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REPORT NO. 9.1-2

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TRIAL DESIGNS MADE IN ACCORDANCE WITH TENTATIVE LIMIT STATES DESIGN STANDARDS FOR REINFORCED MASONRY BUILDINGS

by

JOHN C. KARIOTIS
OMAR M. WAQFI

FEBRUARY 1992

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Kariotis & Associates

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TRIAL DESIGNS MADE IN ACCORDANCE WITH TENTATIVE LIMIT STATES DESIGN STANDARDS FOR REINFORCED MASONRY BUILDINGS

1.0 INTRODUCTION

The end product of the TCCMAR research is the development of seismic design recommendations that will utilize masonry materials in a cost-effective manner. Cost-effective use of masonry in the United States must consider first cost and limitation of possible damage due to natural hazards such as earthquake.

A draft of a Masonry Limit States Design Standard (Tentative Standard) has been written by a number of the Principal Investigators of the TCCMAR program. This draft is being reviewed by a Joint Masonry Standard Committee of the ACI, ASCE and TMS. This draft is also being reviewed by Category 2 Principal Investigators for its effectiveness in specification of seismic design provisions.

The Tentative Standard utilizes the NEHRP Recommended Provisions as a reference for general seismic design provisions. However, the Tentative Standard includes recommendations for seismic design in accordance with a limit states design as the NEHRP Recommended Provisions coordinates its seismic design provisions with working stress standards such as ACI/ASCE 530.

Three buildings that are called the DPC Gymnasium, the TMS Shopping Center and the RCJ Hotel are examples used by the preparers of a Masonry Designers' Guide (MDG). These buildings use ACI/ASCE 530 as the design provisions. These three buildings were selected for these TCCMAR trial designs using the Tentative Limit States Design Standards as the separate efforts will produce results that can have direct comparisons.

2.0 GENERAL PROCEDURE

These trial designs were for seismic design only. The structural materials specified by the MDG were used whenever feasible. Live loading and dead loads that were given by the MDG were used. Snow load, when specified, was reduced to 20 percent of the full snow load as permitted by Sec. 2.1 of the Recommended Provisions for combination with dead loads.

The seismic design generally consisted of the design of the masonry walls for seismic loading normal to the plane of the wall and for seismic loading of masonry walls in their plane as shear walls. For the low-rise buildings, the loading normal to the wall plane was the critical design for determination of vertical reinforcement. The dynamic analysis of the complete building required that a seismic design of the roof diaphragm be made. The path of lateral excitation of ground motion is from the shear wall to the diaphragm edge. The diaphragm couples the mass of the diaphragm and its tributary loads with the shear wall and causes a dynamic displacement at the upper edge of the walls that is parallel to the direction of the ground motion. These walls are excited by the diaphragm motion at their upper edge and by the ground motion at their base. The wall, with its plane normal to the ground motion, is the third oscillator in the path of excitation caused by the horizontal component of the ground motion. In reality, the masonry walls are loaded in real time in their plane and normal to their plane by inertial loads. These loading effects are not combined by current seismic design provisions and these dynamic analyses do not combine the dynamic displacement or stresses in real time.

The calculation of the lateral loading used for determination of required strength used the equations from the Tentative Standard. The specification of seismic hazard used the Appendix to Chapter 1 of the 1991 Edition of the NEHRP Recommended Provisions. This appendix uses spectral acceleration at 0.3 and 1.0 seconds in lieu of A_a and A_v for determination of seismic loading. The seismic design assumed that these buildings are constructed on soils equivalent to an S_2 soil profile. The values for $S_{a(0.3)}$ and $S_{a(1.0)}$ equivalent to A_a and A_v of 0.4 are:

$$S_{a(0.3)} = 1.0$$

$$S_{a(1.0)} = 0.58$$

These values were taken from the commentary to the 1988 Edition of the NEHRP Recommended Provisions, Figure C1-10.

The design forces for parts of the buildings are as follows:

Diaphragms: Loading is $0.35\ S_{a(1.0)}\ W_d$

Walls normal to their plane: Loading is $0.7\ S_{a(1.0)}\ W_w$

Where:

W_d = Weight of diaphragm and the tributary weight of the walls supported by the diaphragm.

W_w = Weight of the wall.

Shear Walls: $V = C_s W$

Where:

$$C_s = \frac{S_{a(1.0)}\ S}{R\ T^n_{es}}$$

and C_s need not be greater than $\dfrac{S_{a\,(0.3)}}{R}$

Where:

S = 1.0 for soil profile S_2

R = $4\frac{1}{2}$

T_{es} = Expected fundamental period

n = 1.0 for $T_{es} \leq 1.0$ and 2/3 for $T_{es} > 1.0$

3.0 DPC GYMNASIUM

3.1 Description of building

A plan of the gymnasium is shown in Figure 3-1. The materials of construction and the design loads are given on Figure 3-1. The effects of the roof diaphragm on the wall design was investigated by use of two alternatives for the roof sheathing. These alternatives were long span steel deck spanning eight feet between the steel trusses and nailed plywood sheathing on 2 X 4 wood rafters spanning the eight feet between the steel trusses.

Elevations of the exterior walls are given in Figure 3-2. All walls are eight inch nominal thickness reinforced concrete masonry. The units are grouted solid. The floor at grade level is a concrete slab and is attached to the walls by dowels. The wall footings are continuous strip foundations placed at one and one-half feet below the level of the floor.

3.2 Design Procedure

The calculations of the design are included in Appendix A-3.1. The procedure and results are summarized in this section.

The concrete masonry walls were designed for seismic loading normal to the face of the wall. The walls were designed as spanning from the floor level to the roof level. No restraint at the floor slab was assumed. The required strength of the walls was determined by loading normal to the wall of 0.7 $S_{a(1.0)}$ W. The weight of the wall was 77 pounds per sq.ft. The expected moment capacity, M_e, of the wall was calculated as:

$$M_e \quad = \quad 0.9 \; [\, A_s \, f_{ye} + P_e \,] \; [d - \frac{a}{2}]$$

Where:

A_s = Area of vertical reinforcement in the width, b, in sq. inches.

f_{ye} = Expected yield stress of the reinforcement, kips per sq. inch.

P_e = Expected axial dead load on wall at mid height of wall, kips.

f_{me} = Expected compression strength of masonry, kips per sq. inch.

a = Length of the compression block measured in the direction of d, inches.

and the distance a is determined by the following method:

$$A_s f_{ye} + P_e \quad = \quad 0.85 \; f_{me} \; a \; b$$

and the quantity of vertical reinforcement is limited by:

$$\rho_v = 0.35\ \rho_{vb}$$

Where:

ρ_v = Ratio of vertical reinforcement to area of wall.

ρ_{vb} = Balanced reinforcement ratio.

The required shear strength of the diaphragm was determined by the use of a loading of 0.35 $S_{a(1.0)}$ W_d, where W_d is the weight of the diaphragm, 20 percent of the snow load and the tributary weight of the masonry walls that are supported laterally by the diaphragm. The expected strength of the diaphragm was taken as twice the published diaphragm strength multiplied by a capacity reduction factor of 0.6.

The required strength of the shear walls was specified by the seismic loading of:

$$V = C_s W$$

and:

$$C_s = \frac{S_{a(1.0)} S}{R\ T_{cs}^{\,n}} \le \frac{S_{a(0.3)}}{R}$$

where:

$S_{a(1.0)}$ = 0.58 for seismic zone 4

S = 1.0

$S_{a(0.3)}$ = 1.0

R = $4\frac{1}{2}$

The limit of C_s is 0.222. The flexural strength of the shear wall, reinforced as required for the loading normal to the wall, greatly exceeds the required strength. The inplane shear strength of the wall must exceed the flexural strength of the wall or 2½ times the required shear strength. The expected inplane shear strength of the shear wall is determined by the equation recommended by Fattal (1991). This equation is:

$$v_u = \left[\left[\frac{0.76}{r_d+0.7} + 0.012\right] \left[4.04\ (\rho_{ve})^{0.3}\right] \sqrt{f_{me}} \right] \frac{d}{L}$$

$$+ (0.1575 \sqrt{\rho_h}\ f_{ye}\ f_{me})\ \delta\ \frac{d}{L} + (0.175\ \sigma_o)\ \frac{d}{L}$$

Where:

r_d	=	Ratio of height to length of the wall.
ρ_v	=	A_{ve}/tL = ratio of vertical reinforcement in one end core to area of wall.
f_{me}	=	Expected compressive strength of the masonry (MPa).
d	=	Distance of end reinforcement bar to opposite end of wall (mm).
L	=	Length of wall (mm).
ρ_h	=	Horizontal reinforcement ratio.
δ	=	1.0 for double curvature, 0.6 for cantilever.
σ_o	=	Nominal axial stress on wall (MPa).

The calculations determined the vertical reinforcement of north and south walls as #5 @ 64 inch spacing. The horizontal reinforcement was taken as equal to the reinforcement in the east and west walls. The maximum height of east and west walls is 32 feet at the center of the gable. The walls have a truss parallel to the wall that make these walls effectively nonbearing. The required vertical reinforcement is #5 @ 16 inch spacing in the center twenty feet of the wall. The vertical reinforcement in the remainder of the wall is #5 @ 24 inches on center. The horizontal shear reinforcement was determined by inplane shear requirements and is #5 @ 40 inches on center.

The steel decking used as a roof diaphragm is an 18 gage, three inch deep unit, 24 inches in width. The decking units have 2 3/4 inch plug welds to the trusses and seam welds at 12 inches on center to the adjacent unit. The alternative sheathing of the diaphragm is ½ inch plywood nailed at 4 inches with 8d nails at all edges.

3.3 Description of the Analytical Model

The analytical model is shown schematically in Figure 3-3. One-half of the building is modeled since the building is symmetrical about both the north-south and east-west axis. The structural elements of the model are the inplane shear wall that transmits the ground motion applied at its lower edge to the edge of the diaphragm; the diaphragm that supports the masonry walls that are loaded by inertial forces normal to their surface; and the masonry wall elements that span from grade to the diaphragm level.

The stiffness properties of each of these elements must be determined by analyses or by prior experimental testing. A nonlinear finite element program (Ewing, 1987) was used to determine the stiffness degradation in the wall beams and determine the force-deformation envelope of the shear wall. The behavioral model of the diaphragm was taken from diaphragm testing performed by ABK (1981).

3.3.1 Wall Beam Element

The out-of-plane walls are modeled as linked beam elements. These beam elements are modeled with degrading stiffness that represents initial cracking, formation of additional cracks and yielding of the vertical reinforcement. Figure 3-4a indicates the segmentation of the north and south walls into beam elements. The width of the beam element is equal to the diaphragm segment of sixteen feet. The height of 24 feet is subdivided into six 4 foot long beam elements. The stiffness-moment-curvature relationship is:

$$\theta = \frac{ML}{EI}$$

and:

$$EI = \frac{ML}{\theta}$$

By applying a moment loading and monitoring θ, a value of effective EI vs. θ is determined for each beam element. The 48 inch long beam element shown in Figure 3-4a was analyzed by a nonlinear finite element program with the mesh shown in Figure 3-4b. The mesh represents two beam elements, the top element was used to apply the loading. Two reinforcement quantities and walls were analyzed. These were the north and south walls that were reinforced with #5 @ 64 inch spacing and the east and west walls with #5 @ 24 inch spacing.

The EI ratio - rotation relationship for each wall is shown in Figures 3-5 and 3-6. The EI ratio is defined as the ratio of EI at any instant to the initial EI. This initial value is calculated as:

$$EI\ (\text{initial}) = 1000\ f_{me}\ I$$

$$= 1000 \times 2.5\ \frac{(16 \times 12)\ (7.625)^3}{12}$$

$$= 1.773 \times 10^7\ \text{k.in}^2$$

The curvature is measured in radians and is that shown for the 48 inch long beam in Figure 3-4a. The mass per wall beam element shown in Figure 3-4a is calculated as:

$$m = \frac{16' \times 4' \times 0.07\ \text{k/sq.ft.}}{386.4} = 0.01275\ \text{k.sec}^2/\text{inch}$$

The mass at the top of the wall is one-half of the typical mass. The stiffness and masses of the wall beam adjacent to the center line of the building is one-half of the adjacent full width element.

Mass damping is used for the beam elements. This damping is equivalent to 7 percent of critical damping for the primary mode for a section of the wall that is 24 feet high and 16 feet wide. This section has a primary mode frequency of W_1 = 10 radian/second.

3.3.2 Diaphragm Parameters

The mass of a segment of the diaphragm (16 x 64 feet) as shown in Figure 3-3 is:

$$m = \frac{64 \times 16 \times 0.015}{386.4} = 0.03975 \text{ kip sec./inch}$$

The stiffness of the 16 feet long segment of the diaphragm was modeled as a Type 14 nonlinear spring, (Kariotis, 1992) with hysteretic behavior. The spring characteristics were obtained from the ABK test program (ABK, 1981). The spring constants used for the plywood and steel deck diaphragms are presented in Table 3-1 and 3-2 respectively. The virgin envelope of these diaphragms are plotted in Figures 3-7 and 3-8 respectively. Damping in the diaphragm is hysteretic and implemented in the behavior of the spring.

Springs S5, S6, S7 and S8, in the plane of the diaphragm, Figure 3-3, are added to connect the lumped mass of the diaphragm with the upper end of the wall beams. These springs were given a stiffness of 3000 kips/inch to represent the connection of the diaphragm to the wall and have 2 percent of critical damping.

The degraded stiffness of the wall beams for each of the ground motions is shown in Table 3-7. Comparison of Tables 3-4 and 3-6 show that the relative deformation of the wall beam is sensitive to the stiffness characteristics of the diaphragm. Additional studies of this effect were made by altering the stiffness of springs S5 through S8 as noted in the model shown in Figure 3-9. These studies were made by introducing the ground motion into the top of the end wall in combination with the application of the ground motion at the base of the out-of-plane walls. Two stiffnesses of springs S5 through S8 were used, k = 500 kips/inch and 10 kips/inch. The comparative deformations of the springs are shown in Figure 3-12. The mid-level relative displacement (curvature) of the six element wall beam is shown in Figure 3-13. The more flexible spring at the top increases the relative deformation of the center of the wall by about 40 percent.

3.3.3 Shear Wall Stiffness

A 64 feet long shear wall with return walls at its ends was analyzed by the FEM program (Ewing, 1987). This analysis determined an initial stiffness, k_i of 19,500 kips/inch. The peak useful displacement of the top of the shear wall was 0.6 inches and the peak force at the top was 1,280 kips.

The mass that is lumped at the top of the shear wall was one-half of the wall weight and one-half of a segment of the diaphragm. This stiffness and mass results in a fundamental frequency of the wall of 48 hz.

3.3.4 Dynamic Analysis Model

Figure 3-3 shows a one-half model of the DPC Gymnasium. The shear wall is spring S13. The diaphragm is segmented into 16 by 64 feet segments and the shear stiffness of each segment is represented by springs S1 through S4. The 16 by 24 feet high wall beams are subdivided into six elements. Beams B1 through B6 at Line B represent the wall. Ground motion is introduced into the model at points E1 through E9.

This dynamic analysis required long running times on a 486 type computer and an alternate of a one-quarter building model was studied. The one-quarter building model is appropriate since the building is symmetrical about the east-west axis. The model is shown in Figure 3-9. The shear wall spring S13 is omitted as one-half of a cantilever shear wall cannot be adequately represented. The stiffness degradation of one-half of a shear wall would not be the same as the degradation of the full shear wall. The stiffness of the diaphragm springs for the one-quarter model are reduced to one-half of half-model stiffness and the lumped masses are reduced by one-half. The number of wall beams are reduced to 24 from 48. The running time of this one-quarter model was significantly reduced.

The results of the two models are compared in Figure 3-10. Figure 3-10 shows the deformation of spring S1 as calculated by the one-quarter model. The second line on Figure 3-10 shows the difference in spring S1 deformation between the half model of the one-quarter model S13. The displacement-time history of spring (13) in the one-half model is shown in Figure 3-11. This displacement was calculated by use of the N-S component of the 1940 El Centro record scaled in amplitude by a factor of 1.31. This study concluded that the one-quarter model shown in Figure 3-9 was adequate for analyses of symmetrical buildings.

3.4 Results of the Analyses

These analyses were for seismic zones having ZPA of 0.4g. The nine ground motions listed in Table 3-3 were used for the analyses.

The results of the analyses are reported in Tables 3-4, 3-5, 3-6 and 3-7. Table 3-4 reports the calculated displacements in the diaphragm. The displacement for the first segment is in COL 2. The displacements at the center of the diaphragm relative to the shear wall is in COL 3. The mid-level displacement of the out-of-plane wall is reported for Grid B, the beam element nearest to the shear wall and for Grid E, the beam element at the center of the building in COL 4 and COL 5, respectively.

The out-of-plane wall for the east -west shaking is 28 feet high. It consists of seven beam elements for each 16 feet wide section. The numbering of the beams is similar to the model shown in Figure 3-9. The beam numbering system in Table 3-7 follows the order of B1 through B7 and corresponds to grid B. Beams B8 through B14 corresponds to grid C and so on. Table 3-7 shows the final degraded stiffness of these beams for east-west shaking. The analysis used the steel deck diaphragm.

3.5 Discussion of the Results

A review of the FEM model of the wall beam confirmed that the beam reinforcement, which was placed in the center of the beam did not yield in any of the dynamic runs. The wall design method of the Tentative Standards does not include an approximation of moment caused by P/Δ effects. Additional studies are needed to fully explore second order effects and the influence of the diaphragm stiffness on the response of the wall that is loaded normal to its plane.

The equations used for seismic design of the diaphragm were adequate for this configuration of diaphragm shape and for these out-of-plane wall stiffnesses. A preliminary run of a one-half building model with linear-elastic wall beams indicated that the design coefficients used for diaphragm loading may be inadequate when the wall stiffness is increased. The steel deck diaphragm is very sensitive to deformations as strength degradation occurs immediately after peak strength is reached. This behavior of steel deck diaphragms is significantly different than that of nailed wood-sheathed diaphragms and may require a different seismic loading coefficient.

TABLE 3-1

PLYWOOD DIAPHRAGM, SPRING 14. MONOTONIC, DPC GYM

Nonlinear spring type number	=	14
Force-deformation path plot flag	=	1
Damping flag for nonlinear spring	=	0

MATERIAL PROPERTY DATA FOR NONLINEAR SPRING

Spring constant, K1	=	4.000E+01
Spring constant, K2	=	2.000E+01
Spring constant, K3	=	2.000E+00
Break point force, F1	=	7.200E+00
Break point force, F2	=	4.800E+01
Break point deformation, E1	=	3.800E-01
Break point deformation, E2	=	2.420E+00
Gap	=	2.000E-01
Viscous damping coefficient, CV	=	0.000E-01
Velocity exponent for CV, EXPCV	=	0.000E-01
Coulomb damping coefficient, CC	=	0.000E-01
Force time history number for CC	=	0.000E-01
Nonlinear spring number for CC	=	0.000E-01
Unused coefficient	=	2.000E-01
Velocity parameter for CC	=	0.000E-01

TABLE 3-2

STEEL DECK DIAPHRAGM, SPRING 14. MONOTONIC, DPC GYM

Nonlinear spring type number	=	14
Force-deformation path plot flag	=	1
Damping flag for nonlinear spring	=	0

MATERIAL PROPERTY DATA FOR NONLINEAR SPRING

Spring constant, K1	=	1.299E+02
Spring constant, K2	=	1.700E+00
Spring constant, K3	=	9.000E-01
Break point force, F1	=	1.102E+01
Break point force, F2	=	7.105E+01
Break point deformation, E1	=	2.848E-01
Break point deformation, E2	=	3.560E+01
Gap	=	2.000E-01
Viscous damping coefficient, CV	=	0.000E-01
Velocity exponent for CV, EXPCV	=	0.000E-01
Coulomb damping coefficient, CC	=	0.000E-01
Force time history number for CC	=	0.000E-01
Nonlinear spring number for CC	=	0.000E-01
Unused coefficient	=	2.000E-01
Velocity parameter for CC	=	0.000E-01

TABLE 3-3

GROUND MOTIONS USED FOR LPM RUNS

No.	Name	Duration	C1	C2
1	EL CENTRO E-W	53.0	0.9255	1.7875
2	EL CENTRO N-S	53.0	0.6777	1.3145
3	PINE UNION 140	29.0	0.8622	1.7067
4	CRUICKSHANK RD. 230	34.0	0.7632	1.4951
5	JAMES ROAD 140	29.0	0.7126	1.3893
6	KERN CO. 69	54.0	1.4080	2.8648
7	CRUICKSHANK RD. 140	34.0	0.6157	1.2024
8	BRAWLEY AIRPORT 315	37.0	1.0644	2.0738
9	KEYSTONE ROAD 140	39.0	0.9485	1.8501

C1: SCALING FACTOR FOR ZPA 0.2g

C2: SCALING FACTOR FOR ZPA 0.4g

TABLE 3-4

COL 1	COL 2	COL 3	COL 4	COL 5
1	0.581 -0.381	1.289 -0.917	3.69 -3.72	3.48 -3.32
2	0.585 -0.525	1.340 -1.210	2.58 -2.66	3.22 -4.06
3	0.482 -0.428	1.070 -0.996	3.16 -2.82	4.35 -3.64
4	0.392 -0.455	1.016 -1.093	3.40 -3.67	3.25 -3.67
5	0.379 -0.328	0.888 -0.804	2.60 -2.50	2.72 -2.06
6	0.438 -0.525	0.999 -1.161	4.30 -4.00	4.63 -4.29
7	0.388 -0.352	0.878 -0.849	2.55 -2.74	3.82 -3.26
8	0.440 -0.344	1.032 -0.851	3.07 -2.56	4.53 -3.63
9	0.503 -0.527	1.134 -1.190	3.29 -3.02	4.02 -3.76

COL1 : GROUND MOTION NO.
COL2 : SPRING 1 MAXIMUM DEFORMATION.
COL3 : TOTAL DIAPHRAGM DEFORMATION FROM WALL TO CENTER LINE.
COL4 : GRID B MID-LEVEL DEFLECTION (RELATIVE CURVATURE).
COL5 : GRID E MID-LEVEL DEFLECTION (RELATIVE CURVATURE).

STEEL DECK DIAPHRAGM. SHAKING IN THE N-S DIRECTION.

TABLE 3-5

COL 1	COL 2	COL 3	COL 4
1	0.357	0.810	4.05
	-0.255	-0.604	-3.68
2	0.413	0.930	3.95
	-0.280	-0.640	-4.61
3	0.308	0.741	4.24
	-0.351	-0.834	-3.28
4	0.473	1.116	3.19
	-0.543	-1.202	-3.56
5	0.323	0.755	2.93
	-0.266	-0.633	-2.43
6	0.255	0.617	4.66
	-0.302	-0.733	-4.88
7	0.204	0.484	4.00
	-0.297	-0.694	-3.28
8	0.319	0.719	4.44
	-0.260	-0.578	-3.76
9	0.405	0.924	3.88
	-0.362	-0.878	-5.11

COL1 : GROUND MOTION NO.
COL2 : SPRING 1 MAXIMUM DEFORMATION.
COL3 : TOTAL DIAPHRAGM DEFORMATION FROM WALL TO CENTER LINE.
COL4 : GRID B MID-LEVEL DEFLECTION (RELATIVE CURVATURE).

STEEL DECK DIAPHRAGM. SHAKING IN THE E-W DIRECTION.

TABLE 3-6

COL 1	COL 2	COL 3	COL 4
1	1.628 -1.342	3.638 -3.037	3.11 -3.18
2	1.446 -1.509	3.621 -3.730	2.14 -2.32
3	1.512 -1.263	3.430 -2.933	2.16 -1.52
4	0.979 -1.490	2.560 -3.334	1.56 -2.01
5	1.072 -1.270	2.554 -2.842	2.59 -2.06
6	1.289 -1.402	2.978 -3.319	2.13 -2.16
7	1.036 -1.640	2.553 -3.447	1.71 -1.57
8	1.102 -1.453	2.765 -3.232	2.17 -1.26
9	1.391 -1.234	3.208 -3.027	2.29 -2.68

COL1 : GROUND MOTION NO.
COL2 : SPRING 1 MAXIMUM DEFORMATION.
COL3 : TOTAL DIAPHRAGM DEFORMATION FROM WALL TO CENTER LINE.
COL4 : GRID B MID-LEVEL DEFLECTION (RELATIVE CURVATURE).

PLYWOOD DIAPHRAGM. SHAKING IN THE N-S DIRECTION.

TABLE 3-7

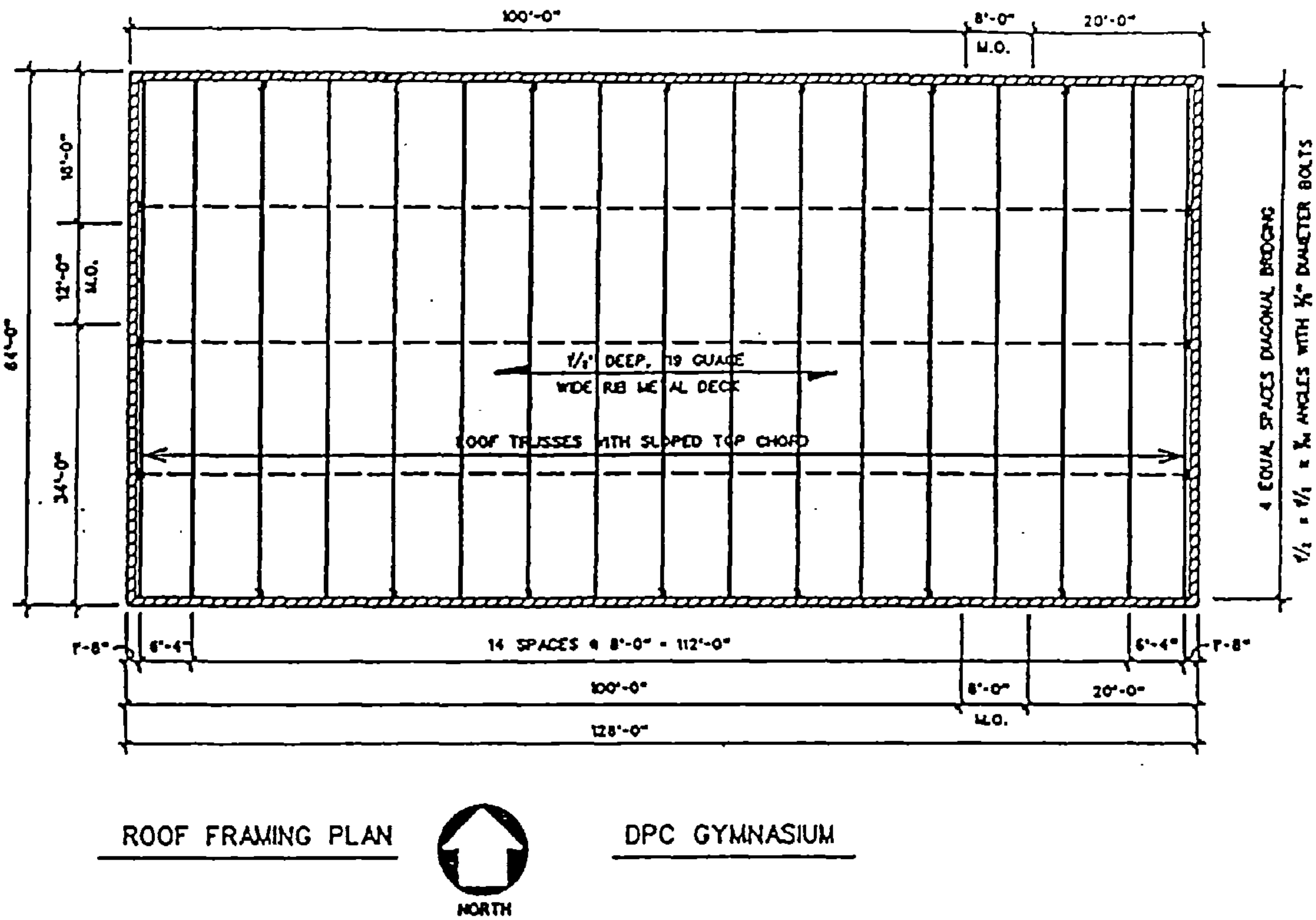
Beam No.	1 C_2	1 C_3	2 C_2	2 C_3	3 C_2	3 C_3	4 C_2	4 C_3	5 C_2	5 C_3	6 C_2	6 C_3	7 C_2	7 C_3	8 C_2	8 C_3	9 C_2	9 C_3
1	0.57	0.45	0.37	0.42	0.6	0.3	0.6	0.3	0.63	0.65	0.3	0.37	0.51	0.47	0.52	0.3	0.3	0.45
2	0.16	0.16	0.15	0.16	0.18	0.15	0.18	0.15	0.18	0.18	0.15	0.15	0.18	0.16	0.16	0.15	0.15	0.16
3	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.16	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
4	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
5	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.16	0.16	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
6	0.18	0.16	0.15	0.16	0.18	0.16	0.18	0.16	0.2	0.18	0.15	0.15	0.18	0.16	0.16	0.16	0.15	0.16
7	0.57	0.45	0.3	0.42	0.63	0.42	0.63	0.42	0.7	0.7	0.37	0.37	0.47	0.49	0.47	0.42	0.3	0.45
8	0.51	0.37	0.3	0.37	0.49	0.33	0.49	0.33	0.57	0.6	0.3	0.33	0.47	0.42	0.42	0.25	0.25	0.3
9	0.16	0.15	0.15	0.15	0.16	0.15	0.16	0.15	0.18	0.18	0.15	0.15	0.16	0.16	0.16	0.15	0.15	0.15
10	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
11	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
12	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.16	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
13	0.16	0.15	0.15	0.16	0.18	0.16	0.18	0.16	0.18	0.18	0.15	0.15	0.16	0.16	0.16	0.16	0.15	0.16
14	0.51	0.37	0.3	0.37	0.6	0.37	0.6	0.37	0.6	0.51	0.33	0.42	0.37	0.45	0.37	0.39	0.25	0.37
15	0.52	0.37	0.25	0.3	0.37	0.3	0.37	0.3	0.45	0.49	0.3	0.33	0.33	0.37	0.47	0.25	0.22	0.3
16	0.16	0.15	0.15	0.15	0.16	0.15	0.16	0.15	0.18	0.18	0.15	0.15	0.16	0.16	0.16	0.15	0.15	0.15
17	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
18	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
19	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.16	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
20	0.16	0.15	0.15	0.16	0.18	0.15	0.18	0.15	0.16	0.16	0.15	0.16	0.15	0.16	0.16	0.15	0.15	0.15
21	0.49	0.37	0.25	0.42	0.49	0.37	0.49	0.37	0.47	0.37	0.3	0.39	0.3	0.42	0.37	0.37	0.25	0.3
22	0.54	0.37	0.25	0.3	0.45	0.3	0.45	0.3	0.39	0.42	0.3	0.3	0.37	0.3	0.45	0.25	0.22	0.25
23	0.16	0.15	0.15	0.15	0.16	0.15	0.16	0.15	0.16	0.16	0.15	0.15	0.15	0.15	0.18	0.15	0.15	0.15
24	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
25	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
26	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15	0.15
27	0.16	0.15	0.15	0.16	0.18	0.15	0.18	0.15	0.16	0.16	0.15	0.15	0.15	0.15	0.16	0.15	0.15	0.15
28	0.49	0.37	0.25	0.37	0.49	0.37	0.49	0.37	0.45	0.37	0.3	0.37	0.3	0.42	0.37	0.37	0.25	0.3

C_2 = Effective EI in the negative direction as a ratio of uncracked stiffness

C_3 – Effective EI in the positive direction

Values are for the steel deck diaphragm shaken in the East-West direction

The wall is subdivided into seven segments of 4 feet in length



NOTES

1. WALL CONSTRUCTION

   SINGLE WYTHE REINFORCED CONCRETE BLOCK MASONRY
   SEE ELEVATION VIEWS FOR WALL DIMENSIONS

2. ROOF CONSTRUCTION (FLEXIBLE DIAPHRAGM)

   A. SINGLE PLY ROOFING, 2" RIGID INSULATION, METAL ROOF DECK ON STEEL TRUSSES WITH SLOPED TOP CHORD

   B. PLYWOOD SHEATHING ON 2 x 4 RAFTERS ON STEEL TRUSSES WITH SLOPED TOP CHORD

   C. TRUSS BEARING PLATES ARE 6" x 12"

3. DESIGN LOADS

   WIND 20 PSF

ROOF SELF WEIGHT	20 PSF (INCLUDING FRAMING)
SNOW ON ROOF	40 PSF OVERALL OR 20 PSF ON ONE SLOPE AND 40 PSF ON THE OPPOSITE SIDE
SEISMIC ZONE	FOUR

4. ALLOWABLE SOIL BEARING 6000 PSF

FIGURE 3-1

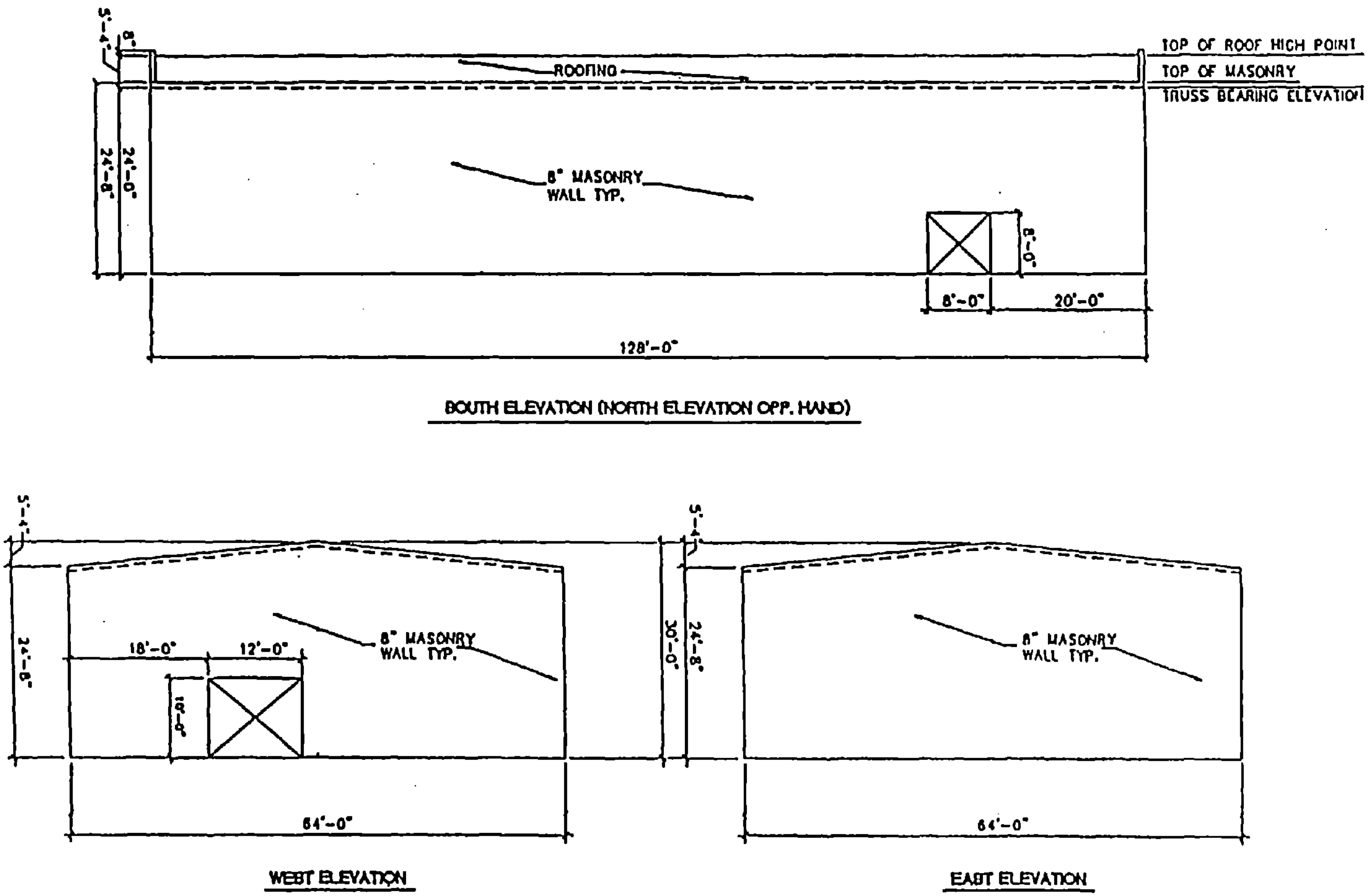

TOP OF ROOF HIGH POINT
TOP OF MASONRY
TRUSS BEARING ELEVATION
ROOFING
8" MASONRY WALL TYP.
5'-4"
8"
24'-0"
24'-8"
8'-0"
8'-0"
20'-0"
128'-0"
SOUTH ELEVATION (NORTH ELEVATION OPP. HAND)
8" MASONRY WALL TYP.
5'-4"
24'-8"
18'-0"
12'-0"
10'-0"
64'-0"
WEST ELEVATION
8" MASONRY WALL TYP.
5'-4"
30'-0"
24'-8"
64'-0"
EAST ELEVATION

FIGURE 3-2

LEGEND

S = Spring

E = Excitation

B = Beam

D = Active DOF

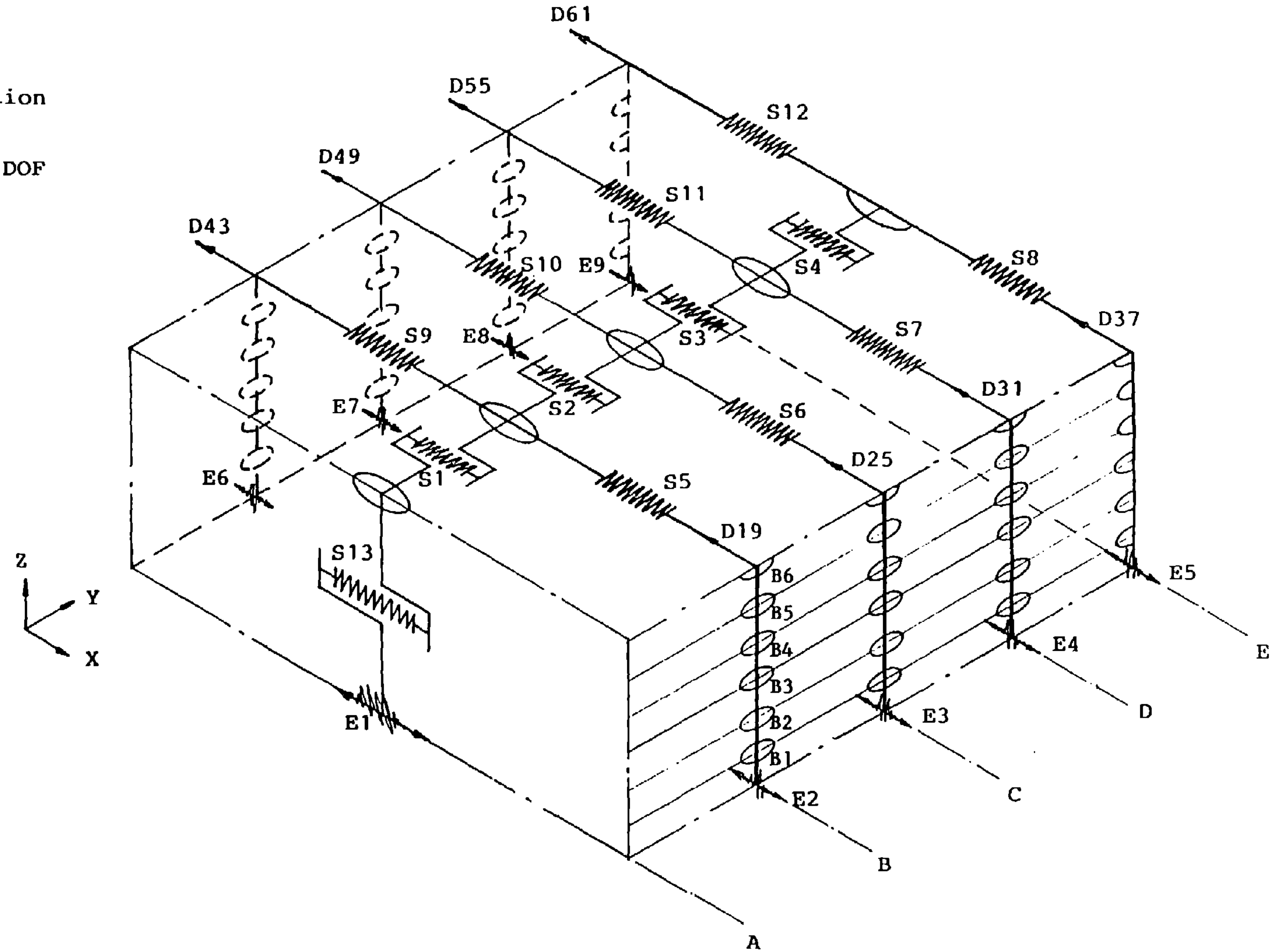


FIGURE 3-3

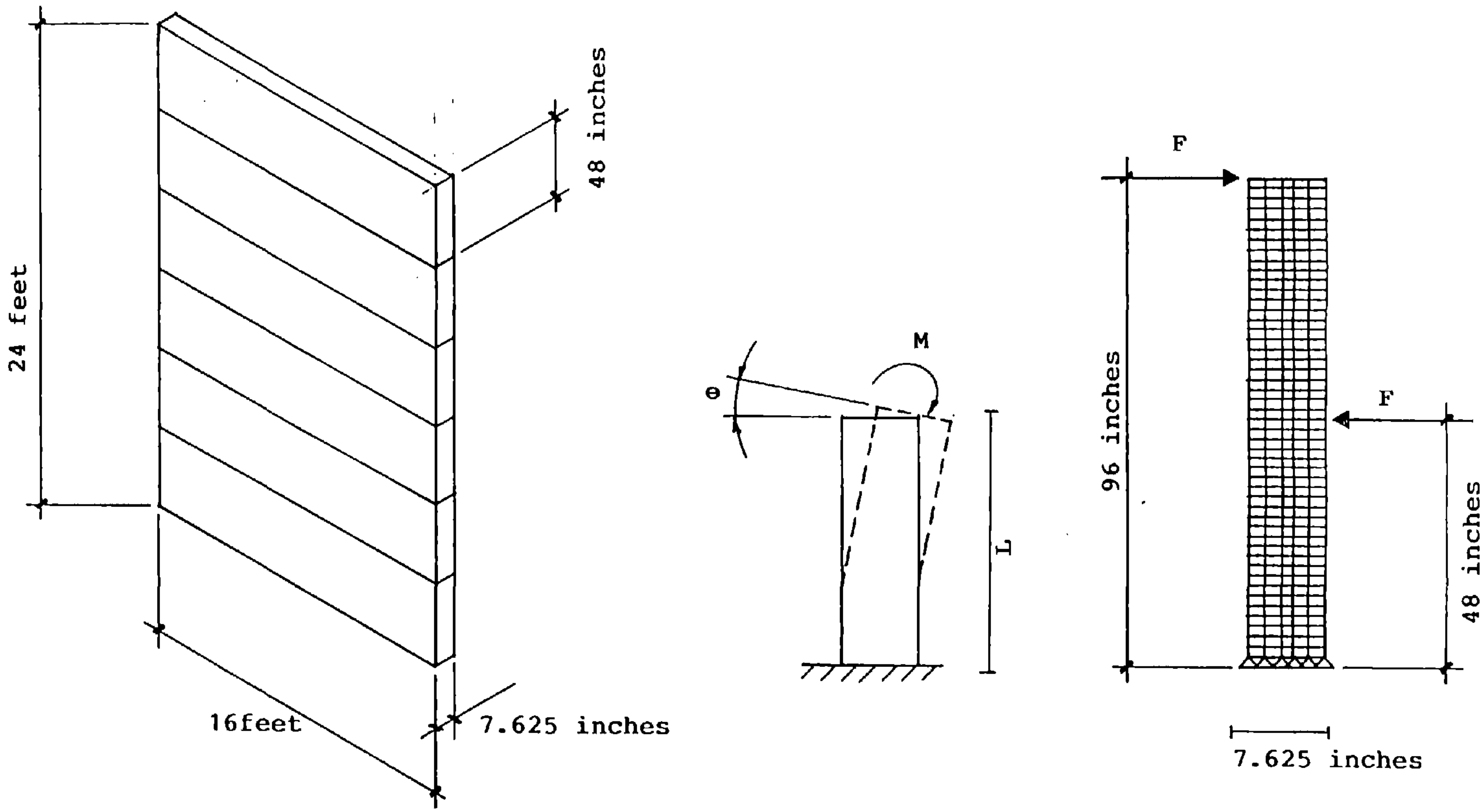
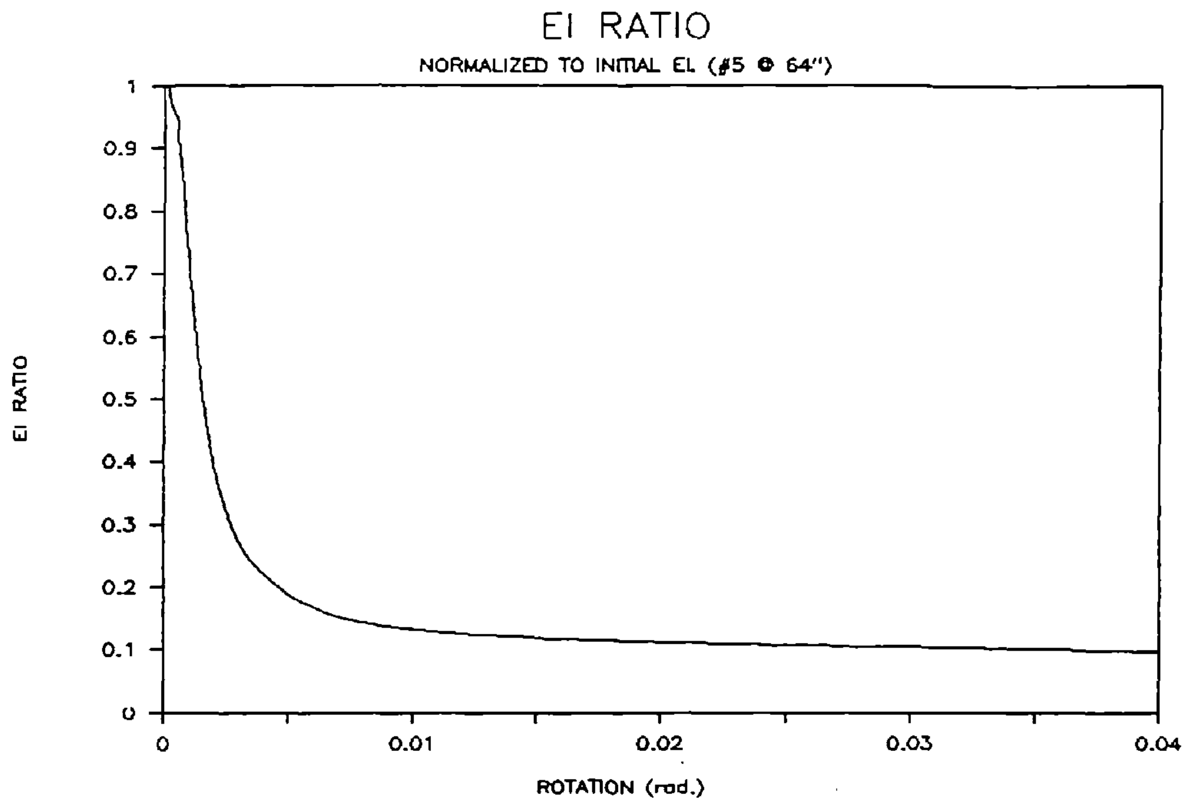

24 feet
16feet
48 inches
7.625 inches
M
θ
L
F
F
96 inches
48 inches
7.625 inches

FIGURE 3-4


EI RATIO
NORMALIZED TO INITIAL EI. (#5 @ 64")
EI RATIO
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.01
0.02
0.03
0.04
ROTATION (rad.)

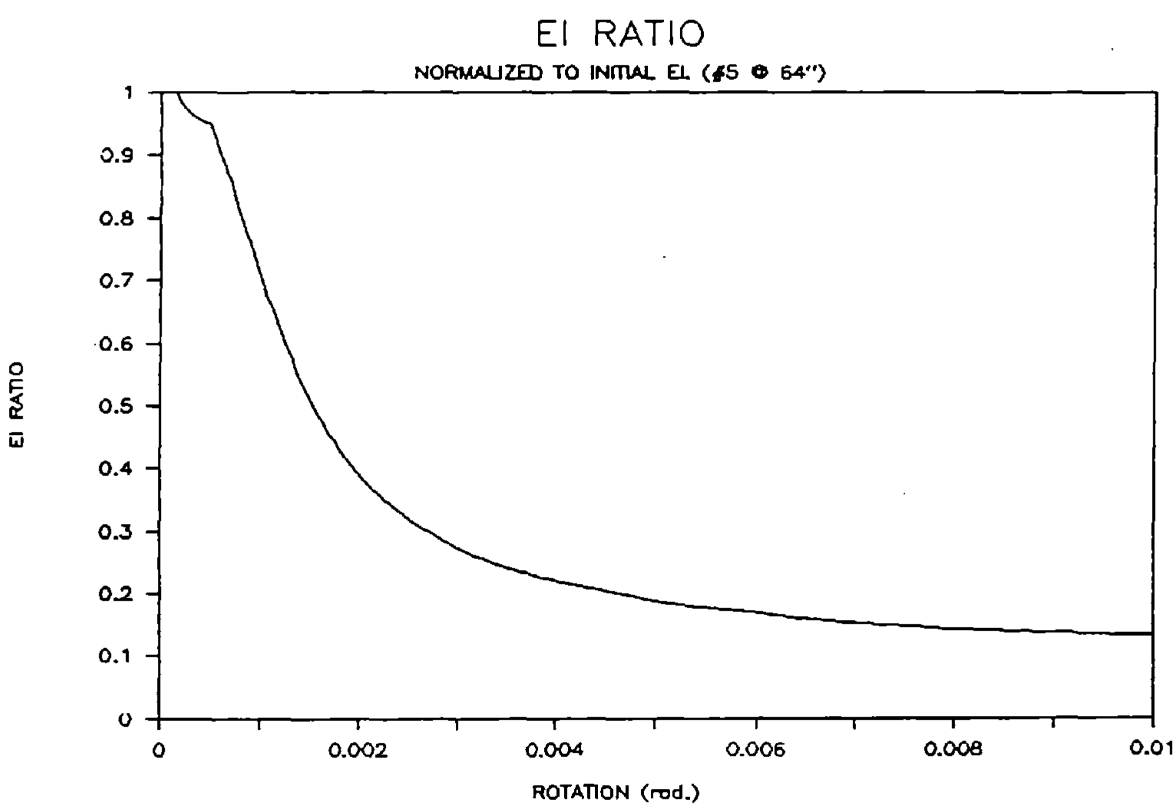
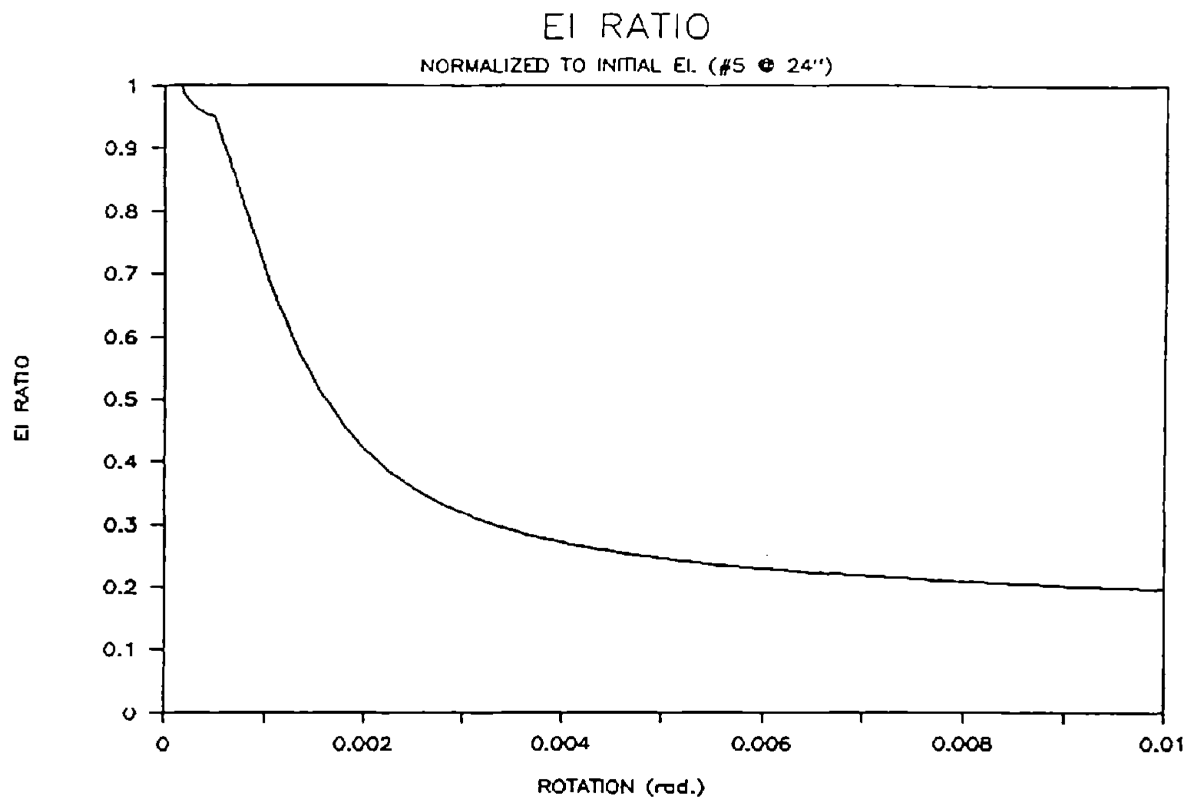

EI RATIO
NORMALIZED TO INITIAL EI. (#5 @ 64")
EI RATIO
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.002
0.004
0.006
0.008
0.01
ROTATION (rad.)

FIGURE 3-5


EI RATIO
NORMALIZED TO INITIAL EI. (#5 @ 24")
EI RATIO
ROTATION (rad.)
0
0.002
0.004
0.006
0.008
0.01
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0

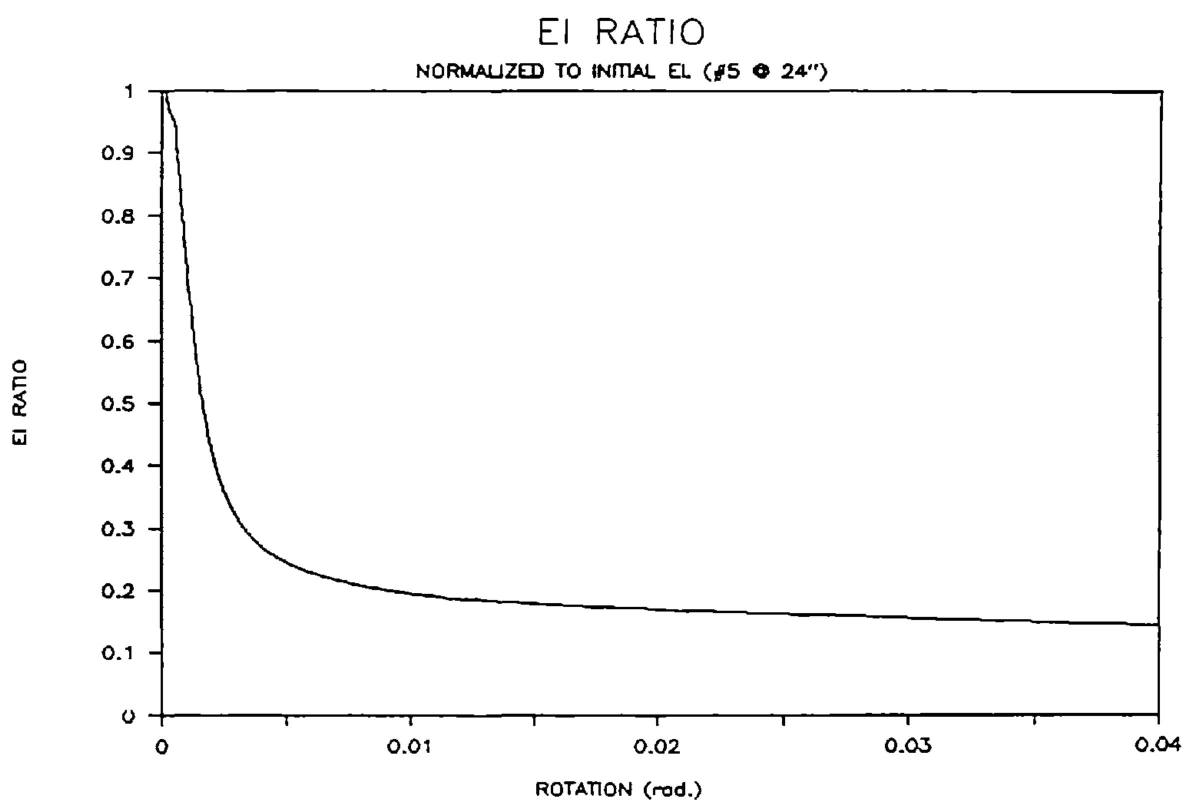

EI RATIO
NORMALIZED TO INITIAL EL (#5 @ 24")
EI RATIO
ROTATION (rad.)
0
0.01
0.02
0.03
0.04
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0

FIGURE 3-6

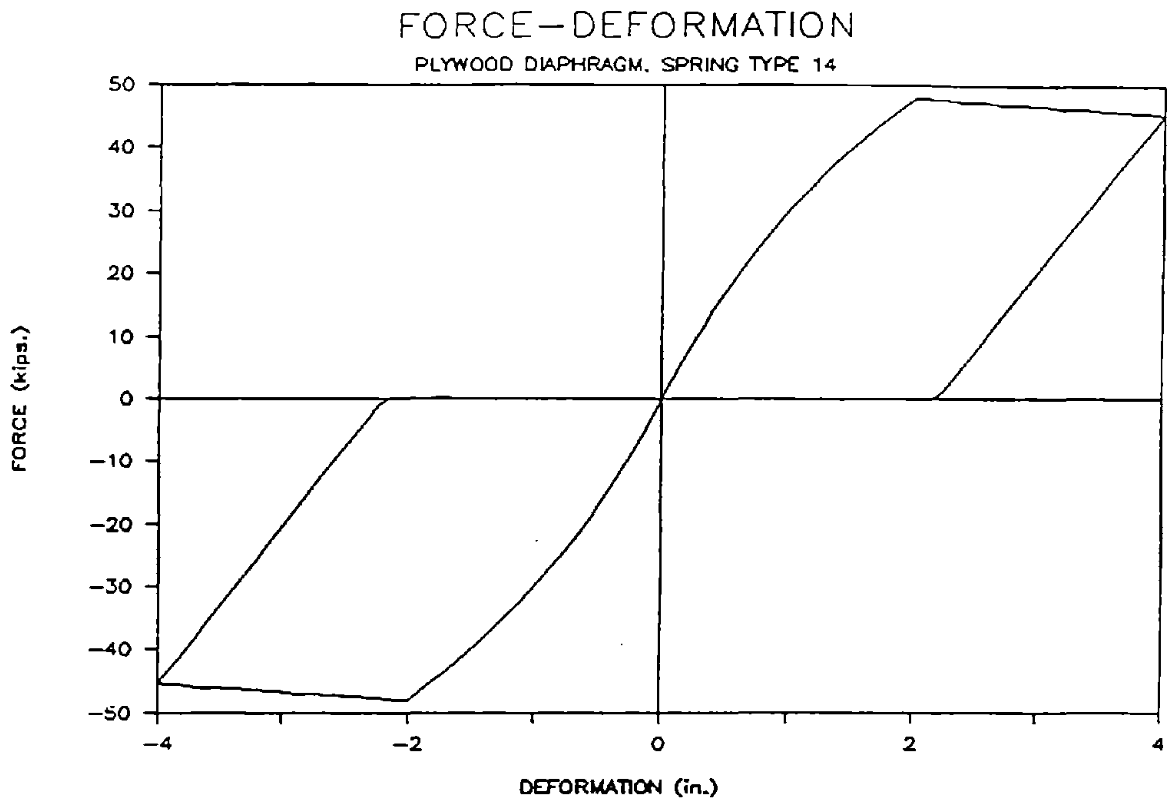


FIGURE 3-7

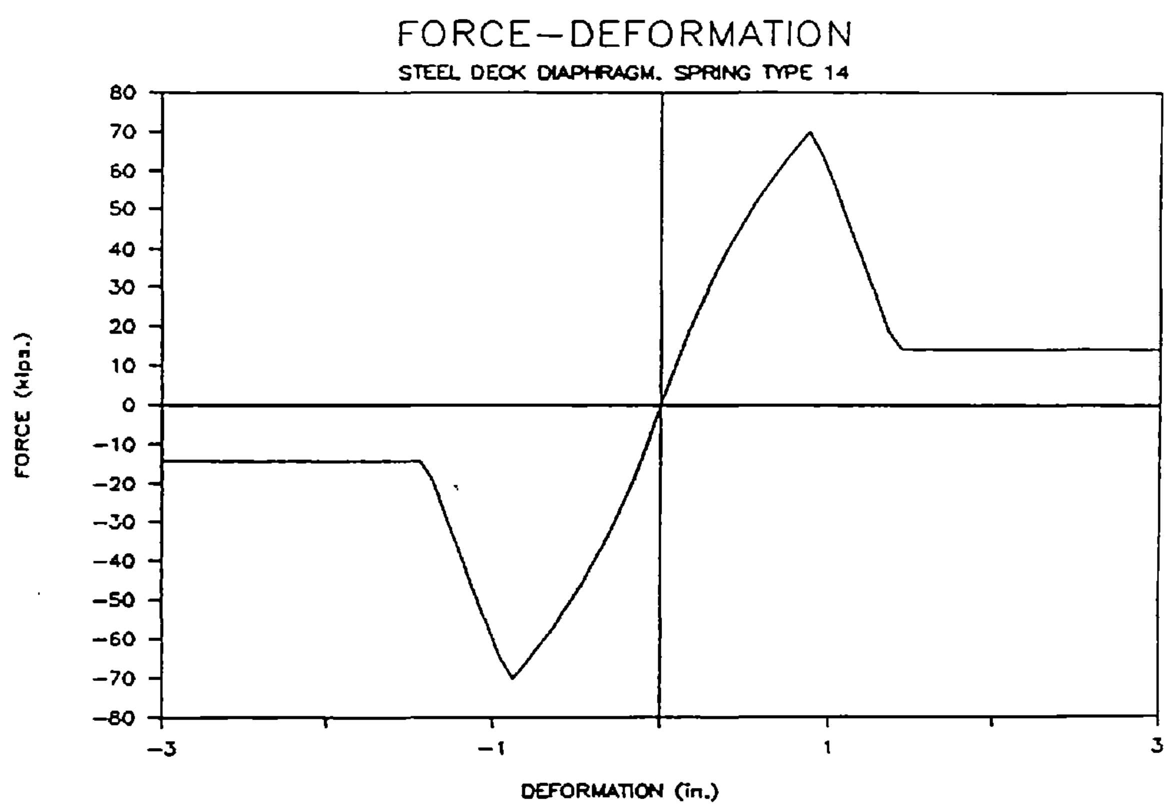


FIGURE 3-8

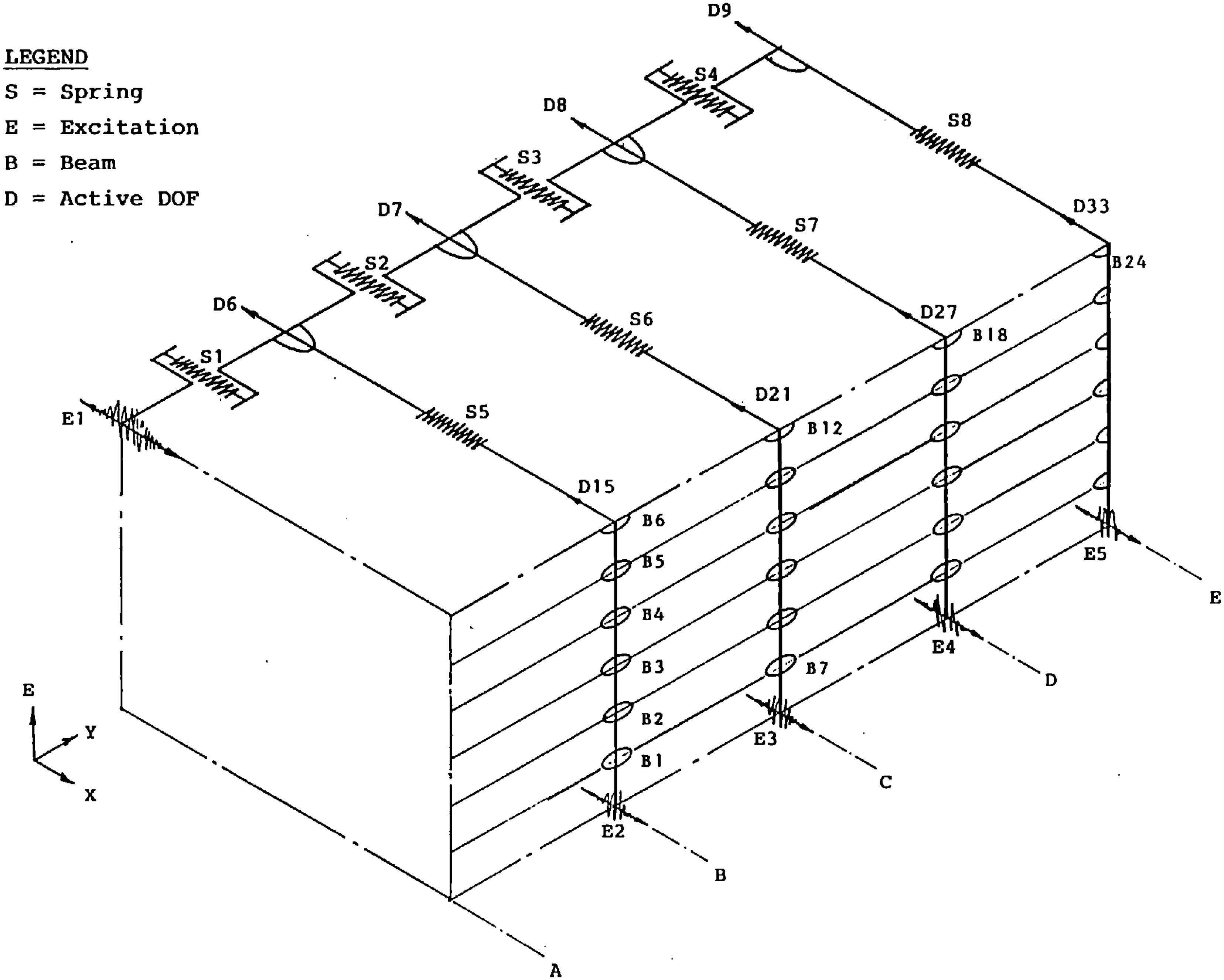

LEGEND
S = Spring
E = Excitation
B = Beam
D = Active DOF
D9
S4
D8
S8
S3
D33
D7
S7
S2
B24
D6
D27
S6
B18
S1
D21
E1
S5
B12
D15
B6
B5
E5
B4
E
B3
B7
E4
E
D
Y
B2
E3
B1
C
X
E2
B
A

FIGURE 3-9

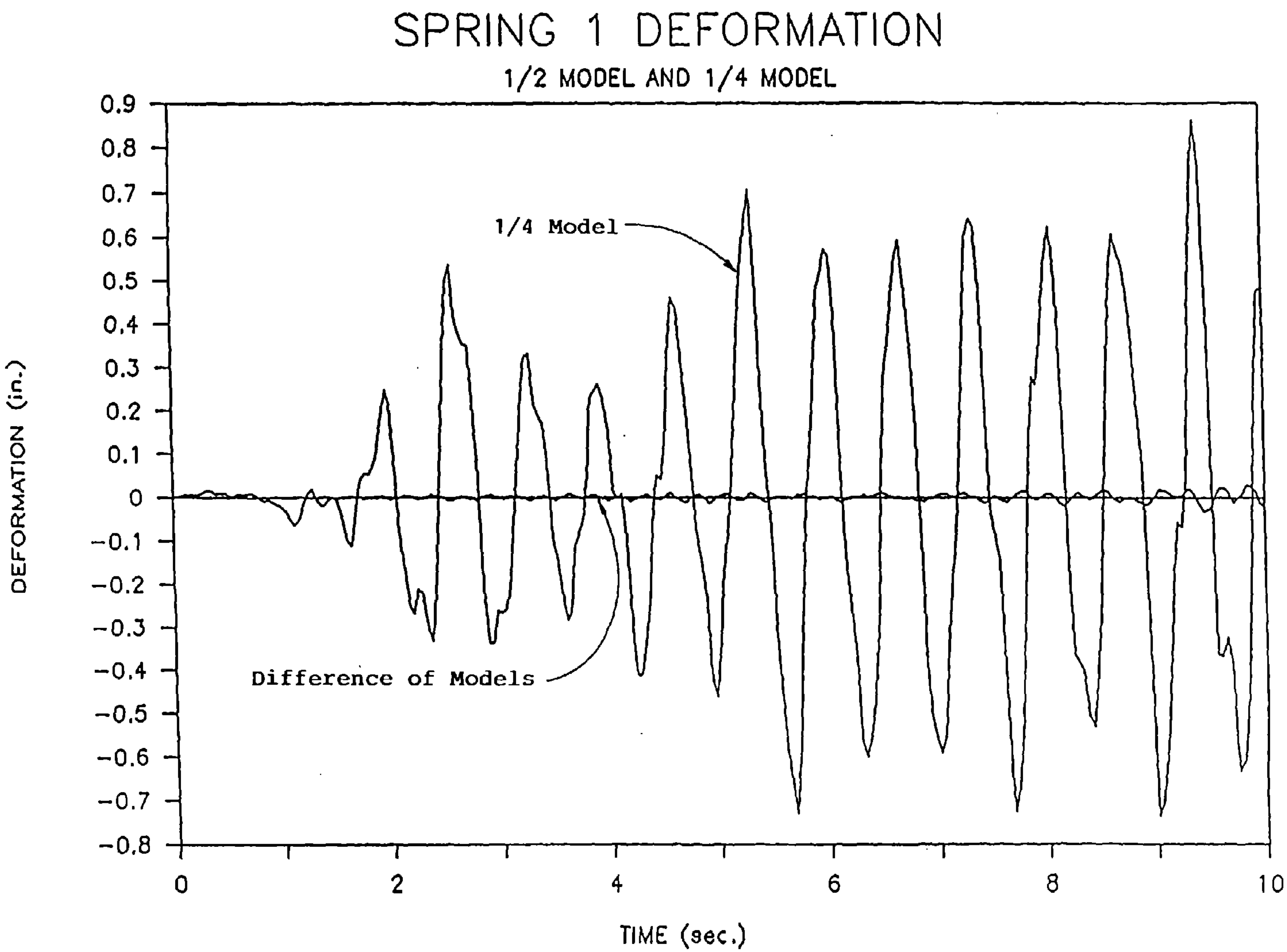

SPRING 1 DEFORMATION
1/2 MODEL AND 1/4 MODEL
1/4 Model
Difference of Models
DEFORMATION (in.)
TIME (sec.)
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
−0.1
−0.2
−0.3
−0.4
−0.5
−0.6
−0.7
−0.8
0
2
4
6
8
10

FIGURE 3-10

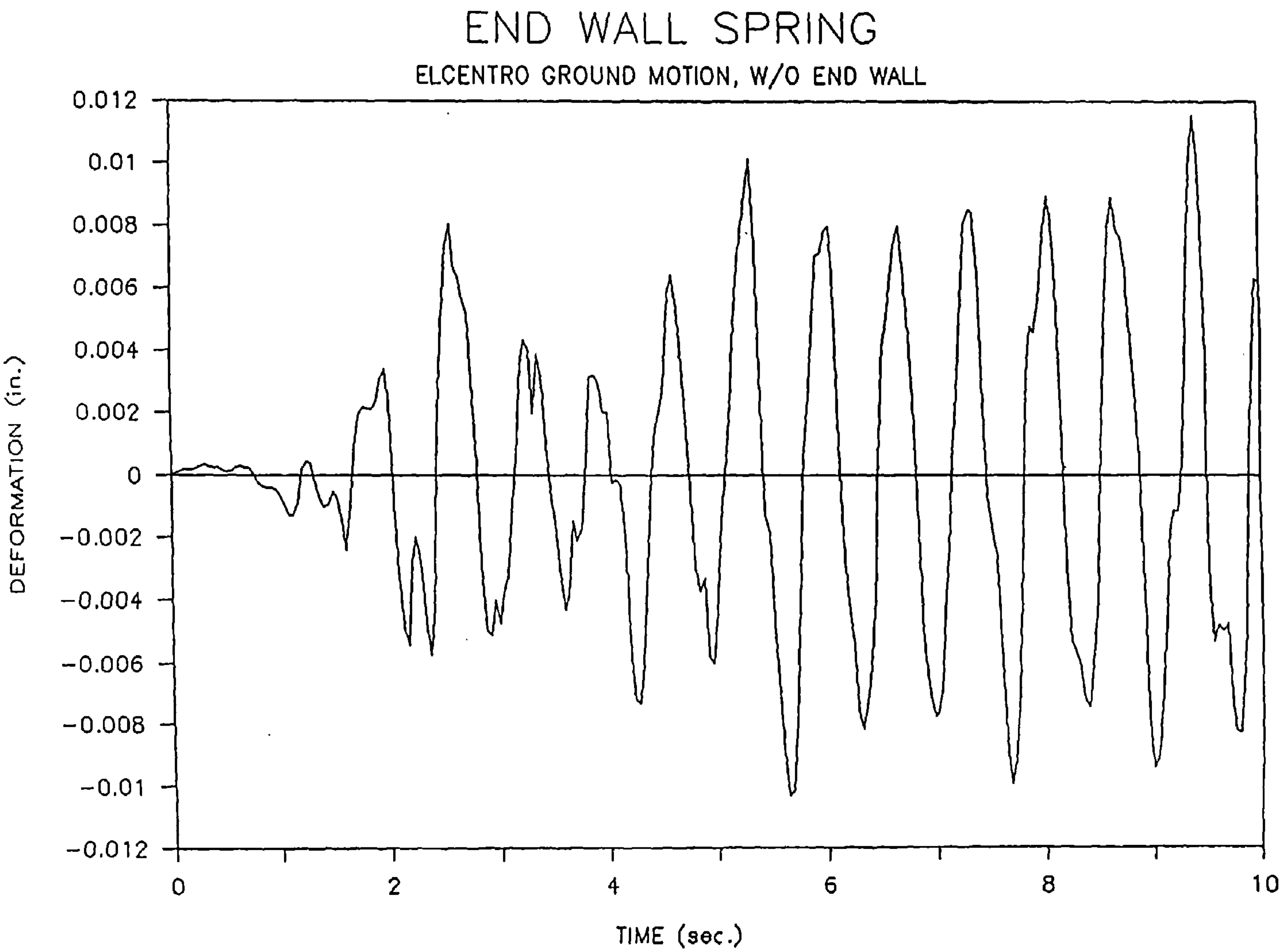

END WALL SPRING
ELCENTRO GROUND MOTION, W/O END WALL
DEFORMATION (in.)
0.012
0.01
0.008
0.006
0.004
0.002
0
-0.002
-0.004
-0.006
-0.008
-0.01
-0.012
0
2
4
6
8
10
TIME (sec.)

FIGURE 3-11

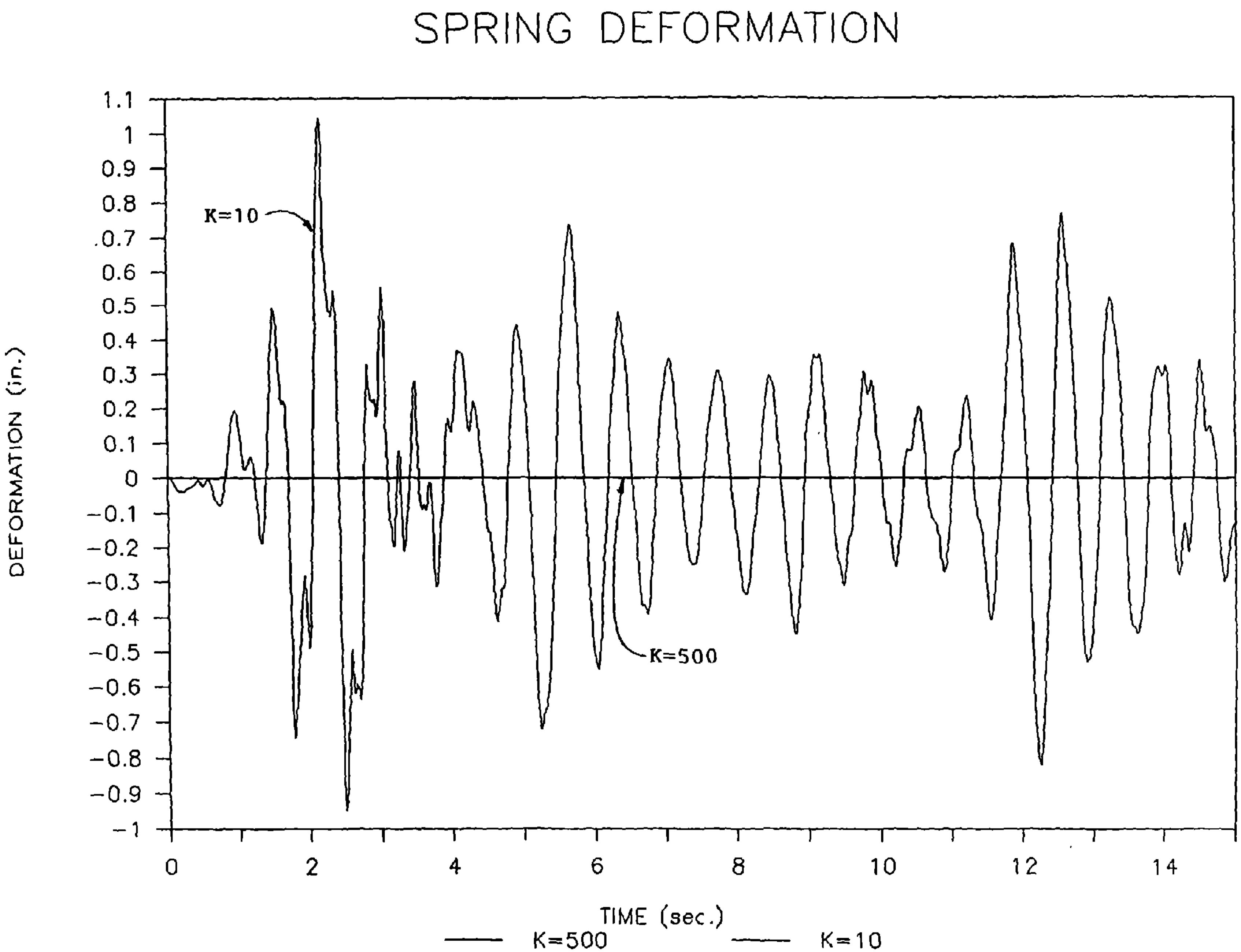

SPRING DEFORMATION
DEFORMATION (in.)
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
−0.1
−0.2
−0.3
−0.4
−0.5
−0.6
−0.7
−0.8
−0.9
−1
0
2
4
6
8
10
12
14
K=10
K=500
TIME (sec.)
K=500
K=10

FIGURE 3-12

MID-LEVEL CURVATURE

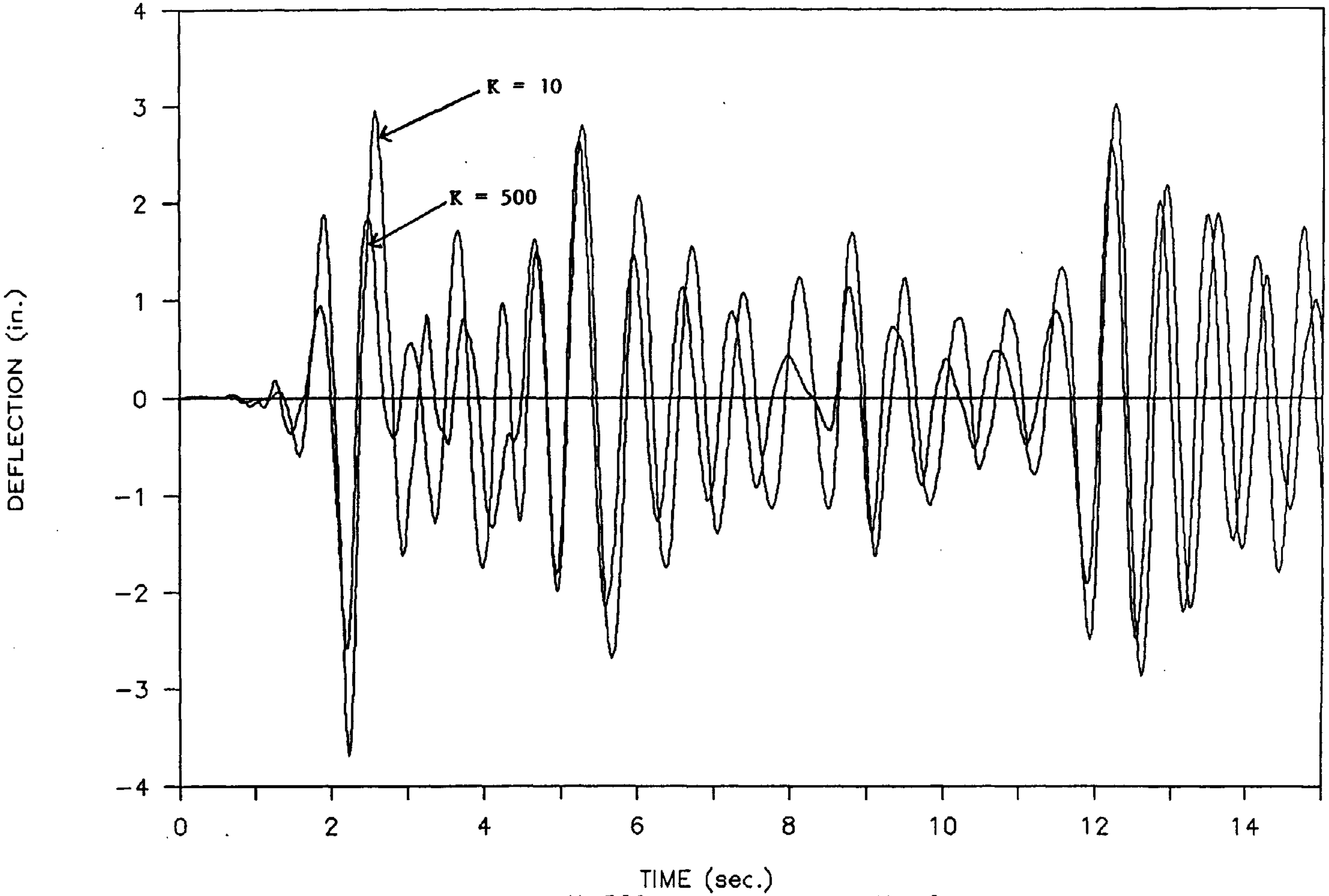


FIGURE 3-13

FIGURE GROUP 3-14

STEEL DECK DIAPHRAGM

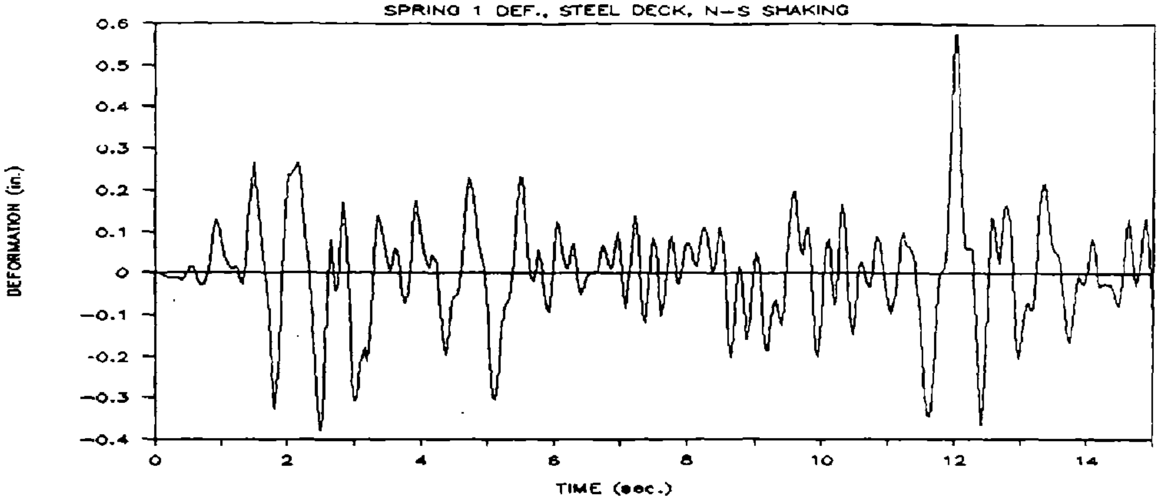
NORTH-SOUTH SHAKING (FLEXIBLE)

DEFORMATION OF SPRING (1),

FIRST SEGMENT OF DIAPHRAGM

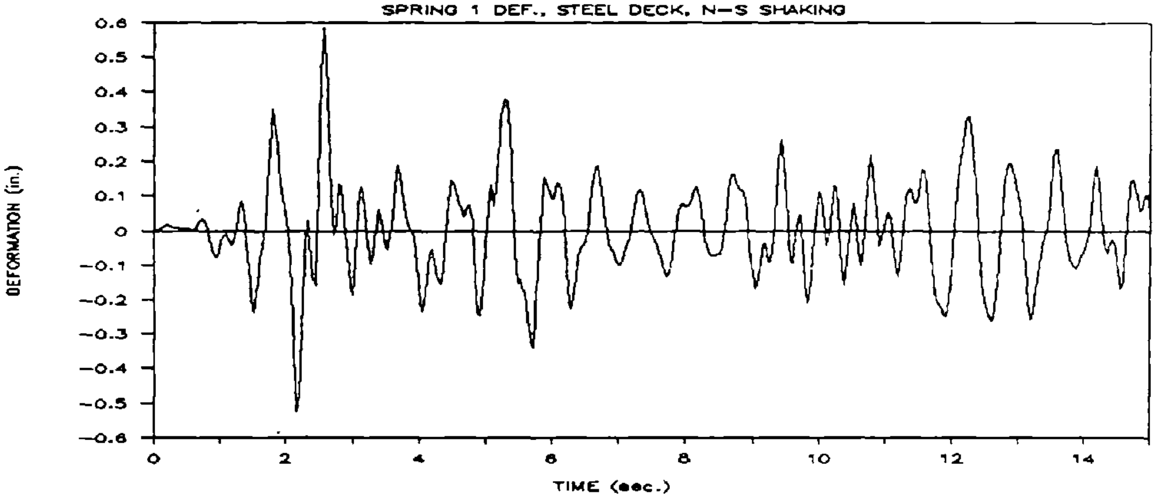
GROUND MOTION 1

SPRING 1 DEF., STEEL DECK, N–S SHAKING



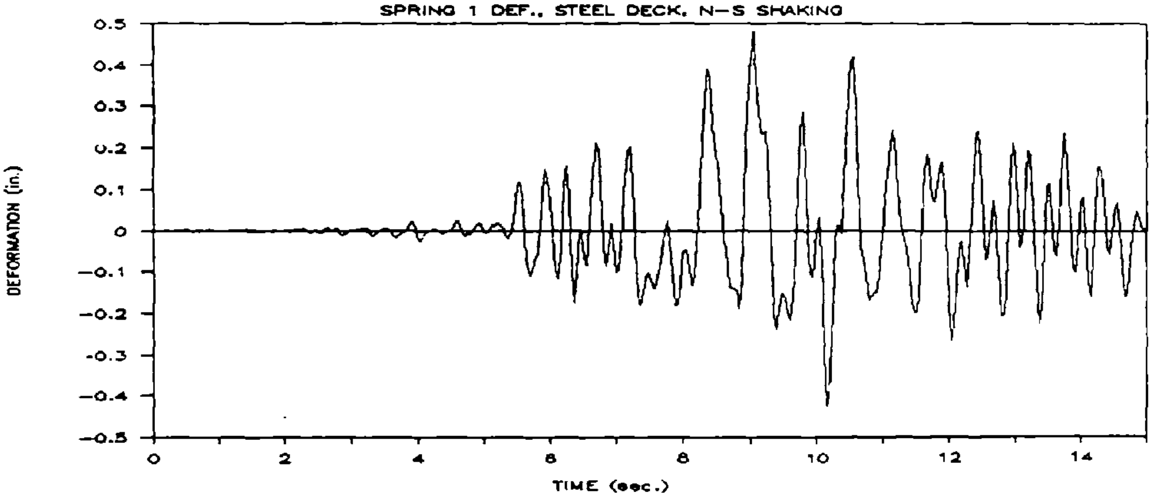
GROUND MOTION 2

SPRING 1 DEF., STEEL DECK, N–S SHAKING



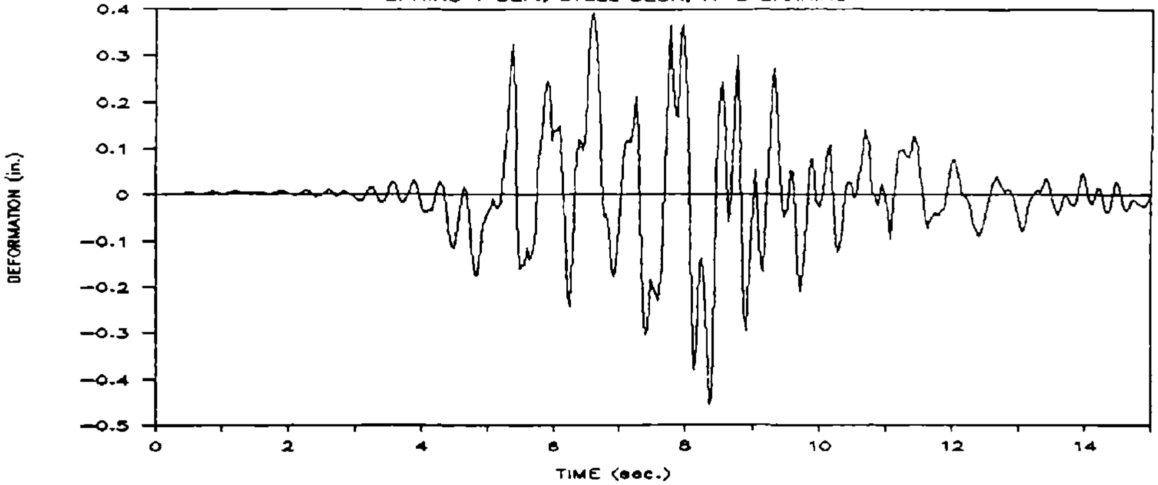
GROUND MOTION 3

SPRING 1 DEF., STEEL DECK, N–S SHAKING



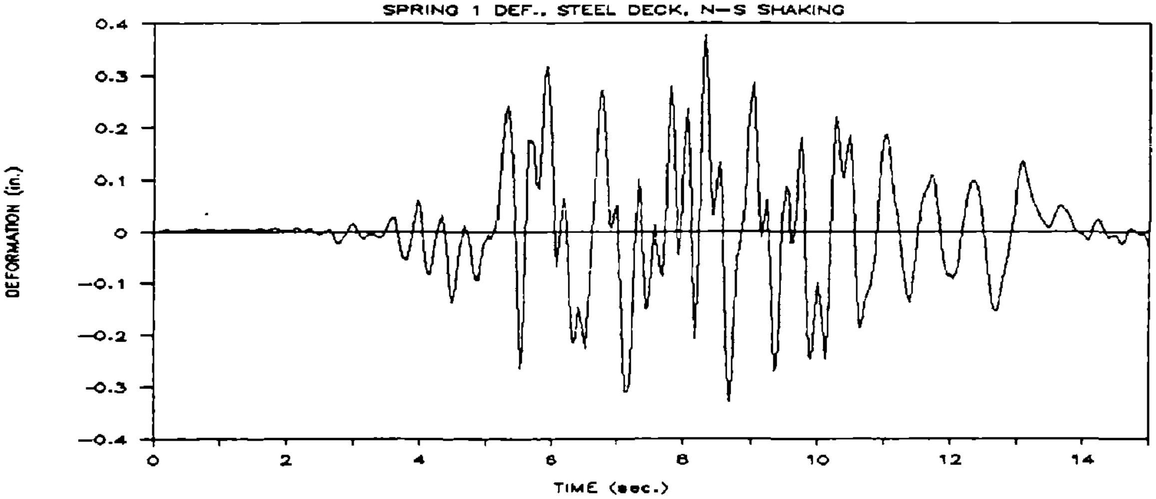
GROUND MOTION 4

SPRING 1 DEF., STEEL DECK, N–S SHAKING



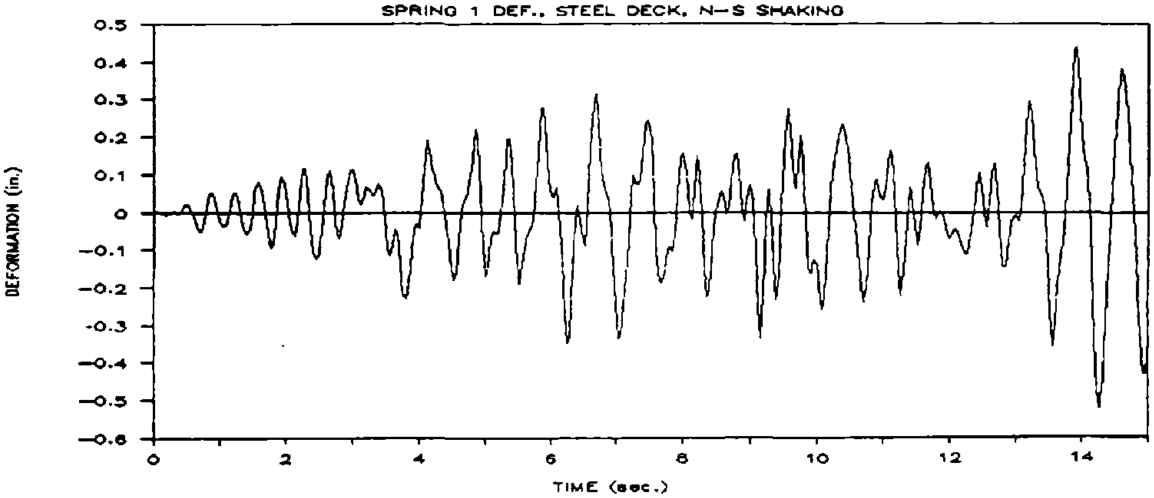
GROUND MOTION 5

SPRING 1 DEF., STEEL DECK, N–S SHAKING



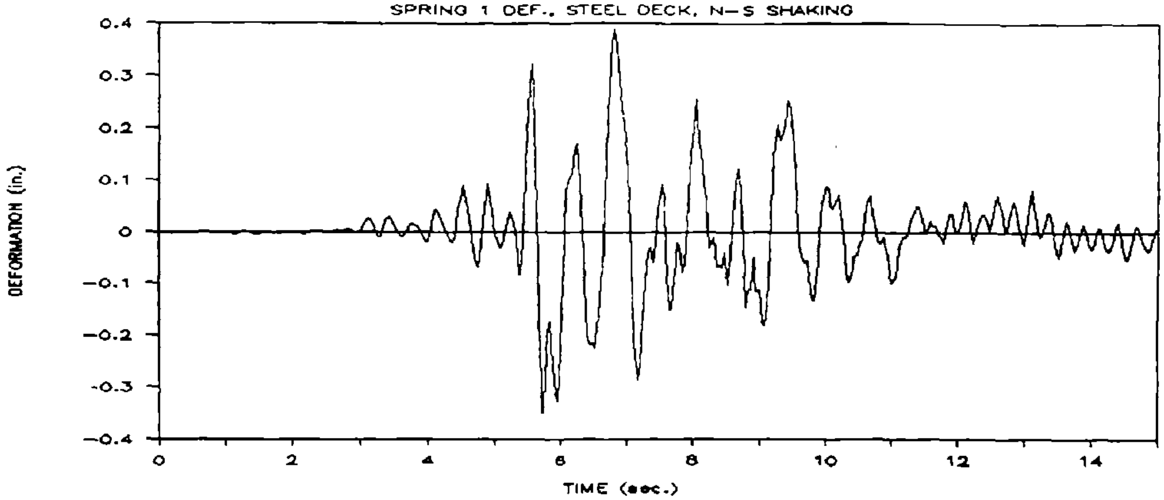
GROUND MOTION 6

SPRING 1 DEF., STEEL DECK, N–S SHAKING



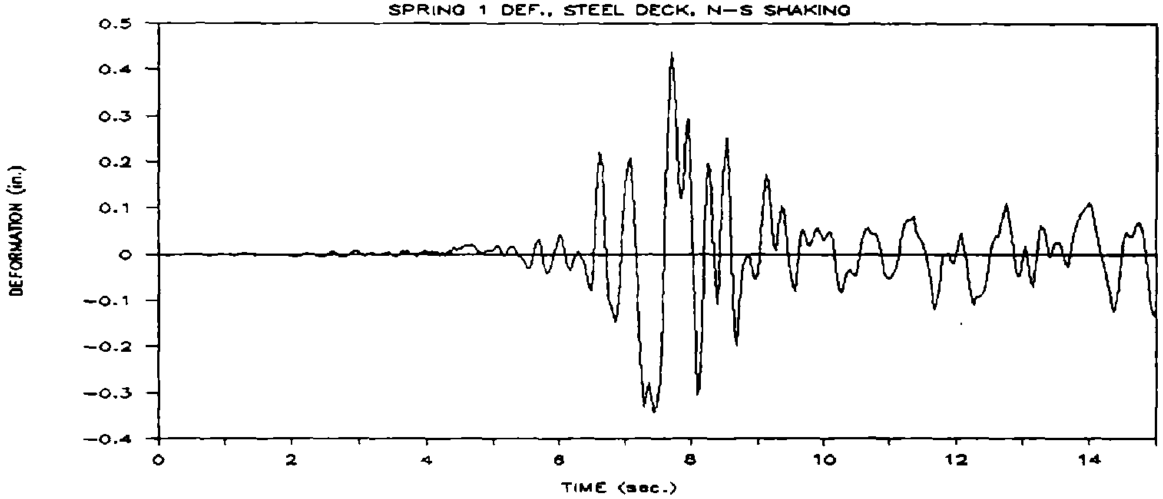
GROUND MOTION 7

SPRING 1 DEF., STEEL DECK, N-S SHAKING



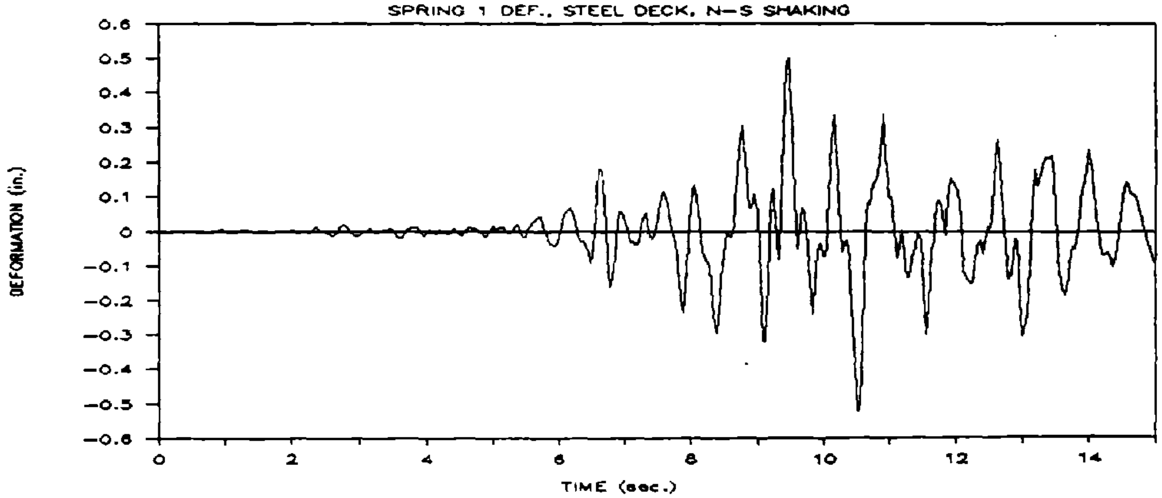
GROUND MOTION 8

SPRING 1 DEF., STEEL DECK, N-S SHAKING



GROUND MOTION 9

SPRING 1 DEF., STEEL DECK, N-S SHAKING



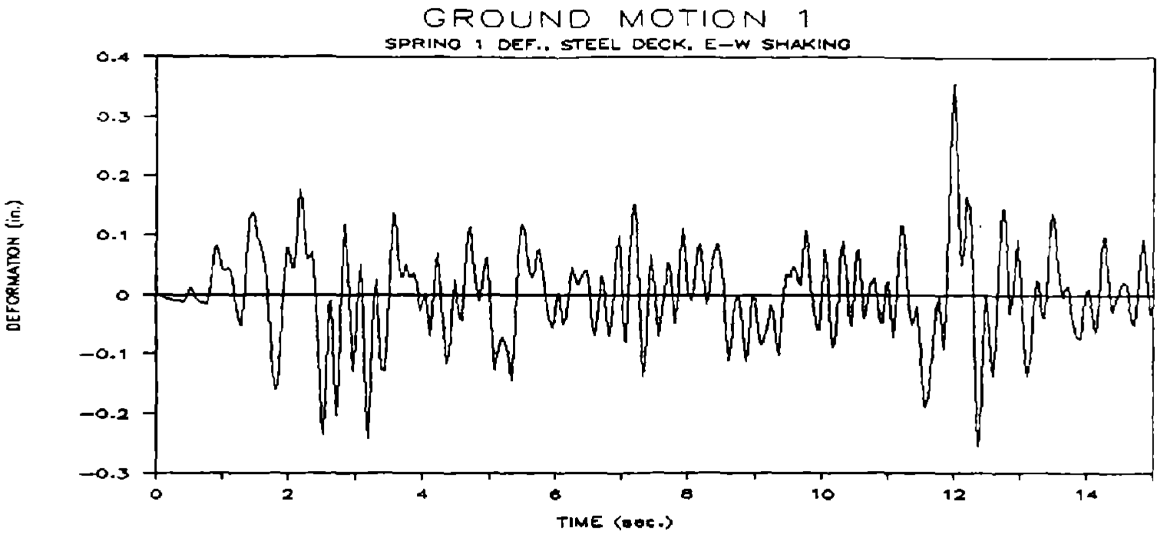
FIGURES GROUP 3-15

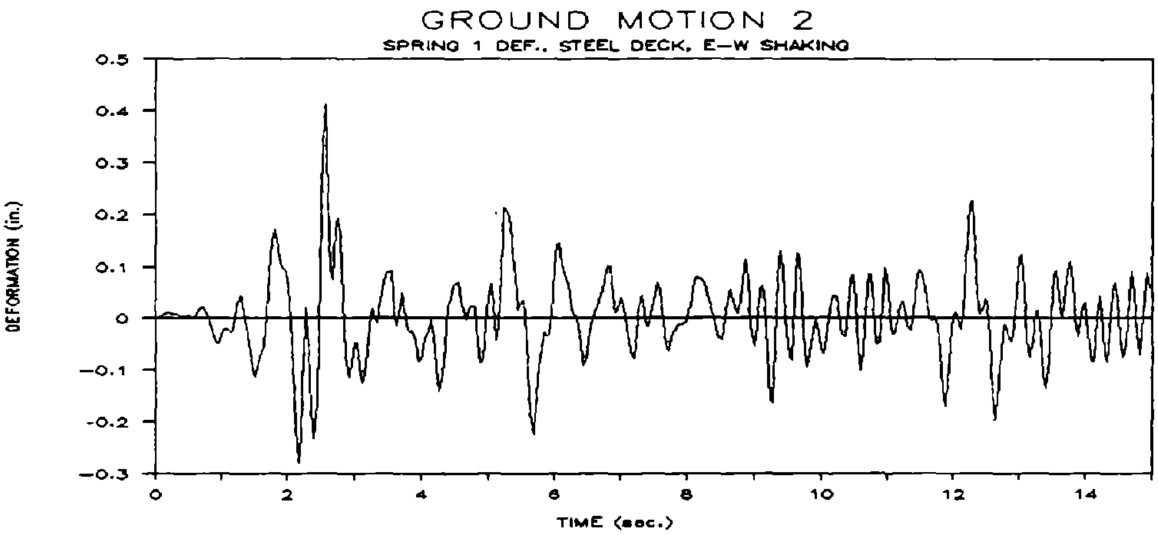
STEEL DECK DIAPHRAGM

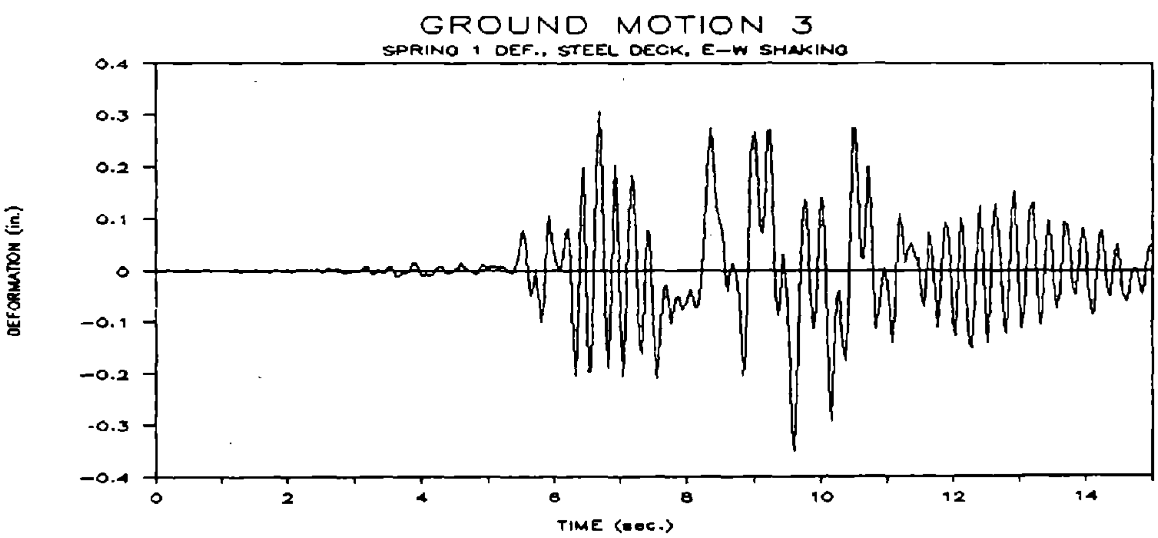
EAST-WEST SHAKING (STIFF)

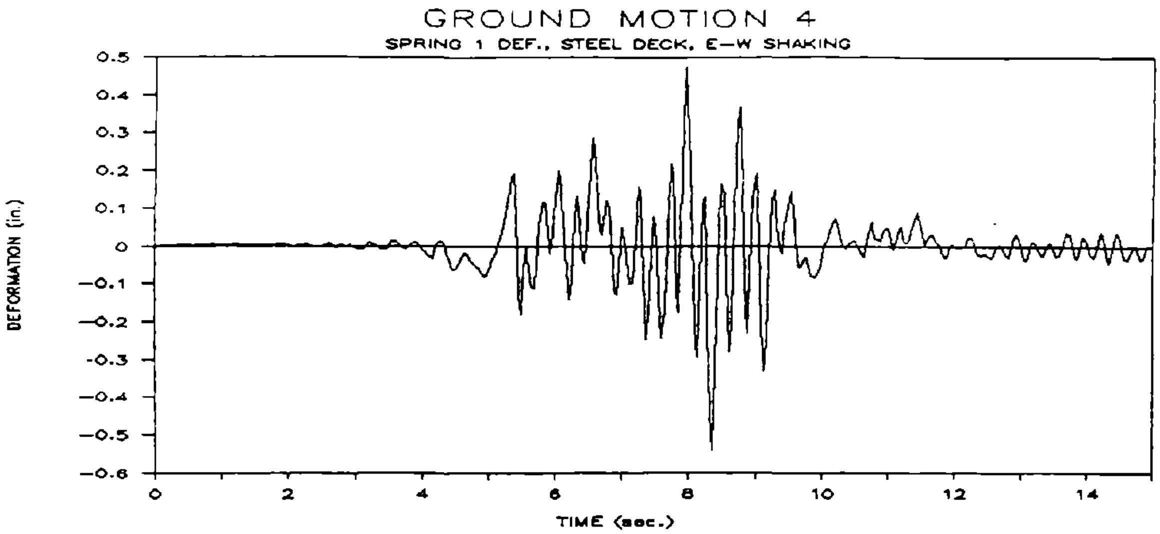
DEFORMATION OF SPRING (1),

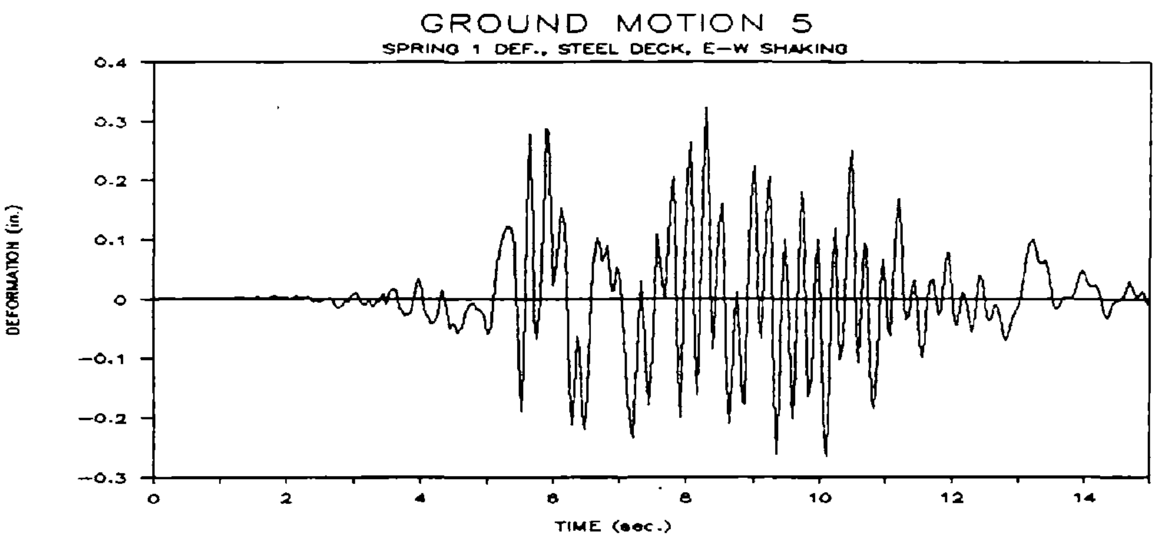
FIRST SEGMENT OF DIAPHRAGM

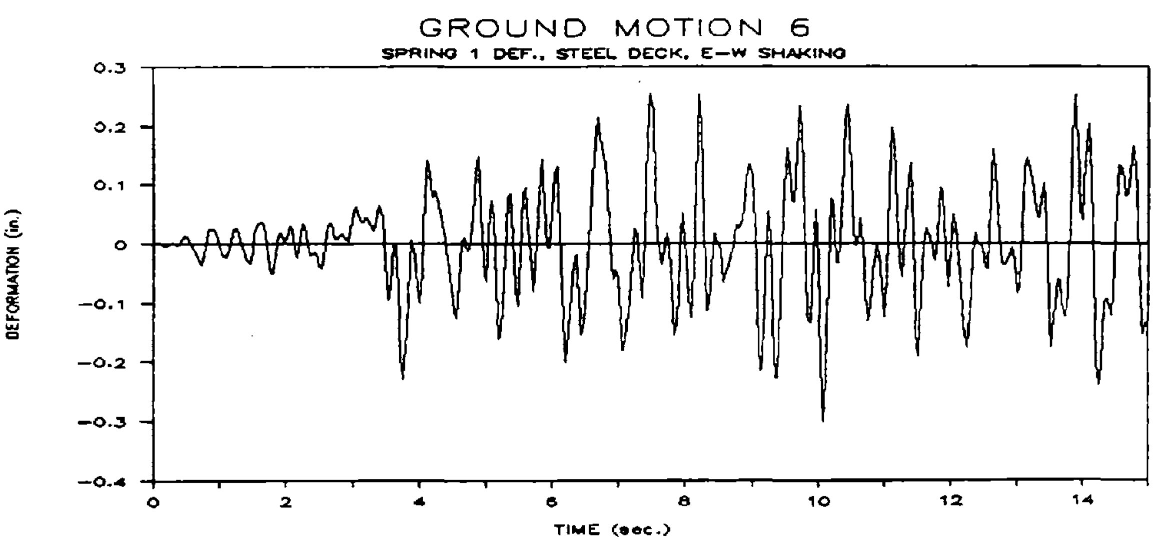

GROUND MOTION 1
SPRING 1 DEF., STEEL DECK, E—W SHAKING
DEFORMATION (in.)
0.4
0.3
0.2
0.1
0
−0.1
−0.2
−0.3
0
2
4
6
8
10
12
14
TIME (sec.)


GROUND MOTION 2
SPRING 1 DEF., STEEL DECK, E—W SHAKING
DEFORMATION (in.)
0.5
0.4
0.3
0.2
0.1
0
−0.1
-0.2
−0.3
0
2
4
6
8
10
12
14
TIME (sec.)


GROUND MOTION 3
SPRING 1 DEF., STEEL DECK, E—W SHAKING
DEFORMATION (in.)
0.4
0.3
0.2
0.1
0
−0.1
−0.2
-0.3
−0.4
0
2
4
6
8
10
12
14
TIME (sec.)

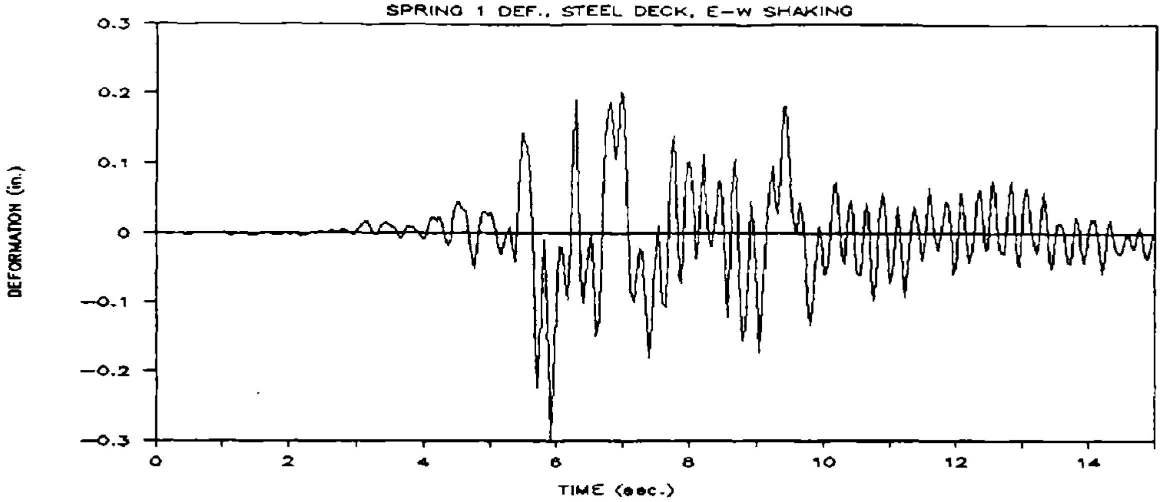

GROUND MOTION 4
SPRING 1 DEF., STEEL DECK, E-W SHAKING
DEFORMATION (in.)
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
0
2
4
6
8
10
12
14
TIME (sec.)


GROUND MOTION 5
SPRING 1 DEF., STEEL DECK, E-W SHAKING
DEFORMATION (in.)
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0
2
4
6
8
10
12
14
TIME (sec.)


GROUND MOTION 6
SPRING 1 DEF., STEEL DECK, E-W SHAKING
DEFORMATION (in.)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
0
2
4
6
8
10
12
14
TIME (sec.)

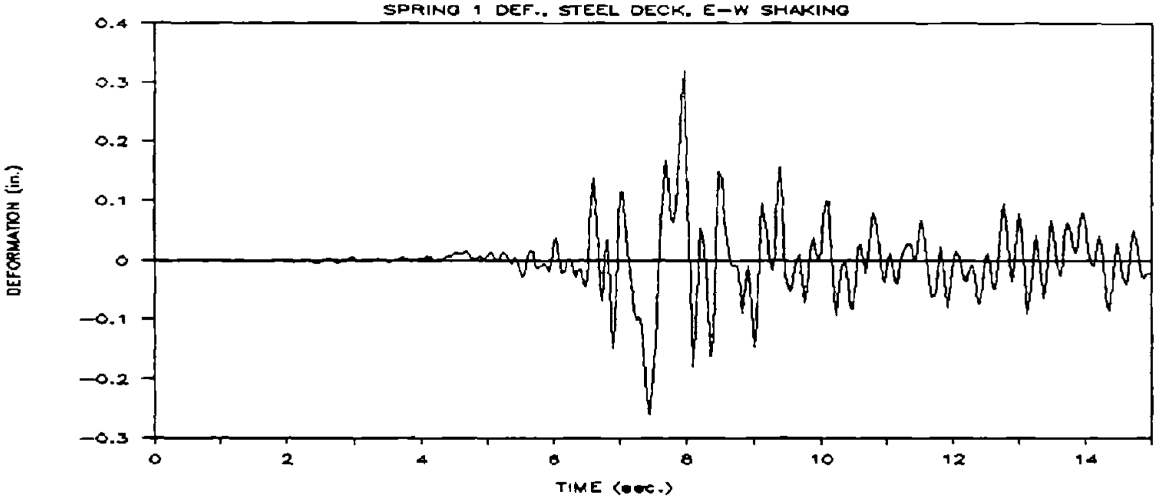
GROUND MOTION 7

SPRING 1 DEF., STEEL DECK, E–W SHAKING



GROUND MOTION 8

SPRING 1 DEF., STEEL DECK, E–W SHAKING



GROUND MOTION 9

SPRING 1 DEF., STEEL DECK, E–W SHAKING

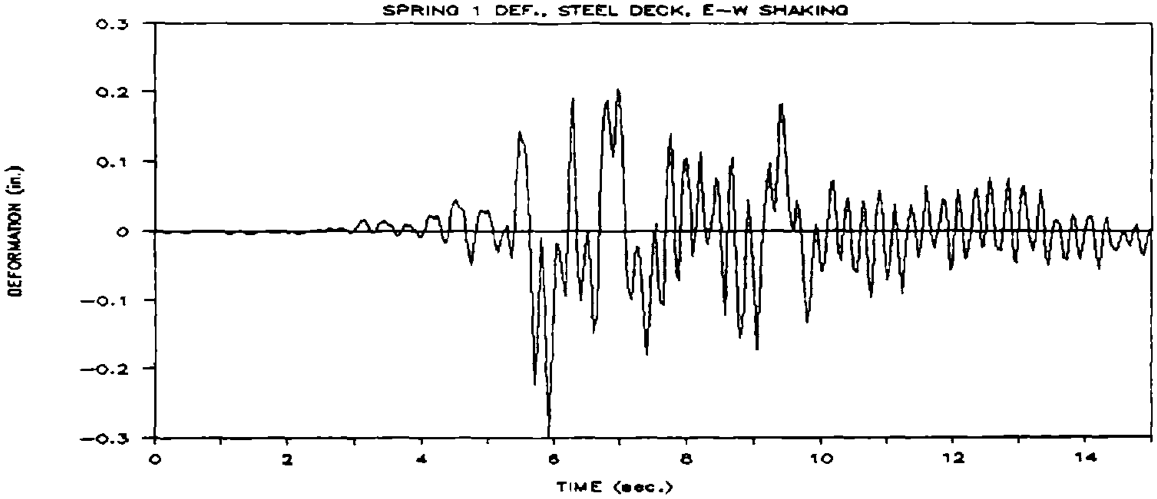


FIGURE GROUP 3-16

PLYWOOD DIAPHRAGM

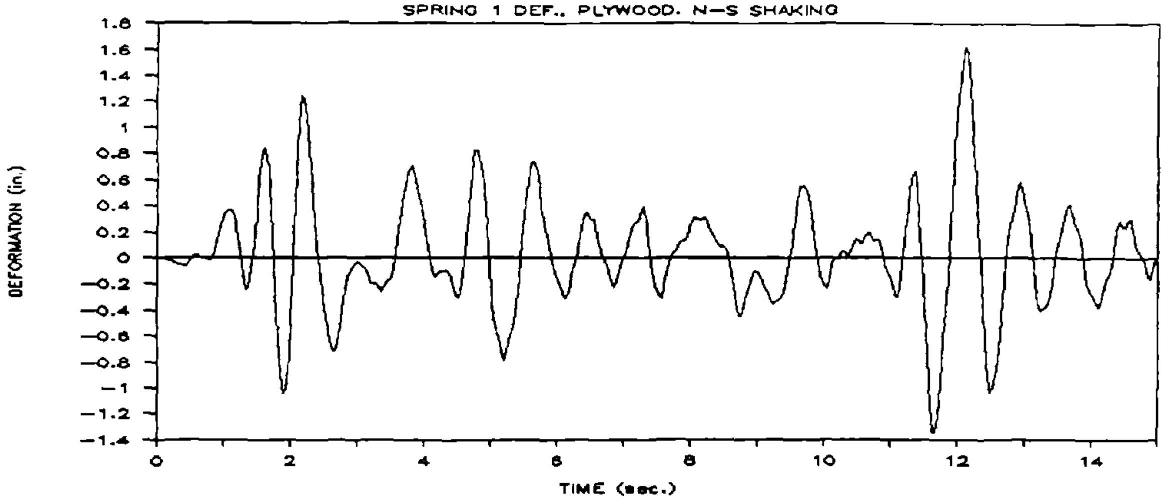
NORTH-SOUTH SHAKING (FLEXIBLE)

DEFORMATION OF SPRING (1),

FIRST SEGMENT OF DIAPHRAGM

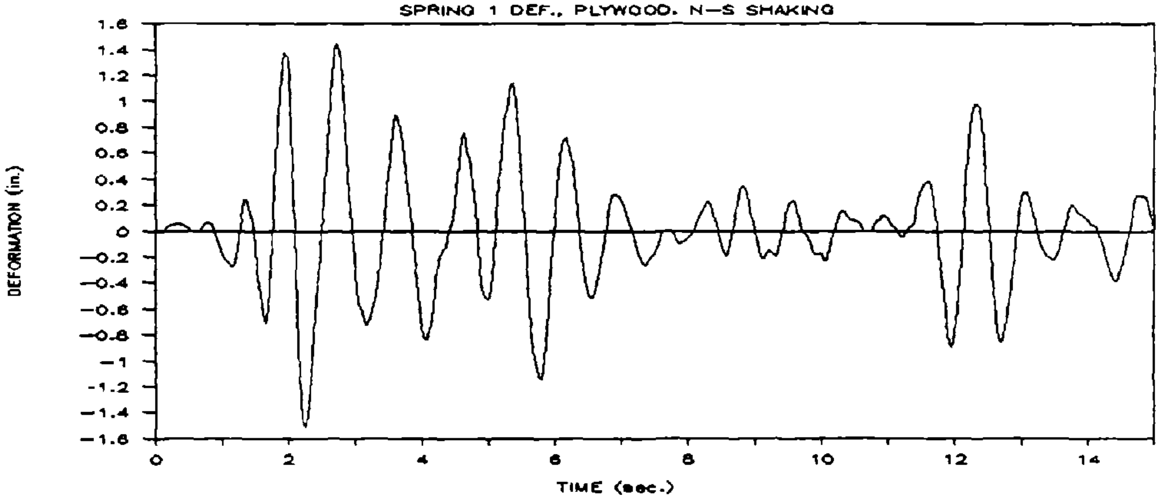
GROUND MOTION 1

SPRING 1 DEF., PLYWOOD. N—S SHAKING



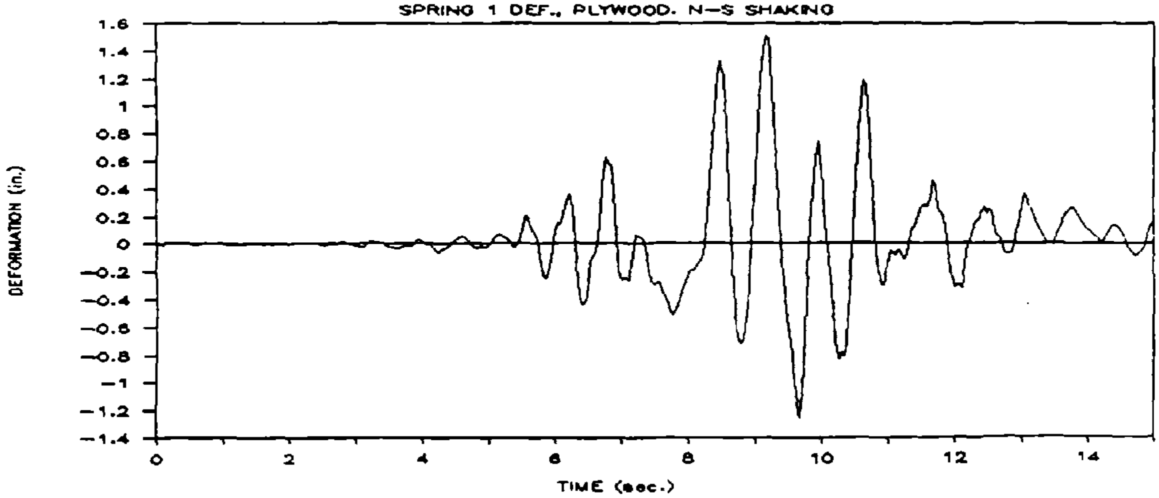
GROUND MOTION 2

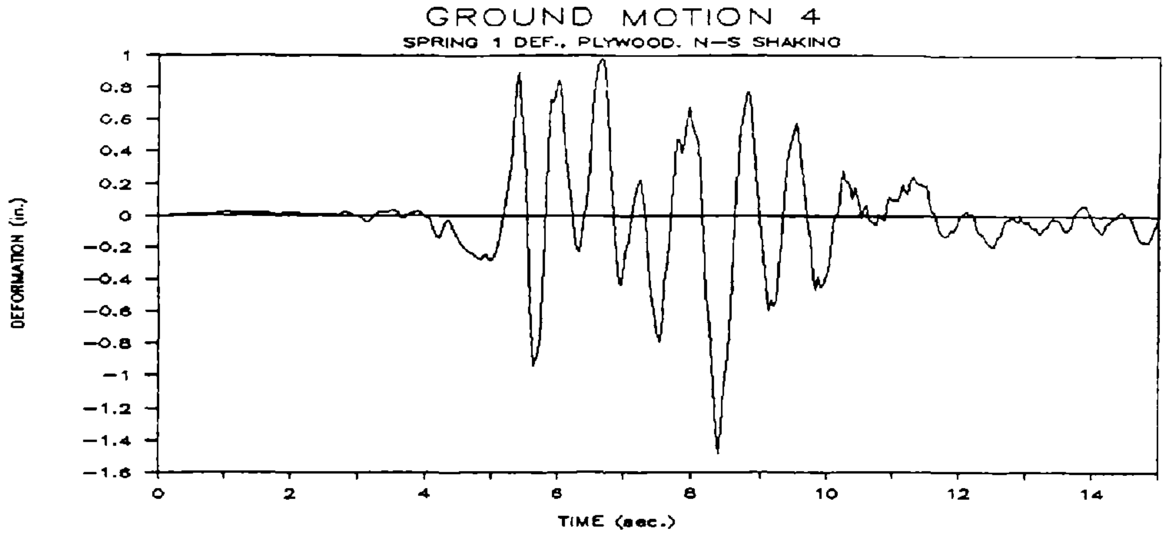
SPRING 1 DEF., PLYWOOD. N—S SHAKING

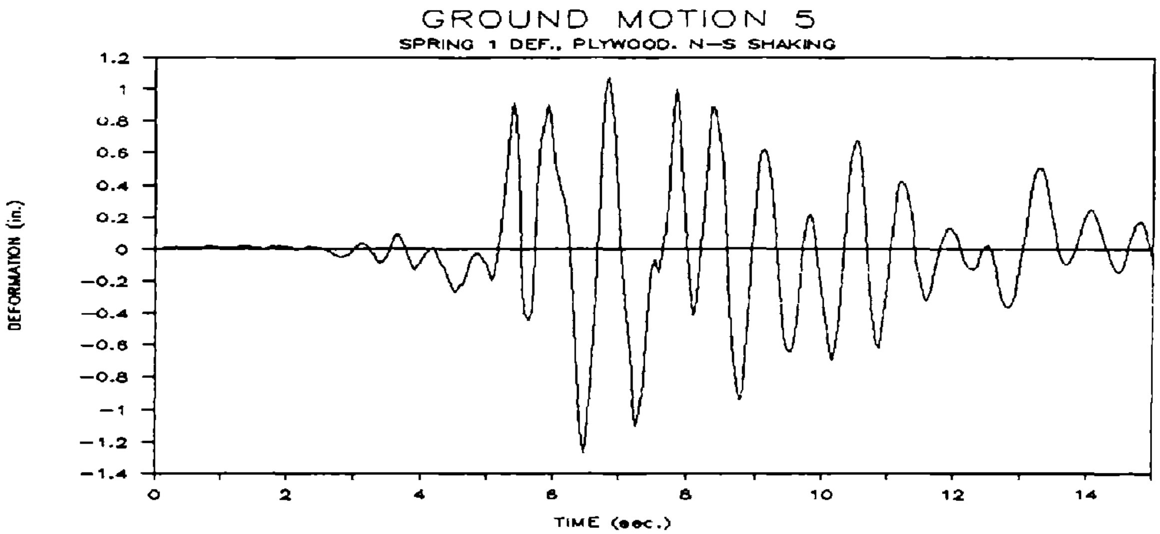


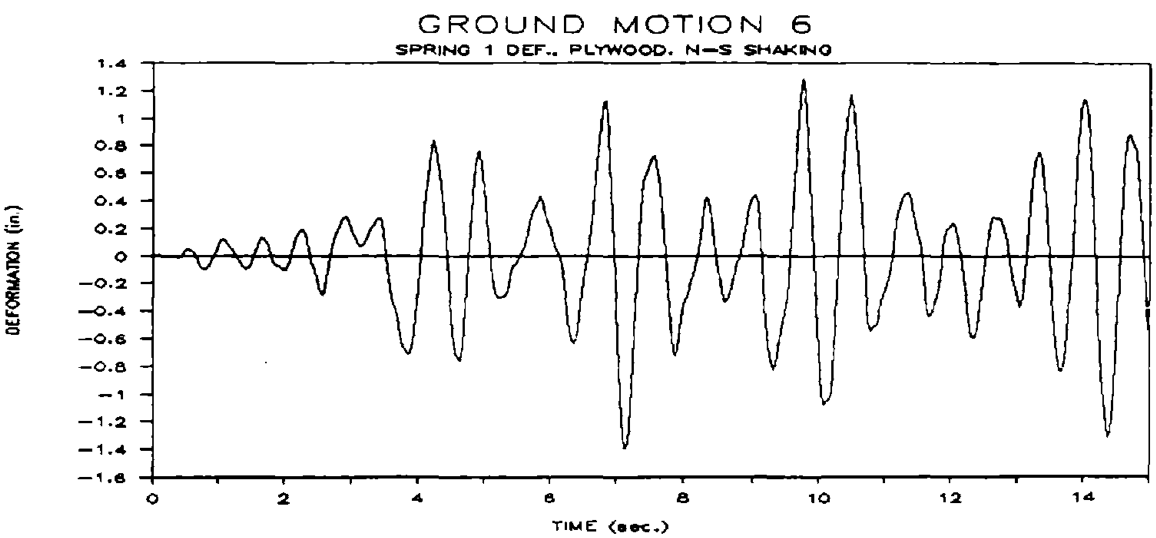
GROUND MOTION 3

SPRING 1 DEF., PLYWOOD. N—S SHAKING



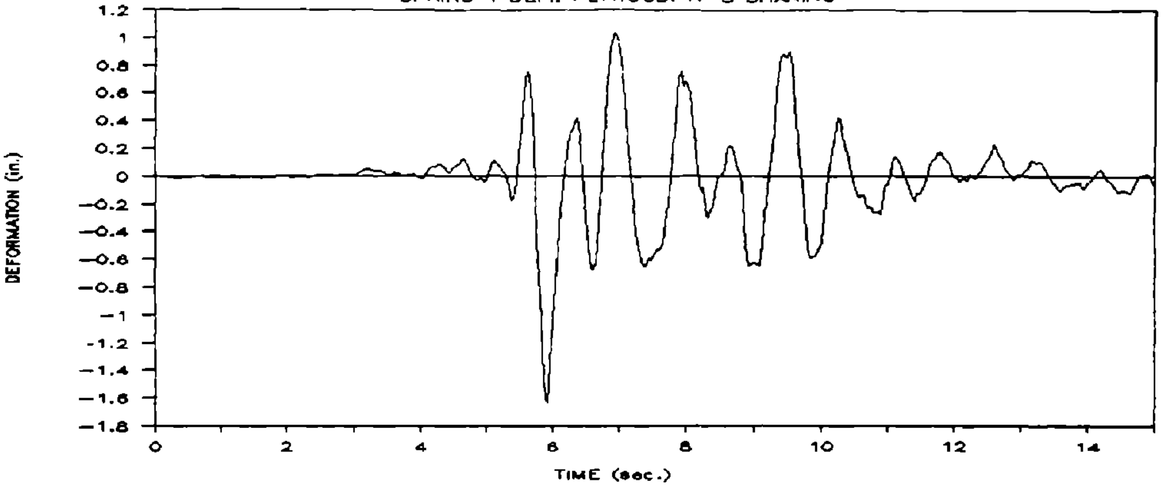

GROUND MOTION 4
SPRING 1 DEF., PLYWOOD. N—S SHAKING
DEFORMATION (in.)
1
0.8
0.6
0.4
0.2
0
−0.2
−0.4
−0.6
−0.8
−1
−1.2
−1.4
−1.6
0
2
4
6
8
10
12
14
TIME (sec.)


GROUND MOTION 5
SPRING 1 DEF., PLYWOOD. N—S SHAKING
DEFORMATION (in.)
1.2
1
0.8
0.6
0.4
0.2
0
−0.2
−0.4
−0.6
−0.8
−1
−1.2
−1.4
0
2
4
6
8
10
12
14
TIME (sec.)


GROUND MOTION 6
SPRING 1 DEF., PLYWOOD. N—S SHAKING
DEFORMATION (in.)
1.4
1.2
1
0.8
0.6
0.4
0.2
0
−0.2
−0.4
−0.6
−0.8
−1
−1.2
−1.4
−1.6
0
2
4
6
8
10
12
14
TIME (sec.)

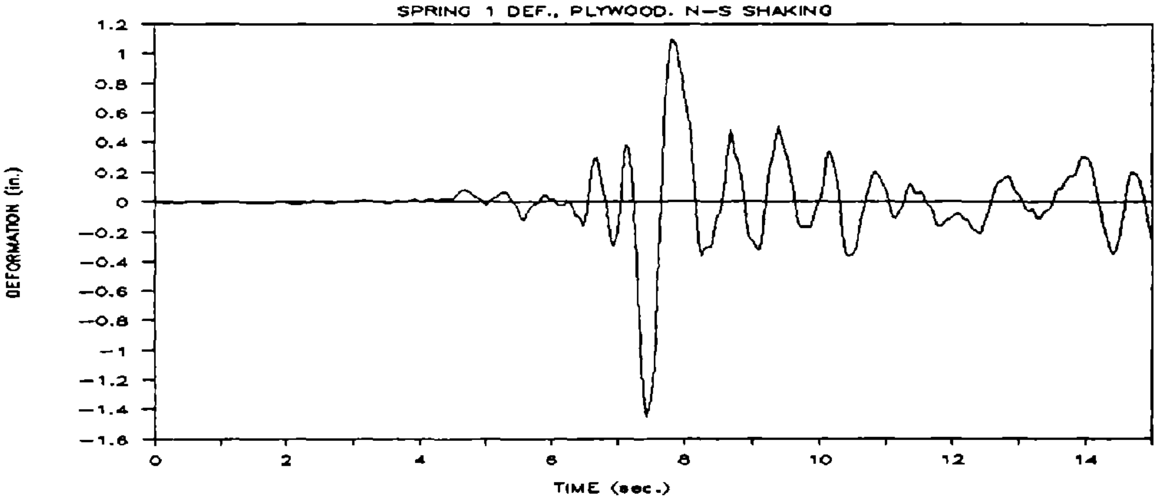
GROUND MOTION 7

SPRING 1 DEF., PLYWOOD. N—S SHAKING



GROUND MOTION 8

SPRING 1 DEF., PLYWOOD. N—S SHAKING



GROUND MOTION 9

SPRING 1 DEF., PLYWOOD. N—S SHAKING

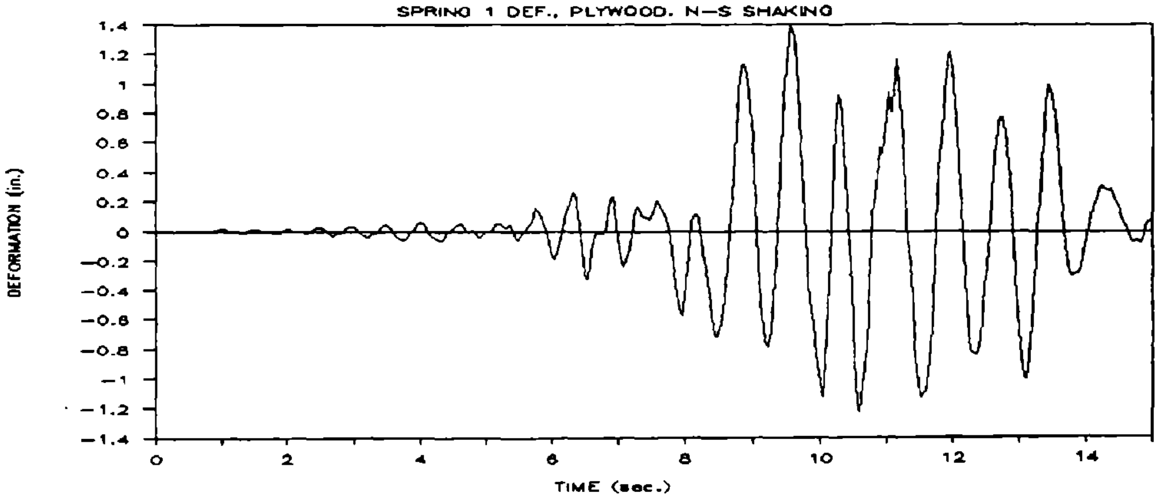


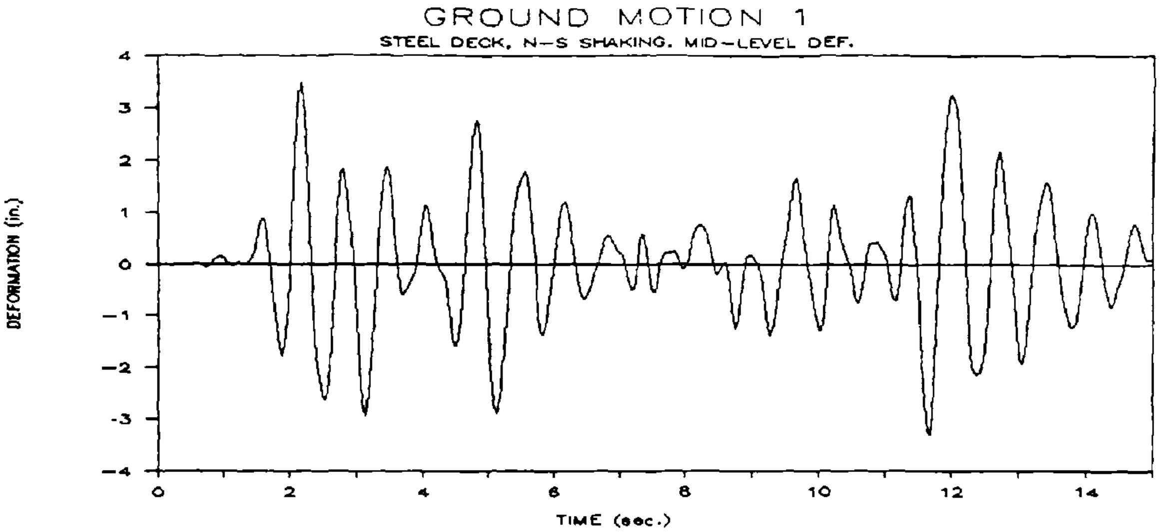
FIGURE GROUP 3-17

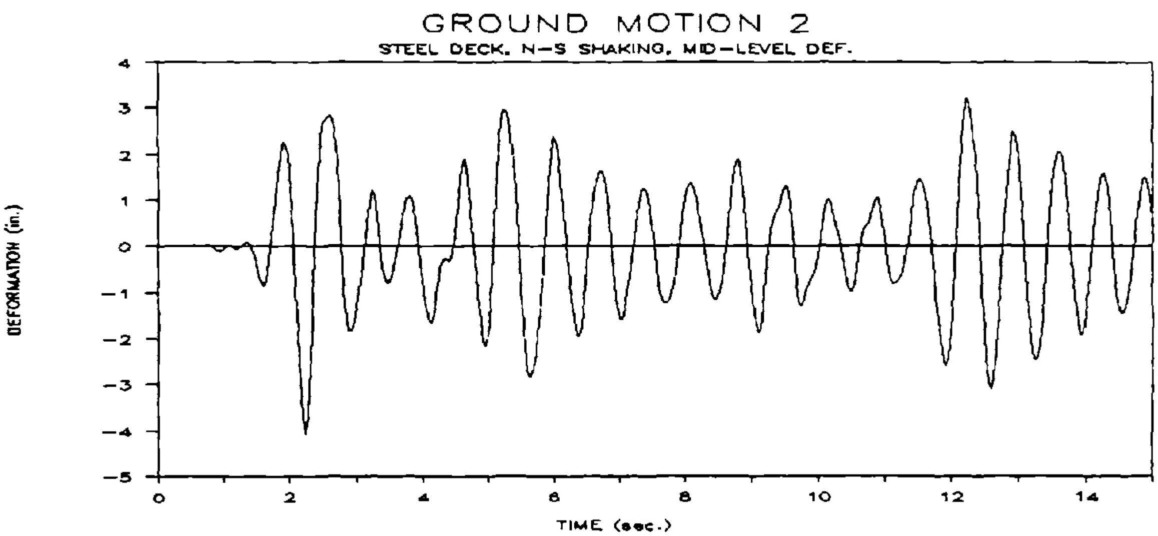
STEEL DECK DIAPHRAGM

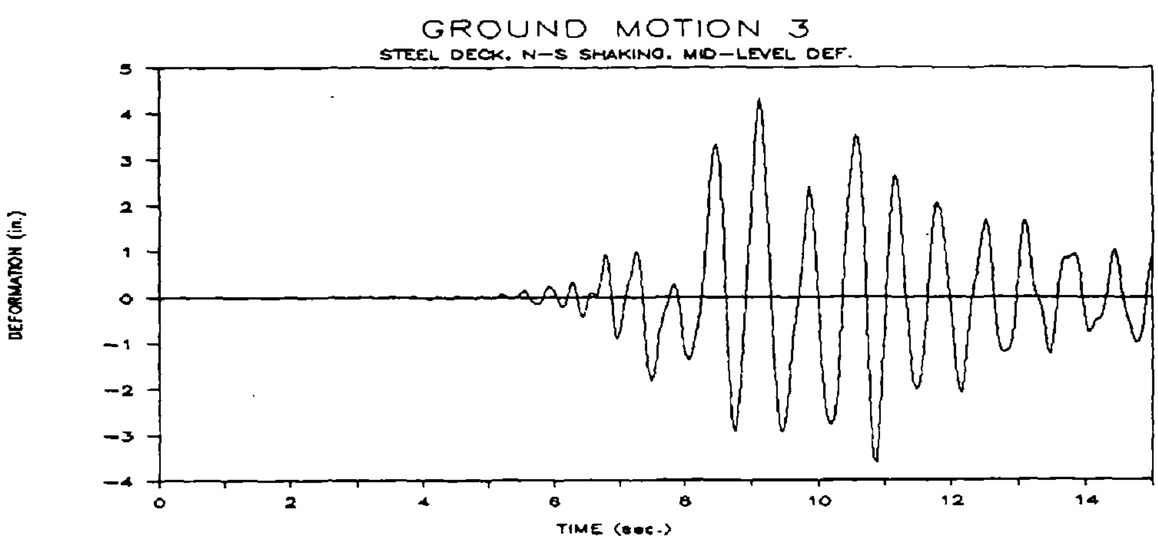
NORTH-SOUTH SHAKING (FLEXIBLE)

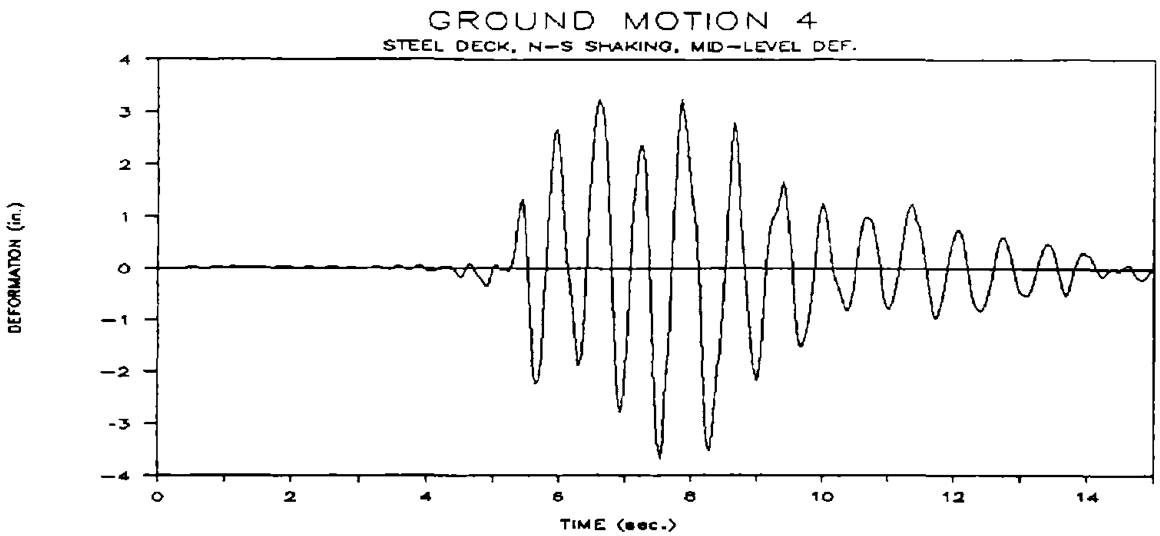
DEFORMATION OF CENTER OF 24 FEET

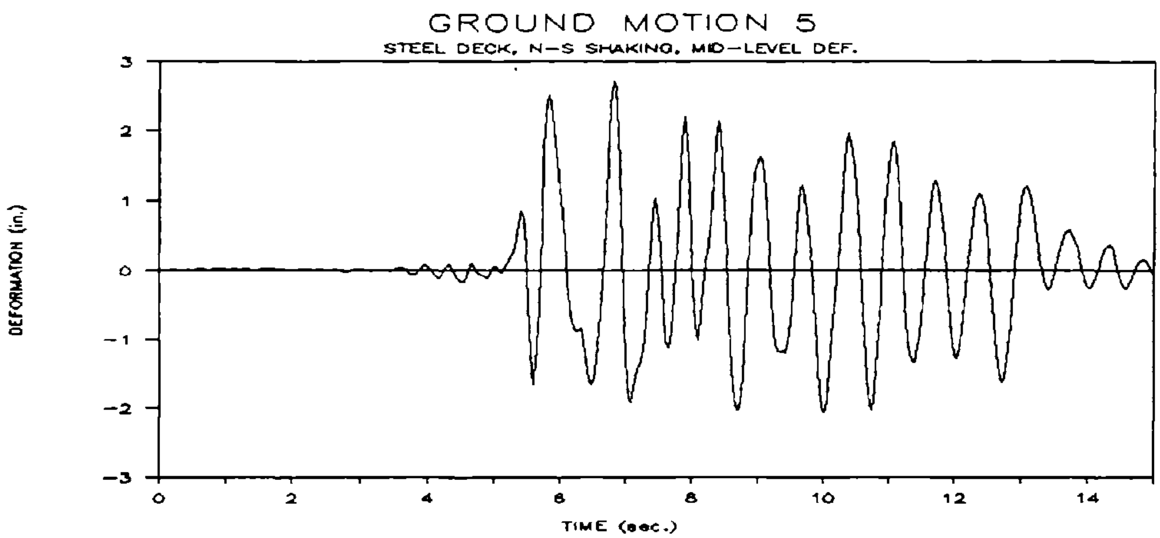
HIGH WALL RELATIVE TO ITS ENDS

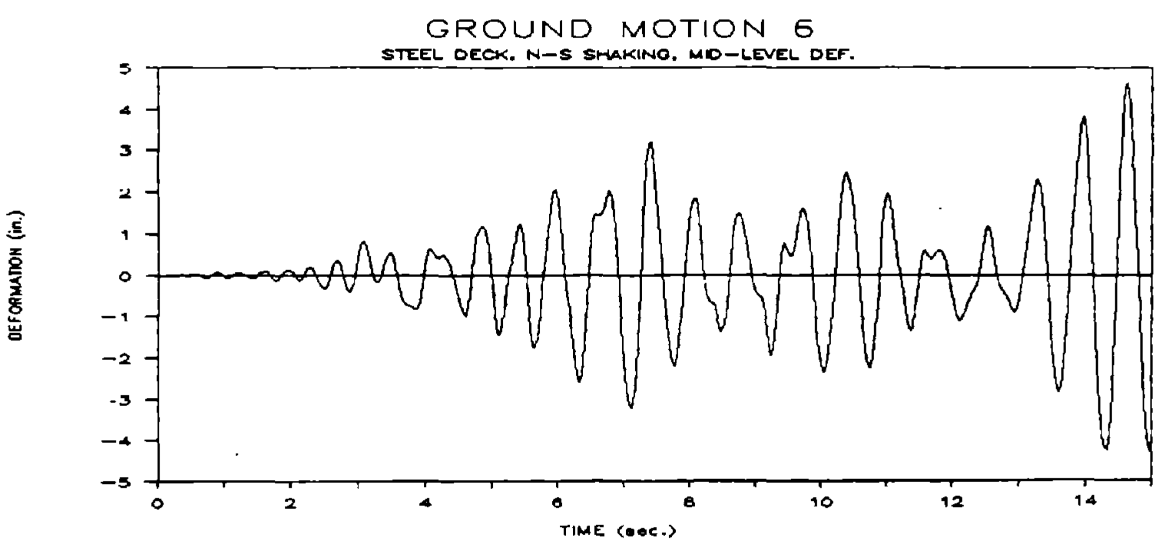

GROUND MOTION 1
STEEL DECK, N-S SHAKING. MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)

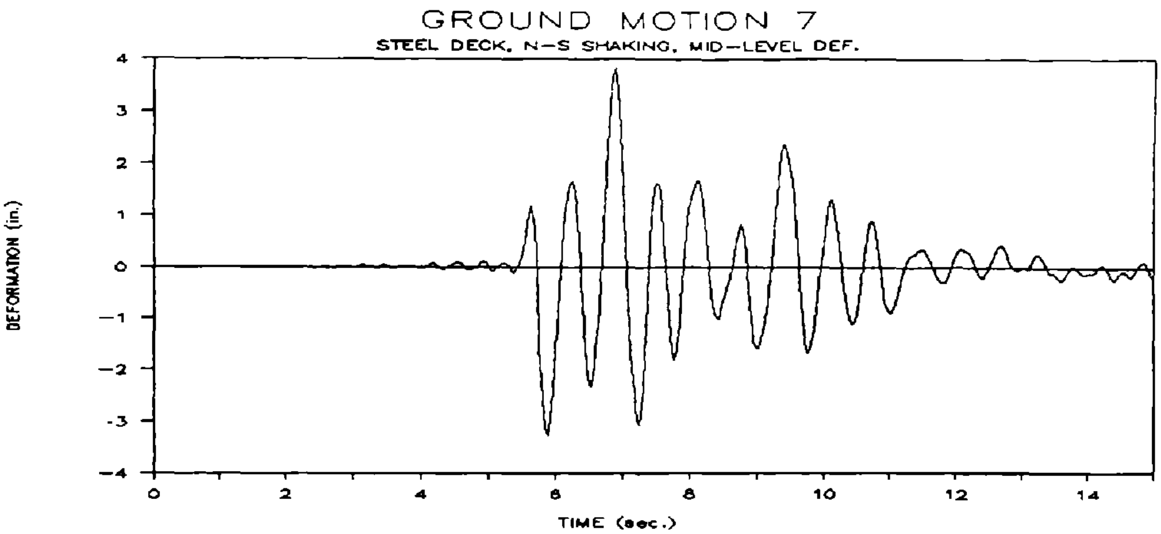

GROUND MOTION 2
STEEL DECK, N-S SHAKING. MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)

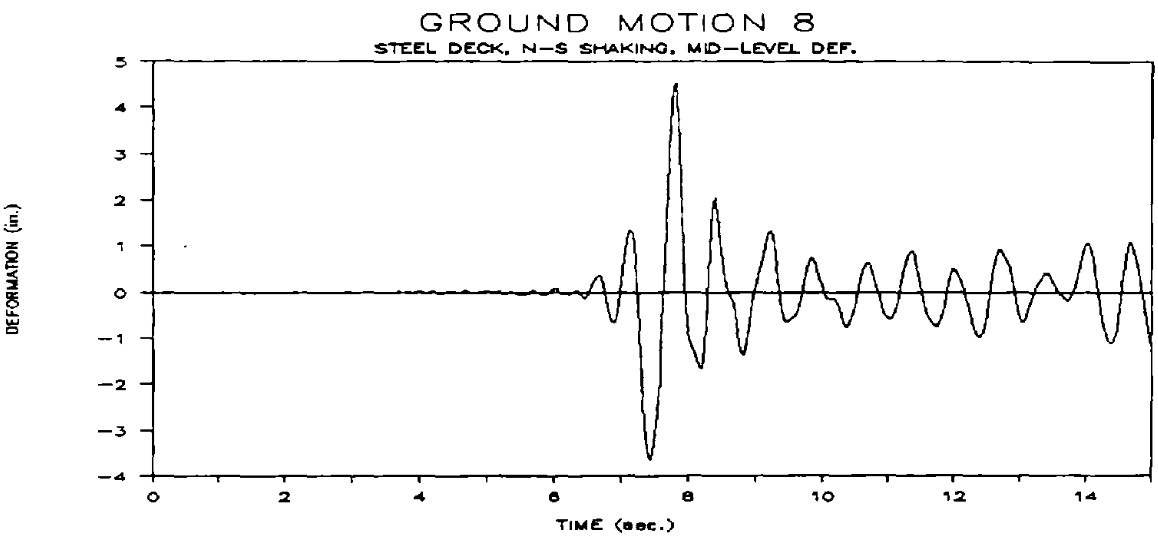

GROUND MOTION 3
STEEL DECK, N-S SHAKING. MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)


GROUND MOTION 4
STEEL DECK, N-S SHAKING, MID-LEVEL DEF.
DEFORMATON (in.)
TIME (sec.)


GROUND MOTION 5
STEEL DECK, N-S SHAKING, MID-LEVEL DEF.
DEFORMATON (in.)
TIME (sec.)


GROUND MOTION 6
STEEL DECK, N-S SHAKING, MID-LEVEL DEF.
DEFORMATON (in.)
TIME (sec.)


GROUND MOTION 7
STEEL DECK, N-S SHAKING, MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)


GROUND MOTION 8
STEEL DECK, N-S SHAKING, MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)

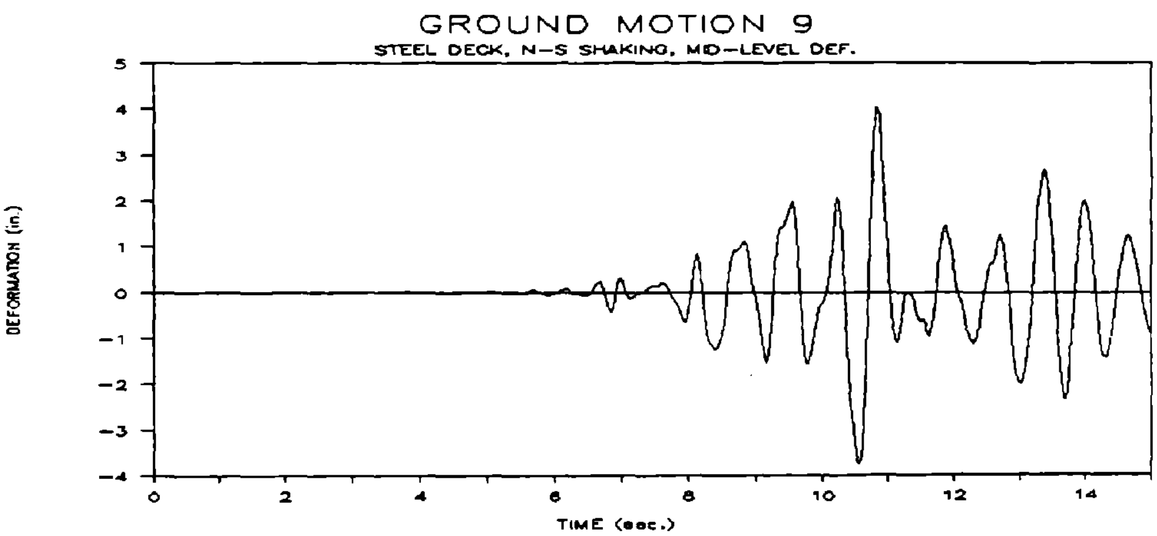

GROUND MOTION 9
STEEL DECK, N-S SHAKING, MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)

FIGURE GROUP 3-18

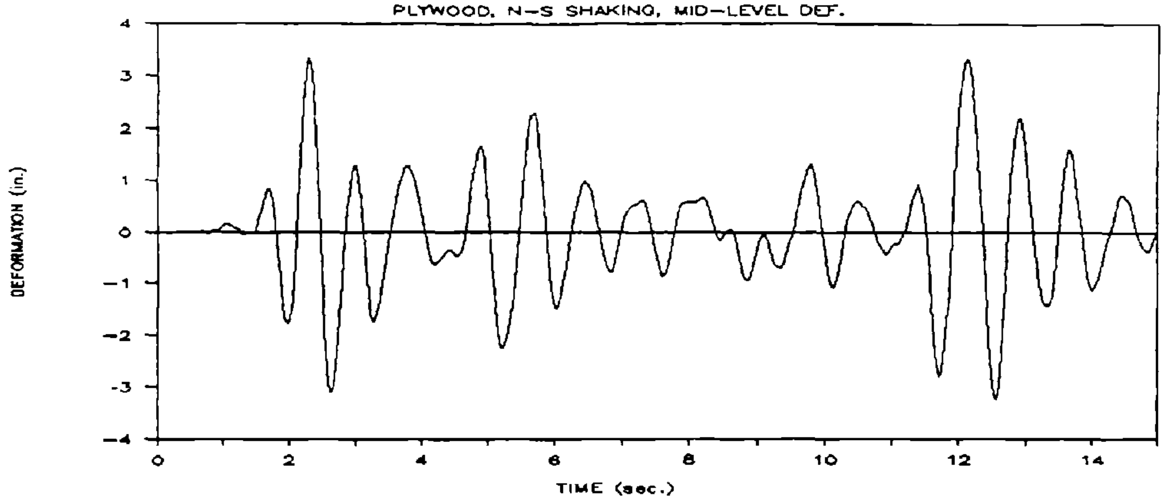
PLYWOOD DIAPHRAGM

NORTH-SOUTH SHAKING (FLEXIBLE)

DEFORMATION OF CENTER OF 24 FEET

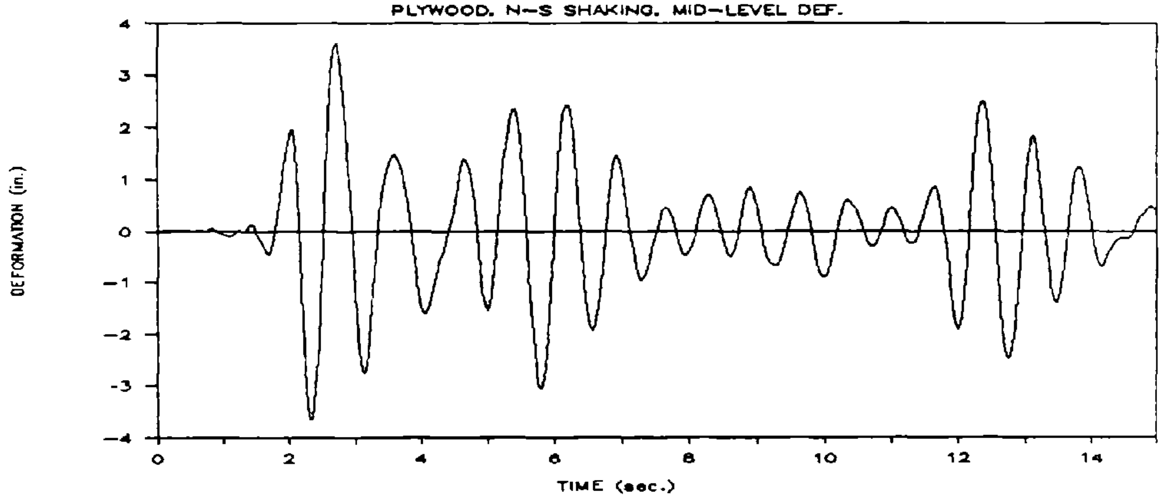
GROUND MOTION 1

PLYWOOD, N-S SHAKING, MID-LEVEL DEF.



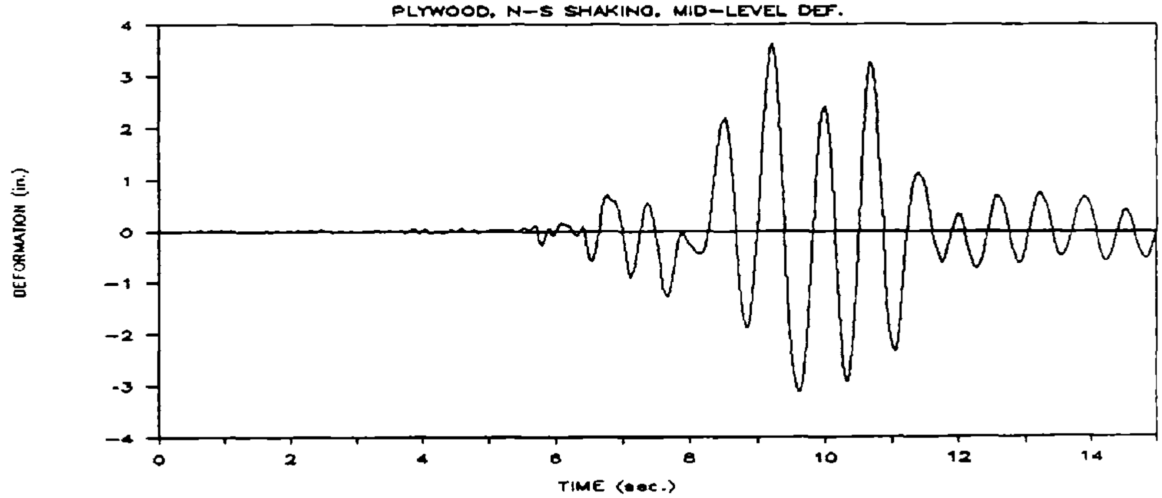
GROUND MOTION 2

PLYWOOD, N-S SHAKING, MID-LEVEL DEF.

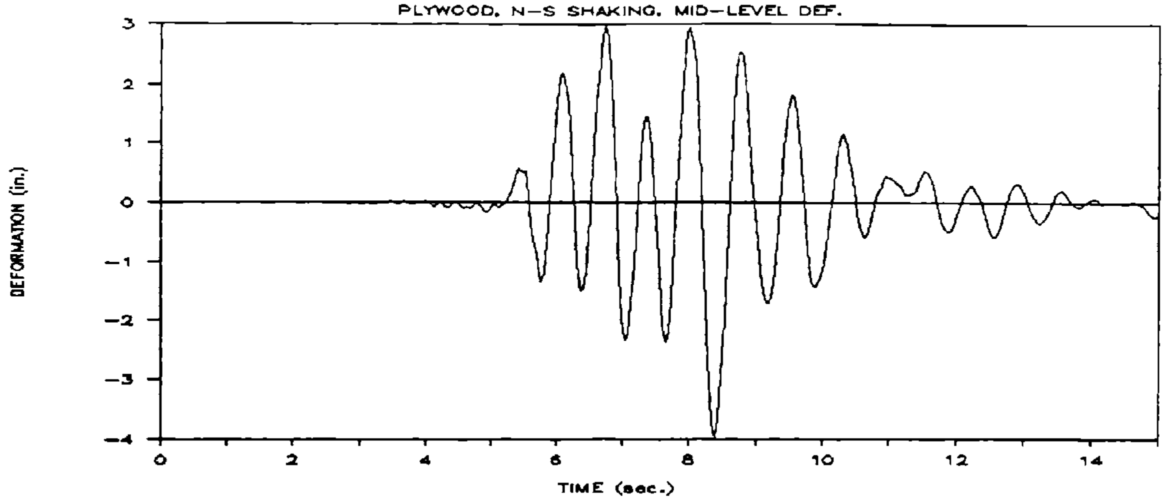


GROUND MOTION 3

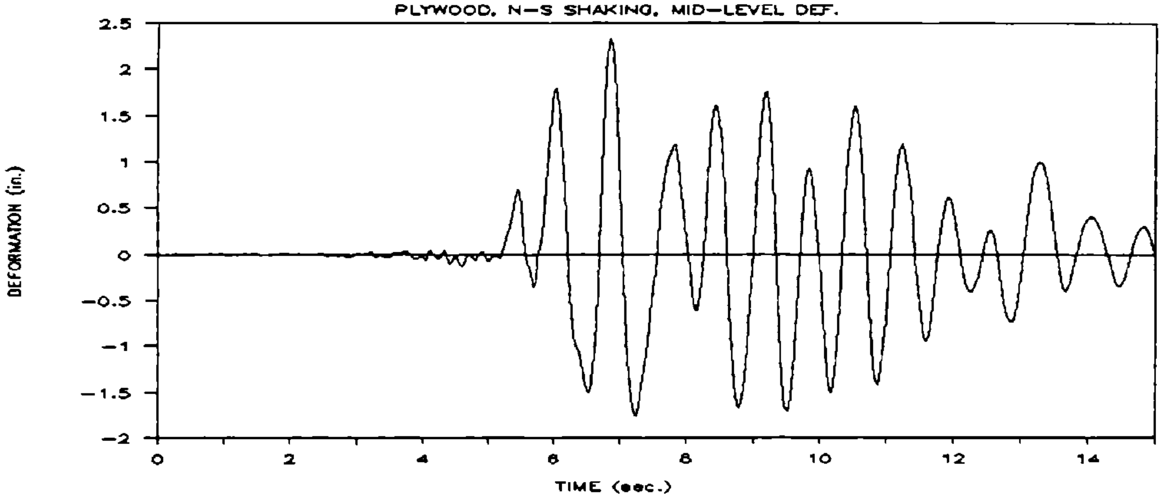
PLYWOOD, N-S SHAKING, MID-LEVEL DEF.



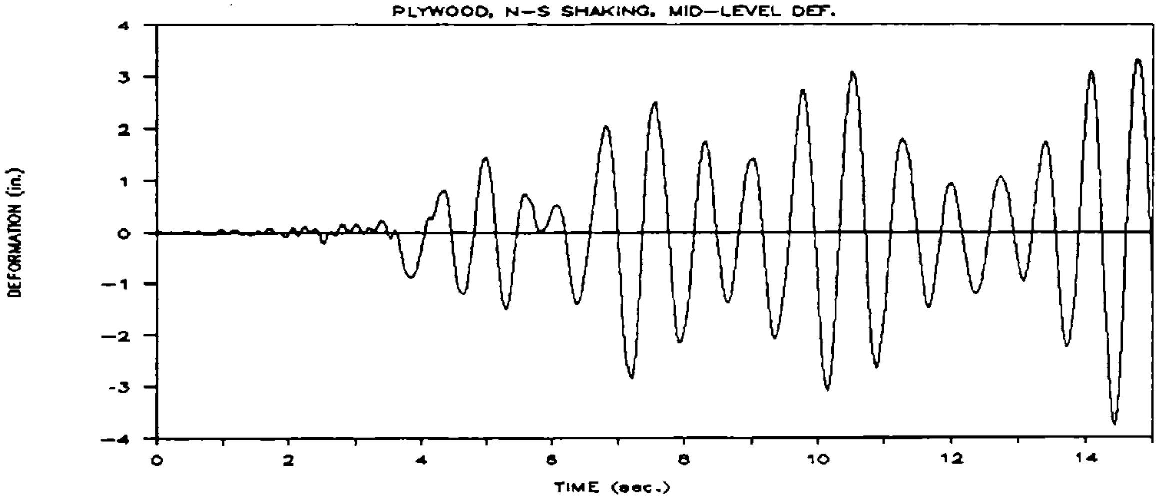
GROUND MOTION 4
PLYWOOD, N-S SHAKING, MID-LEVEL DEF.

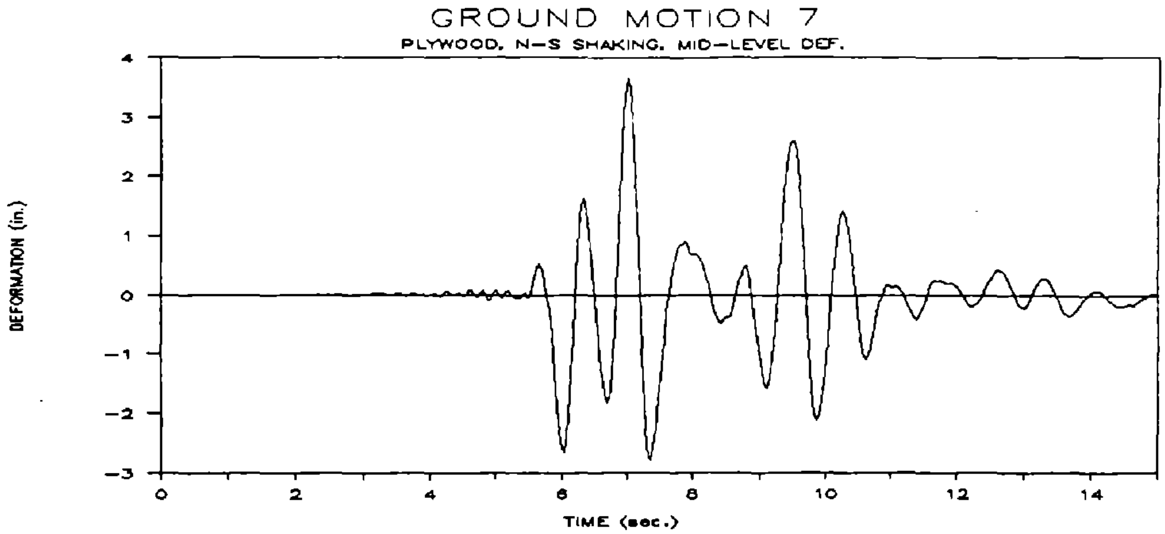


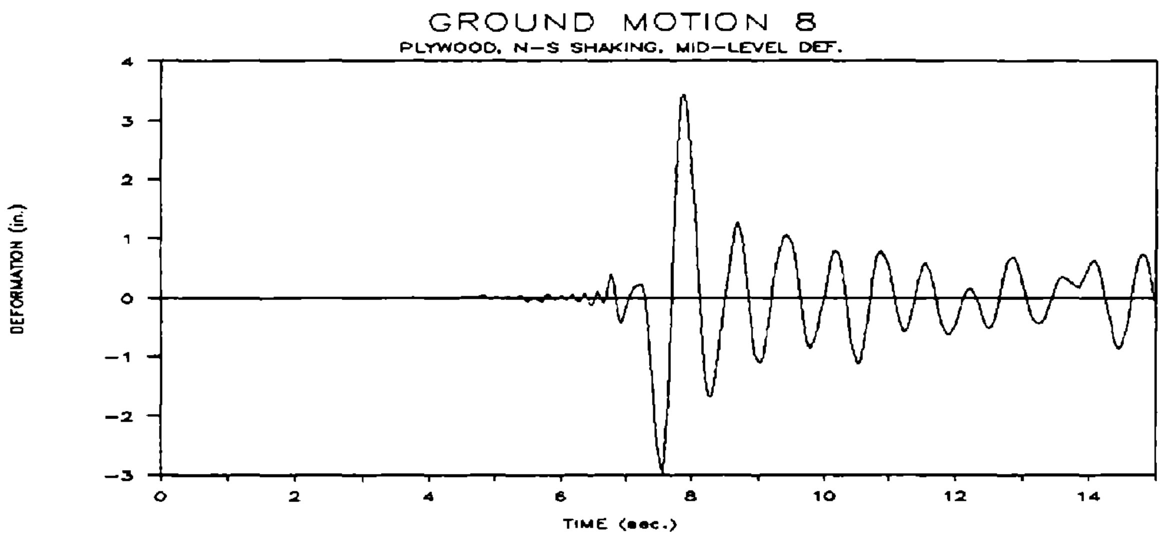
GROUND MOTION 5
PLYWOOD, N-S SHAKING, MID-LEVEL DEF.



GROUND MOTION 6
PLYWOOD, N-S SHAKING, MID-LEVEL DEF.




GROUND MOTION 7
PLYWOOD, N-S SHAKING, MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)


GROUND MOTION 8
PLYWOOD, N-S SHAKING, MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)

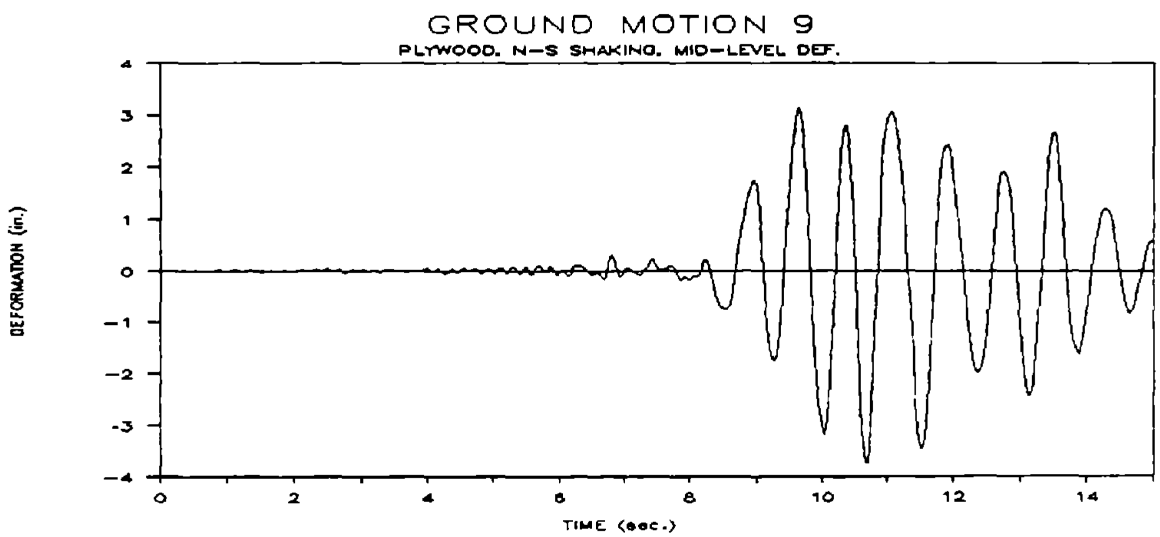

GROUND MOTION 9
PLYWOOD, N-S SHAKING, MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)

FIGURE GROUP 3-19

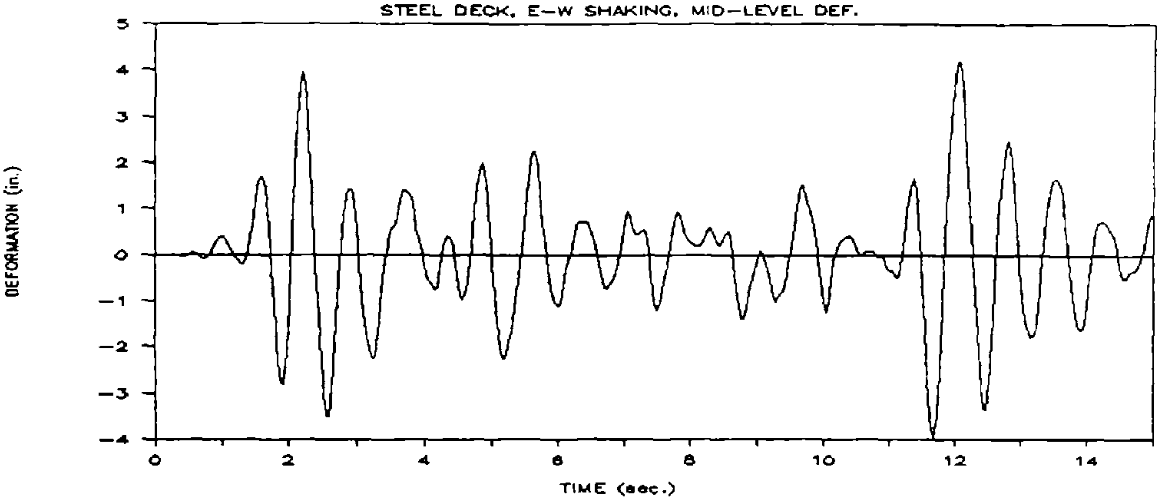
STEEL DECK DIAPHRAGM

EAST-WEST SHAKING (STIFF)

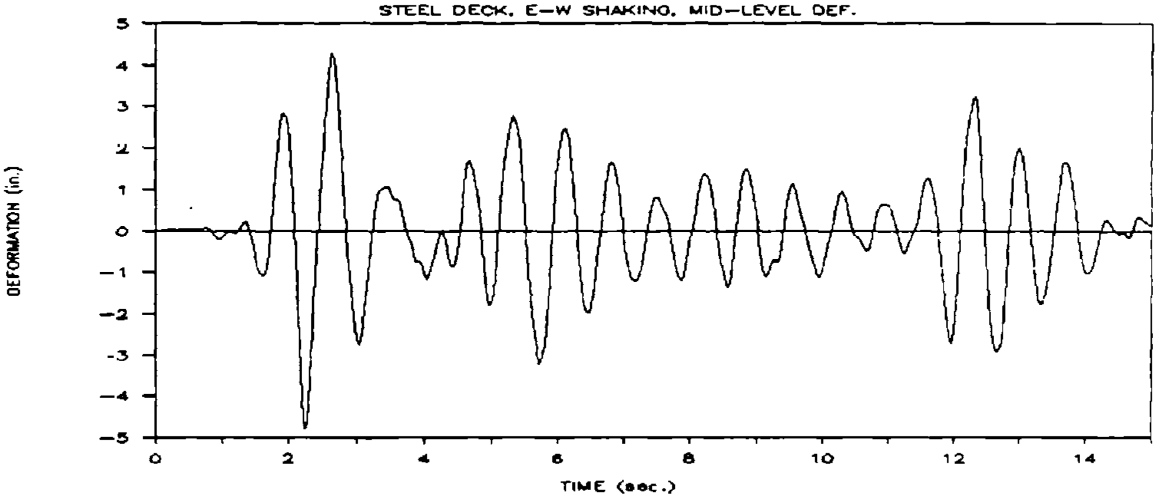
DEFORMATION OF FOURTH NODE ABOVE GRADE

OF 28 FEET HIGH WALL RELATIVE TO ITS ENDS

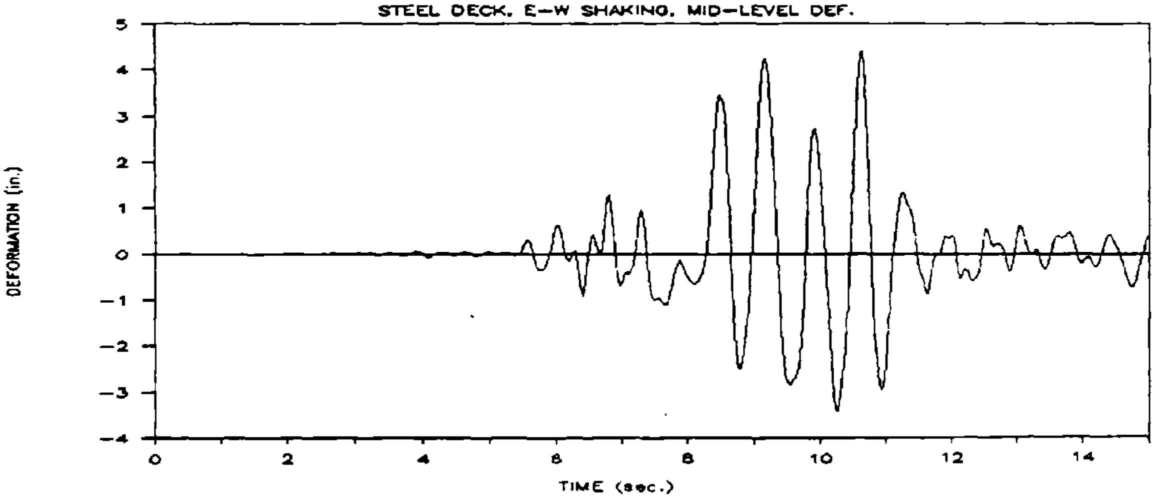
GROUND MOTION 1



GROUND MOTION 2

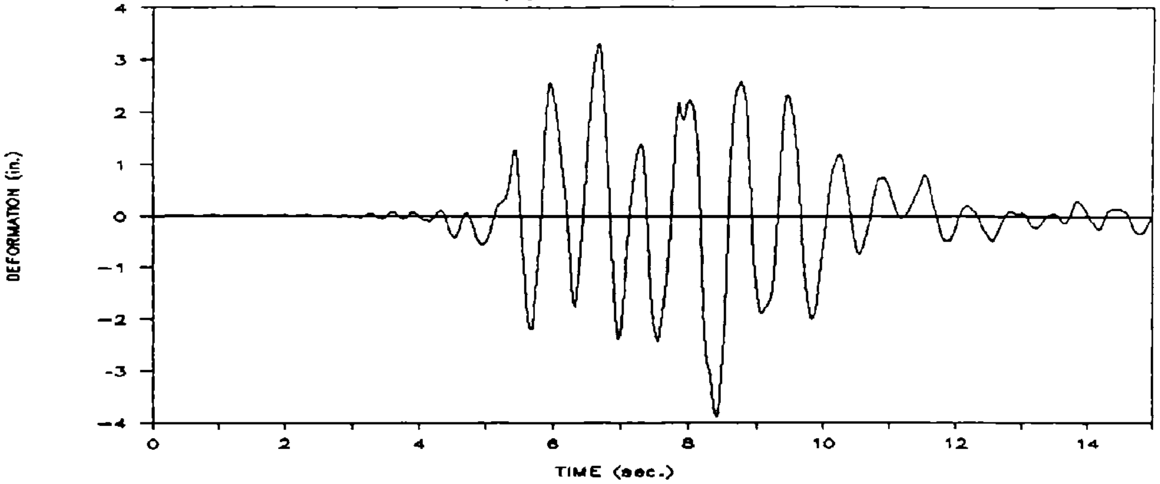


GROUND MOTION 3



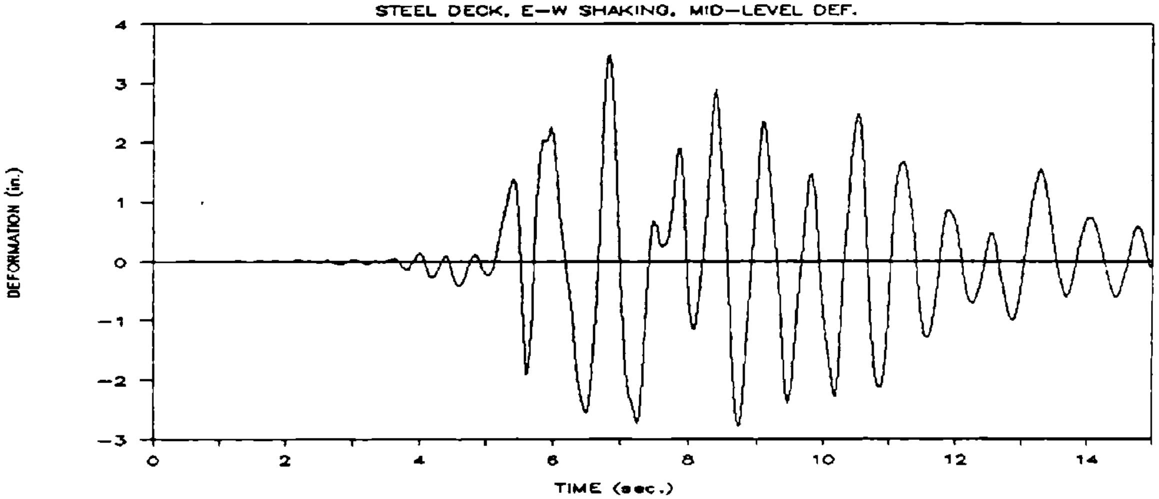
GROUND MOTION 4

STEEL DECK, E-W SHAKING, MID-LEVEL DEF.



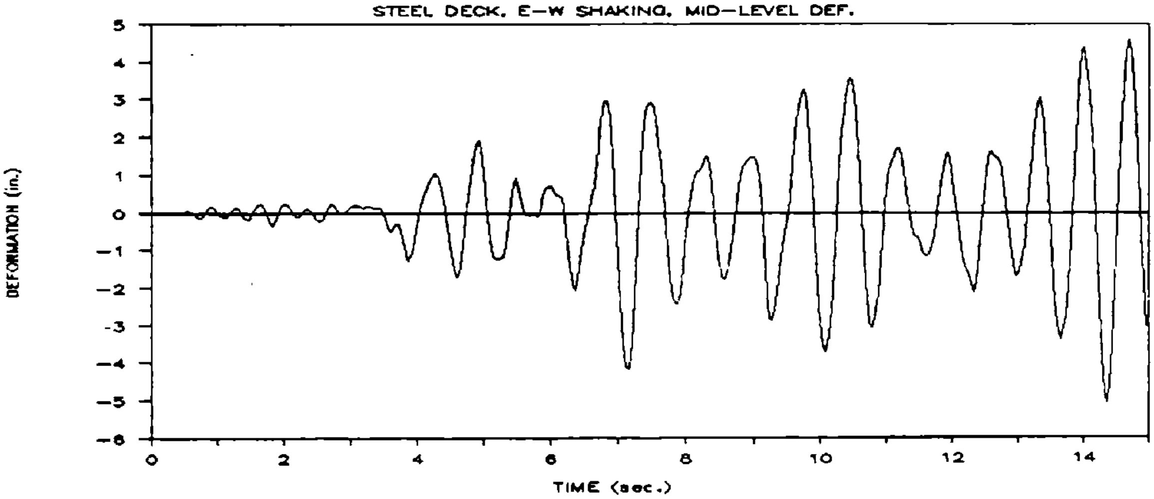
GROUND MOTION 5

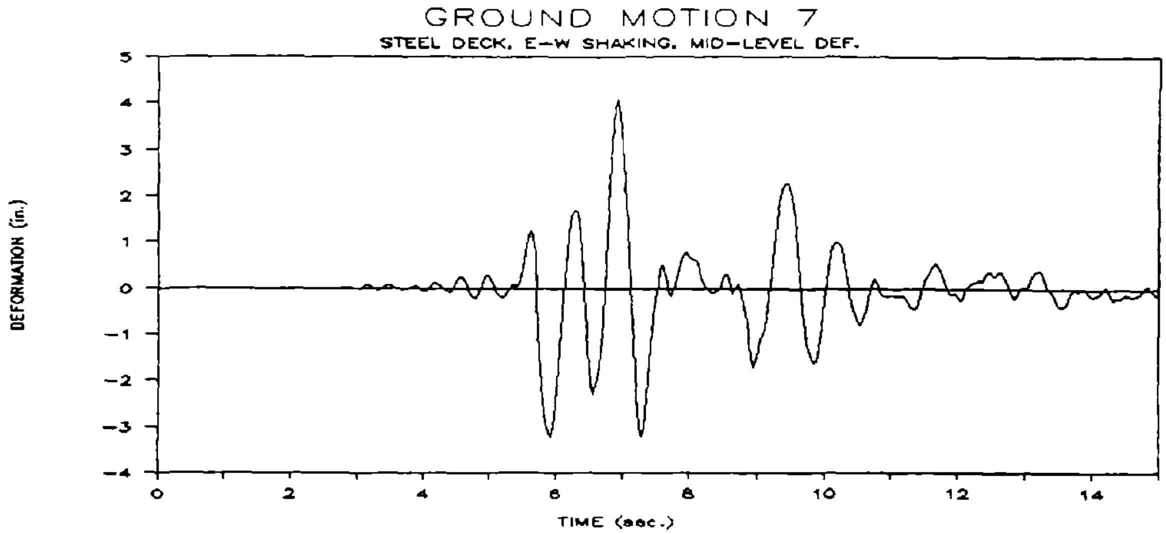
STEEL DECK, E-W SHAKING, MID-LEVEL DEF.

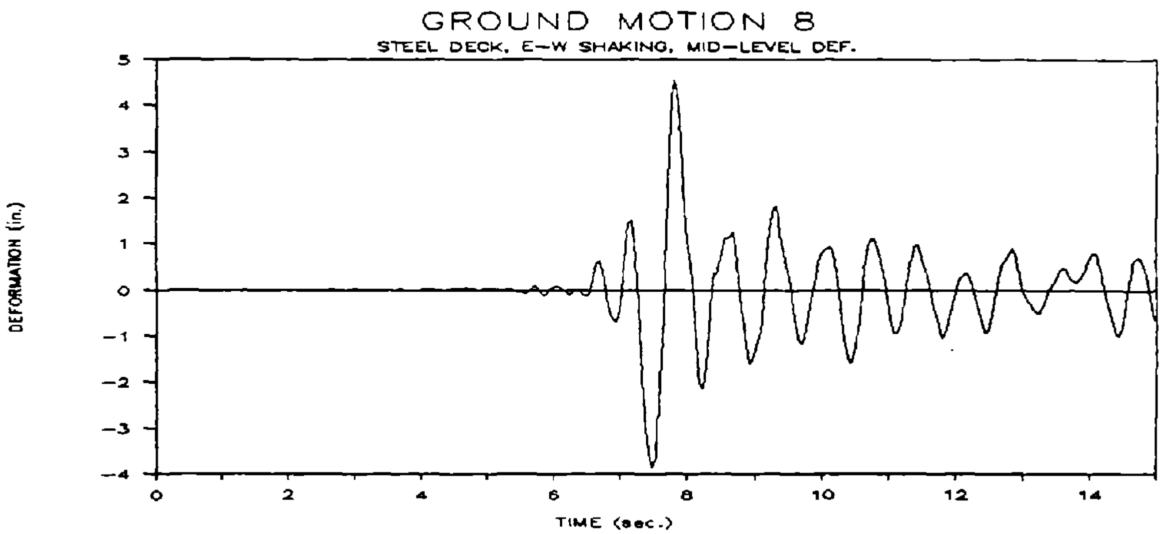


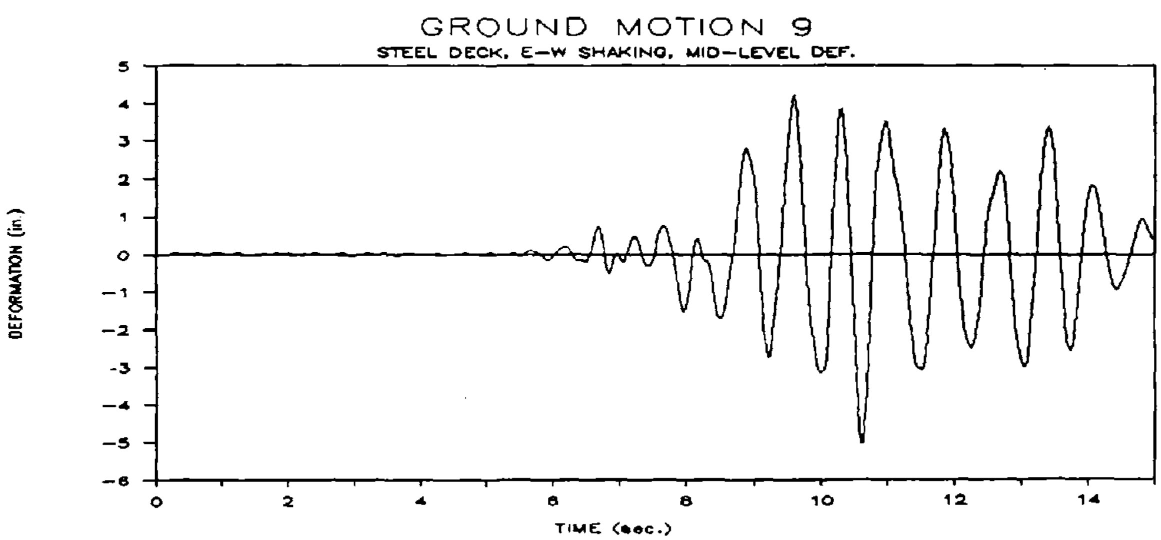
GROUND MOTION 6

STEEL DECK, E-W SHAKING, MID-LEVEL DEF.




GROUND MOTION 7
STEEL DECK, E-W SHAKING, MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)


GROUND MOTION 8
STEEL DECK, E-W SHAKING, MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)


GROUND MOTION 9
STEEL DECK, E-W SHAKING, MID-LEVEL DEF.
DEFORMATION (in.)
TIME (sec.)

4.0 TMS CENTER

4.1 Description of the Building

A roof framing plan of the building and elevations of the concrete block masonry walls are shown in Figures 4-1 and 4-2 respectively. The materials of construction are shown on these figures. The building is similar to one selected for a design example by the preparers of a Masonry Designer's Guide. Figures 4-1 and 4-2 show control joint locations that were used for unreinforced concrete masonry walls. The reinforced masonry design has a single separation joint. The joint is on Line (C) at the junction of the twenty feet long shear wall and the subdividing wall on Line (2).

All walls are eight inch nominal thickness and are grouted solid. The floor at grade level is a concrete slab and attached to the walls by dowels. The walls are supported by continuous strip footings that are placed at one and one-half feet below the floor level.

4.2 Design Procedures

The calculations of the design are included in Appendix A-3.2. The design procedure was near identical to that summarized in Sec. 3.2. The results of the design procedure are summarized in this section.

Twenty percent of the snow load was included in the dead load of the roof diaphragm. The seismic design was for a location where

$S_{a(0.3)}$ is 1.0 and $S_{a(1.0)}$ is 0.58. The first design of the roof diaphragm was for a steel deck as noted on Figure 4-1. The attachment of the decking to the roof joists is three welds per 24 inch wide decking unit. The decking units are attached at their side laps by a one and one-half inch seam welds at mid span of the decking.

Alternative plywood diaphragms of equal strength and of equal initial stiffness were tested by the dynamic analyses when diaphragm failure of the steel decking was caused by two of the nine time-histories that were used for seismic analysis.

The vertical reinforcement in all walls, except for the twenty feet long shear wall is #5 @ 64 inches on center. The vertical reinforcement required for loading normal to the plane of the typical wall was found to be inadequate for the inplane flexural reinforcement of the shear wall.

The vertical reinforcement of the twenty feet long rectangular shear wall was determined as #5 @ 32 inches on center. This reinforcement produced an expected flexural strength of 1603 ft. kips and exceeds the required strength of 1352 ft. kips. The vertical reinforcement ratio is much less than the maximum allowable ratio of 35 percent of the balanced reinforcement ratio. The horizontal reinforcement in the shear wall is #5 @ 32 inches on center. The other shear walls of the TMS Center have horizontal reinforcement of #5 @ 40 inches on center. The flexural capacity of these walls greatly exceeds the required strength and the shear capacity of these shear walls exceeds the required strength by the load factor of 2.5 that is required for this condition.

4.3 Description of the Analytical Model

The analytical model is shown schematically in Figure 4-3. The model is for shaking of the building in the East-West direction. The ground motion is applied to the model at nodes E1 through E2. Node E2 is the top of the north wall. Prior studies of the DPC Gymnasium have shown that this simplification of the model, where the ground motion is applied directly to the diaphragm end, does not affect the dynamic response of the diaphragm or the out-of-plane walls. The model of the diaphragm and the three out-of-plane walls was simplified to a one-third stiffness diaphragm and a single wall of beam elements (Figure 4-3).

4.3.1 Wall Beam Element

The out-of-plane wall is modeled as four, 4 feet long, linked beam elements. These beam elements are identical to the beam elements used for the DPC Gymnasium. Five 16 feet wide elements are equal to the diaphragm span and to the buildings width.

4.3.2 Diaphragm Parameters

The diaphragm model was divided into three equal segments tied together by rigid links. Each segment provides support for one of the three north-south walls and excites the top of the out-of-plane wall with the diaphragm motion. One-third of the diaphragm mass is lumped with each diaphragm segment.

The stiffness displacement at peak strength and peak expected strength of the diaphragm was taken from the ABK diaphragm test program (ABK, 1981). The peak strength of the sixteen feet long segment of steel decking occurs at a relative displacement of 0.9 inches.

4.3.3 Shear Wall Stiffness

The shear wall, shown as spring S6 in Figure 4-3, was analyzed by the nonlinear finite element program. The mesh used for the analysis is shown in Figure 4-4. Peak expected compressive strain occurs in element 12 at 0.63 inches top displacement. Tensile yielding of the vertical reinforcement spread into element 8 at 0.3 inches top displacement. No yielding of the shear reinforcement occurred.

The static nonlinear force-displacement relationship of the south shear wall is shown in Figure 4-5. The envelope curve of the dynamic spring used in the LPM analysis is superimposed on the results of the FEM analysis in Figure 4-5. Spring 11 (Kariotis, 1992) was used for simulation of the shear wall. The strength determined by the FEM analysis was greater than the calculated expected strength of 1603 ft. kips due to use of strain hardening of the vertical reinforcement in the FEM analysis. The peak strength of the shear wall is shown in Figure 4-5.

4.3.4 Dynamic Analysis Model

The dynamic model shown in Figure 4-3 was excited by the nine ground motions described in Appendix A-2. The results of the analyses is presented in Sec. 4.4.

4.4 Results of the Analyses

The nine ground motions used for the analyses are listed in Table 4-1. The results of the analyses are presented in Table 4-2.

The significant elements in this analyses of the TMS Center are the shear wall at the south side on Line C and the diaphragm. The out-of-plane walls are stiff and have minimal out-of-plane curvature. The analysis of this asymmetrical structure produced significantly different results than the analyses of the symmetrical DPC Gymnasium.

Ground motion 5 and 9 caused diaphragm failure adjacent to the rear (north) wall. These ground motions are the 140 degree component of the James Road record scaled by a factor of 1.39 and the 140 degree component of the Keystone Road record scaled by a factor of 1.85. These records are from the 1979 Imperial Valley earthquake.

A typical record of the deformation and force in the shear wall, S6, and in the diaphragm segment adjacent to the rear wall, S6, is shown in Figure 4-6 and 4-7 respectively. The relative displacement of the top of the shear wall exceeds the peak strength displacement but the nonlinear behavior is stable. The dynamic behavior of the diaphragm, Figure 4-7, is linear-elastic. The plots of dynamic behavior are for ground motion No.2 which is the north-South component of the 1940 El Centro record. The scale factor was 1.31.

The failure of the steel deck diaphragm adjacent to the rear (north) wall is shown in Figure 4-8. The displacement associated with the peak strength is exceeded at about 10½ seconds into the record. The strength and stiffness degradation in the post-peak strength region causes a much larger subsequent displacement. This behavior is consistent with the static and dynamic test results of the ABK research program. The time of the diaphragm failure coincided with a top displacement of about 1½ inches at the top of the twenty feet long shear wall, Figure 4-9. A subsequent pulse caused a top displacement of the shear wall of about 2½ inches. At this time, the front (south) shear wall provided the only significant resistance to the lateral displacement of the diaphragm mass.

This diaphragm failure prompted a redesign of the roof diaphragm. Two wood sheathed diaphragm were used for subsequent analyses. These diaphragms are nailed plywood and are described as follows:

Diaphragm No.1	Peak shear strength is 300 kips Displacement at peak strength is 1.9 inches
Diaphragm No.2	Peak shear strength is 200 kips Displacement at peak strength is 1.9 inches

The force-displacement envelopes of the steel decking and wood sheathed diaphragms are shown in Figure 4-10. The initial stiffness of the steel decking and plywood diaphragm No.1 is nearly identical. The peak strength of the steel decking and plywood diaphragm No.2 are identical.

The dynamic model, Figure 4-3, was rerun with wood diaphragm No.1. The deformation and force in the diaphragm segment adjacent to the rear wall is shown in Figure 4-11. This more flexible and stronger diaphragm had near elastic response. The deformations and forces in the shear wall at the front of the building are shown in Figure 4-12. The nonlinear deformation of the shear wall is nearly the same as that caused by the failure of the steel deck diaphragm. However, the peak displacement occurs in the opposite direction and slightly earlier in time.

The more flexible diaphragm of lesser strength, wood diaphragm No.2 had near identical behavior to wood diaphragm No.1. The deformation in the segment adjacent to the rear wall increased. Figure 4-13 shows the deformations and forces in this segment. The forces and deformations in the shear wall at the front of the building is shown in Figure 4-14. The nonlinear deformation in the shear wall is relatively unchanged from that calculated for the stronger wood diaphragm.

The out-of-plane relative deformations of the north-south walls are shown for grid B and grid E in Figure 4-15. These are the wall beam elements adjacent to the front and rear walls. The relative displacement (curvature) of the center of the wall was less than 1/500 of the wall height.

4.5 Discussion of the Results

The TMS Center has a very significant plan irregularity. The rotational response of this building was not modeled into the building specifically but was captured by the analytical method used. The diaphragm was modeled as having shear deformation only. This behavior was ascertained by the dynamic testing conducted by ABK (1981). In this building, such response is highly likely as the north-south walls are very stiff and restrain the adjacent diaphragm edges.

Revisions of the diaphragm stiffness and ductility did not make significant changes in the nonlinear displacements of the south shear wall. Table 4-2 shows that the diaphragm generally coupled more of the diaphragm mass with the stiffer north wall. It is possible that the presence of a shear wall on the south side is not needed. If this shear wall were not present, the design requirement would require the diaphragm strength to be doubled. If this design used a plywood diaphragm, the stiffness would increase with strength increase. It is possible that the dynamic deformations at Line C relative to the ground would be acceptable.

It is possible, but not indicated by the data in Table 4-2 that the south wall represents a "weak" story. The diaphragm maintains its strength as it has near-elastic behavior. When strength degradation occurs in the shear wall, a "weak" story exists.

In summary, the design procedure for all diaphragms was acceptable. Only two ground motions of the nine caused significant damage to the steel deck diaphragm. The drift ratio of the south shear wall was 1/75 maximum and 1/180 average. A maximum drift ratio of 1/100 is proposed by the Tentative Limit States Design Standards.

A prior study of the ground motions recorded during the 1979 Imperial Valley (Kariotis, 1990) earthquake concluded that some recorded ground motions have unique characteristics that cause a significantly greater damage to some class or type of structure. This effect is random and is contained in the probabilistic description of the design ground motion. For this reason nine time histories were used for the nonlinear studies and the <u>average</u> value of the data has more significance than the individual data points.

TABLE 4-1

FINAL GROUND MOTIONS FOR LPM RUNS

No.	Name	Duration	C1	C2
1	EL CENTRO E-W	53.0	0.9255	1.7875
2	EL CENTRO N-S	53.0	0.6777	1.3145
3	PINE UNION 140	29.0	0.8622	1.7067
4	CRUICKSHANK RD. 230	34.0	0.7632	1.4951
5	JAMES ROAD 140	29.0	0.7126	1.3893
6	KERN CO. 69	54.0	1.4080	2.8648
7	CRUICKSHANK RD. 140	34.0	0.6157	1.2024
8	BRAWLEY AIRPORT 315	37.0	1.0644	2.0738
9	KEYSTONE ROAD 140	39.0	0.9485	1.8501

C1: SCALING FACTOR FOR ZPA 0.2g

C2: SCALING FACTOR FOR ZPA 0.4g

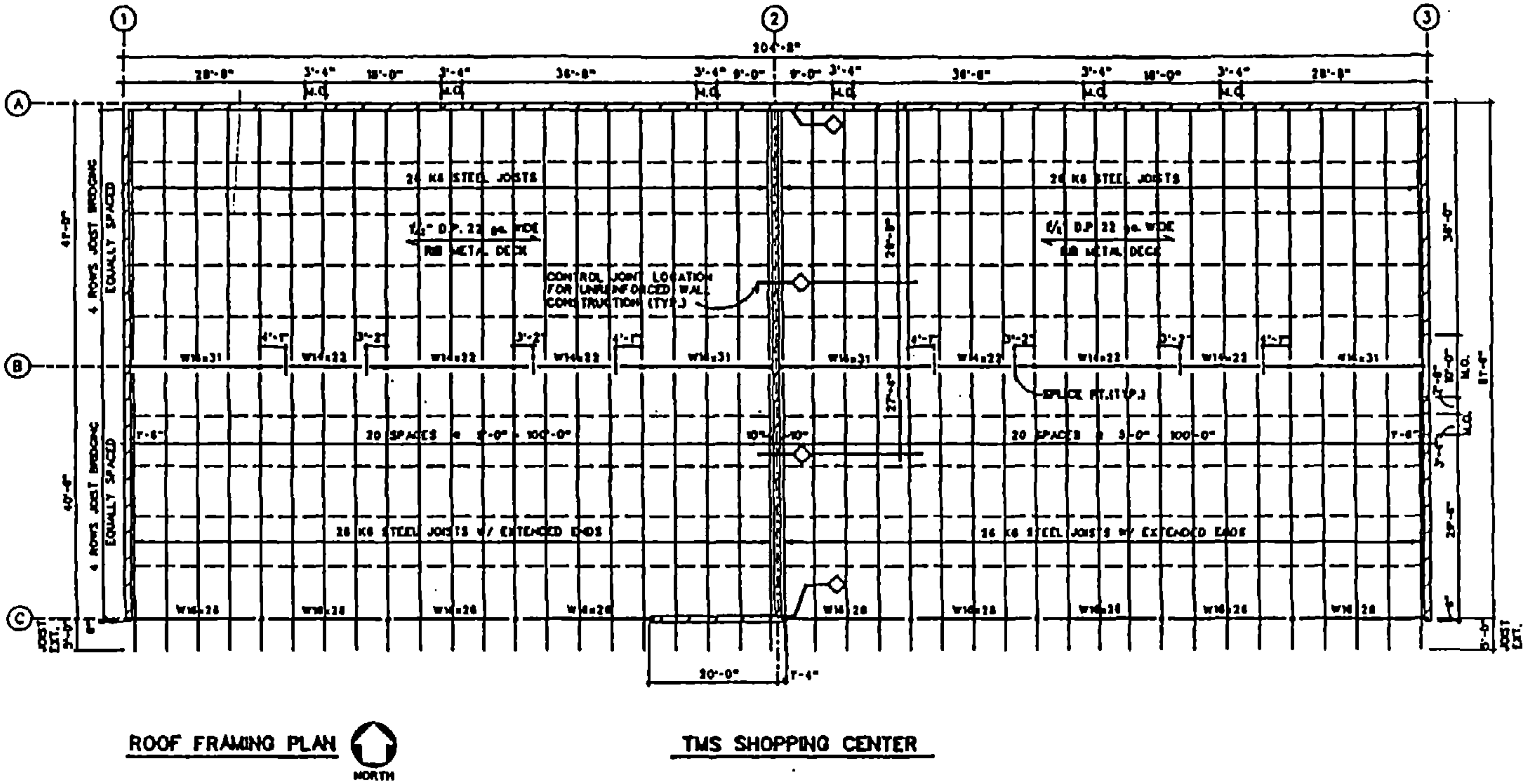
TABLE 4-2

GROUND MOTION

No.	C1	C2	C3	C4	C5	C6	C7	C8
1	0.22	0.15	105.07	107.82	131.70	0.37	0.38	0.22
2	0.25	0.30	116.12	134.10	147.14	0.42	0.51	0.48
3	0.47	0.59	127.31	158.73	151.67	0.47	0.64	1.33
4	0.36	0.34	139.54	142.72	146.23	0.53	0.55	0.37
5	0.48	0.54	162.22	189.91	151.32	0.66	1.66	1.86
6	0.52	0.53	132.67	175.41	151.96	0.51	0.75	1.59
7	0.23	0.26	119.36	147.28	143.45	0.43	0.58	0.34
8	0.42	0.48	116.11	174.00	151.80	0.42	0.75	0.91
9	0.57	0.95	133.06	180.57	151.18	0.50	2.91	2.52
Average								
	0.39	0.46	127.94	156.73	147.38	0.48	0.97	1.07

NOTES:

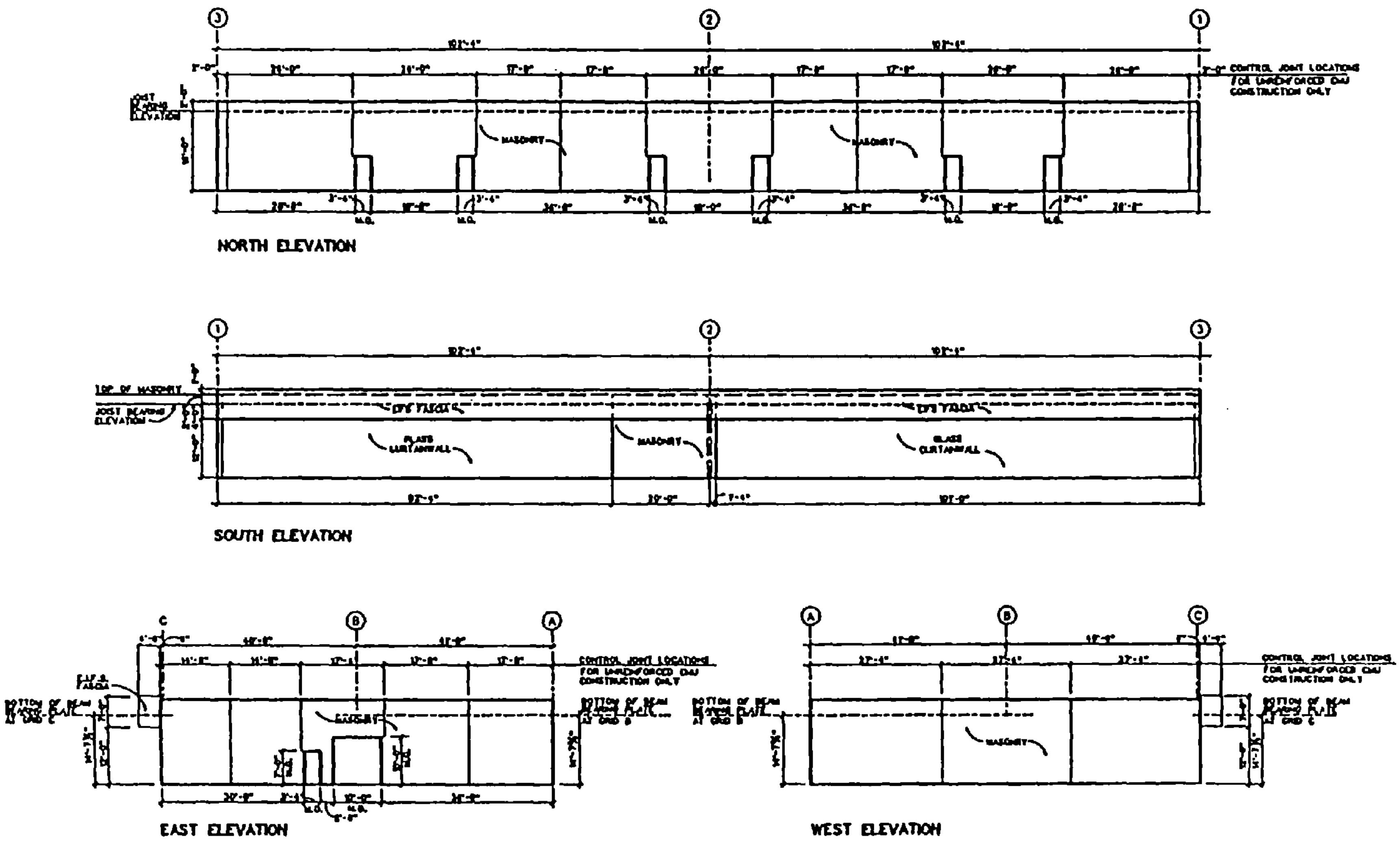
COLUMN 1: GRID B MID-LEVEL CURVATURE FROM ORIGINAL SHAPE.
COLUMN 2: GRID E MID-LEVEL CURVATURE FROM ORIGINAL SHAPE.
COLUMN 3: SPRING 1 MAXIMUM FORCE.
COLUMN 4: SPRING 5 MAXIMUM FORCE.
COLUMN 5: SPRING 6 MAXIMUM FORCE.
COLUMN 6: SPRING 1 MAXIMUM DISPLACEMENT.
COLUMN 7: SPRING 5 MAXIMUM DISPLACEMENT.
COLUMN 8: SPRING 6 MAXIMUM DISPLACEMENT.



NOTES

1. ROOF DEAD LOAD - 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 INSULATION ON METAL ROOF DECK - 15 PSF (FLEXIBLE DIAPHRAGM)
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 4-1



TMS SHOPPING CENTER

FIGURE 4-2

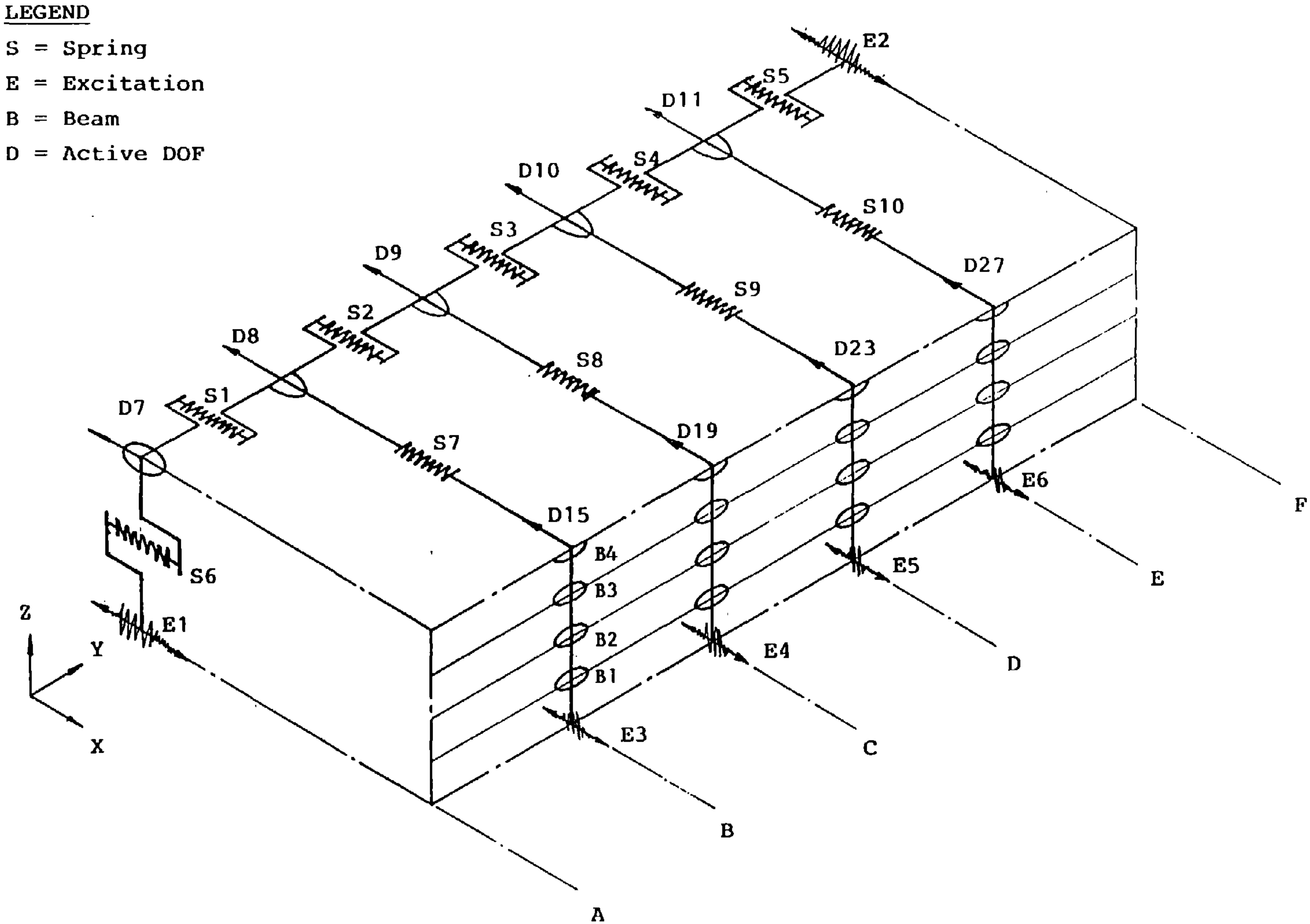

LEGEND
S = Spring
E = Excitation
B = Beam
D = Active DOF
D7
D8
D9
D10
D11
S1
S2
S3
S4
S5
S6
S7
S8
S9
S10
D15
D19
D23
D27
B4
B3
B2
B1
E1
E2
E3
E4
E5
E6
Z
Y
X
A
B
C
D
E
F

FIGURE 4-3

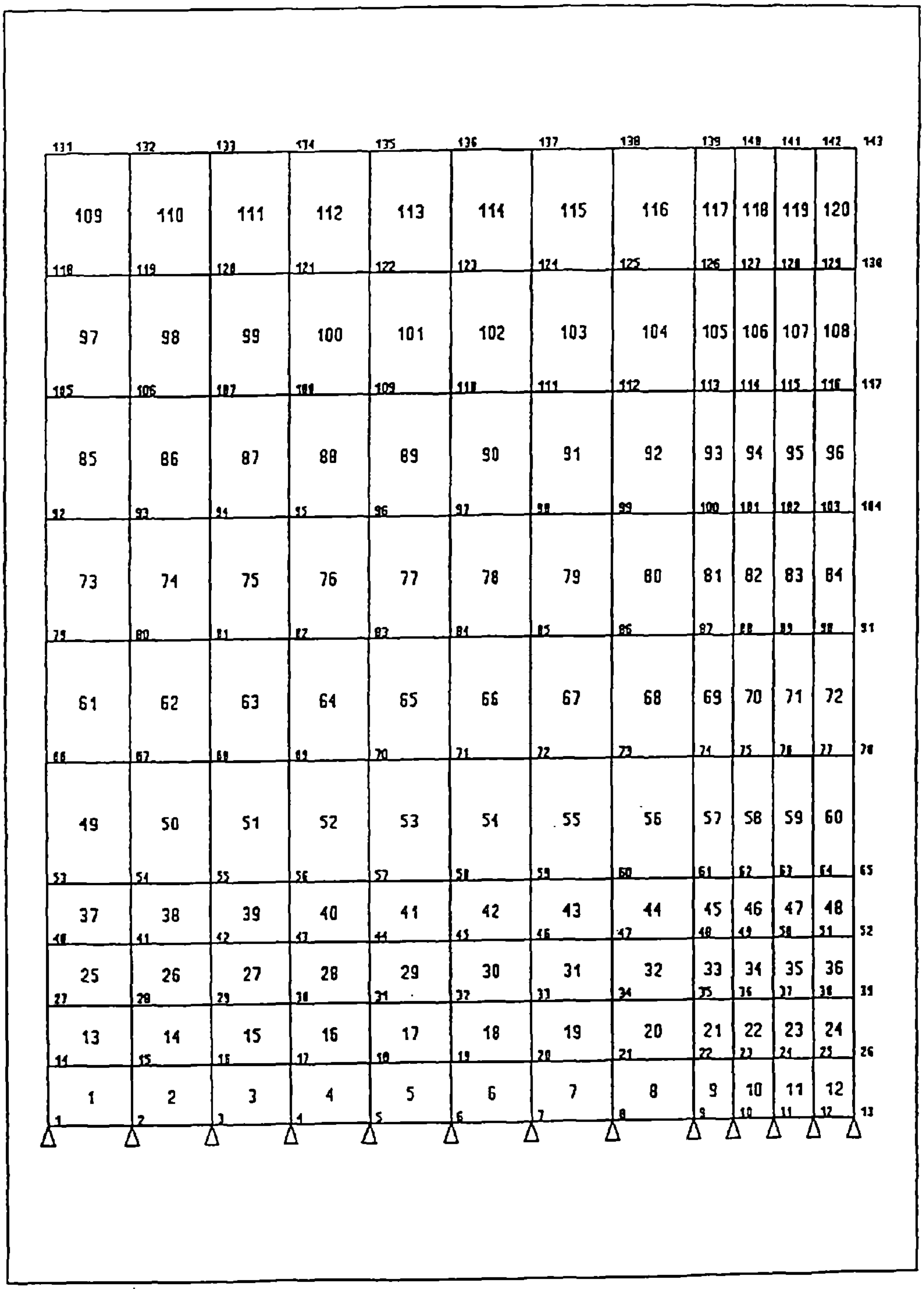

109 110 111 112 113 114 115 116 117 118 119 120
97 98 99 100 101 102 103 104 105 106 107 108
85 86 87 88 89 90 91 92 93 94 95 96
73 74 75 76 77 78 79 80 81 82 83 84
61 62 63 64 65 66 67 68 69 70 71 72
49 50 51 52 53 54 55 56 57 58 59 60
37 38 39 40 41 42 43 44 45 46 47 48
25 26 27 28 29 30 31 32 33 34 35 36
13 14 15 16 17 18 19 20 21 22 23 24
1 2 3 4 5 6 7 8 9 10 11 12

FIGURE 4-4

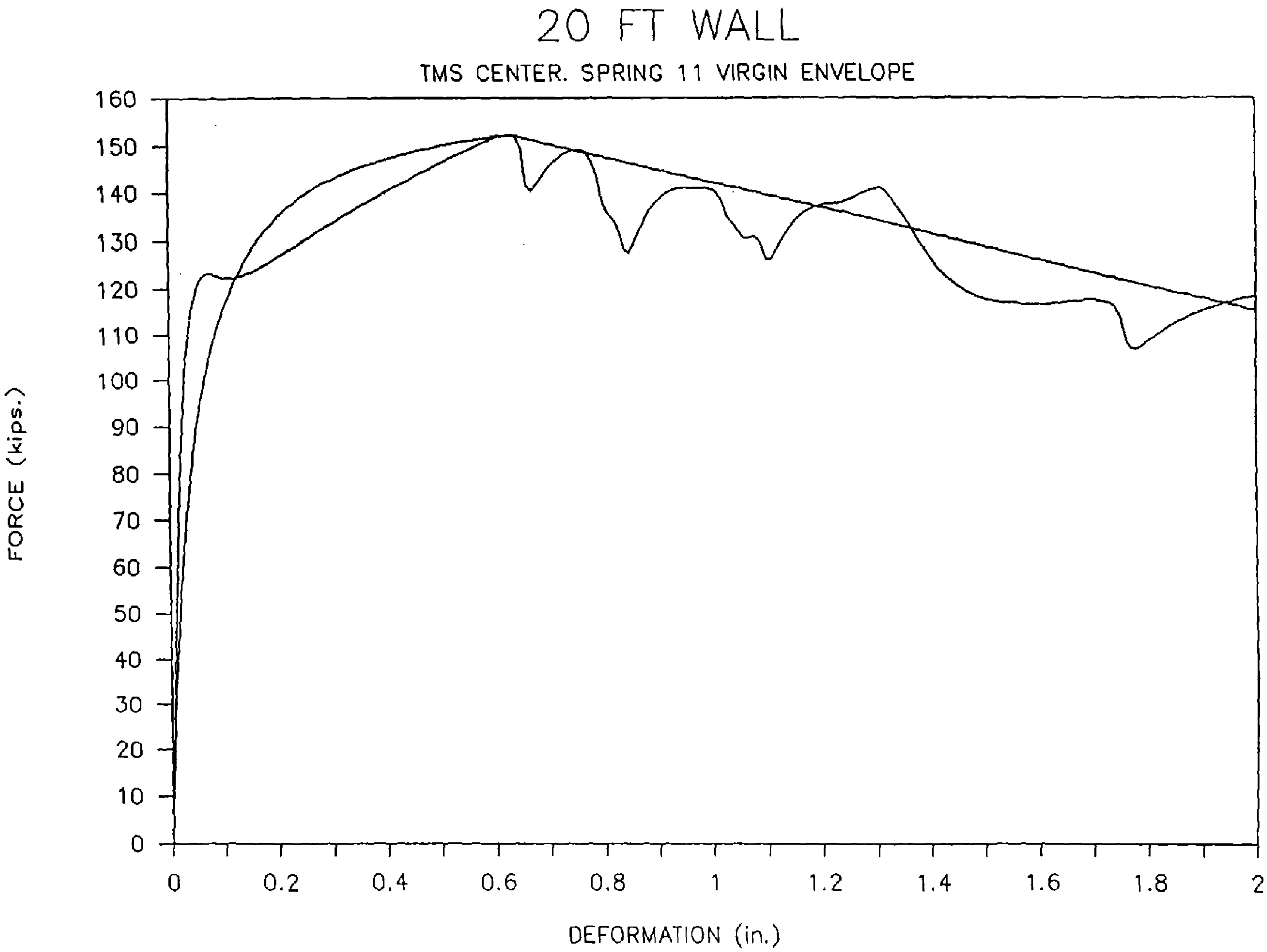
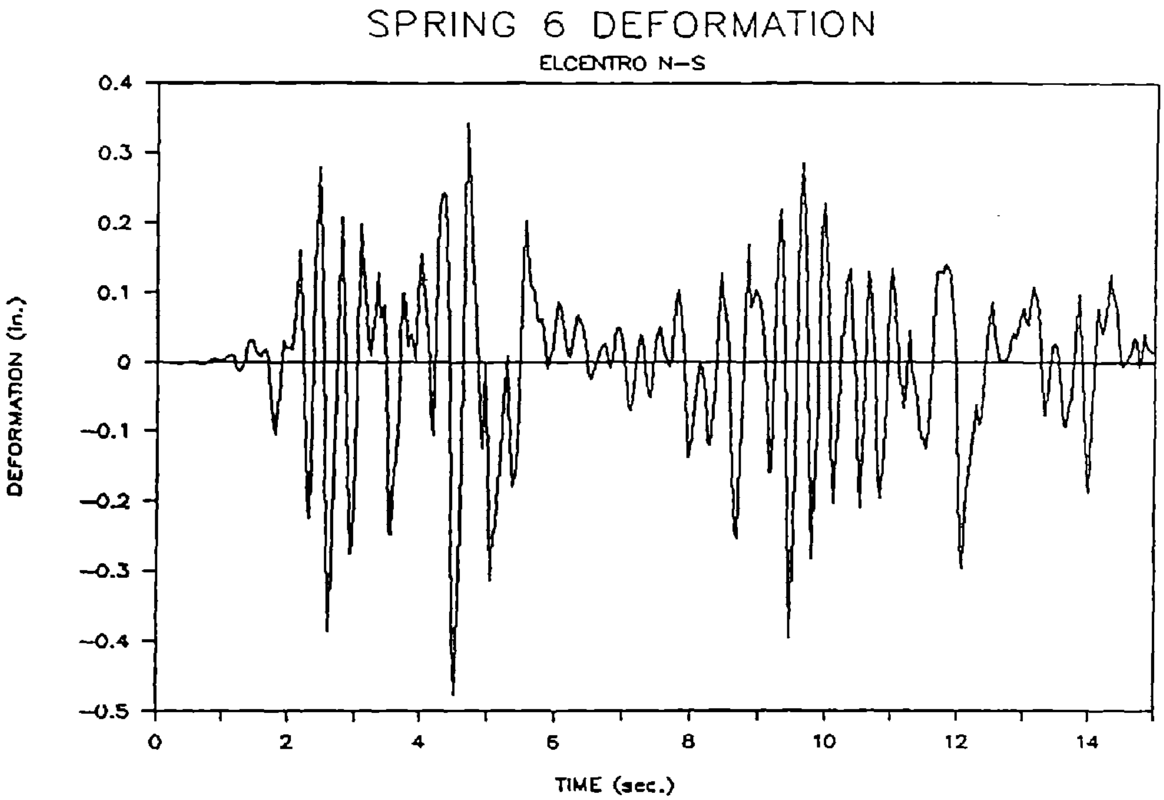

20 FT WALL
TMS CENTER. SPRING 11 VIRGIN ENVELOPE
FORCE (kips.)
DEFORMATION (in.)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
140
150
160
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
1.8
2

FIGURE 4-5


SPRING 6 DEFORMATION
ELCENTRO N-S
DEFORMATION (in.)
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
0
2
4
6
8
10
12
14
TIME (sec.)

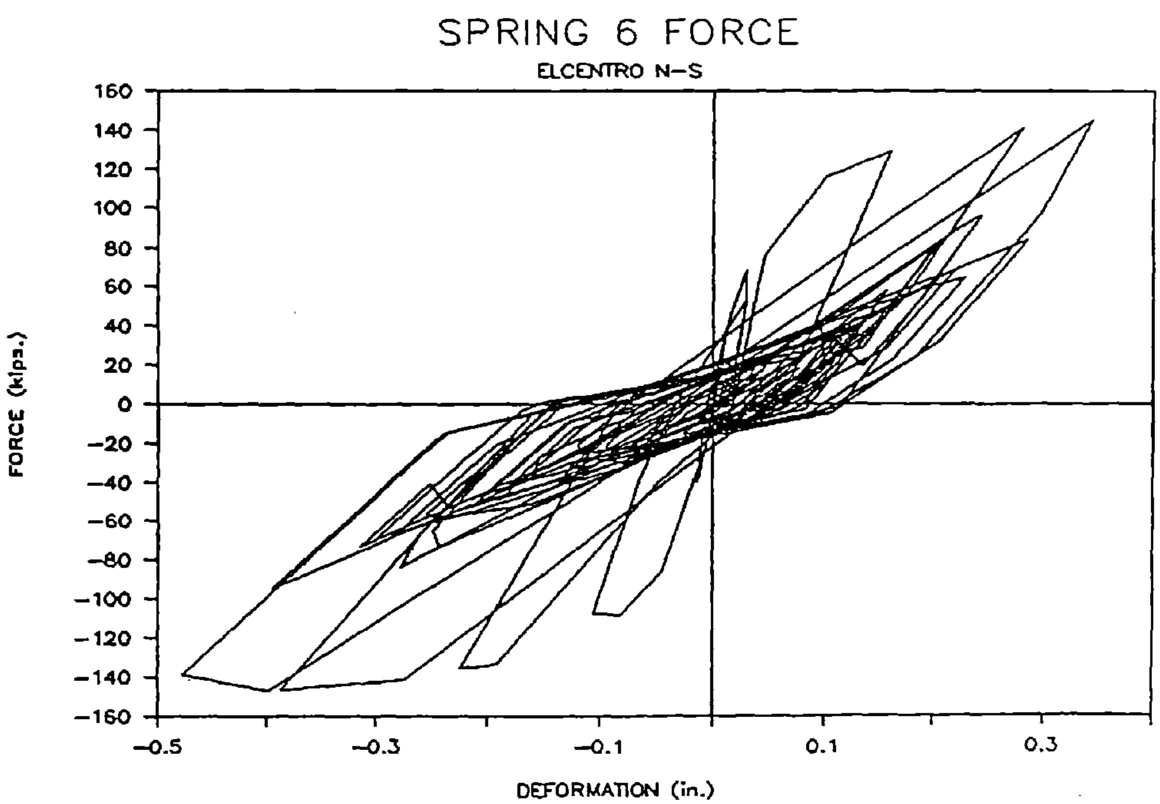
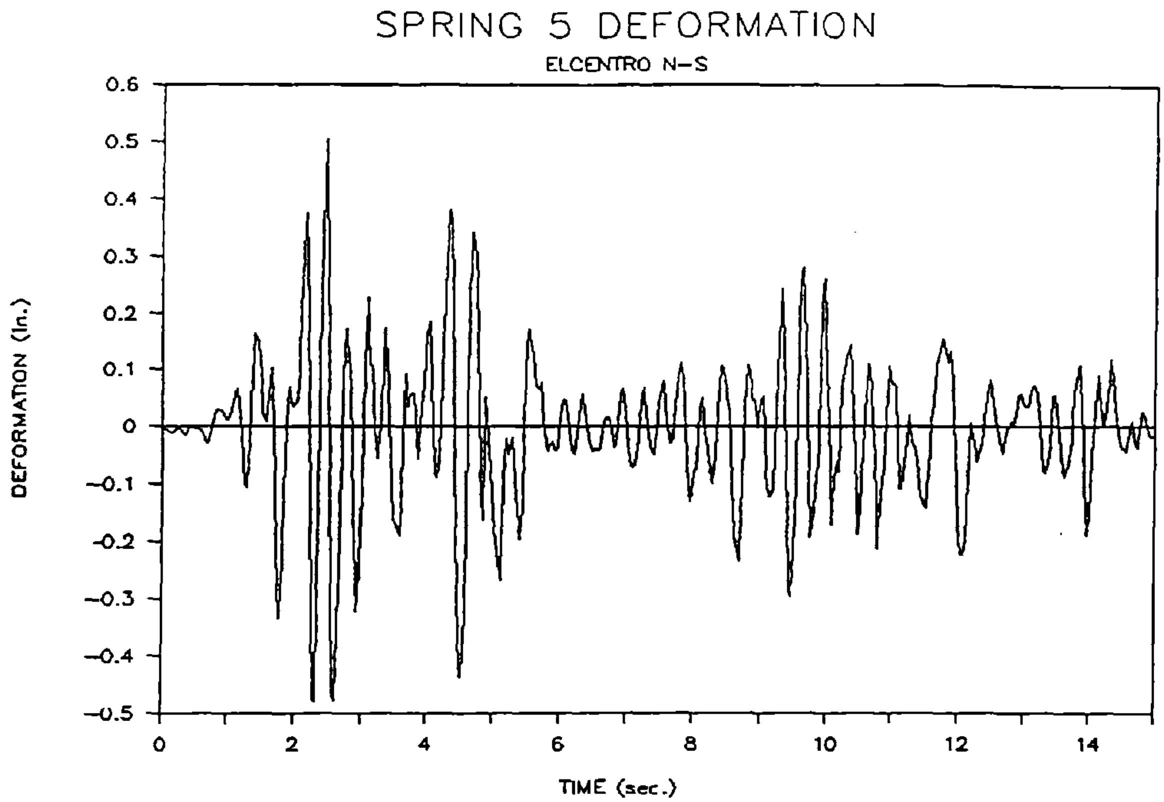
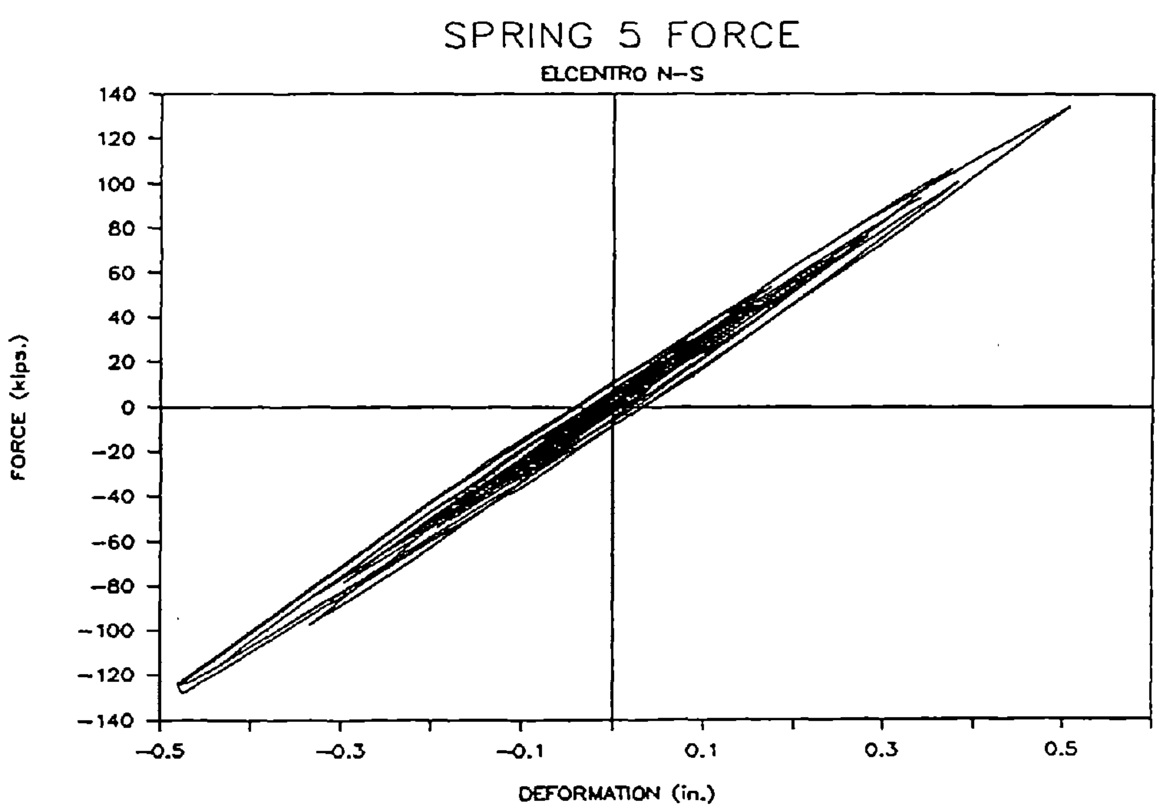

SPRING 6 FORCE
ELCENTRO N-S
FORCE (kips.)
160
140
120
100
80
60
40
20
0
-20
-40
-60
-80
-100
-120
-140
-160
-0.5
-0.3
-0.1
0.1
0.3
DEFORMATION (in.)

FIGURE 4-6

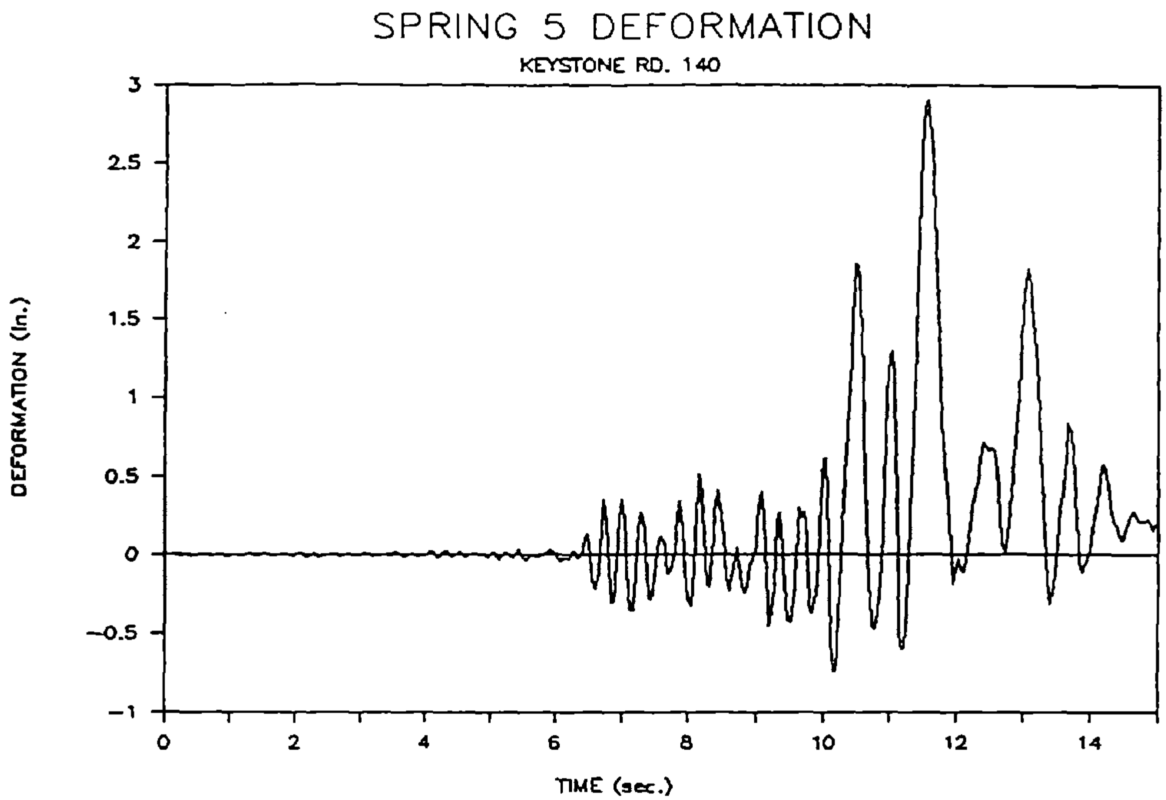


SPRING 5 DEFORMATION
ELCENTRO N-S
DEFORMATION (in.)
0.6
0.5
0.4
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
0
2
4
6
8
10
12
14
TIME (sec.)

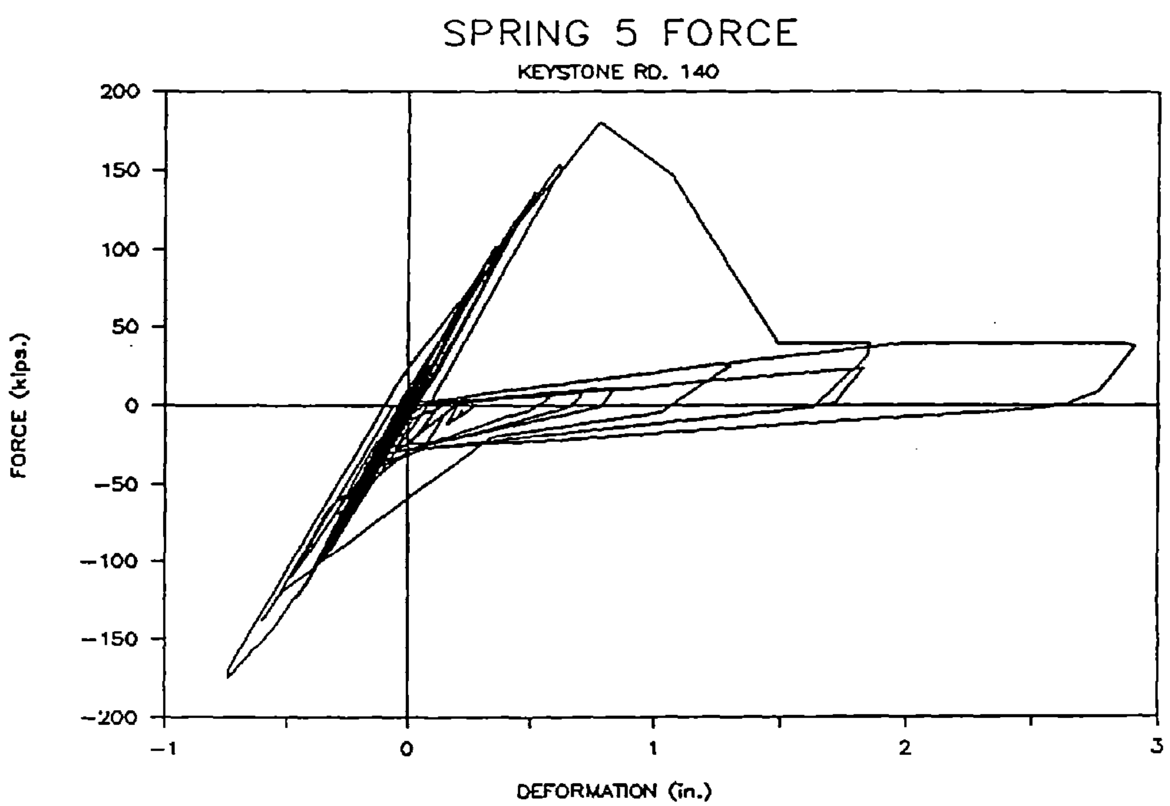


SPRING 5 FORCE
ELCENTRO N-S
FORCE (kips.)
140
120
100
80
60
40
20
0
-20
-40
-60
-80
-100
-120
-140
-0.5
-0.3
-0.1
0.1
0.3
0.5
DEFORMATION (in.)

FIGURE 4-7

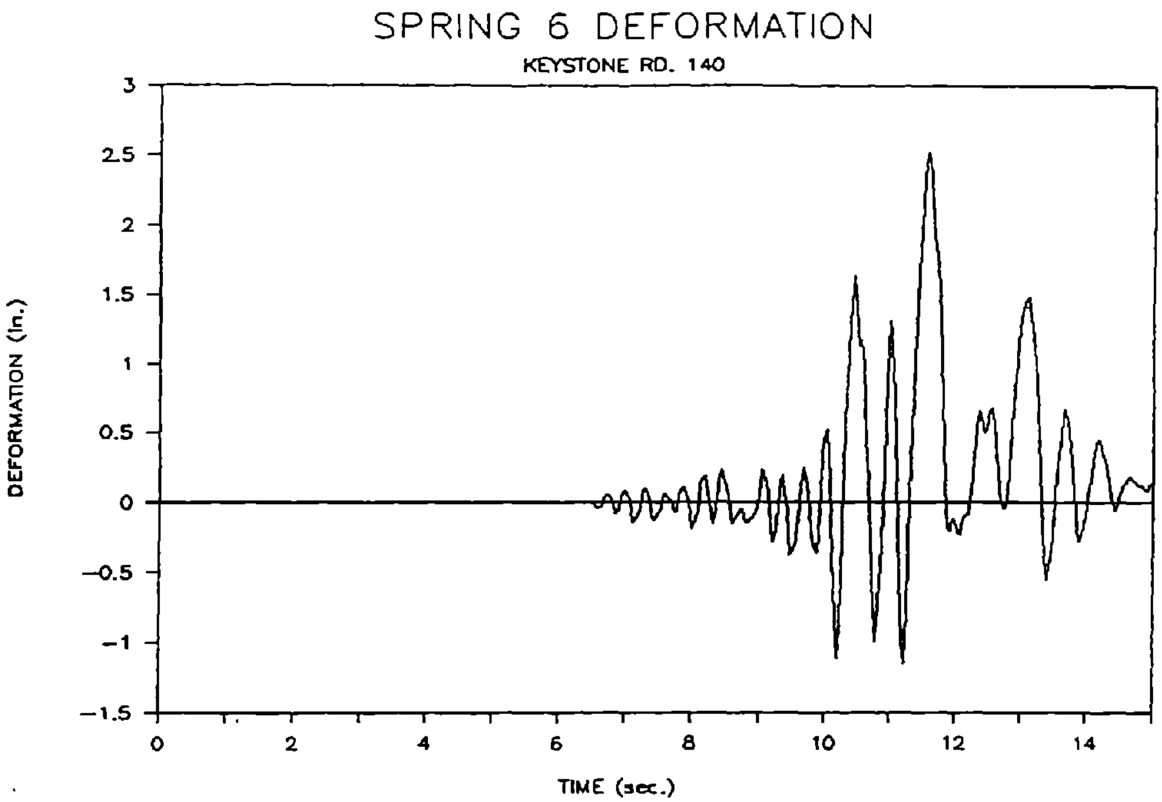


SPRING 5 DEFORMATION
KEYSTONE RD. 140
DEFORMATION (in.)
TIME (sec.)
3
2.5
2
1.5
1
0.5
0
-0.5
-1
0
2
4
6
8
10
12
14



SPRING 5 FORCE
KEYSTONE RD. 140
FORCE (kips.)
DEFORMATION (in.)
200
150
100
50
0
-50
-100
-150
-200
-1
0
1
2
3

FIGURE 4-8


SPRING 6 DEFORMATION
KEYSTONE RD. 140
DEFORMATION (in.)
TIME (sec.)
3
2.5
2
1.5
1
0.5
0
-0.5
-1
-1.5
0
2
4
6
8
10
12
14

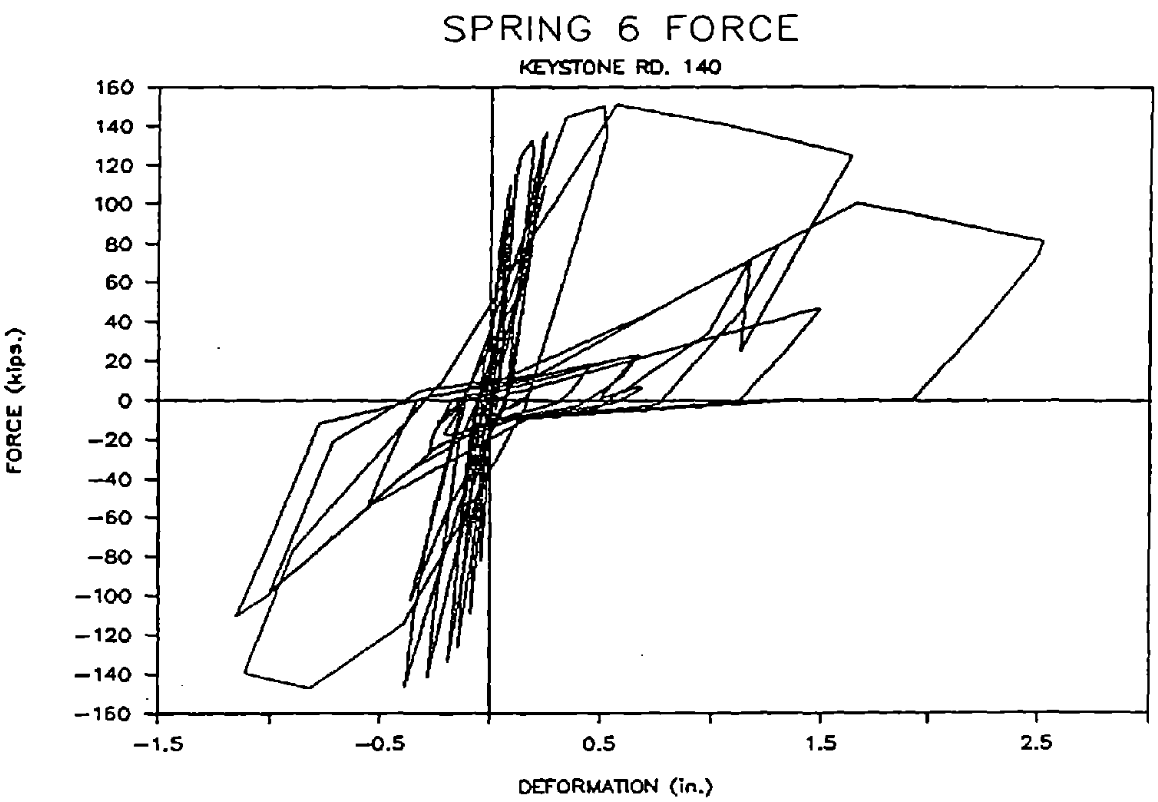
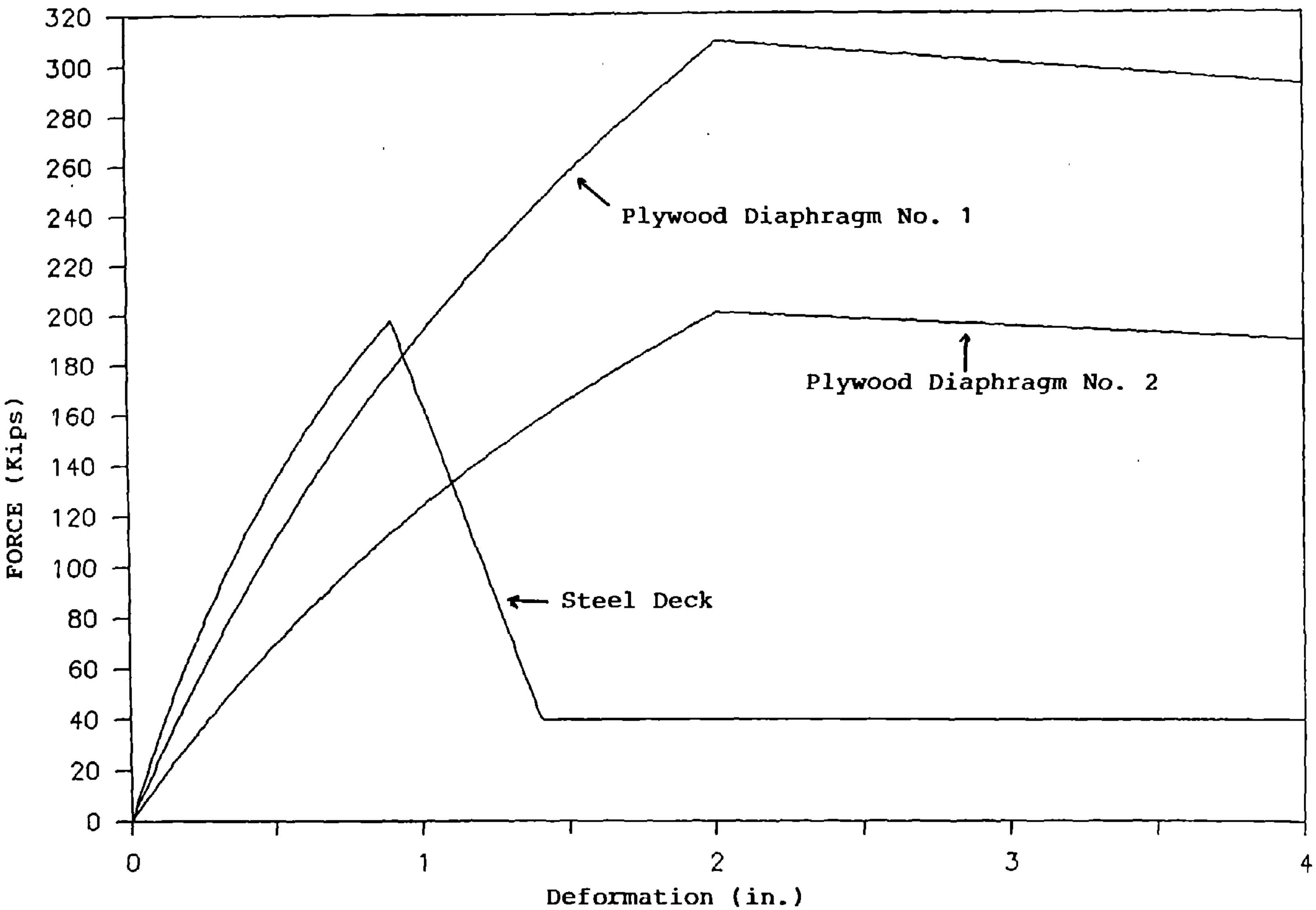
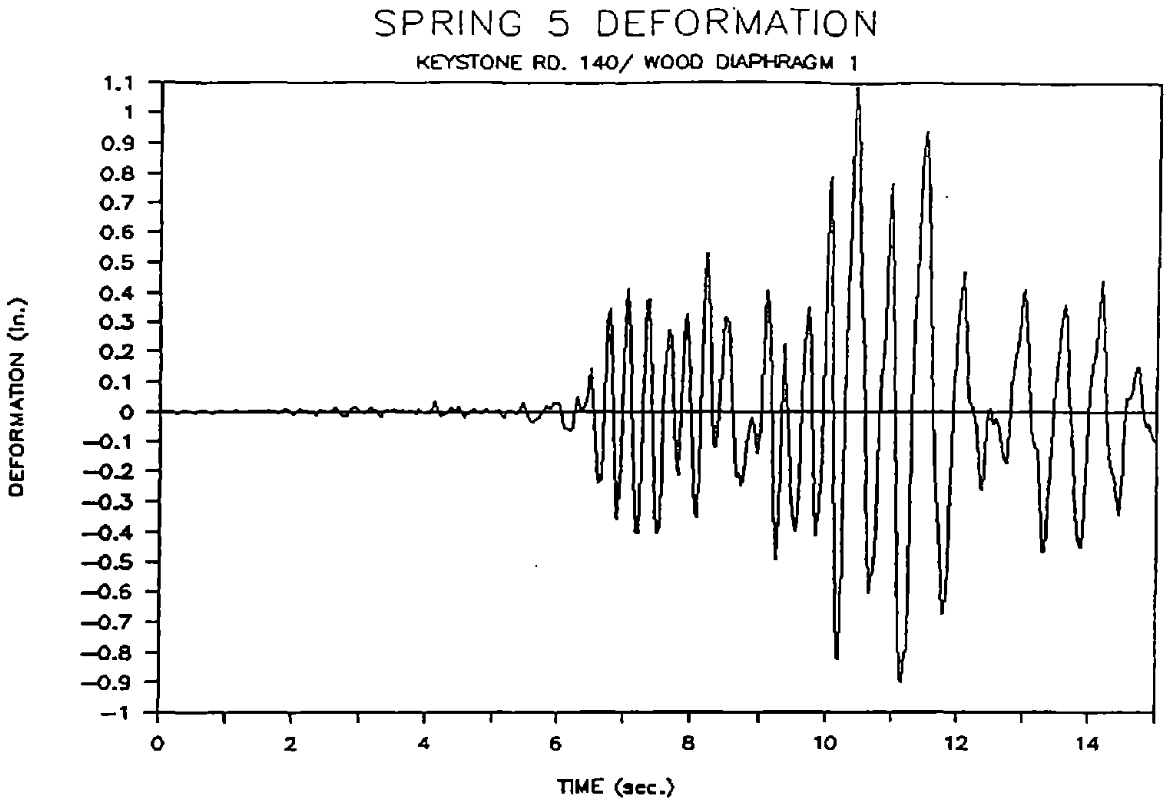

SPRING 6 FORCE
KEYSTONE RD. 140
FORCE (kips.)
DEFORMATION (in.)
160
140
120
100
80
60
40
20
0
-20
-40
-60
-80
-100
-120
-140
-160
-1.5
-0.5
0.5
1.5
2.5

FIGURE 4-9

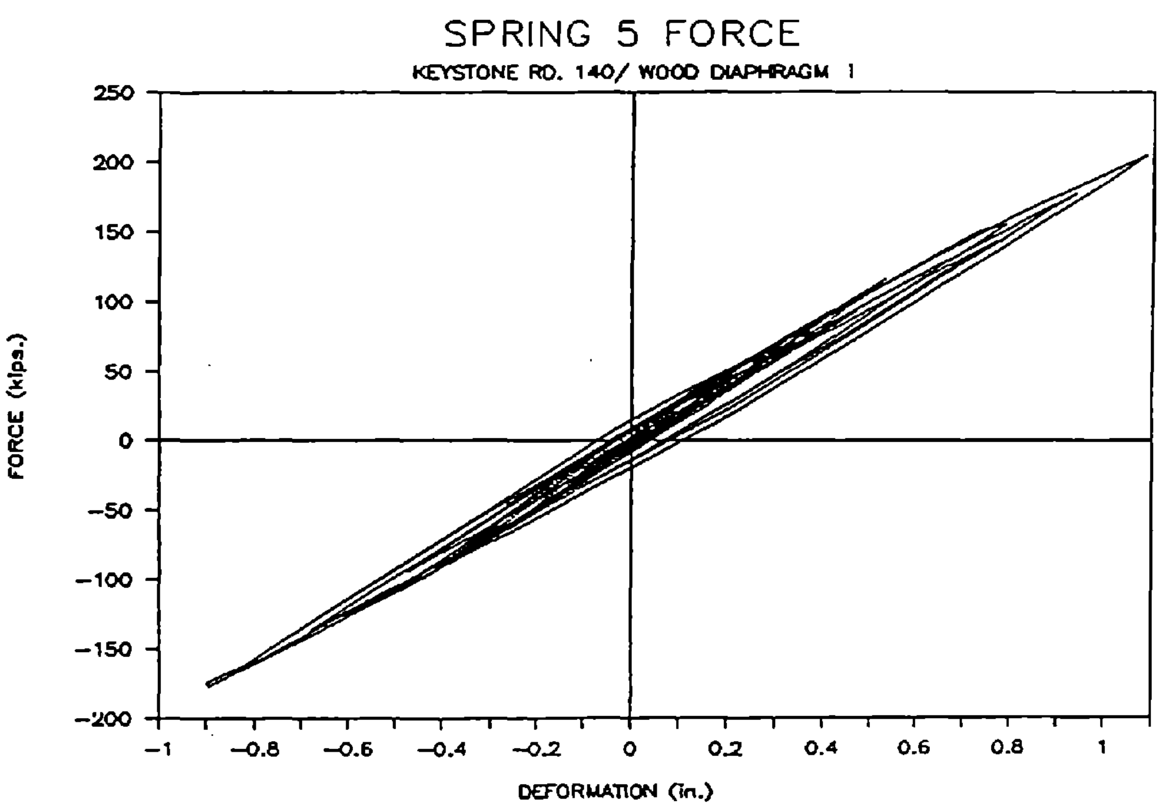


FORCE (Kips)
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
Deformation (in.)
0
1
2
3
4
Plywood Diaphragm No. 1
Plywood Diaphragm No. 2
Steel Deck

FIGURE 4-10

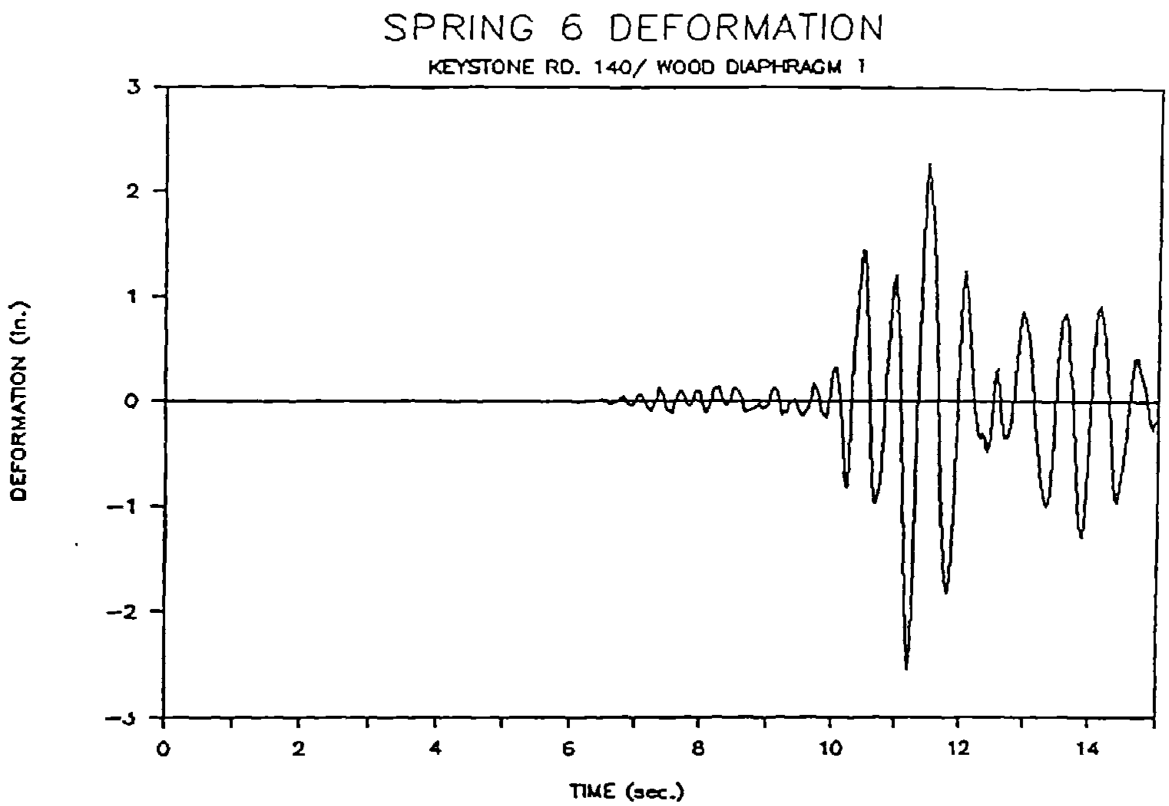


SPRING 5 DEFORMATION
KEYSTONE RD. 140/ WOOD DIAPHRAGM 1
DEFORMATION (in.)
TIME (sec.)



SPRING 5 FORCE
KEYSTONE RD. 140/ WOOD DIAPHRAGM 1
FORCE (kips.)
DEFORMATION (in.)

FIGURE 4-11


SPRING 6 DEFORMATION
KEYSTONE RD. 140/ WOOD DIAPHRAGM 1
DEFORMATION (in.)
3
2
1
0
−1
−2
−3
0
2
4
6
8
10
12
14
TIME (sec.)

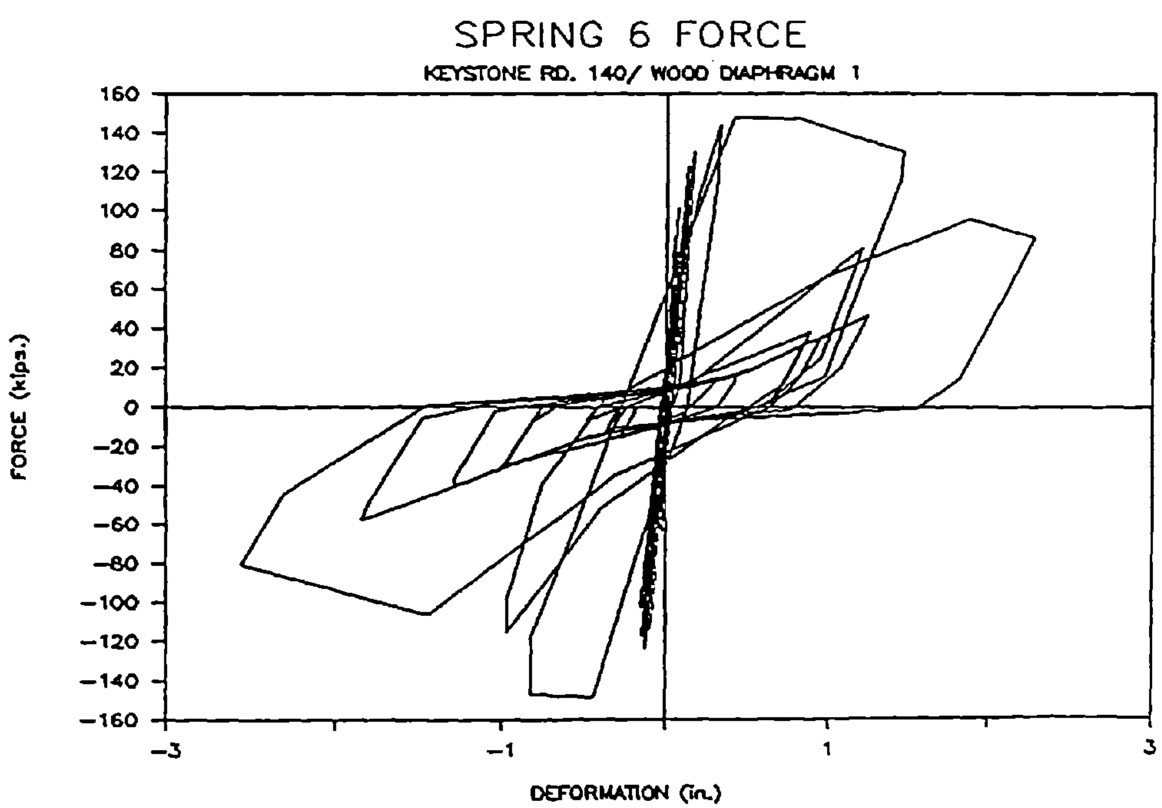
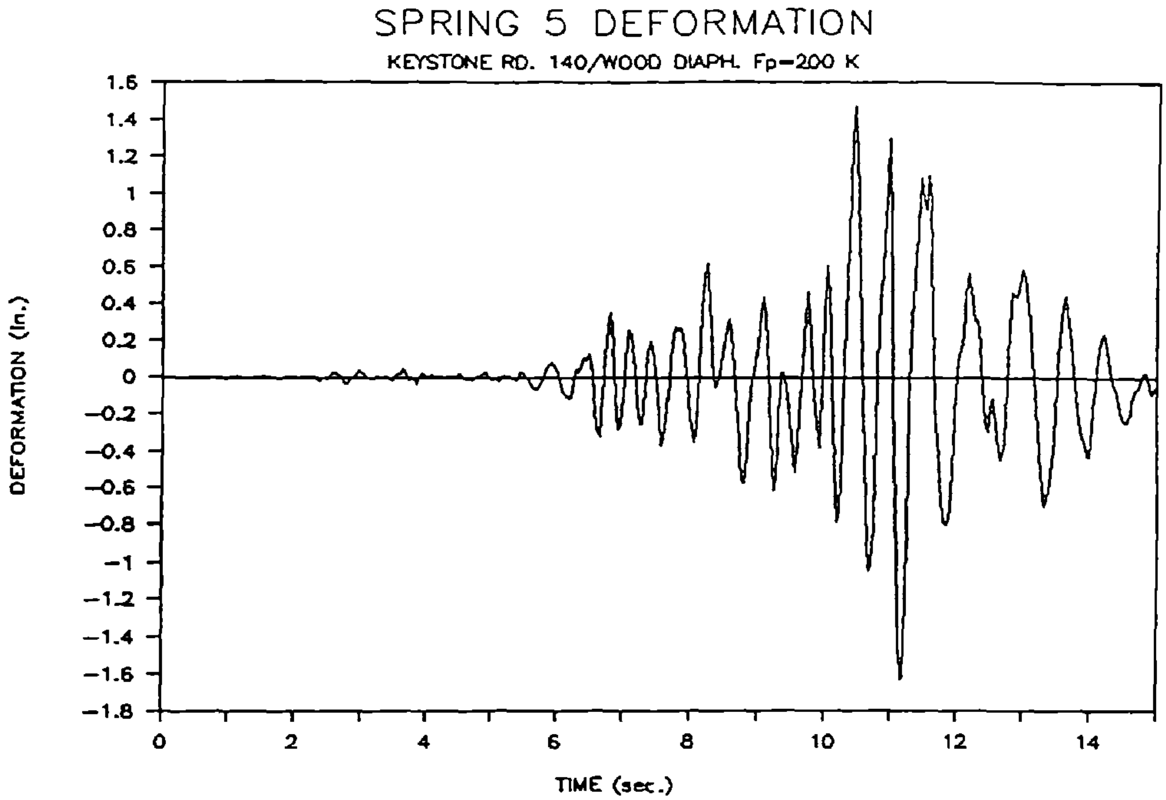

SPRING 6 FORCE
KEYSTONE RD. 140/ WOOD DIAPHRAGM 1
FORCE (kips.)
160
140
120
100
80
60
40
20
0
−20
−40
−60
−80
−100
−120
−140
−160
−3
−1
1
3
DEFORMATION (in.)

FIGURE 4-12


SPRING 5 DEFORMATION
KEYSTONE RD. 140/WOOD DIAPH. Fp=200 K
DEFORMATION (in.)
TIME (sec.)
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
-1.2
-1.4
-1.6
-1.8
0
2
4
6
8
10
12
14

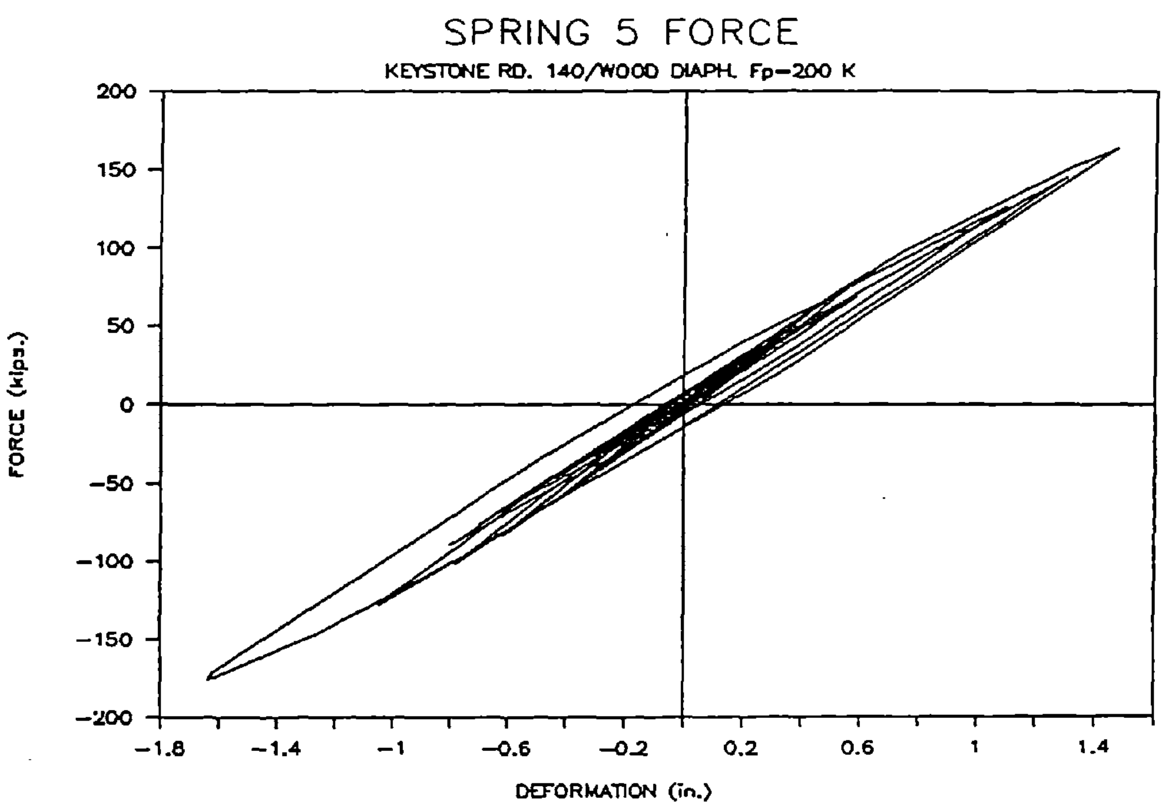
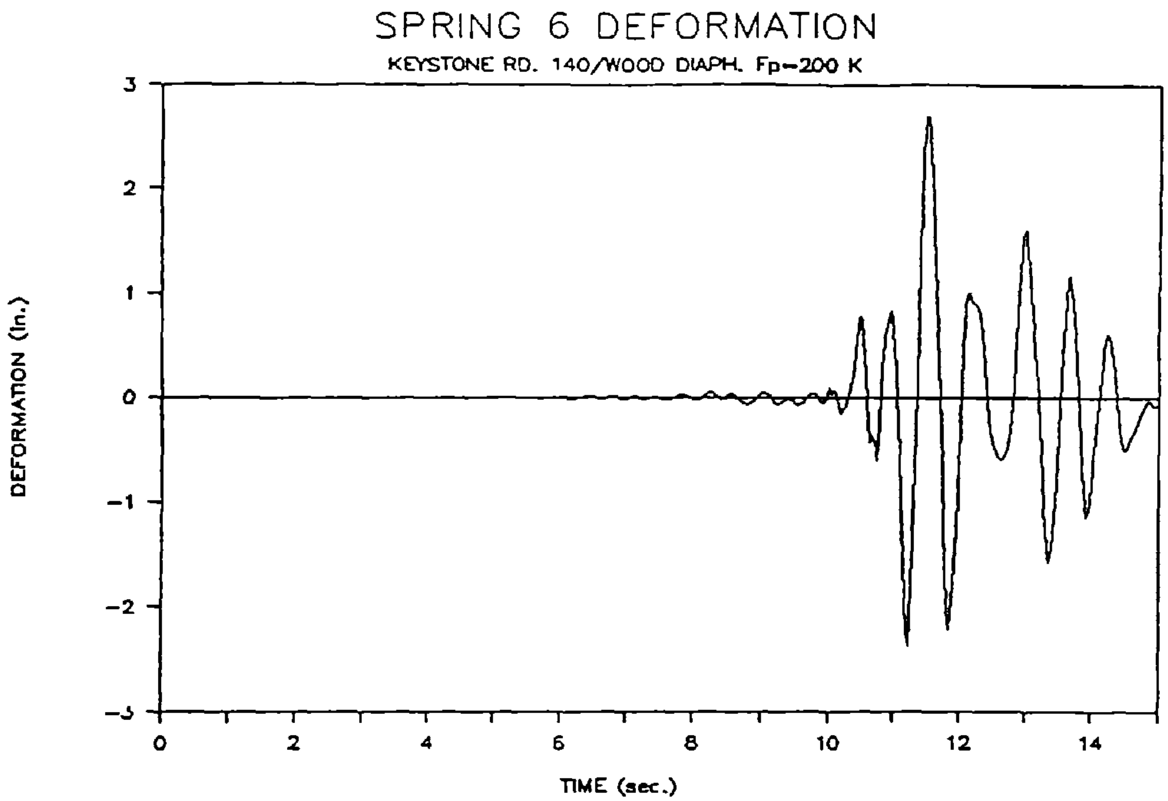
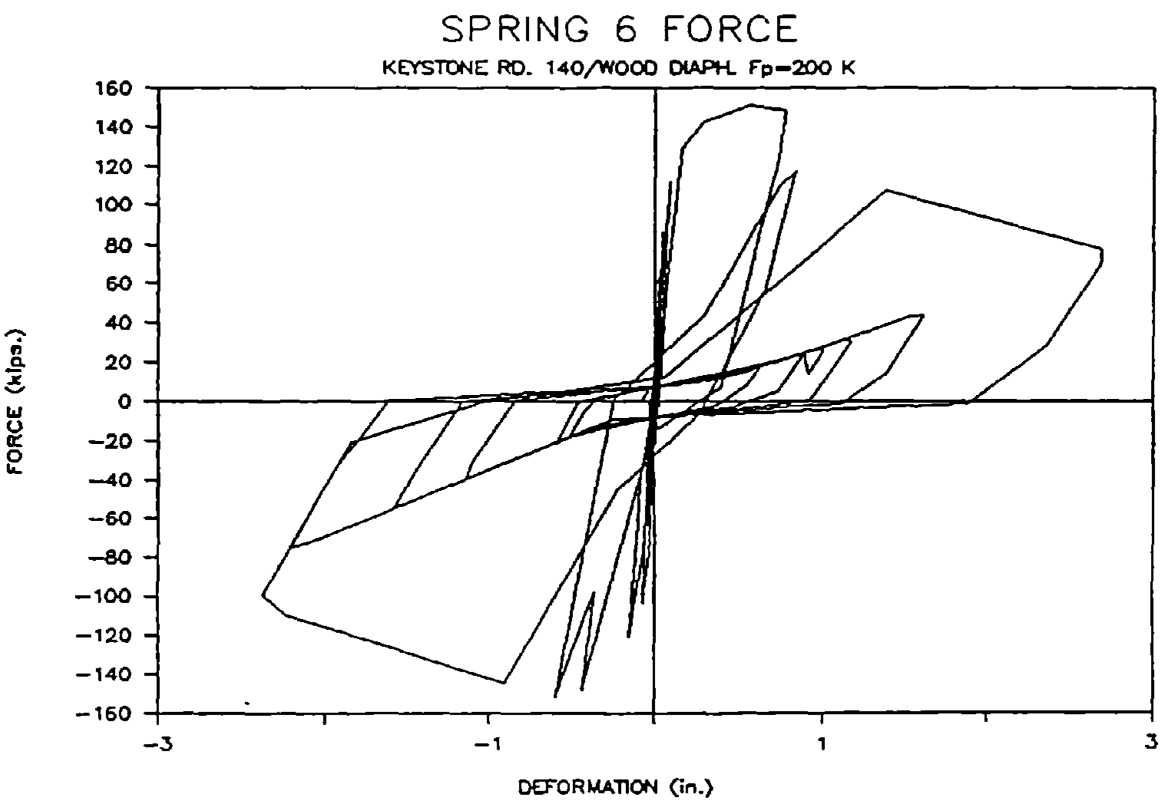

SPRING 5 FORCE
KEYSTONE RD. 140/WOOD DIAPH. Fp=200 K
FORCE (kips.)
DEFORMATION (in.)
200
150
100
50
0
-50
-100
-150
-200
-1.8
-1.4
-1
-0.6
-0.2
0.2
0.6
1
1.4

FIGURE 4-13

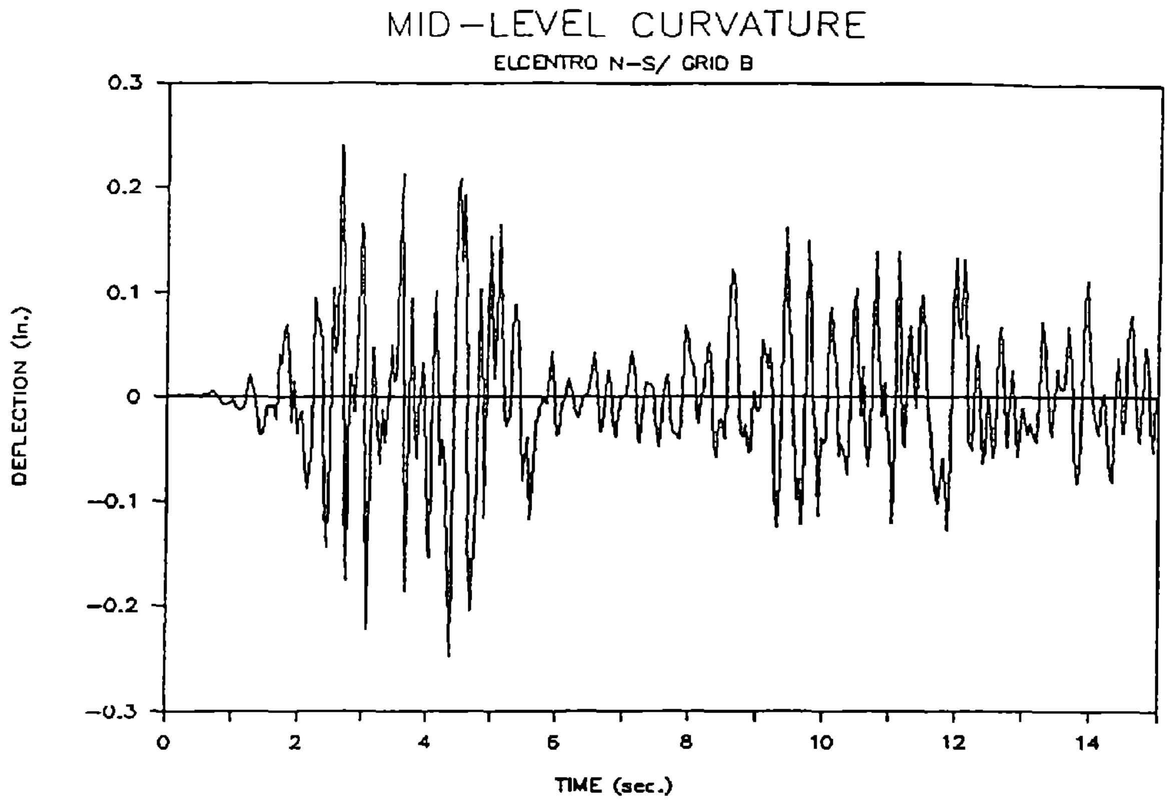


SPRING 6 DEFORMATION
KEYSTONE RD. 140/WOOD DIAPH. Fp=200 K
DEFORMATION (in.)
3
2
1
0
−1
−2
−3
0
2
4
6
8
10
12
14
TIME (sec.)



SPRING 6 FORCE
KEYSTONE RD. 140/WOOD DIAPH. Fp=200 K
FORCE (kips.)
160
140
120
100
80
60
40
20
0
−20
−40
−60
−80
−100
−120
−140
−160
−3
−1
1
3
DEFORMATION (in.)

FIGURE 4-14


MID-LEVEL CURVATURE
ELCENTRO N-S/ GRID B
DEFLECTION (In.)
0.3
0.2
0.1
0
-0.1
-0.2
-0.3
0
2
4
6
8
10
12
14
TIME (sec.)

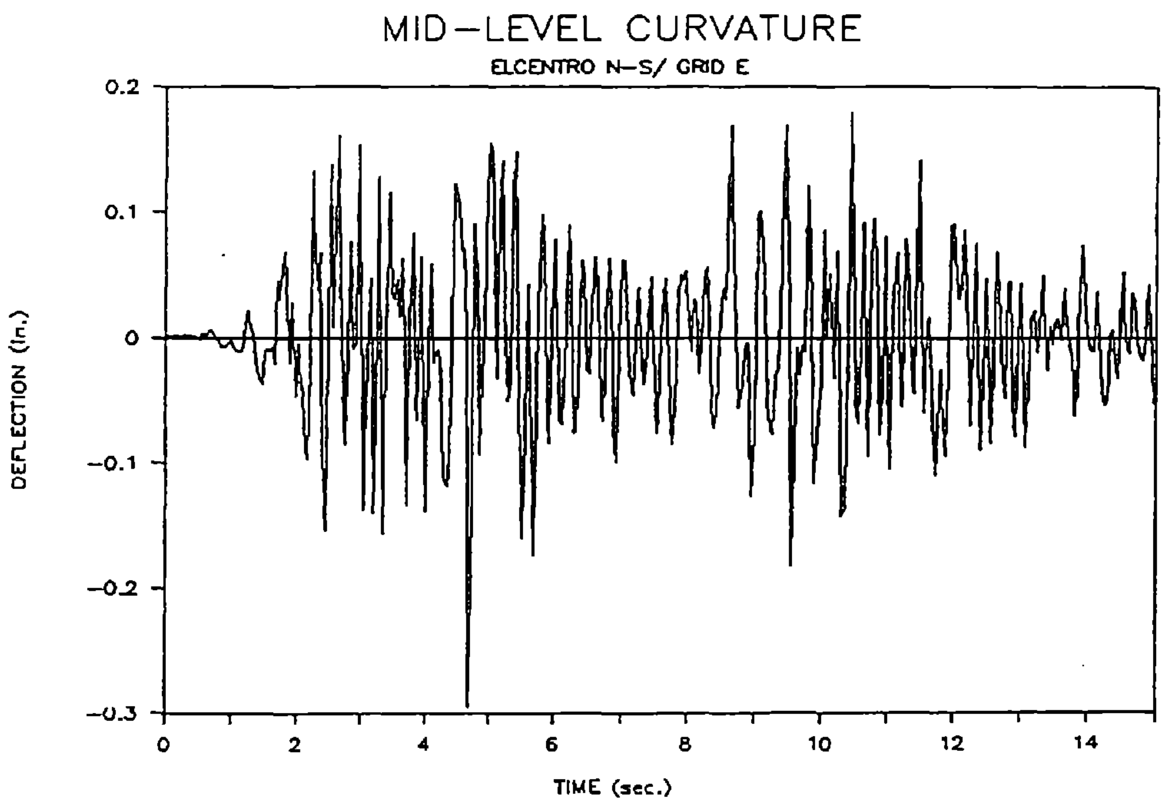

MID-LEVEL CURVATURE
ELCENTRO N-S/ GRID E
DEFLECTION (In.)
0.2
0.1
0
-0.1
-0.2
-0.3
0
2
4
6
8
10
12
14
TIME (sec.)

FIGURE 4-15

5.0 RCJ HOTEL

5.1 Description of the Building

The floor plans of this four story masonry bearing wall building are shown in Figures 5-1 and 5-2. A section through the building, Figure 5-3, shows the east-west shear walls in elevation. These shear walls on Line 2, together with the coupling beams over the doors at each edge, resist the earthquake loading in the east-west direction. The stair enclosures at the east and west ends of the building are seismically separated. The masonry walls that enclose the stairways provide the lateral load resistance. The reason for this design decision was that ties from these walls to the precast floor system cannot be easily made. The precast prestressed floor planks span between the masonry bearing walls on Lines B through F. The elevator shaft and duct space adjacent to Lines 2 and D are the only openings in the floors and roof. The precast planks at this opening are supported by a north-south bearing wall at the east side of this opening. Masonry bearing walls provide support for the floor and the roof at Lines B, C, D, E, F and at the east edge of the opening for the ducts and elevator. These bearing walls act as shear walls in the north-south direction. The wall on Line 2 provides the lateral load resistance in the east-west direction. All other walls are nonstructural and nonbearing.

A description of the loadings used for design is given in the notes, Figure 5-4. This design is for reinforced concrete masonry units. The story heights used for the design of the RCJ Hotel are listed in Figure 5-4. The RCJ Hotel has been designed by alternative methods by the writers of a Masonry Designers' Guide as an unreinforced and a reinforced masonry building for seismic zones 2 and 3. These designs used ACI 530/ASCE 7 as the criteria. This standard prescribes the use of "working stress" design criteria.

5.2 Design Procedure

The calculations made for the design are included in Appendix A-3.3. The design procedure used for the RCJ Hotel was near identical to that summarized in Section 3.2. The results of the design procedure are summarized in this section. The design procedure was limited to a design of the reinforced masonry shear walls on Line 2 for seismic loading in the east-west direction.

The weight of each story level was calculated for computation of a design base shear. The fundamental period was estimated by use of the alternative method of the NEHRP Recommended Provisions rather than the arbitrary definition of period. The contribution of the coupling beams at the floor and roof levels were not specifically considered as contributing to the shear wall stiffness in the calculation of fundamental period. The contribution of the coupling beams was incorporated in the period computation by a decision to utilize twenty percent of the gross section stiffness as the expected degraded stiffness of the shear wall. If the coupling beams were not present, use of ten percent of the gross section stiffness in the period calculations would be appropriate. The value of the calculated base shear was determined by the cutoff value of the base shear as prescribed by the Recommended Provisions.

The quantity of vertical reinforcement in the load bearing shear walls that is required for loading normal to the wall plane was calculated for each story height. The fourth story walls were critical for determination of the maximum quantity of vertical reinforcement. The moment in the bearing wall, due to the precast plank bearing on the face shell of the wall, was added to the moment caused by seismic loading normal to the wall. The required vertical reinforcement for loading normal to the shear wall was #4 @ 64 inches on center. No vertical reinforcement was required at the lower stories as the axial compressive stress exceeds the tensile flexural stresses. However, the quantity of the vertical reinforcement provided at all levels was that quantity calculated for the fourth floor level.

The seismic design of the coupled shear walls required that the expected strength of the floor plank-masonry lintel coupling beam be determined. The prestressing strand in the floor and roof planks acts as reinforcement for the masonry lintel above the doors. These lintels are shown in the elevation of the east-west shear wall, Figure 5-3. The area of the prestressing strand in two planks adjacent to the shear wall was checked against the specified maximum quantity of reinforcement. This maximum is based on a percentage of that calculated for balanced design. The area of prestressing strand required to support the floor loading in these two adjacent planks was calculated and greatly exceeds the limitation of thirty-five percent of the balanced design reinforcement ratio. To comply with the Limit States Design Standards, the configuration of the lintels in the east-west shear walls was revised. A separation joint was made at the ends of the masonry lintel. This joint extends from the top of the door to the underside of the prestressed plank. This joint would be filled with fire-resistant material to retain the fire rating of the masonry wall.

This decision reduced the dimensions of the coupling beam at each level to the plank and its topping. The prestressing strand provides a large expected moment capacity when the coupling beam is deformed such that the bottom of the plank is in tension. Two #8 bars were placed in the topping over the doorway to provide a moment capacity for the coupling beam when the deformed shape places the top of the beam in tension. The masonry over the doorway is reinforced and suspended from the floor and roof system.

The required strength of the four identical east-west shear walls was calculated. The expected "design" strength of the shear wall at its base is required to be equal to or greater than the required strength of the lateral load resisting system at this level minus the expected strengths of all eight coupling beams. This design assumed that the peak expected strength of <u>all</u> coupling beams will be coincident with the expected peak flexural strength of the shear wall at its base. Reinforced masonry shear walls with quantities of flexural reinforcement that are substantially less than thirty-five percent of balanced reinforcement ratio generally have a wide range of displacements that causes a base moment nearly equal to the expected moment. The assumption of the coincidence of peak expected moments in the shear wall and in the coupling beams is reasonable.

This design procedure resulted in a required vertical reinforcement of the shear wall at its base of #7 @ 16 inches on center. The vertical reinforcement ratio is about one-third of the allowable reinforcement ratio. A design decision was made that this quantity of vertical reinforcement would be used in both the first and second story levels. The quantity of vertical reinforcement was reduced to #5 @ 16 inches on center at the third and fourth floor levels. The "Matsumura" (Fattal, 1991) equation was used for calculation of the expected shear strength. The calculated required shear reinforcement of #6 @ 8 inches on center was used at all story levels.

5.3 Description of the Analytical Model

A full description of the development of the analytical model is presented in a companion paper by R.D. Ewing (1992). The shear walls were analyzed by use of a nonlinear beam element computer program, LPM/II, (Kariotis, 1992). Development of the behavioral characteristics of the nonlinear lumped parameter model requires that the characteristics of the coupling beams at each story level and the shear walls below that story level be described by a stiffness degradation versus curvature relationship.

A brief description of the finite element analysis is given here, the full description is given in Ewing (1992). Each coupling beam is modeled as a cantilever extending from the face of the shear wall to the bearing walls on Lines B, C, D, E, and F. The reinforced masonry bearing walls on these lines provide vertical support to the ends of the coupling beams. A point of inflection was assumed at this support point even though the coupling beams do not have symmetry when the free end is deflected either up or down. Moment is induced in the coupling beam by vertical movement of the edge of the shear wall relative to the bearing walls on Lines B through F. Moment is also induced in the coupling beam by curvature of the shear wall at its junction with the coupling beam. The shear force and the induced moment in the coupling beam versus curvature is determined by a nonlinear finite element analysis of the coupling beam. Figure 5-5 shows the finite element subdivision used for analysis of coupling beam 1, the coupling beam at the second story level. Figures 5-6 and 5-7 show the moment induced at the shear wall end versus the rotation of the beam at its point of inflection relative to the face of the shear wall. Figure 5-6 shows the relationship for displacement of the point of inflection upward. Figure 5-7 shows the relationship for displacement downward. The notation on these figures indicates order of events such as cracking, yield, etc.

The behavioral model needed for the LPM/II analysis, (Kariotis, 1992), is the behavioral model of each story height beam element. The "beam element" below each floor level is the shear wall below the level and the coupling beams on each side of the shear wall at that floor level. Figure 5-8 presents the finite element modeling of the shear wall at the first story level. The coupling beams are attached to the shear wall at elements 120 and 136. The analytical model is twice the first story height as a "dummy" wall is used above the wall analyzed to introduce moment into the lower wall section. Displacement is applied in small increments at the top of the "dummy" wall. The force necessary to cause an increment of displacement is calculated and an equal and opposite sense force is applied at the coupling beam level in conjunction with each displacement increment. The rotation at the coupling beam level and the displacements of the edge of the shear wall at the coupling beam level relative to the base are calculated. Shears and moments induced in the coupling beams by this rotation and vertical displacement are applied at the edges of the shear wall at the second floor level in an additional load step. Figures 5-9 and 5-10 show the results of the finite element analysis of two levels of the shear walls. The "base shear" is the shear in the wall below the coupling beam level and should be equal to zero. The base shear was nearly equal to zero except when the stiffness of the beam element had radical changes.

Figure 5-11 shows the stiffness degradation of the nonlinear beam element that represents the combined stiffness of first story shear wall and the coupling beams at the second floor level. The EI factor is the effective secant stiffness ratio of the degraded stiffness element. This factor begins with 100 percent of the stiffness of the uncracked masonry shear wall. The rotation angle is that of the second floor level relative to the base of the wall.

This level is considered to be fixed against rotation. The applied moment is the moment resisted by the system; the shear wall and the coupling beams. The FEM analysis procedure caused a constant moment throughout the story height of the shear wall. Figure 5-12 presents the same data for the second floor level. This beam element has the same quantity of vertical reinforcement but has a lesser story height. Figure 5-13 presents the same data for the beam elements at the third and fourth floor levels that have less vertical reinforcement.

The dynamic model of one of the four identical shear walls is shown in Figure 5-14. One-quarter of the total story mass was lumped at each story height on the weightless beam element. The shear wall is fixed at its base and the base is the point of application of the acceleration-time history of the ground motion. Internal moments are calculated by the dynamic model at each end of the beam elements that represent the story height sections of the shear wall.

5.4 Results of the Analyses

The dynamic model was shaken by the nine ground motions that were selected for representing seismic loading. The selection process, the scaling factors and the titles of the ground motions used for these dynamic analyses are given in Appendix A-2.

Table 5-1 presents the relative displacement caused by each of the ground motions at the second floor level and at the roof level. The first story height is 130 inches. The mean drift ratio at the first story level is 0.87 percent. The standard deviation of the drift ratio is 0.22 percent. The drift ratio of the top of the shear wall is 1.11 percent and has a standard deviation of 0.34 percent. This drift ratio exceeds the recommended maximum drift ratio of 1.0 percent.

Table 5-2 presents the degraded final EI factor that was caused by each of the nine ground motions for each beam element. The stiffness degradation was principally at the base of the shear wall.

A detailed study was made of the effects of the north-south component of the 1940 El Centro earthquake. Table 5-3 presents the floor masses used for this dynamic analysis. This ground motion caused the stiffness degradation listed in Table 5-4. The modal frequencies calculated using initial stiffnesses are given on Table 5-5. Mass damping equivalent to five percent of critical damping of the primary mode based on the initial stiffness was used in the dynamic nonlinear model. Table 5-6 gives the modal frequencies of the degraded model. The percent of critical damping of the fundamental mode has been greatly increased, higher modes of vibration have minor or nearly no damping.

The internal moments at the base of the shear wall, M1, and the internal moments above the second, third, and fourth floors, M3, M5, and M7 respectively, (Figure 5-14) are presented in Figures 5-15a and 5-15b. These moments were used to calculate the base shear presented in Table 5-7. The calculated internal moment at the base exceeds the expected moment capacity shown in Figure 5-11. This is due to the definition of the EI factor as a step function. The acceleration-time plots of each floor due to the N-S El Centro ground motion are given in Figures 5-16a and 5-16b. The displacements at the top of each story level are plotted in Figures 5-17a and 5-17b. These displacements are principally due to primary mode response.

The shear at each story level was calculated by the method shown in Figure 5-18. The calculations were made by use of real-time data. The base shear due to the dynamic loading of the N-S component of the 1940 El Centro earthquake is presented and plotted versus time in Figure 5-19. The story shear effects of higher modes is discernible in this plot as a high frequency response superimposed on the primary mode response. The shear at each story level, first through fourth, is given in Figures 5-20a and 5-20b. The maximum base shear caused by each of the nine ground motions is given in Table 5-7.

5.5 Discussion of the Results

The nonlinear dynamic studies of the east-west shear walls indicated that the recommended drift ratio of 1/100, measured at the top of the shear wall was exceeded. The design calculations, using the estimated effective of stiffness, also predicted excessive drift. The mean displacement determined by the dynamic studies exceeded the recommended limit by eleven percent. The displacement calculated in the design process substantially over-estimated the dynamic displacement. Part of this over-estimation is probably due to the recommendation that the displacement amplification coefficient, C_d, be equal to the response modification coefficient, R. These coefficients would be equal if the design expected strength were near equal to the expected strength that was calculated by the nonlinear finite element method. The "overstrength" of the coupling beams and the shear wall reduces the "ductility" coefficient part of R to less than 4½.

The major difference between expected strengths calculated in the design process and the strengths calculated in the finite element analysis was in the coupling beam strengths. The flexural moment strengths of the coupling beams, calculated in the design process for top in tension and bottom in tension, was 820 inch-kip and 3,900 inch-kips, respectively. The peak strengths of the coupling beams for similar displacements were 1,735 inch-kips and 5,157 inch-kips, respectively when calculated by the FEM analysis. These peak values were due to the tensile strength capacity of the concrete section. This strength capacity is not accounted for by the usual design process. The post-cracking strengths are shown in Figure 5-7 and Figure 5-6 as about 1000 inch-kips and 2,400 inch-kips. These post cracking strength values are increased by strain hardening of the reinforcement. Strain hardening is not considered in the methods used in the design process for calculation of expected strength. The peak combined flexural strength of the system, the first story shear wall and the two coupling beams was determined as 134,300 inch-kip by the finite element analysis. The value calculated in the design process was 108,070 inch-kip. The ratio of the peak strengths calculated by the FEM analysis to that of the design process is 1.25. This overstrength factor indicates that the C_d coefficient should be 3.6. When this value of C_d is used for amplification of the deflection calculated by the equivalent lateral force method, the predicted drift ratio is 0.0112.

The dynamic analysis determined a moment, M1, at the base of the shear wall of 158,563 inch-kips, Figure 5-15a. The reason for this moment that exceeds the peak moment capacity is that the effective stiffness of the beam element at this story level was determined by use of a moment applied at the top of the beam element and having the same value throughout the length of the element. The beam elements used in LPM/II have a moment variation within their length and the moment is calculated as θ, the curvature at the end of the beam, multiplied by the effective EI factor. The effective EI factor is determined by the moment at the top of the element, M2. That moment is equal to M3 which is slightly less than 120,000 inch-kips.

The finite element analysis indicated that the shear capacity of the wall is greater than that predicted by the Matsumura equation. Figure 5-9 and 5-10 shows that the dummy wall above the first and second story wall and coupling beam system was loaded by forces in excess of 1000 kips. This strength is partly due to the use of a larger quantity of horizontal reinforcement in the dummy wall to minimize the possibility of shear overstress in the dummy wall. A major reason for the conservatism in the prediction of Matsumura formula was that a section of the floor and its reinforcement was included in the finite element model. Elements 120 through 136 are heavily reinforced concrete elements with a width of the two precast planks that are adjacent to the shear wall. Equations commonly used for prediction of shear strength use only that reinforcement uniformly distributed in the height of the wall. When the story height of a shear wall is less than the length of the wall and a reinforced concrete floor is used, under prediction of expected shear capacity should be anticipated.

The internal moments in the shear wall system shaken by the N-S component of the 1940 El Centro earthquake peak in the first five seconds of the record (Figure 5-15a). The peak displacement in the first story level and at the top of the shear wall peak just before six seconds into the record (Figure 5-17a and b), and are related to a decrease in flexural moment. The effective fundamental period of the wall also increases. This data indicates that the shear wall has had strength and stiffness degradation as predicted by the finite element analysis when the curvature at peak strength is exceeded. The floor and roof accelerations, Figure 5-16a and b, increase in value as primary mode displacement predominates.

The base shear of the wall shaken by the N-S component of the 1940 El Centro earthquake peaks at about 600 kips. This value occurs early in the response to the ground motion. The base shear drops after six seconds into the record. This is when the peak displacement occurs.

Two ground motions, both recorded near the fault system in the 1979 Imperial Valley earthquake caused a base shear larger than the average. The variation in shear over the height of the shear wall was relatively minor. The shear at the fourth level was about 2/3 of the shear at the base of the building. This data confirms that the design shear should have only a moderate reduction in the levels above the first floor level. The only significant discrepancy between the shear distribution used in the design process and the dynamic analysis is at the fourth story level. The design process assumed 46 percent of the base shear at this level. The dynamic analysis indicated 62 percent of the base shear at this level may be more appropriate.

TABLE 5-1
DISPLACEMENTS AT FIRST STORY AND AT ROOF

GROUND MOTION	DISPLACEMENT 2ND FLOOR	DISPLACEMENT ROOF	
1	1.14	6.89	EL CENTRO E-W
	-0.71	-4.39	
2	0.89	5.41	EL CENTRO N-S
	-0.91	-5.62	
3	0.97	5.89	PINE UNION 140°
	-0.87	-5.59	
4	0.96	5.59	CRUICKSHANK ROAD 230°
	-1.14	-6.70	
5	0.65	4.02	JAMES ROAD 140°
	-0.44	-2.95	
6	0.96	5.83	KERN CO. 69°
	-0.84	-5.12	
7	1.53	8.92	CRUICKSHANK ROAD 140°
	-1.17	-6.77	
8	0.45	3.04	BRAWLEY 315°
	-0.35	-2.25	
9	0.73	4.58	KEYSTONE 140°
	-0.98	-5.88	
MEAN	0.87	5.30	
STD. DEV.	0.29	1.61	

1ST STORY HEIGHT = 130 INCHES
OVERALL HEIGHT = 478 INCHES

TABLE 5-2

GROUND MOTION #1

BEAM NO.	EI FACTOR
1	0.020
2	0.302
3	0.694
4	0.979

GROUND MOTION #2

BEAM NO.	EI FACTOR
1	0.025
2	0.258
3	0.578
4	0.979

GROUND MOTION #3

BEAM NO.	EI FACTOR
1	0.025
2	0.284
3	0.645
4	1.000

GROUND MOTION #4

BEAM NO.	EI FACTOR
1	0.025
2	0.258
3	0.414
4	0.854

GROUND MOTION #5

BEAM NO.	EI FACTOR
1	0.066
2	0.258
3	0.544
4	0.979

GROUND MOTION #6

BEAM NO.	EI FACTOR
1	0.025
2	0.284
3	0.532
4	0.854

GROUND MOTION #7

BEAM NO.	EI FACTOR
1	0.020
2	0.284
3	0.521
4	0.979

GROUND MOTION #8

BEAM NO.	EI FACTOR
1	0.089
2	0.349
3	0.824
4	1.000

GROUND MOTION #9

BEAM NO.	EI FACTOR
1	0.025
2	0.339
3	0.694
4	0.979

TABLE 5-3
FLOOR MASSES

FLOOR NO.	MASS (K-S-S/IN)
1	0.781
2	0.767
3	0.767
4	0.595

TABLE 5-4
DEGRADATION OF STIFFNESS

BEAM NO.	FINAL DEG. STIFFNESS RATIO	INITIAL STIFFNESS INCHES^4
1	0.0253	$3.507X10^{10}$
2	0.2579	$3.507X10^{10}$
3	0.5782	$3.507X10^{10}$
4	0.9792	$3.507X10^{10}$

TABLE 5-5
MODAL FREQUENCY BASED ON ORIGINAL STIFFNESS

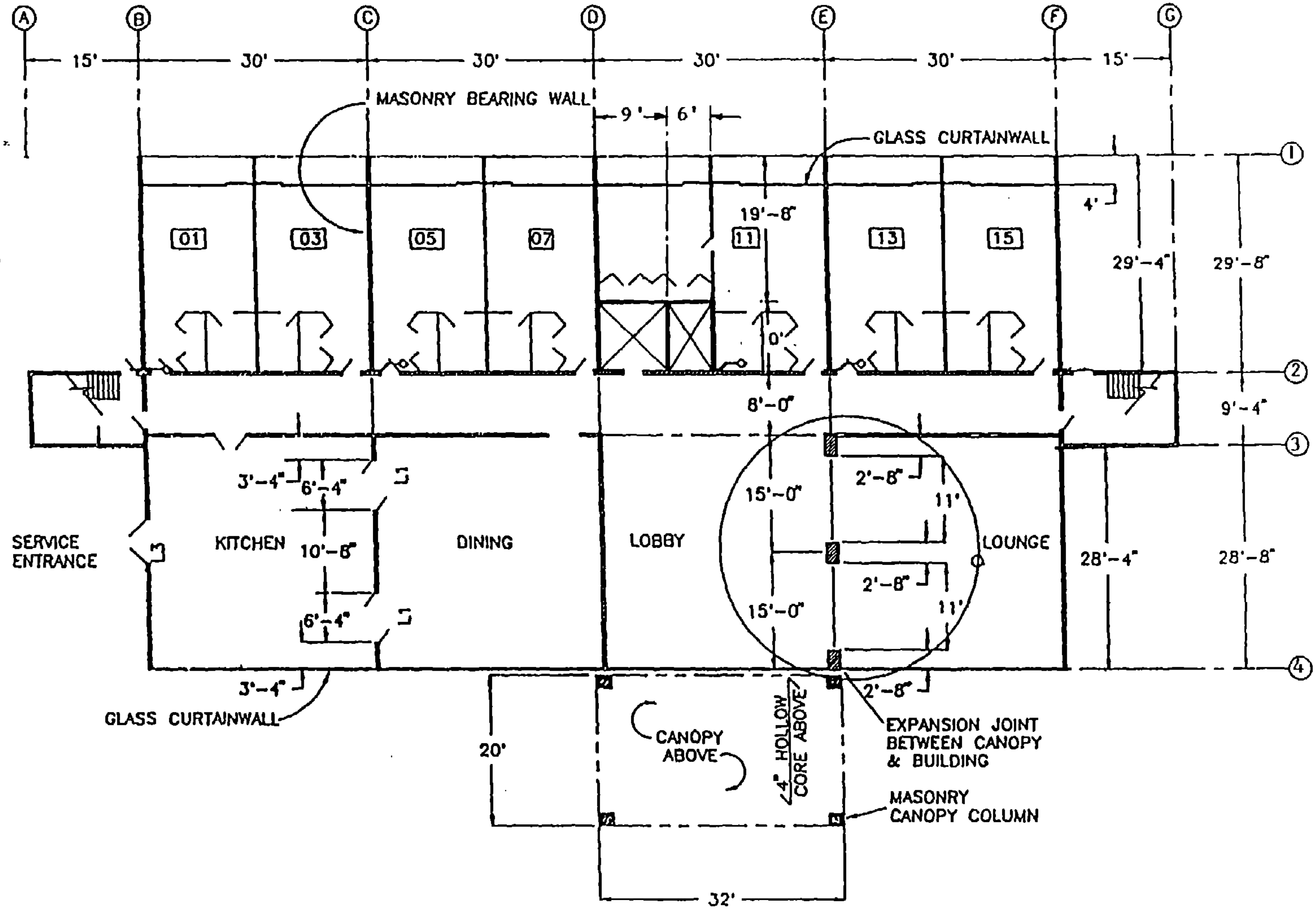
MODE NO.	FREQUENCY (C/SEC)	PERIOD (SEC.)	DAMPING %
1	4.898	0.204	5.00
2	30.156	0.033	0.82
3	83.070	0.012	0.29
4	151.349	0.007	0.16

TABLE 5-6
MODAL FREQUENCY BASED ON FINAL DEGRADED STIFFNESS

MODE NO.	FREQUENCY (C/SEC)	PERIOD (SEC.)	DAMPING %
1	0.913	1.095	26.82
2	11.082	0.090	2.23
3	36.034	0.028	0.68
4	92.107	0.011	0.26

TABLE 5-7
MAXIMUM BASE SHEAR (KIPS)

GROUND MOTION	BASE SHEAR
1	522.65
2	653.77
3	586.27
4	829.08
5	606.39
6	629.93
7	902.28
8	555.37
9	593.51
AVERAGE	653.25
STD. DEV.	127.65



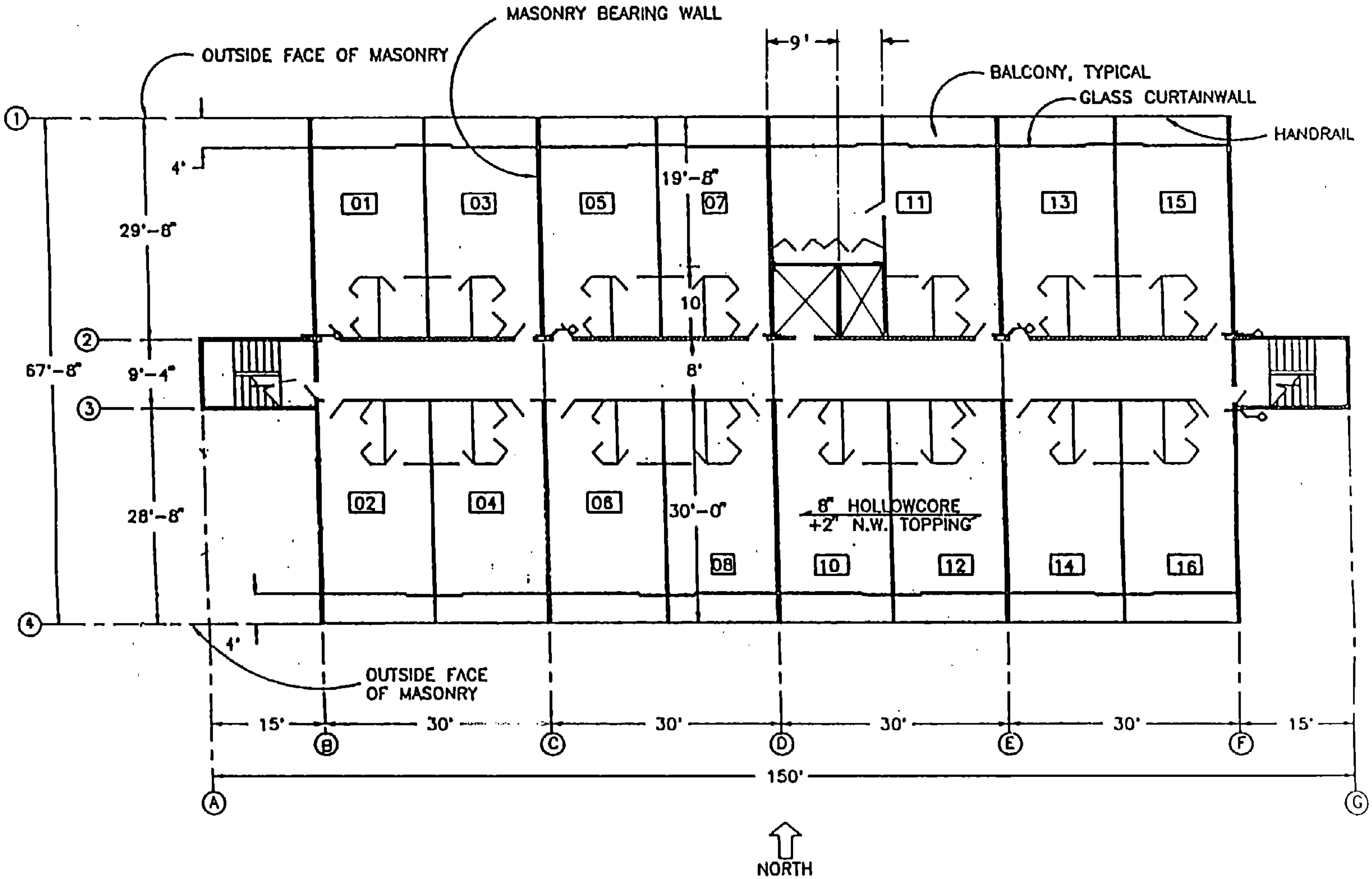
RCJ HOTEL: 1ST FLOOR PLAN

TOP OF FLOOR ELEVATION = 100'-0"

NORTH

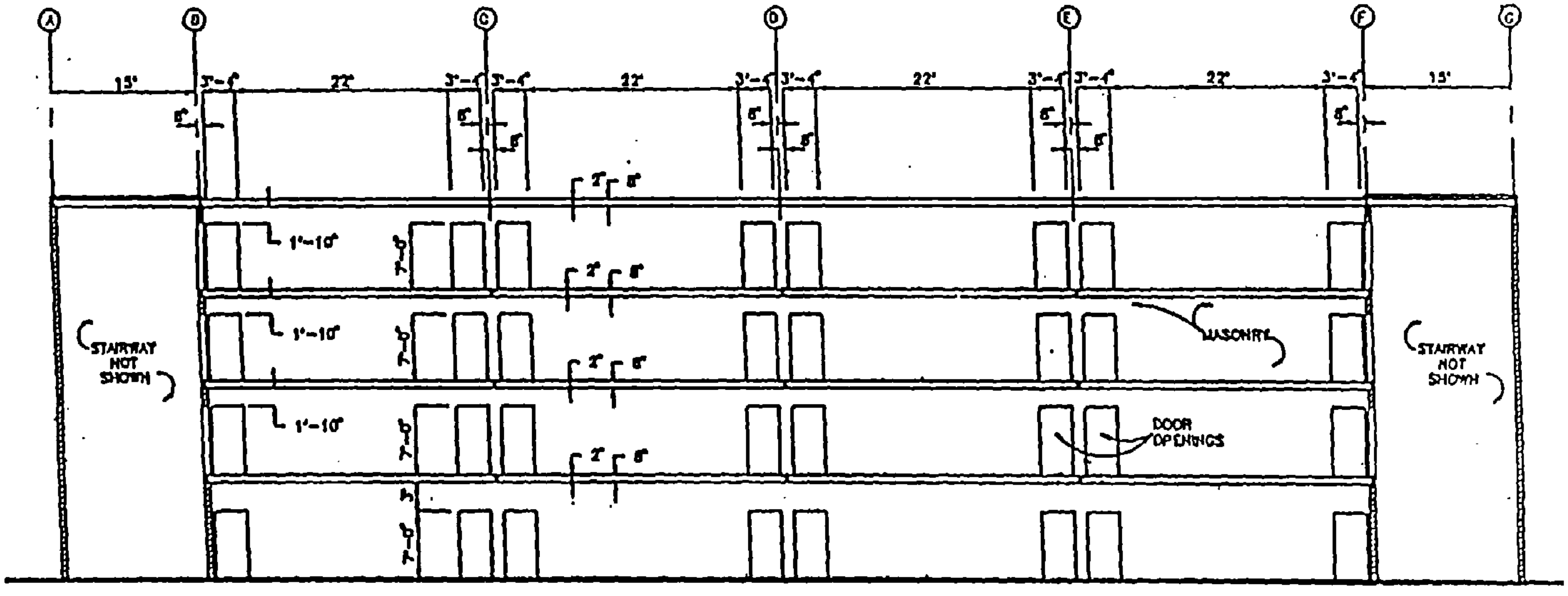
FIGURE 5-1

A DEEP REINFORCED MASONRY BEAM SPANS FROM MASONRY COLUMN AT E-4 TO COLUMN E-3.



2ND, 3RD, & 4TH FLOOR PLAN

FIGURE 5-2



BUILDING SECTION LOOKING NORTH

FIGURE 5-3

<u>RCJ HOTEL NOTES</u>

1. Design Dead Loads

Roof (Hotel)	95 psf	
Roof (Canopy)	50 psf	
Floor (includes partitions)	110 psf	
Glass Curtain wall	10 psf	

2. Design Live Loads

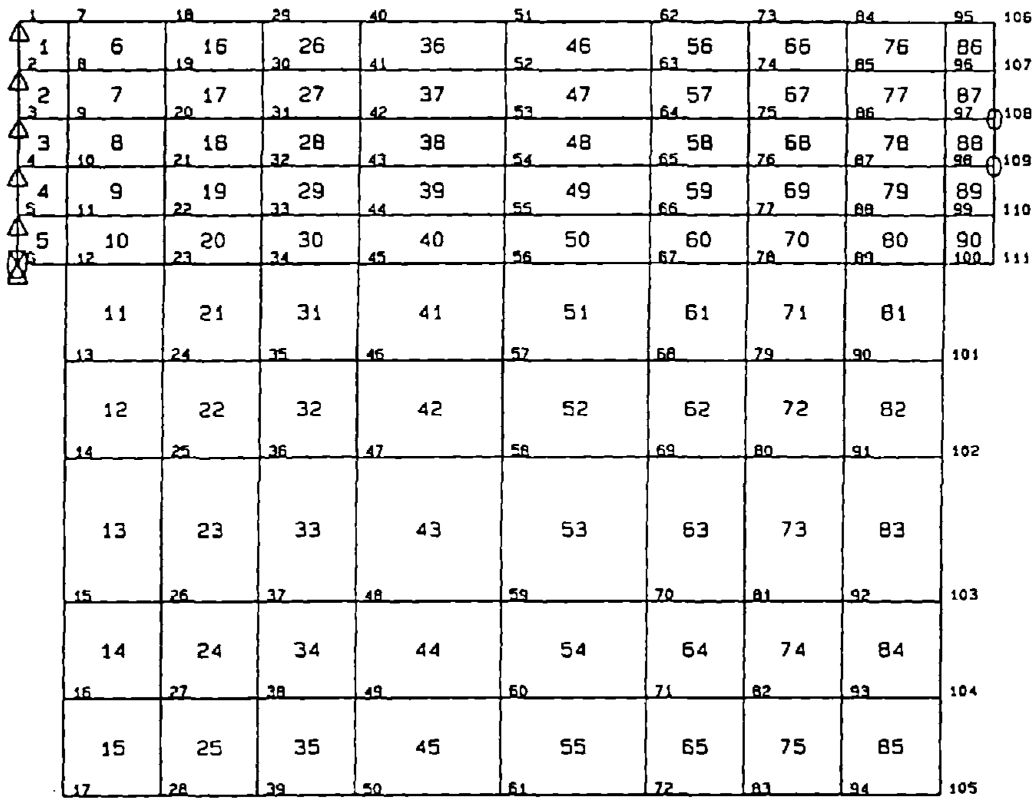
Roof	20 psf	No Snow
Dwelling Rooms	40 psf	
Public Rooms	100 psf	
1st Floor Corridor	100 psf	
Corridors above 1st	100 psf	
Stairways	100 psf	
Wind pressure or suction on vertical surfaces	25 psf	
Wind uplift on open roofs	40 psf	
Seismic Zone	Four	

3. Soil Conditions

Allowable soil bearing pressure	4000 psf
Equivalent fluid pressure	30 pcf

4. Building Construction
   a. Floor and Masonry Elevations
      - 10'-10" First floor to second floor
      - 9'-8" Floor to floor above second floor
      - 40'-4" Overall masonry wall height
   b. Roof and Floor Construction
      - 8" precast hollowcore planks with 2" thick normal-weight topping at Hotel
      - 4" precast hollowcore planks with no topping at Canopy
   c. Wall Construction
      - Reinforced, Seismic Zone 4
      - All walls - single wythe hollow concrete masonry
   d. Canopy Construction
      - Beams and columns - reinforced concrete block masonry
   e. Masonry Openings
      - All door openings are 3'-4" wide x 7'-0" high, unless noted otherwise

FIGURE 5-4

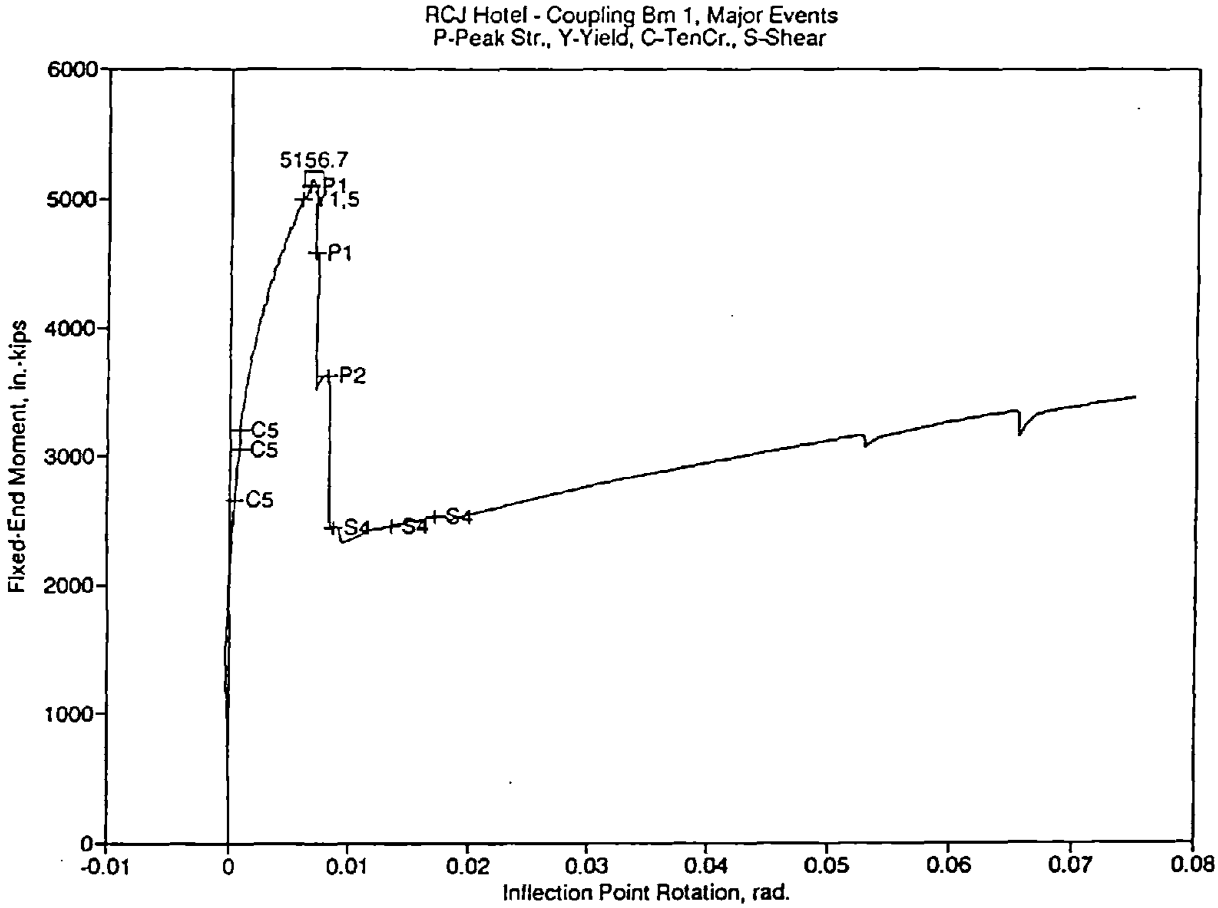


From: R.D. Ewing, (1992) Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard. Ewing & Associates, Rancho Palos Verdes, CA.

Scale = 8.00

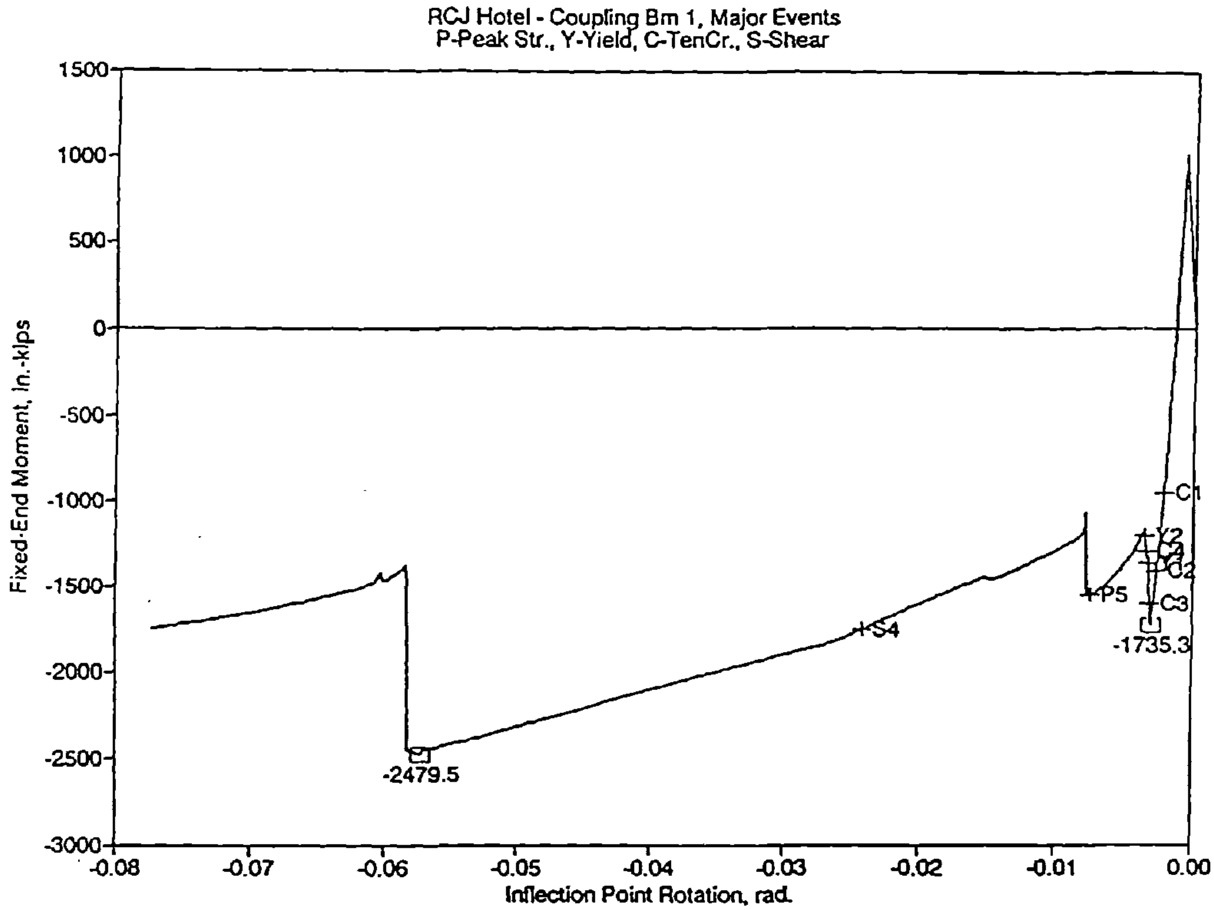
RCJ Hotel - Coupling Beam 1

FIGURE 5-5



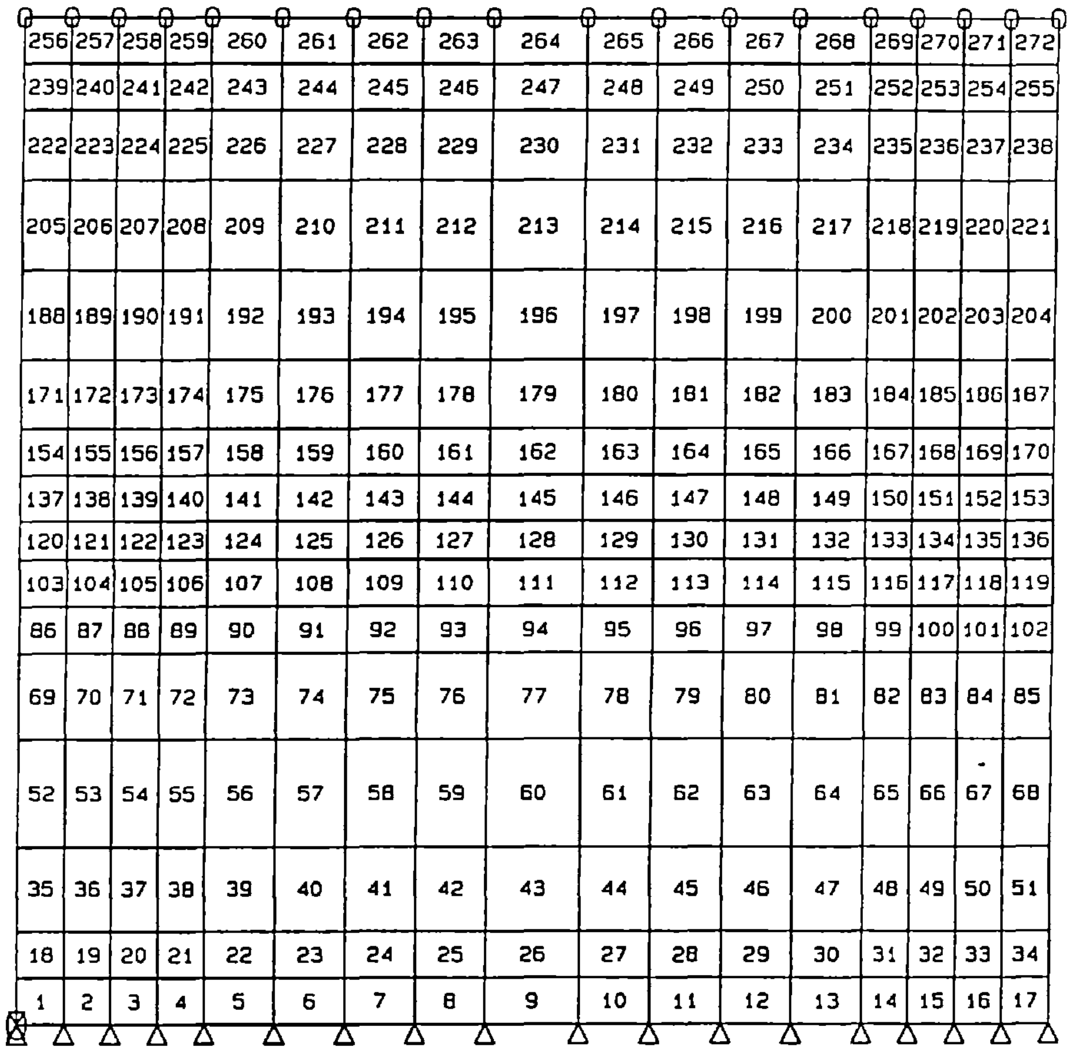
From: R.D. Ewing, (1992) Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard. Ewing & Associates, Rancho Palos Verdes, CA.

FIGURE 5-6



From: R.D. Ewing, (1992) Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard. Ewing & Associates, Rancho Palos Verdes, CA.

FIGURE 5-7



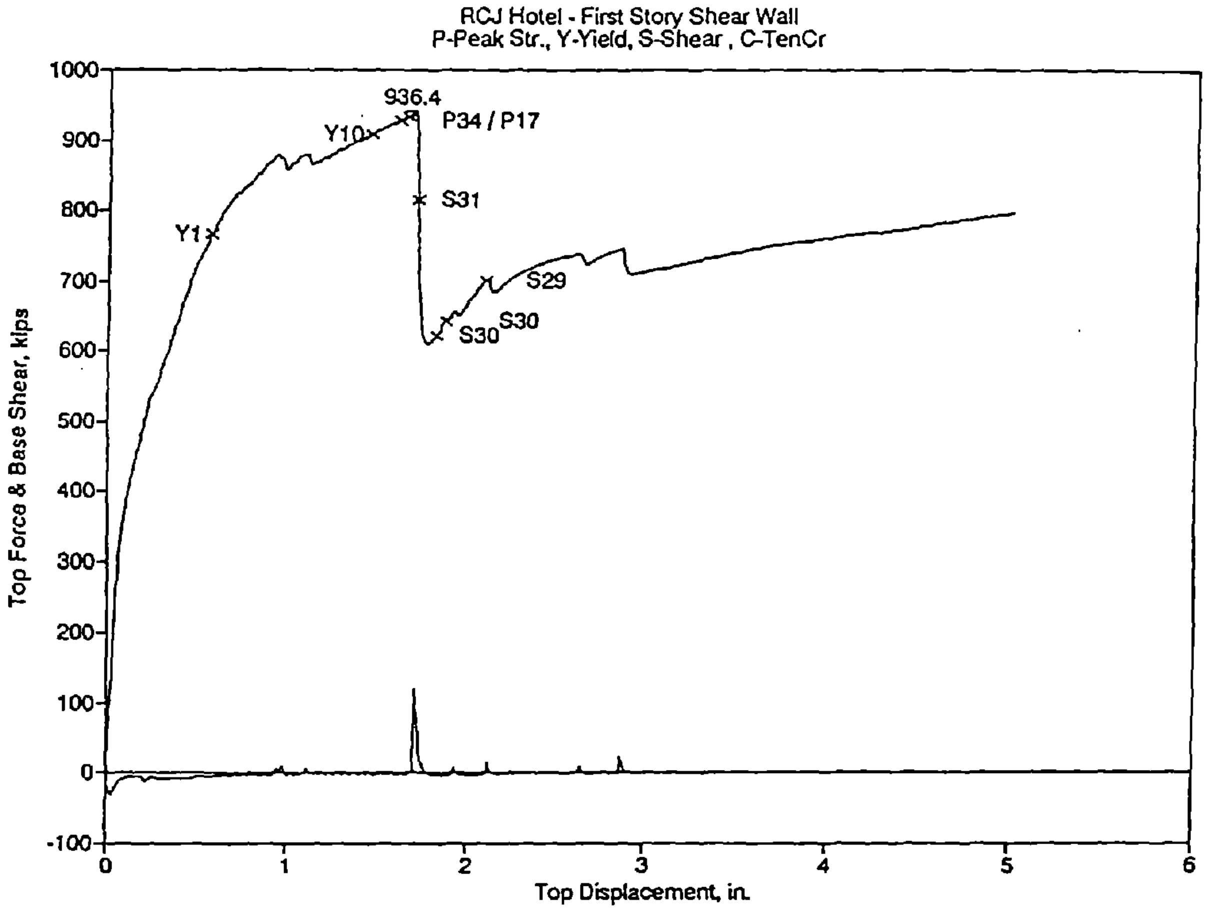
Element Numbers

From: R.D. Ewing, (1992) Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard. Ewing & Associates, Rancho Palos Verdes, CA.

Scale = 50.00

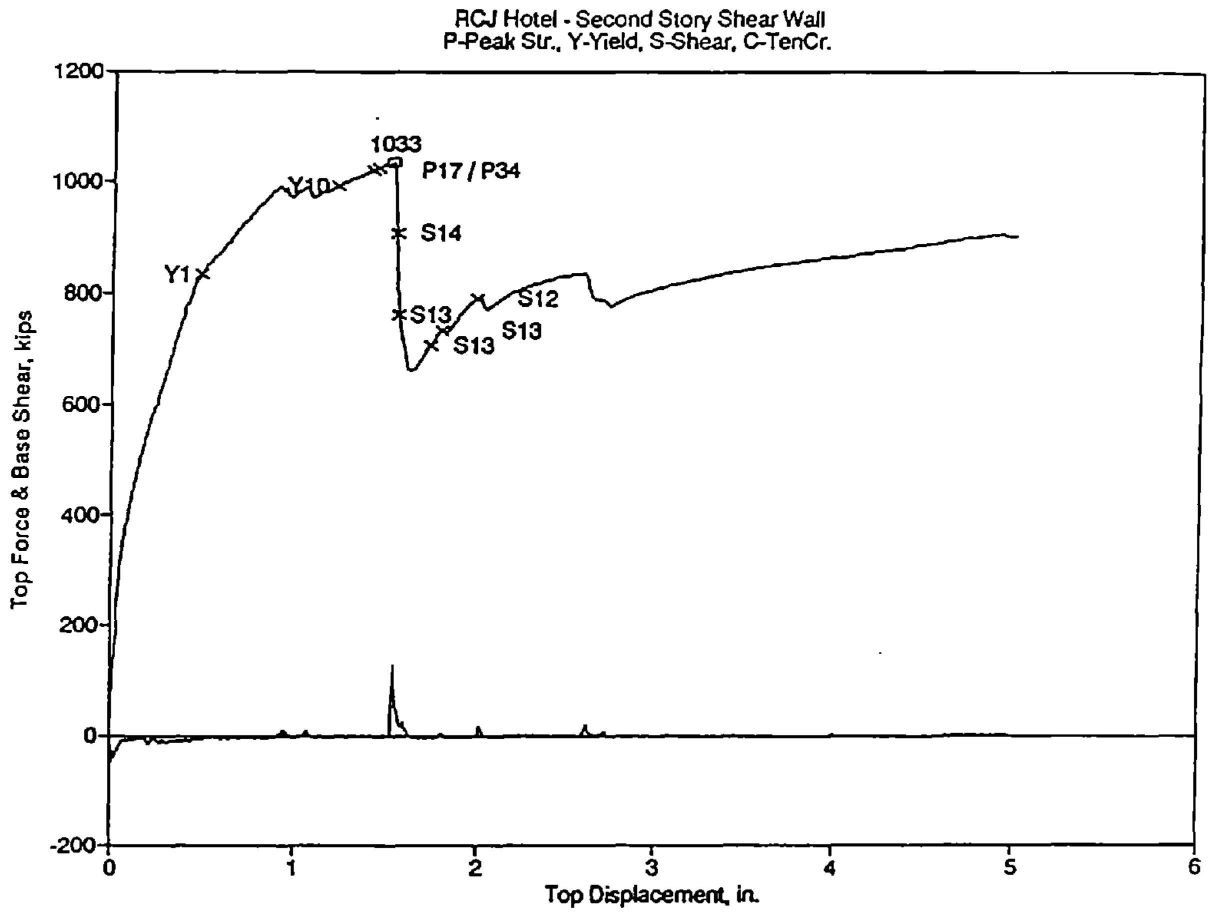
RCJ Hotel - First Story Coupled Shear Wall

FIGURE 5-8



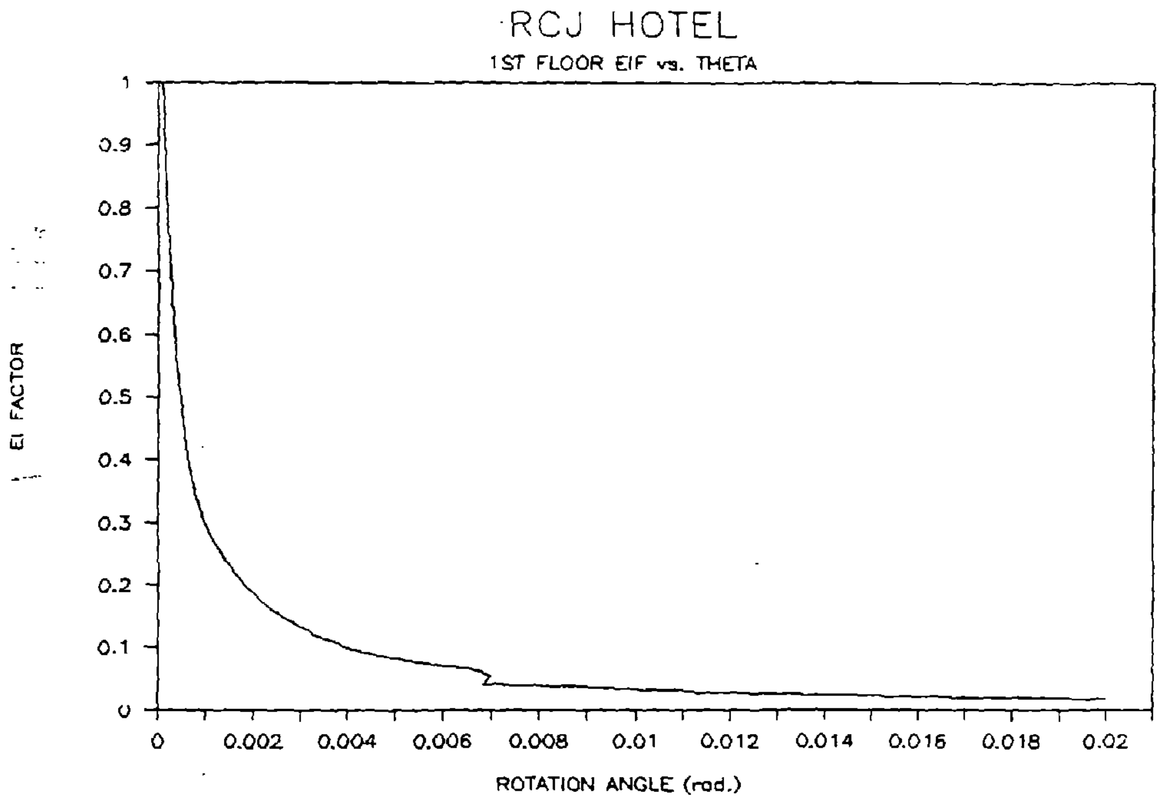
From: R.D. Ewing, (1992) Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard. Ewing & Associates, Rancho Palos Verdes, CA.

FIGURE 5-9



From: R.D. Ewing, (1992) Finite Element Analysis of Reinforced Masonry Building Components Designed by a Tentative Masonry Limit States Design Standard. Ewing & Associates, Rancho Palos Verdes, CA.

FIGURE 5-10


RCJ HOTEL
1ST FLOOR EIF vs. THETA
EI FACTOR
ROTATION ANGLE (rad.)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
0.02

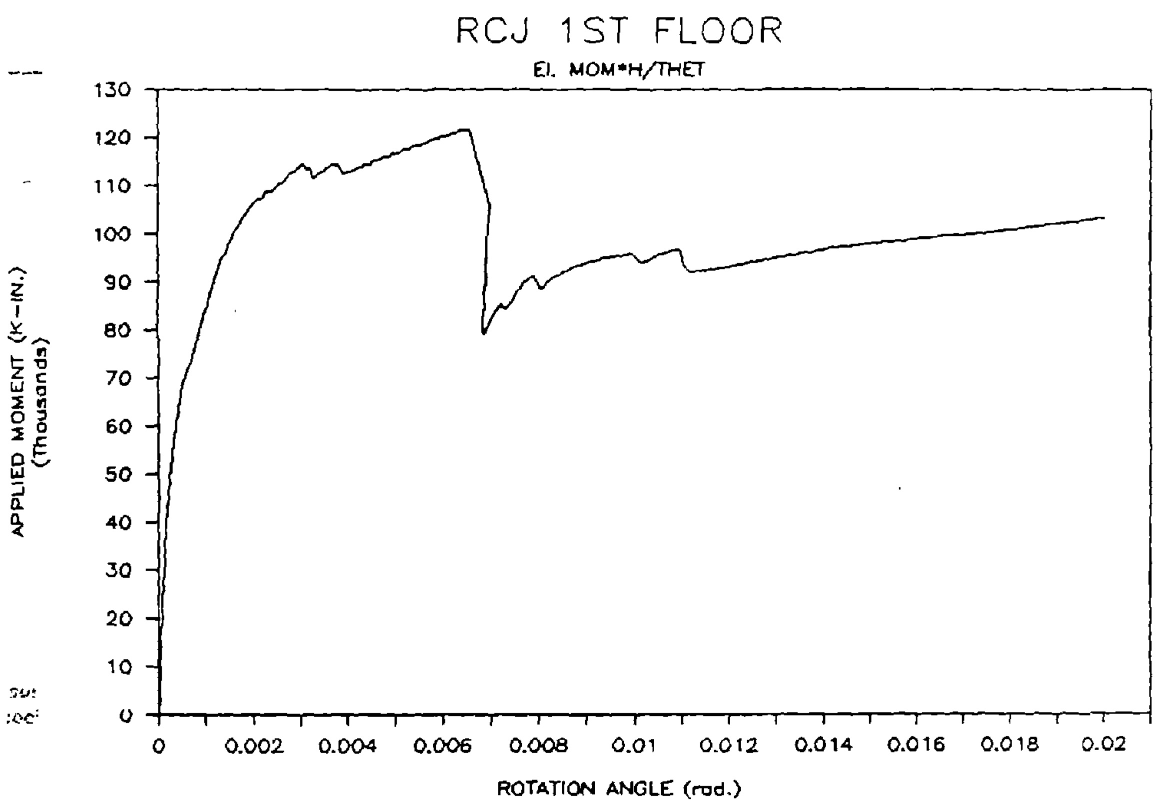
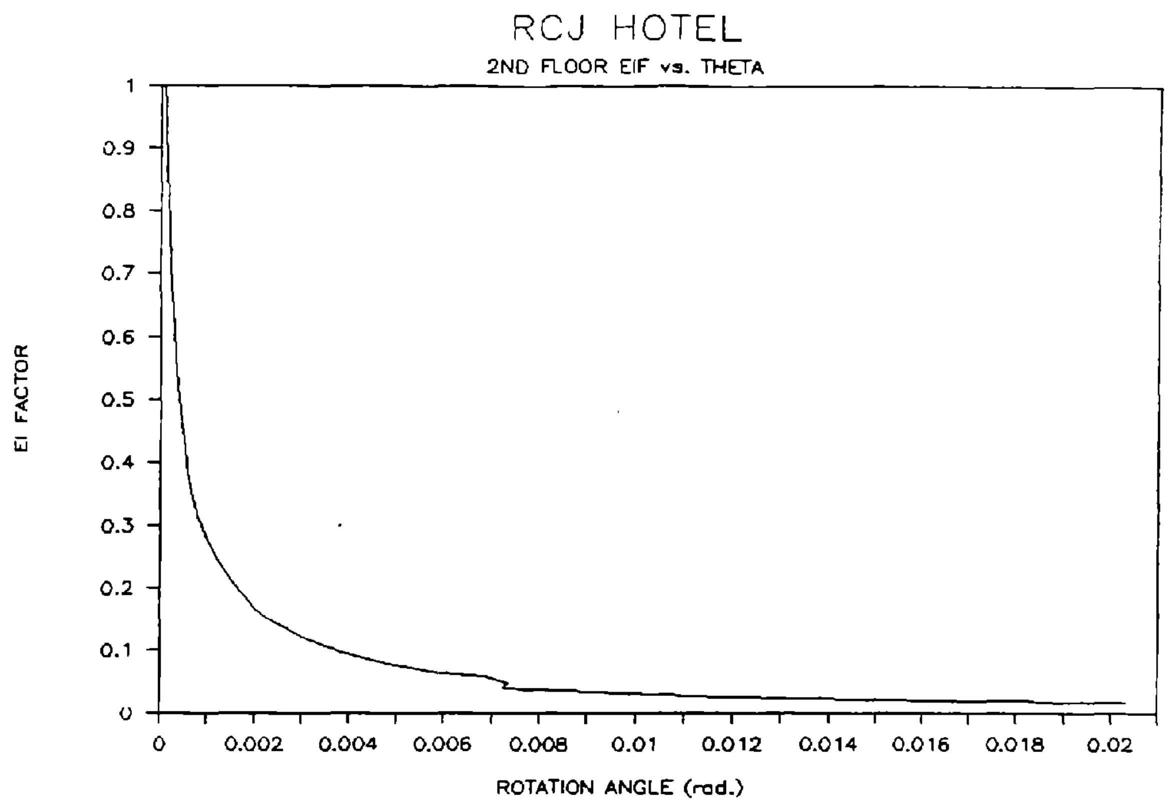

RCJ 1ST FLOOR
EI. MOM*H/THET
APPLIED MOMENT (K-IN.)
(Thousands)
ROTATION ANGLE (rad.)
0
10
20
30
40
50
60
70
80
90
100
110
120
130
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
0.02

FIGURE 5-11


RCJ HOTEL
2ND FLOOR EIF vs. THETA
EI FACTOR
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
0.02
ROTATION ANGLE (rad.)

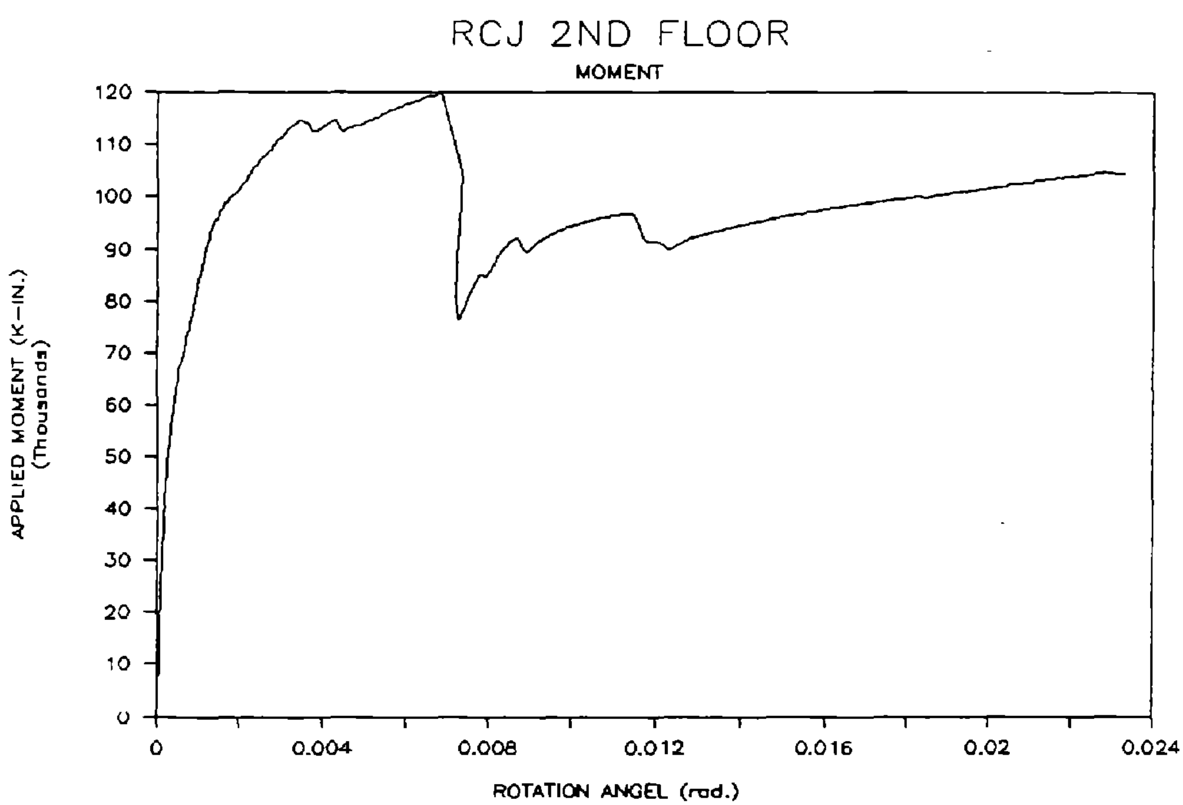

RCJ 2ND FLOOR
MOMENT
APPLIED MOMENT (K-IN.)
(Thousands)
120
110
100
90
80
70
60
50
40
30
20
10
0
0
0.004
0.008
0.012
0.016
0.02
0.024
ROTATION ANGEL (rad.)

FIGURE 5-12

higher than that of the diaphragm used in this study. The stiffness deterioration in the diaphragm, adjacent to the rear wall that has excess strength, would not occur and the damage to the 20 feet long shear wall on the other side of the building would not occur. However, the studies with the two plywood diaphragms did not have a strong indication that a 50 percent increase in strength would have a significant effect. If the diaphragm design had included an analysis that is based on relative rigidity of the shear walls, the required diaphragm strength would be nearly doubled. Additional studies of buildings with plan irregularity should be made to develop diaphragm design recommendations.

The dynamic behavior of the walls loaded by earthquake motions normal to their plane was elastic when defined as no yielding of the reinforcement. The relative deformation of the center of the twenty-four feet tall walls was 1.4 percent of its height. The relative deformation of the middle of the 28 feet high walls was nearly the same even though this wall was shaken at its top by a stiffer diaphragm. The deformation of the center of this wall appeared to be insensitive to the diaphragm stiffness.

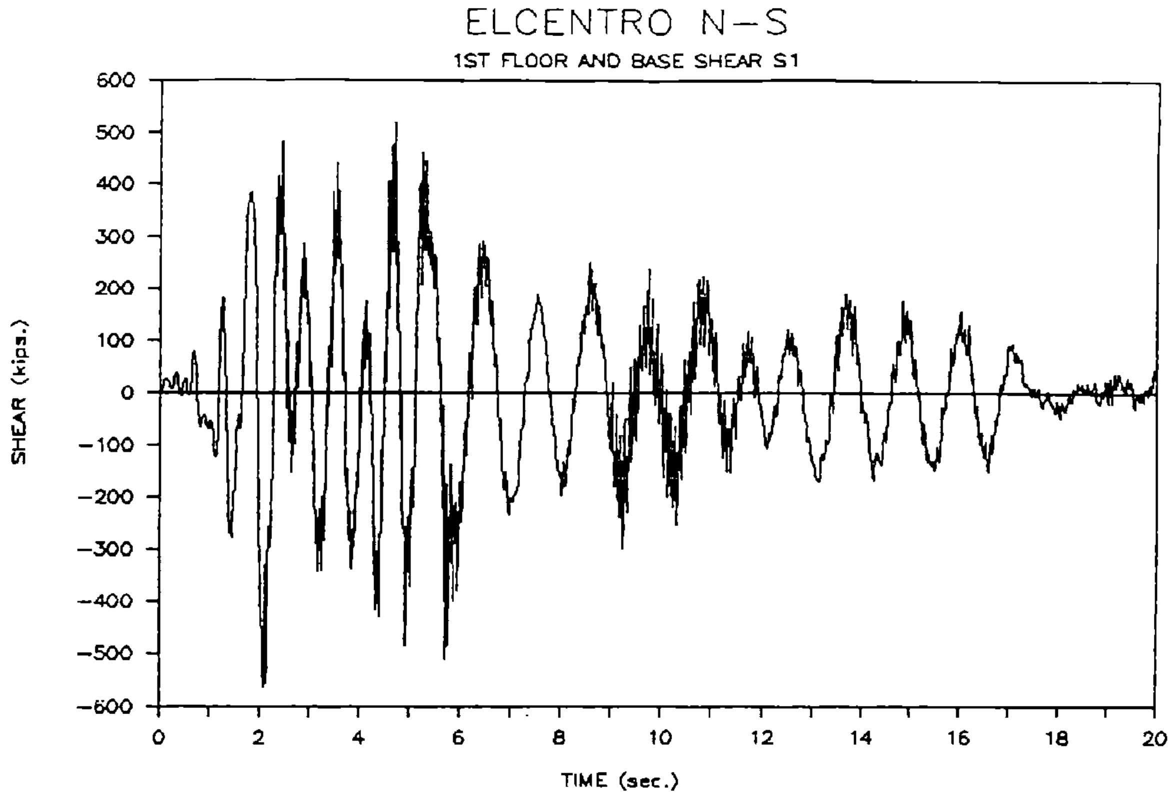
In summation, the draft Limit State Design Standards, have produced adequate designs for single story buildings where the behavior is dominated by diaphragm response. The dynamic behavior of buildings with plan irregularity, such as the TMS Center, does need further studies for improvement of the current Recommended Provisions.

The multi-story, reinforced masonry building, the RCJ Hotel, also indicated that the draft Standards produces buildings that have adequate dynamic behavior. The design calculations indicated that the drift limits for the coupled masonry shear walls were exceeded. The dynamic analyses confirmed the predictions of the calculations.

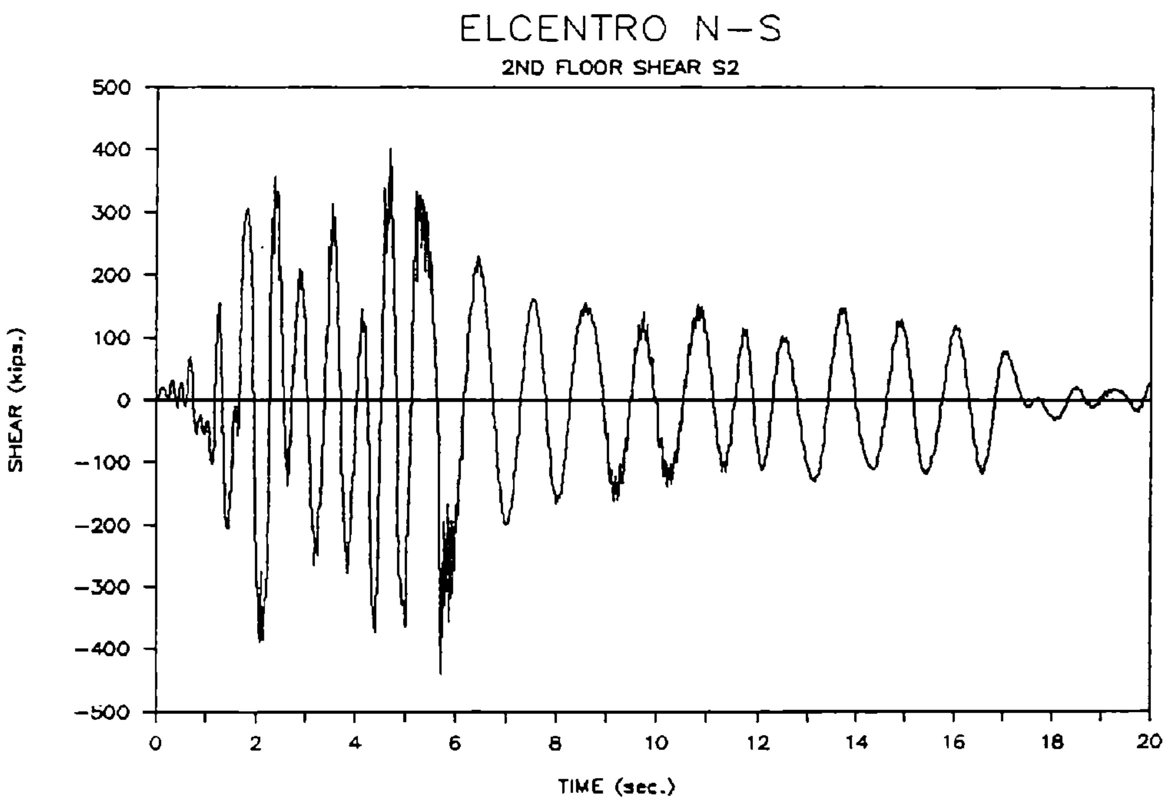
The dynamic analyses of the multi-story shear wall showed that the interstory dynamic shear is substantially different from the shears that are determined by the seismic design loading. The design base shear of one of the four identical shear walls was 250 kips. The mean dynamic base shear calculated by the nine ground motions was 653 kips. Time-history studies of this base shear (Figure 5-19) indicated that higher modes of vibration had a very significant influence on the value of the base shear. The studies of Section 5 did not use structural damping. Additional analyses were made to examine the effects of use of structural damping. Figure 6-1a and 6-1b show that two percent of critical structural damping for the highest mode significantly reduced the contribution of higher modes of vibration to the base shear. The mass damping reduced the base shear to about 550 kips. The shear at the fourth floor was reduced to about 250 kips from nearly 400 kips. The small variation from the base to the top of the wall in the dynamic response shear was unexpected. Additional studies of the probable dynamic interstory shear and the influence of higher modes of vibration on the interstory shear values are needed.

The design procedure assumed the expected strength of the coupling beams and the wall could be added and used as the total expected strength. The strength of the coupling beams of the RCJ Hotel was very poorly estimated by usual strength computations. The reason was that the cracking strength of the prestressed planks greatly exceeded the post-cracked strength. The coupled shear wall had a 30 percent drop in flexural strength at a drift ratio, measured as top displacement vs. overall height, of 1.35 percent. The expected base shear associated with the peak flexural strength was 936 kips. The flexural strength calculated by the finite element analysis exceeded that calculated by the procedure recommended by the draft Standards. These discrepancies indicated that a small amount of overstrength must be included in the response modification factor, R, and that the displacement amplification factor, C_d, should be less than R.

These studies indicated that the procedure used in this research, design of buildings in accordance with a recommended procedure and followed by analysis, nonlinear analysis using scaled time-histories is useful for the confirmation of the adequacy of the recommended procedures. These studies used low-rise buildings for examples. Further research should design and analyze more complex buildings. Each building should be analyzed as a complete three-dimensional structure. The studies of the TMS Center provided insight into the dynamic behavior of irregular buildings. This type of buildings comprise the majority of masonry buildings that are constructed in the United States.

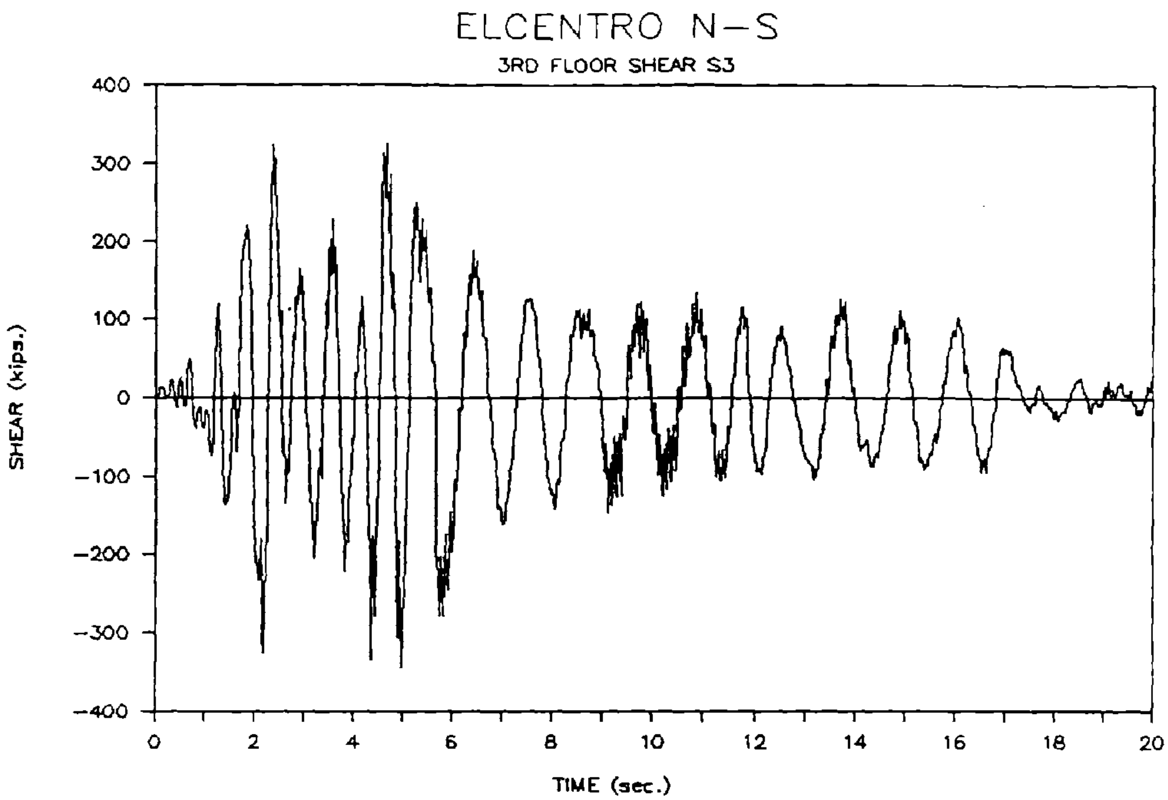


ELCENTRO N-S
1ST FLOOR AND BASE SHEAR S1
SHEAR (kips.)
TIME (sec.)



ELCENTRO N-S
2ND FLOOR SHEAR S2
SHEAR (kips.)
TIME (sec.)

FIGURE 6-1a


ELCENTRO N-S
3RD FLOOR SHEAR S3
SHEAR (kips.)
TIME (sec.)
400
300
200
100
0
-100
-200
-300
-400
0
2
4
6
8
10
12
14
16
18
20

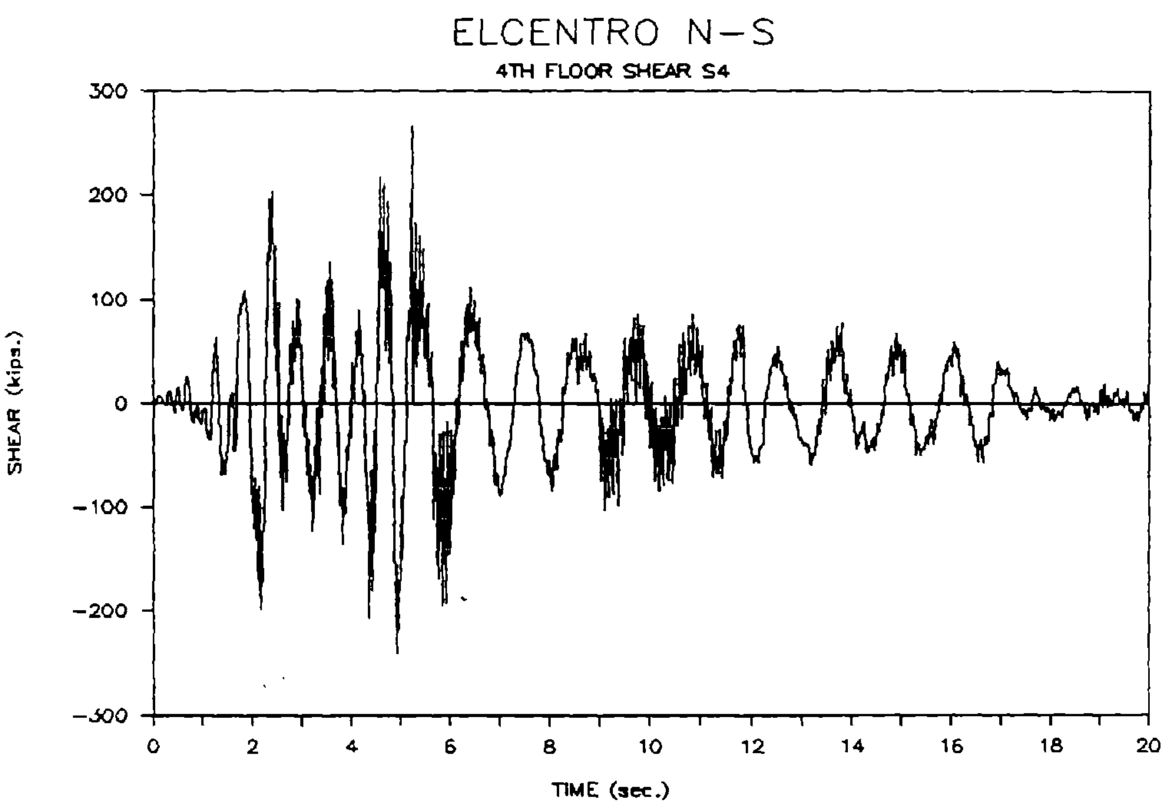

ELCENTRO N-S
4TH FLOOR SHEAR S4
SHEAR (kips.)
TIME (sec.)
300
200
100
0
-100
-200
-300
0
2
4
6
8
10
12
14
16
18
20

FIGURE 6-1b

SECTION 7

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APPENDIX A-1

TENATIVE MASONRY LIMIT STATES

DESIGN STANDARD

CHAPTERS 6, 7, AND 8

DRAFT OF MAY 8, 1992

CHAPTER 6

GENERAL DESIGN REQUIREMENTS

6.0 NOTATIONS

A_s	=	area of tension reinforcement, sq. in.
A_1	=	bearing area, sq. in.
A_2	=	effective bearing area, sq. in.
E_{me}	=	expected modulus of elasticity of masonry, psi.
f_{me}	=	expected compressive strength of masonry at the age of 28 days, psi.
f_{re}	=	expected modulus of rupture of masonry, psi.
f_{ye}	=	expected yield strength of reinforcement, psi.
G_{me}	=	expected shear modulus of masonry, psi
U	=	required strength to resist factored loads, or related internal moments and forces.
ϕ	=	capacity reduction factor.

6.1 GENERAL

This Standard covers the design of structures constructed of materials meeting the requirements of Chapter 2 and designed in accordance with the Limit States criteria set forth herein.

6.2 STRENGTH LIMIT STATE

The nominal strength of the structure as a whole and of each structural element, reduced by the appropriate ϕ factor, shall equal or exceed the required strength, U. In members required to deform inelastically, the nominal strength as governed by brittle failure modes (shear, compression, anchorage), reduced by the appropriate ϕ factor, shall equal or exceed the actions consistent with the expected flexural strength of the element, unreduced by any ϕ factors.

6.2.1 REQUIRED STRENGTH

The required strength, U to resist design loads shall be determined in accordance with Chapter 7.

6.2.2 DESIGN STRENGTH

The design strength shall be taken as the expected strength multiplied by a capacity reduction factor, ϕ.

6.3 STRUCTURAL CONTINUITY CONSIDERATIONS

The structural system shall be composed of structural elements designed to provide a continuous load path between each point of load application and the foundation.

6.3.1 DIAPHRAGM REQUIREMENTS

In structural systems relying on floor and roof diaphragms to transfer lateral loads to shear walls or other lateral load resisting elements, the diaphragm and connections shall be designed to transmit the design loading and thus provide a continuous load path.

The deflection in the plane of the diaphragm (as determined from analysis) shall not exceed the permissible deflection of the attached elements. Permissible deflection shall be that deflection which will permit the attached element to maintain is structural integrity under the individual loading and continue to support the prescribed loads without endangering the occupants of the building.

6.3.2 LATERAL LOAD RESISTING ELEMENTS

Shear walls, frames, and other lateral load resisting elements and connections shall be designed to resist design loads in accordance with principles of mechanics which shall include a load distribution analysis based on the elements' expected stiffness and their end support conditions.

6.4 EXPECTED MATERIAL VALUES

6.4.1 MASONRY

6.4.1.1 EXPECTED COMPRESSIVE STRENGTH

The engineer/architect of record shall specify the value of the expected compressive strength used in the design. This value shall be equal to or greater than 1500 psi. Compliance of the compressive masonry strength with the expected compressive strength shall be based on tests (ASTM CXXX, Test Method for Masonry Prisms). In lieu of testing, expected compressive strength requirements may be satisfied by using masonry units with strength as shown in Table 6.4.1.

Table 6.4.1 Compliance with the Specified Compressive Strength of Masonry		
f_{me}	Required Compressive Strength of Masonry Units, psi	
Clay Masonry:	Type M or S Mortar:	Type N Mortar
1500	4400	5500
2000	6400	8000
2500	8400	10500
3000	10400	13000
3500	12400	---
4000	14400	---
Concrete Masonry:		
1500	1900	2150
2000	2800	3050
2500	3750	4050
3000	4800	5250

6.4.1.2 EXPECTED MODULUS OF RUPTURE (TENSILE STRENGTH OF MASONRY IN FLEXURE)

The expected modulus of rupture for masonry shall be

$$f_{re} = 4.5 \sqrt{f_{me}} \qquad \text{Eq. 6.4-1}$$

6.4.1.3 EXPECTED BEARING STRENGTH

The expected bearing strength on masonry is $\phi(0.85 f_{me} A_1)$, except when the supporting surface is wider on all sides than the loaded area, the design bearing strength on the loaded area may be multiplied by $(A_2/A_1)^{0.5}$, but not more than 2.

6.4.1.4 EXPECTED MODULUS OF ELASTICITY OF MASONRY

The expected modulus of elasticity for concrete masonry shall be:

$$E_{me} = 550\ f_{me} \qquad \text{Eq. 6.4-2}$$

and for clay masonry shall be

$$E_{me} = 700\ f_{me} \qquad \text{Eq. 6.4-3}$$

$$G_{me} = 0.4\ E_{me} \qquad \text{Eq. 6.4-4}$$

6.4.2 REINFORCEMENT

6.4.2.1 EXPECTED YIELD STRENGTH OF REINFORCEMENT

The value of expected yield strength of reinforcement shall be taken as the values in Table 6.4.2 or shall be the average based on mill test data.

Table 6.4.2
Values of Expected Yield Strength of Reinforcement

Grade of Steel	f_{ye}
40	54 ksi
60	66 ksi
75	82 ksi

6.4.2.2 EXPECTED MODULUS OF ELASTICITY OF STEEL REINFORCEMENT

The value of expected modulus of elasticity of steel reinforcement shall be taken as 29,000,000 psi.

CHAPTER 7
DESIGN LOADS

7.0 NOTATIONS

C_d = deflection amplification factor given in Table 7.3-1.
C_s = base shear coefficient as determined by Eq. 7.3-3.
I_e = effective moment of inertia.
I_g = moment of inertia of full cross section.
I_{cr} = moment of inertia of cracked cross section.
M_{cr} = section modulus of full section multiplied by modulus of rupture.
M_a = maximum moment in the length of the member.
n = 1.0 for $T_e \leq 1.0$ sec. and 2/3 for $T_e > 1.0$ sec.
Q_D = effect of dead load.
Q_E = effect of seismic (earthquake) forces.
Q_F = effect of fluid pressure.
Q_H = effect of earth presure.
Q_L = effect of live load.
Q_s = effect of snow load.
Q_T = effect of differential settlement, creep, shrinkage or temperature.
Q_W = effect of wind load.
R = response modification coefficient given in Table 7.3-1.
S = soil profile coefficient given in Table 7.3-2, S_1 to S_4 are coefficients for Soil Types 1 - 4.
$S_{a(0.3)}$ = coefficient representing the spectral acceleration at 0.3 sec. The value of the coefficient shall not exceed 1.0.
$S_{a(1.0)}$ = coefficient representing the spectral acceleration at 1.0 sec. The value of the coefficient shall not exceed 0.6.
T_e = expected period using the expected stiffness of the reinforced masonry shear wall as determined in Sec. 7.3-4.
U = required strength to resist factored loads, or related internal moments and forces.
V = seismic base shear.
V_x = base shear at level x
W = total gravity load of the building.

7.1 GENERAL

This Chapter sets forth design loads and combination of load effects for masonry structures which shall be considered in the design. Loads required to be considered in Section 7.2 shall be determined in accordance with the General Building Code except that seismic loads shall be determined in accordance with Sec. 7.3.

7.2 COMBINATIONS OF LOAD EFFECTS

7.2.1 STRUCTURAL COMPONENT LOAD EFFECTS

All masonry building components shall have strengths sufficient to resist the effects of gravity loading from dead, live and snow loads in combination with the effects of wind and seismic loading specified herein.

The direction of application of wind and seismic forces used in design shall be that which will produce the most critical load effect in each component. The second-order effects shall be included where applicable.

7.2.2 COMBINATION OF LOAD EFFECTS FOR STRENGTH AND DISPLACEMENT LIMIT STATES

The effects on the building and its components shall include load combinations for strength limit states in accordance with the following:

$U = 1.4\ Q_D$	Eq. 7.2-1
$U = 1.2\ Q_D + 1.6\ Q_L$	Eq. 7.2-2
$U = 1.2\ Q_D + 0.5\ Q_L + 1.3\ Q_w + 1.3\ Q_F$	Eq. 7.2-3
$U = 1.2\ Q_D + 0.5\ Q_L + 1.3\ Q_w$	Eq. 7.2-4
$U = 0.9\ Q_D - 1.3\ Q_w$	Eq. 7.2-5
$U = 1.2\ Q_D + 1.6\ Q_L + 1.6\ Q_H + 1.3\ Q_F$	Eq. 7.2-6
$U = 0.9\ Q_D + 1.6\ Q_H + 1.3\ Q_F$	Eq. 7.2-7
$U = 1.3\ Q_F + 1.2\ Q_D + 1.2\ Q_I + 0.5\ Q_L$	Eq. 7.2-8
$U = 1.2\ Q_D + 1.2\ Q_T$	Eq. 7.2-9
$U = (1.1 + 0.3\ S_{a(1.0)})\ Q_D + 1.0\ Q_L + 1.0\ Q_S + 1.0\ Q_E$	Eq. 7.2-10
$U = (0.9 - 0.3\ S_{a(1.0)})\ Q_D \pm 1.0\ Q_E$	Eq. 7.2-11

For plain masonry and brittle connections.

$U = (0.7 - 0.3\ S_{a(1.0)})\ Q_D \pm 1.0\ Q_E$	Eq. 7.2-12

7.2.2.1 Each load 1.3 Q_w, 1.0 Q_S, 1.0 Q_E, may be replaced with a corresponding load from a site - specific study where the corresponding load has a probability of 10 percent or less of being exceeded in a 50 year exposure time.

7.3 SEISMIC LOADING

7.3.1 GENERAL

The 1991 Edition of the NEHRP Recommended Provisions for the Development of Seismic Regulations for New Buildings shall be applicable except as modified herein.

7.3.1.1 ANCHORAGE OF MASONRY WALLS

Masonry walls shall be anchored to the roof and all floors that provide lateral support for the wall. The anchorage shall provide a direct connection between the walls and the roof or floor construction. The connections shall be capable of resisting the greater of a lateral force (F_p) induced by the wall or 700 times the spectral acceleration (1.0), $S_{a(1.0)}$, (pound) per lineal foot of wall. Walls shall be designed to resist bending between anchors where the anchor spacing exceeds 4 feet.

7.3.1.2 DIAPHRAGMS

Floor and roof diaphragms shall be designed to resist the following seismic forces: A minimum force equal to 35 percent of the spectral acceleration (1.0), $S_{a(1.0)}$, times the weight of the diaphragm and other elements of the building attached thereto plus the portion of the seismic shear force at that level (V_x) required to be transferred to the components of the vertical seismic force resisting system because of offsets or changes in stiffness of the vertical components above and below the diaphragm.

7.3.1.3 BEARING WALLS

Exterior and interior bearing walls and their anchorage shall be designed for a force equal to 70 percent of the spectral acceleration (1.0), $S_{a(1.0)}$, times the weight of the wall (W_c), normal to the wall surface, with a minimum force of 10 percent of the wall weight. Interconnection of wall elements and connections to support framing systems shall have sufficient ductility, rotational capacity, or sufficient strength to resist shrinkage, thermal changes, and differential foundation settlement when combined with seismic forces.

7.3.2 SEISMIC DESIGN COEFFICIENT

The seismic design coefficient shall be determined from the following:

$$C_s = \frac{S_{a(1.0)}\ S}{R\ T_c^{n}} \qquad \text{Eq. 7.3-1}$$

but the value of C_s need not exceed

$$C_s \leq \frac{S_{a(0.3)}}{R} \qquad \text{Eq. 7.3-2}$$

The value of $S_{a(0.3)}$ and $S_{a(1.0)}$ shall be determined from Maps 5, 6, 7, or 8 of the NEHRP Recommended Provisions.

7.3.3 SEISMIC BASE SHEAR

The seismic base shear, V, in a given direction shall be determined from the following:

$$V = C_s\ W \qquad \text{Eq. 7.3-3}$$

TABLE 7.3-1
Deflection Amplification Factors and
Response Modification Coefficients
BEARING WALL AND BUILDING FRAME SYSTEM

	C_d	R
Reinforced Masonry Shear Walls	3½	4½
Plain Masonry Shear Walls	1½	2
FRAME WALL SYSTEMS		
Reinforced Masonry Frame Wall	4	5½

TABLE 7.3-2

Soil Profile Type	Coefficient S
S_1	0.7
S_2	1.0
S_3	1.6
S_4	1.9

7.3.4 PERIOD DETERMINATION

The expected fundamental period of the seismic load resisting system may be established using the structural properties and deformational characteristics of the resisting elements in a properly substantiated analysis. The uncracked elastic stiffness of the load resisting system may be decreased by the effects of the foundation flexibility.

The expected stiffness of reinforced masonry cantilever shear walls may be estimated as 10 percent of the uncracked stiffness. This value of expected stiffness assumes the masonry shear wall has an expected flexural strength nearly equal to the required strength. The ratio of the expected stiffness to the uncracked stiffness for flexural strengths that exceed the required design strength shall be calculated by Equation 7.3-4. In any case, the value of the expected stiffness need not be greater than that value determined for an uncracked shear wall with foundation flexibility.

The determination of the expected stiffness of coupled reinforced masonry shear walls shall consider the influence of the coupling beams in addition to the expected stiffness of the individual shear walls. Foundation flexibility may be included in the determination of expected stiffness.

Unless stiffness values are obtained from a more comprehensive analysis, the expected stiffness of reinforced masonry elements may be computed from the modulus of elasticity E_{mc} and the effective moment of inertia, I_e, by Equation 7.3-4.

$$I_e = \left(\frac{M_{cr}}{M_a}\right)^3 I_g + \left[1 - \left(\frac{M_{cr}}{M_a}\right)^3\right] I_{cr} \qquad \text{Eq. 7.3-4}$$

CHAPTER 8
DESIGN FOR STRENGTH AND DISPLACEMENT LIMIT STATES OF REINFORCED MASONRY

8.0 NOTATION

A_n	=	net cross-sectional area perpendicular to axial load, sq. in.
A_s	=	area of tension reinforcement, sq.in.
A_v	=	area of vertical reinforcement at end of wall (mm^2)
a_b	=	length of compressive stress block, in.
b	=	effective width of a member, in.
C_d	=	deflection amplification factor.
c	=	distance from neutral axis to extreme compressive fiber, in.
d	=	effective depth of a member, in
E_{mc}	=	expected chord modulus of elasticity of masonry, psi.
E_{sc}	=	expected modulus of elasticity of reinforcement, psi.
e	=	eccentricity of P_{ou}, in.
f_{mc}	=	expected compressive strength of masonry at the age of 28 days, psi.
f_{rc}	=	expected modulus of rupture, psi
f_{yc}	=	expected yield strength of reinforcement, psi.
h	=	height of the wall between points of support, in.
h_p	=	height of the pier measured at adjacent opening.
I_c	=	effective moment of inertia of cracked wall section, $in.^4$
I_{cr}	=	cracked section moment of inertia of wall section, $in.^4$
I_g	=	gross moment of inertia of the wall cross section, $in.^4$
k	=	effective length factor for compression members.
L	=	length between coupled shear walls.
L_w	=	length of shear wall, in.
ℓ_u	=	unsupported length of compression member, in.
M_a, M_b	=	flexural moment capacity at ends of coupling beams, piers or columns, in.-lbs.
M_{crc}	=	expected cracking moment strength of the masonry, in.-lbs.
M_c	=	expected moment strength, in.-lb.
P_c	=	expected axial load strength, lb.
P_{ou}	=	factored axial load on member, lbs.
P_u	=	required axial load strength of member, lbs.
P_w	=	weight of the wall tributary to section under consideration, lb.
P_{wu}	=	factored weight of the wall tributary to the section under consideration, lb.
Q_E	=	effect of seismic (earthquake) forces
r	=	radius of gyration, in.
r_d	=	h/d
S	=	section modulus, in^3.
V_u	=	required shear strength.
v_u	=	expected shear strength, MPa
ρ	=	ratio of the area of reinforcement to the cross sectional area of masonry on a plane perpendicular to the reinforcement.
ρ_b	=	reinforcement ratio producing balanced strain conditions.

ρ_h = horizontal reinforcement ratio.
ρ_{min} = minimum reinforcement ratio
Δ_u = horizontal deflection at mid-height under factored load, in.
ϕ = capacity reduction factor.
ϵ_{muc} = maximum usable compressive strain of masonry, in/in.
ρ_v = A_v/tL
δ = 1.0 for pier that is restrained at each end, 0.6 for cantilever walls
σ_o = nominal axial stress on wall, MPa
t = wall thickness

8.1 GENERAL

This chapter establishes requirements for the specification of strength and displacement limit states for reinforced masonry. Design assumptions common to design for all combinations of load effects are given in Section 8.2. General requirements for reinforced masonry elements and systems are given in Section 8.3. Section 8.4 describes the required strength for limit states of loading that do not include earthquake loading or when the strength required by combinations of load effects other than earthquake loading exceeds the strength required by combinations of load effects including earthquake loading by 100 percent. Section 8.5 describes the limit state of required strength and displacement that is applicable to load combinations that include earthquake loading.

8.2 DESIGN ASSUMPTIONS

Limit state design of members for flexure and axial loads shall be in accordance with principles of engineering mechanics, satisfaction of applicable conditions of equilibrium, compatibility of strains and in accordance with the provisions of 8.2.1 through 8.2.6

8.2.1 Maximum usable strain, ϵ_{mu}, at the extreme masonry compression fiber shall be assumed equal to 0.0025 for calculation of balanced reinforcement ratio.

8.2.2 Stress in reinforcement below the expected yield strength, f_{ye}, shall be taken as E_{se} times the steel strain.

8.2.3 Tensile strength of masonry shall be neglected in calculating the flexural strength of a reinforced masonry cross section.

8.2.4 The maximum ratio of flexural reinforcement to the masonry cross section shall be limited as specified in Section 8.4.2 and Section 8.5.2. Balanced strain conditions exist at a cross section when tension reinforcement reaches the strain corresponding to its expected yield strength f_{ye} just as masonry in compression reaches its maximum usable strain, ϵ_{mu}. The value of the maximum usable strain shall not exceed 0.0025 unless substantiated by testing. The calculation of balanced strain condition shall include all unfactored axial loads on the section under consideration. The contribution of compression reinforcement to resistance of compressive loads shall not be considered.

8.2.5 The flexural reinforcement shall be uniformly distributed throughout the elements depth and shall meet the requirements for reinforcement required for loading effects normal to the wall plane. All reinforcement parallel to the length of the element shall be used to calculate the expected flexural strength and the maximum ratio of flexural reinforcement.

8.2.6 The expected shear strength for fully grouted masonry shall be determined by Equation 8.2-1.

$$v_u = \left[\left(\frac{0.76}{r_d + 0.7} + 0.012\right)\left(4.04\rho_v^{0.3}\sqrt{f_{me}}\right)\right]\frac{d}{L} + \left[0.1575(\rho_h f_{ye})^{1/2}\sqrt{f_{me}}\right]\frac{\delta d}{L} + (0.175\sigma_o)\frac{d}{L} \qquad \text{Eq. 8.2-1}$$

8.3 GENERAL REQUIREMENTS

8.3.1 DISTANCE BETWEEN LATERAL SUPPORTS OF FLEXURAL MEMBERS

8.3.1.1 The spacing of lateral supports shall be determined by the required strength for out-of-plane loading but it shall not be less than the following:
(a) The least width of beams with horizontal reinforcement of a minimum of two bars placed in a horizontal plane and enclosed by confinement reinforcement shall not exceed 50 times the spacing of lateral supports.
(b) The spacing of lateral supports of beams not complying with Section 8.1.3.1(a) shall not exceed 32 times the least width of the beam.

8.3.1.2 Effects of eccentricity of axial loads shall be taken into account in determining spacing of lateral supports.

8.3.2 SLENDERNESS EFFECTS IN COMPRESSION MEMBERS

8.3.2.1 Design of compression members shall be based on forces and moments determined from analysis of the structure. Such analysis shall take into account influence of axial loads and variable moment of inertia on member stiffness and fixed-end moments, effect of deflections on moments and forces, and the effects of duration of loads.

8.3.2.2 In lieu of the procedure prescribed in Section 8.3.2.1, slenderness effects in compression members are evaluated in accordance with the approximate procedure presented in Section 8.3.3. The detailed requirements of Section 8.3.3, need not be applied if slenderness effects in compression members are evaluated in accordance with Section 8.3.2.1.

8.3.3 APPROXIMATE EVALUATION OF SLENDERNESS EFFECTS

8.3.3.1 Unsupported length ℓ_u of a compression member shall be taken as the clear distance between floor slabs, beams, or other members capable of providing lateral support for that compression member.

8.3.3.2 For compression member braced against sideway, effective length factor k shall be taken as 1.0, unless analysis shows that a lower value may be used.

8.3.3.3 For compression members not braced against sideway, effective length factor k shall be determined with due consideration of effects of cracking and reinforcement on effective stiffness, and shall be greater than 1.0.

8.3.3.4 Radius of gyration, r may be taken equal to 0.30 times the overall dimension in the direction stability is being considered for rectangular compression members, and 0.25 times the diameter for circular compression members. For other shapes, r may be computed for the gross concrete section.

8.3.4 CONSIDERATION OF SLENDERNESS EFFECTS

8.3.4.1 For compression members braced against sideways, effects of slenderness may be neglected when $k\ell_u/r$ is less than 30.

8.3.4.2 For compression members not braced against sideways, effects of slenderness may be neglected when $k\ell_u/r$ is less than 22.

8.3.4.3 For all compression members with $k\ell_u/r$ greater than 100, an analysis as defined in Section 8.3.2.1 shall be made.

8.3.5 LIMITS OF REINFORCEMENT OF COMPRESSION MEMBERS

The reinforcement corresponding to balanced strain conditions shall be calculated. The moments in the member due to rotational restraint and interstory drift shall be calculated. The maximum reinforcement provided in the member for combined axial and flexure loads shall be limited as specified in Section 8.4.2 and Section 8.5.2.

8.3.6 MINIMUM AND MAXIMUM REINFORCEMENT OF MEMBERS SUBJECTED TO AXIAL AND/OR FLEXURAL LOADS

The minimum reinforcement ratio shall be 0.0005. The maximum reinforcement ratio shall be limited as specified in Section 8.4.2 and Section 8.5.2.

8.3.7 CONFINEMENT OF COMPRESSIVE STRESS ZONE

Confined compressive stress zones shall be as follows:

8.3.7.1 Reinforced masonry elements where the modifications of limit states allowed for confinement are utilized in the design shall have confinement ties conforming to Section 4.6.

8.3.7.2 The minimum horizontal length of the confinement region shall be three times the thickness of the wall.

8.3.7.3 Confinement ties shall consist of a minimum of No. 3 bars at a maximum of 8-inch spacing or equivalent within the region defined by the length of the compression zone and one-quarter of the length to the point of inflection or one-sixth of the length of a cantilever member.

8.3.7.4 Confinement of the compressive stress zone does not require vertical reinforcement at the corners and bends of the confinement reinforcement.

8.3.8 REINFORCEMENT

Reinforcement shall be in accordance with the following:

8.3.8.1 All continuous reinforcement shall be anchored or spliced in accordance with Section 4.5.? (with $f_s < 0.5f_{ye}$);Section 4.5.? (with $f_{se} = f_{ye}$).

8.3.8.2 The minimum amount of vertical reinforcement shall not be less than required by the design loading normal to the wall plane.

8.3.8.4 Maximum spacing of horizontal reinforcement shall not exceed six times nominal wall thickness or 48 inches, whichever is less.

8.3.9 FLANGED WALLS

Wall intersections shall meet one of the following requirements:
Design of flanged walls shall conform to the provisions of Section 8.4.2 or 8.5.2 as applicable or the transfer of shear between walls shall be prevented. If the flange is considered effective in resisting applied loads, the width of flange considered effective in compression on each side of the web shall be taken equal to 1/3 of the wall height or shall be

equal to the actual flange on either side of the web wall, whichever is less. The width of flange considered effective in tension on each side of the web shall be taken equal to 1/3 of the wall height or shall be equal to the actual flange on either side of the web wall, whichever is less. The face shells of the masonry units shall be removed and the intersection shall be fully grouted, and all horizontal reinforcement shall be continuous through the intersection.

8.3.10 WALL FRAMES

8.3.10.1 REQUIRED STRENGTH

The required strength of members of wall frames shall be determined by the combined loadings given in Chapter 7. The calculation of required strength of the members shall be in accordance with engineering mechanics and shall consider the effects of the stiffness degradation of the beams and columns. The frame analysis may be an iterative process in which the stiffness of the members are reduced to their effective stiffness by use of Equation 7.3-4 and the ratio of calculated moment to cracking moment.

8.3.10.2 DESIGN STRENGTH

Design strength provided by frame member cross sections in terms of axial force, shear and moments shall be computed as the expected strength multiplied by the capacity reduction factor ϕ.

All frame members shall be proportioned such that the design strength exceeds the required strength.

Expected strength of all frame members shall be based on assumptions prescribed in Section 8.2. The maximum useable strain ϵ_{mu} at the extreme masonry compression fiber shall not exceed 0.0025 for calculation of the balanced reinforcement ratio. If confinement ties as defined in Section 8.3.7 are utilized, increases in the maximum reinforcement ratio and drift limit states may be used.

8.3.10.3 REINFORCEMENT

The expected moment strength at any section along a member shall not be less than 1/2 of the higher moment strength provided at the two ends of the member. Lap splices are permitted only within the center half of the member length. Welded splices and mechanical connections may be used for splicing the reinforcement at any section, provided not more than alternate longitudinal bars are spliced at a section, and the distance between splices on alternate bars is at least 24 inches along the longitudinal axis.

8.3.10.4 FLEXURAL BEAMS OF WALL FRAMES

(a) Factored axial compression force on the beam shall not exceed 0.10 A_n f_{mc}.
(b) Clear span for the beam shall not be less than 4 times its depth.
(c) Nominal depth of the beam shall not be less than 4 units or 32 inches, whichever is greater. The nominal depth to nominal width ratio shall not exceed 4.
(d) Nominal width of the beam shall not be less than 8 inches or that required by Section 8.3.1.1 (b) or 1/26 of the clear span between column faces whichever is less.
(e) At any section of a beam, each masonry unit throughout the beam depth shall contain longitudinal reinforcement.
(f) The variation in the longitudinal reinforcement area between units at any section shall not be greater than 50 percent of the minimum area of longitudinal reinforcement contained by any one unit.
(g) Minimum reinforcement ratio shall be $130/f_{ye}$.
(h) Maximum reinforcement ratio shall be specified by Section 8.4.2 and Section 8.5.2.

8.3.10.5 TRANSVERSE REINFORCEMENT IN FLEXURAL BEAMS OF WALL FRAMES

(a) Transverse reinforcement shall be hooked around top and bottom longitudinal bars with a standard 180 degree hook.
(b) Within a region extending one beam depth from wall frame column faces and at any region at which plastic hinges may form, maximum spacing of transverse reinforcement shall not exceed one-fourth the nominal depth of the beam.
(c) The maximum spacing of transverse reinforcement shall not exceed 1/4 the nominal depth of the beam or that required for shear strength.
(d) Minimum transverse reinforcement ratio shall be 0.0015.

8.3.10.6 MEMBERS SUBJECTED TO AXIAL FORCE AND FLEXURE (WALL FRAME COLUMNS)

(a) Requirements of Section 8.2 apply to wall frame columns of masonry frames that are proportioned to resist flexure in conjunction with axial load.
(b) Factored axial compression force on the wall frame column shall not exceed 0.30 $A_n f_{mc}$. The allowable compression force shall also be limited by the maximum reinforcement ratio.
(c) Nominal depth of the wall frame column shall not be less than two full units or 24 inches, whichever is greater.
(d) Nominal width of the wall frame column shall not be less than 8 inches, or 1/14 of the clear height between beam faces, whichever is greater.
(e) A minimum of 4 longitudinal bars shall be provided at all sections of every wall frame column member.
(f) The flexural reinforcement shall be essentially uniformly distributed across the member depth.

(g) The expected member strength at any section along a member shall be less than 1.3 times the cracking moment strength and the minimum reinforcement ratio shall be $130/f_{ye}$.
(h) Maximum reinforcement ratio shall conform with Section 8.4.2 or Section 8.5.2.

8.3.10.6 TRANSVERSE REINFORCEMENT OF WALL FRAME COLUMNS

(a) Transverse reinforcement shall be hooked around the extreme longitudinal bars with standard 180 degree hook.
(b) The maximum spacing of transverse reinforcement shall not exceed 1/4 the nominal depth of the column.
(c) Minimum transverse reinforcement ratio shall be 0.0015.

8.3.10.7 WALL FRAME BEAM-COLUMN INTERSECTION

(a) Beam-column intersections in masonry wall frame shall be proportioned such that the wall frame column depth in the plane of the frame exceeds 60 times the diameter of longitudinal reinforcement passing through the beam-column intersection.
Beam depth shall exceed 40 times the diameter of frame column longitudinal reinforcement passing through the beam-column intersection.
(b) Beam-column intersection shear forces shall be calculated on the assumption that all flexural beams develop the expected flexural moments.
(c) Shear strength of beam column intersections shall be governed by the appropriate strength reduction factor specified in Section 8.4.2.1 or Section 8.5.2.1.
(d) Longitudinal beam reinforcement terminating in a wall frame column shall be extended to the far face of the column and anchored by a standard 90 degree or 180 degree hook.

8.3.10.8 TRANSVERSE REINFORCEMENT IN WALL FRAME BEAM-COLUMN INTERSECTION

Special horizontal shear reinforcement crossing a potential diagonal beam column shear crack shall be provided such that:

$$A_v = \frac{0.5V}{f_{ye}} \qquad \text{Eq. 8.3-1}$$

Special horizontal shear reinforcement shall be anchored by a standard hook around the wall frame column reinforcing bars.

8.3.10.9 SHEAR STRENGTH OF WALL FRAME BEAM-COLUMN INTERSECTION

The nominal horizontal shear stress at the beam-column intersection shall not exceed 350 psi.

8.4 DESIGN FOR LOADING NOT INCLUDING EARTHQUAKE

8.4.1 DESIGN ASSUMPTIONS

8.4.1.1 GENERAL

The limit state of reinforced masonry elements and systems designed in accordance with this Section is the yield limit state. The design strength shall be equal to or greater than the strength required by combinations of load effects specified in Chapter 7. The expected yield strength is reduced by a capacity reduction factor ϕ to a design strength.

8.4.1.2 BALANCED REINFORCEMENT RATIO

The balanced reinforcement ratio shall be calculated by assuming a maximum usable strain, ϵ_{mue}, at the extreme fiber of the element and a yield strain at the location of the edge tension reinforcement of f_{ye}/E_{se}. The effects of reinforcement in the compression zone shall be disregarded. The distribution of strain across the section shall be assumed linear and the compressive forces shall be in equilibrium with the tension forces in the reinforcement and the unfactored axial loads on the section. The reinforcement shall be assumed to be uniformly distributed across the section and the balanced reinforcement ratio shall be calculated as the area of reinforcement distributed uniformly in the depth of the element divided by the area of the element.

8.4.1.3 EXPECTED YIELD STRENGTH

The expected yield strength of the element shall be calculated by the following assumptions:
A masonry stress of 0.85 f_{mc} shall be assumed uniformly distributed over a compression zone bounded by the edges of the cross section and a straight line parallel to the neutral axis a distance of a_b from the fiber of maximum strain.

The force in the compression zone shall be equated to the total tensile forces in all of the reinforcement outside of the compression zone and the factored axial loads on the section.

The tensile forces in the reinforcement shall be $A_s f_{ye}$ in the edge reinforcement and $A_s E_{se}$ times the strain in the reinforcing that is distributed over the depth of the section. The strain shall be assumed to be zero at the distance a_b from the edge of the element compressed by the axial loads and moment.

8.4.2 GENERAL REQUIREMENTS

8.4.2.1 MAXIMUM REINFORCEMENT RATIO

The maximum reinforcement ratio provided in a member for axial or flexural loads or combined axial and flexural loads shall be limited to 50 percent of the balanced reinforcement ratio. The maximum reinforcement ratio may be 65 percent of the balanced reinforcement ratio if confinement ties are used in the compressive stress zones.

8.4.2.2 DESIGN STRENGTH

Design strength provided by the reinforced masonry cross section shall be calculated as the expected strength multiplied by a strength reduction factor ϕ. The strength reduction factors shall be as follows:

(a) Flexure and flexure with axial load and with reinforcement placed in a single line parallel to the edge under compression.
$\phi = 0.90$

(b) Flexure and flexure with axial load and with reinforcement distributed along the depth of the member.
$\phi = 1.0$

(c) Inplane shear in any reinforced masonry element. $\phi = 0.85$

(d) When the element limit state is flexure, the expected flexural strength shall be at least 1.3 times the cracking moment strength determined by Equation 8.4.1.

$$M_{cre} = f_{re}\ S \qquad \text{Eq. 8.4-1}$$

8.4.3 DESIGN FOR LOADING NORMAL TO THE WALL PLANE

All moment and deflection calculations are based on simple support conditions top and bottom. For other support and fixity conditions, moments and deflections shall be calculated using established principles of mechanics.

Required moment shall be determined at the mid-height of the wall and shall be used for design. The required moment strength, M_u, shall be at least equal to:

$$M_u = \frac{w_u\ h^2}{8} + P_{ou}\frac{e}{2} + \left(\frac{P_w}{2} + P_{ou}\right)\Delta_e \qquad \text{Eq. 8.4-2}$$

where:

$$\Delta_e = \left[\frac{5M_u h^2}{48\ E_{me}\ I_e}\right] + \left[0.064\frac{P_{ou} e h^2}{E_{me}\ I_e}\right] \qquad \text{Eq. 8.4-3}$$

where: I_e shall be calculated by Equation 7.3.4.

8.4.4 DESIGN FOR LOADING IN THE PLANE OF THE WALL

8.4.4.1 GENERAL

Reinforced masonry walls loaded in their plane are designed assuming that they are fixed against rotation at the base of the building. The design shall include effects of load combinations that cause maximum values of moment and shear. Secondary moments in wall piers and spandrel beams caused by openings in the wall plane shall be additive to primary moments.

Four general conditions for design of walls for loading in their plane are given in Sections 8.4.4.2 through 8.4.4.5. These conditions are:

Section 8.4.4.2	Walls with openings.
Section 8.4.4.3	Cantilever shear walls.
Section 8.4.4.4	Coupled cantilever shear walls.
Section 8.4.4.5	Wall frames.

Established principles of mechanics shall be utilized to design wall systems that may fit two of these general conditions. However, the design of wall frame is restricted to systems that conform with the requirements for wall frames given in Section 8.3.

8.4.4.2 SHEAR WALLS WITH OPENINGS

Shear walls with openings shall be designed with due consideration of restraint of piers by lintel beams and reduction in the effective moment of inertia, I_e, due to cracking of the reinforced masonry. The pier-lintel intersection may be considered as a rigid element for this analysis and all design moments may be reduced to the face of the pier-lintel intersection.

The shear capacity of the piers shall be determined by Equation 8.2.1. The value of δ may be interpolated between 0.6 for cantilever piers and 1.0 for piers with fully restrained ends in accordance with the degree of restraint at the top of the pier.

8.4.4.3 CANTILEVER SHEAR WALLS

Cantilever shear walls shall be unrestrained against rotation at their top and at each floor level. Shear walls restrained against rotation at their top or at any floor level shall be designed as coupled cantilever shear walls.

8.4.4.4 COUPLED CANTILEVER SHEAR WALLS

Established principles of mechanics shall be utilized to design coupled cantilever shear walls. However, when the sum of the uncracked moments of inertia of the coupling beams is less than 1/10 of the uncracked moment of inertia of the cantilever shear wall, the shear wall may be designed as a cantilever wall. The design moment in the coupling beams shall be calculated by consideration of the moment induced by the rotation of the shear at that level. Moments due to vertical displacements at the ends of the coupling beams may be neglected.

8.4.4.5 WALL FRAMES

The combined expected strength of the columns at any beam-column intersection shall equal or exceed 1.1 times the sum of the expected strength of all beams framing into the beam-column intersection.

An additional limit state for wall frames is the interstory drift or the overall drift at the top of the wall frame. The stiffness of the wall frame used for calculation of the required strength shall be used for calculation of the interstory and overall drift. The limit of interstory or overall drift shall be 0.0133 times the story or overall height or 0.0150 if confinement ties are utilized in the compressive stress zones of the columns.

8.5 DESIGN FOR LOADING INCLUDING EARTHQUAKES

8.5.1 DESIGN ASSUMPTIONS

8.5.1.1 GENERAL

The combinations of loadings specified in Chapter 7 that include earthquake loading are intended to provide a required strength and do not provide an elastic strength equal to the probable earthquake loads on an elastic structure in nonlinear behavior of the elements and the lateral load resisting system. Use of Section 8.5 for design is mandatory unless the expected strength calculated as prescribed by Section 8.4 exceeds by two the required strength prescribed by Section 8.5. If the expected strength determined by the loading not including earthquake loading is less than twice the strength required by loading combinations including earthquakes, the flexural strength required by Section 8.4 shall be provided and the inplane shear strength required by Section 8.5, expected shall be provided. The required shear strength shall be based on the expected flexural strength provided.

Design for loadings including earthquake requires that expected flexural strength be the limit state. The expected shear strength of walls with earthquake loading shall exceed by 140 percent the base shear of the wall determined by the limit state of flexural strength. The limit state moment shall be calculated by distributing the base shear in accordance with the NEHRP Recommended Provisions. However, the expected shear strength of the wall need not exceed the base shear determined by the stability moment calculated at the bottom of the foundation by the summation of the moments of the dead and live loads, floor and roof loads, weight of the wall, its foundations, the soil overlying the foundations, and the restraint of foundation beams. The moment shall be taken about the extreme edge of the foundation of the wall.

And, the design shear strength of the wall need not exceed 200 percent of the required strength of the wall in any case.

8.5.1.2 BALANCED REINFORCEMENT RATIO

The balanced reinforcement ratio shall be calculated as prescribed by Section 8.4.1.2.

8.5.1.3 EXPECTED FLEXURAL STRENGTH

The expected flexural strength shall be calculated by the following assumptions:
A masonry stress of 0.85 f_{me} shall be assumed uniformly distributed over a compression zone bounded by the edges of the cross section and a straight line parallel to the neutral axis a distance a_b from the fiber of maximum compressive strain. The force in the compression zone shall be equated to the total of the tensile forces in all of the reinforcement outside of the compression zone and the factored axial loads on the section. The tensile forces in the reinforcement shall be $A_s f_{ye}$.

8.5.2 GENERAL REQUIREMENTS

8.5.2.1 MAXIMUM REINFORCEMENT RATIO

The maximum reinforcement ratio provided in an element for axial or flexural resistance or combined axial and flexural resistance shall be limited to 35 percent of the balanced reinforcement ratio. If confinement ties are used in the compressive stress zones, the maximum reinforcement ratio may be increased to 50 percent of the balanced reinforcement ratio.

8.5.2.2 DESIGN STRENGTH

Design strength provided by the reinforced masonry cross section shall be calculated as the expected strength multiplied by a strength reduction factor, ϕ. The strength reduction factors shall be as follows:

(a) Flexure and flexure with axial load and reinforcement placed in a single line parallel to the compression face, $\phi = 0.90$

(b) Flexure and flexure with axial load and reinforcement distributed along the depth of the member, $\phi = 1.0$

(c) Inplane shear in any reinforced masonry element, $\phi = 0.85$

(d) The required strength for load combinations including earthquake shall be determined for loading normal to the wall plane and for loading in the plane of the wall. The quantity of reinforcement provided in the wall shall be the maximum determined by either loading. If loading normal to the wall plane determines the maximum quantity of reinforcement, that quantity of reinforcement shall be used to determine the expected inplane flexural strength and the required shear strength.

8.5.3 DESIGN FOR LOADING NORMAL TO THE WALL PLANE

All moment and deflection calculations are based on simple support conditions top and bottom. For other support and fixity conditions, moments and deflections shall be calculated using established principles of mechanics.

The moment due to loading shall be determined at the mid-height of the wall and shall be used for design. The required moment strength, M_u, shall be equal to:

$$M_u \frac{w_u h^2}{8} + P_{ou} \frac{e}{2} \qquad \text{Eq. 8.5-1}$$

8.5.4 DESIGN FOR LOADING IN THE PLANE OF THE WALL

8.5.4.1 GENERAL

Design requirements for reinforced masonry walls for earthquake loading in the plane of the wall inherently assumes that the limit states are flexural yielding and displacements caused by the specified loadings. Walls are categorized into four general descriptions as follows:

Walls where the flexural limit state will occur in piers between opening rather than at the base of the wall.

Walls where the flexural limit state will occur at the base of the wall and the rotation of the cross section of the wall above the base is not restrained.

Walls where the flexural limit state will occur at the base of the wall and in beams that couple the walls.

Wall frames where the member sizes are restricted by this standard and the flexural limit state will occur at the beam face of the beam-column intersections and in the columns at the base of the wall frame.

These inplane wall systems range in the order given from those having a limited potential for ductile behavior to that having the highest probability of having adequate energy dissipation, displacement ductility and overstrength. This desirable behavior is attained by strict limitation of member sizes, relationship of member strength and rigorous design requirements. The system with the least probability of having adequate displacement ductility, the wall with randomly placed openings, is burdened with increased strength requirements, especially shear strength.

These limit state design standards are intended to provide flexural yielding in preference to shear yielding. The standards are intended to provide reinforced masonry walls that can have adequate displacement ductility to cope with the displacements caused by earthquake motions. Shear yielding has a very limited ductility and is prevented to the maximum extent possible. Cantilever shear walls form a single yield zone at their base and the drift of the shear wall must be limited to minimize damage to this yield zone. Coupled shear walls cause yield hinges to form in coupling beams and at the wall base. The quantity of energy dissipated is greater than that dissipated by cantilever shear walls but the design requirements are more rigorous.

8.5.4.2 SHEAR WALLS WITH OPENINGS

Shear walls with openings shall be designed with due consideration of restraint of piers by lintel beams and of reduction in the effective moment of inertia, I_e, due to cracking of the reinforced masonry. The pier-lintel intersection may be considered as a rigid element for this analysis and all design moments may be reduced to the face of the pier-lintel intersection.

The sum of the expected flexural strengths of the piers in all story levels shall be compared to the flexural strength of the wall at its base. If the expected flexural strength of all of the piers is less than the expected flexural strength of the wall at its base, the capacity reduction factor, ϕ, used for the determination of the shear reinforcement of the piers shall be 0.65. The design shear for each pier shall be determined by:

$$V = \frac{M_a + M_b}{h_p} \qquad \text{Eq. 8.5-2}$$

The shear capacity of the piers shall be determined by Equation 8.2.1. The value of δ may be interpolated between 0.6 for cantilever piers and 1.0 for piers with fully restrained ends in accordance with the degree of restraint at the top of the pier.

If the expected flexural strength of the wall at its base is less than the sum of the expected flexural strengths of all of the piers at any level of openings, the wall may be designed as a cantilever shear wall.

The design moments for the piers shall include the effects of the cantilever moment and the secondary moments of the shear in the piers. The effects of the cantilever moment may be calculated assuming plane sections remain plane.

8.5.4.3 CANTILEVER SHEAR WALLS

Cantilever shear walls shall be unrestrained against rotation at their top and at each floor level. Shear walls restrained against rotation at their top or at any floor level shall be designed as coupled cantilever shear walls.

The required shear strength of the wall shall be determined in conformance with Section 8.5.1.1. The shear capacity of the wall shall be determined by Equation 8.2.1. If the shear wall has intersecting reinforced concrete floors at each floor level, the height h used for calculation of r_d shall be the story height.

8.5.4.4 COUPLED CANTILEVER SHEAR WALLS

Established principles of mechanics shall be used to design coupled shear walls. Moments induced in the coupling beams shall be determined by calculating the rotation of the shear wall at each story level. The effective stiffness of the shear wall, I_e, shall be calculated by use of Equation 7.3.1, for each story level and used for determination of the rotation at each story level. Moments due to vertical displacements at the ends of the coupling beams may be neglected.

The expected flexural strength of the coupled shear wall shall be taken as the sum of the expected strength of all of the coupling beams and the shear wall. The required shear strength shall be calculated as prescribed in Section 8.5.1.1 using this expected strength. If the coupling beams are part of a reinforced concrete floor or roof system, the height h used for calculation of r_d shall be the story height.

The dimensions of the coupling beams may be decreased by isolation joints at their ends to meet the requirements of Chapter 8. The expected moment and the design shear may be calculated using this reduced depth. The design shear in the coupling beams shall be calculated using Equation 8.5.2 by substituting the coupling beam length between the edges of the shear walls, L, for h_p.

8.5.4.5 WALL FRAMES

The required strength of members of wall frames shall be determined by the combined loadings given in Chapter 7. The calculations of required strength of the members shall be in accordance with engineering mechanics and shall consider the effects of the relative stiffness degradation of the beams and columns. The frame analysis shall be an iterative process in which the stiffness of the members are reduced to their effective stiffness by use of Equation 7.3.1.

The limit state of flexural yielding of members is limited to the flexural beams at the face of the columns and to the bottom of the columns at the base of the building.

An additional limit state is the interstory drift or the overall drift limit of the top of the wall frame. The stiffness used for the wall frame in the drift calculations shall be that used for calculation of the effective period and for calculation of required member strength.

Expected strength of all frame member cross sections shall be based on assumptions prescribed in Section 8.2.

The shear strength of all members of the wall frame shall exceed the shear value calculated by Equation 8.5.2 where h_p for columns shall be taken as the clear height between spandrel beams.

The wall frame shall be investigated for discontinuities in strength at story levels in accordance with Section 3.4.2 of the NEHRP Recommended Provisions.

8.5.4.6 DRIFT LIMIT STATES

Cantilever shear walls and coupled shear walls shall be proportioned such that the expected top displacement of the shear wall is less than 0.01 h. The displacement shall be calculated using the effective moment of inertia, I_e, and shall be multiplied by C_d for comparison with the drift limit state.

Wall frames shall be proportioned such that the interstory drift and overall drift does not exceed 0.013 h. The procedure for determining the drift shall be that specified for cantilever shear walls. The interstory and overall drift may be 0.0150 h if confinement ties are used in the compressive stress zones of the columns.

8.6 ELEMENTS NOT PART OF THE LATERAL LOAD RESISTING SYSTEM

Reinforced masonry elements that are isolated from the lateral load resisting system such as columns supporting floors and roofs, shall be designed for effects of load combinations and for displacements caused by story drift of the lateral load resisting system.

APPENDIX A-2

GROUND MOTIONS USED FOR ANALYSIS

The objective of this study is to choose several ground motions for scaling to the S_2 soil profile spectrum in the period range between 0.5 - 1.5 seconds. The following steps were taken:

1. 18 ground motions were selected that have reasonable match with the UBC spectrum in the desired range of period. The scaling factor was calculated such that the area under both the UBC and the selected spectra plot of spectral acceleration was equal for the desired range of period. The scaled spectrum for each of these ground motions is shown in Figure Group 1.

2. Further study of plots in Figure Group 1 suggested a reduction of these ground motions to 12. The reasons for abandoning some of these ground motions were either a high scaling factor or high degree of variation from the UBC spectrum. The spectrum for each of these 12 scaled ground motions and their comparison with the UBC spectrum is shown in Figure Group 2.

The names of these ground motions, their duration, and the scaling factors for S_2 soil profile spectrum 0.4g and 0.2g are listed in Table A-1. Scaling factors for ZPA of 0.4 and 0.2g are given in Table A-1.

3. A SDOF model representing a MDOF system was used to determine the dynamic response of this system to these ground motions. The SDOF system represents a wall 60 feet high and 20 feet wide with an effective mass equivalent to 108 kips located at

a height of 47 feet. The stiffness of the wall is defined as the equivalent force at the mass center needed to deflect the mass one unit. The equivalent force is the moment at the base divided by the mass height where the moment is determined by assuming a triangular loading. The stiffness is represented by a modified Spring Type 11 that is described in Figure No. 3a, 3b and 3c. Figure No. 3a shows the cycle behavior. Figure No. 3b shows the force displacement envelope of Spring Type 11. A plot of the variation of the period in terms of the internal deformation of the spring is shown by Figure 3c. The period is defined in terms of the secant stiffness of the system and the equivalent mass of the SDOF system.

The responses of this SDOF system to the scaled ground motions are shown in Figure Group 4. The maximum displacements of the mass for these ground motions are listed in Table A-2 with the mean and standard deviations of the maximum relative displacement for the ground motions.

4. Further study of the dynamic response to the ground motions suggested elimination of three ground motions, Olympia 86, Imperial Valley College 140 and Coalinga, Pleasant Valley 45. The reason was the large value of scaling factor needed or in the case of Imperial Valley College, the small value of the response. The nine ground motions selected as candidates are tabulated in Table A-3 with the recommended scaling factors. An average spectrum of these final 9 ground motions spectra was determined and is compared to the S_2 soil profile UBC spectrum in Figure 5. The mean and

standard deviation of these selected ground motions is presented in Table A-4.

TABLE A-1

GROUND MOTIONS FOR LPM RUNS

No.	Name	Duration Seconds	C1	C2
1	EL CENTRO E-W	53.0	0.9255	1.7875
2	EL CENTRO N-S	53.0	0.6777	1.3145
3	PINE UNION 140	29.0	0.8622	1.7067
4	CRUICKSHANK RD. 230	34.0	0.7632	1.4951
5	JAMES ROAD 140	29.0	0.7126	1.3893
6	KERN CO. 69	54.0	1.4080	2.8648
7	OLYMPIA 86	88.0	1.3553	2.6210
8	IMP. VALLEY COLLEGE 140	34.0	0.5290	1.0769
9	CRUICKSHANK RD. 140	34.0	0.6157	1.2024
10	BRAWLEY AIRPORT 315	37.0	1.0644	2.0738
11	KEYSTONE ROAD 140	39.0	0.9485	1.8501
12	COALINGA PLES.V.P. 45	19.0	0.3743	0.7088

C1: SCALING FACTOR FOR ZPA 0.2g

C2: SCALING FACTOR FOR ZPA 0.4g

TABLE A-2

MAXIMUM DISPLACEMENT OF A SDOF MODEL FOR 11

GROUND MOTIONS SCALED TO A UBC S_2 SPECTRUM

BETWEEN 0.5 TO 1.5 SECONDS

GROUND MOTION	ZPA = 0.4 MAXIMUM DISPLACEMENT INCHES	ZPA = 0.2 MAXIMUM DISPLACEMENT INCHES
EL CENTRO E-W	5.123	0.5898
EL CENTRO N-S	3.862	0.7526
PINE UNION 140	4.749	0.7986
CRUICKSHANK RD. 230	4.449	0.4857
JAMES RD. 140	4.278	1.4871
KERN CO. 69	3.693	0.9448
OLYMPIA 86	4.920	1.7371
CRUICKSHANK RD. 140	4.065	0.4814
BRAWLEY 315	3.156	0.6804
KEYSTONE 140	6.000	1.1931
COALINGA 45	3.592	0.4604
MEAN	4.353	0.874
STAND. DEV.	0.773	0.410

TABLE A-3

SELECTED GROUND MOTIONS FOR LPM RUNS

No.	Name	Duration	C1	C2
1	EL CENTRO E-W	53.0	0.9255	1.7875
2	EL CENTRO N-S	53.0	0.6777	1.3145
3	PINE UNION 140	29.0	0.8622	1.7067
4	CRUICKSHANK RD. 230	34.0	0.7632	1.4951
5	JAMES ROAD 140	29.0	0.7126	1.3893
6	KERN CO. 69	54.0	1.4080	2.8648
7	CRUICKSHANK RD. 140	34.0	0.6157	1.2024
8	BRAWLEY AIRPORT 315	37.0	1.0644	2.0738
9	KEYSTONE ROAD 140	39.0	0.9485	1.8501

C1: SCALING FACTOR FOR ZPA 0.2g

C2: SCALING FACTOR FOR ZPA 0.4g

TABLE A-4

MAXIMUM DISPLACEMENT OF A SDOF MODEL FOR THE

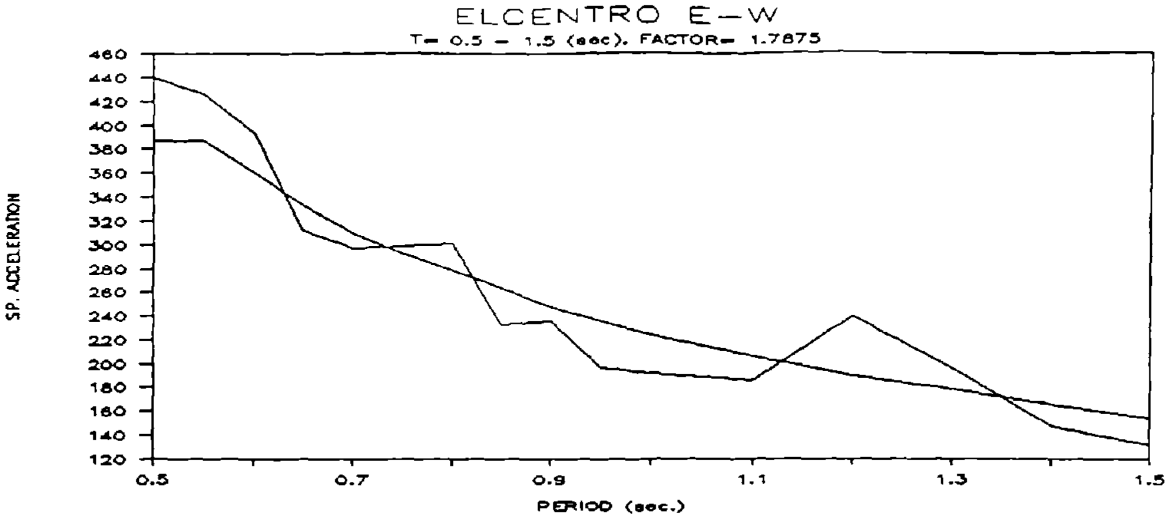
9 GROUND MOTIONS SCALED TO A UBC S_2 SPECTRUM

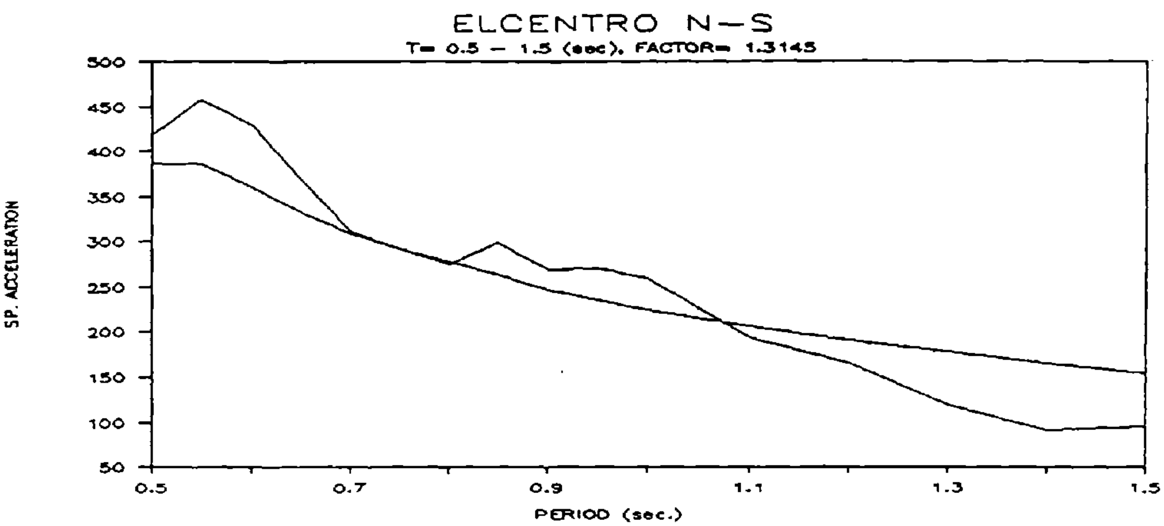
BETWEEN 0.5 TO 1.5 SECONDS

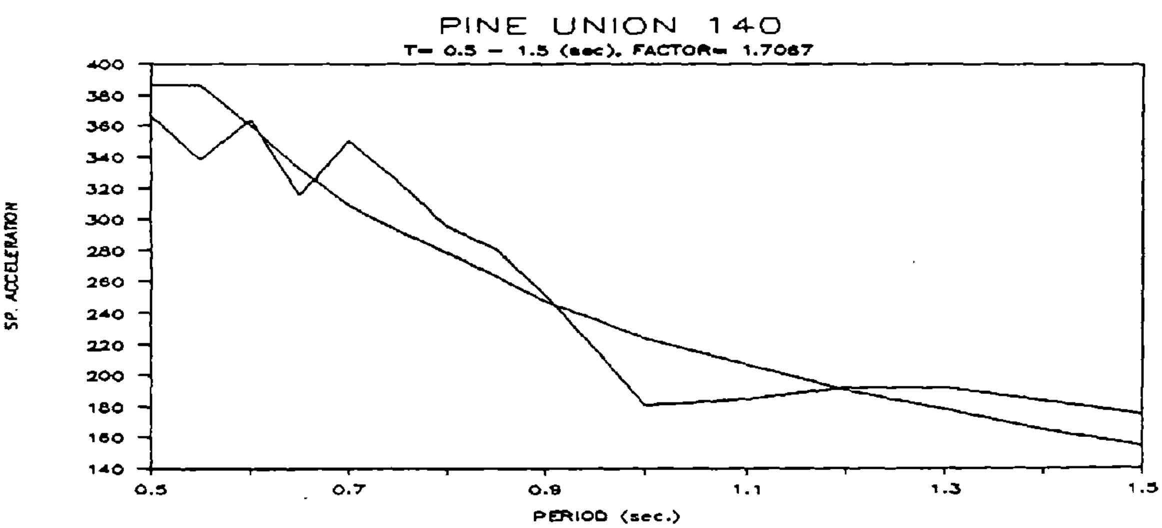
GROUND MOTION	ZPA = 0.4 MAXIMUM DISPLACEMENT INCHES	ZPA = 0.2 MAXIMUM DISPLACEMENT INCHES
EL CENTRO E-W	5.123	0.5898
EL CENTRO N-S	3.862	0.7526
PINE UNION 140	4.749	0.7986
CRUICKSHANK RD. 230	4.449	0.4857
JAMES RD. 140	4.278	1.4871
KERN CO. 69	3.693	0.9448
CRUICKSHANK RD. 140	4.065	0.4814
BRAWLEY 315	3.156	0.6804
KEYSTONE 140	6.000	1.1931
MEAN	4.375	0.824
STAND. DEV.	0.793	0.317

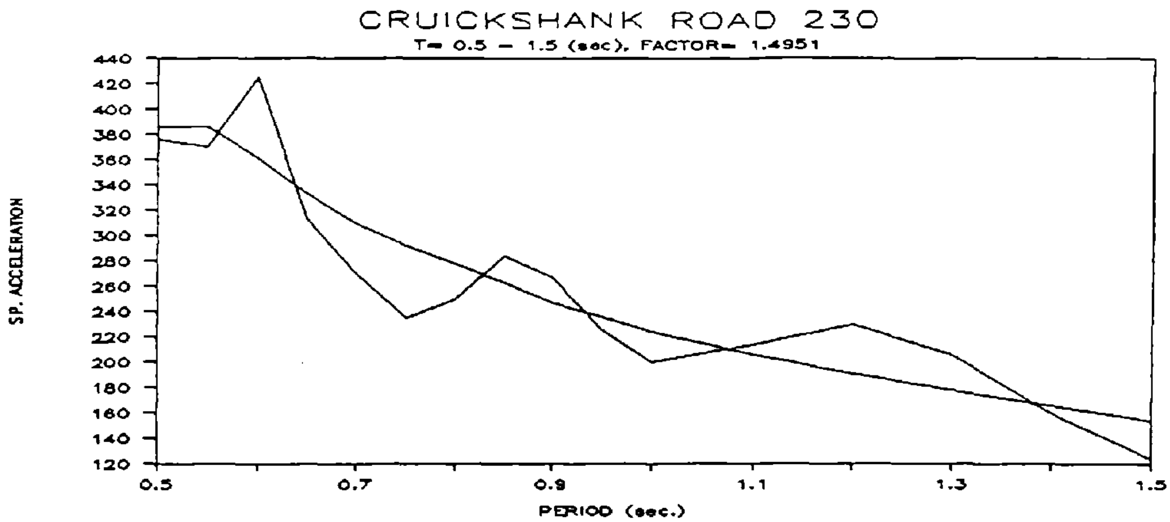
FIGURE GROUP 1

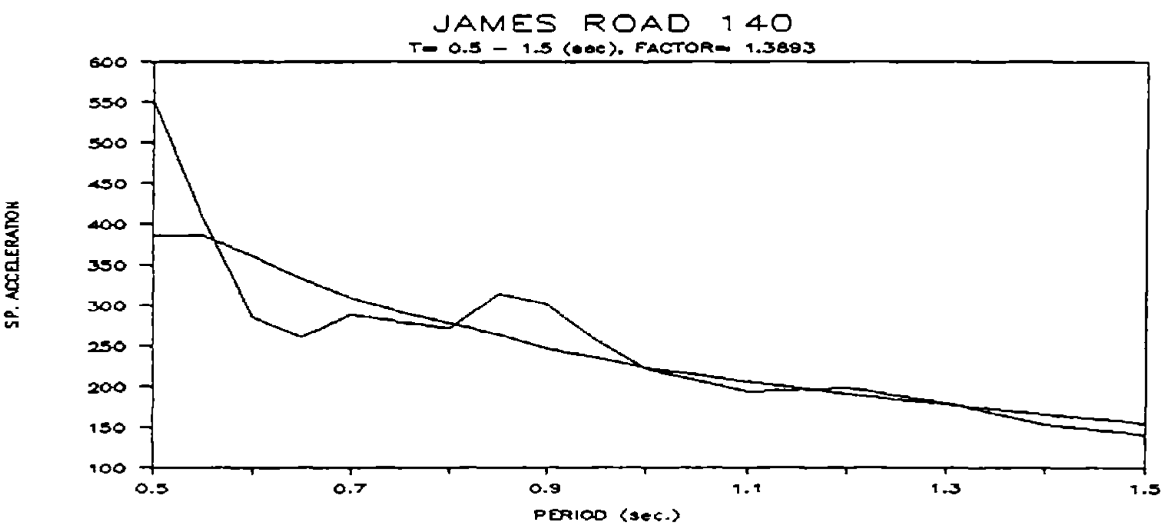
SPECTRA OF 18 GROUND MOTIONS

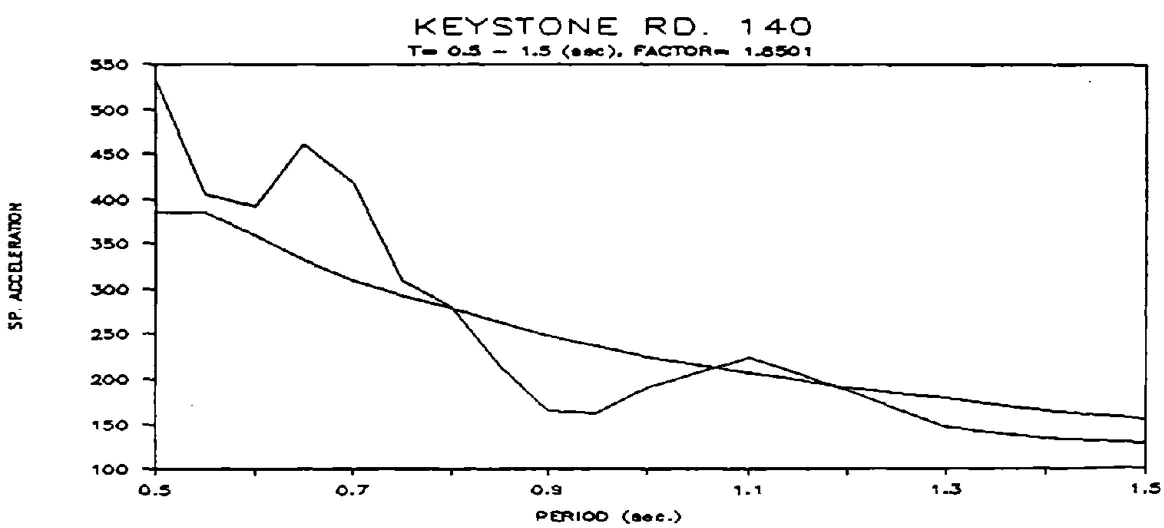

ELCENTRO E—W
T= 0.5 – 1.5 (sec). FACTOR= 1.7875
SP. ACCELERATION
PERIOD (sec.)


ELCENTRO N—S
T= 0.5 – 1.5 (sec). FACTOR= 1.3145
SP. ACCELERATION
PERIOD (sec.)

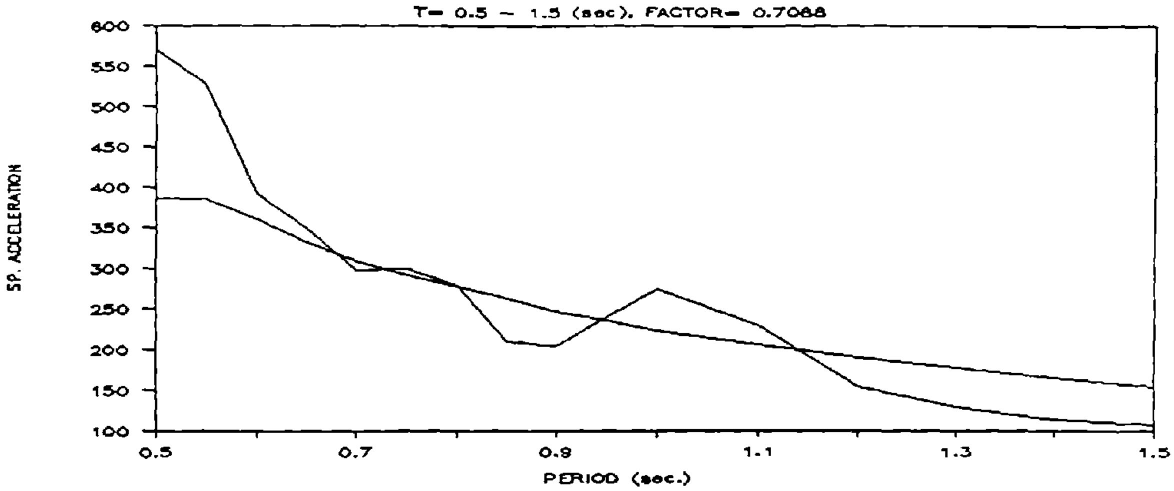

PINE UNION 140
T= 0.5 – 1.5 (sec). FACTOR= 1.7067
SP. ACCELERATION
PERIOD (sec.)


CRUICKSHANK ROAD 230
T= 0.5 - 1.5 (sec), FACTOR= 1.4951
SP. ACCELERATION
440
420
400
380
360
340
320
300
280
260
240
220
200
180
160
140
120
0.5
0.7
0.9
1.1
1.3
1.5
PERIOD (sec.)

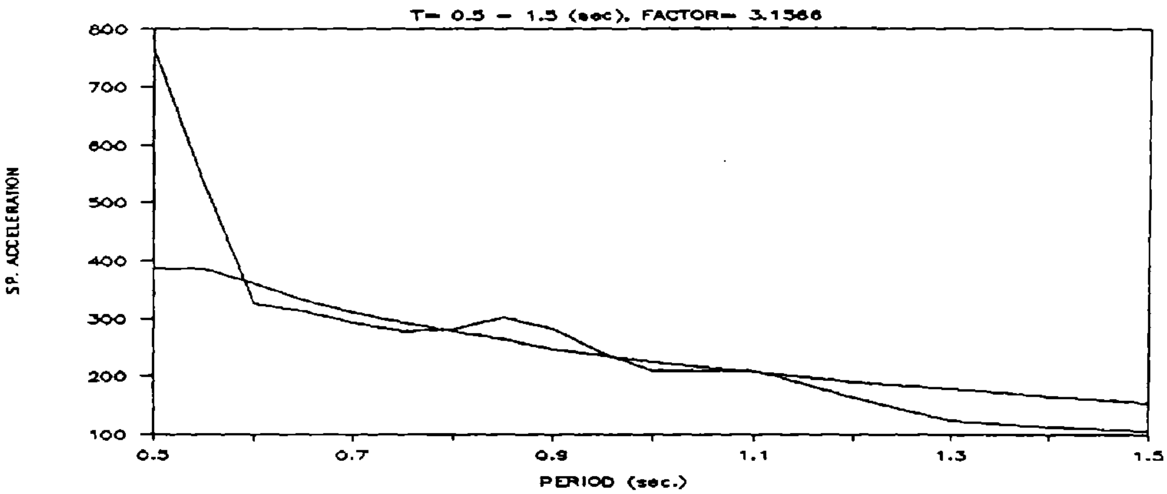

JAMES ROAD 140
T= 0.5 - 1.5 (sec), FACTOR= 1.3893
SP. ACCELERATION
600
550
500
450
400
350
300
250
200
150
100
0.5
0.7
0.9
1.1
1.3
1.5
PERIOD (sec.)


KEYSTONE RD. 140
T= 0.5 - 1.5 (sec), FACTOR= 1.8501
SP. ACCELERATION
550
500
450
400
350
300
250
200
150
100
0.5
0.7
0.9
1.1
1.3
1.5
PERIOD (sec.)

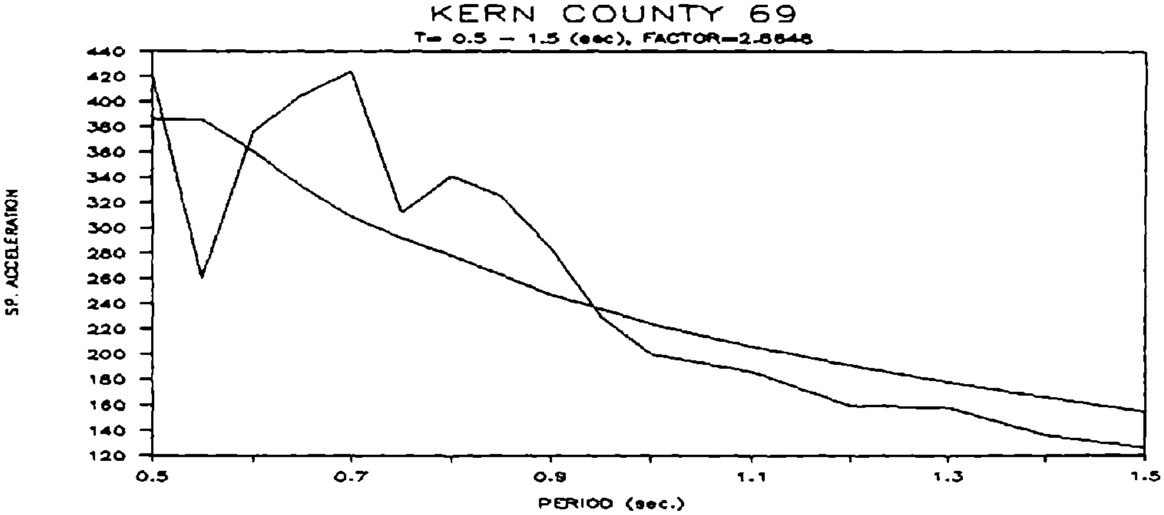
COALINGA 45



CASTAIC OLD RIDGE 21

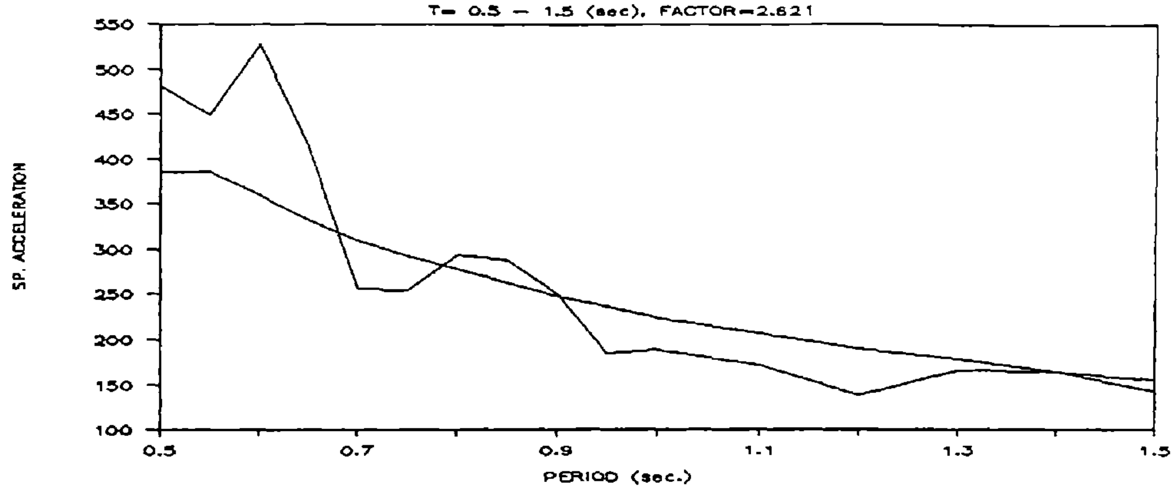


KERN COUNTY 69



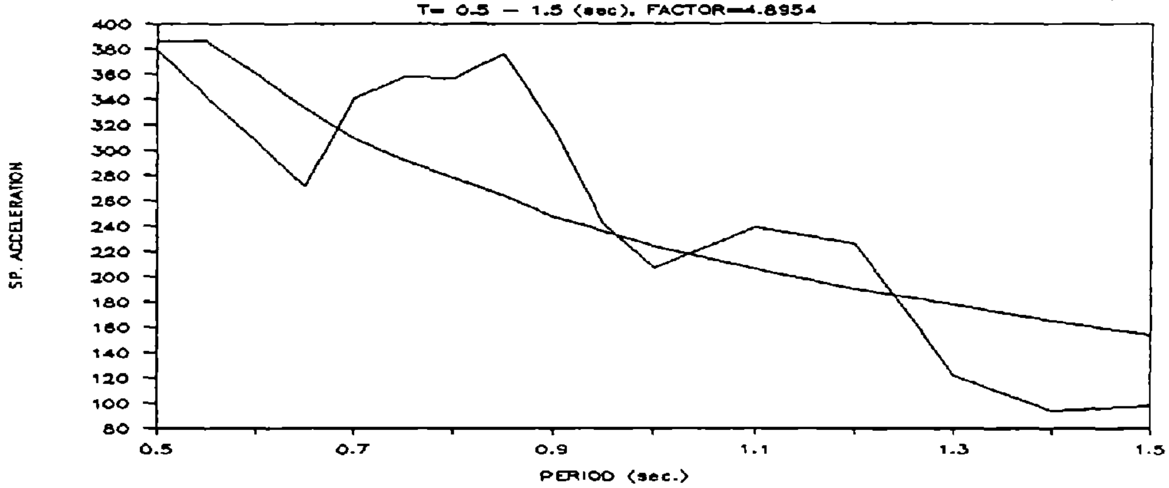
OLYMPIA 86

T= 0.5 – 1.5 (sec), FACTOR=2.621



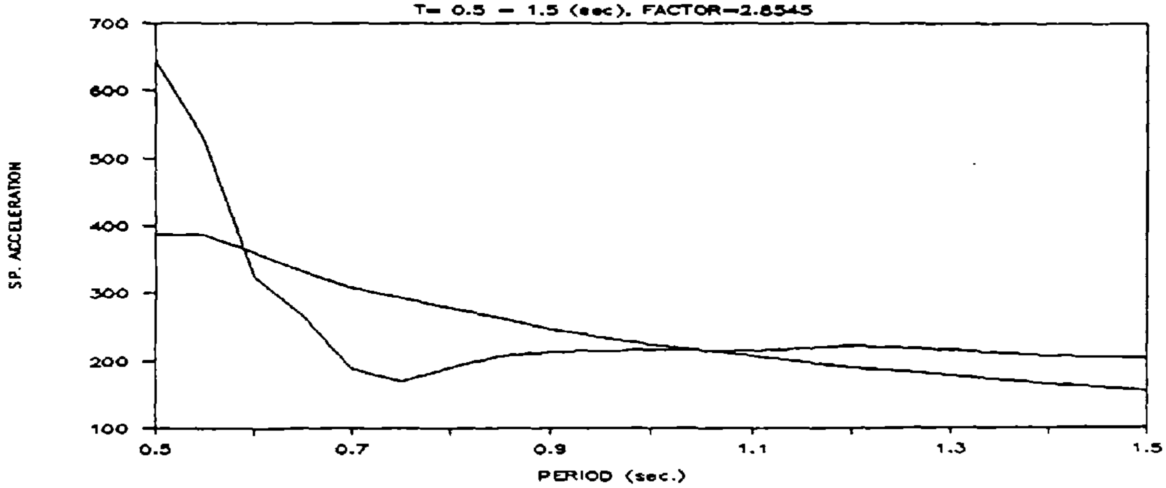
HOLLYWOOD STORAGE 90

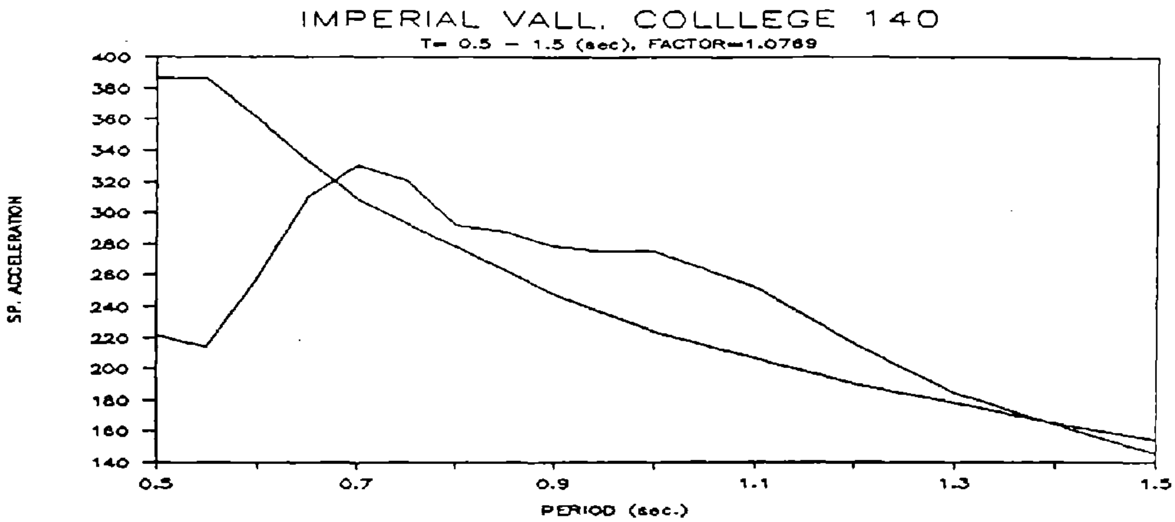
T= 0.5 – 1.5 (sec), FACTOR=4.8954

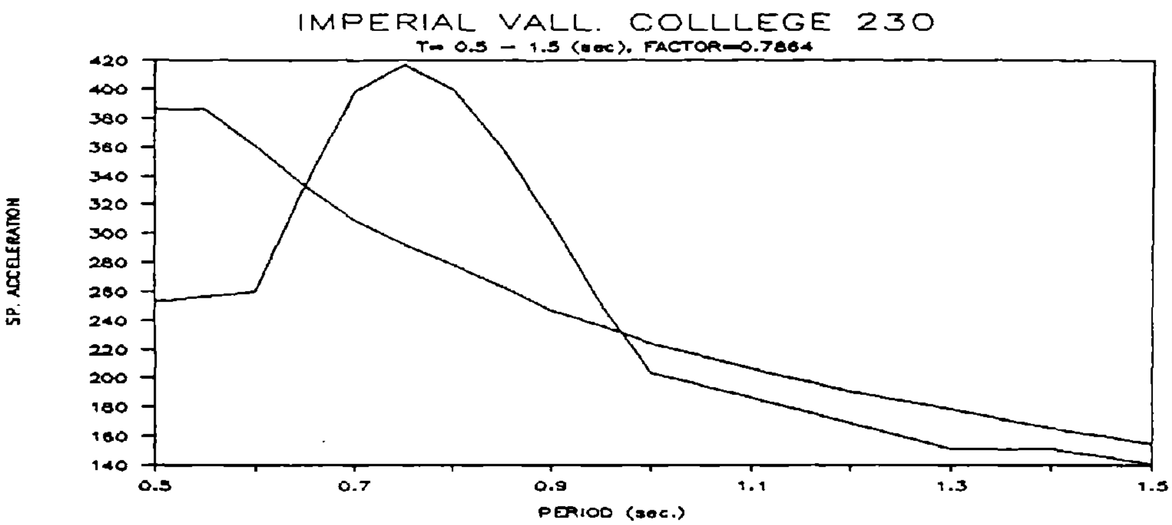


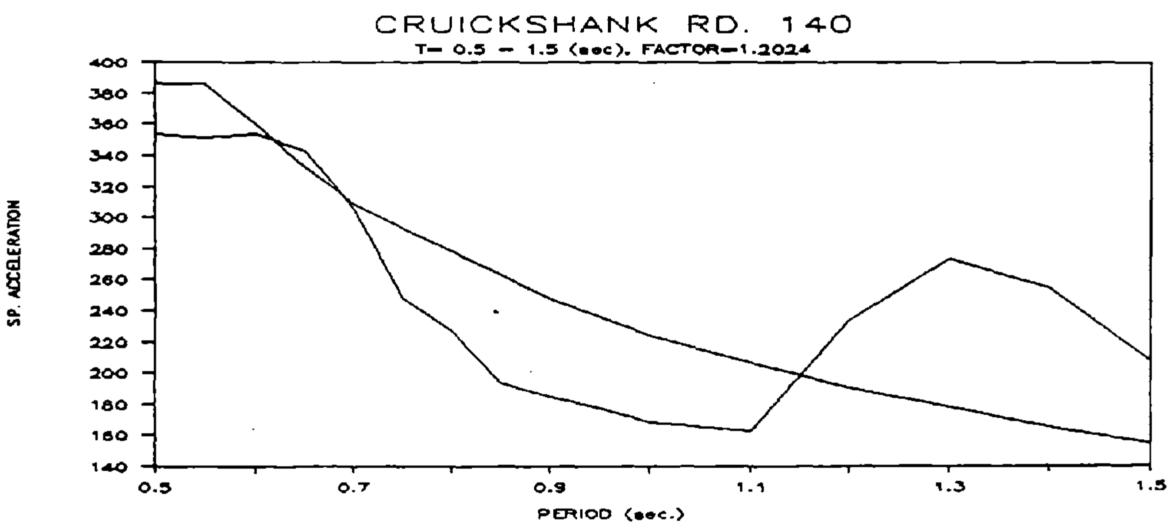
COMMUNITY HOSPITAL 50

T= 0.5 – 1.5 (sec), FACTOR=2.8545



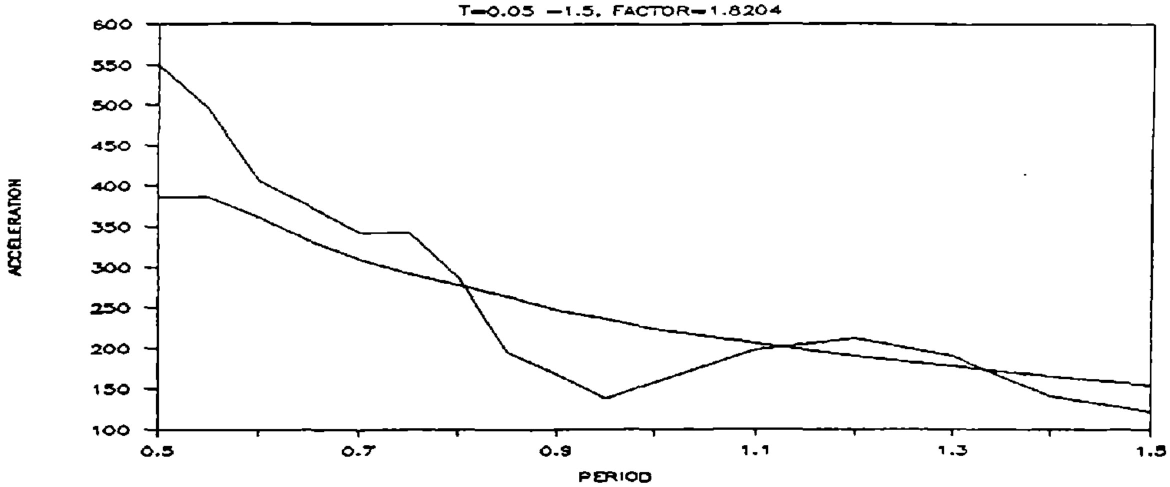

IMPERIAL VALL. COLLLEGE 140
T= 0.5 – 1.5 (sec), FACTOR=1.0769
SP. ACCELERATION
PERIOD (sec.)


IMPERIAL VALL. COLLLEGE 230
T= 0.5 – 1.5 (sec), FACTOR=0.7864
SP. ACCELERATION
PERIOD (sec.)


CRUICKSHANK RD. 140
T= 0.5 – 1.5 (sec), FACTOR=1.2024
SP. ACCELERATION
PERIOD (sec.)

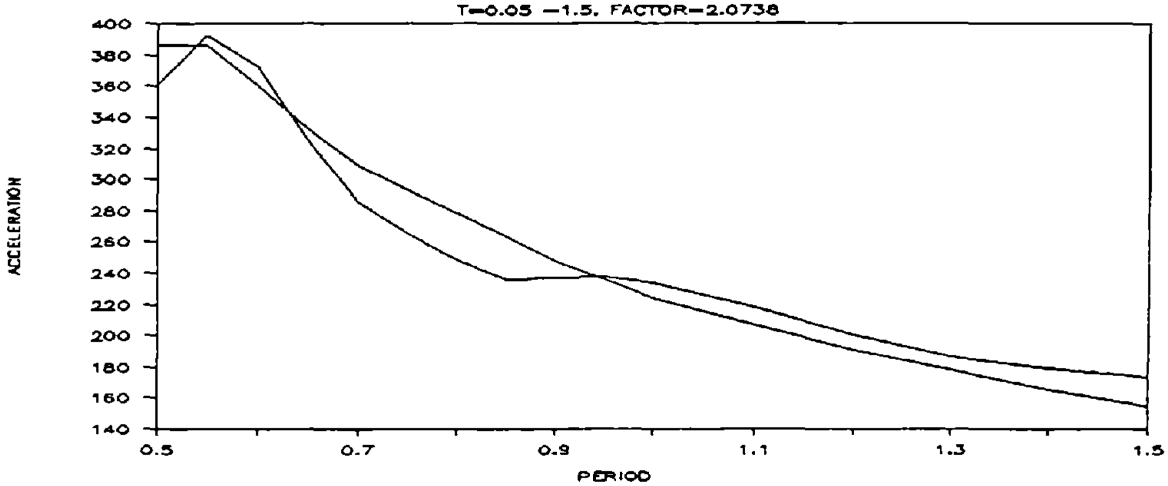
MCCABE SCHOOL 230

T=0.05 -1.5. FACTOR=1.8204



BRAWLEY AIRP 315

T=0.05 -1.5. FACTOR=2.0738



CALIPATRIA FIRE STN 225

T=0.05 -1.5. FACTOR=6.6300

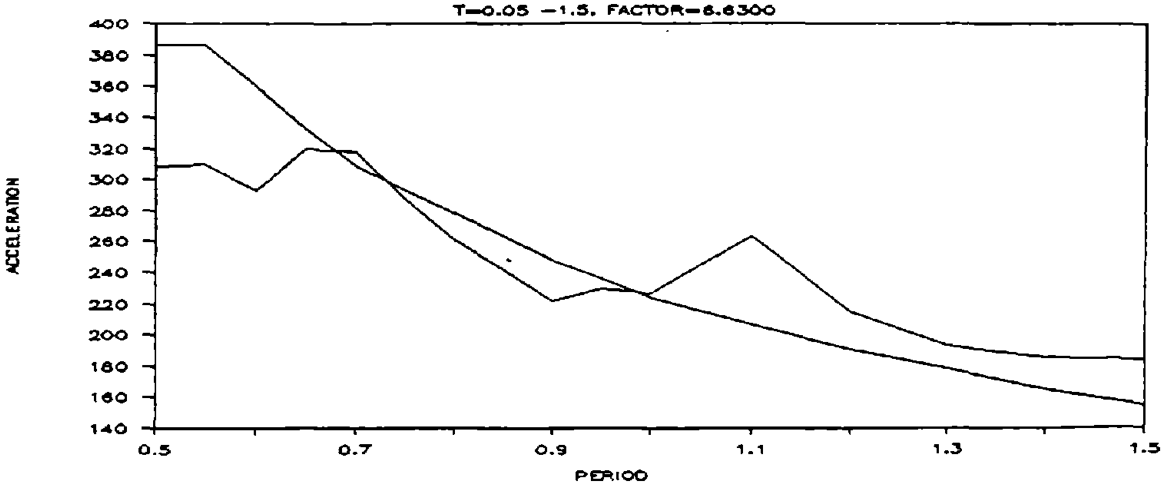
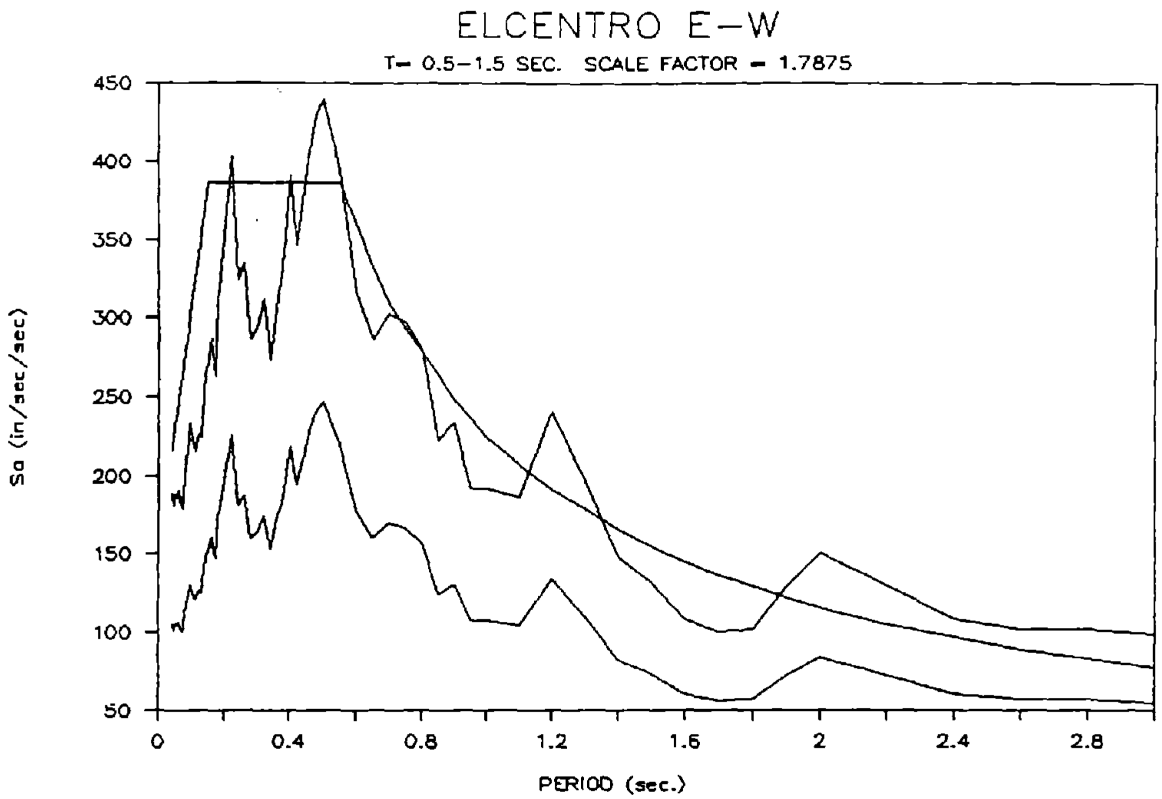
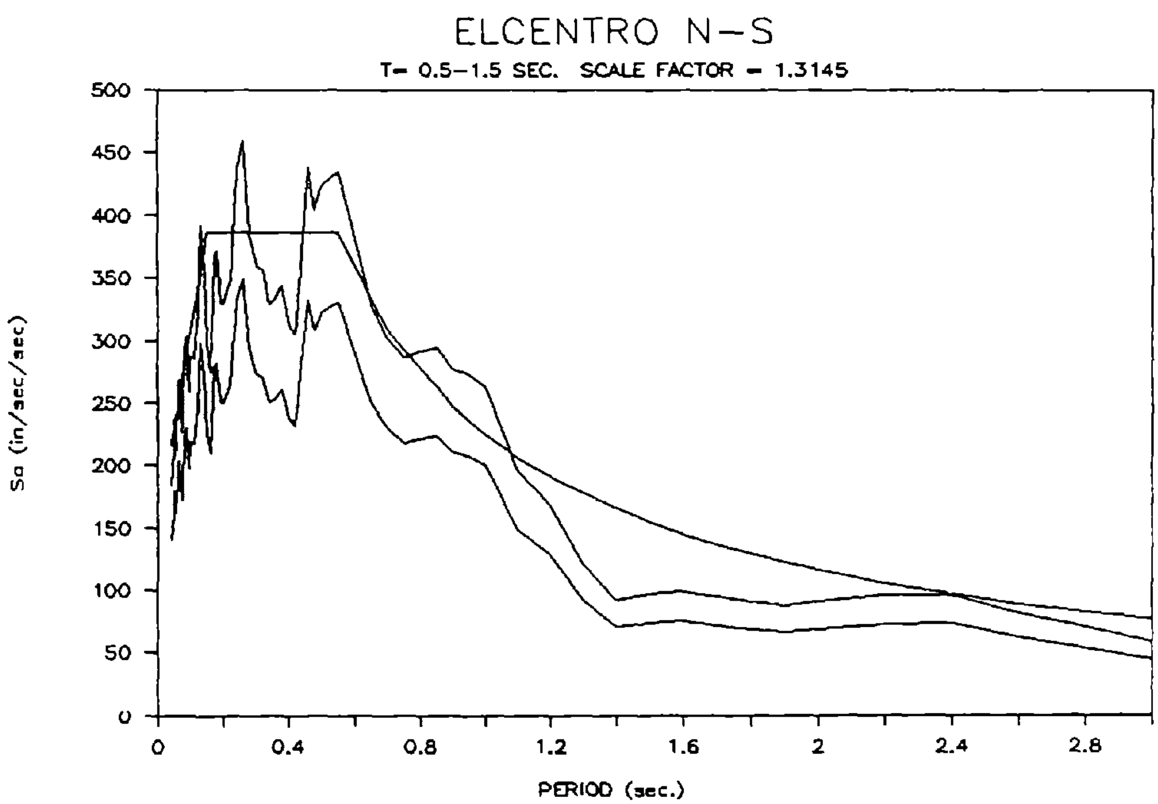


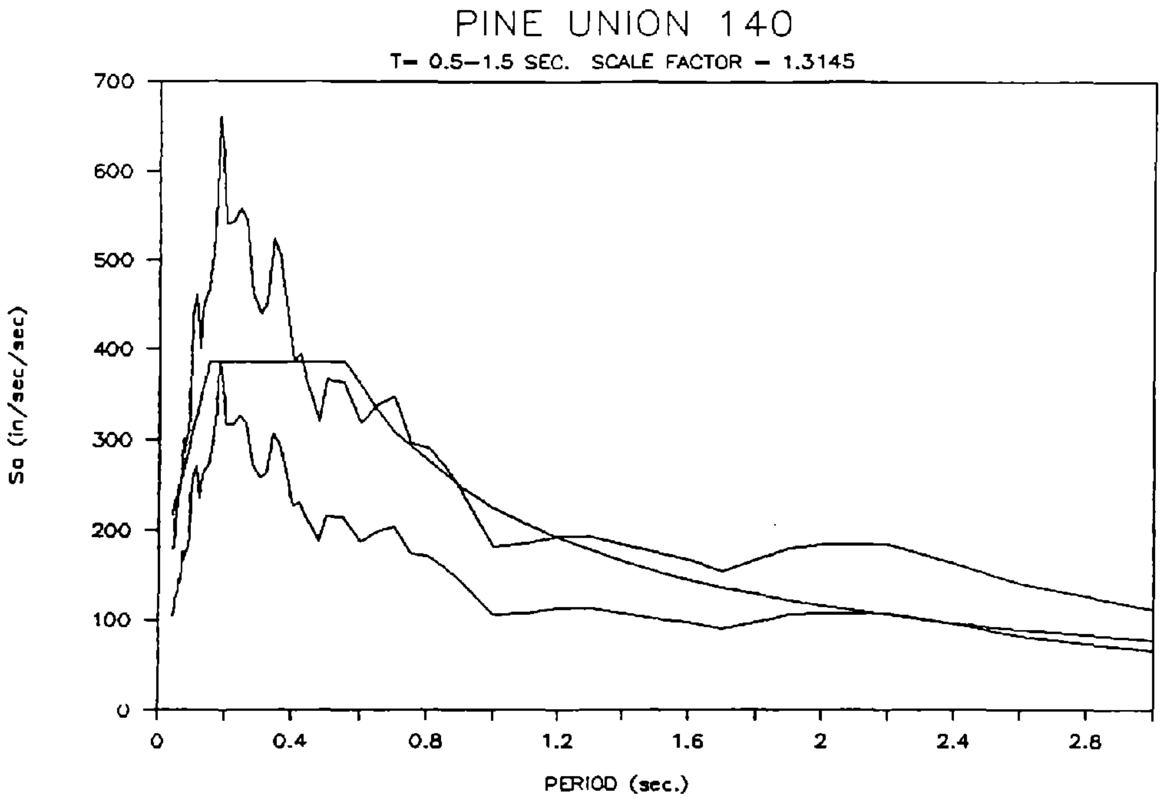
FIGURE GROUP 2

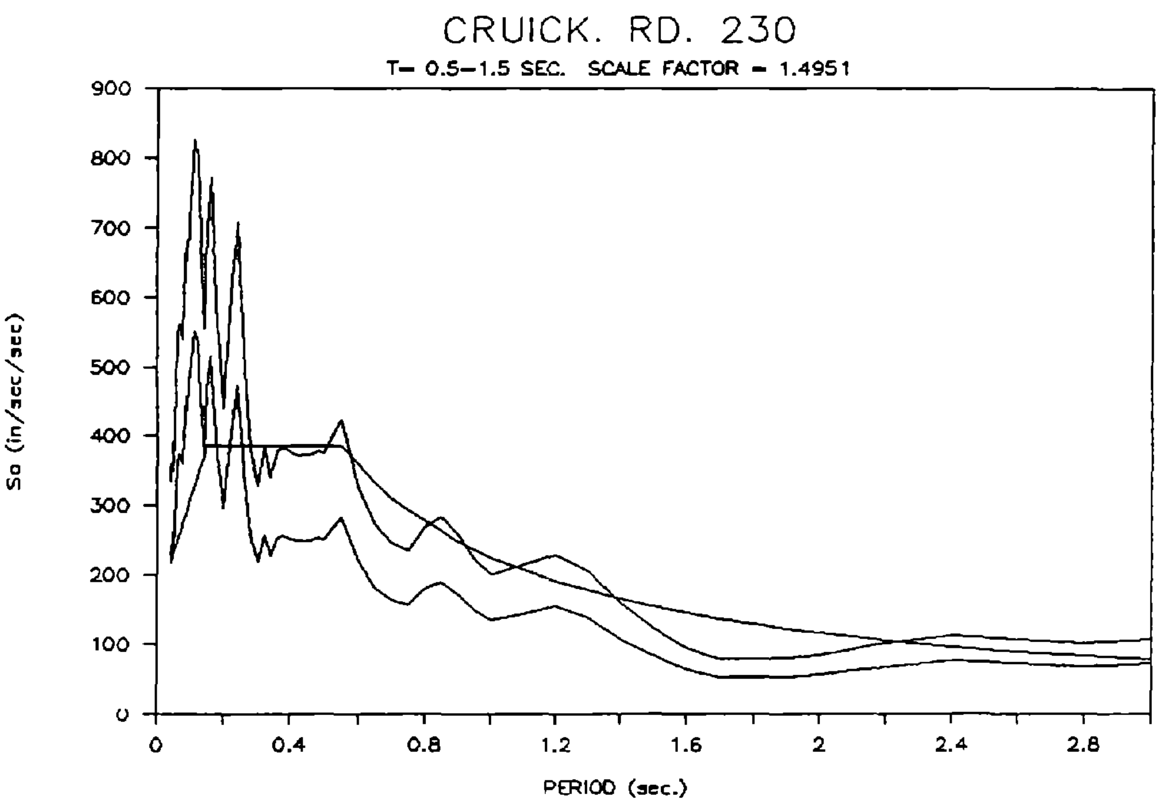
SPECTRA OF 12 GROUND MOTIONS

SELECTED FOR FURTHER INVESTIGATION


ELCENTRO E-W
T= 0.5-1.5 SEC. SCALE FACTOR = 1.7875
Sa (in/sec/sec)
450
400
350
300
250
200
150
100
50
0
0.4
0.8
1.2
1.6
2
2.4
2.8
PERIOD (sec.)

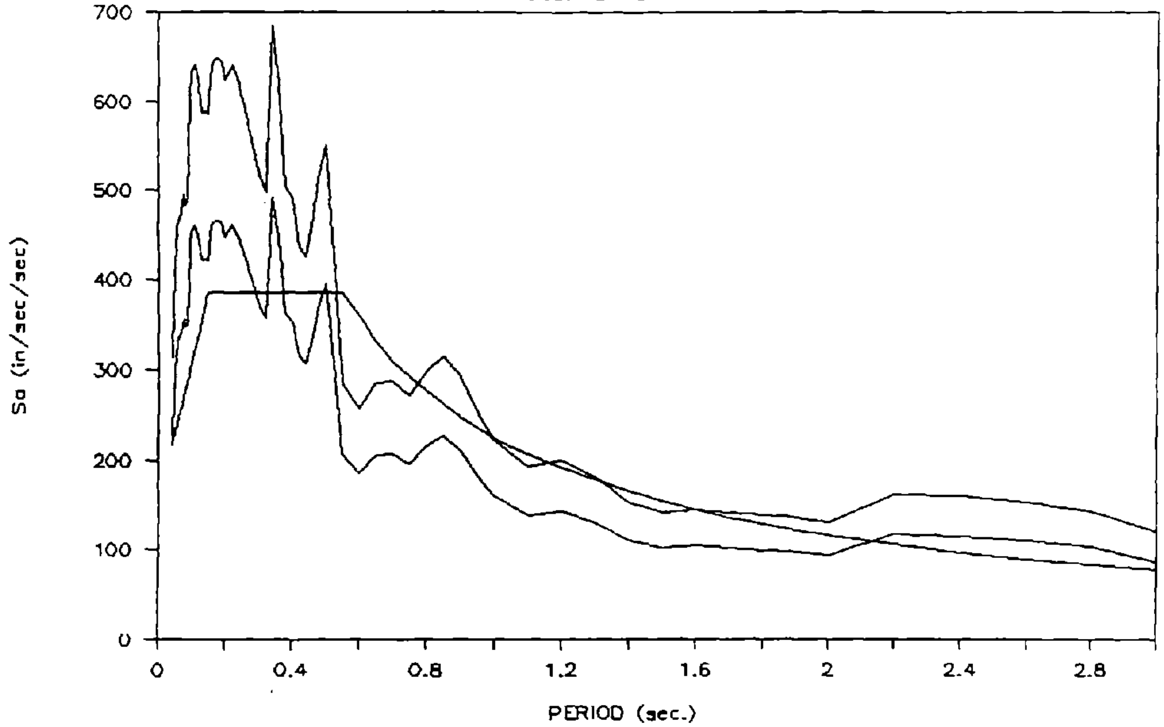

ELCENTRO N-S
T= 0.5-1.5 SEC. SCALE FACTOR = 1.3145
Sa (in/sec/sec)
500
450
400
350
300
250
200
150
100
50
0
0
0.4
0.8
1.2
1.6
2
2.4
2.8
PERIOD (sec.)


PINE UNION 140
T= 0.5–1.5 SEC. SCALE FACTOR = 1.3145
Sa (in/sec/sec)
PERIOD (sec.)


CRUICK. RD. 230
T= 0.5–1.5 SEC. SCALE FACTOR = 1.4951
Sa (in/sec/sec)
PERIOD (sec.)

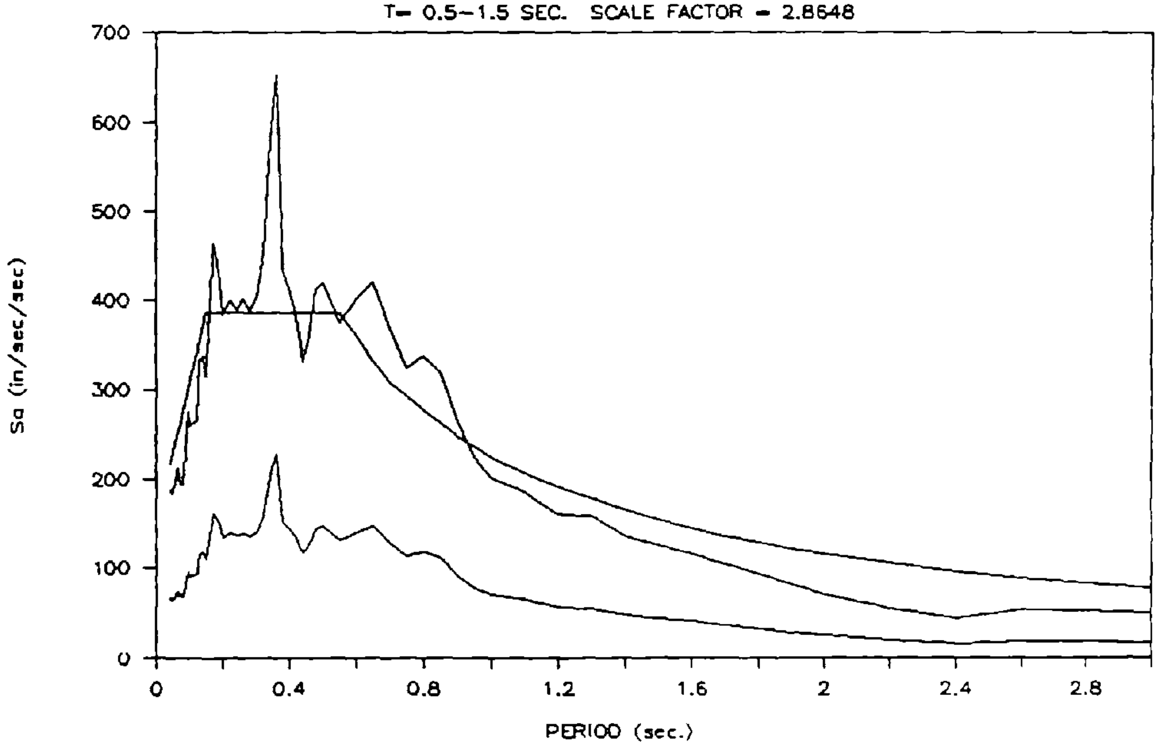
JAMES ROAD 140

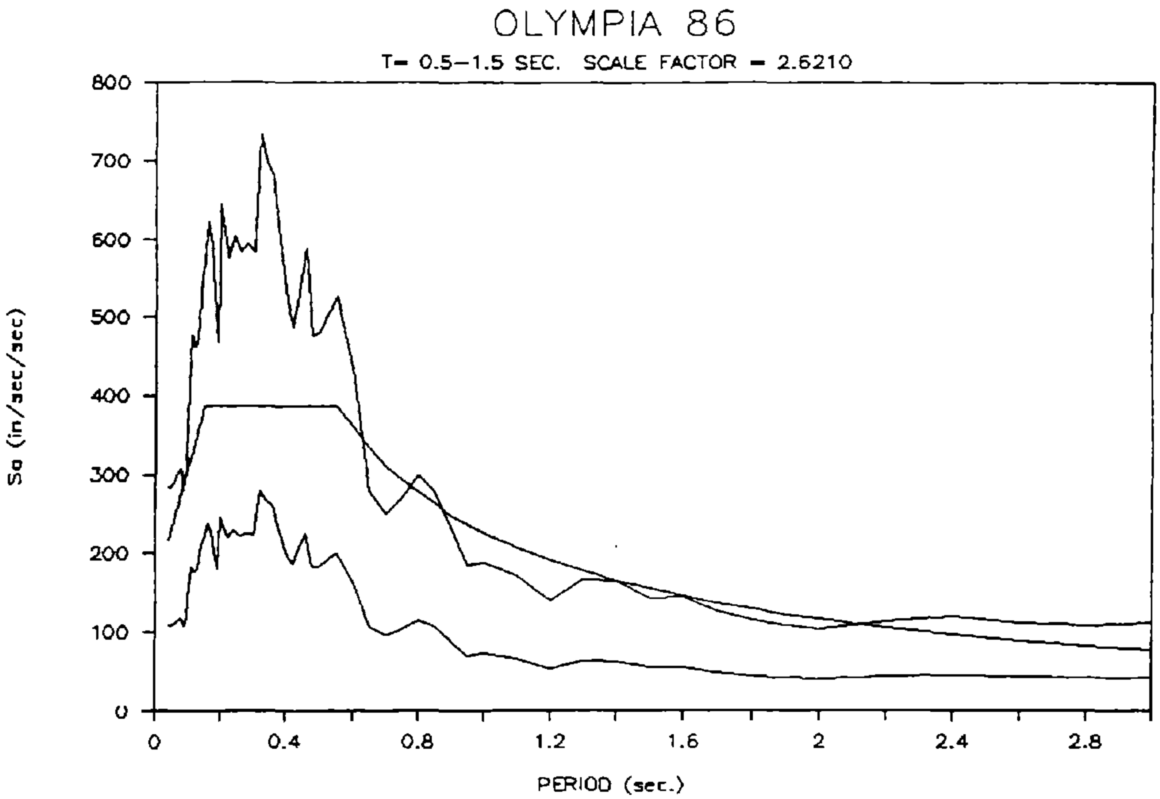
T= 0.5–1.5 SEC. SCALE FACTOR = 1.3893



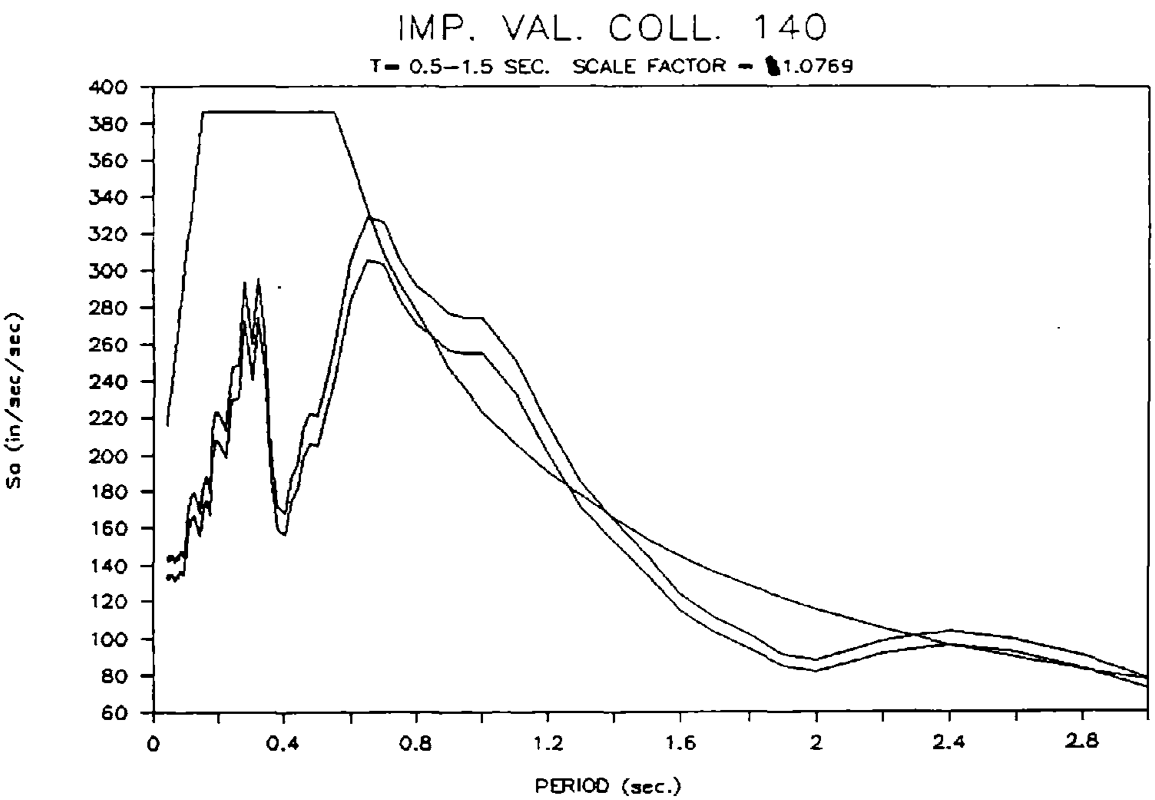
KERN CO. 69

T= 0.5–1.5 SEC. SCALE FACTOR = 2.8648

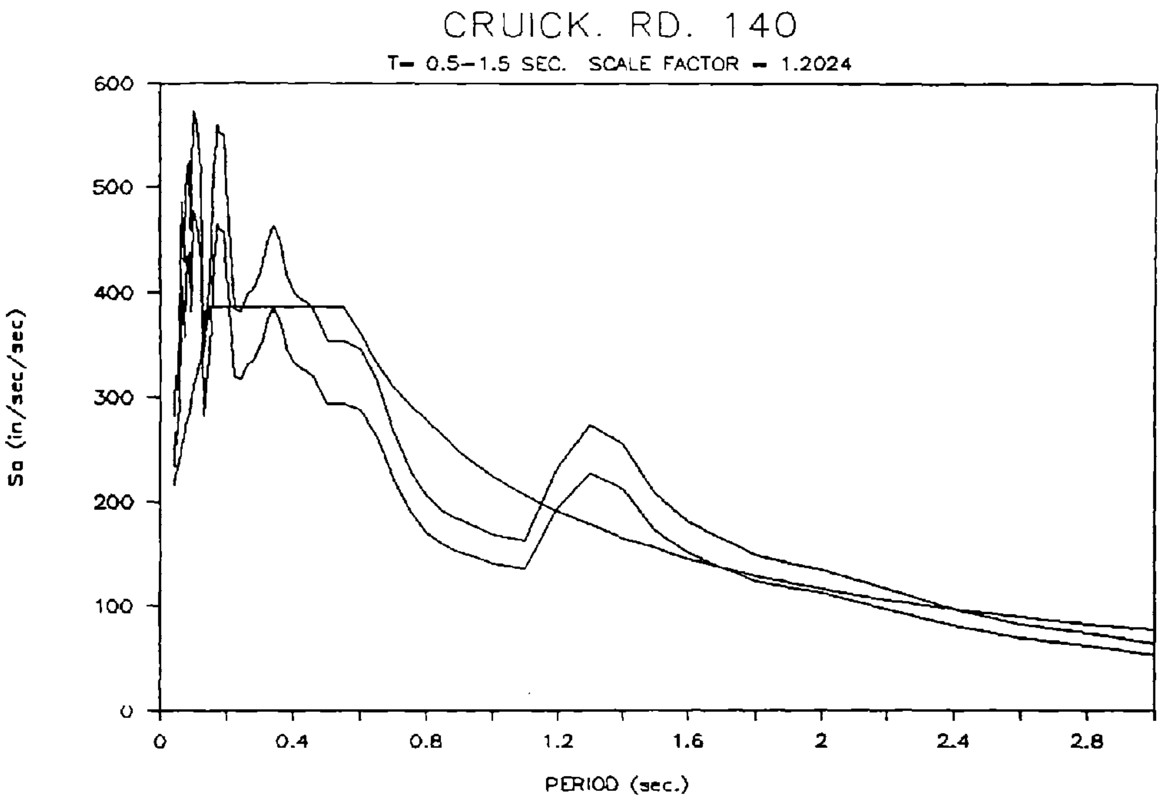


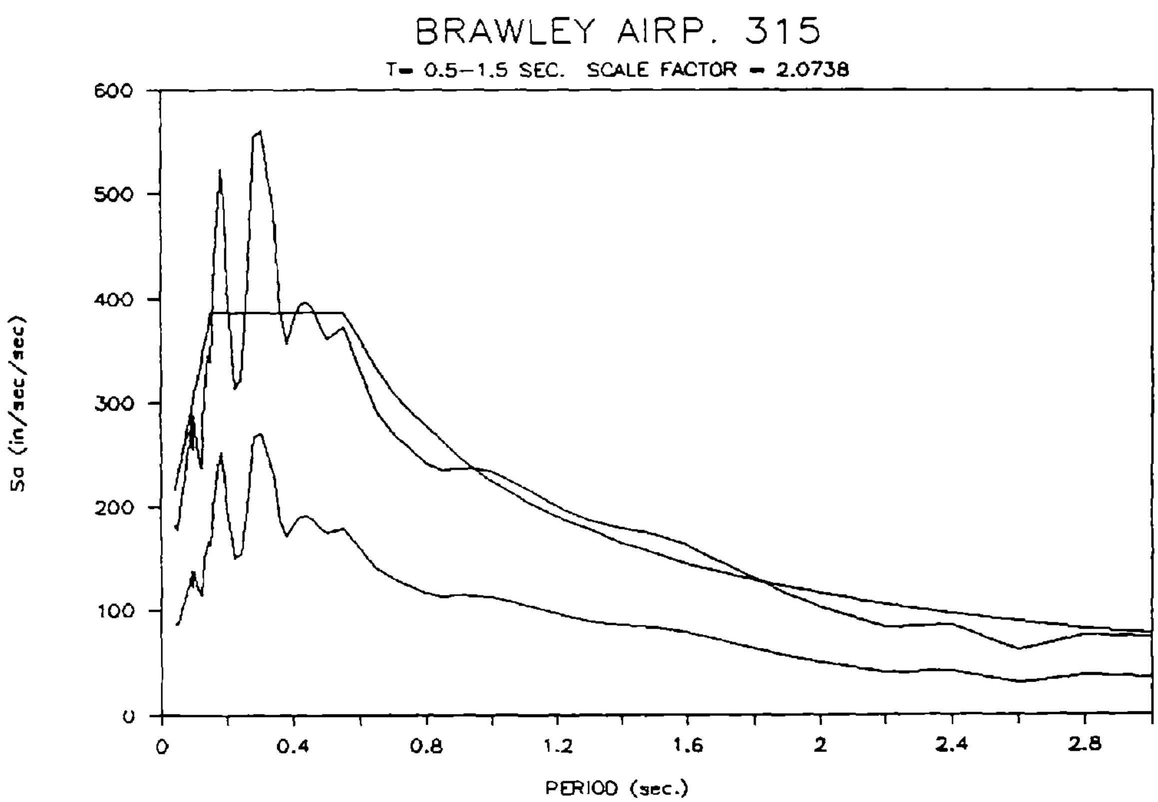


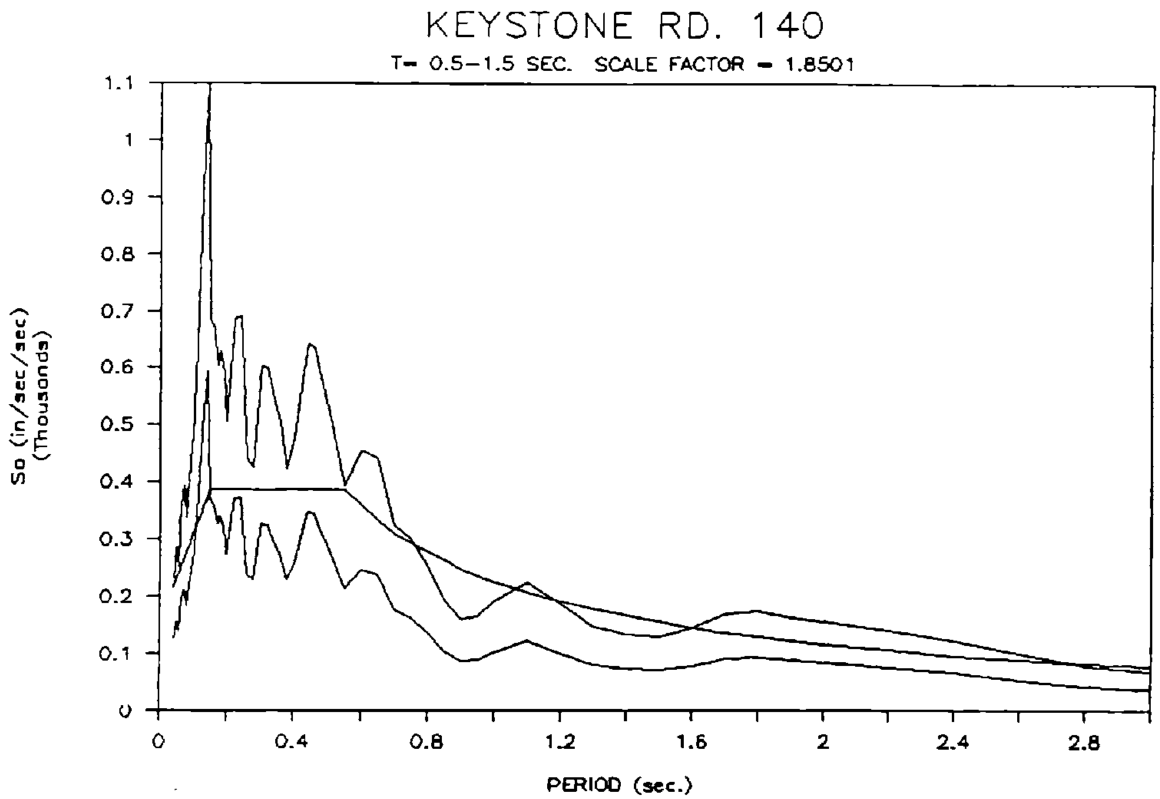
OLYMPIA 86
T= 0.5–1.5 SEC. SCALE FACTOR = 2.6210
Sa (in/sec/sec)
800
700
600
500
400
300
200
100
0
0
0.4
0.8
1.2
1.6
2
2.4
2.8
PERIOD (sec.)

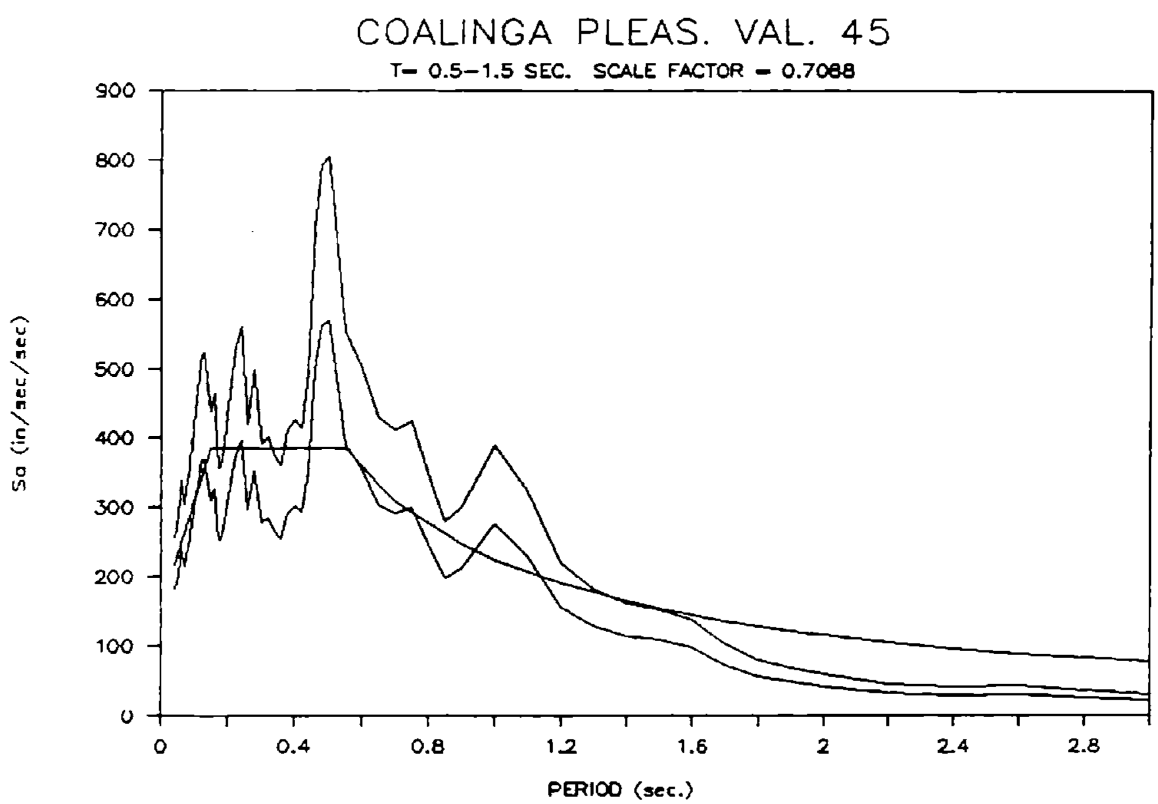


IMP. VAL. COLL. 140
T= 0.5–1.5 SEC. SCALE FACTOR = 1.0769
Sa (in/sec/sec)
400
380
360
340
320
300
280
260
240
220
200
180
160
140
120
100
80
60
0
0.4
0.8
1.2
1.6
2
2.4
2.8
PERIOD (sec.)


CRUICK. RD. 140
T= 0.5–1.5 SEC. SCALE FACTOR = 1.2024
Sa (in/sec/sec)
600
500
400
300
200
100
0
0
0.4
0.8
1.2
1.6
2
2.4
2.8
PERIOD (sec.)


BRAWLEY AIRP. 315
T= 0.5–1.5 SEC. SCALE FACTOR = 2.0738
Sa (in/sec/sec)
600
500
400
300
200
100
0
0
0.4
0.8
1.2
1.6
2
2.4
2.8
PERIOD (sec.)


KEYSTONE RD. 140
T= 0.5–1.5 SEC. SCALE FACTOR = 1.8501
Sa (in/sec/sec) (Thousands)
PERIOD (sec.)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
1.1
0
0.4
0.8
1.2
1.6
2
2.4
2.8


COALINGA PLEAS. VAL. 45
T= 0.5–1.5 SEC. SCALE FACTOR = 0.7088
Sa (in/sec/sec)
PERIOD (sec.)
0
100
200
300
400
500
600
700
800
900
0
0.4
0.8
1.2
1.6
2
2.4
2.8

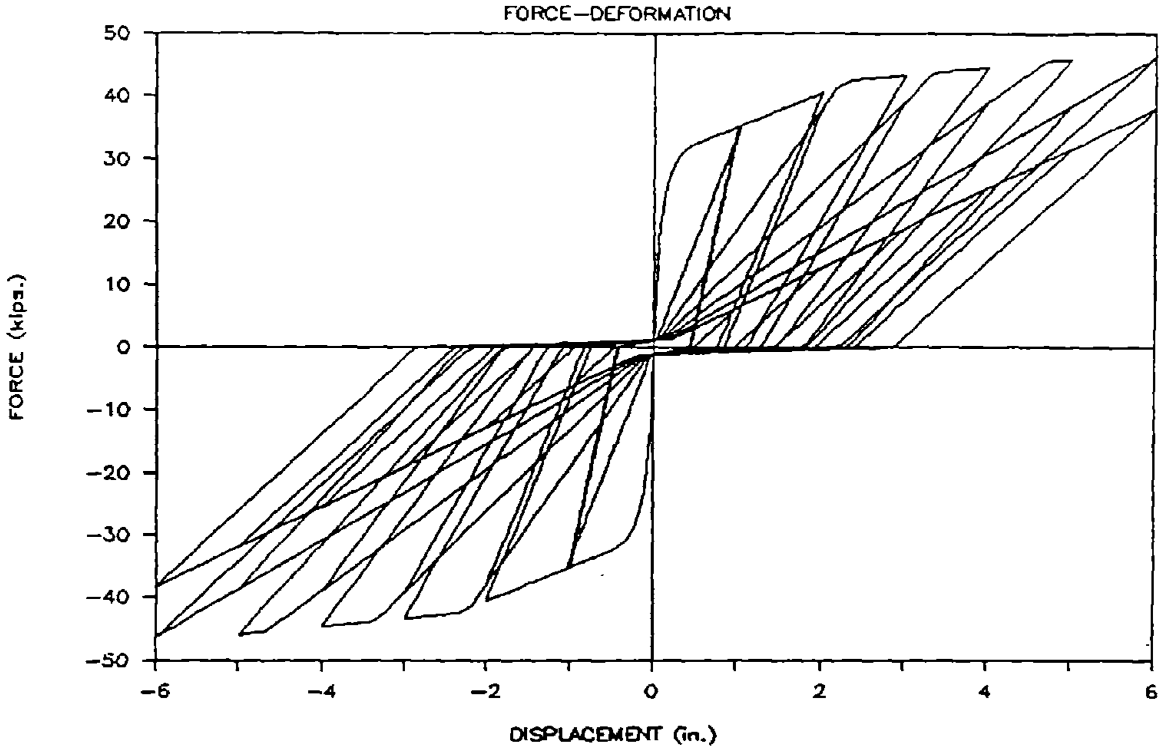


FIGURE 3a

SPRING 11

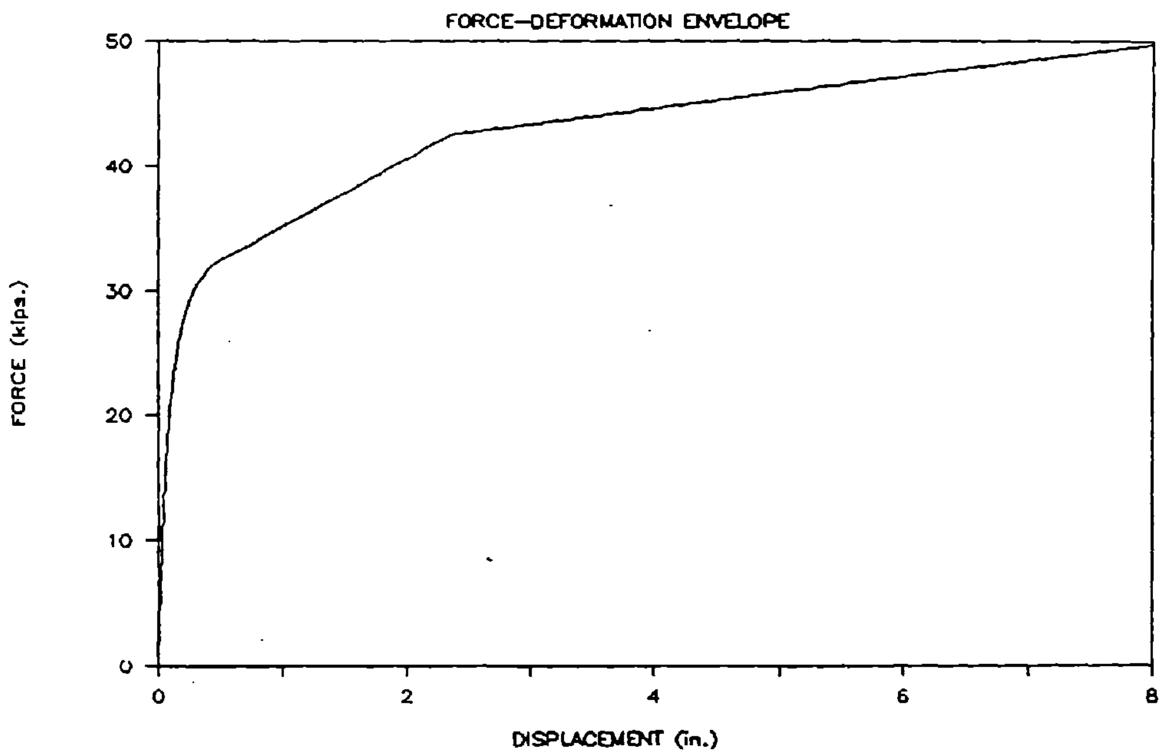


FIGURE 3b

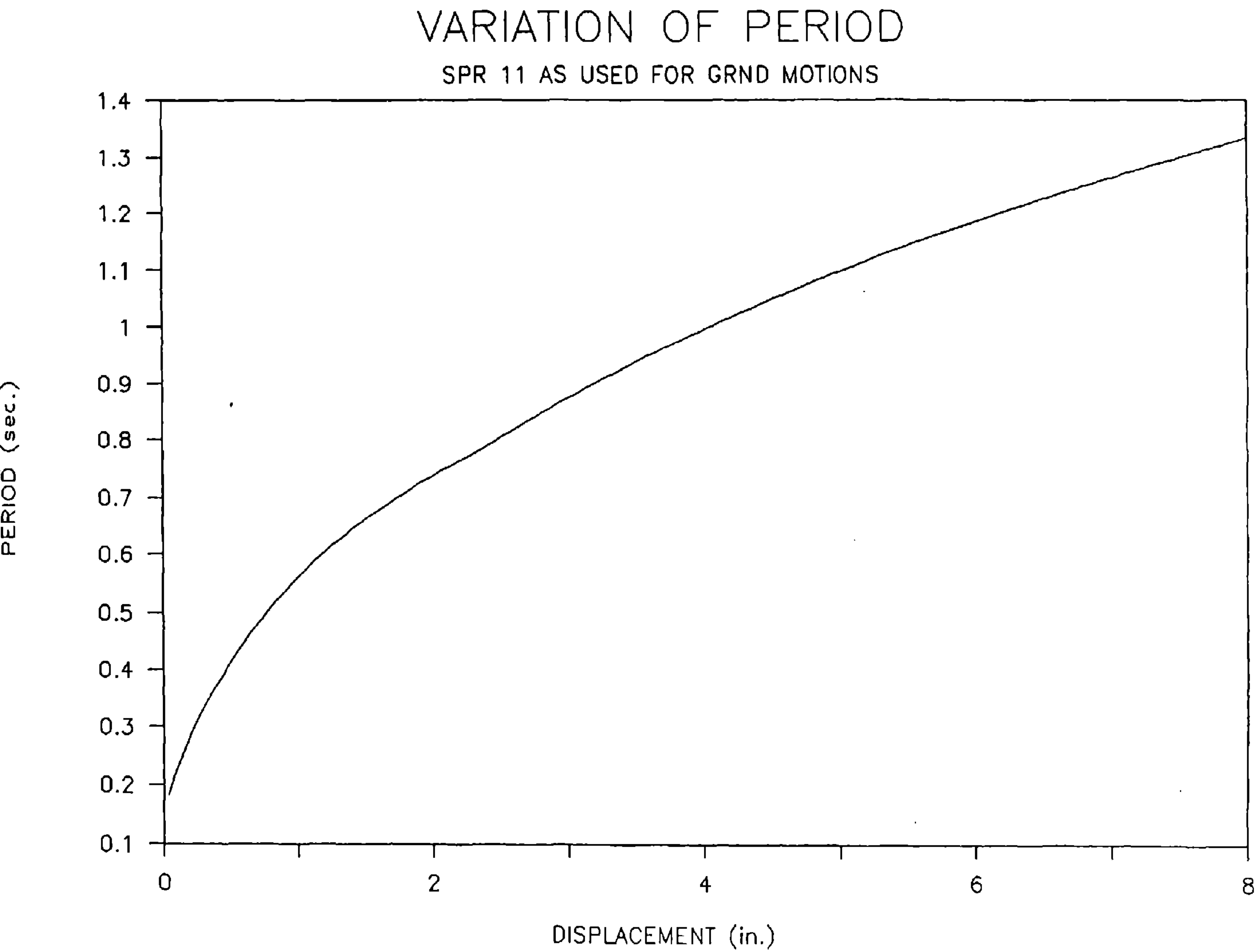

VARIATION OF PERIOD
SPR 11 AS USED FOR GRND MOTIONS
PERIOD (sec.)
1.4
1.3
1.2
1.1
1
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0
2
4
6
8
DISPLACEMENT (in.)

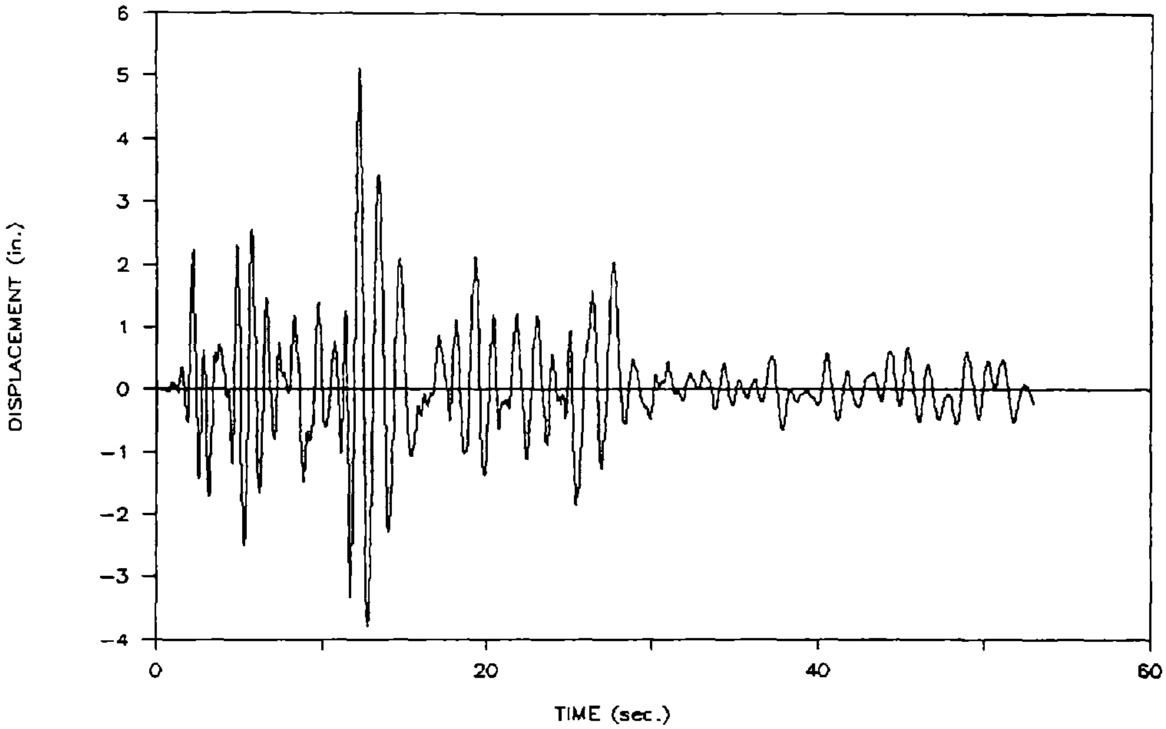
FIGURE 3c

FIGURE GROUP 4

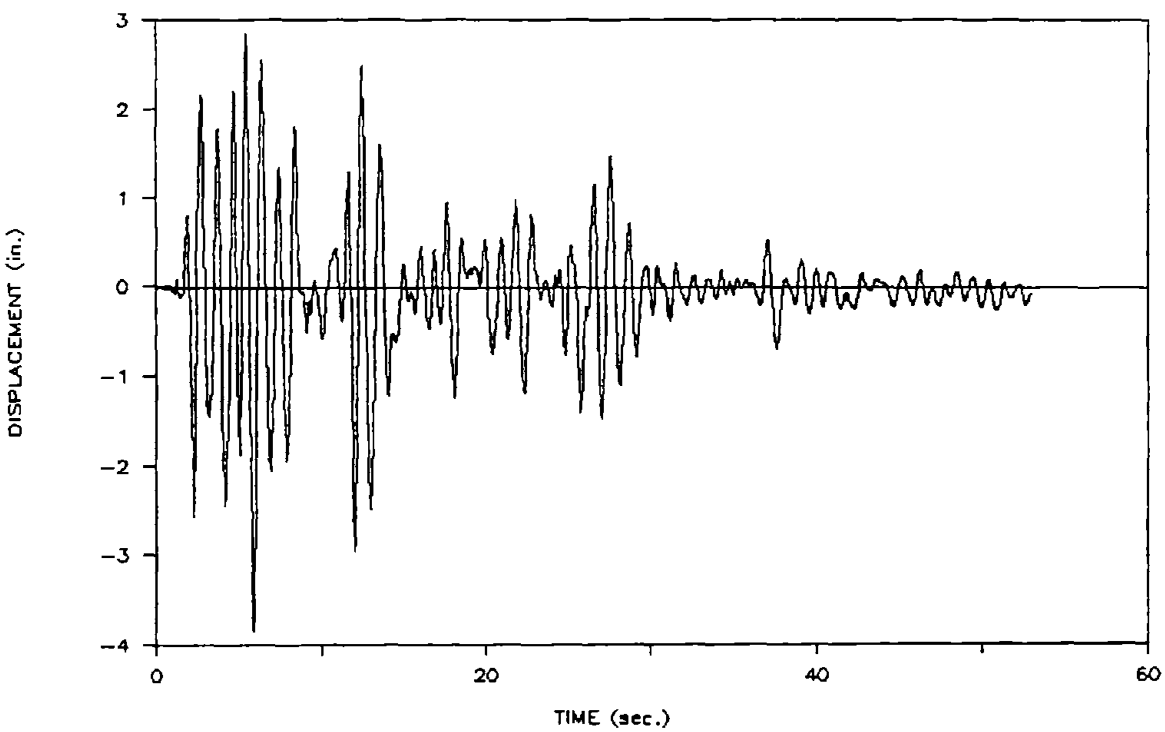
RESPONSE OF SDOF SYSTEM

TO 12 SCALED GROUND MOTIONS

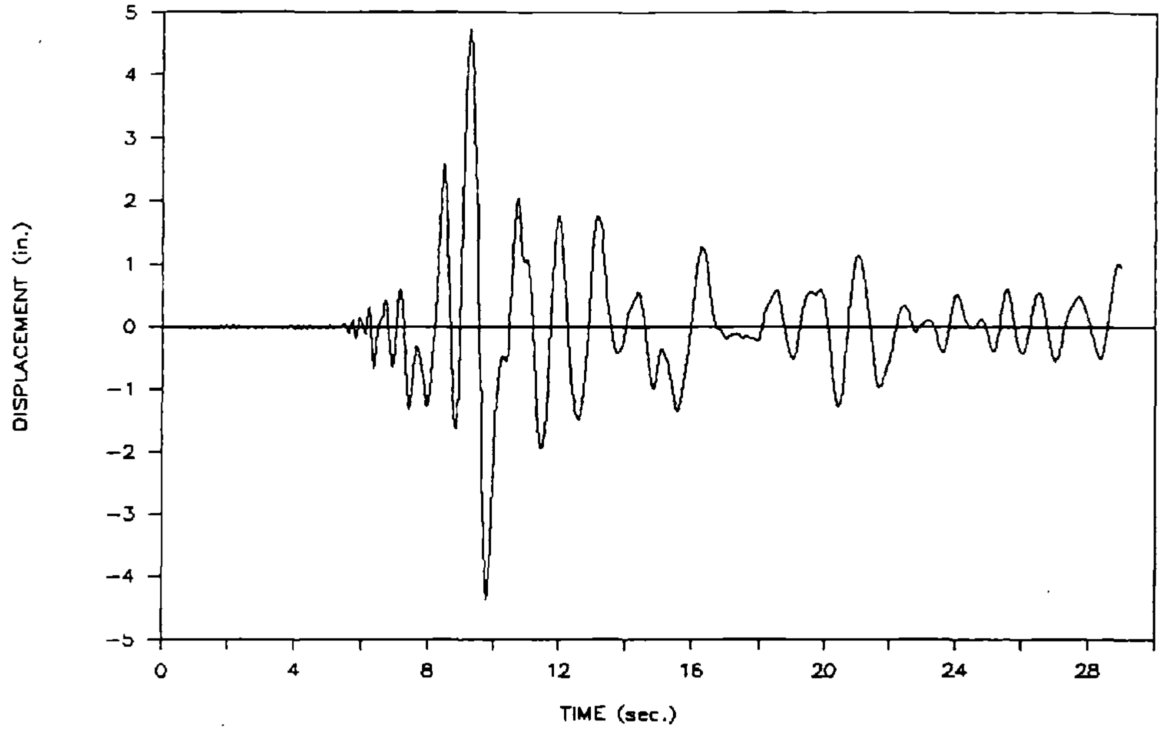
ELCENTRO E—W



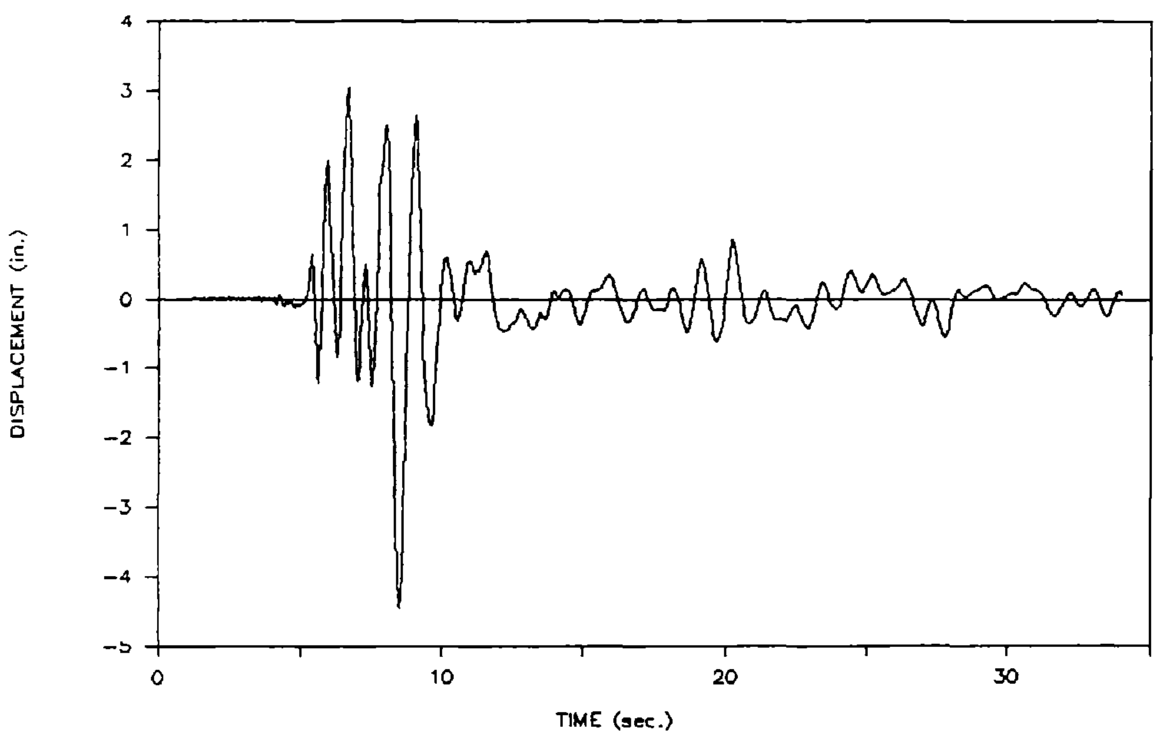
ELCENTRO N—S



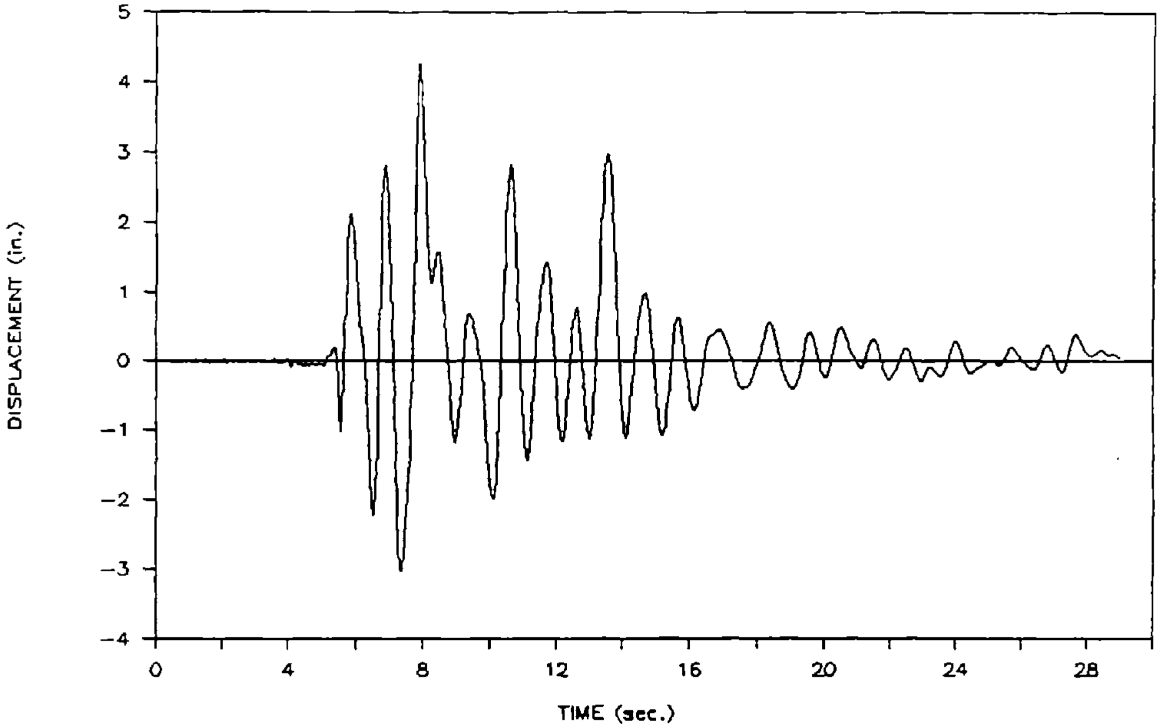
PINE UNION 140



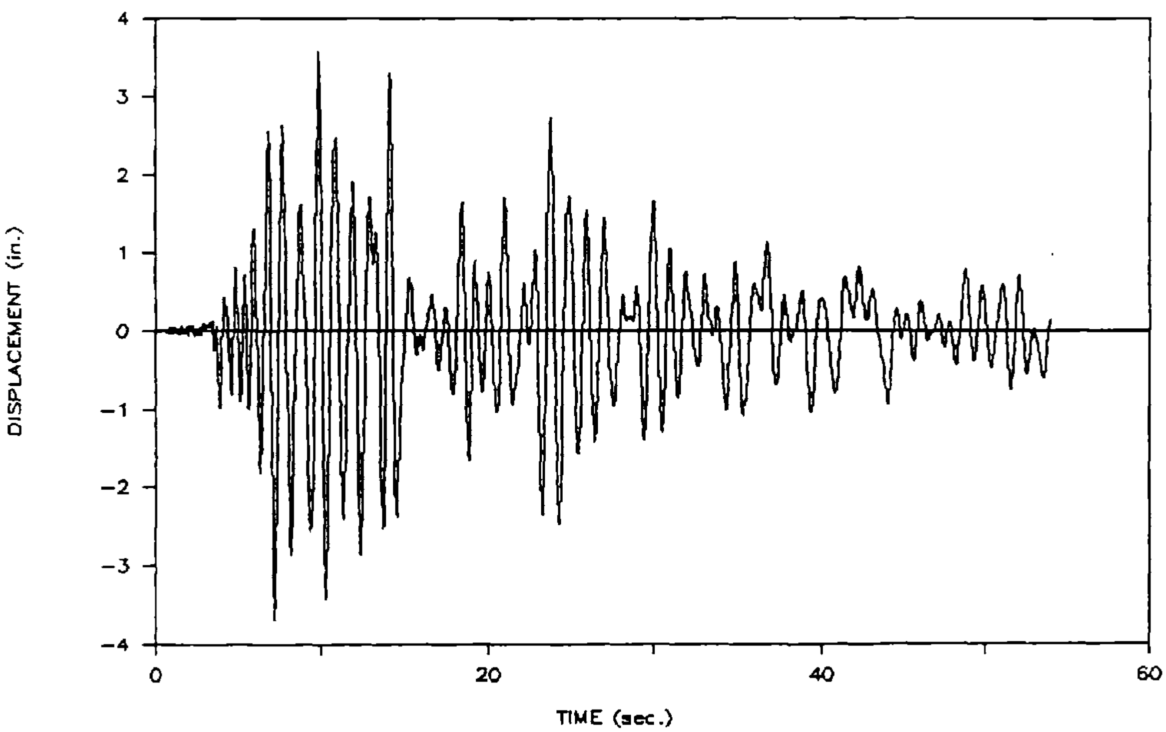
CRUICKSHANK RD. 230



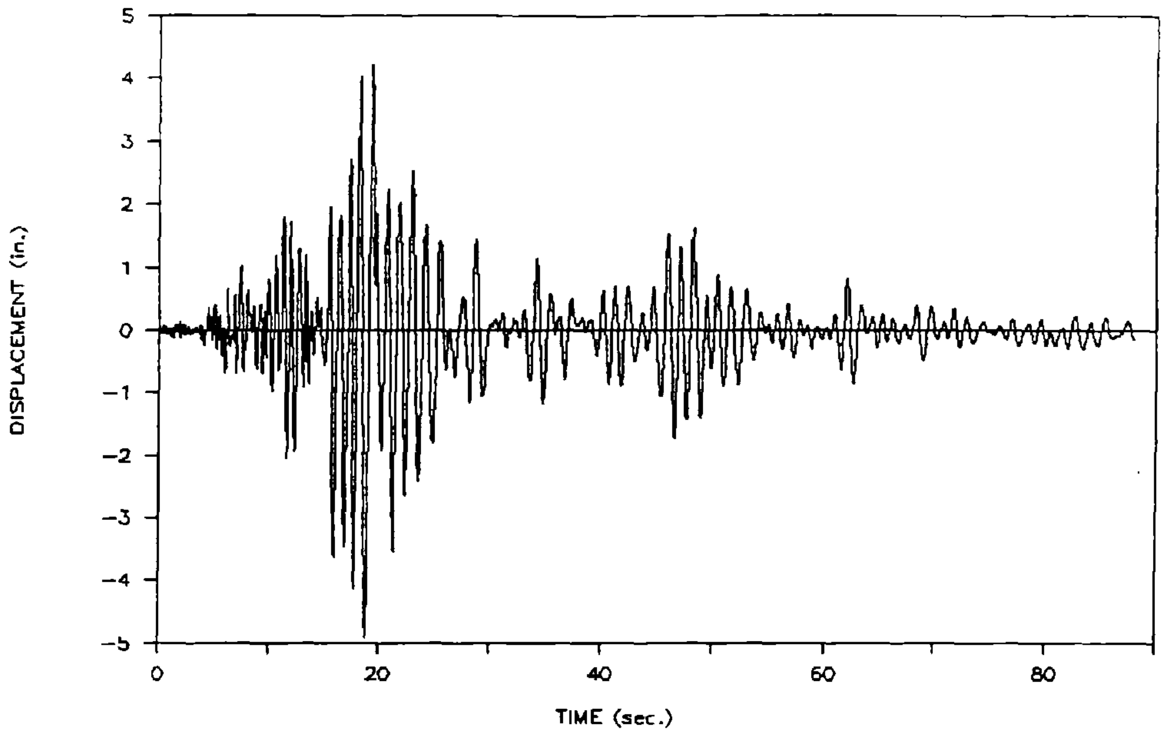
JAMES ROAD 140



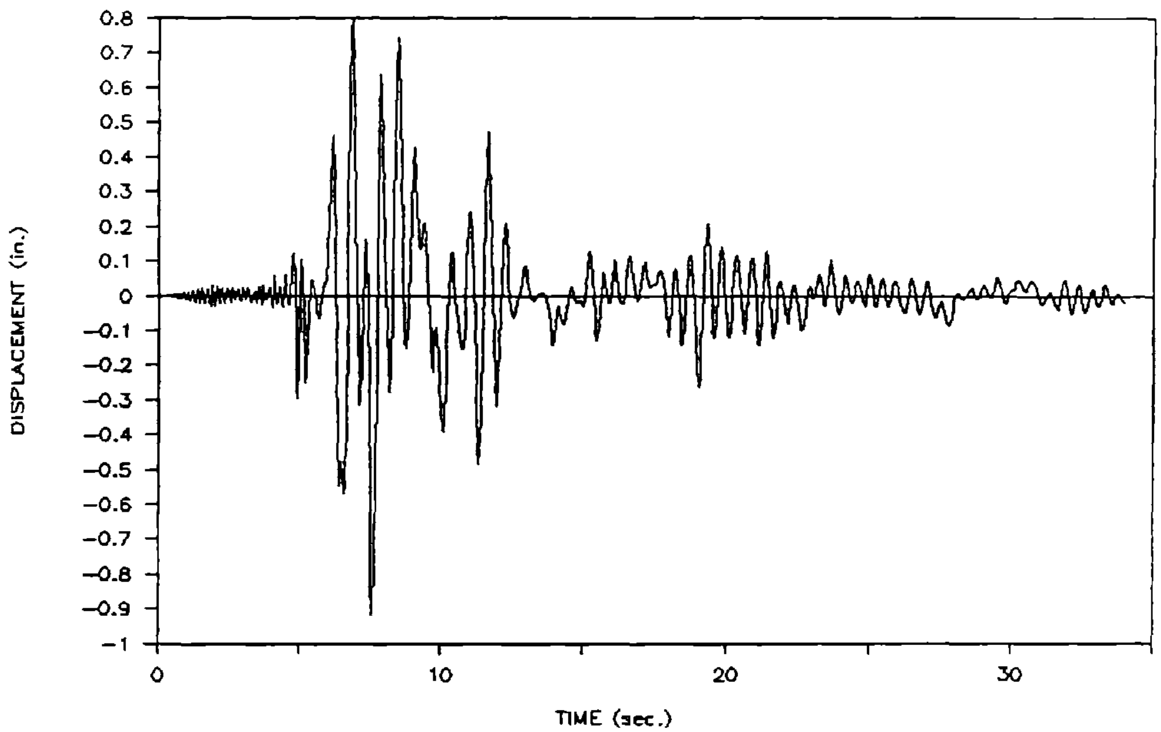
KERN CO. 69



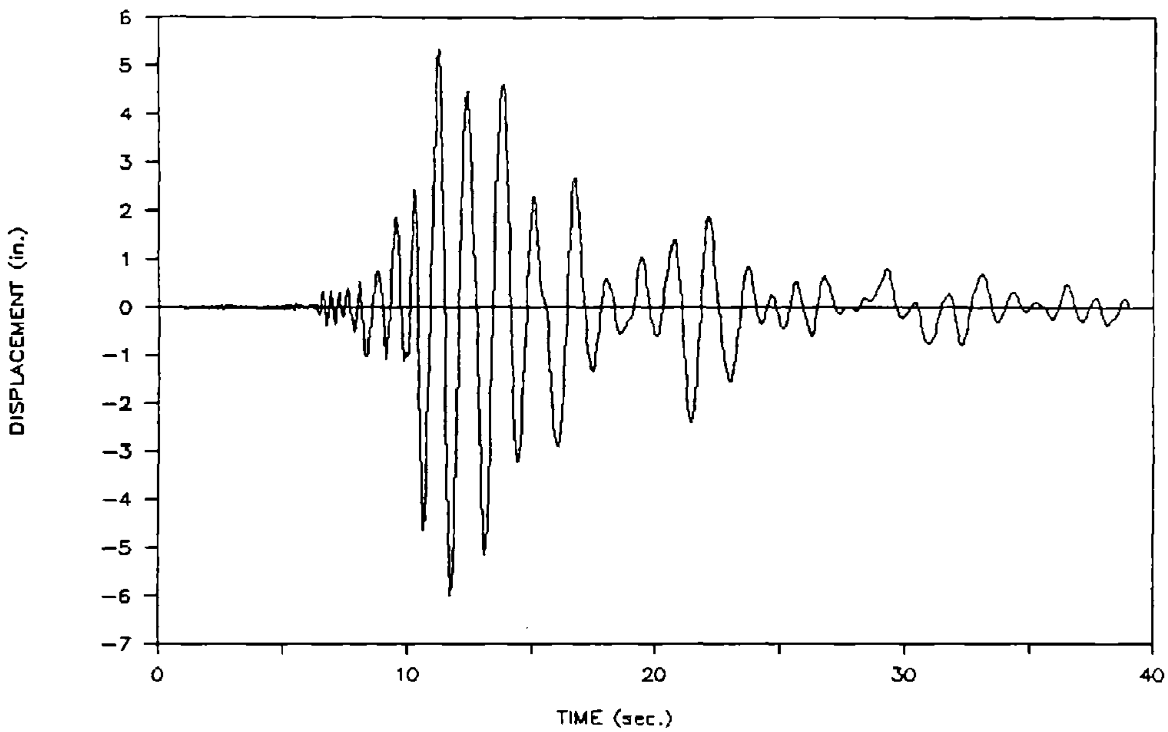
OLYMPIA 86



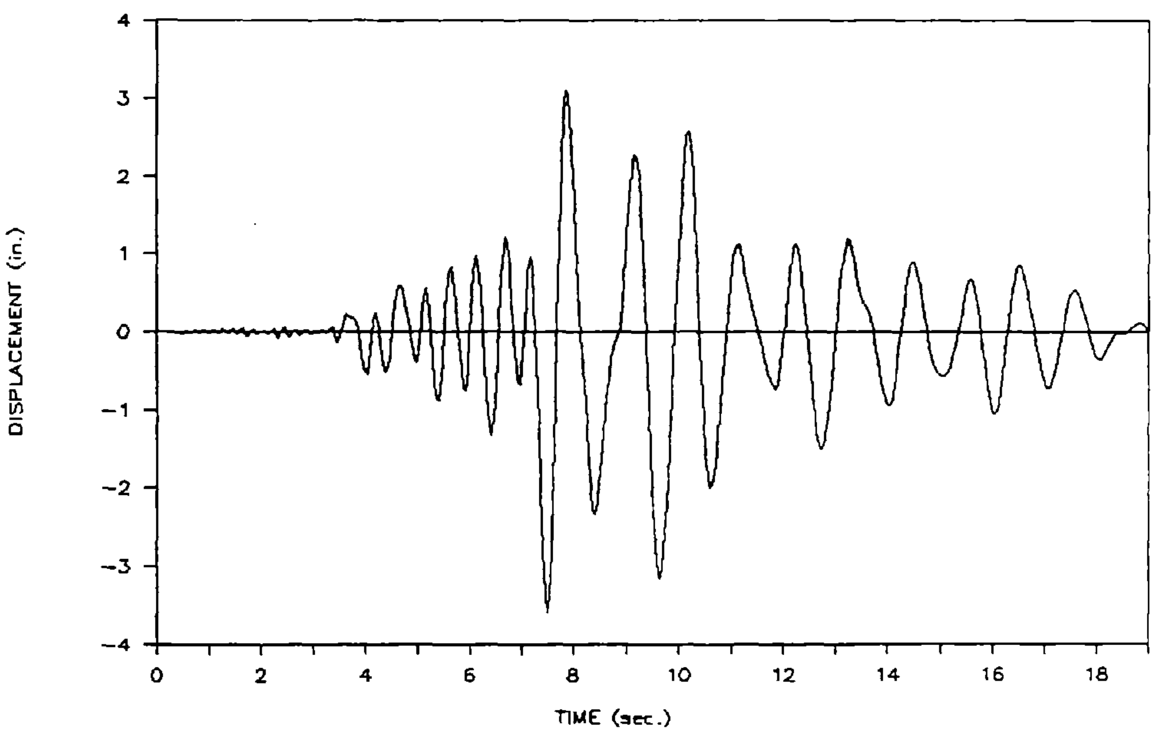
IMP VAL. COLL. 140

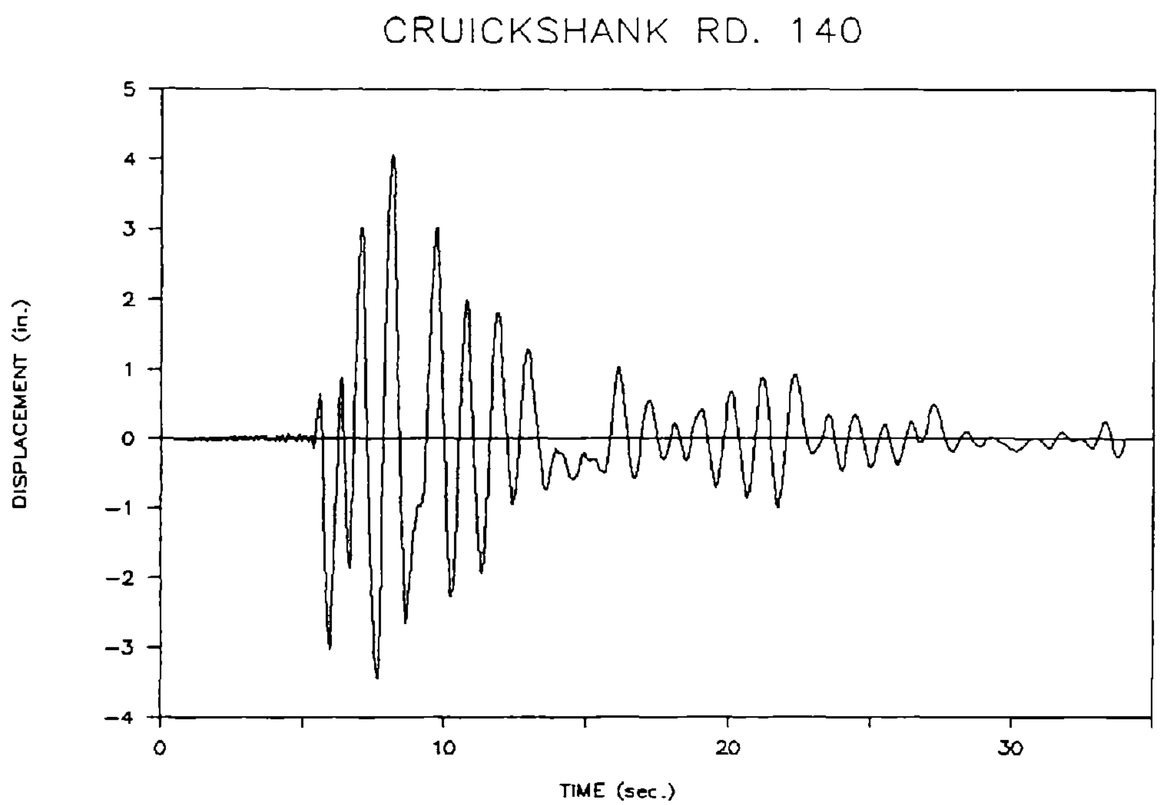


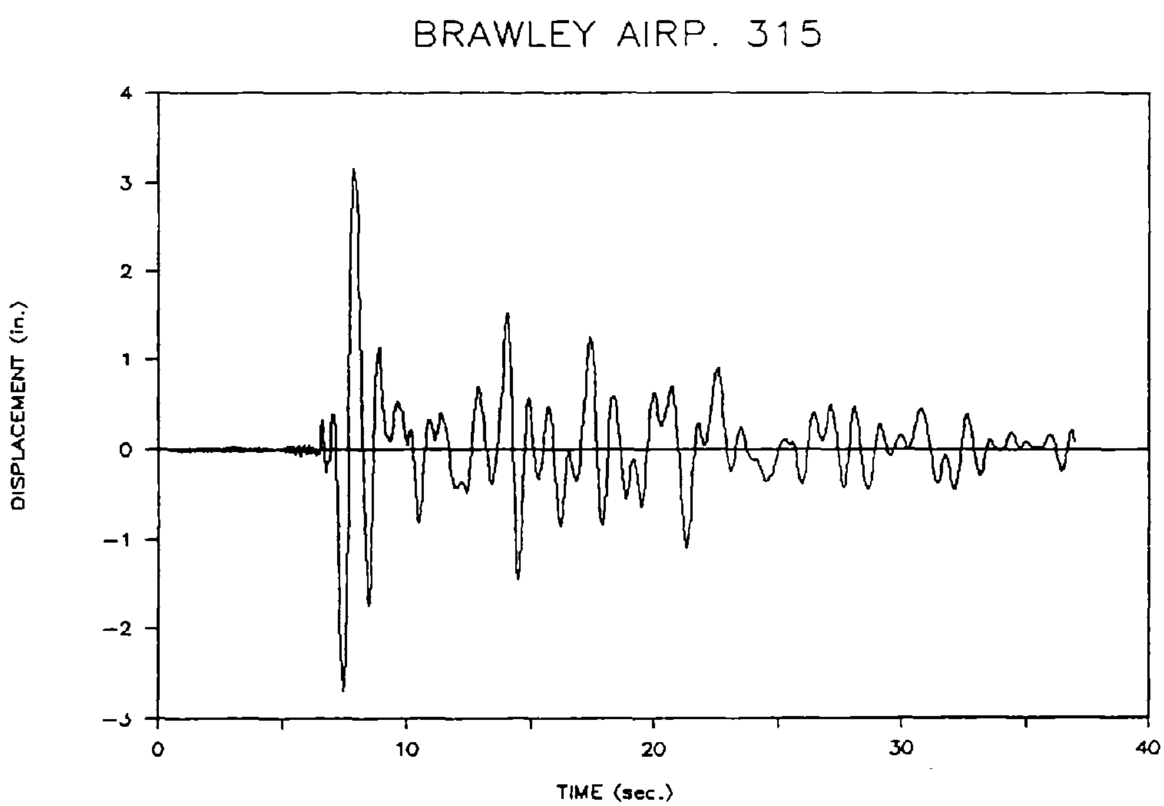
KEYSTONE RD. 140

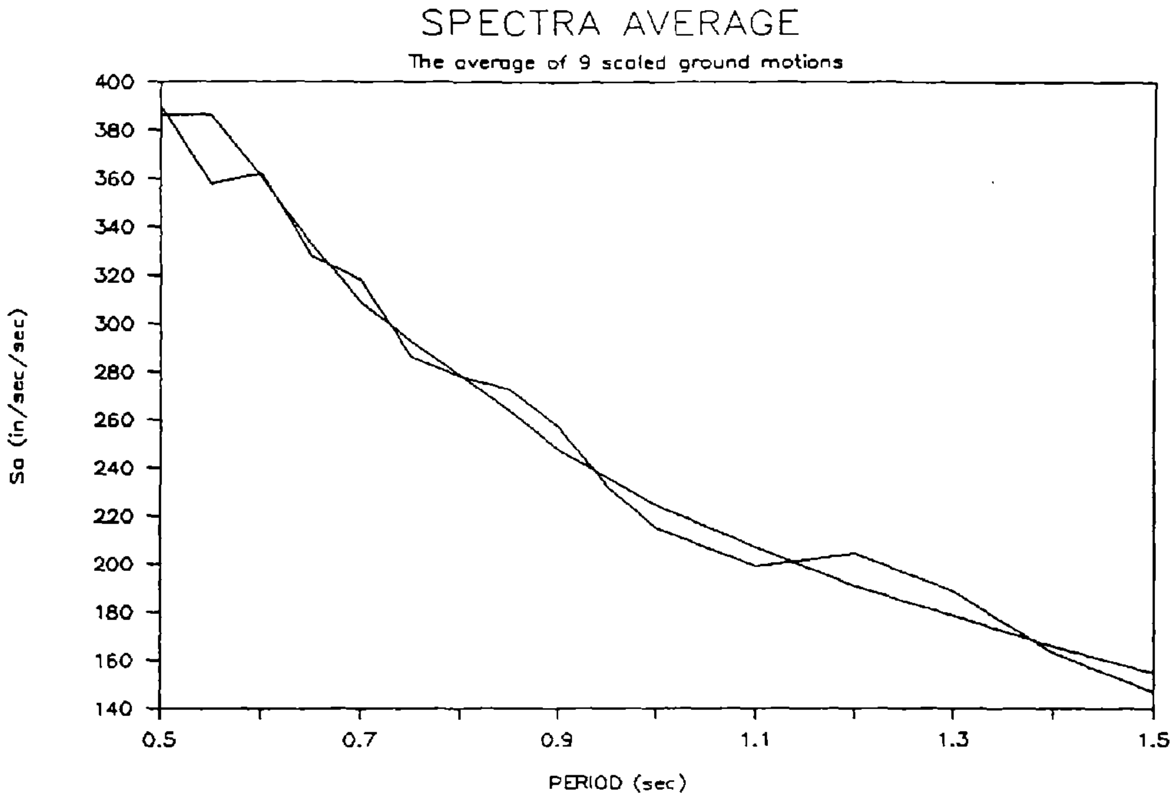


COALINGA PLES. VAL. 45




CRUICKSHANK RD. 140
DISPLACEMENT (in.)
TIME (sec.)
5
4
3
2
1
0
-1
-2
-3
-4
0
10
20
30


BRAWLEY AIRP. 315
DISPLACEMENT (in.)
TIME (sec.)
4
3
2
1
0
-1
-2
-3
0
10
20
30
40


SPECTRA AVERAGE
The average of 9 scaled ground motions
Sa (in/sec/sec)
400
380
360
340
320
300
280
260
240
220
200
180
160
140
0.5
0.7
0.9
1.1
1.3
1.5
PERIOD (sec)

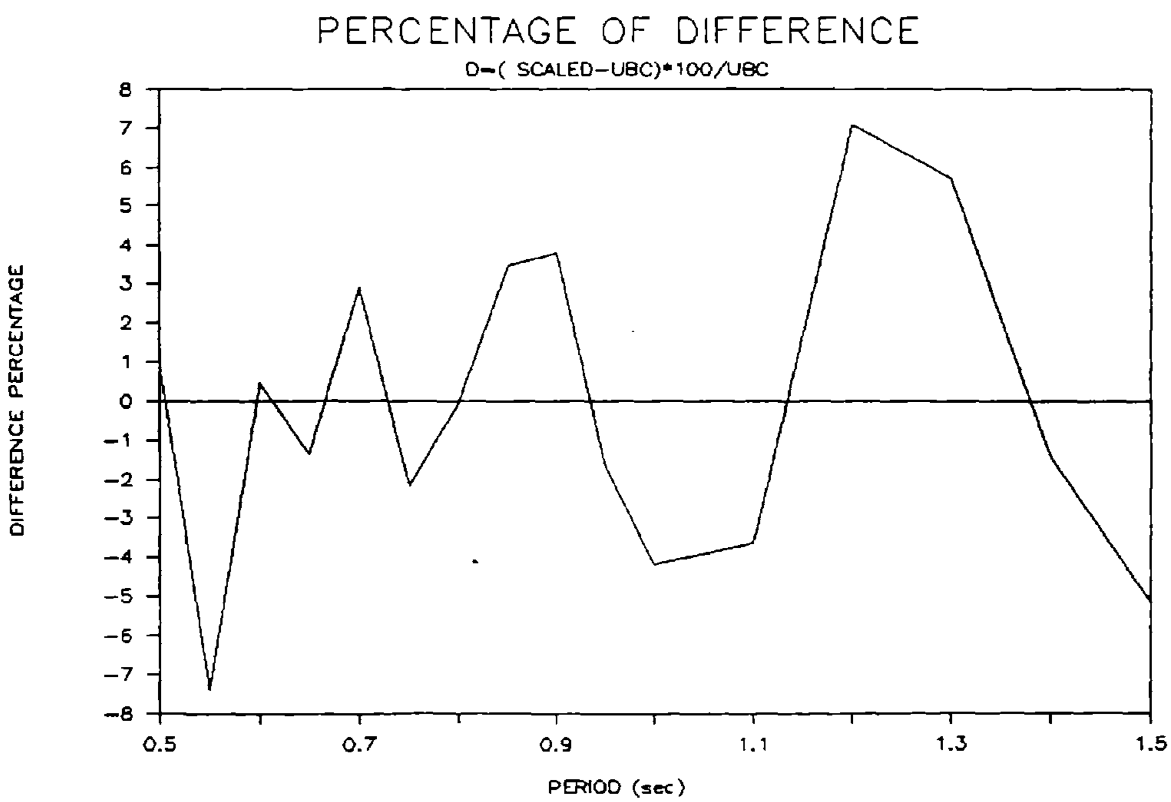

PERCENTAGE OF DIFFERENCE
D=(SCALED-UBC)*100/UBC
DIFFERENCE PERCENTAGE
8
7
6
5
4
3
2
1
0
-1
-2
-3
-4
-5
-6
-7
-8
0.5
0.7
0.9
1.1
1.3
1.5
PERIOD (sec)

FIGURE 5

APPENDIX A-3.1

DESIGN CALCULATIONS FOR THE DPC GYMNASIUM

Design of diaphragm:

Design Load = $0.35\ S_a(1.0) \times W_D$

Roof Load = 15 psf + (0.2 x 40 Ar/snow) = 23 psf

$S_a(1.0) = 0.58\,g$

W = Roof Weight + Walls weight

$= (128' \times 64' \times 0.023) + (2 \times 128' \times \frac{24'}{2}) \times 0.077$

$= 188.4 + 236.5 = 425.0^{k}$

Design Load = $0.35 \times 0.58 \times 425 = 86.3^{k}$.

Diaphragm is supported by two walls, so

Maximum Shear = $\frac{86.3}{2} = 43.2^{k}$.

Shear/L.F. = $\frac{43.2 \times 1000}{64}$ = 675 lb/L.F.

Use A steel deck diaphragm Type N, 18 gauge,

3" deep, 2 welds per sheet to support, seam welds @ 12" o.c

Shear Capacity = $0.6 \times 2 \times 760 = 912$ lb/L.F. > 675

Roof loading = 15 + (0.2 × 40) = 23 psf.

Total Roof Weight = 188.4^{k}.

Walls weight = $\left\{12' \times \left[2 \times 128 + 2 \times 64\right] + 2 \times \frac{32 \times 5}{2}\right] \times .077$

$= 367^{k}$.

Base Shear = V = C_s W

$$C_s = \frac{S_a(1.0)\ S}{T_{cs}^{n}\ R} \leq \frac{S_a(0.3)}{R}$$

$S = 1.0$, $n = 1.0$

$R = 4\frac{1}{2}$

$T_{cs} = 0.05 \frac{h_n}{\sqrt{L}}$

$A_a = 0.4g$

N-S Direction $T_{cs} = 0.05 \times \frac{30}{\sqrt{64}} = 0.188$ (sec)

$C_s = \frac{0.58 \times 1.0}{.188\,R} = \frac{3.1}{R}$

or $C_s = \frac{1.0}{4.5} = \underline{0.222}$ (Governs)

E-W Direction $T_{cs} = 0.05 \times \frac{24}{\sqrt{128}} = 0.106$

$C_s = \frac{5.47}{R}$

or $C_s = \underline{0.222}$

$V = 0.222 \times [188.4 + 367] = 123.3^{k}$

$V/\text{wall} = 61.7^{k}$

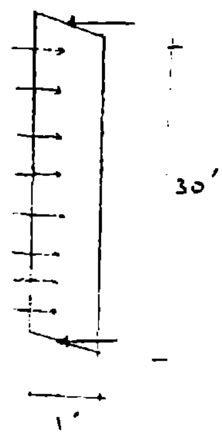
CLIENT NSF	KARIOTIS & ASSOCIATES STRUCTURAL ENGINEERS	SHT. 3 OF 5
		BY DW
SUBJECT: DPC Gymnasium		DATE 1/8/92
ITEM WALL REINFORCEMENT / Out of Plane		JOB NO. 88-507

East & West Walls: Out of plane: Consider a 1 ft wide section

Lateral Loading $= 0.7\ S_a(1.0)\ W_c$

$= 0.7 \times 0.58 \times [30 \times .077]$

$= 0.938^{k}$



Max. Moment @ Mid-height

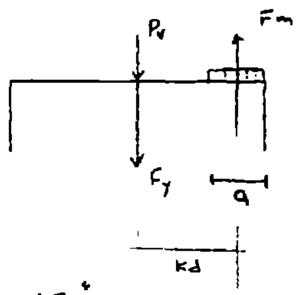
$= \frac{WL}{8} = \frac{938 \times 30}{8}$

$= 3517$ lb-ft.

Vertical Load @ Mid-height $= \left(1 \times \frac{30}{2} \times 77\right) + (1 \times 4 \times 15)$

$= 1155 + 60 = 1215$ lbs.

$[A_s f_y + P_v] = 0.85\ f'_{mc}\ a \times 12''$



Try #5 @ 16"

$A_s = 0.31 \times \frac{12}{16} = 0.233\ in^2.$

$a = \frac{[.233 \times 66 + 1.235]}{.85 \times 2.5 \times 12} = 0.65''$

kd $= 3.49''$

Expected Moment $= 0.9 \times 0.85 \times 2.5 \times 0.65 \times 12 \times 3.49$

$= 52.1$ k-in $= 4338$ lb-ft > 3517

OK.

185

The wall smallest height = 24'

Max. Moment = $\frac{750 \times 24}{8}$ = 2250 lb-ft.

Try #5 @ 32"

Expected Moment = 2375 > 2250 <u>OK</u>

Use #5 @ 16" For 20' of center of wall to be reduced to #5 @ 24" the rest of the wall.

<u>E-W Walls.</u>

Try #5 @ 66"

A_s = 0.058 in²/ft. P_v = (1 x 12 x .077) + (8 x 72 x .015) = 4.7

$a = \frac{[.058 \times 66 + 4.76]}{.85 \times 2.5 \times 12}$

= 0.3367"

kd = 3.644"

Expected Moment = 0.9[0.85 x 2.5 x 0.737 x 12 x 3.64]

= 28.2 k-in = 2349 lb-ft > 2250

OK.

Use #5 @ 66"

Try #5 @ 40" Horizontal Reinforcement.

Shear Capacity $v_u = \left(\left(\frac{0.76}{r_d + 0.7} + 0.012 \right) (4.04 \; \rho_{vc}) \sqrt{f'_m} \right) \frac{d}{L}$

$+ \left[0.01575 \sqrt{\rho_h \, f_{yh} \, f'_{mc}} \right) \frac{\delta d}{L}$

$+ \left(0.175 \; \sigma_o \right) \frac{d}{L}$

N-S wall :

$r_d = \frac{h}{d} = \frac{24 \times 12}{(64 \times 12 - 4)} = 0.377$

$\rho_{vc} = \frac{0.31}{7.62 \times 64 \times 12} = 0.0053\ \%$

$\frac{d}{L} = 0.995$

$\rho_h = \frac{0.31}{40 \times 7.62} = 0.1017\ \%$

$\sigma_o = \frac{80 \times 4.448}{12 \times 7.62 \times (25.4)^2} = 0.006 \ N/mm^2$.

$v_u = \left\{ \left(\frac{.76}{.377 + .7} + .012 \right) (4.04 \times .0053) \sqrt{17.24} + 0.01575 \sqrt{.1017 \times 455.1 \times 17.24} \right.$

$\left. + .175 \times .006 \right\} 0.995$

$= 0.507$

$V_u = v_u \, t \, L = 0.507 \times (7.62 \times 64 \times 12) \times (25.4)^2 = 1.916 \times 10^6$

$= 431^k > 61.7^k$ OK.

APPENDIX A-3.2

DESIGN CALCULATIONS FOR THE

TMS CENTER

Diaphragm Design: Consider Half the diaphragm.

Roof load = 15 psf + (0.2 * 30 psf)

= 21 psf

Weight of diaphragm = 86.5' * 102.3' x (.021)

= 185.9 k

Weight of South Wall = [4' x 82.3' x (.01)] (Glass)

+ [10' x 20' x (0.077)] (Masonry)

+ {(7' x 82.33') + (20' x 1')} x 0.015 (FASCIA)

= 27.6 k

Weight of North Wall = 102.3' x 10' x 0.077 = 78.8 k

Total Weight = 185.9 + 27.6 + 78.8 = 292.3 k.

Sa(1.0) = 0.58 g (ZONE 4)

Design Load = 0.35 Sa(1.0) X W

= 0.35 * 0.58 * 292.3 = 59.3 k.

Diaphragm is supported by two walls:

The maximum shear = $\frac{59.3}{2}$ = 29.7 k

Shear = $\frac{29.7}{81.5}$ = .364 k = 364 lb/L.F.

East-West Direction:

Walls Weight = 3[10' x 11.5' x .077] = 188 k

Diaphragm Weight = 86.5' x 204.7' x .021 = 372 k

Design Load = .35 x 560 x .58 = 113.6 k

Shear = $\frac{(113.6/2)}{204.7}$ = .278 k = 278 lb/L.F.

N-S Direction Controls.

Use 22 Gauge metal deck, 3 welds per sheet to support and 1 1/2" side lap welds @ 1/4 span.

Allowable Shear = 0.6 x 2 x 330 = 396 lb/L.F > 364
OK

WALL REINFORCEMENT

Out of plane: West Wall, consider a 1' wide section.

Lateral loading = 0.7 Sa(1.0) x Wc
= .7 x .58 x [1 x 16' x 77]
= 500 lbs.

Max. Moment = $\frac{WL}{8}$ = $\frac{500 \times 16}{8}$ = 1000 lb-ft @ Mid height

Vertical Load @ Mid-height = P_{wall} + $P_{DIAPH.}$
= [1 x 10' x 77] + [1' x 2.5' x 15]
= 807.5 lbs.

$A_s f_y + P_v = 0.85 f'_{mc} \times a \times 12''$

Try #5 @ 64"

$A_s = 0.31 \times \frac{12}{64} = 0.058$ in²/LF.

$a = \frac{.058 \times 66 + .808}{.85 \times 2.5 \times 12} = .182''$

$Kd = 3.72''$

Expected Moment = $0.9 \times 0.85 \times 2.5 \times 12 \times .182 \times 3.72$

= 15.54 k-in = 1295 lb-ft > 1000 lb-ft.

Use #5 @ 64" For East, West and Middle Walls.

North Wall $P_v = 770^{lb} + [1 \times 20.25' \times 15]$

= 1074 lbs. > 807 lb (West Wall)

#5 @ 64" in Satisfactory For North Wall.

INPLANE ANALYSIS

Total Roof Weight = 372 k

North Wall Wt. = 158 k

N-S Walls Wt [Grid 1, 2, and 3] = 188 k

South Wall Wt = [20' x 10' x .077]

+ [184.7' {(7' x .015) + (4' x .01)}]

= 42.4 k

Total Weight (Diaphragm and Upper half of walls)

$= 372 + 158 + 188 + 42$

$= 760^{k}$.

$$C_s = \frac{S_A(1.0)\ S}{T_{es}\ R} \leq \frac{1}{R}$$

$S_A(1.0) = 0.58\ g$

$S = 1.0 \qquad R = 4.5$

$T_{es} = T_a = 0.05 \dfrac{h_n}{\sqrt{L}}$

N-S direction : $T_{es} = 0.05 * \dfrac{16}{\sqrt{81.5}} = 0.09$ (sec.)

$C_s = \dfrac{.58 \times 1}{.09} = \dfrac{6.55}{R}$

or $C_s = \dfrac{1}{4.5} = 0.222$ (Governs.) .

E-W direction $T_{es} = .05 \dfrac{\times 16}{\sqrt{204.5}} = .056$ (sec)

So $C_s = .222$

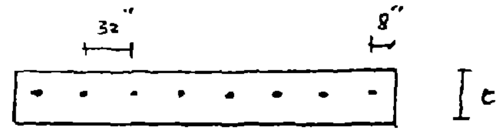
Base Shear $= C_s W = 0.222 * 760 = 169^{k}$

South Wall Shear $= \dfrac{169}{2} = 84.5^{k}$

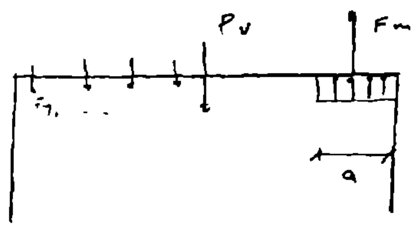
SOUTH WALL

Try #5 @ 32"

8 #5 distributed as shown.



Assume 7 bars are in tension.



$P_v = P_{DIA.} + P_{WALL}$

$= \frac{45.5'}{2} \times 20' \times 15$ psf

$+ 10' \times 20' \times 77 = 22,975^{lb} = 23^k$

$A_s f_y + P_v = 0.85 f'_{mc}\ t\ a$

$a = \frac{7 \times 0.31 \times 66 + 23}{.85 \times 25 \times 7.625} = 10.3"$ (7 bars Yielded).

Expected Moment Capacity $= 0.9 \times 0.31 \times 66 \times \{34.9 + 66.9 + 98.9 + 130.9 + 162.9 + 194.9 + 226.9\}$

$+ 0.9 \times 23 \times 114.9$

$= 19,246^{k-in} = 1603^{k-ft}$.

Required Moment $= 84.5 \times 16 = 1352^{k-ft} < 1603$ Ok

Balanced steel Ratio $= \rho_{bal} = 0.0425$ (See attached Sheet)

$A_{max} = 0.35 \times 0.0425 \times (20' \times 12 \times 7.625) = 27.2\ in^2$

As(provided) $= 8 \times .31 = 2.45\ in^2 < 27.2$ Ok

Shear Design

Expected Shear Capacity $= \frac{M}{H} = \frac{1603}{16} = 100^{k}$.

Using Matsamura formula for Shear Capacity:

$$\nu_u = \left\{ \left(\frac{.76}{r_d + 0.7} + .012 \right) \left(4.04\ \rho_{ve}^{1/3} \right) \sqrt{f'_m} \right\} \frac{d}{L}$$

$$+ \left[.01575 \sqrt{\rho_h f_{yh} f'_m} \right] * \frac{\delta d}{L}$$

$$+ \left(0.175\ \sigma_o \right) \frac{d}{L}$$

$$r_d = h/d = \frac{16 \times 12}{20 \times 12 - 4} = 0.814$$

$$\rho_{ve} = \frac{.31}{7.625 \times 240} = 0.00017$$

$$f'_m = 2.5 \text{ ksi} = 2.5 \times 6.895 = 17.24 \text{ MPa}.$$

$$\frac{d}{L} = \frac{236}{240} = 0.983$$

$$\rho_h = \frac{.31}{72 * 7.625} = \text{[illegible]}$$

$$\sigma_o = \frac{379 * 20 \times 4448}{(240 \times 7.625) * (25.4)^2} = \text{[illegible]}6 \text{ N/mm}^2$$

$$\nu_u = \left(\frac{.76}{.814 + .7} + .012 \right) \left(4.04 * (0.00017)^{1/3} \right) \sqrt{17.24} * 0.983$$

$$+ \left(0.01575 * \sqrt{\text{[illegible]}27 * 455.1 * 17.24} \right) \times 0.983$$

$$+ (.175 \times .0286) * 0.983 = 0.6\text{[illegible]}$$

Wind Loading : Pressure = 20 psf

Exposure B Use a factor of 0.62

Windward Wall " " " 0.8

$P = 20 \times 0.62 \times 0.8 = 9.92$ psf

Max Moment $= 1.3 \times \dfrac{9.92 \times (16)^2}{8} = 412.7$ lb-ft

$A_s f_y + 0.9 P_v = 0.85 f'_m \times 12 a$

$a = \dfrac{0.029 \times 60 + (0.9 \times 0.81)}{.85 \times 2.5 \times 12}$

$= 0.097$"

$kd = 3.76$"

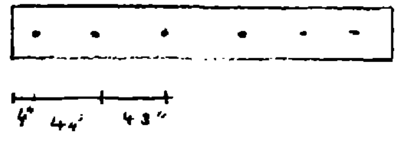
Expected Moment $= 0.8 \times [0.85 \times 2.5 \times 0.097 \times 3.764 \times 12]$

$= 7.45$ k-in $= 621$ lb-ft > 413 lb-ft

Inplane Analysis : South Wall.

Try #4 @ 48"

Assume 5 bars yield



$a = \dfrac{(5 \times 0.2 \times 66) + 23}{.85 \times 2.5 \times 7.625}$

$= 5.5$"

Expected Moment Capacity

$$= 0.9 * 0.2 * 66 [45.25 + 93.25 + 141.25 + 189.25 + 237.25] + 0.9 \times 23 \times 117.25$$

$$= 10,817^{k-in} = 903^{k-ft}$$

$$\text{Required Moment} = VH = \left(\frac{0.111\,W}{2}\right) \times 16 = \frac{.111 * 760}{2} \times 16$$

$$= 675 < 903^{k-ft}.$$

Shear Design: Expected Shear Capacity $= \frac{M}{H} = \frac{903}{16} = 56.4^{k}$

$0.85\, V_u > V_c$

$V_u = 167.3^{k} > \frac{56.4}{.85} = 66.4^{k}$ ok.

ZONE 2 Design:

South Wall: Use #4 @ 48" Both Ways.

The Remaining Walls. Use #4 @ 80" Vertical.

#4 @ 48' Horizontal.

$$V_u = v_u \cdot t \cdot L = 0.681 * (7.62 \times 240) * (25.4)^2$$

$$= 804 \text{ KN}$$

$$= 181 \text{ kips}$$

$$0.85 V_u = 0.85 * 181 = 154^k > 100.^k \quad \underline{OK.}$$

Also $1.54^k < 2.5 * 100 = 250^k$

Use $\underline{\#5 @ 32''}$ Both ways

<u>West Wall</u> Use #5 @ 40" Horizontal Reinforcement.

Shear:

$$v_u = \left(\frac{.76}{.197+.7} + .012\right)(4.04 \times .004)\left(\sqrt{17.24}\right)$$

$$+ (0.0158)\left(\sqrt{.102 * 455.1 \times 17.24}\right)$$

$$+ .175 * 0.00565 = 0.5$$

$$V_u = 0.5 * (81.5 \times 12 \times 7.625) \times (25.4)^2 = 2.4 \times 10^6 \text{ N}$$

$$= 540^k$$

$$0.85 V_u = 0.85 \times 540 = 459^k > 84.5^k$$

SOUTH WALL Reinforcement #5 @ 32" (Vertical)
#5 @ 40" (Horizontal)

All Other Walls Use #5 @ 64" (Vertical)
#5 @ 40" (Horizontal)

DIAPHRAGM DESIGN

$Sa(1.0) = 0.29\ g$

Design Load = $0.35 * .29 * 292.3 = 29.7^{k}$

Maximum Shear = $\dfrac{(29.7/2)}{81.5} = 0.182^{k} = 182\ lb/L.F.$

Use 22 gauge metal deck, No side lap fastening,

3 welds per sheet to support.

WALL REINFORCEMENT :

West Wall / Out of Plane ;

(Seismic) Lateral Loading = $0.7 * .29 \times [1 \times 16 \times 77\ psf)$

$= 250\ lb$

Max. Moment = $\dfrac{WL}{8} = \dfrac{250 \times 16}{8} = 500\ lb\text{-}ft.$

Try #4 @ 80"

$A_s f_y + P_t = 0.85\ f'_m\ a \times 12$

$a = \dfrac{0.029 \times 66 + 0.31}{.85 \times 2.5 \times 12} = 0.103"$

$kd = 3.76"$

Expected Moment = $0.9 \times .85 \times 2.5 \times .103 \times 3.76 \times 12$

$= 9.32^{k\text{-}in} = 776^{lb\text{-}ft} > 500$ OK.

APPENDIX A-3.3

DESIGN CALCULATIONS FOR THE

RCJ HOTEL

Floor Weight = Area * W (psf)

$= \{29'8'' + 9'4'' + 28'8''\} * \{4 \times 30'\} * 110$ psf

$= 893.2 \times 10^3$ lb $= 893.2^k$.

Roof Weight = $8120'^2 * 0.095$ ksf $= 771.4^k$.

WALL Weights:

1) WALL RUNNING EAST-WEST IN THE MIDDLE OF THE HOTEL (LONG WALL):

Wall Wt / Floor Ht $= \left[\{9'8'' * 120'\} - \{8 * 7' * 3'4''\}\right] * 0.077$

$= 75^k$.

2) WALLS RUNNING N-S (Cross WALLS).

Wall Wt / Floor Height $= \{28'8'' + 29'3''\} * 9'8'' * 0.077$

$= 43.4^k$.

Long Wall Net length (Exclude openings) $= 120 - (8 \times 3'4'')$

$= 93'4''$.

Floor No.	Slab Wt.	Long Wall Wt	Cross Walls Wt	Total Weight
Roof	771.4	40.2	108.5	920 k
4th	893.2	75	217	1,185.2 k
3rd	893.2	75	217	1,185.2 k
2nd	893.2	84.3	230	1,207.5 k
				4498 k

$V = C_s W$

$$C_s = \frac{S_a(1.0)\ S}{T_{cs}^{\,n}} \leq \frac{S_a(0.3)}{R}$$

$S = 1.0$

$S_a(1.0) = 0.58\,g$ (ZONE 4)

$n = 1$

$S_a(0.3) = 0.4\,g$

T_{cs} Period Evaluation (East - West Direction).

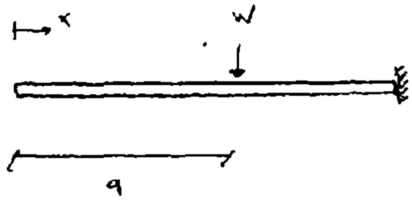
Use. $T = 2\pi \sqrt{\sum_{i=1}^{n} w_i \delta_i^2 \Big/ \left(g \sum f_i \delta_i\right)}$ UBC 34-5

1st Iteration: Try $f = 0.2\,W = 0.2 \times \left(\frac{4498}{4}\right) = 225^k$

(Consider only 1/4 the weight of building / there are 4 similar Panels).

Def: $Y(x) = -\frac{w}{6EI}\left(-a^3 + 3a^2 l - 3a^2 x\right)$

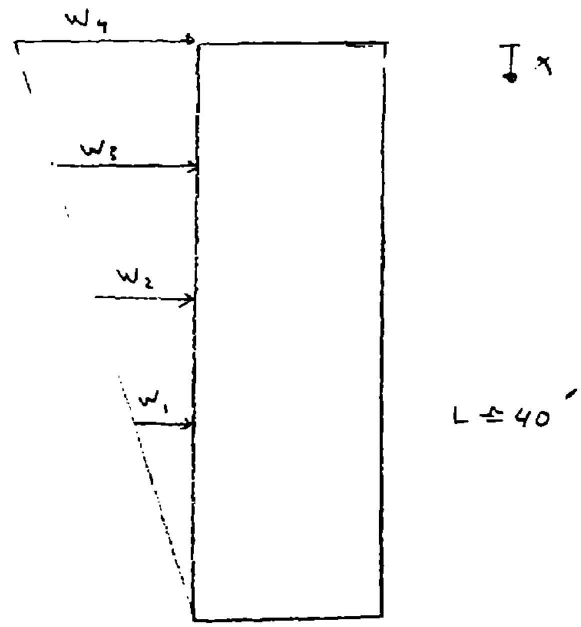
for $x < a$



$Y(x) = -\frac{w}{6EI}\left((x-b)^3 - 3a^2(x-b) + 2a^3\right)$ $\quad l > x > a$

where $b = L - x$

Location	w	a	x
Roof	88.4^k	0	0
			9.67
			19.33
			29.0
4th	67^k	9.67	0
			9.67
			19.33
			29.0
3rd	45.5^k	19.33	0
			9.67
			19.33
			29.0
2nd	24^k	29.0	0
			9.67
			19.33
			29.0



$H_1 = 10.83'$

$H_2 = 20.5'$

$H_3 = 30.2'$

$H_4 = 39.8'$

$EI = \left[\frac{(22 \times 12)^3}{12} \times 7.615\right] \times 1923$

Floor	δ	f	W	$W\delta^2$	$f\delta$
2	0.028	24	302	0.237	0.672
3	0.086	45.5	296	2.189	3.91
4	0.162	67	296	7.770	10.85
Roof	0.245	88.4	230	13.8	21.66
				24.0	37.1

$$T = 2\pi \sqrt{\frac{24}{386.4 \times 37.1}} = 0.257 \text{ (sec.)}$$

This period correspond to 100% EI For a degraded EI equivalent to 0.20 of initial EI, the deflection increases by a factor of 5.

So $T_{(EI=0.2)} = 0.257 \times \sqrt{5} = 0.575$ (sec.)

$$C_s = \frac{0.58}{0.575} \times \frac{1}{R} = \frac{1.01}{R} \leq \frac{1}{R}$$

So $C_s = \frac{1}{R} = \frac{1}{4.5} = 0.222$

$V = C_s W = 0.222 \times 4498^k = 1000^k$

Out of Plane Design of East Wall:

Check the wall section between Roof and 3rd floor, and between ground to 1st floor.

4th : $P_v = P_{DIAP.} + P_{WALL}$

For a 1 ft wide section:

$P_{DIAP.} = 1' \times 15' \times 0.095$

$= 1.425^k$

$P_{WALL} = 1 \times \frac{9.67}{2} \times .077$

$= 0.37^k$

Lateral Load $= 0.7\ S_A(1.0) \times W$

$= 0.7 \times 0.58 \times (9.67 \times 1 \times 0.077)$

$= 0.302^k /LF.$

$M_{max} = \frac{WL}{8} + P_{DIAP} \times \text{eccentricity} \times \text{Load Factor}$

$= \frac{.302 \times 9.67'}{8} + \left\{1.425 \times \left(\frac{3'}{12}\right)\right\} \times \left[1.1 + 0.3 \times 0.58\right]$

$= 0.365 + 0.454 = 0.819^{k\text{-}ft}$

Ground – 2nd Floor: $P_{DIAPH.} = 1.425 + \{3 \times 1 \times 15 \times .11\} = 6.375^k$

$M_{max} = \frac{WL^2}{8} + P \times \text{eccentricity}$

$= \frac{0.031 \times (0.33)^2}{8} + 1.65 \times \frac{3}{12} \times 1.274 = .455 + 0.526 = 0.981^{k\text{-}ft}$

CLIENT NSF

KARIOTIS & ASSOCIATES
STRUCTURAL ENGINEERS

BY OW

SUBJECT: RCT HOTEL
DATE 1/2/92

ITEM WALL REINFORCEMENT
JOB NO. 86-507

Roof - 3rd floor

$As\,f_y + P_v = 0.85\ f'_{me}\ 12\,a$

Assume No steel is used

$P_v = 0.85\ f'_{me} \times 12''\ a$

$P_v = 1.425 + 0.37 = 1.8^{k}$

$a = \dfrac{1.8}{.85 \times 2.5 \times 12} = 0.07$ in.

$kd = \dfrac{7.625}{2} - \dfrac{0.07}{2} = 3.777$ (in.)

Expected Moment $= 0.9 \times .85 \times 2.5 \times 12 \times .07 \times 3.777$

$= 6.07^{k\text{-}in} = 0.51^{k\text{-}ft} < 0.819^{k\text{-}ft}$

Steel Reinforcement is needed.

Use #4 @ 64 $As = 0.058$ in²/L.F.

$a = \dfrac{0.037 \times 66 + 1.8}{.85 \times 2.5 \times 12} = 0.166''$ $kd = 3.73''$

Expected Moment $= 0.9 \times .85 \times 2.5 \times 12 \times .166 \times 3.73$

$= 14.2 = 1.18$ k-ft > 0.819 k-ft.

For the floors below; The additional vertical load compensates for the steel, and no steel is needed for out of plane bending.

Consider a 22' panel of long wall: The floor shears are equivalent to:

Floor	W	H	$WH^2 \times 10^6$	V (kip)	V H
4	920^k	39.8'	1.457	458	18,228
3	1185	30.2'	1.081	340	10,268
2	1185	20.5'	.498	157	3,219
1	1208	10.83'	0.142	45	487
	4498^k		3.178×10^6	1000^k	32,202 k-ft.

Max. Moment at Base

$= \frac{32,202}{4}$ k-ft

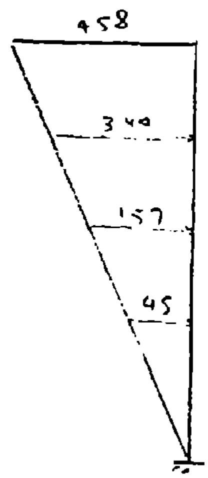
Max. Shear at Base

$= \frac{1000}{4}$ k.

Axial load $= 22' * \{2 \times \frac{40''}{12}\} * \{.095 + 3 \times .11\}$

$+ 22' * \{39.8 \times .077\}$

$= 129.8^k$.



M = 8050 k-ft

Shear/compare to Moment = $\frac{8050}{\frac{?}{3} + 37S}$ =

Shear = 250 k.

P_t = 130 k.

Clear span 29'-4"

Total floor load 110 #/sf.

6" normal wt. spancrete 64 #/□.

Topping 2" 25 #/□.

89 #/□.

Partition 20#

Allowance ceiling 1.0

110 #/□.

LL Rm 40 #/□

corridor 100 #/□.

f'_c Spancrete 4000 psi

f'_c topping 4000 psi

$Q = 1.2D + 1.6L$

Properties of Spancrete — 4.02" / 3.98"

$S_t = 377^{in^3}$ $I = 1515$ all for 40" wide unit

$S_b = 380^{in^3}$ $A = 216\ in^2$

c.g composite 2" topping $= \dfrac{216^{\square''} \times 3.98 + 80^{\square''} \times 9''}{296''}$

$= 5.33''$

$I_{comp} = \underbrace{\dfrac{2^3 \times 40}{12}}_{26.67} + \underbrace{80 \times (9 - 5.33)^2}_{1{,}077.5}$

$+ \underbrace{216\,(5.33 - 3.98)^2}_{397.3} + 1515^{in^4}$

$= 3{,}016.5^{in^4}$

$S_b = 566^{in^3}$

$$M = \Big[\underbrace{110\,\#/_{0'} \times \frac{29.33^2}{8} \times 1.2}_{14,194\ 1\#} + \underbrace{40\,\#/_{0'} \times \frac{29.33^2}{8} \times 1.6}_{6,882}\Big] \times \frac{40''}{12''}$$

$= 70,253\ ^{1\#} \quad 8.43 \times 10^5\ ^{\prime\prime\#}$

max tensile stress @ bottom $6\sqrt{f'_c} = 379$ psi

Use 270^k strand 7 wire

3/8" dia $u_{ef} = 21.57^k$

60% $u_{ef} = 12.94^k$

Use 3/4" cover on bottom $e = 3.98 - 0.8''$

$= 3.18''$

4 strand stress @ bottom after losses

$$f_{cb} = \underbrace{\frac{4 \times 12.94}{218^{\square''}}}_{0.24} + \underbrace{\frac{4 \times 12.94 \times 3.18''}{380^{''3}}}_{0.43} = 0.67\ ^{ksi}$$

$f_b = \dfrac{8.43 \times 10^2\ ^{\prime\prime}\ kips}{566} - 0.67 = 0.82^{ksi} > .379$ allow

6 strand $F = 0.6 \times 270 \times 0.0799 \times 6 = 77.66^k$

$f_{cb} = 1.01^{ksi}$

$f_{be} = 0.48 > .397$

Use 6-7/16" strand A_s total $= 0.1089 \times 6 = 0.65^{\square''}$

250^k $F = 97.5^k$ final force.

Plank @ corridor

$M = 1.255 \times 10^3\ ^{\prime\prime K}$

6 1/2 250^k strand $F = 129.4^k$ $f_{cb} = 1.68^{ksi}$

$f_b = 0.54^{ksi}$ tension $> .397$ allow.

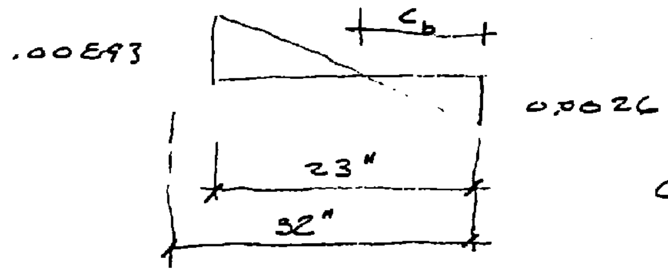
Use 8-1/2" strand - reduce eccentricity

Check coupling bm for balanced reinforcement ratio

Strand $E = 26 \times 10^6$ psi

yield = 250^{ksi} $\qquad f_{me} = 2500$ psi

$e_{ye} = 0.00893$ $\qquad e_{mue} = 0.0026$



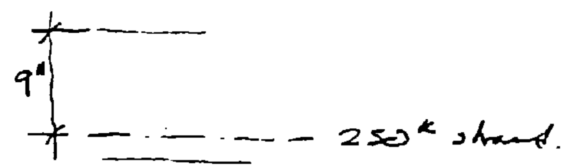
$C_b = 5.19''$

$A_s f_{ye} = 0.85 \times 5.19'' \times 2500 \times 7.62'' = 83.98^k$

$A_s = 0.34^{\square''}$ $\qquad 6\text{-}7/16 + 8\text{-}1/2 = 1.80^{\square''}$

maximum allowable reinforcement ratio is exceeded.
separate lintel below plank from adjacent
masonry shear wall.

$f_{ce} = 4000$ psi $\qquad b = 88''$ $\qquad d =$

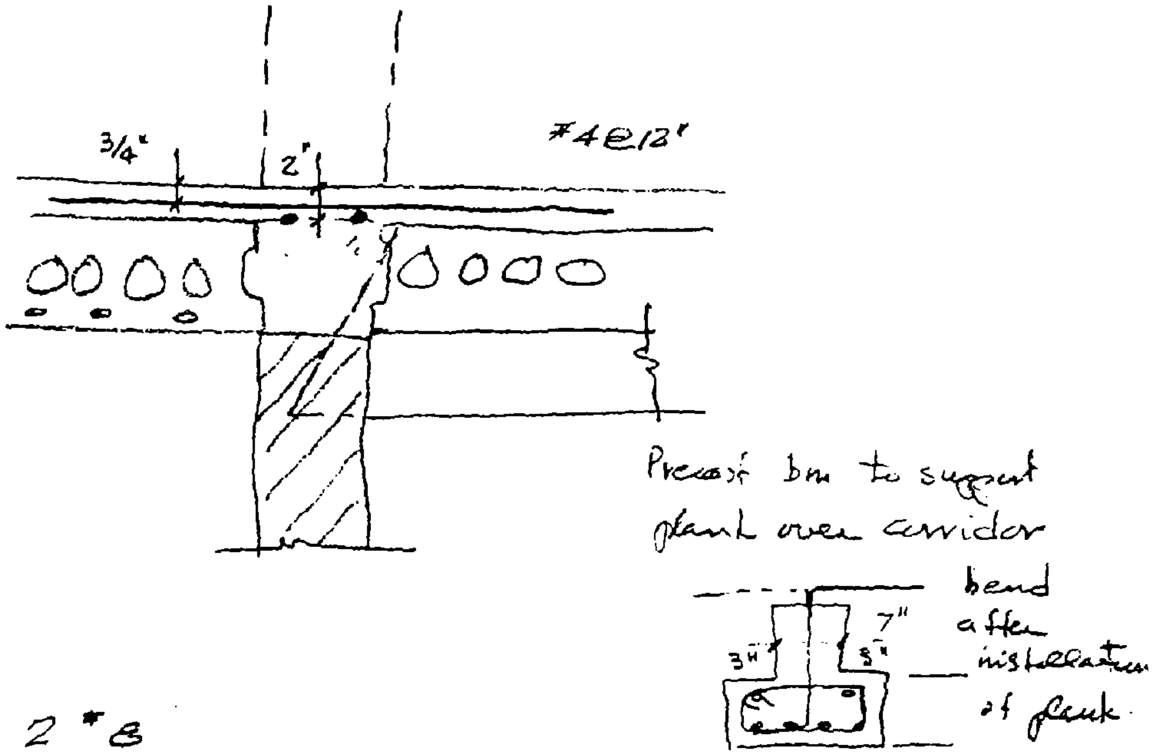


$$Ad = \frac{1.80^{\square''} \times 265^{ksi}}{0.85 \times 88'' \times 4.0}$$

$= 1.59$

$M_c = 1.80 \times 265 \times (9 - \frac{1.59}{2}) = 3{,}912^{''k}$

$326^{'k}$



2 #8

$$Ad = \frac{2 \times 0.79 \times 66^{ksi}}{0.85 \times 88'' \times 4.0} = 0.35''$$

$$M_e = 66 \times 2 \times 0.79 \times \left(8 - \frac{0.35}{2}\right) = 816''^{k}$$

$$68'^{k}$$

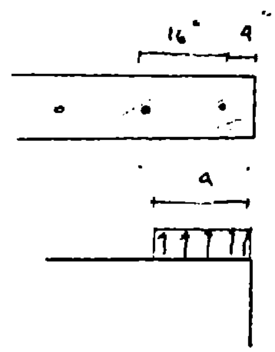
Planks provide restraining Moment equivalent to.

$4 \times 326 + 4 \times 66 = 1576^{k\text{-}ft}$

Required Moment $= 8050 - 1576 = 6474^{k\text{-}ft}$.

Try #7 @ 16"

17 bars.



Assume 14 bars are in tension.

$$a = \frac{14 \times .60 \times 66 + 130}{0.85 \times 2.5 \times 7.625}$$

$= 42.3"$

Balanced steel ratio $= 0.0391$

Maximum steel allowed $= 0.35 \times .0391 = 0.0137$

$As = \frac{0.6}{16 \times 7.625} = 0.0049 < 0.0137$

Expected Moment $= 0.9 \times 130 \times 110.8$

$+ 0.9 \times 0.60 \times 66 \times \{52" + 68" + 84" + 100 + 116$

$+ 132 + 148 + 164 + 180 + 196 + 212$

$+ 228 + 244 + 260 - (14 \times \frac{42.3}{2})\}$

$= 12,964 + 67,285 = 80,249$

$= 6687$ k-ft > 6474

OK.

Shear : Using Matsumura equation ;

$r_a = \frac{h}{d} = \frac{10}{22} = 0.45$ $\qquad t = \frac{7.625 \times 25.4}{1000} = 0.19$ m

$\rho_{vc} = \frac{0.6}{7.625 \times 264} = 0.0003$ $\qquad k_p = 1.16\ (\rho_{vc})^{0.7} = 0.404$

$\rho_h = 0.0072$

$\sigma_o = \frac{30\ kin^2 \times 4.448}{22 \times 12 \times 7.625 \times (25.4)^2} = 0.445\ N/mm^2$

$f'_m = 17.24$ MPa $\qquad f_{yh} = 455.1$ MPa

$\delta = 0.6$ Cantilever loading

$v_u = \{ [(\frac{0.76}{0.455 + 0.7} + 0.012) \sqrt{17.24} \times 0.404$

$+ [0.18 \times 0.6 \sqrt{.0072 \times 455.1 \times 17.24}]$

$+ [0.2 \times 0.445] \} \times 0.19 \times \frac{7}{8} \times \left(\frac{22 \times 12 \times 25.4}{1000} \right) \times 1000$

$= 253 + 905 + 99 = 2257\ (kN) = 507$ kips.

Required Shear Capacity $= \frac{6687}{(\frac{2}{3})\ 30.8 \times 0.9} = 280^k < 507^k$ OK

CLIENT NSF

KARIOTIS & ASSOCIATES
STRUCTURAL ENGINEERS

BY OW

SUBJECT: RCJ

DATE 3/24

ITEM Drift Calculations:

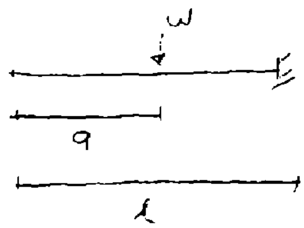
JOB NO. 22-507

Wall Drift Calculation for applied Shear.

Shear distribution:

Floor	Shear^k	a	Δ_{top}
Roof	114.5	0.0	0.077
4th	85	9.6'	0.0369
3rd	39.25	19.3'	0.0087
2nd	11.25	29'	0.0008
			0.1234

$$\Delta_{top} = -\frac{w}{6EI}\left(2l^3 - 3l^2a + a^3\right)$$



$l = 39.3$

$$EI = \ldots \times \left[\frac{(??\times 12)^3}{12} \times 7.675\right] \times .923$$

$= 31.22 \times 10^6$ k-ft^2

$\Delta_{top} = 0.1234$ ft $= 1.48$ in.

Drift $= C_d \times \Delta_{top} = 4.5 \times 1.48 = 6.66$ (in.)

$$\text{Drift Ratio} = \frac{6.66}{39.3 \times 12} = \frac{1}{72}$$