STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION
MATERIALS AND TESTING DIVISION
GEOTECHNICAL SECTION

GEOTECHNICAL REPORT

SEISMIC CHARACTERIZATION FOR RETROFIT DESIGN OF THE INTERSTATE 80 TRUCKEE RIVER BRIDGES NEAR VERDI BRIDGE NOS. G-772 E/W

March, 1993

E.A. 71686

WASHOE COUNTY, NEVADA

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INTRODUCTION

A pair of parallel Interstate 80 bridges across the Truckee River near Verdi are scheduled for seismic retrofit in the near future. Hence, the Bridge Department asked the Geotechnical Section of the Materials Division to determine site seismic characteristics for retrofit design (1). The ensuing study of the Bridge Nos. G-772 E/W site developed foundation seismic stiffness coefficients, determined a relationship between moment and axial shear load at foundation failure, and evaluated other factors that could influenced retrofit design.

Geotechnical personnel under my direction began the study in May, 1992, by running three seismic refraction lines to determine both shear (V_s) and compressional (V_p) wave velocities for the site soils. Subsequently in the office, I reviewed existing literature and bridge plans, evaluated field information, and developed and used an analytical method that included extensive use of the computer application LOTUS 123 (2). Upon completion of this analysis, Drs. Gary Norris and Raj Siddharthan, Geotechnical Engineers and Associate Professors in the Civil Engineering Department at the University of Nevada, reviewed, checked and commented on the method and final results that are presented in this report.

SITE DESCRIPTION

The two Interstate 80 highway bridges cross the Truckee River and the Southern Pacific Railroad tracks about two mile east of the town of Verdi in western Nevada near the California border. Here the Truckee River has cut a channel twenty to sixty feet into the existing hilly terrain on the east side of the Sierra Nevada Mountains. Consequently, the natural relief under the bridges is fairly severe with pier footings varying across elevation differences of one hundred feet.

GEOLOGY AND SEISMICITY

The region that includes the site, is on the western edge of the Great Basin in an area transitional between the Basin and Range and Sierra Nevada structural provinces. It has been subjected to mountain building for three to four hundred million years. The present episode began in the middle Tertiary, and is characterized by the mostly normal north-south trending block faulting that characterizes the Basin and Range Province. Although normal faults are a prominent part of the geological structure in this area, much of the relief is due to warping of mountain blocks. The Truckee Meadows to the east of the site is on one of these downwarped blocks (3).

This is a seismically active region. Since about 1840, ten earthquakes with Richter magnitude greater than 6.5 have shook the ground in this area. Six of these were probably magnitudes of 7 or greater (4). Indicating that M7 or greater occur on the average of about every 27 years. Although none of these large earthquakes were near the site, there have been a number of smaller events of magnitude 5 or less nearby. And, therefore, seismologists conclude that the seismic activity of the region is about average for the western Basin and Range Province (4).

Within the surrounding region, there are a number of recognized tectonic faults that have ruptured within Holocene (about the last 12,000 years) time. It would be generally accepted to assume that these faults are active and capable of producing a significant earthquake. Table I is a list of recognized nearby faults that are presumed to be capable of these large magnitude earthquakes.

Additionally, unidentified faults close to the site may produce lower magnitude earthquakes. These faults do not have a known surface expression or may not have ever ruptured the surface, but they still could generate an earthquake with high intensity shaking at the site.

Active Faults	
Faults	**Expected Mag.**
East Reno Basin Fault	6.9
Freds Mountain Fault	7.0
Long Valley Fault Zone	7.2
North Genoa Fault System	7.1
North Peavine Mountain Fault	7.2
Olinghouse Fault Zone	6.9
Spanish Springs Valley Fault	6.9
Dog Valley Fault Zone	6.9
Warm Springs Fault Zone	7.1

TABLE I

The intensity of bedrock earthquake shaking described as an acceleration coefficient was a fundamental parameter needed prior to the body of the analysis. There were essentially two ways that bedrock acceleration could have been developed. One was to do an individual evaluation of seismological and geological characteristics that might influence site ground shaking using predetermined criteria defining active faulting, historical data, attenuation relationships, and so on. The second was to use seismic acceleration maps developed by others who have evaluated these characteristics. For consistency with standard practice for highway bridges, we used a map.

We considered three possible sources of maps in the selection of the rock acceleration value -- AASHTO Specifications (5), NEHRP Recommended Provisions (6), and a preliminary University of Nevada, Reno (UNR) report (7). Because bridge performance criteria is generally in the purview of the bridge engineer, NDOT Bridge Department engineers selected the map and consequently the coefficient used in this study. They selected the AASHTO criteria. This gave a design level bedrock acceleration level of .38 to .40 g (8).

Besides ground shaking, liquefaction, slope failure, or fault rupture are the most likely hazards to structures at this site during an earthquake. The spread footing pier foundations of the bridges are on granular soils composed of sand, gravel, cobbles and boulders of the Tahoe and Donner Outwashes, and Truckee River Terraces (9). The outwash and terrace deposits would not generally be susceptible to liquefaction due to high relative density and gravelly character even though near river level they are saturated. I, therefore, do not believe that liquefaction during an earthquake will endanger the bridges. Bridge abutment footings are in approach fills. The high embankment on the west side is sixty to seventy feet. The east side embankment is about twenty feet. Slope failure of the west side embankment in particular is a possibility, and is discussed later in the report. There is no strong evidence of Holocene faulting at the site and bridge damaging fault rupture is not believed to be a credible event.

Examination of the very limited available as-built sub-surface information, geological maps (9), and the site indicate that the depth to rock or "rocklike" material (Hunter Creek Sandstone) is less than 200 feet. Because bedrock is close to the surface, I recommend using an AASHTO Soil Profile Type I Site Coefficient and Normalized Response Spectra (5).

WAVE VELOCITIES

Access

Two of the three seismic refraction lines were under the bridges east of the Truckee River near the Southern Pacific Railroad Tracks. The third was on the approach fill to the bridges on the west side. Locations are shown on the plan map in figure 1. There wasn't a clear road or vehicle path except along the severely restrictive railroad tracks themselves on the east side (10). Consequently, we had to laboriously hand carry equipment into the site and work under restricted conditions. Later, we easily approached the west side highway median location with vehicles, and work proceeded there without the east side problems.

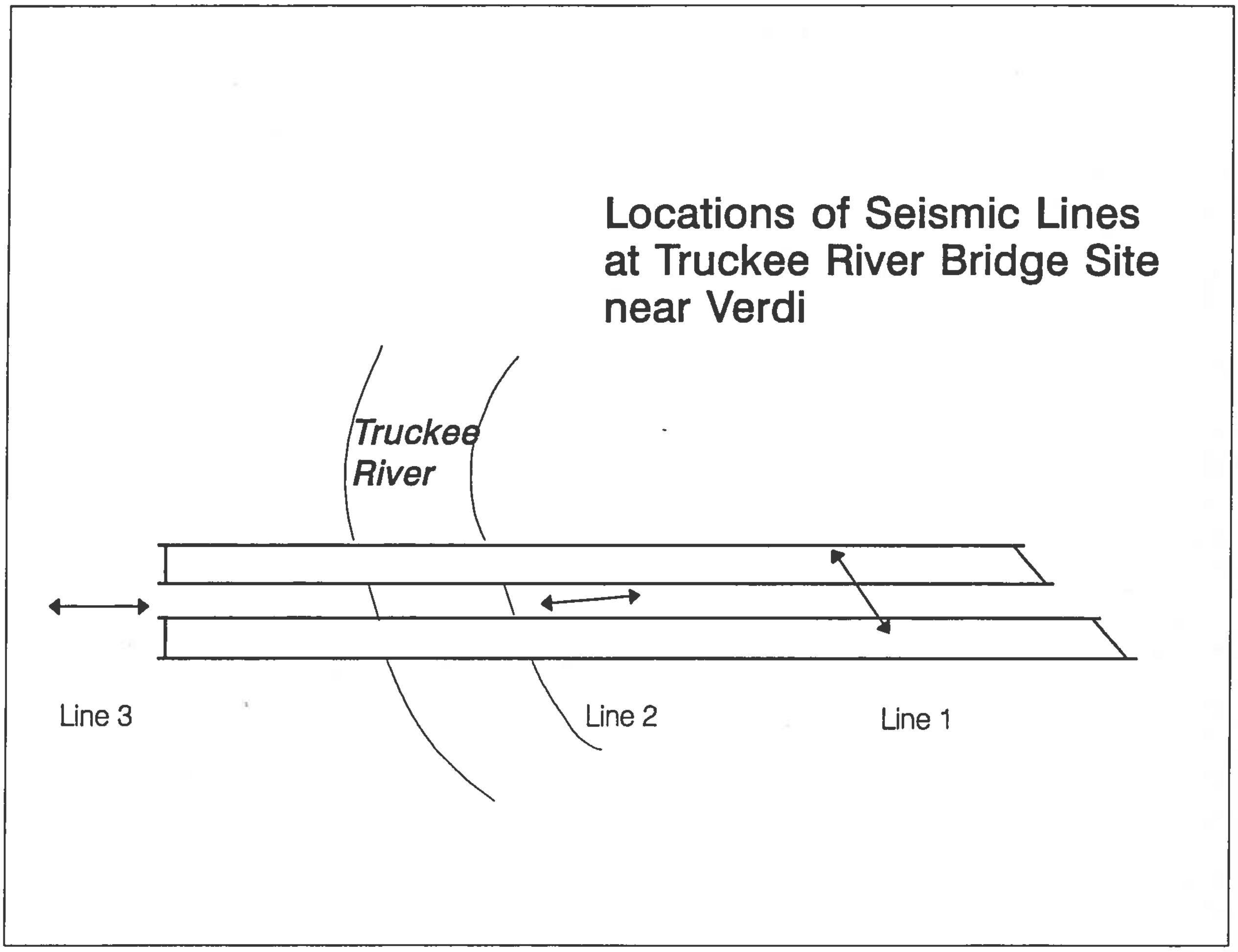

Figure 1

Equipment and Field Methods

Seismograms, on paper media and high density 3 1/2 inch diskettes, were obtained with a EG&G Geometrics Model 2401 4k memory seismograph set to operate at twelve channels. This unit had signal stacking ability (signal enhancement), and analog and digital filters. I used all these features to evaluate records.

Marks Products L-10 geophones transduced the seismic signals for the seismograph.

Twelve horizontal phones were used for V_s determination; twelve vertical phones were used for V_p. Phones were placed on ten foot centers in straight lines one hundred ten feet long.

We placed a seven foot long plank at the ends on the geophone lines perpendicular to line axis, and struck the ends horizontally with a large wooden mallet to generate shear wave energy. At the east site, the plank was weighted down for firm contact against the ground with large canvas sample bags filled with soil shoveled from the site. At the west site, the front wheels of a truck furnished weight. The plank was struck on both ends for different record sets, thus generating polarized, oppositely phased, shear wave rich records of good quality. Compressional energy was produced with vertical sledgehammer blows to a steel plate placed flat on the ground at the ends of the lines. This produced good quality first arrival records.

Office Analysis

Shear wave onset was identified by the phase reversal of arriving wave forms between one geophone record from striking one end of the plank compared to another record for the same geophone from striking the other end of the plank. I used the seismograph's digital filters on several occasions to remove high frequency signals for aid in identification of shear energy wave forms. Arrival picks, however, were made from unfiltered records to avoid the time and phase shifts of filtering. Compressional wave picks were at first arrival waveforms. I constructed time distance graphs using the arrival picks and calculated velocities from them. Time--distance graphs are in Appendix A.

Shear modulus and Poisson's ratio of the soil (G_{max} and μ) at small levels of strain and short times was calculated by using V_s and V_p in the following relationships:

$G_{max} = \rho V_s^2$ where ρ = *in situ* density, and

$\mu = \{(V_p/V_s)^2/2-1\}/\{(V_p/V_s)^2-1\}$.

SEISMIC FOUNDATION STIFFNESS COEFFICIENTS

The FHWA report on seismic design of foundations (11) describes formulas for deriving foundation stiffness coefficients for six modes -- three translational and three rotational for bridge piers and abutments. In all six coefficient calculations, shear modulus is a linear factor, and as such can be easily removed from the calculations and treated as an individual parameter. Therefrom we can developed a table of values for abutment and pier footings that would give the corresponding stiffness coefficients when multiplied by the modulus. Determining the value of shear modulus to use, however, is a different matter.

The shear modulus can be determined accurately from the shear wave velocity for very low soil strain levels. As the soil strain increases, however, the modulus decreases. How it can change with strain has been determined for various soils and is described in the literature (12). And, if the initial modulus is determined from shear waves, then a reasonably good value can be derived for other levels of strain. The initial site shear modulus at footing depths can be determined from near surface shear wave velocities. This gives the maximum footing stiffness coefficient possible for the site without earthquake effects (K_{max}). Then, the maximum modulus for foundation stiffness can be derived by reducing the initial modulus by a factor equivalent to the soil strain anticipated at design level free field ground motions (K_{eq}). Greater soil strain levels caused by soil-structure interaction during an earthquake will further reduce the modulus along established relationships until the soil begins to fail.

I acknowledge, that a clear relationship between seismically induced forces on the footings and strain levels in the foundation soils is not apparent, and that this creates a problem for bridge engineers trying to use a relationship between shear modulus or stiffness coefficients and foundation soil strains. For this study, therefore, I developed a correlation between seismically induced footing forces and moments with stiffness coefficients.

The correlation is founded on the following assumptions:

1. If the soils fail, seismically induced forces on the footings become constant regardless of foundation displacement.

2. The maximum possible seismically induced lateral forces ($P_{f(ub)x,y}$) and torsional moments ($M_{f(ub)z}$) that can cause failure occur when the root mean square (RMS) of the dynamic lateral forces or torsional moments are equivalent to static forces or torsional moments at ultimate foundation bearing, with the footing static axial loads equal to the dead load (L_D) with no eccentricity.

3. The maximum possible seismically induced axial forces ($P_{f(ub)z}$) that can cause failure occur when the RMS of the dynamic forces are equivalent to static axial forces at ultimate foundation bearing with no eccentricity.

4. The maximum possible seismically induced rocking moments ($M_{f(ub)x,y}$) that can cause failure, occur when the RMS of the dynamic moments are equivalent to static moments at foundation soil failure, and the footing axial loads are at one-half the bearing capacity with no eccentricity. This is not strictly correct. But, I believe it is a reasonable approximation.

5. The minimum possible seismically induced moments ($M_{f(lb)x,y,z}$) and forces ($P_{f(lb)x,y,z}$) that can cause failure occur when peak seismic acceleration of soil particles exceed the internal residual frictional resistances at static foundation dead load conditions (soil fluidation)(x). Except were these values exceed the static values described above in numbers 2 and 3, then the static values are the only failure modes.

6. Poission's ratio becomes equal to .5 when soils have failed.

7. Correlations and mathematical relationships that compare G/G_{max} to strain (12), Standard Penetration Test to soil strength (13), and others that are in generally accepted geotechnical literature are correct.

8. Significant portions of the G/G_{max}-strain curve can be reasonably approximated by straight line segments.

Once the soils begin to fail under the footings due to earthquake loads, the stiffness coefficients will vary linearly with the inverse of the displacement. The soil shear stresses (τ_f) associated with failure can be calculated, however, and by converting the G/G_{max}-strain curve to G-strain curve and inverting for τ, the maximum shear modulus (G_f) at failure can be determined. I have done this for both the static foundation and soil fluidation conditions, and converted G_f into stiffness coefficients. The larger of these values is at the upper-bound ($K_{f(ub)}$); the lower is at the lower-bound ($K_{f(lb)}$).

The area bounded by K_{eq}, $K_{f(lb)}$, $K_{f(ub)}$, P and $M_{f(lb)}$, P and $M_{f(ub)}$, and the coefficient-force curve equivalent to the G/G_{max} approximation straight line segment describes a range of uncertainty for stiffness coefficient values as shown in Figure 2. The $P_{eq(lb)}$,$M_{eq(lb)}$, and $P_{eq(up)}$,$M_{eq(up)}$ intercepts on the K_{eq} line shown in the figure are easily calculated from the relationship:

$$P_{eq},M_{eq} = [P_f,M_f][(K_{eq}-K_f)/(K_{max}-K_f)]$$

The application of these computations to the more than fifty bridge footings on the site was facilitated by using the spreadsheet program LOTUS 123 (2). Tables III through IX show the values each of the parameters identified in Figure 2 for each mode at each bridge support.

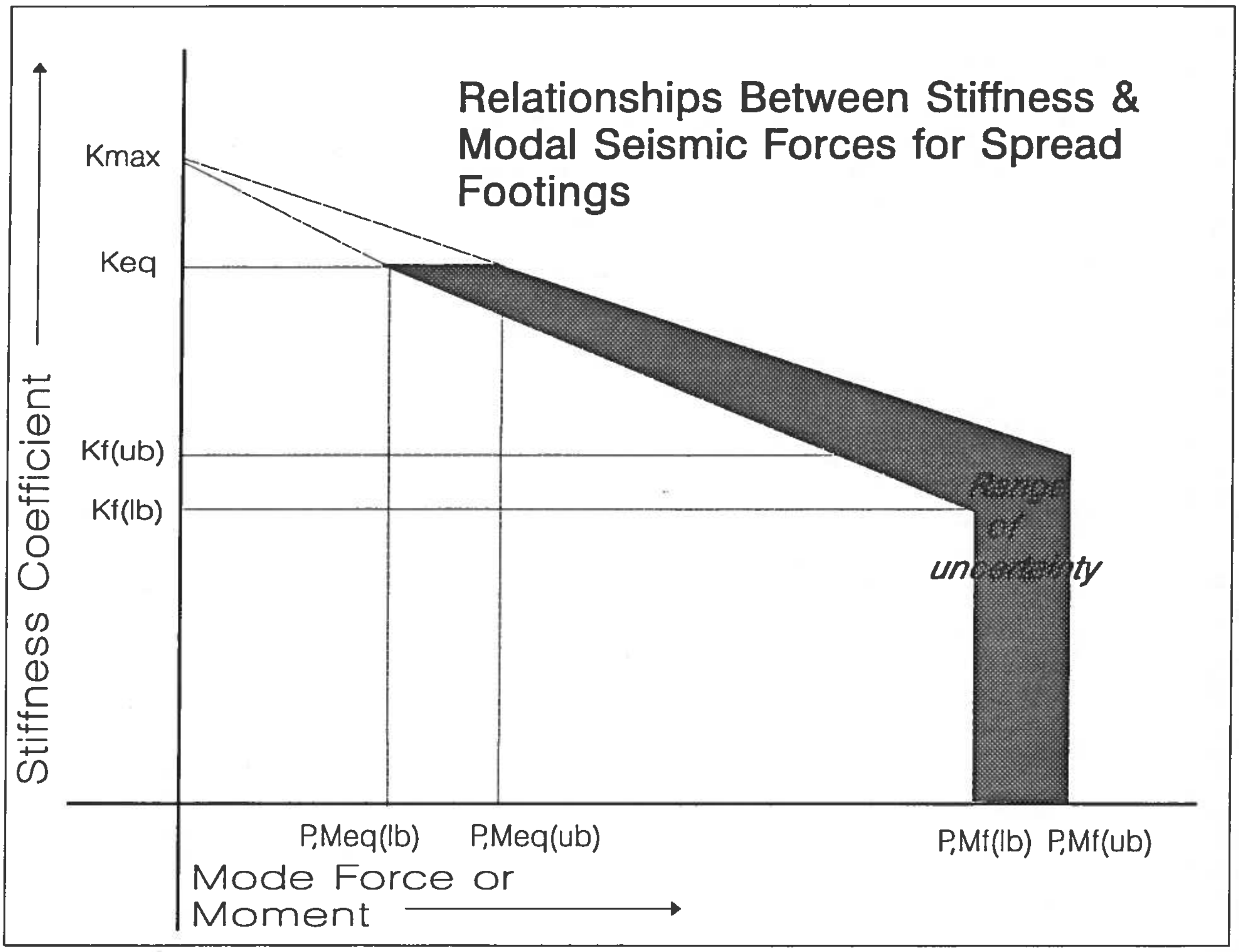

Figure 2

DETERMINATION OF OTHER FACTORS

The Bent 2 columns of both bridges are in part buried by the embankment supporting the west side abutment. As part of this study, I calculated the downslope pressures against the columns and estimated possible downslope movement of the fill that might occur during a design level earthquake. The Mononobe-Okabe (5) equations were used to determine the downslope pressures. The program XSTABL (14) was used to determine slope instability and identify possible failure surfaces. And, the method described by Makdisi and Seed (15) was used to estimate seismically induced deformations.

In a memorandum dated July 29, 1992 (16), the Bridge Department requested curves of ultimate bearing axial capacity vs. ultimate moment capacity for the footings supporting the two bridges. The relationships that had been used previously to develop these curves(x) were an oversimplification of footing behavior at bearing capacity failure. I used generally accepted relationships for determining bearing capacity in the geotechnical field (17) and based my curves on the following assumptions:

1. There was not a lateral load or incline load, only and eccentric load.
2. Static analysis was valid.
3. Residual conditions of angle of internal friction (ϕ) controlled, (i.e. $\phi = 32°$) (18).

The resulting relationship was computed iteratively with LOTUS 123 (2) using the following equations:

CASE 1 $M_x=0$

$$P_x=\{[B-e_x][N_\gamma][1-.4(B-e_x)]/L+zN_q[1+\tan\ \phi][B-e_x]/L\}\{4L(B-e_x)\gamma\}$$

CASE 2 $M_y=0$

A. Where $(L-e_y)>B$

$$P_y=\{BN_\gamma[1-.4B/(L-e_y)]+[zN_q(1+\tan\ \phi)B]/[L-e_y]\}\{4B(L-e_y)\gamma\}$$

B. Where $(L-e_y)<B$

$$P_y=\{[L-e_y][N_\gamma][1-.4(L-e_y)]/B+zN_q[1+\tan\ \phi][L-e_y]/B\}\{4B(L-e_y)\gamma\}$$

and $My = P_x e_x$, $Mx = P_y e_y$

Definitions:

Depth of Embedment	z
Soil Unit Weight	γ
Angle of internal friction	ϕ
Bearing Capacity factors (from AASHTO)	N_q , N_γ
1/2 Footing Width	B
1/2 Footing Length	L
Eccentricity along x-axis (parallel to B)	e_x

Eccentricity along y-axis (parallel to L) e_y
Rocking Moment Around x-axis M_x
Rocking Moment Around y-axis M_y

CONCLUSIONS

Site Characteristics

Table II shows the site characteristics used in the determination of seismic design parameters.

Table II

EARTHQUAKE

MAXIMUM BEDROCK ACCELERATION "a" = 0.4 g
SOIL PROFILE TYPE = I
WITH SITE COEFFICIENT "S" = 1.0

SOILS

NATIVE: for the bents
V_s = 1500 fps V_p = 2500 fps
Poisson's ratio μ = 0.22 ϕ = 44°

δ = 31° γ = 0.135 k/ft.^{3}

FILLS: for the abutments
V_s = 1100 fps V_p = 2200 fps
Poisson's ratio μ = 0.33 ϕ = 35°
δ = 29° γ = 0.130 k/ft.^{3}

Stiffness Coefficients

Table III shows the stiffness coefficients for the initial stress level associated with the design level earthquake (K_{eq}) for the six modes of stress for the various supports of the bridges. Table IV shows the stiffness coefficients for the lower bound failure ($K_{f(lb)}$) of six modes for the supports. Table V shows the stiffness coefficients for the upper bound failure ($K_{f(ub)}$) of the six modes at the supports. Table VI shows the lower bound translational forces, and rocking and torsional moments where dynamic load stresses begin to exceed free field earthquake stresses ($\{P,M\}_{eq(lb)}$) for the various bridge supports. Table VII shows the upper bound forces and moments where load stresses exceed earthquake stresses ($\{P,M\}_{eq(ub)}$) for the supports. Table VIII shows the lower bound dynamic foundation failure forces and moments ($\{P,M\}_{f(lb)}$) for the supports. Table IX shows the upper bound dynamic foundation failure forces and moments ($\{P,M_{f(ub)}$) for the supports.

TABLE III

Keq DO TO EARTHQUAKE STRESSES IN THE SOIL

	x−trans (k/ft.) (*10E6)	y−trans (k/ft.) (*10E6)	z−trans (k/ft.) (*10E6)	x−rock (k−ft.) (*10E6)	y−rock (k−ft.) (*10E6)	z−tors (k−ft.) (*10E6)
Abutments						
1	0.303732	0.273772	0.308615	93.25470	4.758111	97.33230
12	0.323264	0.291377	0.330098	136.8988	5.407678	133.9224
Bents						
2	1.228406	1.214094	0.861549	571.0049	422.5854	525.8953
3	0.807322	0.797916	0.658331	125.1829	92.64457	204.6459
4	0.940665	0.929706	0.713539	209.3361	154.9240	272.2183
5	0.771851	0.752554	0.631326	117.9912	68.46015	179.1399
6	0.709661	0.691920	0.558973	88.11119	51.60338	131.4199
7	0.709661	0.691920	0.558973	88.11119	51.60338	131.4199
8 out	0.507814	0.490839	0.392963	32.21781	16.60610	44.07902
8 in	0.507814	0.490839	0.394099	32.21781	16.60610	44.07902
9 out	0.557907	0.539257	0.417839	44.75165	23.06645	53.80578
9 in	0.557907	0.539257	0.417839	44.75165	23.06645	53.80578
10 out	0.550216	0.531315	0.431735	42.29533	20.89837	59.90187
10 in	0.550216	0.531315	0.431735	42.29533	20.89837	59.90187
11 out	0.498043	0.485592	0.386841	29.12280	16.77575	39.74134
11 in	0.498043	0.485592	0.386841	29.12280	16.77575	39.74134
Abut.Walls						
	x−trans				y−rock	
1	0.414197				5.683760	
12	0.491257				6.741203	

TABLE IV

Kf(lb) DO TO FOUNDATION FAILURE IN THE SOIL

	x−trans (k/ft.) (*10E6)	y−trans (k/ft.) (*10E6)	z−trans (k/ft.) (*10E6)	x−rock (k−ft.) (*10E6)	y−rock (k−ft.) (*10E6)	z−tors (k−ft.) (*10E6)
Abutments						
1	0.303732	0.273772	0.308615	93.25470	4.758111	97.33230
12	0.323264	0.291377	0.330098	136.8988	5.407678	133.9224
Bents						
2	1.063681	1.051288	0.746018	494.4352	365.9183	455.3747
3	0.476395	0.470845	0.388477	73.86966	54.66896	120.7602
4	0.589880	0.583007	0.447452	131.2721	97.15101	170.7048
5	0.512945	0.500121	0.419558	78.41289	45.49625	119.0502
6	0.482494	0.470431	0.380042	59.90616	35.08476	89.35146
7	0.491360	0.479076	0.387026	61.00702	35.72949	90.99342
8 out	0.432738	0.418272	0.334866	27.45464	14.15101	37.56225
8 in	0.167198	0.161609	0.129758	10.60777	5.467590	14.51309
9 out	0.492791	0.476318	0.369071	39.52847	20.37425	47.52585
9 in	0.182851	0.176738	0.136944	14.66710	7.559900	17.63454
10 out	0.491516	0.474632	0.385675	37.78305	18.66882	53.51123
10 in	0.199152	0.192311	0.156267	15.30890	7.564218	21.68163
11 out	0.433220	0.422390	0.336492	25.33233	14.59230	34.56881
11 in	0.108358	0.105649	0.084164	6.336209	3.649877	8.646469

TABLE V

Kf(ub) DO TO FOUNDATION FAILURE IN THE SOIL

	x-trans (k/ft.) (*10E6)	y-trans (k/ft.) (*10E6)	z-trans (k/ft.) (*10E6)	x-rock (k-ft.) (*10E6)	y-rock (k-ft.) (*10E6)	z-tors (k-ft.) (*10E6)
Abutments						
1	0.303732	0.273772	0.308615	93.25470	4.758111	97.33230
12	0.323264	0.291377	0.330098	136.8988	5.407678	133.9224
Bents						
2	0.882830	0.872545	0.813975	539.4747	399.2508	318.4974
3	0.807322	0.797916	0.658331	125.1829	92.64457	181.5892
4	0.912084	0.901458	0.713539	209.3361	154.9240	222.4274
5	0.771851	0.752554	0.631326	117.9912	68.46015	161.1219
6	0.709661	0.691920	0.558973	88.11119	51.60338	118.2016
7	0.709661	0.691920	0.558973	88.11119	51.60338	118.2016
8 out	0.507814	0.490839	0.392963	32.21781	16.60610	40.72533
8 in	0.507814	0.490839	0.394099	32.21781	16.60610	40.72533
9 out	0.557907	0.539257	0.417839	44.75165	23.06645	48.39398
9 in	0.557907	0.539257	0.417839	44.75165	23.06645	48.39398
10 out	0.550216	0.531315	0.431735	42.29533	20.89837	55.34433
10 in	0.550216	0.531315	0.431735	42.29533	20.89837	55.34433
11 out	0.498043	0.485592	0.386841	29.12280	16.77575	36.71768
11 in	0.498043	0.485592	0.386841	29.12280	16.77575	36.71768
Abut.Walls	x-trans				y-rock	
1	0.414197				5.683760	
12	0.491257				6.741203	

TABLE VI

{P,M}eq(lb) DO TO EARTHQUAKE SOIL STRESSES

	x–trans (kips)	y–trans (kips)	z–trans (kips)	x–rock (kip–ft.)	y–rock (kip–ft.)	z–tors (kip–ft.)
Abutments						
1	131.5549	86.24843	339.7241	1776.698	168.1761	1712.815
12	143.2521	88.14966	404.6355	2521.057	200.9536	2074.678
Bents						
2	203.6211	200.5451	3609.658	8626.680	6198.819	2653.551
3	100.0317	98.90306	595.6274	1329.260	978.1371	1333.490
4	130.3586	128.8656	1002.198	2298.197	1675.043	1736.031
5	82.95072	80.77757	550.0862	1366.892	789.5219	1062.022
6	63.14499	61.22169	398.2297	867.8402	503.5119	692.5264
7	62.54815	60.57245	409.0799	891.4853	517.2305	683.1089
8 out	24.08721	21.73943	180.7518	289.4064	147.0741	145.7408
8 in	35.36792	34.55177	62.83417	100.6053	51.12688	275.3029
9 out	27.15216	24.33151	248.2535	408.9791	205.1350	160.2281
9 in	40.45400	39.53686	80.71964	132.9795	66.69966	315.2745
10 out	29.16977	25.86417	286.7839	524.9766	257.2893	192.0270
10 in	42.96084	41.93981	88.58179	162.1548	79.47156	379.6053
11 out	22.21939	20.21385	178.7641	257.6758	146.2374	124.1304
11 in	38.41059	37.83752	51.08095	73.62955	41.78662	288.9382

TABLE VII

{P,M}eq(ub) DO TO EARTHQUAKE SOIL STRESSES

	x−trans (kips)	y−trans (kips)	z−trans (kips)	x−rock (kip−ft.)	y−rock (kip−ft.)	z−tors (kip−ft.)
Abutments						
1	346.5949	291.9804	10792.38	57583.29	5723.749	3073.069
12	365.2257	298.8026	12820.26	81222.50	6783.970	3722.310
Bents						
2	10466.08	8634.261	643303.0	1572281.	1121283.	2405.056
3	1900.615	1713.877	264606	612301.3	444874	3652.615
4	3265.647	2809.382	366468.1	868908.7	626051.3	3349.458
5	1489.921	1231.717	216960.9	570749.3	321691.8	2696.735
6	1202.702	981.3835	155720.9	358884.9	203335.8	1703.080
7	1184.214	962.8955	155720.9	358884.9	203335.8	1635.363
8 out	388.4670	284.7240	48988.04	83814.83	41328.21	257.7638
8 in	822.9355	719.1924	48988.04	83814.83	41328.21	1400.679
9 out	606.0768	421.6446	58769.81	103092.3	50273.98	223.9008
9 in	1059.033	874.6011	58769.81	103092.3	50273.98	1354.946
10 out	429.0398	304.5482	64848.53	127426.0	60412.83	289.4613
10 in	953.1752	828.6835	64848.53	127426.0	60412.83	1852.552
11 out	352.0036	269.0091	45806.07	69865.44	38707.61	205.6062
11 in	939.9226	856.9282	45806.07	69865.44	38707.61	1674.882

TABLE VIII

{P,M}f(lb) DO TO FAILURE OF THE SOIL

	x−trans (kips)	y−trans (kips)	z−trans (kips)	x−rock (kip−ft.)	y−rock (kip−ft.)	z−tors (kip−ft.)
Abutment						
1	131.5549	86.24843	339.7241	1776.698	168.1761	1712.815
12	143.2521	88.14966	404.6355	2521.057	200.9536	2074.678
Bent						
2	264.8709	260.8696	4695.453	11221.61	8063.441	3451.746
3	354.6250	350.6237	2111.574	4712.397	3467.619	4727.391
4	349.3453	345.3441	2685.769	6158.888	4488.913	4652.353
5	267.2791	260.2769	1772.457	4404.325	2543.953	3421.987
6	197.0506	191.0487	1242.718	2708.187	1571.262	2161.101
7	190.0110	184.0092	1242.718	2708.187	1571.262	2075.173
8 out	51.31393	46.31236	385.0628	616.5338	313.3177	310.4773
8 in	216.7430	211.7414	385.0628	616.5338	313.3177	1687.122
9 out	48.14614	43.14457	440.2024	725.2005	363.7448	284.1160
9 in	220.6148	215.6132	440.2024	725.2005	363.7448	1719.341
10 out	52.96258	46.96070	520.7039	953.1825	467.1516	348.6570
10 in	252.5334	246.5315	520.7036	953.1825	467.1516	2231.404
11 out	44.33001	40.32876	356.6532	514.0902	291.7591	247.6533
11 in	268.1873	264.1860	356.6532	514.0902	291.7591	2017.400

TABLE IX

{P,M}f(ub) DO TO FOUNDATION FAILURE

	x−trans (kips)	y−trans (kips)	z−trans (kips)	x−rock (kip−ft.)	y−rock (kip−ft.)	z−tors (kip−ft.)
Abutments						
1	346.5949	291.9804	10792.38	57583.29	5723.749	3073.069
12	365.2257	298.8026	12820.26	81222.50	6783.970	3722.310
Bents						
2	17070.73	14082.93	722986.3	1767033.	1260172.	4532.681
3	1900.615	1713.877	264606	612301.3	444874	6207.802
4	3712.627	3193.911	366468.1	868908.7	626051.3	6109.265
5	1489.921	1231.717	216960.9	570749.3	321691.8	4493.602
6	1202.702	981.3835	155720.9	358884.9	203335.8	2837.863
7	1184.214	962.8955	155720.9	358884.9	203335.8	2725.026
8 out	388.4670	284.7240	48988.04	83814.83	41328.21	407.7050
8 in	822.9355	719.1924	48988.04	83814.83	41328.21	2215.454
9 out	606.0768	421.6446	58769.81	103092.3	50273.98	373.0886
9 in	1059.033	874.6011	58769.81	103092.3	50273.98	2257.763
10 out	429.0398	304.5482	64848.53	127426.0	60412.83	457.8410
10 in	953.1752	828.6835	64848.53	127426.0	60412.83	2930.182
11 out	352.0036	269.0091	45806.07	69865.44	38707.61	325.2073
11 in	939.9226	856.9282	45806.07	69865.44	38707.61	2649.161

Seismic Pressures and Downslope Movement at Bent 2

Calculations at the design level acceleration indicate that unbalanced downslope soil pressures against the Bent 2 piers will be about 630 kips. And, the soil will migrate downslope during the earthquake. Soil movement is estimated to be a little less than one-half foot in a M 6.5 earthquake, and a little more than one-half foot in a M 7.5 earthquake.

Bearing Capacity / Moment Capacity

Table X shows the static axial forces (P_x,P_y) vs. the static moment (M_x,M_y) that a footing could support at calculated bearing capacity for the various bridge footings.

TABLE X

COMPARISON OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE
DURING AN EARTHQUAKE
STATIC ANALSIS

Abutment 1 & 12
Bents 2 through 11

twelve pages

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support:	Br. No. G–772 E/W – – Abutment 1		
PARAMETERS			
Depth of Embed	z	2 feet	2
Soil Unit Weight	gamma	0.13 k/sf	0.13
Angle of fric	phi	34 degrees	34
Capacity factors	Nq	29.44	29.44
from AASHTO	Ngamma	41.06	41.06
Width /2	B	3 feet	3
Length/2	L	21.5 feet	21.5

****** Max Axial = 6061.565 MaxMomX= 32340.51 MaxMomY= 3227.195

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
6061.565	0	6061.565	0
5574.773	836.2160	5450.936	11719.51
5106.281	1531.884	5145.622	16594.63
4656.263	2095.318	4840.307	20813.32
4224.891	2534.935	4534.992	24375.58
3812.339	2859.254	4229.678	27281.42
3418.779	3076.901	3924.363	29530.83
3044.385	3196.604	3619.048	31123.82
2689.329	3227.195	3313.734	32060.37
2353.784	3177.609	3008.419	32340.51
2037.924	3056.886	2703.105	31964.21
1741.921	2874.169	2397.790	30931.49
1465.947	2638.706	2092.475	29242.35
1210.177	2359.846	1787.161	26896.77
974.7836	2047.045	1481.846	23894.77
759.9385	1709.861	1176.532	20236.35
565.8152	1357.956	871.2173	15921.49
392.5868	1001.096	504.1558	9755.416
240.4261	649.1506	186.0196	3799.451
109.5062	312.0927	6.5E–13	1.4E–11
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support: Br. No. G–772 E/W – – Abutment 12

PARAMETERS

Depth of Embed	z	2 feet	2
Soil Unit Weight	gamma	0.13 k/sf	0.13
Angle of fric	phi	34 degrees	34
Capacity factors	Nq	29.44	29.44
from AASHTO	Ngamma	41.06	41.06
Width /2	B	3 feet	3
Length/2	L	25.5 feet	25.5

****** Max Axial = 7197.620 MaxMomX= 45599.69 MaxMomY= 3823.785

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
7197.620	0	7197.620	0
6617.514	992.6271	6473.385	16507.13
6059.552	1817.865	6111.268	23375.60
5523.907	2485.758	5749.150	29320.66
5010.752	3006.451	5387.033	34342.33
4520.259	3390.194	5024.916	38440.60
4052.602	3647.342	4662.798	41615.48
3607.954	3788.352	4300.681	43866.95
3186.487	3823.785	3938.564	45195.02
2788.375	3764.307	3576.446	45599.69
2413.790	3620.686	3214.329	45080.97
2062.906	3403.795	2852.212	43638.84
1735.895	3124.611	2490.094	41273.32
1432.930	2794.214	2127.977	37984.39
1154.184	2423.788	1765.860	33772.07
899.8315	2024.620	1403.742	28636.35
670.0432	1608.103	1041.625	22577.23
464.9929	1185.732	643.4088	14766.23
284.8537	769.1050	237.1099	5743.989
129.7984	369.9254	6.5E–13	1.7E–11
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support: Br. No. G–772 E/W – – Bent 2

PARAMETERS

Depth of Embed	z	36 feet
Soil Unit Weight	gamma	0.135 k/sf
Angle of fric	phi	34 degrees
Capacity factors	Nq	29.44
from AASHTO	Ngamma	41.06
Width /2	B	9 feet
Length/2	L	11 feet

****** Max Axial = 101217.6 MaxMomX= 250629.6 MaxMomY= 184362.3

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
101217.6	0	101217.6	0
94331.98	42449.39	93576.17	102933.7
87609.30	78848.37	89755.44	148096.4
81054.47	109423.5	84630.75	186187.6
74672.32	134410.1	77370.39	212768.5
68467.70	154052.3	70331.51	232094.0
62445.47	168602.7	63522.96	244563.4
56610.46	178322.9	56953.58	250595.7
50967.55	183483.1	50632.25	250629.6
45521.56	184362.3	44567.80	245122.9
40277.35	181248.1	38769.10	234553.0
35239.77	174436.9	33244.99	219416.9
30413.67	164233.8	28004.33	200230.9
25803.90	150952.8	23055.97	177531.0
21415.31	134916.4	18408.77	151872.4
17252.74	116456.0	14071.58	123829.9
13321.04	95911.54	10053.26	93998.01
9625.077	73631.84	6362.654	62990.27
6169.679	49974.40	3008.615	31440.02
2959.704	25305.47	−9.1E−12	−1.0E−10
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support: Br. No. G–772 E/W – – Bent 3

PARAMETERS

Depth of Embed	z	9 feet
Soil Unit Weight	gamma	0.135 k/sf
Angle of fric	phi	34 degrees
Capacity factors	Nq	29.44
from AASHTO	Ngamma	41.06
Width /2	B	9 feet
Length/2	L	11 feet

****** Max Axial = 35272.01 MaxMomX= 82919.78 MaxMomY= 61302.58

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
35272.01	0	35272.01	0
32797.57	14758.90	31879.97	35067.97
30368.86	27331.97	30183.95	49803.53
27990.73	37787.48	28214.68	62072.30
25668.03	46202.45	25794.04	70933.61
23405.60	52662.61	23419.71	77285.04
21208.31	57262.43	21100.54	81237.10
19080.99	60105.12	18845.40	82919.78
17028.49	61302.58	16663.13	82482.53
15055.67	60975.48	14562.59	80094.29
13167.37	59253.20	12552.64	75943.47
11368.45	56273.84	10642.11	70237.97
9663.750	52184.25	8839.882	63205.15
8058.118	47139.99	7154.789	55091.87
6556.405	41305.35	5595.691	46164.45
5163.462	34853.37	4171.442	36708.69
3884.137	27965.79	2890.894	27029.86
2723.279	20833.08	1762.903	17452.74
1685.737	13654.47	796.3204	8321.548
776.3617	6637.892	–2.3E–12	–2.5E–11
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support: Br. No. G–772 E/W ––Bent 4

PARAMETERS

Depth of Embed	z	15 feet
Soil Unit Weight	gamma	0.135 k/sf
Angle of fric	phi	34 degrees
Capacity factors	Nq	29.44
from AASHTO	Ngamma	41.06
Width /2	B	9 feet
Length/2	L	11 feet

****** Max Axial = 49926.59 MaxMomX= 120181.1 MaxMomY= 88453.82

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
49926.59	0	49926.59	0
46471.88	20912.34	45590.24	50149.26
43088.96	38780.06	43422.06	71646.41
39782.67	53706.61	40751.59	89653.50
36557.87	65804.17	37255.45	102452.4
33419.40	75193.66	33844.55	111687.0
30372.12	82004.73	30527.75	117531.8
27420.87	86375.75	27313.89	120181.1
24570.50	88453.82	24211.82	119848.5
21825.87	88394.78	21230.42	116767.3
19191.81	86363.17	18378.52	111190.0
16673.19	82532.30	15664.97	103388.8
14274.84	77084.16	13098.64	93655.34
12001.62	70209.51	10688.38	82300.57
9858.385	62107.82	8443.044	69655.11
7849.969	52987.29	6371.474	56068.97
5981.228	43064.84	4482.532	41911.67
4257.012	32566.14	2785.070	27572.19
2682.169	21725.57	1287.941	13458.98
1261.548	10786.24	–3.8E–12	–4.2E–11
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support: Br. No. G–772 E/W – – Bent 5

PARAMETERS

Depth of Embed	z	8 feet
Soil Unit Weight	gamma	0.135 k/sf
Angle of fric	phi	34 degrees
Capacity factors	Nq	29.44
from AASHTO	Ngamma	41.06
Width /2	B	8 feet
Length/2	L	11.5 feet

****** Max Axial = 28968.82 MaxMomX= 76525.93 MaxMomY= 44450.62

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
28968.82	0	28968.82	0
26905.05	10762.02	26166.87	30091.90
24885.61	19908.49	24765.89	42721.16
22913.92	27496.71	23364.91	53739.31
20993.38	33589.41	21963.94	63146.33
19127.39	38254.79	20562.96	70942.23
17319.36	41566.48	18605.54	74887.33
15572.70	43603.58	16636.07	76525.93
13890.81	44450.62	14719.21	76171.94
12277.10	44197.58	12865.09	73974.29
10734.97	42939.90	11083.82	70105.20
9267.836	40778.48	9385.528	64760.14
7879.090	37819.63	7780.315	58157.86
6572.144	34175.15	6278.304	50540.35
5350.404	29962.26	4889.612	42172.90
4217.275	25303.65	3624.354	33344.05
3176.163	20327.44	2492.647	24365.63
2230.474	15167.22	1504.608	15572.70
1383.613	9962.016	670.3541	7323.618
638.9868	4856.299	3.6E–12	4.2E–11
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support: Br. No. G–772 E/W – – Bent 6

PARAMETERS

Depth of Embed	z	8 feet
Soil Unit Weight	gamma	0.135 k/sf
Angle of fric	phi	34 degrees
Capacity factors	Nq	29.44
from AASHTO	Ngamma	41.06
Width /2	B	7 feet
Length/2	L	10 feet

****** Max Axial = 20928.52 MaxMomX= 48406.34 MaxMomY= 28305.08

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
20928.52	0	20928.52	0
19448.13	6806.848	18951.80	18951.80
17999.74	12599.82	17963.45	26945.17
16585.62	17414.90	16975.09	33950.18
15208.06	21291.28	15986.73	39966.84
13869.32	24271.32	14998.38	44995.14
12571.71	26400.60	13532.97	47365.40
11317.49	27727.86	12101.58	48406.34
10108.96	28305.08	10710.87	48198.93
8948.384	28187.41	9367.491	46837.45
7838.050	27433.17	8078.085	44429.47
6780.241	26103.92	6849.310	41095.86
5777.237	24264.39	5687.816	36970.80
4831.321	21982.51	4600.256	32201.79
3944.773	19329.39	3593.282	26949.61
3119.876	16379.35	2673.544	21388.35
2358.911	13209.90	1847.696	15705.41
1664.159	9901.749	1122.387	10101.49
1037.902	6538.787	504.2720	4790.584
482.4223	3208.108	0	0
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support: Br. No. G–772 E/W – – Bent 7

PARAMETERS

Depth of Embed	z	8 feet
Soil Unit Weight	gamma	0.135 k/sf
Angle of fric	phi	34 degrees
Capacity factors	Nq	29.44
from AASHTO	Ngamma	41.06
Width /2	B	7 feet
Length/2	L	10 feet

****** Max Axial = 20928.52 MaxMomX= 48406.34 MaxMomY= 28305.08

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
20928.52	0	20928.52	0
19448.13	6806.848	18951.80	18951.80
17999.74	12599.82	17963.45	26945.17
16585.62	17414.90	16975.09	33950.18
15208.06	21291.28	15986.73	39966.84
13869.32	24271.32	14998.38	44995.14
12571.71	26400.60	13532.97	47365.40
11317.49	27727.86	12101.58	48406.34
10108.96	28305.08	10710.87	48198.93
8948.384	28187.41	9367.491	46837.45
7838.050	27433.17	8078.085	44429.47
6780.241	26103.92	6849.310	41095.86
5777.237	24264.39	5687.816	36970.80
4831.321	21982.51	4600.256	32201.79
3944.773	19329.39	3593.282	26949.61
3119.876	16379.35	2673.544	21388.35
2358.911	13209.90	1847.696	15705.41
1664.159	9901.749	1122.387	10101.49
1037.902	6538.787	504.2720	4790.584
482.4223	3208.108	0	0
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support:	Br. No. G–772 E/W – – Bent 8	
PARAMETERS		
Depth of Embed	z	6 feet
Soil Unit Weight	gamma	0.135 k/sf
Angle of fric	phi	34 degrees
Capacity factors	Nq	29.44
from AASHTO	Ngamma	41.06
Width /2	B	4.5 feet
Length/2	L	7 feet

****** Max Axial = 6642.252 MaxMomX= 11313.90 MaxMomY= 5811.668

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
6642.252	0	6642.252	0
6173.823	1389.110	6027.494	4219.246
5716.105	2572.247	5720.115	6006.120
5269.706	3557.052	5412.735	7577.830
4835.232	4351.709	5105.356	8934.374
4413.289	4964.950	4797.977	10075.75
4004.482	5406.051	4490.598	11001.96
3609.419	5684.834	4040.678	11313.90
3228.704	5811.668	3579.016	11273.90
2862.945	5797.464	3132.465	10963.62
2512.747	5653.682	2703.306	10407.73
2178.717	5392.326	2293.822	9634.054
1861.461	5025.945	1906.293	8673.636
1561.584	4567.635	1543.002	7560.710
1279.694	4031.036	1206.229	6332.703
1016.395	3430.335	898.2566	5030.237
772.2954	2780.263	621.3659	3697.127
547.9996	2096.098	377.8385	2380.382
344.1143	1393.663	169.9560	1130.207
161.2457	689.3256	1.1E–12	8.0E–12
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support: Br. No. G–772 E/W – – Bent 9

PARAMETERS

Depth of Embed	z	8 feet
Soil Unit Weight	gamma	0.135 k/sf
Angle of fric	phi	34 degrees
Capacity factors	Nq	29.44
from AASHTO	Ngamma	41.06
Width /2	B	4.5 feet
Length/2	L	7 feet

****** Max Axial = 8078.085 MaxMomX= 14055.77 MaxMomY= 7178.965

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
8078.085	0	8078.085	0
7517.235	1691.378	7363.172	5154.220
6969.269	3136.171	7005.715	7356.001
6434.793	4343.485	6648.259	9307.562
5914.413	5322.972	6290.802	11008.90
5408.735	6084.827	5933.345	12460.02
4918.366	6639.794	5575.889	13660.92
4443.910	6999.159	5019.917	14055.77
3985.976	7174.757	4447.753	14010.42
3545.168	7178.965	3895.955	13635.84
3122.093	7024.709	3366.803	12962.19
2717.357	6725.458	2862.579	12022.83
2331.566	6295.228	2385.566	10854.32
1965.326	5748.580	1938.044	9496.416
1619.244	5100.619	1522.295	7992.049
1293.925	4366.998	1140.600	6387.364
989.9765	3563.915	795.2426	4731.693
708.0033	2708.112	488.5020	3077.562
448.6120	1816.878	222.6606	1480.693
212.4089	908.0481	1.5E–12	1.1E–11
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support:	Br. No. G–772 E/W ––Bent 10	
PARAMETERS		
Depth of Embed	z	6 feet
Soil Unit Weight	gamma	0.135 k/sf
Angle of fric	phi	34 degrees
Capacity factors	Nq	29.44
from AASHTO	Ngamma	41.06
Width /2	B	5 feet
Length/2	L	8 feet

****** Max Axial = 8749.743 MaxMomX= 17104.11 MaxMomY= 8450.501

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
8749.743	0	8749.743	0
8127.903	2031.975	7924.753	6339.802
7520.479	3760.239	7512.258	9014.710
6928.303	5196.227	7099.763	11359.62
6352.207	6352.207	6687.267	13374.53
5793.022	7241.278	6274.772	15059.45
5251.580	7877.371	5862.277	16414.37
4728.712	8275.247	5345.035	17104.11
4225.250	8450.501	4734.869	17045.53
3742.025	8419.558	4143.304	16573.21
3279.869	8199.673	3573.747	15724.49
2839.613	7808.936	3029.603	14542.09
2422.088	7266.264	2514.278	13074.24
2028.126	6591.411	2031.176	11374.59
1658.559	5804.956	1583.705	9502.235
1314.217	4928.315	1175.270	7521.731
995.9334	3983.733	809.2766	5503.080
704.5381	2994.287	489.1297	3521.734
440.8631	1983.884	218.2356	1658.590
205.7399	977.2648	–8.5E–13	–6.8E–12
0	0	0	0

COMPARISION OF AXIAL LOAD TO MOMENT ON A FOOTING AT SOIL FAILURE DURING AN EARTHQUAKE– STATIC ANALYSIS

for Support: Br. No. G–772 E/W – – Bent 11

PARAMETERS

Depth of Embed	z	6 feet
Soil Unit Weight	gamma	0.135 k/sf
Angle of fric	phi	34 degrees
Capacity factors	Nq	29.44
from AASHTO	Ngamma	41.06
Width /2	B	4.5 feet
Length/2	L	6.5 feet

****** Max Axial = 6203.139 MaxMomX= 9474.650 MaxMomY= 5434.408

ROTATION ABOUT FOOTING LONG AXIS		ROTATION ABOUT FOOTING SHORT AXIS	
Axial Load(kips)	Moment(k–ft)	Axial Load(kips)	Moment(k–f)
6203.139	0	6203.139	0
5767.329	1297.649	5632.292	3660.990
5341.108	2403.498	5346.868	5213.197
4925.083	3324.431	5061.445	6579.878
4519.861	4067.875	4776.021	7761.035
4126.047	4641.803	4490.598	8756.666
3744.247	5054.734	4074.172	9268.742
3375.068	5315.732	3644.096	9474.650
3019.115	5434.408	3226.768	9438.298
2676.995	5420.916	2824.015	9178.051
2349.315	5285.958	2437.664	8714.652
2036.679	5040.781	2069.542	8071.216
1739.694	4697.176	1721.475	7273.234
1458.967	4267.480	1395.290	6348.572
1195.104	3764.578	1092.814	5327.470
948.7102	3201.897	815.8736	4242.542
720.3920	2593.411	566.2951	3128.780
510.7558	1953.641	345.9054	2023.547
320.4076	1297.650	156.5315	966.5823
149.9536	641.0518	–7.6E–13	–5.0E–12
0	0	0	0

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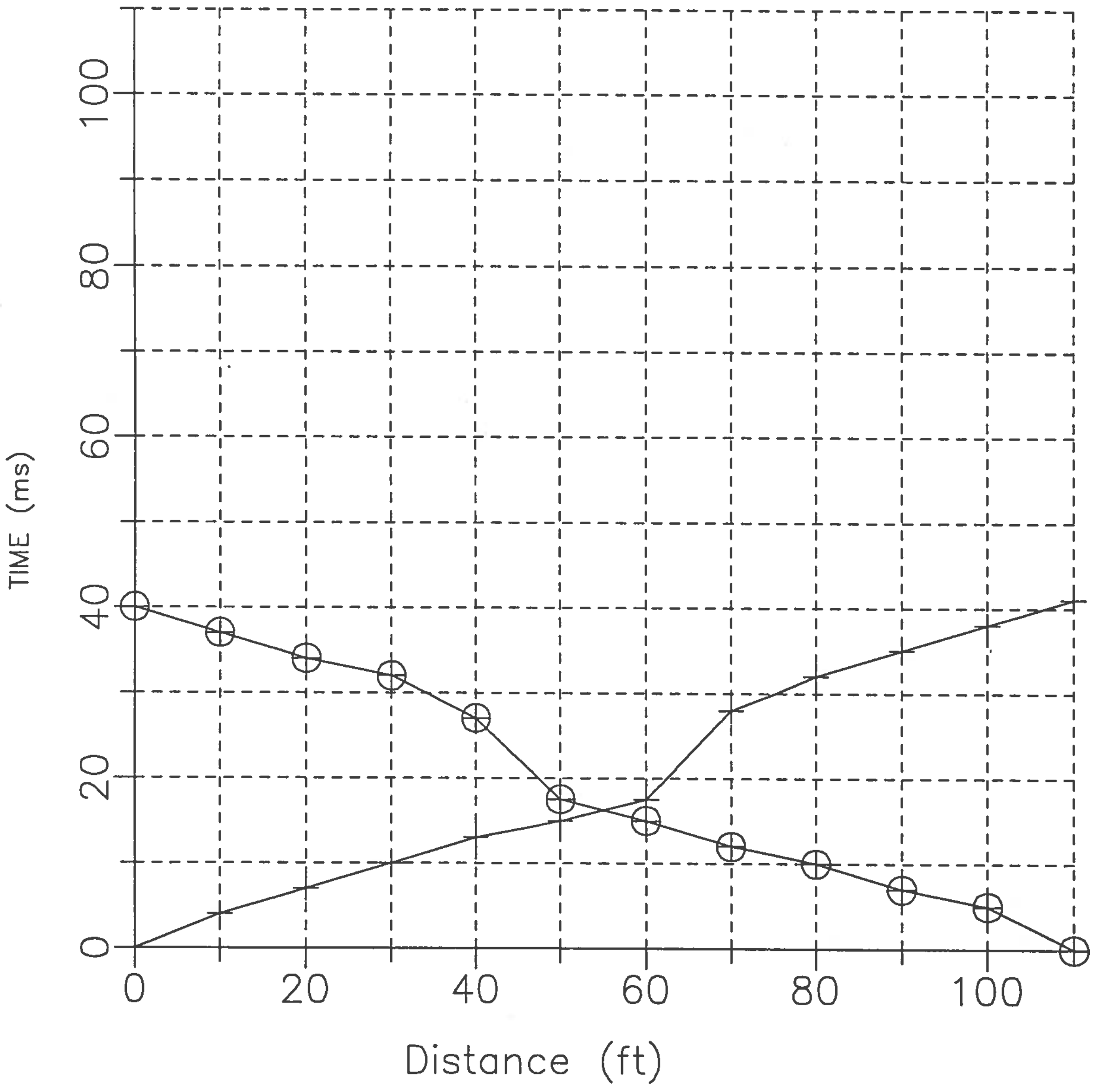
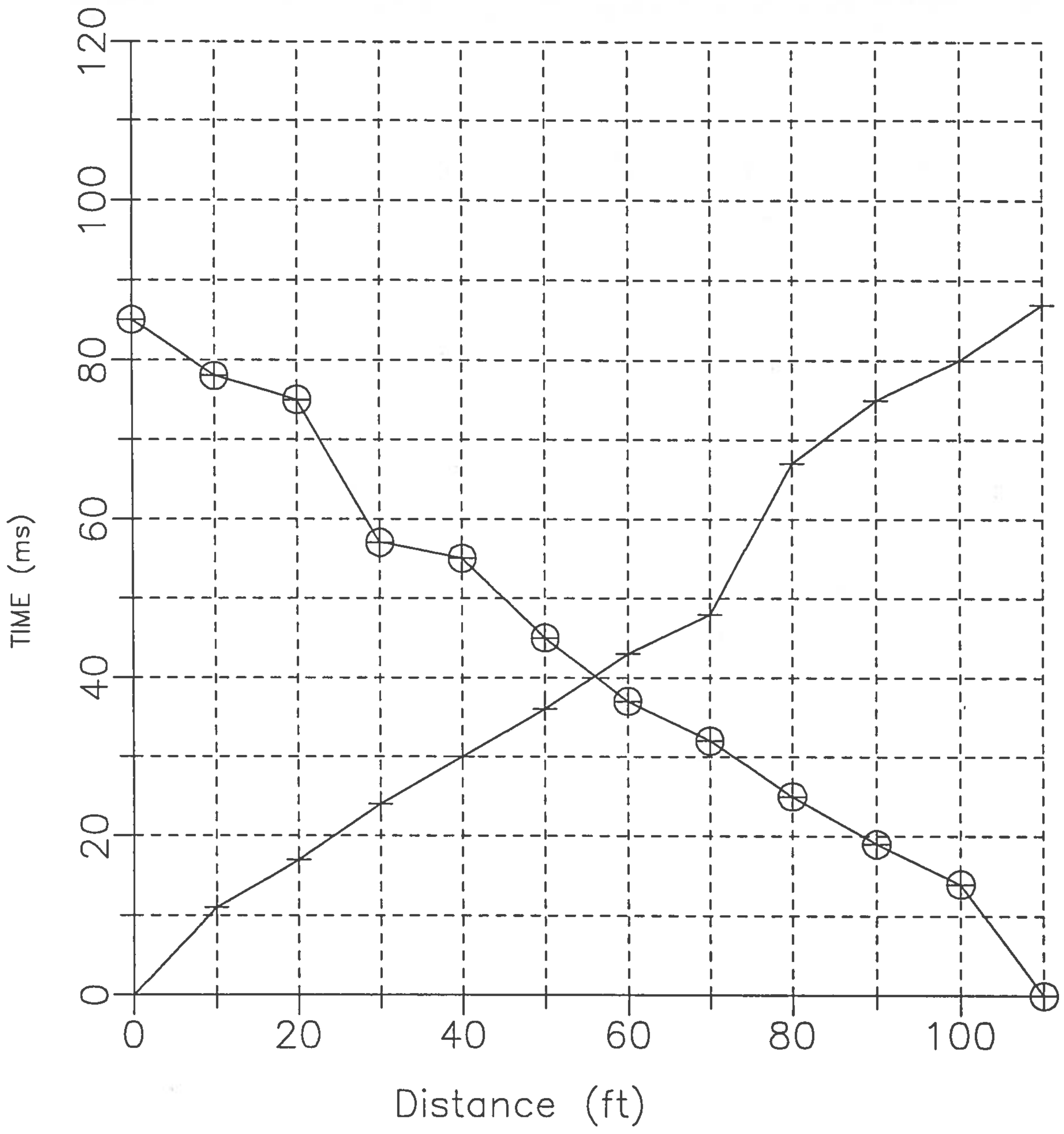
APPENDIX A



Line 1 P–Wave Time–Distance Graph



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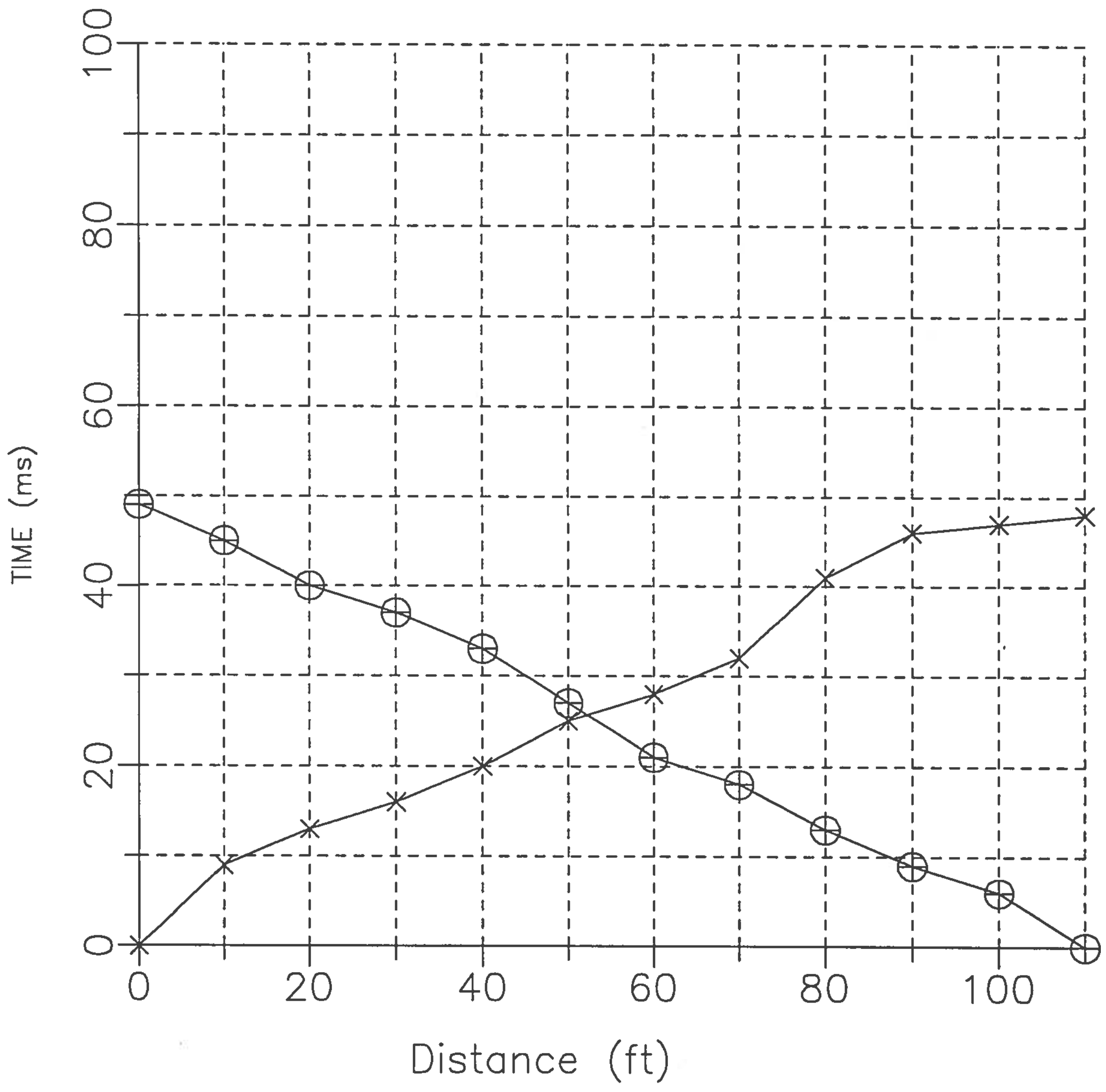

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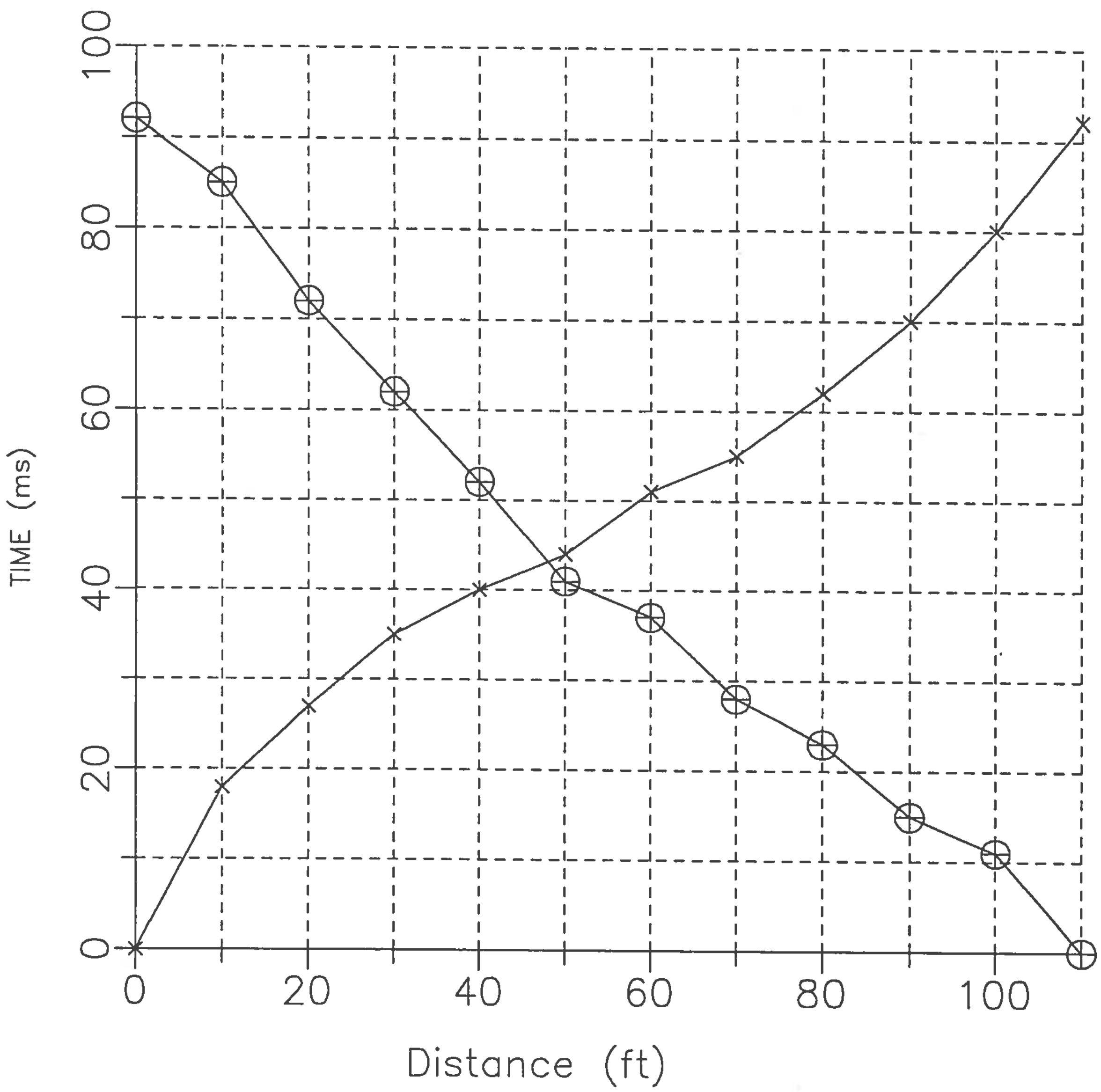

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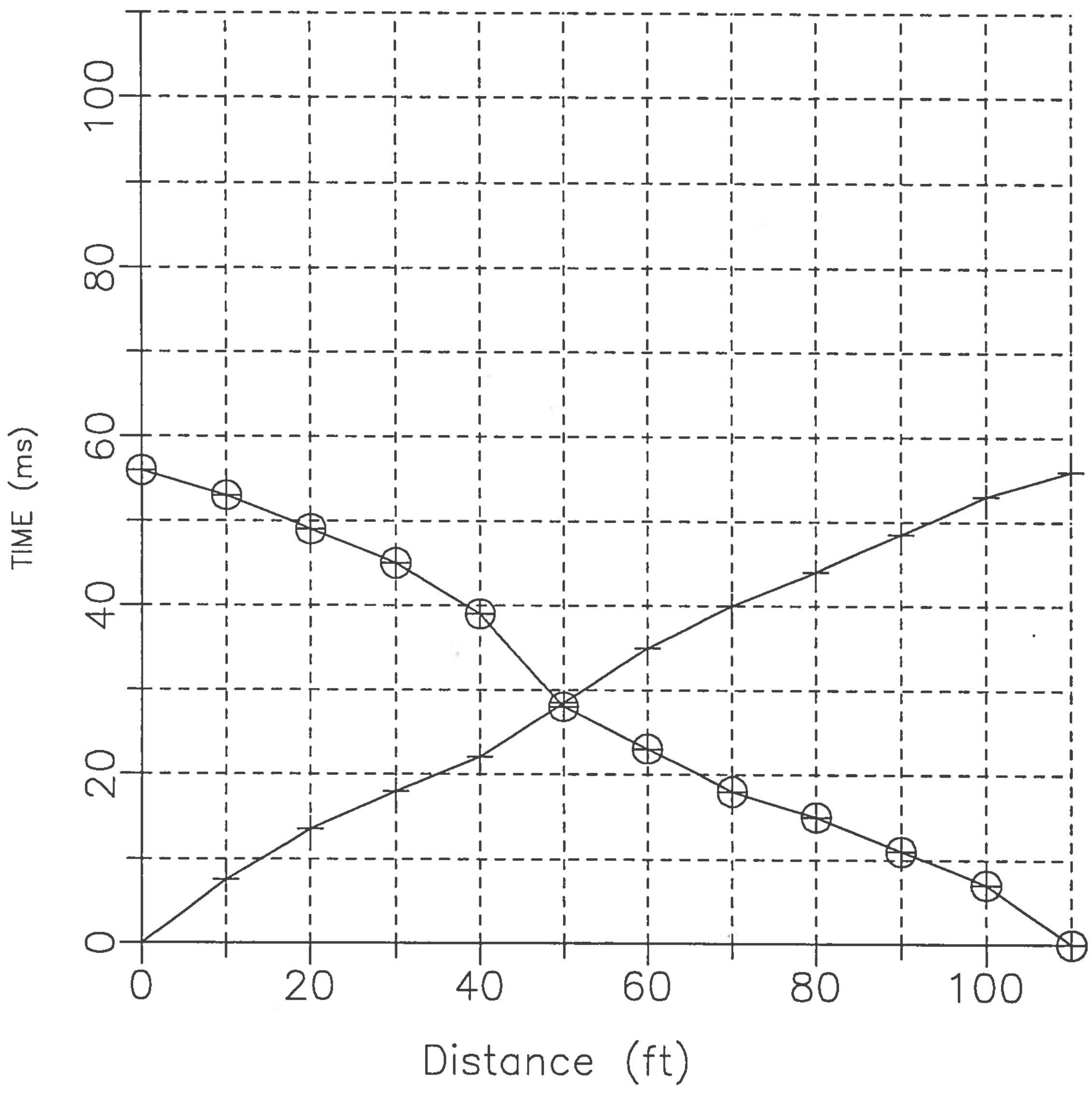

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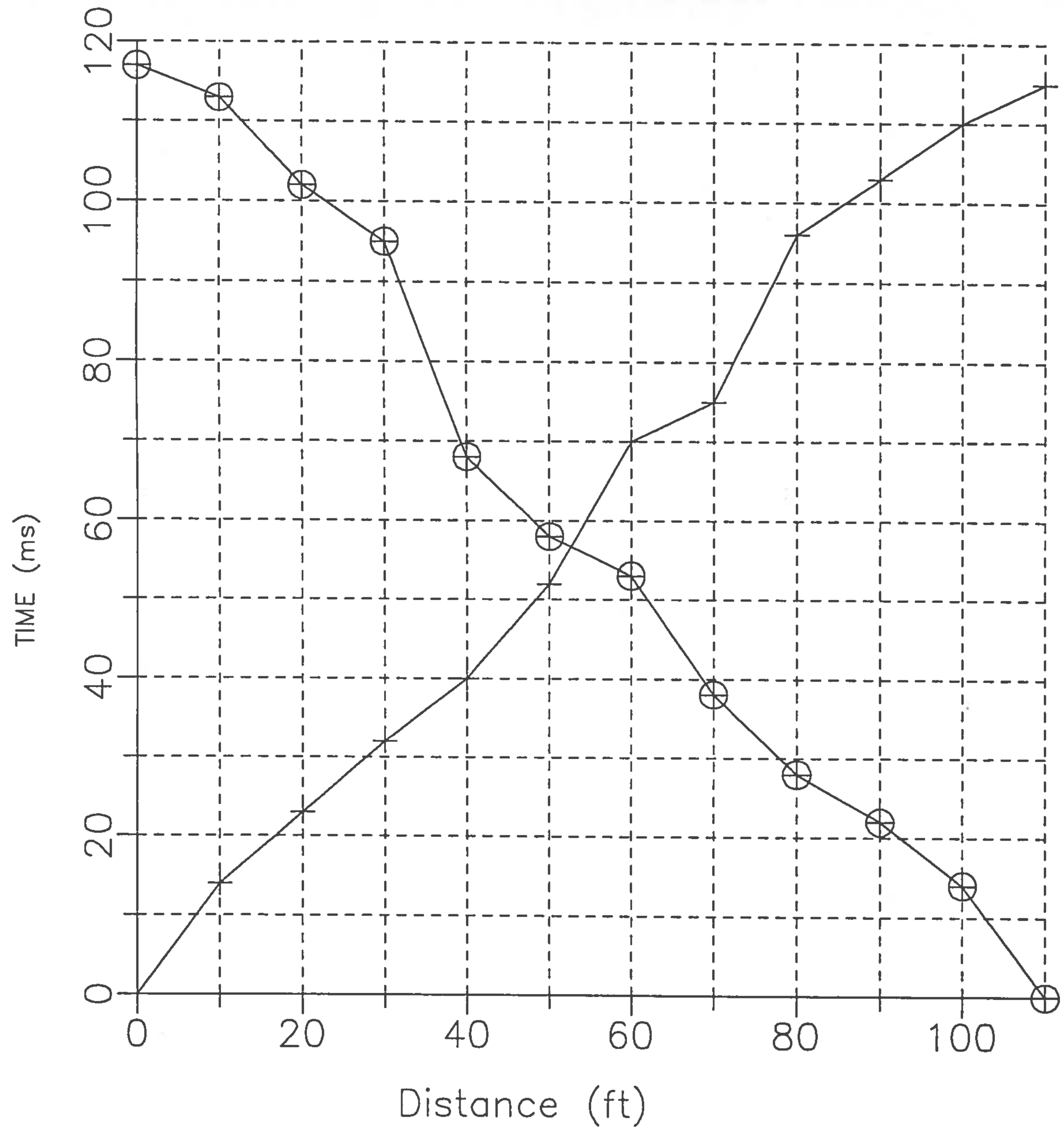

Line 3 S-Wave Time-Distance Graph