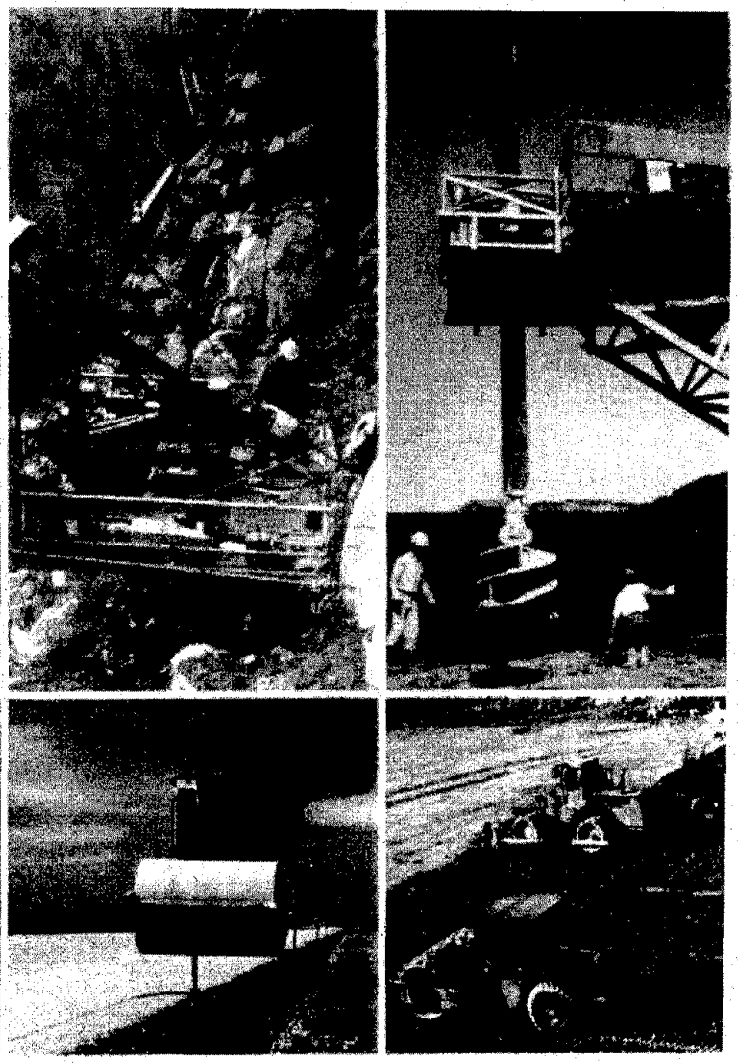
Training Course in Geotechnical and Foundation Engineering

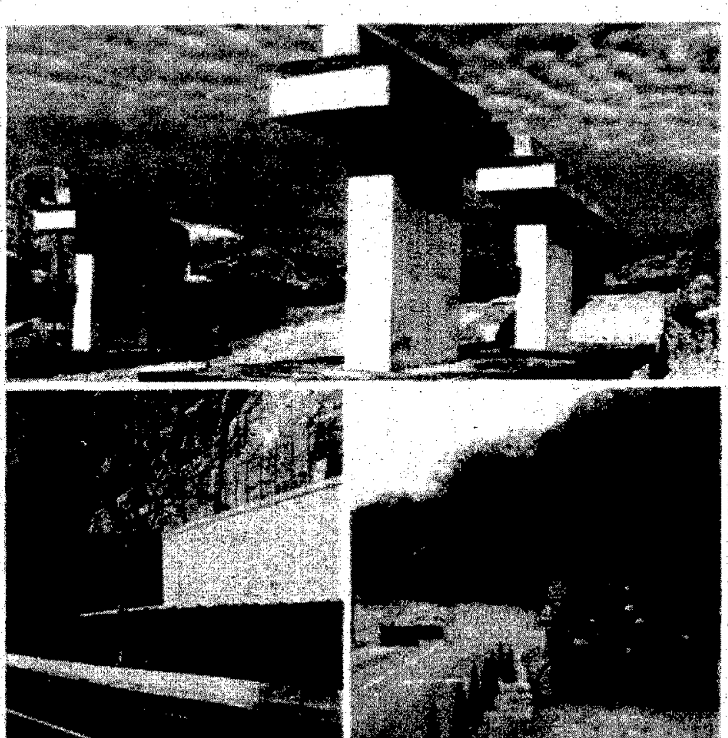
NHI Course No. 13239 - MODULE 9

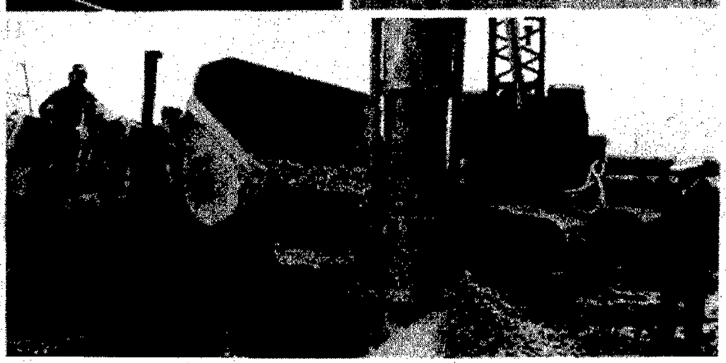
Publication No. FHWA HI-99-014
February 1999

Geotechnical Earthquake Engineering

Student Exercises







U.S. Department
of Transportation

Federal Highway Administration

NHI National Highway Institute

THIS STUDENT EXERCISE BOOK (FHWA-99-014) IS INTENDED ONLY TO BE USED AS AN INTERACTIVE TEACHING TOOL FOR NHI COURSE NO. 13239 – MODULE 9 "GEOTECHNICAL EARTHQUAKE ENGINEERING", AND IS NOT INTENDED TO BE USED AS AN INDIVIDUAL EXERCISE BOOK.

DETAILED DESIGN EXAMPLES ILLUSTRATING THE PRINCIPLES AND ANALYSES OF GEOTECHNICAL EARTHQUAKE ENGINEERING ARE INCLUDED IN PART II OF THE REFERENCE MANUAL (FHWA-HI-99-012) FOR THE SAME COURSE.

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16. Abstract

This student exercise book has been developed for use as an interactive teaching tool and a companion workbook for NHI Course No. 13239 - Module 9 "Geotechnical Earthquake Engineering", and is not intended to be used as an individual exercise book. The extents and depths of the problems presented in this exercise book are limited due to the time constraint of the 2.5-day course schedule. Detailed design examples illustrating the principles and analyses of geotechnical earthquake engineering are included in Part II of Module 9"Geotechnical Earthquake Engineering" Reference Manual (FHWA-HI-99-012).

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Geotechnical earthquake engineering, response spectra, site response analysis, liquefaction, slopes, foundations, retaining walls	No restrictions.

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UNCLASSIFIED	UNCLASSIFIED	162	

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PREFACE

This student exercise book is intended only to be used as an interactive teaching tool and a companion workbook for NHI Course No. 13239 - Module 9 "Geotechnical Earthquake Engineering", and is not intended to be used as an individual exercise book. The extents and depths of the problems presented in this exercise book are limited due to the time constraint of the 2.5-day course schedule.

Module 9 "Geotechnical Earthquake Engineering" is the ninth module in a series of twelve modules that constitute a comprehensive training course in geotechnical and foundation engineering. Sponsored by the National Highway Institute (NHI) of the Federal Highway Administration (FHWA), the training course is given at different locations in the U.S. The course is tailored to the needs of both geotechnical and structural engineers who are involved in the analysis, design, and construction of surface transportation facilities in seismic areas.

A reference manual (FHWA-HI-99-012) was developed to provide information on how to apply principles of geotechnical earthquake engineering to planning, design, and retrofit of highway facilities. Detailed design examples illustrating the principles and analyses of geotechnical earthquake engineering are included in Part II of the reference manual.

Finally, this student exercise book is developed to be used as a living document. Additional student exercises or case histories may be given separately during the training session.

NOTICE

The information in this document has been funded wholly or in part by the US Department of Transportation, Federal Highway Administration (FHWA), under Contract No. DTFH 61-94-C-00104 to Parsons Brinckerhoff Quade & Douglas, Inc. The document has been subjected to peer and administrative review by FHWA, and it has been approved for publication as a FHWA document.

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CONVERSION FACTORS

Approximate Conversions to SI Units			Approximate Conversions from SI Units		
When you know	Multiply by	To find	When you know	Multiply by	To find
(a) Length					
inch	25.4	millimeter	millimeter	0.039	inch
foot	0.305	meter	meter	3.28	foot
yard	0.914	meter	meter	1.09	yard
mile	1.61	kilometer	kilometer	0.621	mile
(b) Area					
square inches	645.2	square millimeters	square millimeters	0.0016	square inches
square feet	0.093	square meters	square meters	10.764	square feet
acres	0.405	hectares	hectares	2.47	acres
square miles	2.59	square kilometers	square kilometers	0.386	square miles
(c) Volume					
fluid ounces	29.57	milliliters	milliliters	0.034	fluid ounces
gallons	3.785	liters	liters	0.264	gallons
cubic feet	0.028	cubic meters	cubic meters	35.32	cubic feet
cubic yards	0.765	cubic meters	cubic meters	1.308	cubic yards
(d) Mass					
ounces	28.35	grams	grams	0.035	ounces
pounds	0.454	kilograms	kilograms	2.205	pounds
short tons (2000 lb)	0.907	megagrams (tonne)	megagrams (tonne)	1.102	short tons (2000 lb)
(e) Force					
pound	4.448	Newton	Newton	0.2248	pound
(f) Pressure, Stress, Modulus of Elasticity					
pounds per square foot	47.88	Pascals	Pascals	0.021	pounds per square foot
pounds per square inch	6.895	kiloPascals	kiloPascals	0.145	pounds per square inch
(g) Density					
pounds per cubic foot	16.019	kilograms per cubic meter	kilograms per cubic meter	0.0624	pounds per cubic feet
(h) Temperature					
Fahrenheit temperature(°F)	5/9(°F- 32)	Celsius temperature(°C)	Celsius temperature(°C)	9/5(°C)+ 32	Fahrenheit temperature(°F)

Notes: 1) The primary metric (SI) units used in civil engineering are meter (m), kilogram (kg), second(s), newton (N) and pascal ($Pa=N/m^2$).
2) In a "soft" conversion, an English measurement is mathematically converted to its exact metric equivalent.
3) In a "hard" conversion, a new rounded metric number is created that is convenient to work with and remember.

MODULE 9
GEOTECHNICAL EARTHQUAKE ENGINEERING
TABLE OF CONTENTS

STUDENT EXERCISES

STUDENT EXERCISE NO. 1

Derivation of Site-Specific Design Ground Motion Parameters and Response Spectra.

Objective:

Design Earthquake and Site-Specific Response Spectra for a Firm Ground Site (V_s = 760 m/s) and Soft Clay Site (V_s = 120 m/s) in Boston, Massachusetts for 2% in 50-Year Probability of Exceedance.

Source Materials:

Reference Manual Part I: Figures 3-4, 3-5, 4-17, and 4-18, Tables 4-3 and 4-5.

USGS Website Data: 2% in 50-Year Seismic Hazard Data, Deaggregated Hazard Data for Boston, Massachusetts (attached).

1. Establish Free-Field Firm Ground Peak Horizontal Ground Acceleration from USGS Hazard Map (Figure 3-5):

   PGA_{FF} = ____________

2. Establish T_O and T_S for Normalized Spectra (Figure 4-18) For **Firm Ground** (V_s = 760 m/s)

   A. Establish Z Factor From Figure 3-4:

      Z = ____________

   B. Establish Site Class From Table 4-3

      Site Class = ____________

   C. Establish C_v From Table 4-5:

      C_v = ____________

D. Establish T_s From Inset on Figure 4-18:

$$T_s = \frac{C_v}{2.5\,Z} = \_\_\_\_\_\_\_\_\_\_\_\_$$

E. Establish T_o From Inset on Figure 4-18:

$$T_o = 0.2\,T_s = \_\_\_\_\_\_\_\_\_\_\_\_$$

3. Plot Spectra on Attached Graph Paper

4. Repeat Steps 2 and 3 for **Soft Clay** Site

5. Plot Spectral Accelerations from USGS Website on Attached Graph

6. Subjectively Assess Design Earthquake Magnitude and Distance from USGS Website Data

A. M = ______________

B. D = ______________

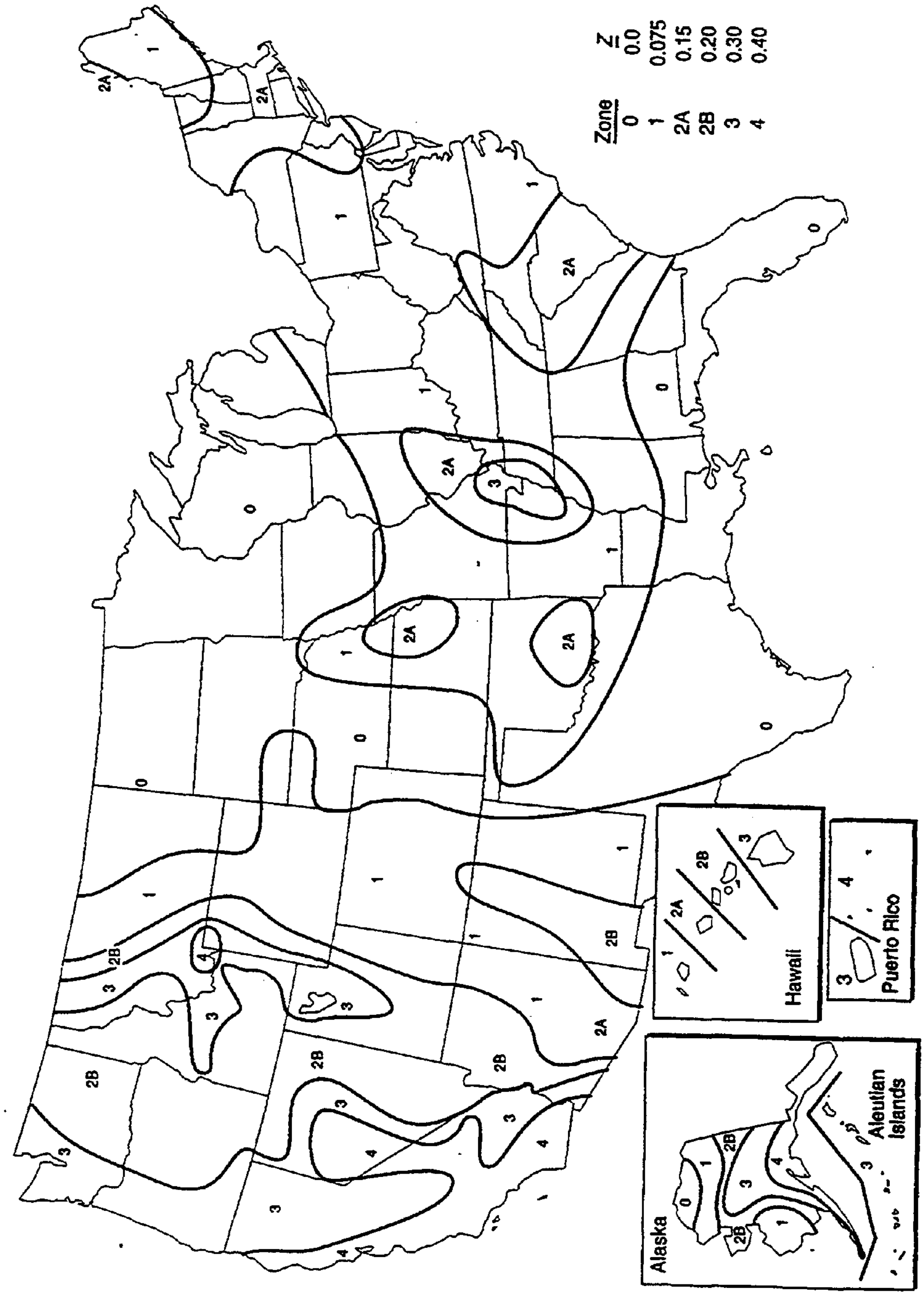


Figure 3-4: Map and Table for Evaluation of UBC Seismic Zone Factor, Z. (Reproduced from the Uniform Building Code™, Copyright© 1994, with the Permission of the Publisher, the International Conference of Building Officials)

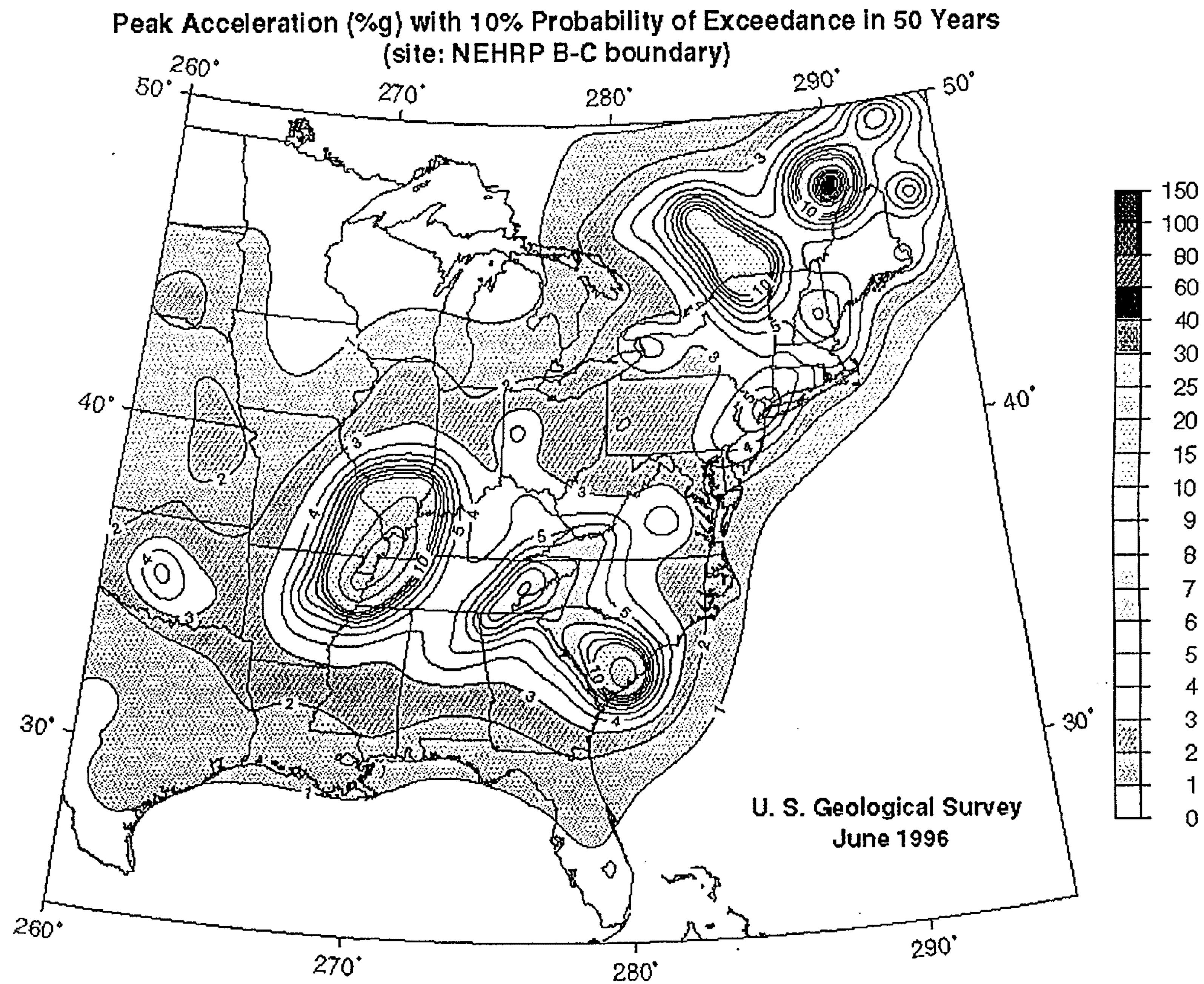


Figure 3-5: Peak Horizontal Ground Acceleration in Bedrock for Central and Eastern U.S. with a 10 Percent Probability of Exceedance in 50 Years. (Frankel, et al., 1996)

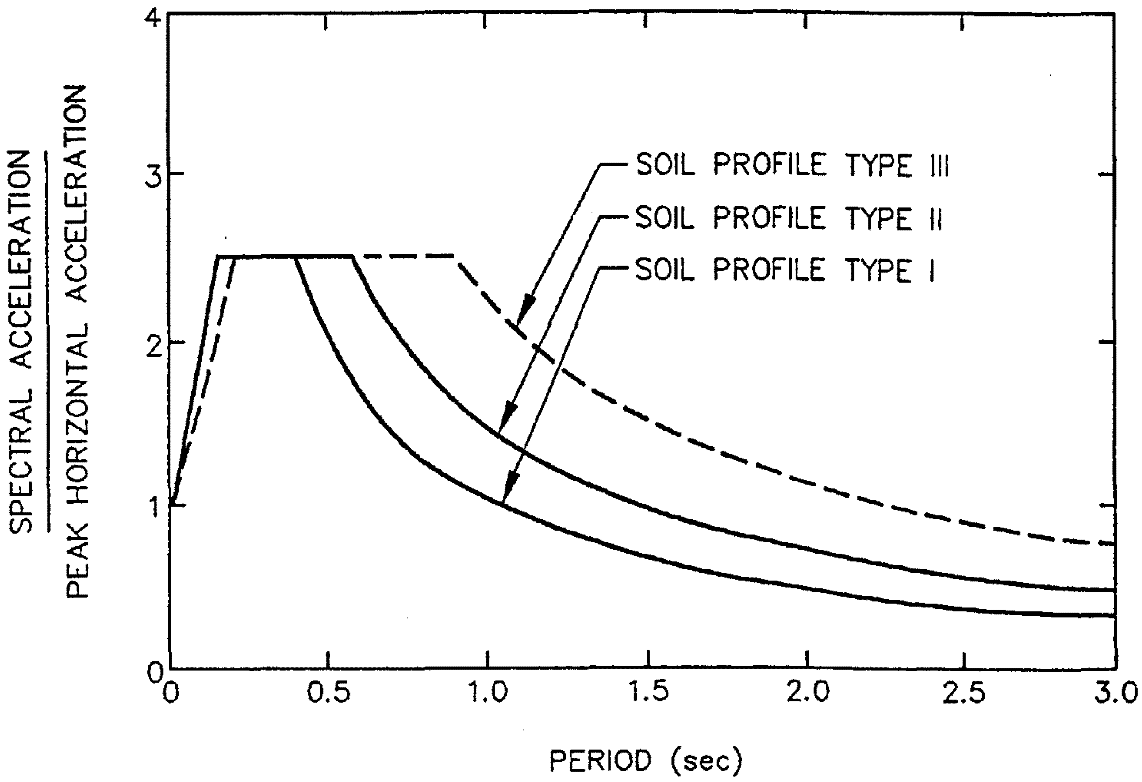


Figure 4-17: Normalized 1994 Uniform Building Code Response Spectra. (UBC, 1994, reproduced from the Uniform Building Code™, copyright© 1994, with the permission of the publisher, the International Conference of Building Officials)

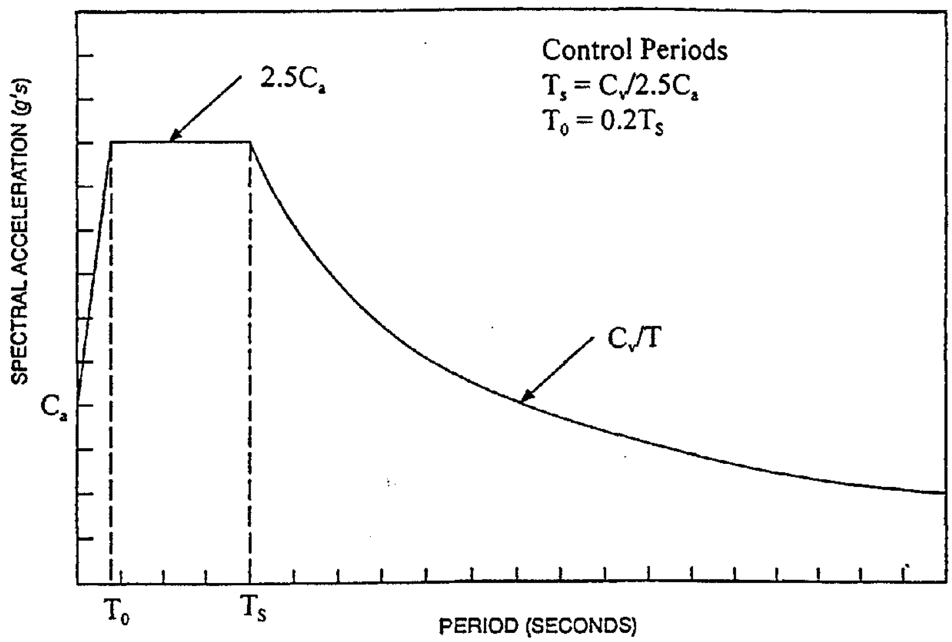


Figure 4-18: 1997 Uniform Building Code Design Response Spectra (UBC, 1997, reproduced from the Uniform Building Code™, copyright© 1997, with the permission of the publisher, the International Conference of Building Officials)

TABLE 4-3
1997 UBC SITE CLASSIFICATION

Designation	Site Class	Shear Wave Velocity[1]	Other Characteristics[2]
S_A	Hard Rock	>1500 m/s	
S_B	Rock	760 m/s to 1500 m/s	
S_C	Very Dense Soil and Soft Rock	360 m/s to 760 m/s	$N > 50$, $S_u > 100$ kPa
S_D	Stiff Soil	180 m/s to 360 m/s	$15 < N < 50$ 50 kPa $< S_u <$ 100 kPa
S_E	Soft Soil	Less than 180 m/s	More than 3m of soil with $PI > 20$, $W_n > 40\%$, and $S_u < 25$ kPa
S_F	Special Soils		Collapsible, liquefiable, sensitive soils; More than 3m of peat or highly organic; More than 7.5m of clay with $PI > 75$; More than 36m of soft to medium clay.

Notes: 1. Average shear wave velocity for upper 30m.
2. N = standard Penetration Test Blow Count
Su = Undrained Shear Strength
PI = Plasticity Index
W_n = Moisture content

TABLE 4-4
SEISMIC COEFFICIENT C_a

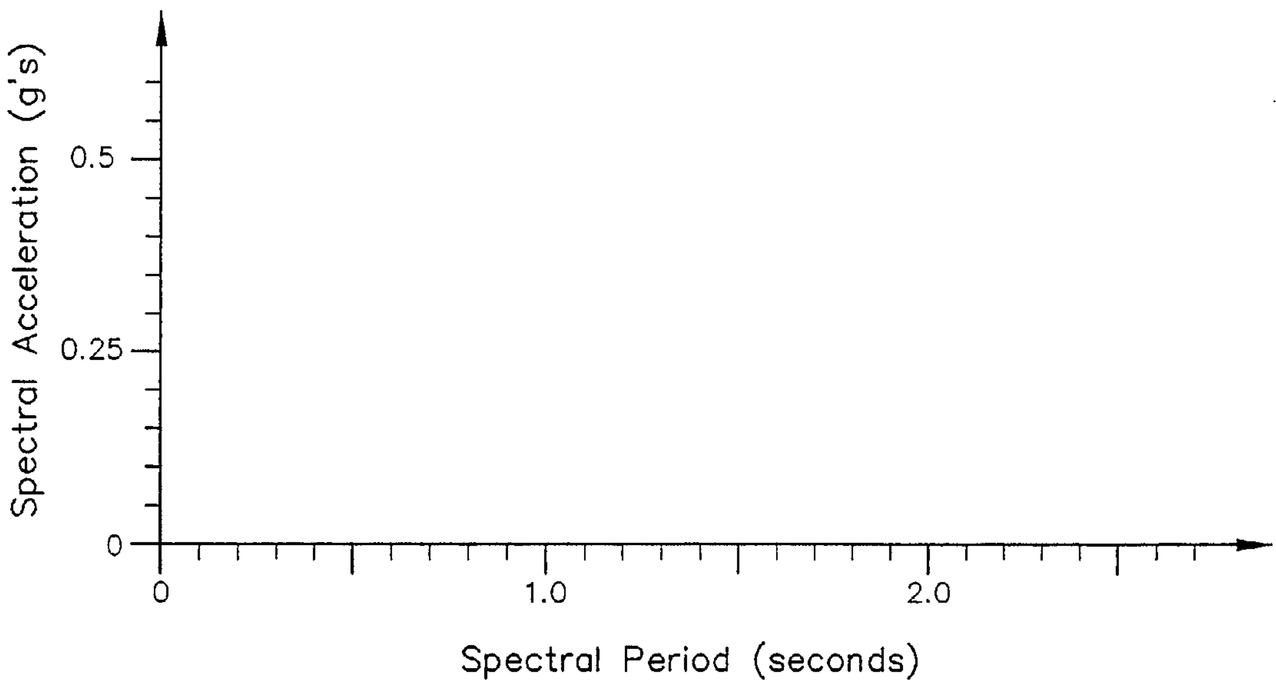
Soil Profile Type	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_a$
S_B	0.08	0.15	0.20	0.30	$0.40N_a$
S_C	0.09	0.18	0.24	0.33	$0.40N_a$
S_D	0.12	0.22	0.28	0.36	$0.44N_a$
S_E	0.19	0.30	0.34	0.36	$0.36N_a$
S_F	See Footnote 1				

Notes: [1] Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.

TABLE 4-5
SEISMIC COEFFICIENT C_V

Soil Profile Type	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S_A	0.06	0.12	0.16	0.24	$0.32N_V$
S_B	0.08	0.15	0.20	0.30	$0.40N_V$
S_C	0.13	0.25	0.32	0.45	$0.56N_V$
S_D	0.18	0.32	0.40	0.54	$0.64N_V$
S_E	0.26	0.50	0.64	0.84	$0.96N_V$
S_F	See Footnote 1				

Notes: [1] Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S_F.



Spectral Acceleration (g's)
0.5
0.25
0
0
1.0
2.0
Spectral Period (seconds)

USGS

PROJECT INFO

REVIEW MATERIAL

1996 HAZARD MAPS

SEISMIC HAZARD

MAP INFO

GENERAL

NATIONAL SEISMIC HAZARD MAPPING PROJECT

USGS, Central Region, Geologic Hazards Team
Golden, Colorado

Welcome to the USGS Zip Code earthquake ground motion hazard look-up page. Here you will be able to enter a 5 digit integer zip code and ground motion hazard values, expressed as a percent of the acceleration of gravity, (%g), will be returned to you. The ground motion hazard values returned will be Peak Ground Acceleration, (PGA), 0.2 second period spectral acceleration, (SA), 0.3 second period (SA), and 1.0 second period (SA) for 10%, 5%, and 2% probability of exceedence, (PE), in 50 years.

(These ground motion values are calculated for 'firm rock' sites which correspond to a shear-wave velocity of 760 m/sec. in the top 30m. Different soil sites may amplify or de-amplify these values.)

- The original zip code file was a freebee download from the Census Bureau, dated approximately January 1996, and thus may not reflect the most recent Zip Codes in use today.
- It has been determined that the latitude and longitude associated with each zip code is the average of the northern and southern most latitudes and the average of the eastern and western most longitudes of the zip code area. This location is not necessarily the Post Office location nor the centroid of the zip code area.
- In this look-up program each zip code location is associated with the nearest point on a grid of points 1/10 of a degree apart on which earthquake ground motions have been calculated covering the 48 adjacent states.

To find the ground motion values enter a 5 digit zip code in each of the blank boxes in the following table. Use the TAB key to move to the next table element. You may request from 1 to 12 Zip Codes.

NO EXTENSIONS

NO ALPHA CHARACTERS

NO DECIMAL NUMBERS

Enter Zip Code: 02115	Enter Zip Code:	Enter Zip Code:
Enter Zip Code:	Enter Zip Code:	Enter Zip Code:
Enter Zip Code:	Enter Zip Code:	Enter Zip Code:
Enter Zip Code:	Enter Zip Code:	Enter Zip Code:

Submit Query

The URL of this page is
http://gldage.cr.usgs.gov/eq/html/zipcode.shtml
Contact: Stanley L. Hanson (hanson@usgs.gov)
Updated: Thursday, 04-Sep-97 14:16:37 MDT

```
The input zip-code is 02115.
ZIP CODE                              2115
LOCATION                             42.3419 Lat.    -71.0968 Long.
DISTANCE TO NEAREST GRID POINT       4.6581 kms
NEAREST GRID POINT                   42.3 Lat.       -71.1 Long.
Probabilistic ground motion values, in %g, at this point are:
            10%PE in 50 yr   5%PE in 50 yr   2%PE in 50 yr
   PGA           4.75            8.20           15.87
0.2 sec SA      10.65           17.44           31.40
0.3 sec SA       8.12           13.02           24.45
1.0 sec SA       2.83            4.87            8.78
- - - - - - - - - - - - - - - - - - - - - - - - - - - - - - - -
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USGS

PROJECT INFO

REVIEW MATERIAL

1996 HAZARD MAPS

SEISMIC HAZARD

MAP INFO

GENERAL

NATIONAL SEISMIC HAZARD MAPPING PROJECT

USGS, Central Region, Geologic Hazards Team
Golden, Colorado

At 56 cities in the Central and Eastern U.S. (CEUS) and 44 cities in the Western U.S. (WUS), the seismic hazard corresponding to a two per cent probability of exceedance in 50 years is deaggregated by magnitude (Mw, or moment magnitude) and by epicentral distance (CEUS) or hypocentral distance (WUS). Hazard with respect to magnitude is binned into intervals of width 0.5 Mw. Hazard with respect to epicentral distance is binned into intervals of 25 km width. The hazard probabililities are deaggregated for the following ground motion parameters: PGA, 1.0, 0.3 and 0.2 second PSA.

Four matrices of per cent contribution to hazard are available at this web site. The matrices are organized with magnitude intervals corresponding to columns and distance intervals corresponding to rows. The first row of numbers gives the upper endpoint of the magnitude interval. For example, the number 6 means that seismic sources with magnitudes in the interval 5.5 < Mw <= 6 are included in hazard calculations for that column. The first column of numbers gives the upper endpoint of the epicentral distance interval. For example, the number 150. means that source-to-station distances in the interval 125 < d <= 150 km are included in the hazard calculations for that row. Missing rows, or gaps in the matrix, correspond to distance ranges for which the greatest per cent contribution to hazard is less than 0.0005, yielding a row of zeros to the level of precision given in the below data.

For the CEUS, the lowest magnitude considered for hazard calculations is MbLg 5.0. This magnitude corresponds to Mw = 4.7 using the Johnston (1996) relationship between the two magnitudes. Thus, for CEUS cities, the interval width for the first column of contribution to hazard is about 0.3 Mw units, rather than 0.5 units, the usual interval width. For the WUS, the lowest magnitude considered for hazard calculations is Mw = 5.0.

An example graph of deaggregated seismic hazard for 1 Hz spectral acceleration for Washington, D.C., for 2% probability of exceedance in 50 years.

To obtain the four hazard matrices, go to the CEUS map or WUS map and click on the city (red dot). The entries are per cent contribution to hazard. They will sum to 100 per cent for each matrix.

The URL of this page is
http://geohazards.cr.usgs.gov/eq/html/deagg.shtml
Web Contact: Nancy Dickman (dickman@usgs.gov)
Last Modified: Thursday, 04-Sep-97 13:49:59 MDT

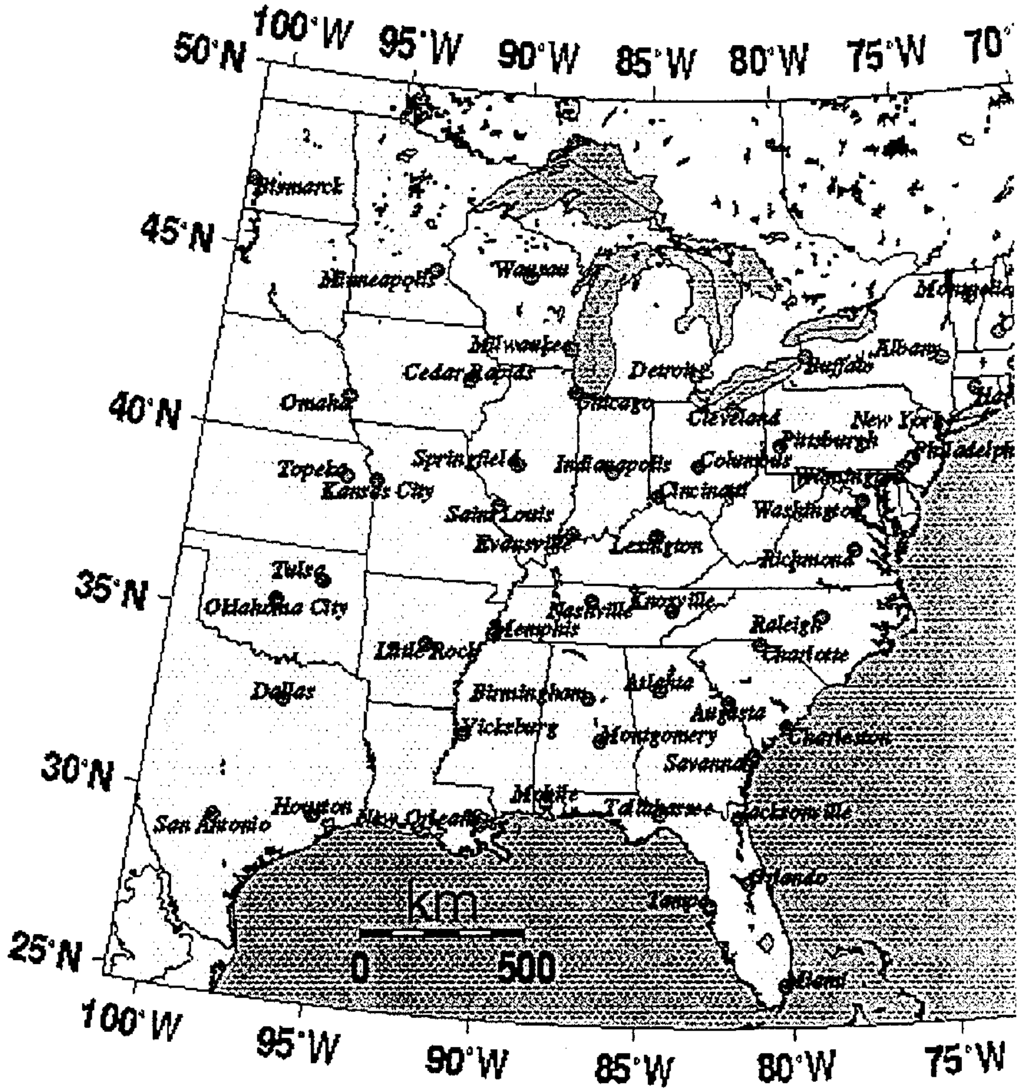
1996 HAZARD MAPS

SEISMIC HAZARD

MAP INFO

GENERAL

CEUS Cities



GMT Jan 30 09:59 Deaggregated Seismic Hazard, Prob. Exceedance 2% 50 yr.

The URL of this page is http://geohazards.cr.usgs.gov/eq/html/ceu
Web Contact: Nancy Dickman (dickman@usgs.gov)
Last Modified: Thursday, 04-Sep-97 13:47:33 MDT

USGS

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USGS, Central Region, Geologic Hazards Team
Golden, Colorado

REVIEW MATERIAL

Please note that the image map on this page is a client-side image map.
YOU WILL NEED A BROWSER WHICH SUPPORTS CLIENT-SIDE IM USE IT!!!
(such as Netscape Navigator 2.0 or Microsoft Internet Explorer 3.0 or the eq

To obtain the four hazard matrices click on the city (red dot). The entries are per hazard. They will sum to 100 per cent for each matrix.

Deaggregated Seismic Hazard PE = 2% in 50 years pga
Boston MA 42.333 deg N 71.083 deg W PGA=0.15820 g

M<=	5.0	5.5	6.0	6.5	7.0	7.5
d<= 25.	14.782	10.319	5.586	2.673	0.972	0.580
50.	6.324	7.939	7.158	5.055	2.338	1.587
75.	0.992	2.138	3.299	3.692	2.387	2.037
100.	0.179	0.583	1.361	2.146	1.788	1.884
125.	0.047	0.211	0.643	1.306	1.286	1.530
150.	0.012	0.072	0.275	0.701	0.824	1.088
175.	0.003	0.021	0.101	0.323	0.461	0.734
200.	0.001	0.006	0.034	0.140	0.240	0.470
225.	0.000	0.002	0.013	0.067	0.137	0.322
250.	0.000	0.001	0.006	0.036	0.087	0.239
275.	0.000	0.000	0.003	0.019	0.054	0.171
300.	0.000	0.000	0.001	0.010	0.033	0.121
325.	0.000	0.000	0.001	0.005	0.020	0.088
350.	0.000	0.000	0.000	0.003	0.013	0.065
375.	0.000	0.000	0.000	0.002	0.008	0.050
400.	0.000	0.000	0.000	0.001	0.005	0.034
425.	0.000	0.000	0.000	0.000	0.003	0.023
450.	0.000	0.000	0.000	0.000	0.002	0.014
475.	0.000	0.000	0.000	0.000	0.001	0.008
500.	0.000	0.000	0.000	0.000	0.000	0.004

Deaggregated Seismic Hazard PE = 2% in 50 years 1.0 hz (1.0 s)
Boston MA 42.333 deg N 71.083 deg W SA= 0.08500 g

M<=	5.0	5.5	6.0	6.5	7.0	7.5
d<= 25.	0.778	2.581	3.391	2.317	0.934	0.571
50.	0.161	1.358	3.656	4.160	2.235	1.572
75.	0.034	0.518	2.283	3.881	2.726	2.262
100.	0.009	0.223	1.439	3.219	2.705	2.527
125.	0.003	0.126	1.044	2.747	2.563	2.493
150.	0.002	0.075	0.739	2.204	2.228	2.222
175.	0.001	0.040	0.466	1.586	1.759	1.945
200.	0.000	0.021	0.281	1.091	1.358	1.650
225.	0.000	0.012	0.189	0.825	1.132	1.501
250.	0.000	0.008	0.145	0.705	1.048	1.501
275.	0.000	0.005	0.109	0.589	0.945	1.449
300.	0.000	0.004	0.083	0.492	0.849	1.390
325.	0.000	0.003	0.064	0.418	0.773	1.355
350.	0.000	0.002	0.050	0.363	0.721	1.354
375.	0.000	0.001	0.041	0.326	0.690	1.387
400.	0.000	0.001	0.030	0.260	0.588	1.261
425.	0.000	0.001	0.022	0.208	0.500	1.139
450.	0.000	0.000	0.014	0.150	0.380	0.922
475.	0.000	0.000	0.010	0.112	0.265	0.681
500.	0.000	0.000	0.007	0.080	0.177	0.482

SOLUTIONS TO EXERCISE NO. 1

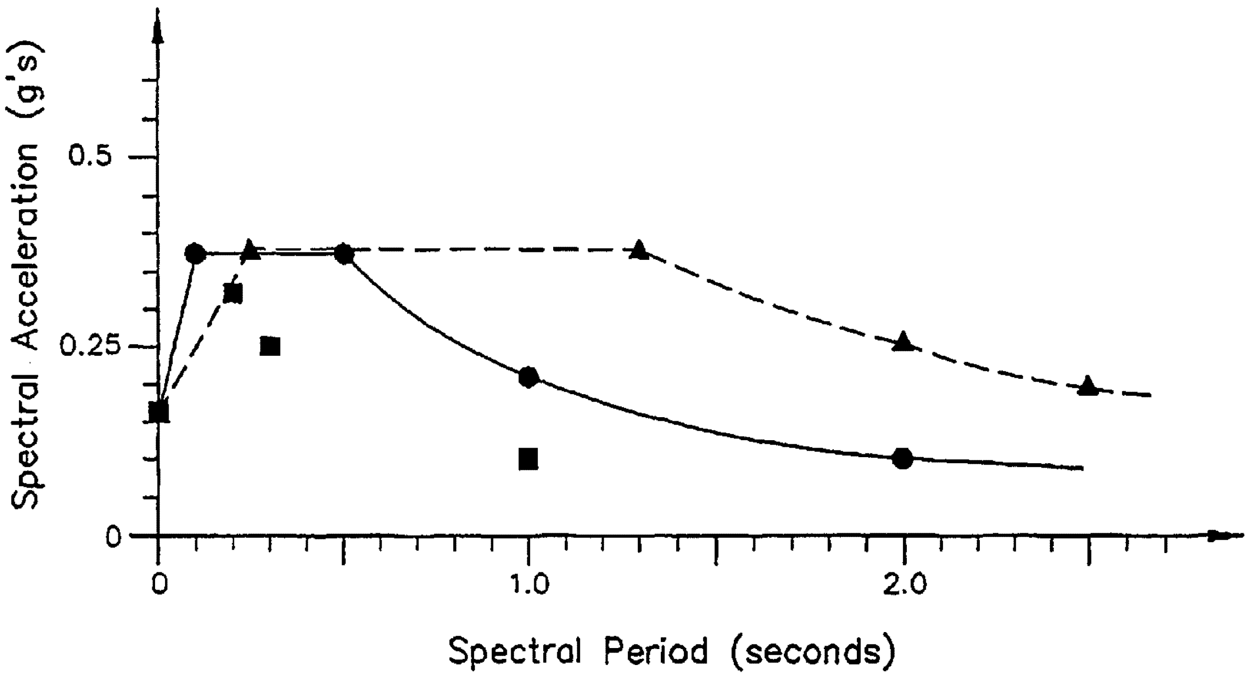
For Firm Ground Site:

1. $PGA_{FF} = 0.16$ g
2A. $Z = 0.15$
2B. Site Class = S_B/S_C (On Boundary)
2C. $C_v = 0.20$ (Interpolate Between Site Classes S_B and S_C)
2D. $T_s = 0.53$ s
2E. $T_o = 0.11$ s

For Soft Ground Site:

4A. $Z = 0.15$
4B. Site Class = S_E
4C. $C_v = 0.50$
4D. $T_s = 1.33$ s
4E. $T_o = 0.27$ s

See Attached Plot of Spectra, USGS Data.



■ USGS – 2% in 50 years (S_B – S_C Boundary)

● 1997 UBC Firm Ground (S_B – S_C Boundary)

▲ 1997 UBC Soft Soil (S_E)

From Deaggregated USGS Hazard Data:

6A. $M = 6.5$ (Subjective)
6B. $D = 75$ km

STUDENT EXERCISE NO. 2A

Computation of Standardized and Normalized SPT Blow Count Number, $(N_1)_{60}$

Objective:
Derive Normalized and Standardized SPT Blow Counts, $(N_1)_{60}$, at 6 m Below Ground Surface at a Site Shown in Figure S2A-1.

- Soil Properties and Uncorrected Field SPT-N Values Are Shown in Figure S2A-1.
- SPTs Were Performed Using a Donut Hammer with Rope and Pulley.
- Standard SPT Sampler (with Room for Liners) Was Used without Liners.
- Borehole Diameter: 100 mm

Source Materials:
Reference Manual Part I: Section 5.4.2 Equations 5-6, 5-7, 5-8, 5-10, and 5-11, and Tables 5-2 and 5-3.

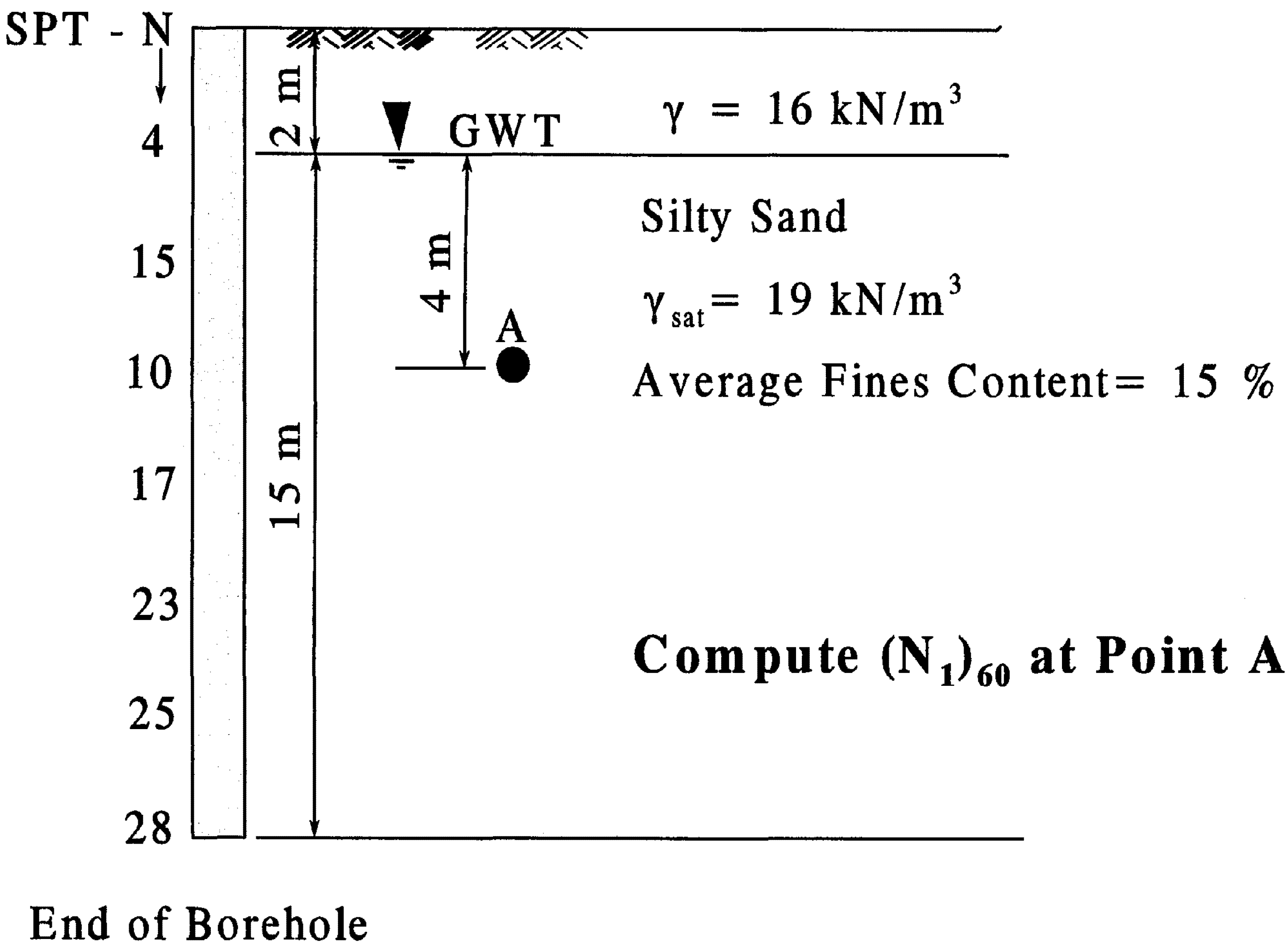


Figure S2A-1: Soil Profile and Field SPT-N Values

Compute $(N_1)_{60}$

1. Use Equations 5-6 and 5-11

   $$(N_1)_{60} = C_N \cdot N_{60} = C_N C_{60} N$$

2. Standardization

   Use Equations 5-7 and 5-8 and Tables 5-2 and 5-3.

   $$C_{60} = C_{HT} C_{HW} C_{SS} C_{RL} C_{BD}$$

   C_{60} = ______________

3. Normalization

   Compute Effective Overburden Pressure, σ_v'

   σ_v' = ___________

Compute Overburden Correction Factor Using Equation 5-10

$$C_N = 9.79\left(\frac{1}{\sigma_v'}\right)^{1/2}$$

C_N = ______________

4. Compute $(N_1)_{60} = C_N C_{60} N$

$(N_1)_{60}$ = ______________

TABLE 5-3
CORRECTION FACTORS FOR NON-STANDARD SPT PROCEDURE AND EQUIPMENT
(Richardson, *et al.*, 1995; Youd and Idriss, 1997)

Correction for	Correction Factor	Reference
Nonstandard Hammer Type (DH = doughnut hammer; ER = energy ratio)	C_{HT}=0.75 for DH with rope and pully C_{HT}=1.33 for DH with trip/auto & ER=80	Seed, *et al.* (1985)
Nonstandard Hammer Weight or Height of Fall (H = height of fall in mm; W = hammer weight in kg)	$C_{HW} = \dfrac{H \cdot W}{63.5 \cdot 762}$	calculated per Seed, *et al.* (1985)
Nonstandard Sampler Setup (standard samples with room for liners, but used without liners)	C_{SS} = 1.10 for loose sand C_{SS} = 1.20 for dense sand	Seed, *et al.* (1985)
Nonstandard Sampler Setup (standard samples with room for liners, and liners are used)	C_{SS} = 0.90 for loose sand C_{SS} = 0.80 for dense sand	Skempton (1986)
Short Rod Length	C_{RL} = 0.75 for rod length 0-4 m C_{RL} = 0.85 for rod length 4-6 m C_{RL} = 0.95 for rod length 6-10 m C_{RL} = 1.0 for rod length 10-30 m C_{RL} < 1.0 for rod length > 30 m	Seed, *et al.* (1983); Youd and Idriss (1997)
Nonstandard Borehole Diameter	C_{BD} = 1.05 for 150 mm borehole diameter C_{BD} = 1.15 for 200 mm borehole diameter	Skempton (1986)

Notes: N = Uncorrected SPT blow count.
$C_{60} = C_{HT} \cdot C_{HW} \cdot C_{SS} \cdot C_{RL} \cdot C_{BD}$
$N_{60} = N \cdot C_{60}$
C_N = Correction factor for overburden pressure.
$(N_1)_{60} = C_N \cdot N_{60} = C_N \cdot C_{60} \cdot N$

SOLUTIONS TO EXERCISE NO. 2A

1. $(N_1)_{60} = C_N C_{60} N$

2. Standardization

$C_{60} = C_{HT}\, C_{HW}\, C_{SS}^{(1)}\, C_{RL}^{(2)}\, C_{BD}$ (Equations 5-7 and 5-8)

$C_{60} = 0.75 \text{ x } 1 \text{ x } 1.1 \text{ x } 0.95 \text{ x } 1$

$C_{60} = \underline{\quad 0.78 \quad}$

Note:
(1) For loose sand (N= 10), use C_{SS}= 1.10 (Table 5-3)
(2) For rod length greater than 6 m, use C_{RL}= 0.95 (Table 5-3)

3. Normalization

Compute Effective Overburden Pressure, $\sigma_v^/$

$$\sigma_v^/ = \gamma \times 2\text{m} + (\gamma_{sat} - \gamma_w) \times 4\text{m}$$

$$\sigma_v^/ = 16 \times 2\text{m} + (19 - 9.81) \times 4\text{m}$$

$$\sigma_v^/ = \underline{\quad 69 \text{ kPa} \quad}$$

Compute Overburden Correction Factor, C_N

$$C_N = 9.79 \left(\frac{1}{69} \right)^{1/2}$$ (Equation 5-10)

C_N = $\underline{\quad 1.18 \quad}$

4. Compute $(N_1)_{60} = C_N C_{60} N$

$(N_1)_{60} = 1.18 \times 0.78 \times 10$

$(N_1)_{60} \approx$ $\underline{\quad 9 \quad}$

STUDENT EXERCISE NO. 2B

Development of Soil Profile Using CPT Data.

Objective:
Develop Soil Stratigraphy Using Cone Penetration Data Shown in Figure S2B-1. Estimate the Equivalent SPT-N Values Using CPT-SPT Correlation Charts.

Source Materials:
Reference Manual Part I: Sections 5.4.2, Figures 5-6 and 5-7.

DEVELOP SOIL PROFILE USING CPT DATA

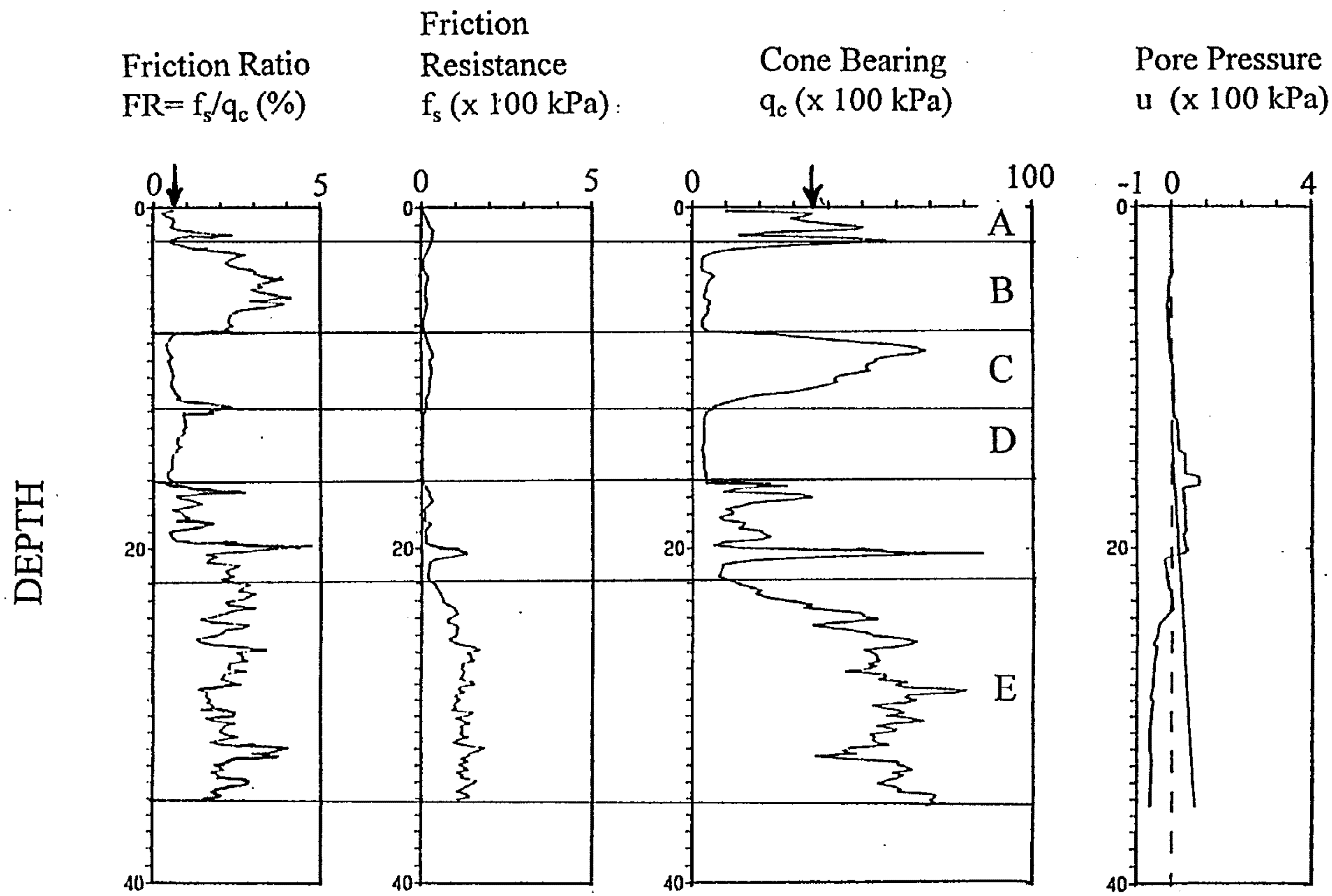


Figure S2B-1: CPT Data Profile

Stratum	Effective Vertical Stress (kPa)	Mean Grain Size (mm)	Cone Resistance q_c (x 100 kPa)	Normalized Cone Resistance q_{c1} (x 100 kPa)	Friction Ratio FR (%)	Soil Type	Equivalent SPT-N Value
A	6 kPa	0.2 mm	35	88	0.6	Sand to Silty Sand	7
B	25 kPa	0.003 mm					
C	40 kPa	0.2 mm					
D	50 kPa	0.003 mm					
E	85 kPa	0.2 mm					

Table S2B-1: Effective Vertical Stress and Mean Grain Size for Each Stratum

For Stratum A:

1. Determine the normalized cone resistance q_{C1}.

   $q_C = 3{,}500$ kPa (from Figure S2B-1)

   $\sigma'_v = 6$ kPa (form Table S2B-1)

   (Using Figure 5-6)

   $q_{C1} = q_C\,(3.5 - 1.25 \log_{10} \sigma'_v)$

   $= 88$ (x 100 kPa)

2. Determine the soil behavior type.

   $q_{C1} = 88$ (x 100 kPa)

   Friction Ration = 0.6% (from Figure S2B-1)

   $\Rightarrow$ Stratum A is **Sand to Silty Sand** (using Figure 5-6)

3. Determine the equivalent SPT-N value.

Mean Grain Size $D_{50} = 0.2$ mm (from Table S2B-1)

$q_c / N = 4.7$ (using Figure 5-7)

$\Rightarrow \quad N = q_c / 4.7 = 35 / 4.7 \cong 7$

(Note: q_c in bars. 1 bar = 100 kPa)

Determine Soil Behavior Types and the Equivalent SPT-N Values for Strata B, C, D, and E.

$$q_{C1} = q_C\,(3.5 - 1.25 \log_{10} \sigma_v')$$
$$\text{with } \sigma_v',\ q_C,\ q_{C1} \text{ in kPa.}$$

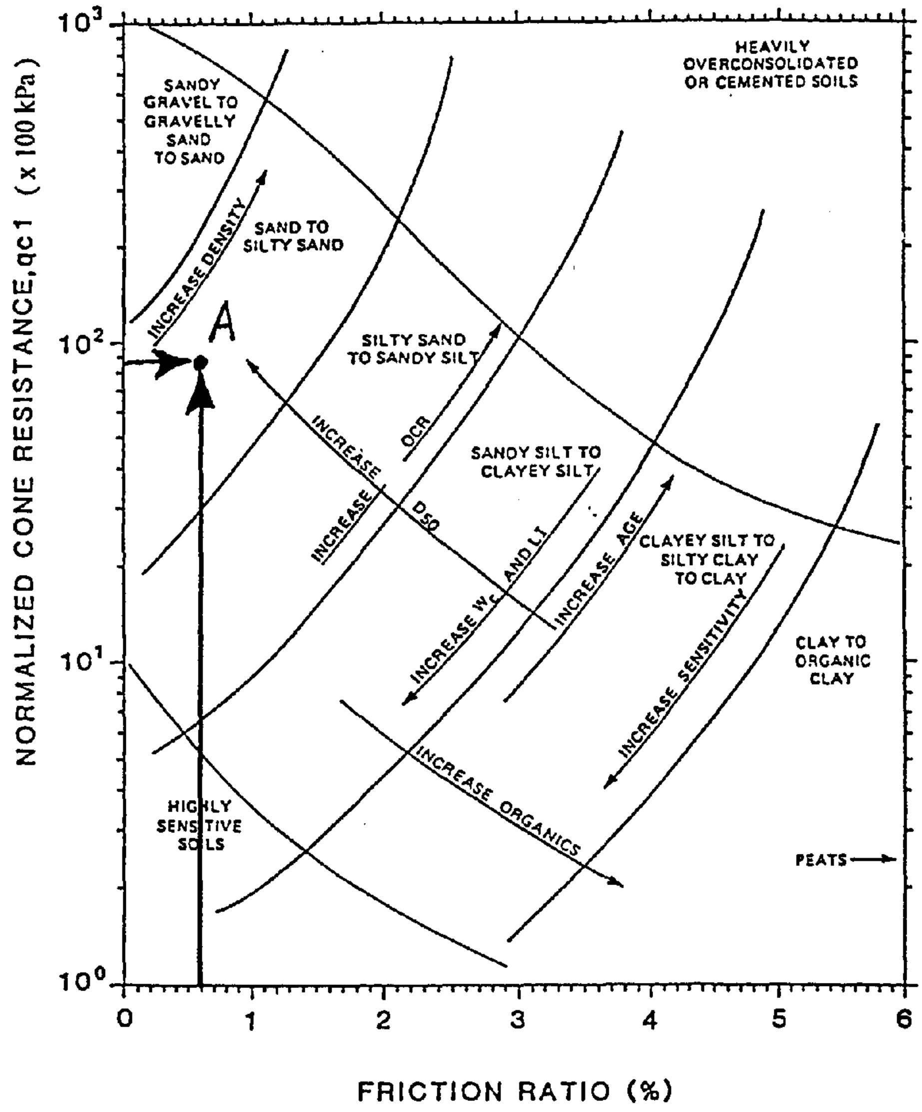


Figure 5-6: Soil Behavior Type Classification Chart Based on the CPT. (Douglas, 1984, 1981, reprinted from FHWA-SA-91-043, 1992.)

BASED ON ENERGY RATIO OF 60% (N_{60})

q_c in bars (1 bar = 100 kPa)

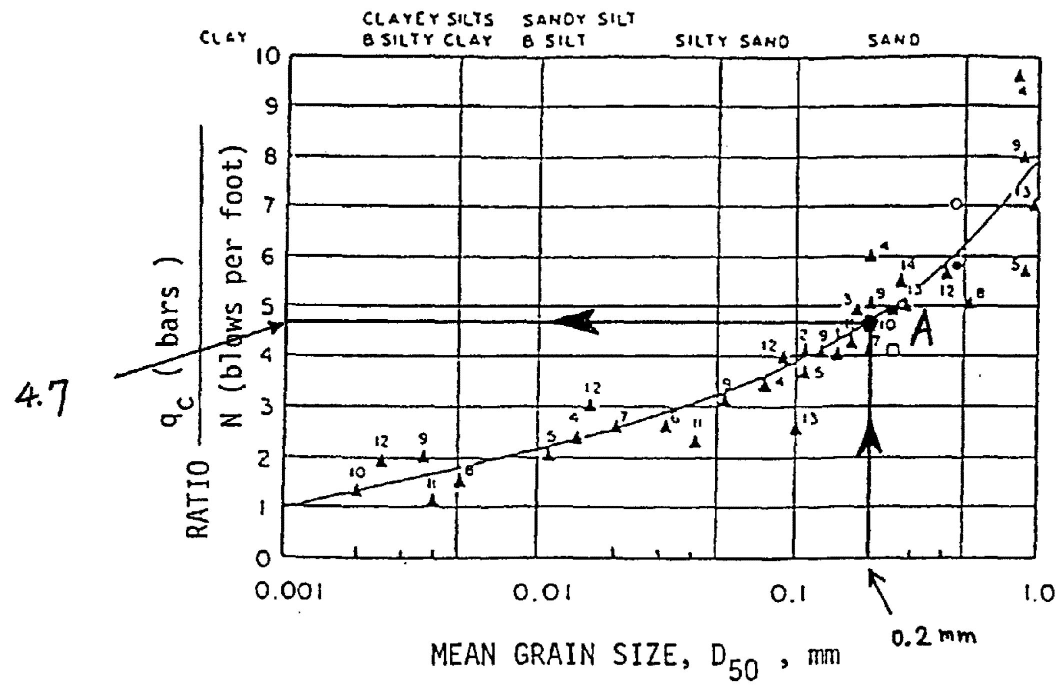


Figure 5-7: CPT-SPT Correlation Chart. (Robertson et al., 1983, reprinted from FHWA-SA-91-043, 1992.)

SOLUTIONS TO EXERCISE 2B

Stratum	Effective Vertical Stress (kPa)	Mean Grain Size (mm)	Cone Resistance q_c (x 100 kPa)	Normalized Cone Resistance q_{c1} (x 100 kPa)	Friction Ratio FR (%)	Soil Type	Equivalent SPT-N Value
A	6 kPa	0.2 mm	35	88	0.6	Sand to Silty Sand	7
B	25 kPa	0.003 mm	4	7	3.5	Silty Clay	3
C	40 kPa	0.2 mm	45	67	0.8	Sand to Silty Sand	9
D	50 kPa	0.003 mm	3.5	5	1	Sensitive Silt	2
E	85 kPa	0.2 mm	55	60	2	Silty Sand to Sandy Silt	12

$$q_{C1} = q_C\,(3.5 - 1.25\log_{10}\sigma_v')$$
$$\text{with } \sigma_v',\ q_C,\ q_{C1} \text{ in kPa.}$$

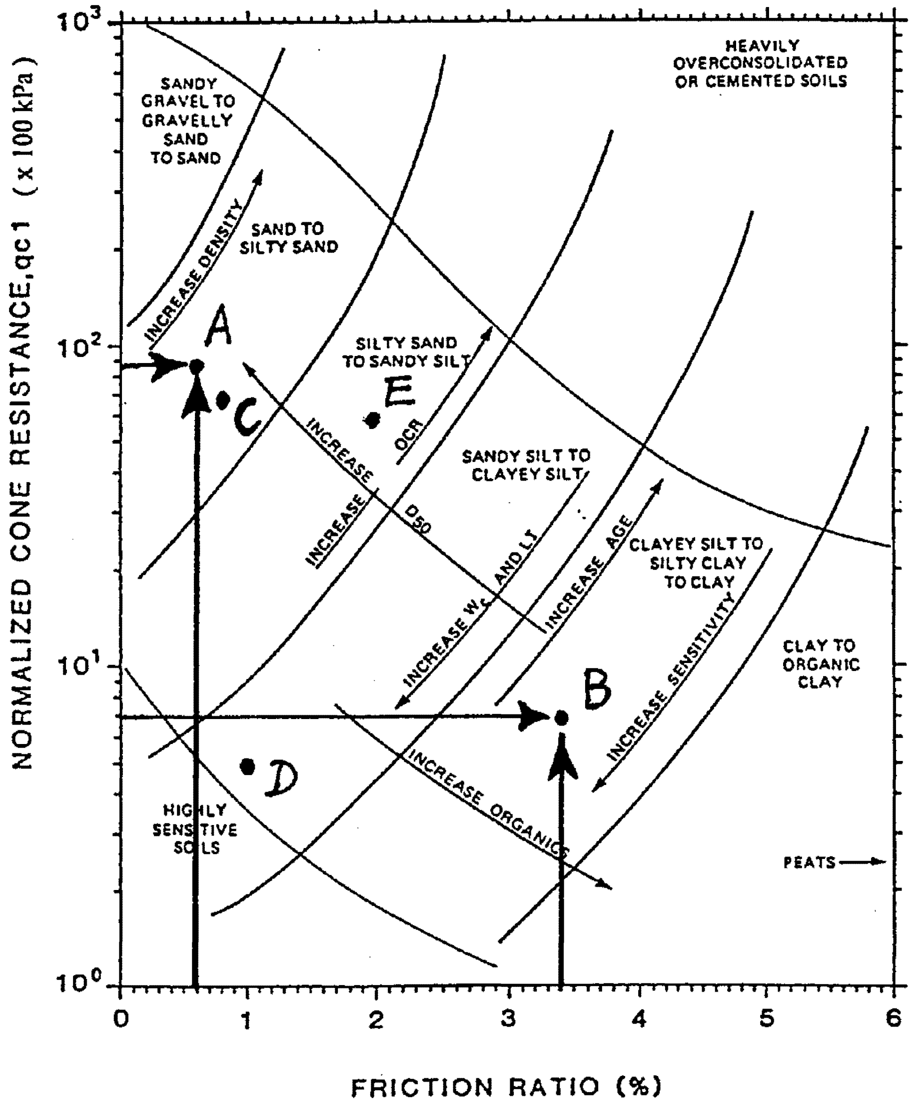


Figure 5-6: Soil Behavior Type Classification Chart Based on the CPT. (Douglas, 1984, 1981, reprinted from FHWA-SA-91-043, 1992.)

BASED ON ENERGY RATIO OF 60% (N_{60})

q_c in bars (1 bar = 100 kPa)

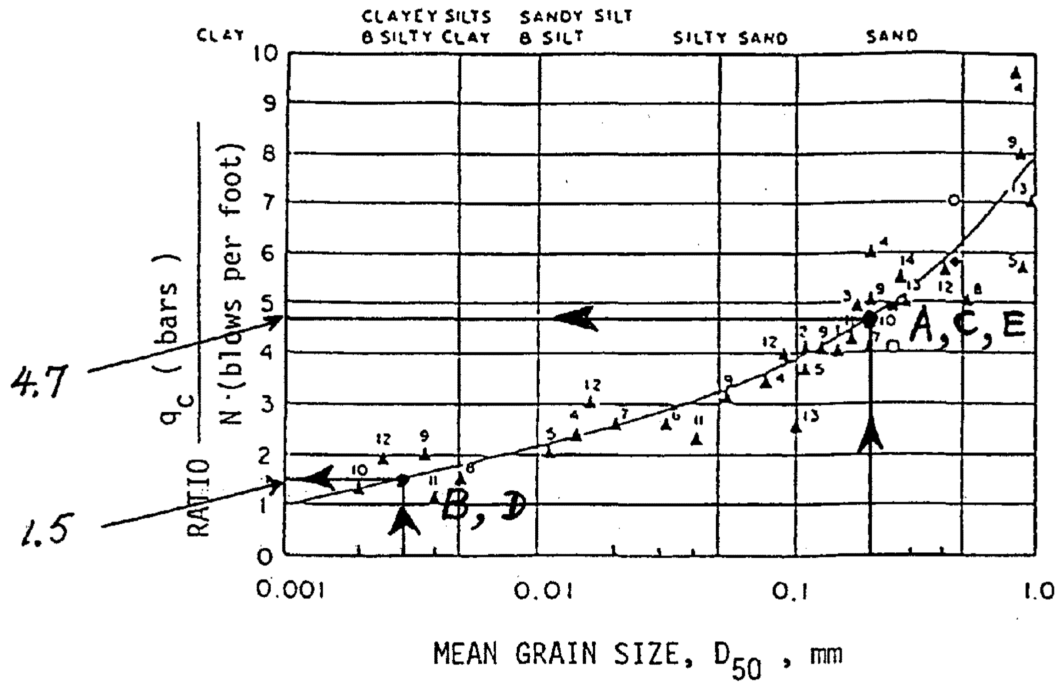


Figure 5-7: CPT-SPT Correlation Chart. (Robertson et al., 1983, reprinted from FHWA-SA-91-043, 1992.)

STUDENT EXERCISE NO. 3

G_{max} Derivation by Empirical Correlations.

Objective:

Evaluate Dynamic Material Properties for Sand at 25 meters with a Void Ratio of 0.5 and the Water Table at 10 m Using Empirical Correlations. Assume a Moist Unit Weight of 18 kN/m^3 and a Saturated Unit Weight of 20.5 kN/m^3. Assume a Friction Angle $\phi = 30^\circ$ for the Sand.

Source Materials:

Reference Manual Part I: Table 5-5, Figures 5-12, 5-13, 5-14, Equations 5-11, 5-12, 5-13, and 5-14.

1. Calculate Mean Effective Stress Using Equation 5-12:

$\sigma_v' =$ ____________

$K_o =$ ____________

$\sigma_m' =$ ____________

2. Assign $(K_2)_{max}$ from Figure 5-12:

$(K_2)_{max}$ = ____________

3. Calculate G_{max} from Equation 5-14:

G_{max} = ____________

4. Assign modulus reduction and damping curves from Figures 5-12, 5-13, and 5-14:

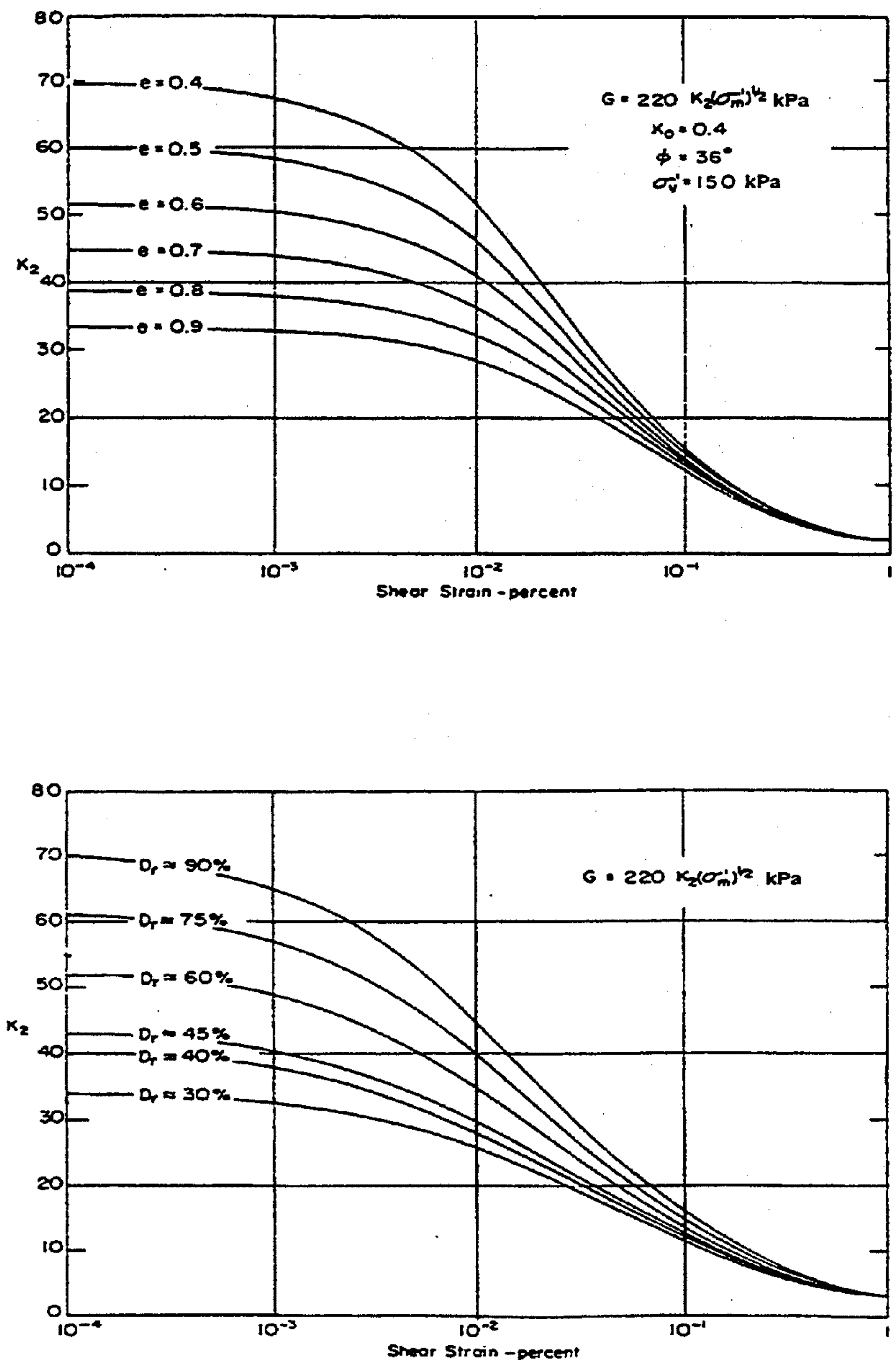


Figure 5-12: Shear Modulus Reduction Curves for Sands. (Seed and Idriss, 1970, reprinted by permission of ASCE)

TABLE 5-5
CORRELATIONS FOR ESTIMATING INITIAL SHEAR MODULUS

Reference	Correlation	Units	Limitation
Seed, *et al.* (1984)	$G_{max} = 220\ (K_2)_{max}\ (\sigma'_m)^{½}$ $(K_2)_{max} \approx 20(N_1)_{60}^{1/3}$	kPa	$(K_2)_{max} \approx$ 30 for very loose sands and 75 for very dense sands; $\approx$ 80-180 for dense well graded gravels; Limited to cohesionless soils
Imai and Tonouchi (1982)	$G_{max} = 15{,}560\ N_{60}^{0.68}$	kPa	Limited to cohesionless soils
Hardin (1978)	$G_{max} = \dfrac{625}{(0.3 + 0.7\, e_o^2)} (P_a \cdot \sigma'_m)^{0.5} OCR^{\,k}$	kPa(1) (3)	Limited to cohesive soils P_a = atmospheric pressure
Jamiolkowski, *et al.* (1991)	$G_{max} = \dfrac{625}{e_o^{1.3}} (P_a \cdot \sigma'_m)^{0.5}\ OCR^{\,k}$	kPa(1) (3)	Limited to cohesive soils P_a = atmospheric pressure
Mayne and Rix (1993)	$G_{max} = 99.5 (P_a)^{0.305} (q_c)^{0.695} / (e_o)^{1.13}$	kPa(2)	Limited to cohesive soils P_a = atmospheric pressure

Notes: (1) P_a and σ'_m in kPa
(2) P_a and q_c in kPa
(3) The parameter k is related to the plasticity index, PI, as follows:

PI	k
0	0
20	0.18
40	0.30
60	0.41
80	0.48
>100	0.50

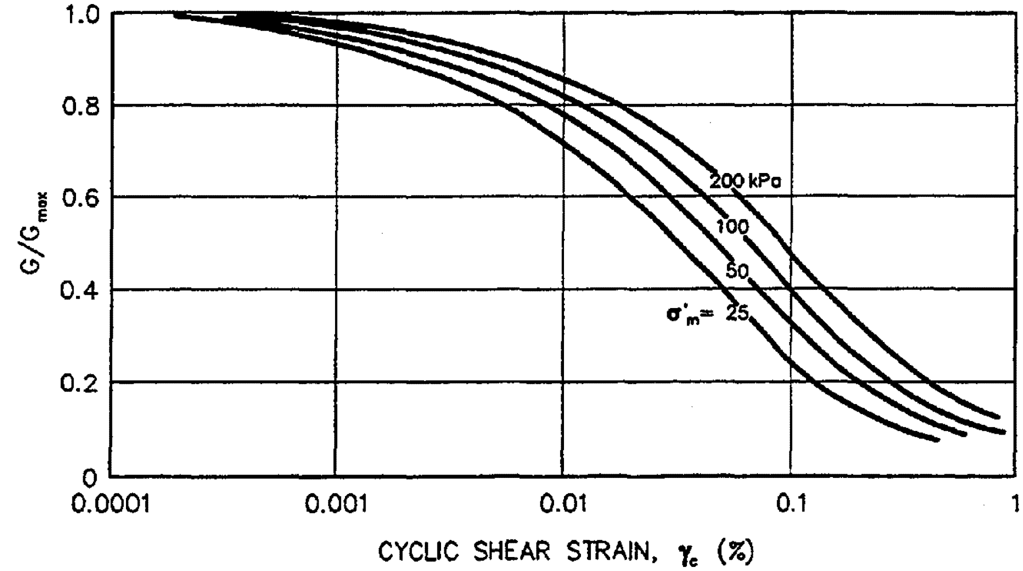


Figure 5-13: Shear Modulus Reduction Curves for Sands. (Iwasaki, *et al.*, 1978, reprinted by permission of Japanese Society of Soil Mechanics and Foundation Engineering)

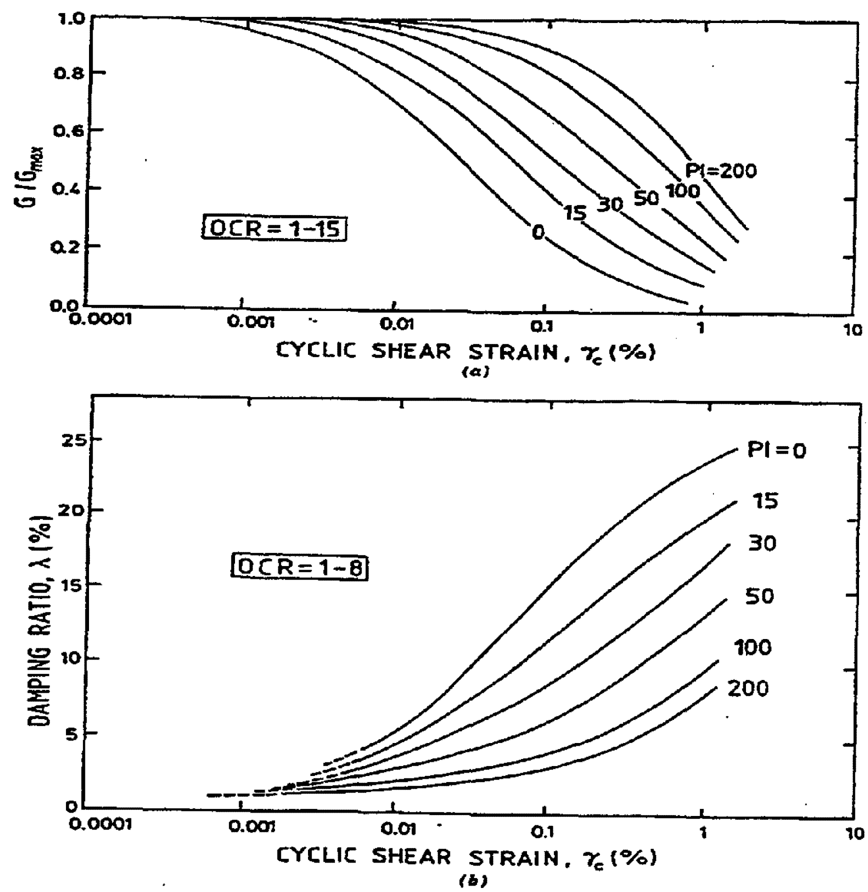


Figure 5-14: Shear Modulus Reduction and Damping Ratio as a Function of Shear Strain and Soil Plasticity Index. (Vucetic and Dobry, 1991, reprinted by permission of ASCE)

SOLUTIONS TO EXERCISE NO. 3

1. $\sigma_v' = (10 \times 18) + (15 \times 10.7) = 340.5$ kPa

   $K_o = 0.5$ (See note 1)

   Use Equation 5-12

   $$\sigma_m' = \left[\frac{1+2k_o}{3}\right] \times \sigma_v' = 227 \text{ kPa}$$

2. From Figure 5-12, for e = 0.5 (see note 2), $(K_2)_{max} = 60$

3. Use Equation 5-14

   $G_{max} = 220(K_2)_{max}(\sigma_m')^{1/2} = 20(60)(227)^{0.5} = 198{,}878$ kPa
   $= 200$ MPa

4. Use either e = 0.5 curve from Figure 5-12, $\sigma_m' = 200$ kPa curve from Figure 5-13, or PI = 0 curve from Figure 5-14 for modulus reduction.

   Use PI = 0 curve from Figure 5-14 for damping

Notes: 1) K_o can be derived as $(1 - \sin\phi)$

2) Void ratio can be derived from unit weight using $\gamma = \frac{G_s\,\gamma_w}{1+e}$, where G_s = Specific Gravity

STUDENT EXERCISE NO. 4

Simplified Site Response Analysis.

Objective:

For Site Profile in Figure S4-1, with a Free-Field Peak Horizontal Ground Acceleration on Firm Ground Equal to 0.16 g, Evaluate:

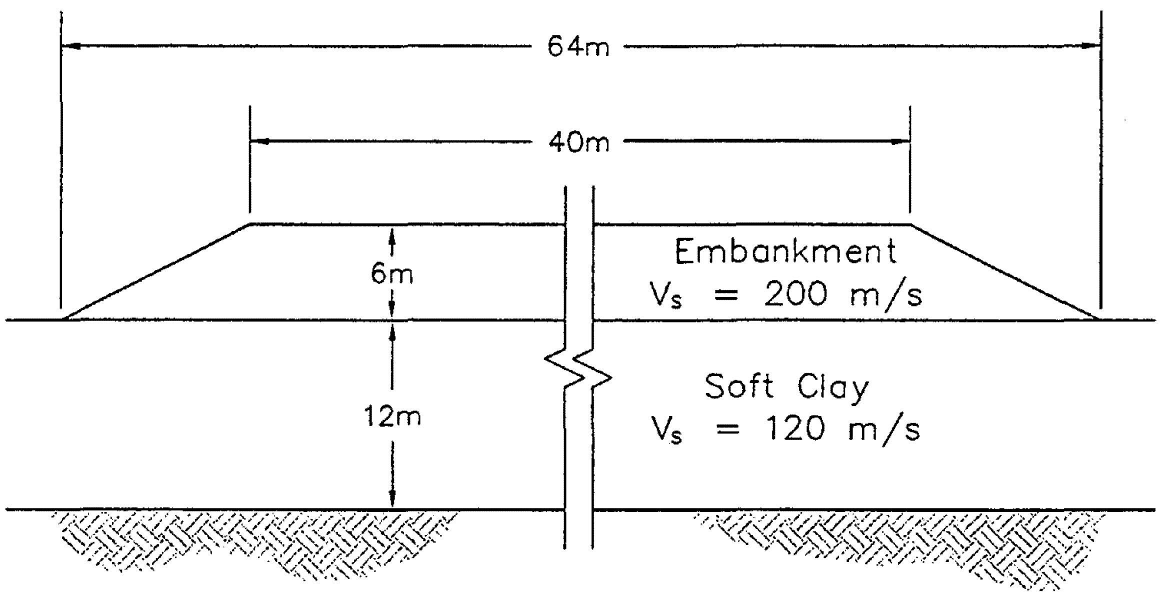
- Free Field Peak Ground Acceleration;
- Peak Acceleration at Top of Embankment;
- Fundamental Period of Clay Deposit in Free Field;
- Fundamental Period of Embankment.

Source Materials:

Reference Manual Part I: Figures 4-19, 6-3, 6-4, and Equation 4-5.

1. Establish Free-Field Peak Ground Acceleration at Top of Clay From Figure 6-3:

   PGA_{FF} = ____________



EXAMPLE

Figure S4-1: Soil Profile

2. Evaluate Peak Acceleration at Top of Embankment from Figure 6-4:

PGA_{EMB} = ____________

3. Evaluate Fundamental Period of Clay Layer, T_o (= $1/f_o$), from Equation 4-5:

$$(T_o)_{FF} = \frac{4H}{V_s}$$

$(T_o)_{FF}$ = ____________

4. Evaluate Fundamental Period of Embankment from Figure 4-19:

A. $H =$ ______________

B. $h =$ ______________

C. $\lambda = \frac{h}{H} =$ ______________

D. $a_n =$ ______________

E. $(T_o)_{EMB} = \frac{a_n H}{V_s} =$ ______________

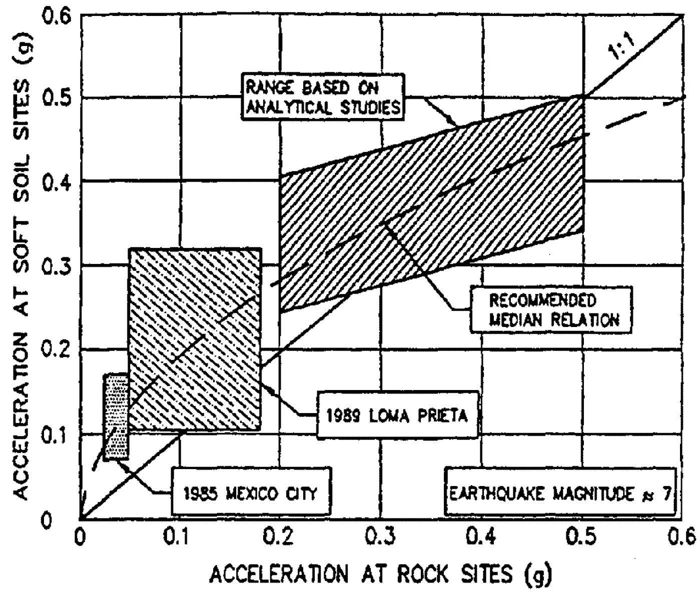


Figure 6-3: Relationship Between PHGA on Rock and on Soft Soil Sites. (Idriss, 1990)

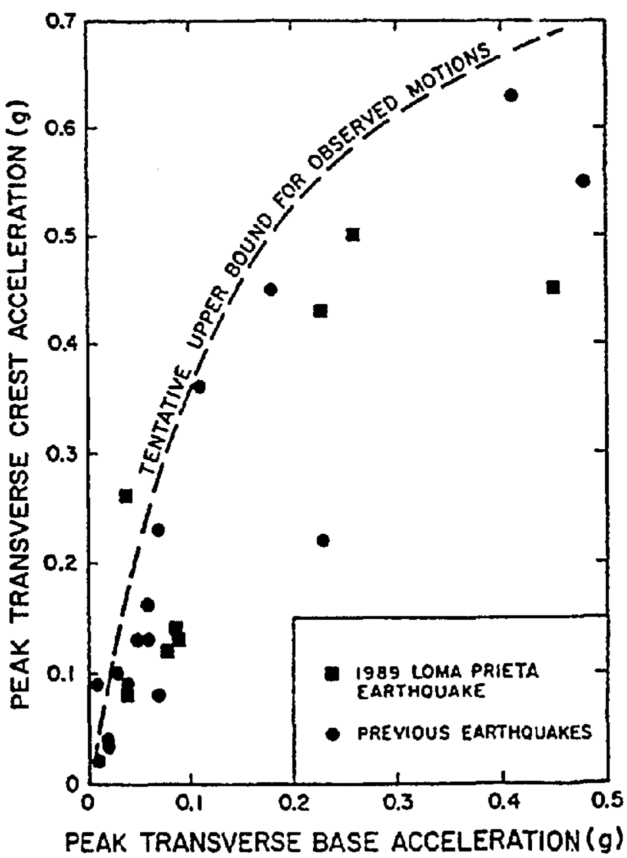
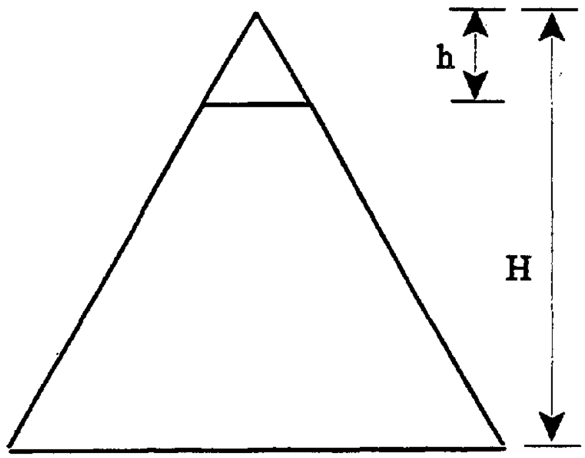


Figure 6-4: Comparisons of Peak Base and Crest Accelerations Recorded at Earth Dams. (Harder, 1991)



$$\lambda = \frac{h}{H} \qquad f_e = \frac{(V_s)_{avg}}{a_n H}$$

f_e = Fundamental Frequency
H-h = Height of the Dam/Embankment
V_S = Shear Wave Velocity
a_n = coefficient

λ	a_n
0.00	2.405
0.03	2.409
0.05	2.416
0.10	2.448
0.15	2.501
0.20	2.574
0.25	2.668
0.30	2.786
0.35	2.930
0.40	3.107
0.45	3.323
0.50	3.588
1.00	4.0

Note: For $0.5 \leq \lambda \leq 1.0$, a_n may be derived by linear interpolation from $a_n = 3.6$ for $\lambda = 0.5$ to $a_n = 4.0$ for $\lambda = 1.0$.

Figure 4-19: Fundamental Frequency of Trapezoidal Dam/Embankment

SOLUTIONS TO EXERCISE NO. 4

1. $PGA_{FF} = 0.26$ g (see attached figure)
2. $PGA_{EMB} = 0.56$ g (see attached figure)
3. $(T_o)_{FF} = \frac{4\,H}{V_s} = \frac{4 \times 12}{120} = 0.4$ s

4A. $H = 16$ m

4B. $h = 10$ m

4C. $\frac{h}{H} = \frac{10}{16} = 0.625$

4D. $a_n = \frac{0.625 - 0.5}{1 - 0.5}(4 - 3.588) + 3.588 = 3.69$

4E. $(T_o)_{EMB} = \frac{3.69 \times 6}{200} = 0.11$ s

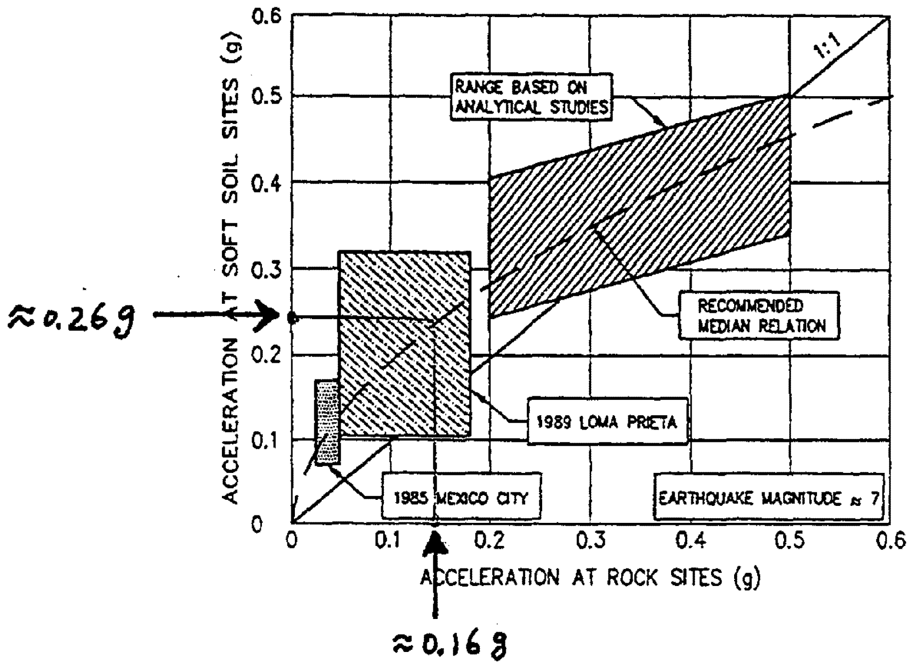


Figure 6-3: Relationship Between PHGA on Rock and on Soft Soil Sites. (Idriss, 1990)

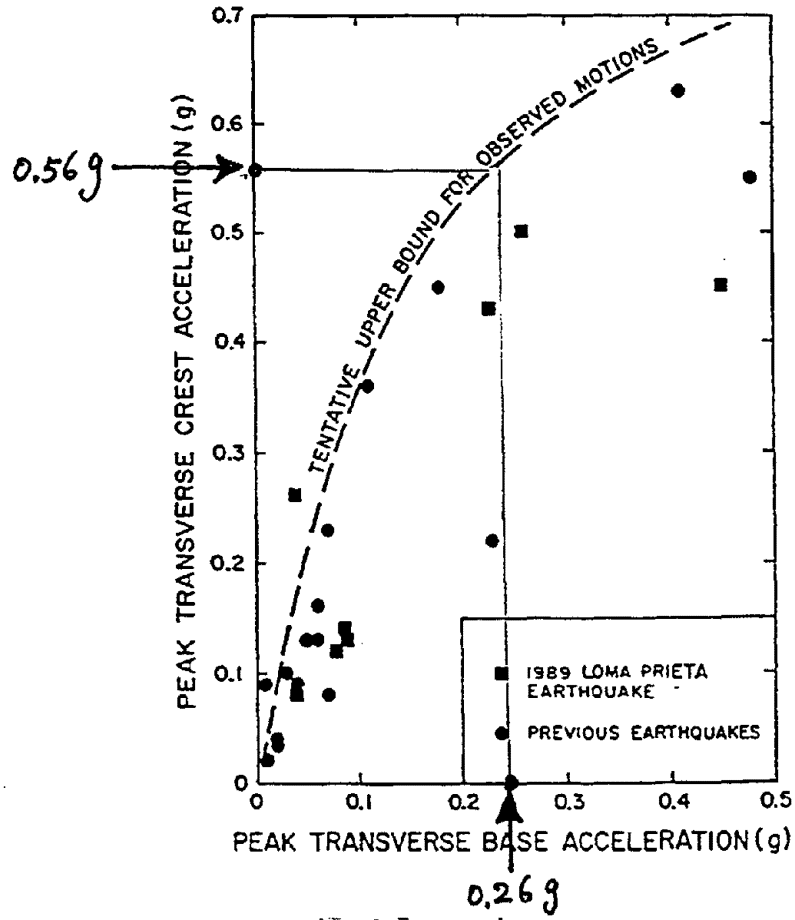


Figure 6-4: Comparisons of Peak Base and Crest Accelerations Recorded at Earth Dams. (Harder, 1991)

STUDENT EXERCISE NO. 5

Preparation of SHAKE Input.

Objective:

Develop Input Data for SHAKE Analysis of Soil Profile Shown in Figure S5-1 Using Soil Data Provided in Table S5-1.

Source Materials:

Reference Manual Part I: Figures 4-19, 5-12, 5-13, 5-14, Table 5-5, and Equations 5-2, 5-3, and 5-12.

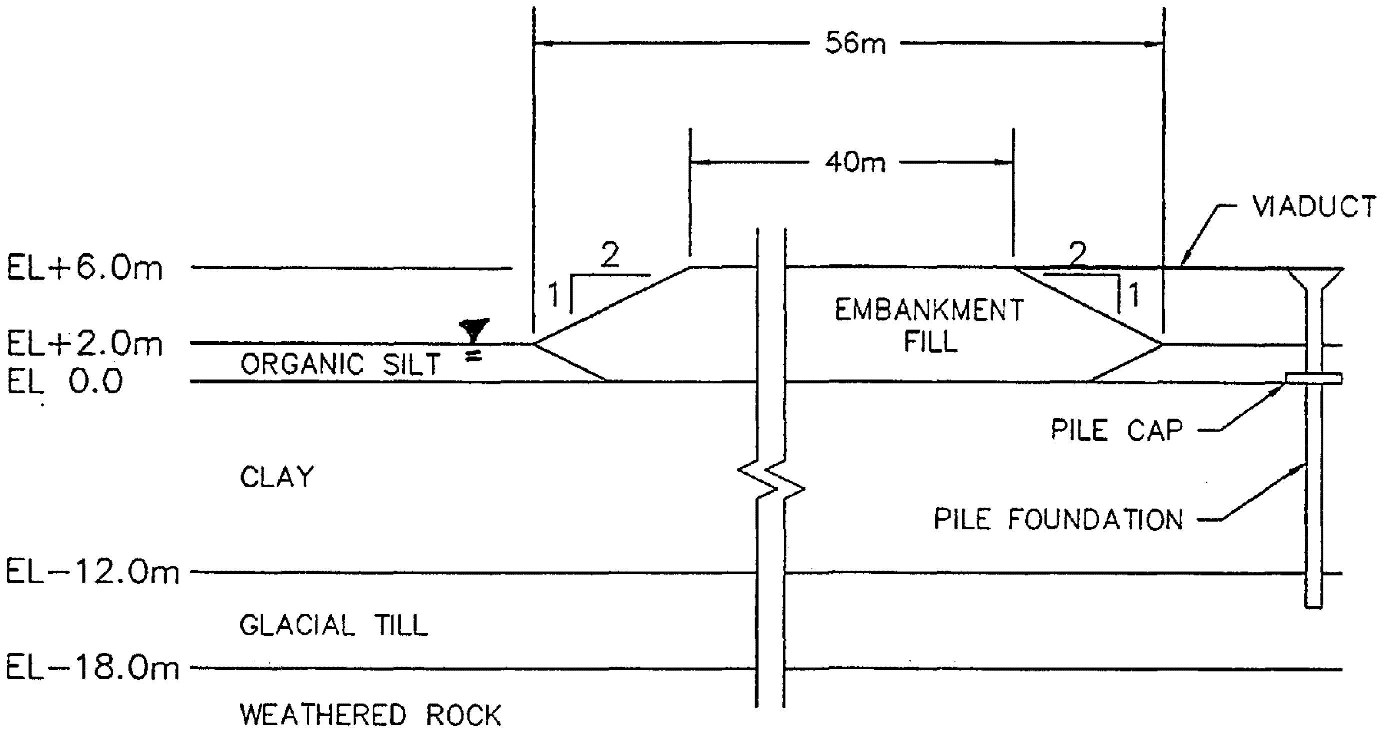
Reference Manual Part II: Figures 4-3a through 4-10.

1. Evaluate Properties at Center of Silt

   A. Evaluate Mean Normal Effective Stress Using Equation 5-12

      $\sigma_v' =$ ______________

      $\sigma_m' =$ ______________



EXAMPLE

Figure S5-1: Subsurface Profile

TABLE S5-1
SUMMARY OF AVAILABLE INFORMATION

	Unit Weight (kN/m^3)	PI (%)	OCR	$(N_1)_{60}$	D_r (%)	e_o
Embankment Fill	19.5	0	—	—	75	—
Organic Silt	12.0	50	1	—	—	3
Clay	16.0	15	1	—	—	1 - 1.2
Glacial Till	20.5	5	> 10	75	—	—
Weathered Rock	21.2	—	—	—	—	—

B. Evaluate Small Strain Shear Modulus Using Table 5-5 (Jamiolkowski, 1991):

G_{max} = ____________

C. Evaluate Shear Wave Velocity Using Equations 5-2 and 5-3:

V_s = ____________

2. Evaluate Properties of Clay Layer:

A. Evaluate Effective Stresses at Top and Bottom of Clay

$(\sigma_v')_{TOP}$ = ____________	$(\sigma_v')_{BOTTOM}$ = ____________
$(\sigma_m')_{TOP}$ = ____________	$(\sigma_m')_{BOTTOM}$ = ____________

B. Evaluate G_{max} at Top and Bottom of Clay

$(G_{max})_{TOP}$ = ______________ $(G_{max})_{BOTTOM}$ = ______________

C. Evaluate Shear Wave Velocity at Top and Bottom of Clay

$(V_s)_{TOP}$ = ______________ $(V_s)_{BOTTOM}$ = ______________

3. Evaluate Properties in Embankment 1 m from Top (Top) and 1 m Above Clay (Bottom)

A. Evaluate Effective Stresses

$(\sigma_v')_{TOP}$ = ______________ $(\sigma_v')_{BOTTOM}$ = ______________

$(\sigma_m')_{TOP}$ = ______________ $(\sigma_m')_{BOTTOM}$ = ______________

B. Evaluate Small Strain Modulus Using Table 5-5 (Seed, et. al, 1984) and Figure 5-12

$(G_{max})_{TOP}$ = ______________ $(G_{max})_{BOTTOM}$ = ______________

C. Evaluate Shear Wave Velocity

$(V_s)_{TOP}$ = ______________ $(V_s)_{BOTTOM}$ = ______________

4. Evaluate Properties in Center of Till

A. Evaluate Effective Stresses

σ_v' = ______________

σ_m' = ______________

B. Evaluate Small Strain Modulus Using Table 5-5 (Imai and Tonouchi, 1982)

G_{max} = ______________

C. Evaluate Shear Wave Velocity

V_s = ____________

5. Assign Modulus Reduction and Damping Curves from Figures 5-12, 5-13, and 5-14

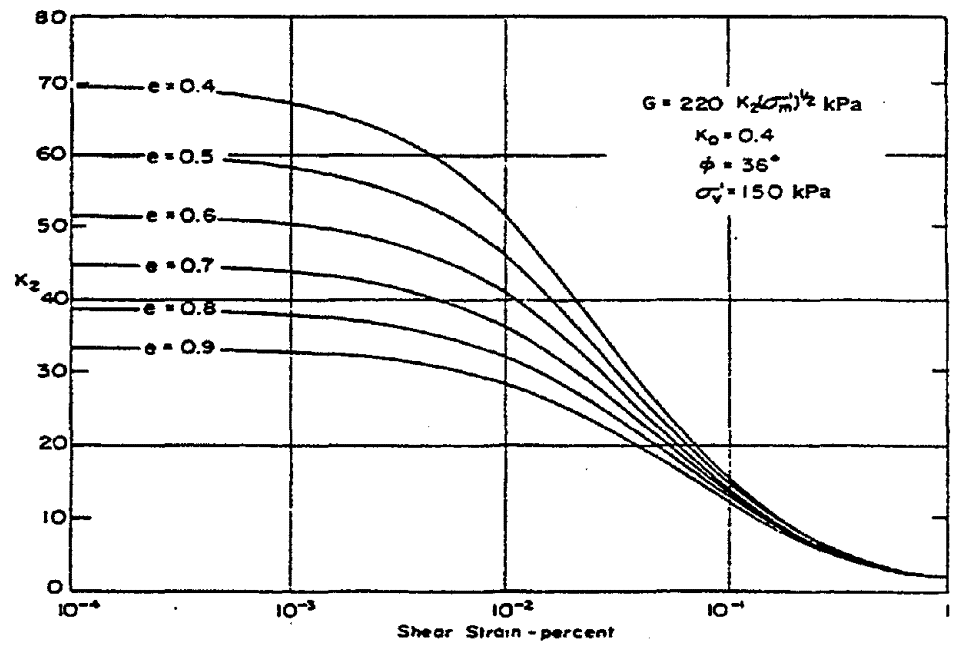
Soil	**Figure Number**	**Curve**
Silt		
Clay		
Embankment		
Till		

6. Calculate Fundamental Period of Clay Layer Beneath the Embankment From Figure 4-19

A. $(V_s)_{AVE}$ = ______________

B. a_n = ______________

C. T_o = ______________



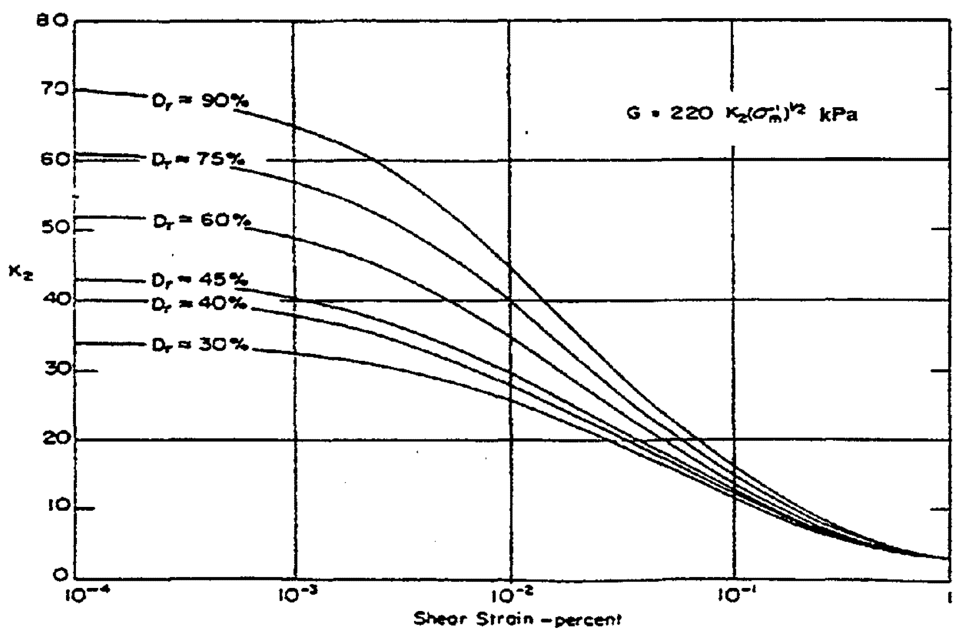


Figure 5-12: Shear Modulus Reduction Curves for Sands. (Seed and Idriss, 1970, reprinted by permission of ASCE)

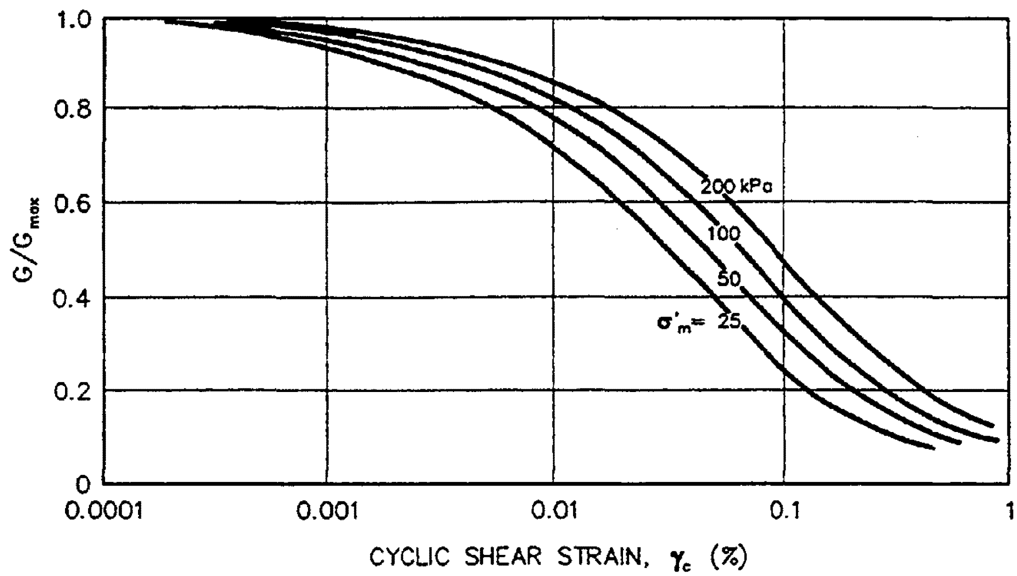


Figure 5-13: Shear Modulus Reduction Curves for Sands. (Iwasaki, *et al.*, 1978, reprinted by permission of Japanese Society of Soil Mechanics and Foundation Engineering)

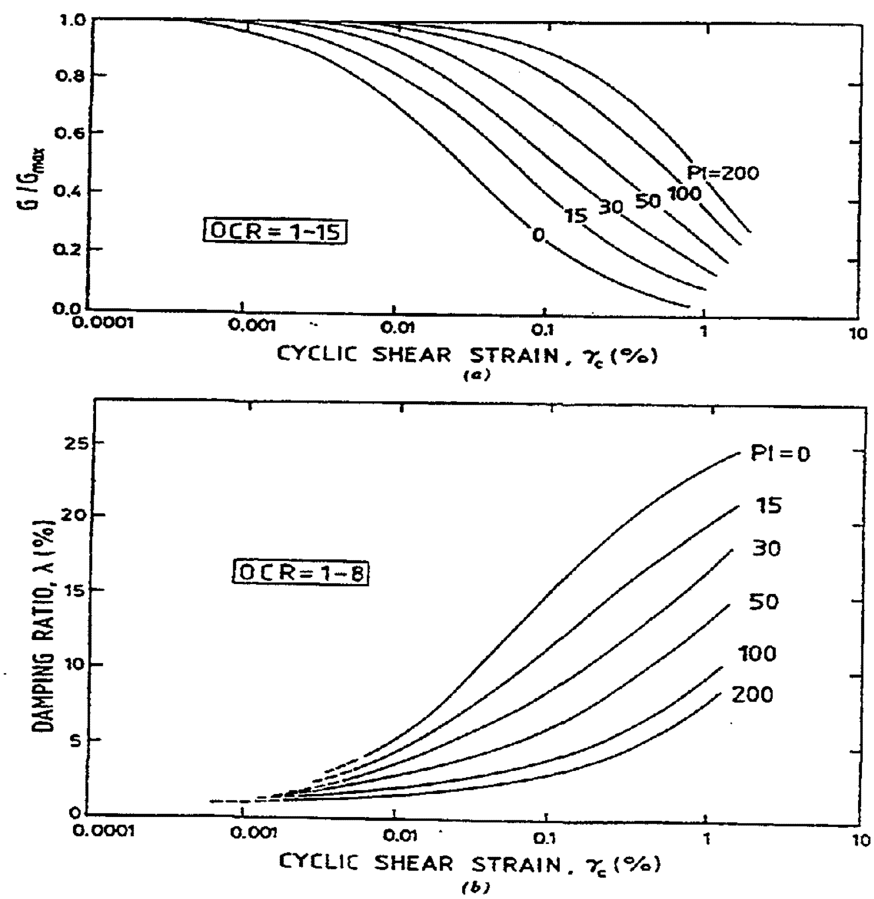
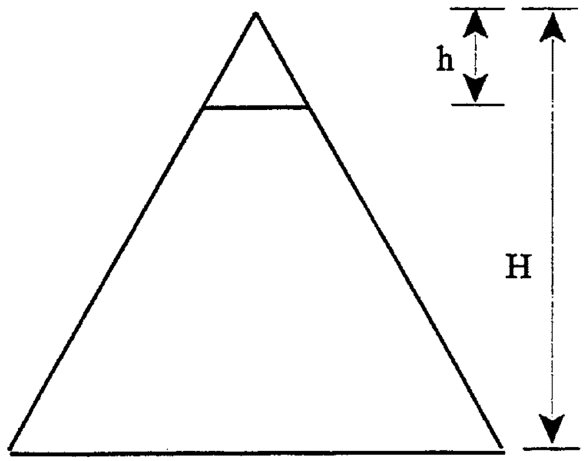


Figure 5-14: Shear Modulus Reduction and Damping Ratio as a Function of Shear Strain and Soil Plasticity Index. (Vucetic and Dobry, 1991, reprinted by permission of ASCE)



$$\lambda = \frac{h}{H} \qquad f_e = \frac{(V_s)_{avg}}{a_n H}$$

f_e = Fundamental Frequency
H-h = Height of the Dam/Embankment
V_S = Shear Wave Velocity
a_n = coefficient

λ	a_n
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0.03	2.409
0.05	2.416
0.10	2.448
0.15	2.501
0.20	2.574
0.25	2.668
0.30	2.786
0.35	2.930
0.40	3.107
0.45	3.323
0.50	3.588
1.00	4.0

Note: For $0.5 \leq \lambda \leq 1.0$, a_n may be derived by linear interpolation from $a_n = 3.6$ for $\lambda = 0.5$ to $a_n = 4.0$ for $\lambda = 1.0$.

Figure 4-19: Fundamental Frequency of Trapezoidal Dam/Embankment

SOLUTIONS TO EXERCISE NO. 5

Material	Location	γ kN/m^3	σ'_v kPa	σ'_m kPa	G_{max} kPa	V_s m/s	Modulus Reduction and Damping
Embankment	1 m from top 1 m above clay	19.5	19.5 87.7	19.5 64.3	60,230 109,390	174 234	PI = 0
Silt	Middle	12	2.2	1.5	1,820	39	PI = 50
Clay	Top Bottom Top w/ Embankment Bottom w/ Embankment	16	4.4 78.8 97.4 171.8	2.9 52.5 64.9 114.5	8,440 45,300 50,350 76,700	72 167 175 217	PI = 15
Till	Middle	20.5	110.9	N/A	293,000	375	PI = 5
Bedrock	Everywhere	21.2	N/A	N/A	N/A	760	N/A

6. Fundamental Period of Soil Layer Beneath Embankment:

6A. $(V_s)_{AVE} = \frac{175+217}{2} = 196$ m/s

6B. $a_n = 4$

6C. $T_o = \frac{4H}{V_s} = \frac{48}{196} = 0.25$ s

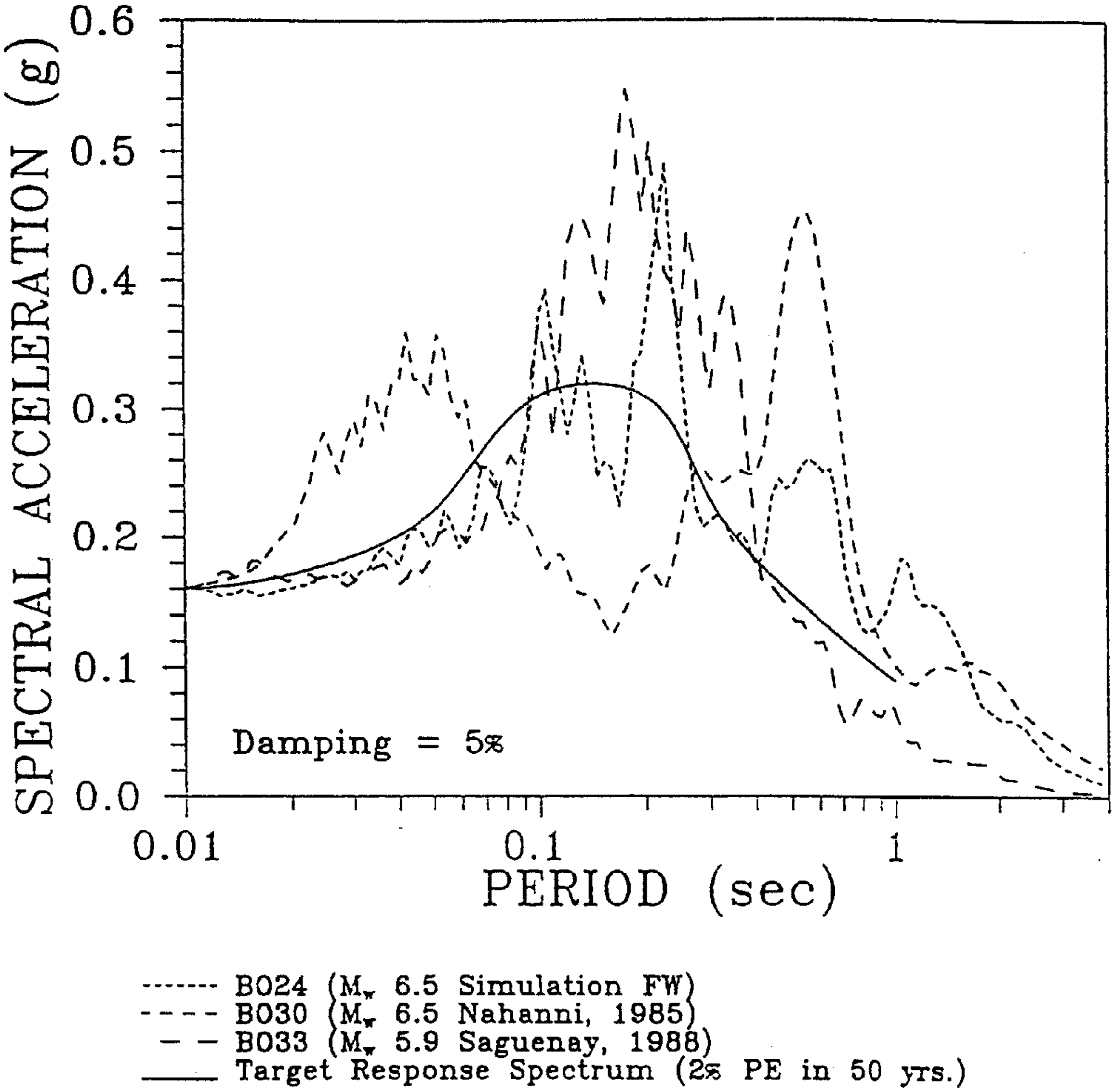


Figure 4-3a: Design Ground Motions- Peak Acceleration Scaling

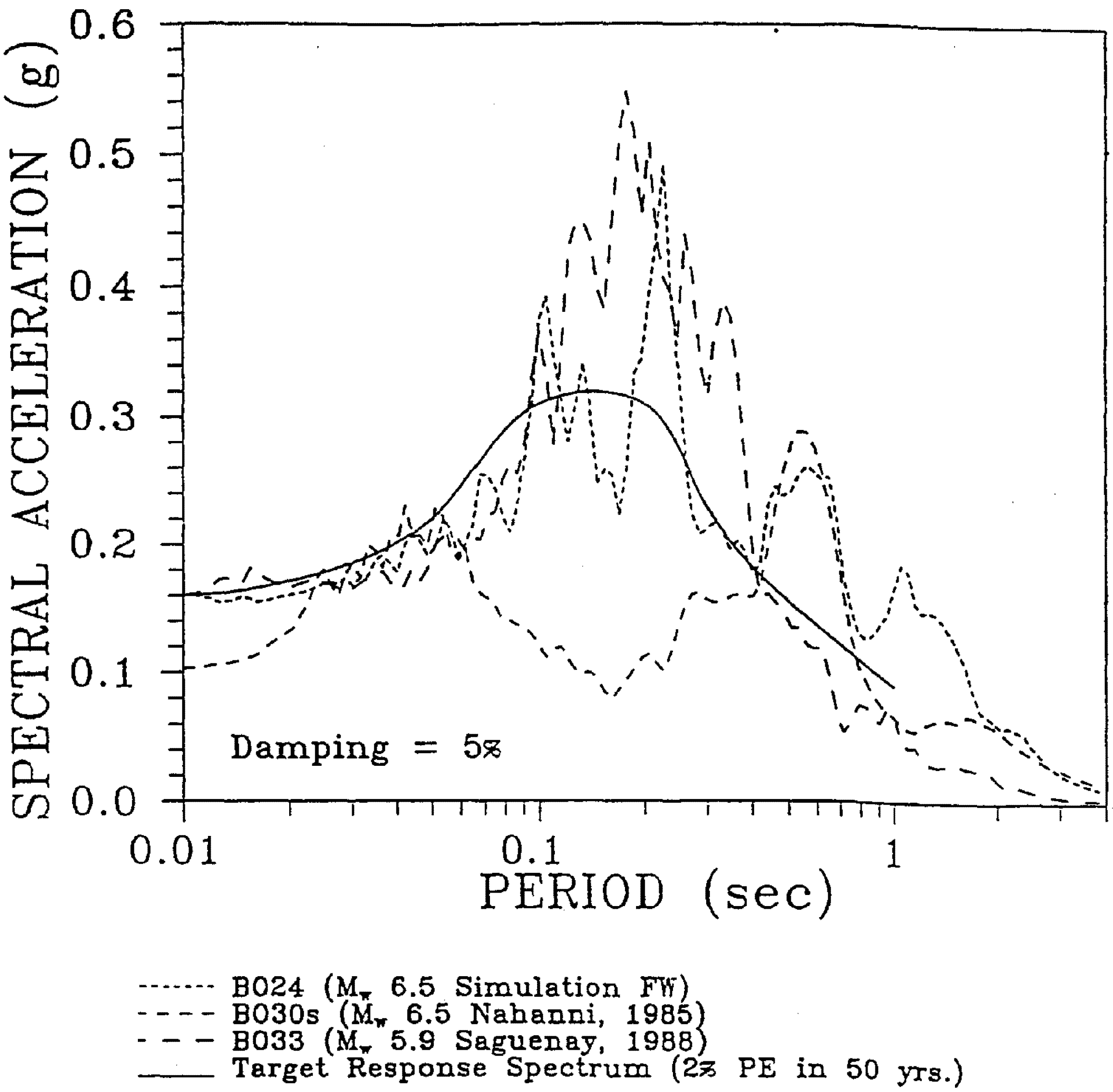


Figure 4-3b: Design Ground Motions- Spectral Acceleration Scaling

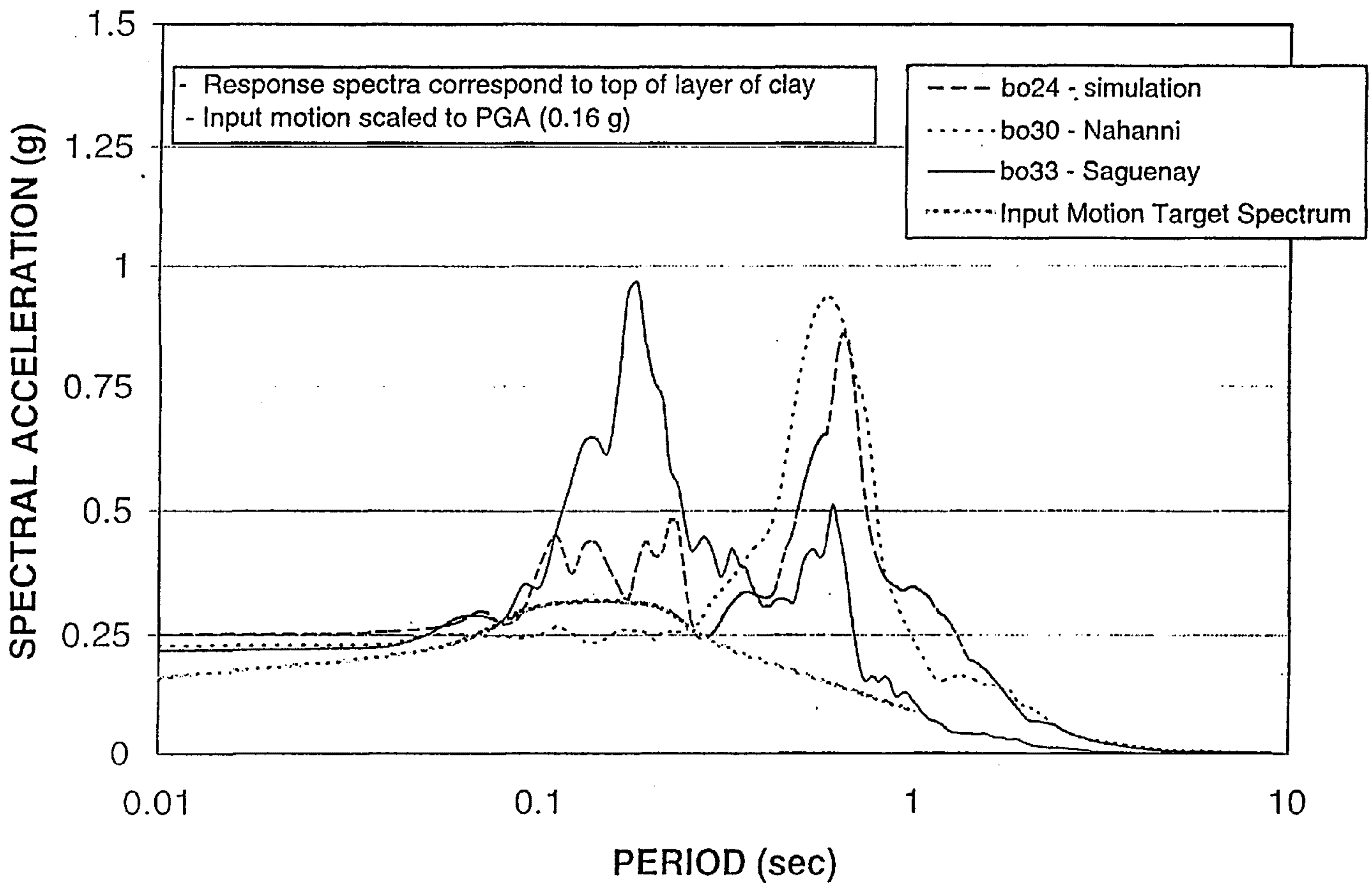


Figure 4-4: Free-Field Acceleration Response Spectra- Peak Acceleration Scaling

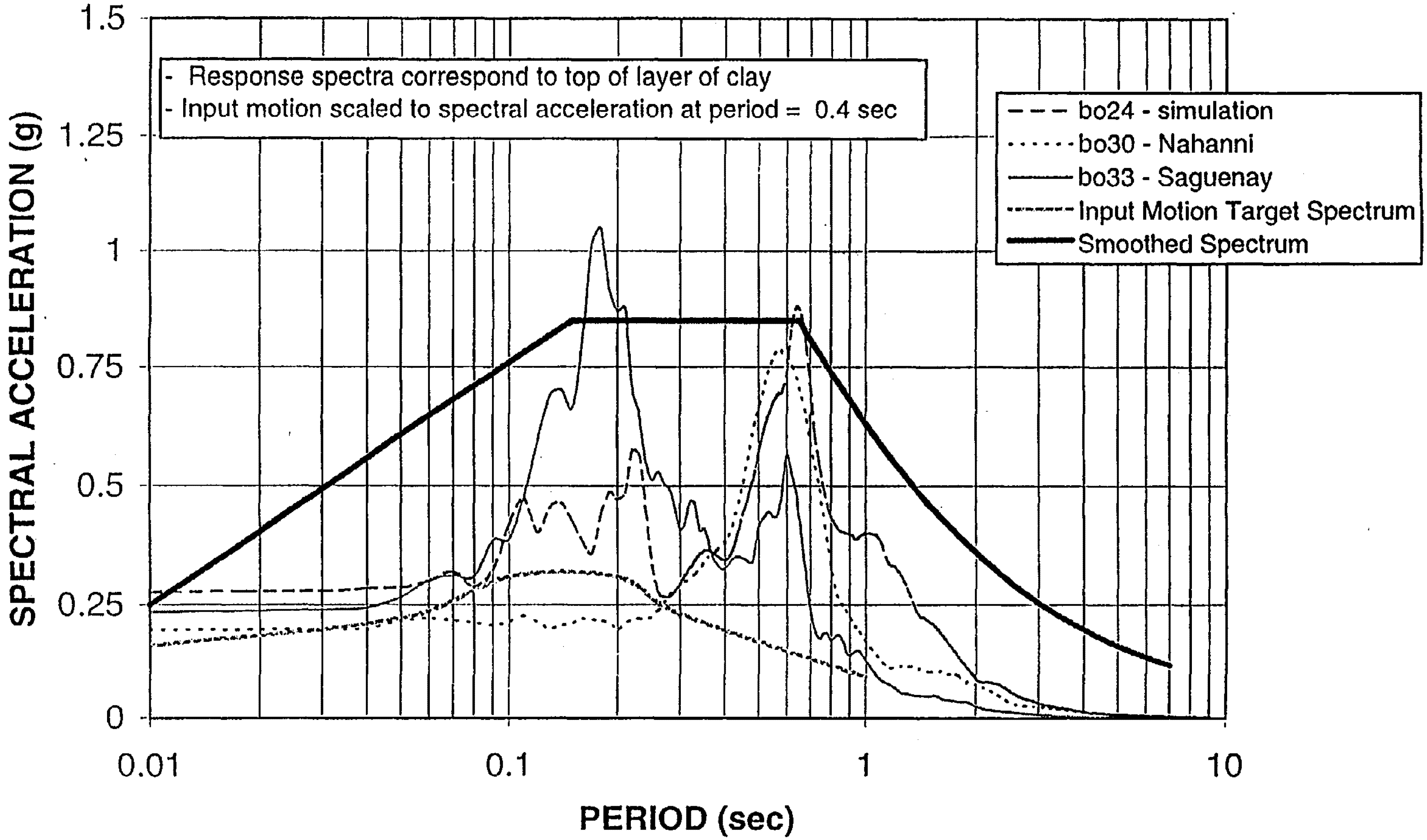


Figure 4-5: Free-Field Acceleration Response Spectra- Spectral Acceleration Scaling

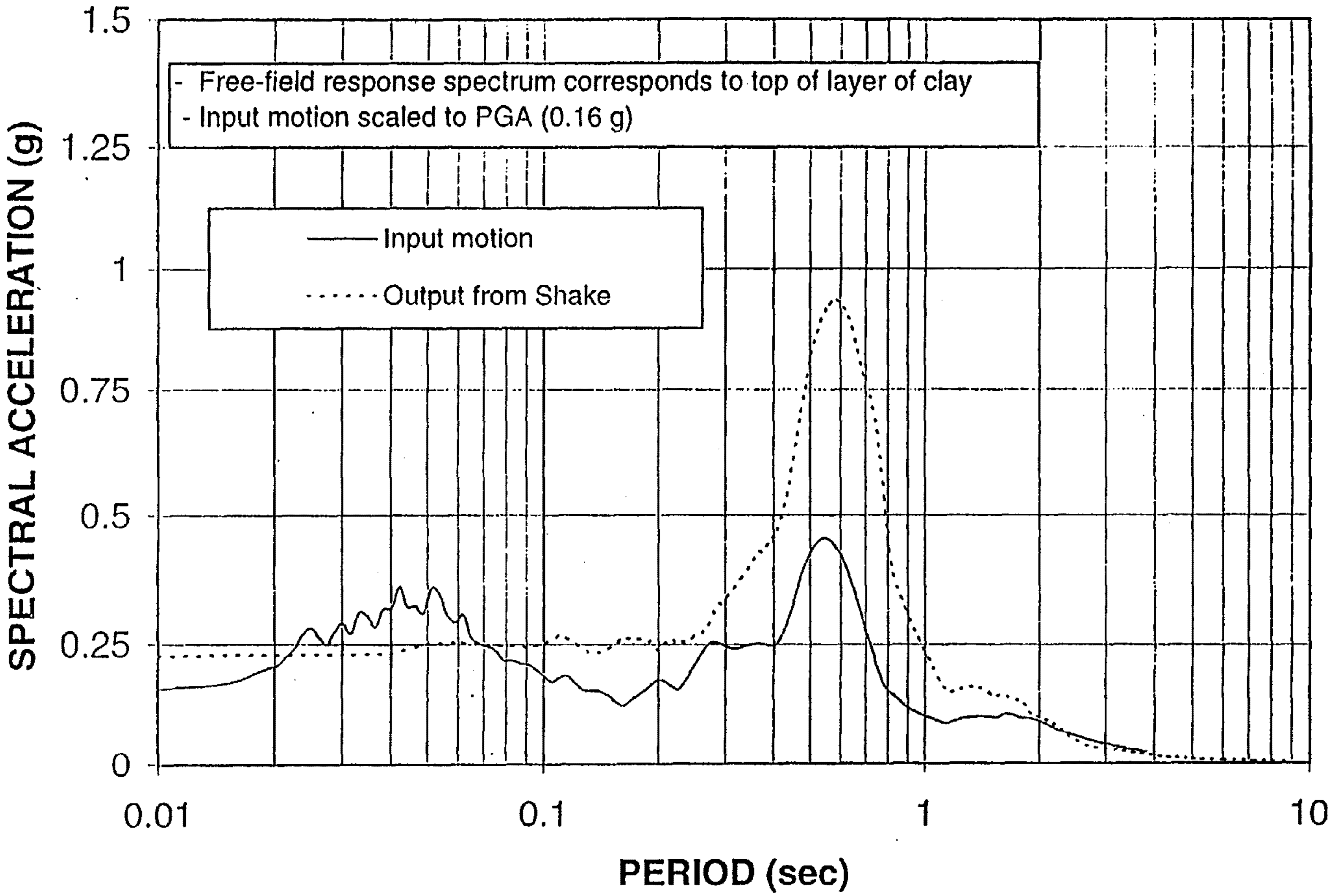


Figure 4-6: Acceleration Response Spectra- Peak Acceleration Scaling (Input vs. Free-Field for Nahanni Record).

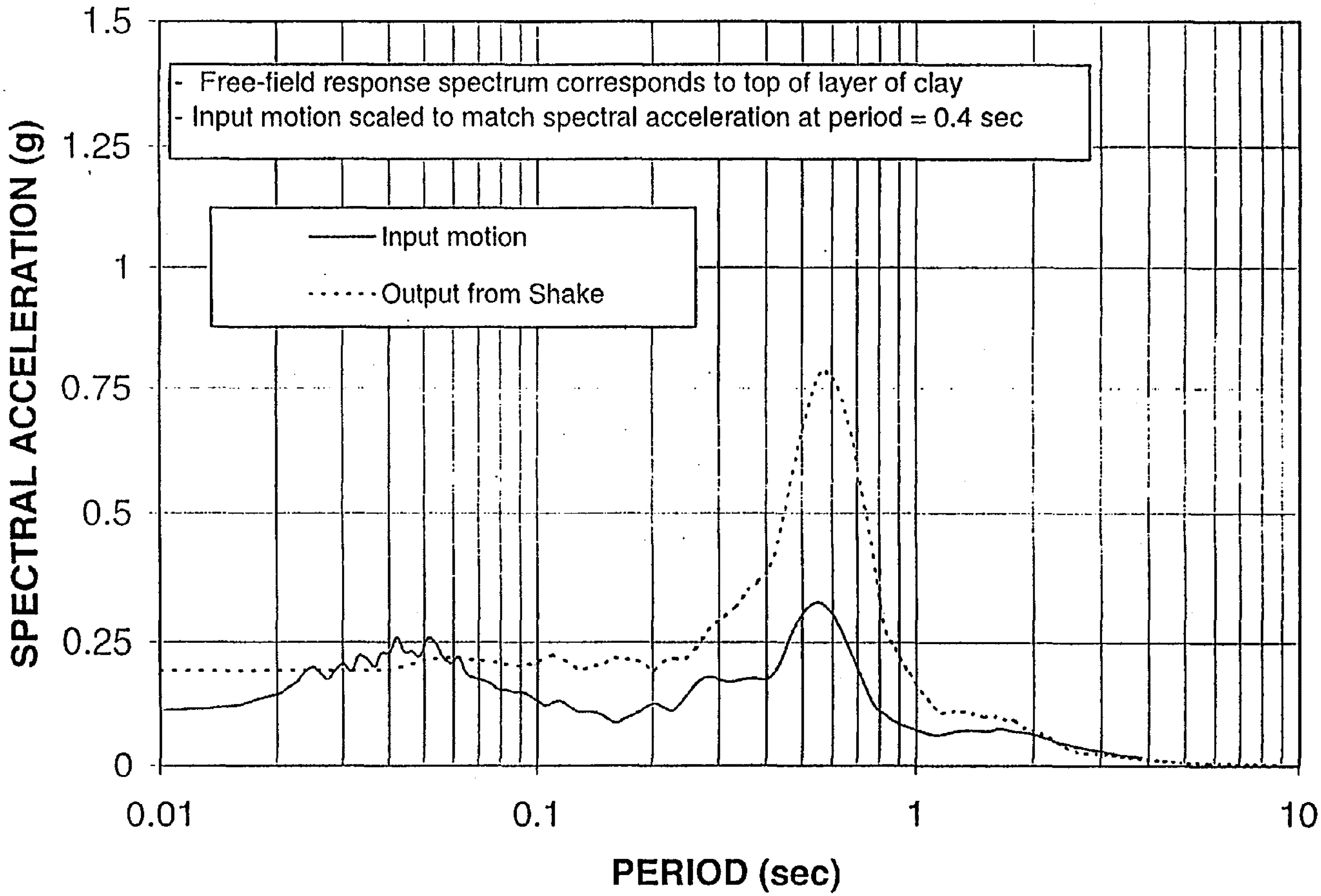


Figure 4-7: Acceleration Response Spectra- Spectral Acceleration Scaling (Input vs. Free-Field for Nahanni Record).

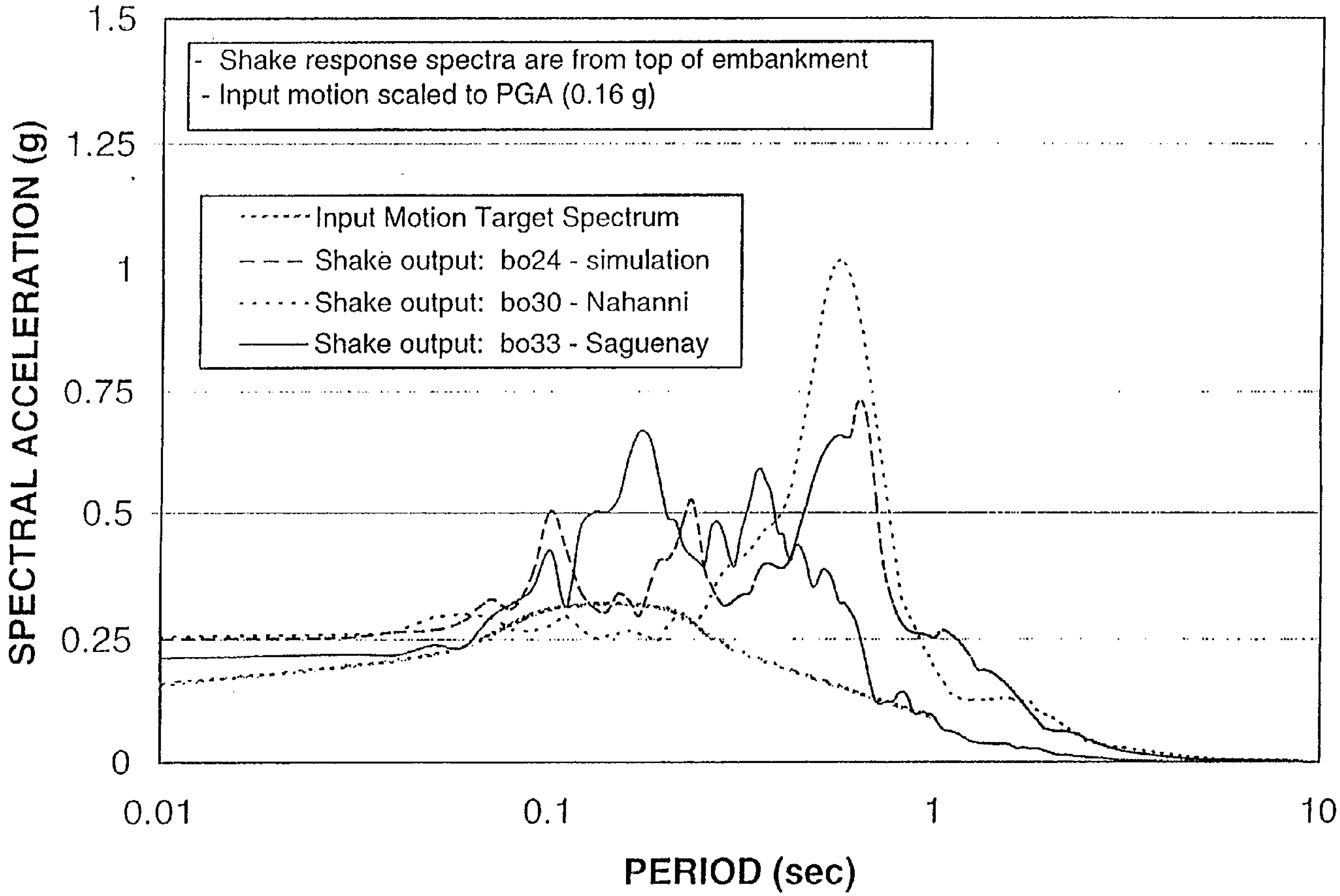


Figure 4-8: Acceleration Response Spectra- SHAKE Analysis vs. Target Spectra

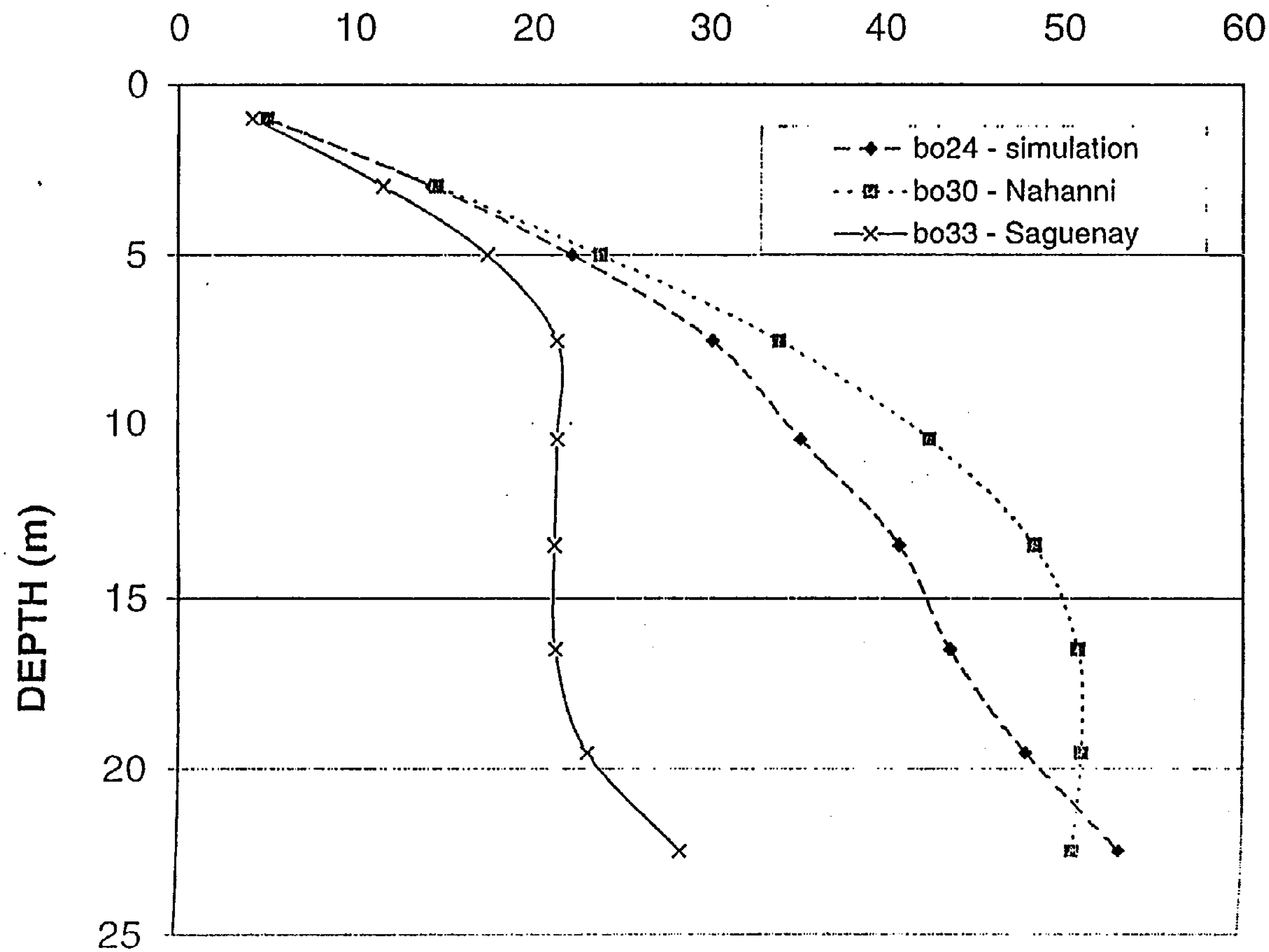


Figure 4-9: Maximum Shear Stresses with Depth.

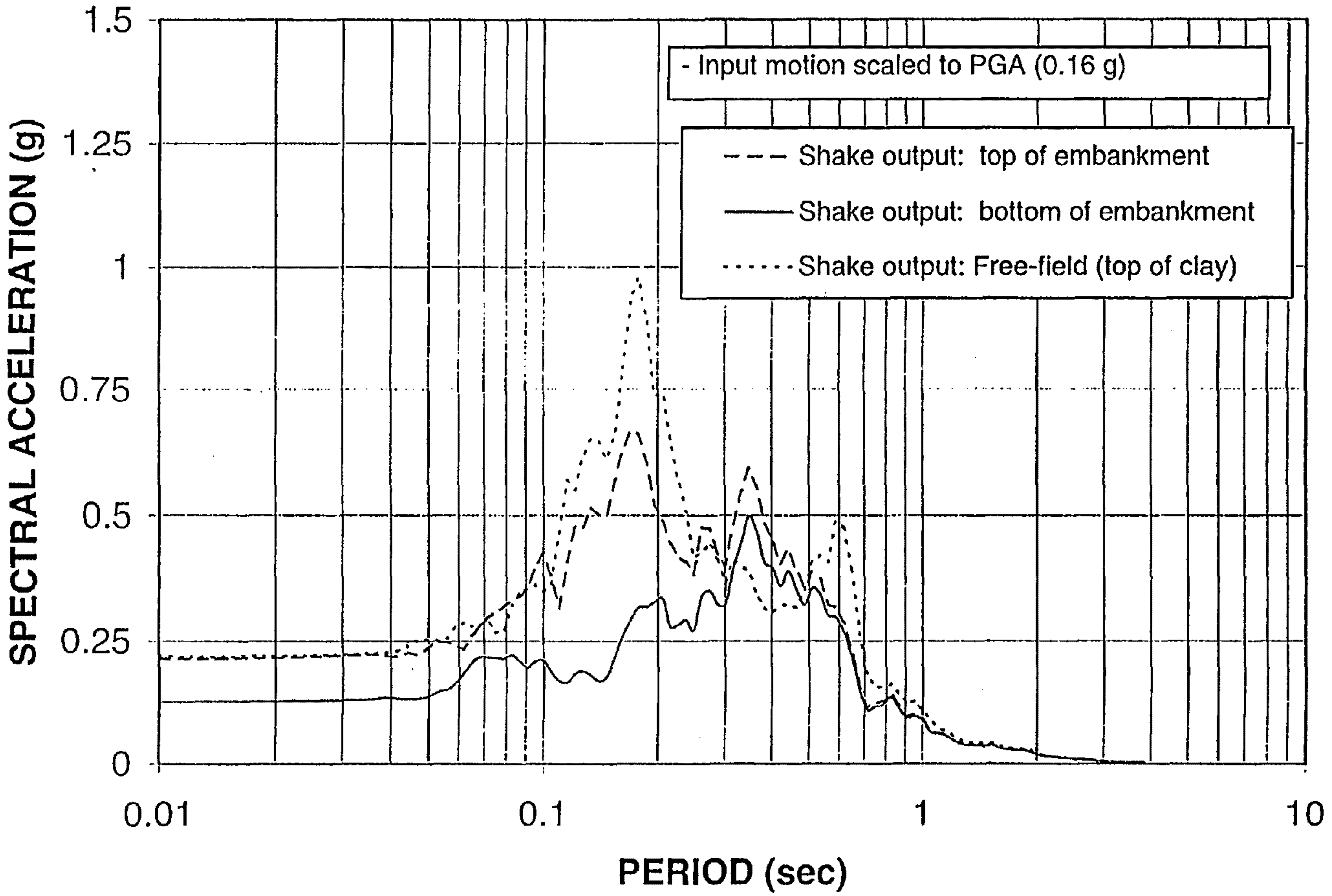


Figure 4-10: Acceleration Response Spectra- Embankment vs. Free-Field for Saguenay Record.

```
OPTION 1 - dynamic soil properties - (max is thirteen):
    1
    5
    9     #1 Modulus for sand (PI=0) (Vucetic and Dobry, 1991)
0.0001    0.000316  0.001     0.00316   0.01      0.0316    0.1       0.316
1.
1.000     1.0       .960      0.87      0.715     0.49      0.25      0.1
0.02
    9     #1 Damping for sand (PI=0) (Vucetic and Dobry, 1991)
0.0001    0.000316  0.001     0.00316   0.01      0.0316    0.1       0.316
1.
2.0       2.0       2.0       3.0       5.5       10.5      16.0      20.0
24.0
   10     #2 Modulus for silt  (PI=50) (Vucetic and Dobry, 1991)
0.0001    0.000316  0.001     0.00316   0.01      0.0316    0.1       0.316
1.        3.16
1.000     1.000     1.000     0.99      0.95      0.83      0.67      0.45
0.22      0.02
   10     #2 Damping for sil  (PI=50) (Vucetic and Dobry, 1991)
0.0001    0.000316  0.001     0.00316   0.01      0.0316    0.1       0.316
1.        3.16
2.0       2.0       2.0       2.0       3.0       4.1       6.0       9.3
13.2      18.0
    9     #3 Modulus for CL (PI=15) (Vucetic and Dobry, 1991)
0.0001    0.0003    0.001     0.00316   0.01      0.0316    0.1       0.316
1.
1.000     1.000     1.000     0.95      0.810     0.63      0.400     0.200
0.1
    9     #3 Damping for CL (PI=15) (Vucetic and Dobry, 1991)
0.0001    0.0003    0.001     0.00316   0.01      0.0316    0.1       0.316
1.
2.580     2.580     2.580     2.580     4.645     7.77      11.67     16.085
20.12
    9     #4 Modulus for till         (PI=5) (Vucetic and Dobry, 1991)
0.0001    0.0003    0.001     0.00316   0.01      0.0316    0.1       0.316
1.
1.000     1.000     0.980     0.900     0.77      0.51      0.32      0.16
0.06
    9     #4 Damping for till not yet (PI=5) (Vucetic and Dobry, 1991)
0.0001    0.0003    0.001     0.00316   0.01      0.0316    0.1       0.316
1.
2.0       2.000     2.2       2.6       4.6       9.0       14.0      19.5
23.8
    8     #5 ATTENUATION OF  ROCK  AVERAGE
 .0001     0.0003   0.001     0.003     0.01      0.03      0.1       1.0
 1.000     1.000    0.9875    0.9525    0.900     0.810     0.725     0.550
    5     #5 DAMPING IN ROCK
 .0001     0.001    0.01      0.1       1.
 0.4       0.8      1.5       3.0       4.6
    5   1    2    3    4    5
OPTION 2 -- Soil Profile
    2
    1    8        FHWA without embankment
    1    2             6.56                 0.05      0.076     128
    2    3             9.84                 0.05      0.102     275
    3    3             9.84                 0.05      0.102     353
    4    3             9.84                 0.05      0.102     431
    5    3             9.84                 0.05      0.102     509
    6    4             9.84                 0.05      0.131     1230
    7    4             9.84                 0.05      0.131     1230
```

a-f-30.ini   Created Wed Jun 10 10:58:48 1998

```
    8    5                                0.10      0.135     2493
OPTION 3 -- input motion:
    3
 3752 4096      .005                   bo30.sar        (8F9.6)
                0.16  25.          2    8
OPTION 4 -- sublayer for input motion (within (1) or outcropping (0):
    4
    8    0
OPTION 5 -- number of iterations & ratio of avg strain to max strain
    5
    0    8    0.59
OPTION 9 -- RESPONse
    9
    2    1
    1    0    981
    0.05
STOP      -- execution will stop when program encounters 0
    0
```

```
OPTION 1 - dynamic soil properties - (max is thirteen):
    1
    5
    9      #1 Modulus for sand (PI=0) (Vucetic and Dobry, 1991)
0.0001     0.000316  0.001     0.00316   0.01      0.0316    0.1       0.316
1.
1.000      1.0       .960      0.87      0.715     0.49      0.25      0.1
0.02
    9      #1 Damping for sand (PI=0) (Vucetic and Dobry, 1991)
0.0001     0.000316  0.001     0.00316   0.01      0.0316    0.1       0.316
1.
2.0        2.0       2.0       3.0       5.5       10.5      16.0      20.0
24.0
   10      #2 Modulus for silt  (PI=50) (Vucetic and Dobry, 1991)
0.0001     0.000316  0.001     0.00316   0.01      0.0316    0.1       0.316
1.         3.16
1.000      1.000     1.000     0.99      0.95      0.83      0.67      0.45
0.22       0.02
   10      #2 Damping for sil  (PI=50) (Vucetic and Dobry, 1991)
0.0001     0.000316  0.001     0.00316   0.01      0.0316    0.1       0.316
1.         3.16
2.0        2.0       2.0       2.0       3.0       4.1       6.0       9.3
13.2       18.0
    9      #3 Modulus for CL (PI=15) (Vucetic and Dobry, 1991)
0.0001     0.0003    0.001     0.00316   0.01      0.0316    0.1       0.316
1.
1.000      1.000     1.000     0.95      0.810     0.63      0.400     0.200
0.1
    9      #3 Damping for CL (PI=15) (Vucetic and Dobry, 1991)
0.0001     0.0003    0.001     0.00316   0.01      0.0316    0.1       0.316
1.
2.580      2.580     2.580     2.580     4.645     7.77      11.67     16.085
20.12
    9      #4 Modulus for till        (PI=5) (Vucetic and Dobry, 1991)
0.0001     0.0003    0.001     0.00316   0.01      0.0316    0.1       0.316
1.
1.000      1.000     0.980     0.900     0.77      0.51      0.32      0.16
0.06
    9      #4 Damping for till not yet (PI=5) (Vucetic and Dobry, 1991)
0.0001     0.0003    0.001     0.00316   0.01      0.0316    0.1       0.316
1.
2.0        2.000     2.2       2.6       4.6       9.0       14.0      19.5
23.8
    8      #5 ATTENUATION OF ROCK  AVERAGE
 .0001      0.0003   0.001     0.003     0.01      0.03      0.1       1.0
 1.000      1.000    0.9875    0.9525    0.900     0.810     0.725     0.550
    5      #5 DAMPING IN ROCK
 .0001      0.001    0.01      0.1       1.
  0.4       0.8      1.5       3.0       4.6
    5    1    2    3    4    5
OPTION 2 -- Soil Profile
    2
    1   10       FHWA with    embankment
    1    1           6.56                  0.05      0.124     571
    2    1           6.56                  0.05      0.124     669
    3    1           6.56                  0.05      0.124     768
    4    3           9.84                  0.05      0.102     591
    5    3           9.84                  0.05      0.102     626
    6    3           9.84                  0.05      0.102     660
    7    3           9.84                  0.05      0.102     695
```



```
    8    4           9.84                  0.05      0.131     1230
    9    4           9.84                  0.05      0.131     1230
   10    5                                 0.10      0.135     2493
OPTION 3 -- input motion:
    3
 3752 4096      .005                     bo30.sar        (8F9.6)
                0.16  25.          2     8
OPTION 4 -- sublayer for input motion (within (1) or outcropping (0):
    4
   10    0
OPTION 5 -- number of iterations & ratio of avg strain to max strain
    5
    0    8    0.59
OPTION 9 -- RESPONse
    9
    1    1
    1    0    981
  0.05
STOP     -- execution will stop when program encounters 0
    0
```

STUDENT EXERCISE NO. 6

Evaluation of Liquefaction Potential

Objective:

Evaluate the Liquefaction Potential at 6 m Below Ground Surface at a Site Shown in Figure S6-1.

- Soil Properties and SPT- N Values are Shown in Figure S6-1. (Same as in Exercise 2A)
- SPT- N Values and Sampling Procedure are the Same as in Exercise 2A.
- Design Earthquake Magnitude M = 6.0
- Peak Ground Acceleration = 0.18 g

Source Materials:

Reference Manual Part I: Section 8.3.2, Figures 8-2, 8-3, 8-4, 8-5, and 8-6 and Equations 8-1, 8-3a, 8-4 and 8-5.

Student Exercise No. 2A

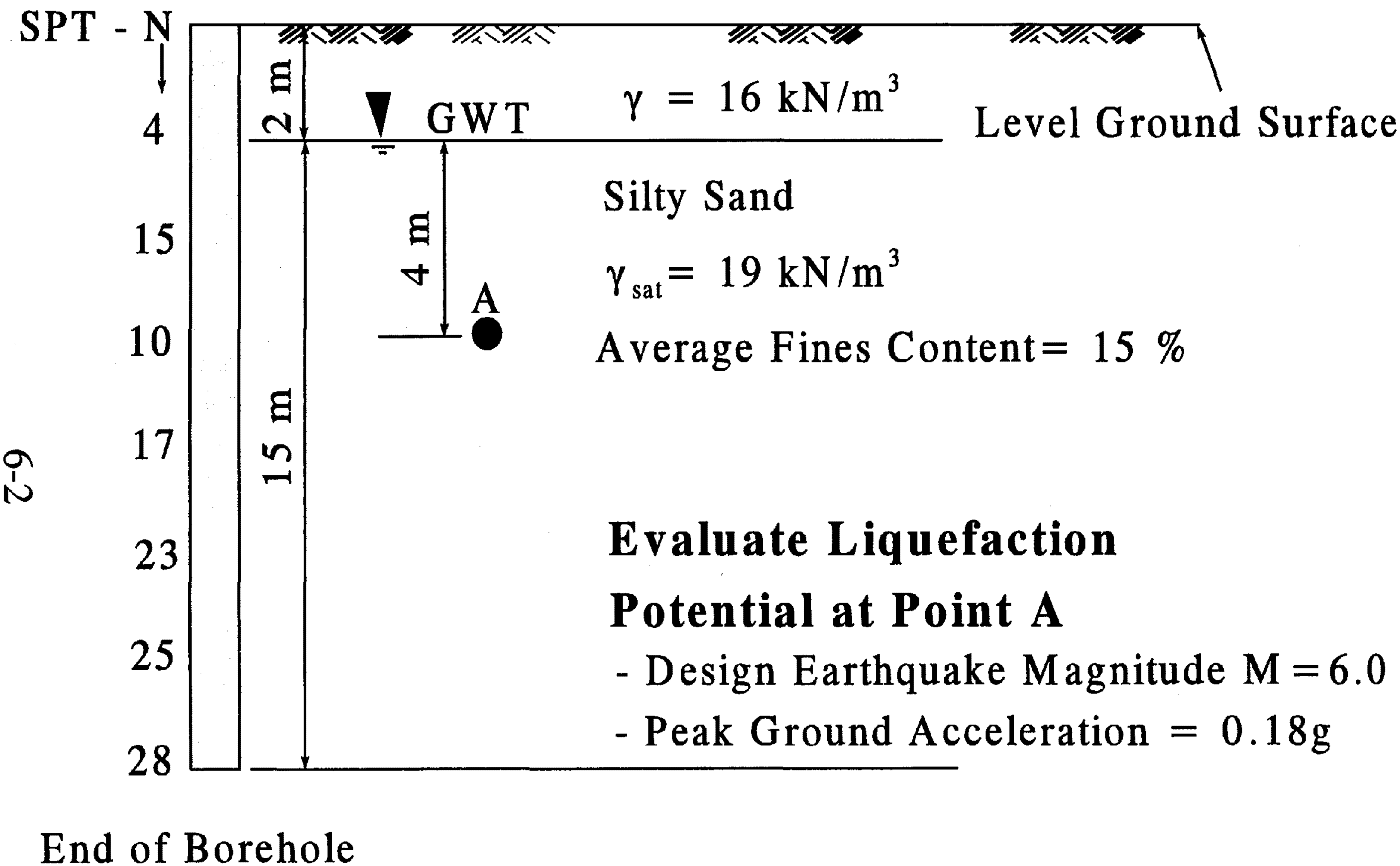


Figure S6-1: Soil Profile

Step 1: Develop Subsurface Profile
(Boring with SPT-N Values Given)

Step 2: Compute Effective Overburden Pressure, σ_v'

σ_v' = 69 kPa (From Student Exercise No. 2A)

Compute Total Overburden Pressure, σ_v

σ_v = ______________

Evaluate Intitial Shear Stress, τ_{ho}

τ_{ho} = ____________

Step 3: Evaluate Stress Reduction Factor, r_d (Equation 8-1 or Figure 8-2)

r_d = ____________

Step 4: Calculate Cyclic Stress Ratio Induced by Earthquake, CSR_{EQ} (Equation 8-3a)

$$CSR_{EQ} = 0.65 \left(\frac{a_{max}}{g} \right) r_d \left(\frac{\sigma_v}{\sigma_v^{/}} \right)$$

CSR_{EQ} = ____________

Step 5: Standardized SPT N Value

$C_{60} = 0.78$ (From Student Exercise No. 2A)

Step 6: Normalized SPT N Vlaue

$C_N = 1.18$ (From Student Exercise No. 2A)

$(N_1)_{60} = C_N C_{60} N$

$(N_1)_{60} = 1.18 \times 0.78 \times 10$

$(N_1)_{60} \approx 9$ (From Student Exercise No. 2A)

Step 7: Determine Soil Resistance to Liquefaction in Terms of Cyclic Stress Ratio (Use Figure 8-3)

$CSR_{M=7.5}$ = ____________

Step 8: Correct CSR for Earthquake Magnitude, Initial Shear Stress and Effective Overburden Pressure

$CSR_L = CSR_{M=7.5}\ k_M\ k_\alpha\ k_\sigma$ (Use Figures 8-4,8-5 and 8-6)

CSR_L = ____________

Step 9: Calculate Factor of Safety (Equation 8-5)

$$FS_L = \frac{CSR_L}{CSR_{EQ}}$$

FS_L = ____________

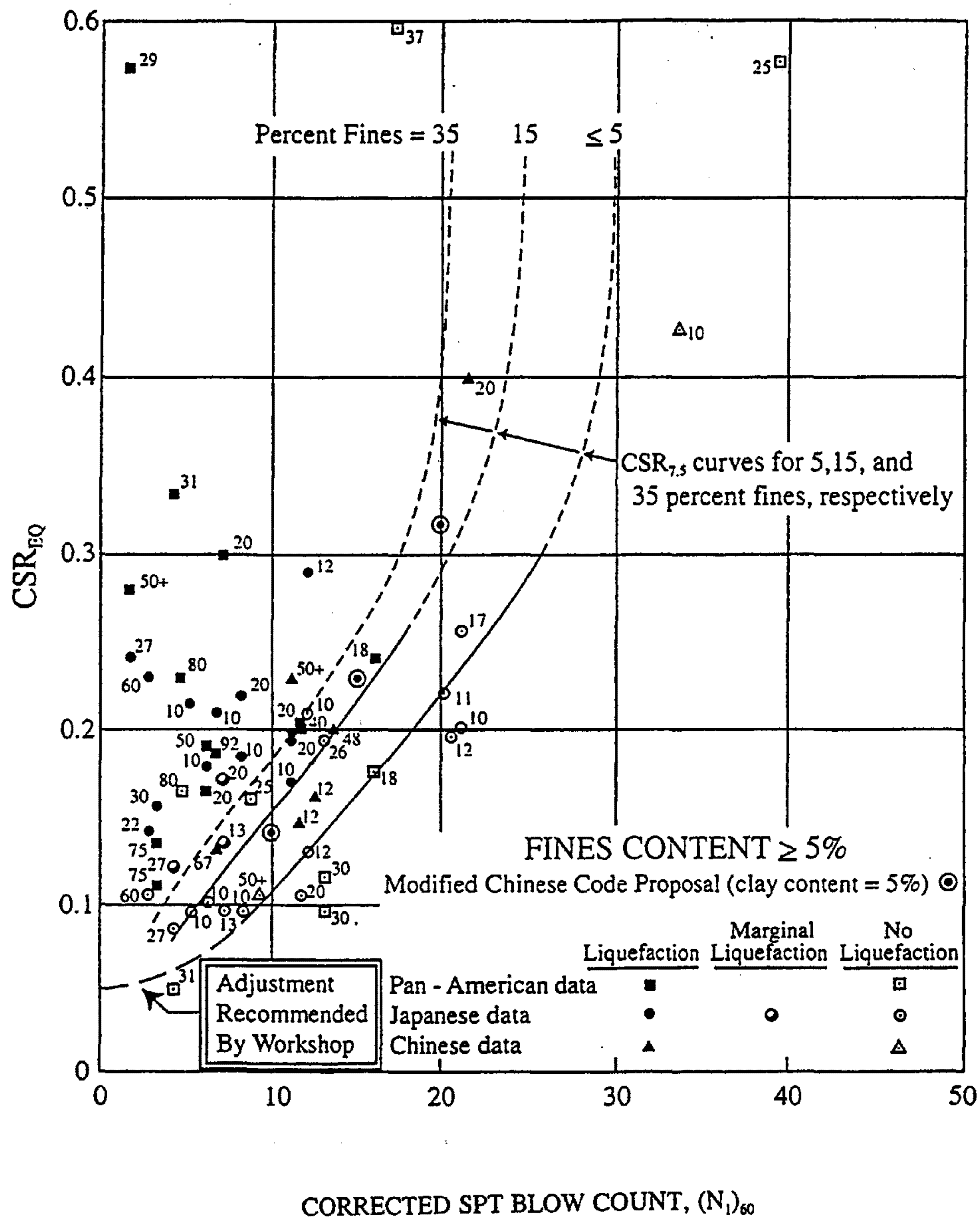


Figure 8-3: Relationship Between Cyclic Stress Ratio Causing Liquefaction and SPT $(N_1)_{60}$ Values for Sands for M= 7.5 Earthquakes (modified From Seed et al., 1985)

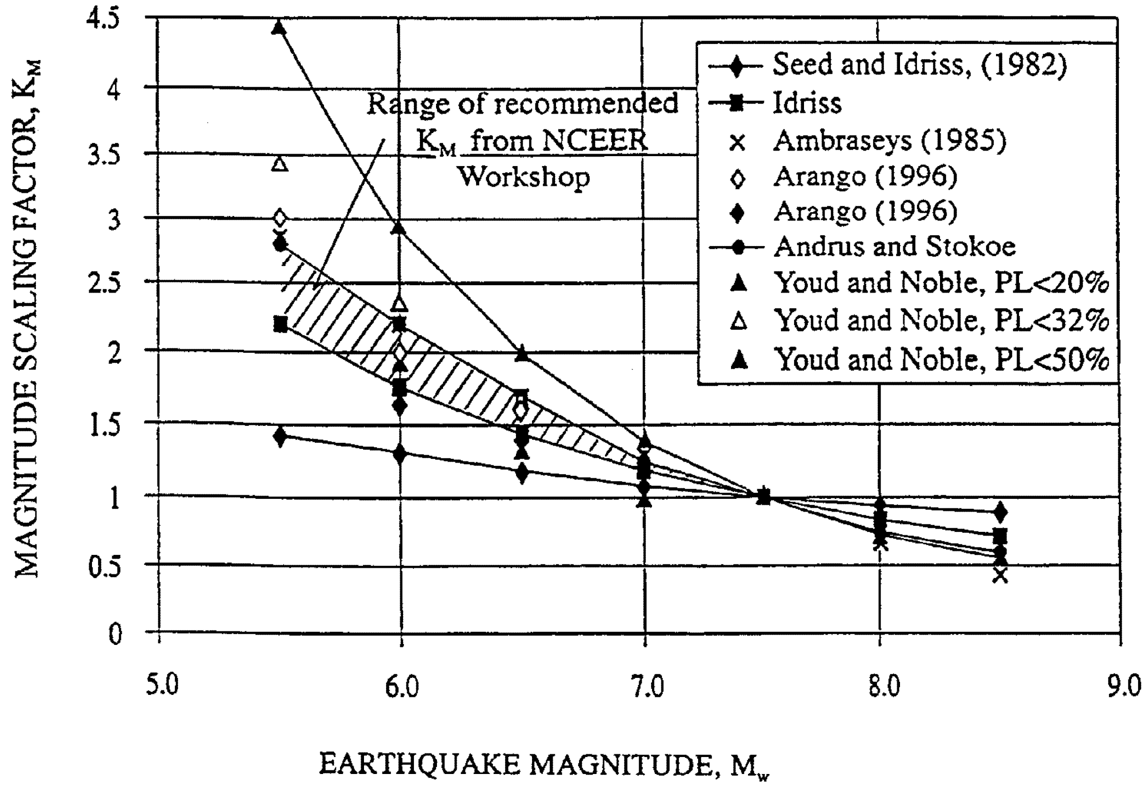


Figure 8-4: Magnitude Scaling Factors Derived by Various Investigators (After Youd and Idriss, 1997)

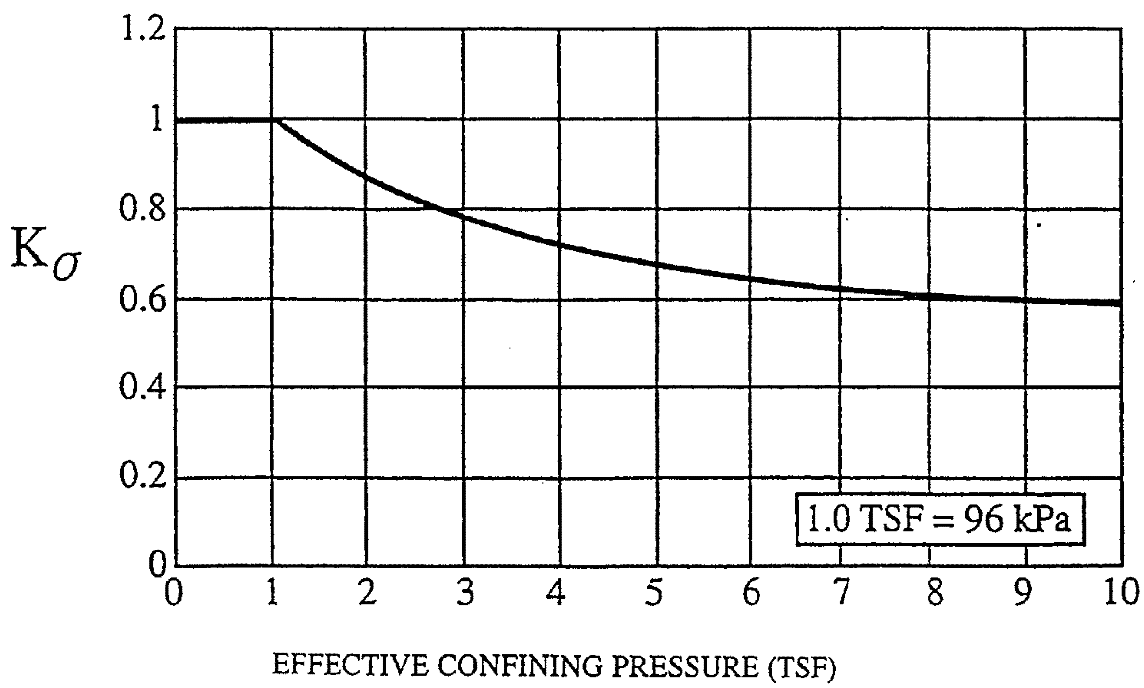


Figure 8-5: Recommended Correction Factor k_σ. (After Youd and Idriss, 1997)

SOLUTIONS TO STUDENT EXERCISE NO. 6

Step 1: Develop Subsurface Profile
(Boring with SPT-N Values Given)

Step 2: Compute Effective Overburden Pressure, σ_v'

$\sigma_v' = 69$ kPa (From Student Exercise No. 2A)

Compute Total Overburden Pressure, σ_v

$$\sigma_v = \gamma \times 2m + \gamma_{sat} \times 4m$$

$$\sigma_v = 16 \times 2m + 19 \times 4m$$

$$\sigma_v = \underline{\quad 108 \text{ kPa} \quad}$$

Evaluate Intitial Shear Stress, τ_{ho}

$\tau_{ho} = \underline{\quad 0 \quad}$ (Level Ground)

Step 3: Evaluate Stress Reduction Factor, r_d (Equation 8-1 or Figure 8-2)

$$r_d = 1 - 0.00765 \cdot z = 1 - 0.00765 \times 6$$

$$= \underline{\quad 0.95 \quad} \qquad \text{(For } z = 6\text{ m)}$$

Step 4: Calculate Cyclic Stress Ratio Induced by Earthquake, CSR_{EQ} (Equation 8-3a)

$$CSR_{EQ} = 0.65 \left(\frac{a_{max}}{g} \right) r_d \left(\frac{\sigma_v}{\sigma_v^{/}} \right)$$

$$CSR_{EQ} = 0.65\ (0.18)\ 0.95 \left(\frac{108}{69} \right)$$

$$CSR_{EQ} = \underline{\quad 0.174 \quad}$$

Step 5: Standardized SPT N Value

$C_{60} = 0.78$ (From Student Exercise No. 2A)

Step 6: Normalized SPT N Value

$C_N = 1.18$ (From Student Exercise No. 2A)

$(N_1)_{60} = C_N C_{60} N$

$(N_1)_{60} = 1.18 \times 0.78 \times 10$

$(N_1)_{60} \approx 9$ (From Student Exercise No. 2A)

Step 7: Determine Soil Resistance to Liquefaction in Terms of Cyclic Stress Ratio (Use Figure 8-3)

$CSR_{M=7.5} = \underline{0.14}$ (Based on $(N_1)_{60} = 9$ and Fines Content = 15%)

Step 8: Correct CSR for Earthquake Magnitude, Initial Shear Stress and Effective Overburden Pressure

$CSR_L = CSR_{M=7.5}\, k_M\, k_\alpha\, k_\sigma$

$k_M = 1.75$ (for M=6.0, Figure 8-4)

$k_\alpha = 1.0$ (for Level Ground)

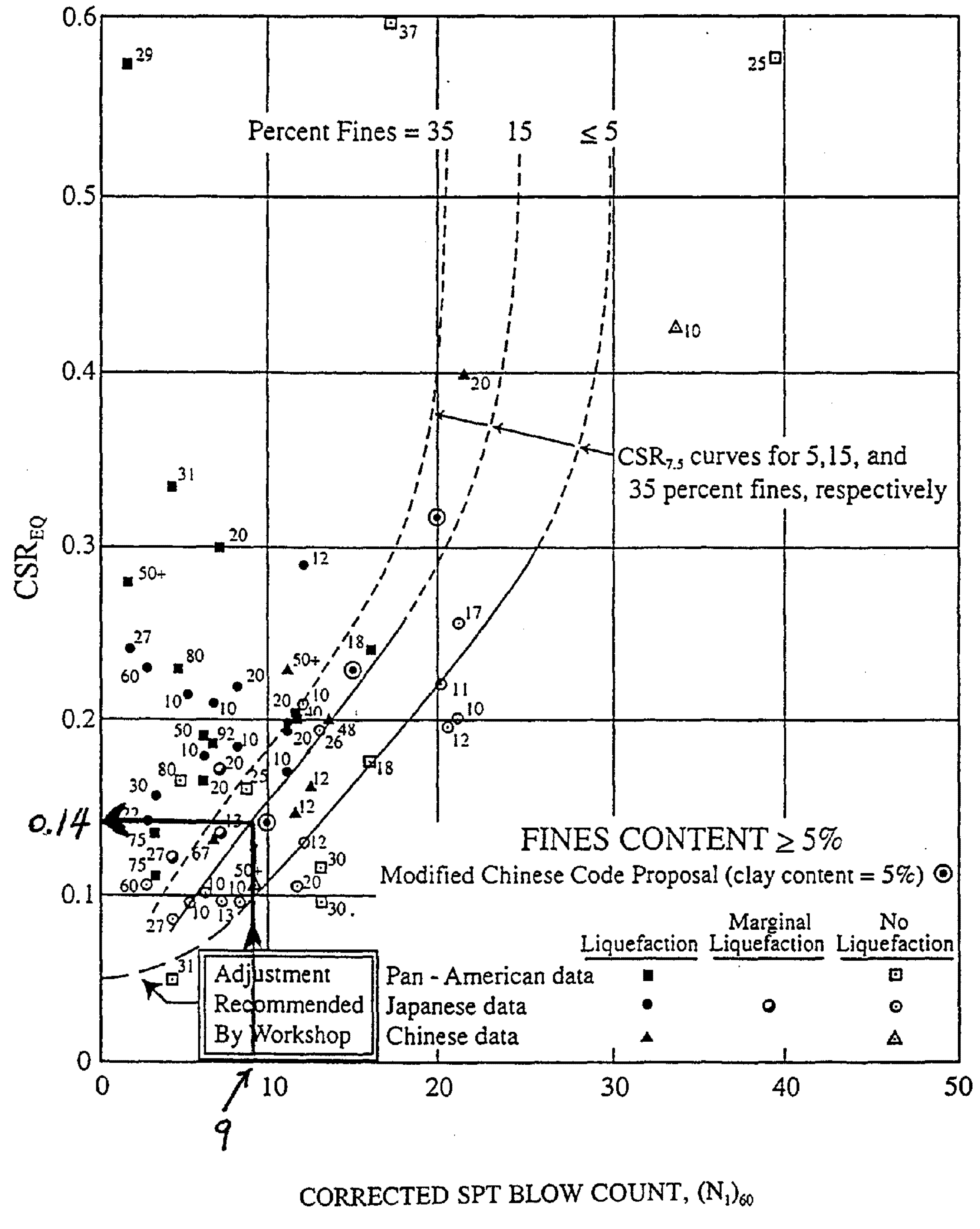


Figure 8-3: Relationship Between Cyclic Stress Ratio Causing Liquefaction and SPT $(N_1)_{60}$ Values for Sands for M = 7.5 Earthquakes (modified From Seed et al., 1985)

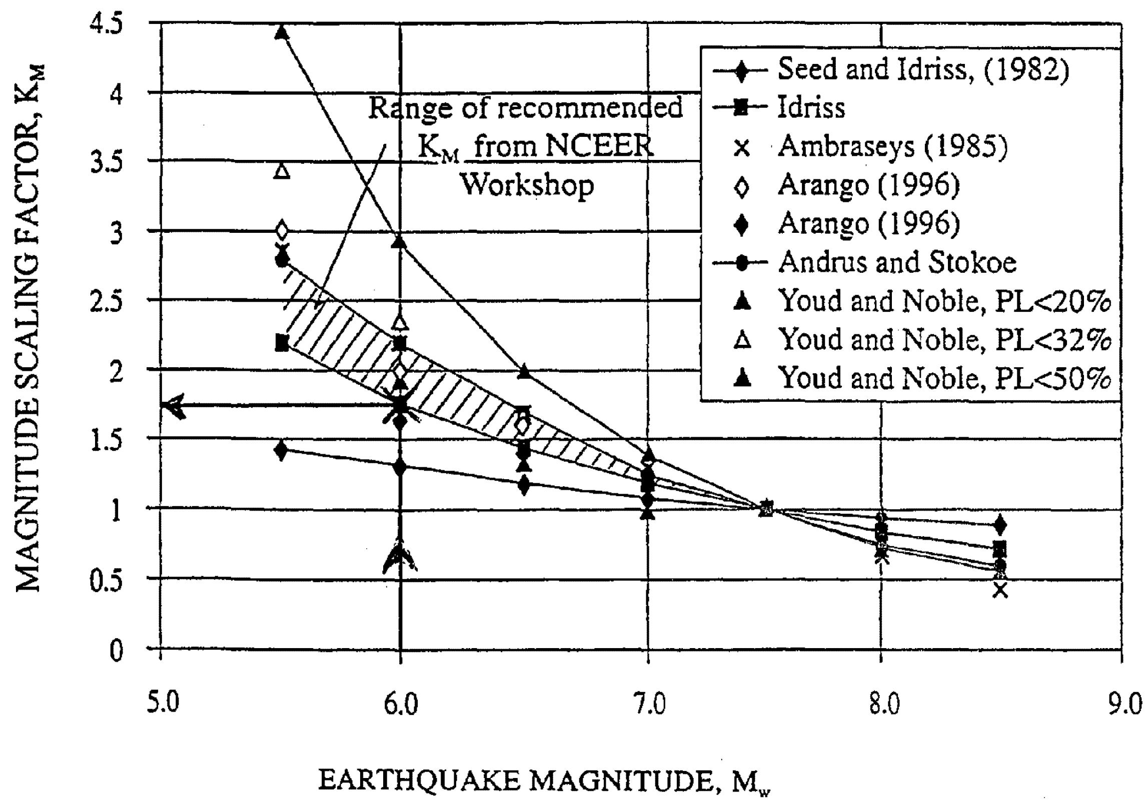


Figure 8-4: Magnitude Scaling Factors Derived by Various Investigators (After Youd and Idriss, 1997)

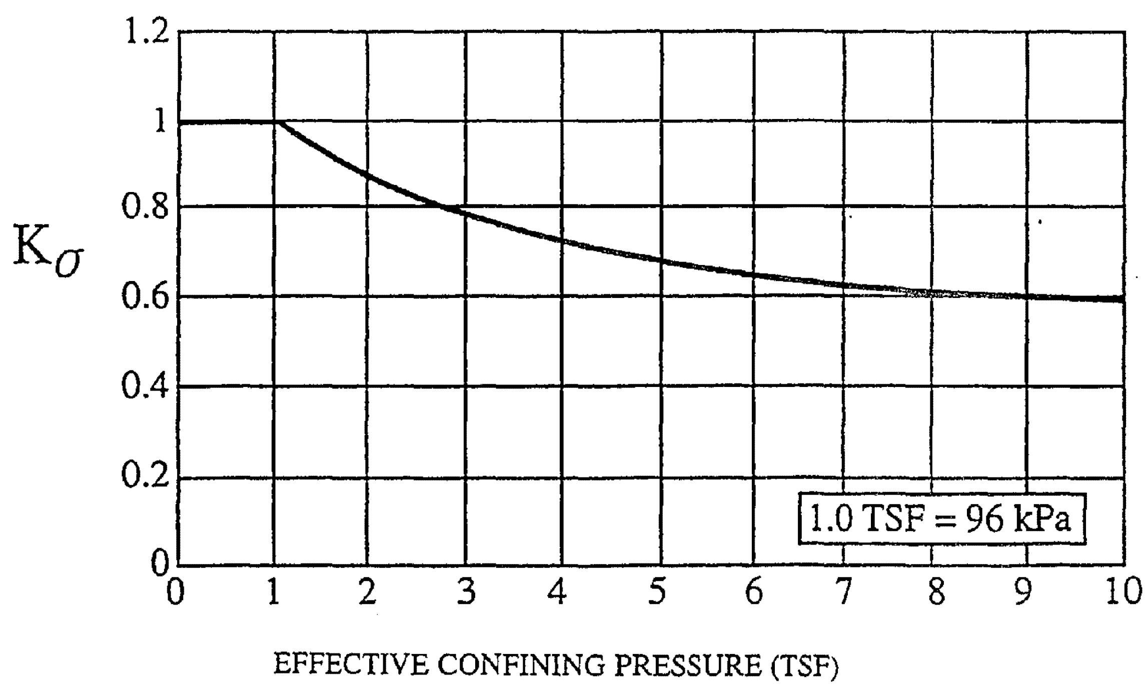


Figure 8-5: Recommended Correction Factor k_σ. (After Youd and Idriss, 1997)

$k_\alpha = 1.0$ (No Correction for $\sigma'_m < 100$ kPa)
(Figure 8-5)

$$CSR_L = 0.14 \times 1.75 \times 1.0 \times 1.0$$

$$CSR_L = \underline{\quad 0.245 \quad}$$

Step 9: Calculate Factor of Safety

$$FS_L = \frac{CSR_L}{CSR_{EQ}}$$

$$FS_L = \frac{0.245}{0.174}$$

$$FS_L = \underline{\quad 1.4 \quad}$$

STUDENT EXERCISE NO. 7

Simplified Seismic Deformation Analysis.

Objective:

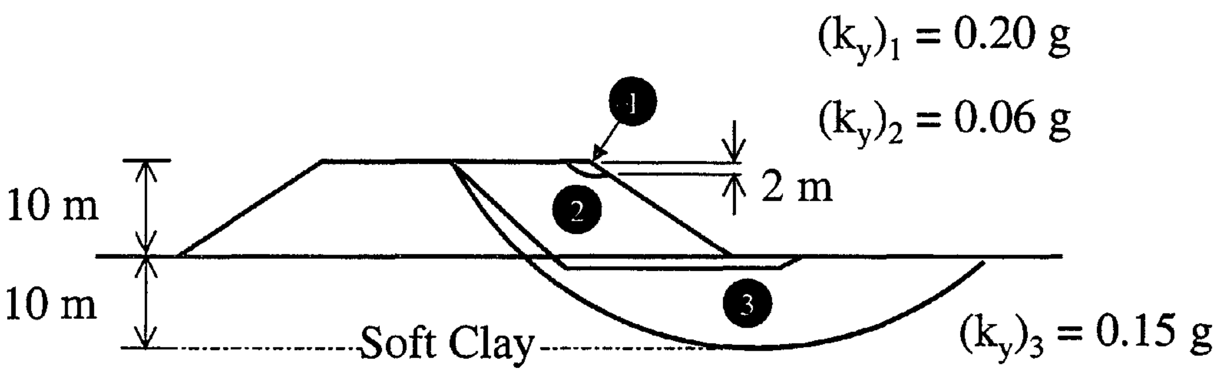
Use Newmark Analysis Design Charts to Estimate Permanent Seismic Deformations for Three Potential Failure Surfaces Shown in Figure S7-1 for a M 6.5 Earthquake with Free-Field, Firm Ground Peak Ground Acceleration = 0.16 g.

Source Materials:

Reference Manual Part I: Figures 6-2, 6-3, 6-4, 6-5, 7-4, 7-9, and 8-2.

1. Evaluate Peak Ground Acceleration at Top of Embankment Using Figures 6-3, and 6-4

$(PGA)_{EMB}$ = ______________



Exercise 7 - Figure S7-1

2. Evaluate PAA for Failure Surface 1, 2, and 3

A. Evaluate Z/H for Use in Figure 6-5

Failure Surface 1: Z/H = ________

Failure Surface 2: Z/H = ________

Failure Surface 3: Not Applicable

B. Find PAA/PGA Using Figures 6-5, and 8-2

Failure Surface 1: $\left(\frac{PAA}{PGA}\right)_1$ = ____________

Failure Surface 2: $\left(\frac{PAA}{PGA}\right)_2$ = ____________

Failure Surface 3: $\left(\frac{PAA}{PGA}\right)_3$ = ____________

C. Find PAA for Each Failure Surface

$(PAA)_1$ = ____________

$(PAA)_2$ = ____________

$(PAA)_3$ = ____________

3. Evaluate k_y/PAA for Each Failure Surface

$$\left(\frac{k_y}{PAA}\right)_1 = \_\_\_\_\_\_\_\_\_\_\_\_\_\_$$

$$\left(\frac{k_y}{PAA}\right)_2 = \_\_\_\_\_\_\_\_\_\_\_\_\_\_$$

$$\left(\frac{k_y}{PAA}\right)_3 = \_\_\_\_\_\_\_\_\_\_\_\_\_\_$$

4. Calculate Permanent Seismic Deformation, PSD, Using Figure 7-4

PSD_1 = ______________

PSD_2 = ______________

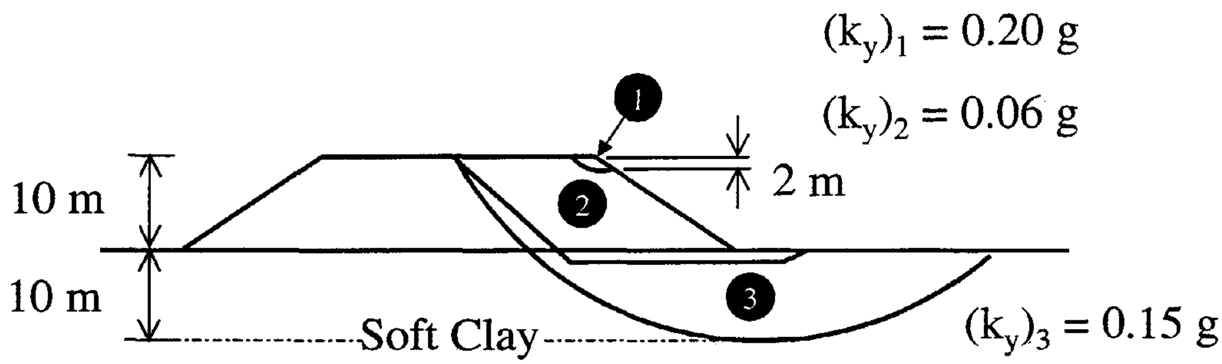
PSD_3 = ______________

5. Calculate Permanent Seismic Deformation, PSD, Using Figure 7-9

PSD_1 = ____________

PSD_2 = ____________

PSD_3 = ____________



Exercise 7 - Figure S7-2

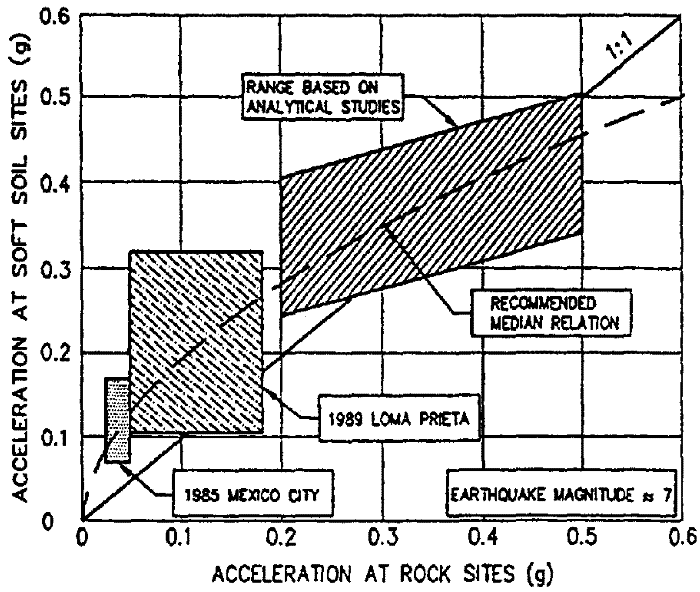


Figure 6-3: Relationship Between PHGA on Rock and on Soft Soil Sites. (Idriss, 1990)

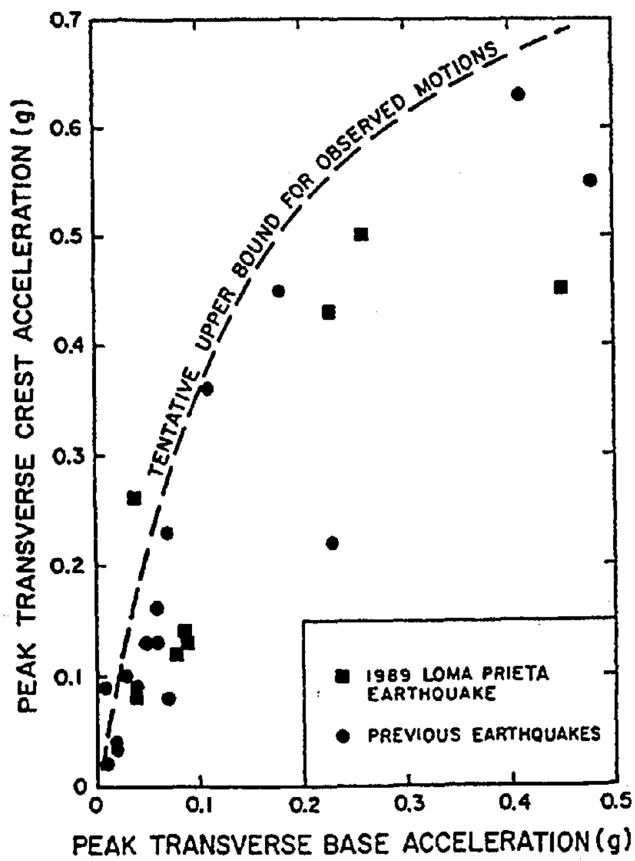


Figure 6-4: Comparisons of Peak Base and Crest Accelerations Recorded at Earth Dams. (Harder, 1991)

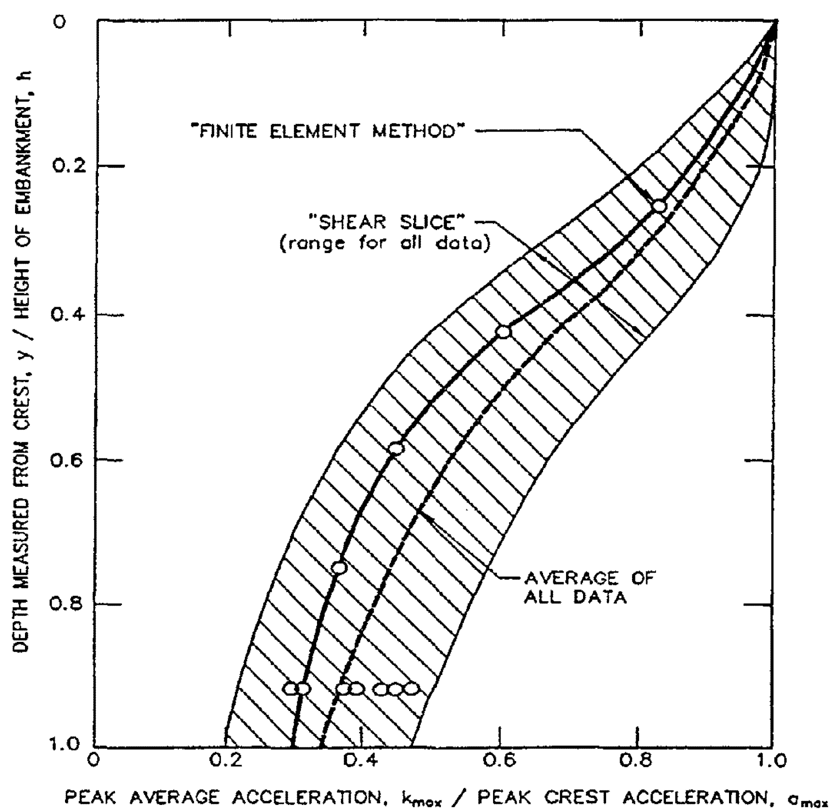


Figure 6-5: Variation of Peak Average Acceleration Ratio with Depth of Sliding Mass. (Makdisi and Seed, 1978, reprinted by permission of ASCE)

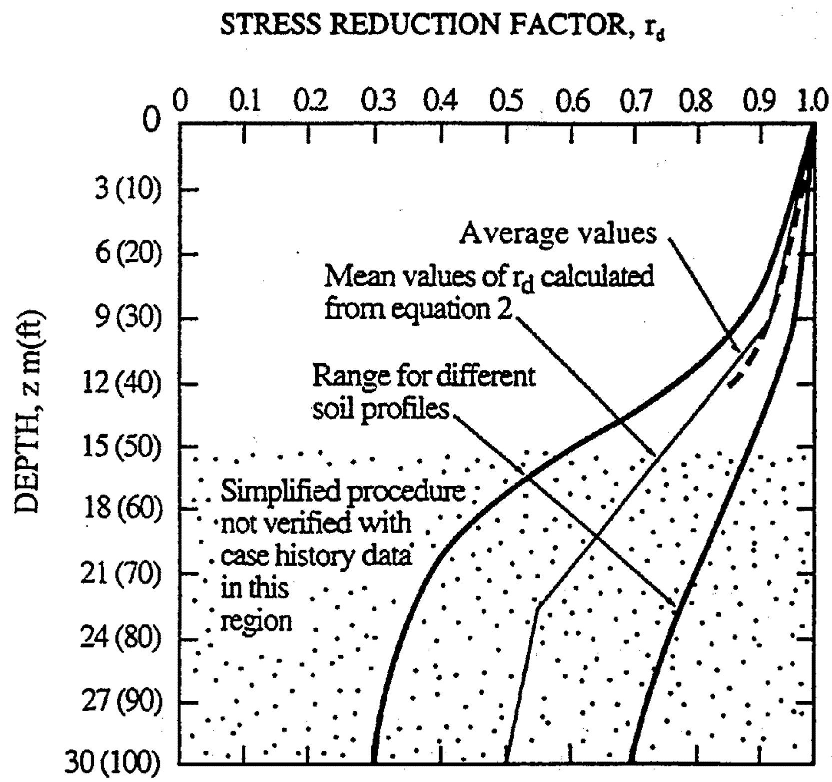


Figure 8-2: Stress Reduction Factor, r_d, Versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean Value Lines from Equation 8-1.

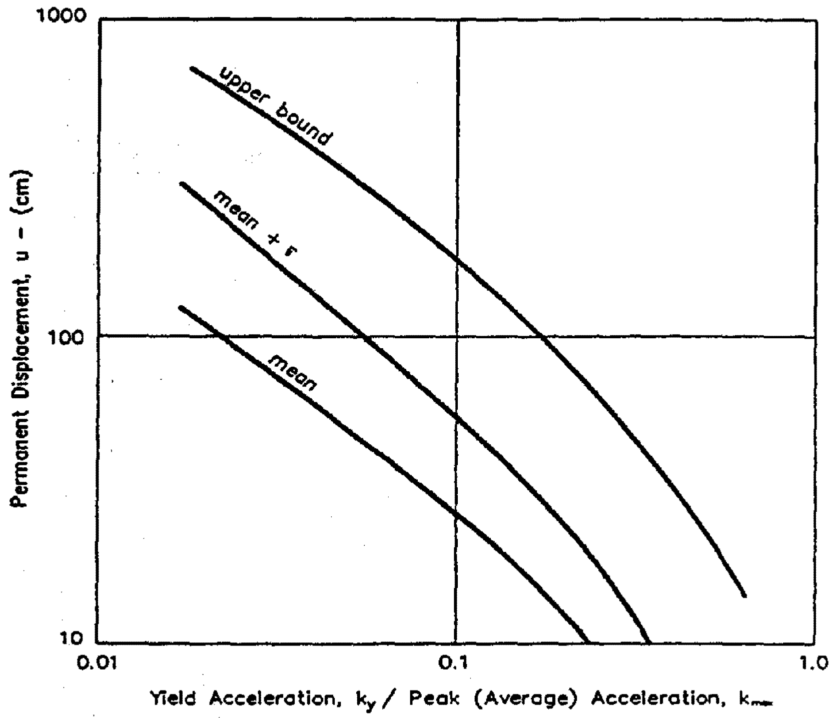


Figure 7-4: Permanent Seismic Deformation Chart. (Hynes and Franklin, 1984, reprinted by permission of U.S. Army Engineer Waterways Experiment Station)

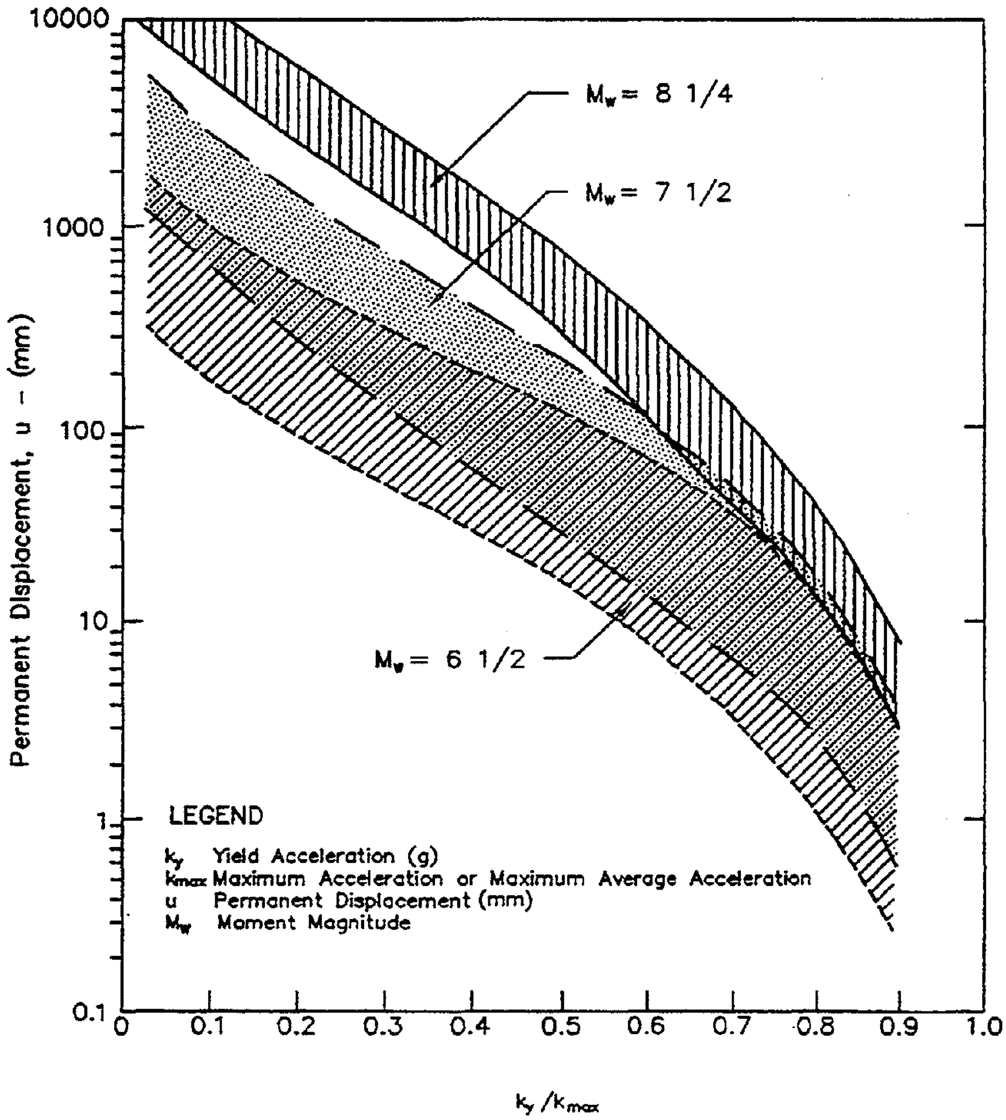


Figure 7-9: Permanent Displacement Versus Normalized Yield Acceleration for Embankments. (After Makdisi and Seed, 1978, reprinted by permission of ASCE).

SOLUTIONS TO EXERCISE 7

1. From Figure 6-4, PGA = 0.56

2. A. $(Z/H)_1 = 2/10 = 0.2$
   $(Z/H)_2 = 10/10 = 1.0$

   B. From Figure 6-5, $(PAA)_1 = 0.56\ (0.85) = 0.48$ g

   From Figure 6-5, $(PAA)_2 = 0.56\ (0.40) = 0.22$ g

   From Equation 8-1, $(PAA)_3 = 0.56\ (0.64) = 0.36$ g

3. $\left(\frac{k_y}{PAA}\right)_1 = \left(\frac{0.2}{0.48}\right) = 0.42$

   $\left(\frac{k_y}{PAA}\right)_2 = \left(\frac{0.06}{0.22}\right) = 0.27$

   $\left(\frac{k_y}{PAA}\right)_3 = \left(\frac{0.12}{0.36}\right) = 0.33$

4. $PSD_1 = < 10$ cm
   $PSD_2 = 10$ cm
   $PSD_3 = < 10$ cm

5. $PSD_1 = 10$ cm
$PSD_2 = 20$ cm
$PSD_3 = 15$ cm

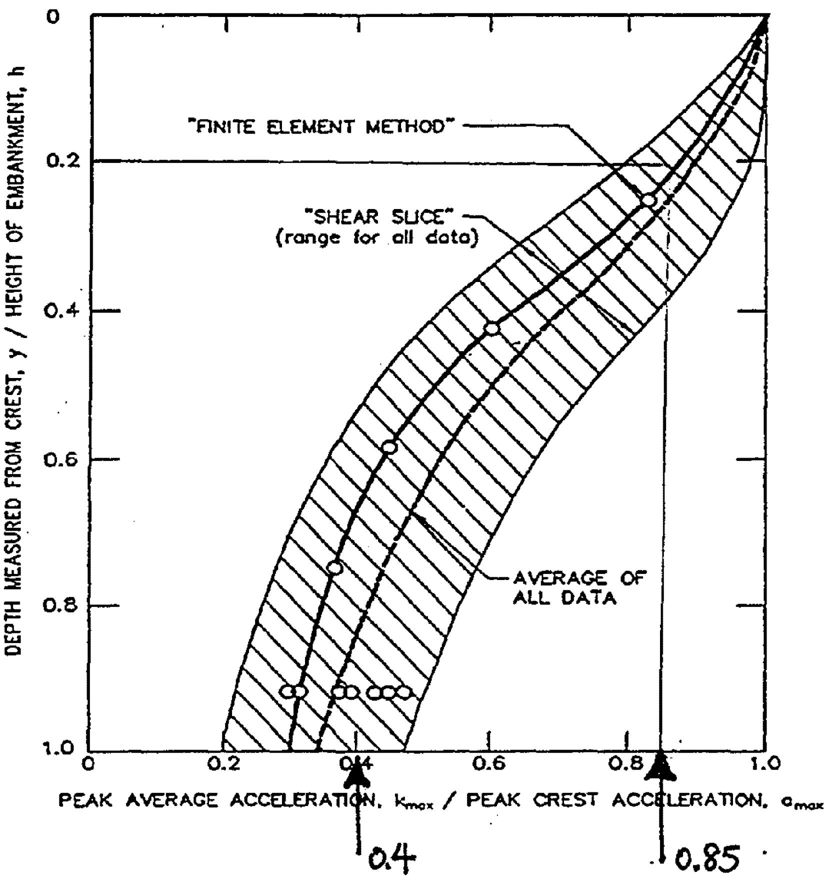


Figure 6-5: Variation of Peak Average Acceleration Ratio with Depth of Sliding Mass. (Makdisi and Seed, 1978, reprinted by permission of ASCE)

STUDENT EXERCISE NO. 8

Stiffness Matrix for Spread Footings.

Objective:

Evaluate the Stiffness Matrix for the Center Pier of the Bridge Shown in Figure S8-1. The Pier Footing is 25 m in Length, 4.25 m Wide, and 1 m Thick. The Soil Profile at the Site is Shown in Figure S8-2. The Design Earthquake is a Moderate Magnitude (M_w = 6.5) Event.

Source Materials:

Reference Manual Part I: Figures 5-12, 5-13, 5-14, 9-7 9-8 and 9-9, Tables 5-5 and 9-2, and Equations 9-11 and 9-12a through 9-12d.

1. Evaluate Equivalent Circular Radius Using Equations in Table 9-2

   A. Translational Modes

   R_z = ____________

   R_x = ____________

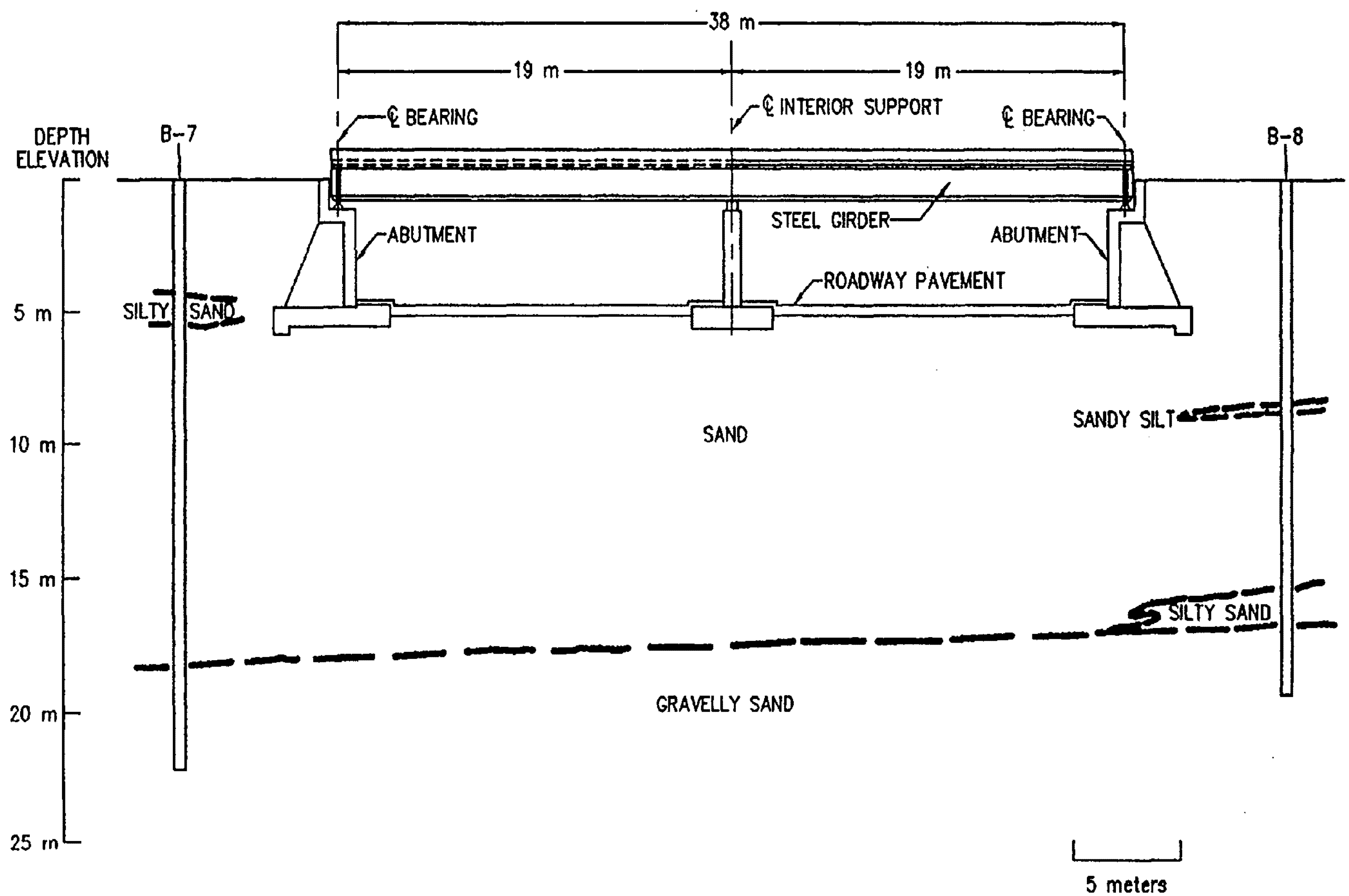


Figure S8-1: Cross Section of Proposed Bridge and Soil Stratigraphy.

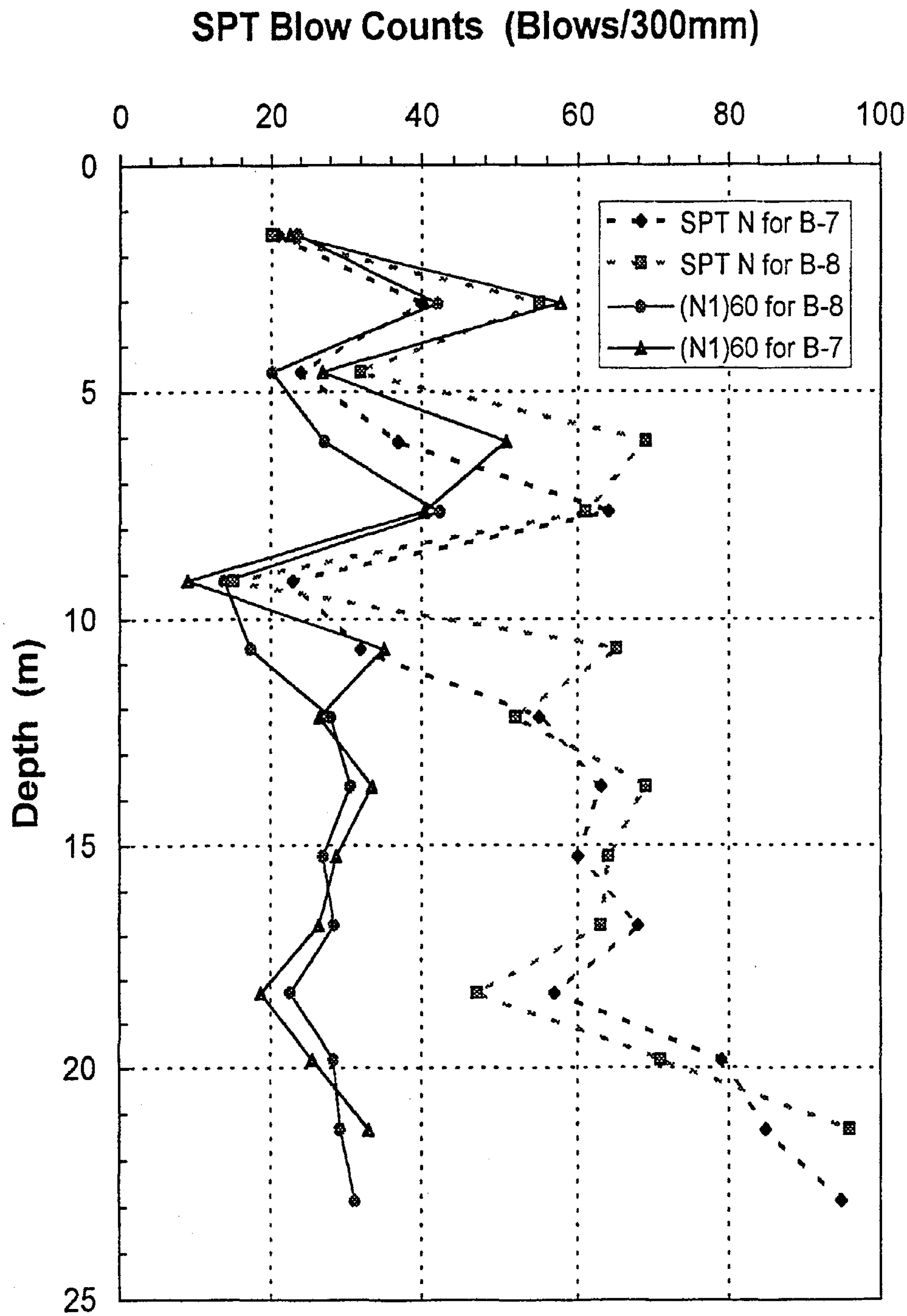


Figure S8-2: Corrected SPT Results for Example No. 8 Test Borings.

B. Rotational Modes

$R_{\psi x}$ = ______________ (X-Axis Rocking)

$R_{\psi y}$ = ______________ (Y-Axis Rocking)

$R_{\psi z}$ = ______________ Torsion

2. Evaluate Maximum Shear Modulus Using Imai and Tonouchi Equation from Table 5-5

N_{60} = ______________ (From Figure S8-2)

G_{max} = ______________ (From Table 5-5)

3. Reduce Maximum Shear Modulus For Strain Softening Using Figures 5-12 and/or 5-13

Moderate Magnitude Event - Assume $\gamma = 0.02\%$

$\left(\frac{G}{G_{max}}\right)$ = ______________

G = ______________

4. Calculate Stiffness Coefficients for Equivalent Circular Footing Using Equations 9-12a through 9-12d

ν = ____________ (Section 5.3.3)

k_z = ____________ (Equation 9-12a)

$k_x = k_y$ = ____________ (Equation 9-12b)

$k_{\psi x}$ = ____________ (Equation 9-12d)

$k_{\psi y}$ = ____________ (Equation 9-12d)

$k_{\psi z}$ = ____________ (Equation 9-12c)

5. Calculate Rectangular Footing Stiffness Using Equation 9-11, Figures 9-7 and 9-8

α_z = _______________ (Figure 9-7)

α_x = _______________ (Figure 9-7)

α_y = _______________ (Figure 9-7)

$\alpha_{\psi x}$ = _______________ (Figure 9-7)

$\alpha_{\psi y}$ = _______________ (Figure 9-7)

$\alpha_{\psi z}$ = _______________ (Figure 9-7)

b_z = _______________ (Figure 9-8)
b_x = _______________ (Figure 9-8)
b_y = _______________ (Figure 9-8)
$b_{\psi x}$ = _______________ (Figure 9-8)
$b_{\psi y}$ = _______________ (Figure 9-8)
$b_{\psi z}$ = _______________ (Figure 9-8)

$k_z = \alpha_z \, \beta_z \, k_z =$ ________________

$k_x = \alpha_z \, \beta_z \, k_z =$ ________________

$k_y = \alpha_z \, \beta_z \, k_z =$ ________________

$k_{\psi x} = \alpha_{\psi z} \, \beta_{\psi z} \, k_{\psi z} =$ ________________

$k_{\psi y} = \alpha_{\psi z} \, \beta_{\psi z} \, k_{\psi z} =$ ________________

$k_{\psi z} = \alpha_{\psi z} \, \beta_{\psi z} \, k_{\psi z} =$ ________________

TABLE 5-5
CORRELATIONS FOR ESTIMATING INITIAL SHEAR MODULUS

Reference	Correlation	Units	Limitation
Seed, *et al.* (1984)	$G_{max} = 220\ (K_2)_{max}\ (\sigma'_m)^{1/2}$ $(K_2)_{max} \approx 20(N_1)_{60}^{1/3}$	kPa	$(K_2)_{max} \approx 30$ for very loose sands and 75 for very dense sands; $\approx$ 80-180 for dense well graded gravels; Limited to cohesionless soils
Imai and Tonouchi (1982)	$G_{max} = 15{,}560\ N_{60}^{0.68}$	kPa	Limited to cohesionless soils
Hardin (1978)	$G_{max} = \dfrac{625}{(0.3 + 0.7 e_o^2)} (P_a \cdot \sigma'_m)^{0.5} OCR^{\,k}$	kPa[1][3]	Limited to cohesive soils P_a = atmospheric pressure
Jamiolkowski, *et al.* (1991)	$G_{max} = \dfrac{625}{e_o^{1.3}} (P_a \cdot \sigma'_m)^{0.5}\ OCR^{\,k}$	kPa[1][3]	Limited to cohesive soils P_a = atmospheric pressure
Mayne and Rix (1993)	$G_{max} = 99.5 (P_a)^{0.305} (q_c)^{0.695} / (e_o)^{1.13}$	kPa[2]	Limited to cohesive soils P_a = atmospheric pressure

Notes: [1] P_a and σ'_m in kPa
[2] P_a and q_c in kPa
[3] The parameter k is related to the plasticity index, PI, as follows:

PI	k
0	0
20	0.18
40	0.30
60	0.41
80	0.48
>100	0.50

TABLE 9-2
EQUIVALENT DAMPING RATIOS FOR RIGID CIRCULAR FOOTINGS
(After Richart, *et al.*, 1970)

Mode of Vibration	Mass (or Inertia) Ratio	Damping Coefficient	Damping Ratio	Equivalent Radius
Vertical Translation	$B_z = \frac{(1-v)}{4} \frac{m}{\rho r_o^3}$	$c_z = \frac{3.4\, r_o^2}{1-v} \sqrt{\rho G}$	$D_z = \frac{0.425}{\sqrt{B_z}}$	$r_o = R_z = \sqrt{BL/\pi}$
Horizontal Translation (Sliding)	$B_x = \frac{(7-8v)}{32(1-v)} \frac{m}{\rho r_o^3}$	$c_x = \frac{4.6\, r_o^2}{2-v} \sqrt{\rho G}$	$D_x = \frac{0.288}{\sqrt{B_x}}$	$r_o = R_x = \sqrt{BL/\pi}$
X- and Y-axis Rocking	$B_\psi = \frac{3(1-v)}{8} \frac{I_\psi}{\rho r_o^5}$	$c_\psi = \frac{0.8\, r_o^4 \sqrt{\rho G}}{(1-v)(1+B_\psi)}$	$D_\psi = \frac{0.15}{(1+B_\psi)\sqrt{B_\psi}}$	$r_o = R_{\psi_x} = \left[\frac{16(B)(L)^3}{3\pi}\right]^{1/4}$ $r_o = R_{\psi_y} = \left[\frac{16(B)^3(L)}{3\pi}\right]^{1/4}$
Z-axis Rotation (Torsion)	$B_\theta = \frac{I_\theta}{\rho r_o^5}$	$c_\theta = \frac{4\sqrt{B_\theta \cdot \rho G}}{1+2B_\theta}$	$D_\theta = \frac{0.5}{1+2B_\theta}$	$r_o = R_{\psi_t} = \left[\frac{16BL(B^2+L^2)}{6\pi}\right]^{1/4}$

Notes:
- m = mass of the foundation
- c = damping coefficient (c_z, c_x, c_ψ, c_θ)
- I = moment of inertia of the foundation
- ρ = mass density of foundation soil
- r_o = equivalent radius (R_x, R_z, R_ψ)
- B = width of the foundation (along axis of rotation for rocking)
- L = length of the foundation (in the plane of rotation for rocking)
- G = shear modulus of the soil
- v = Poisson's ratio of the soil
- D = damping ratio (D_z, D_x, D_ψ, D_θ)

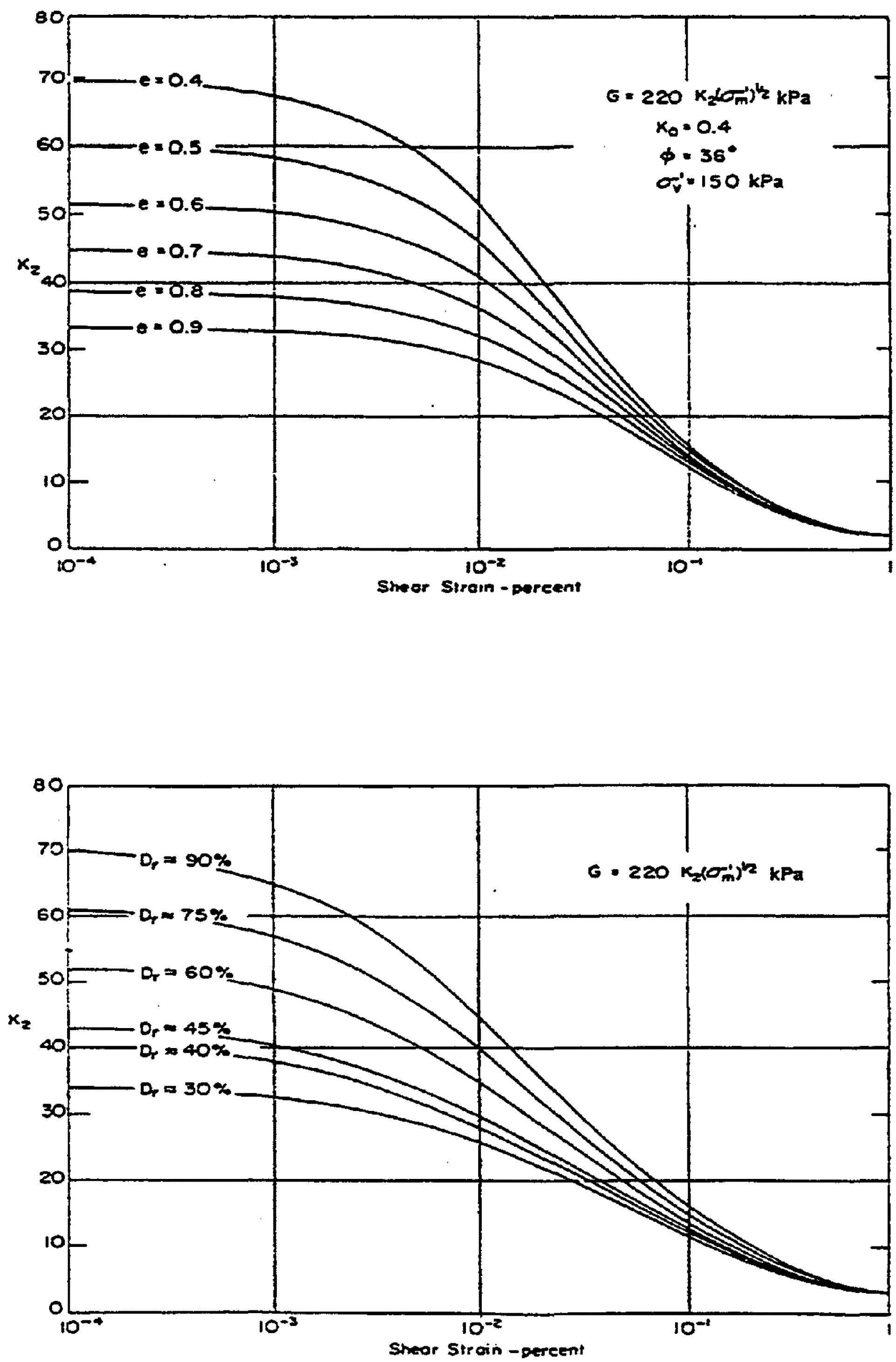


Figure 5-12: Shear Modulus Reduction Curves for Sands. (Seed and Idriss, 1970, reprinted by permission of ASCE)

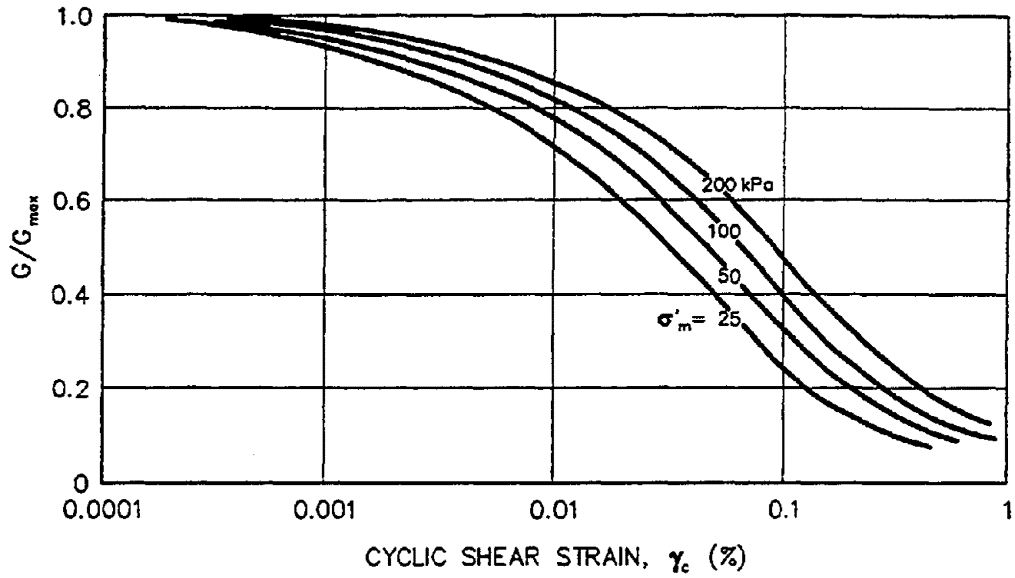


Figure 5-13: Shear Modulus Reduction Curves for Sands. (Iwasaki, *et al.*, 1978, reprinted by permission of Japanese Society of Soil Mechanics and Foundation Engineering)

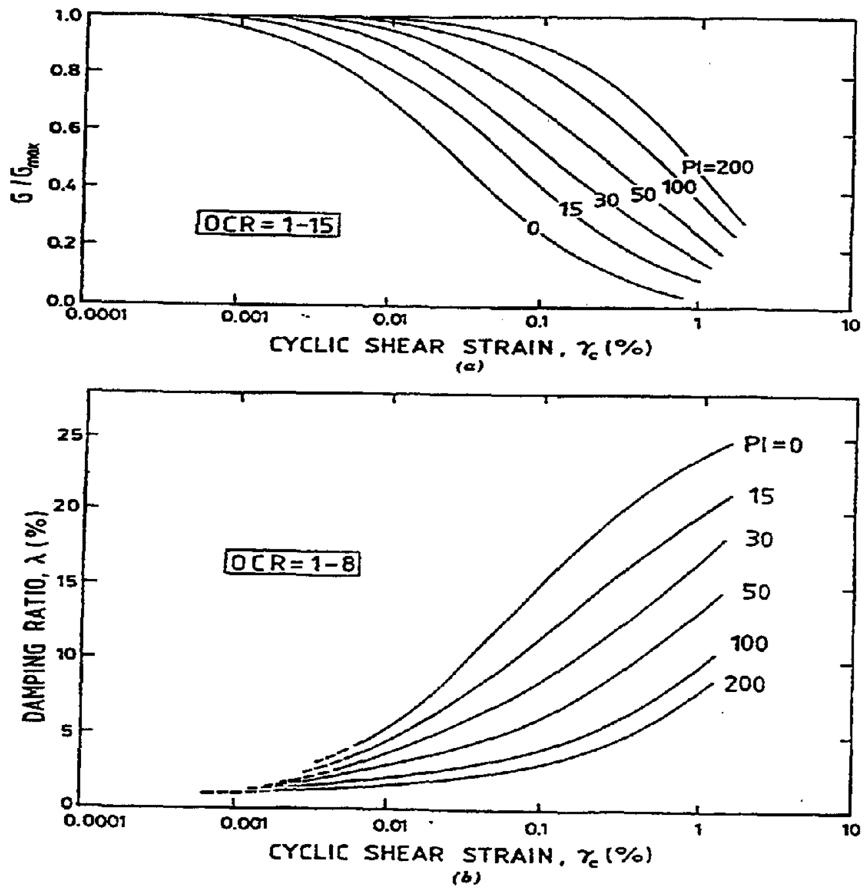


Figure 5-14: Shear Modulus Reduction and Damping Ratio as a Function of Shear Strain and Soil Plasticity Index. (Vucetic and Dobry, 1991, reprinted by permission of ASCE)

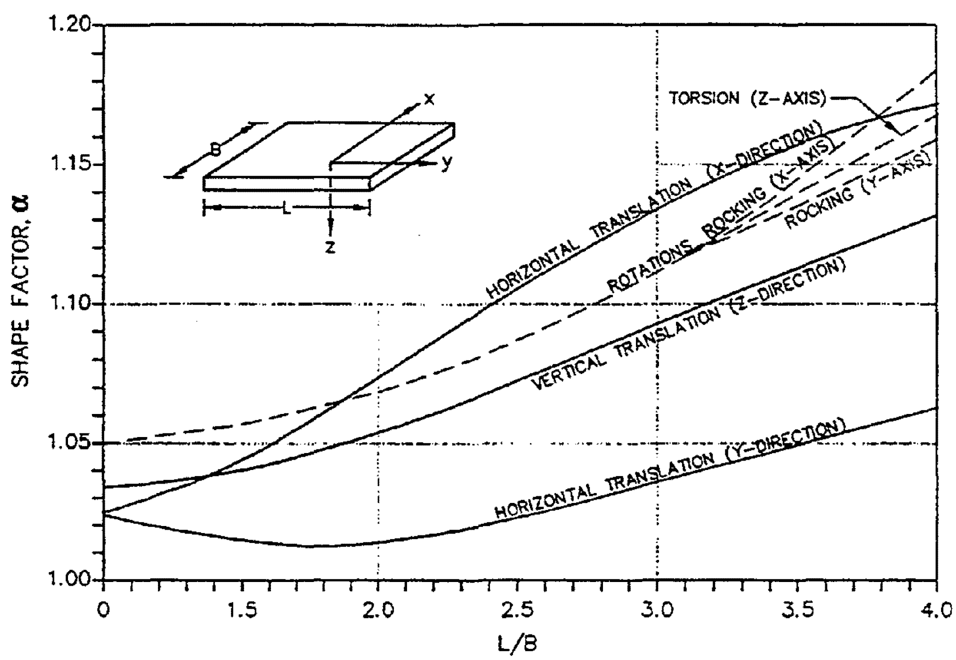


Figure 9-7: Shape Factor α for Rectangular Footings. (Lam and Martin, 1986)

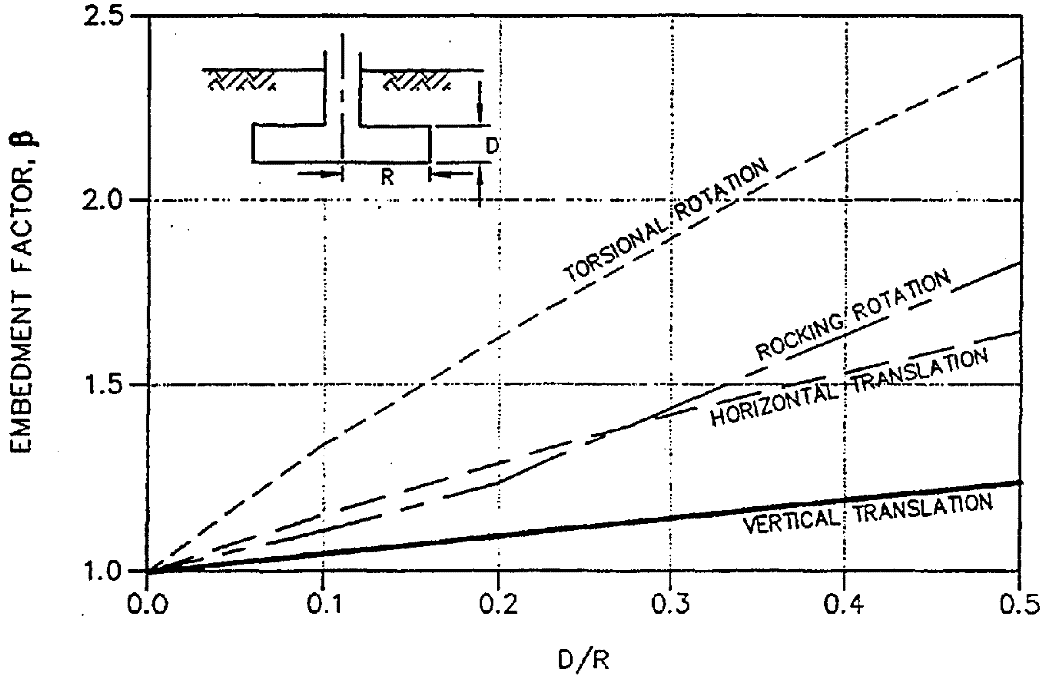


Figure 9-8: Embedment Factors for Footings with D/R < 0.5. (Lam and Martin, 1986)

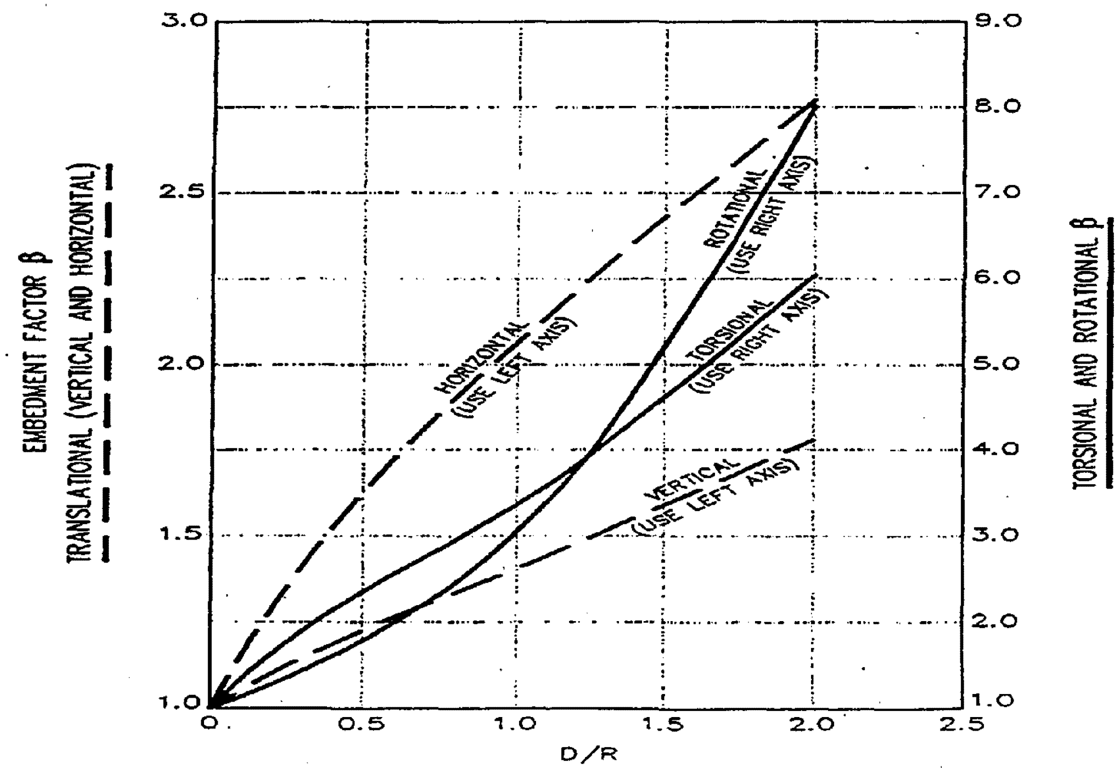


Figure 9-9: Embedment Factors for Footings with D/R > 0.5. (Lam and Martin, 1986)

SOLUTIONS TO EXERCISE NO. 8

1. Equivalent Radius

$$R_z = R_y = R_x = \sqrt{\frac{BL}{\pi}} = 5.82 \text{ m}$$

$$R_{\psi x} = \left[\frac{16(B)(L)^3}{3\pi}\right]^{1/4} = 18.32 \text{ m}$$

$$R_{\psi y} = \left[\frac{16(B)^3 L}{3\pi}\right]^{1/4} = 7.56 \text{ m}$$

$$R_{\psi z} = \left[\frac{16BL(B^2 + L^2)}{6\pi}\right]^{1/4} = 15.52 \text{ m}$$

2. Small Strain Shear Modulus

$(N_{60}) = 28$

$G_{max} = 15{,}560 \; N_{60}^{0.68} = 150{,}000$ kPa

3. Modulus Reduction

$\left(\frac{G}{G_{max}}\right) = 0.6$

$G = 90$ MPa

4. Stiffness Coefficients for Equivalent Circle

MODE	Equivalent Circular Stiffness Mpa - m	α	β	Stiffness Mpa - m
Vertical	3,260	1.13	1.09	4,015
Translation (X-Direction)	2,568	1.17	1.26	3,785
Translation (Y-Direction)	2,568	1.07	1.26	3,430
Rocking (X-Axis)	2,295,478	1.18	1.1	2,980,000
Rocking (Y-Axis)	161,310	1.16	1.2	225,000
Torsion	1,814,326	1.17	1.3	2,760,000

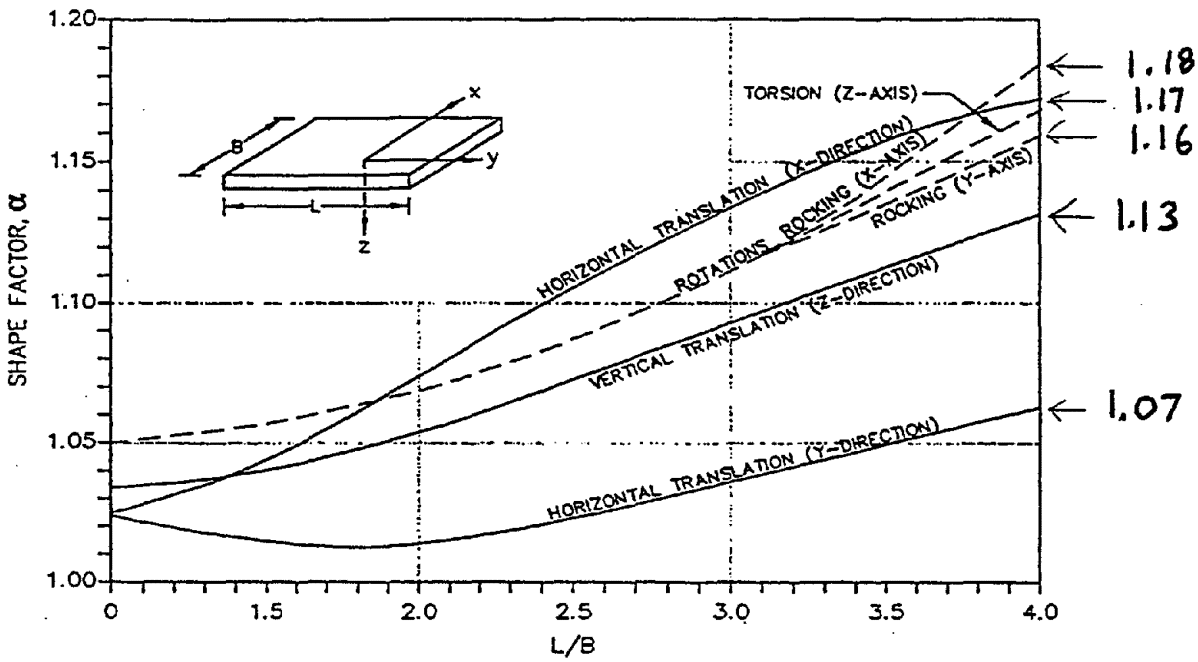


Figure 9-7: Shape Factor α for Rectangular Footings. (Lam and Martin, 1986)

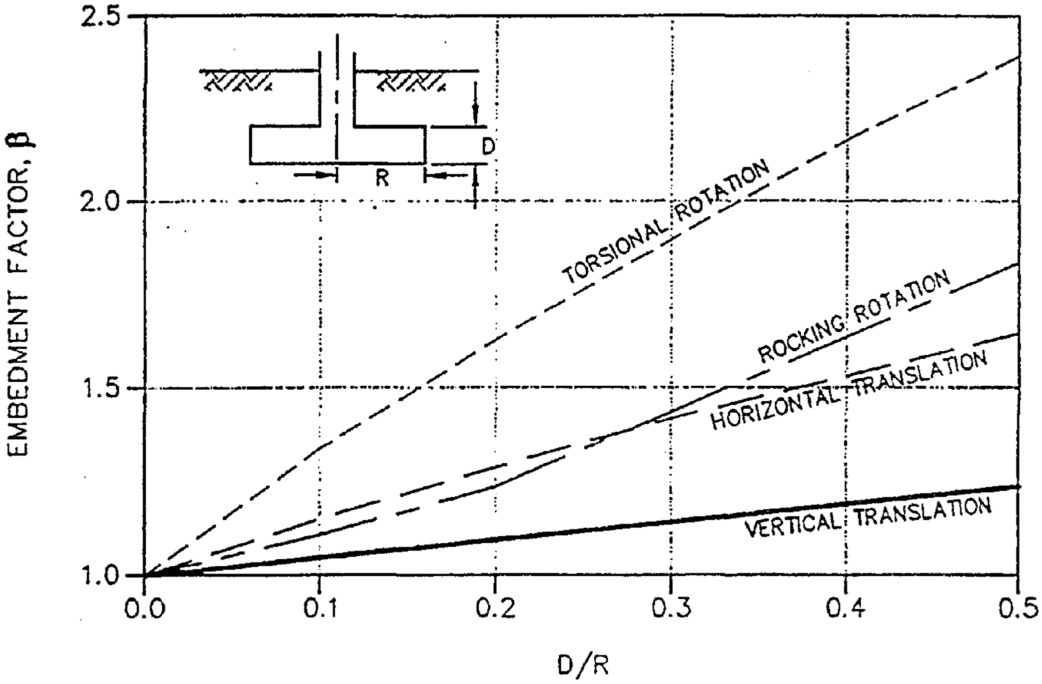


Figure 9-8: Embedment Factors for Footings with D/R < 0.5. (Lam and Martin, 1986)

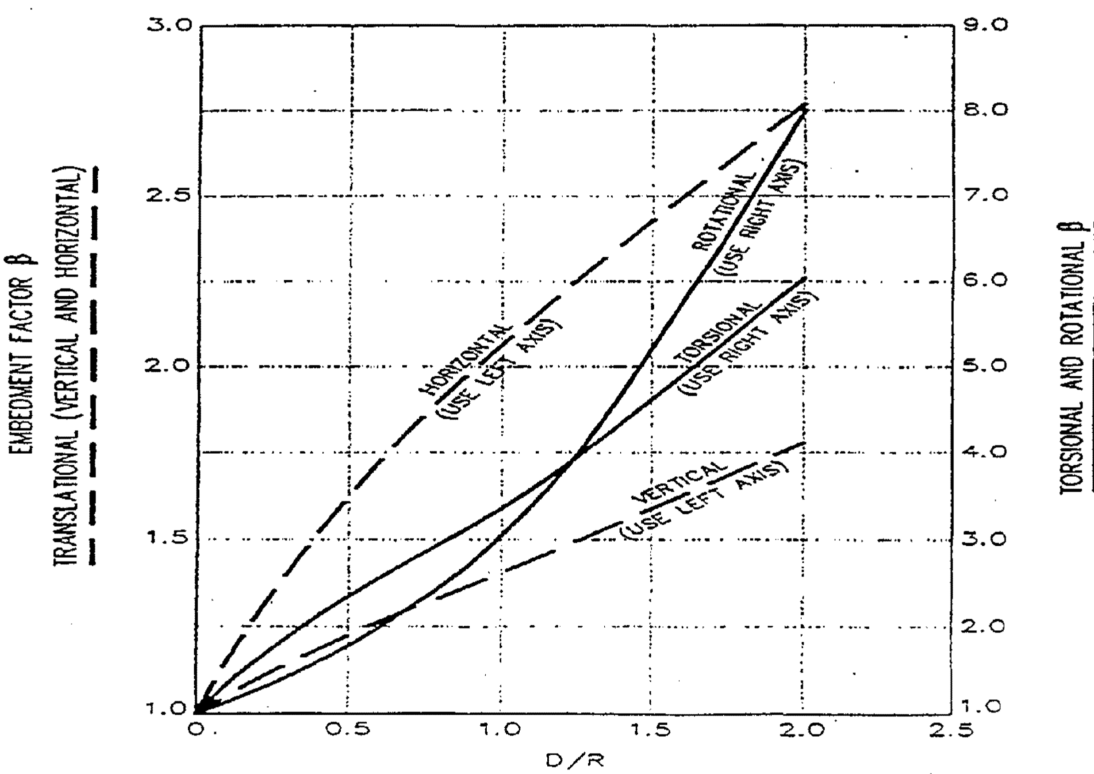


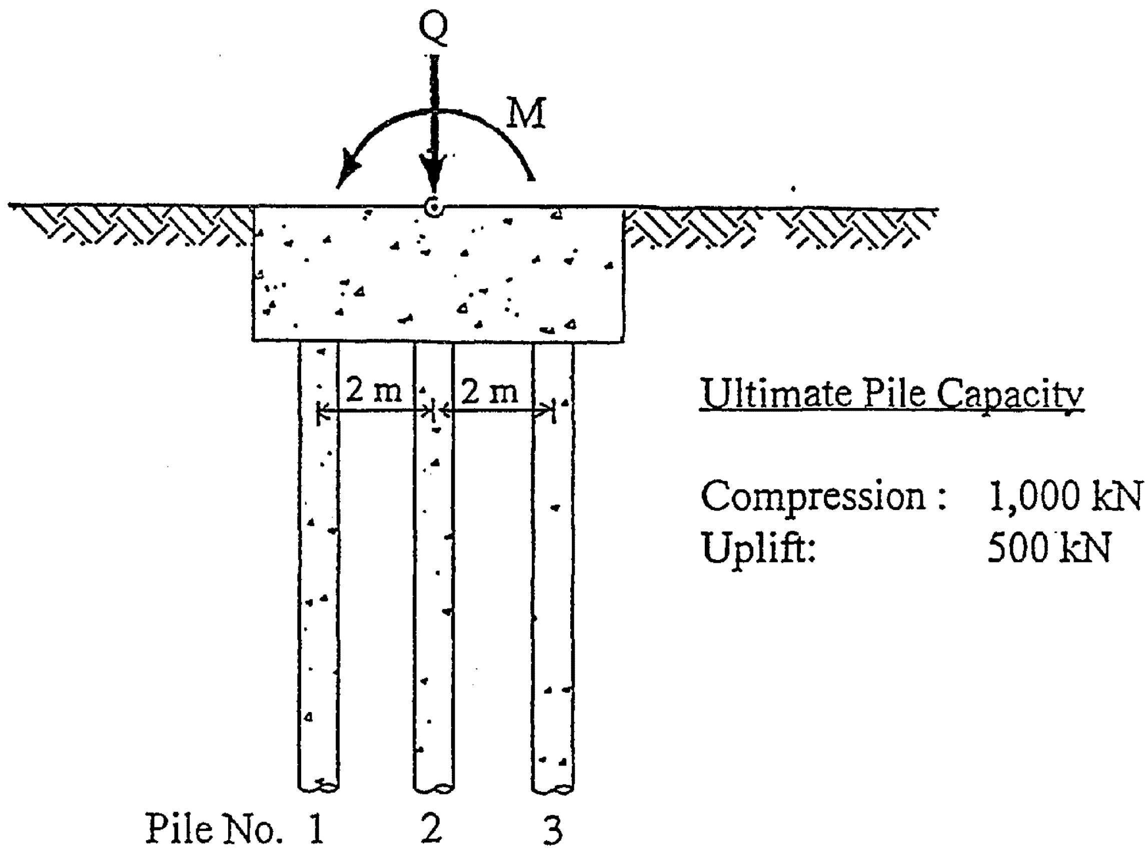
Figure 9-9: Embedment Factors for Footings with D/R > 0.5. (Lam and Martin, 1986)

STUDENT EXERCISE NO. 9A

Geotechnical Pile Capacity Evaluation Under Seismic Loading.

Objective:
Evaluate the Effects of Seismic Loading on the Geotechnical Capacity of a Pile Group Foundation, as Shown in Figure S9A-1.

Source Materials:
Reference Manual Part II: Step 3 in Section 3.4



$$p = \frac{Q}{n} \pm \frac{M \cdot d_i}{\sum_{i=1}^{n} d_i^2}$$

p =vertical load on pile;
Q =static vertical load on pile cap;
n =number of piles in group (3 for this example);
d_i =distance from center of gravity of pile group to pile i;
M = design moment.

Figure S9A-1: Pile Group Foundation

Static Case

$Q = 1{,}500\ \text{kN}, \qquad M = 0$

Maximum Vertical Load on Pile $= \dfrac{1{,}500\text{kN}}{3} = 500\ \text{kN}$

(Compression)

$$F.S._{(\text{compression})} = \frac{1{,}000\,\text{kN}}{500\,\text{kN}} = \underline{\quad 2.0 \quad}$$

Pile No.	Compression (kN)	Uplift (kN)	FS
1	500	-	2.0
2	500	-	2.0
3	500	-	2.0

Seismic Load Case 1

Q=1,500 kN,   M=3,000 m-kN

Pile No.	Compression (kN)	Uplift (kN)	FS
1			
2			
3			

Seismic Load Case 2

Q=300 kN, M=3,000 m-kN

Pile No.	Compression (kN)	Uplift (kN)	FS
1			
2			
3			

SOLUTIONS TO EXERCISE NO. 9A

Seismic Load Case 1

Q=1,500 kN,   M=3,000 m-kN

- Seismic Load on Piles 1, 2 and 3

$$P_1 = \frac{1{,}500}{3} + \frac{3{,}000 \times 2}{2^2 + 2^2} = 500 + 750 = 1{,}250 \text{ kN}$$

$$P_2 = \frac{1{,}500}{3} + 0 = 500 \text{ kN}$$

$$P_3 = \frac{1{,}500}{3} - \frac{3{,}000 \times 2}{2^2 + 2^2} = 500 - 750 = -250 \text{ kN}$$

Pile No.	Compression (kN)	Uplift (kN)	FS
1	1,250	-	$\frac{1{,}000}{1{,}250} = 0.8$
2	500	-	$\frac{1{,}000}{500} = 2.0$
3	-	-250	$\frac{500}{250} = 2.0$

- Pile 1: Compression Failure (FS < 1.0)

Seismic Load Case 2

Q=300 kN, M=3,000 m-kN

$$P_1 = \frac{300}{3} + \frac{3,000 \times 2}{2^2 + 2^2} = 100 + 750 = 850 \text{ kN}$$

$$P_2 = \frac{300}{3} + 0 = 100 \text{ kN}$$

$$P_3 = \frac{300}{3} - \frac{3,000 \times 2}{2^2 + 2^2} = 100 - 750 = -650 \text{ kN}$$

Pile No.	Compression (kN)	Uplift (kN)	FS
1	850	-	$\frac{1,000}{850} = 1.18$
2	100	-	$\frac{1,000}{100} = 10$
3	-	-650	$\frac{500}{650} = 0.77$

- Pile 3: Uplift Failure (FS < 1.0)

STUDENT EXERCISE NO. 9B

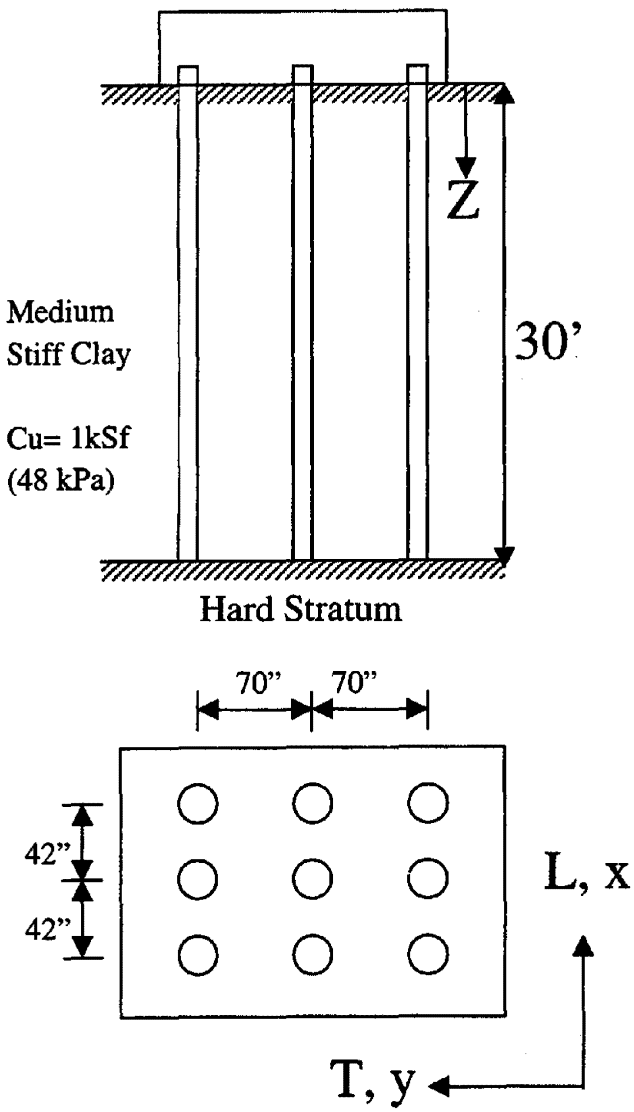
Derivation of Foundation Stiffness for Pile Group

Objective:

Evaluate the Foundation Stiffness for a Pile Group Shown in Figure S9B-1 Using Simplified Design Charts. The Piles Are End Bearing Piles on a Hard Stratum. The Soil Overburden at the Site Consists of 30 ft (9.15 m) of Medium Stiff Clay.

Source Materials:

Reference Manual Part I: Section 9.3.6, Table 9-5, Figures 9-20, 9-21 and 9-22.



- 14-in diameter vertical cast-in-place concrete piles.
- End bearing piles on Very Hard Stratum.
- Pile Length = 30 ft.
- Pile head 6" into pile cap.
- Pile group layout 3 x 3 as shown
- Center-to-center pile spacing
  s = 42" = 3d (Longitudinal)
  s = 70" = 5d (Transverse)

- $A = \Pi \times (14)^2 \times \frac{1}{4} = 154\ in^2$
- $I = \Pi \times (14)^4 \times \frac{1}{64} = 1885\ in^4$
- L = 30 ft
- E = 3,600 ksi

Figure S9B-1: Pile Group Foundation

Step 1: Solve for the stiffness of a single pile under lateral loading.

First, determine pile head boundary condition. Assume pile head has a hinged end condition due to small pile head embedment into the pile cap (i.e., a free-headed pile analysis).

A. Determine the coefficient of variation of subgrade modulus for clay using Figure 9-20:

f = ____________

B. Consider the pile group effect. Estimate the overall stiffness reduction factor (i.e., p-multiplier) using Table 9-5:

Pile Spacing in Longitudinal Direction = 3d
Pile Spacing in Transverse Direction = 5d
(Note: d is the diameter of the pile.)

$(\text{p-multiplier})_{ave}$ in Longitudinal Direction = ________
$(\text{p-multiplier})_{ave}$ in Transverse Direction = ________

C. Determine effective coefficient of subgrade modulus $f_{eff} = (\text{p-multiplier})_{ave} \bullet f$:

Longitudinal: $f_{eff,\,L}$ = ____________

Transverse: $f_{eff,\,T}$ = ____________

D. Calculate the Bending Stiffness of the Pile

EI = ____________

E. Derive lateral stiffness for free-head condition using Figure 9-21:

$k'_{\delta,L}$ = ____________ (Longitudinal)

$k'_{\delta,T}$ = ____________ (Transverse)

Step 2: Solve for the stiffness of a single pile under axial loading.

For end bearing piles (ignore skin friction), calculate the axial stiffness.

$$k_v = \frac{AE}{L} = \underline{\hspace{4cm}}$$

Step 3: Calculate pile group stiffness by combining the stiffness contribution of each individual pile.

A. Lateral Stiffness

$$K_L = n \cdot k'_{\delta,L} = \underline{\hspace{4cm}}$$

$$K_T = n \cdot k'_{\delta,T} = \underline{\hspace{4cm}}$$

Note: n is the total number of piles.

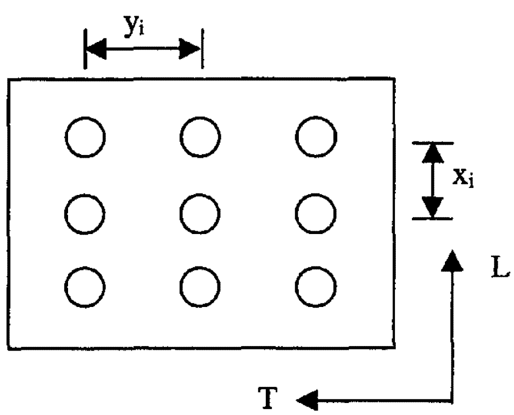
B. Axial Stiffness

$$K_V = n \cdot k_V = \underline{\hspace{4cm}}$$

C. Torsional Stiffness

$$K_{TOR} = \sum_{i=1}^{n} k'_{\delta,L} \cdot y_i^2 + \sum_{i=1}^{n} k'_{\delta,T} \cdot x_i^2$$

$$= \_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_$$



D. Rocking Rotational Stiffness about Transverse Axis.

$$K_{RT} = k_V \cdot \sum_{i=1}^{n} x_i^2$$

$$= \_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_$$

E. Rocking Rotational Stiffness about Longitudinal Axis.

$$K_{RL} = k_V \cdot \sum_{i=1}^{n} y_i^2$$

$= \_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_\_$

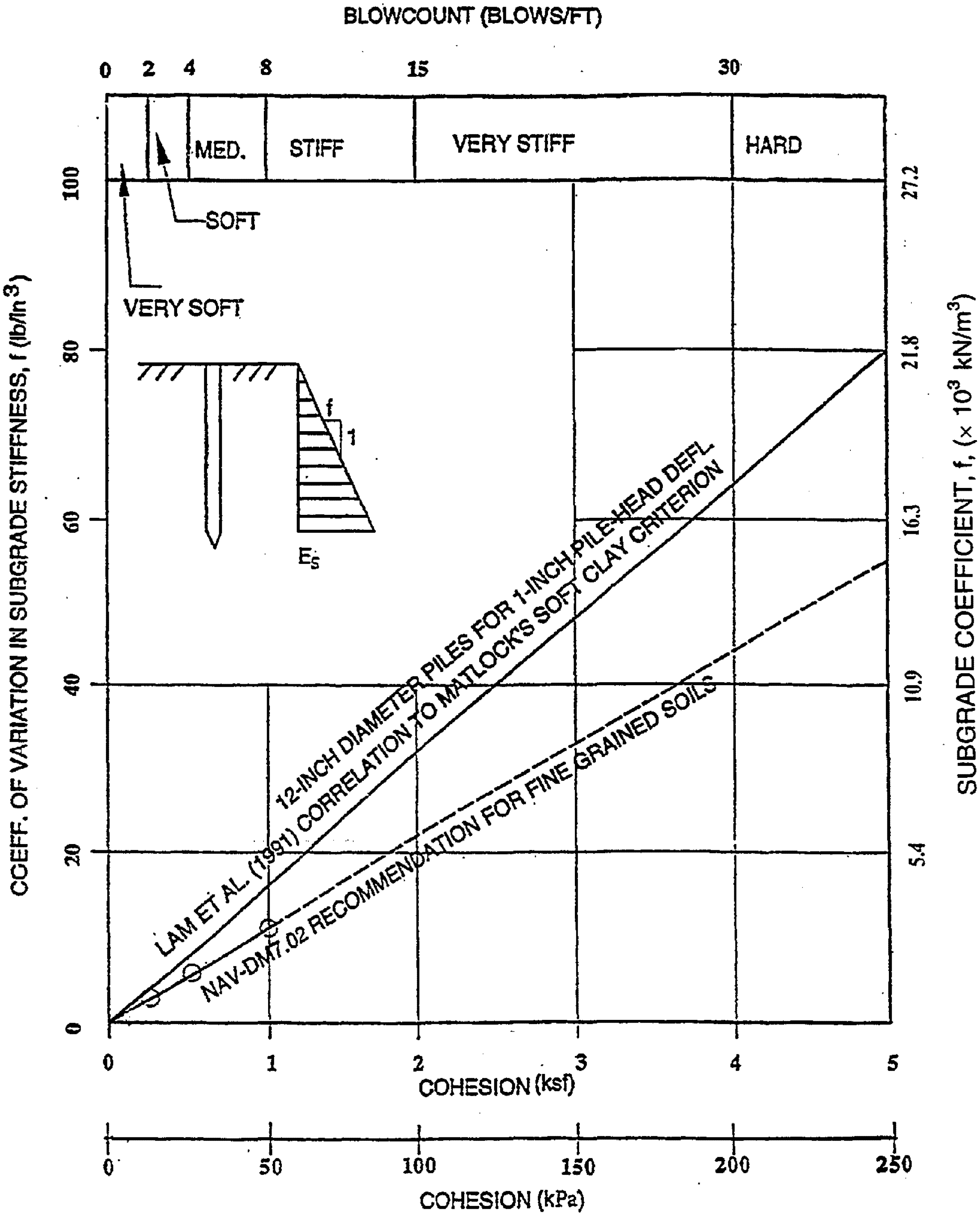
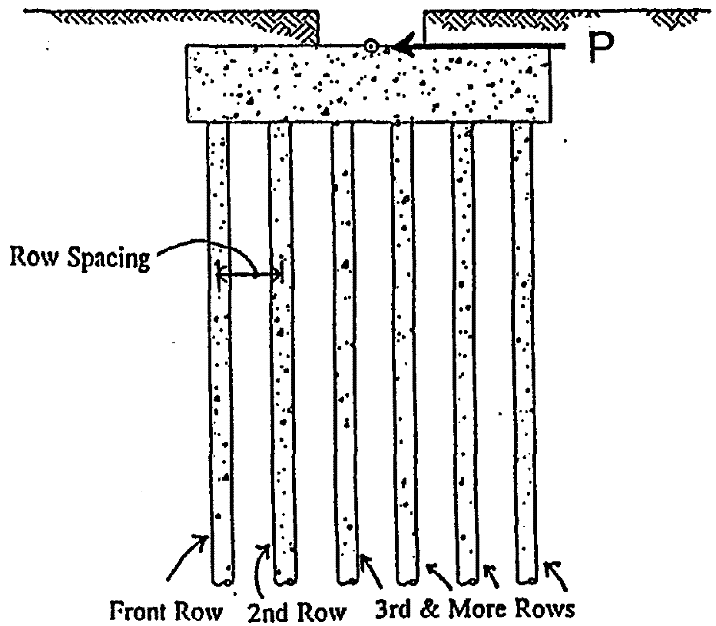
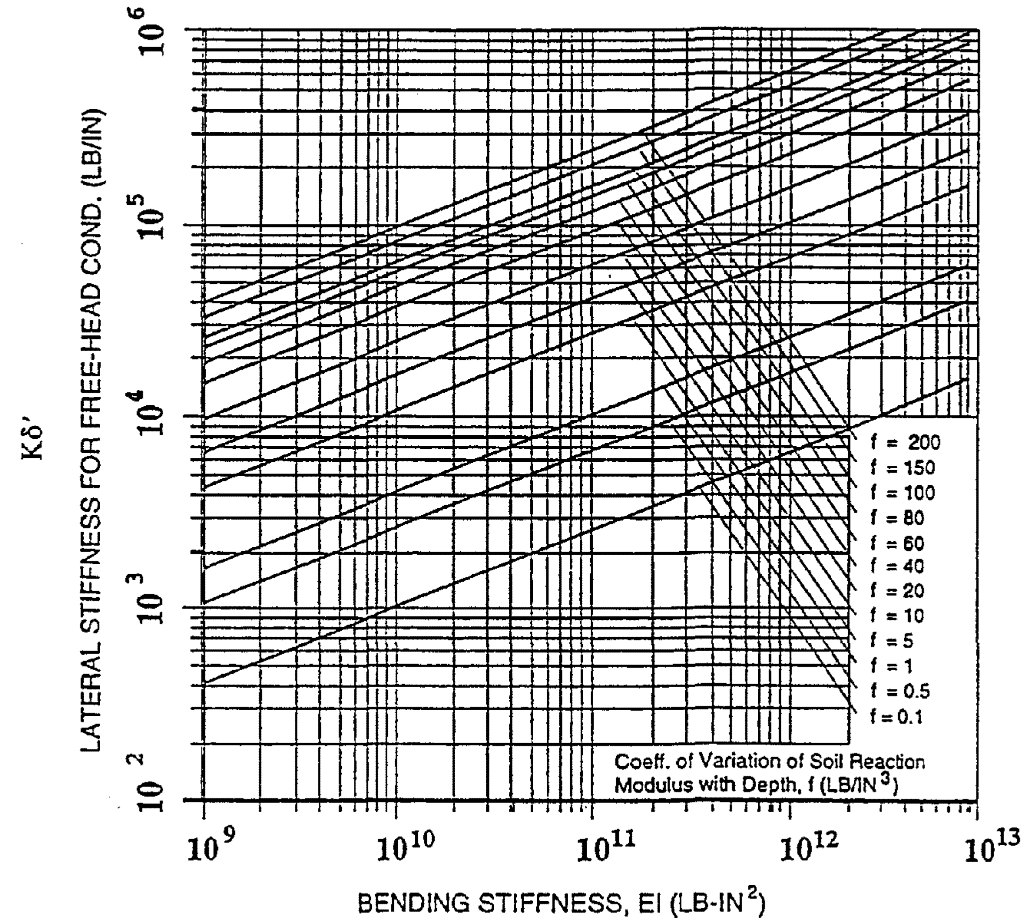


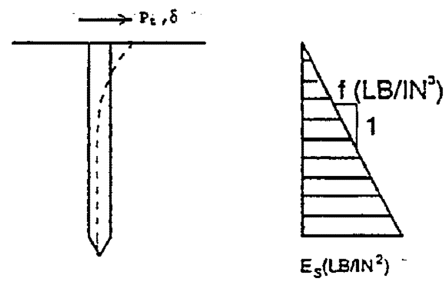
Figure 9-20: Coefficient of Variation of Subgrade Modulus for Clay.

Recommended p-Multiplier For Group Effects



Row Spacing	Front Row	2nd Row	3rd & More Rows
3D	0.8	0.45	0.35
4D	0.9	0.65	0.55
5D	1.0	0.85	0.75





FREE HEAD PILE STIFFNESS $K\delta'$

$$= K_\delta - \frac{K_{\delta\theta}^2}{K_\theta}$$

$$= 0.41 \frac{E \cdot I}{T^3}$$

$$T = \left(\frac{E \cdot I}{f}\right)^{1/5}$$

1 lb/in³ = 272 kN/m³
1 ksf = 48 kPa
1 lb/in = 0.1752 kN/m
1 lb-in² = 2.87x10⁻⁶ kN-m²

Figure 9-21: Lateral Stiffness of Free-Headed Piles.

SOLUTIONS TO EXERCISE NO. 9B

Step 1: Solve for the stiffness of a single pile under lateral loading.

First, determine Pile Head Boundary Condition. Assume pile head has a hinged end condition due to small pile head embedment into the pile cap (i.e., a free-headed pile analysis).

A. Determine the coefficient of variation of subgrade modulus for clay using Figure 9-20:

$$f = \underline{16 \text{ lbs/in}^3}$$

B. Consider the pile group effect. Estimate the overall stiffness reduction factor (i.e., p-multiplier) using Table 9-5:

$(\text{p-multiplier})_{ave}$ in Longitudinal Direction =

$$\underline{(0.8 + 0.45 + 0.35) / 3 = 0.53}$$

$(\text{p-multiplier})_{ave}$ in Transverse Direction =

$$\underline{(1.0 + 0.85 + 0.75) / 3 = 0.87}$$

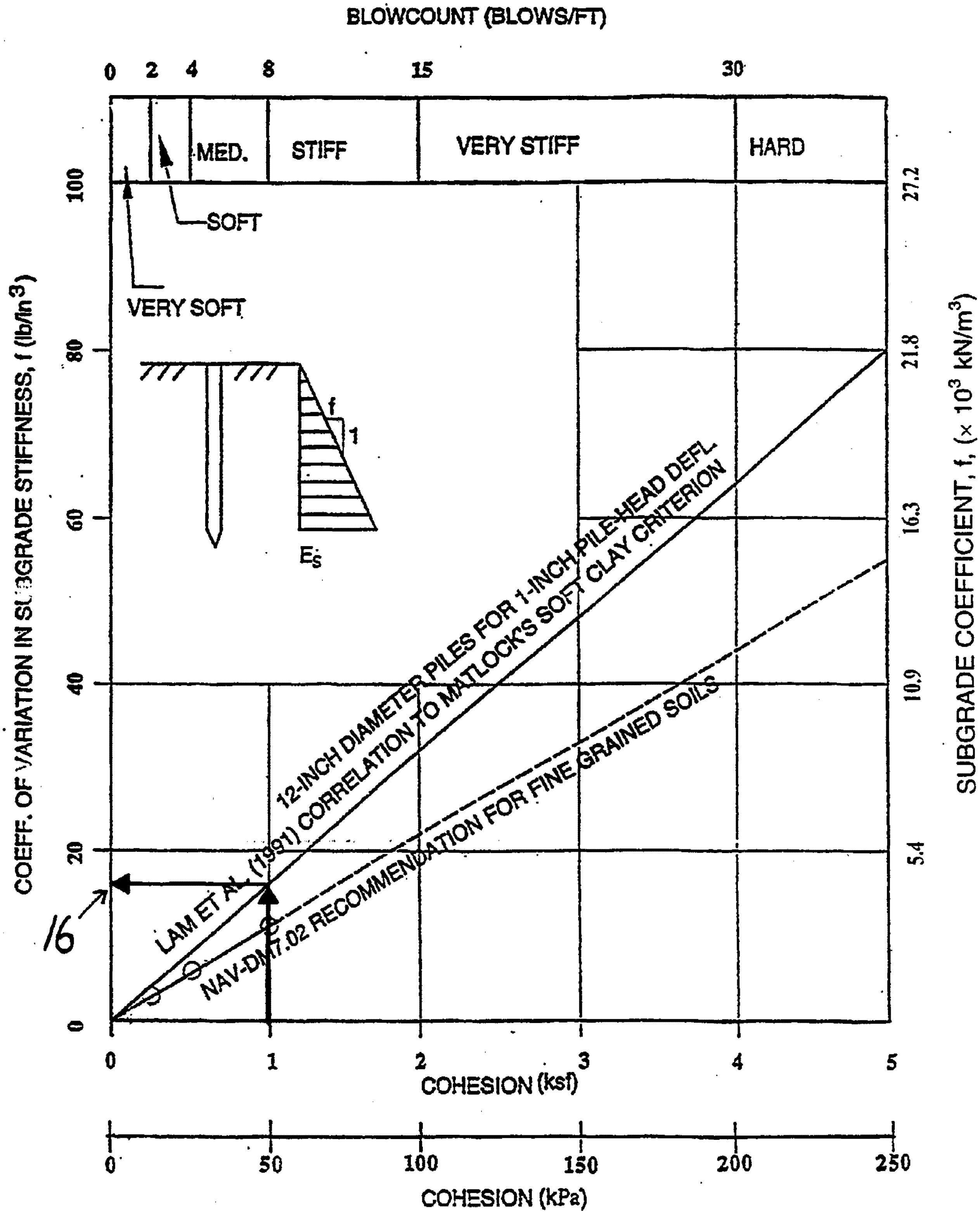


Figure 9-20: Coefficient of Variation of Subgrade Modulus for Clay.

C. Determine the Effective Coefficient of subgrade modulus $f_{eff} = (\text{p-multiplier})_{ave} \bullet f$:

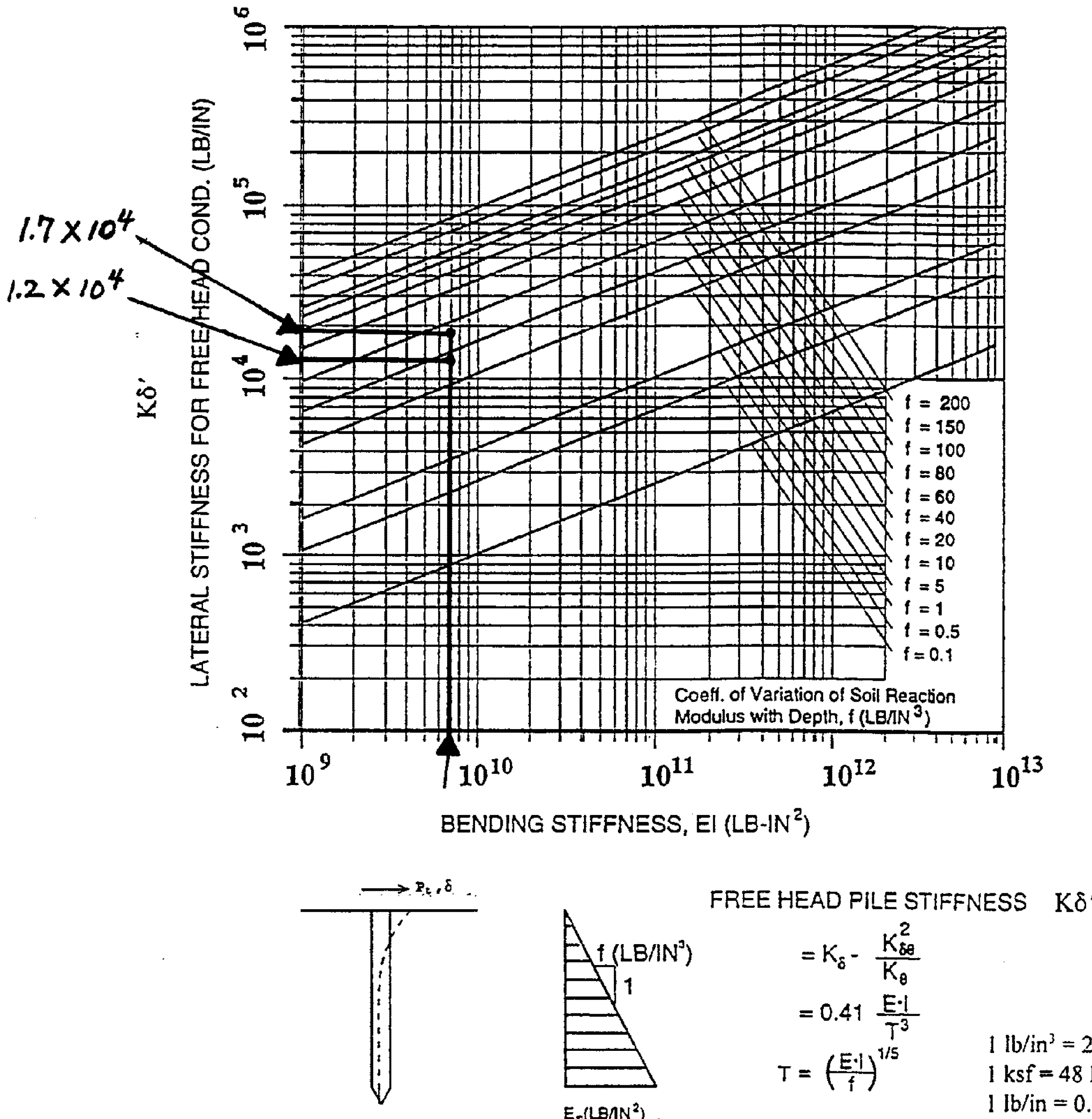
Longitudinal: $f_{eff,\,L}$ = $\underline{0.53 \text{ x } 16 = 8.5 \text{ lb/in}^3}$
Transverse: $f_{eff,\,T}$ = $\underline{0.87 \text{ x } 16 = 13.9 \text{ lb/in}^3}$

D. Calculate the Bending Stiffness of the Pile

EI = $\underline{3{,}600{,}000 \text{ x } 1885 = 6.8 \text{ x } 10^9 \text{ lb-in}^2}$

E. Derive lateral stiffness for free-head condition using Figure 9-21:

$k'_{\delta,L} = \underline{1.2 \text{ x } 10^4 \text{ lb/in}}$ (Longitudinal)
$k'_{\delta,T} = \underline{1.7 \text{ x } 10^4 \text{ lb/in}}$ (Transverse)



$$= K_\delta - \frac{K_{\delta\theta}^2}{K_\theta}$$

$$= 0.41\ \frac{E \cdot I}{T^3}$$

$$T = \left(\frac{E \cdot I}{f}\right)^{1/5}$$

1 lb/in³ = 272 kN/m³
1 ksf = 48 kPa
1 lb/in = 0.1752 kN/m
1 lb-in² = 2.87x10⁻⁶ kN-m²

Figure 9-21: Lateral Stiffness of Free-Headed Piles.

Step 2: Solve for the stiffness of a single pile under axial loading.

For end bearing piles (ignore skin friction), calculate the axial stiffness.

$$k_v = \frac{AE}{L} = \frac{154 \times 3{,}600{,}000}{30 \times 12"} = \underline{1{,}540{,}000 \text{ lb/in}}$$

Step 3: Calaculate pile group stiffness by combining the stiffness contribution of each individual pile.

A. Lateral Stiffness

$$K_L = n \cdot k'_{\delta,L} = 9 \text{ x } (1.2 \text{ x } 10^{4)}$$
$$= 1.1 \text{ x } 10^5 \text{ lb/in (Longitudinal)}$$

$$K_T = n \cdot k'_{\delta,T} = 9 \text{ x } (1.7 \text{ x } 10^4)$$
$$= 1.53 \text{ x } 10^5 \text{ lb/in (Transverse)}$$

B. Axial Stiffness

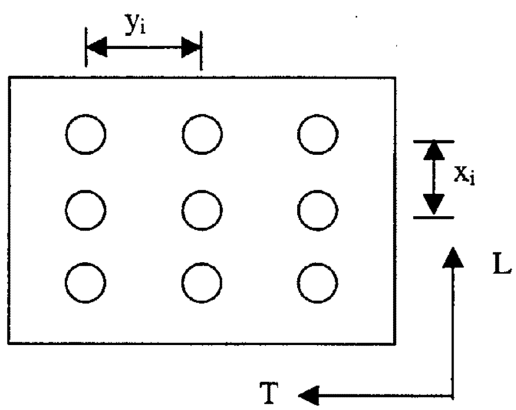
$$K_V = n \cdot k_V = 9 \text{ x } 1{,}540{,}000 = 1.39 \text{ x } 10^6 \text{ lb/in}$$

C. Torsional Stiffness

$$K_{TOR} = \sum_{i=1}^{n} k'_{\delta,L} \cdot y_i^2 + \sum_{i=1}^{n} k'_{\delta,T} \cdot x_i^2$$

$$= \underline{1.2 \times 10^4 \times 6 \times (70)^2 + 1.7 \times 10^4 \times 6 \times (42)^2}$$

$$= \underline{5.3 \times 10^8 \text{ lb-in/rad}}$$



D. Rocking Rotational Stiffness about Transverse Axis.

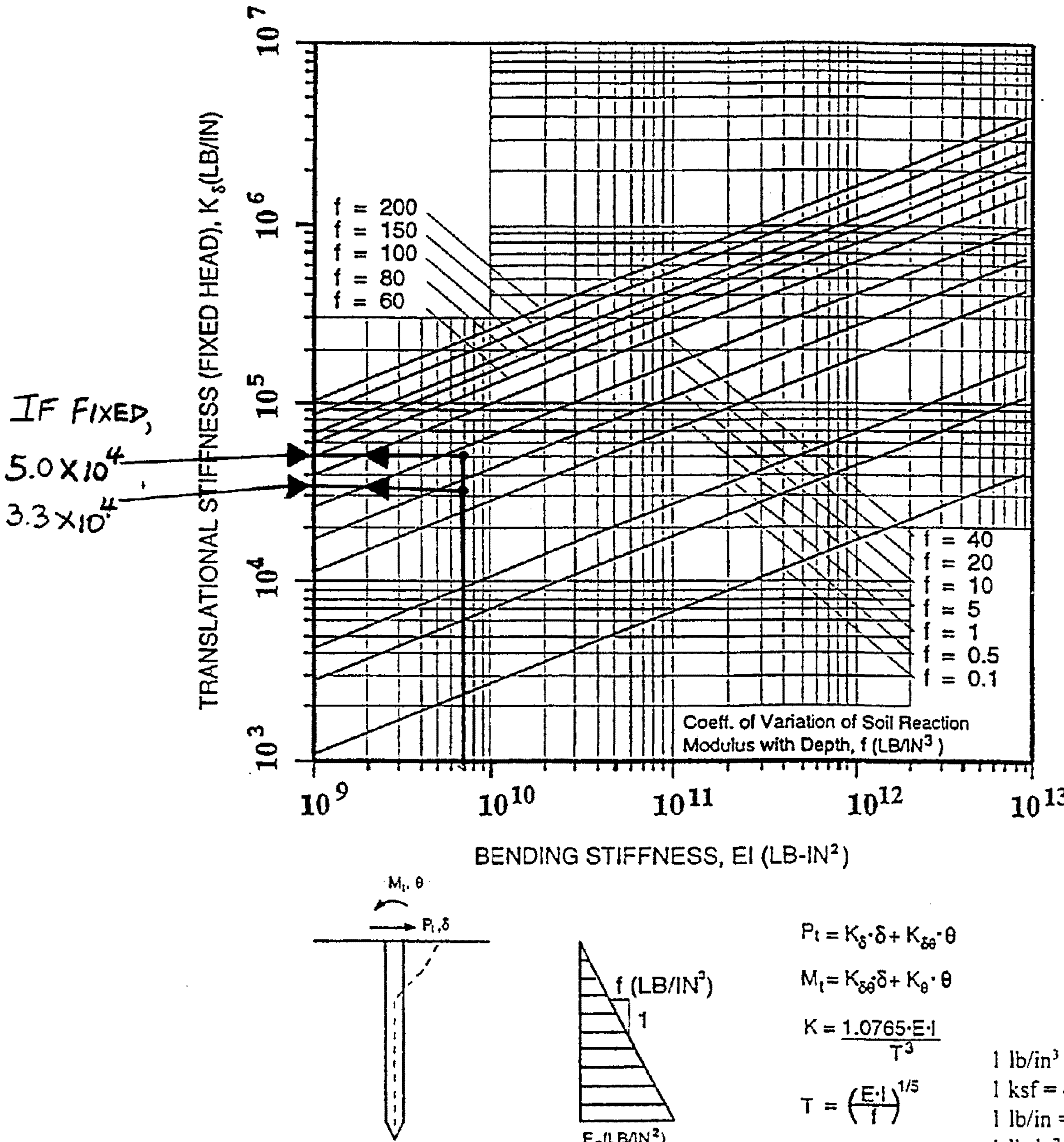
$$K_{RT} = k_V \cdot \sum_{i=1}^{n} x_i^2$$

$$= \underline{1{,}540{,}000 \times 6 \times (42)^2 = 1.6 \times 10^{10} \text{ lb-in/rad}}$$

E. Rocking Rotational Stiffness about Longitudinal Axis.

$$K_{RL} = k_V \cdot \sum_{i=1}^{n} y_i^2$$

$$= \underline{1,540,000 \times 6 \times (70)^2 = 4.5 \times 10^{10} \text{ lb-in/rad}}$$



1 lb/in³ = 272 kN/m³
1 ksf = 48 kPa
1 lb/in = 0.1752 kN/m
1 lb-in² = 2.87x10⁻⁶ kN-m²

Figure 9-22: Lateral Stiffness of Fixed Head Piles.

STUDENT EXERCISE NO. 10A

Dynamic Earth Pressure Approach for Seismic Design of Retaining Walls.

Objective:

Derive the Dynamic Earth Pressure for a Cantilever Wall Retaining a Highway Enbankment in Collinsville, IL. The Wall Geometry, Soil Properties behind the Wall and other Assumptions are Presented in Figure S10A-1.

Source Materials:

Reference Manual Part I: Section 9.4.2, Equations 9-13a and 9-13b, and Figure 9-28.

- Location: Collinsville, IL
- Peak Acceleration on Firm Ground (or Rock)

  A_{cc} = 0.15g (per current AASHTO)
- Site is Underlain by Soft Soil
- Soil Amplification Factor = 1.5
- Assume the Wall Can Yield Sufficiently to Mobilize Active Soil Wedge, But Permanent Sliding Displacement is Not Allowed.
- Other Relevant Parameters Are Given in the Sketch Below.

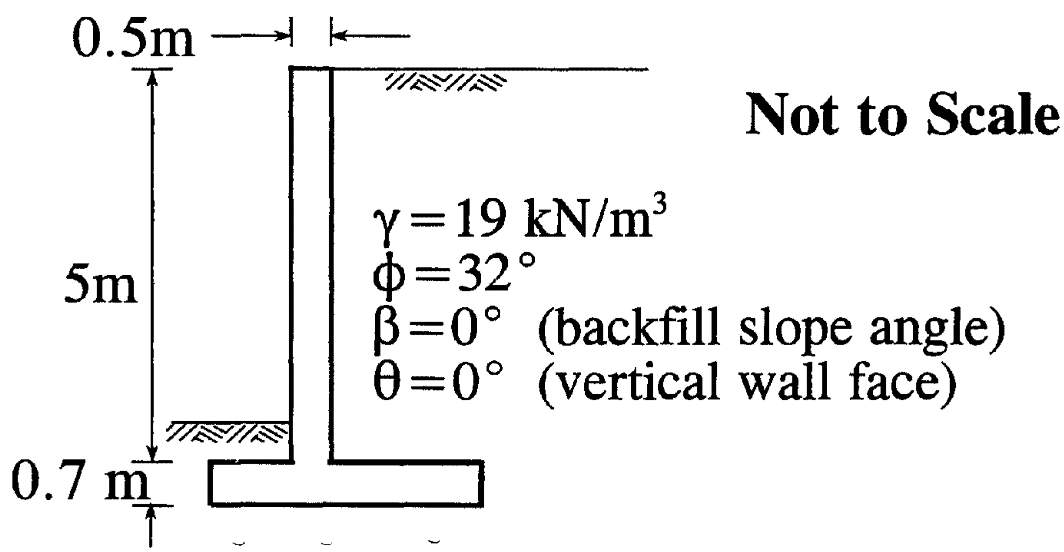


Figure S10A-1: Retaining Wall Geometry and Assumptions

1. Total Wall Height, $H =$ __________

2. $\delta = \phi =$ ______

3. k_h = Amplified Peak Ground Acceleration/g
   $= 1.5 \times 0.15 =$ __________

   Ignore Vertical Ground Motion $k_v = 0$

4. $\psi = \arctan [k_h/(1-k_v)] =$ _________

5. Compute Active Earth Pressure Coefficient (Eq.9-13b)

$$K_{ae} = \frac{\cos^2(\phi-\psi-\theta)}{\cos\psi \ \cos^2\theta \ \cos(\delta+\theta+\psi)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\ \sin(\phi-\psi-\beta)}{\cos(\delta+\theta+\psi)\ \cos(\beta-\theta)}}\right]^2}$$

$K_{ae} =$ __________

6. Total Dynamic Active Earth Force (Eq. 9-13a)

$$P_{ae} = \frac{1}{2} K_{ae} \gamma H^2 = \underline{\hspace{4cm}}$$

7. Uniformly Distributed Dynamic Earth Pressure

$$p_{ae} = P_{ae}/H = \underline{\hspace{4cm}}$$

SOLUTION TO EXERCISE NO. 10A

1. Total Wall Height, $H = 5 + 0.7 = \underline{5.7\ m}$

2. $\delta = \phi = \underline{32^\circ}$

3. k_h = Amplified Peak Ground Acceleration/g
   $= 1.5 \times 0.15 = \underline{0.225}$

   Ignore Vertical Ground Motion $k_v = 0$

4. $\psi = \arctan\,[k_h/(1-k_v)] = \underline{12.7^\circ}$

5. Compute Active Earth Pressure Coefficient (Eq.9-13b)

$$K_{ae} = \frac{\cos^2(\phi-\psi-\theta)}{\cos\psi\ \cos^2\theta\ \cos(\delta+\theta+\psi)\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\ \sin(\phi-\psi-\beta)}{\cos(\delta+\theta+\psi)\ \cos(\beta-\theta)}}\right]^2}$$

$$K_{ae} = \frac{\cos^2(32^\circ-12.7^\circ-0^\circ)}{\cos 12.7^\circ \cos^2 0^\circ \cos(32^\circ+0^\circ+12.7^\circ)\left[1+\sqrt{\dfrac{\sin(32^\circ+32^\circ)\sin(32^\circ-12.7^\circ-0^\circ)}{\cos(32^\circ+0^\circ+12.7^\circ)\cos(0^\circ-0^\circ)}}\right]^2}$$

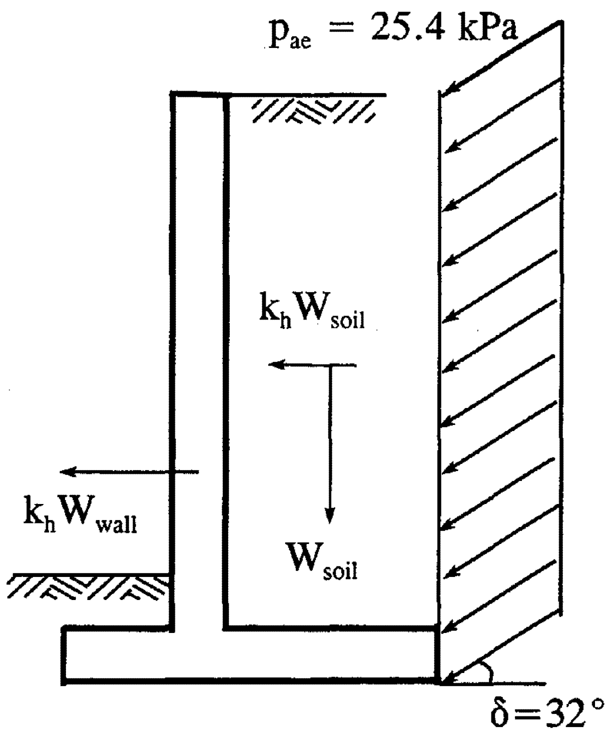
$K_{ae} = \underline{0.47}$

6. Total Dynamic Active Earth Force (Eq. 9-13a)

$$P_{ae} = \frac{1}{2} K_{ae} \gamma H^2 = \frac{1}{2} \times 0.47 \times 19 \times 5.7^2 = \underline{145\,\text{kN/m}}$$

7. Uniformly Distributed Dynamic Earth Pressure

$$p_{ae} = P_{ae}/H = 145/5.7 = \underline{25.4\ \text{kPa}}$$



For External Stability Analysis

STUDENT EXERCISE NO. 10B

Permissible Displacement Approach for Seismic Design of Retaining Walls.

Objective:

Derive the Reduced Seismic Coefficient, K_h, Using the Permissible Displacement Approach for the Same Retaining Wall in Student Exercise NO. 10A. Wall is Allowed to Displace by 50 mm. A Peak Ground Velocity of 250 mm/sec was Obtained from a Site Response Analysis.

Source Materials:

Reference Manual Part II: Section 9.4.2, Equation 9-14.

1. d_R = Allowable Displacement = ________

2. Peak Ground Acceleration Coefficient, A = ______

3. Peak Ground Velocity, V = __________

4. Use Eq. 9-14:

$$d_R = 0.087 \left(\frac{V^2}{A.g} \right) \cdot \left(\frac{N}{A} \right)^{-4}$$

$\Rightarrow$ $N = K_h =$ ______ (Transmittable Acceleration Coefficient)

SOLUTIONS TO EXERCISE NO. 10B

1. d_R = Allowable Displacement = 50 mm

2. Peak Ground Acceleration Coefficient, A = 0.225

3. Peak Ground Velocity, V = 250 mm/sec

4. Use Eq. 9-14:

$$d_R = 0.087 \left(\frac{V^2}{A.g}\right) \cdot \left(\frac{N}{A}\right)^{-4}$$

$$\Rightarrow \quad 50 = 0.087 \left(\frac{250^2}{0.225 \text{x} 9{,}810}\right) \cdot \left(\frac{N}{0.225}\right)^{-4}$$

$\Rightarrow \quad N = K_h = 0.106$ (Transmittable Acceleration Coefficient)

5. Use k_h = 0.106 and $k_v = 0$ for External Stability Analysis of the Wall