



PB99-142549

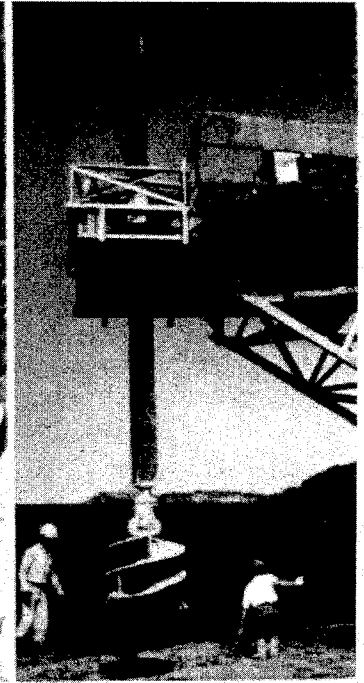
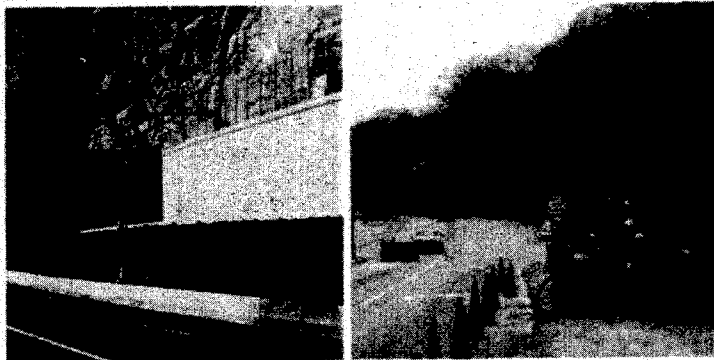
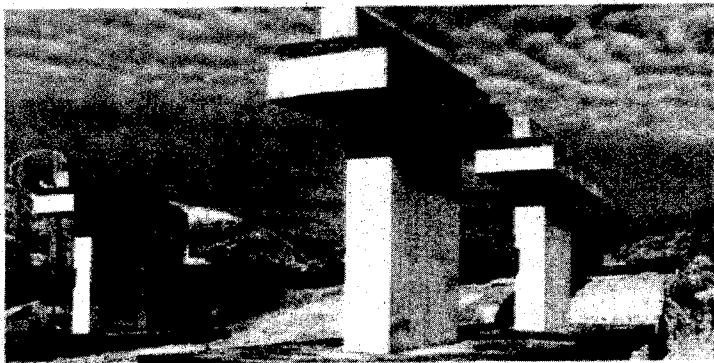
# TRAINING COURSE IN GEOTECHNICAL AND FOUNDATION ENGINEERING

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**




National Highway Institute

REPRODUCED BY: **NTIS**  
U.S. Department of Commerce  
National Technical Information Service  
Springfield, Virginia 22161

**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.**

Technical Report Documentation Page

1. Report No. FHWA-HI-99-014		 PB99-142549		3. Recipient's Catalog No.	
4. Title and Subtitle GEOTECHNICAL EARTHQUAKE ENGINEERING STUDENT EXERCISES				5. Report Date February 1999	
				6. Performing Organization Code	
7. Author(s) Principal Investigator: George Munfakh* Authors: Edward Kavazanjian, Jr. <sup>▲</sup> , Neven Matasović <sup>▲</sup> , Tarik Hadj-Hamou <sup>▲</sup> , and Jaw-Nan (Joe) Wang <sup>*</sup>				8. Performing Organization Report No.	
9. Performing Organization Name and Address * Parsons Brinckerhoff Quade & Douglas, Inc. One Penn Plaza, New York, NY 10119  In association with: ▲ GeoSyntec Consultants 2100 Main St., Suite 150, Huntington Beach, CA 92648				10. Work Unit No. (TRAIS)	
				11. Contract or Grant No. DTFH61-94-C-00104	
12. Sponsoring Agency Name and Address National Highway Institute U.S. Department of Transportation Federal Highway Administration Washington, D.C.				13. Type of Report and Period Covered	
				14. Sponsoring Agency Code	
15. Supplementary Notes FHWA Technical Consultants - J.A. DiMaggio, A. Munoz and P. Osborn FHWA Contracting Officer - J. Mowery III; COTR - L. Jones, National Highway Institute					
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).					
17. Key Words Geotechnical earthquake engineering, response spectra, site response analysis, liquefaction, slopes, foundations, retaining walls			18. Distribution Statement  No restrictions.		
19. Security Classif. (of this report) UNCLASSIFIED	20. Security Classif. (of this page) UNCLASSIFIED	21. No. of Pages 162	22. Price		

## 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.**

PROTECTED UNDER INTERNATIONAL COPYRIGHT  
ALL RIGHTS RESERVED.  
NATIONAL TECHNICAL INFORMATION SERVICE  
U.S. DEPARTMENT OF COMMERCE

Reproduced from  
best available copy. 

## 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.

In this document, certain products may have been identified by trade name. Also, photographs of these products may have been included in the document for illustration purposes. Other products which are not identified in this document may be equally viable to those identified. The mention of any trade name or photograph of a particular product does not constitute endorsement or recommendation for use by either the authors or FHWA.

## CONVERSION FACTORS

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

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<sup>2</sup>).  
 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**

		Page
No. 1	Ground Motion Parameters	1-1
No. 2A	Derivation of SPT- $(N_1)_{60}$	2A-1
No. 2B	Soil Profiling Using CPT	2B-1
No. 3	$G_{max}$ Derivation- Empirical Correlations	3-1
No. 4	Simplified Site Response Analysis	4-1
No. 5	SHAKE Program	5-1
No. 6	Liquefaction Potential	6-1
No. 7	Simplified Seismic Deformation Analysis	7-1
No. 8	Stiffness Matrix for Spread Footings	8-1
No. 9A	Pile Capacity Evaluation	9A-1
No. 9B	Foundation Stiffness for Pile Group	9B-1
No. 10A	Dynamic Earth Pressure- Retaining Walls	10A-1
No. 10B	Permissible Displacement Approach- Retaining Walls	10B-1

## 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):

$$PGA_{FF} = \underline{\hspace{2cm}}$$

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:

$$Z = \underline{\hspace{2cm}}$$

- B. Establish Site Class From Table 4-3

$$\text{Site Class} = \underline{\hspace{2cm}}$$

- C. Establish  $C_v$  From Table 4-5:

$$C_v = \underline{\hspace{2cm}}$$



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

$$T_s = \frac{C_v}{2.5Z} = \underline{\hspace{2cm}}$$

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

$$T_o = 0.2 T_s = \underline{\hspace{2cm}}$$

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.  $M = \underline{\hspace{2cm}}$
  - B.  $D = \underline{\hspace{2cm}}$

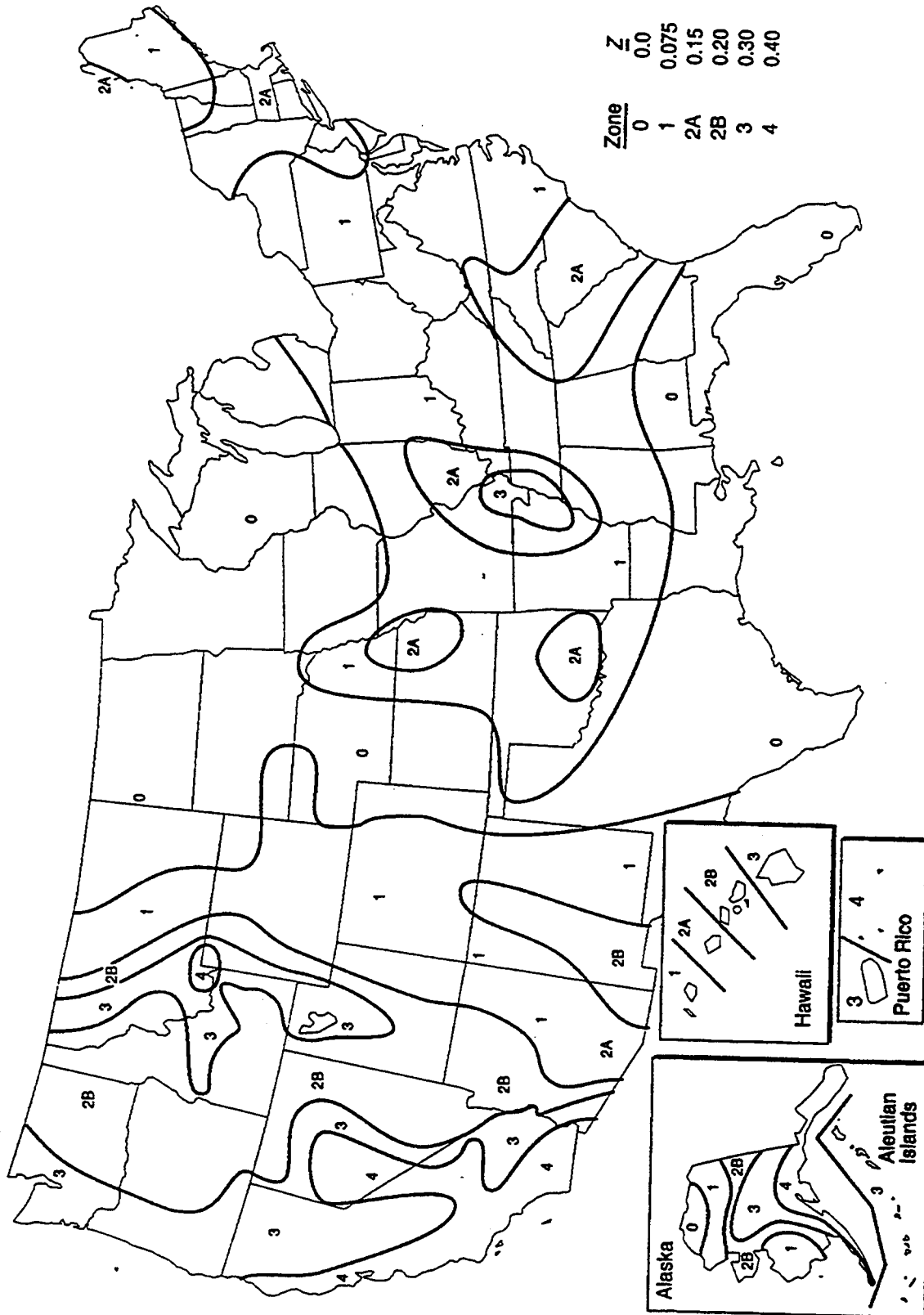


Figure 3-4: Map and Table for Evaluation of UBC Seismic Zone Factor, Z. (Reproduced from the Uniform Building Code™, Copyright© 1994, with the Permission of the Publisher, the International Conference of Building Officials)

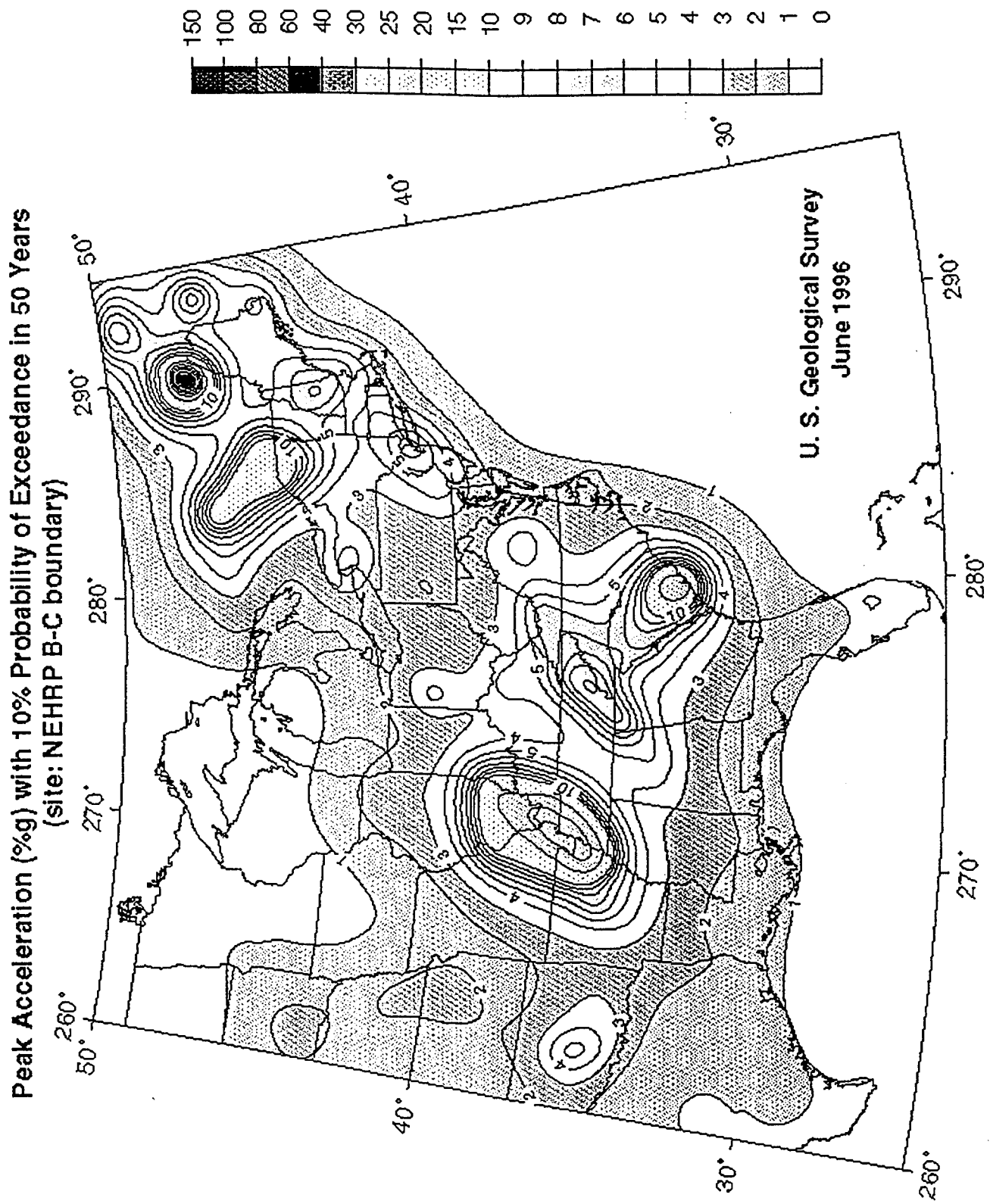


Figure 3-5: Peak Horizontal Ground Acceleration in Bedrock for Central and Eastern U.S. with a 10 Percent Probability of Exceedance in 50 Years. (Frankel, et al., 1996)

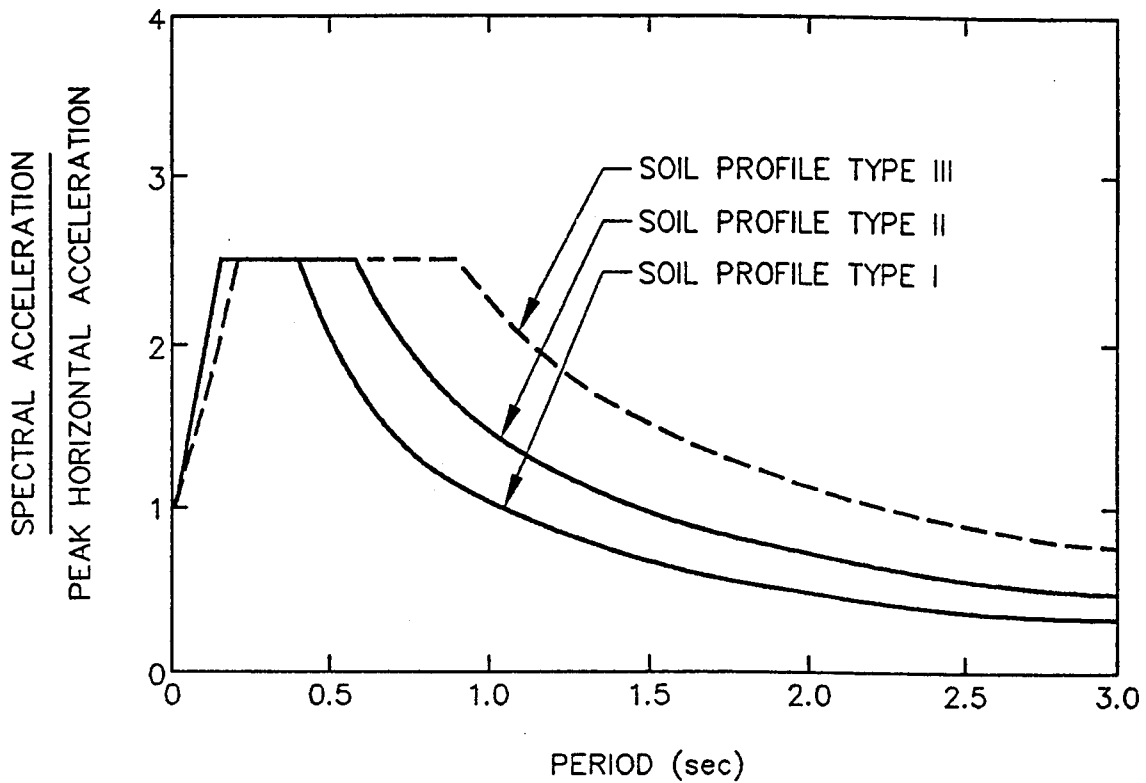


Figure 4-17: Normalized 1994 Uniform Building Code Response Spectra. (UBC, 1994, reproduced from the Uniform Building Code™, copyright© 1994, with the permission of the publisher, the International Conference of Building Officials)

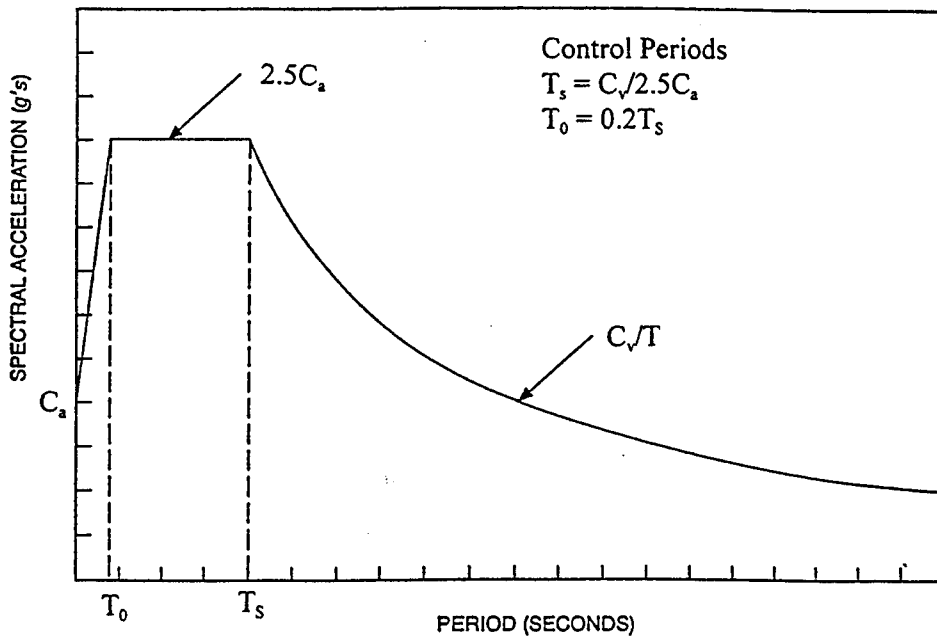


Figure 4-18: 1997 Uniform Building Code Design Response Spectra (UBC, 1997, reproduced from the Uniform Building Code™, copyright© 1997, with the permission of the publisher, the International Conference of Building Officials)

**TABLE 4-3  
1997 UBC SITE CLASSIFICATION**

Designation	Site Class	Shear Wave Velocity <sup>1</sup>	Other Characteristics <sup>2</sup>
S <sub>A</sub>	Hard Rock	> 1500 m/s	
S <sub>B</sub>	Rock	760 m/s to 1500 m/s	
S <sub>C</sub>	Very Dense Soil and Soft Rock	360 m/s to 760 m/s	N > 50, S <sub>u</sub> > 100 kPa
S <sub>D</sub>	Stiff Soil	180 m/s to 360 m/s	15 < N < 50 50 kPa < S <sub>u</sub> < 100 kPa
S <sub>E</sub>	Soft Soil	Less than 180 m/s	More than 3m of soil with PI > 20, W <sub>n</sub> > 40%, and S <sub>u</sub> < 25 kPa
S <sub>F</sub>	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  
 S<sub>u</sub> = Undrained Shear Strength  
 PI = Plasticity Index  
 W<sub>n</sub> = Moisture content

**TABLE 4-4  
SEISMIC COEFFICIENT C<sub>s</sub>**

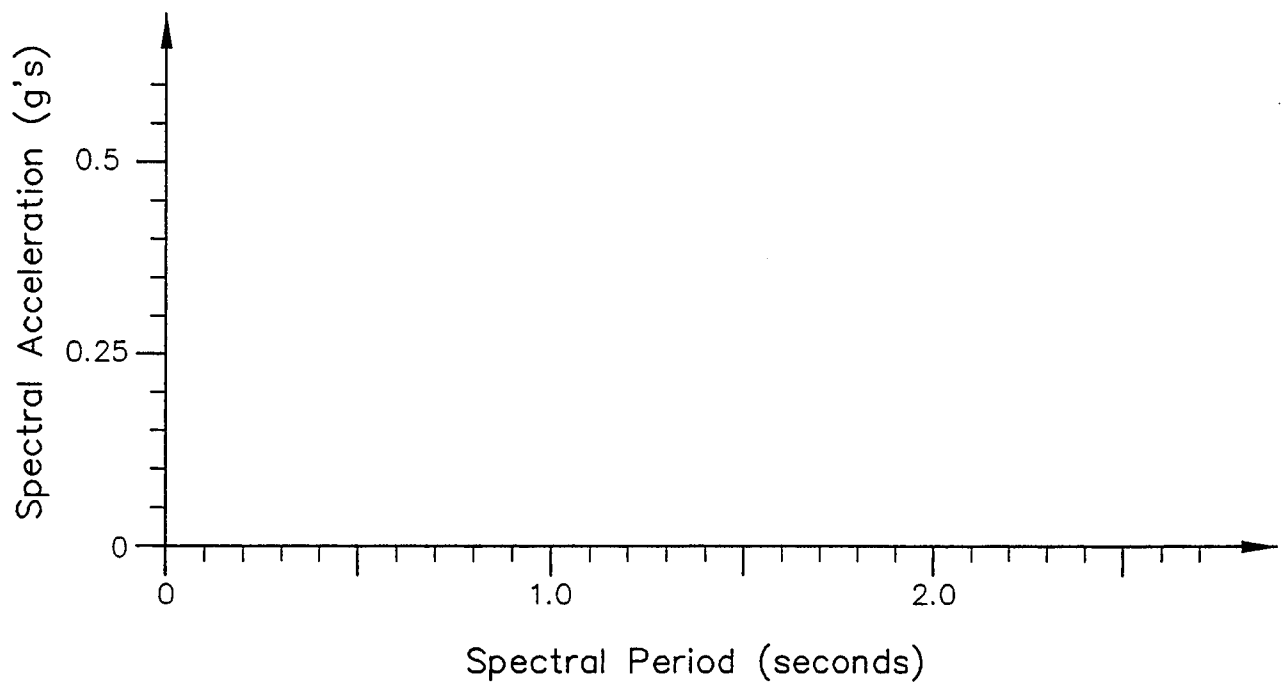
Soil Profile Type	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S <sub>A</sub>	0.06	0.12	0.16	0.24	0.32N <sub>s</sub>
S <sub>B</sub>	0.08	0.15	0.20	0.30	0.40N <sub>s</sub>
S <sub>C</sub>	0.09	0.18	0.24	0.33	0.40N <sub>s</sub>
S <sub>D</sub>	0.12	0.22	0.28	0.36	0.44N <sub>s</sub>
S <sub>E</sub>	0.19	0.30	0.34	0.36	0.36N <sub>s</sub>
S <sub>F</sub>	See Footnote 1				

- Notes: <sup>1</sup> Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S<sub>F</sub>.

**TABLE 4-5  
SEISMIC COEFFICIENT C<sub>v</sub>**

Soil Profile Type	Seismic Zone Factor, Z				
	Z = 0.075	Z = 0.15	Z = 0.2	Z = 0.3	Z = 0.4
S <sub>A</sub>	0.06	0.12	0.16	0.24	0.32N <sub>v</sub>
S <sub>B</sub>	0.08	0.15	0.20	0.30	0.40N <sub>v</sub>
S <sub>C</sub>	0.13	0.25	0.32	0.45	0.56N <sub>v</sub>
S <sub>D</sub>	0.18	0.32	0.40	0.54	0.64N <sub>v</sub>
S <sub>E</sub>	0.26	0.50	0.64	0.84	0.96N <sub>v</sub>
S <sub>F</sub>	See Footnote 1				

- Notes: <sup>1</sup> Site-specific geotechnical investigation and dynamic site response analysis shall be performed to determine seismic coefficients for Soil Profile Type S<sub>F</sub>.





**NATIONAL SEISMIC HAZARD MAPPING PROJECT**

*USGS, Central Region, Geologic Hazards Team  
Golden, Colorado*

**PROJECT INFO**



**REVIEW MATERIAL**

**1996 HAZARD MAPS**

**SEISMIC HAZARD**

**MAP INFO**

**GENERAL**



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](mailto: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
    -----
    
```





## NATIONAL SEISMIC HAZARD MAPPING PROJECT

USGS, Central Region, Geologic Hazards Team  
Golden, Colorado

### PROJECT INFO



### REVIEW MATERIAL

Great Earthquake Maps

### 1996 HAZARD MAPS

### SEISMIC HAZARD

### MAP INFO

### GENERAL

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 ( $M_w$ , 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  $M_w$ . Hazard with respect to epicentral distance is binned into intervals of 25 km width. The hazard probabilities 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  $5.5 < M_w \leq 6$  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  $125 < d \leq 150$  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  $M_b L_g$  5.0. This magnitude corresponds to  $M_w = 4.7$  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  $M_w$  units, rather than 0.5 units, the usual interval width. For the WUS, the lowest magnitude considered for hazard calculations is  $M_w = 5.0$ .

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](mailto:dickman@usgs.gov))  
Last Modified: Thursday, 04-Sep-97 13:49:59 MDT*

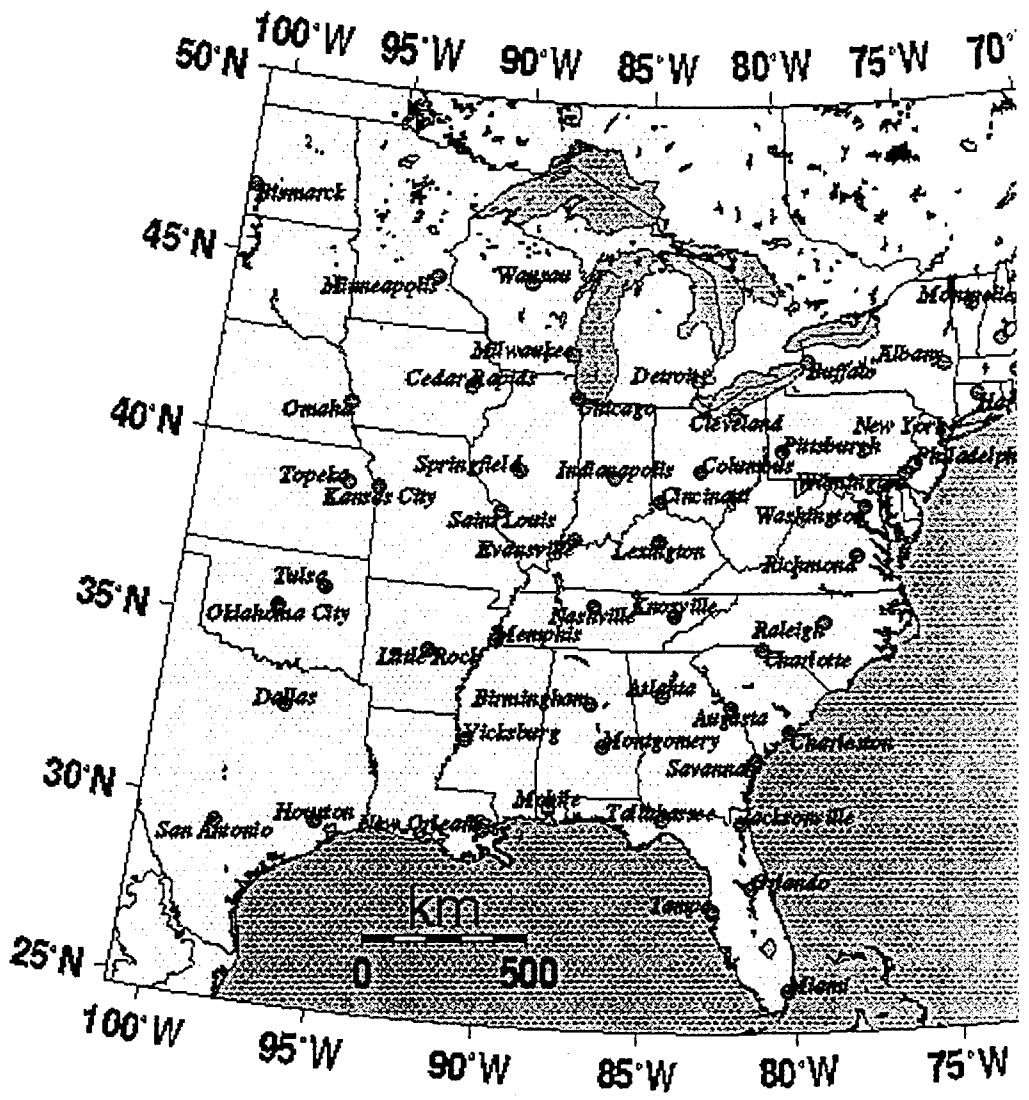
**1996 HAZARD MAPS**

# CEUS Cities

**SEISMIC HAZARD**

**MAP INFO**

**GENERAL**



**GIAT** Jan 30 09:53 Deaggregated Seismic Hazard, Prob. Exceedance 2%-50 yr.

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



NATIONAL SEISMIC HAZARD MAPPING PROJECT

USGS, Central Region, Geologic Hazards Team  
Golden, Colorado

PROJECT INFO

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



REVIEW MATERIAL

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.

USGS (Central Region)

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

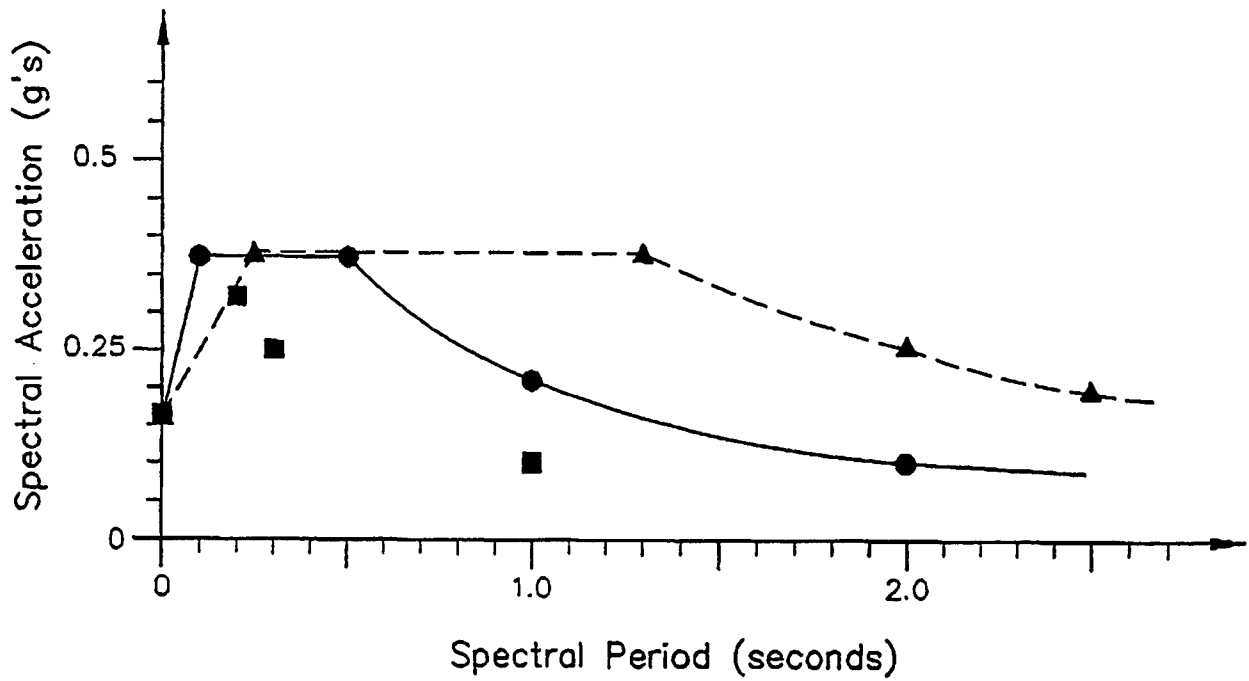
For Firm Ground Site:

1.  $PGA_{FF} = 0.16 \text{ g}$
- 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.  $T_s = 0.53 \text{ s}$
- 2E.  $T_o = 0.11 \text{ s}$

For Soft Ground Site:

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

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.

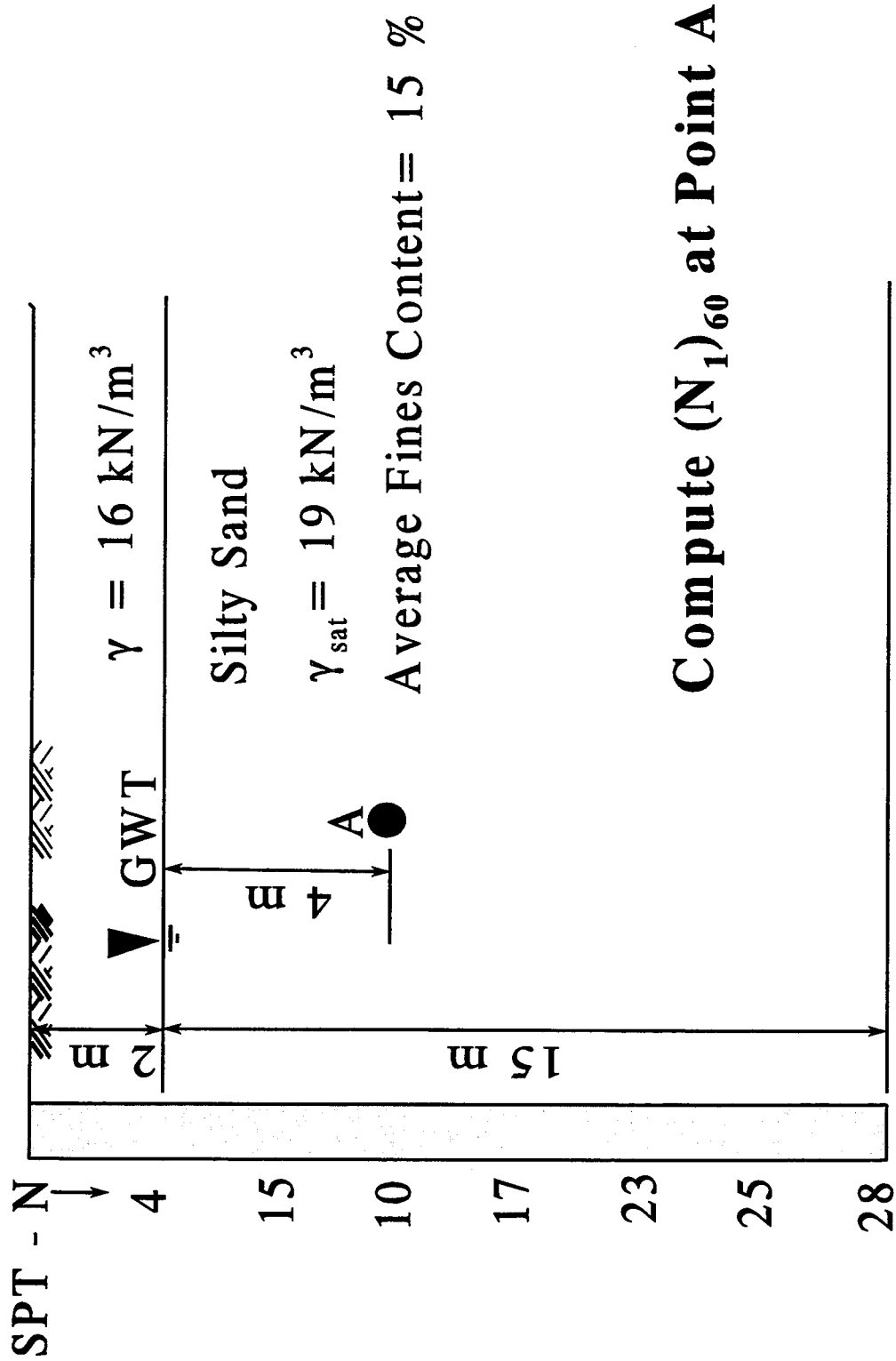


Figure S2A-1: Soil Profile and Field SPT-N Values

## 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.

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

$$C_{60} = \underline{\hspace{4cm}}$$

3. Normalization

Compute Effective Overburden Pressure,  $\sigma'_v$

$$\sigma'_v = \underline{\hspace{4cm}}$$

Compute Overburden Correction Factor Using Equation 5-10

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

$$C_N = \underline{\hspace{2cm}}$$

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

$$(N_1)_{60} = \underline{\hspace{2cm}}$$

**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 pulley $C_{HT} = 1.33$ for DH with trip/auto & ER = 80	Seed, <i>et al.</i> (1985)
Nonstandard Hammer Weight or Height of Fall (H = height of fall in mm; W = hammer weight in kg)	$C_{HW} = \frac{H \cdot W}{63.5 \cdot 762}$	calculated per Seed, <i>et al.</i> (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, <i>et al.</i> (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, <i>et al.</i> (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

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

$$C_{60} = 0.75 \times 1 \times 1.1 \times 0.95 \times 1$$

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

- 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,  $\sigma'_v$

$$\sigma'_v = \gamma \times 2\text{m} + (\gamma_{\text{sat}} - \gamma_w) \times 4\text{m}$$

$$\sigma'_v = 16 \times 2\text{m} + (19 - 9.81) \times 4\text{m}$$

$$\sigma'_v = \underline{\quad 69 \text{ kPa} \quad}$$

Compute Overburden Correction Factor,  $C_N$

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

$$C_N = \underline{1.18}$$

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

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

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





## **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

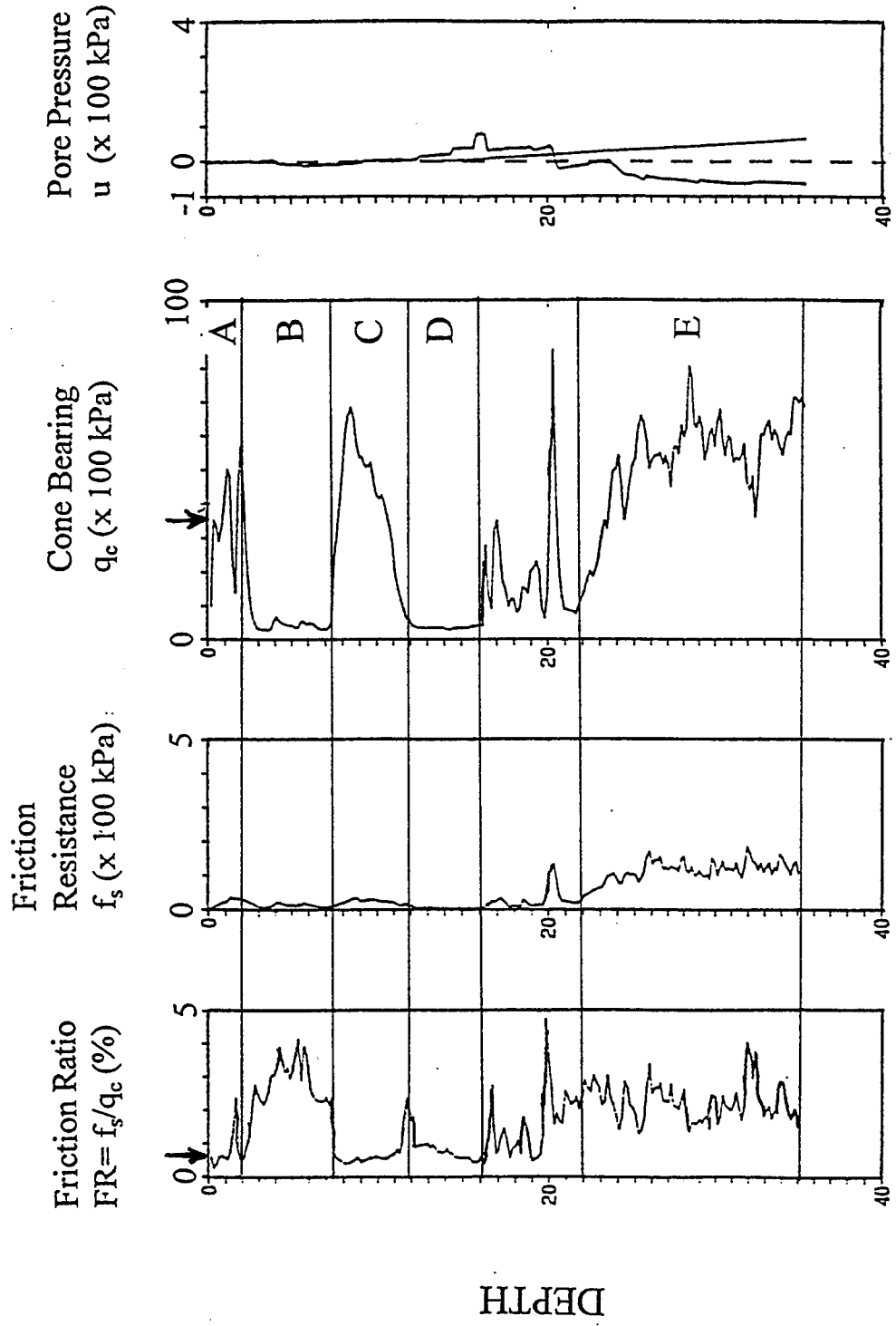


Figure S2B-1: CPT Data Profile

Stratum	Effective Vertical Stress (kPa)	Mean Grain Size (mm)	Cone Resistance $q_c$ (x 100 kPa)	Normalized Cone Resistance $q_{ct}$ (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  $q_{c1}$ .

$$q_c = 3,500 \text{ kPa (from Figure S2B-1)}$$

$$\sigma'_v = 6 \text{ kPa (from Table S2B-1)}$$

(Using Figure 5-6)

$$\begin{aligned} q_{c1} &= q_c (3.5 - 1.25 \log_{10} \sigma'_v) \\ &= 88 \text{ (x 100 kPa)} \end{aligned}$$

2. Determine the soil behavior type.

$$q_{c1} = 88 \text{ (x 100 kPa)}$$

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

⇒ 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)

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

(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.

$$q_{c1} = q_c ( 3.5 - 1.25 \log_{10} \sigma_v' )$$

with  $\sigma_v'$ ,  $q_c$ ,  $q_{c1}$  in kPa.

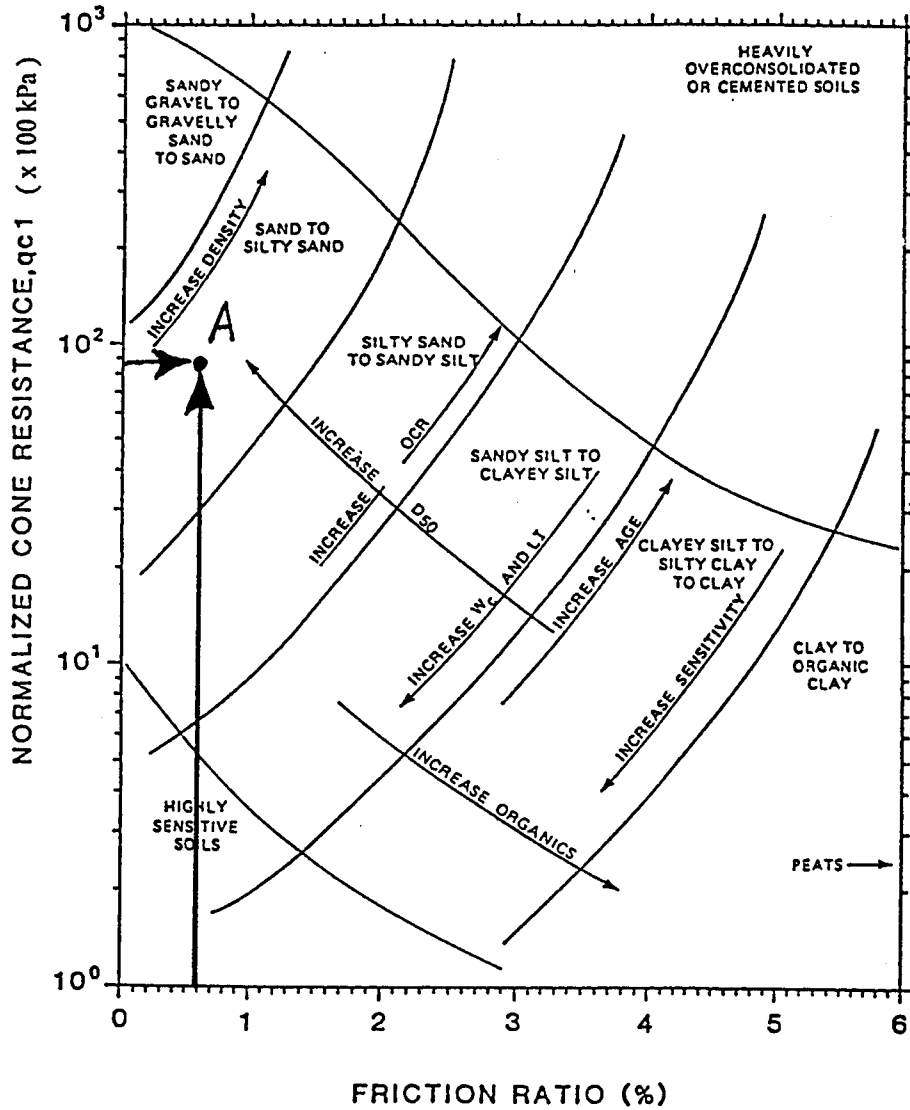


Figure 5-6: Soil Behavior Type Classification Chart Based on the CPT. (Douglas, 1984, 1981, reprinted from FHWA-SA-91-043, 1992.)

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

$q_c$  in bars (1 bar = 100 kPa)

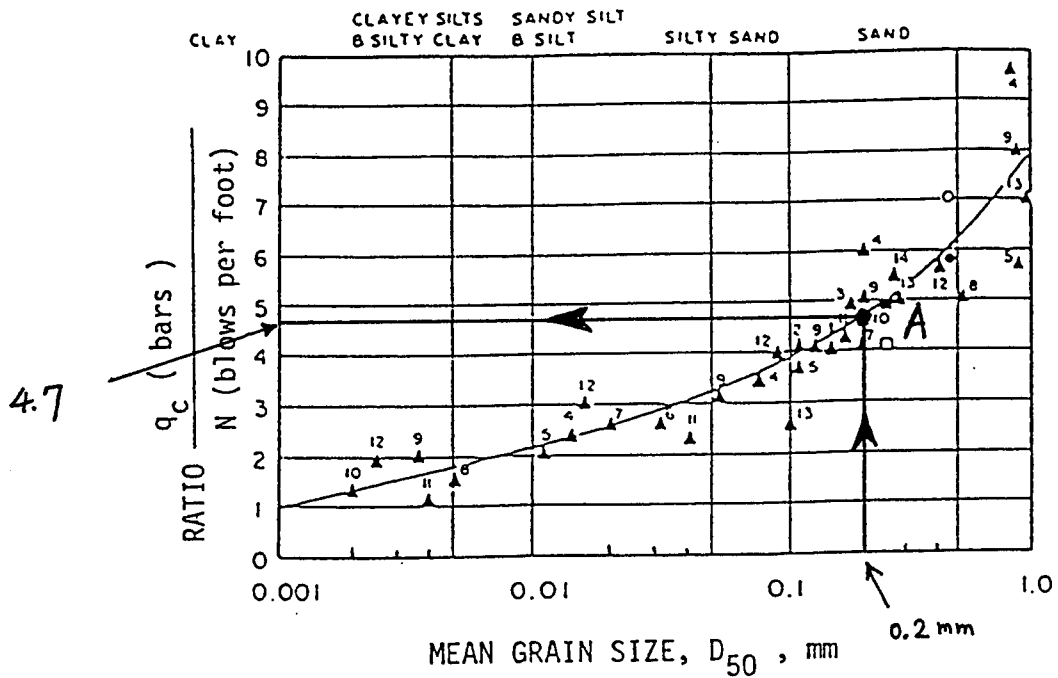


Figure 5-7: CPT-SPT Correlation Chart. (Robertson et al., 1983, reprinted from FHWA-SA-91-043, 1992.)

## 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



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

with  $\sigma_v'$ ,  $q_C$ ,  $q_{C1}$  in kPa.

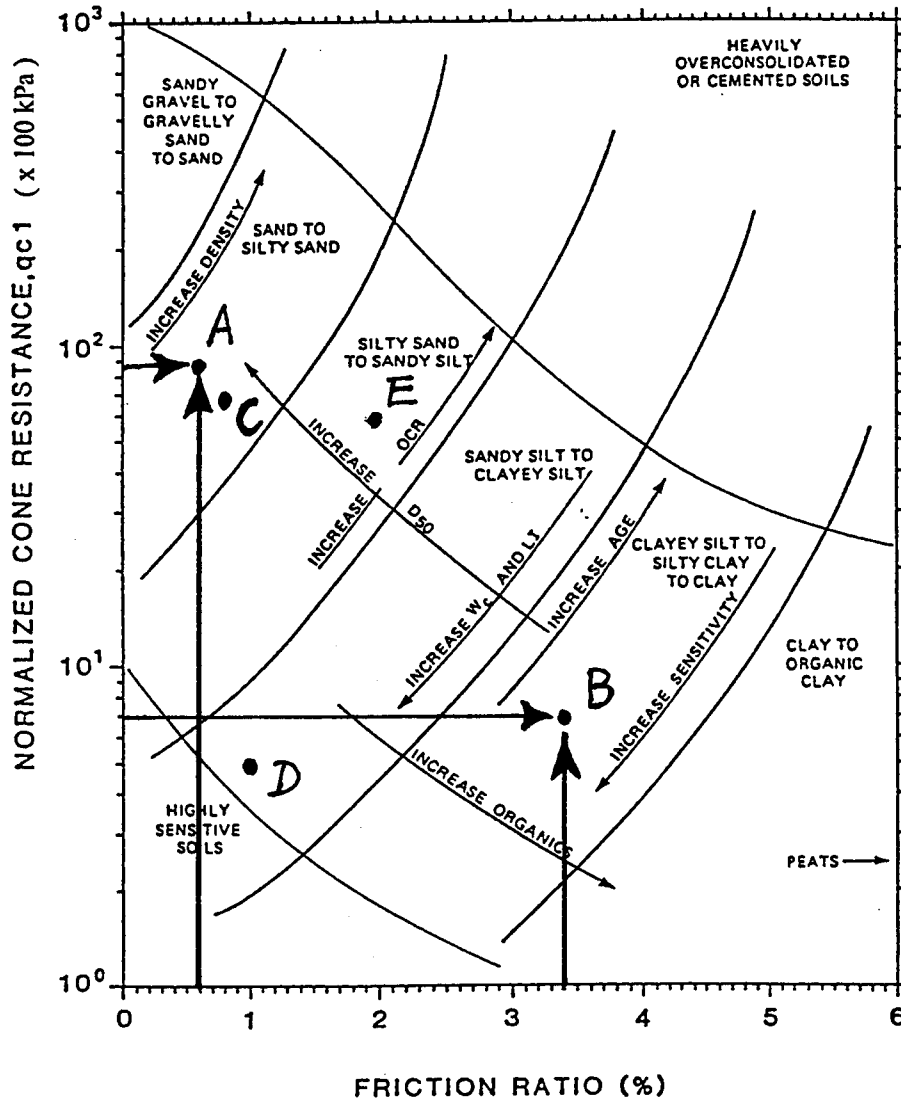


Figure 5-6: Soil Behavior Type Classification Chart Based on the CPT. (Douglas, 1984, 1981, reprinted from FHWA-SA-91-043, 1992.)

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

$q_c$  in bars (1 bar = 100 kPa)

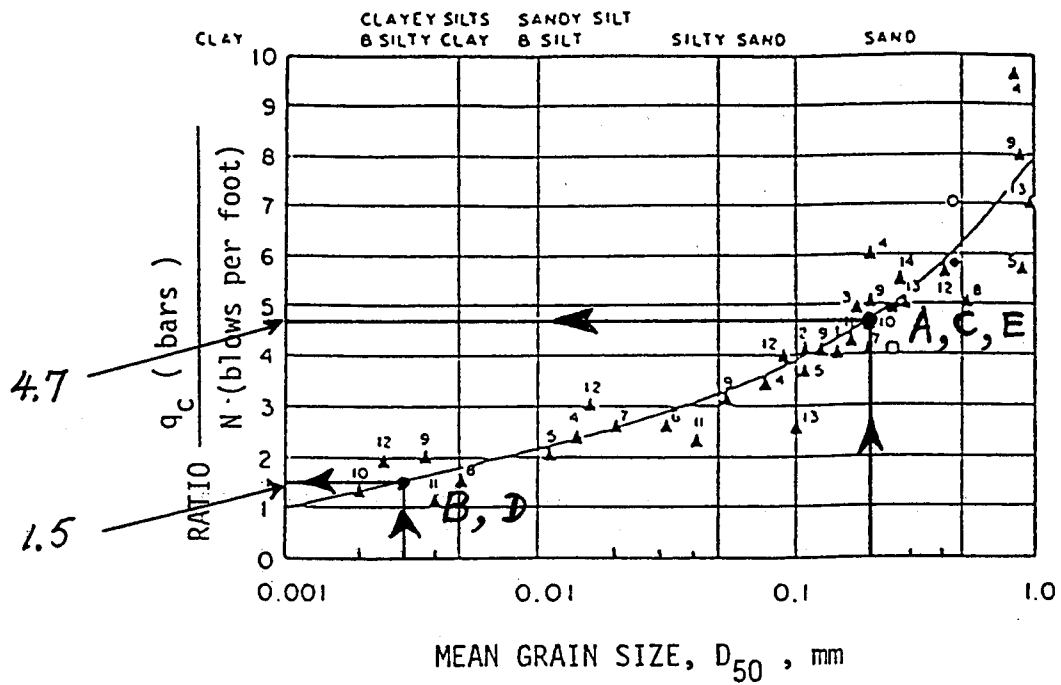


Figure 5-7: CPT-SPT Correlation Chart. (Robertson et al., 1983, reprinted from FHWA-SA-91-043, 1992.)

## 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  $18 \text{ kN/m}^3$  and a Saturated Unit Weight of  $20.5 \text{ kN/m}^3$ . 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:

$$\sigma_v' = \underline{\hspace{4cm}}$$

$$K_o = \underline{\hspace{4cm}}$$

$$\sigma_m' = \underline{\hspace{4cm}}$$

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

$$(K_2)_{\max} = \underline{\hspace{2cm}}$$

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

$$G_{\max} = \underline{\hspace{2cm}}$$

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

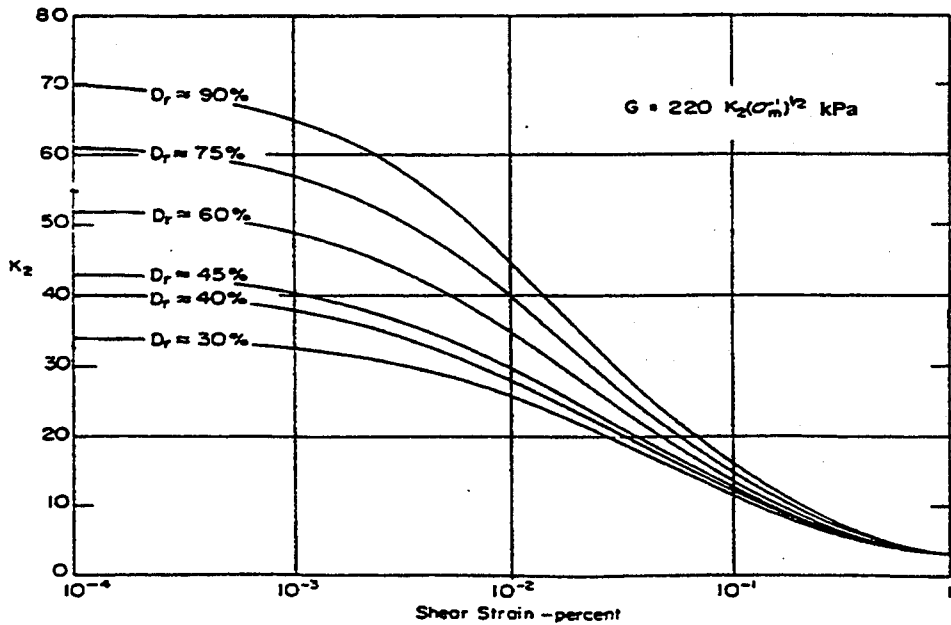
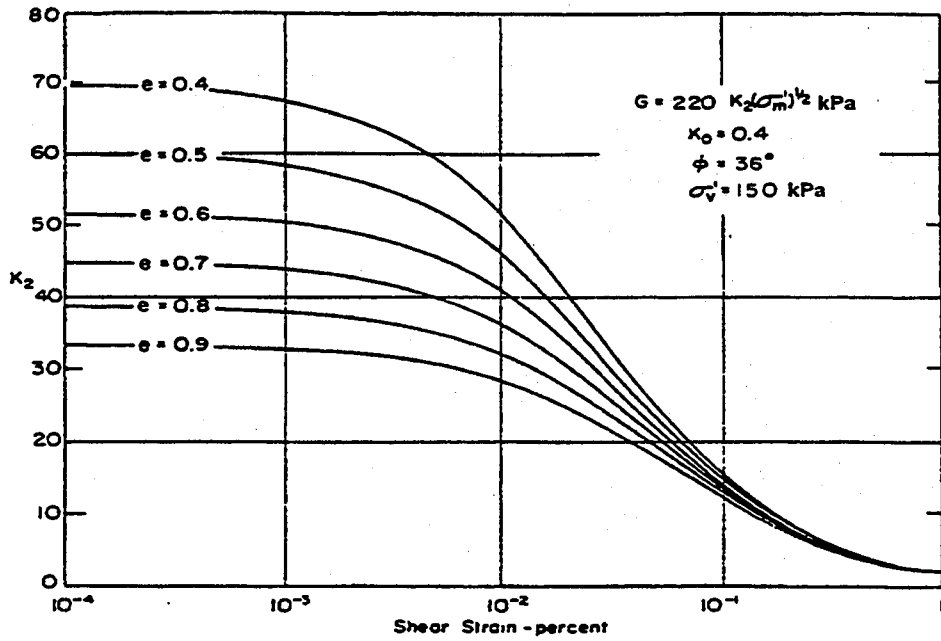


Figure 5-12: Shear Modulus Reduction Curves for Sands. (Seed and Idriss, 1970, reprinted by permission of ASCE)

**TABLE 5-5**  
**CORRELATIONS FOR ESTIMATING INITIAL SHEAR MODULUS**

Reference	Correlation	Units	Limitation
Seed, <i>et al.</i> (1984)	$G_{\max} = 220 (K_2)_{\max} (\sigma'_m)^{1/2}$ $(K_2)_{\max} \approx 20(N_1)^{1/3}$	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)	$G_{\max} = \frac{625}{(0.3 + 0.7e_0)^2} (P_a \cdot \sigma'_m)^{0.5} \text{OCR}^k$	kPa <sup>(1) (3)</sup>	Limited to cohesive soils $P_a =$ atmospheric pressure
Jamiolkowski, <i>et al.</i> (1991)	$G_{\max} = \frac{625}{e_0^{1.3}} (P_a \cdot \sigma'_m)^{0.5} \text{OCR}^k$	kPa <sup>(1) (3)</sup>	Limited to cohesive soils $P_a =$ atmospheric pressure
Mayne and Rix (1993)	$G_{\max} = 99.5 (P_a)^{0.305} (q_c)^{0.695} / (e_0)^{1.13}$	kPa <sup>(2)</sup>	Limited to cohesive soils $P_a =$ atmospheric pressure

Notes: <sup>(1)</sup>  $P_a$  and  $\sigma'_m$  in kPa

<sup>(2)</sup>  $P_a$  and  $q_c$  in kPa

<sup>(3)</sup> 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

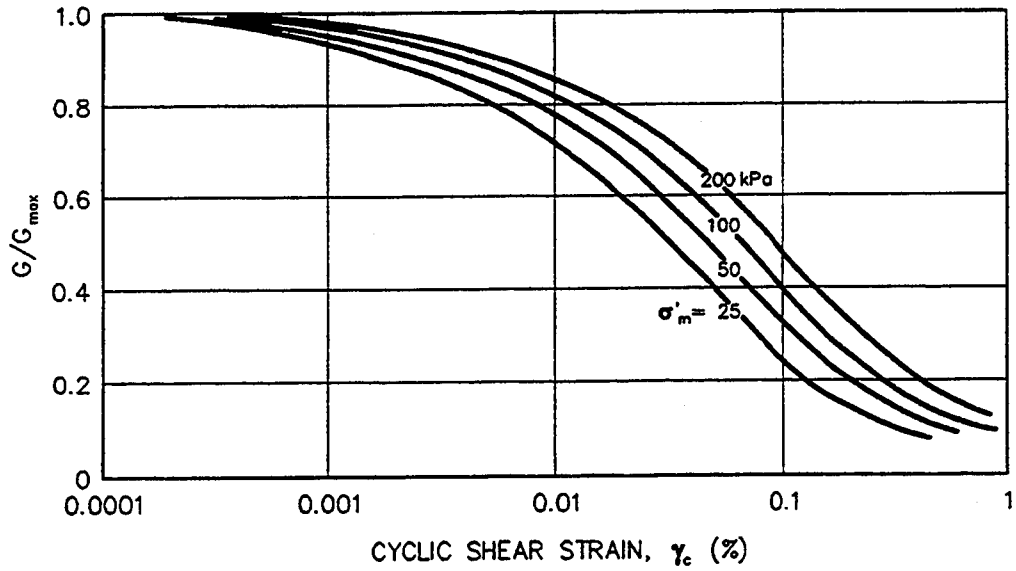


Figure 5-13: Shear Modulus Reduction Curves for Sands. (Iwasaki, *et al.*, 1978, reprinted by permission of Japanese Society of Soil Mechanics and Foundation Engineering)

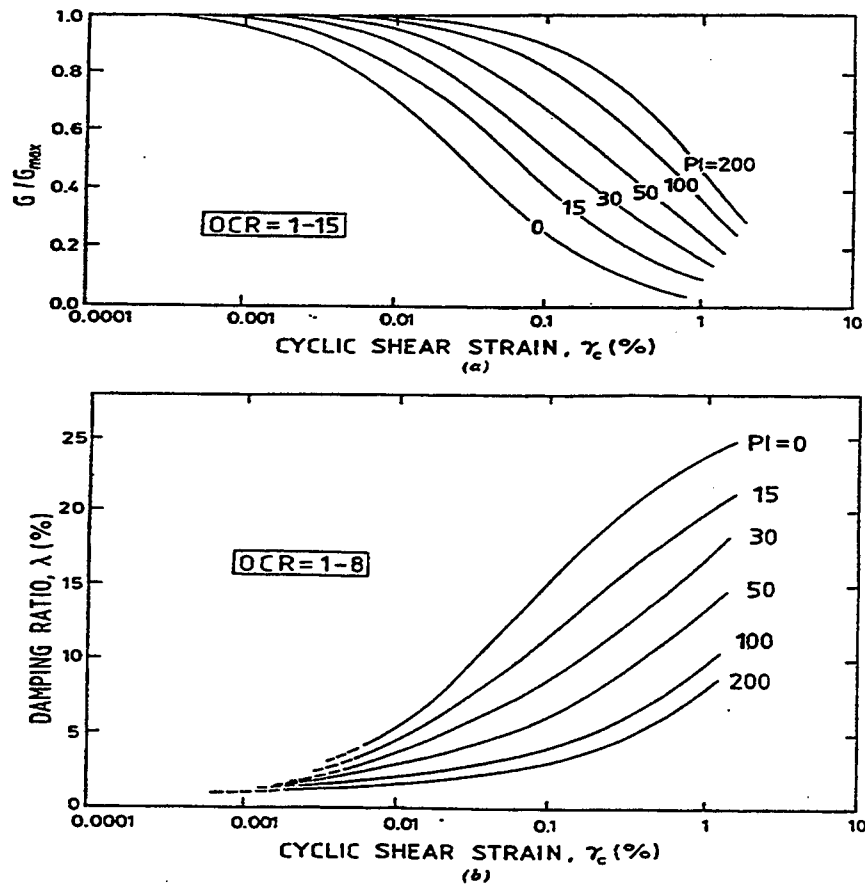


Figure 5-14: Shear Modulus Reduction and Damping Ratio as a Function of Shear Strain and Soil Plasticity Index. (Vucetic and Dobry, 1991, reprinted by permission of ASCE)

### SOLUTIONS TO EXERCISE NO. 3

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

$$K_o = 0.5 \quad (\text{See note 1})$$

Use Equation 5-12

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

2. From Figure 5-12, for  $e = 0.5$  (see note 2),

$$(K_2)_{\max} = 60$$

3. Use Equation 5-14

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

4. Use either  $e = 0.5$  curve from Figure 5-12,

$\sigma_m' = 200 \text{ kPa}$  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  $G_s = \text{Specific Gravity}$



## 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:

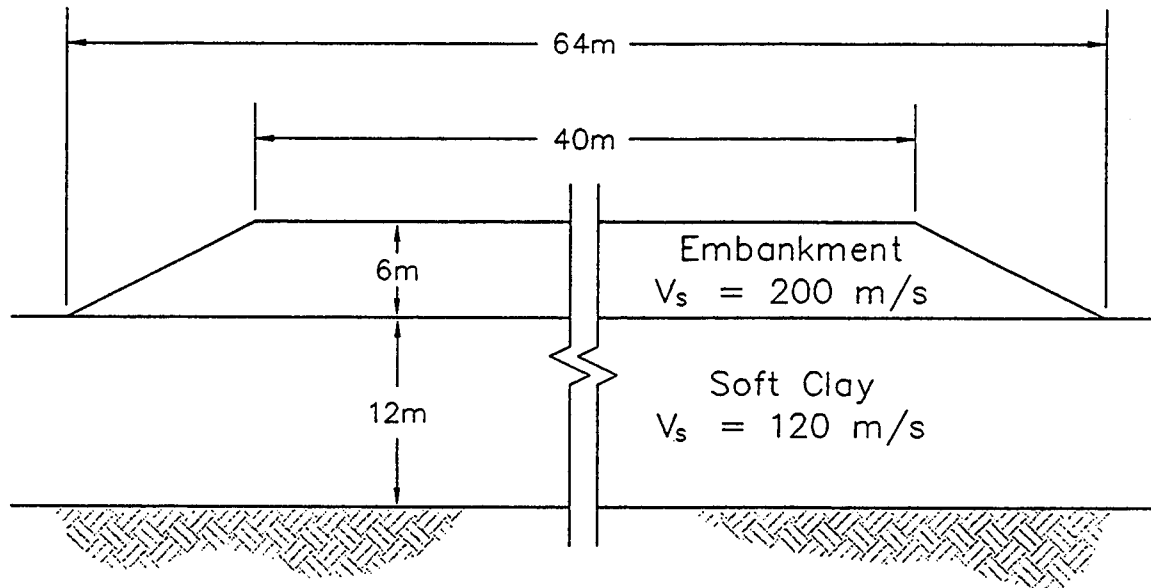
- Free Field Peak Ground Acceleration;
- Peak Acceleration at Top of Embankment;
- Fundamental Period of Clay Deposit in Free Field;
- Fundamental Period of Embankment.

#### **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:

$$PGA_{FF} = \underline{\hspace{2cm}}$$



## EXAMPLE

Figure S4-1: Soil Profile

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

$$PGA_{EMB} = \underline{\hspace{2cm}}$$

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

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

$$(T_o)_{FF} = \underline{\hspace{2cm}}$$

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

A.  $H =$  \_\_\_\_\_

B.  $h =$  \_\_\_\_\_

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

D.  $a_n =$  \_\_\_\_\_

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

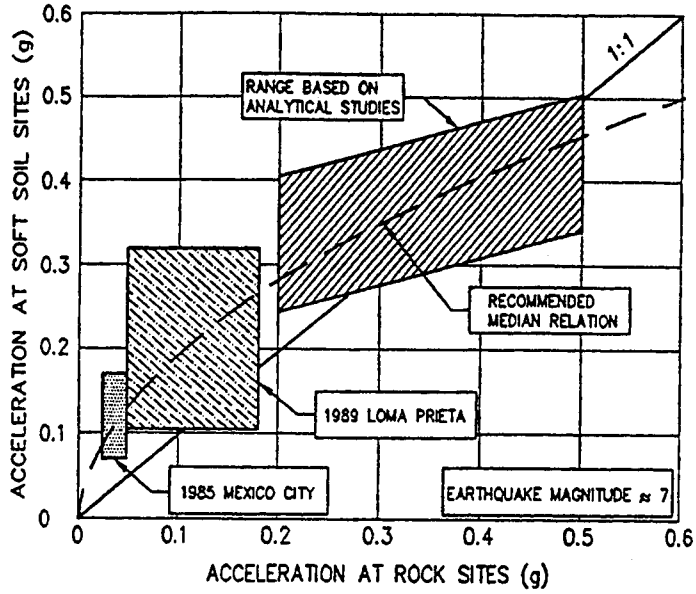


Figure 6-3: Relationship Between PHGA on Rock and on Soft Soil Sites. (Idriss, 1990)

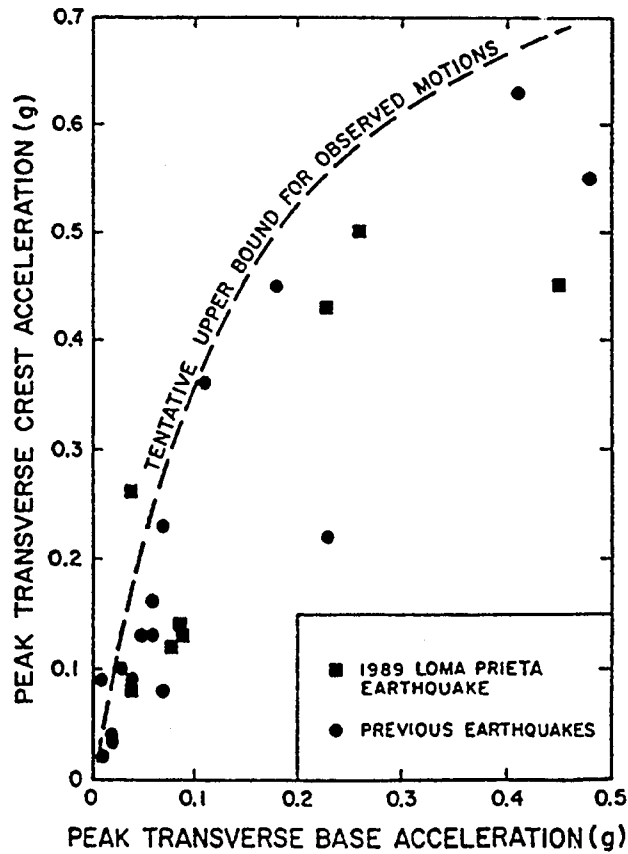
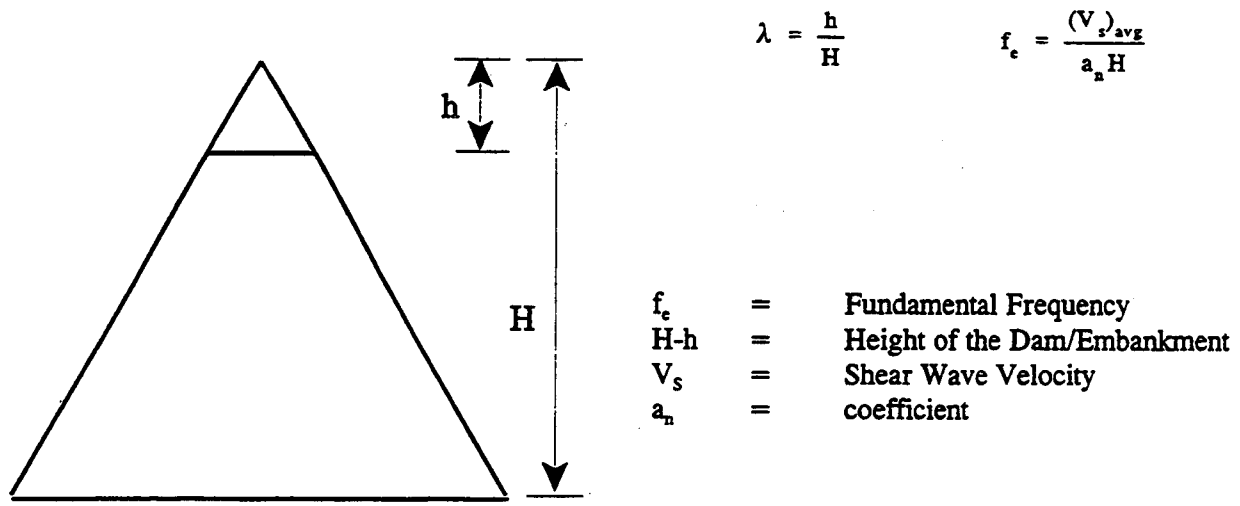


Figure 6-4: Comparisons of Peak Base and Crest Accelerations Recorded at Earth Dams. (Harder, 1991)



$\lambda$	$a_n$
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.  $PGA_{FF} = 0.26 \text{ g}$  (see attached figure)
2.  $PGA_{EMB} = 0.56 \text{ g}$  (see attached figure)
3.  $(T_o)_{FF} = \frac{4H}{V_s} = \frac{4 \times 12}{120} = 0.4 \text{ s}$
- 4A.  $H = 16 \text{ m}$
- 4B.  $h = 10 \text{ m}$
- 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.  $(T_o)_{EMB} = \frac{3.69 \times 6}{200} = 0.11 \text{ s}$

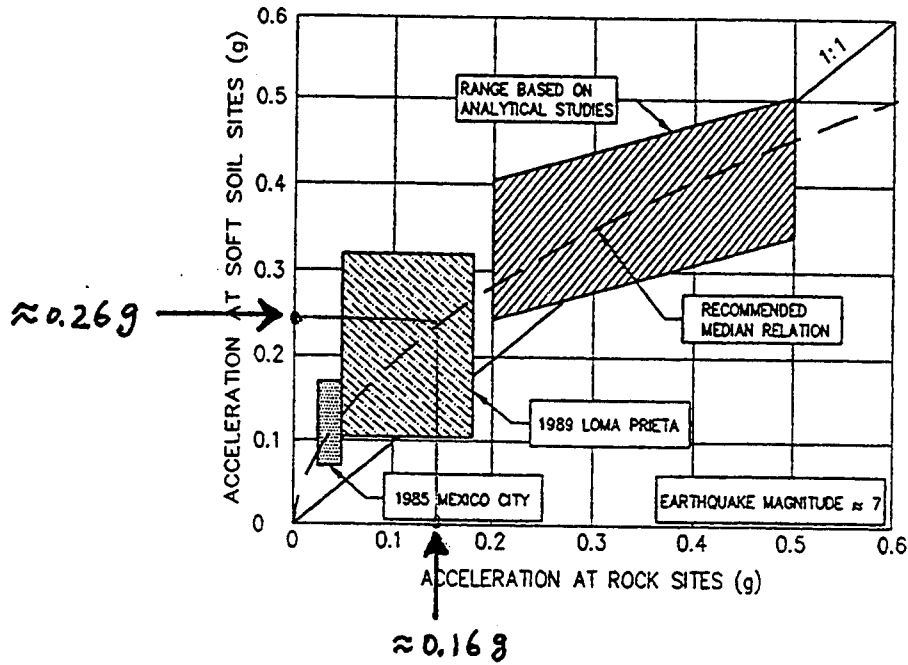


Figure 6-3: Relationship Between PHGA on Rock and on Soft Soil Sites. (Idriss, 1990)

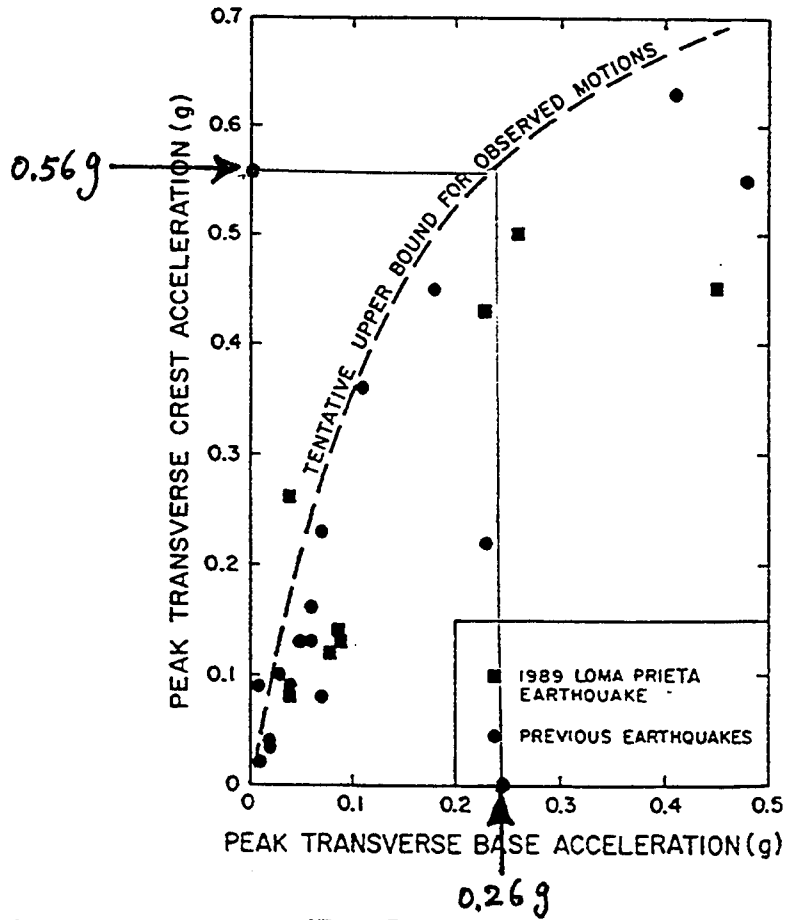


Figure 6-4: Comparisons of Peak Base and Crest Accelerations Recorded at Earth Dams. (Harder, 1991)



## 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:

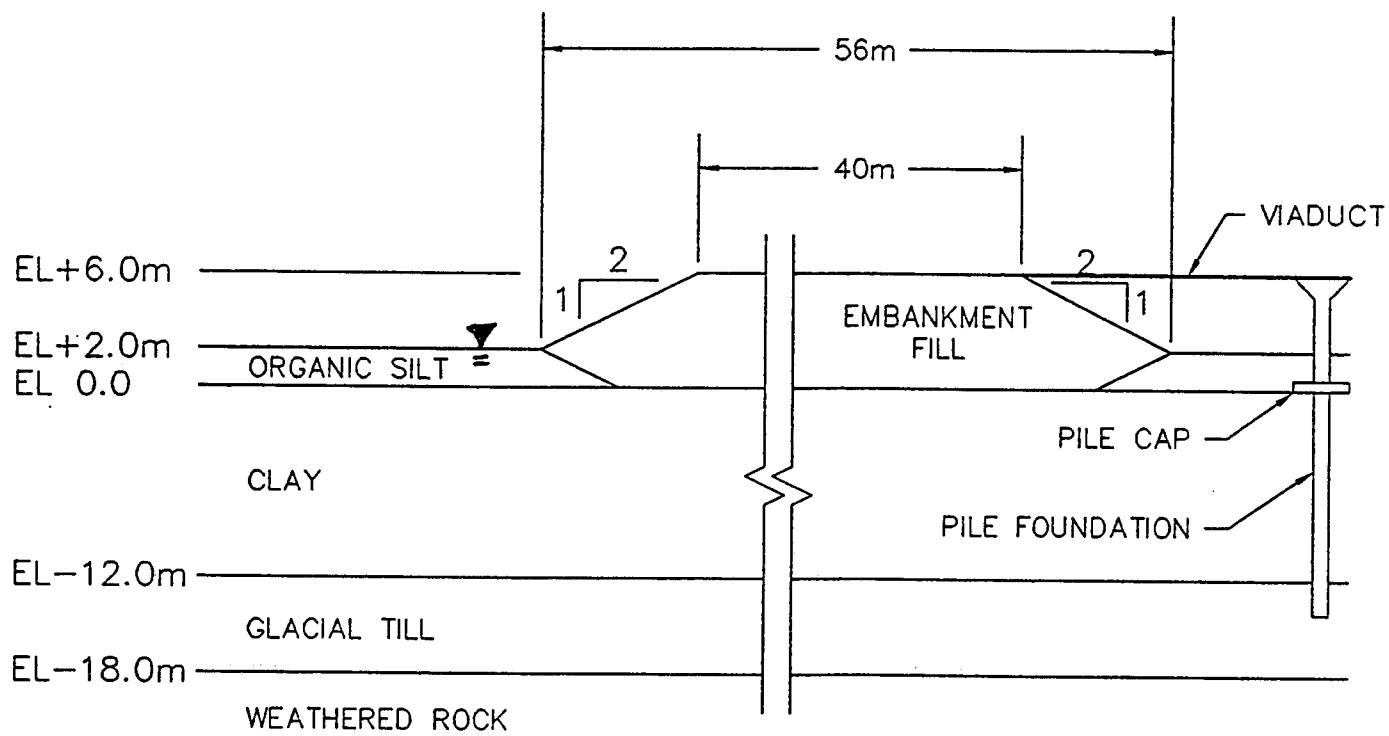
Reference Manual Part I: Figures 4-19, 5-12, 5-13, 5-14, Table 5-5, and Equations 5-2, 5-3, and 5-12.

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

$$\sigma_v' = \underline{\hspace{2cm}}$$

$$\sigma_m' = \underline{\hspace{2cm}}$$



**EXAMPLE**

**Figure S5-1: Subsurface Profile**

**TABLE S5-1  
SUMMARY OF AVAILABLE INFORMATION**

	Unit Weight (kN/m <sup>3</sup> )	PI (%)	OCR	(N <sub>1</sub> ) <sub>60</sub>	D <sub>r</sub> (%)	e <sub>0</sub>
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):

$$G_{\max} = \underline{\hspace{4cm}}$$

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

$$V_s = \underline{\hspace{4cm}}$$

2. Evaluate Properties of Clay Layer:

A. Evaluate Effective Stresses at Top and Bottom of Clay

$$(\sigma_v')_{\text{TOP}} =$$

\_\_\_\_\_

$$(\sigma_v')_{\text{BOTTOM}} =$$

\_\_\_\_\_

$$(\sigma_m')_{\text{TOP}} =$$

\_\_\_\_\_

$$(\sigma_m')_{\text{BOTTOM}} =$$

\_\_\_\_\_

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

$$(G_{\max})_{\text{TOP}} = \underline{\hspace{2cm}} \qquad (G_{\max})_{\text{BOTTOM}} = \underline{\hspace{2cm}}$$

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

$$(V_s)_{\text{TOP}} = \underline{\hspace{2cm}} \qquad (V_s)_{\text{BOTTOM}} = \underline{\hspace{2cm}}$$

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

A. Evaluate Effective Stresses

$$(\sigma_v')_{\text{TOP}} = \underline{\hspace{2cm}} \qquad (\sigma_v')_{\text{BOTTOM}} = \underline{\hspace{2cm}}$$

$$(\sigma_m')_{\text{TOP}} = \underline{\hspace{2cm}} \qquad (\sigma_m')_{\text{BOTTOM}} = \underline{\hspace{2cm}}$$

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

$$(G_{\max})_{\text{TOP}} = \underline{\hspace{2cm}} \qquad (G_{\max})_{\text{BOTTOM}} = \underline{\hspace{2cm}}$$

- C. Evaluate Shear Wave Velocity

$$(V_s)_{\text{TOP}} = \underline{\hspace{2cm}} \qquad (V_s)_{\text{BOTTOM}} = \underline{\hspace{2cm}}$$

4. Evaluate Properties in Center of Till

- A. Evaluate Effective Stresses

$$\sigma_v' = \underline{\hspace{2cm}}$$

$$\sigma_m' = \underline{\hspace{2cm}}$$

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

$$G_{\max} = \underline{\hspace{2cm}}$$

C. Evaluate Shear Wave Velocity

$$V_s = \underline{\hspace{2cm}}$$

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

<b>Soil</b>	<b>Figure Number</b>	<b>Curve</b>
Silt		
Clay		
Embankment		
Till		

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

A.  $(V_s)_{AVE} = \underline{\hspace{2cm}}$

B.  $a_n = \underline{\hspace{2cm}}$

C.  $T_o = \underline{\hspace{2cm}}$



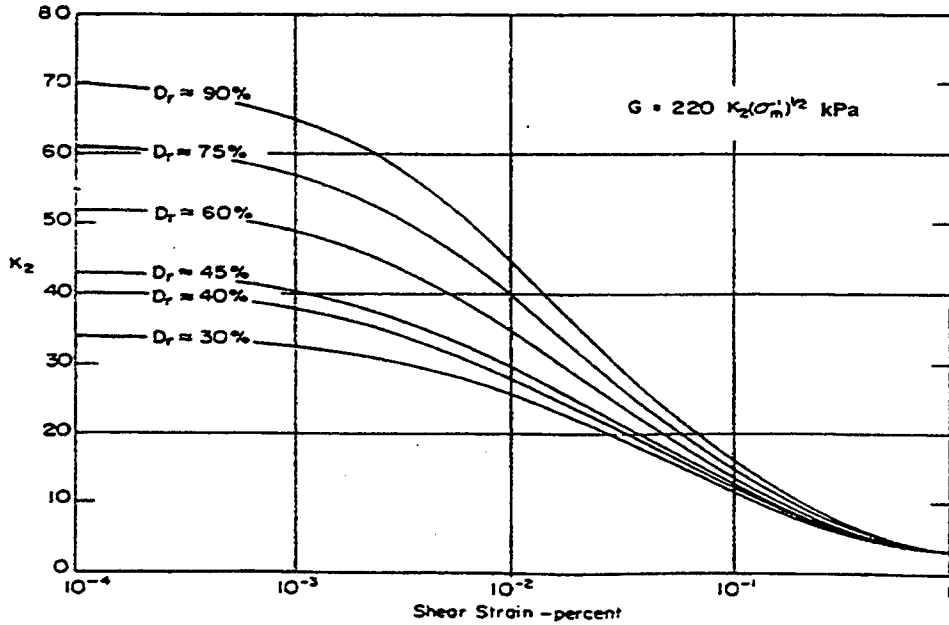
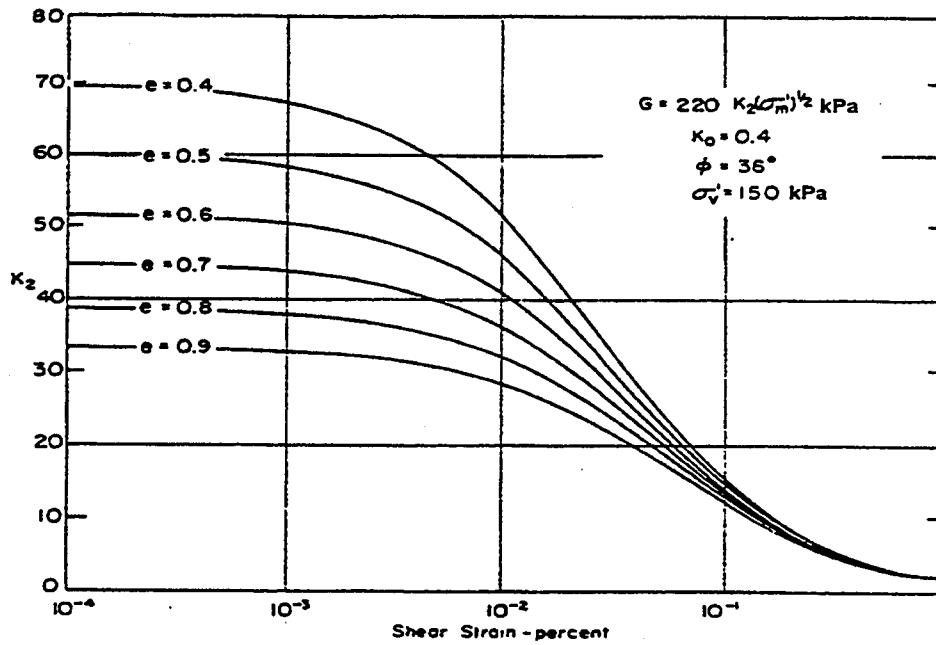


Figure 5-12: Shear Modulus Reduction Curves for Sands. (Seed and Idriss, 1970, reprinted by permission of ASCE)

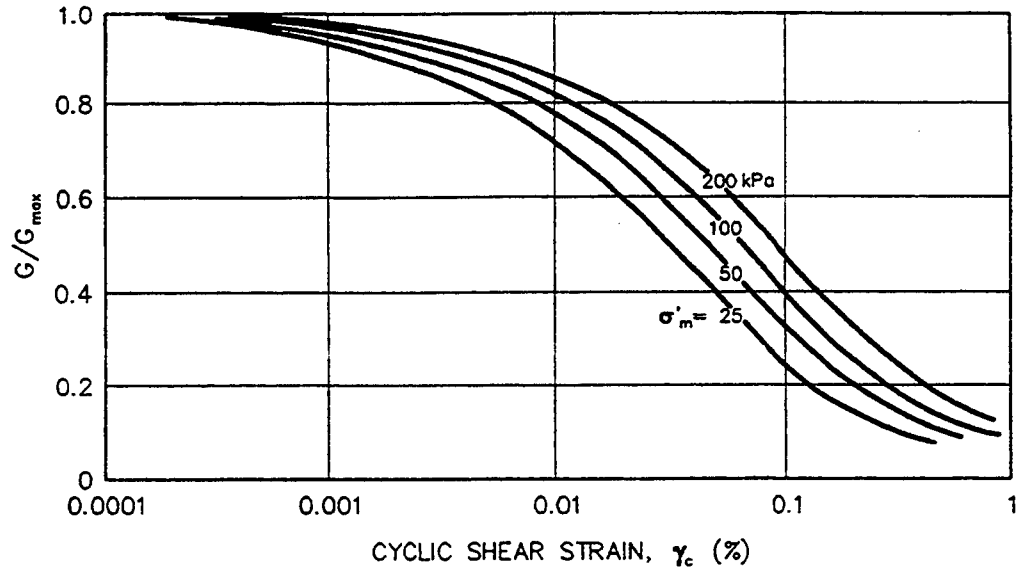


Figure 5-13: Shear Modulus Reduction Curves for Sands. (Iwasaki, *et al.*, 1978, reprinted by permission of Japanese Society of Soil Mechanics and Foundation Engineering)

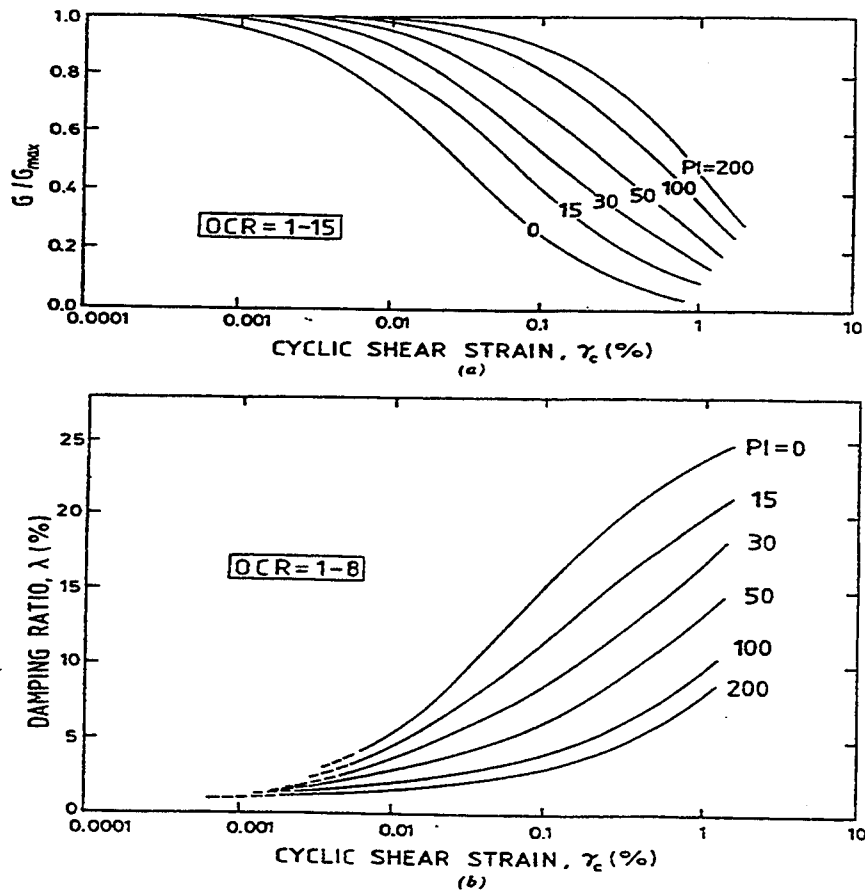
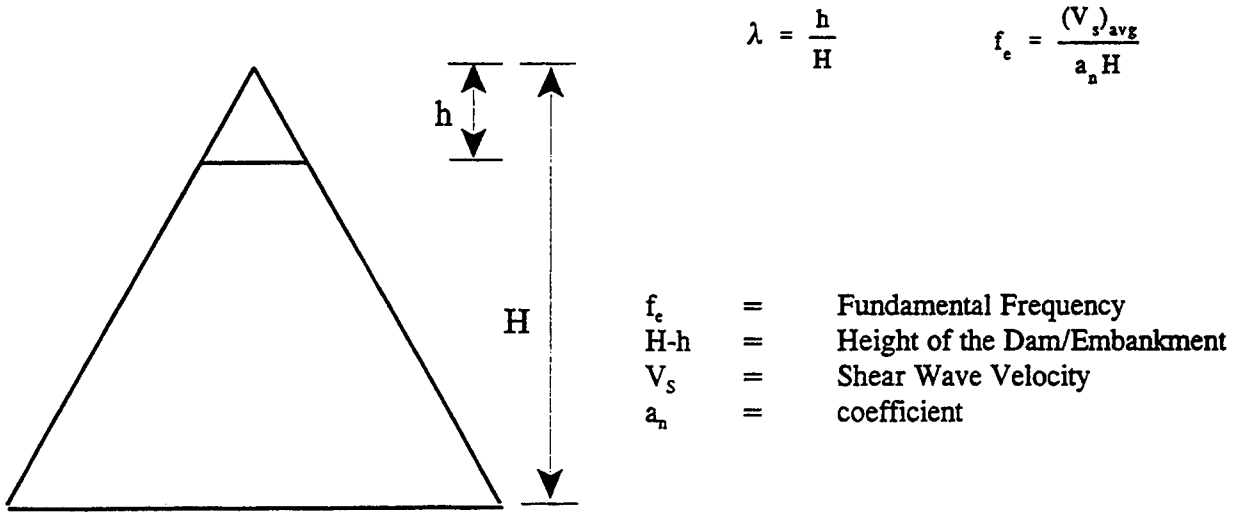


Figure 5-14: Shear Modulus Reduction and Damping Ratio as a Function of Shear Strain and Soil Plasticity Index. (Vucetic and Dobry, 1991, reprinted by permission of ASCE)



$\lambda$	$a_n$
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. 5

<b>Material</b>	<b>Location</b>	$\gamma$ kN/m <sup>3</sup>	$\sigma'_v$ kPa	$\sigma'_m$ kPa	$G_{max}$ kPa	$V_s$ m/s	<b>Modulus Reduction and Damping</b>
Embankment	1 m from top	19.5	19.5	19.5	60,230	174	PI = 0
	1 m above clay		87.7	64.3	109,390	234	
Silt	Middle	12	2.2	1.5	1,820	39	PI = 50
Clay	Top	16	4.4	2.9	8,440	72	PI = 15
	Bottom		78.8	52.5	45,300	167	
	Top w/ Embankment		97.4	64.9	50,350	175	
	Bottom w/ Embankment		171.8	114.5	76,700	217	
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:

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

$$6B. a_n = 4$$

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

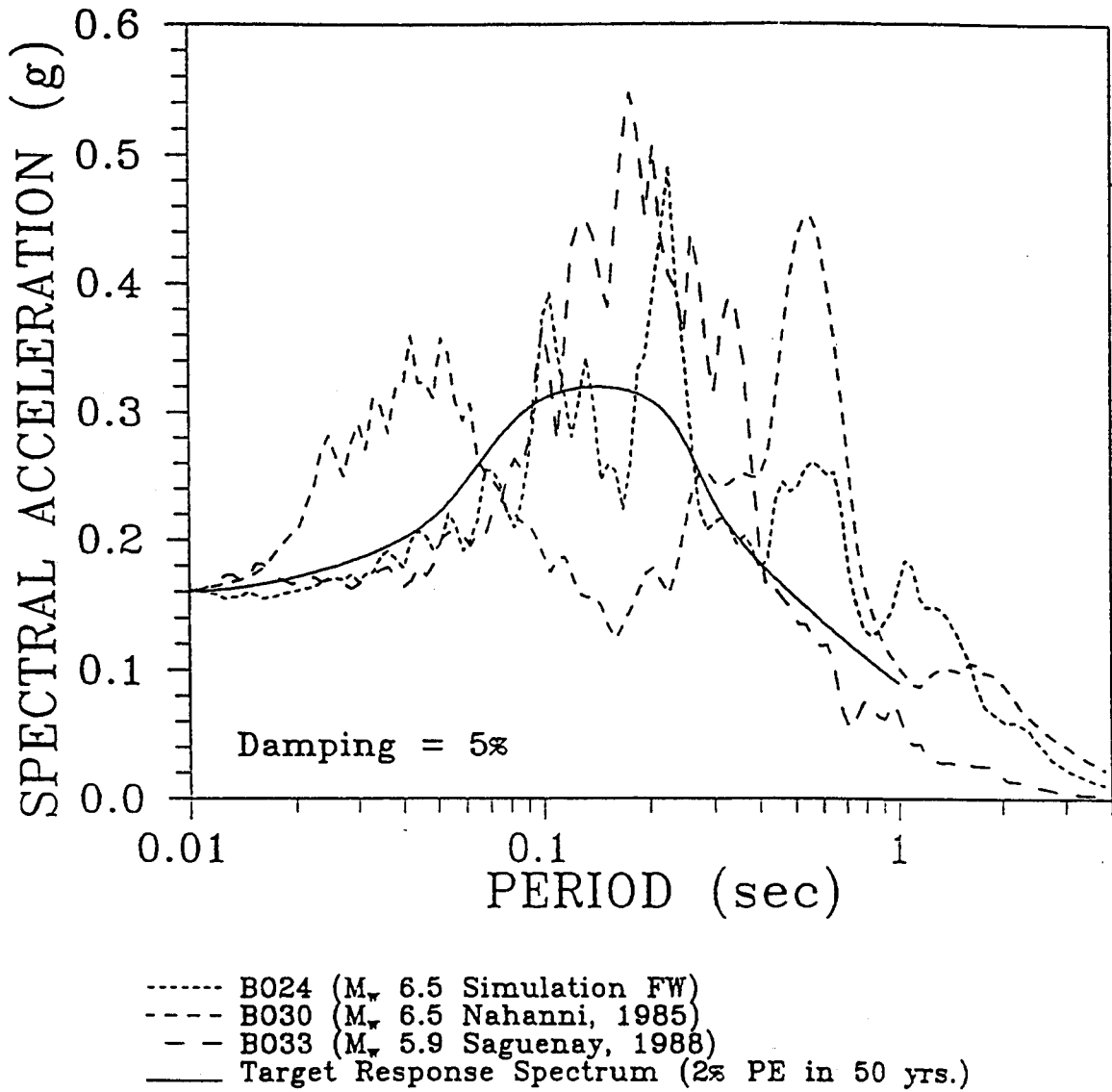


Figure 4-3a: Design Ground Motions- Peak Acceleration Scaling

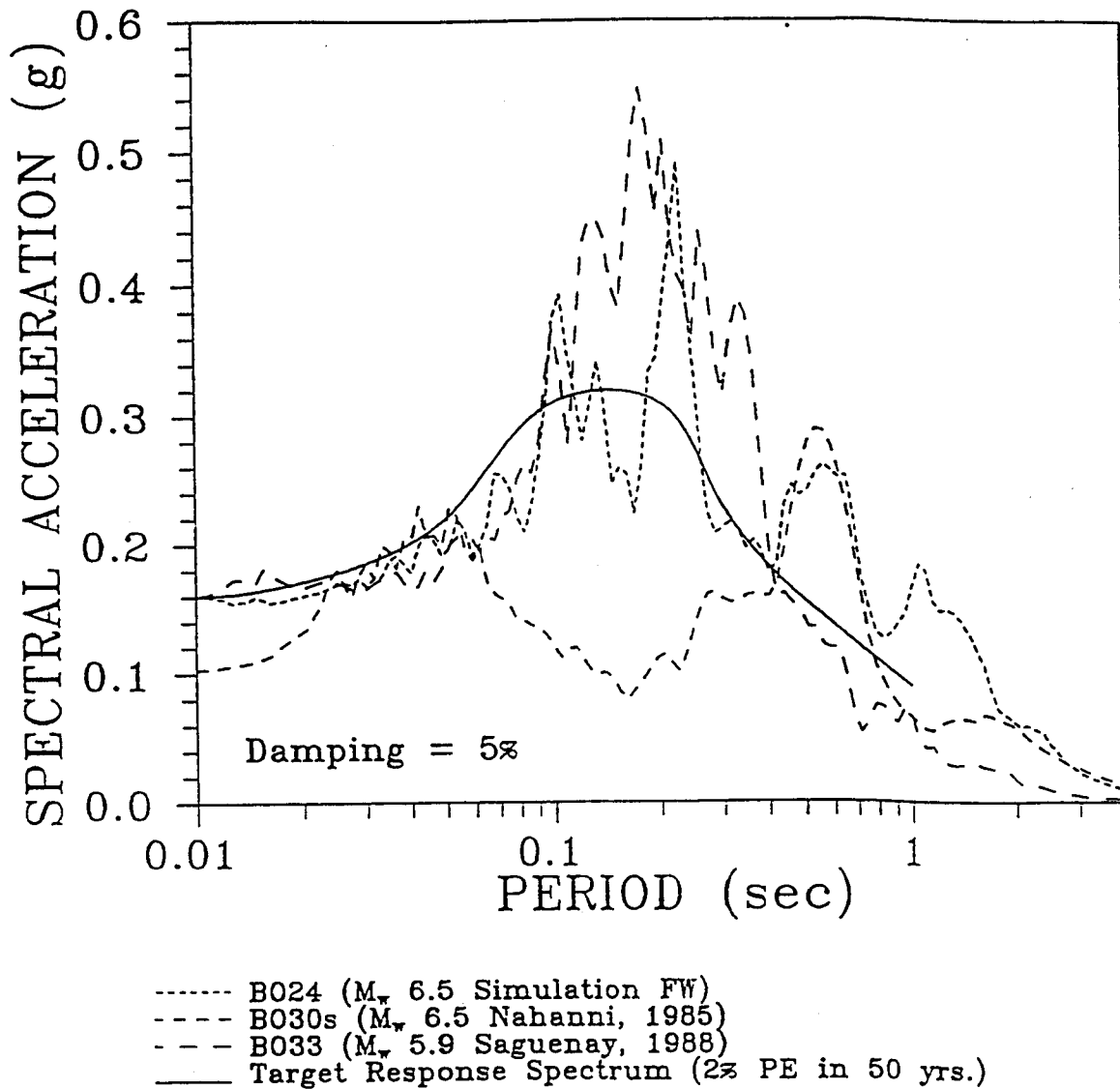


Figure 4-3b: Design Ground Motions- Spectral Acceleration Scaling

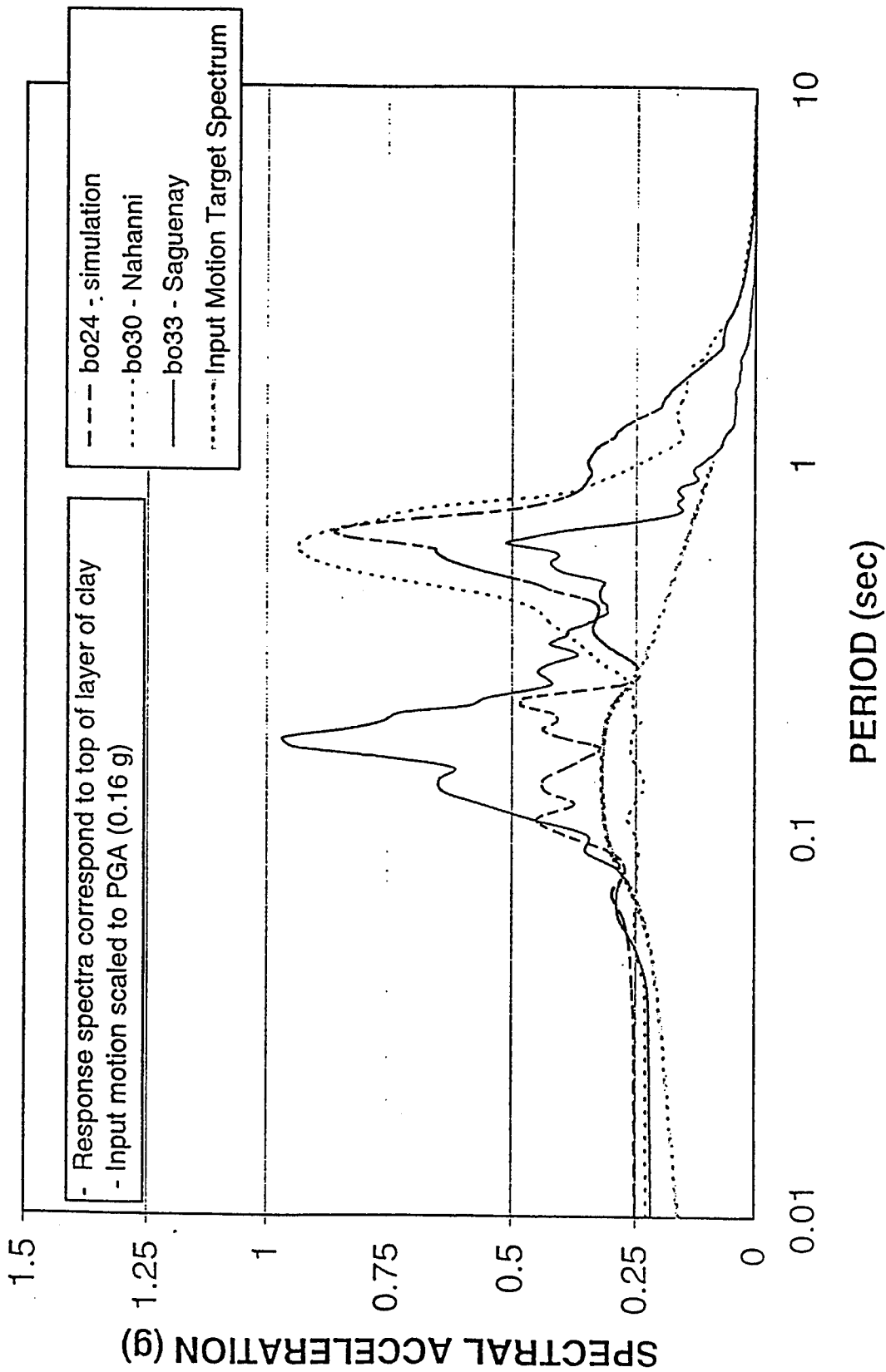


Figure 4-4: Free-Field Acceleration Response Spectra- Peak Acceleration Scaling



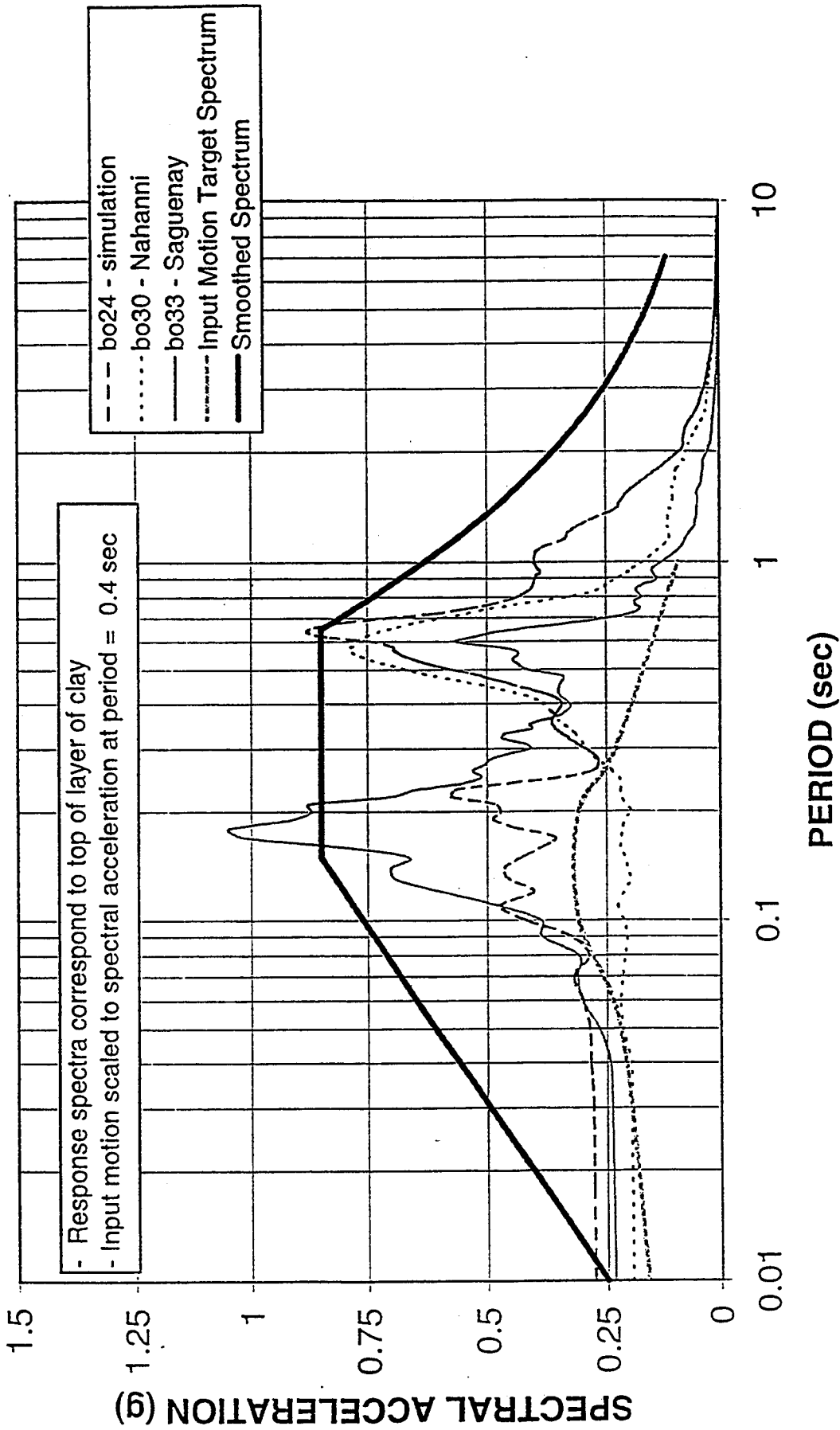


Figure 4-5: Free-Field Acceleration Response Spectra- Spectral Acceleration Scaling

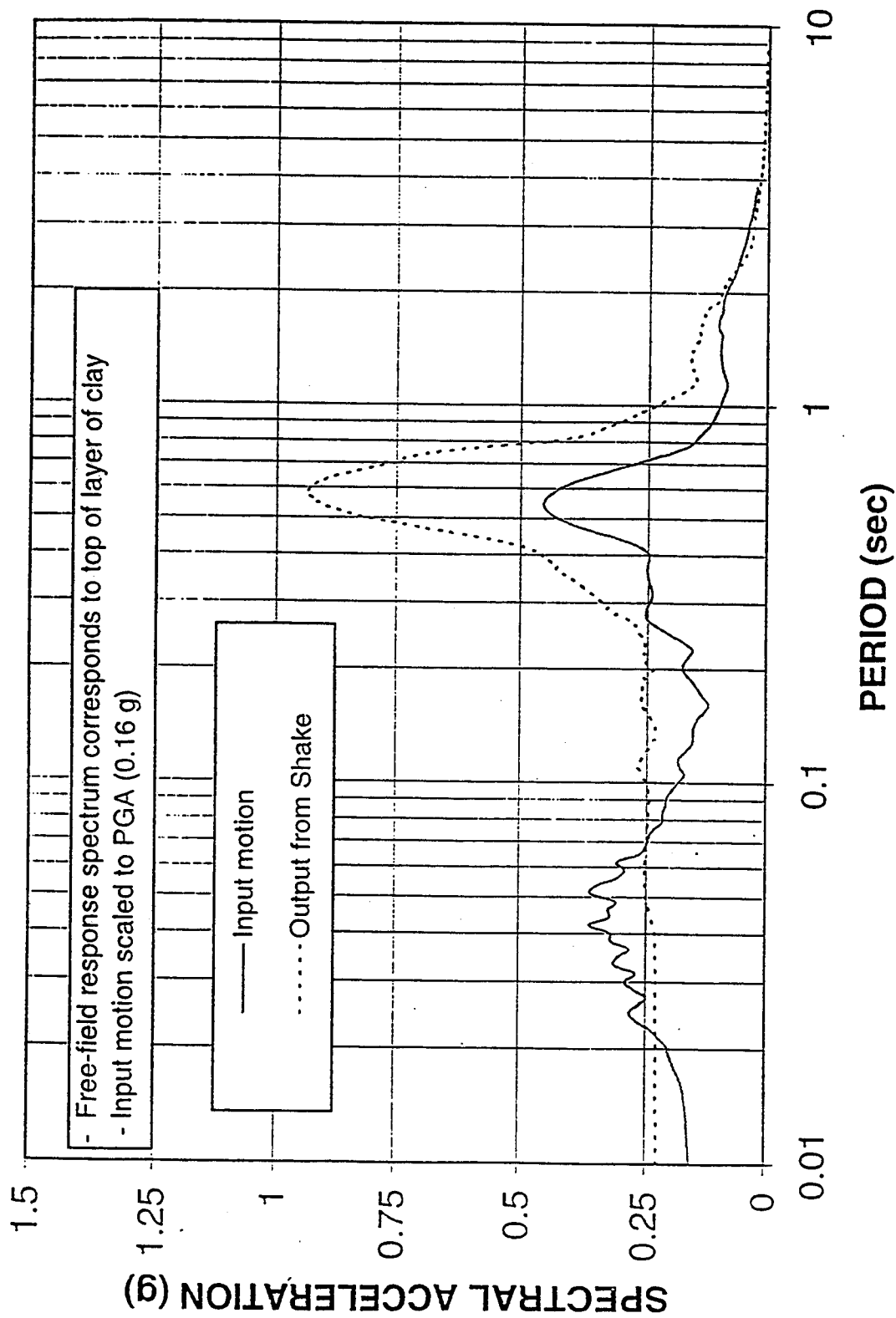


Figure 4-6: Acceleration Response Spectra- Peak Acceleration Scaling (Input vs. Free-Field for Nahanni Record).

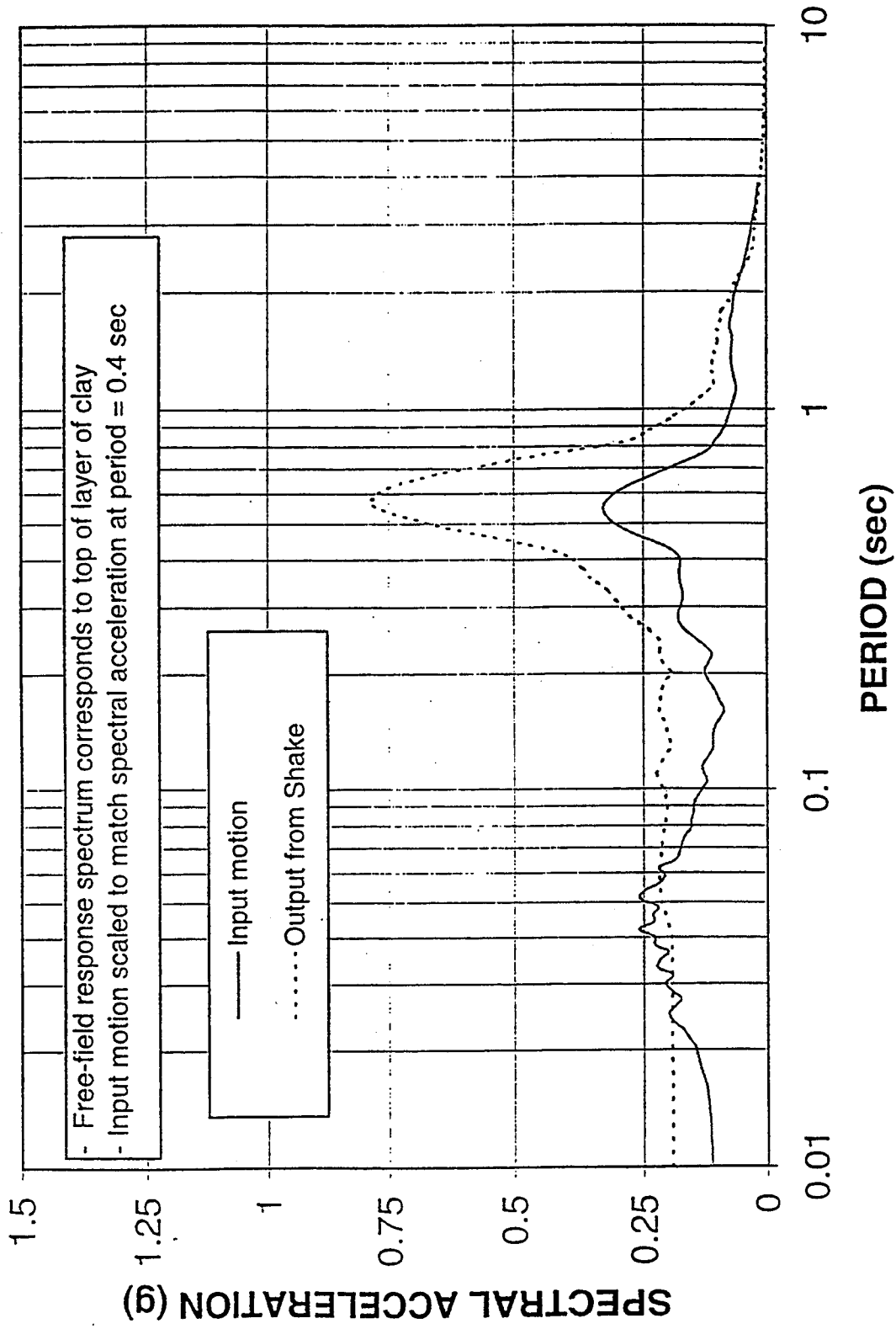


Figure 4-7: Acceleration Response Spectra- Spectral Acceleration Scaling (Input vs. Free-Field for Nahanni Record).

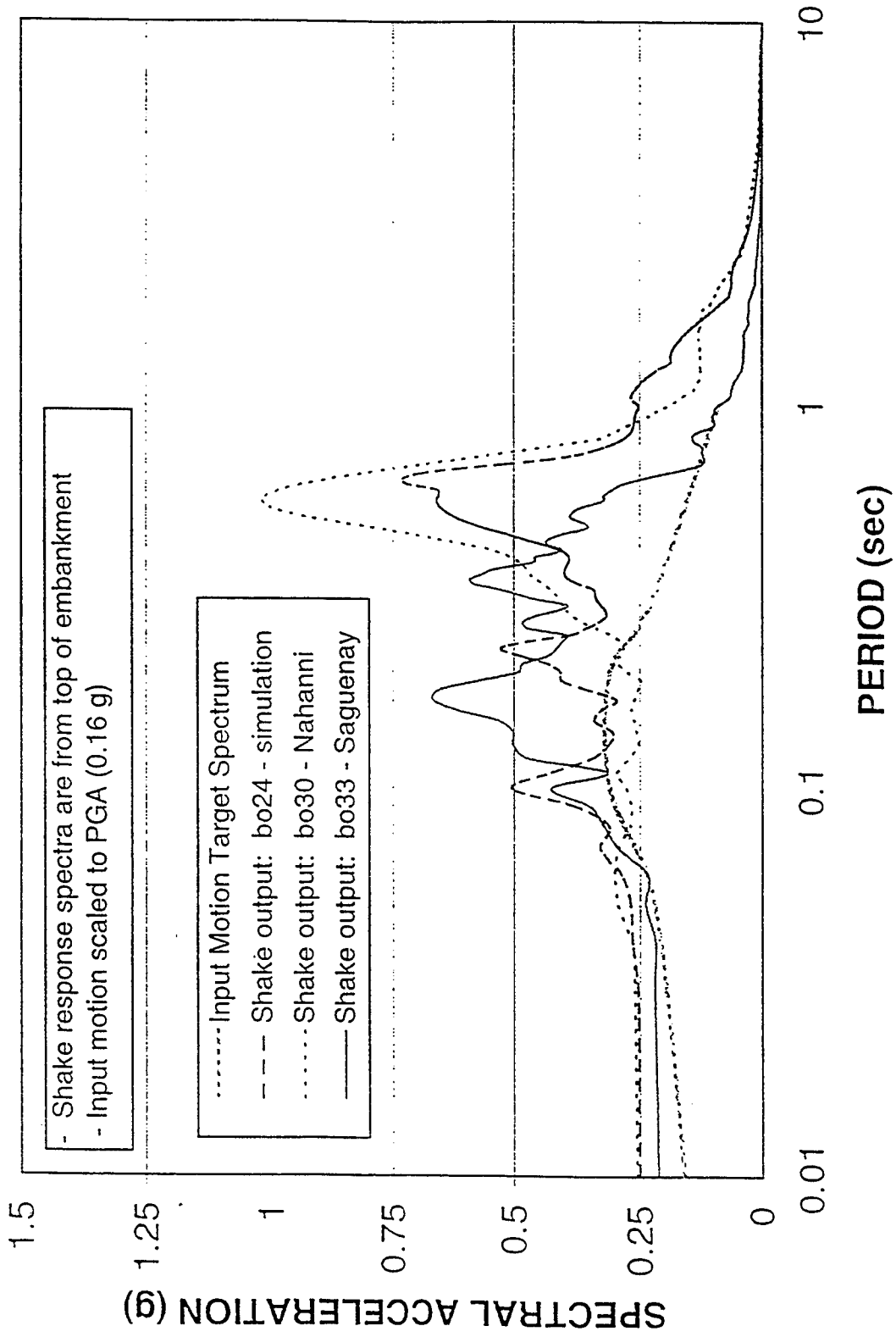


Figure 4-8: Acceleration Response Spectra- SHAKE Analysis vs. Target Spectra

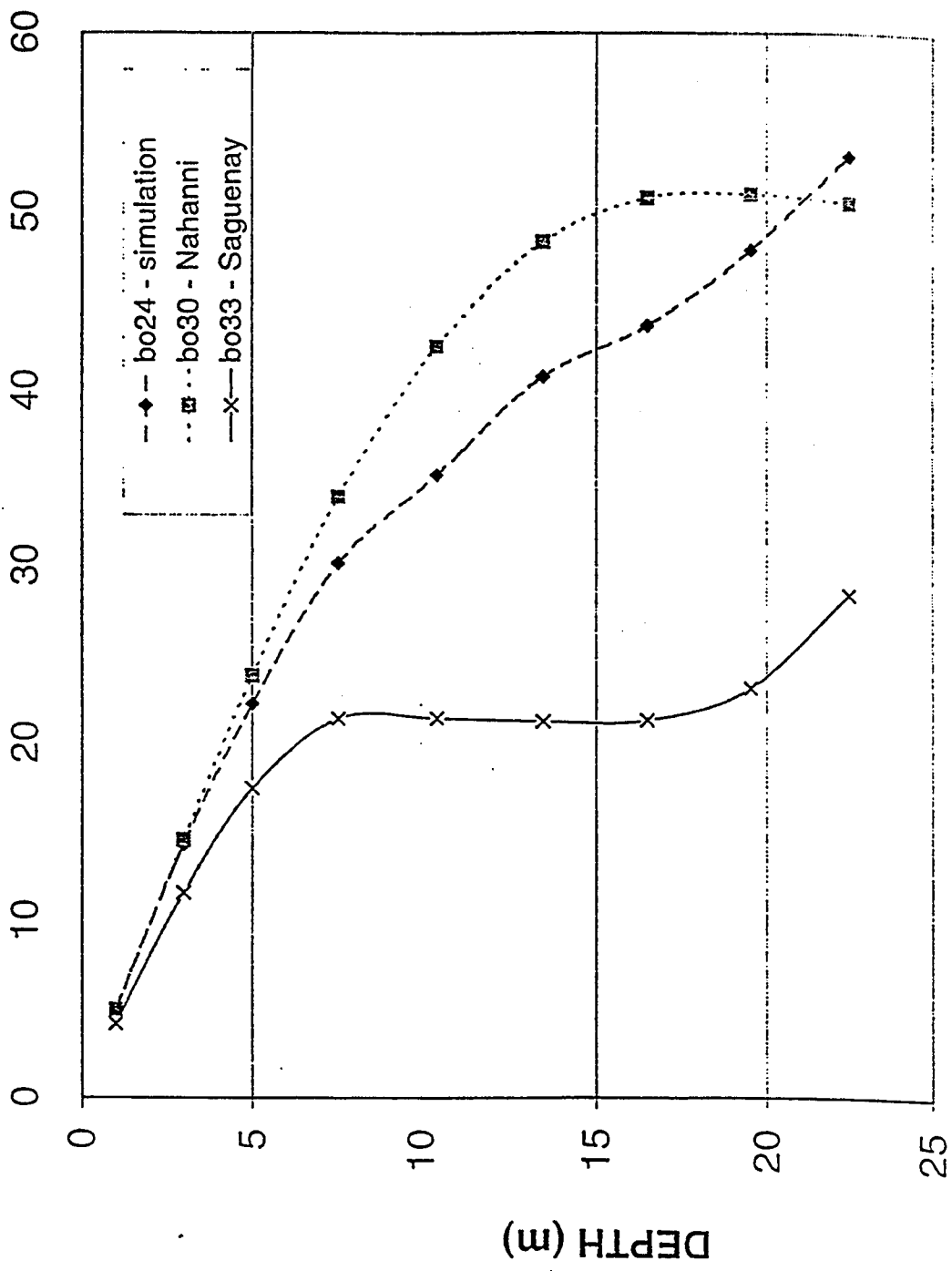


Figure 4-9: Maximum Shear Stresses with Depth.

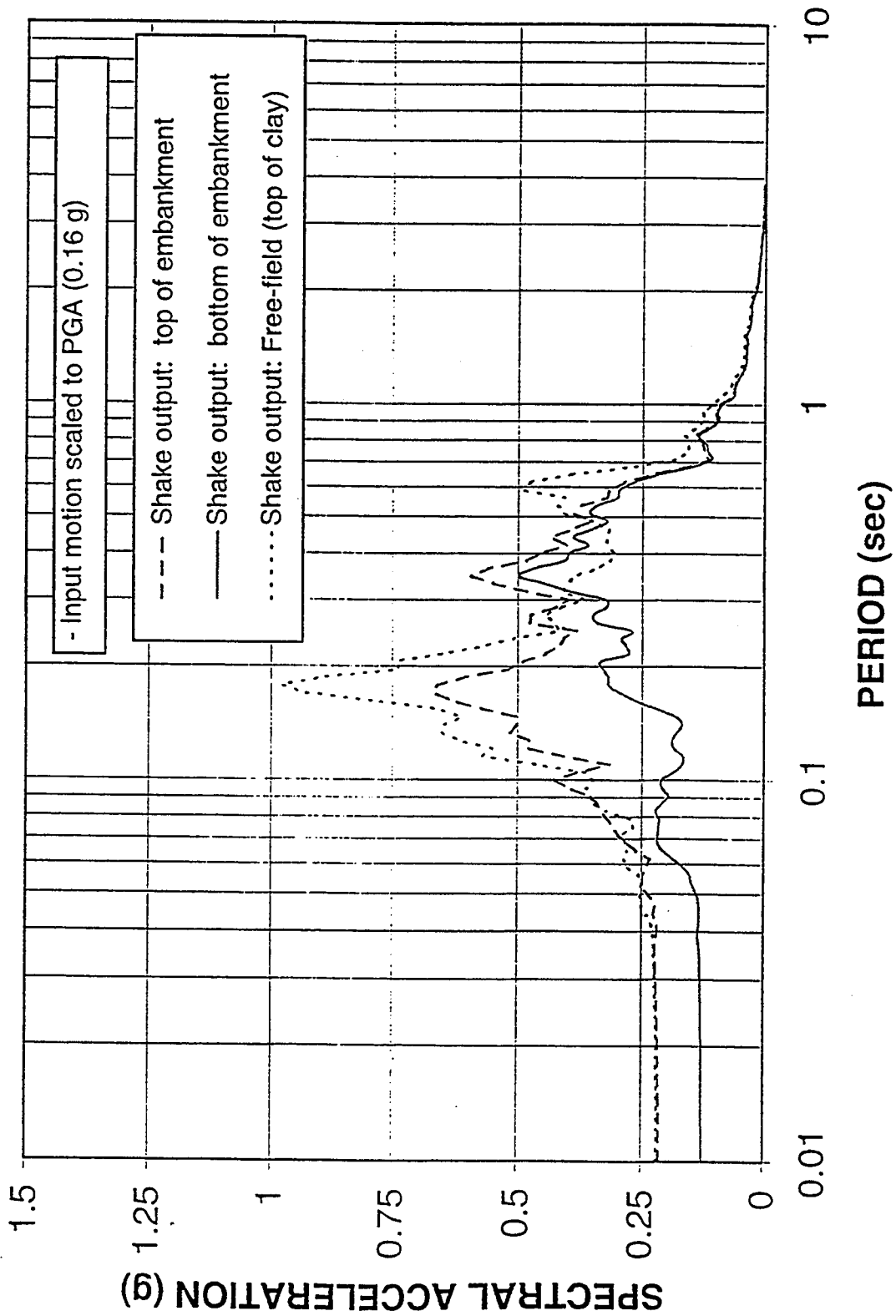


Figure 4-10: Acceleration Response Spectra- Embankment vs. Free-Field for Saguenay Record.

```

OPTION 1 - dynamic soil properties - (max is thirteen):
1
5
9
0.0001
1.0
1.000
0.02
0.0001
1.0
2.0
24.0
0.0001
1.0
1.000
0.22
0.0001
1.0
13.2
0.0001
1.000
0.1
0.0001
1.000
0.1
0.06
0.0001
1.0
2.0
23.8
0.0001
1.000
0.0001
0.4
0.4
5
OPTION 2 -- Soil Profile
2
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33
34
35
36
37
38
39
40
41
42
43
44
45
46
47
48
49
50
51
52
53
54
55
56
57
58
59
60
61
62
63
64
65
66
67
68
69
70
71
72
73
74
75
76
77
78
79
80
81
82
83
84
85
86
87
88
89
90
91
92
93
94
95
96
97
98
99
100
101
102
103
104
105
106
107
108
109
110
111
112
113
114
115
116
117
118
119
120
121
122
123
124
125
126
127
128
129
130
131
132
133
134
135
136
137
138
139
140
141
142
143
144
145
146
147
148
149
150
151
152
153
154
155
156
157
158
159
160
161
162
163
164
165
166
167
168
169
170
171
172
173
174
175
176
177
178
179
180
181
182
183
184
185
186
187
188
189
190
191
192
193
194
195
196
197
198
199
200
201
202
203
204
205
206
207
208
209
210
211
212
213
214
215
216
217
218
219
220
221
222
223
224
225
226
227
228
229
230
231
232
233
234
235
236
237
238
239
240
241
242
243
244
245
246
247
248
249
250
251
252
253
254
255
256
257
258
259
260
261
262
263
264
265
266
267
268
269
270
271
272
273
274
275
276
277
278
279
280
281
282
283
284
285
286
287
288
289
290
291
292
293
294
295
296
297
298
299
300
301
302
303
304
305
306
307
308
309
310
311
312
313
314
315
316
317
318
319
320
321
322
323
324
325
326
327
328
329
330
331
332
333
334
335
336
337
338
339
340
341
342
343
344
345
346
347
348
349
350
351
352
353
354
355
356
357
358
359
360
361
362
363
364
365
366
367
368
369
370
371
372
373
374
375
376
377
378
379
380
381
382
383
384
385
386
387
388
389
390
391
392
393
394
395
396
397
398
399
400
401
402
403
404
405
406
407
408
409
410
411
412
413
414
415
416
417
418
419
420
421
422
423
424
425
426
427
428
429
430
431
432
433
434
435
436
437
438
439
440
441
442
443
444
445
446
447
448
449
450
451
452
453
454
455
456
457
458
459
460
461
462
463
464
465
466
467
468
469
470
471
472
473
474
475
476
477
478
479
480
481
482
483
484
485
486
487
488
489
490
491
492
493
494
495
496
497
498
499
500
501
502
503
504
505
506
507
508
509
510
511
512
513
514
515
516
517
518
519
520
521
522
523
524
525
526
527
528
529
530
531
532
533
534
535
536
537
538
539
540
541
542
543
544
545
546
547
548
549
550
551
552
553
554
555
556
557
558
559
560
561
562
563
564
565
566
567
568
569
570
571
572
573
574
575
576
577
578
579
580
581
582
583
584
585
586
587
588
589
590
591
592
593
594
595
596
597
598
599
600
601
602
603
604
605
606
607
608
609
610
611
612
613
614
615
616
617
618
619
620
621
622
623
624
625
626
627
628
629
630
631
632
633
634
635
636
637
638
639
640
641
642
643
644
645
646
647
648
649
650
651
652
653
654
655
656
657
658
659
660
661
662
663
664
665
666
667
668
669
670
671
672
673
674
675
676
677
678
679
680
681
682
683
684
685
686
687
688
689
690
691
692
693
694
695
696
697
698
699
700
701
702
703
704
705
706
707
708
709
710
711
712
713
714
715
716
717
718
719
720
721
722
723
724
725
726
727
728
729
730
731
732
733
734
735
736
737
738
739
740
741
742
743
744
745
746
747
748
749
750
751
752
753
754
755
756
757
758
759
760
761
762
763
764
765
766
767
768
769
770
771
772
773
774
775
776
777
778
779
780
781
782
783
784
785
786
787
788
789
790
791
792
793
794
795
796
797
798
799
800
801
802
803
804
805
806
807
808
809
810
811
812
813
814
815
816
817
818
819
820
821
822
823
824
825
826
827
828
829
830
831
832
833
834
835
836
837
838
839
840
841
842
843
844
845
846
847
848
849
850
851
852
853
854
855
856
857
858
859
860
861
862
863
864
865
866
867
868
869
870
871
872
873
874
875
876
877
878
879
880
881
882
883
884
885
886
887
888
889
890
891
892
893
894
895
896
897
898
899
900
901
902
903
904
905
906
907
908
909
910
911
912
913
914
915
916
917
918
919
920
921
922
923
924
925
926
927
928
929
930
931
932
933
934
935
936
937
938
939
940
941
942
943
944
945
946
947
948
949
950
951
952
953
954
955
956
957
958
959
960
961
962
963
964
965
966
967
968
969
970
971
972
973
974
975
976
977
978
979
980
981
982
983
984
985
986
987
988
989
990
991
992
993
994
995
996
997
998
999
1000
1001
1002
1003
1004
1005
1006
1007
1008
1009
1010
1011
1012
1013
1014
1015
1016
1017
1018
1019
1020
1021
1022
1023
1024
1025
1026
1027
1028
1029
1030
1031
1032
1033
1034
1035
1036
1037
1038
1039
1040
1041
1042
1043
1044
1045
1046
1047
1048
1049
1050
1051
1052
1053
1054
1055
1056
1057
1058
1059
1060
1061
1062
1063
1064
1065
1066
1067
1068
1069
1070
1071
1072
1073
1074
1075
1076
1077
1078
1079
1080
1081
1082
1083
1084
1085
1086
1087
1088
1089
1090
1091
1092
1093
1094
1095
1096
1097
1098
1099
1100
1101
1102
1103
1104
1105
1106
1107
1108
1109
1110
1111
1112
1113
1114
1115
1116
1117
1118
1119
1120
1121
1122
1123
1124
1125
1126
1127
1128
1129
1130
1131
1132
1133
1134
1135
1136
1137
1138
1139
1140
1141
1142
1143
1144
1145
1146
1147
1148
1149
1150
1151
1152
1153
1154
1155
1156
1157
1158
1159
1160
1161
1162
1163
1164
1165
1166
1167
1168
1169
1170
1171
1172
1173
1174
1175
1176
1177
1178
1179
1180
1181
1182
1183
1184
1185
1186
1187
1188
1189
1190
1191
1192
1193
1194
1195
1196
1197
1198
1199
1200
1201
1202
1203
1204
1205
1206
1207
1208
1209
1210
1211
1212
1213
1214
1215
1216
1217
1218
1219
1220
1221
1222
1223
1224
1225
1226
1227
1228
1229
1230
1231
1232
1233
1234
1235
1236
1237
1238
1239
1240
1241
1242
1243
1244
1245
1246
1247
1248
1249
1250
1251
1252
1253
1254
1255
1256
1257
1258
1259
1260
1261
1262
1263
1264
1265
1266
1267
1268
1269
1270
1271
1272
1273
1274
1275
1276
1277
1278
1279
1280
1281
1282
1283
1284
1285
1286
1287
1288
1289
1290
1291
1292
1293
1294
1295
1296
1297
1298
1299
1300
1301
1302
1303
1304
1305
1306
1307
1308
1309
1310
1311
1312
1313
1314
1315
1316
1317
1318
1319
1320
1321
1322
1323
1324
1325
1326
1327
1328
1329
1330
1331
1332
1333
1334
1335
1336
1337
1338
1339
1340
1341
1342
1343
1344
1345
1346
1347
1348
1349
1350
1351
1352
1353
1354
1355
1356
1357
1358
1359
1360
1361
1362
1363
1364
1365
1366
1367
1368
1369
1370
1371
1372
1373
1374
1375
1376
1377
1378
1379
1380
1381
1382
1383
1384
1385
1386
1387
1388
1389
1390
1391
1392
1393
1394
1395
1396
1397
1398
1399
1400
1401
1402
1403
1404
1405
1406
1407
1408
1409
1410
1411
1412
1413
1414
1415
1416
1417
1418
1419
1420
1421
1422
1423
1424
1425
1426
1427
1428
1429
1430
1431
1432
1433
1434
1435
1436
1437
1438
1439
1440
1441
1442
1443
1444
1445
1446
1447
1448
1449
1450
1451
1452
1453
1454
1455
1456
1457
1458
1459
1460
1461
1462
1463
1464
1465
1466
1467
1468
1469
1470
1471
1472
1473
1474
1475
1476
1477
1478
1479
1480
1481
1482
1483
1484
1485
1486
1487
1488
1489
1490
1491
1492
1493
1494
1495
1496
1497
1498
1499
1500
1501
1502
1503
1504
1505
1506
1507
1508
1509
1510
1511
1512
1513
1514
1515
1516
1517
1518
1519
1520
1521
1522
1523
1524
1525
1526
1527
1528
1529
1530
1531
1532
1533
1534
1535
1536
1537
1538
1539
1540
1541
1542
1543
1544
1545
1546
1547
1548
1549
1550
1551
1552
1553
1554
1555
1556
1557
1558
1559
1560
1561
1562
1563
1564
1565
1566
1567
1568
1569
1570
1571
1572
1573
1574
1575
1576
1577
1578
1579
1580
1581
1582
1583
1584
1585
1586
1587
1588
1589
1590
1591
1592
1593
1594
1595
1596
1597
1598
1599
1600
1601
1602
1603
1604
1605
1606
1607
1608
1609
1610
1611
1612
1613
1614
1615
1616
1617
1618
1619
1620
1621
1622
1623
1624
1625
1626
1627
1628
1629
1630
1631
1632
1633
1634
1635
1636
1637
1638
1639
1640
1641
1642
1643
1644
1645
1646
1647
1648
1649
1650
1651
1652
1653
1654
1655
1656
1657
1658
1659
1660
1661
1662
1663
1664
1665
1666
1667
1668
1669
1670
1671
1672
1673
1674
1675
1676
1677
1678
1679
1680
1681
1682
1683
1684
1685
1686
1687
1688
1689
1690
1691
1692
1693
1694
1695
1696
1697
1698
1699
1700
1701
1702
1703
1704
1705
1706
1707
1708
1709
1710
1711
1712
1713
1714
1715
1716
1717
1718
1719
1720
1721
1722
1723
1724
1725
1726
1727
1728
1729
1730
1731
1732
1733
1734
1735
1736
1737
1738
1739
1740
1741
1742
1743
1744
1745
1746
1747
1748
1749
1750
1751
1752
1753
1754
1755
1756
1757
1758
1759
1760
1761
1762
1763
1764
1765
1766
1767
1768
1769
1770
1771
1772
1773
1774
1775
1776
1777
1778
1779
1780
1781
1782
1783
1784
1785
1786
1787
1788
1789
1790
1791
1792
1793
1794
1795
1796
1797
1798
1799
1800
1801
1802
1803
1804
1805
1806
1807
1808
1809
1810
1811
1812
1813
1814
1815
1816
1817
1818
1819
1820
1821
1822
1823
1824
1825
1826
1827
1828
1829
1830
1831
1832
1833
1834
1835
1836
1837
1838
1839
1840
1841
1842
1843
1844
1845
1846
1847
1848
1849
1850
1851
1852
1853
1854
1855
1856
1857
1858
1859
1860
1861
1862
1863
1864
1865
1866
1867
1868
1869
1870
1871
1872
1873
1874
1875
1876
1877
1878
1879
1880
1881
1882
1883
1884
1885
1886
1887
1888
1889
1890
1891
1892
1893
1894
1895
1896
1897
1898
1899
1900
1901
1902
1903
1904
1905
1906
1907
1908
1909
1910
1911
1912
1913
1914
1915
1916
1917
1918
1919
1920
1921
1922
1923
1924
1925
1926
1927
1928
1929
1930
1931
1932
1933
1934
1935
1936
1937
1938
1939
1940
1941
1942
1943
1944
1945
1946
1947
1948
1949
1950
1951
1952
1953
1954
1955
1956
1957
1958
1959
1960
1961
1962
1963
1964
1965
1966
1967
1968
1969
1970
1971
1972
1973
1974
1975
1976
1977
1978
1979
1980
1981
1982
1983
1984
1985
1986
1987
1988
1989
1990
1991
1992
1993
1994
1995
1996
1997
1998
1999
2000
2001
2002
2003
2004
2005
2006
2007
2008
2009
2010
2011
2012
2013
2014
2015
2016
2017
2018
2019
2020
2021
2022
2023
2024
2025
2026
2027
2028
2029
2030
2031
2032
2033
2034
2035
2036
2037
2038
2039
2040
2041
2042
2043
2044
2045
2046
2047
2048
2049
2050
2051
2052
2053
2054
2055
2056
2057
2058
2059
2060
2061
2062
2063
2064
2065
2066
2067
2068
2069
2070
2071
2072
2073
2074
2075
2076
2077
2078
2079
2080
2081
2082
2083
2084
2085
2086
2087
2088
2089
2090
2091
2092
2093
2094
2095
2096
2097
2098
2099
2100
2101
2102
2103
2104
2105
2106
2107
2108
2109
2110
2111
2112
2113
2114
2115
2116
2117
2118
2119
2120
2121
2122
2123
2124
2125
2126
2127
2128
2129
2130
2131
2132
2133
2134
2135
2136
2137
2138
2139
2140
2141
2142
2143
2144
2145
2146
2147
2148
2149
2150
2151
2152
2153
2154
2155
2156
2157
2158
2159
2160
2161
2162
2163
2164
2165
2166
2167
2168
2169
2170
2171
2172
2173
2174
2175
2176
2177
2178
2179
2180
2181
2182
2183
2184
2185
2186
2187
2188
2189
2190
2191
2192
2193
2194
2195
2196
2197
2198
2199
2200
2201
2202
2203
2204
2205
2206
2207
2208
2209
2210
2211
2212
2213
2214
2215
2216
2217
2218
2219
2220
2221
2222
2223
2224
2225
2226
2227
2228
2229
2230
2231
2232
2233
2234
2235
2236
2237
2238
2239
2240
2241
2242
2243
2244
2245
2246
2247
2248
2249
2250
2251
2252
2253
2254
2255
2256
2257
2258
2259
2260
2261
2262
2263
2264
2265
2266
2267
2268
2269
2270
2271
2272
2273
2274
2275
2276
2277
2278
2279
2280
2281
2282
2283
2284
2285
2286
2287
2288
2289
2290
2291
2292
2293
2294
2295
2296
2297
2298
2299
2300
2301
2302
2303
2304
2305
2306
2307
2308
2309
2310
2311
2312
2313
2314
2315
2316
2317
2318
2319
2320
2321
2322
2323
2324
2325
2326
2327
2328
2329
2330
2331
2332
2333
2334
2335
2336
2337
2338
2339
2340
2341
2342
2343
2344
2345
2346
2347
2348
2349
2350
2351
2352
2353
2354
2355
2356
2357
2358
2359
2360
2361
2362
2363
2364
2365
2366
2367
2368
2369
2370
2371
2372
2373
2374
2375
2376
2377
2378
2379
2380
2381
2382
2383
2384
2385
2386
2387
2388
2389
2390
2391
2392
2393
2394
2395
2396
2397
2398
2399
2400
2401
2402
2403
2404
2405
2406
2407
2408
2409
2410
2411
2412
2413
2414
2415
2416
2417
2418
2419
2420
2421
2422
2423
2424
2425
2426
2427
2428
2429
2430
2431
2432
2433
2434
2435
2436
2437
2438
2439
2440
2441
2442
2443
2444
2445
2446
2447
2448
2449
2450
2451
2452
2453
2454
2455
2456
2457
2458
2459
2460
2461
2462
2463
2464
2465
2466
2467
2468
2469
2470
2471
2472
2473
2474
2475
2476
2477
2478
2479
2480
2481
2482
2483
2484
2485
2486
2487
2488
2489
2490
2491
2492
2493
2494
2495
2496
2497
2498
2499
2500
2501
2502
2503
2504
2505
2506
2507
2508
2509
2510
2511
2512
2513
2514
2515
2516
2517
2518
2519
2520
2521
2522
2523
2524
2525
2526
2527
2528
2529
2530
2531
2532
2533
2534
2535
2536
2537
2538
2539
2540
2541
2542
2543
2544
2545
2546
2547
2548
2549
2550
2551
2552
2553
2554
2555
2556
2557
2558
2559
2560
2561
2562
2563
2564
2565
2566
2567
2568
2569
2570
2571
2572
2573
2574
2575
2576
2577
2578
2579
2580
2581
2582
2583
2584
2585
2586
2587
2588
2589
2590
2591
2592
2593
2594
2595
2596
2597
2598
2599
2600
2601
2602
2603
2604
2605
2606
2607
2608
2609
2610
2611
2612
2613
2614
2615
2616
2617
2618
2619
2620
2621
2622
2623
2624
2625
2626
2627
2628
2629
2630
2631
2632
2633
2634
2635
2636
2637
2638
2639
2640
2641
2642
2643
26
```

```

OPTION 1 -- dynamic soil properties - (max is thirteen):
1
5
9
0.0001 #1 Modulus for sand (PI=0) (Vucetic and Dobry, 1991) 0.316
1.0 0.000316 0.001 0.00316 0.01 0.0316 0.1 0.316
0.02 1.0 .960 0.87 0.715 0.49 0.25 0.1
9
0.0001 #1 Damping for sand (PI=0) (Vucetic and Dobry, 1991) 0.316
1.0 0.000316 0.001 0.00316 0.01 0.0316 0.1 0.316
2.0 2.0 3.0 5.5 10.5 16.0 20.0
24.0
10
0.0001 #2 Modulus for silt (PI=50) (Vucetic and Dobry, 1991) 0.316
1.0 0.000316 0.001 0.00316 0.01 0.0316 0.1 0.316
3.16 1.000 1.000 0.99 0.95 0.83 0.67 0.45
0.22 0.02
10
0.0001 #2 Damping for silt (PI=50) (Vucetic and Dobry, 1991) 0.316
1.0 0.000316 0.001 0.00316 0.01 0.0316 0.1 0.316
3.16 2.0 2.0 2.0 3.0 4.1 6.0 9.3
13.2 18.0
9
0.0001 #3 Modulus for CL (PI=15) (Vucetic and Dobry, 1991) 0.316
1.0 0.0003 0.001 0.00316 0.01 0.0316 0.1 0.316
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) 0.316
0.0001 0.0003 0.001 0.00316 0.01 0.0316 0.1 0.316
2.580 2.580 2.580 4.665 7.77 11.67 16.085
20.12
9
#4 Modulus for till (PI=5) (Vucetic and Dobry, 1991) 0.316
0.0001 0.0003 0.001 0.00316 0.01 0.0316 0.1 0.316
1.000 1.000 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) 0.316
0.0001 0.0003 0.001 0.00316 0.01 0.0316 0.1 0.316
2.0 2.000 2.2 2.6 4.6 9.0 14.0 19.5
23.0
8
#5 ATTENUATION OF ROCK AVERAGE
0.0001 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
0.0001 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 FIWA with embankment 0.05 0.124 571
1 1 6.56 0.05 0.124 669
2 1 6.56 0.05 0.124 768
3 1 6.56 0.05 0.102 591
4 3 9.84 0.05 0.102 626
5 3 9.84 0.05 0.102 660
6 3 9.84 0.05 0.102 695
7 3 9.84

```

```

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:
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:**

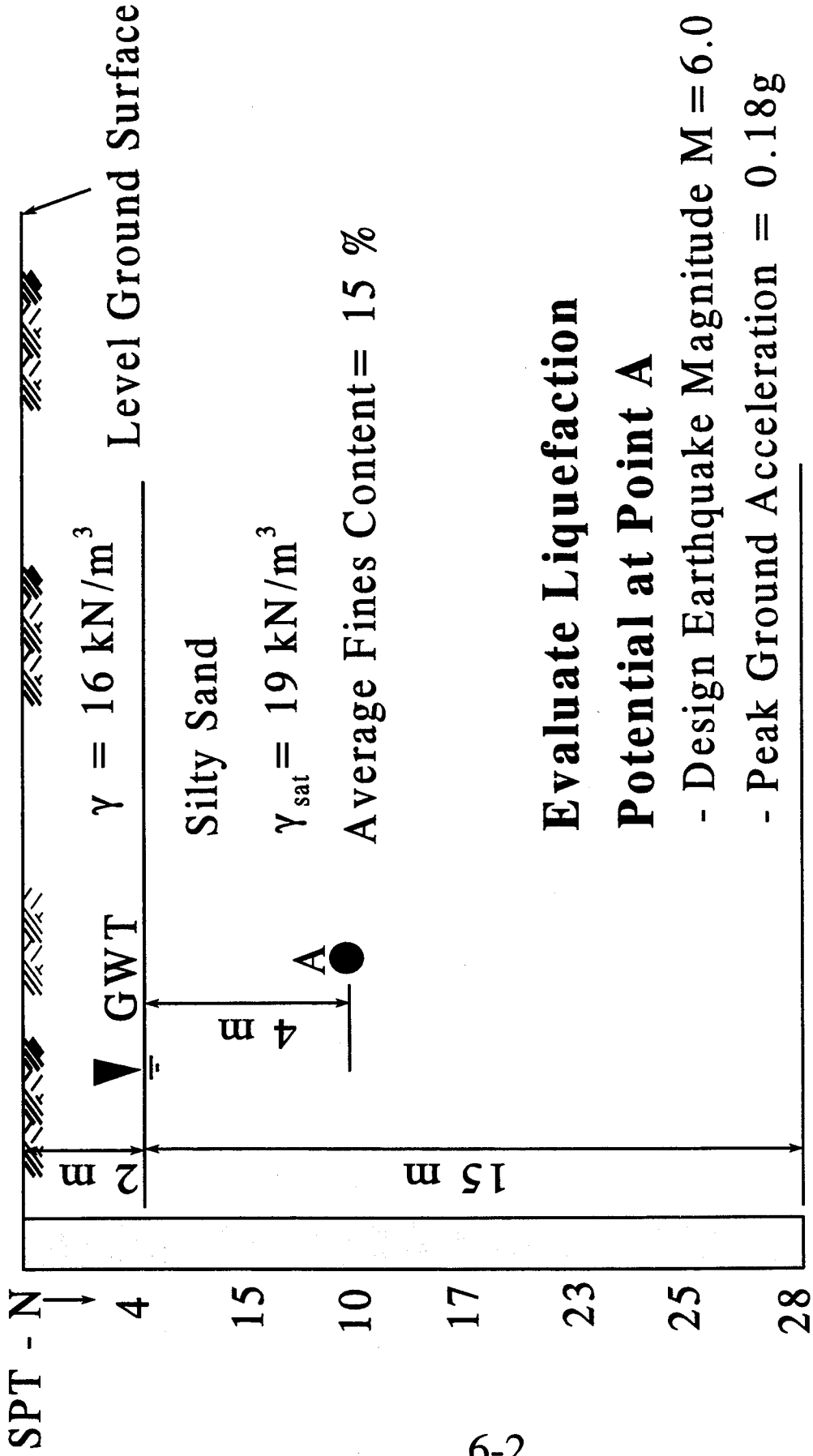
Evaluate the Liquefaction Potential at 6 m Below Ground Surface at a Site Shown in Figure S6-1.

- 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



**Evaluate Liquefaction  
Potential at Point A**

- Design Earthquake Magnitude  $M = 6.0$
- Peak Ground Acceleration =  $0.18g$

End of Borehole

Figure S6-1: Soil Profile

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

Step 2: Compute Effective Overburden Pressure,  $\sigma'_v$

$$\sigma'_v = 69 \text{ kPa} \quad (\text{From Student Exercise No. 2A})$$

Compute Total Overburden Pressure,  $\sigma_v$

$$\sigma_v = \underline{\hspace{4cm}}$$

Evaluate Initial Shear Stress,  $\tau_{ho}$

$$\tau_{ho} = \underline{\hspace{4cm}}$$

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

$$r_d = \underline{\hspace{2cm}}$$

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

$$CSR_{EQ} = 0.65 \left( \frac{a_{\max}}{g} \right) r_d \left( \frac{\sigma_v}{\sigma'_v} \right)$$

$$CSR_{EQ} = \underline{\hspace{2cm}}$$

Step 5: Standardized SPT N Value

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

Step 6: Normalized SPT N Value

$$C_N = 1.18 \quad (\text{From Student Exercise No. 2A})$$

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

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

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

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

$$CSR_{M=7.5} = \underline{\hspace{2cm}}$$

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

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

$$CSR_L = \underline{\hspace{2cm}}$$

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

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

$$FS_L = \underline{\hspace{2cm}}$$

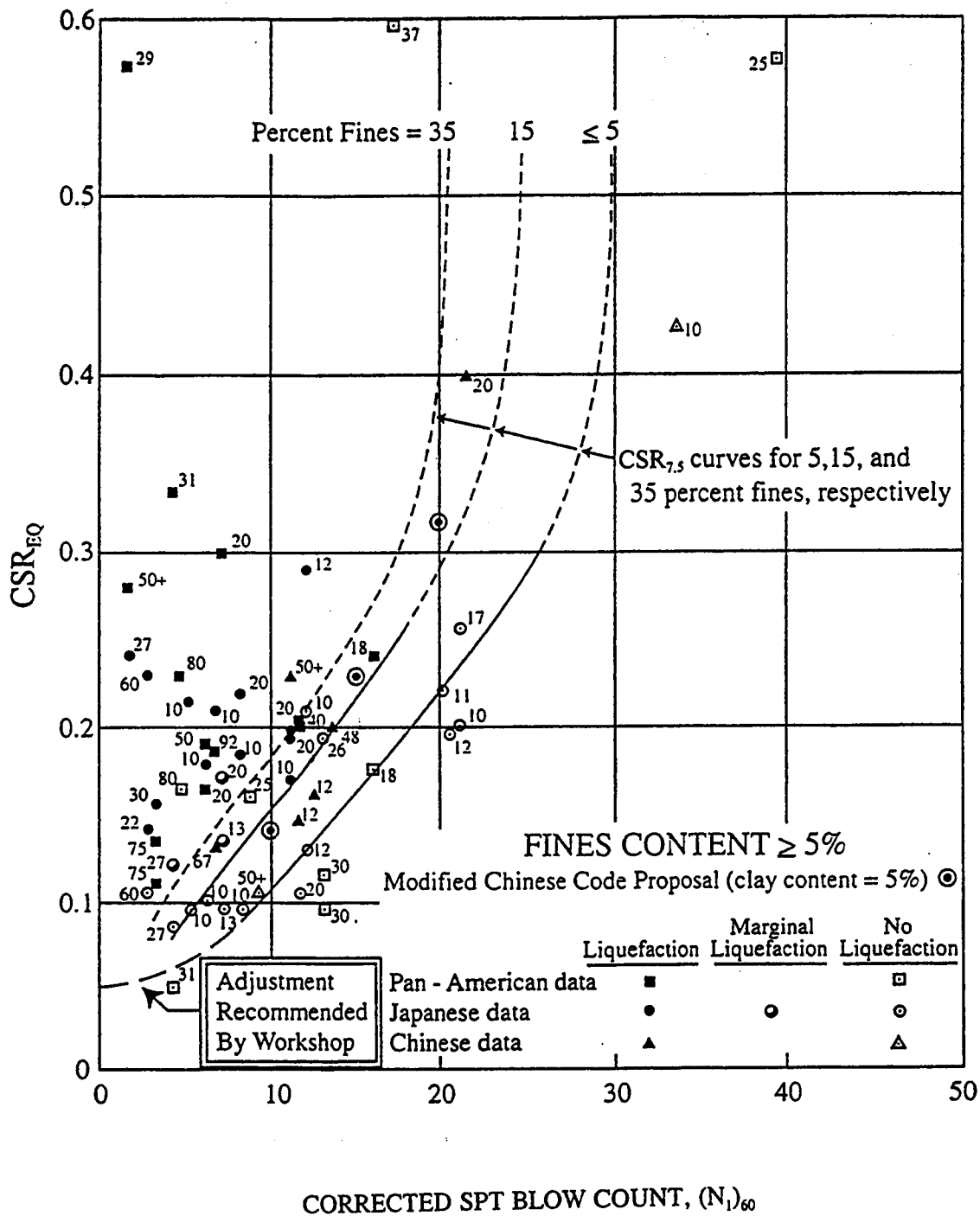


Figure 8-3: Relationship Between Cyclic Stress Ratio Causing Liquefaction and SPT (N<sub>1</sub>)<sub>60</sub> Values for Sands for M = 7.5 Earthquakes (modified From Seed et al., 1985)

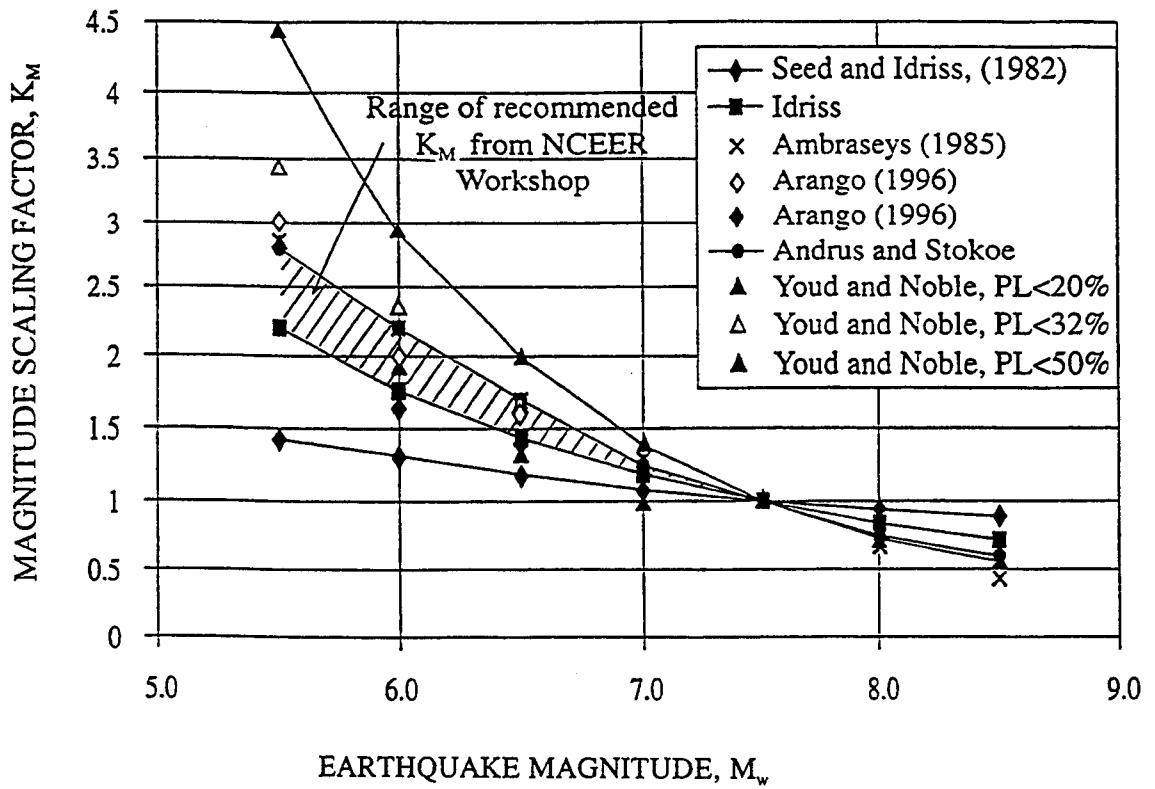


Figure 8-4: Magnitude Scaling Factors Derived by Various Investigators (After Youd and Idriss, 1997)

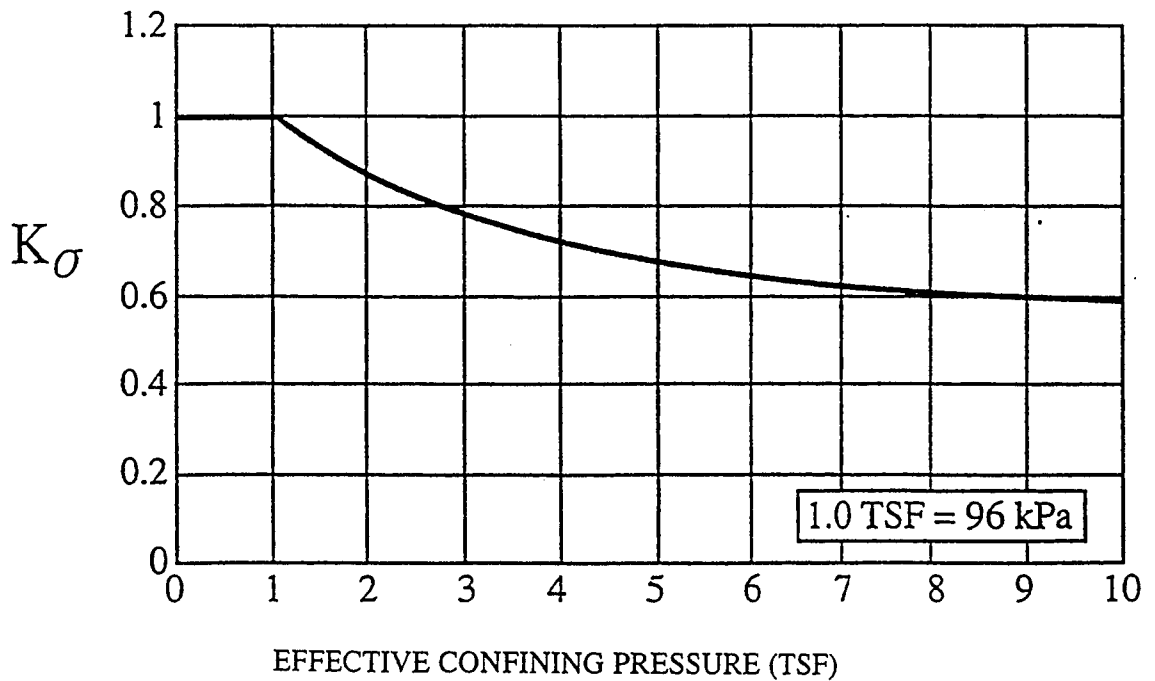


Figure 8-5: Recommended Correction Factor  $k_\sigma$ . (After Youd and Idriss, 1997)



## SOLUTIONS TO STUDENT EXERCISE NO. 6

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

Step 2: Compute Effective Overburden Pressure,  $\sigma'_v$

$$\sigma'_v = 69 \text{ kPa} \quad (\text{From Student Exercise No. 2A})$$

Compute Total Overburden Pressure,  $\sigma_v$

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

$$\sigma_v = 16 \times 2\text{m} + 19 \times 4\text{m}$$

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

Evaluate Initial Shear Stress,  $\tau_{ho}$

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

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

$$r_d = 1 - 0.00765 \cdot z = 1 - 0.00765 \times 6 \\ = \underline{0.95} \quad (\text{For } z = 6 \text{ m})$$

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

$$CSR_{EQ} = 0.65 \left( \frac{a_{\max}}{g} \right) r_d \left( \frac{\sigma_v}{\sigma'_v} \right)$$

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

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

Step 5: Standardized SPT N Value

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

**Step 6: Normalized SPT N Value**

$$C_N = 1.18 \quad (\text{From Student Exercise No. 2A})$$

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

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

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

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

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

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

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

$$k_M = 1.75 \quad (\text{for } M=6.0, \text{ Figure 8-4})$$

$$k_\alpha = 1.0 \quad (\text{for Level Ground})$$

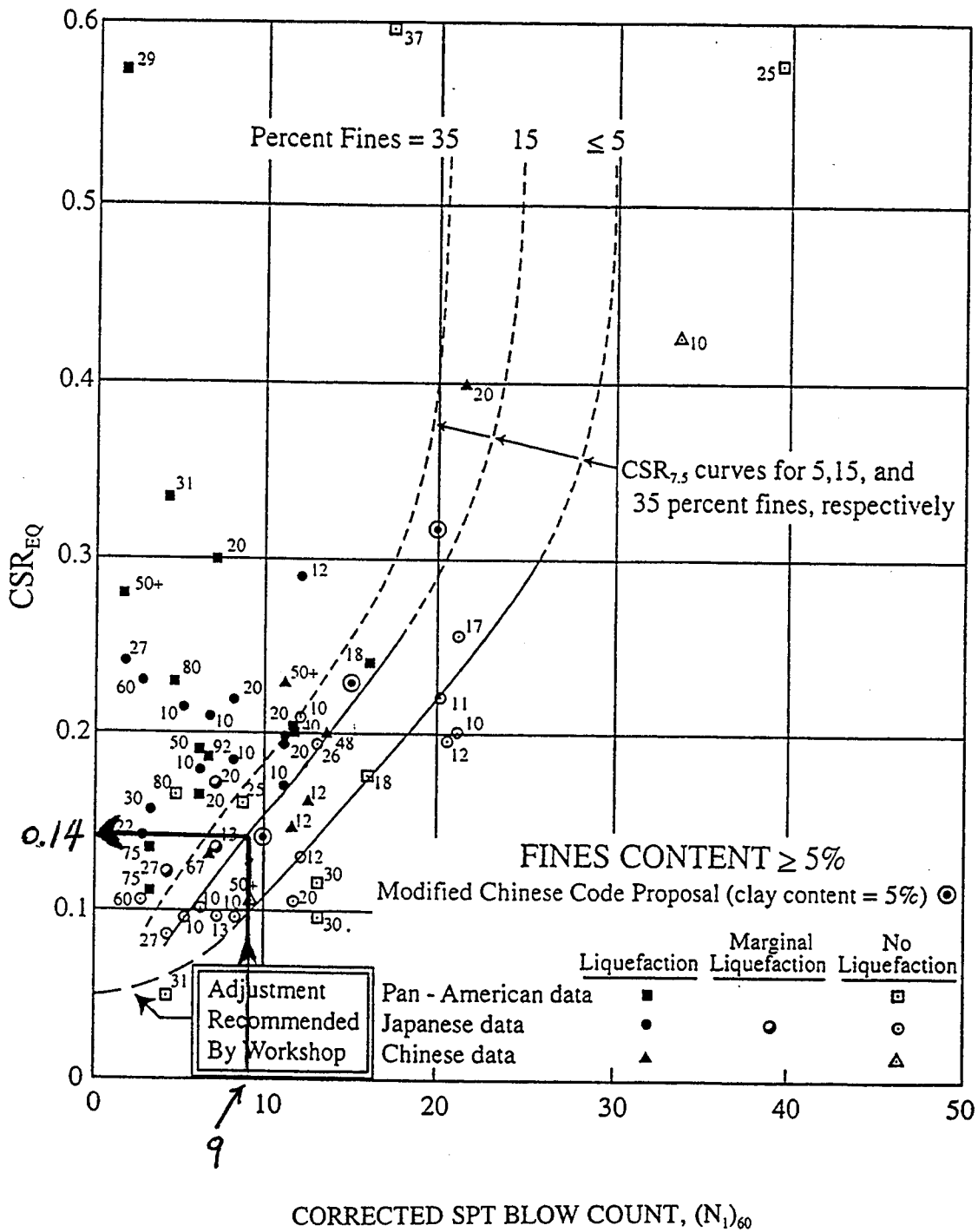


Figure 8-3: Relationship Between Cyclic Stress Ratio Causing Liquefaction and SPT (N<sub>1</sub>)<sub>60</sub> Values for Sands for M = 7.5 Earthquakes (modified From Seed et al., 1985)

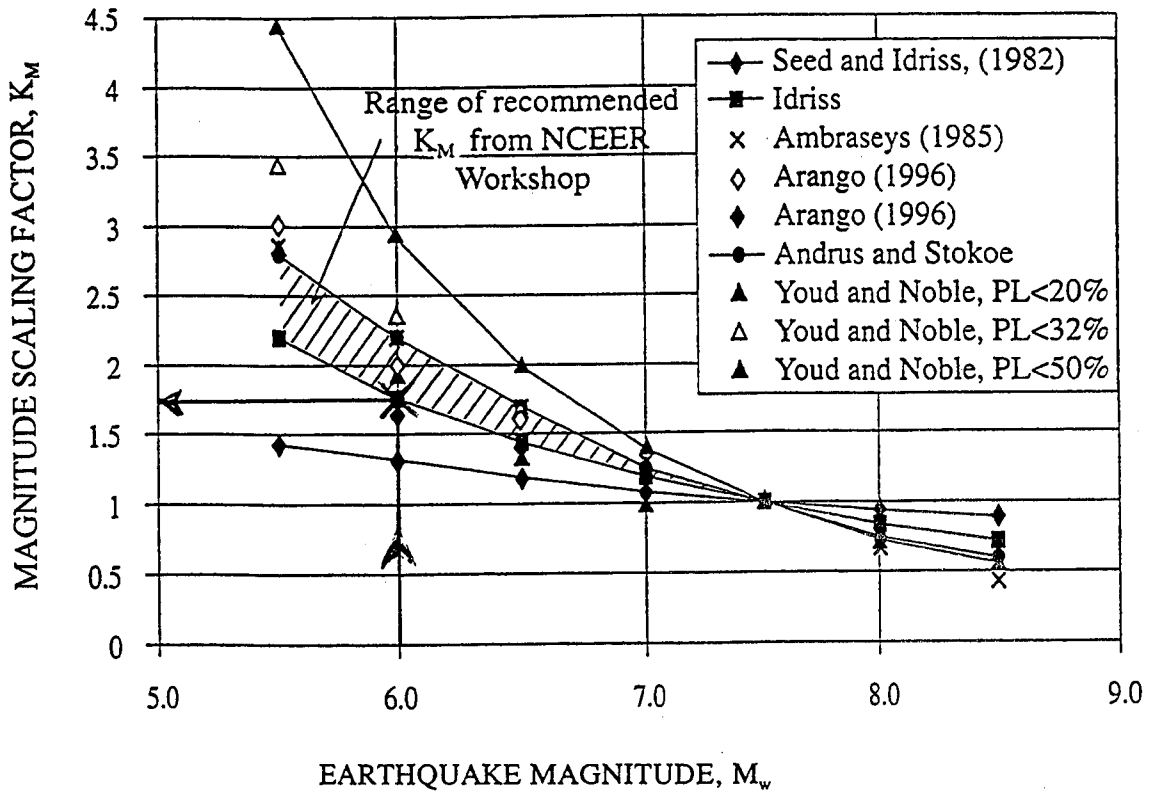


Figure 8-4: Magnitude Scaling Factors Derived by Various Investigators (After Youd and Idriss, 1997)

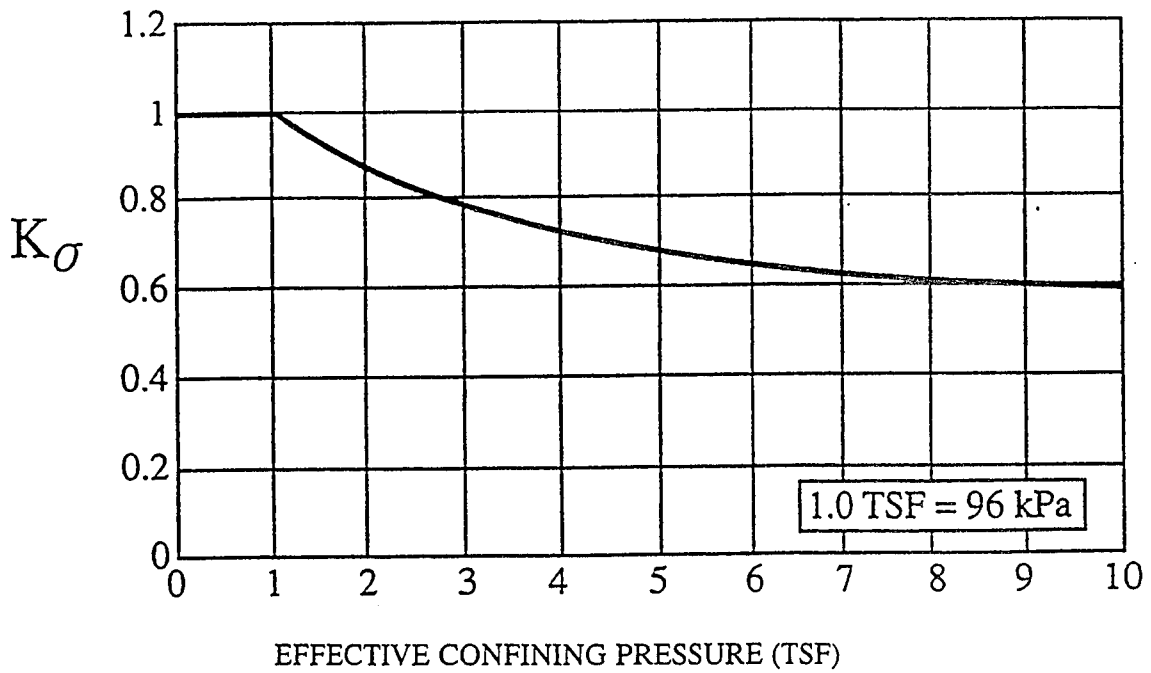


Figure 8-5: Recommended Correction Factor  $k_\sigma$ . (After Youd and Idriss, 1997)

$$k_{\alpha} = 1.0 \text{ (No Correction for } \sigma'_m < 100 \text{ kPa)}$$

(Figure 8-5)

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

$$CSR_L = \underline{0.245}$$

Step 9: Calculate Factor of Safety

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

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

$$FS_L = \underline{1.4}$$

## STUDENT EXERCISE NO. 7

### **Simplified Seismic Deformation Analysis.**

#### **Objective:**

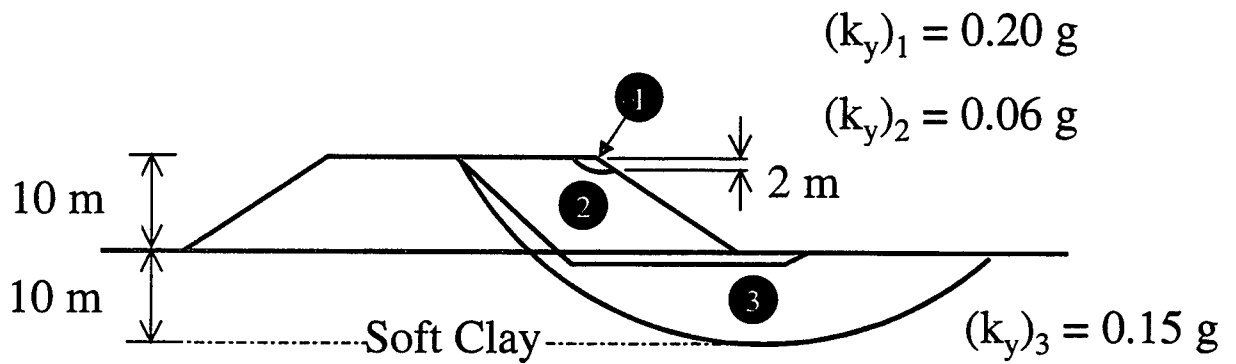
Use Newmark Analysis Design Charts to Estimate Permanent Seismic Deformations for Three Potential Failure Surfaces Shown in Figure S7-1 for a M 6.5 Earthquake with Free-Field, Firm Ground Peak Ground Acceleration = 0.16 g.

#### **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

$$(PGA)_{EMB} = \underline{\hspace{2cm}}$$



**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

$$\left( \frac{k_y}{PAA} \right)_1 = \underline{\hspace{2cm}}$$

$$\left( \frac{k_y}{PAA} \right)_2 = \underline{\hspace{2cm}}$$

$$\left( \frac{k_y}{PAA} \right)_3 = \underline{\hspace{2cm}}$$

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

$$PSD_1 = \underline{\hspace{2cm}}$$

$$PSD_2 = \underline{\hspace{2cm}}$$

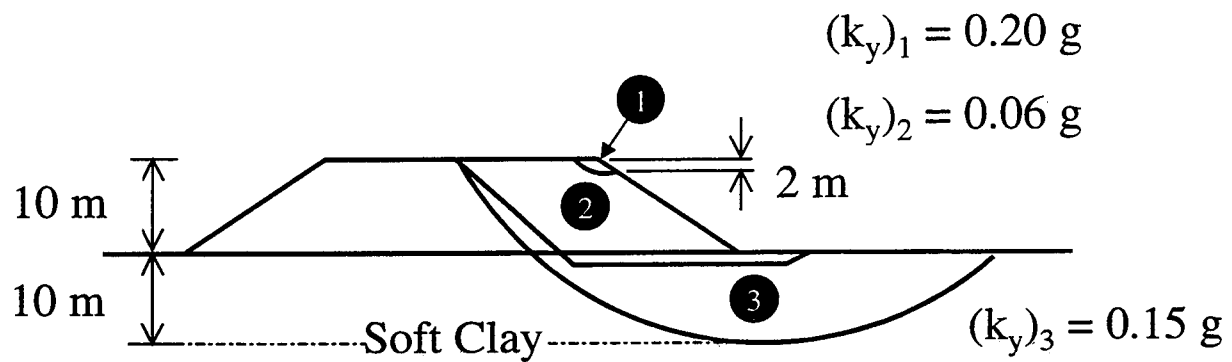
$$PSD_3 = \underline{\hspace{2cm}}$$

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

$PSD_1 = \underline{\hspace{4cm}}$

$PSD_2 = \underline{\hspace{4cm}}$

$PSD_3 = \underline{\hspace{4cm}}$



**Exercise 7 - Figure S7-2**

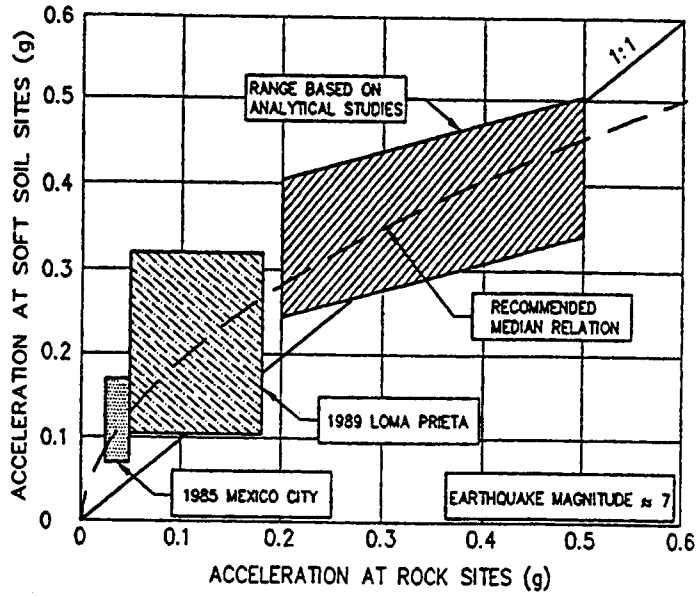


Figure 6-3: Relationship Between PHGA on Rock and on Soft Soil Sites. (Idriss, 1990)

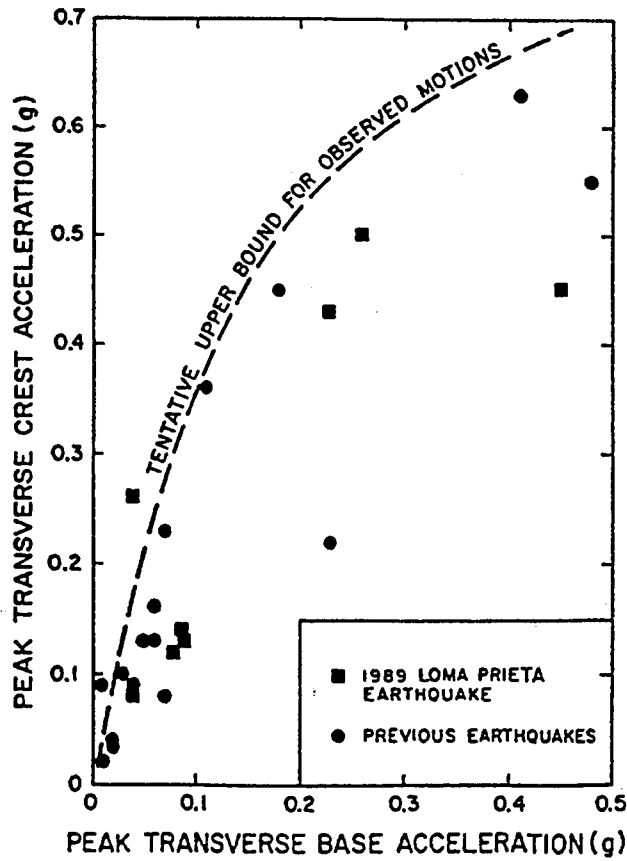


Figure 6-4: Comparisons of Peak Base and Crest Accelerations Recorded at Earth Dams. (Harder, 1991)

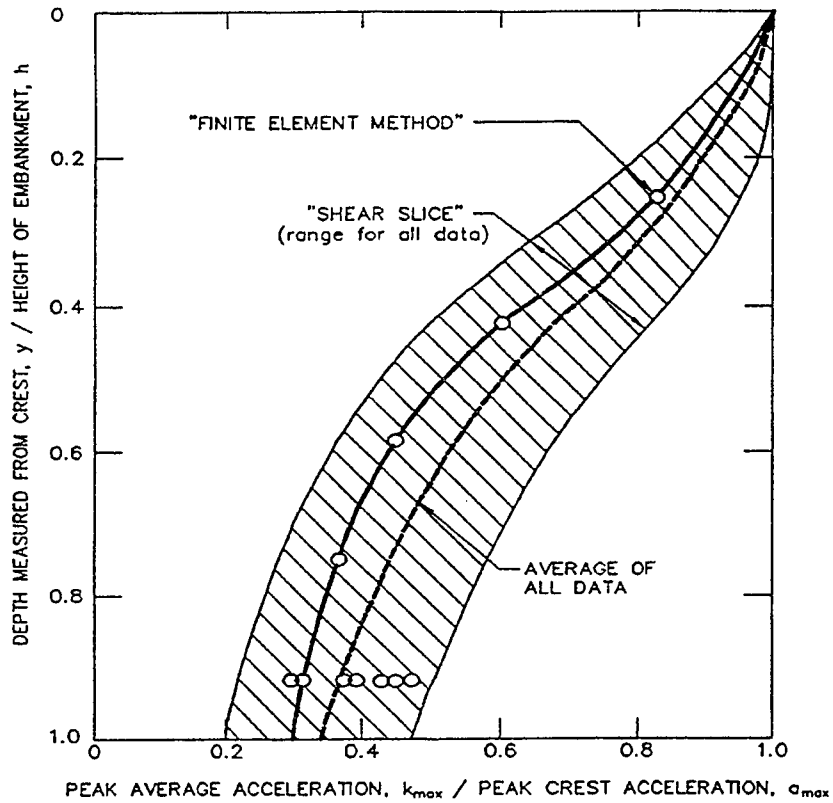


Figure 6-5: Variation of Peak Average Acceleration Ratio with Depth of Sliding Mass. (Makdisi and Seed, 1978, reprinted by permission of ASCE)

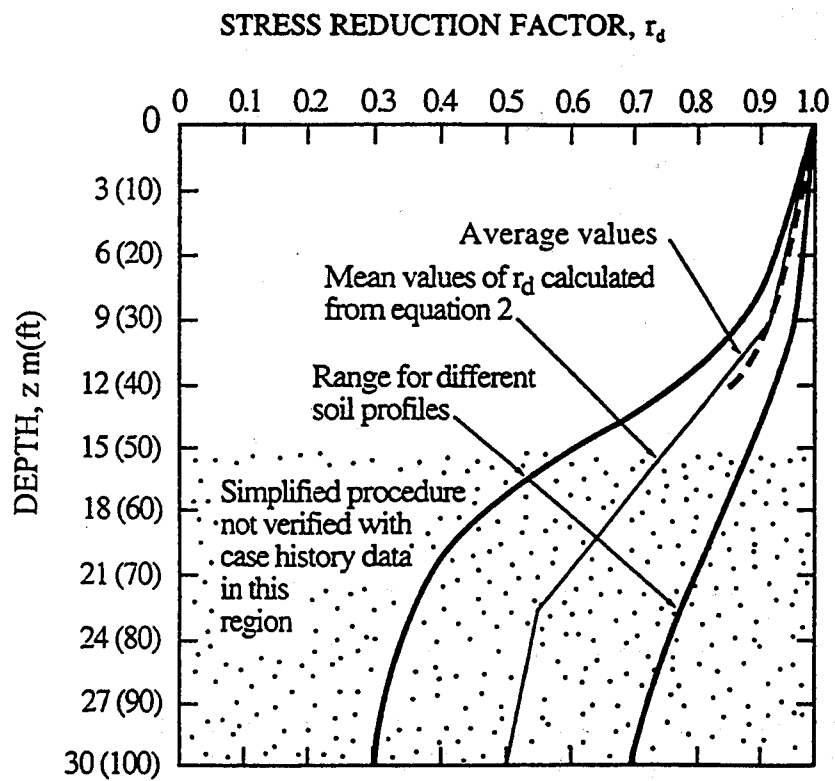


Figure 8-2: Stress Reduction Factor,  $r_d$ , Versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean Value Lines from Equation 8-1.

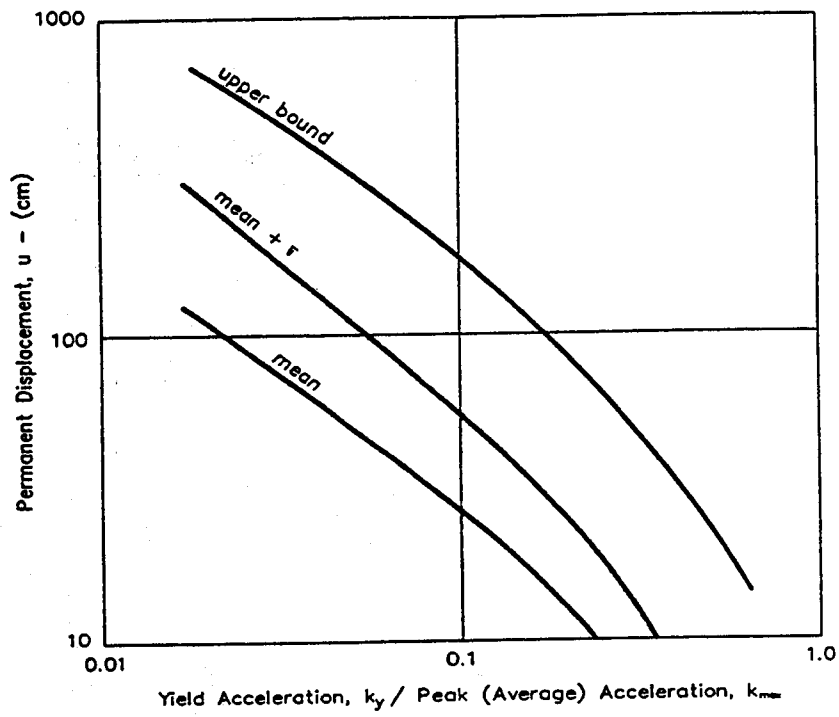


Figure 7-4: Permanent Seismic Deformation Chart. (Hynes and Franklin, 1984, reprinted by permission of U.S. Army Engineer Waterways Experiment Station)

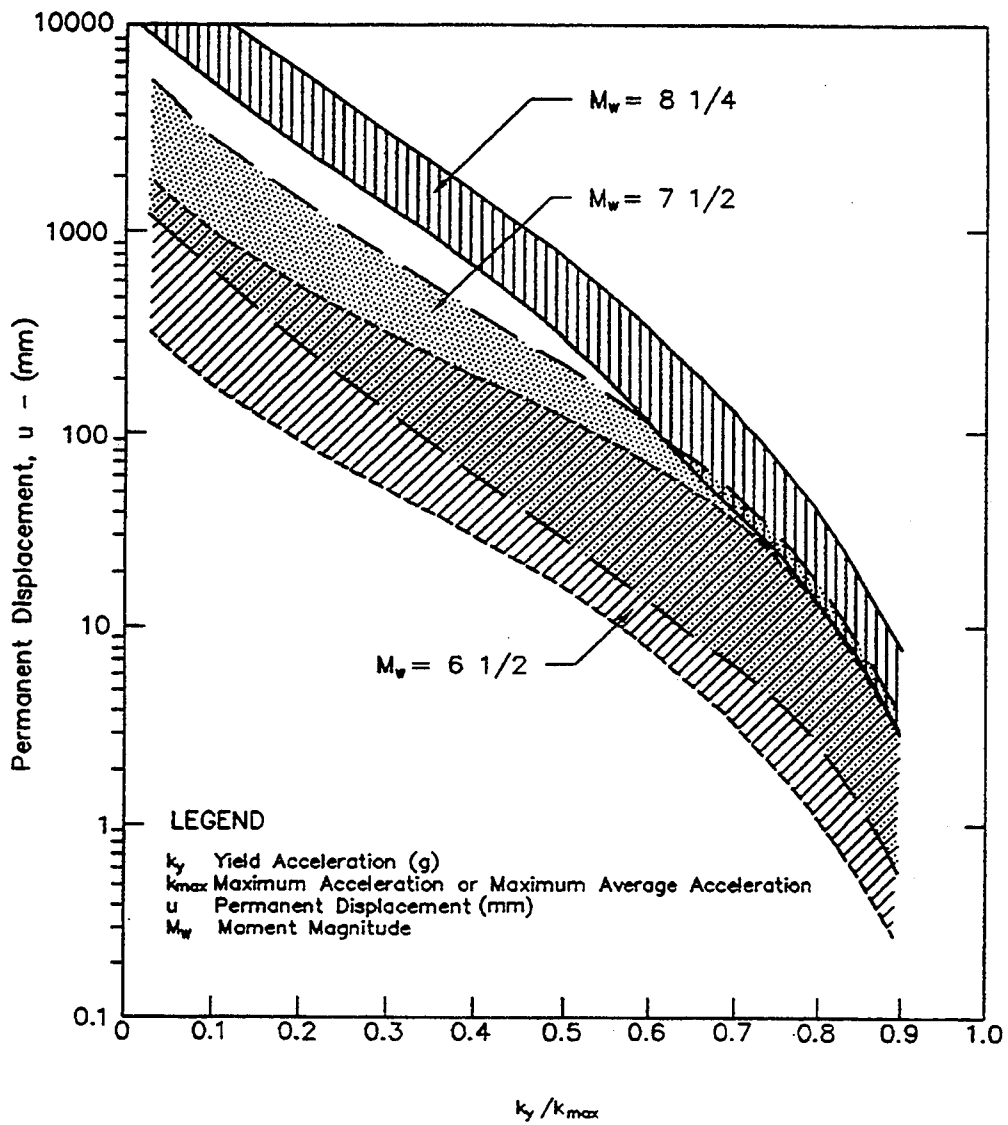


Figure 7-9: Permanent Displacement Versus Normalized Yield Acceleration for Embankments. (After Makdisi and Seed, 1978, reprinted by permission of ASCE).



## 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,  $(PAA)_1 = 0.56 (0.85) = 0.48 \text{ g}$

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

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

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.  $PSD_1 = < 10 \text{ cm}$

$PSD_2 = 10 \text{ cm}$

$PSD_3 = < 10 \text{ cm}$

5.  $\text{PSD}_1 = 10 \text{ cm}$

$\text{PSD}_2 = 20 \text{ cm}$

$\text{PSD}_3 = 15 \text{ cm}$

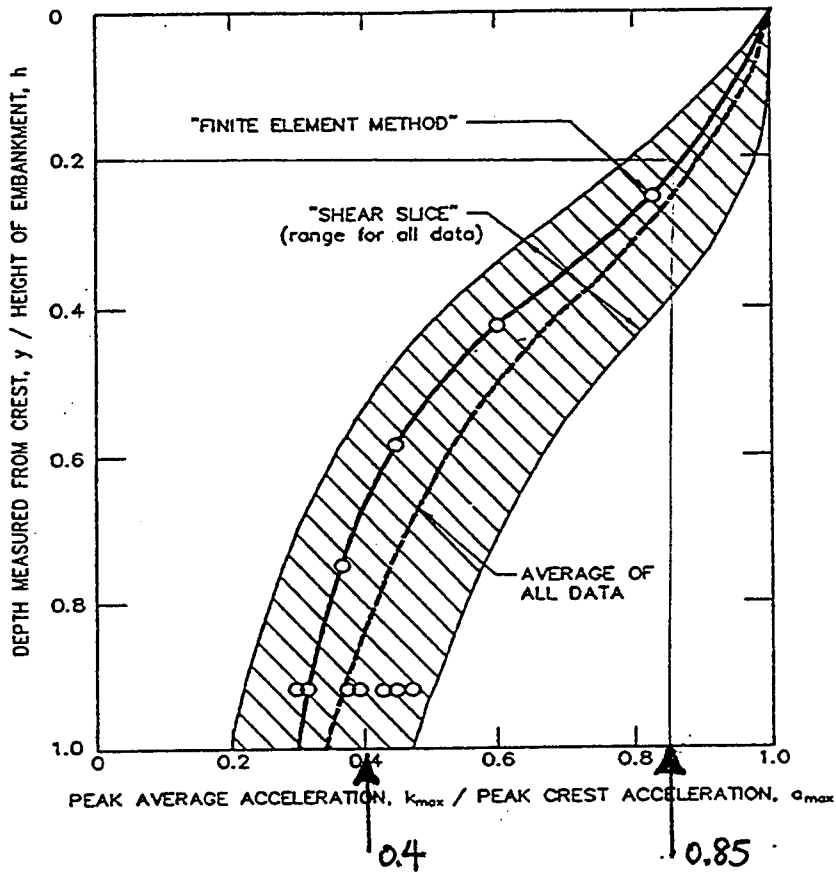


Figure 6-5: Variation of Peak Average Acceleration Ratio with Depth of Sliding Mass. (Makdisi and Seed, 1978, reprinted by permission of ASCE)



## 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

$$R_z = \underline{\hspace{4cm}}$$

$$R_x = \underline{\hspace{4cm}}$$

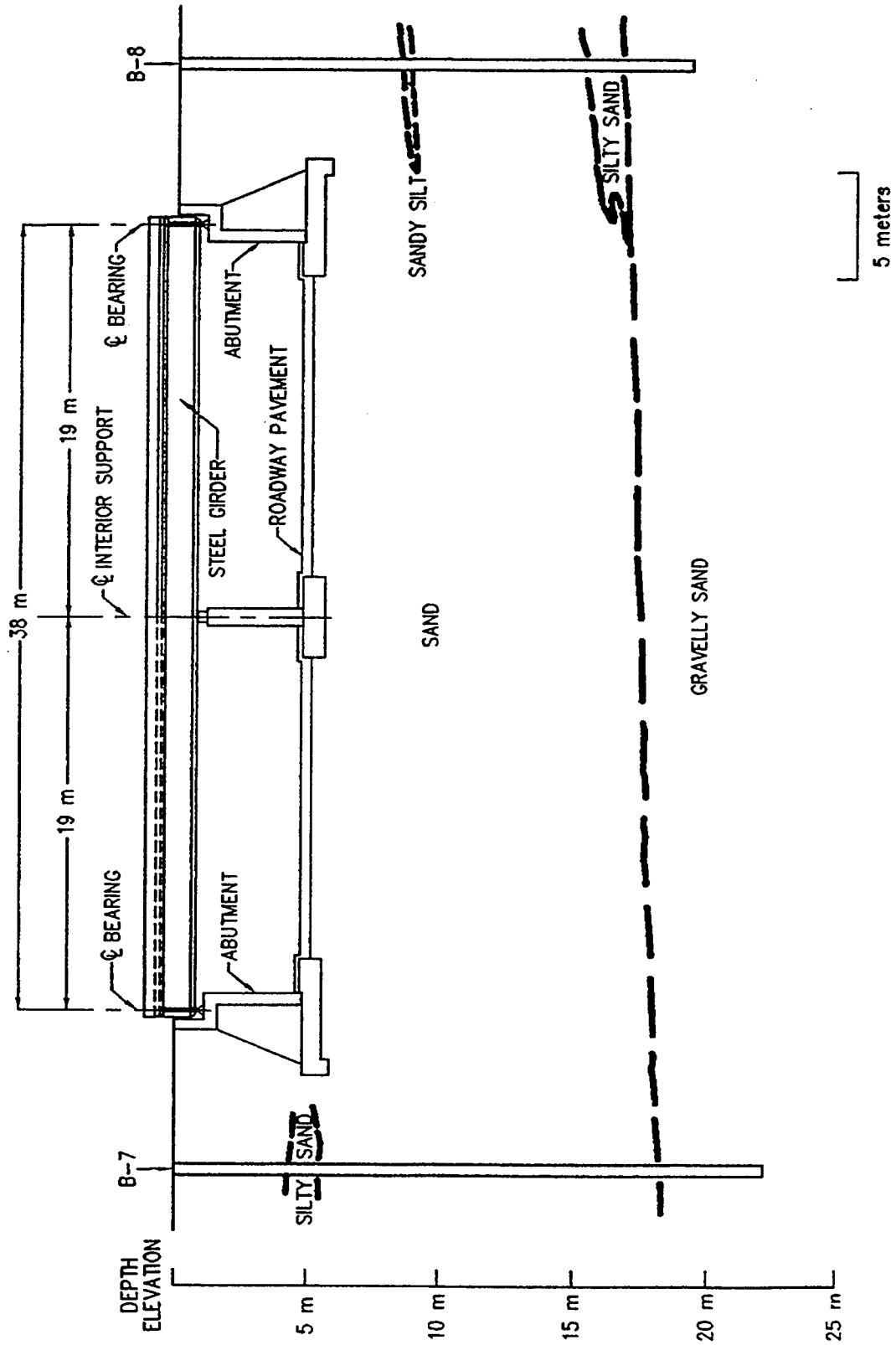


Figure S8-1: Cross Section of Proposed Bridge and Soil Stratigraphy.

# SPT Blow Counts (Blows/300mm)

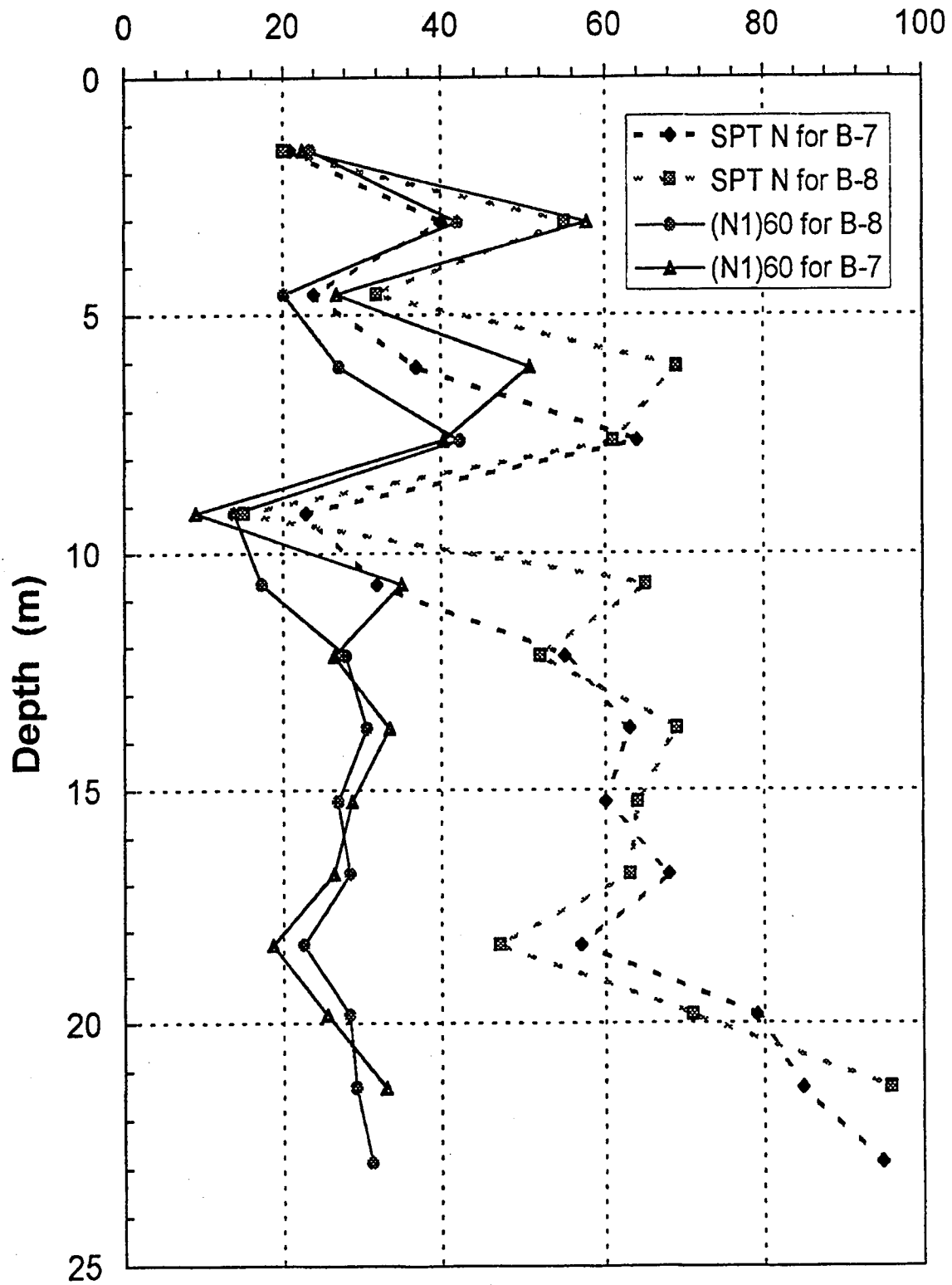


Figure S8-2: Corrected SPT Results for Example No. 8 Test Borings.

## B. Rotational Modes

$$R_{\psi x} = \underline{\hspace{2cm}} \text{ (X-Axis Rocking)}$$

$$R_{\psi y} = \underline{\hspace{2cm}} \text{ (Y-Axis Rocking)}$$

$$R_{\psi z} = \underline{\hspace{2cm}} \text{ Torsion}$$

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

$$N_{60} = \underline{\hspace{2cm}} \text{ (From Figure S8-2)}$$

$$G_{\max} = \underline{\hspace{2cm}} \text{ (From Table 5-5)}$$

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

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

$$\left( \frac{G}{G_{\max}} \right) = \underline{\hspace{2cm}}$$

$$G = \underline{\hspace{2cm}}$$



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

$$v = \underline{\hspace{2cm}} \text{ (Section 5.3.3)}$$

$$k_z = \underline{\hspace{2cm}} \text{ (Equation 9-12a)}$$

$$k_x = k_y = \underline{\hspace{2cm}} \text{ (Equation 9-12b)}$$

$$k_{\psi x} = \underline{\hspace{2cm}} \text{ (Equation 9-12d)}$$

$$k_{\psi y} = \underline{\hspace{2cm}} \text{ (Equation 9-12d)}$$

$$k_{\psi z} = \underline{\hspace{2cm}} \text{ (Equation 9-12c)}$$

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

$$\alpha_z = \underline{\hspace{2cm}} \text{ (Figure 9-7)}$$

$$\alpha_x = \underline{\hspace{2cm}} \text{ (Figure 9-7)}$$

$$\alpha_y = \underline{\hspace{2cm}} \text{ (Figure 9-7)}$$

$$\alpha_{\psi x} = \underline{\hspace{2cm}} \text{ (Figure 9-7)}$$

$$\alpha_{\psi y} = \underline{\hspace{2cm}} \text{ (Figure 9-7)}$$

$$\alpha_{\psi z} = \underline{\hspace{2cm}} \text{ (Figure 9-7)}$$

$$b_z = \underline{\hspace{2cm}} \text{ (Figure 9-8)}$$

$$b_x = \underline{\hspace{2cm}} \text{ (Figure 9-8)}$$

$$b_y = \underline{\hspace{2cm}} \text{ (Figure 9-8)}$$

$$b_{\psi x} = \underline{\hspace{2cm}} \text{ (Figure 9-8)}$$

$$b_{\psi y} = \underline{\hspace{2cm}} \text{ (Figure 9-8)}$$

$$b_{\psi z} = \underline{\hspace{2cm}} \text{ (Figure 9-8)}$$

$$k_z = \alpha_z \beta_z k_z = \underline{\hspace{2cm}}$$

$$k_x = \alpha_z \beta_z k_z = \underline{\hspace{2cm}}$$

$$k_y = \alpha_z \beta_z k_z = \underline{\hspace{2cm}}$$

$$k_{\psi x} = \alpha_{\psi z} \beta_{\psi z} k_{\psi z} = \underline{\hspace{2cm}}$$

$$k_{\psi y} = \alpha_{\psi z} \beta_{\psi z} k_{\psi z} = \underline{\hspace{2cm}}$$

$$k_{\psi z} = \alpha_{\psi z} \beta_{\psi z} k_{\psi z} = \underline{\hspace{2cm}}$$

**TABLE 5-5**  
**CORRELATIONS FOR ESTIMATING INITIAL SHEAR MODULUS**

Reference	Correlation	Units	Limitation
Sced, <i>et al.</i> (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)	$G_{\max} = \frac{625}{(0.3 + 0.7e_0^2)} (P_a \cdot \sigma'_m)^{0.5} \text{OCR}^k$	kPa <sup>(1),(2)</sup>	Limited to cohesive soils $P_a = \text{atmospheric pressure}$
Jamiolkowski, <i>et al.</i> (1991)	$G_{\max} = \frac{625}{e_0^{1.3}} (P_a \cdot \sigma'_m)^{0.5} \text{OCR}^k$	kPa <sup>(1),(2)</sup>	Limited to cohesive soils $P_a = \text{atmospheric pressure}$
Mayne and Rix (1993)	$G_{\max} = 99.5(P_a)^{0.305}(q_c)^{0.695}/(e_0)^{1.13}$	kPa <sup>(2)</sup>	Limited to cohesive soils $P_a = \text{atmospheric pressure}$

Notes: <sup>(1)</sup>  $P_a$  and  $\sigma'_m$  in kPa

<sup>(2)</sup>  $P_a$  and  $q_c$  in kPa

<sup>(3)</sup> 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	$B_z = \frac{(1-\nu)}{4} \frac{m}{\rho r_o^3}$	$c_z = \frac{3.4 r_o^2}{1-\nu} \sqrt{\rho G}$	$D_z = \frac{0.425}{\sqrt{B_z}}$	$r_o = R_z = \sqrt{BL/\pi}$
Horizontal Translation (Sliding)	$B_x = \frac{(7-8\nu)}{32(1-\nu)} \frac{m}{\rho r_o^3}$	$c_x = \frac{4.6 r_o^2}{2-\nu} \sqrt{\rho G}$	$D_x = \frac{0.288}{\sqrt{B_x}}$	$r_o = R_x = \sqrt{BL/\pi}$
X- and Y-axis Rocking	$B_\psi = \frac{3(1-\nu)}{8} \frac{I_\psi}{\rho r_o^5}$	$c_\psi = \frac{0.8 r_o^4 \sqrt{\rho G}}{(1-\nu)(1+B_\psi)}$	$D_\psi = \frac{0.15}{(1+B_\psi) \sqrt{B_\psi}}$	$r_o = R_{\psi_x} = \left[ \frac{16(B)(L)^3}{3\pi} \right]^{1/4}$
				$r_o = R_{\psi_y} = \left[ \frac{16(B)^3(L)}{3\pi} \right]^{1/4}$
Z-axis Rotation (Torsion)	$B_\theta = \frac{I_\theta}{\rho r_o^5}$	$c_\theta = \frac{4 \sqrt{B_\theta \cdot \rho G}}{1+2B_\theta}$	$D_\theta = \frac{0.5}{1+2B_\theta}$	$r_o = R_{\psi_z} = \left[ \frac{16BL(B^2 + L^2)}{6\pi} \right]^{1/4}$

Notes:

- m = mass of the foundation
- c = damping coefficient ( $c_x, c_y, c_\theta$ )
- I = moment of inertia of the foundation
- $\rho$  = mass density of foundation soil
- $r_o$  = equivalent radius ( $R_x, R_y, R_\theta$ )
- 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
- $\nu$  = Poisson's ratio of the soil
- D = damping ratio ( $D_x, D_y, D_\theta$ )

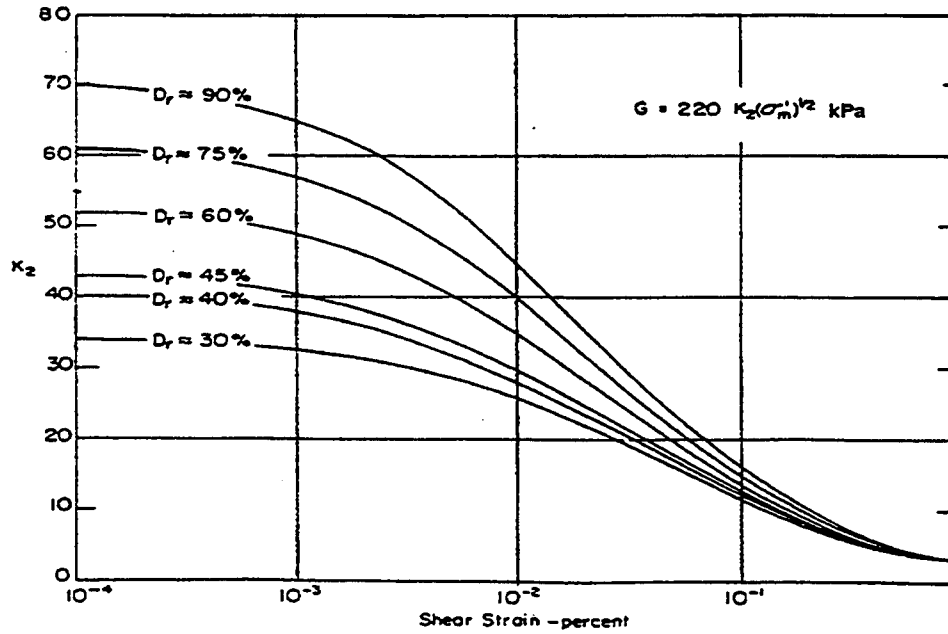
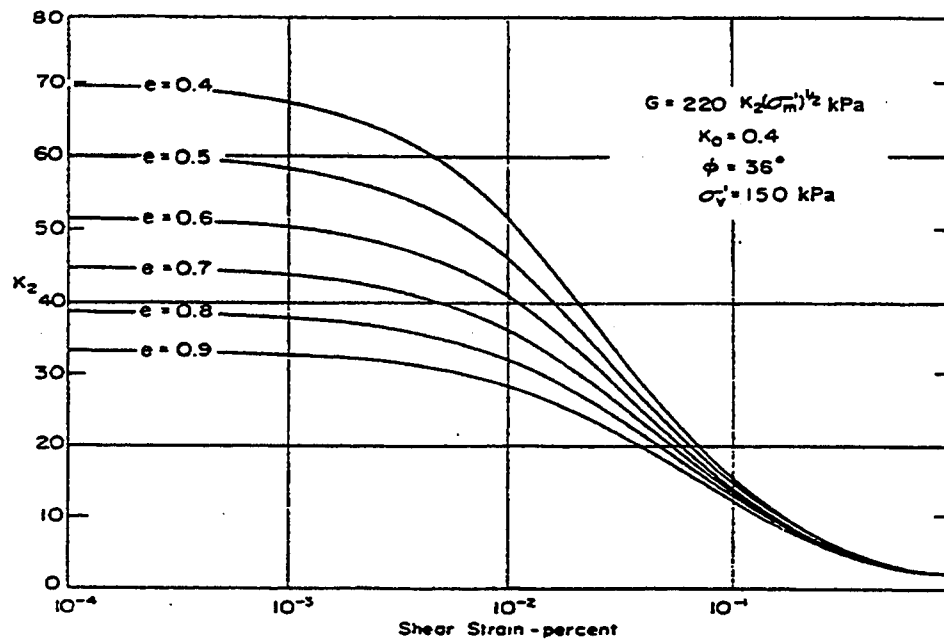


Figure 5-12: Shear Modulus Reduction Curves for Sands. (Seed and Idriss, 1970, reprinted by permission of ASCE)

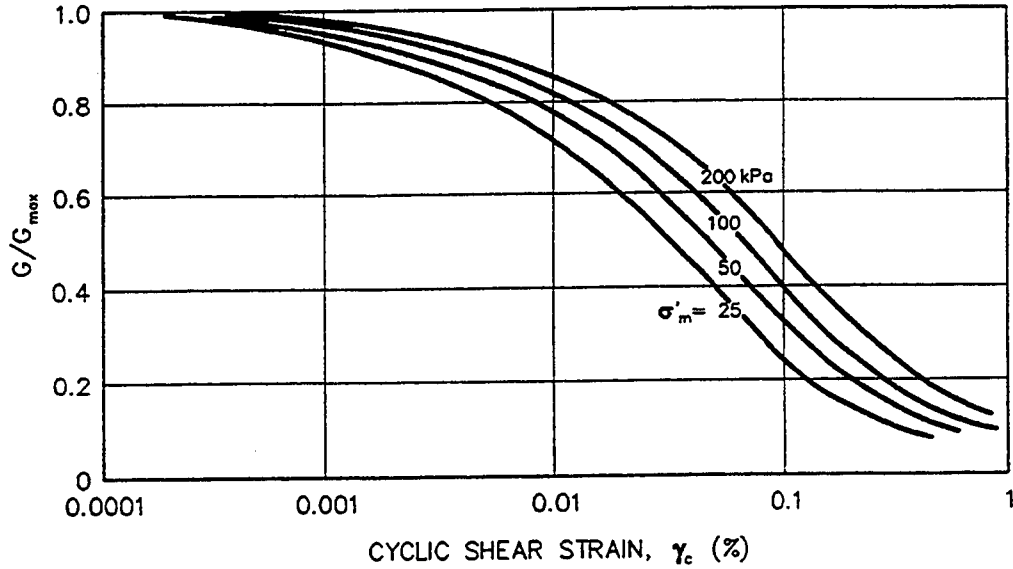


Figure 5-13: Shear Modulus Reduction Curves for Sands. (Iwasaki, *et al.*, 1978, reprinted by permission of Japanese Society of Soil Mechanics and Foundation Engineering)

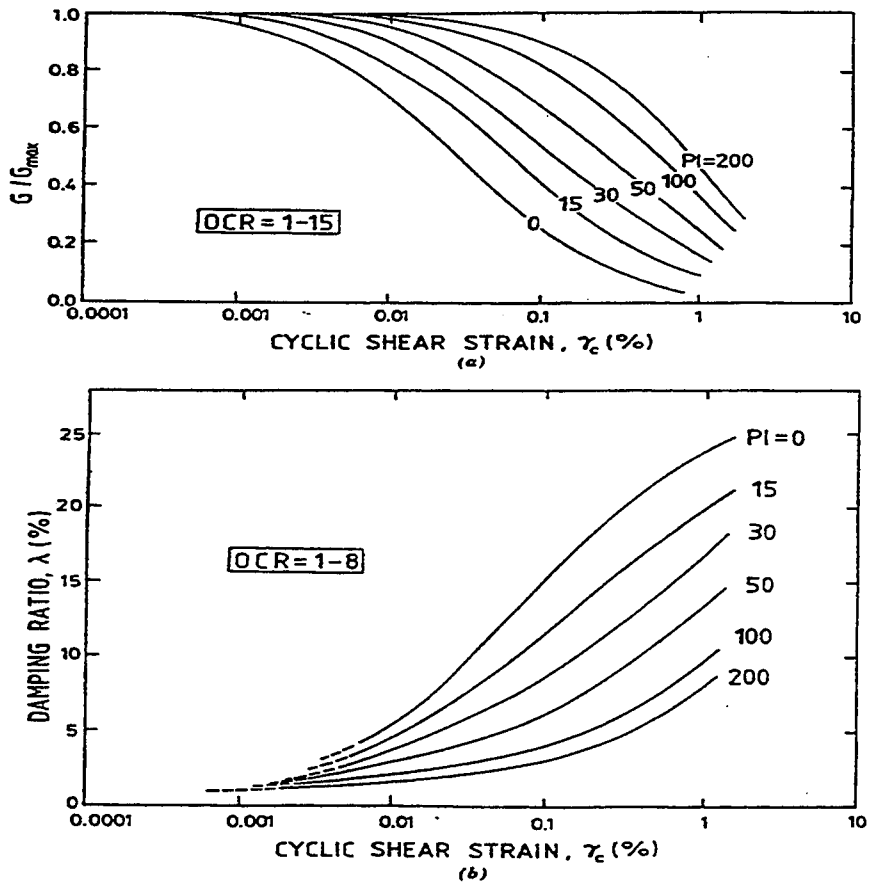


Figure 5-14: Shear Modulus Reduction and Damping Ratio as a Function of Shear Strain and Soil Plasticity Index. (Vucetic and Dobry, 1991, reprinted by permission of ASCE)

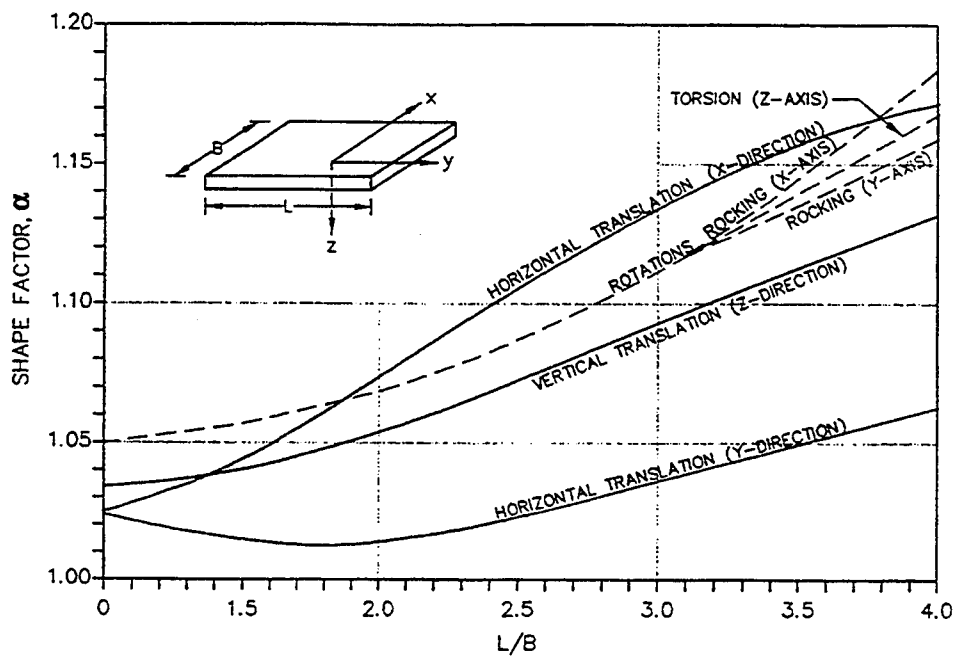


Figure 9-7: Shape Factor  $\alpha$  for Rectangular Footings. (Lam and Martin, 1986)



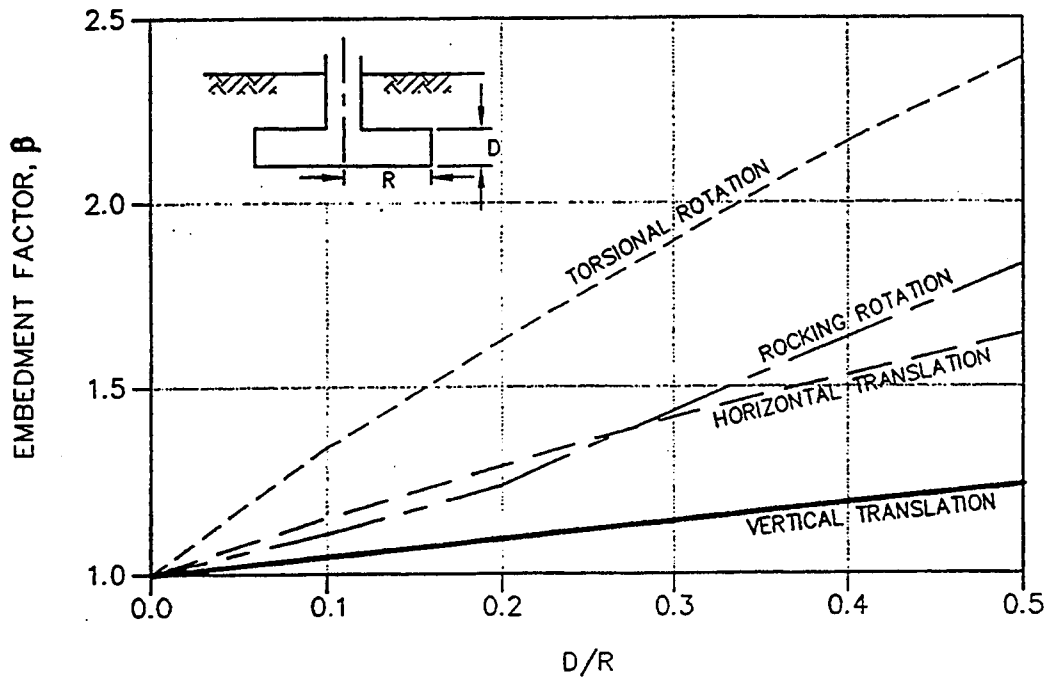


Figure 9-8: Embedment Factors for Footings with  $D/R < 0.5$ . (Lam and Martin, 1986)

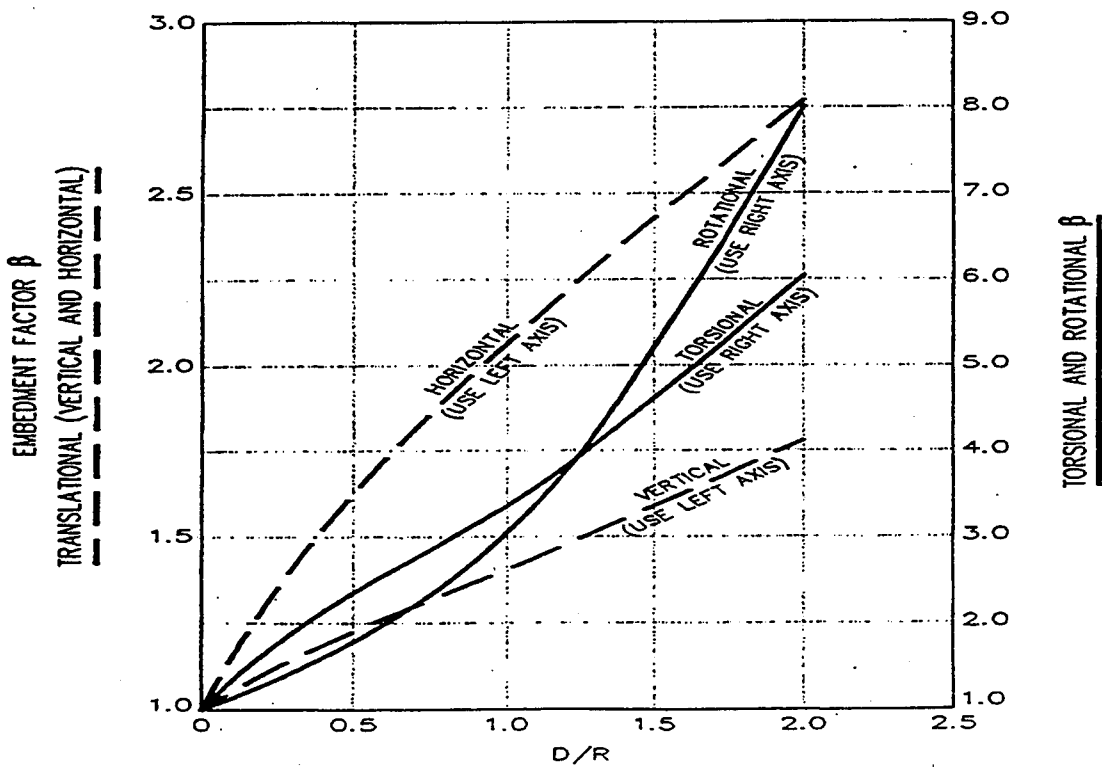


Figure 9-9: Embedment Factors for Footings with  $D/R > 0.5$ . (Lam and Martin, 1986)

## 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

$$(N_{60}) = 28$$

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

### 3. Modulus Reduction

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

$$G = 90 \text{ MPa}$$

4. Stiffness Coefficients for Equivalent Circle

<b>MODE</b>	<b>Equivalent Circular Stiffness Mpa - m</b>	$\alpha$	$\beta$	<b>Stiffness Mpa - m</b>
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

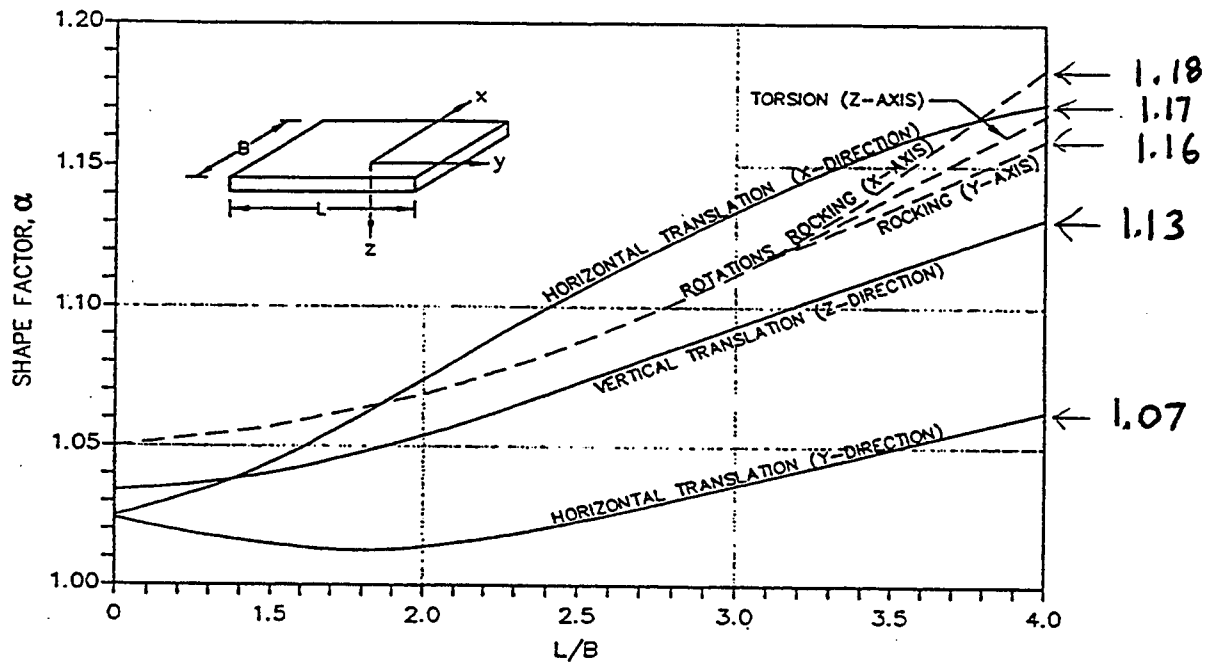


Figure 9-7: Shape Factor  $\alpha$  for Rectangular Footings. (Lam and Martin, 1986)

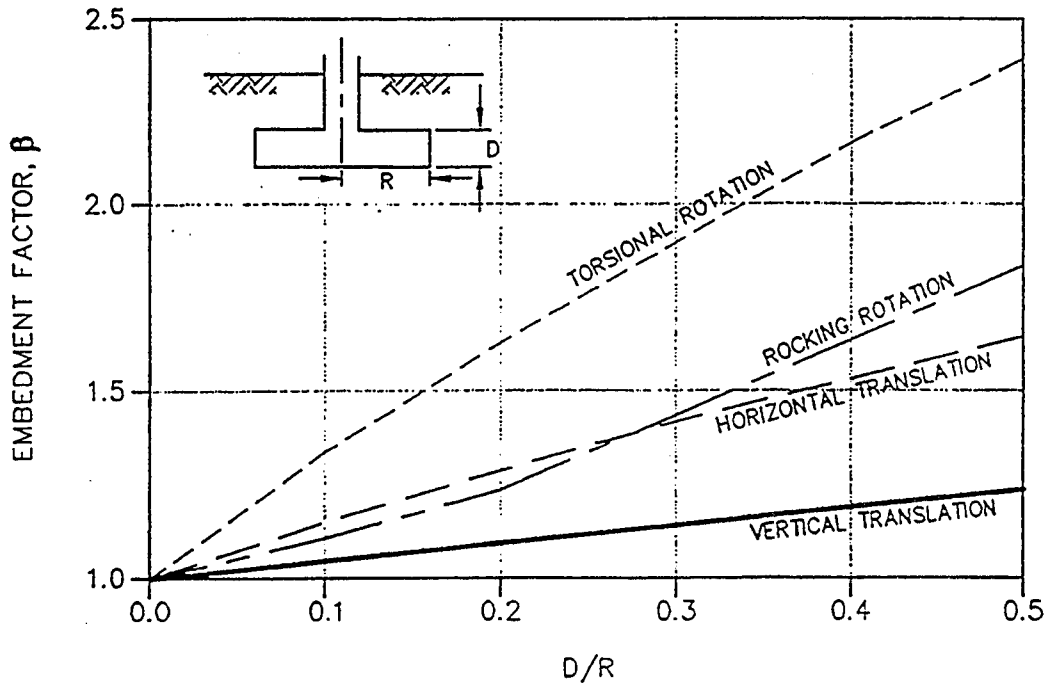


Figure 9-8: Embedment Factors for Footings with  $D/R < 0.5$ . (Lam and Martin, 1986)

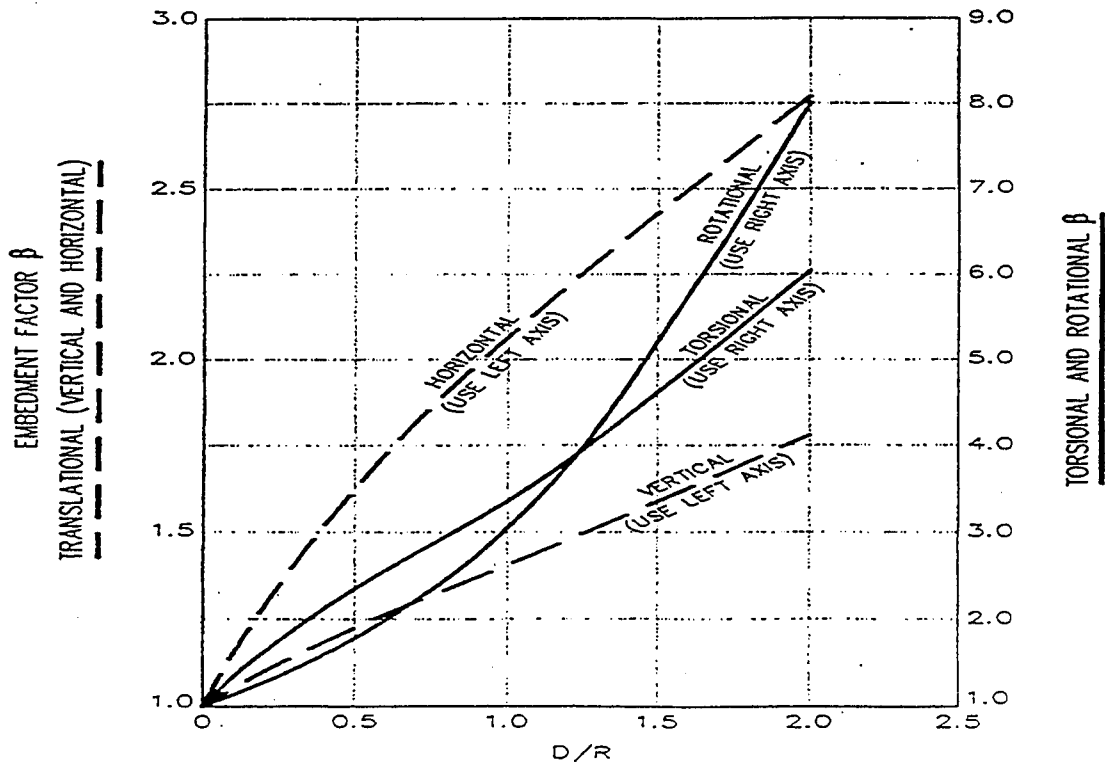


Figure 9-9: Embedment Factors for Footings with  $D/R > 0.5$ . (Lam and Martin, 1986)



## **STUDENT EXERCISE NO. 9A**

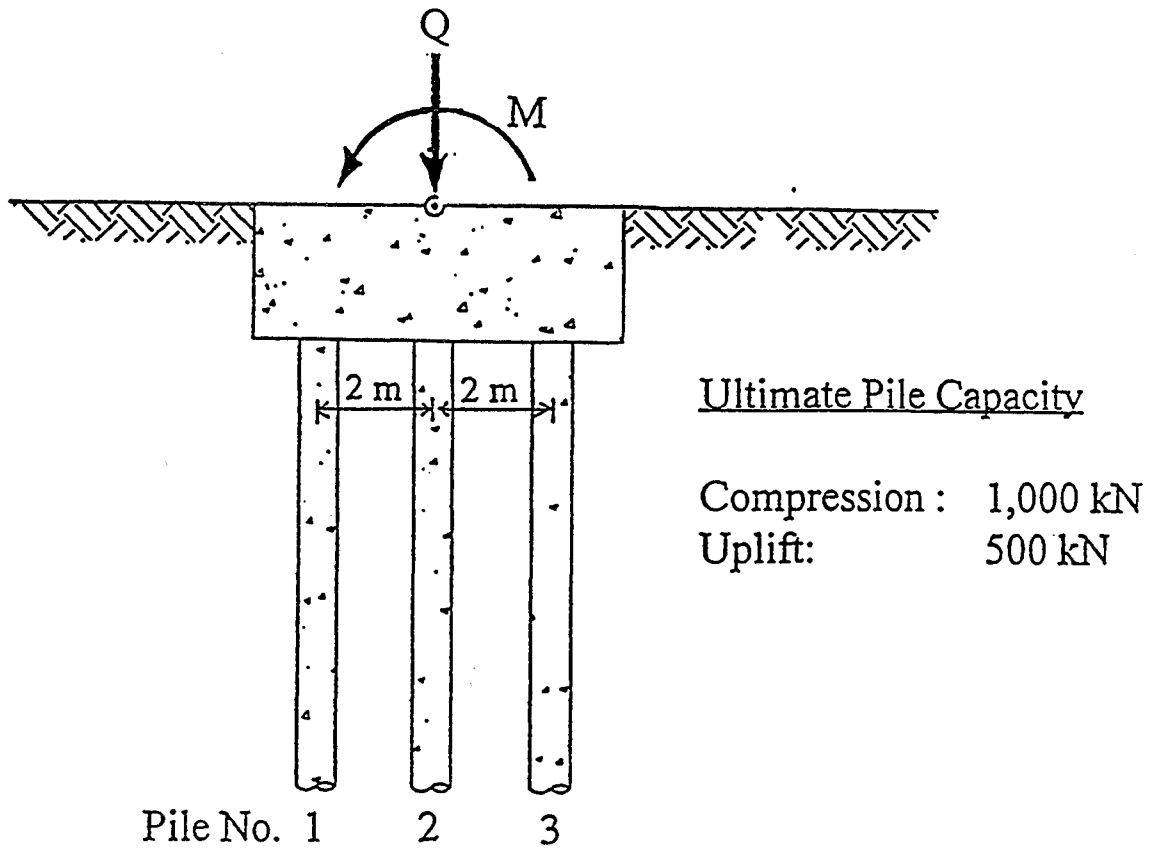
### **Geotechnical Pile Capacity Evaluation Under Seismic Loading.**

#### **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

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

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

(Compression)

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

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

## Seismic Load Case 1

$Q = 1,500 \text{ kN}$ ,  $M = 3,000 \text{ m-kN}$

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

## Seismic Load Case 2

$Q = 300 \text{ kN}$ ,  $M = 3,000 \text{ m-kN}$

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

## SOLUTIONS TO EXERCISE NO. 9A

### Seismic Load Case 1

$$Q=1,500 \text{ kN}, \quad M=3,000 \text{ m-kN}$$

- 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**

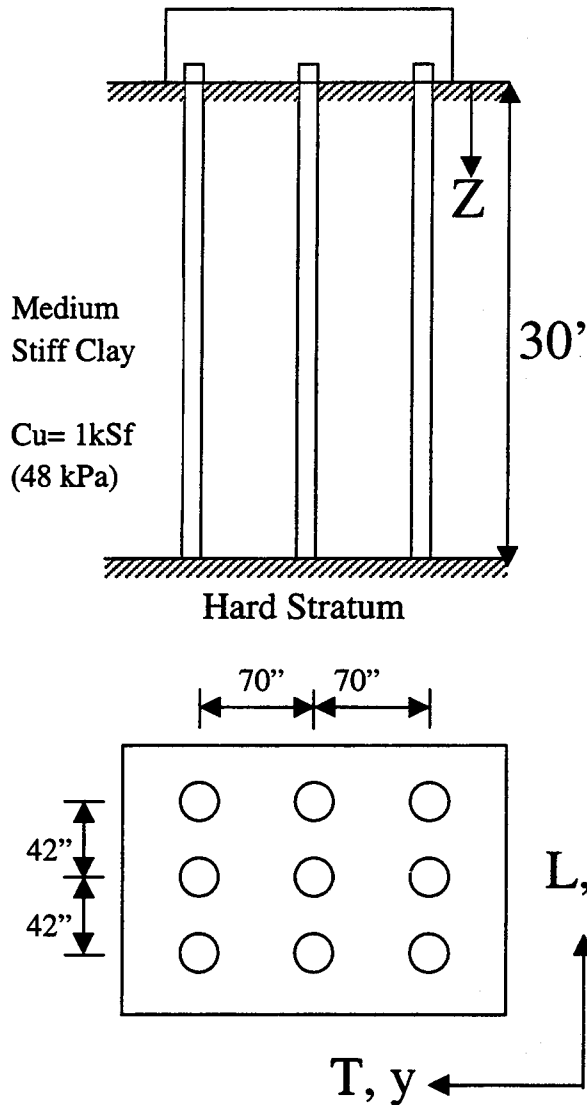
### **Derivation of Foundation Stiffness for Pile Group**

#### **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)

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

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:

$$f = \underline{\hspace{2cm}}$$

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.)

$(p\text{-multiplier})_{\text{ave}}$  in Longitudinal Direction =  $\underline{\hspace{2cm}}$

$(p\text{-multiplier})_{\text{ave}}$  in Transverse Direction =  $\underline{\hspace{2cm}}$

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

Longitudinal:  $f_{\text{eff}, L} = \underline{\hspace{4cm}}$

Transverse:  $f_{\text{eff}, T} = \underline{\hspace{4cm}}$

- D. Calculate the Bending Stiffness of the Pile

$EI = \underline{\hspace{4cm}}$

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

$k'_{\delta, L} = \underline{\hspace{4cm}}$  (Longitudinal)

$k'_{\delta, T} = \underline{\hspace{4cm}}$  (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.

$$k_v = \frac{AE}{L} = \underline{\hspace{2cm}}$$

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

A. Lateral Stiffness

$$K_L = n \cdot k'_{\delta,L} = \underline{\hspace{2cm}}$$

$$K_T = n \cdot k'_{\delta,T} = \underline{\hspace{2cm}}$$

Note: n is the total number of piles.

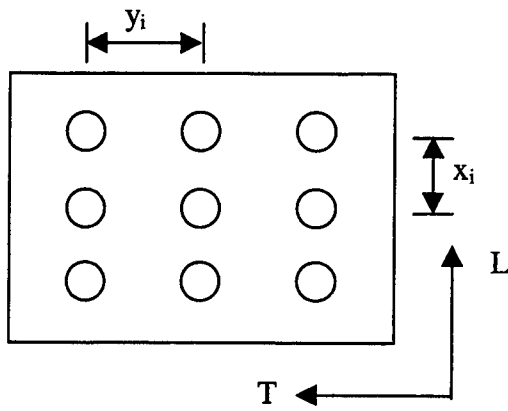
B. Axial Stiffness

$$K_v = n \cdot k_v = \underline{\hspace{2cm}}$$

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.

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

= \_\_\_\_\_

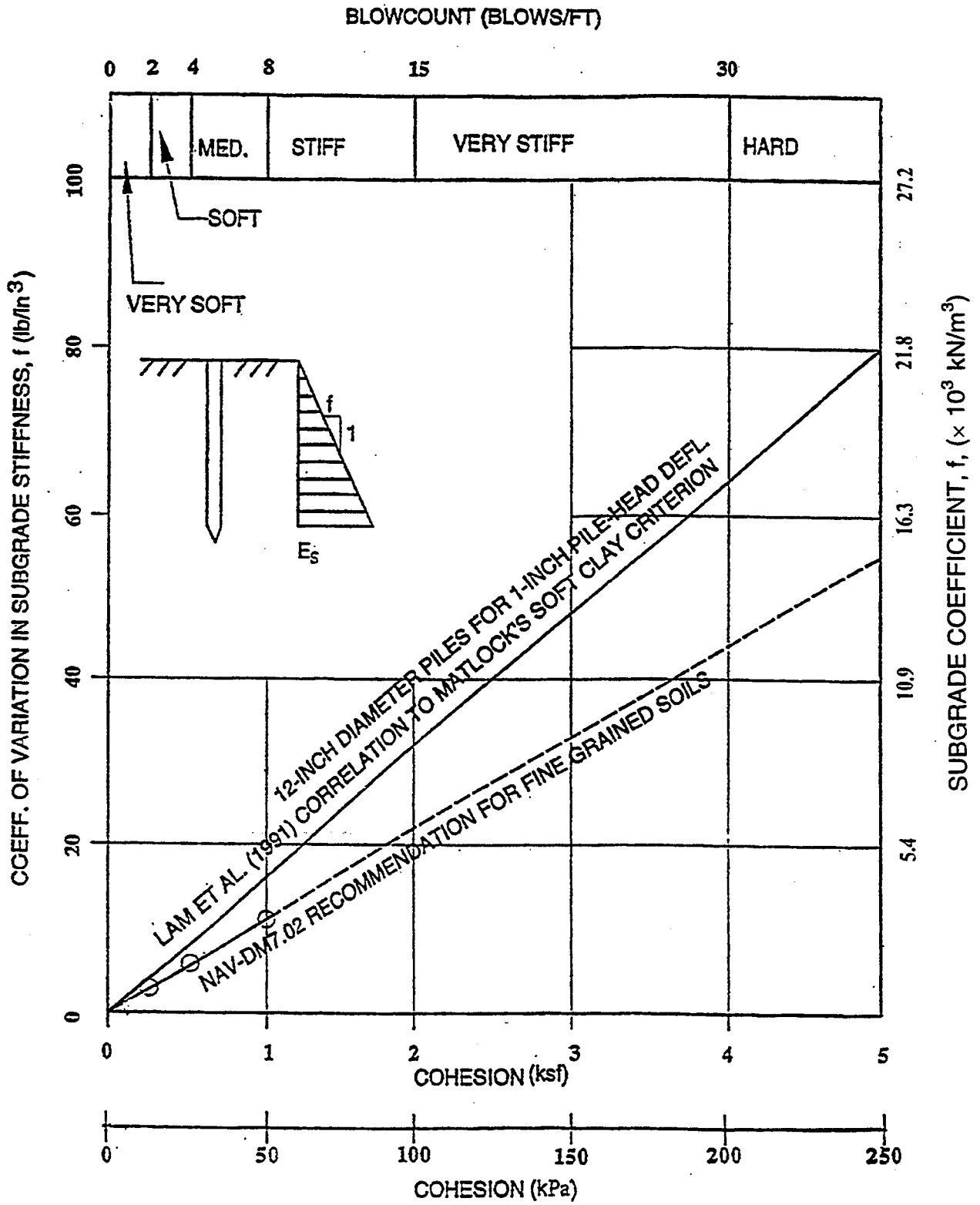
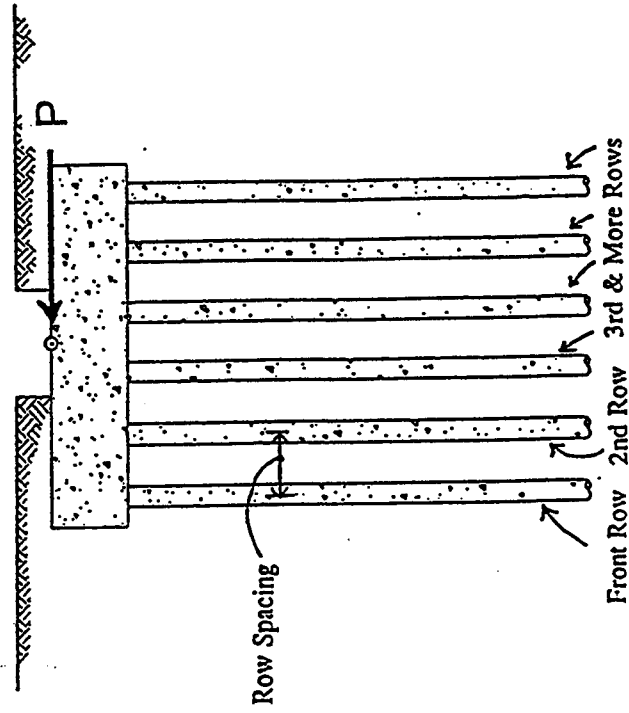
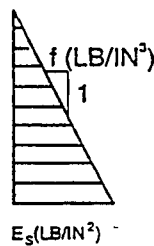
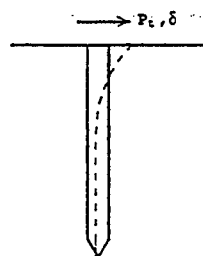
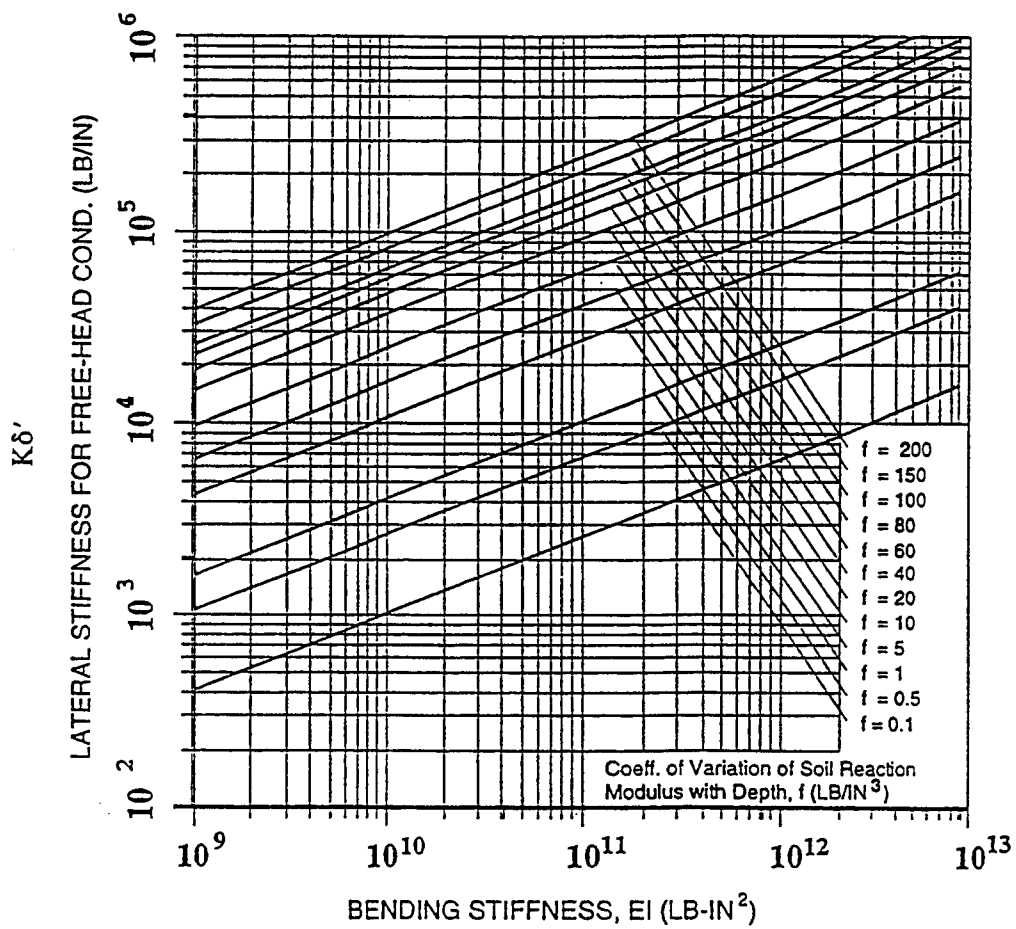


Figure 9-20: Coefficient of Variation of Subgrade Modulus for Clay.

# 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/in<sup>3</sup> = 272 kN/m<sup>3</sup>
- 1 ksf = 48 kPa
- 1 lb/in = 0.1752 kN/m
- 1 lb-in<sup>2</sup> = 2.87x10<sup>-6</sup> kN-m<sup>2</sup>

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:

$$\begin{aligned} (\text{p-multiplier})_{\text{ave}} \text{ in Longitudinal Direction} &= \\ &= \underline{(0.8 + 0.45 + 0.35) / 3 = 0.53} \end{aligned}$$

$$\begin{aligned} (\text{p-multiplier})_{\text{ave}} \text{ in Transverse Direction} &= \\ &= \underline{(1.0 + 0.85 + 0.75) / 3 = 0.87} \end{aligned}$$

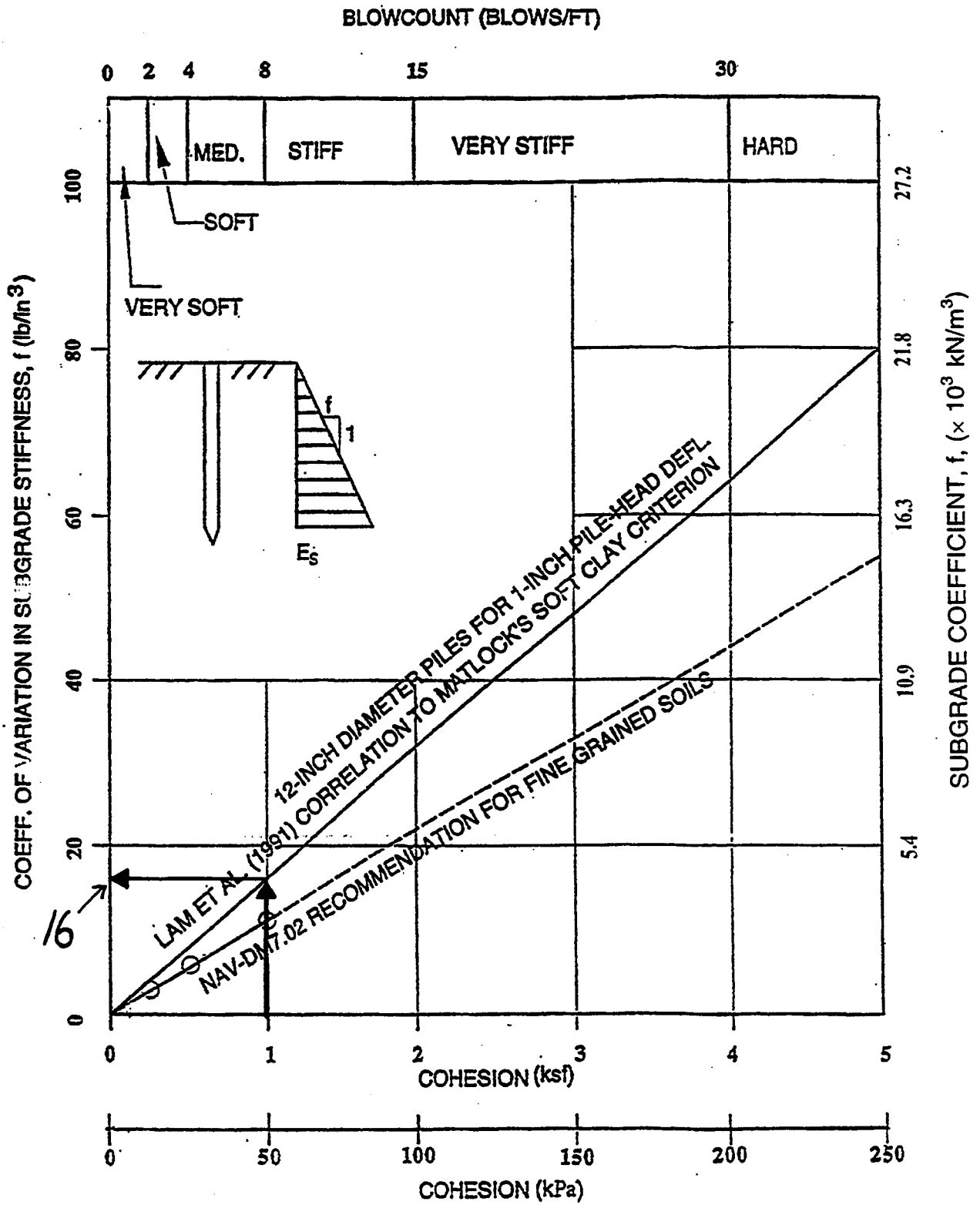


Figure 9-20: Coefficient of Variation of Subgrade Modulus for Clay.



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

$$\text{Longitudinal: } f_{\text{eff}, L} = \underline{0.53 \times 16 = 8.5 \text{ lb/in}^3}$$

$$\text{Transverse: } f_{\text{eff}, T} = \underline{0.87 \times 16 = 13.9 \text{ lb/in}^3}$$

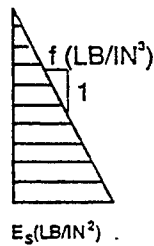
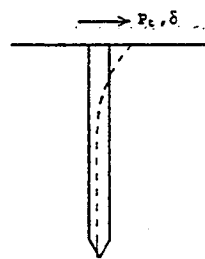
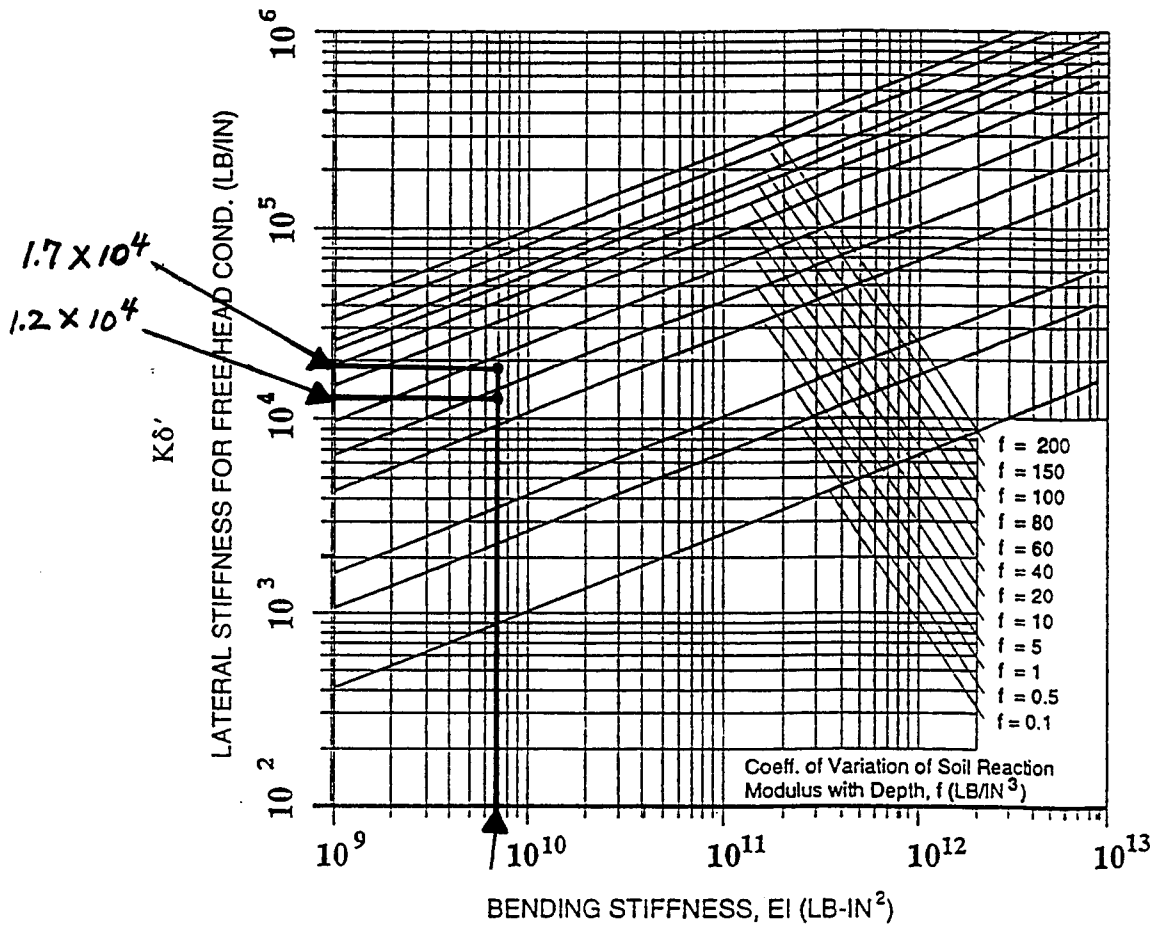
- D. Calculate the Bending Stiffness of the Pile

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

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

$$k'_{\delta, L} = \underline{1.2 \times 10^4 \text{ lb/in}} \text{ (Longitudinal)}$$

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



FREE HEAD PILE STIFFNESS  $K\delta'$

$$= K_g - \frac{K_{g0}^2}{K_g}$$

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

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

- 1 lb/in<sup>3</sup> = 272 kN/m<sup>3</sup>
- 1 ksf = 48 kPa
- 1 lb/in = 0.1752 kN/m
- 1 lb-in<sup>2</sup> = 2.87 x 10<sup>-6</sup> kN-m<sup>2</sup>

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

A. Lateral Stiffness

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

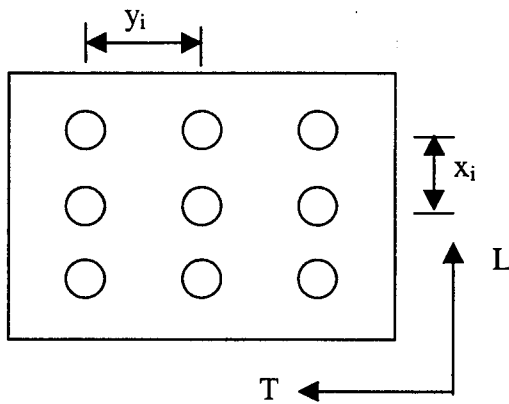
$$\begin{aligned} K_T &= n \cdot k'_{\delta,T} = 9 \times (1.7 \times 10^4) \\ &= 1.53 \times 10^5 \text{ lb/in (Transverse)} \end{aligned}$$

B. Axial Stiffness

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

C. Torsional Stiffness

$$\begin{aligned}
 K_{TOR} &= \sum_{i=1}^n k'_{\delta,L} \cdot y_i^2 + \sum_{i=1}^n k'_{\delta,T} \cdot x_i^2 \\
 &= \underline{1.2 \times 10^4 \times 6 \times (70)^2 + 1.7 \times 10^4 \times 6 \times (42)^2} \\
 &= \underline{5.3 \times 10^8 \text{ lb-in/rad}}
 \end{aligned}$$



D. Rocking Rotational Stiffness about Transverse Axis.

$$\begin{aligned}
 K_{RT} &= k_V \cdot \sum_{i=1}^n x_i^2 \\
 &= \underline{1,540,000 \times 6 \times (42)^2 = 1.6 \times 10^{10} \text{ lb-in/rad}}
 \end{aligned}$$

E. Rocking Rotational Stiffness about Longitudinal Axis.

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

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

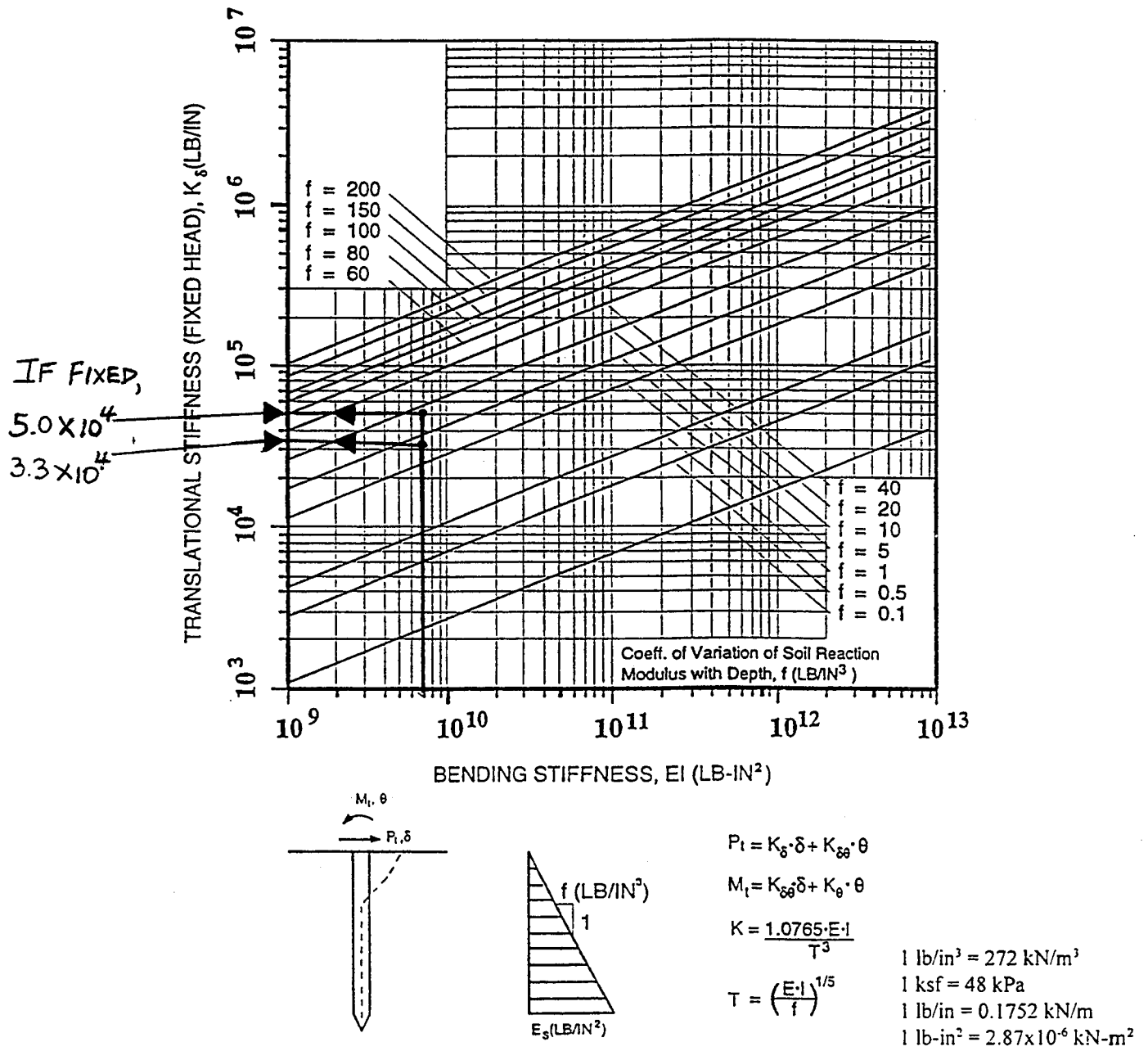


Figure 9-22: Lateral Stiffness of Fixed Head Piles.

## **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 Embankment 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.

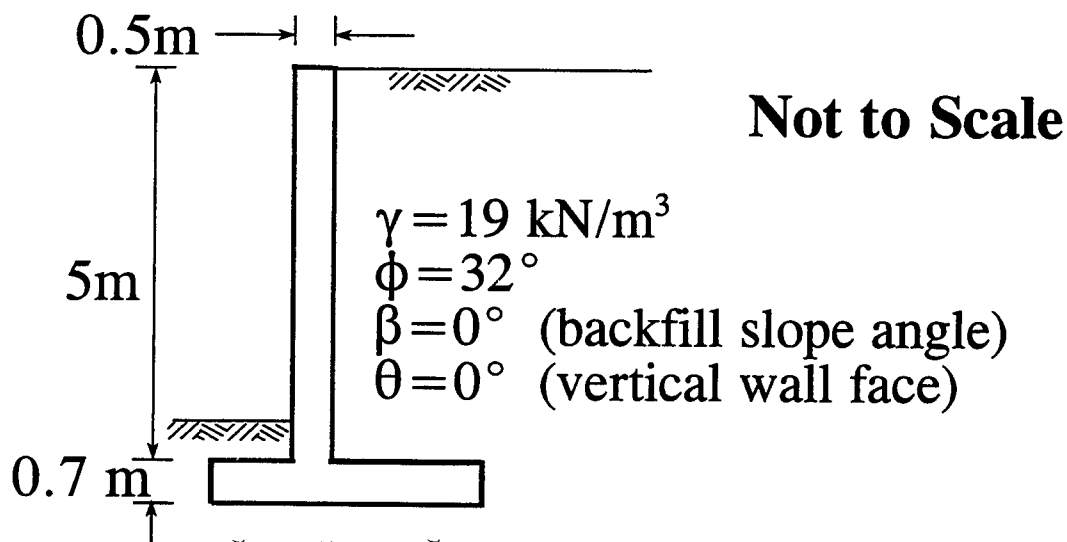


Figure S10A-1: Retaining Wall Geometry and Assumptions



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)

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

$K_{ae} =$  \_\_\_\_\_

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,  $H = 5 + 0.7 = \underline{5.7 \text{ m}}$
2.  $\delta = \phi = \underline{32^\circ}$
3.  $k_h = \text{Amplified Peak Ground Acceleration}/g$   
 $= 1.5 \times 0.15 = \underline{0.225}$

Ignore Vertical Ground Motion  $k_v = 0$

4.  $\psi = \text{arc tan } [k_h/(1-k_v)] = \underline{12.7^\circ}$
5. Compute Active Earth Pressure Coefficient (Eq.9-13b)

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

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

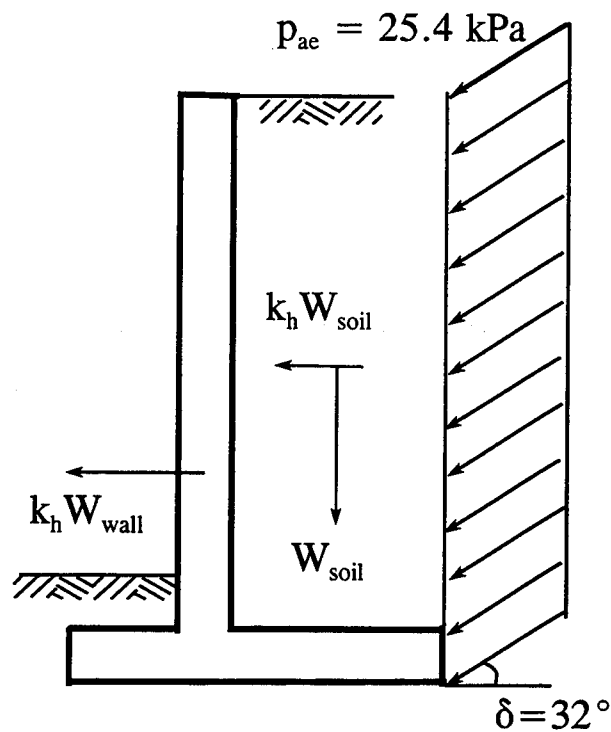
$$K_{ae} = \underline{0.47}$$

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:

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

$$\Rightarrow N = K_h = \text{_____ (Transmittable Acceleration Coefficient)}$$

## SOLUTIONS TO EXERCISE NO. 10B

1.  $d_R = \text{Allowable Displacement} = 50 \text{ mm}$
2. Peak Ground Acceleration Coefficient,  $A = 0.225$
3. Peak Ground Velocity,  $V = 250 \text{ mm/sec}$
4. Use Eq. 9-14:

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

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

$$\Rightarrow N = K_h = 0.106 \text{ (Transmittable Acceleration Coefficient)}$$

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

