TRB WORKSHOP ON NEW APPROACHES TO LIQUEFACTION ANALYSIS

EXAMPLE 1 LIQUEFACTION POTENTIAL ASSESSMENT

Background: Soil borings have identified a 1-m-thick layer of potentially liquefiable soil 6.5 m below ground surface at the site of a bridge abutment near Charleston, South Carolina.

Problem: Evaluate the potential for liquefaction at the site using the Seed-Idriss simplified method as presented in Chapter 8 of the training manual for the NHI Training Course in Geotechnical Foundation Engineering - Module 9, Geotechnical Earthquake Engineering.

Consider the peak ground acceleration with a 10 percent probability of being exceeded in 50 years in accordance with AASHTO requirements.

Resources: Soil profile shown in Figure 1.

SPT energy data shown in Figure 2.

Data on seismic hazard for Charleston from USGS website (http://geohazard.cr.usgs.gov/eq/) presented in Table 1.

Chapter 8 from training manual for NHI Course No. 13239, Module 9 (excerpts attached).

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Written by: N. M. Date: 99 / 01 / 06 (YY MM DD) Reviewed by: EK Date: 99 / 01 / 06 (YY MM DD)

Client: TRB Project: LIQUEFACTION WRKSHP Project/Proposal No.: – Task No.: –

TABLE 1: USGS SEISMIC HAZARD DATA

http://geohazards.cr.usgs.gov/eq/cgi-bin/zipcode.cgi

NATIONAL SEISMIC HAZARD MAPPING PROJECT

USGS, Central Region, Geologic Hazards Team
Golden, Colorado

The input zip-code is 29401.

ZIP CODE	29401
LOCATION	32.7786 Lat. -79.9377 Long.
DISTANCE TO NEAREST GRID POINT	4.2550 kms
NEAREST GRID POINT	32.8 Lat. -79.9 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	16.530781	34.432880	75.529732
0.2 sec SA	31.080429	62.704430	138.920105
0.3 sec SA	23.144159	48.232422	114.840698
1.0 sec SA	6.990796	16.655899	40.275890

- -

http://geohazards.cr.us...q/deagg/charleston.data

Deaggregated Seismic Hazard PE = 2% in 50 years pga
Charleston SC 32.800 deg N 79.967 deg W PGA=0.75790 g

	M<= 5.0	5.5	6.0	6.5	7.0	7.5
d<= 25.	3.112	4.486	4.639	3.929	2.164	57.547
50.	0.067	0.238	0.571	1.018	0.986	15.824
75.	0.001	0.006	0.028	0.095	0.153	3.767
100.	0.000	0.000	0.002	0.012	0.028	0.839
125.	0.000	0.000	0.000	0.003	0.009	0.374
150.	0.000	0.000	0.000	0.001	0.003	0.086
175.	0.000	0.000	0.000	0.000	0.001	0.008
200.	0.000	0.000	0.000	0.000	0.000	0.002
225.	0.000	0.000	0.000	0.000	0.000	0.001

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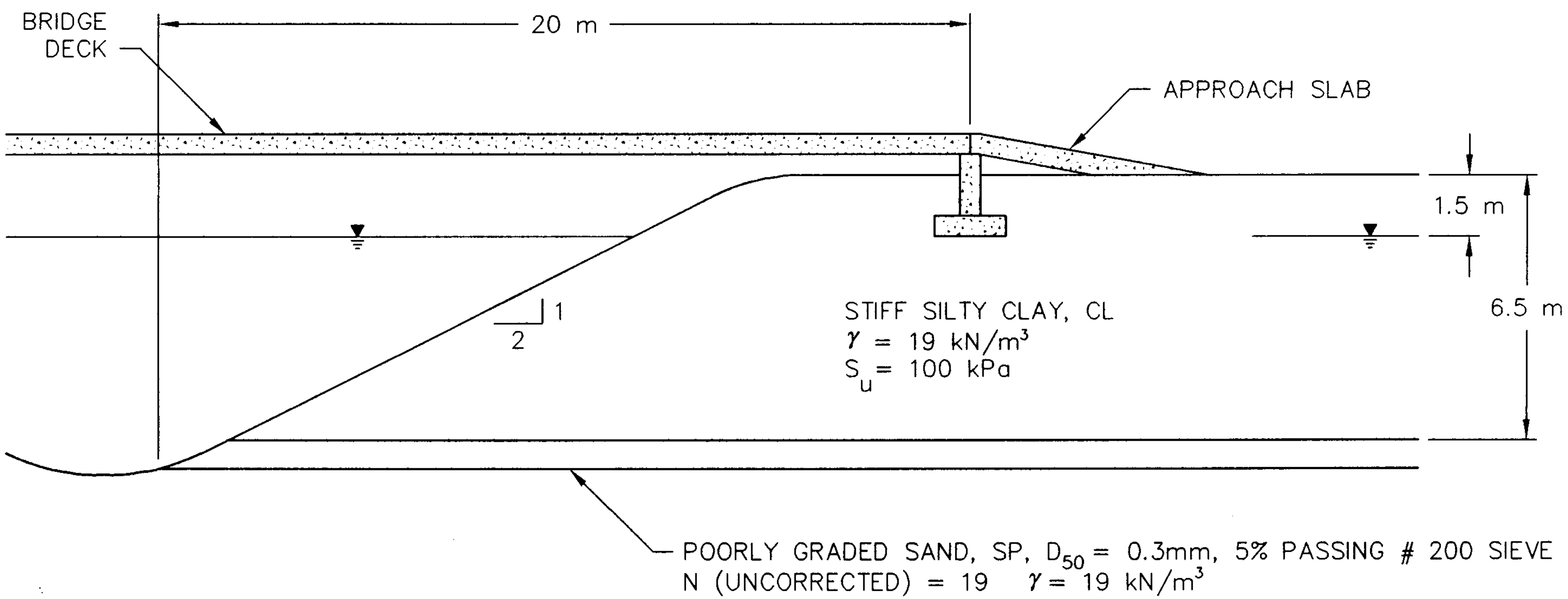
FIGURE 1: SITE PROFILE



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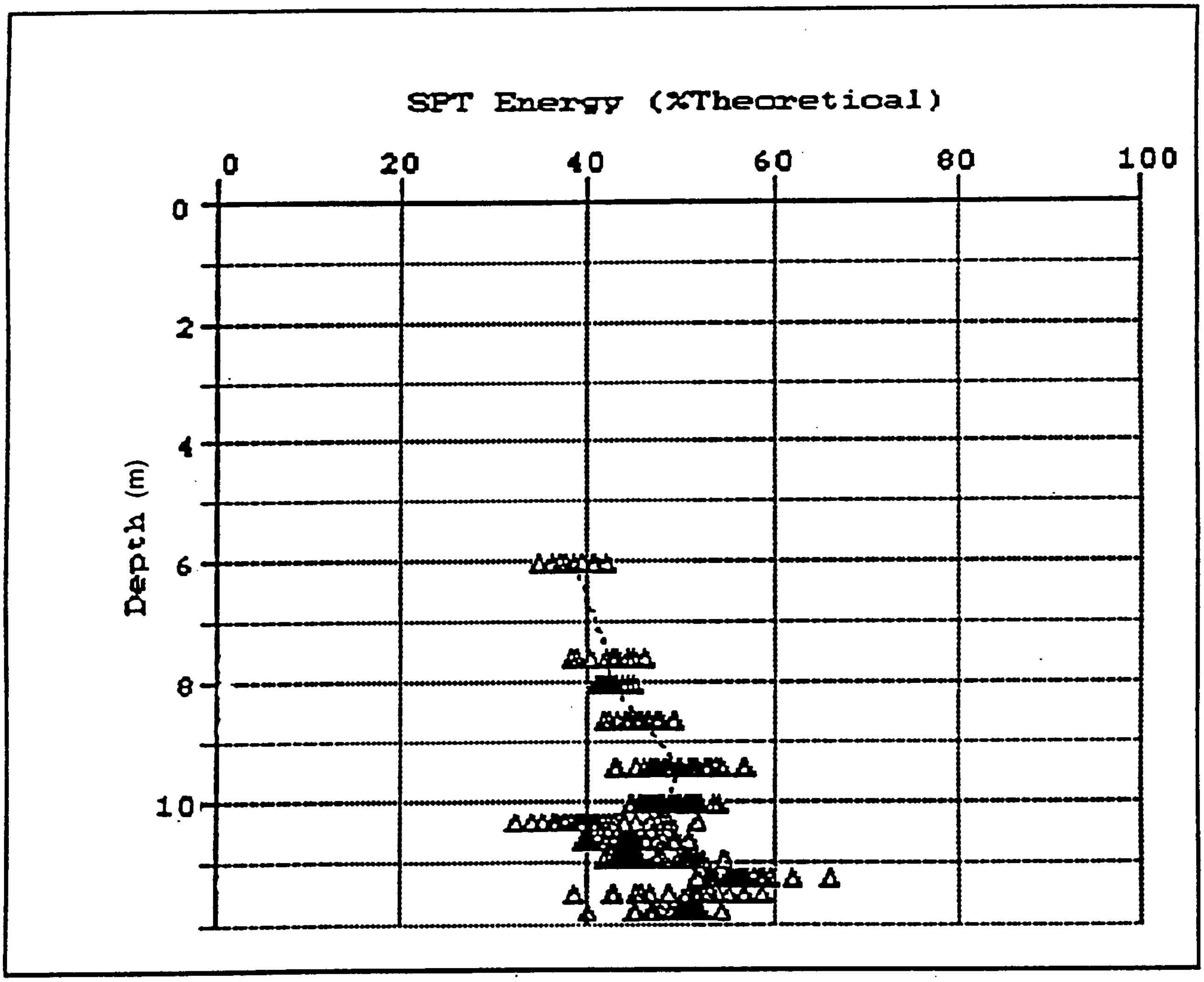
Written by: N.M Date: 99/01/06 Reviewed by: EK Date: 99/01/06

Client: TRB Project: LIQUEFACTION WRKSHP Project/Proposal No.: — Task No.: —

FIGURE 2: SPT ENERGY CALIBRATION DATA



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TRB WORKSHOP EXAMPLE ON LIQUEFACTION POTENTIAL ASSESSMENT

SOLUTION PROCEDURE

I. Establish Design Ground Motion from Website Data

A. Pick PHGA for 10 percent in 50 years from Table 1.

B. Estimate magnitude of design event from deaggregated hazard data for 10 percent in 250 years from Table 1.

C. Estimate epicentral distance from deaggregated hazard data for 10 percent in 250 years from Table 1.

D. Consider influence of local site conditions on PHGA (i.e., consider the potential for site amplification) (Chapter 6 of Training Manual - not included herein).

II. Calculate Earthquake-Induced Shear Stress Ratio, CSR_{EQ}

A. Calculate flexibility ratio, r_d, from Equation 8-1 or Figure 8-2 (attached).

B. Calculate total overburden stress, σ_v.

C. Calculate effective overburden stress, σ'_v.

D. Calculate CSR_{EQ} using Equation 8-3a.

III. Calculate Normalized SPT blow count, $(N_1)_{60}$, Using Equations 5-6 to 5-10 (attached)

A. Evaluate hammer efficiency ratio, H_E, from Figure 2 and Equation 5-9.

B. Evaluate rod length factor, C_{RL}, from Table 5-3 (attached).

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C. Evaluate non-standard sampler set-up correction factor, C_{SS}, from Table 5-3 (attached).

D. Evaluate non-standard borehole correction factor, C_{BD}, from Table 5-3 (attached).

E. Calculate N_{60} using Equation 5-6.

F. Calculate overburden correction factor, C_N, using Equation 5-10 or Figure 5-5 (attached).

G. Calculate $(N_1)_{60}$ using Equation 5-11.

IV. Calculate Stress Ratio Inducing Liquefaction, CSR_L, Using Equation 8-4

A. Calculate stress ratio inducing liquefaction in a magnitude 7.5 earthquake, $CSR_{7.5}$, using Figure 8-3 (attached).

B. Calculate magnitude correction factor, k_m, using Figure 8-4 (attached).

C. Calculate overburden correction factor, k_σ, using Figure 8-5 (attached).

D. Calculate initial static shear stress factor, k_α, using Figure 8-6 (not included, set equal to 1.0 for level ground).

E. Calculate CSR_L using Equation 8-4.

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TRB WORKSHOP EXAMPLE ON LIQUEFACTION POTENTIAL ASSESSMENT

SOLUTION

I. Establish Design Ground Motion from Website Data

A. PHGA for 10 percent in 50 years = 0.17 g

B. Magnitude (moment scale) = Subjective: Based on the Internet data, the magnitude is no greater than 7.5. To be conservative, in the absence of additional data, say M = 7.0. However, argument can be made for a magnitude of 5.5 to 6.5 based upon the magnitude – distance combinations likely to generate a 0.17 g PHGA.

C. Epicentral distance = $\leq$25 km from the web site data. Also, based upon the depth of earthquake sources in the eastern and central U.S., it is probably certain to be at least 10 km.

D. No amplification - use PHGA without modification

II. Calculate Earthquake-Induced Shear Stress Ratio, CSR_{EQ}

A. Flexibility factor, r_d = 0.95 (directly underneath the footing at a depth of 6.5 m)

B. Total overburden stress, σ_v = 123.5 kPa

C. Effective overburden stress, $\sigma'v$ = 74.5 kPa

D. $CSR_{EQ} = 0.65\ (\text{PHGA})\ r_d\left(\dfrac{\sigma v}{\sigma' v}\right) =$ 0.17

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III. Calculate Normalized SPT blow count, (N1)60, Using Equations 5-6 to 5-10 (attached)

A. Hammer efficiency, H_E, = 67 % (Actual hammer efficiency is 40 percent from Figure 2. But H_E is normalized with respect to an efficiency of 60 percent for a standard hammer. Therefore, H_E equals (40/60) in percent, or 67 percent.

B. Rod length correction factor, C_{RL} = 0.95

C. Non-standard sampler correction factor, C_{SS}= 1.0

D. Non-standard borehole correction factor, C_{BD} = 1.0

E. $N_{60} = 19 \times H_E \times C_{RL} \times 1.0 \times 1.0 =$ 13.6

F. Overburden correction factor, C_N = 1.13

G. $(N_1)_{60} = N_{60} \times C_N =$ 15.4

IV. Calculate Stress Ratio Inducing Liquefaction, CSR_L, Using Equation 8-4

A. Stress ratio inducing liquefaction for M 7.5 event, $CSR_{7.5}$ = 0.17

B. Magnitude correction factor, k_m = 1.3 for M = 7.0. Also consider k_m = 1.6 for M = 6.5 and k_m = 1.0 for M = 7.5.

C. Overburden correction factor, k_σ = 1.0

D. Initial static shear stress factor, k_α = 1.0

E. $CSR_L = CSR_{7.5} \times k_m \times k_\sigma \times 1.0 =$ 0.22 for M = 7.0 Also CSR_L = 0.27 for M = 6.5 and CSR_L = 0.17 for M = 7.5.

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V. Calculate the Factor of Safety, CSR_L/CSR_{EQ}

A. FS = 1.3 for M = 7.0 Also, FS = 1.6 for M = 6.5 and FS = 1.0 for M = 7.5.

This example was intended the importance of magnitude in evaluating liquefaction potential as well as the difficulty in establishing the appropriate magnitude from the information currently available from the USGS and NEHRP.

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TRB WORKSHOP
ON
NEW APPROACHES TO LIQUEFACTION ANALYSIS

EXAMPLE 2
LATERAL SPREADING AND

RESIDUAL SHEAR STRENGTH

Background: Same site and soil profile as example on liquefaction potential assessment.

Problem: Evaluate anticipated lateral spreading and post-liquefaction stability.

Resources: Soil profile shown in Figure 1.

SPT energy data shown in Figure 2.

Data on seismic hazard for Charleston from USGS website (http://geohazard.cr.usgs.gov/eq/) presented in Table 1.

Chapter 8 from training manual for NHI Course No. 13239, Module 9 (excerpts attached).

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TRB WORKSHOP EXAMPLE ON LATERAL SPREADING AND RESIDUAL SHEAR STRENGTH

SOLUTION PROCEDURE

I. Calculate Anticipated Lateral Spreading, Δ_L, Using Equation 8-10a

A. Evaluate H_{15}, cumulative thickness of granular layers with $(N_1)_{60}$ less than or equal to 15 (in meters).

B. Evaluate $D50_{15}$, average mean grain size of layers included in H_{15}, in millimeters.

C. Evaluate F_{15}, average fines content of layers included in H_{15}, in percent.

D. Evaluate S, ground slope, in percent.

E. Evaluate W, ratio of height of free face, H, to distance from free face to point in question, L, in percent (i.e., W = 100 H/L).

F. Use M and R from USGS website data.

G. Calculate Δ_L, estimated lateral displacement, from Equation 8-10a, in meters.

II. Evaluate Residual Shear Strength, S_r, from Figure 5-15 (attached)

A. Evaluate fines correction, N_{CORR}, from Figure 5-15.

B. Calculate corrected $(N_1)_{60\text{-}CS}$ from Equation 5-15.

C. Find S_r from Figure 5-15.

III. Evaluate Post-Liquefaction Static Factor of Safety Using S_r

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TRB WORKSHOP EXAMPLE ON LATERAL SPREADING AND RESIDUAL SHEAR STRENGTH

SOLUTION PROCEDURE

I. Calculate Anticipated Lateral Spreading, Δ_L, Using Equation 8-10a

A. Cumulative thickness of granular sediments with $(N_1)_{60}$ less than or equal to 15, H_{15} = 1.0 m

B. Mean grain size of layers included in H_{15}, $(D_{50})_{15}$ = 0.3 mm

C. Average fines content for layers included in H_{15}, F_{15} = 5%

D. Ground Slope, S = 0%

E. Ratio of height of free face, H, to distance from free face, L, W = H/L, in percent = 32 %

F. Magnitude, M = 6.5 to 7.5 (from Example 1)
Epicentral Distance, R = 10 to 25 km (from Example 1)

G. Δ_L = Antilog (-16.366 + 1.178 M - 0.927 Log R - 0.013R + 0.657 Log W + 0.348 Log H_{15} + 4.527 Log (100 - F_{15}) - 0.922 $D50_{15}$) = See Table 1 Below

Table 1 Lateral Spreading, Δ_L, as a Function of Magnitude and Distance

	M = 6.5	M = 7.0	M = 7.5
R = 10 km	0.8 m	3.1 m	12.1 m
R = 15 km	0.5 m	1.8 m	7.1 m
R = 25 km	0.2 m	0.9 m	3.3 m

This example illustrates the importance of magnitude and distance in evaluating the potential for lateral spreading.

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II. Evaluate Residual Shear Strength, S_r, from Figure 5-15 (attached)

A. Fines correction, N_{CORR} = 0

B. Corrected blow count, $(N_1)_{60\text{-}CS}$ = 15.4

C. Residual shear strength, S_r = 400 to 600 psf (19.2 to 28.8 kPa). Use lower bound (400 psf or 19.2 kPa) and see if site is stable. If not, consider validity of using lower quartile (500 psf or 24 kPa) or median (600 psf or 29.8 kPa) values.

III. Evaluate Post-Liquefaction Static Factor of Safety Using S_r

See attached PC STABL analysis. Assign residual strength to liquefied layer and calculate static factor of safety. Include dead load of bridge pier (Not given in earlier information. Assume it to be 3 ksf over 8 ft (or 143.6 kPa over 2.4 m).

Result of static Factor of Safety equal to 1.9 with lower bound value residula strength (S_r equal to 400 psf , or 19.2 kPa) indicates acceptable post-earthquake stability and very low potential for flow failure. (Note: High psot-earthquake static FS is undoubtedly due to very high undrained strength of 100 kPa assigned to silty clay).

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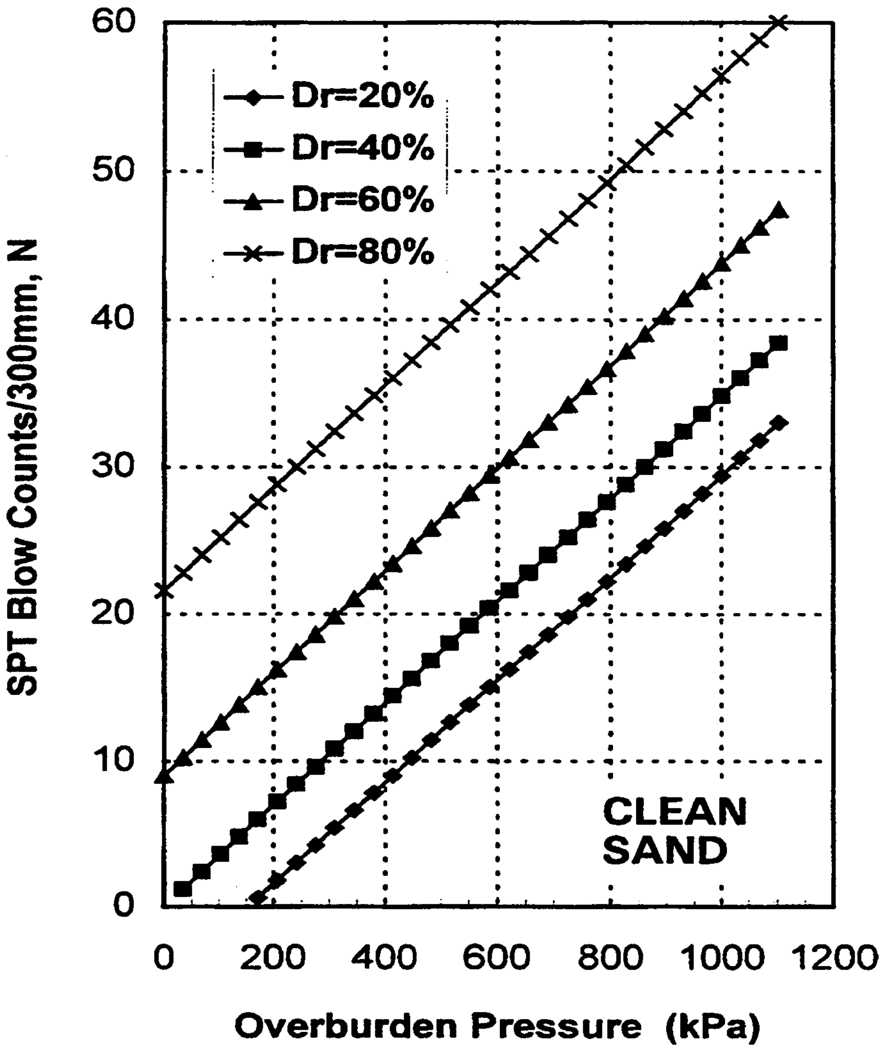

Figure 5-4: SPT-relative Density Correlation. (After Marcuson and Bieganousky, 1977, reprinted by permission of ASCE)

The procedure used to account for the effects of energy variations and overburden pressure on the field SPT blow counts is presented below.

Step 1: Evaluate the *standardized* SPT blow count, N_{60}, which is the standard penetration test blow count for a hammer with an efficiency of 60 percent (60 percent of the nominal SPT energy is delivered to the drill rod). The "standardized" equipment corresponding to an efficiency of 60 percent is specified in Table 5-2. If nonstandard equipment is used, N_{60} is obtained from the equation:

$$N_{60} = N \cdot C_{60} \tag{5-6}$$

where C_{60} is the product of various correction factors. The equation for the global correction factor, C_{60}, in Equation 5-6 and the contributing correction factors recommended by various investigators for some common non-standard SPT configurations are provided in Table 5-3 (Richardson, *et al.*, 1995). The correction factors for Non-standard Hammer Type, C_{HT}, and Non-

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standard Hammer Weight or Height of Fall, C_{HW}, combine to represent a hammer energy factor, H_E:

$$H_E = C_{HT} \cdot C_{HW} \tag{5-7}$$

Therefore, the global SPT correction factor may be written as:

$$C_{60} = H_E \cdot C_{SS} \cdot C_{RL} \cdot C_{BD} \tag{5-8}$$

and C_{SS}, C_{RL}, and C_{BD} are the Non-standard Sampler Setup, Short Rod Length, and Non-standard Borehole Diameter factors presented in Table 5-3.

For important projects, H_E may be calculated directly, by measuring the hammer energy. There are two commercially available methods for measuring hammer energy: the Force Squared (F2) method and the Force Velocity (FV) method. In the F2 method, strain gauge load cells are used to measure the force transmitted to the drill rods. The square of the force is integrated over time to calculate the hammer energy. In the FV method, the product of the force times the velocity is integrated over time. The FV method requires both load cells to measure the transmitted force and an accelerometer to measure the velocity time history. Equipment for making FV measurements are similar to pile driving analyzer equipment for dynamic load testing of driven piles.

In general, the F2 method is not considered as reliable as the FV method and is not recommended for correcting SPT blow counts. Using the energy measured by the FV method, F_{VE} the energy correction factor may be evaluated as:

$$H_E = \frac{F_{VE}}{0.6\,F_{max}} \tag{5-9}$$

where F_{max} is the theoretical maximum energy of the SPT hammer (1,151 kg m^2/s^2).

If CPT data are available, N_{60} can be obtained from the chart relating N_{60} to q_c and D_{50} presented in Figure 5-7 (Robertson et al.,1983).

Step 2: Calculate the normalized and standardized SPT blow count, $(N_1)_{60}$. $(N_1)_{60}$ is the standardized blow count normalized to an effective overburden pressure of 96 kPa in order to eliminate the influence of confining pressure. The most commonly used technique for normalizing blow counts is via the correction factor, C_N, shown in Figure 5-5 (Seed, *et al.*, 1983). However, the closed-form expression proposed by Liao and Whitman (1986) may also be used:

$$C_N = 9.79\ (1/\sigma_v')^{1/2} \tag{5-10}$$

where σ_v' equals the vertical effective stress at the sampling point in kPa.

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TABLE 5-2
RECOMMENDED "STANDARDIZED" SPT EQUIPMENT
(After Seed, *et al.*, 1985 and Riggs, 1986, Reprinted by Permission of ASCE)

Element	Standard Specification
Sampler	Standard split-spoon sampler with: (a) Outside Diameter, O.D. = 51 mm, and (b) Inside Diameter, I.D. = 35 mm (constant - i.e., no room for liners in the barrel)
Drill Rods	A or AW-type for depths less than 15.2 m; N- or NW-type for greater depths
Hammer	Standard (safety) hammer with: (a) weight = 63.5 kg; (b) drop = 762 mm (delivers 60% of theoretical free fall energy)
Rope	Two wraps of rope around the pulley
Borehole	100- to 130-mm diameter rotary borehole with bentonite mud for borehole stability (hollow stem augers where SPT is taken through the stem)
Drill Bit	Upward deflection of drilling mud (tricone or baffled drag bit)
Blow Count Rate	30 to 40 blows per minute
Penetration Resistance Count	Measured over range of 150 to 460 mm of penetration into the ground

Notes: (1) If the equipment meets the above specifications, $N = N_{60}$ and only a correction for overburden is needed.

(2) This specification is essentially the same to the ASTM D 1586 standard.

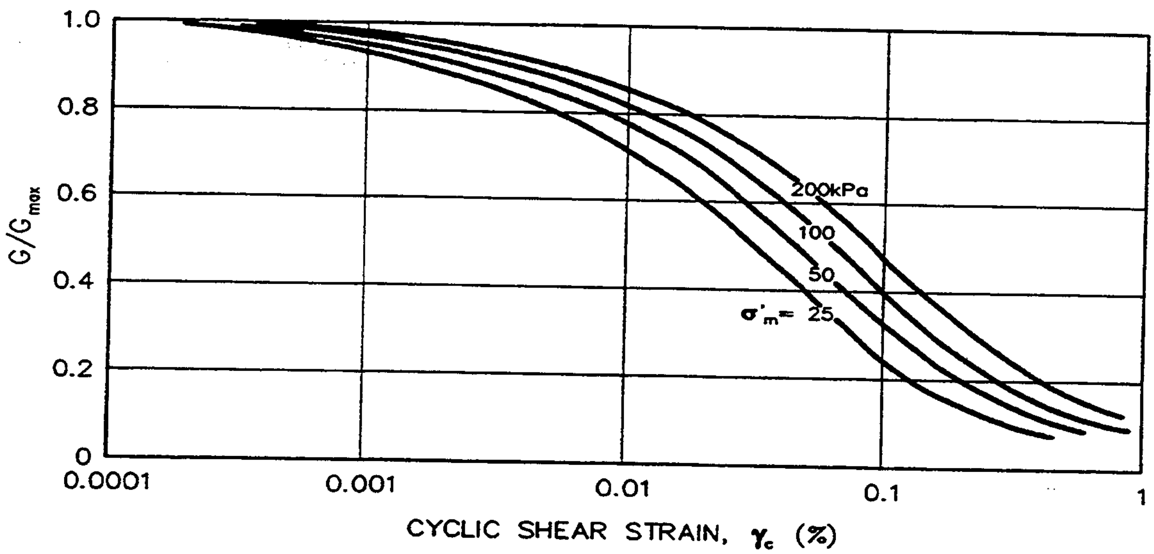

Figure 5-13: Shear Modulus Reduction Curves for Sands. (Iwasaki, *et al.*, 1978, reprinted by permission of Japanese Society of Soil Mechanics and Foundation Engineering)

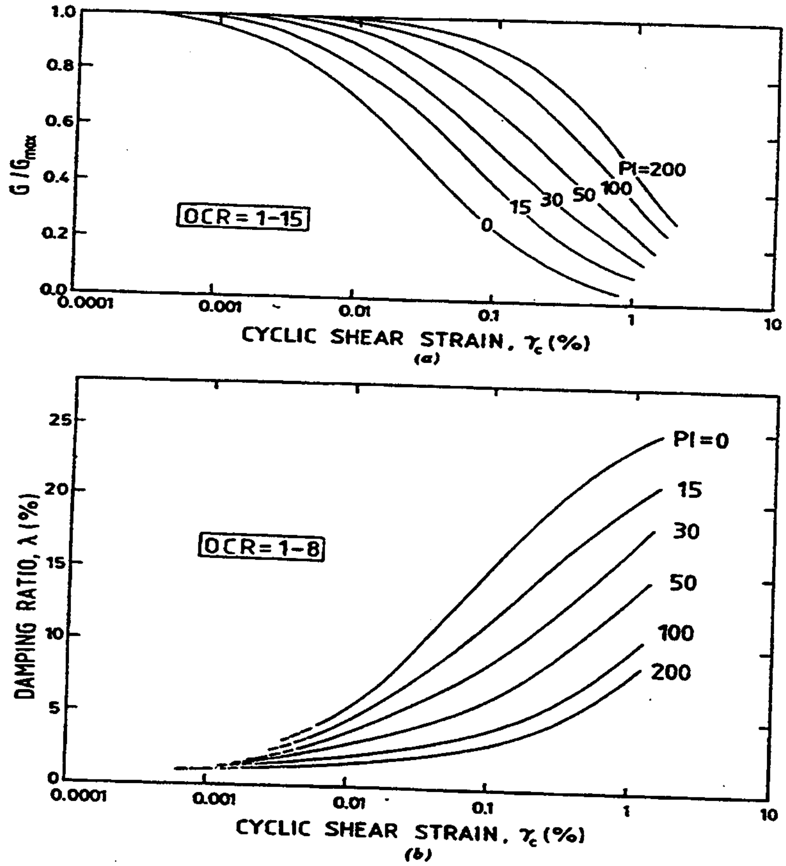

Figure 5-14: Shear Modulus Reduction and Damping Ratio as a Function of Shear Strain and Soil Plasticity Index. (Vucetic and Dobry, 1991, reprinted by permission of ASCE)

The dynamic undrained shear strength of a soil may be influenced by the amplitude of the cyclic deviator stress, the number of applied loading cycles, and the plasticity of the soil. For saturated cohesionless soils, even relatively modest cyclic shear stresses can lead to pore pressure rise and a significant loss of undrained strength. However, Makdisi and Seed (1978) point out that substantial permanent strains may be produced by cyclic loading of clay soils to stresses near the yield stress, while essentially elastic behavior is observed for large numbers of (>100) cycles of loading at cyclic shear stresses of up to 80 percent of the undrained strength. Therefore, these investigators recommend the use of 80 percent of the undrained strength as the "dynamic yield strength" for soils that exhibit small increases in pore pressure during cyclic loading, such as clayey materials, and partially saturated cohessionless soils.

Evaluation of the potential for shear strength reduction in a saturated or almost saturated cohesionless soil (low plasticity silt, sand, or gravel) subjected to dynamic loading may require sophisticated cyclic laboratory testing. Alternatively, a residual strength may be assigned to the soil based upon either undrained laboratory tests or in situ test results.

The residual shear strength after cyclic loading is of critical importance in assessing the post-liquefaction stability of a foundation or earth structure. Saturated soils which liquefy typically possess some "residual" shear strength even when in the liquefied state. In initially loose soils, this residual strength may be very small and of little consequence. In initially dense soils, particularly in dense granular soils which tend to dilate, or expand in volume, when sheared, this residual strength can be significant and of great consequence in acting as a stabilizing force subsequent to liquefaction.

Evaluation of residual shear strength from laboratory tests is not typically recommended due to the difficulties associated with testing. Use of residual strengths derived from in situ testing is, in general, considered more reliable than use of laboratory test results. However, use of residual strengths in assessments of the pseudo-static factor of safety and/or yield acceleration can result in very conservative values (Marcuson, *et al.*, 1990), as discussed in Chapter 7.

The *steady-state* shear strength, S_{su}, governs the behavior of liquefied soil. Poulos, *et al.* (1985) proposed a methodology for evaluation of the in situ S_{su} based on obtaining high-quality soil samples with minimal disturbance. The high-quality samples were tested in the laboratory and the laboratory strengths were then adjusted for field conditions using specially developed techniques to correct the resulting laboratory S_{su} values for effects of void ratio changes due to sampling, handling, and test set-up. Due to the very high sensitivity of S_{su} to even small changes in void ratio, the laboratory techniques proposed by Poulos, *et al.* presently do not appear to represent a reliable basis for engineering analyses unless very conservative assumptions and high factors of safety are employed to account for the considerable uncertainties involved.

Because of difficulties in measuring steady-state strength in the laboratory, Seed (1987) proposed an alternate technique for evaluation of in situ undrained residual shear strength based on the results of SPT testing. He back analyzed a number of liquefaction-induced failures from which residual strength could be calculated for soil zones in which SPT data was available, and proposed a correlation between *residual strength*, S_r, and $(N_1)_{60\text{-}cs}$. $(N_1)_{60\text{-}cs}$ is a "corrected" normalized standardized SPT blow count, as discussed in Section 5.4.2, with a correction, N_{corr}, for fines content to generate an equivalent "clean sand" blow count as:

$$(N_1)_{60-cs} = (N_1)_{60} + N_{corr} \qquad (5\text{-}15)$$

where N_{corr} is a function of percent of fines. Recommendations for selecting N_{corr} are given in the insert of Figure 5-15. Since there is no guarantee that all the conditions for steady-state of deformation were

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8.3 EVALUATION OF LIQUEFACTION POTENTIAL

8.3.1 Introduction

Due to the difficulties in obtaining and testing undisturbed representative samples from most potentially liquefiable soil materials, in situ testing is the approach preferred by most engineers for evaluating the liquefaction potential of a soil deposit. Liquefaction potential assessment procedures involving both the SPT and CPT are widely used in practice (e.g., Seed and Idriss, 1982; Ishihara, 1985; Seed and De Alba, 1986; Shibata and Teparaska, 1988; Stark and Olson, 1995). For gravelly soils, the Becker Penetration Test (BPT) is commonly used to evaluate liquefaction potential (Harder and Seed, 1986). Geophysical techniques for measuring shear wave velocity have recently emerged as potential alternatives for liquefaction potential assessment (Tokimatsu, *et al.*, 1991; Youd and Idriss, 1997).

8.3.2 Simplified Procedure

The most common procedure used in engineering practice for the liquefaction potential assessment of sands and silts is the *Simplified Procedure* originally developed by Seed and Idriss (1982). Since its original development, the original Simplified Procedure as proposed by Seed and Idriss has been progressively revised, extended, and refined (Seed, *et al.*, 1983; Seed, *et al.*, 1985; Seed and De Alba, 1986; Liao and Whitman, 1986). The Simplified Procedure may be used with either SPT or CPT data. Recent summaries of the various revisions to the Simplified Procedure are provided by Marcuson, *et al.*, (1990) and Seed and Harder (1990). A 1996 workshop sponsored by the National Center for Earthquake Engineering Research (NCEER) reviewed recent developments on evaluation of liquefaction resistance of soils and arrived consensus on improvements and augmentation to the simplified procedure (Youd and Idriss, 1997). Based primarily on recommendations from these studies, the Simplified Procedure for evaluating liquefaction potential at the site of highway facilities can be performed using the following steps:

Step 1: From borings and soundings, in situ testing and laboratory index tests, develop a detailed understanding of the project site subsurface conditions, including stratigraphy, layer geometry, material properties and their variability, and the areal extent of potential problem zones. Establish the zones to be analyzed and develop idealized, representative sections amenable to analysis. The subsurface data used to develop the representative sections should include the location of the water table, either SPT blow count, N, or tip resistance of a standard CPT cone, q_c, mean grain size, D_{50}, unit weight, and the percentage of fines in the soil (percent by weight passing the U.S. Standard No. 200 sieve).

Step 2: Evaluate the total vertical stress, σ_v, and effective vertical stress, σ_v', for all potentially liquefiable layers within the deposit both at the time of exploration and for design. Vertical and shear stress design values should include the stresses resulting from facility construction. Exploration and design values for vertical total and effective stress may be the same or may differ due to seasonal fluctuations in the water table or changes in local hydrology resulting from project development. Note that for underwater sites, the total weight of water above the mudline should not be included in calculating the total vertical stress. Also evaluate the initial static shear stress on the horizontal plane, τ_{ho}, for design.

Step 3: If results of a site response analysis are not available, evaluate the *stress reduction factor*, r_d as described below. The stress reduction factor is a soil flexibility factor defined as the ratio of the peak shear stress for the soil column, $(\tau_{max})_d$, to that of a rigid body, $(\tau_{max})_r$. There are several ways to obtain r_d. For non-critical projects, the following equations for r_d were recommended by

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Use of τ_{max} from site response analysis (or use of the results of a site response analysis to evaluate r_d) is considered to be generally more reliable than any of the simplified approaches to estimate r_d, and is strongly recommended for sites that are marginal with respect to liquefaction potential (i.e., sites where the factor of safety for liquefaction is close to 1.0).

Step 4: Calculate the *critical stress ratio induced by the design earthquake,* CSR_{EQ}, as:

$$CSR_{EQ} = 0.65\ (a_{max}/g)\ r_d\ (\sigma_v/\sigma_v') \tag{8-3a}$$

If the results of a seismic site response analysis are available, CSR_{EQ} can be evaluated from τ_{max} as:

$$CSR_{EQ} = 0.65\ \tau_{max}/\sigma_v' \tag{8-3b}$$

Note that the ratio τ_{max}/σ_v' corresponds to the peak average acceleration denoted by k_{max} in Chapter 6.

Step 5: Evaluate the *standardized* SPT blow count, N_{60}, using the procedure presented in Chapter 5.

Step 6: Calculate the normalized and standardized SPT blow count, $(N_1)_{60}$, using the procedure presented in Chapter 5

Step 7: Evaluate the critical stress ratio $CSR_{7.5}$ at which liquefaction is expected to occur during an earthquake of magnitude $M_w = 7.5$ as a function of $(N_1)_{60}$. Use the chart developed by Seed, *et al.* (1985) as modified by NCEER, shown in Figure 8-3, to find $CSR_{7.5}$. It should be noted that this chart was developed using a large database from sites where liquefaction did or did not occur during past earthquakes. The general conditions for the case history data presented in this chart are as follows: (1) all sites evaluated were under level ground condition, (2) the effective overburden pressure for all cases does not exceed 96 kPa, and (3) the magnitude of the earthquakes considered in all cases was in the neighborhood of 7.5.

Step 8: Calculate the *corrected* critical stress ratio resisting liquefaction, CSR_L. CSR_L is calculated as:

$$CSR_L = CSR_{7.5} \cdot k_M \cdot k_\sigma \cdot k_\alpha \tag{8-4}$$

where k_M is the correction factor for earthquake magnitudes other than 7.5, k_σ is the correction factor for stress levels larger than 96 kPa, and k_α is the correction factor for the initial driving static shear stress, τ_{ho}. Previous investigators have derived various recommendations on the magnitude correction factor, K_M, as shown in Figure 8-4. Upon review of all the data, the NCEER workshop participants have recommended a range of K_M values for design and analysis purposes. Their recommendations are presented in Figure 8-4. For effective confining pressures σ'_m larger than 96 kPa, k_σ can be determined from Figure 8-5 (Youd and Idriss, 1997). For σ'_m less than or equal to 96 kPa, no correction is required.

The value of k_α depends on both τ_{ho} and the relative density of the soil, D_r. On sloping ground, or below structures and embankments, τ_{ho} can be estimated using various closed-form elastic solutions (e.g., Poulos and Davis, 1974) or using the results of finite element (static) analyses. Once τ_{ho} and σ_v' are estimated, k_α can be determined from Figure 8-6, originally proposed by Seed

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analyses are warranted.

(2) If $(N_1)_{60}$ values are less than 15, then the evaluation of lateral displacement is performed using the following equations:

For free-face conditions:

$$\begin{aligned}\text{Log}\Delta_L &= -16.366 + 1.178M - 0.927\text{Log } R - 0.013R + 0.657\text{Log } W \\ &\quad +0.348\text{Log } H_{15} + 4.527\text{Log } (100 - F_{15}) - 0.922D50_{15}\end{aligned} \qquad (8\text{-}10a)$$

For ground slope conditions:

$$\begin{aligned}\text{Log}\Delta_L &= -15.787 + 1.178M - 0.927\text{Log } R - 0.013R + 0.429\text{Log } S \\ &\quad +0.348\text{Log } H_{15} + 4.527\text{Log } (100 - F_{15}) - 0.922D50_{15}\end{aligned} \qquad (8\text{-}10b)$$

Where:

Δ_L = Estimated lateral ground displacement in meters
H_{15} = Cumulative thickness of saturated granular layers with corrected blow counts, $(N_1)_{60}$, less than or equal to 15, in meters.
$D50_{15}$ = Average mean grain size in granular layer included in H_{15} in mm.
F_{15} = Average fines content for granular layers included in H_{15} in percent.
M = Earthquake magnitude (moment magnitude).
R = Horizontal distance from seismic energy source, in kilometers.
S = Ground slope, in percent.
W = Ratio of the height (H) of the free face to the distance (L) from the base of the free face to the point in question, in percent (i.e., 100H/L).

Step 3: In areas of significant ground slope, or in situations when a deep failure surface may pass through the body of the facility or through underlying liquified layers, a flow slide can occur following liquefaction. The potential for flow sliding should be checked using a conventional limit equilibrium approach for slope stability analyses (discussed in Chapter 7) together with residual shear strengths in zones in which liquefaction may occur. Residual shear strengths can be estimated from the penetration resistance values of the soil using the chart proposed by Seed, *et al.* (1988) presented in Figure 5-15. Seed and Harder (1990) and Marcuson, *et al.* (1990) present further guidance for performing a post-liquefaction stability assessment using residual shear strengths.

If liquefaction-induced vertical and/or lateral deformations are large, the integrity of the highway facility may be compromised. The question the engineer must answer is "What magnitude of deformation is too large?" The magnitude of acceptable deformation should be established by the design engineer on a case-by-case basis. Calculated seismic deformations on the order of 0.15 to 0.30 m are generally deemed to be acceptable in current practice for highway embankments in California. For highway system components other than embankments, engineering judgement must be used in determining the allowable level of calculated seismic deformation. For example, components that are designed to be unyielding, such as bridge abutments restrained by batter piles, may have more restrictive deformation requirements than structures which can more easily accommodate foundation deformations. At the current time,

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