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Screening Guide for Rapid Assessment of Liquefaction Hazard at Highway Bridge Sites

by

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Publication Date: June 16, 1998
Submittal Date: September 21, 1995

Technical Report MCEER-98-0005

Task Number 106-E-3.1(b)

FHWA Contract Number DTFH61-92-C-00106

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SECTION 5
ASSESSMENT OF GROUND DISPLACEMENT HAZARD

Liquefaction by itself is not a cause of bridge damage. Structural damage occurs when liquefaction induces intolerable ground displacements or deformations or loss of foundation bearing strength. Displacements occur as a consequences of embankment instability, inertially induced deformation, lateral spread, or ground settlement. Analyses of embankment instability and inertial deformation require complex and sophisticated computational procedures and engineering expertise beyond that required for routine screening analyses. Thus only general procedures are given for these evaluations in the following paragraphs. Geotechnical specialists should be consulted where applications of these procedures are required.

Insufficient case history data have been compiled or analyzed to develop quantitative relationships between horizontal ground displacement and bridge damage. Reviews of past bridge performance indicates that most bridges can withstand up to 100 mm of lateral ground displacement without significant damage (Youd, 1993). Displacements of 100 mm or less are generally accommodated through shear or compression of soils rather than through bridge deformation. Even relatively weak bridges, such as light timber structures, have withstood 100 mm of ground displacement without significant distress to the bridge structure. Stronger bridges should be able to withstand much larger ground displacements without distress. Performance criteria have not been developed for bridge damage or distress as a consequence of ground displacements greater than 100 mm. Thus for this screening guide, estimated lateral ground displacements less than 100 mm are considered nonhazardous, while displacements greater than 100 mm are considered potentially hazardous.

Similarly, vertical ground displacements or ground settlements less than 25 mm seldom cause damage to bridges supported by shallow foundations or 100 mm for bridges supported on deep foundations. Thus for screening purposes, vertical displacements less than 25 mm for shallow foundations and 100 mm for deep foundations are also classed as non-hazardous, while vertical displacements greater than these values are classed are classed as potentially hazardous.

5.1 Embankment or Slope Instability

Liquefaction of soil layers temporarily reduces the shear strength of contractive soil. If strength loss beneath an embankment or a steep slope, such as a river bank, reduces shear resistance to a level less than that required for static equilibrium, catastrophic flow failure may ensue. Such failures are generally associated with large ground displacements that could cause fracture or displace bridge foundation elements or collapse approach embankments.

5.1.2 Data Required

The primary data required for analysis of slope stability include the geometry of the slope, soil stratigraphy with appropriate density and strength properties for each soil layer. An estimate of the post-liquefaction or residual shear strength is also required for each liquefiable layer. Most of these

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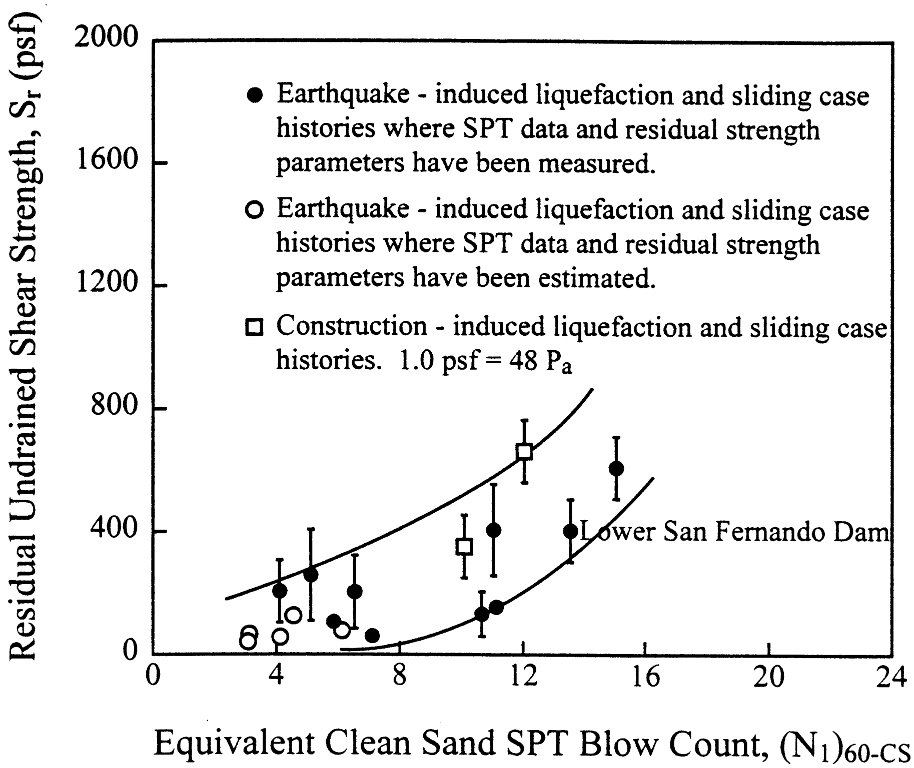

Figure 5-1 Empirical Relationship Between Residual Shear Strength and $(N_1)_{60cs}$ (After Seed and Harder, 1990)

properties are not determined as part of routine foundation investigations. Generally, additional drilling, testing, and data interpretation are required for stability analyses. The present state-of-the-art is to estimate residual strengths for liquefiable layers from corrected standard penetration resistance, $(N_1)_{60}$, using criteria (Figure 5-1) developed by Seed and Harder (1990). For critical structures, such as large or heavily traveled bridges, the lower bound of the undrained residual strength plot (Figure 5-1) is commonly used. For less critical structures rivers, residual strengths may be selected from the mid to lower part of the plot.

5.1.3 Analysis and Classification

The stability of a slope or embankment can be checked using standard limit-equilibrium procedures. Commercial computational programs are available to aid this analysis. The analysis is conducted with undrained residual strengths assigned to all liquefiable layers and conservative estimates of drained strengths and undrained strengths for other materials above and below the watertable, respectively. For the stability analysis, only static (gravitational) forces are applied to the embankment or slope. Inertial forces generated by the earthquake are usually neglected at this stage of the investigation. If the calculated static factor of safety (FS) is equal to or less than 1.1, the site could become unstable during or after an earthquake, leading to large soil deformations. Such

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deformations would likely generate large embankment or slope displacements that could adversely affect the integrity of the bridge foundation. Thus if the stability analysis indicates a static factor of safety equal to or less than 1.1, the bridge site is classed as potentially hazardous and prioritized for further investigation and possible remediation.

If the static factor of safety is greater than 1.1, the embankment is classed as stable against catastrophic slope failure, but the embankment may still undergo damaging deformation due to soil softening. In such instances, the evaluation continues to the next step in the analysis (Figure 2.1).

5.2 Analysis of Embankment or Slope Deformation

Damaging ground deformations may occur within or beneath embankments or slopes as a consequence of liquefaction, even though the site may be stable against catastrophic flow failure. Such deformations occur as a consequence of soil softening and yielding due to liquefaction and the inertial forces generated by the earthquake. In these instances, cyclic mobility and limited strains within liquefied layers may lead to ground deformations and displacements that could damage the bridge structure. Analyses of embankment or slope deformation at liquefiable sites is complicated because of the complex nature of constitutive relations for liquefied soils. In particular, stress-strain relations are very complex for moderately dense or dilative soils that may deform under either undrained or partially drained conditions. The nonhomogeneity of natural soils also complicates the characterization of soil properties. The erratic nature and general unpredictability of inertial loads generated by the earthquake additionally complicates the analysis. Thus deformation analyses are generally performed by specialists in geotechnical earthquake engineering.

5.2.2 Data Required

All of the data required for analysis of slope stability analyses (Section 5.1) also are required for analysis of embankment or slope deformation. In addition, a strong motion accelerogram, or set of accelerograms, is required. These accelerograms should be representative of ground motions expected at the site for earthquakes expected in the region. Soil properties required for the analysis include stiffness and damping values for each soil layer and constitutive relations between stresses and strains and between strains and pore-pressure changes. Stiffness is usually couched in terms of small-strain shear modulus, G_{max}, and modulus and damping relationships that are functions of shear strain, soil plasticity, and over-consolidation ratio. Some of the more sophisticated analysis packages apply finite element analysis and require constitutive relationships for the liquefied soil. As noted above, these relationships tend to be complex and difficult to define and quantify for in-place soils at natural sites. The more simple analyses assume plastic soil deformation and simple stress-strain behavior that can be estimated from soil strength data. Although perhaps not as exact, the simpler procedures are much easier to apply and require less soil testing and analysis. Even with the simplified procedures, specialized expertise is required, however, to properly perform the analysis and interpret the results. Because this level of expertise exceeds that of typical practicing geotechnical engineers, outside specialists should be consulted or employed to perform embankment deformation analyses.

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5.2.3 Analysis and Classification

No single procedure has gained widespread acceptance for analysis of embankment or slope deformations at liquefiable sites. The most widely used analyses are based on the simplified sliding-block model of Newmark (1965). For example, the procedures developed by Makdisi and Seed (1978) and by Byrne (1991) use the sliding-block mechanism. The procedure requires ground response evaluation for critical points in the slope or embankment. Only acceleration pulses with magnitudes greater than a critical or yield acceleration, are can activate the sliding block and generate slope displacement. Displacement continues for the short period of time while inertial and kinetic forces are sufficient to sustain movement. Once movement generated by the acceleration pulse is arrested by soil shear resistance, no additional displacement occurs until a subsequent acceleration pulse exceeds the yield acceleration. The incremental displacements generated by each acceleration pulse are then summed to yield an estimate of total slope displacement.

The yield acceleration is a function of the factor of safety calculated for static slope stability (described in Section 5.1.2). For a slope with a factor of safety of 1.0, the yield acceleration is zero and all down-slope horizontal accelerations generated by an earthquake induce soil yielding and slope deformation. As the static factor of safety increases, the yield acceleration also increases, reducing the number and widths of acceleration pulses that exceed the yield level. These reductions of mobilizing forces markedly decrease the amount of calculated slope displacement. Past experience and analytical calculations indicate that slope deformations are generally negligible for sites with an adequate static factor of safety. Adequate factors of safety that generally reduce displacements to acceptable limits (less than 100 mm) are approximately 1.5 for magnitude 6.5 earthquakes, 2.0 for magnitude 7.5 earthquakes, and 2.5 for magnitude 8.5 earthquakes. For screening applications, if the static factor of safety against slope failure is greater than these values for the given earthquake magnitudes, the embankment or slope can be classed as minimally deformable and non-hazardous, and the screening analysis proceeds to the next step.

If the static factor of safety is less than the above limits, a dynamic deformation analysis should be conducted. As noted above, expert assistance will usually be required to conduct this analysis. The result of the analysis will be a tabulation or contour plot of predicted slope displacements at various points on and within the slope or embankment. Very little guidance is available from analysis or case history observations on amounts of ground displacement bridge structures can withstand without unacceptable distress to the foundation or bridge structure. Such thresholds are a function of several factors including bridge strength, foundation configuration, and the distribution of ground displacements acting on the structure. As noted in the introduction to Section 5, common highway bridges can generally withstand up to 100 mm of ground displacement without significant distress to abutments or foundation elements. Using the 100 mm displacement as a threshold for screening yields the following criterion. If the predicted slope deformations at points of intersection with the bridge structure exceed 100 mm, the bridge site should be classed as potentially hazardous and prioritized for further investigation and possible remediation. If predicted slope displacements are less than 100 mm, the site can be classed immune to local slope deformation, and the screening evaluation proceeds to the next step (Figure 2.1).

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5.3 Lateral Spread Displacement

If steep slopes and embankments are stable against slope failure and excess deformation, the next mode of ground displacement to be evaluated is liquefaction-induced lateral spread. This type of failure induces lateral displacement of natural or filled ground down gentle slopes or toward free faces such as incised river and stream channels. Lateral spread displacements generate lateral earth pressures that press against abutments and bridge foundation and may fracture and displace these elements. As noted in Section 1, lateral spread has been the most common cause of liquefaction-induced bridge damage during past earthquakes. Nearly all of this damage was due to displacement of flood plain or fill deposits toward river channels or other incised water bodies.

For routine analyses such as hazard screening, the empirical procedure of Bartlett and Youd (1995) has been widely used for estimating lateral ground displacements. This procedure is easy to apply and only requires information developed during typical high-quality foundation investigations. From a multiple linear regression (MLR) analysis of lateral spread case history data, Bartlett and Youd (1995) developed two empirical equations for predicting lateral ground displacement, one for free faces, such as incised river channels and the other for gently sloping ground conditions.

For free-face conditions:

$$\begin{aligned} \mathrm{LOG}\ D_H = {} & -16.3658 + 1.1782\ M - 0.9275\ \mathrm{LOG}\ R - 0.0133\ R + 0.6572\ \mathrm{LOG}\ W \\ & + 0.3483\ \mathrm{LOG}\ T_{15} + 4.5270\ \mathrm{LOG}\ (100 - F_{15}) - 0.9224\ D50_{15} \end{aligned} \tag{5.1a}$$

and for ground slope conditions:

$$\begin{aligned} \mathrm{LOG}\ D_H = {} & -15.7870 + 1.1782\ M - 0.9275\ \mathrm{LOG}\ R - 0.0133\ R + 0.4293\ \mathrm{LOG}\ S \\ & + 0.3483\ \mathrm{LOG}\ T_{15} + 4.5270\ \mathrm{LOG}\ (100 - F_{15}) - 0.9224\ D50_{15} \end{aligned} \tag{5.1b}$$

where D_H is the estimated lateral ground displacement, in meters; M is the estimated moment magnitude of the earthquake; R is horizontal distance from the seismic energy source, in kilometers; T_{15} is the cumulative thickness in meters of saturated granular layers with corrected blow counts, (N1)60, less than 15; F_{15} is the average fines content (fraction of sediment sample passing a No. 200 sieve) for granular layers included in T_{15}, in percent; $D50_{15}$ is the average mean grain size in granular layers included in T_{15}, in millimeters; S is the ground slope, in percent; and W is the free-face ratio defined as the height (H) of the free face divided by the distance (L) from the base of the free face to the point in question, in percent. Because of the empirical nature of Equations 5.1a and 5.1b, these relationships are valid only for the limited range of values for each independent variable incorporated in the compiled data base. The ranges for these limiting values are listed in Table 5-1. Extrapolation beyond the limits will lead to uncertain results.

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Table 5-1 Ranges of Values for Independent Variables for Which Lateral Spread Equations Are Verified by Case-History Observations (after Bartlett and Youd, 1992)

Input Factor	Range of Values in Case History Database
Magnitude	$6.0 < M < 8.0$
Free-Face Ratio	$1.0\% < W < 20\%$
Ground Slope	$0.1\% < S < 6\%$
Thickness of Loose Layer	$0.3\ m < T_{15} < 12\ m$
Fines Content	$0\% < F_{15} < 50\%$
Mean Grain Size	$0.1\ mm < D50_{15} < 1\ mm$
Depth to Bottom of Section	Depth to Bottom of Liquefied Zone < 15 m

Bartlett and Youd (1995) note that there is some statistical uncertainty in D_H calculated from Equations 5.1a and 5.1b, even when the variables are with in the ranges noted in Table 5-1. Doubling the calculated D_H, however, will yield estimates with a high probability of not being exceeded. Thus for screening applications, calculated values of D_H should be doubled to assure adequate conservatism.

5.3.1 Data Required

The data required to apply the Bartlett and Youd procedure is that necessary to define each of the independent variables in the equations. Requirements for each variable are briefly discussed below. More guidance, detailed explanations, and example calculations for selecting values for each variable are given by Youd (1993b).

Earthquake Magnitude, M. The moment magnitude, M, is specified for calculation of D_H from Equations 5.1a and 5.1b. This same magnitude is generally specified for calculation of liquefaction resistance as noted in Section 4.3.1. Magnitudes estimated from other scales may be substituted, however, within the limits noted in Section 4.3.2.2.

Seismic Source Distance, R. The seismic source distance, R, is defined as the horizontal distance, in kilometers, from the site in question to the nearest point on a surface projection of the seismic source zone. For many localities in the western U.S., the distance from known active faults may be used as an estimate for R. For regions where seismicity is associated with broad source zones rather than discrete faults, the distance R is measured from the nearest point on or within the source zone boundary. Minimal values for R for use with either faults or source zones are listed in Table 5-2. For earthquakes with magnitudes less than 6, epicentral distances provide adequate estimates for R. For magnitudes greater than 6 a single point--the epicenter--is not adequate to represent a large fault

TABLE 5-2 Minimum Values of R for Use in Equations 5.1 (after Bartlett and Youd, 1992)

Magnitude M_w	Minimum Value of R km
6.0	0.5
6.5	1
7.0	5
7.5	10
8.0	20-30

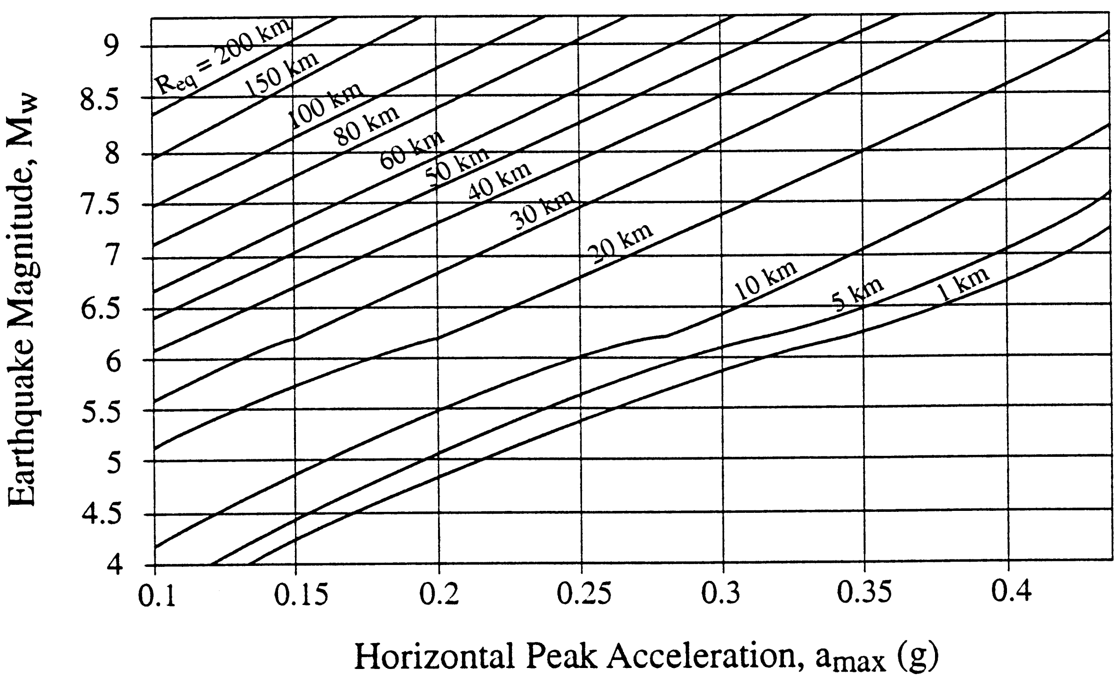

Figure 5-2 Curves for Determining Equivalent Source Distance, R_{eq}, From Magnitude and Estimated a_{max} (After Bartlett and Youd, 1992)

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rupture zone. Thus distance to the nearest bound on a surface projection of the source zone should be used rather than epicentral distance for large events.

For earthquakes in the eastern U.S. or for sites underlain by soft soils where amplification of ground motions may be large, an equivalent distance, R_{eq}, should be used in place of R. R_{eq} is estimated from M and a_{max} as follows: An estimated a_{max} is developed for the site based on local ground response analyses or use of amplification factors or ratios as described in Section 4.3.2.2. The a_{max} determined, however, must be a mean-expected value rather than a more conservative value, such as an a_{max} with mean plus one standard deviation probability of occurrence, as is often used for conservative engineering design. Use of a_{max} values that are more conservative than the mean will yield overly conservative estimates of D_H. The mean a_{max} is then plotted against magnitude on Figure 5-2, and R_{eq} is interpolated from the curves on the plot. That R_{eq} is then used in Equations 5.1a or 5.1b to estimate D_H. The R_{eq} procedure is only valid for a_{max} less than 0.4 g and magnitudes less than 8. Extrapolation beyond these values will lead to uncertain predictions.

Thickness of loose granular sediment, T_{15}. T_{15} is an estimate of the thickness of loose granular sediment at a liquefiable site. Bartlett and Youd (1995) define T_{15} as the thickness of liquefiable granular sediments in a soil profile characterized by an $(N_1)_{60}$ equal to or less than 15. Where there are distinct lithologic changes in granular sediments, such as distinct layers of clean and silty sand, separate displacement calculations should be made for each layer and the displacements for the all the layers summed to provide the final estimate of D_H.

Average fines content, F_{15}. F_{15} is defined as the average fines content (i.e., the percent of material passing a No. 200 sieve) from all samples taken from a layer characterized by a thickness, T_{15}.

Average mean-grain size, $D50_{15}$. Bartlett and Youd (1995) characterized the coarseness of a layer by the parameter, $D50_{15}$, which is defined as the average mean-grain size of materials included in layer T_{15}. This variable is determined by averaging mean-grain sizes measured in each layer characterized by a thickness T_{15}.

Ground Slope, S. The ground slope, S, corresponds to the standard engineering definition of slope, or the rise of elevation over the horizontal run of the slope, expressed in percent. S is the average ground slope and generally disregards minor topographic rises and depressions.

Free-Face Ratio, W. The free-face ratio is defined as the height, H, of a free face divided by the distance, L, from the point in question to the base of the free face:

$$W = (H/L)(100), \text{ in percent} \tag{5.2}$$

The height, H, is commonly determined by subtracting the elevation at the base of a depression, such as a river bottom or the toe of a fill, from the elevation at the crest of the bank or fill. The distance, L, is measured from the base or toe of the free face to the locality in question.

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5.3.2 Analysis and Classification

Estimates of D_H are calculated from Equations 5.1a and 5.1b, with input variables determined as noted in the previous section. Because of statistical uncertainty, the calculated D_H should be doubled to provide adequate conservatism for screening applications. If the conservatively estimated displacement is less than 100 mm, the site can be classed as not susceptible to significant lateral ground displacement. If the estimated displacements are 100 mm or greater, the bridge should be classed as possibly hazardous and high priority for further investigation. If insufficient data are available to apply Equations 5.1a or 5.1b or if the available data fall outside the limiting ranges in Table 5-1, the bridge should also be classified as possibly hazardous and prioritized for further investigation.

5.4 Analysis of Ground Settlement

Earthquake shaking is an effective compactor of granular soils--dry, moist, or saturated-- leading to vertical ground displacement or ground settlement. Laboratory studies show that compaction is enhanced by liquefaction of granular materials (Lee and Albaisa, 1974). Tokimatsu and Seed (1987) compiled data from laboratory tests and case histories of earthquake-induced ground settlements and developed a simplified empirical method for estimating settlement. That procedure is recommended here for estimating liquefaction-induced settlement for hazard screening purposes. Ishihara and Yoshimine (1992) also developed a technique for estimating ground settlement; the predicted results, however, are usually comparable to those determined from the Tokimatsu and Seed procedure.

The premise of the Tokimatsu and Seed procedure is that earthquake shaking generates cyclic shear strains that compact granular soils, causing volumetric strain. Where drainage cannot occur rapidly, the tendency to compact also generates transient pore water pressures that prevent immediate decrease in volume. However, as pore pressures dissipate, the layer consolidates, producing volumetric strain and ground settlement. Tokimatsu and Seed show that the induced volumetric strains are primarily a function of amplitude of the cyclic shear strains generated by the earthquake and the initial relative density of the sand. The cyclic shear strains are a function of the cyclic stress ratio (CSR), relative density, and earthquake magnitude. They corrected the cyclic stress ratio for magnitude by dividing the CSR by an appropriate magnitude scaling factor from Table 4-4. Relative density was estimated directly from corrected penetration resistance, $(N_1)_{60}$. Figure 5-3 is a synthesis diagram developed by Tokimatsu and Seed from available laboratory tests data and field observations of earthquake-induced settlements in clean sands. They recommend use of this diagram to estimate volumetric strains from magnitude-corrected CSR and $(N_1)_{60}$ determined for the particular site and sand layer. That volumetric strain is then multiplied by the layer thickness, assuming one-dimensional consolidation, to estimate the change in thickness. The changes in thickness from all layers at the site are then summed to provide an estimate of the total settlement.

The Tokimatsu and Seed procedure was developed for clean sands. For screening purposes the procedure may be applied to silty sands by converting the blow count to an equivalent clean sand value, $(N_1)_{60cs}$, using Equations 4.7 to 4.9.

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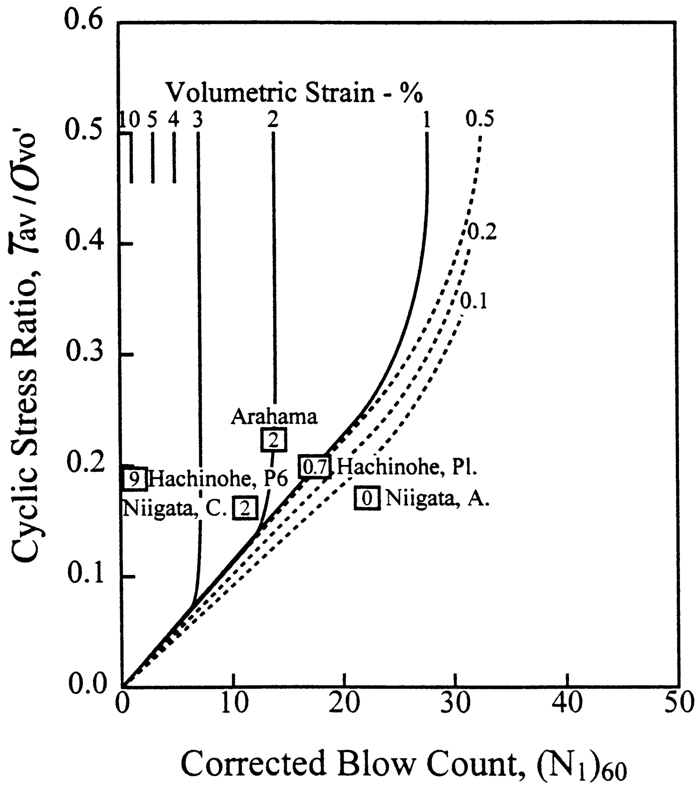

Figure 5-3 Curves for Estimating Volumetric Strain at Liquefiable Sites (After Tokimatsu and Seed, 1987)

5.4.1 Data Required

The data required to estimate ground settlement by the Tokimatsu and Seed (1987) procedure are the same as those required to apply the simplified procedure for calculating liquefaction resistance (Section 4.3.2), plus estimates of layer thicknesses. Layer thicknesses can be estimated from borehole or CPT logs or interpolated stratigraphic cross-sections.

5.4.2 Analysis and Classification

The analysis of ground settlements consists of estimating CSR for the site and $(N_1)_{60cs}$ values for each granular layer beneath the site. These factors are then used with Figure 5-3 to estimate volumetric strains for each layer. The increment of ground settlement for each layer is calculated by multiplying volumetric strain by the layer thickness. The total settlement is then calculated by summing up the incremental settlements for all of the layers.

Design criteria for most structures on shallow foundations limit ground settlements to 25 mm or less. Structures on deep foundations can generally withstand settlements of 100 mm or more, so long as

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structural loads are transferred downward to competent strata and the foundation itself does settle not more than 25 mm. These limits are applicable to settlements caused by liquefaction as well as those caused by static consolidation of compressible layers. Based on this general guidance, the following conservative criteria are recommended for hazard screening for highway bridges: (1) Bridges supported on shallow foundations may be classed as nonvulnerable to settlement damage if estimated ground settlements are 25 mm or less. In this instance, the screening analysis proceeds to the final step of assessing foundation bearing capacity (Section 5.5). If predicted settlements are greater than 25 mm, the structure should be classed as possibly hazardous and prioritized for further investigation. (2) For structures supported by deep foundations, if predicted settlements are 100 mm or less, the structure may be classed as nonvulnerable to ground settlement, and the load capacity of the foundation should evaluated (Section 5.5). If predicted settlements are greater than 100 mm, the structure should be classed as possibly hazardous and prioritized for further investigation.

5.5 Bearing Capacity Analysis

If liquefaction-induced ground deformations and ground settlements are tolerable, the remaining possible liquefaction-induced hazard to bridges is loss of foundation bearing strength. Loss of bearing strength could lead to penetration of shallow or deep foundations into the liquefied sediment or to lateral displacement or buckling of piles as a consequence of reduced lateral resistance in liquefied soil layers.

5.5.1 Data Required

A standard bearing capacity analysis may be used to assess bearing capacity for shallow foundations, with residual strengths assigned to liquefiable layers. For calculation of axial load capacity for deep foundations, liquefiable layers are commonly assumed to have negligible strength. Lateral load resistance of deep foundations is usually estimated by assigning a multiplier ranging from 0.1 to 0.3, depending on relative density, to lateral resistance values calculated for nonliquefied layers.

5.5.2 Analysis and Classification

If the load capacity analyses indicate and adequate factor of safety (say 1.5 or greater) against each of the various modes of failure, the site may be classed as non-hazardous and immune to detrimental effects of liquefaction even though liquefaction of some subsurface layers may occur. At this juncture of the investigation, all of the possible detrimental effects of liquefaction have been considered and determined to be nondamaging to the Bridge. Conversely, if the analysis indicates a marginal factor of safety (less than 1.5), unacceptable foundation displacements may occur during an earthquake and further investigation should be recommended for the site to confirm the hazard and remedial measures designed if the hazard is confirmed.

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