PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

# Recommended Design Practice for Pile Foundations in Laterally Spreading Ground

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PEER 2011/04
JUNE 2011

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PEER Report 2011/04
Pacific Earthquake Engineering Research Center
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June 2011

## ABSTRACT

This report presents recommended procedures and practices for the design and performance evaluation of pile foundations for bridges in areas subject to lateral spreading hazards. The past decade of research has produced numerous insights into the behavior and performance of pile foundations and bridges impacted by liquefaction and lateral spreading. The purpose of this report is to develop a set of recommended procedures and practices for analysis and design that are based on a synthesis of research findings when supporting research is available, and on the professional opinions of the Principal Investigators when supporting research is lacking. The scope of the report is to summarize those recommendations in a concise document that provides references to supporting materials and/or identifies areas where supporting materials are lacking.

This report is intended for engineers who are familiar with geotechnical and structural design practice for static and seismic loading of bridges. The pile foundations covered by these recommendations include all piles included in the Caltrans Standard Specifications (Caltrans 2006). The term "lateral spreading" in this document refers to global movements of soil due to liquefaction of underlying cohesionless soil and hence includes cases that might be described as flow liquefaction (e.g., slope instability). The recommendations presented herein focus on the effects of liquefaction and lateral spreading, and assume familiarity with the necessary background information.

# ACKNOWLEDGMENTS

This project was sponsored by the Pacific Earthquake Engineering Research Center's (PEER's) Program of Applied Earthquake Engineering Research of Lifelines Systems supported by the California Department of Transportation and the Pacific Gas and Electric Company. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect those of the sponsors.

The authors are greatly appreciative of the opportunity to work closely with Tom Shantz of Caltrans through this entire project. The authors also wish to express their gratitude to Steve Kramer of the University of Washington and Lelio Mejia of the URS, Corporation. Their reviews and suggestions for improvement to the report were a valuable contribution to the project.

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# 1 Introduction

This report presents recommended procedures and practices for the design and performance evaluation of pile foundations for bridges in areas subject to lateral spreading hazards. The past decade of research has produced numerous insights into the behavior and performance of pile foundations and bridges impacted by liquefaction and lateral spreading. The purpose of this report is to develop a set of recommended procedures and practices for analysis and design that are based on a synthesis of research findings when supporting research is available, and on the professional opinions of the Principal Investigators when supporting research is lacking. The scope of the report is to summarize those recommendations in a concise document that provides references to supporting materials and/or identifies areas where supporting materials are lacking.

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## 1.1 LIQUEFACTION AND NONLIQUEFACTION CASES

The steps for design or performance evaluation of pile foundations for a bridge include:

- Designing the piles or evaluating their performance for the inertia loading that would occur in the absence of liquefaction.

- Estimating the potential for liquefaction, and quantifying any expected lateral and vertical ground displacements.
- Designing the piles or evaluating their performance for the lateral spreading and inertia demands that would occur if liquefaction is triggered.

## 1.2 LOCAL AND GLOBAL EFFECTS OF LIQUEFACTION

The effects of liquefaction on a bridge system are evaluated first for the local subsystems. These subsystems may include pile groups in lateral spreads and piled abutments in approach embankments (see Figure 1.1). In both cases, the restraining or "pinning" effects of the piles and bridge superstructure may reduce the lateral spreading displacements of the soil near the piles, which in turn reduces the demands imposed on the piles. As will be discussed in Sections 4 and 5, this interaction between the pile foundation and lateral spreading soil can be accounted for using different analysis approaches for the two subsystems shown in Figure 1.1. The effects of liquefaction on the global response of a bridge are discussed in Section 6.

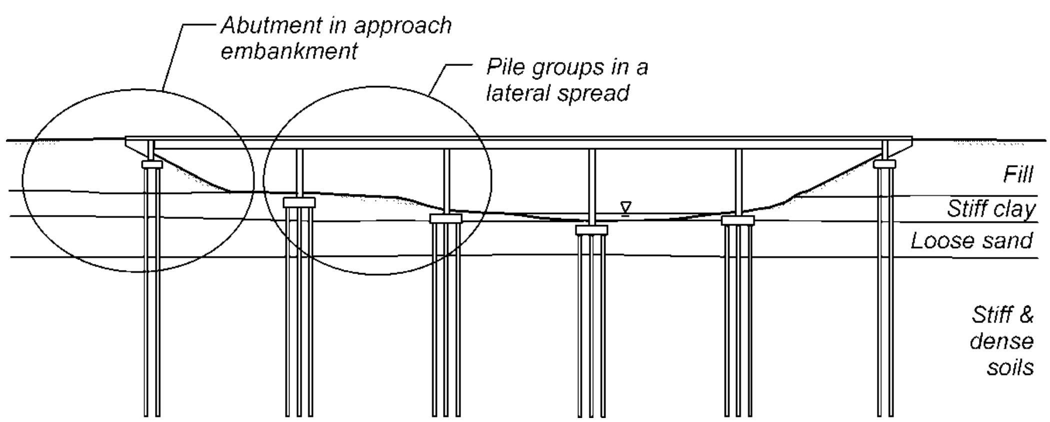



**Figure 1.1 Schematic of bridge showing two local subsystems for analysis: (1) pile groups in laterally spreading ground, and (2) pile-supported abutment in approach embankment.**

## 1.3 ANALYSIS METHODS FOR LIQUIFACTION EFFECTS

The analysis of pile foundations for liquefaction effects may include nonlinear equivalent static analyses (ESA) or nonlinear dynamic analyses.

Nonlinear analyses, whether equivalent static or dynamic, are required for pile foundations in liquefied ground because the soil-pile-structure interaction is highly nonlinear in such conditions. Equivalent static nonlinear analyses may be performed using equivalent-linear methods that iterate to obtain strain-compatible properties for the soils and piles. Linear elastic analyses that do not account for strain-compatibility of the soil and pile properties are often used to analyze piles in nonliquefied ground, but are not appropriate for analyzing piles in liquefied ground.

# 2 Pile Design for Nonliquefaction Case

The capacity and stiffness of pile foundations under axial and lateral inertia loading can be estimated with a progressively increasing level of effort as warranted. The general hierarchy of approaches is as follows:

- Presumptive values for capacity and stiffness based on general soil conditions, pile type, and local experience.
- Analyses that either use estimated or measured soil strength and stiffness parameters, or direct correlations to results of *in situ* tests (e.g., SPT or CPT).
- Analyses of dynamic monitoring records such those compiled as during pile driving or pile re-strikes (e.g., CASE method or CAPWAP analyses).
- Pile load tests.

The uncertainty in the pile foundation's estimated capacity and stiffness will depend on the approach taken, as well as other factors. Two issues of concern—the required margin of safety on computed capacity (accounted for by either load and resistance factors or a factor of safety), and the uncertainty in computed stiffness—can generally be reduced with increasing level of effort. For axial and/or lateral stiffness, sensitivity analyses should be performed to check that the design—and the resulting effect on the response of the superstructure—is satisfactory for an appropriate level of uncertainty in the estimated stiffness.

A beam on nonlinear Winkler foundation (BNWF) approach can be used for both the nonliquefaction and liquefaction cases. Assembly of a BNWF model requires selection of lateral (*p-y*), axial (*t-z*), and tip bearing (*q-z*) spring parameters for the piles and pile cap. The determination of the stiffness, capacity, and nonlinear shape of these spring elements may be estimated using various levels of effort, as previously described. These estimates may require further adjustments for pile group effects and pile set up with time. Furthermore, the spring parameters will be different for nonliquefaction and liquefaction cases. For seismic design it is important that this step identify best estimates and some measure of uncertainty for these different spring parameters (e.g., upper- and lower-bound design values) because it is not always evident whether a conservative design will correspond to under-estimating or over-estimating the spring parameters.

The inertia loads from the bridge superstructure may be estimated using different analysis methods, including estimates based on the local subsystem and/or a global analysis of the entire bridge system, as described by Aviram et al. (2008). The analyses should include both transverse and longitudinal shaking. The analysis of the pile foundation may be directly coupled to, or separated from, the analysis of the superstructure. These analyses involve specification of the design linear-elastic acceleration response spectra (ARS) for the site, which will correspond to the nonliquefaction case. Lateral loads and overturning moments imposed on the foundation by the inertia of the bridge superstructure are generally limited by the lateral strength (with allowance for over-strength) of the supporting columns or piers.

The pile foundation is then analyzed for the lateral loads and overturning moments that are produced by the inertia of the bridge superstructure. Piles are also checked for their maximum uplift and compressive axial loads. Uplift or plunging of the outer piles under the imposed overturning moments can contribute to the cyclic accumulation of permanent displacements and rotations at the pile cap. Although these should be evaluated as well, they may also contribute to energy dissipation.

Kinematic loading from ground deformation is generally not included in this analysis provided that both transient and permanent ground deformations are expected to be insignificant. Special analyses are required in cases where ground deformations may be significant [e.g., in soft clays or liquefiable soils, or when a bridge crosses a fault rupture zone (Goel and Chopra 2008)].

If an estimate of the pile foundation stiffness is needed for estimating the superstructure's dynamic response, then such that a round of iteration may be required between these two steps. It is generally preferable to have the piles remain elastic because subsurface damage is difficult to assess or repair, but there are cases where allowing a limited amount of yielding in the piles can provide significant economy.

# 3 Estimating Lateral Spreading Displacements

The evaluation of potential liquefaction-induced ground deformations involves the following major steps.

- Site characterization and evaluation of liquefaction susceptibility.
- Evaluation of the potential for liquefaction triggering in susceptible soils.
- Estimation of expected lateral and vertical ground displacements or instability of embankments and slopes due to liquefaction.

## 3.1 SITE CHARACTERIZATION

The susceptibility of soils to liquefaction during earthquakes varies with the nature of the deposit and its age (Youd and Perkins 1987). The most susceptible soils are recent fills and Holocene deposits of alluvial, fluvial, marine, deltaic, and wind-blown sediments that include significant amounts of cohesionless soils (gravels, sands, and non-plastic or very low plasticity silts).

The site characterization should begin with an interpretation of the local geology, aided by a review of aerial photographs and historical records. Knowledge of the expected geologic conditions can be used to guide the planning of site specific explorations. This effort should include a search for any historical or geologic evidence of prior liquefaction or ground failure at the site.

Site explorations should include an appropriate combination of SPT borings and CPT soundings and laboratory tests. All SPT tests should adhere to detailed specifications in ASTM D-6066 (ASTM 2008) when the data will be used for liquefaction evaluations (Youd et al. 2001).

Detailed subsurface cross sections should then be developed that show the *in situ* test data (e.g., penetration resistances versus depth) and the interpreted primary geologic strata. A key factor is to identify the spatial extent and continuity of soil strata that are susceptible to liquefaction. The quality of the site exploration work and the geologic interpretation are often the most important part of any liquefaction evaluation. Therefore, such analyses should be performed under close supervision by personnel familiar with liquefaction effects.

## 3.2 TRIGGERING OF LIQUEFACTION

The potential for triggering liquefaction in cohesionless soils can be evaluated using the Seed-Idriss (1971) Simplified Procedure for estimating earthquake-induced cyclic stress ratios (CSR) and various *in situ* test-based liquefaction correlations for estimating the cyclic resistance ratio (CRR) of cohesionless soils. The most commonly used SPT- and CPT-based liquefaction correlations for the past ten years have been documented in Youd et al. (2001). Although these correlations have since been updated by a number of investigators [including Cetin et al. (2004), Idriss and Boulanger (2006), and Moss et al. (2006)] these updated correlations have some significant differences among each other that are still being evaluated in the engineering community; the correlations in Youd et al. (2001) may be used until consensus is reached on the newer methods.

The SPT- and CPT-based liquefaction correlations are considered applicable to nonplastic and very low-plasticity silts, but not to plastic silts or clays. According to Boulanger and Idriss (2006): (1) the seismic behavior of silts and clays [i.e., soils having greater than 50% fines content per the Unified Soil Classification System (ASTM 2008)] with a PI ≥ 7 can be evaluated using procedures appropriate for cohesive (or clay-like) sediments; and (2) fine-grained soils that fail to meet this criterion should be evaluated for liquefaction potential using SPT- or CPT-based liquefaction correlations, unless a detailed program of laboratory testing is performed to evaluate the soil behavior and potentially justify the use of greater cyclic resistances. Bray and Sancio (2006) showed that fine-grained soils with PI values between 7 and 20 and with high ratios of water content to Liquid Limit can have cyclic loading responses that are similar to those of saturated sands. While they used the term "liquefaction" to describe the behavior of these fine grained soils, they also recommended that the best way to determine the cyclic strength of such soils is by laboratory testing. The preceding guidance may be extended to silty or clayey sands when the fines fraction represents the load-carrying matrix of the soil. This transition may be estimated to occur at a fines fraction of roughly 35%, although a detailed program of laboratory and *in situ* testing may be needed to justify the use of this criterion.

## 3.3 GROUND DEFORMATIONS DUE TO LIQUEFACTION

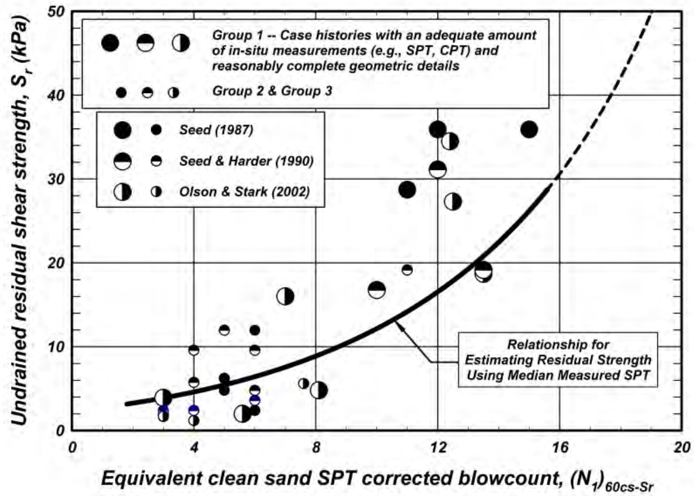
Ground deformations as a result of liquefaction may develop in different ways for which different analysis methods are used. Three specific cases are considered.

- Instability of a slope or embankment due to shear strength loss in liquefied zones.
- Lateral spreading of level or mildly sloping ground (e.g., ground oscillation).
- Settlements due to one-dimensional reconsolidation of liquefied soils.

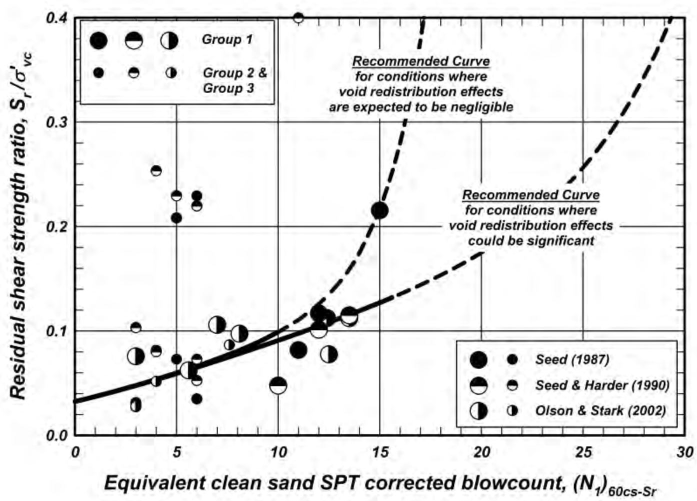
The first step should identify whether the slope would be stable under static driving stresses after liquefaction or whether a flow-failure would develop. Post-liquefaction stability can be evaluated using limit equilibrium slope stability software, other types of slope stability analysis methods, and nonlinear finite element or finite difference analyses. Residual shear strengths should be assigned to those cohesionless soils that have a factor of safety less than or equal to 1.1 against triggering of liquefaction ($FS_{liq}$). Soils with a factor of safety greater than or equal to 1.3 can be assigned their full drained shear strength, subject to the limitation that the soil is dense of its critical state (i.e., drained strength is less than undrained strength). Conceptually, a soil with very low relative density and/or very high confining stress may be loose in its critical state. Here, the monotonic undrained strength should be used instead of the drained strength for soils, even though the $FS_{liq}$ may be greater than 1.3. Only very loose soils have the potential to be loose of critical under the stress conditions common to bridge applications, and since such soils have low cyclic strengths, this latter case is not likely to be encountered in most practical situations. Soils with a factor of safety between 1.1 and 1.3 can be assigned shear strengths based on linear interpolation between the above recommended shear strengths at $FS_{liq}$ = 1.1 and 1.3.

The residual shear strength of a liquefied soil can be estimated from empirical correlations to SPT or CPT data. These correlations are approximate at best and do not explicitly account for a variety of phenomena like void redistribution (e.g., the formation of water films beneath lower permeability soil interlayers), which are currently not possible to explicitly quantify by calculation in practice. The earliest correlations had related SPT data directly to residual shear strength, such as illustrated by data and correlation in Figure 3.1(a). More recent correlations have related SPT and CPT data to normalized residual shear strength ratios (residual shear strength normalized by effective overburden stress), as shown by the correlation in Figure 3.1(b). This latter correlation includes a recommended distinction between cases where void redistribution effects could be significant versus cases where they are expected to be negligible (Idriss and Boulanger 2007). Note that the curve in Figure 3.1(a) is approximately equal to the lower-third of the range given by Seed and Harder (1990) for blow counts less than about 15, and that the lower curve in Figure 3.1(b) is approximately equal to the curve by Olson and Stark (2002) for sands having blow counts less than about 12. The curves in Figure 3.1 provide guidance on the unavoidable task in practice of having to extrapolate these relationships to higher blow counts.

The residual strengths obtained from correlations to $S_r$ will be greater than the strengths obtained from correlations to $S_r/\sigma'_{vc}$ when the effective overburden stress is relatively small, and vice-versa when the effective overburden stress is large. This systematic difference between approaches is illustrated in Figure 3.2 using the two correlations by Idriss and Boulanger (2007; Figure 3.1). Both approaches are unavoidably approximate and the choice between them is subject to debate. Designers may have a preference for either relationship, but they should evaluate how the alternative approach would affect expected performance. Life safety evaluations should not be allowed to depend on the distinction between these relationships.




(a)




(b)

**Figure 3.1 Correlations between equivalent clean sand corrected SPT blow count: (a) residual shear strength of liquefied sand, and (b) normalized residual shear strength ratio of liquefied sand (Idriss and Boulanger 2007).**

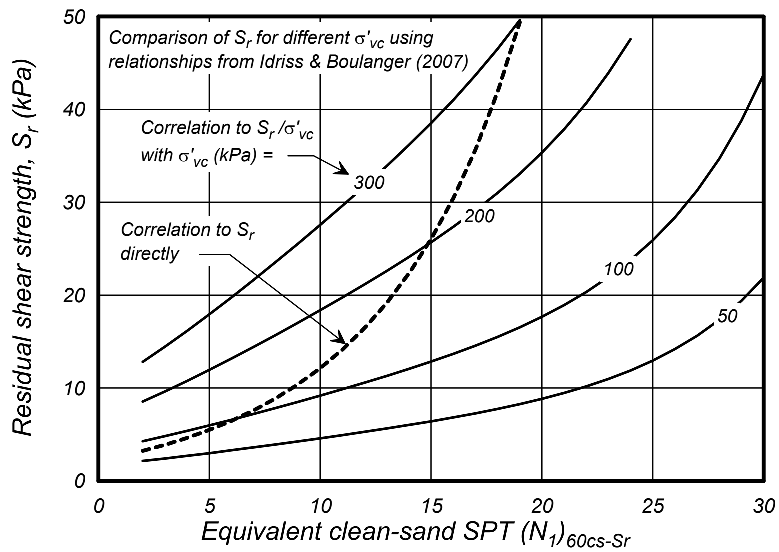



**Figure 3.2 Comparison of residual shear strengths obtained from $S_r$ and $S_r/\sigma'_{vc}$ correlations at different effective overburden stresses.**

For cases where the slope is stable under static driving shear stresses after liquefaction, lateral spreading may still occur as the driving stresses transiently exceed the available strength or as the cyclic ratcheting behavior of the soils results in a progressive accumulation of permanent strains. Free-field lateral spreading displacements may be estimated in a number of ways, including the following:

- Integration of shear strain profiles estimated in conjunction with SPT- and CPT-based liquefaction analyses. The maximum potential shear strains may be estimated using existing relationships, such as the three SPT-based correlations compared in Figure 3.3. The computed ground surface displacement is known as the "Lateral Displacement Index," or simply LDI (Zhang et al. 2004). This method does not account for two- or three-dimensional effects; therefore, the results for individual borings can be misleading on their own. A benefit of the integration of strain method is that an estimate of the soil displacement profile is obtained over the depth of the foundation, which can in turn be used as an analysis input.

- Empirical relationships based on regression against case history data and broad indices of the seismic loading and site characteristics. Models by Youd et al. (2002), Bardet et al. (2002), and Rauch and Martin (2000) are examples. These models have little physical basis and cannot be extended to approach embankments (outside the empirical dataset).

- Newmark sliding block analyses, including both regression models and methods that require integrating site-specific earthquake acceleration time series. Displacements estimated using Newmark sliding block methods can depend heavily on the residual strength of the liquefied zones, which in turn contains significant uncertainty (e.g., see the dispersion in the normalized residual strength values in Figure 3.1). Nevertheless, they can be useful for evaluating cases with isolated pockets of liquefiable material, wherein shear stresses can be transmitted to nonliquefied material, and cases where medium dense soils provide sufficient residual strength to limit deformations to reasonable levels. Furthermore, Newmark methods can provide a rational approach for quantifying the beneficial reduction of lateral spreading demands caused by pinning forces that tend to increase the yield acceleration of the spreading mass.
- Nonlinear dynamic numerical analyses, including one-dimensional shear beam analyses or two-dimensional continuum analyses. One-dimensional shear beam analyses can be performed with and without horizontal static shear stresses (to represent the influence of a sloping ground surface), and thus provide insight into the effects of liquefaction on both the dynamic site response and the permanent ground deformations. One- and two-dimensional soil models can also be connected to models of the bridge superstructure, such that the soil-structure interaction effects are directly included. These types of analyses require special expertise, but are becoming increasingly common in engineering practice on important projects.

There is considerable uncertainty in estimating lateral spreading displacement based on current methods, with the overall uncertainty including contributions from the uncertainties in ground motion, site characterization, spatial heterogeneities, soil property estimation, and approximations inherent to each analysis method. When possible, a number of independent estimates of ground displacement should be made to quantify a range of anticipated displacements. Expected performance of a bridge may be based on a designer's best estimates of lateral spreading displacements, but safety against collapse should be ensured for an upper range estimate of lateral spreading displacements (e.g., not less than twice the expected lateral spreading displacement).

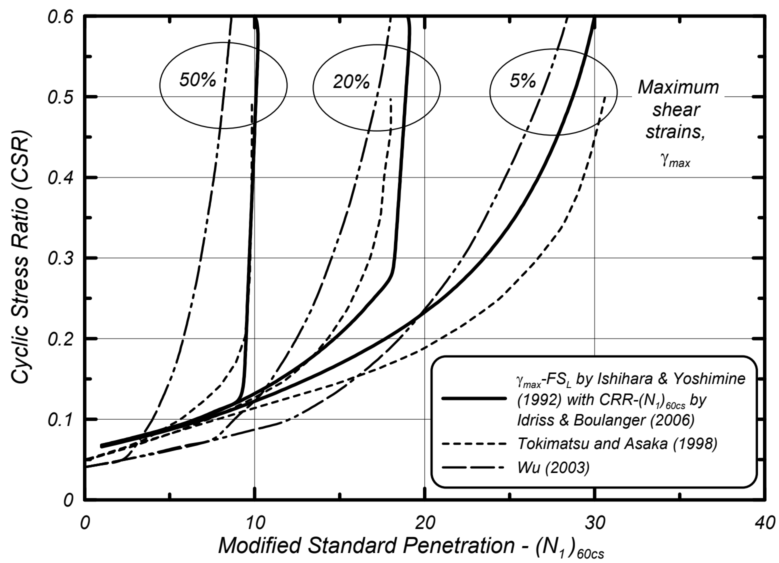



**Figure 3.3 Three relationships between cyclic stress ratio, SPT $(N_1)_{60cs}$, and maximum shear strains for M=7.5 and $\sigma'_{vc}$ =100 kPa.**

The distribution of lateral spreading displacements with depth must also be estimated. For cases where the piles are laterally stiff and strong enough to provide satisfactory performance, the pile head displacement and maximum bending moment are often relatively insensitive to the assumed soil displacement profile shape, such that a simplified profile with linear variations across layers can be assumed for design. For more flexible pile foundations, the bending moment and curvature demands versus depth can be controlled by the assumed shape of the free-field soil displacement profile, such that additional soil displacement profile shapes may need to be considered. The choice of alternative soil displacement profile shapes may be guided by any trends in the SPT or CPT penetration resistances (i.e., strains being larger when the soil is looser), the presence of a low-permeability crust layer (strains being larger immediately beneath the crust), and the designer's judgment.

The timing of lateral spreading displacements relative to the interval of strong ground shaking can be affected by numerous factors, including those that affect the diffusion of earthquake-induced excess pore water pressures. For design purposes, it is prudent to assume that enough lateral spreading displacement occurs during strong shaking to require that the lateral spreading displacements and a fraction of the inertia demands be considered as additive. Additional discussion on the combination of inertial and kinematic loading is provided in Section 4.2.

The transient lurching of liquefied ground during strong shaking can produce significant kinematic loading in the direction transverse to the primary direction of lateral spreading and at level sites that are far from a free face and not prone to lateral spreading. Lateral displacements caused by ground lurching in the direction perpendicular to the primary direction of permanent lateral spreading may be estimated as about 20% of the estimated permanent lateral spreading displacements, according to the recommendations of Tokimatsu and Asaka (1998).

Ground surface settlement can be caused by both: (1) settlements due to reconsolidation of liquefied soils, and (2) vertical displacements due to shear deformation of the soil, such as may occur if the ground deforms toward a free face during lateral spreading. The settlement due to reconsolidation of liquefied soil can be computed by integrating the profiles of potential vertical strains in conjunction with SPT- or CPT-based liquefaction evaluations. Relationships for estimating vertical reconsolidation strains are presented in Figure 3.4. The differences between these different relationships are generally small relative to the other uncertainties involved in predicting liquefaction induced settlements, such that the use of any one relationship is acceptable.

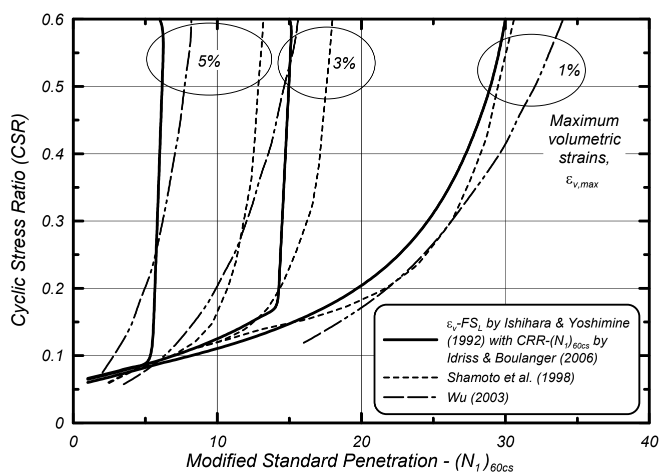



**Figure 3.4 Comparison of three relationships between cyclic stress ratio, SPT $(N_1)_{60cs}$, and maximum volumetric strains for M=7.5 and $\sigma'_{vc}$ = 100 kPa.**

# 4 Analysis of Piles in Response to Lateral Spreading

This section considers the analysis of a single pile or pile group for an individual bridge bent that is located within an area of lateral spreading away from the abutments (see Figure 1.1). The liquefaction susceptibility and potential for liquefaction-induced lateral and vertical ground displacements should have already been estimated.

The response of the piles to lateral spreading is best analyzed using a BNWF approach where the estimated free-field soil displacements are applied to the support ends of the *p-y* springs. Inertial loads are applied at the same time (see Figure 4.1); note that inertial loads can act in either direction.

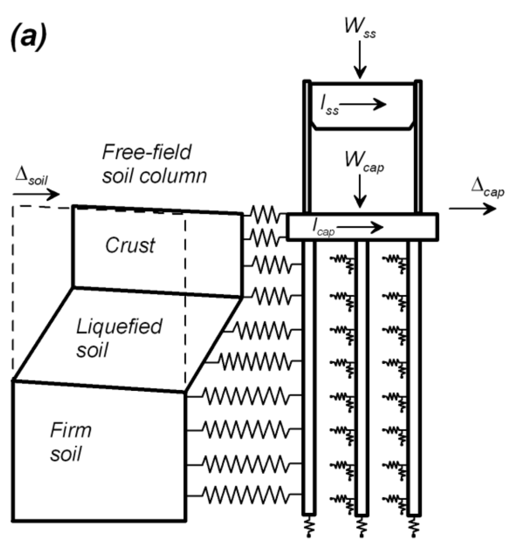



**Figure 4.1 Static BNWF analysis method with imposed soil displacements.**

The alternative to the BNWF analysis approach is to impose limit pressures to the pile foundation over the depth of lateral spreading, which is generally not recommended. The limit pressure approach can be overly conservative in some situations (e.g., Ashford and Juirnarongrit

2003; Brandenberg et al. 2007b), providing less insight than the method with imposed soil displacements and sometimes still requiring the inclusion of soil displacements beneath the zone of liquefaction. In situations where ground displacements are large and the pile foundation relatively strong, the two BNWF approaches can produce similar results.

## 4.1 BNWF MODEL FOR LIQUEFACTION CASE

### 4.1.1 *p-y* Behavior in Liquefied Sand

The influence of liquefaction on BNWF springs for sand can be approximately accounted for by applying scaling factors, or *p*-multipliers ($m_p$), to the *p-y* resistances (Table 4.1). Subgrade reactions have been observed to depend on the same factors that affect the cyclic loading response of liquefying soils (e.g., $D_R$, strain and strain history), plus the factors that affect the local variations of stress and strain around the piles (e.g., pile foundation flexibility, ground motions, and lateral spreading displacements) and the diffusion of pore water pressures between the far field and near field (e.g., permeability, pile diameter, relative velocities). The scaling factors shown in Table 4.1, for example, only account for the first order effects that also influence penetration resistance, primarily relative density ($D_R$). Figure 4.2 summarizes other published recommendations for *p*-multipliers. For cases where the free-field excess pore water pressure ratio ($r_u$) is expected to be less than 100%, a value for $m_p$ may be linearly interpolated between the values that are estimated for free-field $r_u$ values of 0 and 100% (e.g., Dobry et al. 1995). Another approach to compute a *p-y* relation for liquefied sand is to use the sand's estimated residual strength along with a relation appropriate for undrained behavior of clay (e.g., Matlock 1970). Available information is insufficient to determine whether either of these techniques is more accurate than the other.

**Table 4.1 *p*-multipliers, $m_p$, to account for liquefaction.**

| $(N_1)_{60\text{-}CS}$ | $m_p$ |
|---|---|
| <8 | 0.0 to 0.1 |
| 8-16 | 0.05 to 0.2 |
| 16-24 | 0.1 to 0.3 |
| >24 | 0.2 to 0.5 |

The actual *p-y* behavior during liquefaction and lateral spreading is much more complex, as illustrated by the subgrade reactions that have been back-calculated from experimental measurements (e.g., Wilson et al. 2000; Weaver et al. 2005; Tokimatsu et al. 2005; Rollins et al. 2005). In fact, the shape of *p-y* curves back-calculated from full-scale field tests (e.g., Weaver et al. 2005) and small-scale centrifuge experiments (e.g. Wilson et al. 2000) show the shape of *p-y* curves for liquefied soil to be concave upward, rather than concave downward like traditional curves. The cause of the upward concavity is dilatancy, or cyclic mobility behavior, of sand in undrained loading, and the shear strains that cause dilatancy can be imposed by the pile as it pushes through the liquefied sand or by free-field ground shaking. These complex effects cannot reasonably be captured in a static analysis, but can be captured in a dynamic analysis using a soil constitutive model that can capture dilatancy. Traditional, concave downward curves can produce reasonable predictions of pile group behavior, are easy to implement, and are numerically more stable than concave upward curves when lateral spreading displacements are imposed.

Performance evaluations or design for a bridge may use *p*-multipliers that are at the middle of the range recommended by Brandenberg (2005), as shown in Figure 4.2. Sensitivity of the expected foundation performance to factor of 2 increases or decreases in *p*-multipliers should also be evaluated. Expected pile performance can be insensitive to the *p*-multipliers when a strong nonliquefied crust spreads laterally against the bridge component. Similar data or guidance regarding *t-z* and *q-z* behavior is generally not available at this time, so in the absence of such data it seems reasonable that the *p*-multipliers may be assumed to characterize the effects of liquefaction on *t-z* and *q-z* behavior as well.

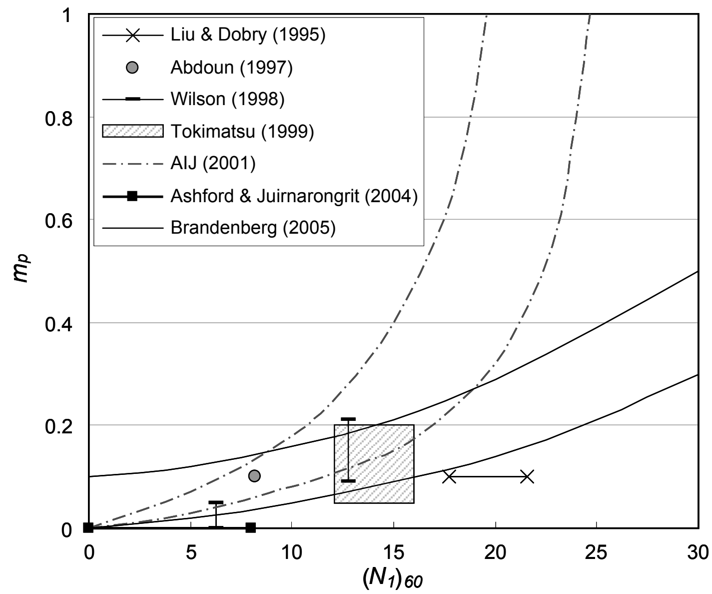



**Figure 4.2** ***p*-multiplier ($m_p$) versus clean sand equivalent corrected blow count, $(N_1)_{60cs}$, from a variety of studies.**

### 4.1.2 Loads from Nonliquefied Crusts

The ultimate lateral loads imposed by a nonliquefied crust against a bridge foundation consist of passive pressure on the upslope face of the pile cap or abutment backwall and frictional resistance along the pile cap sides and base (Brandenberg et al. 2005; 2007b). The lateral loads can be represented using *p-y* springs (e.g., Juirnarongrit and Ashford 2006), and both the capacity and shape of the *p-y* curves must be specified. Load transfer behavior when a nonliquefied crust spreads against a pile cap (or wall) is different than when a pile cap pushes into a nonliquefied soil profile, and the properties of the *p-y* springs must be selected to incorporate these important differences, as discussed below.

Passive pressures can be estimated using conventional earth pressure theories using a log-spiral failure mechanism for cases where vertical friction forces are mobilized at the soil-wall interface. However, the wall friction may be significantly lower when a crust spreads against a wall than when a wall is pushed into a nonliquefied soil profile because: (1) the spreading crust may settle due to extensional strains, cracking, and sand boil formation, and this settlement will somewhat negate the formation of upward frictional stresses on the back of the wall; and (2) the underlying liquefied sand provides a soft and weak boundary condition on the base of the deposit that permits lateral stresses to spread a large distance upslope from the wall, and the resulting failure mechanism is associated more closely with Rankine earth pressure theory than with

Coulomb or log-spiral theories (Terzaghi 1936). Until further research is available to clarify appropriate selection of wall friction parameters for lateral spreading, the friction should be reduced by half from the value that would be used for a nonliquefied soil profile and the earth pressure computed using log-spiral theory. Three-dimensional correction factors should be used to account for the finite width-to-height ratio of pile caps and abutment backwalls (e.g., Mokwa and Duncan 2001). Friction forces along the sides and base of the pile cap may be estimated using appropriate interface friction parameters (e.g., $\sigma_h'{\cdot}\tan(\delta)$ for sand or $\alpha{\cdot}s_u$ for clay). In the absence of site-specific measurements, general relations for retaining walls or piles may be used. Base friction may be affected by formation of a gap beneath the pile cap that may or may not remain open during lateral spreading. Brandenberg et al. (2007b) found that the base friction should be multiplied by a factor of 0.25 based on centrifuge tests involving pile caps whose base was in contact with clay.

The stiffness and capacity of soil springs are often reduced to account for pile group interaction effects (i.e., by applying group *p*-multipliers). Group effects should not be applied in liquefied soil because the data indicate that group effects do not exist in liquefied ground since liquefied soil is weak (e.g., Rollins et al. 2005). Group effects should be used for underlying nonliquefied layers, however, as demonstrated by Juirnarongrit and Ashford (2006); group factors recommended by Mokwa and Duncan (2001) may be used for this purpose. Group effects should not be applied in laterally spreading nonliquefied crust layers because using group effects would constitute an unconservative reduction in lateral spreading forces. If a group of closely-spaced piles has the potential to effectively act as a wall for the nonliquefied crust, the total lateral load capacity against the individual piles is greater than the total lateral load capacity that would develop if the piles act as a wall. If the piles act as a wall, then the ultimate value of the *p-y* springs in the nonliquefied crust layer can be reduced such that the sum of the ultimate values of the *p-y* springs are equal to the lateral load against the equivalent wall.

Corrections for the effects of cyclic loading (Matlock 1970) should not be used for piles in liquefied and laterally spreading ground. The cyclic reduction factors are intended to capture the cumulative effects of a large number of cycles that displace the piles back and forth through the soil (e.g., wave loading against an offshore foundation), thereby causing cyclic degradation of the load transfer behavior through cyclic degradation of the soil properties, gapping at the soil-pile interface, with erosion/scour occurring around the pile as water flows in and out of those gaps. These cyclic reduction factors should not be applied to *p-y* relationships for laterally spreading ground because: (1) lateral spreading is more comparable to monotonic lateral loading due to the downslope displacement bias and comparably small number of significant loading cycles; and (2) reducing the crust load would be unconservative for lateral spreading, whereas it is conservative for many other loading conditions.

The presence of a liquefied layer will reduce the ultimate lateral loads that can develop against a pile in the overlying or underlying nonliquefied layers to a distance of up to a few pile diameters [see (B) in Figure 4.3] from their contact with the liquefied layer [as illustrated by the three-dimensional finite element analyses for 43-cm-square piles in layered soils by Yang and Jeremic (2002)]. However, overlying or underlying strong layers do not appreciably strengthen the *p-y* behavior in the liquefied sand. A "smeared" profile of $p_u$ values can be linearly interpolated from the original profile to account for the weakening effect of the liquefied sand on nonliquefied layers above and below, as shown in Figure 4.3. This type of "smeared" profile can be used in the design of piles in approach embankments discussed next in Section 5, in order to avoid an unrealistically large stiffness contrast between the liquefied layer and base layer. The appropriate distance for smearing against large-diameter pile shafts requires further study, particularly because the distance 2B shown in Figure 4.3 can equal or exceed the thickness of a nonliquefied crust when B is large and the crust is relatively thin. For this reason, the smeared profile in Figure 4.3 should not currently be used to reduce the ultimate passive load that a nonliquefied crust can impose on large-diameter pile shafts.

The stiffness of the soil springs that are used to connect the nonliquefied crust with the pile foundation depend on the point of reference that is being used for specifying input ground displacements. Physically, the restraining or pinning effect of the pile foundation can significantly reduce the lateral spreading displacements of the soil near the pile foundation (e.g., soil in contact with the piles may be restrained to some negligible displacement), but has progressively less effect on the lateral spreading displacements for points located at progressively greater distances from the pile foundation. The soil "*p-y*" spring in a BNWF model relates the load on the pile foundation ($p$) to the relative displacement ($y = y_{soil} - y_{pile}$) between the pile displacement ($y_{pile}$) and the soil displacement ($y_{soil}$). Thus, the loading from the nonliquefied crust can conceptually be represented using any number of possible reference points for $y_{soil}$, with each reference point producing a different spring stiffness and input ground displacement.

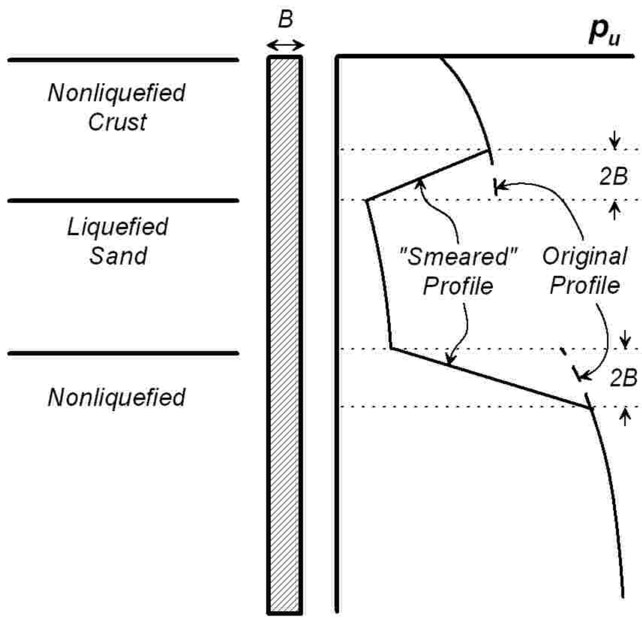



**Figure 4.3 Modification to the profile of ultimate subgrade reaction, $p_u$, to account for the weakening effect the liquefied sand exerts on overlying and underlying nonliquefied layers.**

When the size of a lateral spread feature is large compared with the size of the affected bridge component or subassemblage, it is convenient to specify $y_{soil}$ as a "free-field" condition, such as would be obtained from any lateral spreading calculation that does not account for the pinning effects of the pile foundations. In this case, the amount of displacement required to mobilize the ultimate crust load can be approximated as being 25% (for stiff crust soil) to 70% (for soft or loose crust soil) of the wall (or pile cap) height (Brandenberg et al. 2007a). These load transfer relationships are significantly softer than those for static loading of pile caps in nonliquefied soils, where the ultimate lateral resistance develops at a lateral displacement that is typically between 1% and 7% of the wall (or pile cap) height. This softening of the load transfer between the nonliquefied crust and the pile foundation is caused by loss of stiffness in the underlying liquefied layers.

When the mass of the spreading soil is small relative to the size of a bridge component, the interaction between the lateral spreading soil mass and the pile foundation can be more directly analyzed using "pinning analysis" methods. As discussed later in this report, pinning-analysis methods for piled abutment are appropriate when the dimensions of the spreading soil have well-defined limits relative to the pile foundation dimensions. For example, the zone of soil interacting with a piled abutment in an approach embankment is limited by the width of the embankment, whereas the zone of soil interacting with a single pile group located within a large lateral spread may be essentially unbounded for practical purposes. Possible exceptions are when the bounds of the lateral spread are known (e.g., by geologic controls) and the pile foundation dimensions are large relative to the lateral spread, in which cases the designer may choose to use a pinning-analysis method.

## 4.2 COMBINING LATERAL SPREADING AND DYNAMIC DISPLACEMENT DEMANDS

The demands on a bridge from to the possible combined effects of both lateral spreading and dynamic shaking need to be considered in design. This will generally involve first estimating an inertia demand that accounts for the effects of liquefaction, combining this inertia demand with lateral spreading displacements, and then performing analyses with these demands imposed simultaneously. Inertia demands utilized in design of bridges without liquefaction and lateral spreading are most commonly represented as displacements based on a design displacement response spectrum, and the natural period of the bridge or component. Spectral displacements are the relative displacement between the bridge superstructure and the ground, hence these displacements are an appropriate intensity measure for quantifying demands on the bridge components. When liquefaction-induced lateral spreading occurs, a number of factors complicate selection of inertia demands: (1) liquefaction affects how seismic waves propagate through the soil, thereby altering the ground surface motion (and the response spectrum); (2) the demands imposed by the laterally spreading soil cause the pile cap to displace and rotate, thereby altering the reference point utilized in a relative displacement response spectrum; and (3) the peak lateral spreading demands and peak inertia demands may occur at different times. The goals of the proposed methodology are therefore to select displacement demands that are compatible with the effects of liquefaction and lateral spreading. As shown in Figures 4.4-4.6, multiple analyses may be required to account for the various fixity conditions at the tops of the pier columns that may result from transfer of loads among various components through the superstructure.

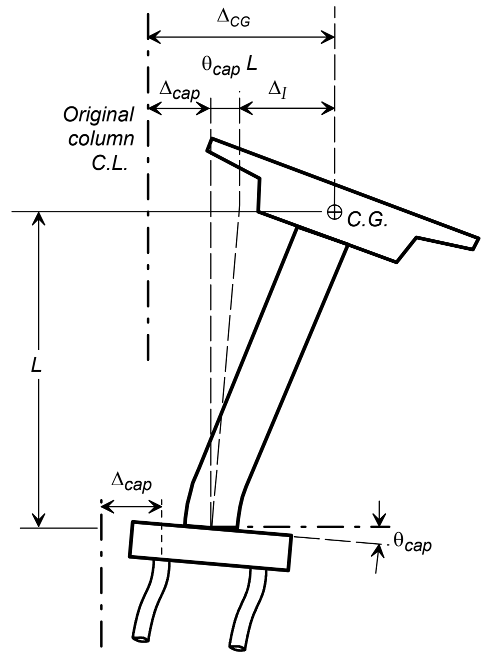
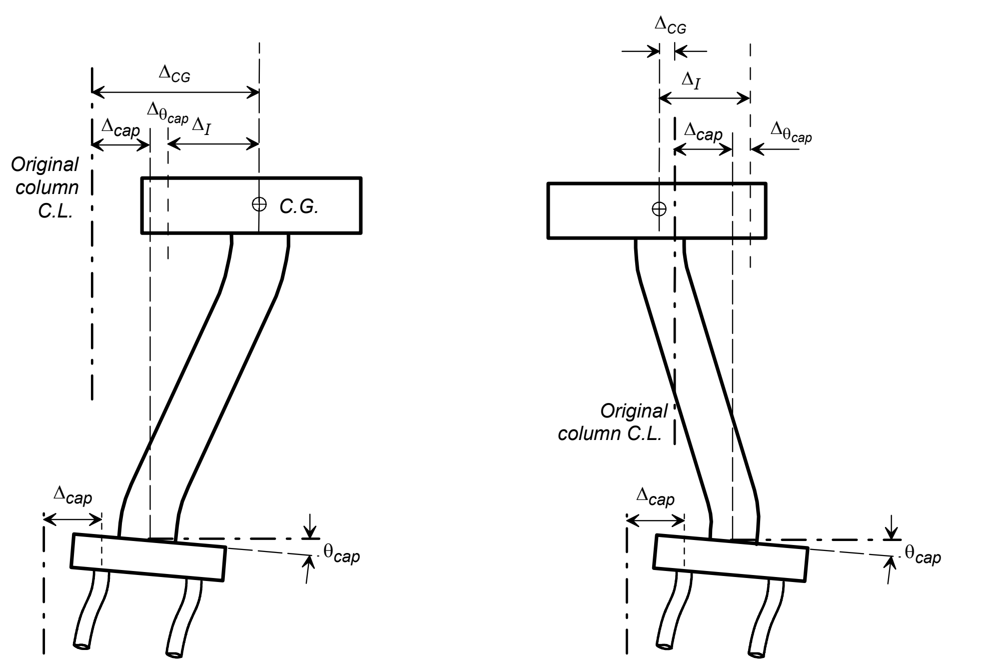



**Figure 4.4 Combining displacement demands from lateral spreading and inertial loading in the transverse direction for an individual single-column bent.**




(a) Demands in same direction (b) Demands in opposing directions

**Figure 4.5 Combining displacement demands from lateral spreading and inertial loading in the longitudinal direction for an individual single-column bent.**

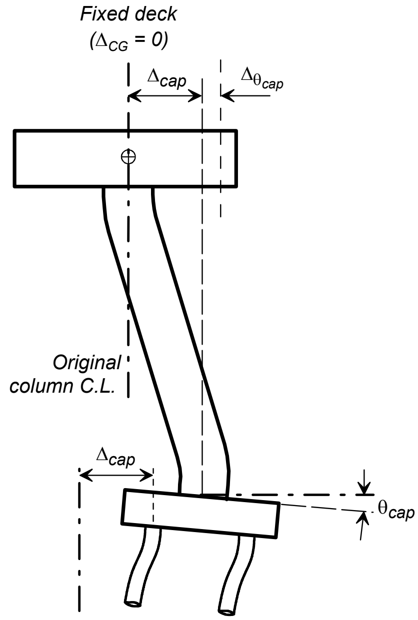



**Figure 4.6 Displacement demands from lateral spreading with the deck restrained from translation in the longitudinal direction for an individual single-column bent**

### 4.2.1 Top of Pier Column(s) Not Restrained by the Superstructure

When the superstructure does not provide restraint at the top of the pier column(s), the spectral displacement can be estimated as a fraction of the spectral displacement for the nonliquefaction case (Boulanger et al. 2007) as shown in Equation 4.1.

$$\Delta_{I,liq} = C_{cc} C_{liq} \Delta_{I,nonliq} \tag{4.1}$$

where $\Delta_{I,liq}$ is the structural displacement demand consistent with the effects of liquefaction, $\Delta_{I,nonliq}$ is the displacement demand without liquefaction, $C_{liq}$ is the ratio of maximum displacement demand with liquefaction to that without liquefaction, and $C_{cc}$ is the fraction of the maximum displacement demand with liquefaction that occurs at the critical loading cycle (i.e., when the maximum pile bending moments and shear forces occur). As shown in Figure 4.4, $\Delta_I$ is a relative displacement that characterizes the structural deformation of the pier column. Additional components of displacement arise from pile cap displacement and rotation. Hence, $\Delta_I$ is not a global displacement, and iteration will generally be required to obtain the global displacement boundary condition that produces the desired relative displacement value. For cases where the pier column remains elastic, the spectral displacement can be replaced by an equivalent force, thereby eliminated the need to iterate. The inertia force is defined in Equation 4.2:

$$I_{liq} = C_{cc} C_{liq} I_{nonliq} \tag{4.2}$$

where $I_{max,liq}$ is the inertia force accounting for the effects of liquefaction, and $I_{max,nonliq}$ is the maximum or peak inertia force for a linear-elastic bridge superstructure in the absence of liquefaction (i.e., $I_{nonliq} = \Delta_{I,nonliq} \cdot K = m \cdot S_{a,nonliq}$, where $K$ is lateral stiffness, $m$ is mass, and $S_{a,nonliq}$ is spectral acceleration).

The values of $C_{liq}$ and $C_{cc}$ were shown to vary with the frequency content characteristics of the input motion, which can be conveniently represented by the ratio $Sa_{T=1s}/Sa_{T=0s}$ for the design spectra for the nonliquefied case. The values of $C_{liq}$ and $C_{cc}$ did not show clear dependence on the structure's elastic period or the nonliquefied site period. Recommended values of $C_{liq}$ and $C_{cc}$ are summarized in Table 4.2 (Boulanger et al. 2007). For example, the linear-elastic inertia force from a bridge superstructure with liquefaction for a typical design acceleration response spectrum having $Sa_{T=1s}/Sa_{T=0s}$ = 0.5 – 1.6 would be about 36% of the inertia force for the nonliquefaction case, based on the $C_{liq}$ = 0.55 and $C_{cc}$ = 0.65 values listed in Table 4.2.

**Table 4.2 Inertia coefficients for BNWF analysis of pile foundations in liquefied ground.**

| Design spectra for nonliquefied condition, $Sa_{T=1s}$ / $Sa_{T=0s}$ | Pile cap | | Superstructure | |
|---|---|---|---|---|
| | $C_{liq}$ | $C_{cc}$ | $C_{liq}$ | $C_{cc}$ |
| 1.7 – 2.4 | 1.4 | 0.85 | 0.75 | 0.65 |
| 0.5 – 1.6 | 0.75 | 0.85 | 0.55 | 0.65 |
| ≤ 0.4 | 0.35 | 0.85 | 0.45 | 0.65 |

Note: $Sa_T$ is the linear-elastic spectral acceleration (5% damping ratio) at period, $T$.

### 4.2.2 Top of Pier Column(s) Restrained by the Superstructure

The superstructure displacement or rotation at an individual bent or frame affected by liquefaction may be restrained by adjacent bents, frames, or abutments. This interaction across frames can be evaluated using global analyses of the full bridge system. If such a global analysis is not performed, then the foundation and pier column(s) should be analyzed using the three loading conditions shown in Figure 4.5 (superstructure restrained against rotation) and Figure 4.6 (superstructure restrained against translation and rotation).

The peak demand on the foundation for the cases illustrated in Figure 4.5 would occur when the lateral spreading and inertia demands act in the same direction, as shown in Figure 4.5(a). However, more demand may be placed on the pier column(s) when the inertia load acts in the opposite direction from lateral spreading displacements due to the combination of pile cap rotation and restraints imposed by the superstructure. Hence, an additional analysis should check the demands in the pier columns with inertia imposed in the opposite direction from lateral spreading displacements, as shown in Figure 4.5(b).

The local system should also be evaluated for the case where the bridge deck is fixed against translation (i.e., $\Delta_{CG}$ set equal to 0) to simulate the condition where loads in the pier column(s) are transferred to other bents, frames, or abutments through compressive stresses in the superstructure, as shown in Figure 4.6. This loading case can result in larger or smaller demands on the pier columns than are obtained for the loading case shown in Figure 4.5(b), depending on whether the $\Delta_{CG}$ value is positive or negative in Figure 4.5(b). If $\Delta_{CG}$ is positive in Figure 4.5(b), then the conditions represented in Figure 4.6 will induce larger demands on the pier column(s) than the conditions in Figure 4.5(b). If $\Delta_{CG}$ is negative in Figure 4.5(b), then the conditions represented in Figure 4.6 will induce smaller demands on the pier column(s) and does not need to be evaluated.

The $C_{cc}$ and $C_{liq}$ values suggested in Table 4.1 were formulated for the case without any restraint at the top of the pier column(s) from the superstructure. Future research is required to better quantify the influence of liquefaction when the superstructure does provide restraint. In the absence of better information, the values in Table 4.1 can be used in the analyses shown in Figure 4.5. For the analysis shown in Figure 4.6, inertia loads were omitted.

In certain cases, a pile foundation's response to lateral spreading is relatively uncoupled (physically) from its response to inertia loading. For example, the lateral spreading of a strong thick crust may cause bending of the piles at large depths, while the superstructure's inertial loads may be transferred to the soil at shallower depths such that the two loading mechanisms have little overlapping influence and could have been analyzed as separate load cases. In other situations, the two loading mechanisms may have overlapping influence such that they cannot reasonably be analyzed as separate load cases. In practice, it is often difficult to predict whether the effects of lateral spreading and inertia loading can be analyzed as separate load cases or not, without actually performing an equivalent static BNWF analysis to determine how strongly they interact. Thus it is recommended, that an equivalent static BNWF analysis with soil displacements and inertia loads applied simultaneously be performed.

## 4.3 ADDITIONAL COMMENTS FOR PILE GROUPS IN LATERAL SPREADS

The sensitivity of the computed foundation response to variations in the major input parameters should always be evaluated. Previous sensitivity studies have demonstrated that peak response parameters (maximum shear forces, maximum bending moments, pile cap displacements) were the most sensitive to inertial loads, lateral spreading displacements, crust properties, and pile foundation characteristics (structural and geotechnical capacities). Other parameter variations can investigate factors such as the shape of the soil displacement profile and the *p*-multipliers for liquefied layers, although previous studies identified that these factors were generally of lesser importance in determining pile cap displacements and loads at the pile-cap connection. If the bending moment distribution beneath the ground surface is important, then the soil displacement profile and p-multipliers for liquefied layers can be of greater importance.

For most parameter variations, it is important to consider best estimates with high and low ranges, as well as other permutations, because it is not always evident which will result in a conservative estimate of foundation response. For example, a conservative estimate of pile response for the nonliquefaction case might correspond to a softer load transfer relationship between the pile cap and surrounding crust (e.g., weaker crust strengths, assumption of zero shear on the base and sides of the pile cap, larger relative displacements to mobilize the crust loads) whereas a conservative estimate of pile response for the liquefaction case might instead correspond to a stiffer estimate of the same load transfer relationship (e.g., stronger crust strengths, inclusion of base shear on the pile cap, etc.). Important parameters for one particular bridge or bridge component may be unimportant for another bridge or bridge component, and therefore sensitivity studies are case-specific and should not be overly generalized.

Free-field soil displacements are typically imposed as displacement constraints on the free ends of the soil springs, which inherently assumes that the soil deforms as a shear beam. A shear-beam soil displacement profile is often associated with an abrupt change in shear strain at layer boundaries, which is associated with infinite curvature. Free-field soil displacement profiles with discontinuous slopes can cause unrealistically large curvature demands on piles, particularly when the pile is flexible relative to the soil profile. Three-dimensional finite element simulations have shown that the soil around a pile exhibits curvature, and therefore does not deform as a shear beam (Lam et al. 2007). It is currently not clear how to incorporate this beneficial effect into BNWF analyses. As such, BNWF analyses that predict large pile curvatures at locations with discontinuities in the slope of the imposed free-field soil displacement profile should be interpreted carefully, and may warrant more detailed analysis if the results are found to significantly affect design decisions.

# 5 Design of Piles in Approach Embankments

This section considers the local analysis of a bridge abutment in an approach embankment (Figure 1.1) for the case where liquefaction has been triggered in the underlying soils. As the embankment soils spread longitudinally, the piles and bridge superstructure can develop reaction forces that are significant relative to the inertia forces driving displacements of a finite-width embankment. These "pinning" forces reduce the embankment displacements relative to those that would occur in the absence of any pinning force. The result is a coupled system wherein demands imposed on the bridge depend on embankment displacements, which in turn depend on the degree to which the piles and bridge superstructure pin the embankment. The beneficial effect of this coupling diminishes as the weight of embankment undergoing spreading increases relative to the available pinning forces.

Embankments can also develop substantial transverse spreading and surface settlements, which are important considerations for evaluating the post-earthquake accessibility or serviceability of a bridge. The ground displacement from the pinning analyses will not account for these effects, and thus should not be used for evaluating serviceability of the embankment after an earthquake.

Procedures for estimating pile pinning effects on longitudinal embankment displacements have been described by Martin et al. (2002) and modified by Boulanger et al. (2006). These procedures can be represented by three primary parts.

- Estimation of the longitudinal displacement of the embankment soil mass for a range of restraining forces from the piles and bridge superstructure.
- Estimation of the longitudinal restraining force exerted on the embankment mass by the piles and bridge superstructure for a range of imposed embankment displacements.
- Determination of the compatible displacement and interaction force between the embankment mass and the piles and bridge superstructure.

Each of these three parts is discussed in more detail in the following sections.

## 5.1 ESTIMATING EMBANKMENT DISPLACEMENTS FOR A RANGE OF RESTRAINING FORCES

First, slope stability analyses of the embankment for a range of restraining forces from the piles and bridge superstructure should be performed. The total shear force ($V_t$) and bending moment ($M_t$) in the piles at the slope stability failure surface (Figure 5.1) can be represented by an equivalent force-couple, such as having the pile shear force act alone at a distance equal to $M_t/V_t$ above the slope stability failure surface. A single large point forces can produce numerical errors in the limit equilibrium analyses. In these cases, $V_t$ is often represented as a distributed force or an equivalent increase in soil shear strength along some portion of the failure surface. Then slope stability analyses are used to determine yield accelerations ($k_y$) for a range of possible restraining forces. For each restraining force, the yield acceleration is the value of the horizontal seismic coefficient that produces a factor of safety of unity against slope instability.

These slope stability analyses must consider a range of possible failure surfaces because the most critical failure surface can increase substantially in size with increasing restraining force (e.g., Figure 5.1). In some cases, the failure surface may be predicted to extend the full length of the approach embankment, rendering the slide mass so large that the pinning force is ineffective. In reality, the length of the failure surface may be limited by geologic boundaries that control the extent of liquefiable soils or by the finite length of an approach embankment. In addition, the net displacement of a slide mass may eventually decrease as the failure surface length becomes very large, because: (1) the average seismic coefficient will be reduced by incoherence of motions within the larger soil mass; and (2) the compensating effect of two-way sliding, which is not included in most Newmark sliding block analyses or regression formula, will tend to be more significant for a larger slide mass. Analyses that account for the above effects can involve considerably more engineering effort. For cases where such efforts are not justified, it is suggested that the distance that the critical failure surface extends behind the abutment may be limited to about four times the embankment thickness for the purpose of estimating the loads on the piled abutment.

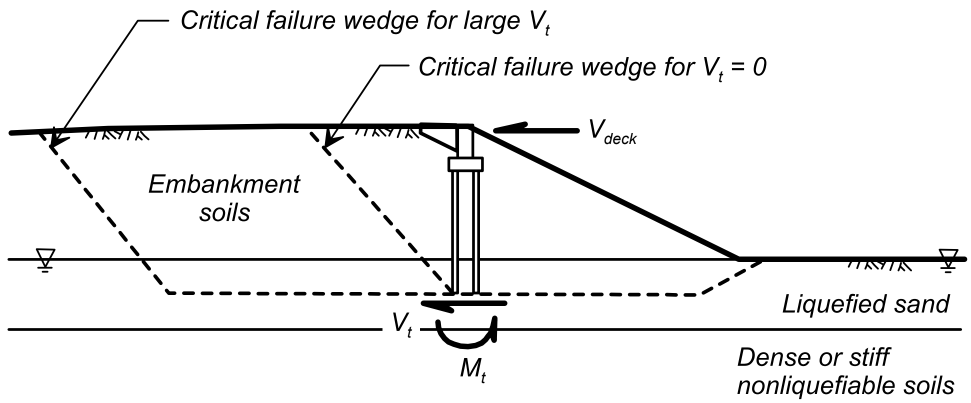



**Figure 5.1 Critical slope stability failure surfaces for different values of pile restraining force.**

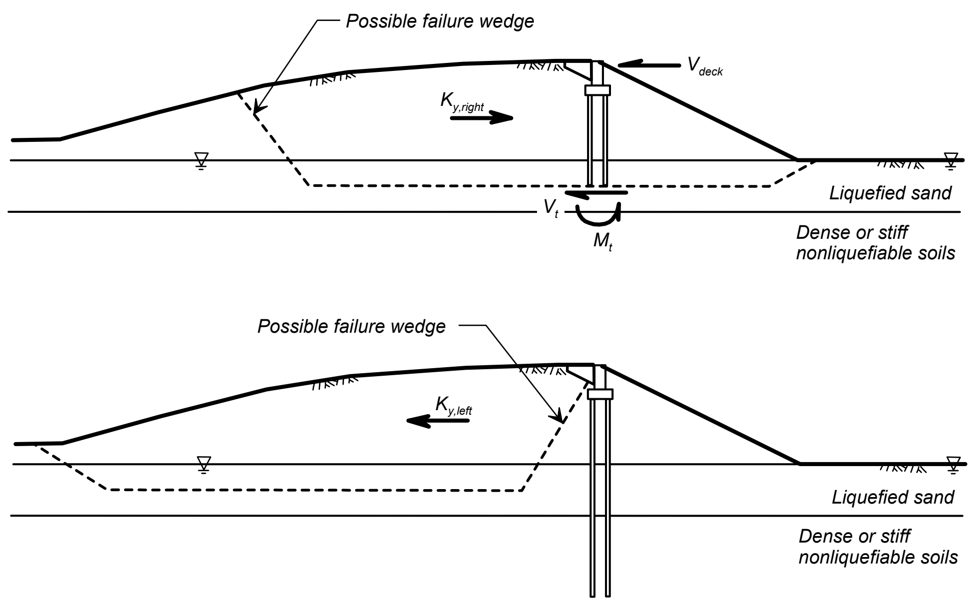



**Figure 5.2 Critical slope stability failure surfaces for movement toward the abutment versus away from the abutment (Armstrong and Boulanger, in progress).**

Possible sliding mechanisms in approach embankments may include movements both toward and away from the abutment. In this case, yield accelerations can be computed for both directions of movement (see Figure 5.2). If the yield acceleration away from the abutment ($K_{y,left}$ in Figure 5.2) is less than or equal to the yield acceleration toward the abutment ($K_{r,\ right}$), then the portion of the embankment that is common to both failure mechanisms would be expected to progressively move away from the abutment. In that case, the critical failure surface for loading of the piled abutment may be limited to a wedge that forms immediately behind the abutment (e.g., similar to the surface for $V_t = 0$ in Figure 5.1).

Embankment displacements for each of the possible restraining forces are then computed based on the yield acceleration and the design ground motion parameters. This step can be performed using a regression model for Newmark sliding block displacements, such as the one developed by Bray and Travasarou (2007) and applied to liquefaction by Ledezma and Bray (2007). These types of models generally assume one-way sliding for the slide mass. The results of these analyses are a plot of embankment displacement versus restraining force per unit thickness of the analyzed section.

The tributary (transverse) width for the embankment mass is used to establish a common dimension between the force-displacement relationships for the embankment and for the pile foundation/bridge superstructure. Consider the embankment transverse cross-section shown in Figure 5.3. The piles and bridge superstructure will restrain movement of an embankment mass that includes the soil defined by the embankment crest width, plus a portion of the side slope masses. This is accounted for by adopting an equivalent tributary width whose mass includes a portion of the side slope masses, with one-half of the side slope mass recommended as a reasonable value for design.

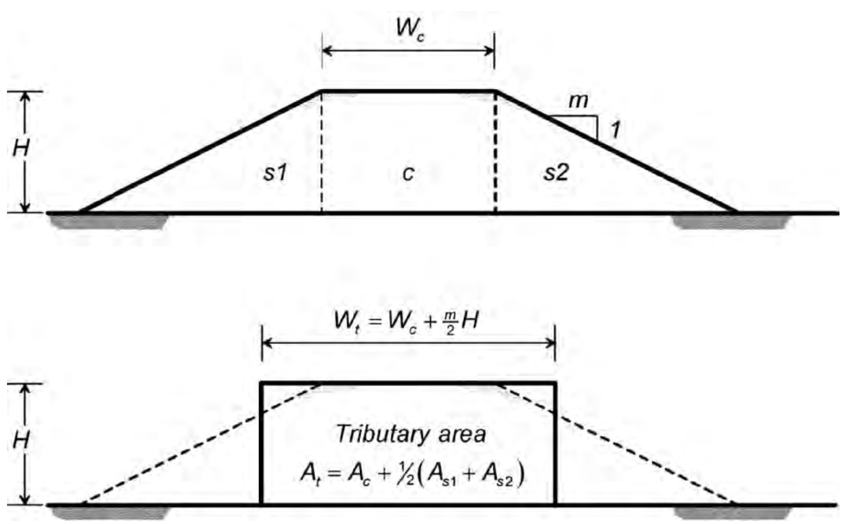



**Figure 5.3 Transverse section of an abutment showing the equivalent tributary width that is assumed to interact with the restraining forces from the pile foundation and bridge superstructure.**

## 5.2 ESTIMATING PILE/BRIDGE RESTRAINING FORCES FOR A RANGE OF DISPLACEMENTS

The restraining forces from the pile foundation and bridge superstructure are estimated for a range of possible embankment displacements. Inertia forces from the bridge structure will alternate between causing an increase and decrease in the restraining force on the embankment. The equivalent static representation of restraining forces from the pile foundation and bridge superstructure neglects the transient influence of bridge inertia forces.

The restraining force from the pile foundation is determined using an equivalent static BNWF pushover analysis. In this analysis, the imposed soil displacements are progressively increased, and the shear forces and bending moments in the piles at the location of the slope stability failure surface are determined. Conventional *p-y* springs are used between the piles and the embankment because the reference ground displacement lies within a restrained soil failure mass near the piles (softer *p-y* springs are used when the reference ground displacement is for a "free-field" condition outside of the influence of the piles). The ultimate shear force that can develop will be limited by plastic hinging in the piles. The moment capacity of the piles, and hence their shear resistance, may be further reduced by geometric effects (i.e., $P$-$\Delta$ or buckling) as the abutment displacements become significant (Martin et al. 2002).

The development of restraining forces from the bridge superstructure with increasing embankment displacement depends on the structural configuration and details (e.g., bearings, expansion joints, shear capacity of seat abutment backwall, and shear keys), the characteristics of the embankment soils (e.g., passive resistance against an abutment backwall that is designed to break away during design loading), as well as the capacity of the opposite abutment. The restraining force that develops at the abutment must be transferred to either the intermediate bents or to the opposite abutment. Pushover analyses of the global bridge structure can be used to estimate this load transfer behavior.

The combined restraining forces from the pile foundation and bridge superstructure will progressively increase as the embankment displacement increases during earthquake shaking. Newmark sliding block analyses for the embankment are, however, most commonly based on the assumption that the restraining forces are constant throughout shaking. To provide consistency between these two uncoupled analyses, the "equivalent constant restraining force" from the piles and bridge superstructure can be approximated as the average resistance that develops between the start of shaking (zero embankment displacement and hence zero resistance) and the end of shaking (the resistance for the final embankment displacement) (Boulanger et al. 2006).

## 5.3 COMPATIBILITY OF EMBANKMENT AND PILE DISPLACEMENTS

A compatible displacement and interaction force between the embankment slide mass and the pile foundation/bridge superstructure can be determined from the relationships developed in the previous steps. Graphically, the solution is the intersection of the force-displacement relationships determined separately for the embankment slide mass and the pile foundation/bridge superstructure, as shown in the example in Figure 5.4. This example shows that the embankment displacements would be expected to range from 1.4 to 2.4 m in the absence of any pile pinning effects, but would be expected to range from 0.5 to 0.7 m when the benefit of the pile pinning effects are taken into consideration.

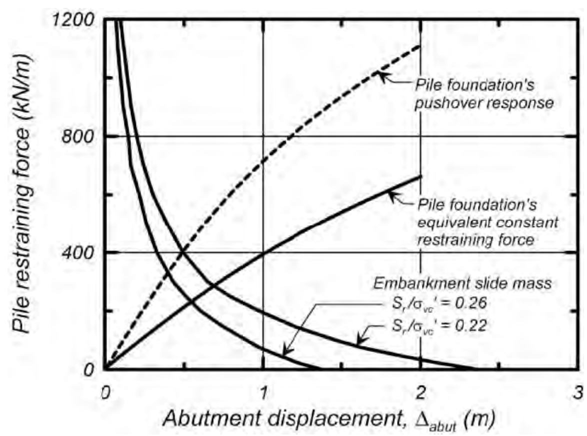



**Figure 5.4 Compatibility between the computed embankment slide mass displacements and the equivalent constant restraining force from the pile foundation.**

# 6 Global Bridge Response for Liquefaction Case

Global analysis of a bridge can provide a more realistic evaluation of the distribution of force and displacement demands throughout the bridge than can be obtained from local analyses of individual bents or frames. Global analyses of ordinary bridges without liquefaction effects are commonly performed using linear-elastic ESA for cases where a dynamic analysis will not add significantly more insight or linearly-elastic dynamic analyses in more complicated cases. The Caltrans SDC indicates that ESA are best suited for bridges or individual frames that have low skew, simple lateral force distributions, and responses that are dominated by the fundamental mode of vibration. Although Aviram et al. (2008) developed guidelines for nonlinear analysis of standard ordinary bridge structures are presented, they provide only limited guidance regarding foundation modeling; they do not provide guidance for modeling of liquefaction and soil spreading for global bridge-soil analysis.

Global analyses for the effects of liquefaction are particularly warranted when the subsurface conditions and expected liquefaction-induced ground displacements vary substantially along the bridge alignment. For example, consider a global analyses using a nonlinear bridge model with two different scenarios of soil displacement profiles, as shown in Figure 6.1. For the case in Figure 6.1(a), soil displacements are imposed only at the abutments with lateral spreading demands toward the center of the bridge where the critical components for the analysis are the piles that support the abutments, which have suffered extensive deformations. The piers suffer only small deformations because the loading on the bridge is nearly symmetric, and lateral spreading forces are transmitted as compressive stresses through the continuous superstructure. For case (b) in Figure 6.1(b), soil displacements are imposed at the left abutment and at the adjacent pier with large demands imposed on the piles supporting the left abutment and adjacent pier. Although little demand is placed on the left pier itself, the other two piers suffer extensive deformations. This deformation pattern results because the entire bridge has shifted from left-to-right, while the pile cap for the left pier has displaced about the same amount as the bridge deck, and the other two pile caps have exhibited little left-to-right translation. This example demonstrates how lateral spreading demands may affect bridge components in competent nonliquefied soil layers, and these demands can best be captured by a global analysis.

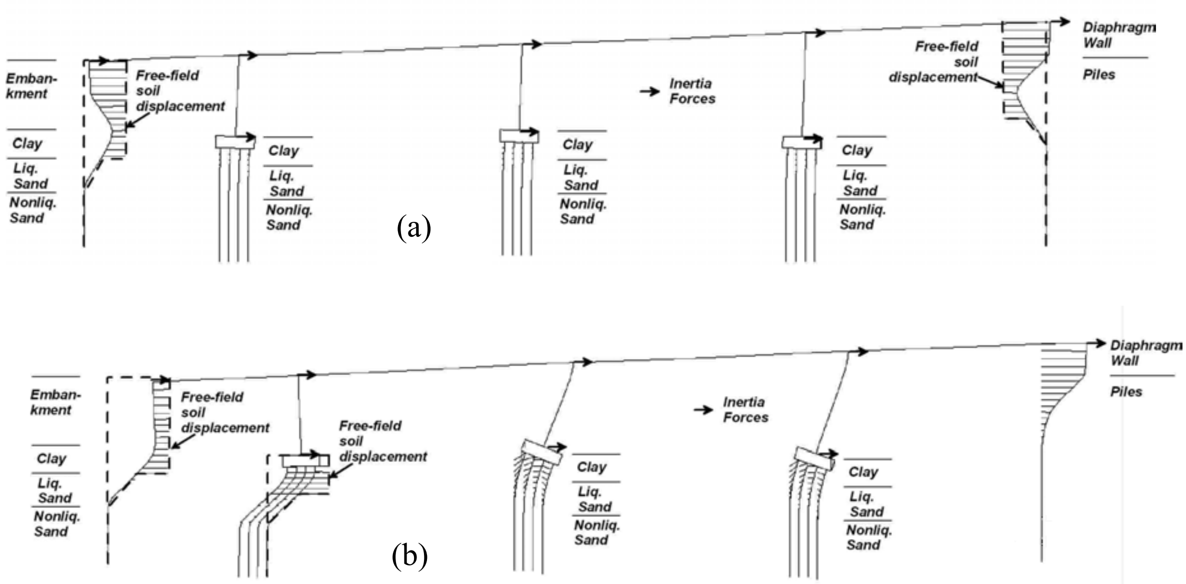



**Figure 6.1 Deformed mesh for two different global analyses in longitudinal direction.**

## 6.2 DYNAMIC ANALYSES

The dynamic response of a bridge with liquefaction effects is highly nonlinear and strongly affected by the accumulation of permanent ground displacements. Linear-elastic dynamic analyses cannot be reasonably adapted to include the effects of liquefaction. Nonlinear dynamic analyses make it possible to investigate complex interaction mechanisms and gain insight into how ground deformation patterns can affect the performance of a structure. Although nonlinear dynamic analyses require a high level of expertise with computational methods with considerable engineering effort required to perform with existing software, these types of analyses are becoming more common on large engineering projects where the additional insights justify the expense. The use of nonlinear dynamic analyses for ordinary bridges is beyond the scope of this report.

## 6.3 GLOBAL EQUIVALENT STATIC ANALYSIS WITH NONLINEAR FOUNDATION MODELS

A global ESA for an ordinary bridge with liquefaction can be performed using a nonlinear model for the bridge with nonlinear BNWF models for the pile foundations. The loading from lateral spreading can be imposed on the global model by imposing the lateral spreading displacements to the support ends of the soil springs. The additional loading due to dynamic response of the superstructure can be modeled either as forces (as shown in Figure 6.1) or as spectral displacements, with each approach having certain limitations and advantages. Global analyses should examine a number of possible loading combinations, with lateral spreading demands imposed on various combinations of components to assess the most critical conditions. Global analyses can also provide additional insights into the pinning effect at abutments, where axial loads in the superstructure can serve to pin back a spreading abutment. Procedures for imposing inertia demands in global ESA for ordinary bridges require further development and research. In particular, various approaches for performing a global ESA will require validation against results from nonlinear dynamic analyses of a wide range of bridge configurations and soil conditions.

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ISSN 1547-0587X