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EVALUATING SEISMIC DISPLACEMENTS AND DAMAGE FOR PILE FOUNDATIONS UNDERGOING LIQUEFACTION-INDUCED LATERAL SPREADING

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ABSTRACT

Liquefaction-induced lateral spreading has caused significant damage to pile foundations during past earthquakes. Ground displacements due to lateral spreading can impose large forces on the overlying structure and large bending moments in the laterally displaced piles. Pile foundations, however, can be designed to withstand the displacement and forces induced by lateral spreading. Piles may actually “pin” the upper layer of soil that would normally spread atop the liquefied layer below it into the stronger soils below the liquefiable soil layer. This phenomenon is known as the “pile-pinning” effect. Piles have been designed as “pins” across liquefiable layers in a number of projects, and this design methodology was standardized in the U.S. bridge design guidance document MCEER/ATC-49-1. A number of simplifying assumptions were made in developing this design procedure, and several of these assumptions warrant re-evaluation. In this paper, some of the key assumptions involved in evaluating the pile-pinning effect are critiqued, and a simplified probabilistic design framework is proposed for evaluating the effects of liquefaction-induced lateral spreading on pile foundations of bridge structures. Primary sources of uncertainty are incorporated in the proposed procedure so that it is compatible with the performance-based earthquake engineering framework. The application of the proposed procedure is illustrated through a realistic example.

Keywords: earthquakes, liquefaction, lateral spreading, pile-pinning effect, performance-based design

INTRODUCTION

Past earthquakes have shown that pile foundations can be vulnerable to liquefaction-induced ground failures. In cases where the post-liquefaction static factor of safety is less than one, flow slides may occur and induce large lateral and vertical deformations causing extensive damage. For example, the Showa Bridge failed catastrophically in part due to liquefaction-induced lateral spreading produced by the 1964 Niigata, Japan earthquake (e.g., Hamada, 1992). Compressional forces in the structure, additional shear forces in connections, and significant relative ground displacements across the supporting piles, which induce large bending moments, may all result due to lateral spreading. Each of these additional loads may lead to severe damage or collapse of structures. Even if flow slides do not occur, earthquake-induced seismic displacements of the ground may progressively develop and damage piles and the overlying structure.

When liquefaction-induced displacements threaten the performance of an engineered structure, soil remediation techniques may be considered to minimize the expected damage. These techniques usually involve ground modification, ground improvement, or stabilizing measures that significantly reduce the liquefaction hazard or reduce the level of ground deformations to an acceptable level. However, mitigation may be difficult and costly at some sites, and it may not be necessary if the structure’s

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foundation piles can be designed to withstand the displacement and forces induced by the lateral spreading. Moreover, if the foundation piles can be designed to “pin” the upper layer of soil, that would normally spread atop the liquefied layer below it, into the stronger soils below the liquefiable soil layer, then other ground remediation techniques may not be required to achieve satisfactory performance. This technique has been coined the “pile-pinning” effect, which is illustrated in Figure 1.

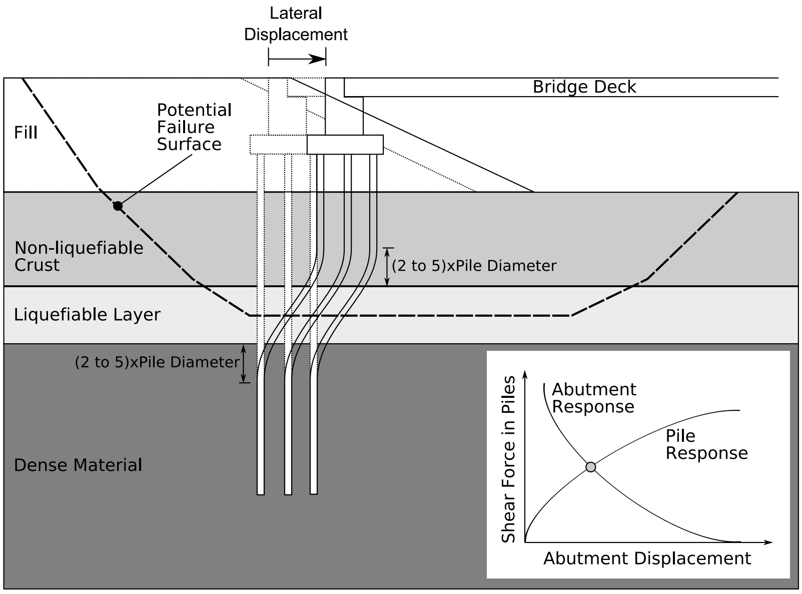


Figure 1. “Pile-pinning” effect for the case of piles that are locked into to both the soil above and below the liquefiable soil layer

PILE PINNING EFFECT

The pile pinning effect was first implemented in the United States as a result of the seismic safety evaluation of Sardis Dam (Finn et al. 1991). Sardis Dam was judged to be potentially unstable due to excessive seismic deformations resulting from liquefaction of a thin layer of silt in the foundation of the dam. The liquefiable silt layer was located between stronger soil strata above and below the potentially weak soil layer. Dynamic effective stress, large-strain finite element analyses (Finn et al. 1997) indicated that the potentially large seismic displacements due to laterally spreading atop the liquefiable silt layer could be arrested by “pinning” the competent soils above the weak layer into the strong soils below the weak layer. A total of nearly 2600 prestressed reinforced concrete piles (60 cm in diameter and spaced 2.4 to 3.7 m apart) were installed in the mid-1990s to remediate Sardis Dam.

The pile pinning effect is now recognized as a legitimate remediation option when bridge or wharf structures built on pile foundations are located in areas susceptible to liquefaction-induced laterally spreading (e.g., Martin and Lam 2000, Martin et al. 2002). Work by Martin and others led to the development of a simplified design procedure for evaluating the effects of liquefaction-induced lateral spreads as proposed in the MCEER/ATC-49-1 recommended seismic design document of bridges (ATC/MCEER Joint Venture, 2003). Some of the principal steps involved in the current MCEER/ATC-49-1 design procedure are:

1. Identify the soil layers that are likely to liquefy.
2. Assign undrained residual strengths (S_{ur}) to the layers that liquefy. Perform pseudo-static seismic stability analysis to calculate the yield acceleration, k_y, for the critical potential sliding mass. Typically, the slide mass with the lowest k_y value is considered as critical.
3. Estimate the maximum lateral spread displacement of the soil.

4. If the assessment indicates that movement of the foundation is likely to occur in concert with the soil, then the structure must be evaluated for adequacy at the maximum expected displacement. This is the mechanism illustrated in Figure 1. The structural remediation alternative makes use of the "pinning" action that the piles contribute as they cross the potential failure surface.
5. Identify the plastic mechanism that is likely to develop in the presence of spreading.
6. From an analysis of the pile response to a liquefaction-induced ground displacement field, the likely shear resistance of the foundation is estimated. This increased resistance is then incorporated into the stability analysis, which increases k_y.
7. Recalculate the overall displacement on the basis of the revised resistance levels, and iterate until the resistance is consistent with the level of displacement estimated. Once a realistic displacement is calculated, the foundation and structural system can be assessed for this level of movement. If necessary, additional piles can be installed to reduce the seismic displacement further.

CRITIQUE OF THE ASSUMPTIONS INVOLVED IN PILE PINNING

It is conventional in engineering practice (e.g., MCEER/ATC-49-1) to evaluate liquefaction potential deterministically using the state-of-the-practice recommendations of Youd et al. (2001). However, Performance-Based Earthquake Engineering (PBEE) requires that significant sources of uncertainty are handled in a consistent framework. With this in mind, the probabilistic liquefaction evaluation procedures proposed by Seed et al. (2003) are employed. The probability of the occurrence of liquefaction given a specified ground intensity measure, $\Pr(L \mid im)$, can be estimated from (Seed et al. 2003):

$$P_L\left(N_{1,60}, CSR, M_w, \sigma_v', FC\right) = \Phi\left(-\frac{\begin{pmatrix} N_{1,60}\cdot(1+0.004FC) - 13.32\cdot\ln(CSR) - \\ 29.53\cdot\ln\left(M_w\right) - 3.70\cdot\ln\left(\sigma_v'\right) \\ +0.05\cdot FC + 44.97 \end{pmatrix}}{2.70} \right) \tag{1}$$

where P_L is the probability that liquefaction occurs, $N_{1,60}$ is the normalized Standard Penetration Test blow count, CSR is the cyclic stress ratio, M_w is the moment magnitude of the controlling earthquake, σ_v' is the vertical effective stress in atmospheres, FC is the fines content (i.e., amount finer than the #200 sieve by weight), and Φ is the standard normal cumulative distribution function (e.g., the function NORMSDIST in Excel). If the values of σ_v' and FC are assumed to be deterministic, Equation (1) provides an estimation of the probability of liquefaction given the values of $N_{1,60}$, CSR, and M_w. The values of CSR and M_w are part of the earthquake ground motion intensity measures, so this relation can provide an estimate of the probability of liquefaction occurring given the earthquake intensity measure and the soil's normalized SPT blow count, i.e., $\Pr(L \mid im, n_{1,60})$.

If liquefaction occurs, the critical layer will involve liquefied material, so the undrained residual shear strength of the liquefied material (S_{ur}) needs to be estimated. Again, this is conventionally done deterministically using procedures such as Seed and Harder (1990) or Olson and Stark (2002), wherein S_{ur} is estimated as a function of $N_{1,60}$. The critical undrained residual shear strength parameter can be estimated probabilistically if the probability density function (PDF) of $N_{1,60}$ and the PDF of S_{ur} for a given SPT value are assumed to be known. In this case, by the total probability theorem

$$f_{S_{ur}}(s_{ur}) = \int_{-\infty}^{+\infty} f_{S_{ur}}(s_{ur} \mid n_{1,60}) f_{N_{1,60}}(n_{1,60}) dn_{1,60} \tag{2}$$

where $f_{S_{ur}}(s_{ur})$ is the resultant PDF of S_{ur}, and $f_{N_{1,60}}(n_{1,60})$ and $f_{S_{ur}}(s_{ur} \mid n_{1,60})$ are the given probability density functions. If the information required to evaluate (2) is not available, one alternative is to assume that S_{ur} has, for instance, a *lognormal* distribution, and that its coefficient of variation (c.o.v.) is similar to the typical c.o.v. of the undrained shear strength, i.e. between 0.13 and 0.40 (from Table 3 in Duncan, 2000). Recently, Eddy and Gutierrez (2006) have proposed a probabilistic framework for estimating S_{ur}, and the c.o.v. proposed herein is consistent with the c.o.v. suggested by them.

Typically (e.g., MCEER/ATC-49-1), the Newmark (1965) "stick-slip" rigid sliding block approach is used to estimate seismic displacement for the case wherein liquefaction flow side does not occur (e.g., pile pinning is effective as indicated in Figure 1). Thus, the sliding mass is assumed to be rigid, and a dominant, distinct failure surface is assumed to exist. Rathje and Bray (1999) showed that the application of the Newmark rigid sliding block analysis can be overly conservative or unconservative depending on the characteristics of the potential sliding mass and the earthquake ground motion. Recent work by Bray and Travasarou (2007) has used a nonlinear deformable sliding block model to estimate probabilistic seismic displacements in a rigorous manner. The probability that the seismically induced permanent lateral displacement (D) exceeds a specific threshold of displacement is a function of the intensity of the ground motion and the resistance of the slope. The maximum dynamic resistance of the slope is represented by its yield coefficient k_y. The ground motion is characterized by its spectral acceleration at its degraded natural period. The relationship developed by Bray and Travasarou (2007) for a deformable sliding block is:

$$\begin{aligned} \ln(D) = -1.10 - 2.83\ln(k_y) - 0.333(\ln(k_y))^2 + 0.566\ln(k_y)\ln(S_a(1.5T_s)) + \cdots \\ 3.04\ln(S_a(1.5T_s)) - 0.244(\ln(S_a(1.5T_s)))^2 + 1.50T_s + 0.278(M-7) \pm \varepsilon \end{aligned} \tag{3}$$

where D (cm) is the seismically induced permanent lateral displacement, T_s (seconds) is the initial fundamental period of the potential sliding mass (i.e., $T_s \sim 4H/V_s$), S_a represents the spectral acceleration at $1.5T_s$ in units of g, and ε is a normally-distributed random variable with zero mean and standard deviation σ = 0.66. To eliminate a bias in the model for very low values of T_s, the first term of Equation (3), i.e., -1.10, should be replaced with -0.22 when $T_s < 0.05$ s.

The mechanism of the pile deformation as a result of lateral spreading needs to be characterized so that the system response can be evaluated. The MCEER/ATC-49-1 procedure assumes that the pile develops full fixity at some distance into the stronger soil strata above and below the liquefied soil stratum. With this assumption, the shear force in the pile can be calculated as

$$V_p = \frac{2M}{H + 2(\alpha \cdot 2R)} \tag{4}$$

where M is the bending moment in the piles, H is the total thickness of the liquefiable material, $2R$ is the diameter of the pile, and α is assumed to be a parameter on the order of 2 to 5 that represents the number of pile diameters into the strong soil layers above and below the liquefiable material. In the proposed probabilistic approach, α is assumed to be a random variable that has a *uniform* distribution between 2 and 5. Whereas it is likely that the pile (if it is sufficiently long) develops full fixity within the soil below the slide plane, it is more likely that the pile does not develop full fixity in the soil crust above the liquefiable soil. The soil crust that spreads laterally atop the liquefied soil is often observed to be significantly cracked and internally deformed in field case histories. Hence, the common assumption of full pile fixity in the upper soil crust warrants re-examination, and this is being evaluated further by a number of researchers. At this time, however, full fixity will be assumed, which implies that this procedure should only be applied when the liquefiable layer is relatively thin and the soil crust is relatively thick.

The sliding surface that initially produces the lowest k_y value is commonly used to define the critical potential sliding mass in the MCEER/ATC-49-1 procedure. However, the presence of piles increases k_y and causes the sliding surface to extend laterally. Thus, the pseudostatic slope stability analysis should be rerun with the increased resistance of the piles included to identify the most critical sliding surface. The potential sliding mass with the lowest k_y/k_{max} ratio, where k_{max} represents the maximum seismic loading should be considered most critical.

In the current procedure, it is also implicitly assumed that the pinning effect will increase the shear strength of the critical layer instantaneously. However, in reality, soil displacement is required to engage the pile-pinning effect. Hence, the pile-pinning effect is initially nonexistent, increases as the soil displaces, and eventually reaches a maximum value, which is the value assumed to instantaneously occur in the MCEER/ATC-49-1 procedure. A displacement-dependent yield coefficient relationship is being developed to handle those cases where intermediate displacements are calculated. For small or large displacements, the MCEER/ATC-49-1 is reasonable, but at intermediate displacements levels it may not be reasonable, and when it is not, it is unconservative.

The present methodology assumes that the effects of ground displacement can be decoupled from the effects of structural inertial loading. According to Martin et al. (2002), in most cases this is a reasonable assumption, because peak vibration response is likely to occur in advance of maximum ground displacements. However, Brandenberg et al. (2005) have indicated that, under some circumstances, the maximum inertial force can occur at essentially the same time as the maximum kinematically-induced force occurs. Moreover, centrifuge studies at U.C. Davis by Dr. Boulanger and others indicate that the tributary mass that loads the piles is greater than that directly behind the piles. The problem is three-dimensional and some additional load from outside the piles is attracted to the ground where the piles are effectively restraining lateral spreading. Guidance on these issues is being developed by researchers at U.C. Davis.

The piles performance criterion is not clear. One recommendation given in the MCEER/ATC-49-1 document is that if the plastic rotation of the piles (θ_p) is less than 0.05 radians, the pile may be considered to be in good condition. However, there is a need to establish a more robust relation of this parameter to reliably assess damage states.

P-Δ effects in the laterally displaced piles are typically ignored, and a simplified sensitivity study using the first-order correction recommended in the MCEER/ATC-49-1 document indicates that this is a reasonable approach given that P-Δ effects affect the estimation of shear force in the piles by less than 10%.

PROBABILISTIC METHODOLOGY

The Pacific Earthquake Engineering Research Center (PEER) Performance-Based Earthquake Engineering (PBEE) methodology is based on a framework that estimates the mean annual frequency of events where a specified decision variable exceeds a given threshold. The framework equation is (Cornell and Krawinkler, 2000):

$$\lambda(dv) = \int_{dm} \int_{edp} \int_{im} G(dv \mid dm) \left| dG(dm \mid edp) \right| \left| dG(edp \mid im) \right| \left| d\lambda(im) \right| \tag{5}$$

in which *im* is an intensity measure, *edp* is an engineering demand parameter, *dm* denotes a damage measure, *dv* denotes a decision variable, $G(x \mid y) = \Pr(X > x \mid Y = y)$ is the conditional complementary cumulative distribution function of the random variable X given $Y=y$, $dG(x \mid y)$ is the differential of $G(x \mid y)$ with respect to x, and $\lambda(x)$ is the mean frequency of $\{X>x\}$ events per year. Der Kiureghian (2005) provides a comprehensive review of the PEER PBEE framework.

To evaluate Equation (5), it is necessary to define the conditional complementary cumulative distribution function $G(edp \mid im)$. The following sections of this paper present a procedure that describes these functions for the case of bridge pile foundations undergoing liquefaction-induced lateral spreading where the pile-pinning effect is included.

PROBABILITY OF SEISMIC DISPLACEMENTS

Seismically Induced Permanent Lateral Displacement of the Soil Crust

A procedure to estimate $G(edp \mid im)$, i.e., the probability that EDP exceeds a certain threshold edp given an intensity measure im, $\Pr(EDP > edp \mid IM = im)$, is presented. The edp that has been initially selected is the seismically induced permanent total lateral displacement of the ground surface (D). The total probability theorem can be used to estimate the required conditional probability $\Pr(D > d \mid im)$.

Let L be the event that liquefaction occurs, and $\overline{L}$ the event that liquefaction does not occur. The probability that the seismic displacement exceeds a specified threshold of displacement given an intensity measure, $\Pr(D > d \mid im)$, can be estimated as

$$\Pr(D > d \mid im) \quad = \Pr(D > d \mid im, L)\Pr(L \mid im) + \Pr(D > d \mid im, \overline{L})\Pr(\overline{L} \mid im) \tag{6}$$

The probability of the occurrence of liquefaction given an intensity measure, $\Pr(L \mid im)$, can be estimated making using Equation (1) following the recommendations of Seed et al. (2003). As stated previously, this relation can provide an estimate of $\Pr(L \mid im, n_{1,60})$ if the other input parameters of Equation (1) are assumed to be constant. Let $f_{N_{1,60}}(n_{1,60})$ be the PDF of the random variable $N_{1,60}$. The total probability theorem can be used to estimate the probability of liquefaction given an intensity measure as

$$\Pr(L \mid im) = \int_{-\infty}^{+\infty} \Pr(L \mid im, n_{1,60}) f_{N_{1,60}}(n_{1,60}) dn_{1,60} \tag{7}$$

where it has been assumed that $N_{1,60}$ is independent of the intensity measure (im). The probability that liquefaction does not occur given an intensity measure can be calculated as

$$\Pr(\overline{L} \mid im) = 1 - \Pr(L \mid im) \tag{8}$$

The probability that the seismic lateral displacement exceeds a specified displacement threshold is evaluated using Equation (3) based on the work of Bray and Travasarou (2007). The evaluation of $\Pr(D > d \mid im, L)$ or $\Pr(D > d \mid im, \overline{L})$ can be thought as the estimation of the probability of failure of a component, where "failure" is defined as the event $\{D > d\}$ given an intensity measure and that liquefaction has (or has not) occurred. In reliability theory terms, the limit-state function of the problem is $g(\underline{x}) = d - D(\underline{x})$, where $\underline{x}$ represents a vector of random variables, and $D(\underline{x})$ comes from Equation (3). The vector of random variables in this case is $\underline{x} = [S_u \quad \alpha \quad \varepsilon]^T$, where $S_u \equiv S_{ur}$ if liquefaction occurs, and the parameter α is related to the distance between points of fixity in the piles.

There are several techniques available to estimate the probability $\Pr(g(\underline{x}) \leq 0)$. In the proposed procedure, the First-Order Reliability Method (FORM) is used. Preliminary results indicate that FORM gives reasonably accurate results when compared to Monte Carlo Simulations or the results from Second-Order Reliability methods (SORM). The program FERUM was used to evaluate the required probabilities (see < http://www.ce.berkeley.edu/FERUM/ >).

Inclusion of the Pile Pinning Effect

If the pile-pinning effect and the passive reaction at the abutment are included, the equivalent shear strength of the critical layer is a function of the level of expected displacement (D), which means that the yield coefficient is also a function of D and that the evaluation of $D(\underline{x})$ needs to be performed iteratively.

Let p be a function that relates the value of k_y and the total equivalent shear strength of the critical layer (S), i.e. $k_y = p(S)$. This function can be estimated by solving the pseudo-static seismic stability problem at the abutment with different "reasonable" values of S, between S_{min} and S_{max}. Let $\overline{S}_u$ be the mean undrained shear strength of the critical layer, N the total number of piles, M_p the plastic bending moment of each pile, H the thickness of the liquefiable layer, F_P^{max} the total maximum passive reaction at the abutment, and A the area of the horizontal portion of the failure surface. A reasonable lower limit for S is the mean undrained shear strength of the critical layer, i.e. $S = S_{min} \approx \overline{S}_u$. An upper limit for S can be estimated considering that the piles have reached their plastic limit, that the distance between points of fixity is equal to the thickness of the critical layer, and that the passive reaction at the abutments has reached its maximum, in that case $S = S_{max} \approx \overline{S}_u + N \times 2M_p / (H \times A) + F_P^{max} / A$. Preliminary results indicate that for cases when there is a distinct weak layer, this function is roughly linear.

When the "pile-pinning" effect is included, the shear force in the piles can be estimated using Equation (4), and the total equivalent shear strength of the liquefiable layer can then be expressed as

$$S = S_u + \frac{N \times 2M(D)}{(H + 2(\alpha \cdot 2R))A} + \frac{F_P(D)}{A} \tag{9}$$

where the term $M(D)$ indicates that the bending moment in the piles is a function of the lateral displacement, and $F_P(D)$ represents the passive reaction at the abutment, which is also a function of D. As it has been assumed that $k_y = p(S)$, then

$$k_y = p\left(S_u + \frac{N \times 2M(D)}{(H + 2(\alpha \cdot 2R))A} + \frac{F_P(D)}{A} \right) \tag{10}$$

The combination of equations (3) and (10) results in an implicit non-linear equation for D. However, through iteration, a consistent set of estimated seismic displacement and yield coefficient that includes the pile pinning effect can be obtained, and the seismic performance of the piles can be assessed.

DESIGN EXAMPLE

Problem

The proposed methodology will be applied to the case shown in Figures 2-4 considering that the seismically induced permanent longitudinal lateral deformation of the piles is the governing engineering demand parameter and the proper index of the overall seismic performance of the bridge foundation.

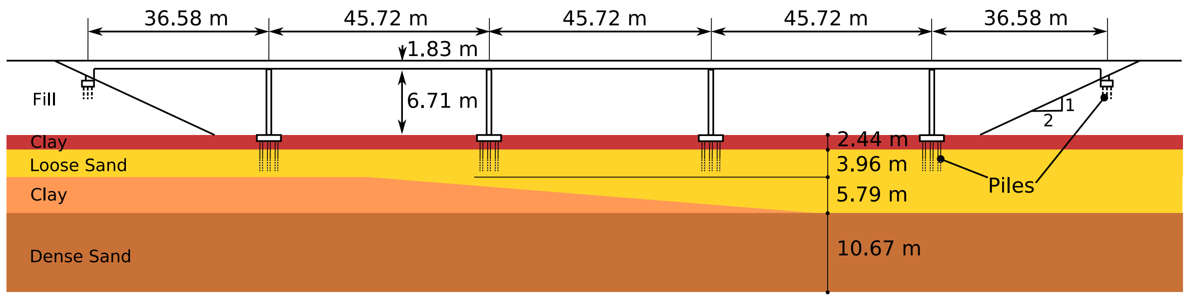
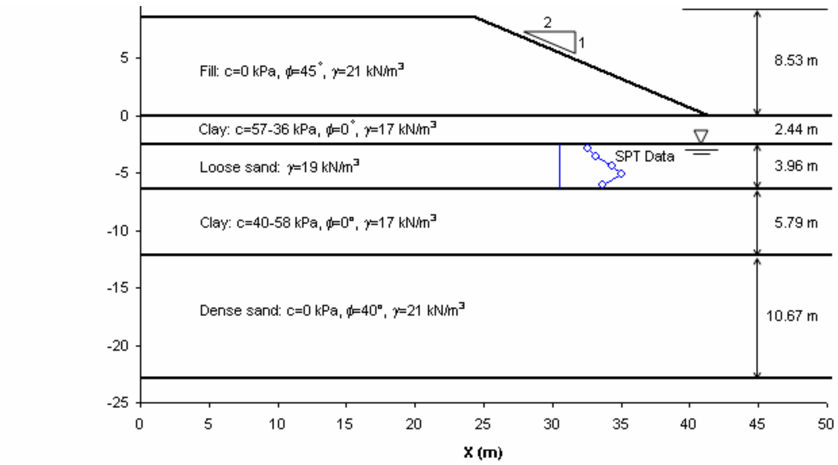


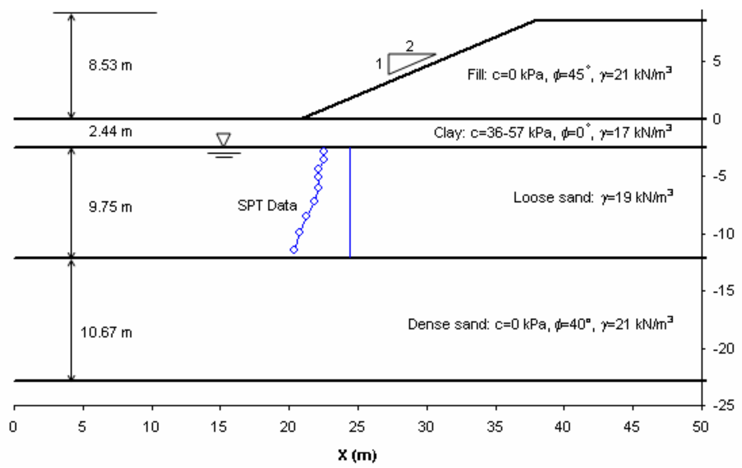
Figure 2. Example bridge profile



SPT Data

Depth (m)	N1,60-CS
2.8	15
3.6	19
4.3	27
5.1	32
6.0	22

Figure 3. Left abutment – soil profile and dimensions (not to scale)



SPT Data

Depth (m)	N1,60-CS
2.83	13
3.57	13
4.33	16
5.12	16
6.00	16
7.19	18
8.53	22
9.91	25
11.43	28

Figure 4. Right abutment – soil profile and dimensions (not to scale)

The foundation system consists of circular open-ended steel pipe piles that are arranged in groups of 3×2 at each interior bent and 6×1 at each abutment. The steel pipe piles have an external diameter of 61 cm, thickness of 1.3 cm, and yield stress of 414 MPa.

Assumptions

– There is only one critical sliding surface at each abutment.
– At the left abutment, if liquefaction occurs, the base of the potential failure surface goes through the bottom of the $N_{1,60\text{-}CS}$=15 material in the SLOPE/W model (GEOSLOPE/W International, Ltd., 2004). Assume c.o.v.= 0.3, which means that the standard deviation of this parameter would

be around 4.5. Hence, at the left abutment, the SPT blow-count at the critical layer has a *lognormal* distribution with mean 15, and a standard deviation of 4.5.

- At the right abutment, if liquefaction occurs, the base of the potential failure surface goes through the bottom of the lower $N_{1,60\text{-CS}}$=13 material. Two liquefiable layers are involved in the potential failure surface. The two layers have the same SPT blow-count, i.e., $N_{1,60\text{-CS}}$=13. Again, assume c.o.v.= 0.3, which means that the standard deviation of this parameter should be around 3.9. Hence, at the right abutment, the SPT blow-count at the equivalent critical layer has a *lognormal* distribution with mean 13, and a standard deviation of 3.9.
- Based on the work presented by Eddy and Gutierrez (2006) assume that the undrained residual shear strength S_{ur} given a value of $N_{1,60\text{-CS}}$ has a *lognormal* distribution with mean $0.1(N_{1,60\text{-CS}})^2 + 0.9(N_{1,60\text{-CS}})$. Their results show that the c.o.v. is around 0.22. Duncan (2000) indicates that a reasonable range for the c.o.v. of the undrained shear strength S_u is 0.13 to 0.40. It will be assumed that the c.o.v. of S_{ur} given $N_{1,60\text{-CS}}$ is 0.25.

Calculations

If liquefaction does not occur, the base of the potential failure surface goes through the bottom of the shallow clay layer at both abutments. In that case, the initial fundamental period (T_s) of the potential sliding mass is 0.13 seconds, and the k_y versus S relationship is $k_y = 0.00835 \cdot S - 0.1473$, with S in kPa (correlation coefficient $R^2 = 0.9981$).

If liquefaction occurs, the initial fundamental period (T_s) of the potential sliding mass is approximately 0.15 seconds for the left and right abutment; and the k_y versus S relationships are $k_y = 0.00627 \cdot S - 0.0587$ and $k_y = 0.00627 \cdot S - 0.0842$, with S in kPa, for the left and right abutment, respectively (R^2 = 0.9981 and 0.9998, respectively).

The in situ cyclic shear stress ratios (CSR) are 0.68·PGA and 0.70·PGA for the left and right abutment, respectively.

According to Caltrans Design Criteria (Caltrans, 2006), the longitudinal stiffness of the abutment is 11.5 (kN/mm)/m × 13.11 m × 1.83/1.7 = 162 kN/mm, and the total passive capacity is 1.83 m × 13.11 m × 239 kPa × 1.83/1.7 = 6170 kN. This means that the lateral displacement required to reach the total passive capacity must be 6170/162 = 38 mm = 3.8 cm (~2% of wall height). Because for the three hazard levels under consideration the probability of exceeding such deformation is high, and for the sake of simplicity, it has been assumed that the total passive capacity is constant and equal to 6170 kN for all levels of abutment displacement.

The peak ground accelerations for the 50%, 10%, and 2% in 50 years events at the example site, which is in Northern California and located about 10 km from the Hayward fault, are 0.34 g, 0.64 g, and 0.90 g, respectively, based on a Probabilistic Seismic Hazard Assessment (PSHA) performed by Somerville and Collins (2005). Also, the spectral accelerations at $1.5T_s$ are 0.87 g, 1.6 g, and 2.3 g, respectively for the left and right abutments ($T_s \sim 0.15$ s).

Results

The probabilities of liquefaction for the 50%, 10%, and 2% in 50 years events are 85%, 99%, and 99.7%, respectively, for the left abutment, and 94%, 99.7%, and 99.9%, respectively, for the right abutment. Resulting estimates of the expected seismic displacement at each abutment based on the proposed procedure are summarized in Table 1 and Figure 5. These estimates include the beneficial effect of the piles and the abutment, which both resist lateral spreading.

Table 1. Expected seismic permanent lateral displacements for the three hazard levels

	Left Abutment			Right Abutment		
	Hazard Level			Hazard Level		
	50%in50yr	10%in50yr	2%in50yr	50%in50yr	10%in50yr	2%in50yr
16th Percentile	0 cm	4 cm	11 cm	2 cm	7 cm	17 cm
Median	2 cm	10 cm	26 cm	4 cm	17 cm	39 cm
84th Percentile	5 cm	23 cm	61 cm	9 cm	38 cm	86 cm

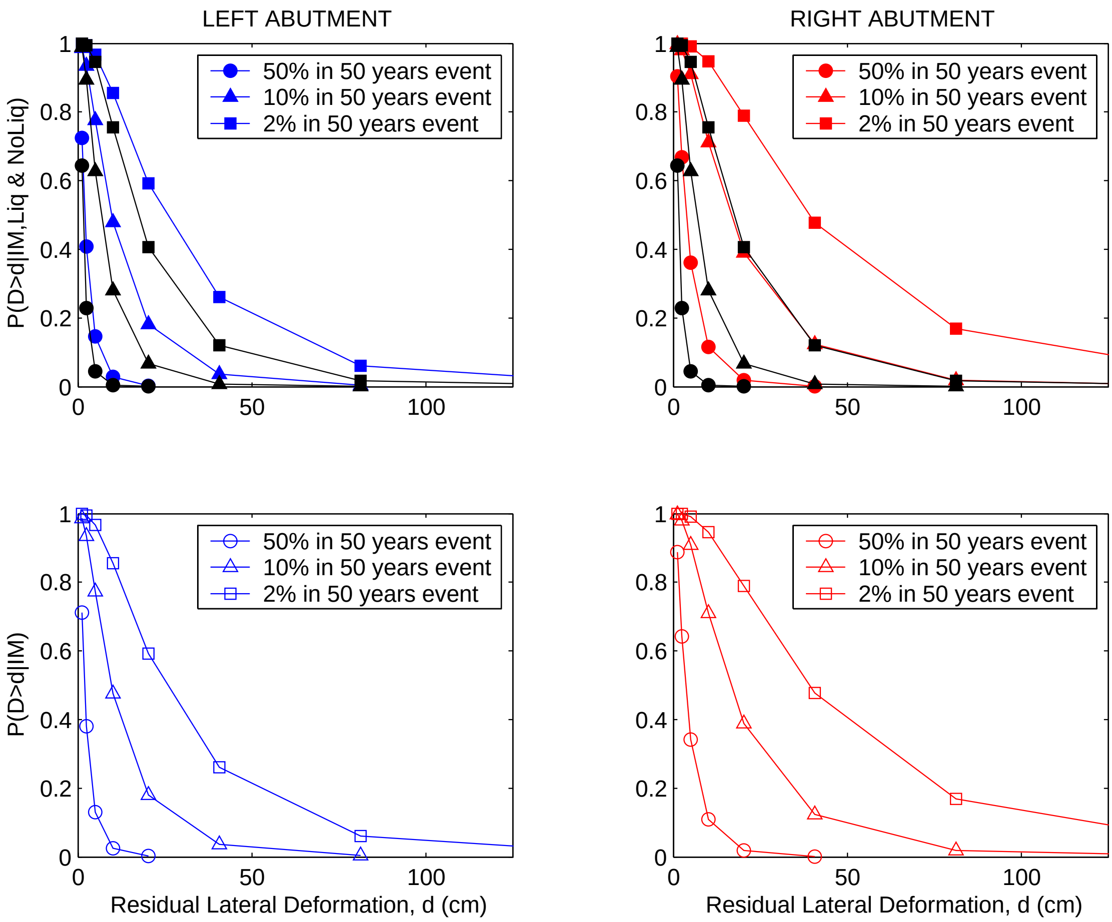


Figure 5. Probability of seismic displacement (*D*) exceeding a threshold of *d* given the *IM* value at each hazard level. Upper figures in each column represent the probability of displacement for each liquefaction case, where the black lines correspond to the case without liquefaction and the color lines represent the case with liquefaction. The bottom figures in each column show the final result, where the cases with and without liquefaction have been combined using the total probability theorem

Comparison with results from a deterministic analysis

The bottom plots in Figure 5 show the probability, $\Pr(D > d \mid im)$, that the seismic displacement exceeds a threshold value of displacement as a function of the ground motion intensity measure of spectral acceleration at the initial fundamental period of the potential sliding mass at each hazard level. As a comparison, the results from a deterministic analysis using mean values for the random variables and considering that liquefaction occurs for all values of *im* are:

- For the left abutment: the seismic displacements are approximately 2 cm, 9 cm, and 22 cm, for events having a probability of exceedance of 50%, 10%, and 2% in 50 years, respectively.
- For the right abutment: the seismic displacements are approximately 2 cm, 11 cm, and 27 cm, for events having a probability of exceedance of 50%, 10%, and 2% in 50 years, respectively.

Although these deterministic estimates of seismic displacement are not inconsistent with the median results from the probabilistic analysis, they do not provide information regarding the uncertainty of the estimate. Having the ability to develop a range of seismic displacements (i.e. using the 16% and 84% exceedance values) is considered to be significant advancement. Additionally, the proposed methodology can be incorporated within a fully probabilistic PBEE evaluation of the entire bridge system.

CONCLUSIONS

The pile pinning effect has been employed as a remediation measure in a number of engineering projects to reduce the amount of movement likely to occur as a result of liquefaction-induced lateral spreading. This remediation measure has been formally incorporated in design guidance documents such as MCEER/ATC-49-1 for the design of bridges with pile foundations. Its use is becoming more widespread in highly seismic regions where pile foundations are installed to support wharfs, bridges, and buildings in ground susceptible to liquefaction-induced lateral spreading. Other ground modification techniques can be employed to reduce the liquefaction hazard, however, they can be costly and unnecessary if the piles can be shown to arrest the lateral spreading and achieve acceptable seismic performance.

A simplified probabilistic procedure for evaluating the seismic performance of pile-supported bridges that are subjected to liquefaction-induced lateral spreading has been developed. The proposed procedure formally incorporates the “pile-pinning” effect. Moreover, the uncertainty in parameters such as the SPT blow counts, the residual undrained shear strength of the critical liquefied soil layer, the distance between points of fixity of the piles, and the probability of occurrence of liquefaction have been incorporated in the proposed procedure so that the uncertainty involved in each of these key parameters can be incorporated in the overall seismic evaluation of the engineered system. Additionally, the assumptions in terms of types of distributions and some derived relationships can be easily modified to study their impact on the final result.

The procedure that has been developed is compatible with the Performance-Based Earthquake Engineering framework proposed by PEER. The final goal of our research is to provide the designer with a robust decision tool for the type of problems described in this paper. In this context, the next step will be to find reasonable relationships between seismic displacement and other important EDPs with damage measures and decision variables.

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