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Risk evaluation of existing piled foundations in liquefiable soils

Risquer l'évaluation de fondations entassées existantes dans les sols liquéfiables

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ABSTRACT
The current methods of pile design under earthquake loading is based on a bending mechanism. These codes do not consider the effect of axial load that the pile continues to carry at full liquefaction. Recent research has revealed, however, that buckling can occur in piles due to axial load and loss of soil stiffness owing to liquefaction. It is, therefore, necessary to evaluate the risk of the existing piled foundations designed by the current codes of practice. This paper identifies the dominant parameters for such a risk evaluation.

RÉSUMÉ
Les méthodes actuelles de conception de tas sous le chargement de tremblement de terre sont basées sur un mécanisme courbant. La recherche récente a révélé, cependant, cet attacher peut arriver dans les tas en raison du chargement et la perte axiales de raideur de sol dû à la liquéfaction. C'est, donc, nécessaire d'évaluer le risque des fondations entassées existantes conçues par les codes actuels de pratique. Ce papier identifie les paramètres dominants pour une telle évaluation de risque.

1 INTRODUCTION

1.1 *Current understanding of pile failure i.e. bending mechanism*

Collapse of piled foundations in liquefiable soils is still observed after strong earthquakes; for example 1995 Kobe or 2001 Bhuj earthquake. The current methods of pile design under earthquake loading, such as Eurocode 8, part 5 (1998), NEHRP (2000), Japanese Highway Code (JRA) or the Indian code (IS 1893) is based on a bending mechanism where inertia and slope movement (lateral spreading of soil) induce bending moments in the pile. Piled foundations often collapse by forming plastic hinges. This suggests that the bending moment or shear forces experienced by the pile exceed the plastic moment capacity of the pile. All current design codes provide a high margin of safety using partial safety factor, yet occurrences of pile failure are abundant. Bhattacharya (2003) has shown that the overall factor of safety against plastic yielding of a typical concrete pile can range between 4 and 8. This is due to the multiplication of partial safety factors on load (1.5), material (1.5 for concrete), fully plastic strength factor (Z_P/Z_E = 1.67 for circular section) and practical factors such as minimum reinforcements. This implies that the actual moments or shear forces experienced by the pile are 4 to 8 times those predicted by their design methods. Bhattacharya and Bolton (2004) have shown that bending mechanism due to lateral loads cannot always explain a pile failure. They have used the well-known case study of the Showa Bridge failure to illustrate that although the design of the piles in the bridge satisfies the latest guidelines of JRA (1996), yet the bridge actually failed during the 1964 Niigata earthquake.

1.2 *Buckling as a feasible pile failure mechanism in areas of seismic liquefaction*

Research carried out by Bhattacharya et al. (2004) has shown that if piles are too slender they require lateral support from the surrounding soil if they are to avoid buckling instability. During earthquake-induced liquefaction, the soil surrounding the pile loses effective confining stress and can no longer offer sufficient support to the pile. A slender pile may then buckle sideways in the direction of least elastic bending stiffness pushing aside the initially liquefied soil, and eventually rupturing under the increased bending moment and shear force. Lateral loading due to slope movement, inertia or out-of-straightness increases lateral deflections, which in turn induces plasticity in the pile and reduces the buckling load, promoting more rapid collapse. These lateral loads are, however, secondary to the basic requirements that piles in liquefiable soil must be checked against Euler's buckling. This theory has been formulated based on a study of fourteen case histories of pile foundation performance and verified using dynamic centrifuge tests. Analytical studies also support this theory of pile failure, see for example Kimura and Tokimatsu (2004). In other words, part of the pile in liquefiable soil needs to be treated as an unsupported structural column. In contrast, the piles in liquefiable soils are erroneously designed as beams.

1.3 *Need for risk evaluation of existing piled foundations designed based on bending mechanism*

Beam bending and column buckling require different approaches in design. Bending is a stable mechanism as long as the pile is elastic, i.e. if the lateral load is withdrawn, the pile comes back to its initial configuration. This failure mode depends on the bending strength (moment at first yield, M_Y; or plastic moment capacity, M_P) of the member. On the other hand, buckling is an unstable mechanism. It is sudden and occurs when the elastic critical load is reached. It is the most destructive mode of failure and depends on the geometrical configuration of the member, i.e. slenderness ratio, and not on the yield strength of the material. Bending failure may be avoided by increasing the yield strength of the material, i.e. by using high-grade concrete or additional reinforcements, but it may not suffice to avoid buckling. To avoid buckling, there is a need for a minimum pile diameter depending on the thickness of the liquefiable soil, such as Fig. 1. Thus there is need to reconsider the safety of the pile-supported structures (buildings and bridges) whose piles are designed based on a bending mechanism.

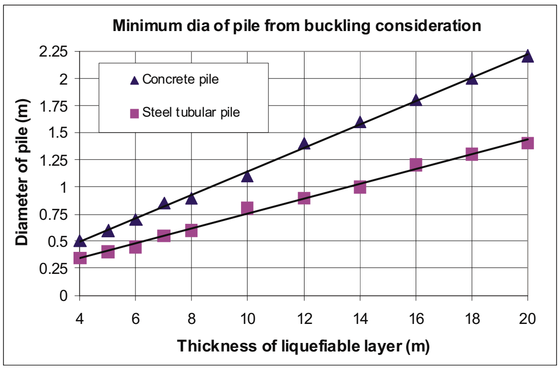

Figure 1: Minimum diameter of pile based on buckling considerations, Bhattacharya and Tokimatsu (2004).

1.4 *Aim of the paper*

The paper aims to identify the parameters involved in the risk evaluation of existing structures founded on end-bearing piles designed based on a bending mechanism. As many of such parameters are likely to be uncertain (random), a probabilistic approach of analysis would be more appropriate. A computational scheme for reliability (risk) analysis has been developed to determine the probability of failure of piled foundations as well as its sensitivity to the different random parameters.

2 THEORETICAL BACKGROUND BEHIND THE RISK ASSESMENT

This section of the paper describes the theoretical background behind the mathematical formulation for the risk assessment. The case of a piled building/ tank (Fig. 2) is considered to demonstrate the methodology. The different stages of loading in a piled foundation are explained schematically in Bhattacharya et al (2004). P is the static axial load acting on each pile beneath the building/tank assuming that each pile is equally loaded during static condition neglecting any eccentricity of loading. However during earthquakes, inertial action of the superstructure will impose dynamic axial loads on the piles which will increase the axial load on some piles. This increase may range between 10% to as high as 50% depending on various factors such as the type of superstructure, height of the centre of mass of the superstructure. This factor can be called as "Dynamic Axial Load Factor" denoted by α.

$$P_{dynamic} = (1+\alpha)P_{static} \tag{1}$$

For buckling analysis, each pile needs to be evaluated with respect to its end conditions (fixed, pinned or free). Each pile in a group of identical piles will have the same buckling load as a single pile. If a group of piles is fixed in a stiff pile cap and embedded sufficiently at the tip, as in Fig. 2, the pile group may buckle in either side sway or in simple vertical displacement. For side sway buckling mode, which is the most likely mode of failure, the effective length (L_{eff}) for the pile is equal to the unsupported length, Bhattacharya et al. (2004). The unsupported length of the pile is equal to the thickness of liquefiable soil (D_L) plus some additional length necessary for fixity at the bottom of the liquefiable soil. The theoretical buckling load (P_{cr}) is given by equation 2.

$$P_{cr} = \frac{\pi^2 EI}{L_{eff}^2} \tag{2}$$

A parameter called "Critical Depth (H_c)" is now defined to identify the unsupported length of the pile required for buckling instability. This is essentially the length of the pile necessary to be unsupported during the liquefaction process for the initiation of buckling. Dynamic centrifuge tests carried out by Bhattacharya (2003) have shown that buckling initiates when a front of zero effective stress often known as liquefaction front reaches this critical depth. This is schematically shown in Figure 2. A piled structure becomes unstable when the critical depth is less than the thickness of the liquefiable soil (D_L) assuming earthquake intensity is strong enough to cause liquefaction to this depth.

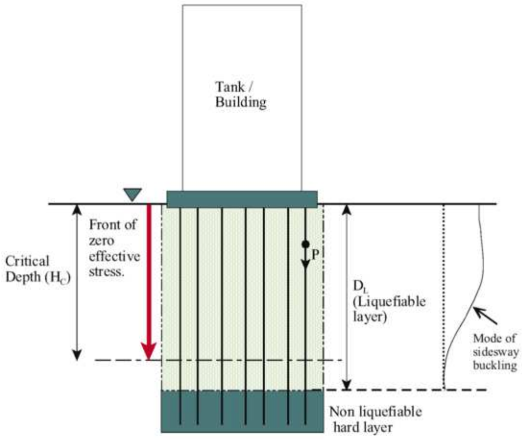

Figure 2: Schematic diagram showing the concept of "Critical depth" and the mode of sway buckling.

The mechanism of failure of piles in liquefiable soil is to a great extent similar to the failure of slender columns. As a result fundamental principles for the analysis/design of slender struts can be applied to pile foundation. Experiments show that the actual failure load ($P_{failure}$) of slender columns is much lower than that predicted by equation 2. Rankine (1866) recognized that the actual failure involved an interaction between elastic and plastic modes of failure. Lateral loads or inevitable geometrical imperfections lead to creation of bending moments in addition to axial loads. Bending moments have to be accompanied by stress redistribution that diminish the cross-sectional area available for carrying the axial load, so the failure loads $P_{failure}<P_{cr}$. Equally, the growth of zones of plastic bending reduces the effective elastic modulus of the section, thereby reducing the critical load for buckling. Furthermore, these processes feed each other. As the elastic critical load is approached, all bending effects are magnified. Stability analysis of elastic columns, Timoshenko and Gere (1961), shows if lateral loads in the absence of axial load would create a maximum lateral displacement δ_0 in the critical mode-shape of buckling, then the displacement δ under the same lateral loads but with co-existing axial load P is given by:

$$\left(\frac{\delta}{\delta_0}\right) = \frac{1}{1-\frac{P_{dynamic}}{P_{cr}}} \tag{3}$$

The term (δ/δ_0) can be termed as "Buckling Amplification Factor" and is amplification of lateral displacements due to the presence of axial load ($P_{dynamic}$). It can be observed from equation 3 that if the applied load is 50% of P_{cr} the amplification of lateral deflection due to lateral loads is about 2 times. At these large deflections, secondary moments will generate which will lead to more deflections and thus more P-Δ moment. It must be mentioned here that structural engineers generally prefer to keep a factor of safety of at least 3 against linear elastic buckling to take into account the eccentricity of load, deterioration of elastic stiffness due to plastic yielding and unavoidable imperfections. The actual failure load ($P_{failure}$) is therefore some factor, ψ (ψ <1) times the theoretical Euler's buckling load given by equation 4.

$$P_{failure} = \psi . P_{cr} \tag{4}$$

For the specific case of the piled building/tank, ψ is taken as 0.35 i.e. a factor of safety of about 3. However, this factor will depend of the axial load ($P_{dynamic}$), imperfections or the residual stresses in the pile due to driving. For the type structure shown in Figure 2, $L_{eff} = H_C$, and thus the limiting axial load that can be applied is given by equation 5 derived from equation 2.

$$P_{dynamic} = 0.35 P_{cr} = \frac{0.35\pi^2 EI}{H_C^2} \tag{5}$$

Rearranging equation 5 gives the estimate of "Critical Depth H_C", equation 6, for a pile for the type of structures shown in Figure 2.

$$H_C = \sqrt{\frac{3.45EI}{P_{dynamic}}} \tag{6}$$

3 RELIABILITY ANALYSIS

In this paper, an attempt has been made to develop a computational scheme to estimate the probability of failure by buckling instability of an existing piled foundation for a future earthquake. Reliability analysis is carried out using Mean-Value First Order Second Moment method (MFOSM), Ang and Tang (1984). This section formulates the reliability analysis for the piled foundation.

3.1 *Performance function and the failure surface equation*

As discussed in the earlier section, the stability of the piled foundation under the action of axial load depends on the depth of liquefaction (D_L) and the "Critical depth" (H_C). The failure surface equation can be thus defined as follows:

$$g(\mathbf{X}) \equiv (H_c - D_L) = 0 \tag{7}$$

where, H_C = Critical pile depth calculated from equation 5;
D_L = Depth of liquefaction;
X = Vector representing the set of random variables;
In the above,

$g(\mathbf{X}) < 0$ indicates failure state; $g(\mathbf{X}) = 0$ indicates limit state; $g(\mathbf{X}) > 0$ indicates safe state

Failure in this context would mean that a piled foundation would become laterally unstable under the axial load and inevitable imperfections when the soil surrounding the piles liquefies.

3.2 *Identification of variables for risk assessment analysis.*

The random variables identified for risk assessment can be categorized under four main headings, such as
Earthquake characteristics. To estimate the depth of liquefiable soil, using Idriss and Boulanger (2004), the random variables identified are M (moment magnitude of the earthquake) and a_{max} ie. Peak Ground Acceleration (P.G.A).
Pile characteristics: Following equation 6, the pile has been characterized by axial load ($P_{dynamic}$), Young's Modulus of the material of the pile (E) and pile diameter (D). From the diameter, moment of inertia (I) can be estimated.
Soil profile and ground condition: To use Idriss and Boulanger (2004), the soil at site has been characterized by SPT profile and an average fines content (FC) over the entire profile. The discrete SPT–N values can be treated as random variable and thus the depth wise variation can be included. The location of the water table (Z_{WT}) is also a random variable as it varies in different seasons. The slope of the terrain (θ), which is also a factor, can also be included in the analysis as a variable.
Type of superstructure: The superstructure will dictate the "Dynamic Axial Load Factor" α defined by equation 1 and "Buckling Amplification Factor due to lateral load"ψ defined in equation 4. The random variables are α and ψ.
It must now be mentioned that most of the parameters are uncertain and may be treated as random or probabilistic rather than deterministic. As a result reliability or probabilistic analysis rather than a conventional deterministic analysis is best suited.

3.3 *Reliability index (β), probability of failure (p_F)*

For the performance function g (X) defined in equation 7, the reliability index (β) based on the MFOSM method of reliability is given by equation 8.

$$\beta = \frac{E[g(X)]}{\sigma[g(X)]}$$

$$= \frac{H_c(\mu_{x_i}) - D_L(\mu_{X_i})}{\sqrt{\sum_{i=1}^{n}\left(\frac{\partial g}{\partial X_i}\right)^2 \sigma^2[X_i] + 2\sum_{i,j=1}^{n}\left(\frac{\partial g}{\partial X_i}\right)\left(\frac{\partial g}{\partial X_j}\right)\rho\,\sigma[X_i]\sigma[X_j]}} \tag{8}$$

where n = Number of random variables. Based on section 3.2, the random varibles are a_{max}, M, E, D (outer dia. as well as inner dia. for a tubular pile), P, FC, α,ψ, a set of SPT values upto sufficient depth, Z_{WT},θ.
E[g(X)]= Expected value i.e., the most likely value of the performance function;
σ[g(X)]= Standard deviation of g(X);
μ_{xi}= Mean value of design parameter X_i;
$\sigma[X_i]$ = Standard deviation of X_i;
ρ= Correlation coefficient between X_i and X_j.

In this study, the random parameters have been assumed to be uncorrelated and therefore, ρ = 0. Equation 8 thus takes the form as given in equation 9.

$$\beta = \frac{H_c(\mu_{X_i}) - D_L(\mu_{X_i})}{\sqrt{\sum_{i=1}^{n}\left(\frac{\partial g}{\partial X_i}\right)^2 \sigma^2[X_i]}} \tag{9}$$

In this study, the partial derivatives have been evaluated based on Finite Difference technique. Once the value of the reliability index, β, is determined, the probability of failure, p_F is then obtained using equation 10.

$$p_F = \Phi(-\beta) \tag{10}$$

where Φ(.) is the standard normal cumulative probability density function values of which are tabulated in standard text books.

3.4 *Sensitivity analysis of the parameters.*

It is often necessary to identify the parameters that strongly influence the failure. Once such parameters are identified, efforts can then be made to make best estimates of such parameters that will lead to a better reliability calculation. The usual technique of identifying such parameters is a thorough parametric study wherein each parameter is varied and the resulting change in the values of the probability of failure noted. However, such procedure involves large computation making it unattractive and sometimes impractical. In structural reliability analysis, sensitivity of a random variable is expressed in terms of its 'Importance Factor' defined by equation 12 following Adhikary and Langley (2002).

$$I_{F_i} = \frac{\left(\frac{\partial \beta}{\partial X_i}\right)}{\sqrt{\sum_{i=1}^{n}\left(\frac{\partial \beta}{\partial X_i}\right)^2}} \tag{12}$$

where I_{Fi} is the importance factor for the i^{th} random parameter. The above computational scheme has been coded in a FORTRAN computer program and has been used to carry out the computations for an example problem.

4 ILLUSTRATIVE EXAMPLE

The section of the paper demonstrates the application of the reliability and sensitivity analysis using a well-documented case history. Figure 3 shows a three storied building that collapsed during the 1995 Kobe earthquake (Moment mag. M = 7.2) along with the soil profile. Details of the case history can be seen in Tokimatsu et al (1998).

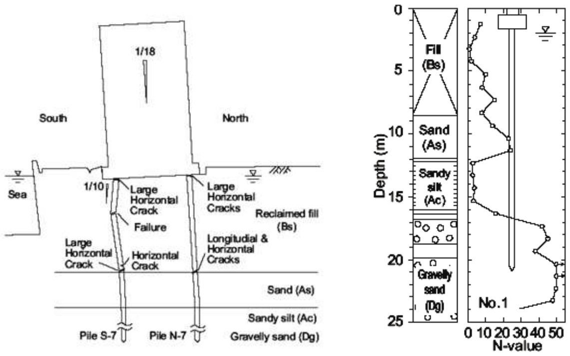

Figure 3: A piled-building that collapsed during the 1995 Kobe earthquake along with the soil profile, Tokimatsu et al (1998).

The static axial load (P_{static}) acting in the building is reported to be 416kN. Input motion measured in the nearby Higashi-Kobe Bridge showed a peak ground acceleration of 0.38g. However, the PGA at the building site is not known. The piles used in the building had an external dia. (D_O) of 0.4m and inside dia. (D_i) of 0.24m. A preliminary analysis incorporating the SPT values and Z_{WT} in the random vector revealed that the p_F for this type of structure is insensitive to these variables. The soil profile and the water table is taken as deterministic and thus not included as a variable. Thus there are altogether nine variables in the analysis, which are uncertain and therefore treated as random. Because the probability distributions and co-variances of these variables are not known, it is assumed that they are normally distributed and uncorrelated. Further due to non-availability of the relevant data, a uniform coefficient of variation of 0.1 has been assumed for all the variables. Table 1 lists the random variables and the result of the analysis. The data available from the case history have been considered as the mean value of the respective parameters/ random variables.

It may be noted that the outside diameter has the highest value of importance factor. The second most dominant factor is the ψ value i.e. the buckling amplification factor for lateral load. The third dominant factor is the internal diameter. α, M, a_{max}, comes next in the order of importance. The analysis predicts that the probability of failure of the building is 73.5% with the input values chosen. Going by the conventional FACTOR OF SAFETY (F.O.S) approach, the FOS against buckling failure will be given by the ratio of H_C to D_L which is 0.83. It must be mentioned that the building actually failed.

Table 1: Statistical property of the parameters and the results of the reliability analysis.

Parameters/ random variables	Input	Output of the program	
	Mean value	Importance factor (I_F)	Summary
a_{max} (P.G.A)	0.38g	-0.014	D_L (mean) = 16.59m H_C (mean) = 13.75m β = -0.63 p_F = 73.5%
M (Earthquake magnitude)	7.2	-0.049	
D_0 (Outside dia.)	0.4m	0.95	
D_i (Inside dia.)	0.24m	-0.21	
E (Young's modulus)	25GPa	3.29×10^{-9}	
P_{static} (Axial load)	416kN	-1.9×10^{-4}	
FC (Fines content)	14	2.13×10^{-4}	
α(Dynamic axial load factor)	0.2	-0.068	
ψ (Buckling amplification factor for lateral loads)	0.35	0.235	

5 CONCLUSIONS

The current methods of pile design in liquefiable soils are based on bending mechanism. A recent study has established that buckling is a feasible pile failure mechanism. There is thus a need to evaluate the safety of the existing piled foundation against buckling instability. The parameters for risk evaluation against buckling instability have been identified. The parameters have been classified into the earthquake characteristics, pile characteristics, soil profile, ground condition at the site and the type of superstructure. As many of the parameters are likely to be uncertain, a probabilistic approach of risk analysis rather than a conventional factor of safety approach is well suited and used. Computational scheme using MFOSM (Mean Value First Order Second Moment) method has been formulated and tested using a well-documented case history. The method predicted a high probability of failure for the parameters used in this case history of a failed piled foundation. Geometric configuration of the pile i.e. the external diameter comes out to be most dominant parameter in the sensitivity analysis.

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