Design of piles in liquefiable soil: A review of design codes and methodologies

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SUMMARY:

Piles are generally viewed as an acceptable foundation solution in liquefiable ground, and there are numerous case histories of piled foundations performing well where soil has liquefied. However, there are also case histories of piles with inadequate resistance to the additional loads imparted by liquefied soil and the associated loss of support. The design of piles in liquefiable soils requires careful consideration, both of the behaviour of the piles themselves and the impact on the supported structures.

This paper presents a comprehensive review of two issues faced by an earthquake engineer designing piles in such zones. Firstly, what are the statutory requirements? National and international standards and guidance documents are reviewed, and specific requirements for pile design and performance, are summarized. Secondly design methodologies, ranging from simplified to complex models are reviewed.

Finally, a brief comparison of selected methodologies is presented. to illustrate the advantages of more rigorous approaches. The implications in terms pile performance are discussed. The paper provides a useful and current overview of this aspect of seismic foundation design for practising engineers.

Keywords: liquefiable, Pile, Design Code

1. INTRODUCTION

Piled foundations are often adopted as a foundation solution in potentially liquefiable soil, due to their proven ability to perform well in past earthquakes where soil has liquefied due to seismic loading. Nonetheless, there are case histories where piles have had inadequate resistance to the additional loads imparted by liquefied soil and the associated loss of support.

The concept of Performance Based Earthquake Engineering (PBEE) places an emphasis on performance based design for geotechnical structures (ISO-23469, 2005). This implies engineering evaluation and design of structures whose seismic performance meets the objectives of modern society (Cubrinovski, 2009). Performance based design recognises that seismic loading is an imposed deformation and therefore the deformation demands for a chosen earthquake level should be quantified. Thus the imposed deformation can be compared against deformation limits at both global and local component levels. Currently there is limited guidance on applying a performance based framework to the performance of piles, particularly in liquefiable soil.

This paper presents a comprehensive review of two issues faced by a geotechnical earthquake engineer when designing piles in seismic zones where ground conditions could lead to liquefaction. Firstly, what are the statutory requirements that should be followed in such circumstances? National and international standards and guidance documents are reviewed, and specific requirements for pile design and performance, both in liquefied and non-liquefied soils, are summarised. Additionally performance criteria are presented and their application in typical cases is briefly discussed in the paper.

2. EARTHQUAKE-INDUCED PILE LOADS

When analysing the behaviour of piles in liquefied soils, it is useful to distinguish between two different phases in the soil-pile interaction process;

- A cyclic phase in the course of the intense ground shaking. The soil will impose a load on the pile due to its transient movement, whether liquefaction occurs or not.
- A permanent deformation phase following the occurrence of liquefaction. This may comprise lateral spreading or flow failure (where a free face is present), and/or vertical settlement. The permanent horizontal deformation of the ground around the pile imposes a load on the pile.

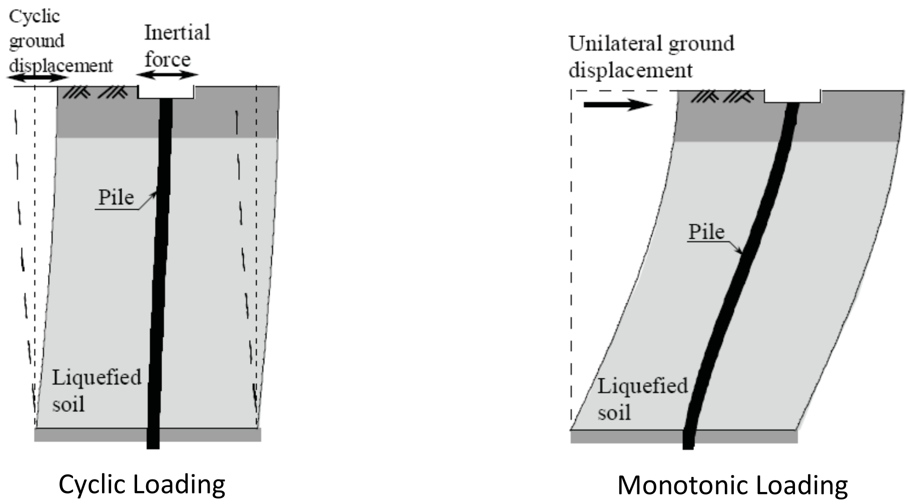


Figure 1: Loads on piles during cyclic loading and lateral spreading (Cubrinovski et al. 2007).

The total earthquake-induced loads on the pile, as shown in Figure 1, comprise:

- Inertial loads imposed by the superstructure to the pile head. This is a function of frequency of the superstructure and the input motion and varies as the stiffness of the soil changes. This is normally greatest in the initial part of the shaking, before the onset of liquefaction.
- Kinematic forces acting along the embedded length of the pile due to the movement of the soil. If there is non-liquefied material above liquefied soil, the combination of stiffer, non-liquefied material and large movements due to the underlying layer are particularly onerous.

3. CODE PROVISIONS FOR DESIGNING PILES IN LIQUEFIABLE SOIL

Design codes typically mandate a large margin of safety against plastic hinge formation in piles (through the use of partial factors). It is often 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 in the overall design. Further detail on specific code provisions is presented below. The effects of inertial and kinematic interaction are discussed in all the codes reviewed herein. Engineers clearly need to evaluate these effects when designing piles in liquefied ground.

3.1 Eurocode 8 (EN 1998-5:2004) Provisions

The default performance objective in EN1998 (subject to National Annexes) is that structures are designed for *No Collapse* under the 10% in 50 year event, with an importance factor to represent higher loads. In areas of potential liquefaction, EN1998:5 (4.1.4) advises that:

- The use of pile foundations alone should be considered with caution due to the large forces induced in the piles by the loss of soil support in the liquefiable layer or layers, and to the inevitable uncertainties in determining the location and thickness of such layers.

- Careful consideration should be given to any additional loading on the piles and the pile cap that arise due to lateral spreading, particularly in the presence of non liquefied soil strata overlying liquefiable soil. In addition, where liquefaction is anticipated it is stated that the contribution of liquefied layers to pile capacity should be ignored.

EN1998:5 specifies that piles should be designed in principle to remain elastic but "*may under certain conditions*" be allowed to develop a plastic hinge at their heads. The region of potential plastic hinging should be detailed according to Clause 5.8.4:

- a region of 2 pile diameters from the pile cap
- a region of ± 2 pile diameters from any interface between two layers with markedly different shear stiffness (ratio of shear moduli > 6)

3.2 JRA Provisions

The Japanese Highway Specification, JRA (1996, 2002) has incorporated the concept of "*top-down*" and "*bottom-up*" effects as shown in Figure 2. The code advises practicing engineers to design piles against bending failure assuming that the non-liquefied crust exerts passive earth pressure on the pile and the liquefied soil offers 30% of total overburden pressure. Dobry *et al*. (2003) studied a similar kind of soil-pile system, where the top portion of the pile is embedded in non-liquefiable clayey crust and terminated in a liquefiable soil. They suggested that the pressure on a pile due to the liquefied soil may be of the order of 10kPa.

Many researchers have verified their experimental results against such pressure distribution. The JRA code also advises designers to check against bending failure due to kinematic forces and inertia separately, i.e., a check against bending failure due to the combination of the two loads (inertial and kinematic) is not required.

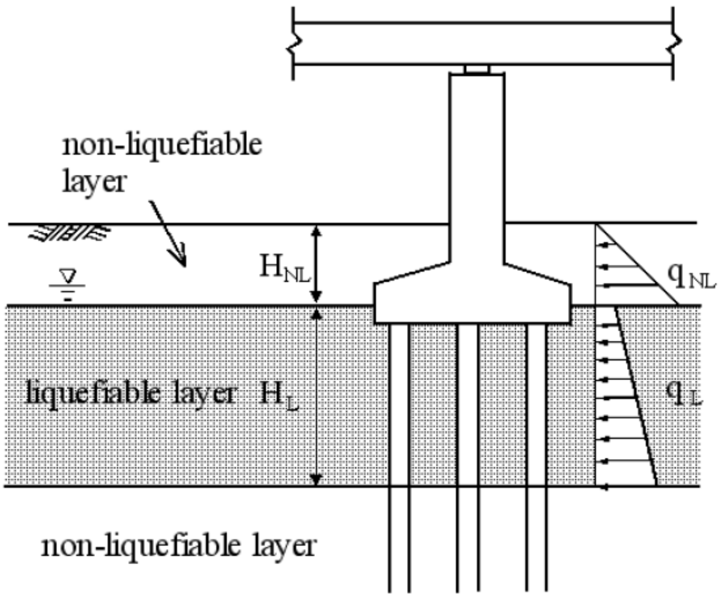


Figure 2: Japanese Road Association (JRA, 2002) design guidelines

3.3 ASCE 7-10

ASCE 7-10 does not provide any specific guidance on methods to be used to design for liquefaction. However according to Clause 12.13.6.3, *piling shall be designed and constructed to withstand deformations from earthquake ground motions and structure response. Deformations shall include* *both free-field soil strains (without the structure) and deformations induced by lateral pile resistance to structural seismic forces; all as modified by soil–pile interaction* (e.g. Figure 1). This implies that the code requires evaluation of the inertial and kinematic effects while designing the pile for liquefaction effects.

3.4 AASHTO (2010)

Clause 10.5.4.2 of AASHTO (2010) specifies that in Seismic Zone 4 (where the acceleration coefficient Sd_1 is greater than 0.5g) that *if liquefaction occurs then the bridge shall be designed and analysed for liquefied and non-liquefied conditions:*

Piles should in principle be designed to remain elastic, however, under certain circumstances a plastic hinge may be allowed to develop at the pile head, noting that *this plastic rotation does imply that the piles and possibly other parts of the bridge will need to be replaced if these levels of deformation do occur"*. Specifically, for sites where lateral flow due to liquefaction is anticipated, significant inelastic deformation may be permitted in the piles (AASHTO 2010). In such cases the elastic moment capacity of the pile shall not be exceeded by more than a factor of 2. The code is not explicit as to whether this would be using factored or unfactored loads, or as to whether this is the case only where lateral spreading is expected, or also where liquefaction with no lateral flow is expected. The commentary also notes that pile group effects are not considered significant for liquefied soil.

3.5 ISO-23469 (2005)

This code addresses issues of liquefaction and dynamic soil structure interaction in a systematic manner within a consistent framework of performance based design. According to clause K.3.1, in a simplified equivalent static analysis the effects of liquefaction are evaluated as follows:
Immediately after the triggering of liquefaction: through a reduction factor for subgrade reaction. The effects of ground displacement may be included if significant.

The performance criteria parameters based on ISO-23469 (2005) for pile foundations can be summarised as:

- Acceptable displacement of pile cap,
- Margins to the elastic limits specified in terms of shear force and overturning moment at the head of pile, and
- Acceptable residual response beyond the elastic limit of piles.

3.6 Analysis requirements

Codes do not tend to specify the complexity of analysis required. Many of them state explicitly that they represent the minimum standard required, and designers may go beyond this minimum standard where appropriate. The level of complexity must be decided on a case by case basis, and should consider the uncertainty of available information, and the uncertainty of the soil-structure-interaction that is to be modelled. The analysis of piles in liquefied soils is complex due to the uncertainties associated with modelling the behaviour of liquefied soil. Any analysis method should be able to handle these uncertainties in a reasonable manner (Cubrinovski *et al*., 2007).

The Caltrans (2011) guidance gives more guidance than most documents on the level of analysis required. However, it is a guidance document rather than a Code of Practice.

4. PERFORMANCE REQUIREMENTS

Performance requirements for piles often have to be inferred from the performance requirements for the superstructure that they are supporting. Consideration of the soil-pile-structure system in its entirety is necessary to understand the expected performance. Design options range from (a) an acceptance of the movements with potentially significant damage to the piles and columns if the movements are large, to (b) designing the piles to resist all forces within the elastic limit. Between these options a range of mitigation measures can be used to limit the amount of movement to tolerable levels for the desired performance objective (ISO, 2005).

Whilst the codes mentioned above make reference to performance-based design criteria such as the allowance of inelastic pile behaviour “under certain conditions”, none provide clear guidance as to appropriate margins of safety to use for different levels of performance.

The performance guidelines for any foundation component maybe set based on an acceptable level of damage to a structure following the PIANC (2001) guidelines as shown in Table 1. These guidelines are drafted specifically for marine structures but provide a useful categorisation method that could be used to define the performance of pile foundations.

The PIANC guidelines provide for four grades of performance. The performance grade should be based on the importance of the structure, the local seismic codes and standards and the requirements of the user/ operators of the facility. Based on these selections, the limits of the deformation, settlement and allowable deformation of the piles may be defined.

Table 1: Acceptable level of damage in Performance Based Design (PIANC, 2001)

Level of Damage	Structural	Operational
Degree 1: Serviceable	Minor or no damage	Little or no loss of serviceability
Degree II: Repairable	Controlled damage	Short term loss of serviceability
Degree III: Near Collapse	Extensive damage or near collapse	Long term or complete loss of serviceability
Degree IV: Collapse	Complete loss of structure	Complete loss of serviceability

5. REVIEW OF INTERNATIONAL PRACTICE AND RECENT DEVELOPMENTS

In order to build on the limited guidance presented in codes, a review of recent published material has been carried out in order to understand best practice in terms of methods of analysis. Caltrans (2011) provides up-to-date and comprehensive guidance on analysis methods. Although the Caltrans document is written for piles in laterally-spreading ground, much of the guidance is applicable for piles in level ground as well. Dash *et al* (2008) provide a useful summary of available methodologies for modelling the load-deflection behaviour of piles in liquefied ground, highlighting inconsistencies between a number of commonly used methods.

Use of simple elastic-plastic models for the behaviour of laterally loaded piles represents ‘state-of-the-practice’. These models can be modified to allow for layers of liquefied soil by using reduced strength and stiffness values in these layers. Pile analysis software such as LPILE or Oasys ALP allows both inertial and kinematic effects to be modelled. In addition piles act as laterally unsupported slender columns in the liquefied zone and are therefore prone to buckling instability (Bhattacharya et al. 2004).

More advanced, but still simplified analyses use non-linear p-y (load deflection) curves to model the behaviour of laterally loaded piles as shown in Figure 3. Pile foundations subjected to lateral loading may also be modelled using the Beam-on-Nonlinear-Winkler-Foundation (BNWF) (e.g. Boulanger et al. 2007, Caltrans, 2011). The BNWF (Figure 3) model is extensively used in practice due to its simplicity, mathematical convenience and ability to incorporate non-linearity of the system (Dash et al., 2008). The beam represents the pile, and the non-linearity of the ‘foundation’ is represented by a set of horizontal springs modelled using non-linear p-y curves. Various published methodologies for p-y curves that represent liquefied soil are reviewed by Dash *et al*., (2008). The simplest and most widely used approach uses a p-reduction factor to modify the p-y curves for non-liquefied soil.

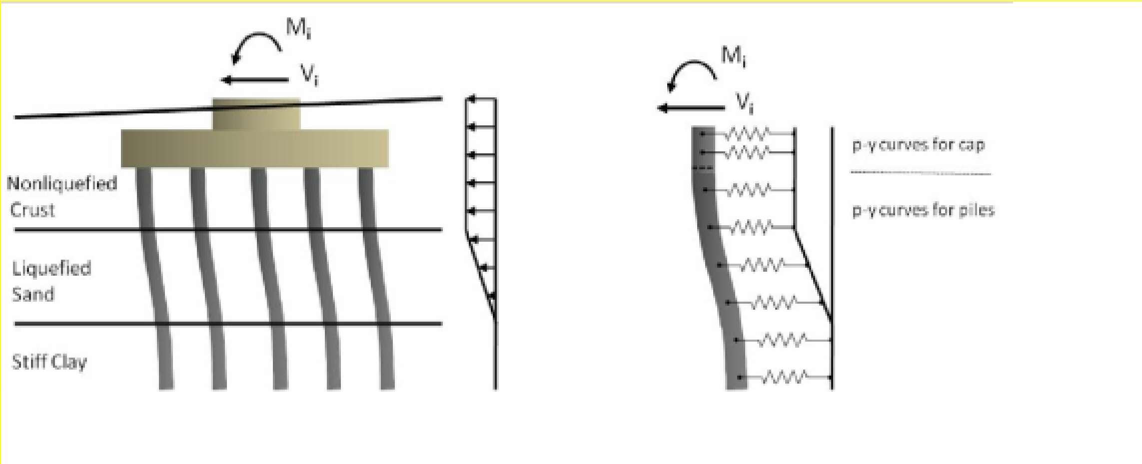


Figure 3: From Caltrans (2011). The displacement based analysis method applies a pseudo-static soil displacement to length of the pile in combination with an inertial load from the superstructure.

In addition to this the analysis can become very complicated by using the advanced models proposed by several researchers (Prevost (1985); (1989); Iai (1991); Arulanandan and Scott (1993); (1994); Muraleetharan et al. (1994); Manzari and Dafalias (1997); and Li and Dafalias (2000) to cite a few). Although different models have been proposed to capture the dynamic behaviour, there is not yet a firm agreement among researchers about the most suitable soil model.

6. CASE STUDY

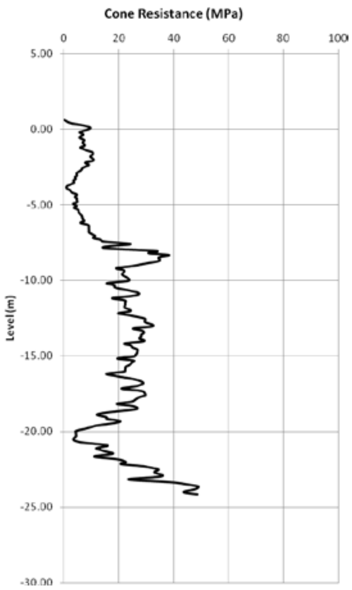
A LNG tank will be constructed at a site very close to the coast in a region of moderate seismicity (peak ground acceleration of 0.17g). The designer is considering the use of pile foundations. The soil properties at the site are given in Table 2 below. The site is underlain by sandy soil interfaced with clay infilled channels. Site investigation consisted of Cone Penetration Testing (CPT) measurements and Standard Penetration Testing (SPT), pocket penetrometer and laboratory testing, including undrained triaxial testing data. The strength properties are derived based on these data. A shear wave velocity profile was developed based on CPT data, using standard conversion correlations by Mayne and Rix (1993) for the cohesive layers and Rix and Stokoe (1991) for the cohesionless layers as shown in Table 2.

Table 2: Soil Properties at the site

Stratum	Thickness of Stratum (m)	Unit Weight, γ (kN/m3)	Internal angle of friction, φ' (°)	Undrained shear strength, c_u (kPa)	Average Vs (m/s) (for the layer)
Sand	20	17	30	-	180
Clay	2	19	-	100	265
Sand	12	19	35		289
Clay	3	19		200	314
Sand	13	19	35		337

6.1 Liquefaction Assessment

In this study, the NCEER methodology (Youd et al., 2001), and the methodology of Moss et al. (2006) was used for liquefaction assessment for the CPT data. The liquefaction assessment was performed for a characteristics earthquake of $6.75M_w$. The CSR (Cyclic Stress Ratio) was determined from 1D site response analysis. Spectrally matched time histories for 2475 year return period were used in the analysis. It can be seen that liquefaction is possible for a layer about 3.5 thick, from approximately 3m below the ground surface as seen in Figure 4.



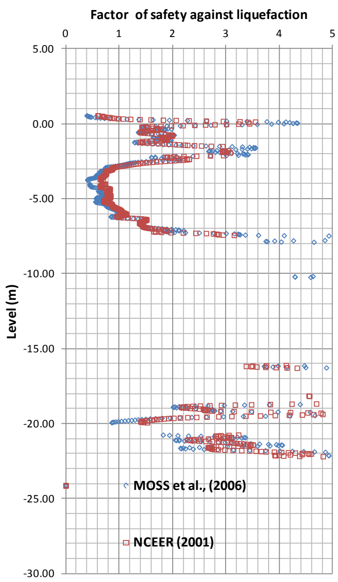


Figure 4: Liquefaction Potential Assessment for one of the CPT traces for Mw =6.75

6.2 Site Response Analysis for Liquefied Soil Profile

Residual soil strengths were determined using the methodology of Idriss and Boulanger (2008), allowing for the presence of lower permeability soils overlying the liquefied material, which would impede the post-earthquake dissipation of earthquake induced excess pore-water pressures.

6.3 Pile Behaviour Under Lateral Load

The following earthquake-induced loads on the piles were considered:

- Inertial lateral forces from the structure
- Overturning moments, which give rise to push-pull effects
- Kinematic interaction from the soil

The kinematic loading from the soil is estimated from the 1D site response analysis results, performed using *Oasys* SIREN (Pappin *et al*., 1991). Pile capacities were estimated based on the geotechnical properties reported in Table 2.

6.3.1 Load Combinations

For the liquefied condition Boulanger (2003) notes that the appropriate kinematic and inertial load combinations are a topic of ongoing research. The most recent guidance on this issue (Caltrans, 2011), which is based on back analysis of centrifuge experiments and numerical simulations, suggest that peak demands can be estimated reasonably well using the following combinations:

- 50% kinematic + 100% inertial = peak pile cap displacement
- 100% kinematic ± 50% inertial = peak bending moment and shear force

Note that in some instances peak pile demands occur when the direction of the inertial loading is opposite to the kinematic loading.

6.3.2 Results

The behaviour of piles under lateral loads was analysed using *Oasys* ALP. This programme predicts the pressures, horizontal movements, shear forces and bending moments induced in a pile when subjected to lateral loads, bending moments and imposed soil displacements. The pile is modelled as a series of elastic beam elements. The soil is modelled as a series of non interactive, non-linear "Winkler type" springs. The soil load-deflection behaviour can be modelled either assuming Elastic-Plastic behaviour, or by specifying or generating load-deflection (i.e. p-y) springs. In this case, the soil was modelled assuming Elastic-Plastic behaviour.

Pile deflections are shown in Figure 5, for piles subjected to both inertial and kinematic loads. Maximum pile head deflections of approximately 40mm are predicted for an 800mm diameter pile. The pile displacement can be seen to occur mainly within the liquefied soil layer. The maximum displacements at the pile head are governed by load-combinations where inertial loading is dominant. The bending moments along the pile profile are also shown in Figure 5. The largest bending moment, approximately 1100kNm, is due to inertial loading at the pile head where a fixed-head condition is assumed at the pile cap. Secondary bending moments and shear forces below the top of the pile head are greater and occur over greater depths due to the presence of liquefied ground.

7. SUMMARY & CONCLUSIONS

Based on previous project experience (e.g Ghosh et al., 2009), detailed performance objectives should be developed on a project-specific basis, with careful consideration of the client's requirements. These may be defined in terms of allowable deformation, either of the piles or of the overall soil-pile-structure system. From experience, performance under the 'no collapse' (i.e. Safe Shutdown) criteria is typically demonstrated without incorporating safety factors.

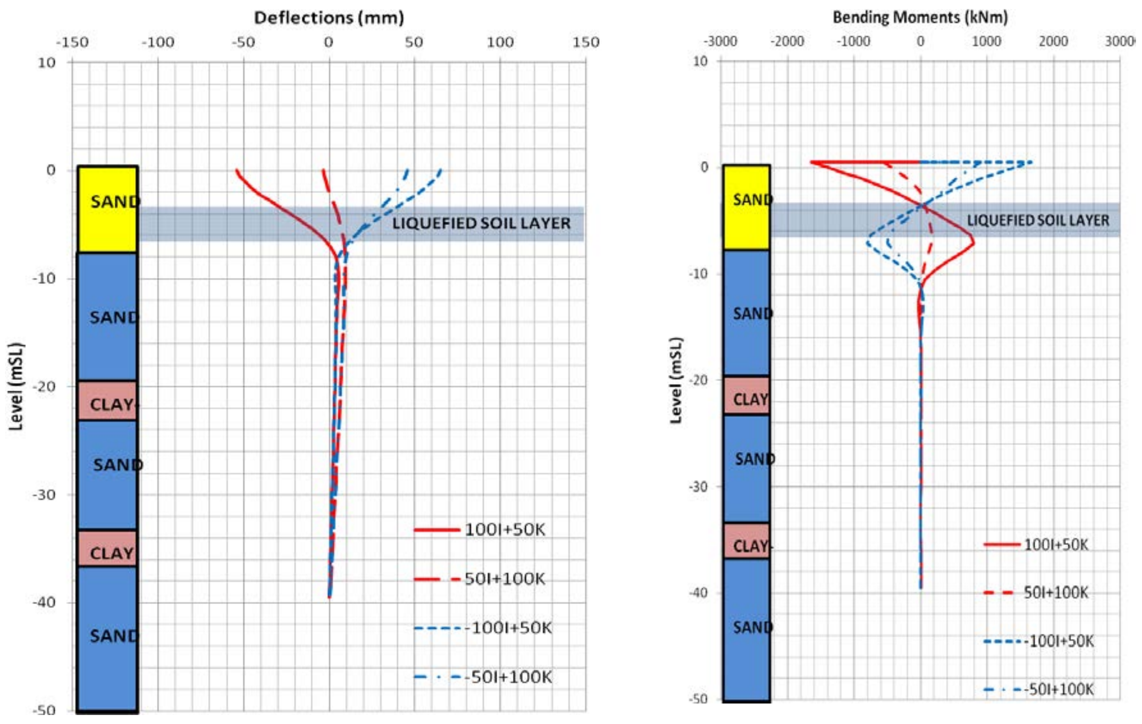


Figure 5: Single Pile displacement and bending moments for various load conditions for pile diameter 800mm

Allowing a plastic hinge to form in a pile means that the performance achieved may not fall into the category of 'repairable', even where it may be possible through advanced analysis to demonstrate 'no collapse'. Thus the detailed performance objectives need careful consideration. Provision of sufficient ductility to ensure that plastic hinges are more likely to form in the superstructure may represent a more appropriate long term solution.

Based on the review of codes and published literature, the following recommendations are made for designing piles in liquefiable soil:

- A collapse mechanism should not form in the piles under the combined action of lateral loads imposed upon by the earthquake (soil and structure) and axial load under the serviceability level earthquake (often termed the Operating Basis Earthquake, or OBE, for LNG facilities).

- At any section of the pile, the bending moment should not exceed allowable elastic moment capacity of the pile section under OBE loads (including appropriate factors). The shear stress load at any section of the pile should not exceed the allowable shear capacity.
- Under a less frequent level of earthquake loading, typically termed the Safe Shutdown Event (SSE), where the performance criteria would allow a level of damage to occur at a facility provided that the facility can be safely shutdown, thus providing life safety, a limited amount of inelastic deformation may be acceptable for piles. Piles with plastic deformations can be assumed to be 'unrepairable'.
- Piles should have sufficient embedment in the non-liquefiable layer below the liquefiable layer, to achieve fixity in order to carry moments induced by the lateral loads.
- Piles should have sufficient capacity to carry the axial load acting on it during the OBE and SSE earthquake without buckling. Lateral loading due to ground movement, inertia, or out-of-straightness, will increase lateral deflections which in turn can cause plastic hinges to form, reducing the buckling load, and promoting more rapid collapse.
- The settlement in the foundation due to the loss of soil support should be within the acceptable tolerances. The settlement should not induce end-bearing failure in the pile.

This paper presents a brief summary of pile design considerations for piles extending through potentially liquefiable soil. Key factors for consideration include the selection of pile performance criteria under extreme loading, and appropriate combination of kinematic and inertial loading. The client should be made aware of the consequences of choosing a particular performance requirements and the performance matrix should be developed in consultation with the designers and the client. This is in the spirit of performance based design.

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