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Seismic risk management of piles in liquefiable soils stabilized with cementation or lattice structures

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Abstract: Liquefaction is an important seismic hazard that can cause extensive damage and high economic impact during earthquakes. Despite the extensive research, methodologies, and approaches for managing liquefaction for pile supported structures, failures of structures due to liquefaction have continued to occur to this day. The main aim of this paper is to develop a simplified methodology to reduce potential structural damage of structures founded in soils susceptible to liquefaction. In order to implement a successful remediation technique, the current methods for pile failure in liquefiable soils and remediation schemes of earthquake-induced liquefaction are critically reviewed and discussed. The cementation and lattice structure techniques to reduce the liquefaction hazard are proposed, while numerical analysis for unimproved and stabilised soil profiles using Finite Element Method (FEM) is carried out to simulate the analysis of both stabilisation techniques. The results showed that the both techniques are effective and economically viable for reduction or avoidance of potential structural damage caused by liquefied soil and can be used in isolation or in combination, depending on the ground profile and pile type.

1 INTRODUCTION

Damaging effects in pile supported structures due to liquefiable soils were extensively observed during and after earthquakes in the past (Tokimatsu et al., 1998, Bhattacharya, 2006, Bhattacharya, et al., 2011, Lombardi and Bhattacharya, 2012), which put the remediation of earthquake-induced liquefaction in the focus of geotechnical earthquake engineering practice. Liquefaction has been shown to occur when, during seismic vibration, the pore water pressure in the usually loosely deposited sandy soil layers increases rapidly and sufficiently which may lead to a decrease in the effective stress in the soil to zero (Booth, 1994). Although through evaluation of the seismic risk and subsequent management the existing piled foundations usually achieve the desired level of safety, failures of structures due to liquefaction still occur. Therefore, there is an urgent need to better understand and clarify this complex phenomenon, as well as to identify how liquefaction affects piles.

During earthquakes, the response of pile-supported structures to liquefiable soils depends on the stiffness of the pile foundation, response of the soil surrounding the pile, and the soil-pile interaction effects (NEHRP, 2012). The interaction effects include the inertial loading exerted by the superstructure and the kinematic loading induced by the soil surrounding the pile (Fig. 1).

Before the earthquake, the axial load on the piles can be estimated based on static equilibrium. Upon commencement of the seismic vibration, and before the excess pore water pressure build-up, this axial compressive load may increase/decrease further due to the inertial effect of the superstructure (due to oscillation of superstructure) and the kinematic effects of the soil flow past the foundation (due to ground movement). This change in loading can be transient (during the vibration, due to the dynamic effects of the soil mass) and residual (after the vibration, due to soil flow, often known as “lateral spreading” (Bhattacharya and Madabhushi, 2008)).

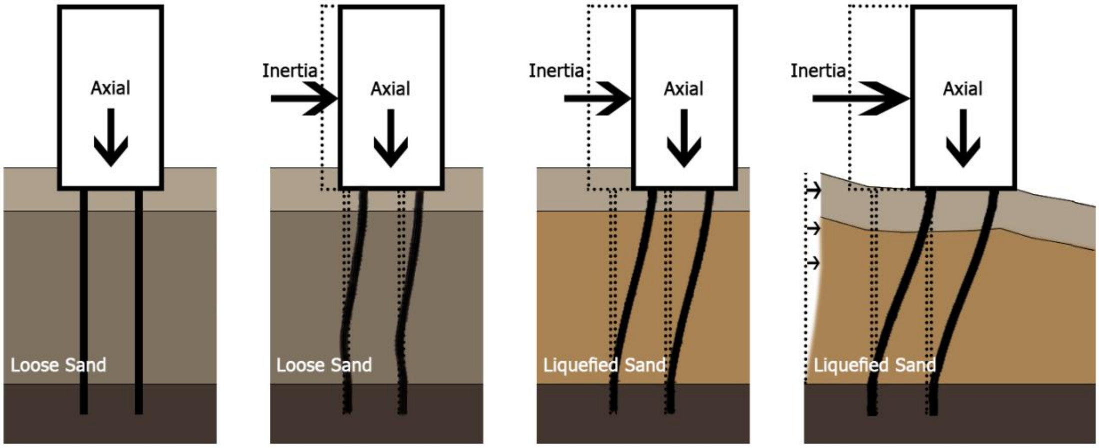


Figure 1 Different stages of loading and failure mechanism of pile during earthquake (adapted from Bhattacharya, 2014)

However, at this stage, with pore water pressure built up (at full liquefaction, the excess pore water pressures reach the overburden vertical effective stress), the soil loses its strength and stiffness, and the pile acts as an unsupported column over the liquefied depth (Lombardi and Bhattacharya, 2014). Most of the efforts have been made to greatly improve understanding of pile failure mechanism due to liquefaction; however, further research is required to develop insight into the effects of liquefaction triggering on seismic response of structures and soil stiffness.

It is widely accepted that the impact of geotechnical hazards is the main contributor in the damage to structures during earthquakes (e.g. Kramer et al. 2014). The assessment of geotechnical hazards is, therefore, essential for quantification of the seismic safety and liquefaction mitigation of these structures. Various ground improvement techniques are used for remediation of piled foundations in liquefiable soils including densification, preferential drainage path provision, soil reinforcement, removal and replacement of the liquefiable soils with competent soils, etc. (Mitchell 2008; Rayamajhi, et al. 2015). However, the behaviour of piled foundations stabilised with these techniques has rarely been modelled or quantified in the past which has affected the acceptance of these techniques in the geotechnical engineering practice and the overall seismic risk management approach to piles in liquefiable soils.

The main aim of this study is to develop a novel approach for seismic risk management by providing a methodology to reduce potential structural damage of pile-supported structures founded in soils susceptible to liquefaction. In order to investigate the feasibility of a successful remediation technique, the current methods for pile failure in liquefiable soils and remediation schemes of earthquake-induced liquefaction will be critically reviewed and discussed. Two viable methods to reduce the liquefaction hazard (cementation and lattice structure techniques) will be proposed, and numerically simulated using Finite Element Method (FEM) in order to establish areas for application of the proposed techniques and methodology.

2 METHODOLOGY

In this study we propose a methodology where the seismic risk management (SRM) for mitigating liquefaction is evaluated by comparing consistent measures of seismic loading that have caused pile failure and liquefaction resistance (Kramer, 2008). Therefore, both the current understanding of pile failure in liquefiable soils and the remediation schemes will have to be investigated and understood (Fig. 2). Once these are critically reviewed, the SRM for mitigating the risks on pile-supported structures in liquefiable soils by using cementation and lattice structure improvement techniques will be proposed and demonstrated through numerical simulation. The numerical modelling using FEM Abaqus will be carried out to analyse both unimproved and stabilised soil profiles. The results of the analysis and simulation will be then used to focus on the behaviour of the improvement (stabilisation) techniques during earthquake as well as on their effects on the soil and structures. Additionally, our proposed methodology will examine and determine the ability and mitigation potential of the proposed techniques in the light of ground deformations for piles. Finally, the findings of the simulations and analyses will be used to perform a seismic risk management by developing a liquefaction remediation strategy.

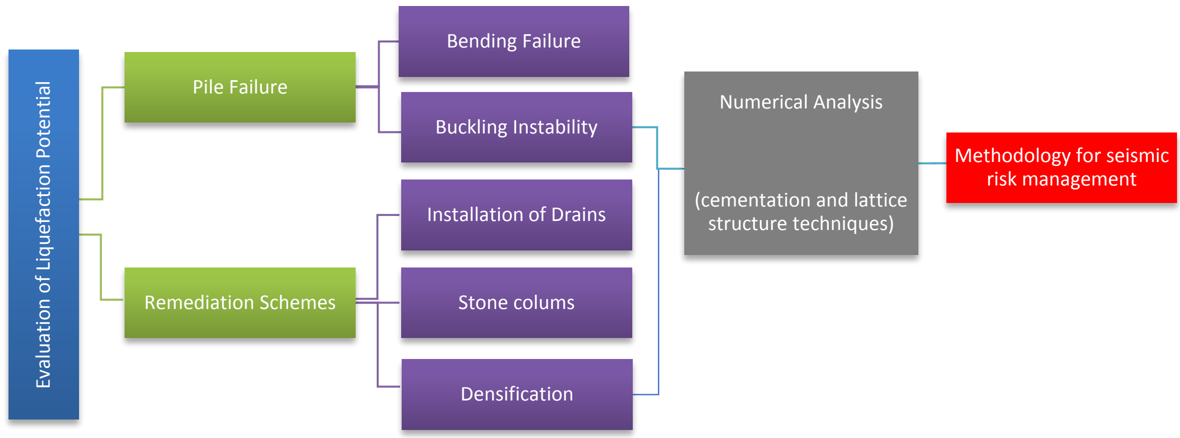


Figure 2 Schematic illustration of the methodology

2.1 Current understanding of pile failure due to seismic liquefaction

A number of research studies have been carried out in the past to predict the response of soil-foundation-structure systems in order to avoid collapse and decrease the damage levels (e.g. Bhattacharya and Goda, 2013; Krishna, et al., 2014; Bhattacharya, et al., 2014; Dammala, et al., 2017). Liquefaction hazard evaluation is generally concerned with two different mechanisms of pile failure: failures due to bending (flexural) or buckling of the pile (Bhattacharya et al. 2004; Dash et al. 2010; Lombardi and Bhattacharya, 2014 and 2016; Rostami et al. 2017). Bending failure (flexural failure) occurs when the soil surrounding the piles liquefies and loses much of its stiffness, causing the piles to act as unsupported slender columns, while buckling failure occurs when piles act as beam-columns under both axial and lateral loading. Evaluating the potential for initiation of liquefaction (i.e. liquefaction potential), involves comparing the anticipated level of loading applied to the structure as a result of an seismic vibration at a particular site with the liquefaction resistance of the soil at the same site.

In practice, different design procedures have been used for the seismic design of pile-supported structures. The Japanese Highway Code of Practice (JRA) (2002), for example, advises the practicing engineers to consider both of the loading conditions mentioned above. However, it suggests a separate bending failure (flexural failure) check for the effects of kinematic and inertial forces. Similarly, BS EN ISO 2008 (Eurocode 8; 2004) advises pile design against bending due to inertial and kinematic forces arising from the deformation of the surrounding soil. In the event of liquefaction, Eurocode 8 also suggests that “the side resistance of soil layers that are susceptible to liquefaction or to substantial strength degradation shall be ignored”. The NEHRP (2000), on the other hand, focuses on the bending strength of the piles by treating them as laterally loaded beams and assuming that the lateral load due to inertia and soil movement causes bending failure. Based on these guidelines, for this study, the pile is modelled as a beam-column element carrying both axial and seismic loads.

2.2 Current Remediation Schemes

Piled foundations of existing buildings are often difficult to access for retrofitting and, in addition, any procedure must ensure that the superstructure is not damaged during remediation (Mitrani and Madabhushi, 2011). Remediation of existing structures founded in liquefiable soils is usually carried out using methods such as installation of drains (Brennan and Madabhushi, 2002), stone columns (Gniel and Bouazza, 2009; Lo et al., 2010; Asgari et al. 2013; Tang et al., 2015) and densification (e.g., using deep dynamic compaction, vibro-compaction, compaction piles) (Baez, 1995; Adalier and Elgamal 2003; Coelho et al., 2007; Mitchell 2008). The densification methods have been widely studied because these techniques are relatively simple and practical, and the resulting remediation success can be easily verified by using in-situ penetration techniques (Mitchell and Solymar, 1984; Charlie et al, 1992; Elias et al., 2006). For example, the effects of sand layers of varying density, thickness and extent on the behaviour of a bridge abutment have been investigated by Balakrishnan and Kutter, (1999) and Kutter et al., (2004). However, Rayamajhi et al. (2014, 2015) reported that the densification and drainage techniques of improvement are often ineffective while the soil-cement columns were relatively ineffective in reducing the potential for liquefaction triggering in saturated silty soils.

The cementation and lattice structure techniques (e.g. grouting injection, deep soil mixing) for soil improvement structures have been studied in the past (e.g. Suzuki et al., 1991; Tokimatsu et al., 1996; Namikawa et al. 2007; Kitazume and Takahashi 2010; Funahara et al., 2012; Nguyen et al., 2012 and 2013; Yamauchi, et al., 2017) and were shown to effectively stabilise liquefiable soils at reduced installation costs.

3 NUMERICAL MODELLING

In the present study, a numerical method was used to investigate the stabilising mechanisms of cementation and lattice structure techniques in liquefiable soils as an extension of the previous research conducted by authors (Rostami et al. 2017) and discussion on the verification of numerical modelling procedures was well explained in previous paper. Three-dimensional (3D) nonlinear dynamic analyses were performed for a piled foundation on a liquefiable soil layer in original (unimproved) and stabilised (cementation and lattice structure techniques) soil profiles. These analyses were carried out in Abaqus and included modelling of a single pile as a beam-column element carrying both axial and seismic loading, within a liquefiable soil which is stabilised using the two chosen techniques. The observed deformation of the pile affected by soil liquefaction was used to demonstrate the pile capacity and predict the thickness of the stabilised soil layer that would be affected in the seismic event. The results of these analyses provide the required thickness and the properties of the zone of liquefiable soils requiring treatment.

3.1 Overview of Models

Figure 3a shows the extent of 3D ground model comprising three soil layers. The liquefiable soil was modelled in between two layers of non-liquefiable soil (Fig. 3b) and a reinforced concrete pile with fixed-head was modelled to span the three soil layers with varying properties (thickness, type, articulation). Due to axial symmetry, only half of the pile and surrounding soil were modelled for the original and stabilised (a cement injected layer in lieu of the liquefiable layer soil stratum) soil profile. Additionally, cases of pile without and with cement injected layer were modelled (Fig. 4).

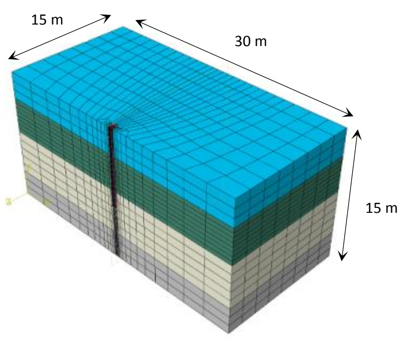


Figure 3(a) The 3D numerical model

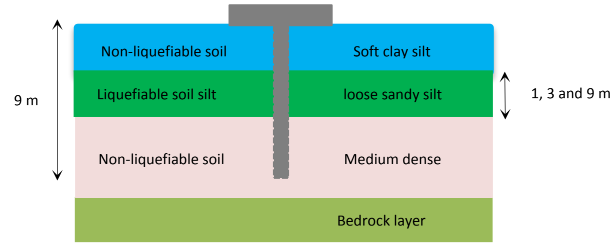


Figure 3(b) Details of the pile and model

The different thickness of liquefiable soil profiles (1, 3 and 9 m) surrounding the pile were considered to be wide enough to identify the effectiveness of the free-field kinematic demand imposed on the soil system. The full model is shown in Figure 5, which was used for lattice structure technique evaluation.

3.2 Modelling the soil-pile system

For the FE model to effectively simulate the pile-soil interaction, it was important to appropriately define the interaction between the pile and the soil near the solid-to-liquefied layer interface. To model the interaction between the soils and pile the “surface-to-surface” contact method (a.k.a. “master-slave” surface) was used, where the more deformable and more rigid surfaces are defined as the “slave” and “master” surfaces, respectively (Abaqus, 2012).

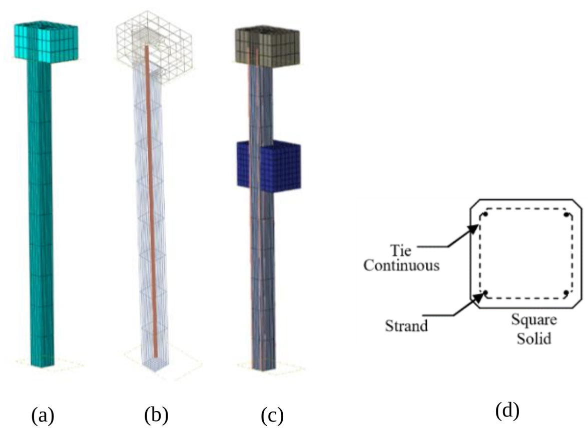


Figure 4 (a) Details of the pile, (b) the flexible beam element along the pile and (c) pile with cement injected (stabilized) layer (d) Cross sections of the piles

The non-linear p–y curves of the liquefied soil used in the modelling of soil–pile–structure interaction were based on the beam on elastic foundation approach (Hetényi, 1946). The p–y curves have been used to model the reaction of the foundation with consideration of inertial effects and seismic soil–pile interaction. In this study, the non-linear spring stiffness (p–y curves) of the liquefied soil is used to evaluate soil–pile interaction analysis performed pile bending moments.

To evaluate the soil–pile interaction of the liquefied soil, analysis is normally performed in terms of shear forces and pile bending moments (McGann, et al., 2012). However, the pile bending moments could not be directly obtained from the Abaqus output as the pile was modelled as a solid element. This restriction was overcome by adding a very flexible beam element along the pile (Banerjee and Shirole, 2014).

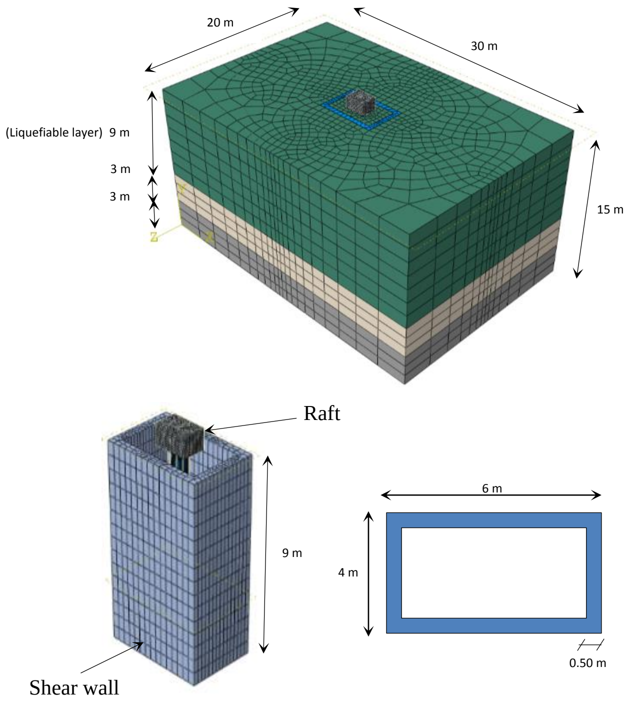


Figure 5 Details of the lattice structure and model

The dynamic load model requires boundary conditions that offer support to the elements whilst restricting unnecessary motions (Abaqus, 2012). For dynamic cases, the ability of the infinite elements to transmit energy out of the FE mesh, without trapping or reflecting it, is optimized by making the boundary (same material for each layer without damping) between meshes as close as possible to orthogonal in the direction from which the waves will impinge on the boundary (i.e. close to a free surface, where Rayleigh or Love waves may be significant; Figure 6) (Abaqus, 2012).

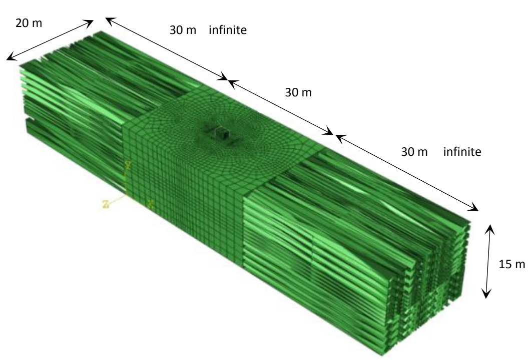


Figure 6 The infinite elements to transmit energy out of the finite element mesh

During earthquakes, the excess pore water pressure in loose, saturated soils increases, thus reducing the effective stress in the layer and, subsequently, significantly decreasing the shear strength. As a result of the pore water pressure build up, the compressibility of the layer cannot change drastically (McGann, et al., 2012) so the soil bulk modulus, K, is assumed to remain constant throughout the soil mass, and the Poisson's ratio of liquefiable soils is assumed as $\upsilon = 0.485$ (McGann, et al., 2012). Additionally, the Mohr–Coulomb failure criterion is used to simulate the soils behaviour (Helwany, 2007), while the hypoelastic model in Abaqus was used to simulate nonlinearity below the yield envelope (Banerjee and Shirole, 2014).

The seismic loading was applied at bedrock level (assumed below the three soil layers) in the horizontal direction in form of an acceleration time history. The input motion of harmonic excitation consisted of waves of unit amplitude and different frequencies for the first 8 seconds of the El-Centro earthquake record scaled to 0.30 g and used as the base input acceleration (Fig. 7a). However, the input motion was applied at 0.15 g due to the larger values of initial effective stress at the lower layers (Rahmani and Pak, 2012). First, the pile is subjected to the axial load of 1100 KN (Fig. 7b) and increasing due to equilibrium is satisfied within the soil layers. Then, the dynamic load applied by time history.

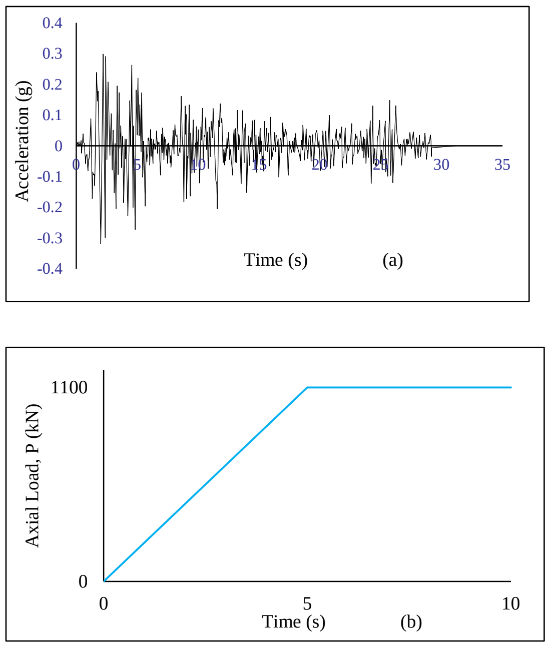


Figure 7 Seismic loading for this study. (a)Acceleration record of El-Centro (1940) earthquake (b) Increase of axial load

3.3 Modelling the pile and lattice structure

The piles in this study include one deep foundation reinforced concrete pile (Fig. 4) modelled using beam-column elements as elastic materials, reflecting a typical precast pile used in construction (0.16 m^2 section, length of 9 m and 12m).

In this study, 3D model of a lattice structure surrounding the pile (Fig.5) is used as a representative of lattice structure used to remediate against the potential effects of earthquake-induced liquefaction phenomenon (Nguyen et al., 2013). The lattice structure walls were modelled as a shear box, which can provide additional shear stiffness and strength for sites to withstand liquefaction (Nguyen et al., 2013).

The properties of piles, raft, cement injection and lattice structure are given in Table 1.

3.4 Modelling the Soil

Three typical soils were modelled in 3D, surrounding the pile, varying the thicknesses of liquefiable layer between the two non-liquefied layers and material properties to explore the effects of liquefaction on the pile. Appropriate values for the soil parameters were chosen from previous case histories (Sarkar, et al., 2014) to ensure valid result. The soil parameters selected for the FE model are summarized in Table 2.

Table 1

Properties of piles, raft, cement injection and lattice structure models

Item Description	Poisson's ratio	Modulus of elasticity (kN/m^2)	Unit weight (kN/m3)	σ_y (MPa)	f'_c(KPa)	ε_c
9m Pile	0.15	30×10^6	24	1860	44816	0.03
12 Pile	0.15	30×10^6	24	1860	44816	0.03
Steel material	0.3	200×10^6	78.5			
Cement injection layer	0.2	25×10^6	23.5			
lattice structure	0.2	25×10^6	23.5			

yield strenght σ_y (MPa) compressive strength f'_c(KPa) concrete strain ε_c

Table 2

Soil parameters

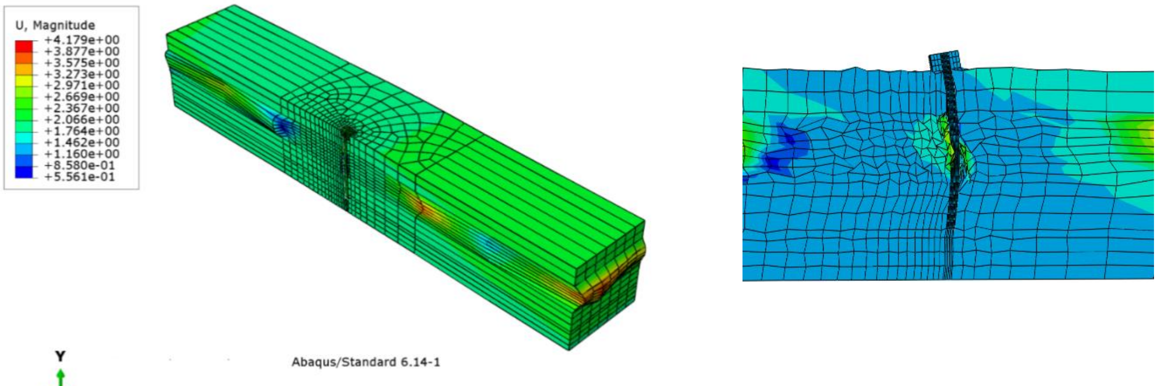
	Layer no.	Basic description	γ (kN/m3)	Cohesion, cu (kPa)	Friction angle, Φ (°)	υ	shear moduli G (KPa)	K (KPa)
	I	Soft silty clay	19.1	40.0	—	0.35	9260	27777.8
	II	Soft clayey silt	18.2	23.0	—	0.35	9260	27777.8
Liquefaible Zone	III	Loose sandy silt	18.0	—	28.0	0.485	824	27777.8
Liquefaible Zone	IV	Medium dense silty sand	19.0	—	30.0	0.485	824	27777.8
Liquefaible Zone	V	Stiff clayey silt	18.4	49.0	—	0.485	824	27777.8
	VI	Medium dense silty sand	19.0	—	32.0	0.35	9260	27777.8

4 ANALYSIS AND RESULTS

In order to implement a successful remediation technique for the seismic risk management of pile-supported structures in liquefiable soils, a parametric study has been carried out on three different soil profiles, varying the thickness of liquefiable soil. To obtain results 12 soil profiles for each of three different thickness of liquefiable soil profiles (1, 3 and 9 m) and the unimproved and stabilised soil for both cementation and lattice structure techniques were modelled.

4.1 Analysis of 3D FEM

As expected, the effect of the remediation technique was dependent on the respective material properties, thickness of cement layer, input wave and the surrounding soil. The behaviour at each incremental point along the pile length was calculated and plotted. An example of deformed shape of the systems and the interaction between the soil and the pile are shown in Figure 8.



(a) (b)

Figure 8 (a) Deformed shape of model of unimproved soil with 3 m thickness of liquefiable soil

(b) Pile deformation

From the deformed shape of the system, it can be observed that the imposed displacement profile triggers bending in the pile. It also shows that the non-liquefiable layers of soil begin to displace laterally with respect to the liquefiable layer. However, the pile provides resistance to this motion as the upper portion is pushed along with the flow of soil. This behaviour is illustrated in the lateral stress distribution curve (Fig. 9) which is shown alongside the maximum bending moment.

4.2 Cement injection improvement

Figure 9 illustrates the maximum bending moment developing along the length of piles embedded in soil layers without and with cement injection layer. It can be seen that the imposed displacement induces bending in the pile. It can also be observed that that the volume of soil improvement could be reduced 90% for 1.0 m of liquefiable layer thickness and 70 % for 3 m thickness of liquefiable soil. However, the 9 m thick liquefiable soil layer can provide 30% resistance to liquefaction and this stability is not satisfied. It can be explained by a number of factors, that decreasing density and stability. The large thickness of liquefiable soil in touch with pile and the lateral stress distribution of the nature of ground motions and containing pore pressure generation put the pile in maximum of bending and increasing shear stress. It is found that for the range of parameters used in this study, the bending moment reduction using cement injection across 1/3 of liquefiable soil thickness may be sufficient to prevent liquefaction (Fig. 9 b) and this solution could be considered for thin liquefiable layers with thickness of less than 1/3 of pile length.

Therefore, it would be prudent for this method to be used as secondary rather than primary mechanism for ground improvement in liquefiable soil with liquefiable layers with thickness of more than 1/3 of pile length, although cement injection may help to prevent liquefaction triggering in stabilised thin liquefiable soil.

4.3 Lattice improvement

Figure 10 shows the bending moment reduction achieved by using lattice structure. Based on the numerical analyses, a new simplified design method was proposed, which better quantifies the level of bending moment reduction in the improved soil. It can be seen that in the improved case, the bending moment is reduced due to dilation of the lattice structure, such that the decrease in lateral soil movement. The results shown on Figures (10a to 10c) that the lattice structure mechanism could be sufficient to prevent liquefaction triggering and ground improvement in liquefiable soil. As illustrated in Figure 10a, this could be improved by 90% for 3m. However, for thicker liquefiable soil layers, the lattice walls would tend to be more flexible and may offer improvements of as little as 0.50% (Fig. 10 b). In such conditions it may be better to consider lattice in conjunction with cement injection for ground improvement in liquefiable soil by 70% (Fig. 10 c). An example of the deformed shape of a lattice structure used for remediation of liquefiable soil is illustrated in figures 11a and 11b sequentially. The figure 11a shows that the dynamic amplitude leads to a change in effective stress of the soil and increasing shear stress with time. It can also be observed that shear wall can stabilise the effective stress path, and provide some additional stiffness of the soil under these conditions.

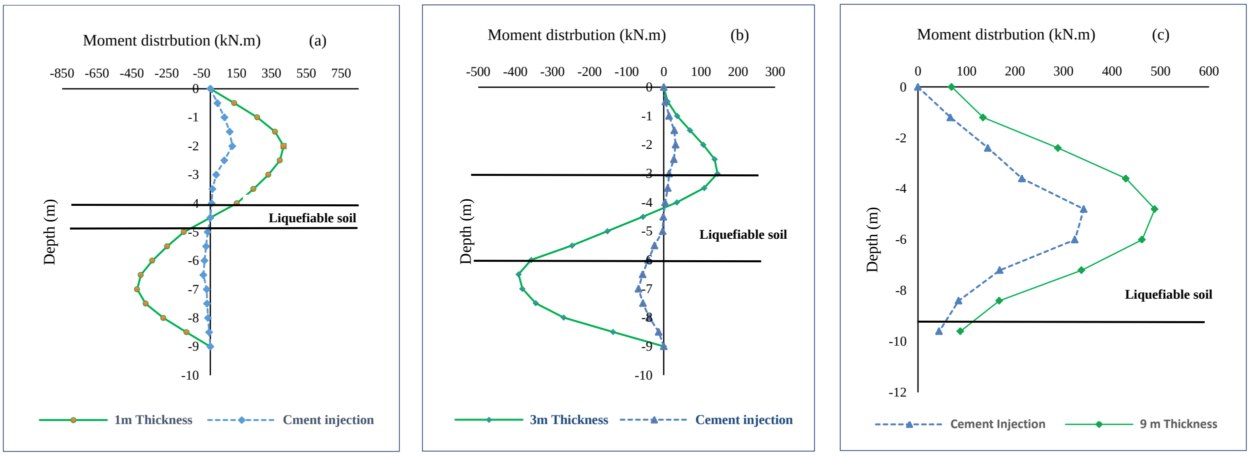


Figure 9 The bending moment without cement injection and with cement injection (a) 1m thickness (b) 3m thickness and (c) 9m thickness of liquefiable soil

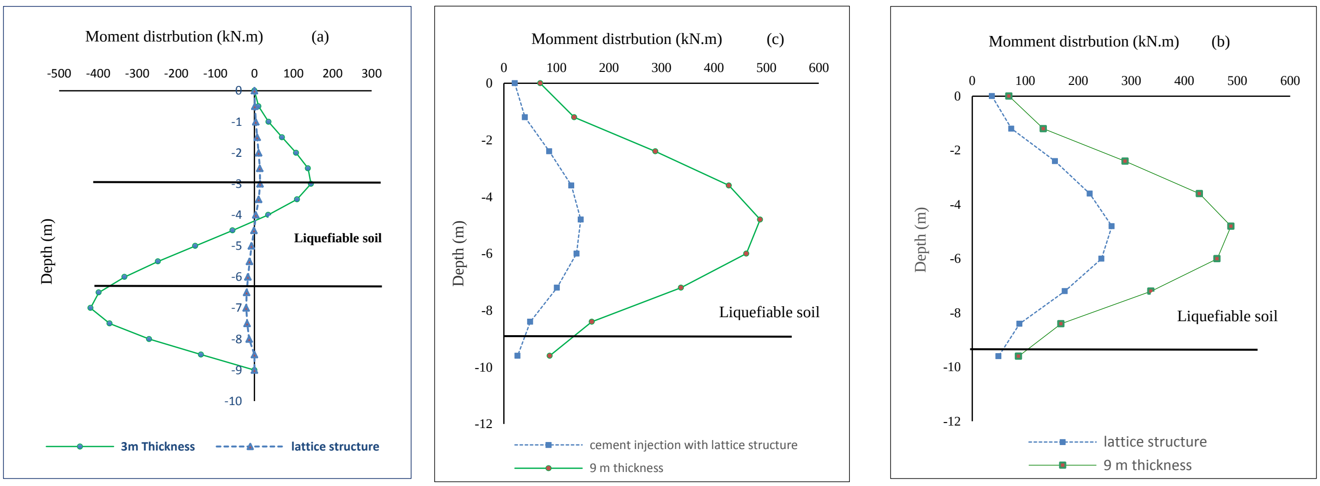


Figure 10 The bending moment without lattice structure and with lattice structure (a) 3m thickness (b) 9m thickness (c) 9m thickness with both cement injection and lattice structure

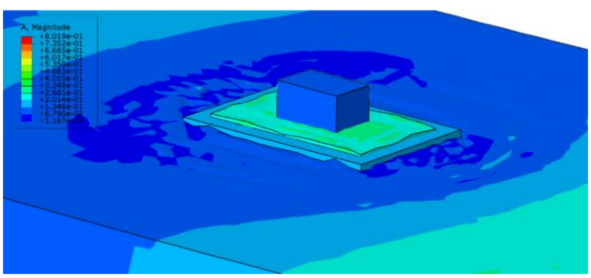


Figure 11a The deformed shape around the shear wall

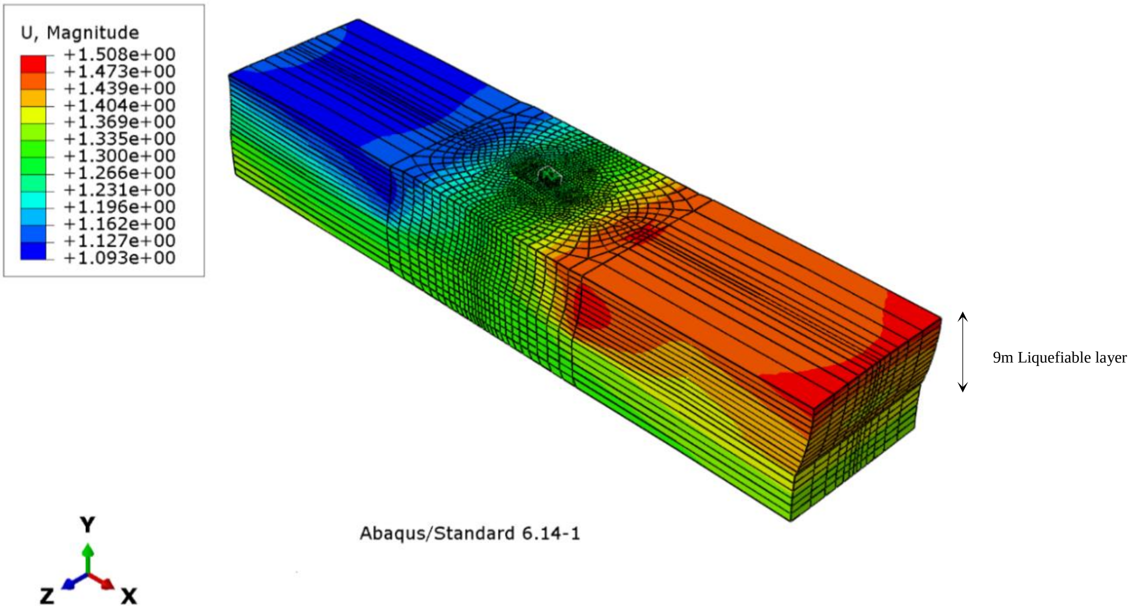
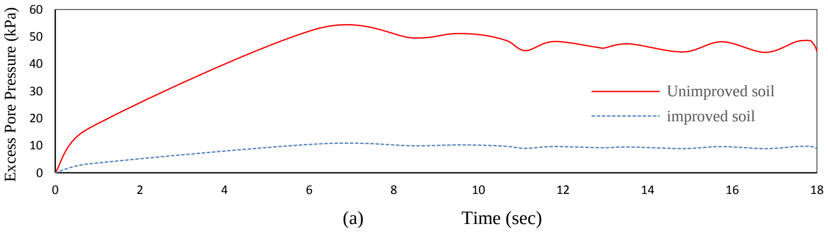
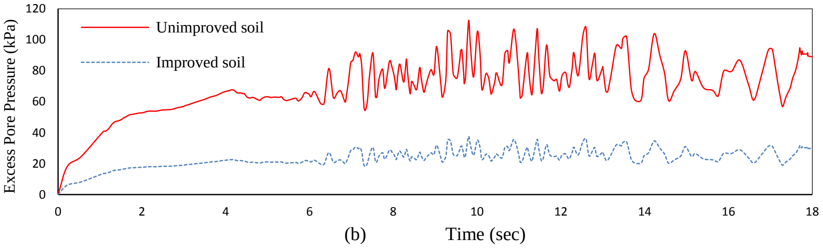


Figure 11b Deformed shape of model 9 m thickness of liquefiable layer with lattice structure





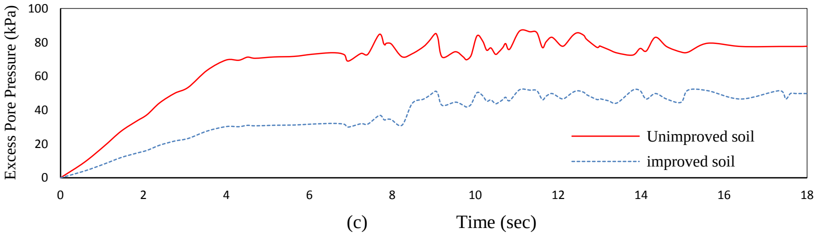


Figure 12 Excess pore water pressure generated near pile (a) for 1m cemented soil, (b) for 3m lattice structure and (c) for 9m lattice structure

Figure 12 shows the excess pore water pressure generated near a pile at 5 m below the soil surface for the case of 1m cement injection improvement and 3m, and 9 m of lattice structure model during and after earthquakes respectively. It can be seen that lower levels of the excess pore water pressure (blue colour) were generated in the stabilised soils. As illustrated (Fig. 12) limiting the excess pore pressure for all cases and the ground improvement can prevent and protect the pile against liquefying. However, the case of 9 m thick liquefiable soil shows that the excess pore pressure decrease slightly. This excess pore pressure behaviour can be understood by hydraulic gradients that drive pore water flow both during and after earthquake shaking (Kramer, 2008). In this case, the flow might migrate upward, even under the structure, thereby decreasing the density, and consequently improving the liquefiable soil layer by densification.

5 SEISMIC RISK MANAGEMENT

The FEM showed that the volume of soil activated during liquefaction dictates the deformations of the structure which, in turn can be controlled by the type and magnitude of stabilization measures. Based on this, we propose the following framework for characterization of seismic loading and resistance to liquefaction (Fig. 13).

Step 1: Identification of the liquefiable layer

The first step in a liquefaction assessment is to identify whether or not the soils are susceptible to liquefaction. The estimate of input ground motion at a site is a critical parameter in the characterization of earthquake loading in conventional liquefaction potential analyses and can be obtained using the regional ground motion prediction equation (GMPE) (Goda and Hong, 2008; Goda and Atkinson, 2009 and 2010). The liquefaction susceptibility can be preliminarily screened by using historical, geological, hydrological, and compositional criteria (e.g. Youd & Perkins, 1987, Seed et al., 2003, Kramer, 2008), and the liquefaction potential defined using established methods (e.g. Seed and Idriss, 1971, 1983, 1985; Idriss & Boulanger, 2008).

Step 2: Characterisation of soil material

The next step is to define local site conditions including stratification, engineering and material properties of different soil layers, possible groundwater conditions, thickness and location of liquefiable soil, and the length of pile in touch with the liquefied soil zone. In Situ Geotechnical Tests such as the Cone Penetration Test (CPT) and Standard Penetration Test (SPT) are the two empirical methods for evaluating liquefaction (Seed and Idriss, 1971, 1982, Seed et al., 1977, 1983, Seed, 1979, Stark and Olson, 1995, Cetin et al. 2002, 2004, Juang et al. 2005, Moss et al. 2006, Goda et al. 2011, Boulanger and Idriss, 2014). Laboratory testing of 'undisturbed samples', typically simple shear, triaxial or torsional cyclic tests, can be also used to derive the soil material properties (e.g. Seed et al., 2003; Boulanger and Idriss, 2005; Bray and Sancio, 2006). Some engineering properties in terms of seismic hazards can be derived from the National Annexes of the relevant Eurocodes. For example, Eurocode 8-Part 5 (2004b) shows two separate empirical approaches for clean sand and silty sand which show liquefaction potential.

Step 3: Site hazard quantification

After the soil materials have been identified and characterised, the site-specific ground response needs to be determined, the liquefaction hazard to be analysed, and the as built details of structure and the response of infrastructure modelled in order to obtain the seismic effects for a particular site and structures (EN 8, (2004); Ghosh and Bhattacharya, (2008), and Govindaraju and Bhattacharya, (2012)).

Step 4: Assessment of unsupported pile length

Next step is to estimate the laterally unsupported length of the pile D_L in the seismic event. This is based on the depth of liquefaction potential evaluation of a soil column and often can be obtained by using simplified stress-based methods (Seed and Idriss (1971), Kramer, (1996); Youd et al., (2001) and Idriss and Boulanger (2008); Khoshnevisan et al., 2015; Kramer and Greenfield, 2017). Indeed, DL can be determined by the thickness of liquefied soil layers plus some additional length necessary for fixity at the bottom of the liquefied soils (Bhattacharya and Goda, 2013). In this study, the criteria to determine of unsupported length (D_L) based on liquefied soil profile (base case is set to a limiting thickness of non-liquefied soil layers for lateral support of a pile) equal to 6.5D was considered.

Step 5: Assessment of maximum critical pile length

The critical pile length resisting buckling failure, Hc, is a function of pile characteristics and pile head loading (Bhattacharya and Goda, 2013) which a pile can sustain without collapse due to combined axial and lateral loading. The critical pile length depends on the type and dimension of superstructure (bridge or building), bending stiffness, axial load acting on the pile, dynamic characteristics of superstructure, and boundary conditions of the pile at the top and bottom of the liquefiable layer. Hc can be estimated using an established method (Bhattacharya and Goda, 2013):

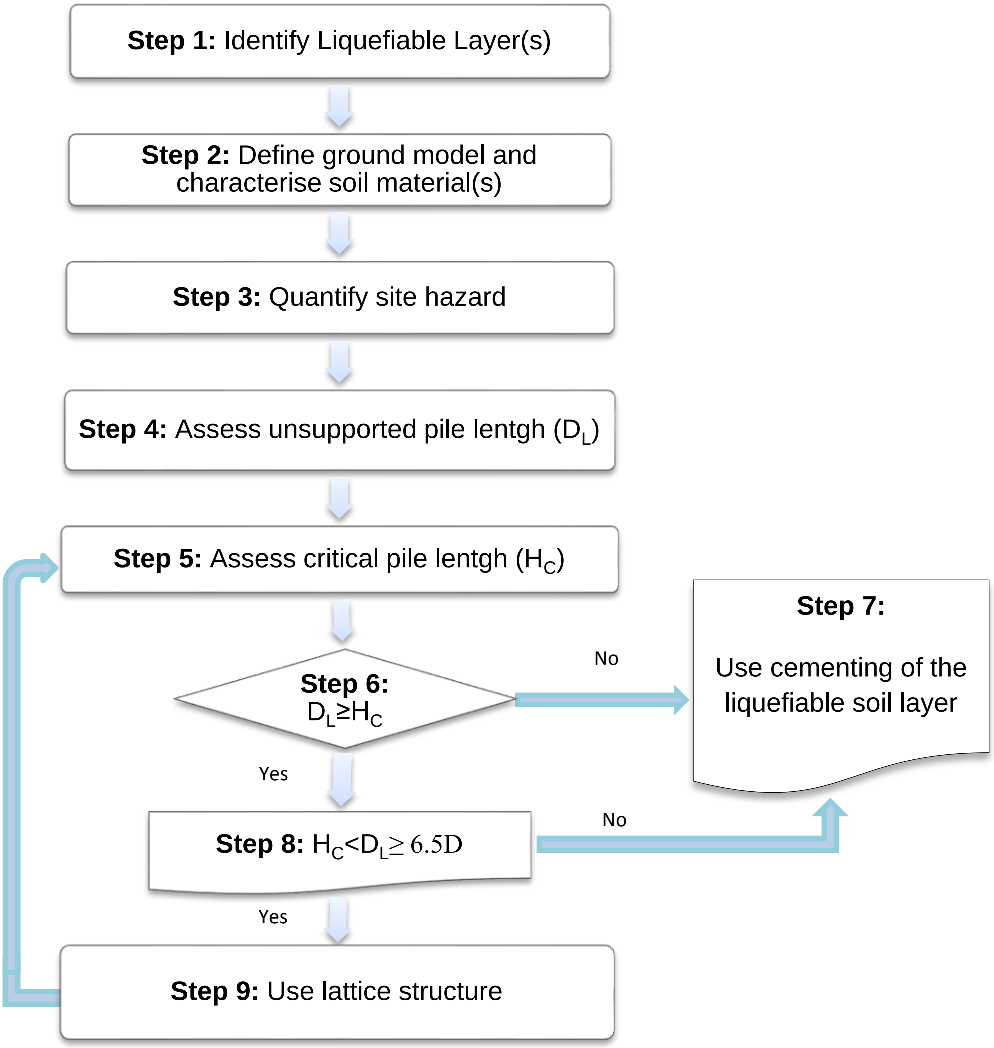
$$H_C = \sqrt{\frac{\emptyset \pi^2 EI}{K^2 P_{dynamic}}} \qquad (1)$$

Where EI is the bending stiffness of the pile, K is the column effective length factor.$\emptyset$<1, it is noted that in reality, this factor depends on the axial load, imperfection of piles, and residual stress in the pile due to driving. An estimate of the maximum axial compressive load acting on a pile can be given by

$$P_{dynamic} = (1+\propto)P_{Static} \qquad (2)$$

In this study, the values of input parameters set to 0.35, and 1.0 for Ø and K, respectively.

where α is termed as the dynamic axial load factor and is a function of type of superstructure, height of the centre of mass of the superstructure, and characteristics of the earthquake shaking (e.g., frequency content and amplitude).



(1) H_C: Critical pile length in touch with liquefiable soil,
D_L: unsupported pile length,
D: The diameter of pile

Figure 13 Seismic requalification methodology of a pile-supported building

Step 6: Comparing H_C with D_L, potential failure

In this step, the critical pile length (H_C) that is in touch with liquefiable soil should be assessed in order to identify appropriate method to retrofit the foundations to resist seismic loading. If $H_C \geq D_L$, most of the pile length will be in touch with liquefiable soil, the pile would be at risk of failure due to buckling and, thus, would require retrofit.

Step 7: Cementing the soil surrounding the pile within the liquefiable soil zone

This step presents an appropriate method for pile-supported structures by using cementation of the soil surrounding the pile within the liquefiable zone. The cement injection technique (see section 4.2) in stabilised soil may be sufficient to prevent triggering of liquefaction where the pile length in touch with the liquefiable soil is within 6.5D of the total pile length. The micro-jet grouting method can be used for the cementation. This method is characterised by its ability to produce soil improvement structures with arbitrary shapes and large diameter including walls, fans, and lattices (Stoel, 2001; Burke, 2004; Stark, 2009; Malinin, et al., 2010; Yamauchi, et al., 2017). This construction method can be used near boundaries of existing structures and the total construction cost, including economic damage, of grouting can be lower than the construction cost of conventional methods (Stoel, 2001; Yoshida, 2010; Saurer, et al., 2011; Yamauchi, et al., 2017).

Step 8: Identify remediation technique

In this step, the critical pile length (H_c) that is in touch with liquefiable soil (estimated in Step 6) should be compared with length of pile to identify an appropriate method to retrofit the foundations to resist seismic loading. Therefore, for $H_c < D_L \geq 6.5D$, the cement injection alone cannot be used for stabilisation.

Step 9: Using lattice structure to mitigate the risk of buckling failure

According to the analysis of the lattice structure mechanism (see section 4.3), it can be seen that this mechanism is sufficient to prevent liquefaction triggering and ground improvement in liquefiable soil when cementation is not enough (i.e. when $D_L \geq 6.5D$). However, if the thickness of liquefied soil layer(s) is higher than the total pile length, it would be recommended to use both techniques.

A systematic evaluation has made to develop this methodology on the basis of understanding of the potential for initiation of liquefaction, the mechanics of the liquefaction process, various aspects of pile failure and the feasibility of a successful remediation technique. Numerical analyses have developed to the point the effects of liquefaction triggering on seismic response of structures and soil stiffness and the results of analysis illustrated a robust framework for mitigation of pile foundations by using the recent design earthquakes. The main differences that are made this framework better and constitutive than the conventional frameworks use of three dimensional nonlinear and effective analysis with recent key parameters and present the simple, effective and economically viable techniques.

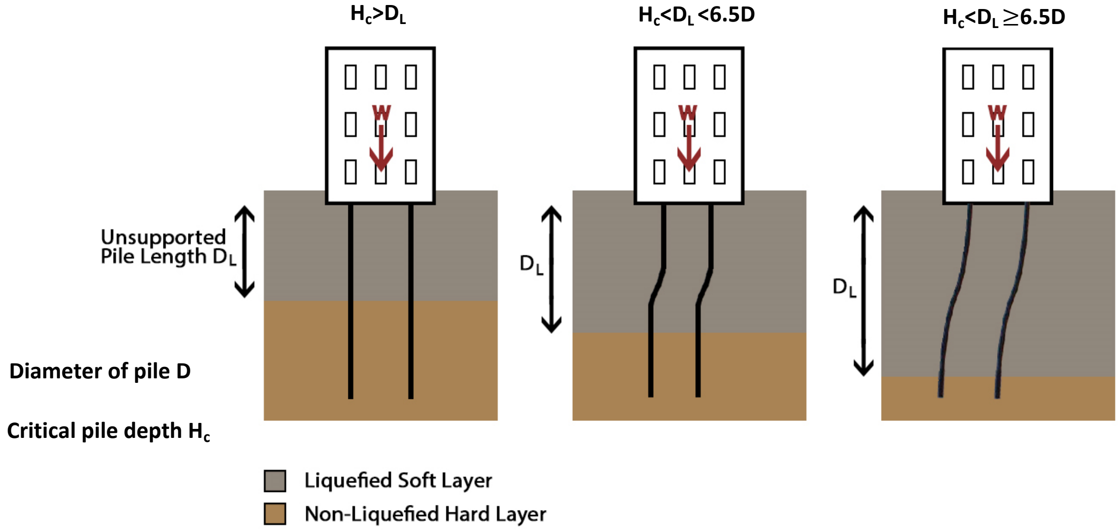


Figure 14 Concept of critical length of the pile and unsupported length of the pile (adapted from Bhattacharya and Goda, 2013)

6 CONCLUSIONS

The seismic risk of liquefaction was evaluated by comparing relevant mitigating measures against pile failure in liquefied soil. Numerical analyses of unimproved and stabilised soil models with cement injection and lattice structure techniques were performed to investigate their effects in liquefiable soil when subject to seismic loading. A reinforced concrete pile constructed in a stratified soil system and carrying both axial and seismic earthquake loading was analysed for both cementation and lattice structure retrofit within the liquefiable soil zone. It was found that for the range of parameters used in this study, the bending moment reduction using cement injection in the liquefiable soil may be sufficient to prevent liquefaction triggering for thicknesses of up to 1/3 (6.5d) of the length of the pile in touch with the liquefiable soil. For conditions other than these, it is recommended that cement injection mechanism should be considered as secondary rather than primary mechanism for ground improvement in liquefiable soil. The lattice structure technique, on the other hand, was found to reduce pore pressure effectively, even in the high thickness of liquefiable soil. This improvement was most likely achieved by wall being prevented and through lateral soil movements being restrained. However, in the higher thickness of the liquefiable soil the walls were flexible and so may just improve 0.50%. These were most likely due to lateral movements or densification of the sand beneath the shear wall. Thus, it is recommended that in these conditions may be better to consider a combination of both techniques for ground improvement. Overall it was found that the both techniques are effective and economically viable to reduce or avoid potential structural damage caused by liquefied soil.

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