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ANALYSIS OF DESIGNED PILES UNDER SEISMIC CONDITIONS IN LIQUID SOILS

Reem Siham Tawfeeq[1*]
Bilal Muiassar M. Salih[2,]
[1, 2,] Department of Civil Engineering, University of Al-Iraqia, Baghdad, Iraq
Email: engreemsiham86@gmail.com, bmm84eng@gmail.com

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Abstract:

In order to successfully design and analyze pile foundations in liquefiable soils, a basic overview of state of art and new technology is offered. Case histories are taken into consideration and used to show two separate design instances. One is the pile's reaction to intense shaking that is followed by the liquefaction of the ground or the creation of high pour water pressures, and the other is pile's reaction to pressures as well as displacements that are brought on by the lateral spread of the liquefied earth. The existing capacity for a valid examination of piles under heavy shaking is shown using analysis of the centrifuge tests on the pile foundations. For demonstrating the benefits of thorough analyses in assessing a suggested design, a practical example that involves 1.5m cast in place reinforced concrete piles that support a 14 story building that is situated on reclaimed ground on liquefiable soils has been provided.

Keywords:: Seismic Analysis; Liquefaction; practical example; Seismic response; water pressures; Centrifuge tests

1. INTRODUCTION

Under the water table, loose, cohesionless sands and silts can liquefy or experience significant porewater pressures. Strongness and stiffness may be significantly reduced as a result of the high pore water pressures (Keefer 1994; Emmer and Cochachin 2013). In these liquefiable soils, the seismic design of pile foundations presents extremely challenging difficulties in analysis and design (Menoni 2011). If liquefaction happens, pile foundation could experience significant shaking even though soil has been entirely liquefied and has a low degree of stiffness (Vlachakis et al. 2020). The pile is vulnerable to significant cracking or perhaps breakage during this shaking phase. Additionally, liquefaction causes a significant rise in the pile cap displacement when compared to non-liquefied situation. Considerable lateral spreading or down-slope displacement can happen after liquefaction if the soil's remaining strength is lower compared to static shear stress values that are brought on by sloped site or free surface, like river bank. The piles may fail as a result of destructive stresses from the shifting soil. Many examples of failures were prevalent over the word caused by earthquakes (Cutter 2018; Ross *etal*. 2019; Selva *etal*. 2021; Stepinac et al. 2021). These two crucial design challenges have just recently started to be effectively addressed by the profession. The advancements in analysis and the results of centrifuge testing are to thank for this. The advancement in analysis has also for deeper, more thorough analyses of case studies, which has increased understanding of the issues of the design. The goal of this study is to provide broad overview of state of the art and developing technologies for designing and analyzing pile foundations in the liquefiable types of soil while taking under consideration lateral pressures caused by post-liquefaction displacements as well as the impacts of seismic shaking.

2. PILE FOUNDATIONS BEHAVIOR DURING EARTHQUAKES

Large displacements of the ground can occur during liquefaction on slopes or in the direction of open face, like the river bank. Figure 1 depicts the displacements caused by lateral spreading during the Burma earthquake (Url-1).

Figure 1. Ground displacements in burma earthquake (adapted from Ref. [1])

After the 1993 earthquake in Japan, utterly unsuccessful pile heads the inside downward view of No. 54 pile at a depth is shown in Figure 2. At 6.5 m depth, one meter of the pile entirely disintegrated, leaving a massive shear deformation and longitudinal fissures that could be observed. In addition, the image showed a lateral shift at the breakup site of half the pile radius (Mori et al, 1994).

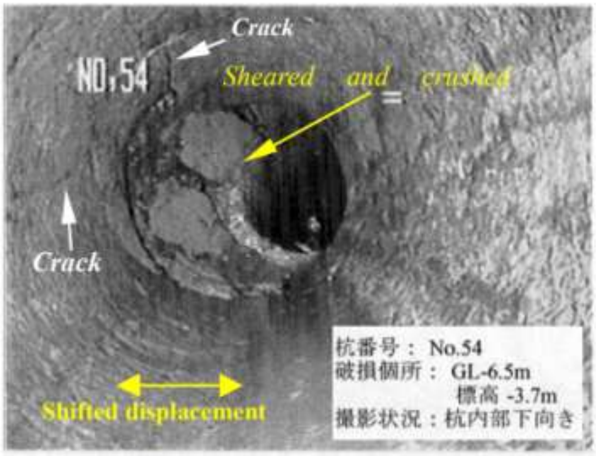


Figure 2 Downward inner view of the pile No.5 at depth of old sea bottom

When the Nihon-Kai-Chubu earthquake occurred in 1983, the foundation soils melted. The approach embankments failed as a result of lateral spreading, however, pile foundations were unharmed. Figure 3 depicts pile holding crane rail on the Port Island, a small island off the coast of Kobe City. After the Kobe earthquake caused liquefaction, the ground in this area moved over 1m. Figure. 3 make it very clear how the earth and pile are moving relative to one another. The pile, however, was built to withstand heavy shears and moments and kept its integrity.

Figure 3. Damage to a pile by 2 m of ground displacement in Niigata earthquake, 1964.

3. SEISMIC ANALYSIS OF PILE FOUNDATIONS

The analysis of the case studies had amply shown design issues that pile foundations provide in the liquefied soils. A trustworthy method for estimating the impacts of the shaking of the earthquake and post-liquefaction displacement on the pile foundations is crucial for dealing with these issues. The benefits and drawbacks of the various strategies will be highlighted in an overview of the techniques now in use. The review's objective is to provide a comprehensive, current evaluation of overall state of art. Technical information won't be discussed. The cited references are recommended reading for these.

4. A SUMMARY OF FUNDAMENTAL IDEAS

As a connected system, the pile foundation-structure system shakes during earthquakes. It should logically be examined as a connected system. This kind of analysis, however, cannot be used in engineering practice. Numerous well-liked structural analysis applications are unable to properly incorporate the pile foundation to computational model. Computing requirements are too great even when it is possible. Different approximation methods of analysis are therefore employed.

Winkler springs and dashpots are used most frequently for the simulation of soil damping and stiffness while analyzing pile foundations. The springs could be linear or elastic. Some organizations, like the American Petroleum Institute (API1995), provide detailed instructions for the creation of non-linear load-deflection (p-y) curves which may be utilized for modeling the nonlinear springs. According to information from static and slow cyclic loading tests that have been conducted in the field, API (p-y) curves, which are the most frequently utilized in engineering practice, were developed. Even under the slow cyclic and static loading, it was demonstrated that the dependability of those (p-y) curves for a study of the pile foundations is relatively poor (Murchison and O'Neill 1996). The evidence for their performance under seismic loads is weak.

Spring and dashpot models are used to simulate the near field interactions between the pile and soil. The motions of the seismic base and the free field motions that are applied to ends of every one of the Winkler springs excite near field pile-soil systems as well as any structural material that has been integrated with pile. Using 1D dynamic analyses and a program like SHAKE (Schnabel *etal*. 1972) or DESRA-2C (Finn et al. 1997), free field motions at necessary altitudes in soil layer have been calculated.

It is possible to utilize finite element continuum analyses that are based upon real soil parameters as a substitute to Winkler type computation model. Due to the time required for the computations, dynamic nonlinear finite element analyses in time domain utilizing complete 3D wave equations is currently impractical for the application in the field of engineering. However, there is a possibility to obtain trustworthy answers for the nonlinear responses of the pile foundations with a significantly decreased computational cost by loosening some boundary constraints that are related to full 3D analyses. The results for excitation caused by vertically propagating horizontally polarized shear waves are quite accurate. Finn and Wu have published a thorough explanation of this strategy along with multiple validation studies (Wu & Finn 1997). The technique has been included in software PILE-3D

5. VERIFICATION OF METHODS OF ANALYSIS

Quantitative information on seismic reaction of the pile foundations is scarce and also difficult to get in practice. The reliability of a variety of the approaches for seismic analyses of pile foundation types has recently been evaluated more realistically thanks to data from seismic load of the model pile foundations in the centrifugation tests. Below is a sample example.The PILE-3D model only preserves these characteristics which had been demonstrated to be crucial in those analyses since the seismic response analyses are typically carried out under the assumption that input movements are vertically moving shear waves that are horizontally polarized. Shear stresses on the horizontal and the vertical planes as well as normal stresses in shaking direction are these characteristics. According to Figure 4, 3-D finite elements are used to simulate the soil. Beam elements or block elements are used to model the pile. We presume that the pile will still be elastic. Such presumption has been consistent with the principle of the design that foundation's structural components shouldn't budge. When deformations go over the cracking limit, the cracked section moduli are employed in the study of concrete piles.

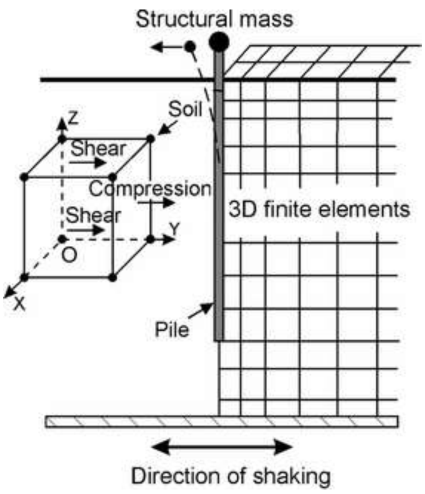


Figure 4: Pile-soil reaction quasi-3D model

5.1. PILE-3D analysis

In the centrifuge test at California Institute of Technology, a single pile's seismic reaction was examined using PILE-3D. You may read about the test's specifics in Finn and (Wu and Finn 1997). The system of soil-pile-structure that has been employed in the test is depicted in Figure 5. a 60g nominal centrifuge acceleration was applied to the system. At the system's foundation, horizontal acceleration record with 0.158 g peak acceleration has been entered. Prior to shaking, shear moduli distribution has been measured with the use of bender elements while centrifuge has been in a state of motion. Figure 6 displays the calculated and observed moment distributions along pile at maximal pile head deflections instant (Jie et al. 2013). The moments calculated by PILE-3D and the measured moments match up fairly well. When compared to a recorded peak value of 325kn m, the peak moment that has been predicted by quasi-3D finite element analysis has been 344kN m.

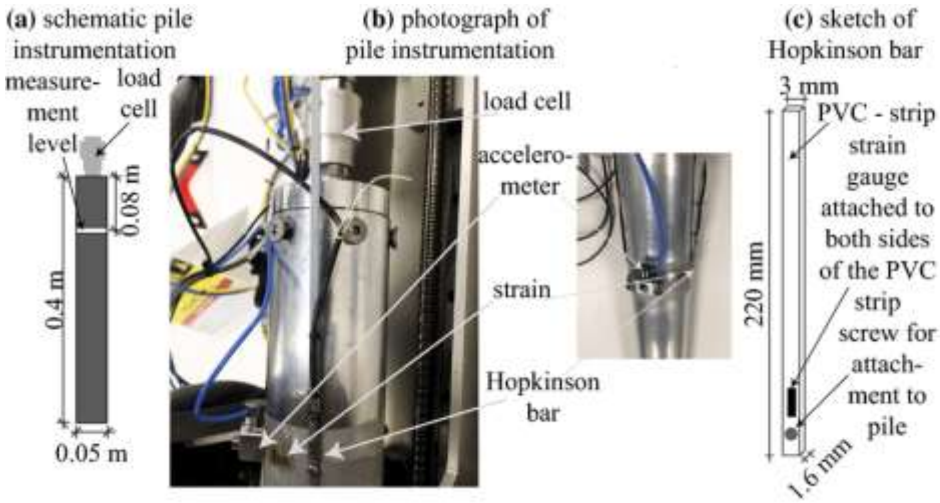


Figure 5. Instrumented pile for centrifuge test.

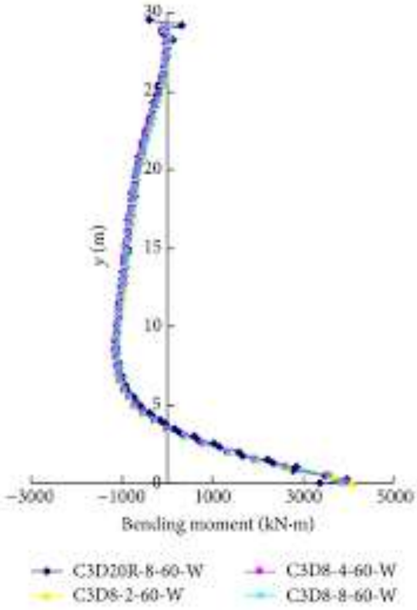


Figure 6. Comparison of measured and computed bending moments

5.2. Analysis using API p–y curves

In order to simulate the interaction between the soil and the piles, dynamic study of foundation-superstructure system has been carried out as well by the use of p - y curves that have been recommended by the API (API, 1995). The equation defines these p-y curves.

$$p = 0.9p_u \, tanh\left[\frac{kH}{0.9p_u} y\right] \quad (1)$$

Where, H represents depth, k represents initial modulus of the subgrade reaction, y represents lateral deflection, and pu represents maximum bearing capacity at that depth. The sand surrounding the pile has a relative density of D 1/4 38%. According to the API standards, this translates to k of about 15,000kN/m3. Figure 7 depicts distributions of the moments for k = 14 15,000kN/m3.

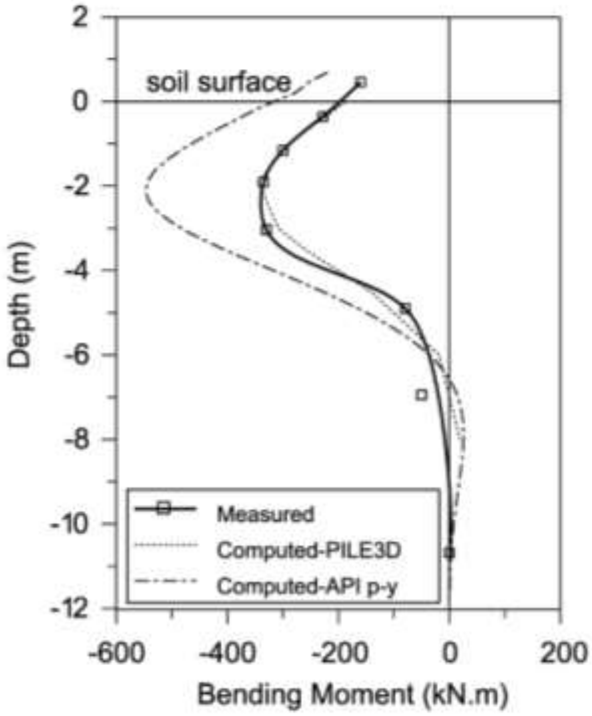


Figure 7. Comparison of measured and computed bending moments

In contrast to the measured moment of 325 kN m, the Winkler model can predict a maximal moment of 550kN m. It appears that API p-y curves are too rigid for such degree of the shaking. K 14 2500 kN/m3 is needed to produce a

reliable approximation of the peak moment in pile, which is just 1/6 of the API-recommended amount. The stiffness of the API k 14 15,000kN/m3 provides a very excellent approximation to recorded bending moments in a different test in same sand that is run at quite low levels of acceleration with an acceleration peak of 0.04 g as seen in Figure 8.
In this situation, the initial stiffness affects the reaction, which is almost elastic. These findings imply that the API p-y curves have a suitable starting stiffness, but that during intense shaking, curves don't deviate quickly enough from initial tangent at origin. As a result, a too wide range of displacements are mobilizing stiffness near to the starting value. These findings imply that the empirical pressure-displacement relations that describe interactions of soil-pile and have been based upon data from slow cyclic or static loading tests might not be very trustworthy in the case of being utilized to carry out the dynamic response analyses.

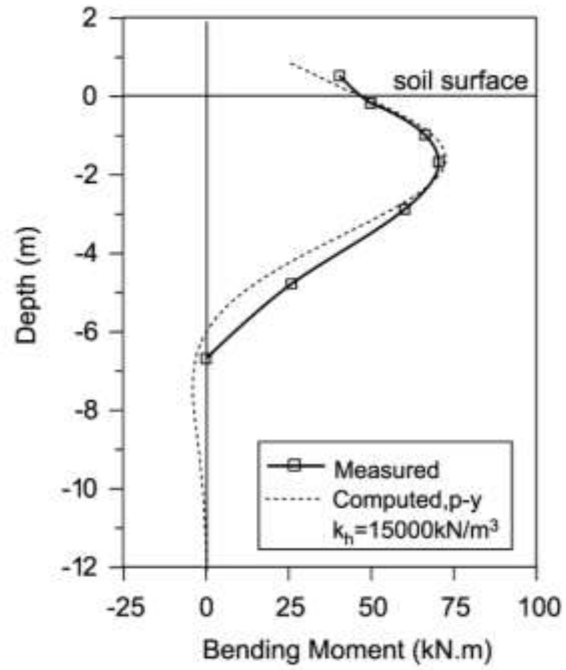


Figure 8. Using the API technique, compare the bending moments that were computed and measured.

6. A PSEUDO-STATIC ANALYSIS OF BUILDING FOUNDATIONS

Building pile foundations are frequently designed using the most basic form of analysis. Although the foundation and building's dynamic interaction has an impact on design loads, this influence is disregarded. Without doing a dynamic analysis, dynamic loads on building have been calculated in accordance with building code requirements. It is presumed that a rigid base supports the structure. The pile foundation is next subjected to the estimated shear and moment values, corresponding to the happening of the yield at the column base, and static analysis has been carried out in order to identify corresponding shear and moment values in pile. In this analysis, pile head has been frequently presumptively locked against the rotation. The analysis has typically been carried out under the assumption that the ground-pile interaction may be approximated using Winkler springs, as shown in Figure 9. Only near field section of overall model in Figure 9 has been utilized for the static analysis which is discussed here.

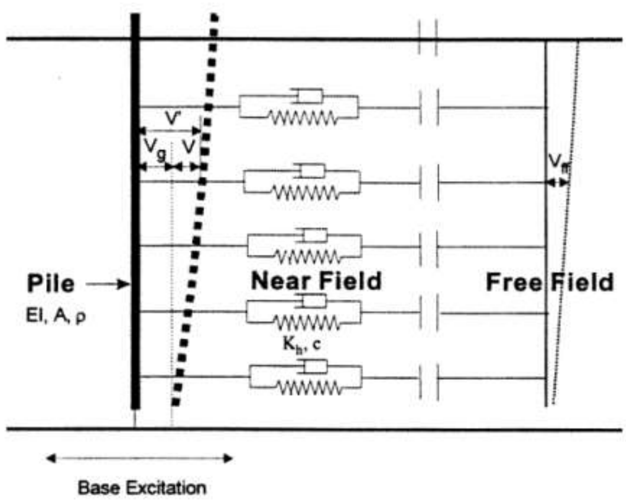


Figure 9. A Winkler spring model for pile foundation analysis

Elastic springs are sometimes used by designers to simulate the interaction between the earth and the pile. When there is significant shaking, nonlinear (p-y) curves should be used in place of elastic springs to account for the soil's nonlinear reaction. The American Petroleum Institute recommends using such curves. The force applied to foundation reflects yield strength of pile-supported column. Which is only a nominal load capacity, though. Numerous structures can hold 1.5–2.0 times their nominal worth. The over strength factor is defined as nominal strength multiple. If the intention of design is for yielding to occur in columns rather than foundation piles, then the over strength of super-structure must be taken under consideration while designing pile foundations

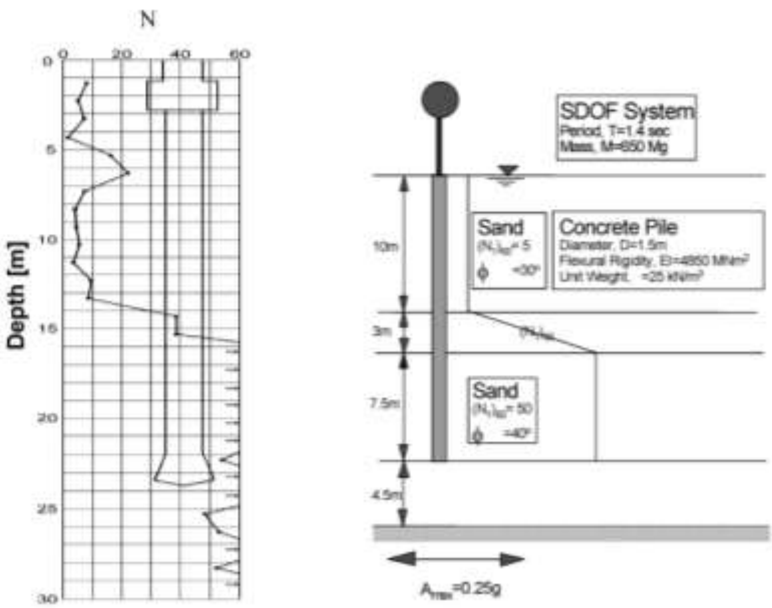
7. CENTRIFUGE TEST DATA VALIDATION OF PILE RESPONSE ANALYSIS FOR LIQUEFIABLE SITES

The soil modulus and strength can be significantly reduced by seismic pore water liquefaction and pressures, which also reduces stiffness of the pile head. By lowering subgrade reaction coefficient, in the case where elastic Winkler model has been employed, and ultimate strength and initial stiffness, in the case where (p-y) curves have been utilized, this impact has been considered in the case of employing semi-empirical computation models. PILE-3DEFF program, which has been created by (Finn and Thavaraj, 1997), performs the tracking of evolution of seismic pore water pressures and continuously adjusts the soil parameters to be suitable with the present condition of the effective stress.

By utilizing modified Martin-Finn-Seed pore water pressure generation model version (Martin *etal*. 1995) which incorporates Byrne's two-constant volume change expression, incremental seismic pore water pressures have been generated in every one of the individual elements, based upon accumulated volumetric strain that is present in that element and current increment in the shear strain. To take under consideration effects of the fluctuating seismic pore water pressure levels, the moduli and the shear strength values of foundation soils have been adjusted continuously. Data from centrifuge testing were used to validate PILE-3DEFF in a collaborative effort with University of California, Davis (Finn and Thavaraj 2001).

8. EXAMPLE FROM PRACTICE

A 14-story apartment structure on reclaimed ground is supported by 1.50-inch-diameter cast-in-place reinforced concrete piles for the columns. In order to highlight some of the concepts stated previously and to demonstrate the kind of findings that can be obtained with the status of the analyses, results from early investigations of pile structure system will be discussed here. (Figure 10,a) depicts the pile and soil conditions. Fig. 22 depicts the idealized site circumstances for demonstration analyses. During the planned earthquake, it is anticipated that the top 10 meters of recovered soil will liquefy. In (Figure 10,b), the mass that has been mounted on pile stands in for mass equivalent of force that a pile is carrying. In order to roughly simulate inertial interaction between superstructure and pile foundation, the mass was placed on the pile. The mass has a 1.4-second period of vibration as a result of being attached on pile head by flexible support, which corresponds to prototype structure's predicted fundamental period. There were two different types of assessments done: total stress dynamic analyses that ignore liquefaction and seismic pore water pressure levels, and the effective stress analyses, which accounts automatically for seismic pore water pressure levels. Generally, the soil characteristics are modified continuously for the shear strains and pore water pressures present. Input acceleration record's max acceleration has been 0.25 g, which has been increased to 0.40g reaching surface. In the presence of liquefaction, the surface accelerations are barely noticeable.



a b

Figure 10. Model of soil–pile-structure system

CONCLUSION

This paper provides a broad review of the critical elements influencing seismic design of the piles in order to withstand the loading of the earthquake in the liquefiable types of soil. Case studies demonstrate that the damage has been concentrated in important locations, like pile head in the case where it's locked against rotation and boundary between non-liquefied and liquefied layers, and occurs more frequently in liquefiable soils. Large loss levels in the strength and stiffness occur when the soil liquefies, which can cause significant moments as a result of increasing displacements. Case studies demonstrate as well that piles may successfully withstand significant displacements and violent shaking when they are appropriately built. Reliable estimate values of environmental load, realistic evaluations of the pile head fixity, and analyses techniques which can effectively gauge how pile-soil-structure systems will react to the intense shaking which could cause liquefaction in the complex layered systems are all essential components of good design.

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