PILE DESIGN IN LIQUEFYING SOIL

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ABSTRACT:

The behavior of piles in seismic areas is significantly affected if the soil liquefies. The strength of the soil and its stiffness decrease due to the increase in pore water pressure. The observed pile damage during earthquakes has provided an insight into the mechanism of soil-pile interaction in liquefying soils. The piles in liquefied soil may undergo large lateral displacements and my fail due to bending or buckling. The paper discusses the observed performance of piles in liquefying soils and the suggested methods for their design.

KEY WORDS: Piles; Soils; Liquefaction; Performance; Design.

1. INTRODUCTION

Pile foundations are regarded as a safe alternative for supporting structures in seismic areas and have been used for this purpose in non-liquefying as well as liquefying soils. The overall design problem in either case is complicated. The seismic loading induces large displacements/strains in the soil. The soil behavior becomes non-linear. The shear modulus of the soil degrades and damping (material) increases with increasing strain. The stiffness of piles should be determined for these strain effects. The stiffness of the pile group is estimated from that of the single piles by using group interaction factors. The contribution of the pile cap, if any, is also included. The response of the single pile or pile groups may then be determined using principles of vibrations.

In liquefiable soils, progressive buildup of pore water pressure may result in loss of strength and stiffness resulting in large bending moments and shear forces on the pile. The mechanism of pile behavior in liquefying soil has been investigated by several investigators in the recent years based on observations of pile performance during earthquakes and studies in the centrifuge. The behavior and design of piles in liquefying soil is discussed in this paper.

The pile behavior in liquefied soil is strongly influenced by non-linearity of soil, resulting in soil's shear modulus degradation and increased material damping with displacement. Soil displacements and lateral spreading associated with liquefaction may exert damaging lateral pressure on the piles (Finn and Fujita, 2004, Ishihara and Cubrinovsky, 2004)).

2. OBSERVATIONS DURING PAST EARTHQUAKES

Excess pore pressure during seismic motion may cause lateral spreading resulting in excess moments in the piles and settlements and tilt of the pile caps and the superstructure. Excessive lateral pressure may lead to failure of the piles which were experienced in 1964 Niigata and the 1995 Kobe earthquake (Finn and Fujita, 2004). Figure 1 shows the damage to a pile under a building in Niigata caused by ground displacement (Yasuda et al, 1999). The ground displacement at this location was more than 1 m. Ishihara and Cubinovski (2004) investigated the performance of piles below an oil-storage tank (Fig.2)

Figure.1 Damage to pile by 2m of lateral ground displacement during 1964 Niigata earthquake (Yoshida et al.1990)

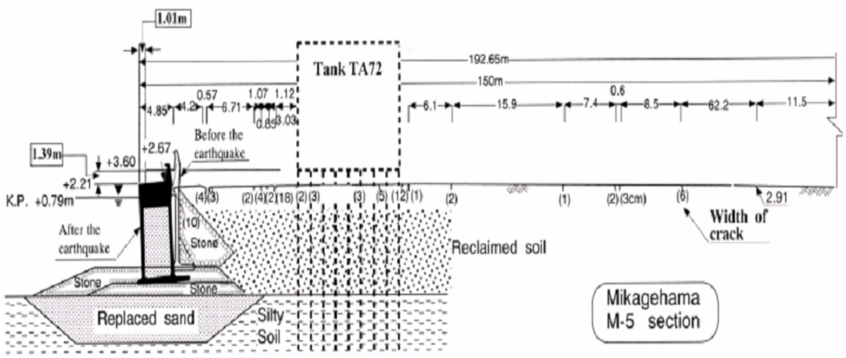

Figure 2 Detailed profile of the quay wall movement and ground distortion in the backfills at Section M-5

during the Kobe earthquake of 1995. The quay wall moved approximately 1m towards the sea. The seaward movement of the quay wall was accompanied by lateral spreading of the backfill soils resulting in a number of cracks on the ground inland from the waterfront. The lateral ground displacement was plotted as a function of the distance from the waterfront, Figure 3. The permanent lateral ground displacement corresponding to the location of Tank TA72 is seen somewhere between 35 and 55 cm (Ishihara and Cubrinovski, 2004). To inspect the damage to the piles of an oil tank site after Kobe (1995) event, 70cm wide and 1m deep trenches were excavated at 4 sections and the upper portion of the pile was exposed. The wall of the cylindrical piles was cut to open a window about 30cm long and 15cm wide. From this window, a bore-hole camera was lowered through the interior hole of the hollow cylindrical piles to examine the damage to the piles throughout the depth (Ishihara and Cubrinovski, 2004). The lateral displacement and cracks on two damaged piles (pile no 2 and 9) in the pile group supporting the tank are shown in figures 4 and 5 respectively. It was further observed that the piles showed cracking and suffered maximum damage at the depth of the interface zone between the liquefied fill deposit and the underlying non-liquefied Silty soil layer. This observation indicates that liquefaction and resulting lateral spreading of the backfill soil seriously affected the pile performance. The maximum moment on the pile thus seems to occur at the interface zone of the liquefying and non-liquefying soil layers.

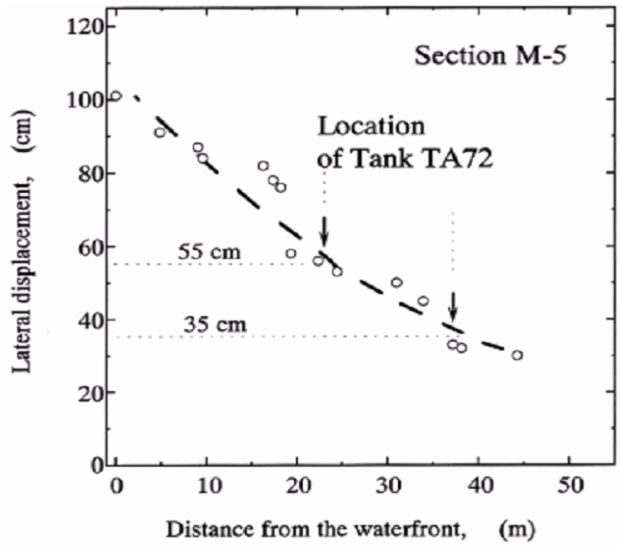

Figure.3 Lateral ground displacement versus distance from the waterfront along Section M-5(Kobe 1995 EQ)

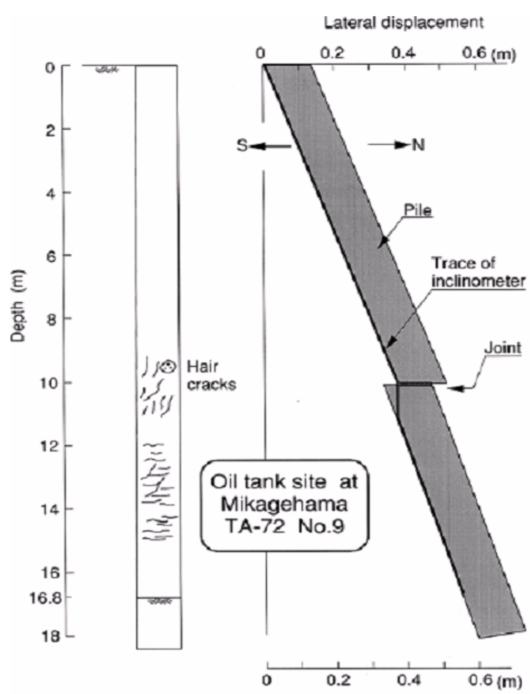

Figure. 4 Lateral displacement and observed cracks on the inside wall of Pile No.9 Kobe 1995 EQ

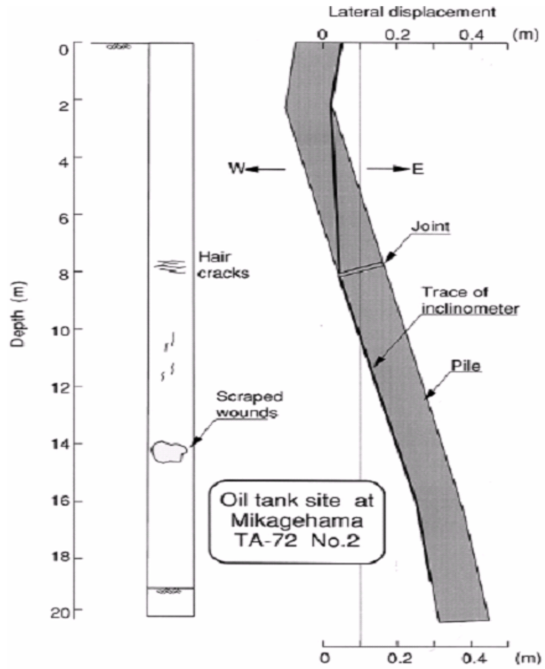

Figure.5 Lateral displacement and observed cracks on the inside wall of Pile No.2 Kobe 1995 EQ

3. LABORATORY STUDIES

In order to determine the various factors affecting the behavior of piles in liquefying soil, studies have also been conducted using large shake table and centrifuges (Abdoun et al., 2003: Boulanger et al., 2003; Suzuki et al., 2005; Tokimatsu et al., 2004, 2005). The development and distribution of excess pore water, subgrade reaction and stiffness of soil-pile system and deflected shape of the pile have been investigated. Based on the results of centrifuge tests Liyanapathirana and Poulos (2005) concluded that for the case of free head piles when a liquefying soil layer is underlain by a non-liquefying soil layer, , maximum bending moment develops at the interface between the two layers irrespective of the thickness of the liquefying soil layer. For fixed head piles, the maximum bending moment develops at the interface if the thickness of the liquefying layer is greater than one third, and less than two thirds, the total thickness of the soil deposit.

Bhattacharya (2006) re-examined the damage to piles during 1964 Niigata and 1995 Kobe earthquakes and noted that pile failure in liquefying soil can be better explained as buckling type failures.

4. METHODS OF DESIGN

The design of pile foundations in liquefied soils requires a reliable method of calculating the effects of earthquake shaking and post liquefaction displacements on pile Foundations (Finn and Fujita,2004). The methods currently in use for design of piles in liquefying soil are ;

1. The force or limit equilibrium analysis and
2. The displacement or p-y analysis.

4.1 The Force or Limit Equilibrium Analysis

This method of analysis is recommended in several Japanese design codes for analysis of pile foundations in liquefied soils undergoing lateral spreading (JWWA, 1997; JRA, 1996). The method involves estimation of lateral soil pressures on pile and then evaluating the pile response. A schematic sketch showing lateral pressures due to non- liquefied and liquefied soil layers is shown in Figure 6. The non-liquefied top layer is assumed to exert passive pressure on the pile. The liquefied layer is assumed to apply a pressure which is about 30% of the total overburden pressure This estimation of pressure is based on back calculation of case histories of performance of pile foundations during the Kobe earthquake . The maximum bending moment is assumed to occur at interface between the liquefied and non-liquefied soil layer .

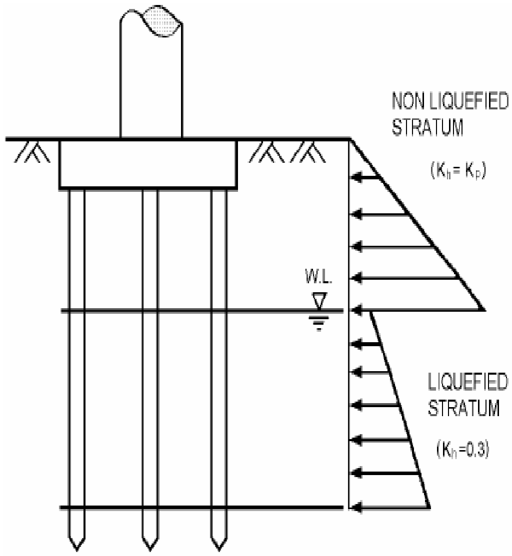

Fig 6. Schematic Sketch Showing Pressure Distribution against the Piles due to Lateral Soil Flow associated with Liquefaction (JWWA, 1997) (Ashford and Juirnarongrit, 2004 and Finn Fujita, 2004).

4.2 Displacement or p-y Analysis

This method involves making Winkler type spring mass model shown schematically in Fig. 7. The empirically estimated post liquefaction free field displacements are calculated. These displacements are assumed to vary linearly and applied to the spings of the soil-pile system as shown in Fig. 7 (Finn and Thavaraj, 2001). Degraded p-y curves may be used for this kind of analysis. In the Japanese practice the springs are assumed to be linearly elastic-plastic and can be determined from the elastic modulus of soil using semi-empirical formulas (Finn and Fujita, 2004). The soil modulus can be evaluated from plate load tests or standard penetration tests. Reduction in spring stiffness is recommended by JRA (1996) to account for the effect of liquefaction. Such reduction is based on FL (factor of safety against liquefaction). These reduction factors are shown in Table 1.

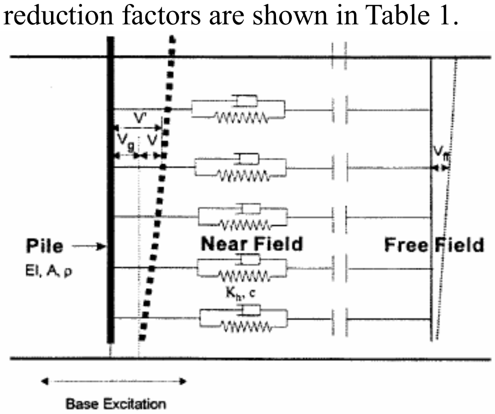

Fig.7. A Schematic Sketch for Winkler Spring Model for Pile Foundation Analysis (Finn and Thavaraj, 2001)

Table.1 Reduction coefficients for soil constants due to liquefaction (JRA, 1996)

Range of F_L	Depth from present ground surface X(m)	Dynamic Shear Strength Ratio R	
		$R \le 0.3$	$0.3 < R$
$F_L \le 1/3$	$0 \le X \le 10$	0	1/3
	$10 \le X \le 20$	1/3	1/3
$1/3 < F_L \le 2/3$	$0 \le X \le 10$	1/3	2/3
	$10 \le X \le 20$	2/3	2/3
$1/3 < F_L \le 1$	$0 \le X \le 10$	1/3	1
	$10 \le X \le 20$	1	1

The North American Practice is to multiply the p-y curves, by a uniform degradation factor p, called the p-multiplier, which ranges in values from 0.3 - 0.1. The values 'p' seems to decrease with pore water pressure increase (Dobry et. al; 1995) and become 0.1 when the excess pore water pressure is 100%. Wilson et al (1999) suggested that the value of 'p' for a fully liquefied soil also depends on the initial relative density D_r. The values of 'p' range from 0.1 to 0.2 for sand at about 35% relative density and from 0.25 to 0.35 for a relative density of 55%. It was found that the resistance of the loose sand did not

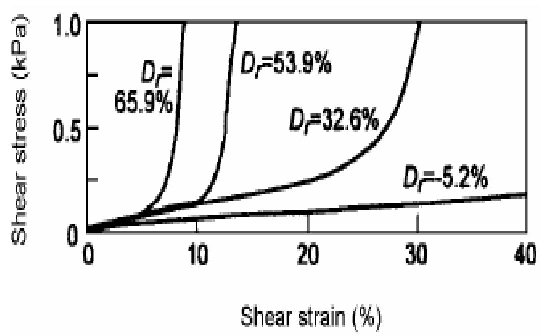

Fig. 8. Post-liquefaction un-drained stress-strain behavior of sand (Yasuda et al 1999)

pick up even at substantial strains but the denser sand, after an initial strain range in which it showed little strength, picked up strength with increasing strain (Fig. 8) . This finding suggests that the good performance of the degraded p-y curves which did not include an initial range of low or zero strength ,must be test specific and the p-multiplier may be expected to vary from one design situation to another. Dilatancy effects may reduce the initial p-y response of the dense sands (Yasuda et al 1999). Ashford and Juirnarongrit (2004) compared the force based analysis and the displacement based analysis for the case of single piles subjected to lateral spreading problems. They observed that the force based analysis reasonably estimated the pile moments but underestimated pile displacements. The displacement analysis was found to make better prediction about the pile moment and the pile displacement

5. DISCUSSION

The 'force based' method of pile design in liquefied soil recommended in the Japanese codes is based on the observation of pile damage during the past earthquakes especially the Kobe earthquake. The pile performance in liquefied soil may be influenced by earthquake parameters, variations in the soil profile and the pile geometry. A question naturally arises as to how far these factors are accounted for in this method. Similarly, the 'displacement' method requires the prediction of surface displacements which are estimated empirically and the development of p-y curves for generating the post-liquefaction behavior introduce an unknown uncertainty. These questions need further study as no satisfactory solutions are available as of today (2008).

6. CONCLUSIONS

The design of pile foundations in liquefying soil needs an understanding of soil liquefaction, behavior of soils following liquefaction and the soil-pile interaction. The practice of pile design in liquefying soil has progressed considerably in the last decade based on observations during the past earthquakes and experimental studies on centrifuge and large shake table. However, there are several parameters and questions which need to be examined further in detail.

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