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REDUCING LIQUEFACTION RISK IN THE SANDY SHORE OF CASPIAN SEA BY USING STEEL PILE DRIVING

Y.Ahadi[1] and M.Tehrani zadeh[2]

ABSTRACT

One of the methods for dynamic compacting and reducing liquefaction risk of soil is pile driving. In the present project for reducing liquefaction risk and to increase soil's bearing capacity, steel piles were used with 36 cm diameter and 2cm thickness for driving in soil and pile hammer k25. For proving soil layers at the centre of the site, a BHA well was dug. After testing the soil patterns, it was detected that the soil's bearing capacity is low and liquefaction risk is high. To reduce the liquefaction risk and to increase soil's bearing capacity, steel pile driving was used as a method for dynamic compacting and after driving 60 piles in the site and repeating the tests, results showed that the liquefaction risk has been vanished and soil's bearing capacity has increased to about 200 percent. at this project 113 steel piles were used.

Introduction

One of the factors of destruction in earthquakes is failure of land. Failure of land could happen because of cracks, openings, unnatural or unequal movements, or the loss of strength. Loss of strength may happen in sandy lands because of increase in pore-water pressure. This phenomenon, which is called liquefaction, could happen in incompact and saturated sands. Increase in the pore-water pressure may lead to the reduction in shear strength and even to the total disappearance of compression strength in soil. The soils which loss their shear strength act as a strong liquid. During the earthquake, the liquefaction of the land and sand fountain is seen. This phenomenon always has been observed in earthquakes. When the soil turns to this state, the buildings, which are built on them, immerses into the soil slowly.

The theory of liquefaction

The strength of sandy soils occurs because of their internal friction of particles.

$$\tau = \sigma_n * \tan \varphi + C \tag{1}$$

for sand C = 0 and we will have:

[1]Graduate Research Assistant, Dept. of Civil Engineering, University of Amir kabir.
[2]Professor, Dept. of Civil Engineering, University of Amir kabir.

$$\tau = \sigma_n * \tan \varphi \tag{2}$$

If the soil is saturated the shear strength will be equal to

$$\tau = (\sigma_n - u) * \tan \varphi \tag{3}$$

In which τ is shear strength and σ or σ_n is the plane normal stress in z depth, φ is the internal friction angle of the soil, c is the viscosity among the particles and u is the pore-water pressure.
If the normal stresses in z depth are being considered in saturated soil:

$$\begin{aligned} \sigma_n &= \gamma_{sat} Z \\ u &= \gamma_w . Z \end{aligned} \tag{4}$$

Thus the shear strength would be as below:

$$\begin{aligned} \tau &= (\gamma_{sat} Z - \gamma_w Z) \tan \varphi \\ \tau &= (\gamma_{sat} - \gamma_w).Z \\ \tau &= \gamma^{'} Z \tan \varphi \end{aligned} \tag{5}$$

In the said equation γ_{sat} is the saturated unit-weight, γ_w water's specific weight and, $\gamma^{'}$ is the submerged unit-weight. Now, if the pore-water pressure increases to $+\Delta u = \gamma_w h_w$ because of earth movements, we will have:

$$\tau = (\gamma^{'} Z - \Delta u) \tan \varphi \tag{6}$$

It is observable that the increase in the pore-water pressure will reduce the soil's shear strength.
When soil's shear strength disappears, then:

$$\begin{aligned} &\gamma^{'} Z - \gamma_w h = 0 \rightarrow \gamma^{'} Z = \gamma_w h \rightarrow \frac{h_w}{Z} = \frac{\gamma^{'}}{\gamma_w} \\ &i_{cr} = \frac{\gamma^{'}}{\gamma_w} = \frac{h_w}{Z} = \frac{G_s - 1}{1 + e} \end{aligned} \tag{7}$$

In which, G_s is the specific weight of the soil's solid particles, i_{cr} the critical hydraulic gradient, and e is the porosity of the soil. The soil's loss of the strength happens because of the transference of stress between the particles, which falls into the particles of the pore-water.

The reduction of effective stress between particles equals with rigidity reduction and will cause the deformation or the subsidence of the sand. Therefore, as soon as a part of the effective stress is transferred to the pore-water pressure, particles start to subside and the effect of this subsidence, which will appear in the upper levels, the upper structures, will subside too.

Piles and Pile Driving Hammers

The piles are divided into several groups according to their use, such as end bearing piles, friction piles, laterally loaded piles, pressure piles. In the present research, considering the soil layers until 91.86 ft in depth, lack of bedrocks, metal piles of 1.12 ft in diameter and 0.8 inch in thickness and average length of 40.2 ft were used. 113 piles were drove into the ground in the land of 10764 ft^2, using the hammer whose power of strokes was 7.5 ton in meter.

Piles generally are driven by hammer, which are called pile hammers. Depending on the condition and position of the piles, the falling direction of the hammer could be vertical or oblique. If the hammer is rose up using a cable and then falls freely on the pile, it is called falling hammer that in the present research the said one has been used. The number of impulses, which falls on the piles, will depend on the layers of the soil and averagely 929 impulses is used to drive the piles of 40.2 ft and totally, for 113 piles, 105000 impulses were used.

The specification of soil layers in BHA well

BHA well was dug in the center of the site in 91.86 ft in depth and the gained samples shows the soil layers in different depths as below:
From the surface of the soil to 8.53 ft in depth: poorly graded sand, light gray, low density wet.
From 8.53 to 19.36 ft : well graded gravel with light grey sand- saturated with water and medium density
From 19.36 to 24.61 ft : lean clay with sand, light grey, soft and saturated low plasticity
From 24.6 to 58.1 ft : well graded gravel with gray sand and round to sub round gravel with shell
From 58.1 to 66.60 ft : gray silt
From 66.60 to 70.86 ft : gravel with gray, low density and saturated sand
From 70.86 to 73.16 ft : soft silt, light gray with low plasticity
From 73.16 to 91.86 ft : gravel with sand and rounded particles and high density along some shells, saturated.
The depth of underground water is 1.6 from the surface.
Table 1 shows the samples along with their specification.

standard penetration test (SPT) test or the field test with ASTM D1886 standard was done which is based on the number of the impulses used for driving a cone core with the weight and height of the standard falling into the soil. In the said method, the number of impulses needed for immersing a standard core sampler by the hammer of 63.5 kg, falling height of 2.5 ft to the depth of 1.5 ft and omitting the first 0.5 ft impulses, the N-VALUE of 1 ft is calculated for the penetration.

Table 1 : The specification of samples from BHA well.

sample	Depth (ft)	description	$\gamma_d (\frac{lb}{in^3})$	$\gamma (\frac{lb}{in^3})$	e	s_r	c
A_0	4.3	Sand	-	-	-	-	-
A_1	10.9	Gravel	54.55	55.64	0.76	5.49	1.55
A_2	20.8	Clay	57.44	73.0	-	-	-
A_3	45.6	Gravel	61.05	64.31	0.57	24.54	2.40
A_4	58.8	Silte	-	-	-	-	-
A_5	70.3	Gravel	62.13	69.73	0.54	60.91	0.42
A_6	80.3	Gravel	-	-	-	-	-
A_7	90.2	Gravel	-	-	-	-	-

Table 2 and 3 illustrates the average number of impulses N in different depths. As it is clear the lowest amount of 11 for the depth of 0 to 17.4 ft has been recorded, which the existence of medium gravels along the stones causes the deviance in counting N and hence there must some amendments in the calculations. In table 4 the specifications of relative density and the strength of the layers has been illustrated.

Table 2 : average of standard penetration number in the analyzed field in different depths.

Depth (ft)	1	7.5	14.1	20.7	27.2	33.8	40.4	43.6	53.5	60.0
N (mean impulse)	8	11	14	6	14	22	29	36	41	8

Table 3 : average N in every strength zone.

range	A	B	C	D
Depth(ft)	0~17.4	17.4~24.0	24.0~56.8	56.8~66.6
N (mean impulse)	11	6	28	5

Table 4 : Specification of relative density and stability of soil layers in region.

Strength range	D_r (sand and gravel)	Stability (sand and gravel)	Shear Strength (clay)	$\gamma_{sat} (\frac{lb}{in^3})$
A	0.15~.3	soft	-	61.42~72.25
B	-	-	0.62	57.80~68.64
C	0.33~0.67	compact	-	68.64~79.48
D	-	-	0.52	61.42~72.25

Analyzing the liquefaction phenomenon

Using the standard penetration number and soil bearing capacity, in order to create a relation between the field amounts in proportion to the stress we use the τ_n / σ_0' equation and relative density (in which τ_n shows the horizontal shear stress created by the earthquake and σ_0' the initial stress of the sand's overload layer). Amounts of correspondent stress proportions would be modified according to the occurrence or lack of occurrence of liquefaction in the field concerning the modified average amount of penetration strength (N1), which in this case the measured number of standard penetrations (N) would be modified according to the effective overload pressure. Therefore:

$$N_1 = C_n . N \quad (8)$$

In the Eq (8), C_n is the correct factor and could be determine according to the following relation:

$$C_n = 0.77 \log(\frac{20}{\bar{p}}) \quad (9)$$

In this equation $\bar{p}$ is the effective overload pressure in a depth which standard penetration test has been done. The maximum amount of the C_n is 2. The amount of produced shear stress in different depths when earthquake occurs could be obtained from the Eq (10):

$$\tau_{av} = 0.65 \gamma h \frac{a_{\max}}{g} r_d \quad (10)$$

In which τ_{av} is the average uniform shear stress. r_d Is the reduction shear stress coefficient and would be less than 1. Therefore, in any point and every amount, for the maximum acceleration of the ground, the possibility of liquefaction could be determined according to the experimental method using the curve Fig.1. By determining a suitable amount of N_1 for the direction of the considered sand layer and reading the lower level of the stress proportions, $(\frac{\tau}{\sigma_0})$ correspondent to the amount of N1 and finally this task would be practical by the comparison of these stresses with the shear stress produced by earthquake (τ_{av}).

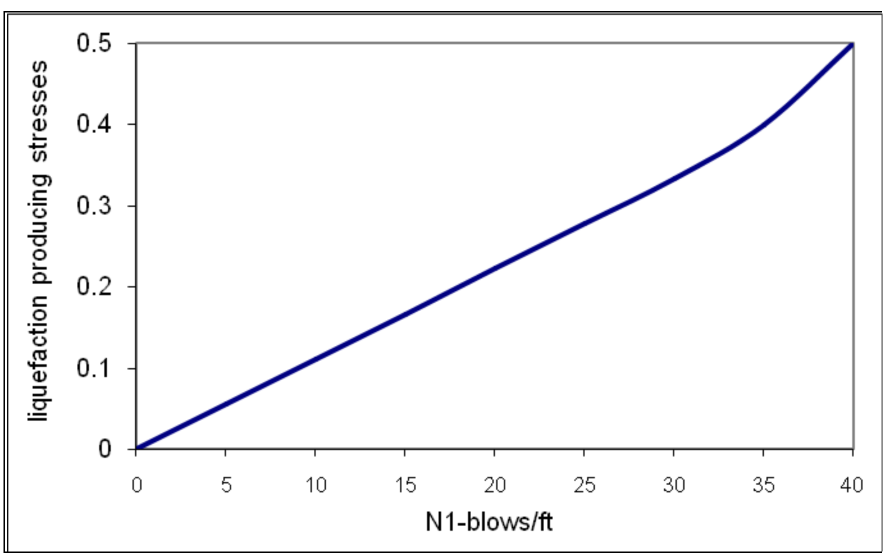

Figure 1 : The relation between liquefaction producing stresses and standard penetration number.

The obtained amounts for occurring liquefaction by an earthquake in 7.5 Richter could be stated by following equation according to the experienced observations by a suitable adjustment:

$$(\frac{\tau}{\sigma_n'}) \cong \frac{1}{90} N_1 \tag{11}$$

In table 5, the number of standard penetration has been illustrated after modifying. It is observable that always the modified standard penetration number is less than 20 and the by-law of Iran earthquakes (standard 2800) permits the building for standard number more than 20. The shear stress, which produces the liquefaction in different depths (τ) and the amount of the produced stress by earthquake (τ_{av}) in the site, has been illustrated in fig. 2. The produced shear stress by the earthquake always is more than the liquefaction producing shear stress. Especially from the depth of 3.50 towards the lower parts, this difference has been more. Therefore, the possibility of liquefaction in lower layers is more and the most dangerous part for liquefaction is in the depth of 6 meter.

Table 5 : standard penetration test before pile driving.

Depth (ft)	$\gamma'(\frac{t}{ft^3})$	$P'(\frac{t}{ft^2})$	C_n	N	$N_1 = C_n.N$
1	0.0531	0.0531	1.984	8	15.87
7.667	0.0531	0.4600	1.262	11	15.88
14.333	0.0531	1.220	0.935	14	13.09
21	0.0468	2.203	0.738	6	4.43
27.667	0.0655	4.017	0.537	14	7.52
34.333	0.0655	6.267	0.388	22	8.54
41	0.0655	8.954	0.269	29	7.80

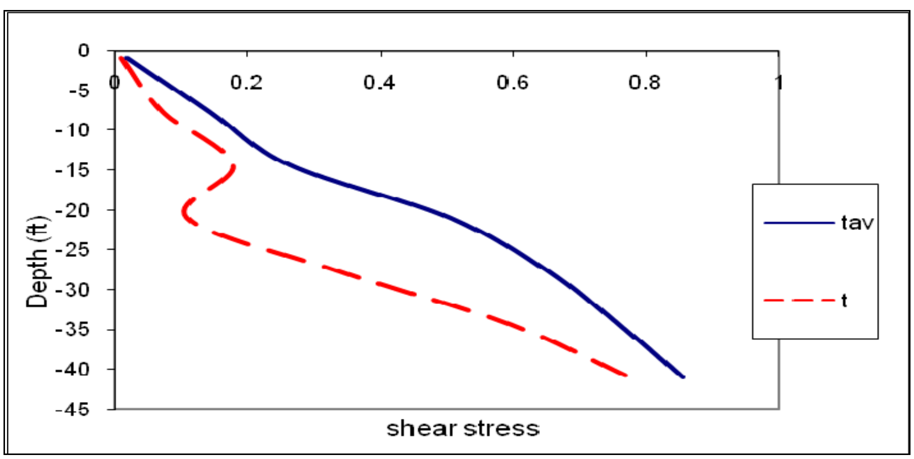

Figure 2: comparing the liquefaction producing stresses τ and produced stresses by earthquake τ_{av} before pile driving.

After driving 60 piles and repeating the standard penetration test in 1, 3, and 6 meter depths, it was observed that the ground has been compacted and the standard penetration number has been increased. The results of which are shown in table 6.

Table 6: The results of standard penetration test after driving 60 piles.

Point Depth (ft)	1	2	3	4	5	6	7
3.28	40	32	53	40	44	32	52
9.85	28	27	28	-	-	-	-
19.70	26	23	26	-	-	-	

After modifying the standard penetration number for the point 2, which includes the lowest amounts according to the results of the table 7, it is observed that the standard penetration number has been increased to more than 20 and the danger of liquefaction has been vanished.

Table 7 : standard penetration number after modified.

Depth (ft)	$\gamma'(\frac{t}{ft^3})$	$p'(\frac{t}{ft^2})$	C_n	N	$N_1 = C_n.N$
28.3	053055.0	17.0	59.1	32	77.50
84.9	053055.0	70.0	12.1	27	32.30
70.19	059297.0	86.1	90.0	23	70.20

Liquefaction producing shear stress and the amount of the produced stress of the earthquake after modifying has been shown , Fig. 3. It deserve to notice that, liquefaction producing shears stress is more than the shear produced by the earthquake, and the danger of liquefaction has been disappeared after pile driving, especially in depth of 6 meter which the difference of the curves has vanished totally.

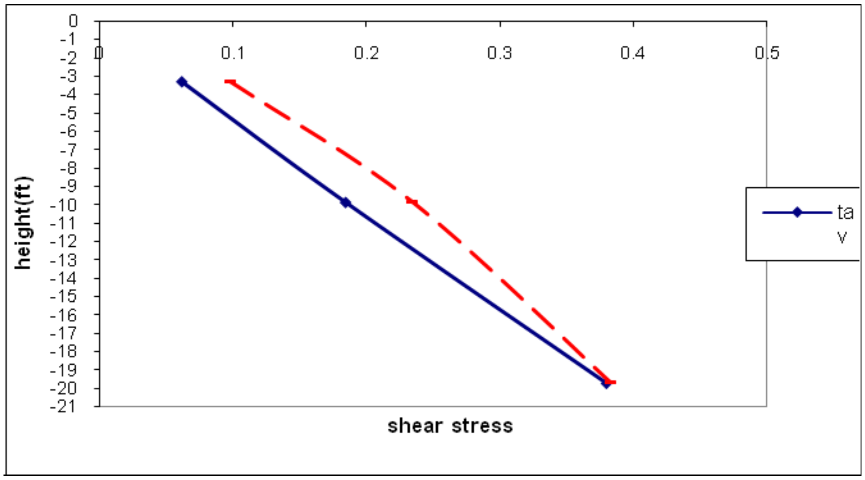

Figure 3 : comparing the liquefaction producing stresses τ and produced stresses by earthquake τ_{av} after pile driving.

Soil allowable strength

Soil allowable strength is the maximum pressure, which a hill can undergo without reaching the braking stage or passing the rapture degree from the allowed level.
Considering the Meyehof equation, Terzaghi and peck diagram, soil strength could be calculated according to the results of the standard penetration test by following eqn (12):

$$q_a = 7.99N(\frac{3.28b + D}{3.28B})^2 * 0.01 \tag{12}$$

In Eq (12), q_a is the Soil allowable strength, N the standard penetration number, B the width and D is depth of foundation.
In the equation above, obtained allowable strength has been calculated according to the maximum amount of allowable subsidence and disposition of the foundation's bed on the untamed, natural, or original ground. Considering the said equation and the results of the standard penetration test before pile driving, the allowed strength of the soil will be according to the table 8 by the depth and width of the different raft foundations. It deserves to mention that since according to table 3, the foundation of the structure has been placed in the depth of range (A) then penetration number of 11 has been selected.

Table 8 : Soil allowable strength using standard penetration number and Meyehof equation before pile driving.

Depth (ft)	Width (ft)	N	$q_a \frac{lb}{in^2}$
7.22	49.21	11	13.52
7.22	59.06	11	13.38
7.22	68.90	11	13.24
7.22	78.74	11	13.10
7.22	88.58	11	12.96
7.22	98.43	11	12.96
7.22	114.83	11	12.82
7.22	131.23	11	12.82

As it is observed, the allowed strength of the soil for different dimensions of the foundation is less than 1 and constructing a huge and tall building on such a ground is dangerous. After driving the 60 piles and repeating the standard penetration test whose results are offered in table 9, it is observed that soil's mechanical specification has been improved and soil's permitted strength has increased to around 200 percent.

Table 9 : Soil allowable strength using standard penetration number and Meyehof equation after pile driving.

Depth (ft)	Width (ft)	N	$q_a (\frac{lb}{in^2})$
7.22	49.21	23	28.17
7.22	59.06	23	27.84
7.22	68.90	23	27.56
7.22	78.74	23	27.35
7.22	88.58	23	27.18
7.22	98.43	23	27.06
7.22	114.83	23	26.87
7.22	131.23	23	26.76

Conclusion

According to the tests, it is observed that driving metal piles in sandy grounds especially coast grounds, which are completely saturated, reduces the liquefaction, increases the soil bearing capacity and transfers the forces to the earth's different layers. It also, reduces the costs and duration in comparison to the cast-in place piles, which only reduces the liquefaction danger and transfers the forces to the different layers of the earth. The needed time for driving a 12-meter metal pile with a k25 hammer is 5 hours. While, the needed time for a concrete cast-in place piles is about three days. Therefore, metal pile driving is suggested as the better method in coastlands in comparison with cast-in place pile driving.

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