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USING MICRO PILES FOR LIQUEFACTION MITIGATION IN LOW COMPACTED SANDY SOILS FOR ONSHORE STRUCTURES , CASE STUDY: KOOHESTAK FISHERY HARBOR

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Key words: Liquefaction, Micro Piles, Low Compacted Sandy Soils, Onshore Structures.

Abstract

Micro-piles are principally used as elements for foundation support to resist static and dynamic loading conditions, less frequently as in-situ reinforcements for slope and excavation stability and construction of onshore coastal structures in low consolidated sandy beds. The study was performed on Koohestak fishery harbour (Fisheries Organization, IFO), Hormozgan Province, IRAN. In this paper, using perforated micro-piles instead of steel piles were studied**.** Case study is located at 200 km east Bandar Abbas and 45 km south coast Minab on the northern shore of Persian Gulf with geographical coordinates equal to 260 8` N -570 1`E. According to the geotechnical tests and analysis, the basement has a significant liquefaction potential which could cause serious settlements in the foundation of the wharf. The design of the micro piles has been done in three steps. First, structural design, the bearing capacity of the elements was determined. For the next step, geotechnical design, the bond strength of the micro pile was calculated. Finally, the punching shear control was performed. Furthermore, based on the field and laboratory tests, especially SPT test, the rectangular and triangular shapes were considered for the micro piles arrangement under the basement. As a result of the design procedure, 180 micro-piles with 12 meter initial length were designed to be installed in the basement of this project instead of using steel piles. Using this method has several advantages. As construction advantages, the installation procedure causes minimal vibration and noise. Also, the construction time will decrease. From geotechnical point of view, using micro-piles in this project will lead to about 30% decrease in liquefaction in comparison with steel bars that have no effect on the soil properties. In addition, from economical point of view, about 15% will be saved by using micro piles instead of steel bars in this project.

Introduction

Safe design of foundations for on-shore structures needs an understanding of soil-structure interaction effects under dynamic loads. The stability and integrity of structures will be at risk if the soil fails due to liquefaction as a result of the shaking of the soil. This kind of failure also can be catastrophic, as observed in the earthquakes in Japan and in Turkey.

Evaluation of liquefaction potential of soils at any site requires a combination and interaction of two sets of parameters; cyclic loads (seismic motion or wave action) and soil properties (such as grain characteristics, relative density, soil structure, fabric, stress history and strain

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history). Liquefaction-induced damage to marine structures has been documented quite extensively in the literature. The procedure for predicting liquefaction resistance of soils was originally developed by Seed and Idriss (1971) using the Standard Penetration Test (SPT) blow counts correlated with a parameter representing the seismic loading on the soil, called cyclic stress ratio. Their endeavors were lead to several revisions and updates and developed a couple of papers between 1971 up to 1982. Additional works were carried out by Tokimatsu and Yoshimi (1983). The results gained by previous researchers lead to introduce the liquefaction criteria which is published by Seed et al (1985). General reviews of the simplified procedure are contained in a report by the National Research Council (1985) and a summary report from the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) workshops on evaluation of liquefaction resistance of soils by Youd et al. (2001).[7],[9],[10].[12]
A port is a dynamic system, growing and changing as the purpose it was constructed for changes. Piled foundations with large diameters are usually used for platforms and wharf's Foundations and onshore structures where normally soft deposits are located. If liquefaction potential exceeds, it would unsustainably damage the wharf. Considering the fact that the construction of a wharf is an expensive task and takes a lot of time, some approaches should be applied to reduce the liquefaction potential of low-density sandy soils. To cope with this problem, Using pile and micro-pile were studied and results compared. [14], [15]

Scope of work

The investigations include the pile and micro-piles calculated ,in the basic design of the steel pile ,the favorable concept for steel pile beside Codes and standards was conform to API RP2A-WSD93. To calculate the required penetration of the piles, maximum axial compressive and tensional forces have to be obtained from serviceability limit state and allowable bearing capacity of steel pile. According to the geotechnical reports of KOOHESTAK project, a total number of 7 boreholes are drilled in the site area among them one deep boreholes which are located near the wharf axis mainly give information about deep formations which will provide required capacity for pile foundations. The evaluation of the bearing capacity of the piles is done based on the geotechnical data extracted from borehole No 7(BH7). We gave the required knowledge on the geotechnical condition at the lower levels, then assessment of the probability of liquefaction occurrence & compare liquefaction in every case, to this aim, Liquefy Pro software was used to simulate and analysis the soil behavior against liquefaction phenomenon. . [2], [3]
In this study, at first the potential of liquefaction for a case study (an onshore coastal structure constructed on low consolidated sand beds) is evaluated and then using micro-piles instead of steel piles in order to mitigate liquefaction has been studied. Finally, the efficiency of pile group and micro-pile group evaluated and results compared geotechnicaly, structurally and economically.

Methods Based on SPT results for liquefaction Analysis

The main intent of empirical methods is to acquire an indication of the relative density of the soil in order to evaluate liquefaction potential. It seems the SPT is a suitable way of assessing the resistance to liquefaction since the test is related to several factors that also affects the liquefaction resistance (for instance: ratio of horizontal to vertical stress). Another important factor affects on liquefaction is the intensity of cyclic load which is evaluated from the earthquake magnitude and the distance to the epicenter of the earthquake. [13]
In the following this empirical methods of assessing the potential of liquefaction on a site will be described. The data from which these methods are developed correspond to level ground.

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Method presented by Seed et al. (1985)

The potential of liquefaction is assessed with the aid of liquefaction charts. In Fig.1, each point corresponds to site specific conditions obtained during some particular earthquake. The resistance of the soil is represented on the horizontal axis by the (N1)60 values, which is the blow count value in the SPT-test corrected for the overburden pressure and the energy delivered to the drill rod. On the vertical axis, the intensity of ground motion is represented by the average cyclic stress ratio, tav/sigma'0, tav is the average shear stress mobilized by earthquake and sigma'0 is the initial vertical effective stress. Tav/sigma'0 is calculated from the peak surface acceleration. [12]

In this method firstly SPT results should be modified and according to the average fines content in percent by mass (FC) and earthquake magnitude, the alternative shear strength ratio (R) is calculated and then this number is compared with average shear strength. If this ratio is less than 1, the existing soil is prone to liquefied.[12]

Simplified Base Curve Recommended for Determination of CRR from SPT Data for M=7.5

Method developed by NCEER-1997

To evaluate the potential of liquefaction in prone regions, an instruction was published by the National Center for Earthquake Engineering Research (NCEER-1997). This method of prediction consists of relevant procedures with respect to soil type are applied to determine liquefaction potential of the soil.[11]

According to NCEER committee recommendations, Two variables are required for the assessment of the liquefaction potential in sandy soils:(1) the seismic demand on a soil layer, ex pressed in terms of cyclic stress ratio(CSR); and(2) the capacity of the soil to resist liquefaction, expressed in terms of the cyclic resistance ratio (CRR).[11]

Cyclic stress Ratio

The CSR is defined as the ratio of the maximum cyclically applied shear stress to the effective normal stress acting at the beginning of shaking on the plane where shear stress is applied. The cyclic (or Dynamic) strength is defined as the CSR value at N=10or20 cycles.

The cyclic stress ratio, CSR, at a particular depth in a level soil deposit can be calculated from [11]:

$$CSR = \left(\frac{\tau_{ave}}{\sigma'_{ave}}\right) = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{v0}}{\sigma'_{v0}}\right) r_d \qquad (1)$$

Where;

amax= maximum acceleration at the ground surface

σ_0= total overburden stress at the depth in question

σ'_0= effective overburden stress at the same level

r_d=stress reduction factor decreasing with depth

Stress reduction coefficient for routine practice and noncritical projects, Youd et al. (2001) suggest the following equations be used to estimate average values of rd (Liao and Whitman 1986): [8], [17]

$$r_d = 1 - 0.00765 * z \quad \text{for } z \leq 9.15 \text{ m} \qquad (2)$$

$$r_d = 1.174 - 0.0267 * z \quad \text{for } 9.15 \text{ m} < z \leq 23\ m$$

Where; z is the depth below the ground surface in meters.

The parameters affecting the liquefaction potential of loose, saturated granular soils that have been investigated in a detailed manner and can be summarized as: relative density or void ratio; confining pressure; fines content; grain characteristics; plasticity of fines; method of

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sample preparation and the degree of saturation. Other factors include prior seismic straining, the coefficient of earth pressure at rest, K0, the overconsolidation ratio of the soil deposit, and increased time under pressure.

The best condition to determine this parameter is to obtain undisturbed soil samples and implement cyclic shear tests. Due to the difficulties in obtaining satisfactory undisturbed samples from granular soils, use of in-situ tests (such as SPT and CPT tests and etc) for liquefaction studies are now encouraged.

In this research, NCEER method is used. In order to use calculation graphs presented by NCEER (1997), we need to modify the SPT data given from site. The corrected SPT values called (N1)60 and it directly related to the fine materials existing in sandy soils. To consider this concept, the equivalent corrected SPT value is defined as follows [11]:

$$(N_1)_{60cs} = K_s (N_1)_{60} \qquad (3)$$

$$K_s = 1 + \left[\left(\frac{0.75}{30}\right)(FC - 5)\right] \qquad (4)$$

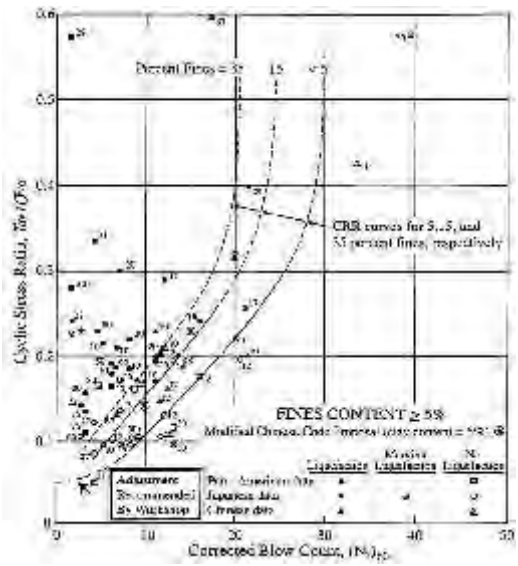

Fig1.Simplified Base Curve Recommended for Determination of CRR from SPT Data for M=7.5 (After Youd and Idriss, 1997)

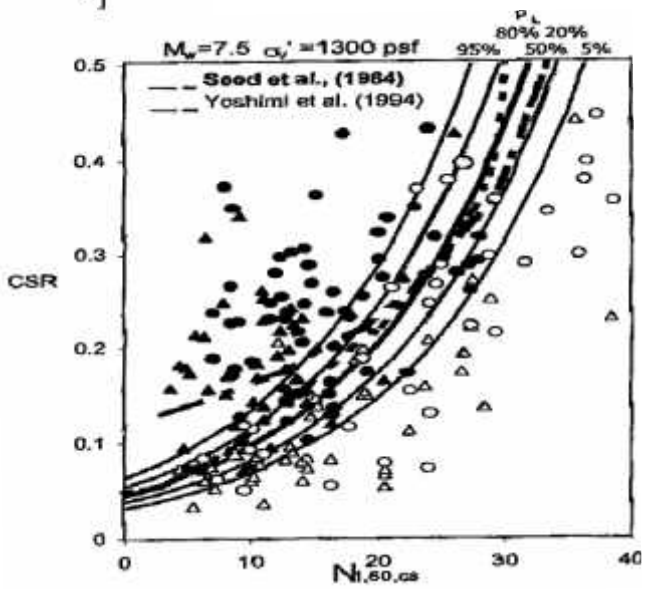

Fig.2.SPT-Based liquefaction Correlation (for M=7.5), Proposed by Seed et al. (1984)

Factor of Safety

Considering the potential for liquefaction is in terms of a factor of safety described as follow:

$$FS = \frac{CRR}{CSR} \qquad (5)$$

A factor of safety smaller than one at any depth indicates liquefaction susceptibility at that depth. However, to assess the effect of liquefaction on the ground surface, the variation of the factor of safety with depth needs to be evaluated to determine the possible impact of liquefaction for the engineering structures on the ground surface.

As compared to Seed method, BSSC (2000) recommends a FS value of 1.2 to 1.5 is suitable when implementing the Seed simplified procedure in engineering design. The acceptable value of FS for a particular site will depend on several factors, including the type and importance of structure and the potential for ground deformation. [16]

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KOOHESTAK Fishery Harbour Project

The study was performed on the koohestak fishery harbour Which is located in IRAN at 200 km east Bandar Abbas & south coast Minab in 45 Km south Minab on the northern shore of the Persian gulf with geographical coordinates equal to 260 8` N -570 1`E. (The vicinity of the road of Minab to koohestak). [2]

To reduction the effect of wave reflection, considered to design of open type wharf (pile and deck structures). This wharf was 52 m long and 12 m wide, the aim of the project to constructed a wharf and access footing concrete pier to the open sea and a breakwater to protect the quay walls of the ports and fishing.

Materials

In order to perform geotechnical investigations, a total number of 7 boreholes are drilled. To evaluate liquefaction potential, only BH7 results are presented. According to the results gained from this borehole, the soil layers from top to bottom mainly consist of SP and SM-SC respectively; as it can be seen from Fig3 the soil profile and main soil properties of each soil layer and the gradation curves (ASTM D422) are presented in Fig3 and fig4 respectively.

Liquefaction Evaluation

In order to evaluate the liquefaction potential seed method and NCEER method are used. The soil properties required for such SPT based analyses (for example at depth 9.0m) is shown in table1.It should be note that with considering the change of ground water level in different seasons, the GWL is assumed at surface. [2]

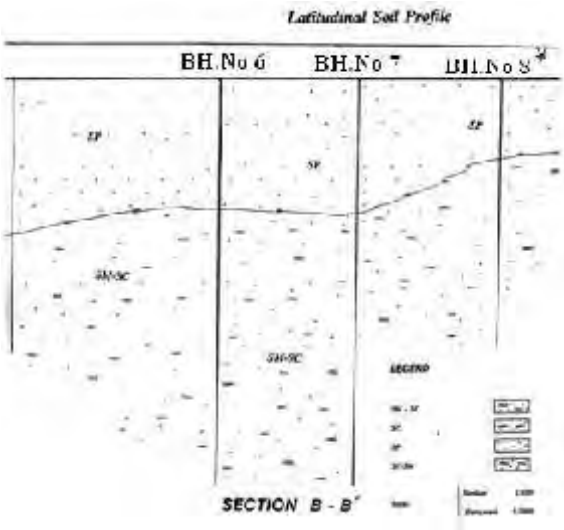

Fig3.Latitudinal soil Profile

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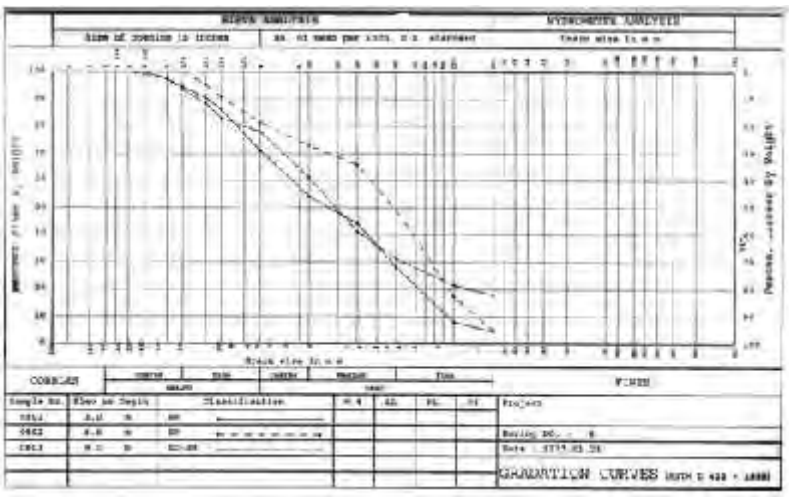

Fig4. Gradation Curves for all type of soil in this project

Table1)Soil properties at liquefaction depth

USCS	$\gamma_{sat}\left(\frac{gr}{cm^3}\right)$	Dr	C(kPa)	Φ	ν	E (kPa)	SPT No.	Fine Content
SM-SC	**2.03**	**46%**	**20**	**30**	**0.3**		**18**	**4%**

The low compacted sandy soils located behind the wharf. The liquefaction potential accrues for earthquake properties: acceleration =0.3g & Magnitude = 7 in the low consolidated sand soil. According to SPT test, Liquefaction was seen to depth 12 -15m. The results of all mentioned methods are summarized in table2.

Table.2) Liquefaction Potential Evaluation based on NCEER Method

B.H	Depth (m)	USCS	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	CRR	CSR	S.F
7	3	SP	606	309	115.8	0.98	17	17	0.375	0.225	0.6
7	5.5	SP	1111	561	210.2	0.97	18	18	0.374	0.238	0.636
7	9	SP	1819	928.5	330	0.93	19	19	0.355	0.262	0.738
7	12	SM-MC	2430	1230	412.2	0.87	18	21	0.335	0.287	0.857
7	15	SM-MC	3048	1548	463.6	0.78	19	21	0.299	0.287	0.96
7	18	SM-MC	3666.6	1866	493.3	0.69	20	23	0.264	0.313	1.18
7	20	SM-MC	4078	2078	493.1	0.62	21	25	0.237	0.338	1.14
7	23	SM-MC	4696	2396	521.9	0.57	11	16	0.218	0.22	1.01
7	26	SM-MC	5314	2714	538.8	0.52	24	29	0.198	0.525	2.65
7	30	SM-MC	6138	3138	574.5	0.48	24	27	0.183	0.387	2.11

Fig5 shows the variation of CSR and CRR with depth for this project. As it can be seen from this graph, in zones which is prone to liquefy, the CRR is less than CSR.

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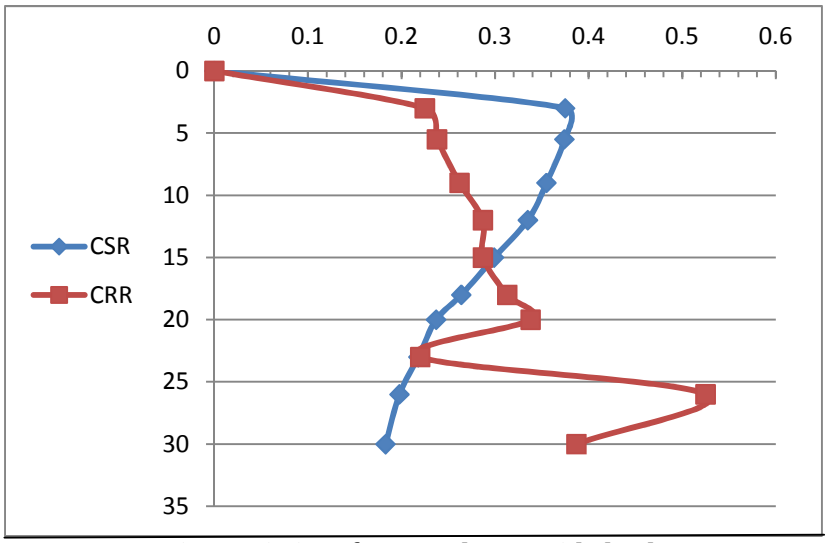

Fig5.Variation of CSR and CRR with depth

If liquefaction potential exceeds the considered and scheduled forces of the structure and piles, it would unsustainably damage the wharf. Considering the fact that the construction of a wharf is an expensive task and takes a lot of time, some approaches should be applied for reduce the liquefaction potential of low-density sandy soils. To do this stabilization and improvement is used methods micro-pile with grout injection.

It should be note to say that, large proportion of exerted load in pile is directly beard by reinforce concrete in spite of that, in micro pile, steel casing has the most important rule in load transition. According to the low diameter of micro pile, the tip resistance is neglected and it is assumed that entire load is transferred by means of skin friction. Final capacity of such this mixed soil is really depend on these factors: soil type, grouting method, grouting pressure and grouting discharge.

Loading Conditions

The main loads applied to the structure described as follow:

Dead Load

This dead load comprises the weight of the different types of structures material and those equipments and structural elements constantly impose their weight onto the structure.

Live Load

To consider the loads imposed by cargo (mainly container stacks), a uniform surcharge equal to 40 kN/m2 was applied to the whole area of the deck.

Seismic Load

Chinese code considers the seismic activity earthquakes for design construction. Load case Ex and Ey defines seismic load in transverse direction with 5% eccentricity.

Thermal Load

Changes in overall temperature of the structures are usually accommodated by allowing expansion and contraction of the deck. The resulting longitudinal movement, assumed to take place to both sides of the midpoint or of a strong point.

Two different thermal loading cases have been applied. The first case is the overall

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change of the structure temperature which is the difference between the concrete temperature when poured and maximum temperature experienced during the lifetime of the structure. For this case we assume the extreme temperatures of the structure to be 15ºC to 45 ºC and the ambient temperature when concrete is poured to be about 30 ºC. The difference will be 15 ºC. The second case is the thermal gradient between upper parts of the structure exposed to sunlight and the sheltered parts. We take this difference to be about 10 ºC.

Soil Pressure

Lateral soil pressure at any level is calculated by factoring the overburden pressure at that level by lateral soil pressure coefficient relevant to the soil stratum. For the purpose of calculating soil pressures, surcharge (live loading) on the surface should be determined, extreme water levels should be derived, pore – water pressures should be determined with reference to tidal range, soil permeability and drainage provisions. Also allowance should be made foe reduced passive resistance due to over dredging or scour.

When no movement of the wall relative to the ground takes place, the earth pressure shall be calculated from the state of stress at rest. The stress history of the soil is of great importance in determination of stress state at rest.

For the horizontal ground surface, the coefficient of earth pressure at rest Ko, may be determined by:

$K0 = (1 - \sin\Phi^{/}).\ (OCR)\ 0.5$ (6)

Japanese code (OCDI) [1] gives closed from solution to active and passive pressure determination based on Coulomb's work. These solutions are recommended for ordinary soil i.e. not liquefied, not improved and not reinforced. In seismic conditions, dynamic soil pressure has to be calculated.

Berthing Action

The berthing force is exerted as the vessel momentum diminishes to zero as it impacts against the structures. The force coming from berthing of a vessel is only considered in the structural design of open type wharf (pile and deck structures). Based on BS6349 the amount of energy to be absorbed by fender was calculated.

Mooring Effect

The external forces generated by the motions of moored vessel should be calculated by carrying out a numerical simulation of vessel motions, with the wave force acting on the vessel, the wind force, the current force due to water currents, being set appropriately.

The tractive force acting on a bollard has been determined based on the wind pressure acting on a vessel in such a way that lightly loaded vessel should be able to moor safety even when the wind velocity is 25~35 m/s. The mooring force is exerted on the bollard in 5 directions horizontally, covering an arc of 180 degrees. In each case, it is assumed that are tractive forces stipulated in table 2 acts horizontally and vertical force.

Temp

Effect of overall change in temperature was defined in this load case. Regarding to design criteria value of overall change in temperature is 15ºC for concrete deck and steel piles.

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Pile Design

STATIC ANALYSIS

In the static analysis, loads applied to the structure were constant in time. Dead loads, uniform live loads, thermal loads were static loads. Although some loads like wind and earthquake were dynamic in nature, we treat them in pseudo-static manner.

Moving loads, berthing and mooring actions were dynamic but these were also applied to the structure as static. These loads have unspecified point of effect so the loads had to be applied at different points to find the extreme effect on the structure. Duo to the very large stiffness matrix, influence line analysis was very time consuming and effectively impossible. Therefore maximum and minimum internal forces were calculated by manually moving the point of effect of loads on the quay Wall. Berthing and mooring effects were also applied simultaneously to each fender and bollard to find the worst effect on the structure. The result would be combined at the end.

MODELLING OF STRUCTRE

As it mentioned, in basic design report quay structure was modeled in "sap 2000" steel piles and beams were modeled by frame element and deck is modeled by orthotropic shell element.

DESIGN VERIFICATIONS

-Pile Axial Bearing Capacity Calculation

For a pile which transfers axial force to the soil, no matter whether it was installed "driven" or "bored", the ultimate axial bearing capacity was a combination of skin friction and end – bearing mechanisms.

For full-displacement piles it was common practice to calculate the ultimate bearing capacity. (API RP2A-WSD 93) [2], BS 8110-1:1997& BS 6349 [3]

-Pile Driving Analysis

Analysis of a pile under the action of an impact hammer was based on an energy balance. It was stated in the way that the energy delivered by the hammer was lost in the driving system.

DESIGN OF THE MAIN ELEMENTS

In order to estimate displacements in both elements, SAFE program was used. It should be note to say that modeling procedure for pile and micro pile is the same. The differences are just in geometry and material properties. In this research displacements of both structure was calculated and compared.

Steel piles were designed at serviceability limit state. Concrete parts were checked at ultimate limit state.

-Pile Penetration Length

To calculate the required penetration of the piles, maximum axial compressive and tensional forces had to be obtained from serviceability limit state.

- Steel Piles Design

Steel piles were designed at serviceability limit state. AISC code [4] was used for the design.

Corrosion effect must be considered in the design of steel pipes. As a common procedure it is assumed that corrosion takes place with a rate of 0.1 mm per year. Thus regarded a service life of 60 years, a total reduction of 6 mm of piles thickness would be allowed.

Sap2000 software would be calculated stress ratio for steel pipe. In each load combination piles must be checked for axial force and bending moment. Pile sections must be designed for shear force.

In this case, the earthquake load was applied to the structure. Thus we increase allowable stresses by 1.33 its value.

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MICROPILE DESIGN

Design of steel micro-piles on the koohestak fishery harbor project has been described as follow. The micro-piles piles design, except as Codes and standards was conform to FHWA-SA-97-070 [5]:

In the first step, structural design, the bearing capacity of the elements had been determined. For the next step, geotechnical design, the frictional strength of the micro piles' casing with the lateral soil was calculated. Finally, the punching shear control had been performed.

Micro pile specifications

For each micro pile, length and diameter is considered to be 12 meter and 200 millimeter respectively. Each micro pile is covered with a casing with ST37 steel and external diameter of 76 millimeter. Micro pile is reinforced with Steel Bars with 32 mm diameter, Grade III, in accordance with DIN 1045 Specification with a minimum yield stress of 4000 kg/cm² or APPROVED equivalent.

MICROPILE STRUCTURAL DESIGN

In some cases, a micro pile will be subject to both axial compression load and bending stresses. A combined stress evaluation is conducted, based on AASHTO (2002).

The allowable tension load and compression load for the length of a micro-pile is given as:

(7)

$$P_{t-all} = 0.55(F_{y-bar}A_{bar} + F_{Y-casing}A_{casing})$$

$$P_{C-all} = 0.4 f'_{C-grout}A_{grout} + 0.47\left(F_{y-bar}A_{bar} + F_{Y-casing}A_{casing}\right)$$

Where, $f'_{c-grout}$ = *unaxial compressive strength of grout*

f_{y-bar} = *minimum steel yield stress*

$f_{y-casing}$ = *minimum yield stress of casing*

A_{bar} = *cross sectional area of bar*

A_{casing} = *cross sectional area of casing*

Final results can be summarized as follow:

In a safe side conservative design:

P_{c-all}=320 kN

P_{t-all}=305kN

MICROPILE GEOTECHNICAL DESIGN

Micro pile Axial Capacity

According to the geotechnical point of view, Bond length design is most usually coupled with assumptions that load is transferred by skin friction only and this distribution is considered to be uniform, which is not really true, but, it is conservative.

According to FHWA recommendation, the nominal factor of safety on the ultimate bond value varies typically from 2 to 2.5 (2.5 is used in this project).

Soil category (according to soil test result):

Soil description: Sand (with some silt)

According to technical specification and considering silt and clay, for primary grout placed under gravity head (Type D) and ground bond nominal selected based on FHWA and considered in a safe side conservative design which can be seen as follow:

(8) $$P_{G-allowable} = \frac{\sim_{\text{bond nominal strength}}}{F.S.} * 3.14 * \text{DIA}_{\text{bond}} * \text{bond } length$$

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Soil description: Silt & Clay (some sand)

$\alpha_{\text{Bond nominal strength}}$ = 85 kN/m2

F.S = 2.5

DIA $_{bond}$ = 0.20 m

$$P^{10}_{G--\text{allowable}} = \frac{85}{2.5} * 3.14 * \text{ DIA}_{\text{bond}} * \text{ bond length} = 213.5\,\text{kN}$$

Final results can be summarized as follow:

In a safe side conservative design:

P $_{c\text{-design}}$=200.00 ^{kN}

P $_{t\text{-design}}$=100.00 ^{kN}

MICROPILE LATERAL CAPACITY

The behavior of a laterally loaded micro pile depends on the properties of the micro-pile such as diameter, depth, bending stiffness, fixity condition of the pile in the footing, and on the properties of the surrounding soils. The lateral stiffness and capacity of micro-pile is limited due to smaller diameter.

According to (FHWA-SA-97-010), (FHWA-IP-84-11), NAVFAC [5] procedure is described as below:

Lateral load required to produce 6.33 mm, lateral displacement in a soil by NAVFAC is:

(9)
$$P = \left[\frac{¢_P}{F_¢}\right] \times \frac{EI}{\left[\left(\frac{EI}{f}\right)^{1/5}\right]^3}$$

According to AISC [4] and NAVFAC [5], and soil characteristics, f = 2,000 KN/m3

(10)
$$T = \left(\frac{EI}{f}\right)^{1/5} = \left(\frac{3{,}006{,}932}{2000}\right)^{1/5} = 4.31^m$$

$$P = \left[\frac{¢_P}{F_¢}\right] \times \frac{EI}{\left[\left(\frac{EI}{f}\right)^{1/5}\right]^3}$$

P = 91.72 kN

In a safe side conservative design:

P = 20.00 KN : Micro-pile lateral capacity

Modeling of micropile

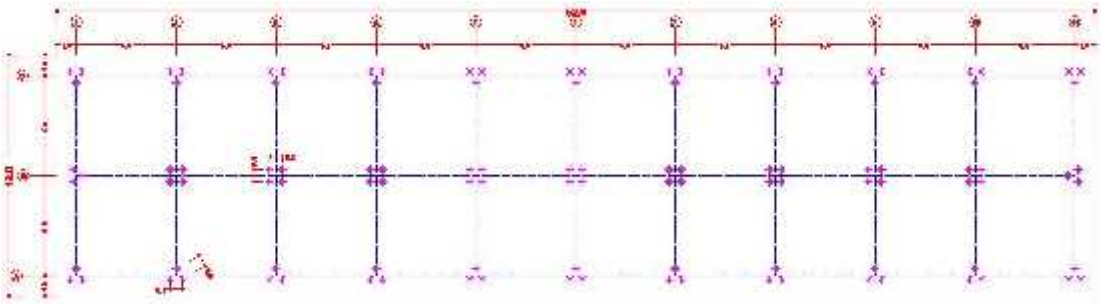

Fig 10.Micro piling Arrangement plan

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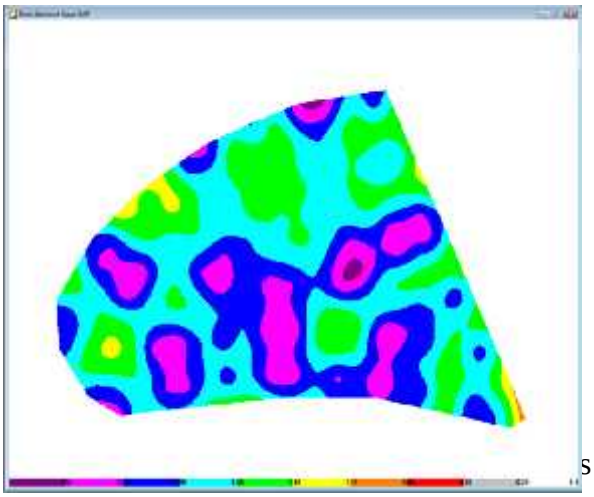

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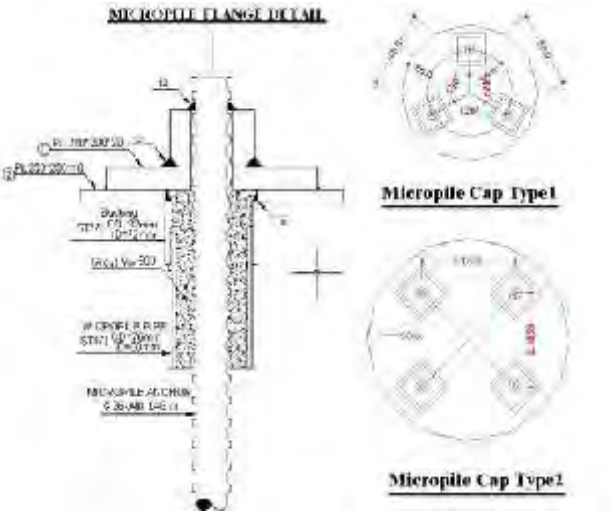

Comparison of displacements in pile and micro pile

In order to compare displacements of pile and micro piles, both of them are modeled with SAFE program. Spring elements are also used to simulate the interaction between the soil and structure. To estimate the stiffness of springs, a full scale test was performed. Based on full scale test results, micro piles stiffness in considered to be 100(ton/cm). Displacements diagram for both elements are shown before, as it can be seen , the maximum displacements in micro pile is 2.8 mm in comparison with pile which number reaches to about 5 mm.
According to the dimensions of pile and micro pile, the bearing capacity of pile is far greater than micro pile, so, the number of micro piles should be increased. The increase of micro piles number could lead to the reduction of displacements and stresses in different points of the foundation, and also the thickness of foundation decreases.

Liquefaction analysis

To observe the effect of using micro-piles against liquefaction phenomenon Liquefy Pro software (Civil Tech Company) has been used. The material used for bolts and foundation bars As compared to soil liquefaction methods in improved methods by using micro piles was considered as follow :

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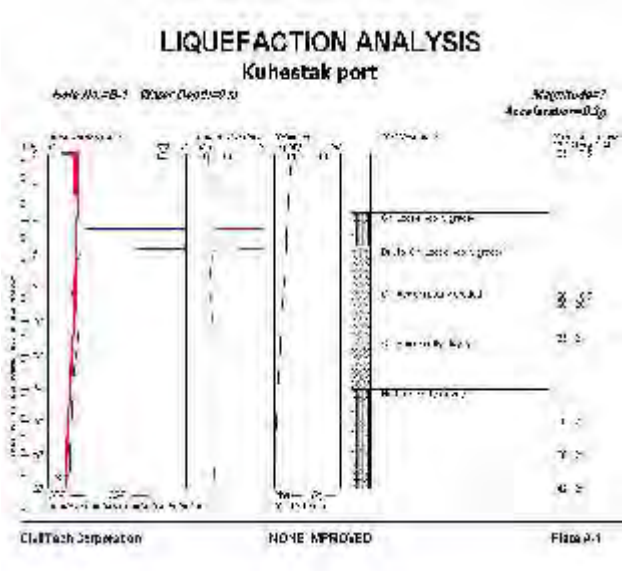

Fig13.Liquefaction analysis (None improved)

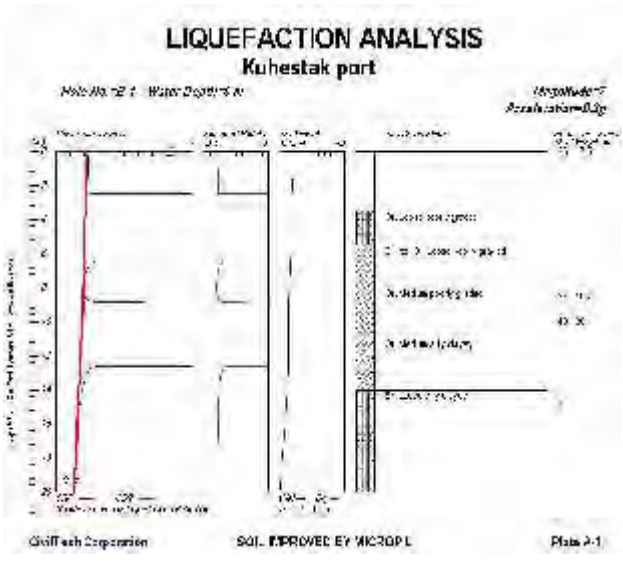

Fig14.Liquefaction analysis (Improved by micropile)

Conclusions

This paper provides a brief overview of the usage of piles and micro-piles in order to reduce the liquefaction effects. Geotechnical studies and the analysis of liquefaction have shown the liquefaction potential for Koohestak Project. To reduce the liquefaction potential of low-density sandy soils stabilization methods was performed by means of micro-pile construction and grout injection. According to the results which is gained from the case study, final capacity of pile is more greater than micropile ,because of the large diameter of pile, and we have to use a group of micropiles instead of single micropile in order to increase the final bearing capacity, this is beneficial because a group of micropiles can reduce the settlement of structure, in comparison with pile.

As construction advantages, the installation procedure causes minimal vibration and noise. Also the construction time will decrease. From geotechnical view, using micro piles in this project will lead to about 30 % decrease to liquefaction and about 50 % decrease in vertical displacements, in comparison with steel bars. In addition, based on Iranian instructions for estimating project costs and from economical view, about 15 % saving and other advantages of the plan during promotion will occur by using micro piles instead of steel bars in this project.

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