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MITIGATION OF EARTHQUAKE INDUCED LIQUEFACTION HAZARDS, A CASE STUDY: BANDAR ABBAS DRY DOCK PROJECT

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ABSTRACT

The effectiveness of the dynamic compaction method as a mitigation measure against the risk of liquefaction has been assessed in a case study involving the foundation of two 300,000 ton dry docks located in port of Bandar Abbas in Iran. Subsurface investigations revealed that there is a high liquefaction potential in a layer beneath the major industrial facilities. A comprehensive Probabilistic Seismic Hazard Assessment (PSHA) has been performed to evaluate the local Peak Ground Acceleration (PGA) based on various risk levels. Site investigation and liquefaction assessment confirmed that ground improvement was required as part of the hazards mitigation. Among several techniques, dynamic compaction method has been selected based on considerations of ground conditions, operational impacts, degree of improvement required, and environmental aspects. Liquefaction susceptibility and the efficiency of dynamic compaction method have been evaluated by in-situ Standard Penetration Test (SPT) data obtained before and after the compaction. The results demonstrate that dynamic compaction as a liquefaction remedial measure decreases the liquefaction potential to an acceptable level and so its efficiency was approved.

Keywords: Liquefaction, seismic hazard assessment, dynamic compaction, SPT, ground improvement, mitigation measure

INTRODUCTION

The behavior of cohesionless granular media under earthquake loading is one of the most important problems facing geotechnical engineers. Loose sands tend to reduce its volume when subjected to seismic ground motions. If the sand is saturated and drainage is prevented, or if the vibrations are rapid enough that drainage is unable to occur, this tendency for volume decrease results in the increase of pore pressure and may cause liquefaction (Martin et al. 1975). This phenomenon has occurred in almost all large earthquakes. The destructive effects of liquefaction were fully realized in the aftermath of the 1964 Anchorage, Alaska and Niigata and more recently 1999 Chi-Chi earthquake in Taiwan. These destructive effects have yielded to develop various techniques capable of providing the required improvement to prevent earthquake induced liquefaction. A number of ground improvement techniques have been considered including dynamic compaction, explosion compaction, and vibro replacement. Applying dynamic compaction to saturated sand induces a controlled liquefaction, allowing the particles to rearrange to a denser packing concurrent with the dissipation of the excess pore water pressures (Mitchell 1981).

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Two dry docks are under construction on the northern banks of the Persian Gulf, near the Iranian port of Bandar-Abbas, featuring the installation of more than 6500 large diameter steel pipe metal piles and 1000 bored piles. The dry docks have dimensions of 470 m x 80 m and 370 m x 80 m, respectively and are designed to repair and renovate VLCC vessels with capacities of up to 370,000 tons. The docks draft is designed to be 9 m in low tide condition; as the result of which the dock depth will be 16 meters below the original ground level. The plan and side view of the project are depicted in Figure 1. Alongside, there are a series of industrial workshops designed for cutting, welding, and painting the components of VLCC vessels.

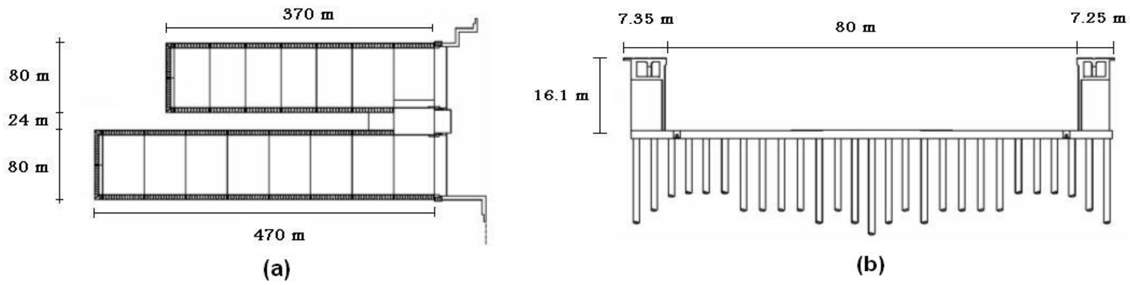

Figure 1. The dry docks, a) plan , b) side view

A series of earthquakes with moderate to strong magnitudes have recently struck the area, the most destructive one hitting the island of Qeshm (around 3 kilometers south of the project site) with a magnitude of M_b=6.0 (IIEES report, 2005). These events caused serious concern of liquefaction occurrence due to the underlying soil layers nature. To evaluate the liquefaction potential, there must be a rational earthquake scenario present for the project site. Since no extensive seismic survey was available for this area, a seismic hazard analysis has been performed using two different probabilistic approaches. The liquefaction potential assessment is performed subsequently using the code presented in the Technical Standards and Commentaries for Port and Harbor Facilities in Japan (The Overseas Coastal Area Development Institute of Japan, 2002). This is referred to as *the Japanese code* here after.

A rather extensive geotechnical subsurface investigation program was followed to provide the characteristics of the underlying soil layers. Boreholes were extended to 65 m deep to reach a dense enough layer that could be properly assumed as bedrock.

Figure 2 illustrates the soil layers characteristics derived from either field or laboratory experimental results. The groundwater level is about 2.5 m below the ground surface and denotes approximately a thorough saturated soil profile. Below the workshops, the upper 10 meters is designated as loose silty sand layer that is underlain by shallow cemented sand (Cap Rock) with a thickness of less than 1 meter. The silty sand layers are assumed to be susceptible to liquefaction.

In order to evaluate the soil modulus (E), pressuremeter tests has been performed. Shear moduli values are subsequently derived from elastic moduli using Hardin and Black, 1968 and Seed and Idriss, 1970 relationships for clay and sand layers, respectively. These values are required to constitute a soil model and evaluate the site response amplification of earthquake motion transmitted from the bedrock towards ground surface.

Since the main issue is the liquefaction susceptibility of the silty sand deposits, the conventional Standard Penetration Test (SPT) is applied. The use of SPT blow count as a tool for evaluation of liquefaction potential first began to evolve in the wake of a pair of devastating earthquakes that occurred in 1964; the 1964 Great Alaskan Earthquake (M=8+) and the 1964 Niigata Earthquake

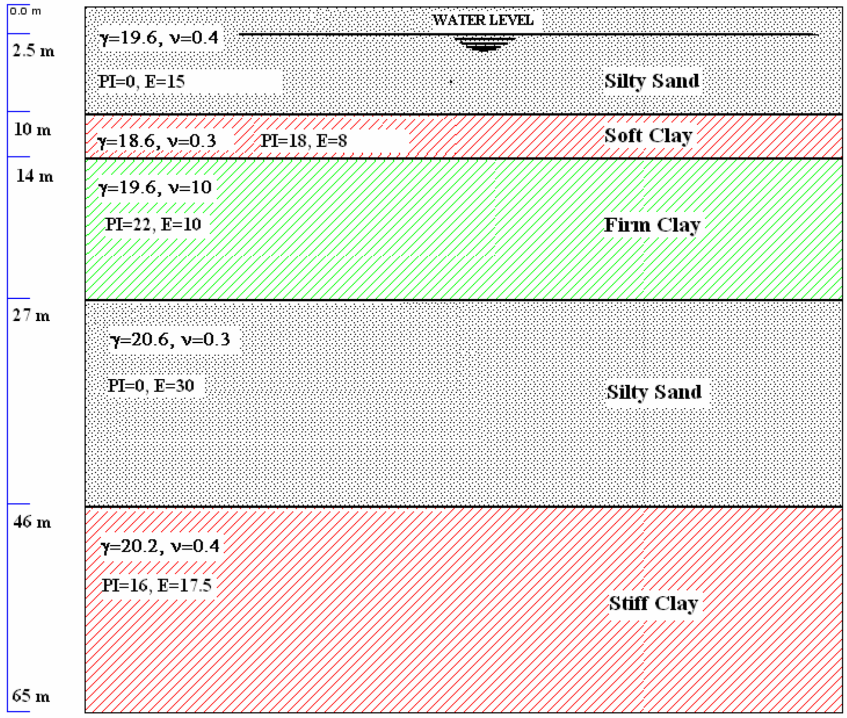

Figure 2. Soil profile and mechanical characteristics of the layers

γ: Soil density (kN/m^3) ν: Poisson ratio PI: Plasticity Index E: Modulus of elasticity (MPa)

(M<7.5), both of which produced significant liquefaction-related damage (e.g., Kishida 1966; Koizumi 1966; Ohsaki 1966; Seed and Idriss 1971). A typical 65m borehole with the obtained SPT values is shown in Figure 3.

Liquefaction potential incurred due to the credible seismic event has been assessed in three major steps as follows:

1. Evaluating the probabilistic local PGA using probabilistic seismic hazard analysis (PSHA) by two separate approaches;
2. Local site response analysis to derive the ground amplification variations in the alluvium;
3. Liquefaction potential assessment utilizing the Technical Standards and Commentaries for Port and Harbor Facilities in Japan, 2002.

PROBABLISTIC LOCAL PGA ESTIMATION

Currently two distinct approaches on PSHA may be distinguished. The first approach, known as deductive, was basically formulated by Cornell (1968) and requires the distinction of potentially active seismic zones or resources. Seismicity parameters of the region are to be determined in this method which includes mean seismic activity rate ν, the maximum magnitude M_{max}, and the Guttenberg-Richter relation for each seismogenic zone. To find out these parameters, the cumulative distribution function (CDF) is established for the required ground motion parameter (e.g. PGA). Integration of individual contributions from each seismogenic one into a site-specific distribution is the final step in the process. The major disadvantage of this approach comes from the necessity of determining the seismic sources (Kijko and Oncel, 2000). Since no official extensive survey was available for the site region's seismic specifications, the seismogenic zones were not well defined in the area. To compensate for imprecision in determining seismogenic sources, a historic PSHA procedure has been simultaneously followed here.

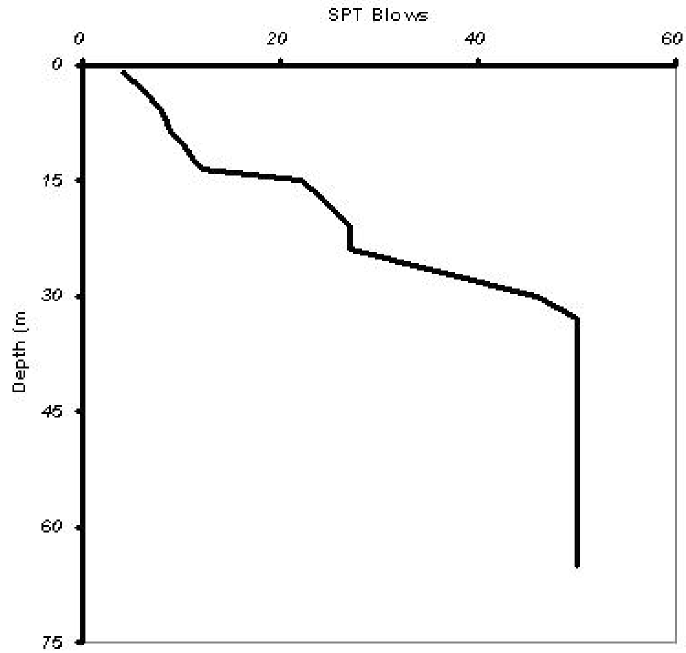

Figure 3. Typical SPT N-value profile

Historic methods are the other category of PSHA approaches that only require past seismicity data in the area and not any specific recognition of the seismogenic zones (Veneziano and Cornell, 1984). A catalogue of past earthquakes in the region must be established for this procedure (including both historic and instrumental events). The empirical distribution of the required seismic hazard parameter (PGA for instance) is estimated using the values for magnitude, epicentral distance, and assumed ground motion attenuation relation for each event in the catalogue (Kijko and Oncel, 2000). Normalizing this distribution for the duration of the seismic event catalogue, one may obtain an annual rate of exceedance of a predetermined value of the hazard parameter. This procedure is not recommended for an area where the seismic event catalogues are highly incomplete. Since no official extensive survey was available for the site region's seismic specifications, the faulting systems are not well defined in the area. This can weaken the reliability of the deductive based approaches for PSHA. The results are then compared for an appropriate selection of PGA value for amplification analysis. The assumptions and input data of the two methods are given in Tables 1 and 2.

Both PSHA approaches result in PGA recurrence curves. These curves show the return period of a given peak ground acceleration. The resulted curves are shown in Figures 4 and 5 for deductive and historic approaches respectively. The Maximum Probable Earthquake (MPE) was determined. Defining the seismic event having 10% probability of exceedance in the structure's lifetime, the assuming the dry docks lifetime to be 50 years, the event is to be affiliated with accelerations of 0.35g and 0.31g from deductive and historic curves, respectively as shown in Figure 4 and 5. The values are close enough, and so simply the larger value (0.35g given by the deductive approach) was selected as the bedrock PGA to be used in amplification analysis.

Table 1. Input variables for deductive PSHA

Source type	Area source (9 zones)
$m_{\circ}$	5.0
m_{max}	7.8
Faulting system	Mainly reverse
Attenuation relation	Boore et al. (1993)
Local site condition	Rock

Table 2. Input variables for historic PSHA

Seismic events time span	107 years
$m_{\circ}$	5.0
$m_{\max}$	7.8
Seismic events distribution function	Gumble type III
Attenuation relation	Boore et al. (1993)
Local site condition	Rock

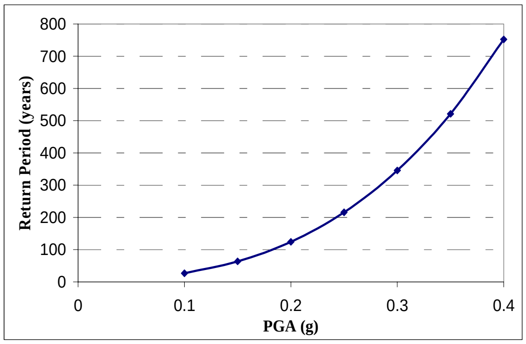

Figure 4. Deductive PSHA return period for a given PGA

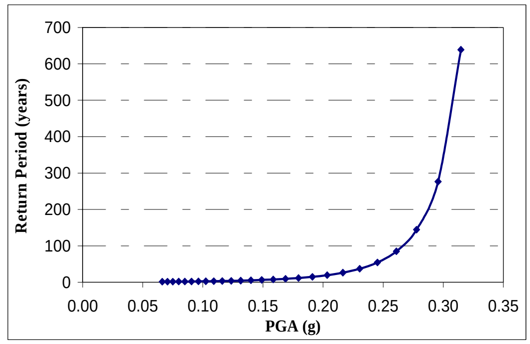

Figure 5. Historic PSHA return period for a given PGA

LOCAL SITE RESPONSE ANALYSIS

A 2-D equivalent linear analysis within a finite element model -run by the geotechnical software package Geo-Slope v.5- is carried out to consider the amplification effects of earthquake pulses transmitting through the soil layers. The analysis was run using the. Equivalent linear analysis, a total stress based approach, has been widely utilized in dynamic analyses of earth structures in geotechnical practice. The values of G_{max}, the shear modulus reduction curve G/G_{max}, and damping ratio curve that are required in the analysis were derived from the measured data. The initial shear modulus was calculated by Equation 1 (Hardin and Black, 1968) and Equation 2 (Seed and Idriss, 1970) for clay and sand, respectively:

$$G_{\max} = 3270.\left(\frac{(2.97-e)^2}{1+e}\right).\sigma'^{0.55} \tag{1}$$

$$G_{\max} = 1000.k_1.\sigma'^{0.5} \tag{2}$$

where e is the void ratio, and σ' the effective confining pressure. The parameter k_1 can be generally assumed equal to 15 for compacted sand.
A recorded time-history of Qeshm earthquake is selected to be used as the base time-history in the analysis after scaling for PGA. This earthquake took place in November 2005 having a magnitude of M_W= 6.0. The original time-history had a peak ground acceleration of 0.32g which was scaled to 0.35g according to the results of the probabilistic seismic hazard analysis implemented in the previous stage. The model is extended large enough in horizontal direction to avoid wave distortion which may occur due to the reflection of seismic waves at the model boundaries. Figure 6 illustrates the result, in which the peak ground acceleration profile is depicted versus depth throughout the alluvium. The peak ground acceleration is observed to have amplified to 0.67g at the ground level.

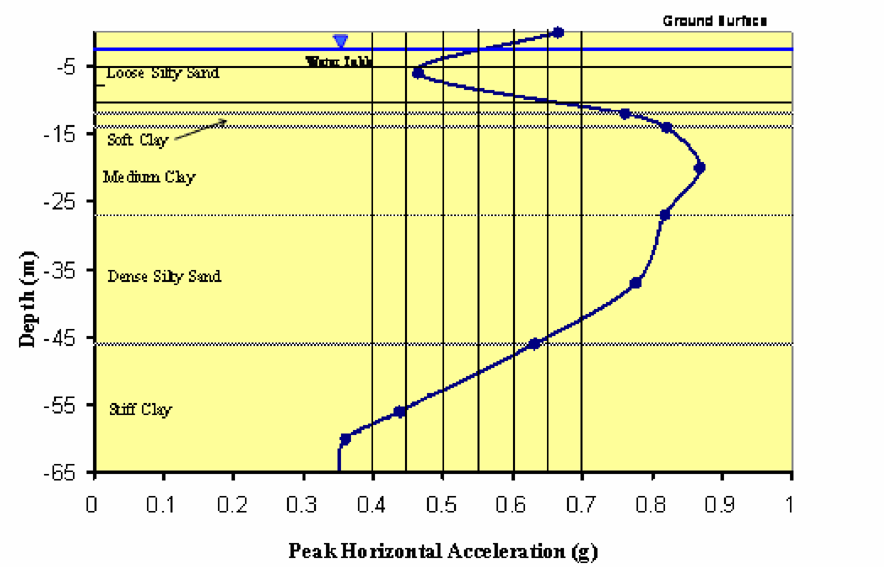

Figure 6. Peak ground motion variations diagram

LIQUEFACTION POTENTIAL ASSESSMENT

To assess the liquefaction potential of the layer, the method proposed by Technical Standards and Commentaries for Port and Harbor Facilities in Japan is utilized. This code offers a rather simple approach to evaluate the possibility of liquefaction using the parameters such as particle-size distribution curve, fines content percentage, plasticity index, maximum shear stress or PGA estimated from local site response analysis, and the SPT results.

Figure 7 presents the particle-size distribution curve of the silty sand layer which is in the range of very large possibility of liquefaction. To verify the liquefaction susceptibility of the layer, the following steps are to be examined. First the SPT blow count shall be corrected for the effective overburden pressure of 65 kN/m^2 using Equation 3 (The Japanese code).

$$N_{65} = \frac{N - 0.19(\sigma_v^{'} - 65)}{0.0041(\sigma_v^{'} - 65) + 1.0} \tag{3}$$

where N_{65} is equivalent N-value, N, N-value of the subsoil, and σ'_v the effective overburden pressure of the subsoil (kN/m^2).

According to the Japanese code, if the fines content is equal or less than 15% or plasticity index is less than 10%, the equivalent N values should be set on N_{65}/C_N where C_N is the compensation factor as shown in Figure 8. Since the fines content of SM layer is 15% here, the corresponding compensation factor is considered as 0.5.

The equivalent acceleration is calculated by Equation 4 (The Japanese code).

$$\alpha_{eq} = 0.7 \times \tau_{\max} / \sigma'_v \times g = 0.7 \times a_{\max} \times \sigma_v / \sigma'_v \tag{4}$$

where α_{eq} is equivalent acceleration (Gal), τ_{max}, maximum shear stress (kN/m^2), σ'_v, effective overburden pressure (kN/m^2), σ_v, total overburden pressure (kN/m^2), g, gravitational acceleration (980 Gal), and a_{max}, maximum acceleration (Gal). The effective overburden pressure should be determined based on the ground elevation during an earthquake. The maximum shear stress and the maximum acceleration of the layer are determined from local response analysis.

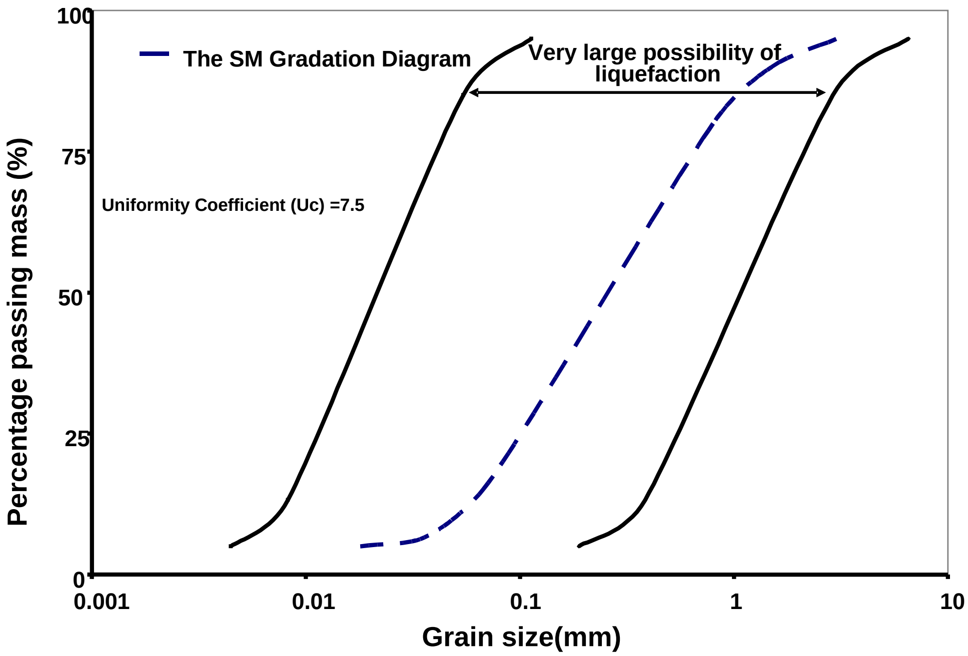

Figure 7. A typical silty sand gradation graph (The Japanese code)

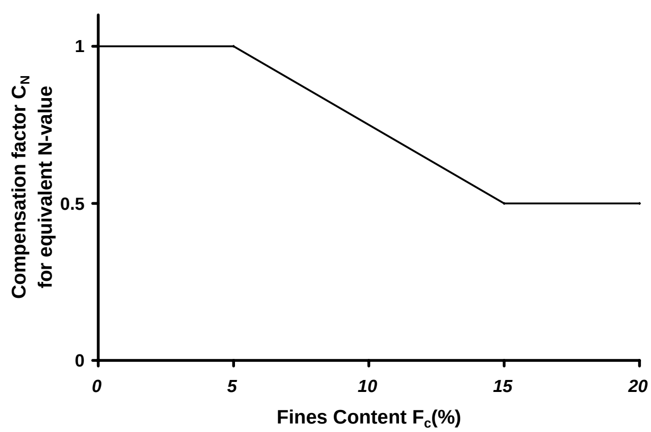

Figure 8. Compensation factor of equivalent N-value corresponding to fine contents (The Japanese code)

Finally, the code brings the analyses output in a diagram shown in Figure 9 to indicate the possibility of liquefaction occurrence. The spots represent several bore logs of the top layer. The figure clearly reveals that the considered layer fall in the range of very high possibility of liquefaction region should the scenario earthquake happens.

SOIL IMPROVEMENT USING DYNAMIC COMPACTION

There are a number of ground improvement techniques capable of reducing the potential of liquefaction susceptible ground including explosive compaction, dynamic compaction, particulate grouting, electrokinetic injection, jet grouting, mix-in-place piles and walls and vibro-replacement stone columns (Ledbetter, 1985, Hausmann, 1990, and Moseley, 1993). Based on review of options for foundation ground improvement, the dynamic compaction method has been selected to improve the top soil deposit which has a very high probability of liquefaction.

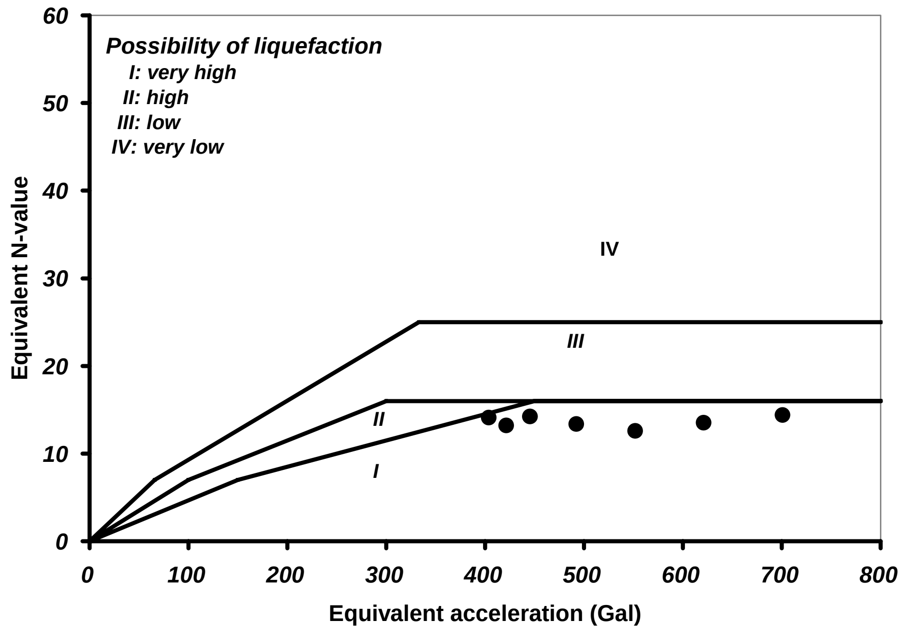

Figure 9. Liquefaction assessment on the top soil, before dynamic compaction (The Japanese code)

Among the different remedial measures, dynamic compaction (DC) is a well recognized and widely used technique that involves the repetitive dropping of large weights in a grid pattern, with the direct application of high energy causing the compaction and strengthening of the ground (Mayne et al, 1984). The design of drop pattern and average energy application depends on the soil types to be improved and desired depth of improvement. Typically a large steel tamper is dropped several times from heights of up to 30 m at a given grid or drop point. The method may be used to treat soils both above and below the water table. In granular soils the effectiveness is controlled mainly by the energy per drop.

In order to design the work, a trial compaction program using dynamic compaction has been performed in a three-stage compaction pattern as follows:

- Conduct 3 drops using 19.2 ton ponder with free fall height of 15.5 m on 4.6 m grids;
- Conduct 6 drops using 19.2 ton ponder with free fall height of 15.5 m on the middle spots;
- Conduct 3 drop using 7.5 ton ponder with free fall height of 12 m in the ironing stage.

The pilot test to establish the dynamic compaction pattern was performed in an area of 15 x 15 meters. Several boreholes have been excavated after dynamic compaction (DC) ranging from 11 to 13 meters to evaluate the effectiveness of the method. Figure 10 depicts a comparison between SPT N-value profile before and after applying deep dynamic compaction. It’s clearly evident that the deep dynamic compaction programme has effectively improved the density of the layer. While the SPT N-values have generally increased everywhere, the SPT profile reveals a decreasing trend at shallow depths which turns to an increasing trend later on. This may be due to the high stress concentration at shallow depths caused by immediate impact effect of the tamper. This impact effect dramatically decreases at greater depths.

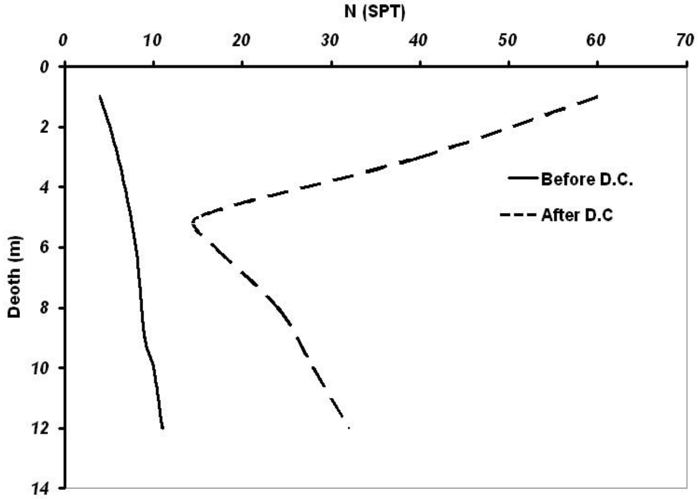

Figure 10. SPT N-Values versus depth in the top layer before and after compaction

The analysis provides that the overall effect of dynamic compaction programme was sufficient to achieve a stable foundation subjected to earthwork as shown in Figure 11 demonstrating the reduction of liquefaction potential after dynamic compaction. It can also be observed that all the spots have moved to the safe zone. This indicates that improvement procedure has been successful enough to prevent the risk of liquefaction due to strong ground motion. Figure 11 suggests that the applied dynamic compaction pattern has perhaps been more than necessary since all the spots are moved safely into the region IV. However, an alternate pattern could not be established for there was only a single pilot test available.

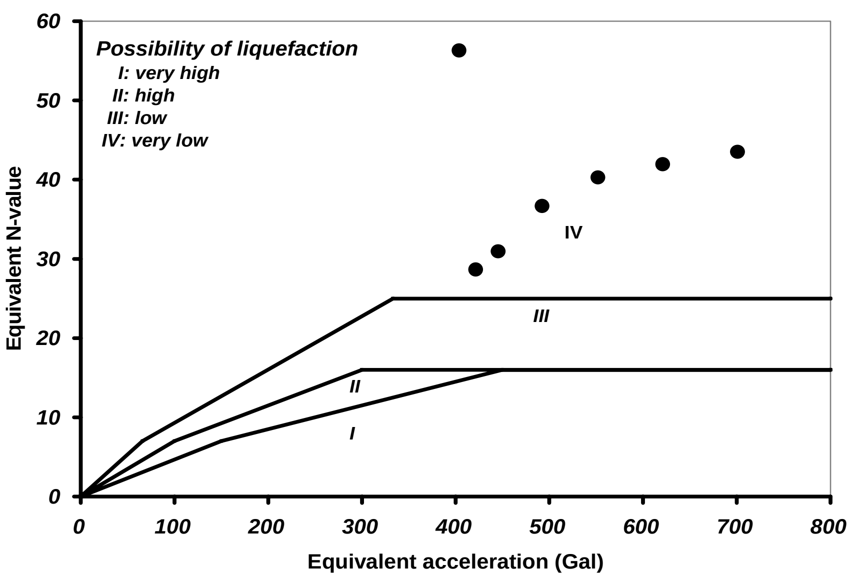

Figure 11: Liquefaction assessment on the top soil layer, after dynamic compaction (The Japanese code)

CONCLUSIONS

The effectiveness of deep dynamic compaction as a countermeasure against liquefaction susceptibility has been assessed. Two 300,000 ton dry docks are being constructed near the southern port of Bandar Abbas, Iran. Recent earthquakes in the region prove the existence of an active zone. On the other hand, geotechnical investigations revealed that the top soil layer is very prone to liquefaction. To address this concern, a seismic hazard assessment analysis has been implemented in the construction site. Two different PSHA approaches, deductive and historic, were concurrently carried out to compensate for the imprecision due to the lack of reliable seismological studies involved in the analysis. A finite elements model has subsequently been developed to determine the PGA variations along the soil layers from bedrock to the ground level. The site response analysis yields an acceleration of 0.67g at the ground level. The Liquefaction assessment then followed for the top soil deposit demonstrating the high probability of liquefaction occurrence. Deep dynamic compaction has been applied as a mitigation countermeasure. The proposed assessment and SPT testing suggest that the dynamic compaction has generally met the performance criteria preventing earthquake induced liquefaction.

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