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Evaluation of Empirical Relationships for Dynamic Compaction in liquefiable Reclaimed Silty Sand Layers using Pre/Post Cone Penetration Tests

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ABSTRACT

This case history presents an application of Dynamic Compaction of the soil layers susceptible to liquefaction behind the main container quay wall of Shahid Rajaee Port Complex Development (SRPCD), situated on shores of Persian Golf near Bandar-Abbas. The liquefaction of such layers during earthquake results in the great lateral earth pressure as well as the settlement and large horizontal deformation of the main wall and anchor wall. Regarding the extension and the depth of the identified liquefiable layers, the improvement method of dynamic compaction was employed to mitigate the liquefaction destructive effect. Generally, the subsoil liquefiable layers of the SRPCD site consisted of reclaimed layers of silty sand with the maximum depths of 7 to 12 meters and the fine content of 20% – 40%. The preliminary compaction patterns were obtained using the energy-based method and the available empirical relationships based on the depth of influence and the required improving energy. Considering the previously conducted researches, the effectiveness of dynamic compaction and the applied energy to subsoil collapsible layers deteriorates due to the presence of fine content. Therefore, the effective influence depth of soil that is affected by this method of improvement is reduced. The effectiveness of the employed dynamic compaction patterns for different parts behind the main quay wall and anchor wall is evaluated comparing the results of pre and post-CPT tests (cone penetration tests performed before and after the compaction) with the criterion. Such criterion is defined as the liquefaction threshold resistance of the soil layers that is obtained using the most recent and distinguished CPT based liquefaction evaluation method. In case the criterion is not satisfied using CPT test results, the pattern (weight and drop height of the tamper, spacing and the passes of compaction) is modified in order to apply the greater amount of energy. Comparing the obtained values for influence depth resulted from empirical formula and the Pre/Post-CPT results, the validity of the preliminary used relationships and empirical constants are studied. In addition, back calculating the constants using the obtained improved depths from pre/post-CPT, the achieved constants for the available relationships are suggested.

INTRODUCTION

Dynamic Compaction as a high-energy impact method to densify loose and medium dense granular soils is one of the most frequently used methods for soil improvement which has been proved to effectively mitigate the liquefaction and the induced consequent devastating settlement and loss of bearing capacity. The theoretical and practical aspects of dynamic consolidation were firstly introduced by French Firm Menard Techniques Limited (Menard and Broise, 1975). This soil-improvement technique has become a well-established method for treating a wide range of soils due to its simplicity and cost-effectiveness. In this method, which is also known as heavy tamping, large pounder weighing typically 60–400 kN is dropped in free fall from a height generally varying between 10 and 40m onto predetermined grid points on the ground surface. The weight (W) and the free fall height (H) of the tampers are determined using empirical methods in which empirical relationships relate the depth of soil improvement (d_{max}) to the energy (W.H) applied to soil sublayers. Generally, the current design of DC work i.e. the tamper weight (W), drop height (H), print spacing of the grid (S), number of drops per pass (N), and number of passes (P), is still essentially empirical in nature, relying mainly on the results of trial compactions and pre/post control in-situ and laboratory tests. However, several methods have been introduced to design the dynamic compaction work using numerical modeling of wave propagation (Chow *et al.*, 1992a and b; 1994; Lee and Gu, 2004) and physical modeling (Merrifield and Davies, 2000).

Since Dynamic Compaction densifies soils through vibration and wave traveling in subsoil layers, the technique is mostly effective in granular material and presence of the fine-grained particles decreases the efficiency of the technique. As in most cases, subsoil strata are consisted of various soil types with different percentage of fine grained material, clean granular layers in which DC is highly effective rarely happen in the nature. DC is also widely used to treat dredged material which has formed reclaimed areas. Such loose and mostly liquefiable material may result in extensive densification and settlements and also apply a substantial lateral earth pressure to the in-situ deep structures such as retaining walls and pile foundations.

This paper presents the results of a DC work in a reclaimed area of dredged material, which mainly consists of sand layers with the fine content of 20-40%. The paper also evaluates the effective depth of improvement considering the applied energy to the ground sublayers and also the fine content.

DESCRIPTION OF THE SITE

The site is located in the western part of Shahid Rajaee port. As shown in Figs. (1) and (2), development Stage consists of two different zones:

a) Zone 1: The two narrow rectangle zones of 910m in 100m for new berths Eastern and western berths with the region behind quay walls for unloading and reloading containers. The narrow rectangular area is next to a region of shallow water open to the harbor, which will be dredged to provide sufficient water depth for berthing of the vessels. As illustrated in Fig. (3), this zone includes the berthing diaphragm wall, the anchor wall and the rear crane bored deep foundation.

b) Zone 2: The area of an approximately rectangular shaped zone with 1300m in 480m dimensions in the north part of the site where the containers are to be stacked with maximum 5 high. This zone the site is land, with existing roads, buildings and port infrastructure.

Different Zones of the site have been previously reclaimed at the level of approximately +4.5mCD.

GEOTECHNICAL SUBLAYER CONDITION

Geotechnical site investigations were conducted in different regions of the site in several comprehensive phases taking advantage of both in-situ tests i.e. Standard Penetration Test, Cone Penetration Test, Vane shear Test, Pressuremeter Test, together with laboratory tests performed on intact and disturbed samples extracted from boreholes. All the boreholes fairly show similar subsoil layers, however there is some variability in the levels and thicknesses of each of these layers in different borings. The five substantial layers generally identified in the boreholes are as follows:

- Existing Fill/ Made Ground (Medium Dense SILTY SAND with N_{SPT} of 10-20 bpf, thickness of 3-4m).

- Upper SAND (Medium Dense SILTY SAND with N_{SPT} of 10-25 bpf, thickness of 5-8m).

- Upper SILT (Medium Dense SANDY and CLAYEY SILT with N_{SPT} of 10-15 bpf, thickness of 2-3m).

- Upper CLAY (Firm to Stiff normally consolidated SILTY CLAY with N_{SPT} of 5-15 bpf, thickness of 7-10m).

- Lower SAND (Dense to very Dense SILTY SAND with N_{SPT} of greater than 50 bpf, thickness of 9-11m).

- Lower SILT (Dense to very Dense SANDY and CLAYEY SILT with N_{SPT} of 30-70 bpf, thickness of 6-15m).

- Lower CLAY (Hard to Very Hard overconsolidated SILTY CLAY with N_{SPT} of 20-40 bpf, upper depth of 33-40m to the end of the boreholes).

SEISMOLOGICAL CONDITION OF THE SITE

Based on Performance-Based Design and according to seismic hazard analysis, two levels of ground motion were adapted for the seismic analysis. The Peak Ground motion Acceleration (PGA) for different design levels were evaluated as below:

Level-1: PGA= 0.25g; (M_w=6.5).
Level-2: PGA= 0.37g; (M_w=7.0).

LIQUEFACTION ASSSESSMENT

High values of PGA obtained from the seismic hazard analysis showed that the region of the project was a seismically active area, thus necessitated the seismic analysis and check all possible mechanisms of failure and deformation. Furthermore, most of the sublayers of the region were generally young sediments, which in the upper levels appeared as dredged material. Such dredged sediments were so loose, thus susceptible to liquefaction and high deformations due to densification during earthquake. As indicated in the previous section, the upper layers i.e. existing fill material and upper sand, are medium dense with N_{SPT} values of 10-20 bpf. Low density of such layers especially in upper levels associated with the greater hazard of liquefaction in superficial layers due to less confinement result in an increasing necessity of liquefaction evaluation and soil improvement. Moreover, in western parts of the site due to stacking of the dredged material a lagoon was formed. Due to the very loose material of the lagoon, even conducting geotechnical investigation was only possible by performing an embankment on the lagoon. Therefore, a primary preloading treatment was applied to strengthen the loose silty and sandy soil in the lagoon.

In addition to high values of liquefaction-induced settlement, a huge extra pressure due to lateral spreading of the liquefied soil strata was also a likely loading.

Fig.1. Plan of the S.R.P.C.D. Stage 2 project: 2 western and eastern container berths and northern stacking yard

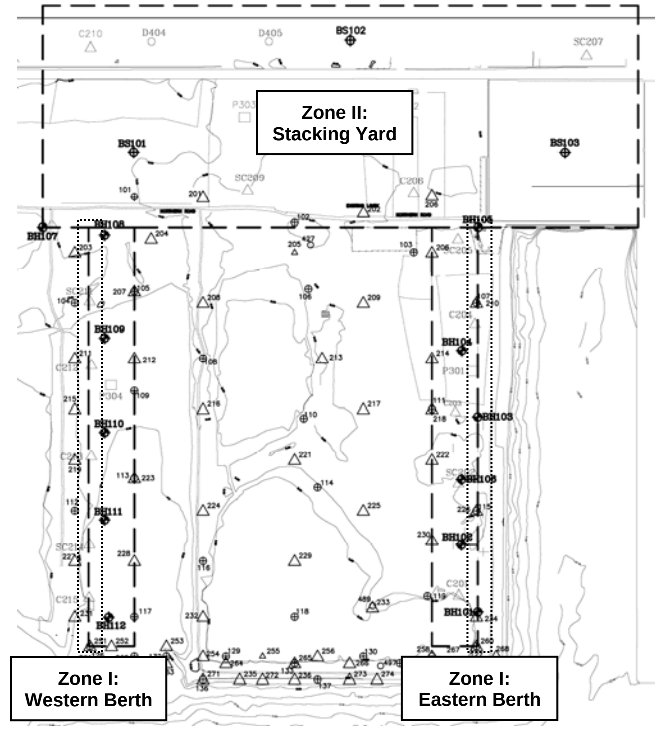

Fig. 2. Zones of S.R.P.C.D. project (Zone I: Western and Eastern container berths; Zone II: Northern container stacking yard) and geotechnical site investigation

The quay wall of S.R.P.C.D. project was an anchored diaphragm wall, which was constructed in 4-meter width in-situ concrete panels. Since the panels were implemented in place without deep backfilling material and the structure was predicted to be fairly deep with the dredging depth of 25 meters (Fig. 3), the liquefaction induced lateral pressure of the liquefiable layers could be severely catastrophic for the retaining wall. Furthermore, the in-situ concrete anchor wall acting as a support for the main wall with the length of approximately 15 meters (Fig. 3) was also prone to loss of stability due to devastating earth pressure of lateral spreading. Negative effects of liquefaction induced lateral pressure on the rear crane piles (Fig. 3) also had to be mitigated.

Therefore, taking advantage of field test results i.e. SPT and CPT data the evaluation of liquefaction susceptibility was conducted using PGA in Level-2 (0.37g for the 475 years of return period earthquake). The factor of safety for liquefaction assessment for a level-2 earthquake was adapted as 1.0 considering the performance-based design methodology and the grade of the berthing structure (PIANC, 2001) together with the recommendation of “Seismic Criteria for California Marine Oil Terminals” (Ferritto *et al.*, 1999).

In the procedure of liquefaction assessment of the S.R.P.C.D. site which is presented in detail by Seyedi and Jalili (2007), NCEER Workshop (Youd and Idriss, 2001) recommendations, originally based on Seed *et al.* (1985) for SPT and Robertson & Wride (1998) for CPT liquefaction evaluation, were used and the maximum depth of liquefying layers for each borehole and cone penetration test was determined in western and eastern zones of the site. For determination of the minimum soil strength required against liquefaction, the criterion based on the N_{SPT}, presented in the OCDI (2002), was used (Seyedi and Jalili, 2007). According to this procedure, the required equivalent N-values (N_{60}), which meets the minimum criteria for densification to mitigate liquefaction, was obtained for three groups of soil based on the fines content of 5%, 10% and 15%. Using the available empirical relationships, i.e. $q_c = C_f \times N_{SPT}$ in which C_f is the coefficient dependent on D_{50}, the obtained criterion for N_{SPT} was defined as cone tip resistance (Seyedi and Jalili, 2007).

Having determined the improvement depths, the western and eastern regions behind berths were divided into several zones of improvement. The different improvement segments determined based on the assessed maximum depth of liquefiable layer for the eastern and western regions behind the quay walls are shown in Figs. (4) and (5) respectively.

DETERMINATION OF COMPACTION PATTERN

In order to determine the mass and drop height of the tamper, the formula originally presented by Menard and Broise (1975) and modified by Lukas (1986) was used:

$$d_{max} = n.(WH)^{0.5} \qquad (1)$$

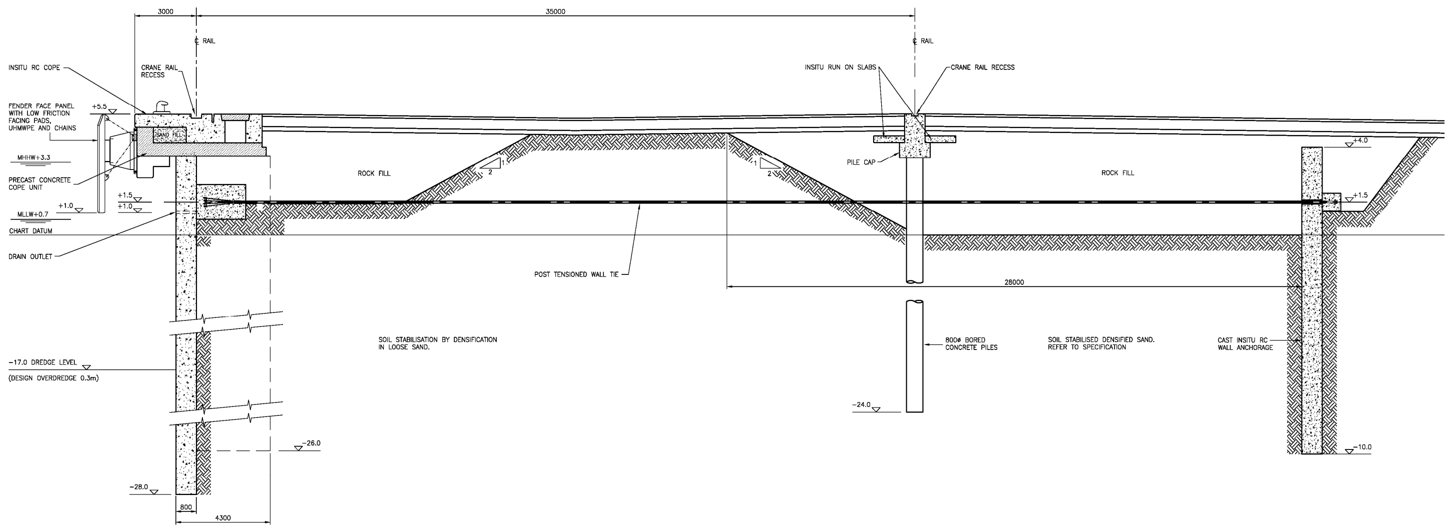

Fig. 3. Typical section of the diaphragm quay wall, rear crane pile and anchor wall in S.R.P.C.D. project

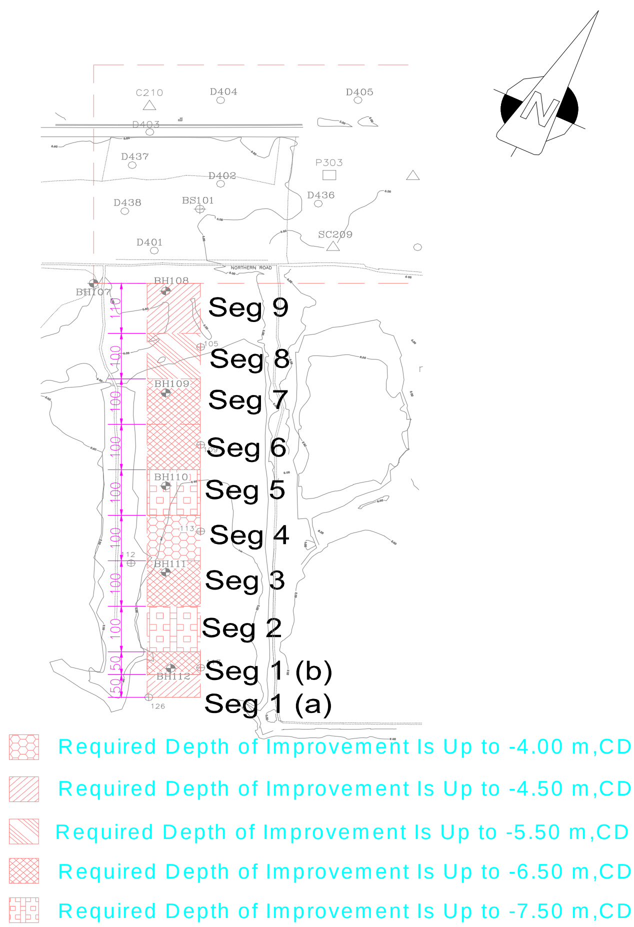

Fig. 4. Improvement segments for the western region behind the quay walls

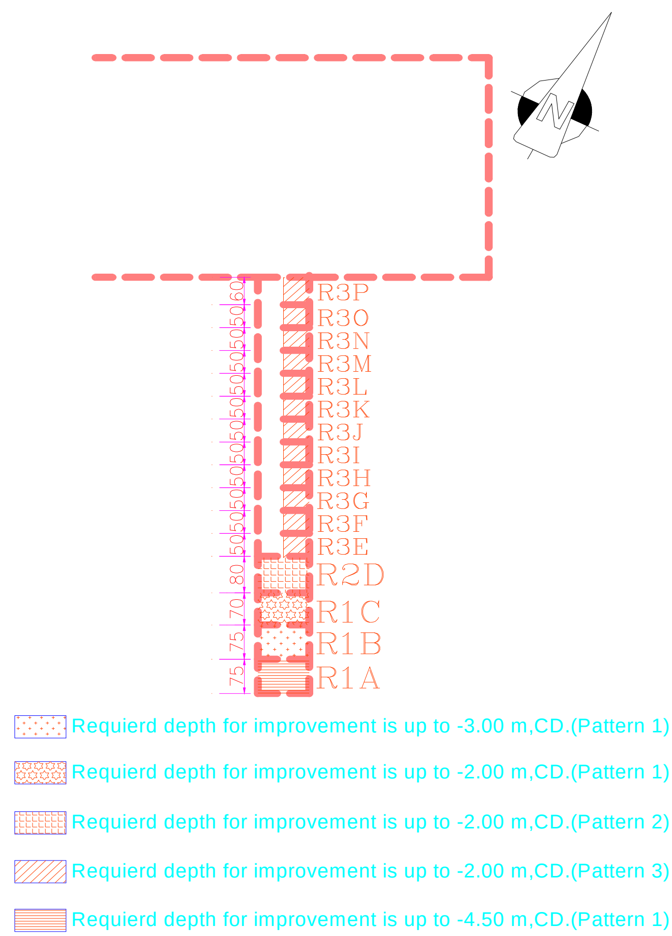

Fig. 5. Improvement segments for the eastern region behind the quay walls

d_{max}= depth of improvement in meters.
W= mass of tamper in tones.
H= drop height in meters.
n= empirical coefficient.

The upper layers needed to be improved were loose silty sand and silt layers as semipervious deposits with low values of plasticity index with low saturation in the first 2 to 3 meters and high saturation in the greater depths. The fine content of these layers varied about 20-40%. Therefore, n coefficient was selected as 0.4. After conducting pre and post Cone Penetration Tests and determination of the improvement depth, having W and H the coefficient was back calculated for n and compared with the assumed value.

Applied energy requirements

To achieve a minimum degree of improvement in soil parameters and meet design purposes sufficient amount of energy must be applied during dynamic compaction to the soil layers. The applied energy generally given as the average energy applied over the entire area is calculated using the following equation:

$$AE = \frac{NWHP}{S^2} \qquad (2)$$

Where *AE* is applied energy per unit area, *N* is number of drops for each specific drop point, *W* and *H* are mass and drop height of the tamper, *P* is number of passes and *S* is the grid spacing in meters. For this project, the target preliminary value of applied energy was adapted using the available values in literature and also the values back calculated from the previous construction stages of the project. According to Lukas (1995) the applied energy for Semipervious fine-grained soils is 250-350 kJ/m^3 (41-60% of Standard Proctor Energy). Having assumed *AE* as well *S* and also *WH* using Equation (2), the product of Number of drops and Passes (*NP*) can be obtained. Due to the high ground water level and low values of permeability, number of passes was selected as 2 in order to allow the generated pore pressure to be dissipated.

Improvement of liquefiable layers in eastern zone

Having determined the depth of improvement in the eastern region (Fig. 5), the specifications of dynamic compaction were obtained using the mentioned procedure. Since the maximum improvement depth in most segments was 7-8m, Patterns 2 and 3 were mainly used for these segments. The specifications for the mentioned dynamic compaction patterns are presented in Tables (1) and (2).

The results of Pre and Post-CPT tests for 6 trial areas of 6 selected segments are depicted in Figs. (6) to (11). Comparing the q_c results of Post-CPT test ($q_{c,Post}$) with the criteria values of q_c obtained from the liquefaction assessment shows that the employed DC patterns suffice for the improvement of the liquefiable layers and transfer energy satisfactorily to the desired depth. It should be noted that because of the high percentage of fine content in the soil layers (20-40%) the graph for (FC>15%) was used as the criterion to compare with the Post-CPT results. Using the Post CPT results and the obtained improvement depth, the average value of the back calculated n was 0.44, which fairly matched the assumed 0.4.

Table 1. Specifications of Pattern 2 for eastern region behind quay walls (applied to R2D)

Pattern	Pass	Phases	Weight (ton)	Height (m)	No. of Drops	Applied Energy	Area of Tamper (m^2)	Grid Dimension (L*M)
P 2	1	1	27	17	8	57	5.72	8*8
		2	27	17	6	43	5.72	8*8
	2	1	15	10	10	47	5.72	8*4
		2	15	10	10	47	5.72	8*4

Table 2. Specifications of Pattern 3 for eastern region behind quay walls (applied to R3E to R3P)

Pattern	Pass	Phases	Weight (ton)	Height (m)	No. of Drops	Applied Energy	Area of Tamper (m^2)	Grid Dimension (L*M)
P 3	1	1	15	17	9	47	5.72	7*7
		2	15	17	7	36	5.72	7*7
	2	1	10	10	10	41	5.72	7*3.5
		2	10	10	10	41	5.72	7*3.5

Improvement of deep liquefiable layers in western zone

As mentioned before in the region behind the western quay walls, the depth of the liquefiable loose material was predicted to be high due to the lagoon of dredged material. The geotechnical site investigations and liquefaction assessment also confirmed the predictions and depicted that the depth of loose material susceptible to liquefaction varies between 9 to 13m, which is fairly high for the regular depth that the dynamic compaction energy can reach. Furthermore, high percentage of fine-grained silty and clayey material in these soil layers might result in a less energy transferred to the desired depth of improvement.

Using the aforementioned procedure, several trial dynamic compaction patterns were designed for different part of the western region behind quay wall. For the segments in which the depth of improvement was high the pattern that was of the highest energy was applied. The details of the designated pattern including the weight, area, and drop height of the tamper, and also the number of passes and grid spacing are presented in Table (3).

Table 3. Specifications of Pattern 1 for western region

Pattern	Pass	Phases	Weight (ton)	Height (m)	No. of Drops	Applied Energy	Area of Tamper (m^2)	Grid Dimension (L*M)
P 1	1	1	30	22	10	103	5.72	8*8
		2	30	22	8	82	5.72	8*8
	2	1	15	15	12	84	5.72	8*4
		2	15	15	10	70	5.72	8*4

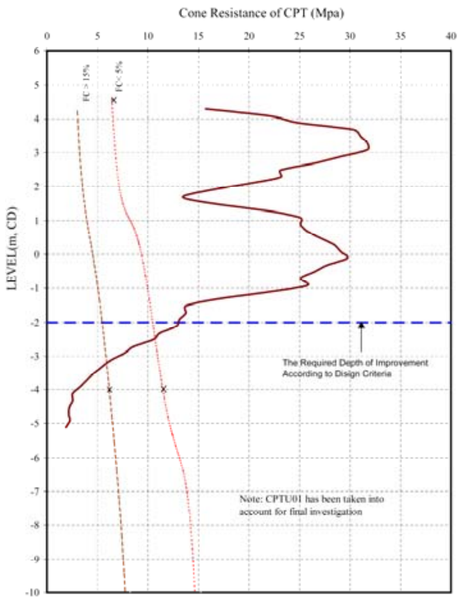

Fig.6. Results of $q_{c,Post}$ for segment R1C

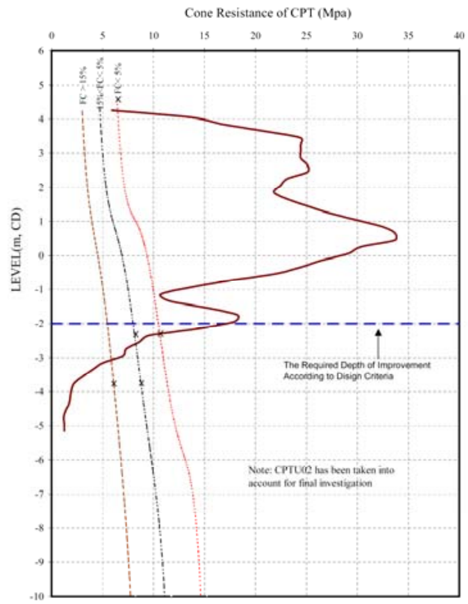

Fig.7. Results of $q_{c,Post}$ for segment R2D

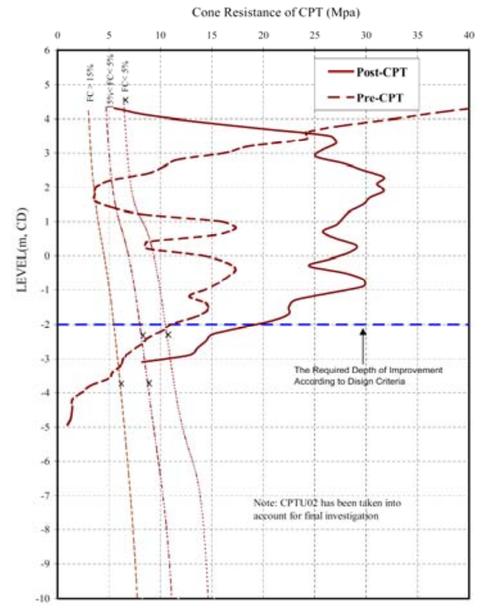

Fig.8. Results of $q_{c,Pre}$ and $q_{c,Post}$ for segment R3E

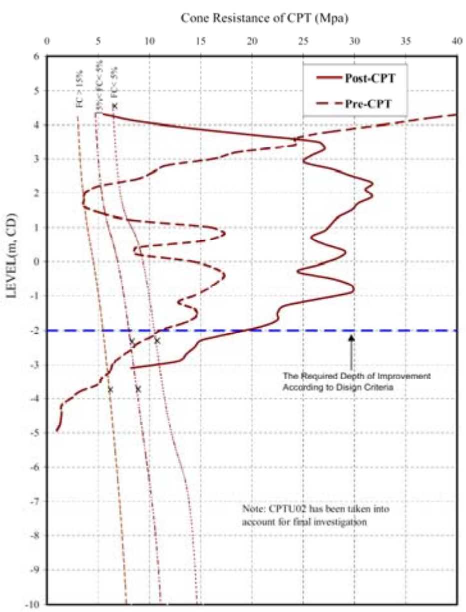

Fig.9. Results of $q_{c,Pre}$ and $q_{c,Post}$ for segment R3F

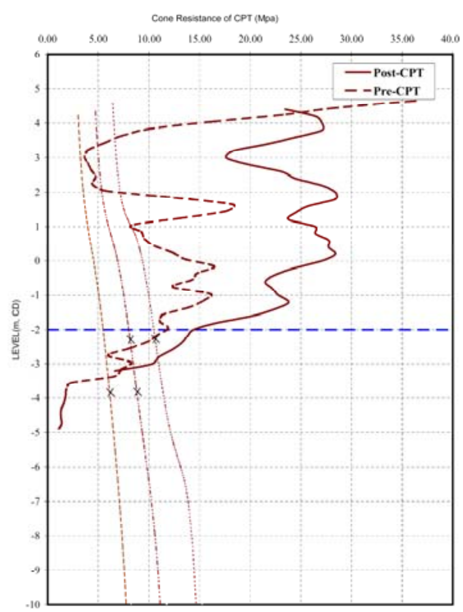

Fig.10. Results of $q_{c,Pre}$ and $q_{c,Post}$ for segment R3H

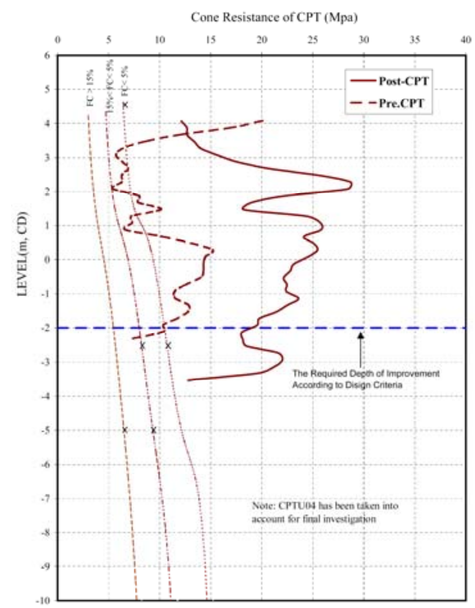

Fig.11. Results of $q_{c,Pre}$ and $q_{c,Post}$ for segment R3K

As the heaviest DC pattern P1 was applied to a trial area located in Segment 6 (Fig. 4) in which the liquefiable layer reached to the depth of 12m. The results of pre and post CPT testing are demonstrated in Fig. (12). Regarding the high percentage of fine content in the soil layers which varies between 20-40%, values of $q_{c,Post}$ were compared with the cone resistance graph of (FC>15%) as the q_c criterion. As it is evident in the Fig. (12), the energy of P1 is transferred to the desired level; however the slight increase in the values of q_c in comparison with the Pre-CPT shows that the energy has not reached deeper levels.

Pre and Post-CPT tests were carried out in between the points of impact. Conducting another Post-CPT in a point exactly beneath the point of tamper impact resulted in fairly the same results as the first Post-CPT.

Although P1 was somehow satisfactory, the pattern was changed to a heavier one as Pattern 1-1. In comparison with P1, the area of the tamper for P1-1 was reduced to 2.56 m^2 from 5.72 m^2 in order to apply more concentrated impact energy to a deeper depth of desire. The grid spacing was also changed from 9×9m to 6×6m to compensate for the decrease in the tamper area and a better overlap of the stress distribution in the underlying layers. The specifications of DC Pattern 1-1 are presented in Table (4).

Table 4. Specifications of Pattern 1-1 for western region behind quay walls

Pattern	Pass	Phases	Weight (ton)	Height (m)	No. of Drops	Applied Energy	Area of Tamper (m^2)	Grid Dimension (L*M)
P 1-1	1	1	32	25	9	222	2.56	6*6
		2	32	25	7	177	2.56	6*6
	2	1	15	15	12	150	2.56	6*3
		2	15	15	10	125	2.56	6*3

The results of Post-CPT after applying Pattern 1-1 presented in Fig. (13) shows a quite satisfactory improving effect of the employed pattern on the obtained values of q_c to a depth of 13m and therefore shows that an adequate amount of energy is transferred to the underlying layers in spite of the high fine content. Using the employed P1-1 the other liquefiable segments of the western region with the maximum depth of 13m can also be densified.

The results also show that the idea of reducing the dimensions of the tamper and also the grid spacing could give rise to a more effective dynamic compaction program and lead to a deeper level of improvement in the soil layers.

It should also be mentioned that using the improvement depth of 12m the back calculation of the coefficient (n) in Equation (1) for patterns P1 and P1-1 are respectively obtained as 0.44 and 0.42. Comparing the mentioned values with the first guess for n as 0.4 shows a reasonable agreement. Therefore, the first assumption for n coefficient was acceptable.

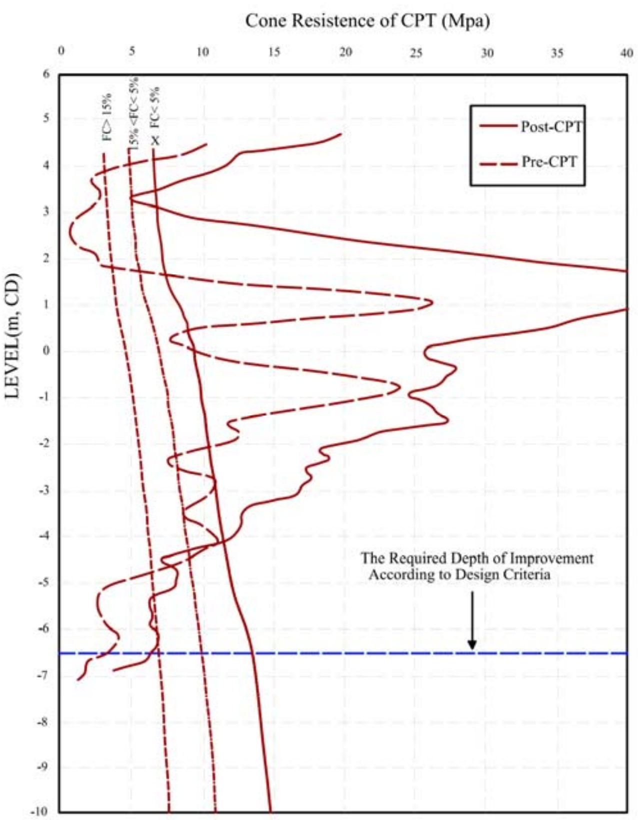

Fig.12. Results of Pre and Post-CPT for a trial area in segment 6 after applying DC Pattern 1

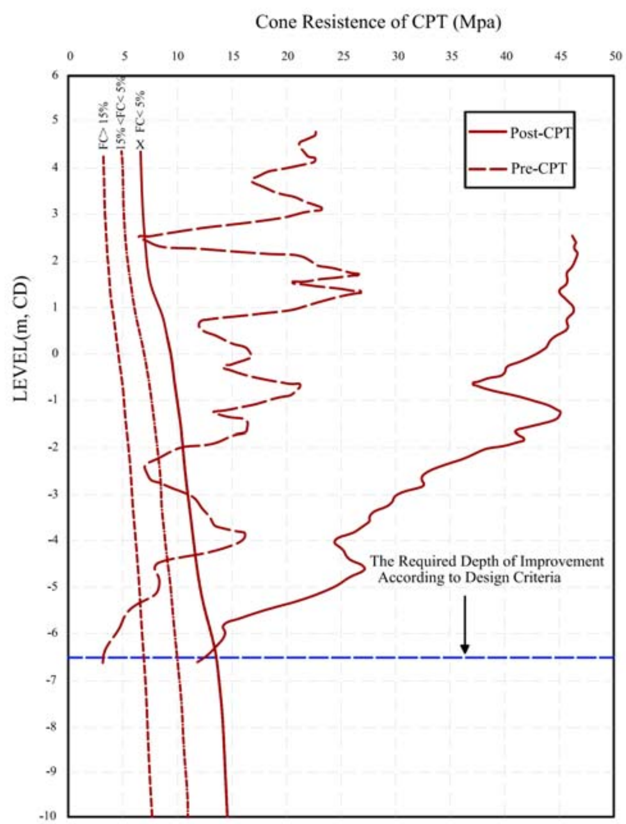

Fig.13. Results of Pre and Post-CPT for a trial area in segment 6 after applying DC Pattern 1-1

CONCLUSION

The paper concentrates mainly on the design, implementing, and control the soil improvement procedure of the reclaimed area. The reclaimed region is of dredging material which accommodates the stacking yard and the area behind berths are required to be treated to mitigate the devastating settlement as well as lateral spreading. Having determined the depth of liquefaction based on the available geotechnical site investigations and using the energy-based method, the primary DC patterns were generated. The results of trial dynamic compaction proved that deep liquefiable layers in could be improved by dynamic compaction patterns. In western region behind quay wall in which the depth of improvement had reached up to 13m, the heavy pattern with the tampers mass of 30-32 ton and free fall of 22-25m were adapted. Furthermore, the area of the tamper was reduced by 55% to transfer a more concentrated energy to the deep soil layers. Such decrease in tamper area together with a 25% reduction in grid spacing to provide a reasonable overlap, resulted in a satisfactory depth of improvement in spite of the high percentage of fines content of the soil layers (20-40%). Back calculating the values of coefficient n in the empirical relationship (d_{max}= n.$(WH)^{0.5}$) resulted in an average value of 0.42 which was in a reasonable agreement with the first assumed n= 0.40.

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