

COMPARISON OF LIQUEFACTION ASSESSMENT RESULTS USING SPT AND CPTU DATA

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ABSTRACT

This article explains how to find required criteria to prevent the liquefaction phenomenon for soil improvement in the Shahid Rajaei Port Project, Bandar Abbas, Iran. The site investigation shows some loose to medium dense sandy layers in different depths. Having low fines content, these layers are susceptible to be liquefied. Liquefaction of the soil behind the quay wall can cause large pressure and thus large deformations that might not be tolerable with the wall. Therefore the occurrence of liquefaction should be prevented by soil improvement. The soil improvement can help the quay wall to reduce the lateral movement and the applied pressure back the wall.

In this paper, the liquefaction assessment of the ground layers behind the wall is performed using the ground investigation in-situ SPT and CPTu tests and finally the depth, thickness of liquefiable layer and the ground settlement are calculated. Based on these calculations, it is concluded that there are good agreements between these two procedures. Finally, a criterion based on SPT procedure is presented not to allow the soil be liquefied, which is dependent on the fines content of the soil. In other words, the soil layer should be so improved that the mentioned criterion will be satisfied. It is noted that it is not necessary to improve the deep liquefiable layers according to in-situ surcharge effect based on Ishihara criterion (1985), which is explained in the paper.

Keywords: quay wall, liquefaction, SPT, CPTu, improvement criteria

INTRODUCTION

Soil liquefaction is a major concern for structures constructed on saturated soils. The major earthquakes of Niigata in 1964 and Kobe 1995 have illustrated the significance and extent of damage caused by soil liquefaction. Soil liquefaction is also a major design problem for large soil structures such as reclaimed lands. To evaluate the potential of soil liquefaction, it is important to determine the soil stratigraphy and state of the soil. Numerous methods have been proposed for evaluating the liquefaction potential of soil deposits including in-situ and laboratory tests which can be divided into stress based, strain based and energy based methods [1, 2, 3, and 4]. In-situ tests (the standard penetration test (SPT), the cone penetration test (CPT), shear-wave velocity measurements (Vs), and the Becker penetration test (BPT)) can often be accepted to laboratory tests because of important advantages such as cost, time effectiveness, the ability of soil assessment in its natural environment and its possibility to estimate the spatial variability of deposits [5].

SPTs and CPTs are generally preferred because of the more extensive databases and past experience, but the other tests may be applied at sites underlain by gravelly sediment or where access by large equipment is limited [6].

In the southern part of Iran, Shahid Rajaei Port construction is in progress which is designed as a diaphragm quay wall. Evaluating the probability of liquefaction of soil strata back the wall is very

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important since it causes large pressure on the wall and makes unexpected or damageable displacements.

This liquefaction-induced lateral spreading is usually restricted to the ground surface behind the retaining walls and thus it would be termed as localized lateral spreading. In this paper, the liquefiable soil layers are evaluated by two in-situ methods i.e. SPT and CPT, and are compared with each other.

SITE DESCRIPTION

The site is located in the south coast of Iran approximately 20 km southwest of “Bandar Abbas. The area consists of an approximately rectangular shaped zone with 1300 m in 480 m dimensions for extended container stacking yards and two regions of 910 m in 100 m for new berths where the quay walls are being constructed. This project is a part of development phases of the existing port (Shahid Rajaei Port). The site area has been previously reclaimed to the level of approximately +4.5mCD. The top level of the quay wall will be at +6.0 mCD.



Figure 1. Presentation of the reclaimed area of Shahid Rajaei Port

GEOTECHNICAL CONDITION

The geotechnical site investigation was performed in the site location. These activities consisted of exploratory boreholes with SPT and vane shear tests, cone penetration tests (CPT), dynamic probing tests. Generally there are five layers with fluctuating levels including (see Figures 2 and 3):

- 1) A layer of FILL MATERIAL (mixture of sand, silt and gravel) from reclamation activities.
- 2) A layer of silty/clayey SAND. The N-SPT values show that this layer is loose to medium dense.
- 3) A layer of fine grained soil including silt and clay. The upper part is the SILT layer which overlays a layer of CLAY.
- 4) The other layer of silty/clayey SAND is placed under the upper SILT/CLAY layer.
- 5) A layer of sandy/clayey SILT is encountered beneath the lower SAND layer up to the end of boreholes.

All these layers incline down toward the sea side, i.e. the land side level of layers are higher than the level of sea side.

LIQUEFACTION ASSESSMENT

Calculation of two variables is required for evaluation of liquefaction resistance of soils: (1) the seismic demand on a soil layer, expressed in terms of CSR; and (2) the capacity of the soil to resist liquefaction, expressed in terms of CRR. The first variable has been termed *the cyclic stress ratio* required to generate liquefaction, and has been given different symbols by different writers [7, 8 and 9].

The evaluation procedure for liquefaction assessment is divided into four parts:

- 1) Determination of soil types which are susceptible to liquefaction.
- 2) Calculation of Cyclic Stress Ratio (CSR) induced in soil by an earthquake.
- 3) Calculation of Cyclic Resistance Ratio (CRR, representing soil “strength”) based on in-situ test data from SPT and CPT tests.
- 4) Evaluation of liquefaction potential by calculating a factor of safety against liquefaction, F.S., by dividing CRR by CSR.

For the site region, two levels of ground motion for seismic analysis are chosen which are based on the Performance Based Design. According to hazard analysis, The Peak Ground motion Acceleration (PGA) for different design levels are estimated as below:

PGA Level 1=0.25g (Mw=6.5)

PGA Level 2=0.37g (Mw=7.0)

For liquefaction analysis, only the PGA in Level 2 is considered. The factor of safety against liquefaction for an earthquake of level 2 is considered as greater than 1.0 [10].

Determination of soil types susceptible to liquefaction

The so-called Chinese criteria, as described by Seed and Idriss (1983), are decided to be used in liquefaction analysis.

Cyclic Stress Ratio (CSR)

The earthquake strength demand is calculated using Seed-Idriss method, first introduced in 1971[6].

Cyclic Resistance Ratio (CRR)

Criteria for evaluation of liquefaction resistance (CRR) can be obtained from SPT and CPT measurements as follows.

CRR computation from SPT results

The Seed & Idriss method is applied to evaluate the CRR [1]. In this method, the N value of SPT is corrected with different factors such as hammer energy, borehole diameter, rod length and the effect of liner. Also, the effect of overburden pressure is considered. Finally, the CRR is calculated using the corrected N-SPT, which is modified according to fines content by equations developed by Idriss [6].

CRR computation from CPT results

For CPT, the method proposed by Robertson & Wride (1998) is applied [6, 12]. This method utilizes the soil behavior type index I_c . In this index, the effect of soil type, overburden pressure and friction ratio are considered. The effect of fines content can be found using I_c for modifying the tip resistance (q_c). Using the modified q_c , CRR can be estimated in a graph.

Factor of Safety

Dividing the CRR by CSR, the safety factor against liquefaction is calculated as follows:

$$S.F. = \frac{CRR}{CSR}$$

If S.F. is smaller than 1.0, the layer in the corresponding depth will be liquefied.

COMPARISON OF RESULTS

The geotechnical profiles and the liquefaction assessment results for two rows in eastern and western parts are presented in Figure 2 and 3. The numbers beside the boreholes indicate the raw values of N_{60} and corrected N'_{60} (considering the overburden depth) in the parentheses obtained from SPT. The N-SPT is not corrected for the fines soil layers. Also, the locations of CPT and the measurements, the values of the cone (on left side) and friction resistance (on right side), are presented. It can be seen that there is a good agreement between the SPT and CPT values in recognizing different layers, where as the N value and the cone of CPT reduces while reaching to the fines layers and in return, the sleeve friction augments. The stratification can be proved with observing the core samples obtained from the boreholes.

As shown, the levels of liquefied soil layers in two profiles of each side are similar to each other according to the correspondent methods. Generally, two liquefiable zones are identified in eastern side, but there is only one surface zone in the western side which is thicker than that of the eastern side. In this part, the deep liquefiable zone is not distinguished. The liquefaction hazard in the western side is larger than in the eastern side.

As stated before, the surface layer is made up of fill materials from previous dredging activities and is constructed about 25 years ago. Although it was expected that the fill layer (mixture of sand, silt and gravel) be liquefiable, the layer was found to be so compacted since it was impossible to push the cone of CPT easily and the pre-drilling was inevitable. So, there is no danger of liquefaction in the upper levels in the filled material.

In eastern side, the thickness of the upper zone towards the sea is about 3.0 meters being thicker and having the lower level towards the south. This layer is found out along all the quay wall line, but in the line behind it, only the southern part is probable to be liquefied with about 7 meters thick. In both lines, a liquefiable zone is found out according to CPT calculations. The levels of this zone are the same, but the thickness is much bigger (about 4 times) in the line toward the sea.

In western side, there is only one liquefiable zone in the upper sandy layer. The thickness of the zone in the sea side is about 10 meters, which is reduced to about 4 meters toward the south. Also, the level becomes down. The situation in the land side line is different, where the thickness and level of the liquefiable is various, but reduces from 7 meters in the north to about 4 meters in the south.

The important experience obtained here is that in both sides, it is observed that the upper level of the zone based on SPT method is higher than the level calculated based on CPT method. In return, the lower level zone based on CPT is calculated lower. In other words, the CPT method can recognize the liquefiable zone in the deeper layers in contrast to the SPT method, since the deep liquefiable zone in the eastern side is distinguished with CPT and not SPT.

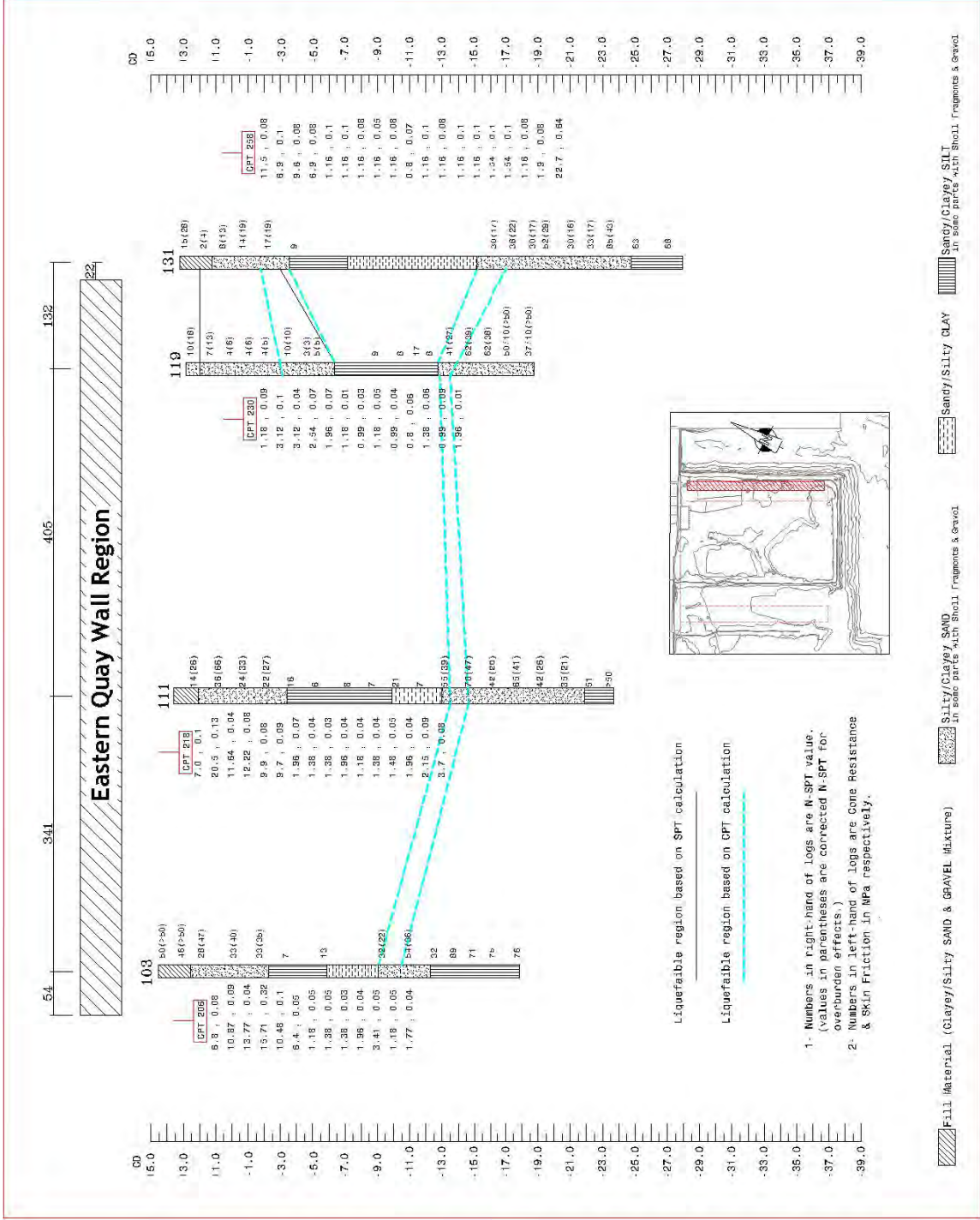


Figure 2 (a). Presentation of liquefiable layers based on CPT and SPT- Eastern side of the site (On the quay wall line)

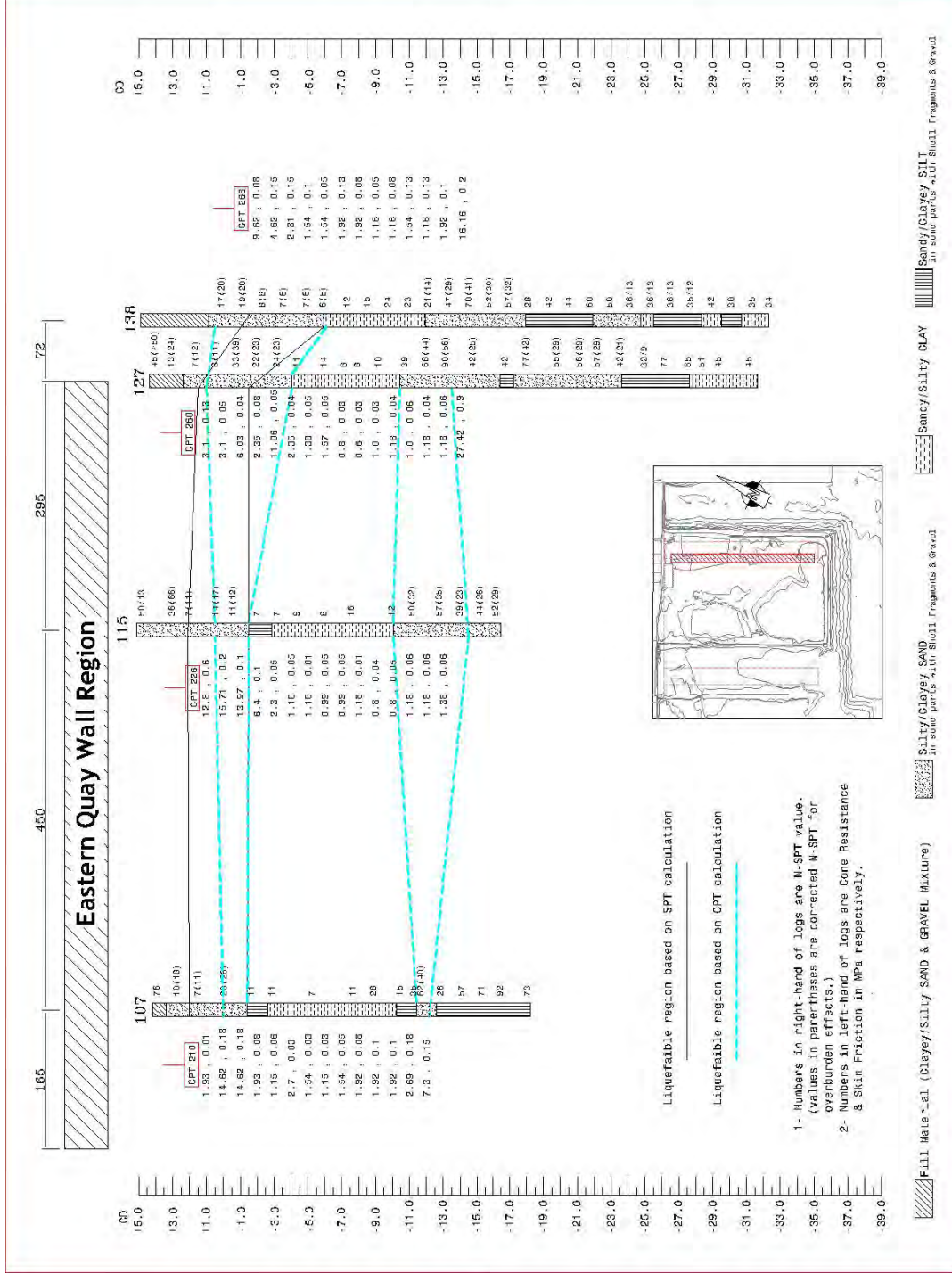


Figure 2 (b). Presentation of liquefiable layers based on CPT and SPT - Eastern side of the site (back the wall)

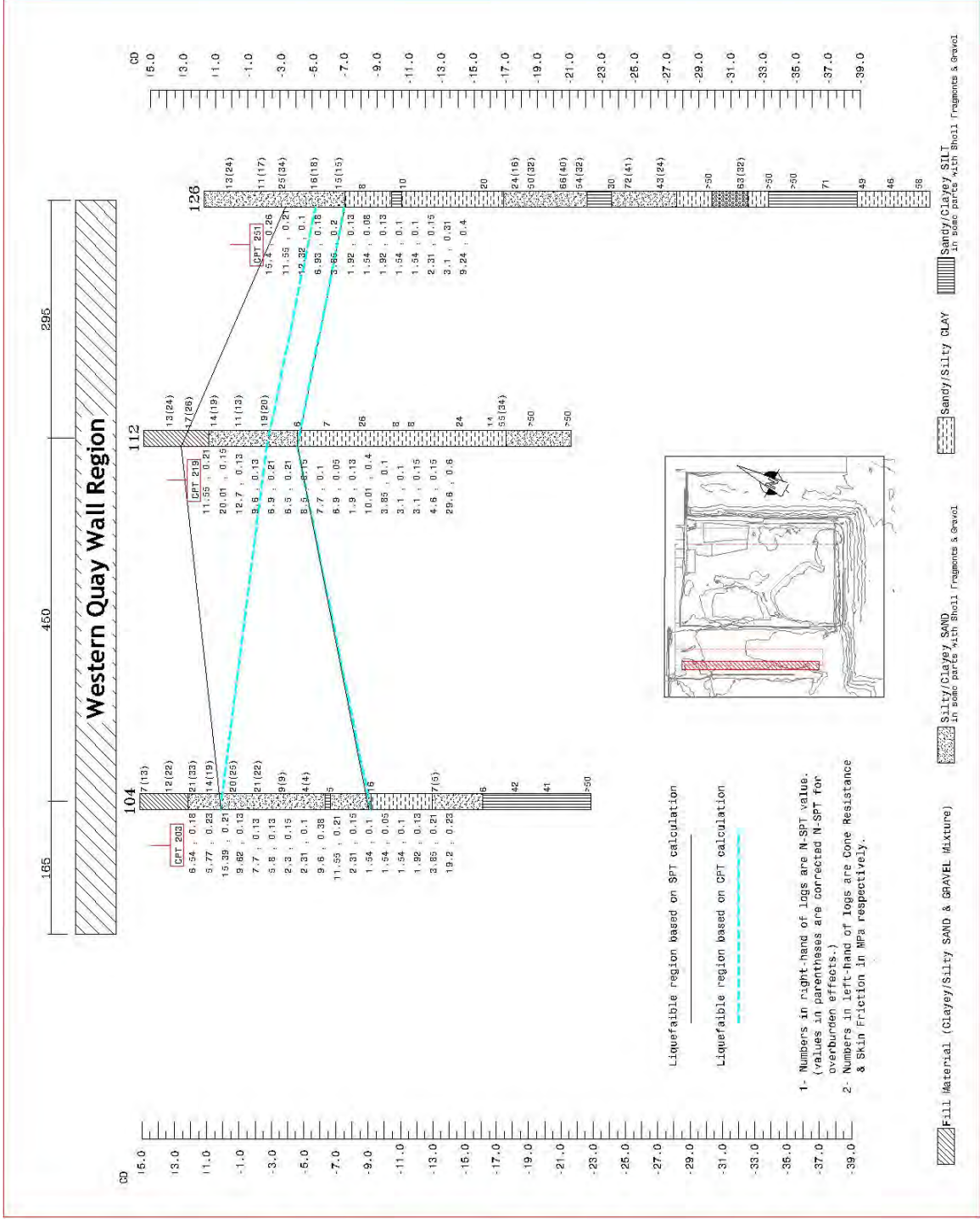


Figure 3 (a). Presentation of liquefiable layers based on CPT and SPT- Western side of the site (on the wall line)

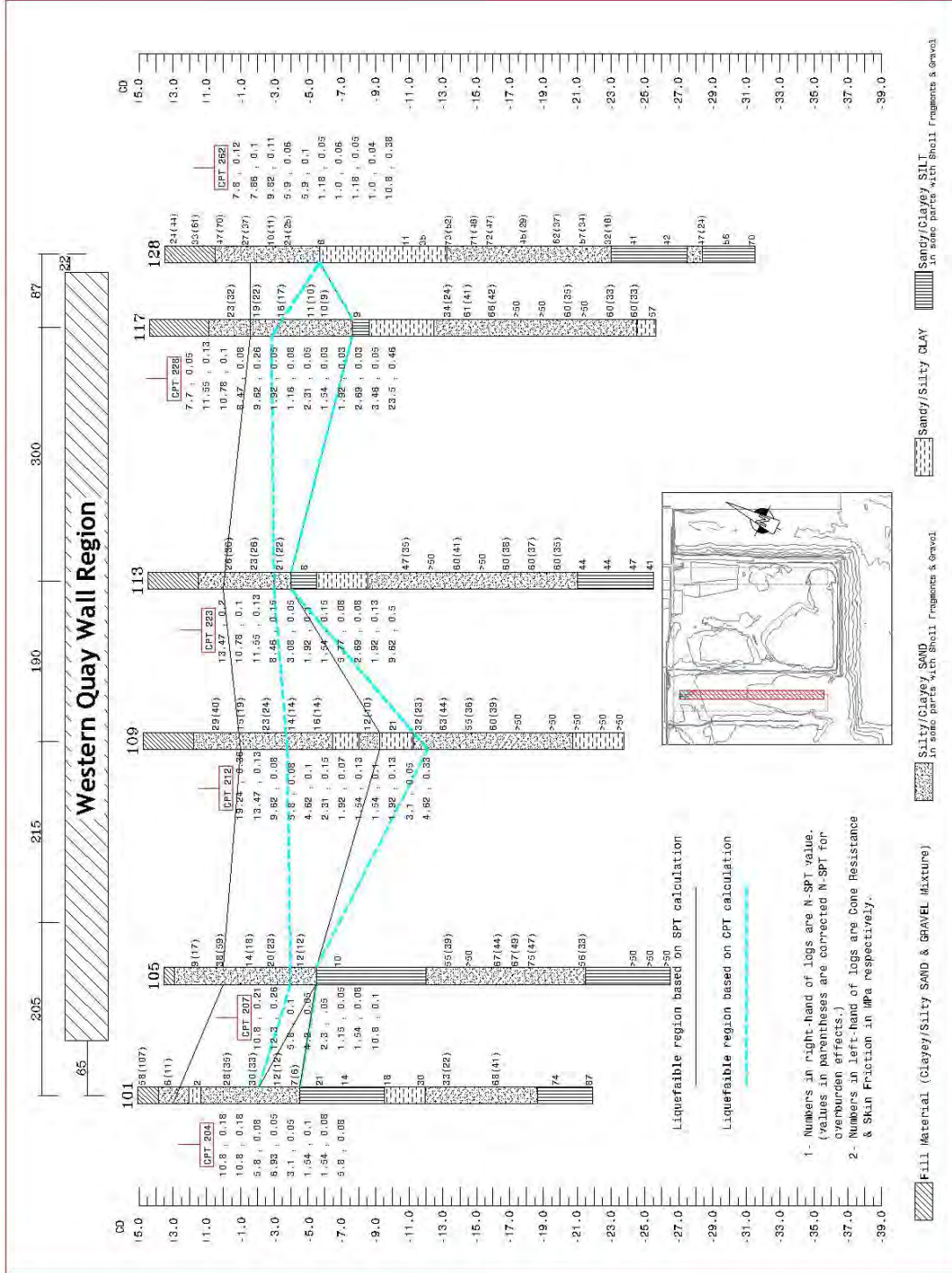


Figure 3 (b). Presentation of liquefiable layers based on CPT and SPT - Western side of the site (back the wall)

DETERMINATION OF DESIGN REQUIREMENTS

For assessment of minimum requisite of soil strength against liquefaction, the criterion based on the N-SPT, which is explained in the OCDI (2002), is applied. As stated in OCDI (Section 5.4.2), the improvement (such as dynamic compaction) shall be sufficient to increase the equivalent N-value such that the treated soil is in the range of above the line in the middle part of region III based on OCDI.

According to this procedure, the required equivalent N-values (N_{60}) for meeting the minimum criteria can be presented in three groups of soil based on the fines content of 5%, 10% and 15%.

As an alternative, it is possible to introduce a minimum q_c value for CPT tests based on SPT results. In the literature, there are some correlation between the value of N-SPT and the cone resistance (q_c) measured from CPT tests as follows:

$$q_c = C_f \times N_{SPT}$$

where C_f is a coefficient dependent on mean particle size of soil and the values can be found in Fang (1996) for different soil types. Figure 4 shows the minimum criteria for soil improvement based on the SPT value and the equivalent q_c for fines content of 5%, 10% and 15%. To use these graphs, one should compare the measured values of N-SPT or cone of CPT measured after the improvement process regarding the fines content and the depth of layer.

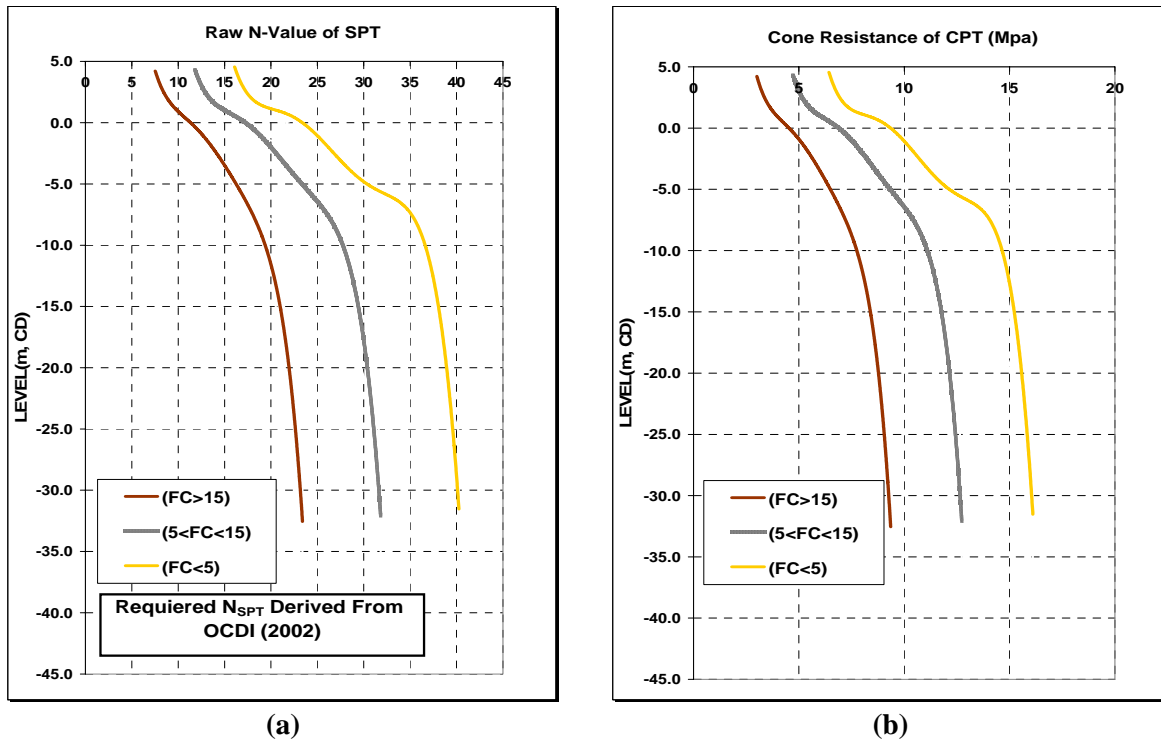


Figure 4. Minimum requirement of improvement criteria against liquefaction based on: (a) SPT value; (b) CPT value

DETERMINATION OF IMPROVEMENT DEPTH

For engineering purposes, it is not only the occurrence of liquefaction itself that is important, but its consequences such as the ground settlement and damage to adjacent structures should be evaluated. After recognizing the locations and depths of liquefiable layers, it is essential to improve the strength of the soil.

For the region just behind the retaining wall, it is necessary to improve all the depths back the wall to prevent large pressure on the wall due to liquefiable layer. It means that deep improvement is needed.

In this region, it is not possible to use dynamic compaction because the wall structure is not tolerable to the blows and the wall might be damaged. In turn, it will be possible to use the less risky improvement methods such as vibro-displacements or grouting. The soil improvement method in the region far from the wall can be different. In the latter region, it can be possible to use dynamic compaction which is an easy way. As an alternative, it is decided not to treat all the depths, but only a part of the upper layers in accordance to the criterion mentioned below.

In several geotechnical site investigations, a layer susceptible to liquefaction has been encountered. Although the layer was liquefied, the effect of liquefied layer was not seen on the ground at that region. That was due to the overburden non-liquefiable layers on the aforementioned layer. To decide whether liquefaction will or will not exert on the ground surface, the thickness of the liquefiable layer can be compared with the thickness of non-liquefiable surficial layers. Some curves are suggested by Ishihara (1985) for induced damage from liquefaction considering two liquefiable and non-liquefiable layers. If the thickness of the surface layer is larger than that of the underlying liquefied layer, resulting damage on the ground surface may be insignificant. This evaluation of the effects of liquefaction on the ground surface is stated in "Manual for Zonation on Seismic Geotechnical Hazards" (Revised Version, 1999).

Consequently, it is sufficient just to improve a portion of liquefiable layer (upper parts) not to allow the effects of liquefaction to the surface. For instance, using this criterion for the region behind the western quay wall, where the depth of liquefiable layer is up to 12 meters, improving the upper 8 meters depth will be adequate.

CONCLUSION

In this paper, the susceptibility of the soil layers against liquefaction in the region of Shahid Rajaei Port is examined by two different in-situ methods based on SPT and CPT. By comparing the liquefaction assessment results from two these methods, it seems that these two methods can figure out the liquefiable layers, but the layer based on CPT would be estimated lower in depth. On the other hand, one can find all the susceptible soil layers to be liquefied in a continuous way, in contrast to the laboratory tests where the probability of soil disturbance would also exist. In the following, the procedure stated in OCDI (2002) is applied for determination of required minimum criteria based on SPT to prevent liquefaction in underground layers. For the region just behind the quay wall, all the depths should be improved to prevent large pressure on the wall, but in the regions far from the wall, one can reduce the improvement depth using the Ishihara (1985) criterion which uses the effect of overburden pressure of the upper non-liquefiable layers.

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