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A CASE STUDY OF LIQUEFACTION ASSESSMENT USING SWEDISH WEIGHT SOUNDING

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ABSTRACT

Evaluation of liquefaction potential of soils is one of the important aspects of geotechnical earthquake engineering practice. After the publication of "simplified procedure" in 1971, it has become a widely used method of practice for liquefaction assessments. Several in-situ tests have been used for evaluating resistance of soils to liquefaction. The most common are SPT, CPT, V_s measurement and BPT. Swedish Weight Sounding test is a common in-situ test in some countries like Nordics, Japan and some east European countries. Also, It has been used in Iran for geotechnical design purposes in some projects. The paper presents a case study of using Swedish Weight Sounding results to assess liquefaction potential in Shahid Rajaei Port development in Iran, which is a very large development plan. The site consists of reclaimed sandy areas filled with dredged materials. An extensive geotechnical site characterization program has been undertaken in the project using different tests such as SPT, CPT, Dynamic Probing and laboratory tests, as well as Swedish Weight Sounding for research purposes. Regarding the case study of the mentioned project, the paper concludes on the validity of using Swedish Weight Sounding for liquefaction assessment.

Keywords: Liquefaction assessment, Low-cost site characterization, In-situ tests, Swedish Weight Sounding test (WST)

INTRODUCTION

Liquefaction can be defined as the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress (Marcuson, 1978). The possible damaging consequences of this phenomenon came to attention after two earthquakes of Good Friday (Alaska) and Niigata (Japan) in 1964. Since then, evaluation of liquefaction resistance of soils has become an important aspect of geotechnical engineering practice. Evaluation of soil resistance to liquefaction can be made using laboratory or in-situ test results. Several in-situ tests have been used for this purpose up to now. In the paper, Swedish Weight Sounding test (WST) has been considered as an in-situ test which can be used for liquefaction assessment.

LIQUEFACTION HAZARD ANALYSIS

Three aspects should be considered in a comprehensive evaluation of liquefaction hazards. These are susceptibility, initiation and effects of liquefaction (Kramer, 1996). Liquefaction susceptibility can be judged by several criteria such as historical, geological, compositional and state criteria. The geologic setting and geologic criteria provide very useful information for preliminary assessment of

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liquefaction susceptibility. Geologic processes that sort soils into uniform grain size distributions and deposit them in loose states, produce soil deposits with high liquefaction susceptibility (Pyke, 2003). Human made soil deposits, particularly loose fills such as hydraulically fills in which soil particles are loosely deposited by settling through water, are very likely to be susceptible to liquefaction. Experiences indicate that liquefaction with engineering consequences is largely happens in hydraulic fills and very recent alluvial and fluvial deposits (Pyke, 2003). Initiation of liquefaction refers to the phenomena of seismic generation of large pore-water pressures and consequent softening of granular soil. As a result, the assessment of liquefaction resistance is only illustrates that liquefaction initiates or not. In the paper we will assess the initiation of liquefaction of the studied area and assessment of post liquefaction phenomena such as residual shear strength, soil deformation and ground failure are out of the scope of this work.

SIMPLIFIED PROCEDURE

Background

Seed and Idriss (1971) developed and published a methodology named “Simplified Procedure” for evaluating liquefaction potential of soils. Since that time the procedure has become the most widely used method of liquefaction hazard assessments. The Simplified Procedure was developed from empirical evaluations of field observations and field and laboratory test data. The database of field evidences collected for this procedure were mostly surface evidences such as ground fissures, sand boils or lateral spreads which indicate that liquefaction was happened in a specific area. Data were collected mostly from sites on level to gently sloping grounds, underlain by Holocene alluvial or fluvial sediments at shallow depth (<15m) (youd et al., 2001).

The Methodology

The evaluation of liquefaction potential in cyclic stress approach, and consequently in simplified procedure as a cyclic stress approach-based method, simply performs by a comparison of loading and soil resistance throughout the mentioned soil deposit. In this approach, the earthquake loading characterizes by the amplitude of an equivalent uniform cyclic stress and liquefaction resistance by the amplitude of the cyclic stress required to initiate liquefaction (in the same number of cycles). By plotting the variation of equivalent cyclic shear stresses of an earthquake loading (τ_{cyc}) and the cyclic shear stress required to cause liquefaction ($\tau_{cyc,L}$), as Kramer (1996) denoted, throughout a soil strata in the same graph, the evaluation can be performed graphically. Liquefaction can be expected at depth, where the loading exceeds the resistance (Fig. 5). It should be noticed that the values of $\tau_{cyc,L}$ must correspond to the same earthquake magnitude, or same number of equivalent cycles as τ_{cyc}. Many engineers find it more convenient to characterize earthquake loading and liquefaction resistance in terms of Cyclic Stress Ration (CSR) and Cyclic Resistance Ratio (CRR), as Youd et al. (2001) denoted, which are both vary over a much smaller range than the cyclic shear stresses themselves. Based on their definitions, the CSR and the CRR are normalized cyclic shear stresses to the effective overburden stress. Also, the comparison can also be made through the concept of Safety Factor (FS) which defines as the ratio of resistance to loading, expressed as Eq. 1. As it is obvious, when FS becomes one or less, the soil will liquefy.

$$FS = \frac{\tau_{cyc,L}}{\tau_{cyc}} = \frac{\tau_{cyc,L} / \sigma'_{V0}}{\tau_{cyc} / \sigma'_{V0}} = \frac{CRR}{CSR} \tag{1}$$

Characterization of Earthquake Loading

Based on the cyclic stress approach, the fundamental reason for generation of excess pore pressure and consequently initiation of liquefaction are cyclic shear stresses which generate because of the Earthquake shakings. Thus, the earthquake loading characterized as a number of equivalent shear stress cycles. The amplitude of induced shear stresses is basically related to the intensity of earthquake shakings which indicates by peak ground acceleration of the earthquake. For estimating the shear stresses induced by earthquake in an element placed in depth of "h" of a soil deposit, a soil column can be assumed above the element. If the soil column assumed to be rigid, the maximum shear stress can be obtained by Eq. 2:

$$\tau_{\max} = \frac{\gamma h}{g} a_{\max} = \sigma_V \frac{a_{\max}}{g} = \frac{\sigma_V}{\sigma'_{V0}} . \frac{a_{\max}}{g} . \sigma'_{V0} \qquad (2)$$

In which γ is unit weight of the soil, h is the depth of the element, g is the gravity acceleration, a_{max} is the peak horizontal acceleration of the ground surface, σ_V and σ'_{V0} are total and effective overburden stress, respectively. But because of the flexibility of the soil profile, the real stress is less than which is estimated by Eq. 2. To account this flexibility Seed and Idriss (1971) proposed a coefficient termed "r_d" which decreases with depth. For routine practice and non-critical projects and for depth below 15m, the mean values of r_d can be chosen for calculations using Eq. 3 (Youd et al., 2001).

$$r_d = 1.00 - 0.00765h \qquad \text{for} \qquad 0 < h < 9.15m \qquad (3)$$

To convert the irregular time-history of shear stresses to a number of equivalent shear stresses, Seed and Idriss (1971) proposed the 65% of the value of τ_{max} as the average value of shear stress. Thus Eq. 4 can be used for calculation of the average cyclic stress. And by normalizing τ_{cyc} to the effective overburden stress (σ'_{V0}), the cyclic stress ratio (CSR) can be obtained by Eq. 5.

$$\tau_{cyc} = 0.65\tau_{\max} = 0.65 \frac{a_{\max}}{g} \sigma_V . r_d \qquad (4)$$

$$CSR = \frac{\tau_{cyc}}{\sigma'_{V0}} = 0.65 \frac{a_{\max}}{g} . \frac{\sigma_{V0}}{\sigma'_{V0}} . r_d \qquad (5)$$

Using SPT Results for Evaluation of the Liquefaction Resistance

Because of the many factors which influence the procedure of soil sampling and testing of the retrieved samples in the laboratory, obtaining acceptable results of liquefaction potential through the laboratory tests is normally difficult. Therefore, using in-situ tests become the routine practice for evaluating the liquefaction potential of soils. Several field tests such as SPT, CPT, V_s measurements and BPT have been used for liquefaction assessments. Each test has its advantages and disadvantages which have been discussed in the literature (Kramer, 1996; Youd et al., 2001). In the presented research, SPT-based criteria have been used for liquefaction assessment.

The SPT was the first in-situ test which was used for liquefaction assessment and criteria for evaluation of liquefaction resistance based on SPT have been verified and modified by large number of case histories. Although, because of low quality control during SPT tests and consequently poor repeatability of its results, the use of SPT for liquefaction assessment has been criticized (Robertson, 2004), but because of the accuracy of SPT-based liquefaction criteria using high quality data obtain acceptable results (as is done in the paper by using consistent correlations). The SPT data should modified for some important effects which are described as follows:

SPT Corrections

It has become obvious that in a same soil, SPT N-value will increase with depth. In fact, the increase of the effective overburden stress increases SPT N-value. Thus, an overburden stress correction factor (C_N) is applied to modify measured N-values (N_m). It has become common to normalize N_m to an overburden pressure of 100kPa. There are different relationships for estimating C_N (e.g. ENV 1997:3-

2000). The formula proposed by Liao and Whitman (1986) which is used commonly in engineering practice is used in the presented research (Eq. 6). Where σ'_{V0} is the effective overburden stress and C_N normalizes N_m to an effective overburden pressure (P_a) of approximately 100kPa (1 ton/sq ft). C_N should not exceed the value of 1.7 (Youd et al., 2001).

$$C_N = \sqrt{P_a / \sigma'_{V0}} \tag{6}$$

SPT results are also influenced by the amount of the energy, transferred from the falling hammer to the SPT sampler. For considering this effect, “Energy Ratio” (ER) is defined as the ratio between actual energy delivered into the drive rod immediately below the anvil, and the theoretical free fall energy of the hammer, expressed in percentage (ENV 1997-3:2000). As Seed proposed for the first time, based on approximate average of U.S. testing practice, an ER of 60% is generally accepted as a reference value for energy correction (Kramer, 1996). There are other corrections for parameters such as borehole diameter, rod length and liner of the sampler (Skempton, 1986), which are not used in the paper.

SPT Liquefaction Resistance Criteria

SPT liquefaction resistance criteria are normally expressed in figures in which CRR is plotted versus $(N_1)_{60}$ (Fig. 2). Where $(N_1)_{60}$ is the SPT blow counts, normalized to an overburden pressure of approximately 100kPa and a hammer energy ratio of 60%. These kinds of relationships between cyclic stress ratios causing liquefaction (CRR) and $(N_1)_{60}$ values in M_W=7.5 earthquakes was firstly proposed by Seed et al. (1985) (Fig. 3a). In the paper we use the modified correlations proposed by Youd et al. (2001) are used (Fig. 3b). As Fig. 3 indicates, the liquefaction resistance increases with the increase of SPT N-values. This increase continues up to $(N_1)_{60}$ =30 and for $(N_1)_{60}$ >30, the granular soils are too dense to liquefy and are classified as non-liquefiable.

An important point in the paper of Seed et al. (1985) was the effect of fines content on SPT resistance. Based on Fig. 3a, if the fines comprise more than 5% of the soil, the CRR increases with the increase of fines content. Thus in further calculations, the effects of fines content on liquefaction resistance are considered. It should be noted that other grain characteristics, such as soil plasticity, may affect liquefaction resistance as well as fines content. But laboratory tests indicate little influence at plasticity indices below 10 (Kramer, 1996). Most of sandy soils which are investigated in the research have plasticity indices less than about 10, so the effect of fines plasticity can be ignored.

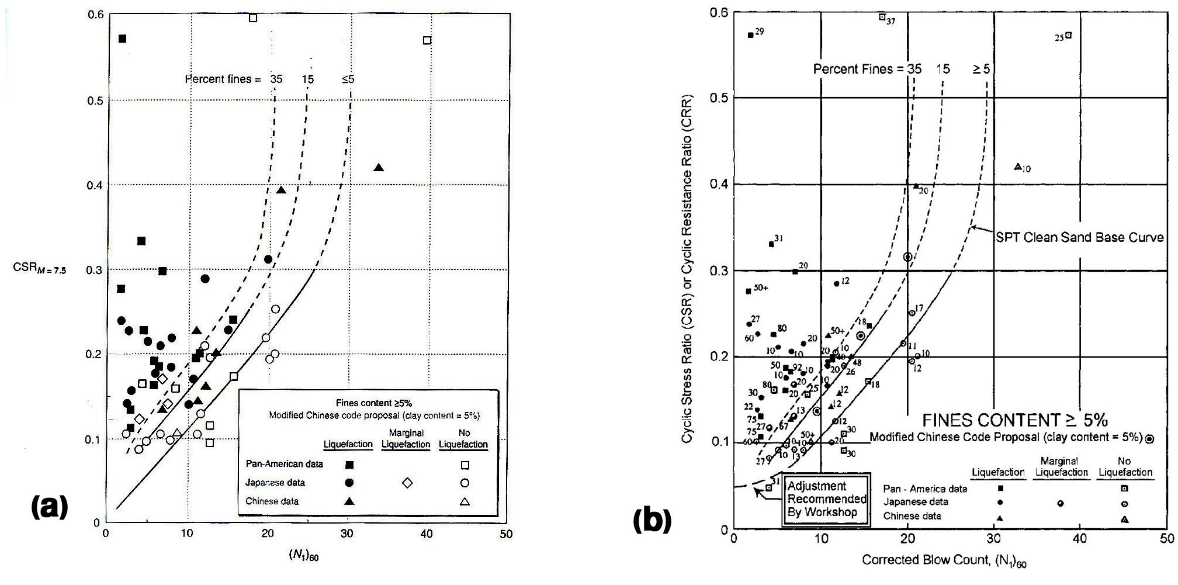

Figure 2. SPT-based resistance criteria [(a) after Seed et al. (1985), (b) after Youd et al. (2001)]

Magnitude Scaling Factors (MSF)

Earthquake magnitude characterize the duration of the ground shakings. Because strong-motion duration and consequently equivalent number of uniform stress cycles increases with earthquake magnitude, the CRR decreases with increasing magnitude. Because the typical CRR versus $(N_1)_{60}$ curves (Fig 2) are only applicable to earthquakes with M_W=7.5, "Magnitude Scaling Factors (MSF)" defined by Seed and Idriss (1982). Thus, the effect of earthquake magnitude on estimated liquefaction hazard can be determined by Eq. 7:

$$FS = \frac{CRR_M}{CSR} = \frac{CRR_M}{CRR_{7.5}} . \frac{CRR_{7.5}}{CSR} = (\frac{CRR_{7.5}}{CSR}).MSF \qquad (7)$$

Where CSR is the cyclic stress ratio generated by earthquake shakings, $CRR_{7.5}$ is the CRR for magnitude 7.5 earthquakes and CRR_M is the CRR for earthquake with magnitude as large as "M". In the paper, the MSFs proposed by Youd et al. (2001) is used which introduce a lower bound (Eq. 8) and an upper bound (Eq. 9) for Magnitude Scaling Factors. In this approach, the engineer can choose the MSF based on the level of the risk, suitable for the project.

$$MSF = 10^{2.24} / M_W^{2.56} \qquad (8)$$

$$MSF = (M_W / 7.5)^{-3.3} \qquad (9)$$

SWEDISH WEIGHT SOUNDING

Swedish Weight Sounding test (WST) is a common penetration test in Nordic countries and also in Japan. It is estimated that about 20,000 Weight Sounding tests are carried out yearly, only in Sweden (Broms and Flodin, 1988). The test has also used in some east European countries as well as countries like Singapore and Algeria (Bergdahl et al., 1988). Also Habibi et al. (2006) reported the use of WST for estimating the level of foundations and bearing capacity for some buildings in southern areas of Tehran, Iran.

Equipments of WST are consist of some pieces of weights (a 5kg clamp, two 10kg and three 25kg weights), a screw shaped point, 22mm extension rods and a handle (or a motor) for rotating the rods. The stated description illustrates that WST equipments are simple and can be easily transported. Fig. 3 shows a schematic view of the apparatus and its screw shaped point.

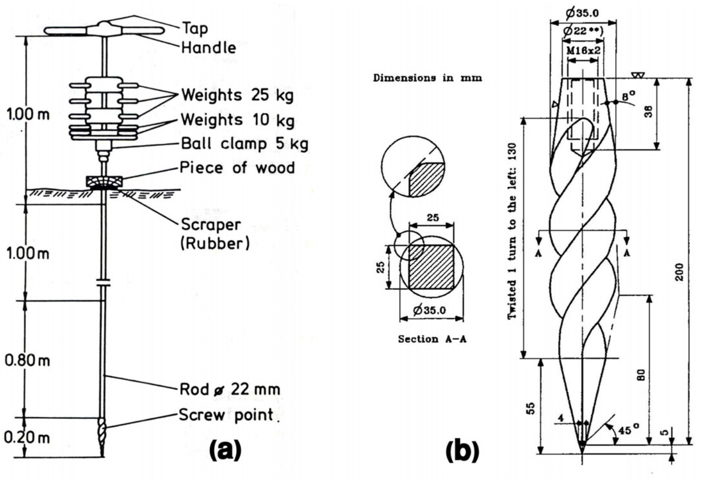

Figure 3. Swedish Weight Sounding (a) Schematic view of the Apparatus (Bergdahl et al., 1988)
(b) Screw shaped point (ENV 1997:3-2000)

Soil resistance measurements obtained from Swedish Weight Sounding method can be reported as W_{WST} or N_{WST}. When sounding is performed in a soft soil, penetration resistance is the weight required for penetration of the rods on the rate of 50mm/sec (ENV 1997:3-2000). It means that the weight increases up to the weight which obtains the mentioned penetration rate (W_{WST}). The levels of static loading used in this test are 0, 5, 25, 50, 75 and 100kg. If the penetration does not occur by 1kN loading, the apparatus is rotated and the number of half turns of rotations required for 0.2m of penetration (N_{WST}) indicates the soil resistance.

THE STUDIED SITE

Location

Shahid Rajaei Port Complex in south of Iran is one of the most important ports of Iran which plays a great role in export, import and transit of goods. Ports and Shipping Organization of Iran is planning the development of port facilities. It is a very large development plan which is aimed to achieve about 5 million TEU container terminal capacities. The development site is located in the western part of Shahid Rajaei Port Complex, situated near the town of “Khunsurkh”, approximately 20km south west of “Bandar Abbas” city, where is the center of “Hormozgan” province (Fig. 4). The geotechnical ground investigations of the project was one of the most extensive site characterization works in Iran. The site investigation program consisted of field works such as drilling of shore and marine boreholes, disturbed and undisturbed sampling, excavating trial test pits and performing a large number of in-situ tests such as SPT, CPT, CPTU, SCPTU, Vane Shear test, Pressuremeter test, Dynamic Probing etc., as well as conventional laboratory tests and sophisticated ones like resonant column test, Cyclic Triaxial tests etc.

Specifications of Investigated Strata

The area of project was rectangular in shape and approximately 85 hectares in size which is divided into two phases with an 1100*480m^2 area in phase I and remaining area for phase II. The phase II consists of two narrow rectangular parts in west and east of the phase I with dimensions 910*100m^2 and a narrow part on the North. Because the ground level of some parts of this project was below the sea level, they were hydraulically filled with dredged materials. Most of these hydraulically filled areas are located on the western zone of the project (phase II) where Swedish Weight Sounding tests were performed. In the mentioned area, the dredged materials were placed above a thin layer of soft rock which exists in most parts of the project. The depth of filled materials above the thin rock layer varies from 3.4m to 2.1m in different areas. The reclaimed area comprises of rather uniform fine sands which are loose and saturated in most of their depth. The level of ground water in the area is rather high and differs from 0.4m to 2.4 m from the ground level. There are also areas in the north of this narrow rectangular which comprise two layer, hydraulic fill in the bottom and a dry coarse sand fill on top. The liquefaction assessment was done for the both areas, with or without the upper fill layer.

Most of the soils in the studied site are saturated loose silty sands that are hydraulically filled. A fast drainage of the loose layer is not possible from the bottom during an earthquake, so it seems that these soils are potentially liquefiable. Thus, the liquefaction hazard assessment of the area is necessary.

LIQUEFACTION HAZARD ASSESSMENT OF THE AREA

Earthquake Characteristics

Peak horizontal acceleration (a_{max}) and magnitude of an earthquake (M_W) could indicate the intensity and the duration of the probable earthquakes, respectively. The Iranian Code of Practice for Seismic Design (Standard 2800-05) proposes the values indicated in Fig 4 for ground acceleration of different parts of the country. The seismic hazard map of Iran indicates that the Shahid Rajaei Port site is located in Zone 2 with high earthquake hazard (Fig. 4). Thus, the horizontal ground acceleration for design purpose in the area is 0.3g which was also confirmed by a site specific study (Sahel, 2005). In the research, the assessment is done for different levels of earthquake magnitude as M_W =7.5, 7 and 6.5. And for the upper and lower bound of Magnitude scaling factors for M_W =7 and 6.5. Therefore there will be five series of results for different levels of magnitudes and conservatism.

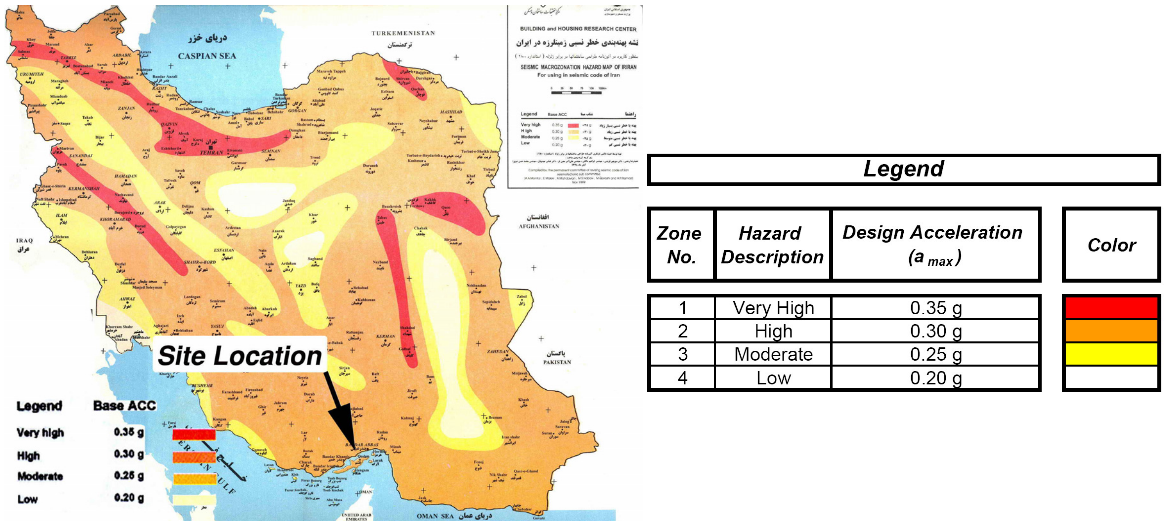

Figure 4. Seismic Hazard Map of Iran (Standard 2800-05)

Soundings

The Swedish Weight Soundings were undertaken in the filled areas which are all located in phase II of the project. Table 1 shows the depth of soundings and soil information such as soil classification, fines content and depth of water table. The information is obtained from boreholes drilled near each WST. Where a sounding was done between two boreholes, the data have obtained from interpolation. Because in-situ measurements for determination of the unit weights of the soil were not performed, the values of 16 and 19.5 KN/m^2 are assumed for unit weights of the soil above and below the ground water level, respectively.

Table 1. Specification of soil layers in Swedish Weight Soundings

Sounding No.	2	3	4	5	11	12	13	14	15	16	18	19
Penetration depth (m)	3.5	2.9	2.8	2.0	2.7	3.1	3.4	3.3	2.1	3.1	3.0	2.9
Depth of water table (m)	2.4	1.7	1.0	0.4	1.4	1.4	1.3	1.3	0.4	1.0	1.0	1.0
Soil Classification	SM	SM	SM	SM	SM	SM	SM	SM	SM	SM	SM	SM
Fines Contents	19	19	24	21	22	22	24	24	26	23	23	19

Because the presented work is a part of comprehensive research (Habibi, 2006), several soundings have been undertaken in every point. Therefore, the data which are used in analysis are average values of penetration resistance obtained from different soundings.

Soil Resistance

Swedish Weight Sounding test characterizes resistance of the soils as W_{WST} or N_{WST}. But these values should be converted to the equivalent results of other tests which are commonly used in liquefaction hazard assessments based on simplified procedure. There are several correlations between WST and SPT results, which are mostly presented by Japanese and Nordics. Because of the rather clearness of the amount of the energy transferred in Japanese SPT practice (e.g. Ishihara, 1993), Japanese correlations were preferred to be used in the presented research. Among the Japanese correlations, Inada (1960) used a robust database and covers almost every kind of soils. Inada's correlations are also proposed by Japanese code of practice for field surveys (JIS A 1221-1995) which the one for cohesionless soils are presented here.

$$N_{30} = 2.0W_{WST} + 0.335N_{WST} \qquad (10)$$

Where N_{30} is the SPT N-value for 30cm of split sampler penetration, W_{WST} is the WST weight required for penetration in kN and N_{WST} is the half turns of rotation, required for 0.2m penetration of rods. It must be noted that Eq. 10 is converted from the original form of the Japanese Code (1995) to the units of KN and ht/0.2m for weight and rotations, respectively. It is also obvious that, however, if the test is performed in rotation phase, the value of 1.0 should be substituted in Eq. 10 for W_{WST}.

SPT Energy Correction Factor

The original simplified procedure was based on $(N_1)_{60}$ which is the SPT N-value corrected for overburden stress and the energy transmission ratio of 60%. Ishihara (1993) pointed out that the value of 72% can be assumed as the average energy transmission ratio for Japanese common practice. Thus, we use the value of 72% was used for conversion of the N_{30} values obtained from Eq. 10.

Depth of Initiation of Liquefaction

After calculating the CSR and CRR, the depth where liquefaction happens can simply be found by drawing the variations of CSR and CRR in depth and see where the CSR is more than CRR (Fig. 5a). Also a comparison can be made by Safety Factor (FS) through the depth of the layer. Where the FS becomes less than one, liquefaction initiates during the earthquake (Fig. 5b).

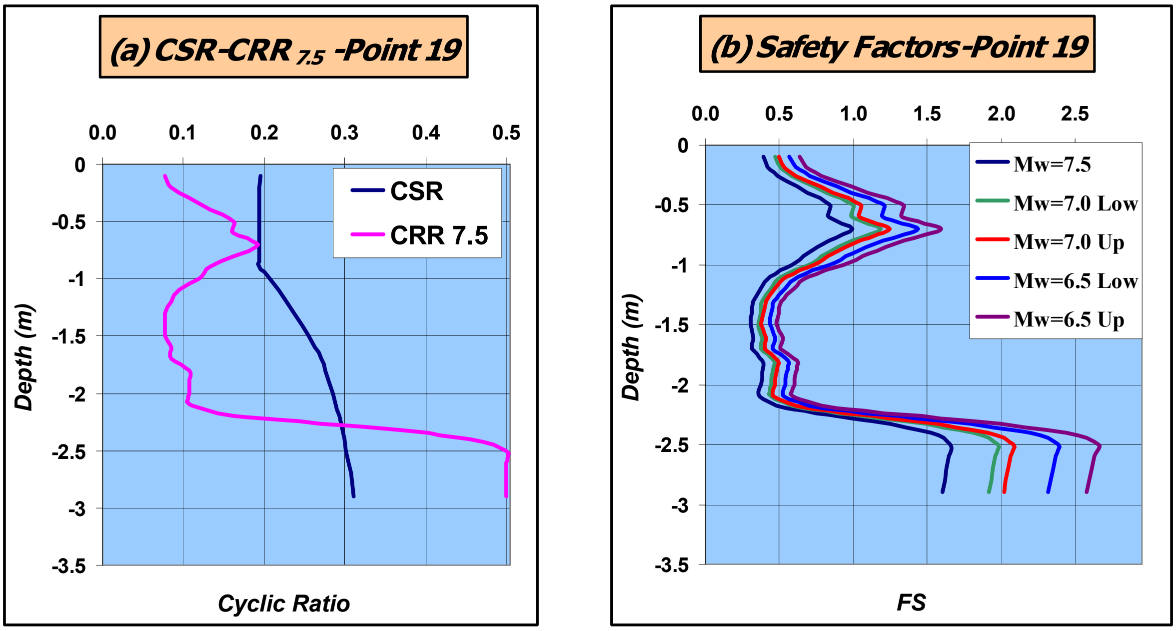

Figure 5. The typical process by which zone of liquefaction will be identified

Because the Analyses are done for different earthquake magnitudes and different levels of conservatism, several levels of liquefaction hazard are obtained. Table 2 shows typical calculations of cyclic shear stresses, liquefaction resistance and Safety Factor for different levels of earthquake magnitudes and their upper and lower bounds, e.g. for point 19 in depth of 0.7m to 1.3m. Based on the performed calculations, the depth of initiation of the liquefaction could be found for every point of penetration and for every earthquake magnitude. Table 3. illustrates the results of the liquefaction assessment of all penetration points. Also, based on these results a Zonation can be drawn for the investigated area, e.g. for lower bound of M_W=6.5, as shown in Fig. 6.

Table 2. Typical calculations of induces shear stresses, liquefaction resistance and the Factor of Safety

Depth	Field Data		Cyclic Stresses Calculations						Resistances Calculations							Safety Factors Calculations				
	W (Kg)	N (ht)	γ (KPa)	γ' (KPa)	σ_v (KPa)	σ'_v (KPa)	r_d	CSR	$(N)_J$	$(N)_{60}$	C_N	$(N_1)_{60}$	FC	$(N_1)_{60CS}$	$CRR_{7.5}$	$FS_{7.5}$	$FS_{7.0}$ Low	$FS_{7.0}$ Up	$FS_{6.5}$ Low	$FS_{6.5}$ Up
0.7		7	16.0	16.0	11.2	11.2	0.99	0.19	6.7	8.0	1.7	13.6	19	18.1	0.19	0.99	1.18	1.25	1.43	1.59
0.8		5	16.0	16.0	12.8	12.8	0.99	0.19	5.4	6.4	1.7	10.9	19	15.1	0.16	0.83	0.99	1.05	1.20	1.34
0.9		3	16.0	16.0	14.4	14.4	0.99	0.19	4.0	4.8	1.7	8.2	19	12.2	0.13	0.69	0.82	0.86	0.99	1.10
1		2	16.0	6.2	16.0	15.0	0.99	0.21	3.3	4.0	1.7	6.8	19	10.7	0.12	0.58	0.69	0.73	0.84	0.93
1.1	100		16.0	6.2	17.6	15.6	0.99	0.22	2.0	2.4	1.7	4.1	19	7.8	0.09	0.43	0.52	0.54	0.63	0.70
1.2	75		16.0	6.2	19.2	16.3	0.99	0.23	1.5	1.8	1.7	3.1	19	6.7	0.09	0.37	0.45	0.47	0.54	0.60
1.3	50		16.0	6.2	20.8	16.9	0.99	0.24	1.0	1.2	1.7	2.0	19	5.6	0.08	0.32	0.38	0.41	0.47	0.52

Table 3. Results of the liquefaction assessment (Depths of initiation of liquefaction [m])

Point No.	2	3	4	5	11	12	13	14	15	16	18	19
Mw=7.5	2.2~2.8	0~2.25	0~2.25	0~2.0	0~2.1	0~2.2	0~0.45 0.95~3.3	0~0.85 1.05~3.4	0~2.1	0~2.9	0~0.7 0.9~3.1	0~2.3
Mw=7.0 Up	2.25~2.75	0~2.2	0~0.6 0.85~2.3	0~2.0	0~0.65 0.9~2.1	0~0.8 1.0~2.2	0~0.4 1.05~3.3	0~0.85 1.1~3.4	0~2.1	0~0.85 0.95~2.9	0~0.6 1.0~3.1	0~0.6 0.8~2.25
Mw=7.0 Low	2.3~2.7	0~2.15	0~0.55 0.95~2.3	0~2.0	0~0.6 0.95~2.1	0~0.6 1.0~2.2	0~0.4 1.1~3.3	0~0.6 1.1~3.4	0~2.1	0~0.75 1.0~2.9	0~0.6 1.0~3.1	0~0.5 0.8~2.25
Mw=6.5 Up	2.35~2.65	0~2.1	0~0.4 1.2~2.25	0~2.0	0~0.5 1.0~2.1	0~0.5 1.05~2.2	0~0.35 1.2~3.3	0~0.45 1.2~3.4	0~2.1	0~0.5 1.1~2.9	0~0.5 1.0~3.1	0~0.4 0.9~2.2
Mw=6.5 Low	2.4~2.6	0~0.4 0.8~2.1	0~0.3 1.3~2.2	0~2.0	0~0.35 1.15~2.1	0~0.4 1.1~2.15	0~0.3 1.35~3.3	0~0.4 1.25~3.4	0~2.1	0~0.3 1.15~2.9	0~0.25 1.15~3.1	0~0.35 0.95~2.2

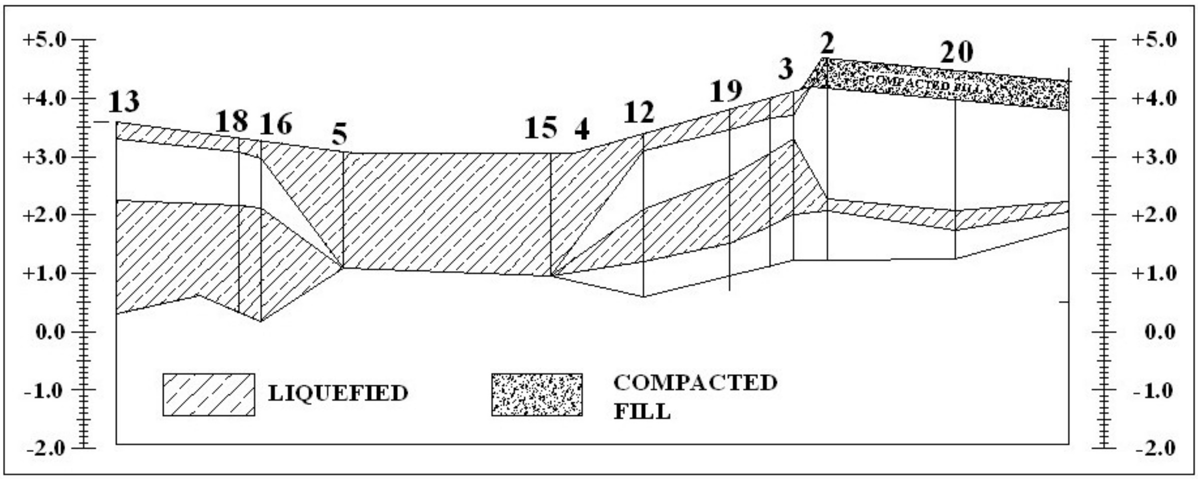

Figure 6. Liquefaction Zonation of the soil strata (Mw=6.5, lower bound)

An extensive assessment of liquefaction was also undertaken by Sahel (2005) using CPT and laboratory cyclic tests of samples. The results of liquefaction assessment presented in the paper (Table 3 and Fig. 6) are confirmed by the extensive study undertaken by Sahel (2005) and concluded that there is rather acceptable agreement between results of two assessment programs.

CONCLUSIONS

- The paper presented the use of WST through the correlations between WST results and SPT N-values to assess liquefaction potential in a large project. The results were confirmed by the results of other assessment for the studied site.

- Swedish Weight Sounding is a simple and low-cost in-situ test which could be used for rapid assessment of liquefaction potential in small projects which have a limited budget for geotechnical site characterization as well as preliminary assessments of large projects.

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