Comparison of SPT and DMT/SDMT for Liquefaction Potential Analysis of Soft Alluvial Soil at British Embassy in Yangon, Myanmar

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ABSTRACT: The city of Yangon, Myanmar is located in a seismic prone area and is therefore susceptible to potentially serious damage and loss as a result of earthquakes. With increasing high rise development now taking place in Yangon it is essential that geotechnical investigations for proposed structures include an assessment of liquefaction potential. This paper focuses on the use of the SDMT/DMT to estimate the horizontal stress index K_D. An updated K_D-CRR correlation is also presented, that may possibly predict the cyclic resistance ratio CRR with lower scatter than the conventional correlations based on the SPT number. It is widely recognized that stress history has a substantial influence on sand liquefaction and deformation behavior. The use of K_D for enhancing the accuracy of liquefaction assessments in sand is discussed.

1 INTRODUCTION

Myanmar is opening up and the new democratic government has enacted the Myanmar Investment Law inviting foreign investment in the nation. Investors from many countries are now showing an interest in Myanmar and the number of businessmen visiting Myanmar is soaring along with an influx of tourists. As a result, many infrastructure development projects are being, or will be undertaken, in Yangon, the gateway to Myanmar. During the development stage of a project, geotechnical investigations are required to assist with foundation design. The purpose of this paper is to inform related professionals, owners and authorities of the application of geo-technology to a site in Yangon.

A ground investigation survey using the SPT and DMT/SDMT was carried out by Mya Yar Pin Engineering Company to assess the potential for liquefaction at the site of the British Embassy in downtown Yangon. Downtown Yangon is known for its leafy avenues and fin-de-siècle architecture. The former British colonial capital has the highest number of colonial period buildings in Southeast Asia. The British Embassy is one of the buildings from this era and is a 4-storey mixed use (residential and commercial) building with 14-foot (4.3 m) ceilings. The embassy is situated on the east bank of Yangon River (Fig. 1).

2 SEISMIC ACTIVITIES OF YANGON

Yangon is the most populous and socio-economically important city in Myanmar. Unfortunately, it is located in a seismic prone area. Moreover, the seismogenic Sagaing fault is located

Fig. 1. Location of project site.

about 50 km away from downtown Yangon and has experienced several earthquakes in the past. The historical earthquakes around Yangon and its environs are shown in Fig. 2 (MEC 2011, Soe Thu Ya Tun 2004).

The past seismic activities show that Yangon is located in a moderate seismic prone area. According to the probabilistic seismic hazard map of Myanmar (Myo Thant et al. 2012), Yangon is located in a zone of peak ground acceleration generally in the range 0.11 g to 0.2 g, as shown in Fig. 3.

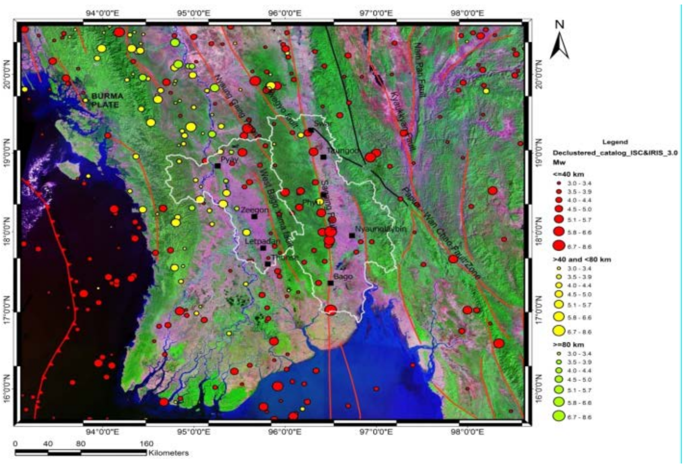

Fig. 2. Seismic activities of Yangon.

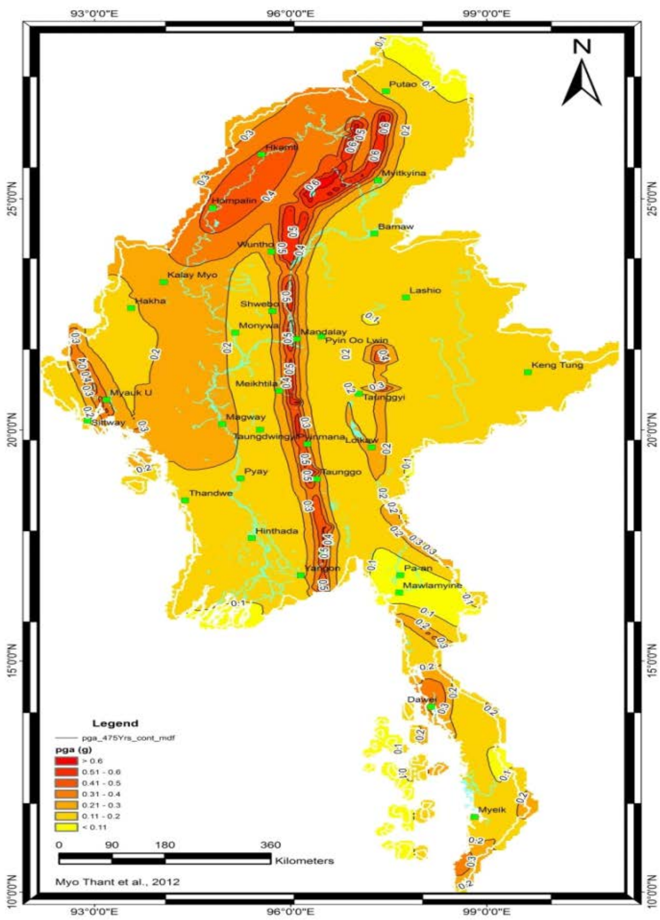

Fig. 3. Seismic hazard map of Myanmar.

3 GENERAL GEOLOGY OF YANGON

The general geology of Greater Yangon is shown in Fig. 4 (Mg Mg Khin 1980). Greater Yangon is characterised by Recent and Quaternary deposits that can be classified into four engineering geological units. They are: (1) Flat marine alluvial soil; (2) Valley filled alluvial soil; (3) Terraces and proluvial soil; and (4) Swamps and peaty alluvial soil.

Most of the townships in Yangon lie on the alluvial plain and are underlain by sand, silt, clay and gravel pockets, where strong motion and higher amplification of local sediments can be expected during earthquakes.

At the site of the British Embassy, valley filled alluvium mixed with marine deposits was encountered. The valley filled alluvium varied in texture, cohesiveness, water content and thickness of the strata in different locations.

4 LIQUEFACTION

Liquefaction occurs in saturated soils, mostly sands, when the strength and stiffness of the soil is reduced by earthquake or rapid load. If the sand is in a saturated condition and has a void ratio greater than the critical void ratio and is subjected to a suddenly

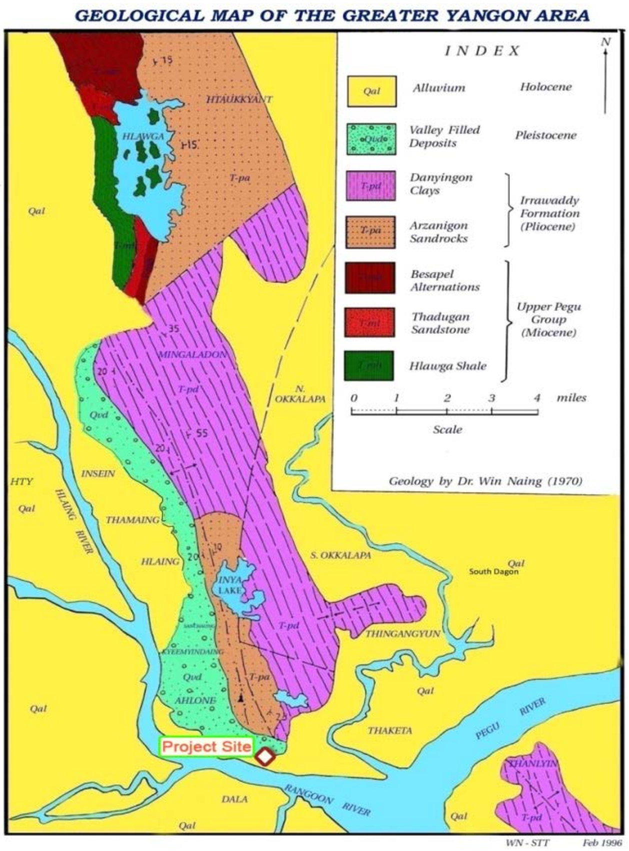

Fig. 4. Geological map of Greater Yangon Area.

applied shearing stress, as from an earthquake, heavy blasting, pile driving or any other dynamic force, the sand tends to decrease in volume. As a result the pore water is subjected to a suddenly applied excess pore water pressure and a portion of the weight of the overlying materials is transferred from inter-granular pressure to pore water pressure. The effective stress in the soil is reduced. Since the shear strength depends upon the effective stress, this transfer of pressure causes a sudden decrease in the shear strength and if this is reduced to a value below the applied shearing stress, the mass will fail in shear. Failure occurs suddenly, and the whole mass appears to flow laterally as if it were liquid.

The following conditions are likely to result in liquefaction: (1) low fines content (< 0.06 mm) of a saturated soil, (2) low SPT value of saturated sandy soil, (3) shallow ground water level and (4) large maximum peak acceleration.

5 STANDARD PENETRATION TEST (SPT)

5.1 *Test procedure and results*

The standard penetration test (SPT) is widely used throughout the world in many soil types. The test involves driving a split spoon sampler (outside diameter 50 mm, inside diameter 35 mm) using a 64 kg hammer falling for a distance of 760 mm. The number of blows required to drive the sampler for the last 300 mm is known as the SPT number.

SPT’s are typically carried out at between 1.0m and 1.5m intervals although closer spacing is feasible. Due to considerable differences in equipment and test procedure, significant variability of measured penetration can occur between different operators even on the same site. The repeatability of the test is questionable and the results are highly affected by drilling and sampling operations such as inadequate cleaning of the borehole, failure to maintain the hydrostatic pressure, variations in the driving of the hammer, worn equipment etc.

Notwithstanding these drawbacks, the SPT is commonly used to assess liquefaction potential.

5.2 *Liquefaction analysis based on SPT*

Liquefaction analysis in Yangon is usually performed by the method presented by Architectural Foundation Design Guideline, Japan Architectural Association (JAA 1977) based on SPT results.

According to past earthquake experiences, the looser the sediment, and the higher the water table, the more susceptible the soil is to liquefaction.

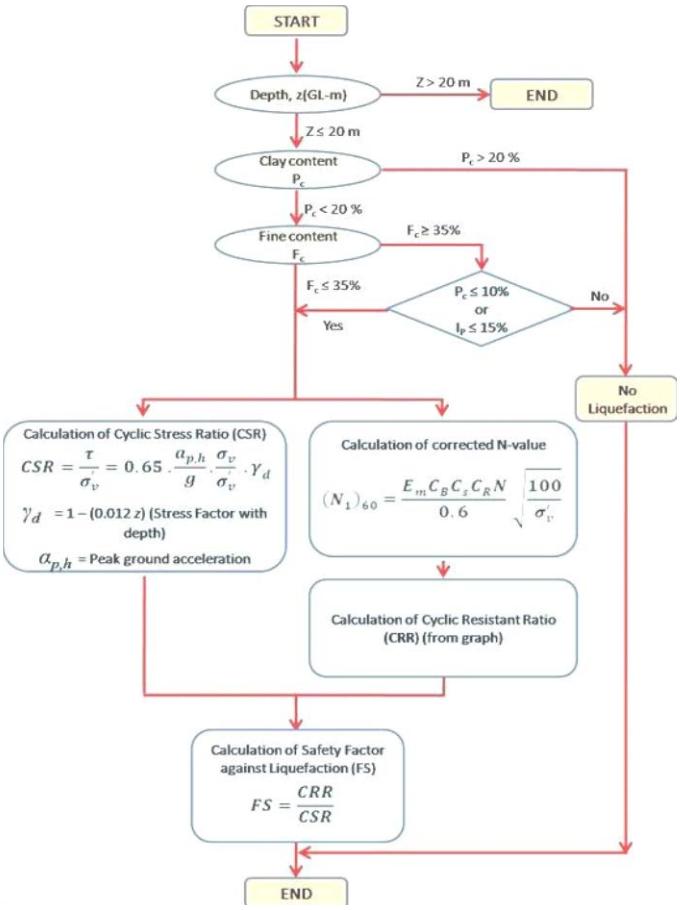

Fig. 5. General analysis procedure (SPT).

Liquefaction has been most prevalent in areas where the ground water lies within 20 m from the ground surface, primarily in deposited sands and silts. However, a few instances of liquefaction have occurred in soils with clay content greater than 35%. Therefore liquefaction analysis has been performed in such soils. The general procedure, described below, is shown in Fig. 5.

I. Identifying soil layers for liquefaction analysis:
1) Depth of soil layers: ground level ± 0 to 20 m,
2) Clay content Pc: Pc < 20%, and
3) Fines content Fc: Fc ≤ 35% (however, even if Fc > 35%, the soil which has Pc ≤ 10% or I_P ≤ 15% is still analysed for liquefaction.)

II. Calculation of Cyclic Stress Ratio (CSR):
According to Seed & Idriss (1971) and Srbulov (2008), the Cyclic Stress Ratio (CSR) for a magnitude M = 7.5 earthquake is calculated according to Eq. (1):

$$CSR = \frac{\tau}{\sigma'_v} = 0.65 \cdot \frac{a_{p,h}}{g} \cdot \frac{\sigma_v}{\sigma'_v} \cdot \gamma_d \qquad (1)$$

where $a_{p,h}$ = peak horizontal ground acceleration, g = acceleration due to gravity, σ_v = total vertical stress, σ'_v = vertical effective stress, z = depth from ground surface (GL) in metres, and γ_d = stress factor with depth = 1 – (0.012 z).

III. Calculation of Cyclic Resistance Ratio (CRR) or Liquefaction Resistance Ratio (R):
A measured SPT blow count N should be normalized to an overburden pressure of 100 kPa (Liao & Whitman 1986) and corrected to an energy ratio of 60% (the average ratio of the actual energy delivered by hammer to theoretical free-fall energy) as shown in Eqns (2) to (4):

$$N_{60} = \frac{E_m C_B C_S C_R N}{0.6} \tag{2}$$

$$(N_1)_{60} = N_{60} C_N \tag{3}$$

$$C_N = \sqrt{\frac{100}{\sigma'_v}} \tag{4}$$

where $(N_1)_{60}$ = corrected SPT N-value to an energy ratio of 60%, N = field SPT N-value, and σ'_v = effective overburden pressure.

The corrected SPT results from boreholes BH-E1 and BH-E2 at the British Embassy site are summarised in Table 1.

Table 1. Summary of SPT Results

Depth (m)	Borehole BH-E1		Borehole BH-E2	
	N	$(N_1)_{60}$	N	$(N_1)_{60}$
1	4	4.51	5	5.64
2	4	3.76	9	8.42
3	3	2.61	11	9.51
4	3	2.26	7	5.24
5	1	0.76	6	4.76
6	4	2.76	7	5.28
7	6	3.82	8	5.77
8	6	3.58	8	5.54
9	9	5.05	8	5.33
10	10	5.47	5	3.22
11	12	6.41	7	4.36
12	9	4.69	8	4.83
13	9	4.59	10	5.87
14	8	4.00	9	5.13
15	9	4.41	9	5.00
16	10	4.80	13	7.04
17	11	5.18	15	7.93
18	12	5.55	18	9.30
19	13	5.91	19	9.60
20	14	6.25	19	9.39

Table 2. Magnitude Scaling Factors

Magnitude (M_w)	5.5	6.0	6.5	7.0	8.0
Scaling Factor	2.86	2.20	1.69	1.30	0.67

The CRR for an earthquake magnitude of 7.5 can then be obtained from Fig. 6 or by using Eq. (5) (AGMU 2010):

$$CRR = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10(N_1)_{60}+45]^2} - \frac{1}{200} \tag{5}$$

For other earthquake magnitudes, the CRR values should be multiplied by the magnitude scaling factors indicated in Table 2 or in Fig. 7 (Ambraseys 1988, Eurocode 8-5).

The factor of safety (FS) against liquefaction is obtained from Eq. (6):

$$FS = \frac{CRR}{CSR} \tag{6}$$

Generally if FS >1.0 liquefaction will not occur. However, it should be noted that even if FS > 1.0, straining of the soil may occur due to the generation of excess pore water pressures by the earthquake.

6 FLAT DILATOMETER (DMT)

6.1 *Test procedure and results*

The flat dilatometer (DMT) is used in most of the world's industrialized countries and is standardized in ASTM and Eurocode 7.

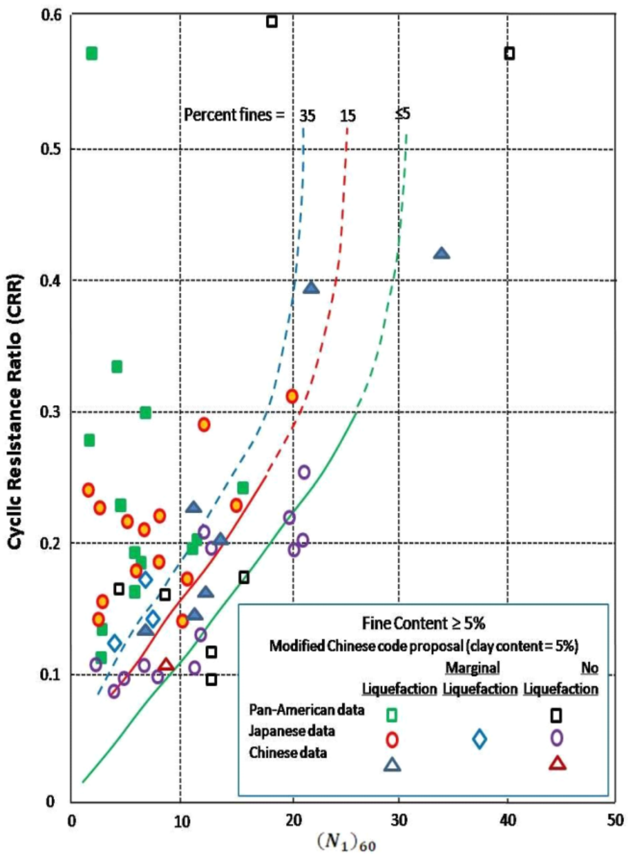

Fig. 6. CRR versus corrected N-value.

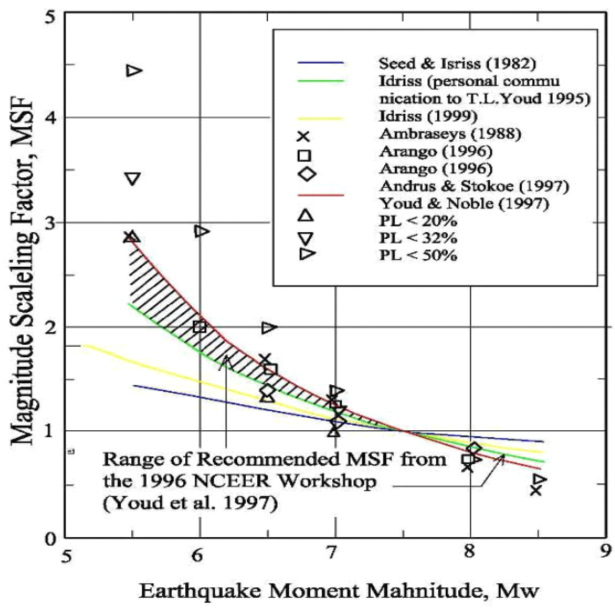

Fig. 7. Earthquake Magnitude Scaling Factors.

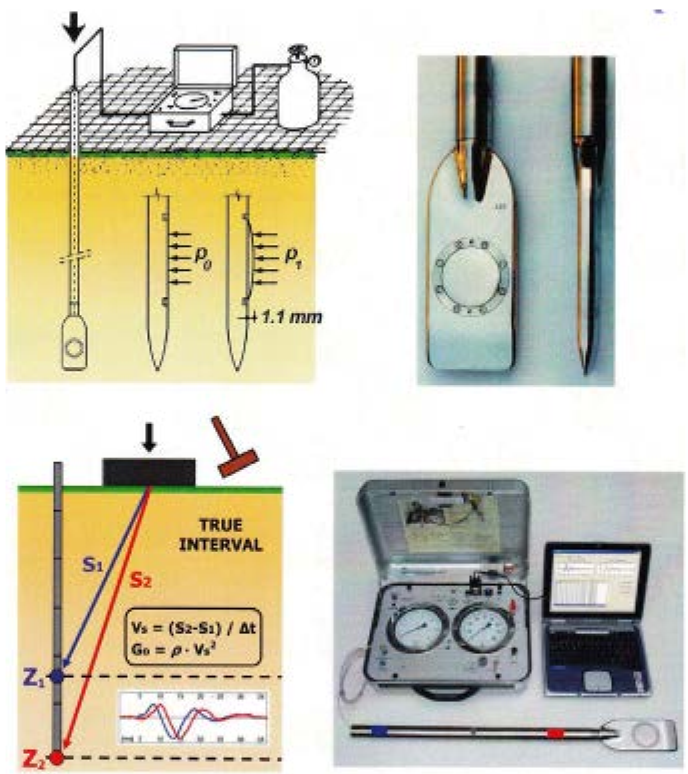

Fig. 8. Flat and seismic dilatometer.

The flat dilatometer (Marchetti 1975) consists of a steel blade, push rods, electric ground cable, control box, pneumatic cable and gas tank (Fig. 8).

The DMT is a penetration and load displacement test, not requiring a bore hole, to provide information on soil stiffness and strength.

The seismic dilatometer SDMT is a DMT which has been adapted to measure shear wave velocity (V_S).

The blade is jacked into the ground using a penetrometer rig. The blade is connected to a control unit on the ground by a nylon tube containing an electric wire. The tube runs through the penetrometer rods. Jacking is stopped at 20 cm depth intervals and, without delay, the membrane is inflated by means of pressurized gas. Readings are taken of the pressure to just begin to move the membrane (A) and of the pressure required to move its center 1.0 mm into the soil (B). The rate of pressure increase is set so that the expansion occurs in about 15 s. A penetration rate of about 2 cm/s is generally adopted.

The pressure readings A and B are corrected to take into account membrane stiffness and then corrected into p_0 and p_1 respectively. Eqns 7 to 9 are then used to calculate the intermediate parameters E_D (dilatometer modulus), I_D (material index) and K_D (horizontal stress index).

$$E_D = 34.7 \cdot (p_1 - p_0) \tag{7}$$

$$I_D = \frac{p_1 - p_0}{p_0 - u_0} \tag{8}$$

$$K_D = \frac{p_0 - u_0}{\sigma'_{v0}} \tag{9}$$

where u_0 = in situ equilibrium pore pressure and σ'_{v0} = effective vertical stress prior to blade insertion.

The SDMT/DMT results for SDMT No. 1 and SDMT No. 5 at the British Embassy site are summarised in Table 3.

Table 3. Summary of SDMT results

Depth (m)	SDMT 1			SDMT 5		
	I_D	K_D	E_D	I_D	K_D	E_D
1	-	4.5	-	1.19	4.6	3.9
2	-	4.3	-	0.79	4.3	3.9
3	-	2.6	-	0.95	6.5	10.6
4	-	3.3	-	0.94	5.0	11.1
5	0.68	3.2	5.7	0.99	3.9	11.5
6	0.98	2.6	8.1	1.02	3.3	12.2
7	1.18	3.0	12.8	0.90	3.3	11.8
8	1.07	3.0	12.5	1.13	3.1	14.9
9	1.24	3.0	15.4	1.57	2.7	19.1
10	3.24	2.0	29.3	1.14	2.4	13.4
11	1.57	2.7	19.9	0.97	2.6	13.0
12	1.26	2.2	13.9	1.71	2.3	21.1
13	1.33	2.3	16.2	1.44	2.3	18.4
14	2.38	2.0	26.2	1.91	1.9	22.2
15	1.75	1.9	19.6	1.92	1.9	22.4
16	1.58	1.9	17.5	1.50	1.6	16.2
17	1.27	1.7	13.8	2.43	1.5	25.2
18	1.51	1.6	15.8	1.22	1.6	14.2
19	1.68	1.5	17.0	2.47	1.4	25.0
20	1.63	1.6	18.3	2.15	1.5	24.5

6.2 *Liquefaction assessment based on SDMT/DMT*

The CRR (Cyclic Resistance Ratio) can be estimated from the horizontal stress index K_D as shown in Eq. (10) (Monaco et al. 2005, Monaco & Marchetti 2007):

$$CRR = 0.0107K_D^3 - 0.0741K_D^2 + 0.2169K_D - 0.1306 \quad (10)$$

The CRR can then be compared with the CSR (Cyclic Stress Ratio) calculated using Eq. (1) as described for the SPT analyses.

Where the CRR is less than the CSR, there is potential for liquefaction.

7 COMPARISON OF SPT AND SDMT

A comparison between the values from the SDMT and SPT is presented in Table 4.

A liquefaction analysis was carried out at the two boreholes (BH-E1 and BH-E2) to depth 20 m based only on the SPT results using the JAA (1977) approach. The results were as follows:

- Comparing the CSR and CRR (M = 6.5, 7.0, 7.5) at BH-E1 (Fig. 11), there is potential for liquefaction for M = 7.5 at depths between 2 m to 7 m and 12 m to 20 m, and for M = 7.0 between 2 m and 7 m, but none for M = 6.5.
- Comparing the CSR and CRR (M = 6.5, 7.0, 7.5) at BH-E2 (Fig. 12), there is potential for liquefaction for M = 7.5 at depths between 3 m and 17 m, and for M = 7.0 at depths between 9 m and 12 m and 13 m and 15 m, but none for M = 6.5.

Table 4. Comparison of SPT and SDMT Results

Depth (m)	Borehole BH-E1				Borehole BH-E2			
	N	I_D	K_D	E_D	N	I_D	K_D	E_D
1	4	-	4.5	-	5	1.19	4.6	3.9
2	4	-	4.3	-	9	0.79	4.3	3.9
3	3	-	2.6	-	11	0.95	6.5	10.6
4	3	-	3.3	-	7	0.94	5.0	11.1
5	1	0.68	3.2	5.7	6	0.99	3.9	11.5
6	4	0.98	2.6	8.1	7	1.02	3.3	12.2
7	6	1.18	3.0	12.8	8	0.90	3.3	11.8
8	6	1.07	3.0	12.5	8	1.13	3.1	14.9
9	9	1.24	3.0	15.4	8	1.57	2.7	19.1
10	10	3.24	2.0	29.3	5	1.14	2.4	13.4
11	12	1.57	2.7	19.9	7	0.97	2.6	13.0
12	9	1.26	2.2	13.9	8	1.71	2.3	21.1
13	9	1.33	2.3	16.2	10	1.44	2.3	18.4
14	8	2.38	2.0	26.2	9	1.91	1.9	22.2
15	9	1.75	1.9	19.6	9	1.92	1.9	22.4
16	10	1.58	1.9	17.5	13	1.50	1.6	16.2
17	11	1.27	1.7	13.8	15	2.43	1.5	25.2
18	12	1.51	1.6	15.8	18	1.22	1.6	14.2
19	13	1.68	1.5	17.0	19	2.47	1.4	25.0
20	14	1.63	1.6	18.3	19	2.15	1.5	24.5

A liquefaction analysis using the SDMT/DMT results found there was no potential for liquefaction at either location (Grasso & Maugeri 2006).

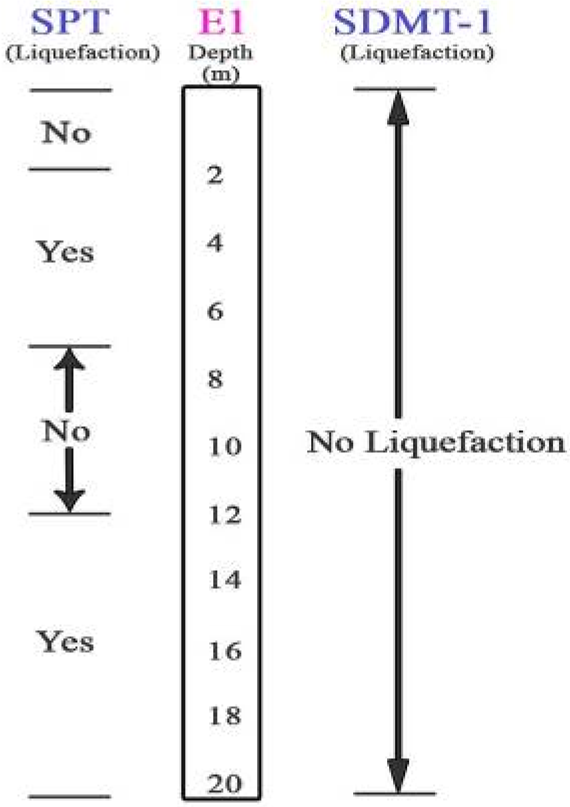

Fig. 11. Comparison of SPT, E1 & SDMT-1.

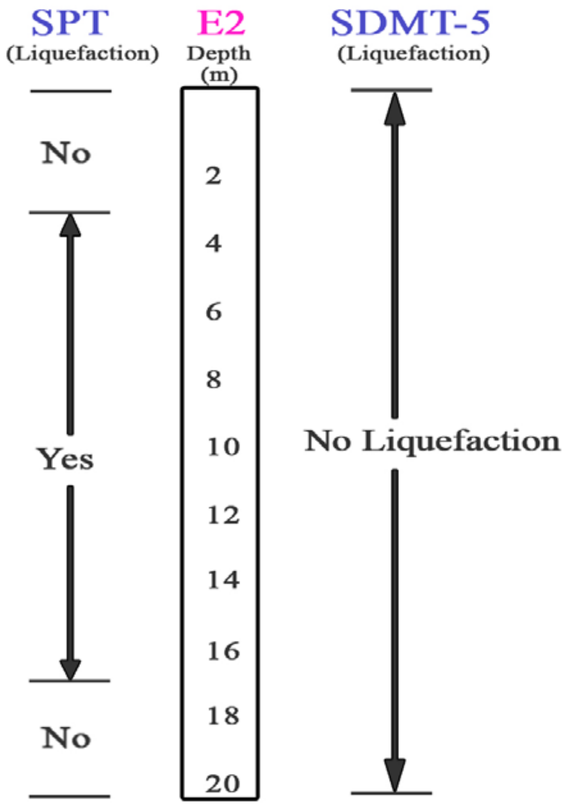

Fig. 12. Comparison of SPT, E2 & SDMT-5.

When the laboratory test results to measure the fines content of the soils were taken into consideration in conjunction with the SPT numbers, the JAA (1977) method also showed no liquefaction potential consistent with the SDMT findings.

8 CONCLUSIONS

The dilatometer in-situ testing device (DMT/SDMT) provides a relatively inexpensive, quick method to estimate a number of parameters used in geotechnical engineering works.

Results from the DMT/SDMT are expressed in terms of I_D, K_D, E_D and interpreted parameters, related to soil stiffness and other important soil properties.

The cyclic resistance ratio CRR can be calculated using correlations based on the horizontal stress index K_D.

On this basis, the liquefaction potential of the foundation soils can easily be assessed through comparison of the CSR and CRR.

By contrast, the assessment of liquefaction potential by the SPT method can take much longer due to the need to supplement the SPT numbers with the results of laboratory tests in order to confidently assess whether the foundation soils will liquefy or not.

In summary, the SDMT provides a relatively inexpensive and rapid method to assess liquefaction when compared with the SPT approach.

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