

# REVIEW OF LIQUEFACTION ASSESSMENT IN SANDS USING SEISMIC DILATOMETER TEST (SDMT)

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#### Abstract

Liquefaction is a complex phenomenon and the liquefaction potential of soil is affected by several factors such as aging, stress history, relative density, structure, bonding and confining stress. There are difficulties in conducting laboratory tests as reconstituted models can often be misleading due to difference in the fabric and structure of the actual soil. Hence strength assessment relies heavily on in-situ testing of soils. The simplified methods suggest using the correlations with the penetration resistance of some common tests i.e. Standard Penetration Test (SPT), Cone penetration Test (CPT) and Tests which estimate Shear Wave Velocity, for obtaining the Cyclic Resistance of soil. The Seismic Dilatometer test (SDMT) has been gaining popularity for its simplicity and repeatability. The DMT is suitable for liquefaction assessments due to its high sensitivity to stress history and aging of soil unlike other tests. The various correlations for assessing the factor of safety using the DMT as proposed by various researchers are discussed. Further the factor of safety by CPT and the DMT are compared at a liquefaction vulnerable area in Gujarat, India. The assessment of factor of safety is done as per the recommended method given in literature, also the liquefaction potential index is evaluated. The results indicate that this site is at a high risk of liquefaction.

Keywords: Liquefaction; Seismic Dilatometer Test; Cone Penetration Test; Horizontal Stress Index; Liquefaction Potential Index

## 1. Introduction

Liquefaction is a complex phenomenon which leads to the temporary loss of shear strength and stiffness of soil due to dynamic loading. The first step in order to mitigate liquefaction is to assess the soil's susceptibility to it. Liquefaction resistance depends on number of factors such as relative density  $D_r$ , earth pressure at rest  $K_0$ , stress and strain histories, aging, bonding and structure. It is difficult to conduct laboratory tests and to isolate the contribution of each of these factors on soil resistance. Remoulded samples obtained from sites and structure of the soil in-situ. It is also very difficult to obtain undisturbed samples, therefore in-situ testing methods are preferred to quantify the liquefaction resistance. The "Simplified Procedure" <sup>[1]</sup> is widely used for liquefaction assessment. This procedure incorporates the results of the most common and widely used penetrometer tests i.e. the standard penetration test (SPT) and the cone penetration test (CPT). Liquefaction resistance is indicated by means of cyclic resistance ratio (CRR) which is an indication of how close is the initial state of soil to the state of failure. It is evaluated from in-situ tests using the cyclic resistance ratio v/s penetration resistance correlation. Likewise the CRR can also be obtained from CRR v/s shear wave velocity (V<sub>S</sub>) correlation. The shear wave velocity is normally ascertained from seismic cone penetration test (SCPT) or crosshole test.

The seismic dilatometer test (SDMT) has applicability in liquefaction assessment because it correlates to the stress history and the age of soil. Unlike the penetration resistance used for correlating CRR, the DMT utilizes the Horizontal Stress Index  $K_D$ . This index is similar to at rest earth pressure  $K_0$  but has been distorted due to blade insertion. It has a profile similar to OCR and hence provides useful information about the stress history of the soil deposit. Stress history is an essential parameter for liquefaction assessment. Details of evaluating this parameter and other parameters obtained from this test are given in TC 16<sup>[2]</sup>. The SDMT can also



correlate the shear wave velocity  $V_s$  with CRR<sup>[3]</sup>. Albeit not being a very common test, its simplicity and repeatability are its advantages.

#### 2. Procedure for Liquefaction Assessment

stresses, respectively, and  $r_d$  is the stress reduction coefficient<sup>[4]</sup>

The simplified procedure <sup>[1]</sup> relies on the estimation of the cyclic stress ratio (CSR) and cyclic resistance ratio. The cyclic stresses that are generated in the ground are due to earthquake induced horizontal vibrations. There are changes in total vertical and horizontal stresses and pore pressure, but the effective stresses in vertical and horizontal direction are not affected. These horizontal cyclic stresses generated in the ground are normalized with the effective consolidation stress at a given depth to obtain the CSR. The cyclic stress ratio is calculated using the following expression <sup>[1]</sup>.

$$CSR = 0.65 \left(\frac{\tau_{max}}{\sigma'_{vo}}\right) = 0.65 \left(\frac{a_{max}}{g}\right) \left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right) r_{d}$$
(1)

Where,  $a_{max}$  is the maximum horizontal acceleration at ground surface level generated by the earthquake, g is the acceleration due to gravity,  $T_{max}$ ,  $\sigma_{vo}$  and  $\sigma'_{vo}$  are peak cyclic, total and effective vertical overburden

Since the cyclic stress time series involves many irregular cycles the same can be represented as a uniform cyclic stress time series with equivalent number of cycles that depend on the uniform cyclic stress amplitude. Hence a stress level equal to 65% peak cyclic stress is used to compute the CSR. <sup>[1]</sup> The stress reduction coefficient is introduced since the CSR equation is obtained assuming a rigid column of soil while the soil behaves deformable with the shear stress varying with depth.

The CRR used to be initially estimated using laboratory testing such as cyclic triaxial test. But due to the lack of undisturbed samples which could truly represent in-situ conditions, in-situ tests are found to be dependable and adopted as standard practice. The liquefaction resistance is empirically associated to the in-situ penetration resistance, which is considered as a fair representations of soil parameters such as density and pore pressure changes which affect liquefaction resistance. Based on past case histories of liquefaction, a plot of load applied versus the liquefaction resistance (CRR) is plotted.

After the CRR has been correctly scaled to the right magnitude of earthquake, the liquefaction potential is given as the ratio of CSR to CRR. If this ratio exceeds one it indicates the soil has liquefied. The equation for determining the CRR are given in equations 2,  $3^{[5]}$  and  $4^{[3]}$  using SPT, CPT and Shear wave velocity test respectively. The V<sub>s</sub> can be estimated from any seismic in situ test including the SDMT.

$$CRR_{M=7.5,\sigma'_{v}=1atm} = exp\left(\frac{\left(N_{1}\right)_{60}}{14.1} + \left(\frac{\left(N_{1}\right)_{60}}{126}\right)^{2} - \left(\frac{\left(N_{1}\right)_{60}}{23.6}\right)^{3} + \left(\frac{\left(N_{1}\right)_{60}}{25.4}\right)^{4} - 2.8\right)$$
(2)

$$\operatorname{CRR}_{M=7.5,\sigma'_{v}=1 \operatorname{atm}} = \exp\left(\frac{q_{c1Ncs}}{540} + \left(\frac{q_{c1Ncs}}{67}\right)^{2} - \left(\frac{q_{c1Ncs}}{80}\right)^{3} + \left(\frac{q_{c1Ncs}}{114}\right)^{4} - 3\right)$$
(3)

$$CRR_{7.5} = \left[ 0.022 \left( \frac{K_{a1} V_{s1}}{100} \right)^2 + 2.8 \left( \frac{1}{V_{s1}^* - K_{a1} V_{s1}} - \frac{1}{V_{s1}^*} \right) \right] K_{a2}$$
(4)

 $(N_1)_{60}$  is the SPT blow count at 1 atm pressure and normalized for 60% hammer efficiency. And  $q_{c1N}$  is the cone penetration resistance normalized at 1 atm pressure.  $V^*_{S1}$  is a limiting upper value of  $V_{S1}$  for liquefaction, it is assumed to change linearly as 200 m/s for soils with fines= 35 % - 215 m/s for soils with fines< = 5 %  $K_{a1}$  is a correction for high  $V_{S1}$  values due to aging,  $K_{a2}$  is a correction for the age and its influence on CRR. Before the resistance values can be used they are corrected for various factors for e.g. overburden, hammer



efficiency, rod length, borehole size, sampler, fines for SPT and overburden, fines and sometimes a thin layer correction for CPT<sup>[5]</sup>.

### 3. Dilatometer test (DMT) and Seismic Dilatometer (SDMT)

In 1975, a new type of penetrometer test, called as the Dilatometer Test (DMT), was developed by Prof. S. Marchetti to study lateral capacity of H-Piles. Its potential for diverse applications was soon understood and extensive research was done to study these applications and develop them. The DMT device consists of a small blade with a cutting edge to ensure minimum distortion during penetration and an expandable circular steel membrane. The blade is pushed into the ground at desired depth and this can be achieved with a CPT driving equipment. Pressure readings are taken at two instances i.e. when there is "lift off" (A reading) and when the membrane expands by 1.1mm (B reading). At times a third reading may be taken optionally, when the membrane deflates back (C reading). These pressure readings can then be used to obtain the material index (I<sub>D</sub>), horizontal stress index (K<sub>D</sub>) and dilatometer modulus (E<sub>D</sub>).The method of obtaining these parameters from A, B and C reading is given in TC  $16^{[2]}$ .Various Calibration tests <sup>[6]</sup>, <sup>[7]</sup> showed that K<sub>D</sub> parameter has found to be sensitive to the stress strain history of the soil and proved to be a key parameter in study of liquefaction assessment.

The SDMT is the DMT device with an additional seismic module, first introduced in 1988 <sup>[8]</sup>, subsequently underwent modifications <sup>[9] [10] [11] [12]</sup>. It consists of a tube element placed above the blade and has two receivers separated by 0.5m distance. An impulse is set on the ground surface using a hammer and the wave generated is propagated through the soil. Arrival of the wave at the top receiver first and then the bottom receiver is recorded by seismograms. The ratio between the distance separating the two receivers and the lag in the arrival of the waves at the two receivers gives the shear wave velocity ( $V_s$ ) at that point. The biggest advantage thus, for the SDMT is that, simultaneously two parameters can be estimated to assess the liquefaction resistance.

#### 3.1 Sensitivity of DMT to stress history and its application for liquefaction assessment

The SPT is the most widely used test and hence is the most obvious choice for liquefaction correlations. It provides soil samples with blow count but has limitations. It has poor repeatability due to different operators handling varied equipment to do the same test. This can lead to measurement errors and give highly variable N values for the same site. The CPT on the other hand has good repeatability, continuous profiling and well established liquefaction database but effects of penetration resistance on various factors cannot be isolated. Moreover CPT cannot capture the effects of stress history accurately nor aging of the soil with penetration resistance <sup>[13]</sup>. For reliable predictions regarding the liquefaction resistance of sands, the knowledge of the stress-strain history of the soil is essential. Hence it would require an equipment other than SPT and CPT which could furnish information about the stress-strain history of the soil <sup>[14]</sup>.

The dilatometer parameter  $K_D$  is found to be sensitive to stress history when compared alongside the cone tip resistance  $q_{c1}$ . Various researches have conducted calibration chamber tests <sup>[6]</sup> <sup>[13]</sup> to investigate effects of prestressing in soil. CPT and DMT were conducted before and after pre-stressing the soil along  $K_0$  line. It was observed that the cone penetration resistance increased the initial modulus by one order magnitude but there was considerable increase in  $K_D$  parameter, which indicated its high sensitivity to stress history.

Calibration chamber tests on Busan Sands<sup>[15]</sup> compared CPT and DMT test results in NC and OC sands. The  $q_c$ - $D_R$ - $\sigma_v$  relation and the  $E_D$ - $D_R$ - $\sigma_v$  relation clearly showed an insignificant influence on stress history whereas  $K_D$ - $D_R$ - $\sigma_v$  relation shows that the horizontal stress index increases with increasing stress history. This confirms that  $K_D$  factor is more sensitive to stress history than the normalized tip resistance of cone penetration test. The reason for the insensitivity of  $q_c$  to stress history can be associated to the distortions caused by the advancing penetrometer into the soil<sup>[13]</sup>.

A very important effect of aging with respect to liquefaction is that using CRR from correlation which do not consider the effect of aging in soil can underestimate the CRR by 60%. This is equivalent to ignoring an important parameter in computations<sup>[16]</sup>.



Simultaneous CPT and DMT sounding were done to record cone resistance and horizontal stress index in the Enel Milano Calibration Chamber Study<sup>[17]</sup>. The parameters were found before and after cyclic pre-straining where vertical and horizontal stress was increased by keeping  $K_0$  as constant, then both increases were removed and initial stress state was achieved for testing. Five such cycles were conducted to imitate the process of aging where grains slip into stable configurations which would otherwise take many years. It was found that increase in  $K_D$  was approximately 3 to 7 times than in cone penetration resistance, showing that it is more sensitive to aging than penetration resistance.

Similarly the Treporti embankment in Venice <sup>[18]</sup> was instrumented and tested at various stages of its construction. The embankment was completed in 6 months and surcharge was applied for 4 years before it was removed. CPT and DMT sounding done before, immediately after construction and after removal of surcharge showed that  $K_D$  was sensitive to aging and stress history while the CPT resistance and the shear wave velocity profiles showed negligible influence on these parameters.

Based on the above discussion it is evident that aging and stress history significantly affect liquefaction resistance and the DMT parameter  $K_D$  is essentially a key parameter in determining liquefaction resistance. Many researchers have put forward correlations of DMT parameters with CRR. The next section discusses in depth about these correlations.

#### **3.2 Evaluation of CRR from DMT parameters**

The very first correlation for liquefaction assessment was given as reported in equation 5.Based on some existing experimental data <sup>[19]</sup> a plot of liquefaction based on stress ratio was obtained and the equation was suggested as follows <sup>[13]</sup>,

$$\frac{\tau}{\sigma_{vo}'} = \frac{K_{\rm D}}{10} \tag{5}$$

Later field and laboratory tests were conducted in Sands by many researchers<sup>[20][21]</sup>. The relations given are based on both laboratory and in-situ testing results.

A correlation based curve was proposed later as given in equation 6 <sup>[22]</sup>. The existing CRR-  $(N_1)_{60}$  and CRR -  $q_{c1}$  correlations were changed to CRR -  $K_D$  correlation using  $D_R$  as an intermediate parameter. The  $D_{R-q_{c1}}^{[23]}$  and  $D_R-(N_1)_{60}^{[24]}$  and the  $K_D-D_R$  correlations<sup>[21]</sup> were used to give the CRR- $K_D$  correlation.

$$CRR=0.0107K_{\rm D}^3-0.0741K_{\rm D}^2+0.2169K_{\rm D}-0.1306$$
(6)

The limitation of this relationship is the transformation uncertainty that is associated with correlating penetration resistance with horizontal stress index. But overall it gives a tentative demarcation of approximately how the liquefaction plot would look like when compared to existing database for CRR with SPT and CPT values. The equation was validated for some sites at Loma Preita after the 1989 earthquake.

Another correlation<sup>[25]</sup> was proposed by conducting side by side DMT , SPT and CPT tests to derive relationship between  $K_D$  with SPT  $(N_1)_{60}$  and CPT resistance  $q_c.K_D-q_{c1}$  and  $K_D-(N_1)_{60}$  relationships were established as follows:

$$N_{1.60} = 0.185 K_{\rm D}^3 - 2.75 K_{\rm D}^2 + 17 K_{\rm D} - 15$$
<sup>(7)</sup>

$$q_{c1} = 0.4K_{\rm D}^3 - 2.75K_{\rm D}^2 + 56K_{\rm D} - 20 \tag{8}$$

Using these established relationship a relationship between  $K_D$ -CRR from existing CRR -( $N_1$ )<sub>60</sub> and CRR-q<sub>c1</sub>relationship was derived as follows

$$CRR = exp\left[\left(\frac{K_{\rm D}}{8.8}\right)^3 - \left(\frac{K_{\rm D}}{6.5}\right)^2 + \left(\frac{K_{\rm D}}{2.5}\right) - 3.1\right]$$
(9)



The curves were validated for some Sand samples and this correlation indicated that previous correlations could underestimate liquefaction potential. Based on the correlations and data points<sup>[25]</sup> an empirical correlation between  $q_{c1}$  and  $K_D$  for sandy soils ( $I_D$ >1.2 and 2< $K_D$ <6) was suggested as follows<sup>[26]</sup>:

$$q_{c1} = 25K_D \tag{10}$$

Based on equation 10 the CRR-K<sub>D</sub> correlation was updated using CPT correlation for CRR<sup>[27]</sup> as follows:

$$CRR=93(0.025K_{\rm p})^3 + 0.08 \tag{11}$$

Later the relation between CRR and  $q_c$ <sup>[5]</sup> was updated using equation 3<sup>[26]</sup> to obtain a CRR-K<sub>D</sub> relationship<sup>[28]</sup>. Hence a combined correlation is obtained by taking a geometric average between both the CRR- $q_c$  and CRR-K<sub>D</sub> relation. This is the recommended correlation to be used<sup>[28]</sup> for important sites where one has to rely on more than one in-situ test for liquefaction assessment.

Average CRR=
$$[(CRR \text{ from } q_c)X(CRR \text{ from } K_D)]^{0.5}$$
 (12)

A relationship between  $V_{s1}$  and  $K_D$  was also established <sup>[29]</sup>. This relation is site specific, but it showed a large scatter indicating that Vs and  $K_D$  cannot be interchanged to estimate CRR, both give different estimates. Moreover  $V_S$  is less sensitive to stress history but sensitive to cementation, hence  $K_D$  correlations are preferred for liquefaction assessment. For computing CRR from Vs measurement equation 4 <sup>[3]</sup> is used.

### 4. In-situ testing at Mundra

Mundra is a port town located in the state of Gujarat in India. The Mundra port handles 110 Million Metric Tons of cargo per year which is highest cargo handling in India. It lies in the Kutch region which falls in zone V as per the classification for seismic vulnerability done by IS 1983:2002<sup>[31]</sup>. The Gujarat region has experienced several earthquakes in the past, the most destructive being the Bhuj earthquake of 2001. Sand boils were formed indicating this region experience liquefaction<sup>[30].</sup> Ports serve as lifeline commodities hence they should be safeguarded against liquefaction failures when they lie in areas having high seismic vulnerability.

#### 4.1 Factor of safety against liquefaction

Side by side CPT and DMT were carried out at a site on this port  $(22.76^{\circ}N \text{ and } 69.65^{\circ}E)$  to assess the liquefaction susceptibility. The factor of safety against liquefaction was evaluated from CPT<sup>[5]</sup>, from DMT<sup>[26]</sup> <sup>[28]</sup> and also from combined CPT and DMT using equation  $12^{[28]}$ . The ground acceleration was ascertained as per IS code provision for zone V as 0.36g for a magnitude of earthquake  $7.7^{[31]}$ . The CPT (q<sub>c</sub>) and DMT (K<sub>D</sub>) profiles recorded at site are shown in figure 1 a and b

The ground water table was found at 1.9 m below the ground surface and the factor of safety was evaluated from the 1.9 m onwards. Due to the presence of a Clay layer from 2.4 m to 5m, with PI=26(based samples recovered at the test site), the procedure used was not applicable and hence no factor of safety has been calculated for this depth interval. Beyond 5 m the factor of safety by all the three methods has been estimated. The factor of safety is interpreted as given in table  $1^{[32]}$ 

FS>13	Non Liquefaction
15/1.5	Non Liquendenon
1 < FS < 1.3	Moderately liquefiable
1 < 1 5 1.5	Moderatery inquenable
0 < FS < 1	Critically Liquefiable
0<15<1	Cinically Liquenable
1	

Table 1: Classification of factor of safety<sup>[32]</sup>



Figure 1: a) CPT and b) DMT test parameters used for liquefaction calculations with soil profile

#### **4.2 Liquefaction potential Index**

The Liquefaction potential index <sup>[33]</sup> is a measure of the severity of liquefaction at a given site. This factor is useful as it can analyze the risk of liquefaction at a given site, based on the factor of safety computed. It is given as

$$I_{L} = \int_{0}^{20} Fw(z)dz$$
(13)

Where F=1-FS when FS<1 and F=0 when FS $\ge$ 1, z= depth below ground surface in m and w (z) = 10-0.5z; is the depth weighting factor. The following classification, (see table 2)<sup>[33]</sup> gives the risk of liquefaction at a given site. Since the test soundings were available from 8.4 m onwards, It has been considered that the portion below this depth is non liquefiable and hence LPI =0 at depth >8.4m.

Table 2	: LPI Classif	ication for Liquefaction Risk <sup>[33]</sup>
	I DI	Line for the Disla

LPI	Liquefaction Risk
0	Very Low
0-5	Low
5-15	High
>15	Very High



## 5. Results

As it can be seen from figure 2 a. the factor of safety is <1 below 5m depth i.e. the soil is critically liquefiable. A small thickness of soil layer is safe against liquefaction as per the DMT test but susceptible to liquefaction as per CPT. Figure 2b indicates the liquefaction potential index for the test site. The values of liquefaction potential index in all cases is > 15 which indicates that the risk of liquefaction is very high.



Figure 2: a) Factor of safety against liquefaction and b) LPI

## 6. Conclusions

The dilatometer is a very robust equipment, giving logs at every 200mm along with the shear wave velocity. But it has certain disadvantages such as the soil profile obtained is not continuous, pore pressures cannot be recorded and the test progress is slower as compared to the CPT.

The  $K_D$  parameter has high sensitivity to stress history, aging, relative density which affect liquefaction resistance, hence it has a potential application in liquefaction assessment. There are no corrections for overburden since the parameter is normalized, hence making computations easier. However  $K_D$ -CRR correlations which have been developed over the years are based on correlations with other in-situ tests, so for more realistic assessment, there is a need for development of  $K_D$ -CRR correlations from case histories. Also due to lack of fines corrections for DMT, at present the application of this method is limited to clean sands only.



The factor of safety from DMT is higher as compared to the CPT - this could be due to the fact that a  $q_c$ - $K_D$  correlation has been used. This can be assessed in a more systematic manner with  $K_D$ -CRR correlations from actual case histories. The in-situ test done at Mundra indicates that the site has high risk of liquefaction from both the tests. The LPI from the CPT and the DMT are in close range to each other. This is well justified given that this area has experienced a similar magnitude of earthquake with occurrence of liquefaction, the chances are very high that this site may liquefy again. A small thickness of 0.4 meters is liquefiable as per the CPT but not as per DMT. However since the crust layer above it being 2 m, this small layer possesses low risk of liquefaction related damages. For better assessments at this site more number of CPT and DMT tests would be recommended.

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### 8. References

- Seed, H. B., and Idriss, I. M. (1971). "Simplified procedure for evaluating soil liquefaction potential." J. Soil Mech. and Found. Div., ASCE, 97(9), 1249–1273
- [2] TC16 (2001), the Flat Dilatometer Test (DMT) in Soil Investigations, A Report by the ISSMGE Committee TC16. May 2001, 41 pp. Reprinted in Proc. 2nd Int. Conf. on the Plant, Proc. ASCE Spec. Conf. on Use of In Situ Tests in Geotechnical Engineering In Situ '86, Virginia Tech, Blacksburg. ASCE Geotech. Spec. Publ. No. 6, 985-1001.
- [3] Andrus, R.D. & Stokoe, K.H., II. (2000). "Liquefaction resistance of soils from shear-wave velocity". *Jnl GGE*, ASCE, 126(11), 1015-1025.
- [4] Idriss, I. M. (1999). An update to the Seed-Idriss simplified procedure for evaluating liquefaction potential, in Proceedings, *TRB Workshop on New Approaches to Liquefaction*, Publication No. FHWARD-99-165, Federal Highway Administration, January.
- [5] Idriss, I. M., and Boulanger, R. W. (2008). *Soil liquefaction during earthquakes*. Monograph MNO-12, Earthquake Engineering Research Institute, Oakland, CA, 261 pp.
- [6] Lambrechts, J.R., Leonard, G.A. (1978)"Effect if stress history on deformation of sand." J. Geotech Engg, 104 (11), 1371-1381.
- [7] Jamiolkowski, M., Robertson, P.K. (1988) Closing address," Future trends for penetration testing, *Penetration testing in the UK*." Thomas Telford, London, pp.321-342.
- [8] Hepton, P. (1988) "Shear wave velocity measurements during penetration testing." *Proc. Penetration Testing* in the U.K., ICE, 275-278.
- [9] Martin, G.K. and Mayne, P.W. (1997)"Seismic flat dilatometer tests in Connecticut Valley Varved Clay", ASTM Geotech. Testing Jnl, 20(3), 357-361.
- [10] Martin, G.K. and Mayne, P.W. (1998)"Seismic flat dilatometer in Peidmont residual soils", *Proc. 2<sup>nd</sup> Int. Conf. on Flat Dilatometer*, Washington D.C., 295-305.
- [11] Mayne, P.W., Schneider, J.A. and Martin G.K. (1999)"Small and large- strain soil properties from seismic flat dilatometer tests" *Proc.* 2<sup>nd</sup> Int. Symp. On pre-failure Deformation Characteristics of Geomaterials, Torino, 1,419-427.
- [12] Marchetti, S. (2008)"In Situ Test by Seismic Dilatometer (SDMT)", ASCE Geot. Special Publication GSP No. 170 Schmertmann Volume.
- [13] Marchetti, S. (1982)"Detection of liquefiable sand layers by means of quasi static penetration probes." GED, 104, GT11:1371-1387.





- [14] Jamiolkowski, M., Baldi, G., Bellotti, R., Ghionna, V. and Pasqualini, E. (1985) "Penetration resistance and liquefaction of sands." *Proc. XI ICSMFE*, Balkema, Rotterdam, the Netherlands 1891-1896.
- [15] Lee, M., Choi, S., Kim, M., Lee, W. (2010)"Effect of stress history on CPT and DMT results in Sand." J. Engg. Geology 117(2011) 259-265.
- [16] Leon, E., Gassman, S.L., Talwani, P. (2006)" Accounting for Soil Aging When Assessing Liquefaction Potential." J.Geotech.Geoenv. Engrg. ASCE, 132(3), 363-377.
- [17] Jamiolkowski, M., Lo Presti, D. (1998)"DMT research in Sand, What can be learned in calibration chamber tests, IN: Robertson P, K., Mayne P.W. (Eds). Proc. Of first Int. conf. on site characterization, Balkema Pub, Rotterdam, Oral Presentation.
- [18] Marchetti, S. (2010)"Sensitivity of CPT and DMT to stress history and aging in sands for liquefaction assessment." *CPT 2010 Int, Symposium*, Huntington Beach, California.
- [19] Vaid, Y.P., Byrne, P.M. and Hughes, J.M.D. (1981), "Dilation Angle and Liquefaction Potential", *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 107, No. GT7
- [20] Robertson, P.K. & Campanella, R.G. (1986). "Estimating LiquefactionPotential of Sands Using the Flat Plate Dilatometer". *ASTMGeotechn. Testing Journal*, 9(1), 38-40.
- [21] Reyna, F. &Chameau, J.L. (1991) "Dilatometer based Liquefaction Potential of SIes in the Imperial Valley ". 2<sup>nd</sup> Int. Conf. on recent Advances in Geot. Earthquake Engrg and Soil Dyn. St. Louis. May 7 pp.
- [22] Monaco, P., Marchetti, S., Totani, G., and Calabrese M., (2005)" Sand liquefiability assessment by Flat Dilatometer Test (DMT)" Proc. 16<sup>th</sup> ICSMGE-Osaka.
- [23] Baldi, G., Bellotti, R., Ghionna, V., Jamiolkowski, M. & Pasqualini, E. (1986.) "Interpretation of CPT and CPTUs. 2nd part: Drained penetration of sands". *Proc. 4th Int. Geotech. Seminar*, Singapore, 143-156.
- [24] Gibbs, K.J. & Holtz, W.G. (1957). "Research on determining the density of sands by spoon penetration testing". *Proc. IV ICSMFE*, 1, 35-39.
- [25] Tsai, P.H., Lee, D.H., Kung, G.T.C. and Juang, C.H. (2009) "Simplified DMT vased methods for evaluation liquefaction resistance of soils." *Engineering Geology*, Vol. 103, No. 1-2, pp. 13-22
- [26] Robertson, P.K. (2012). "Mitchell Lecture. Interpretation of in-situ tests some insight." *Proc. ISC-4*, Porto de Galinhas Brazil, 1, 3-24.
- [27] Robertson, P.K. & Wride, C.E. (1998)." Risk-based investigation" Geotechnical News; 45-47, September 1998
- [28] Marchetti, S. (2016)"Incorporating the Stress History Parameter K<sub>D</sub> of DMT into the Liquefaction Correlations in Clean UncementedSands" *Jnl GGE*, ASCE
- [29] Maugeri M. & Monaco P. (2006) "Liquefaction Potential Evaluation by SDMT" *Proc. Second Introl Conf. on the Flat Dilatometer*, Washington D.C., P. 295-305 Apr.
- [30] Bardet, J.-P., Rathje, E. M., and Stewart, J. P., principal authors, (2002). Chapter 9: Liquefaction, Chapter 8: Ports. *Bhuj, India Earthquake of January 26, 2001 Reconnaissance Report, Jain, S. K., Lettis, W. R., Murty, C. V. R., and Bardet, J. P., eds., Earthquake Spectra, Supplement A to vol. 18, 101–30.*
- [31] IS 1893: Part 1 (2002) Criteria for earthquake resistant design of structures—Part 1: General provisions and buildings. *Bureau of Indian Standards*, New Delhi.
- [32] Mhaske SY, Choudhury D (2011) Geospatial contour mapping of shear wave velocity for Mumbai city. Nat Hazards 59:317–327
- [33] Iwasaki, T., Tokida, K., Tatsuoka, F., Watanabe, S., Yasuda, S. and Sato, H. (1982). "Microzonation for soil liquefaction potential using simplified methods." *Proc. of 3rd Int. Conf. on Microzonation*, Seattle, 3, 1319– 1330.