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An innovative low-cost SDMT marine investigation for the evaluation of the liquefaction potential in the Genova Harbour, Italy

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ABSTRACT: A new container quay namely "Calata Bettolo", will be built inside the Genova harbour. The length of the quay is of about 750 m and the area is of about 22,000 m^2. The depth of water will reach 17 m along the quay. The material to fill the area of the quay derives from the dragging of the bottom of the harbour to increase in some areas the depth of water. Detailed in situ and laboratory tests have been made for site characterisation, to face all geotechnical design aspects of the quay. In this paper will be discussed the in-situ tests for the evaluation of the seismic behaviour of the Calata Bettolo. Among the in situ tests, Seismic Dilatometer Marchetti Tests (SDMTs) have been carried out with the aim of general site characterisation, with particular reference to the evaluation of the soil profile of shear waves velocity (Vs) and horizontal stress index K_d for liquefaction potential analysis. The SDMT test program includes tests on the land and on the sea. The latter have been made by an innovative technique by means of floating barge. This technique allowed to perform the SDMT tests at considerably reduced cost in comparison with tests executed from a self elevating platform. The Seismic Dilatometer Marchetti Tests were performed up to a depth of 42 meters. The results show a very detailed and stable shear waves profile. The SDMT test results enable to evaluate the potential liquefaction hazard by two independent measurements, K_d and V_S. The results obtained for potential liquefaction by K_d and V_S are compared and discussed.

Keywords: Marine harbour, SDMT, shear waves, liquefaction potential, horizontal stress index

1 INTRODUCTION

In this paper the seismic flat dilatometer test (SDMT) was used to provide shear wave velocity (V_s) measurements to supplement conventional inflation readings (p_o and p_1).

Soil stratigraphy and soil parameters are evaluated from the pressure readings while the small strain stiffness (G_o) is obtained from in situ V_s profiles. A comprehensive in situ and laboratory investigation has been carried out to study the new container quay namely "Calata Bettolo", inside the Genova harbour.

The length of the quay is of about 750 m and the area is of about 22,000 m^2. The depth of water reaches 17 m along the quay. The material to fill the area of the quay is dragged by the sea bottom of the harbour to increase the depth of water in some areas of the harbour.

The SDMT test program for soil liquefaction hazard evaluation includes tests on the land and on the sea. The latter have been made by an innovative technique by means of floating barge.

This technique allowed to perform the SDMT tests at considerably reduced cost in comparison with tests executed from a self elevating platform. Moreover the use of SDMT instead of the traditional in-situ and lab tests for soil characterisation allows to reduce consistently the time needed for site characterisation.

2 GEOTECHNICAL CHARACTERISATION FOR SOIL LIQUEFACTION HAZARD EVALUATION

For the geotechnical characterisation of the test sites a survey of supplementary geotechnical tests was carried out during the design of the quay.

This survey consists of a borehole and of some tests with the Seismic Dilatometer Marchetti (SDMT), inland and offshore. The borehole S1 was performed in correspondence of the existing jetty, consisting of filling material of varying size, gravel and stone blocks.

Figure 1. Seismic dilatometer equipment positioned offshore on the floating pontoon.

Below, as highlighted in the stratigraphy of the borehole S1, the soil consists of a fine sand weakly gray silty, which extends from the depth of 24 m up to the bottom of the hole.

The survey was carried out up to 24 m in destruction of core and from 24 to 42 m coring continued. As a result of the survey were collected No. 6 undisturbed samples, starting from the depth of 30 m, to execute laboratory tests.

The geotechnical characterisation of the soil was also deduced from the results of SDMTs, carried out inland (SDMTT1) and offshore (SDMT1-4, SDMT6). As regards the inland test SDMTT1, since the land was performed with coarse material, the test was carried out by a drill-hole that passed this material.

The tests carried out offshore were carried out by a floating pontoon, on which the SDMT equipment was placed by a crane. The floating pontoon was towed to be positioned in test sites (Figure 1).

In the interpretation of the test results of the must be taken into account that the soil below the quays is subjected to a load corresponding to the weight of the quays.

As regards the SDMTs performed offshore, they were carried out at a distance from existing docks to not be affected by the presence of the quays.

As regards the values of the friction angle, it is variable from 30° to 35°, with minimum value equal to 25° at a greater depth, where is the presence of the silt component. The value of the material index (I_d) shows the presence of a soil consisting of a sand that at a greater depth becomes a silty clay sand.

As regards the values of the shear wave velocity, the velocity profile shows a growing trend with the depth of 150–250 m/s, with an average value in the first 30 m higher than 180 m/s, which allows to classify the soil type C, for the purposes of evaluation of amplification factor Ss, which is between 1 and 1.2 (Figure 2). As regards the values of the horizontal stress index K_d, it varies from about 4 in the sandy type soil, up to a value of about 2 more in depth where increases the content of silt (Figure 3). It was not possible to make tests to determine the characteristics of the filler from dredging.

3 CURRENT METHODS FOR EVALUATING LIQUEFACTION POTENTIAL

The traditional procedure, introduced by Seed & Idriss (1971), has been applied for evaluating the liquefaction resistance of Calata Bettolo sandy soils in the Genova harbour. This method requires the calculation of the cyclic stress ratio CSR, and cyclic resistance ratio CRR. If CSR is greater than CRR, liquefaction can occur. The cyclic stress

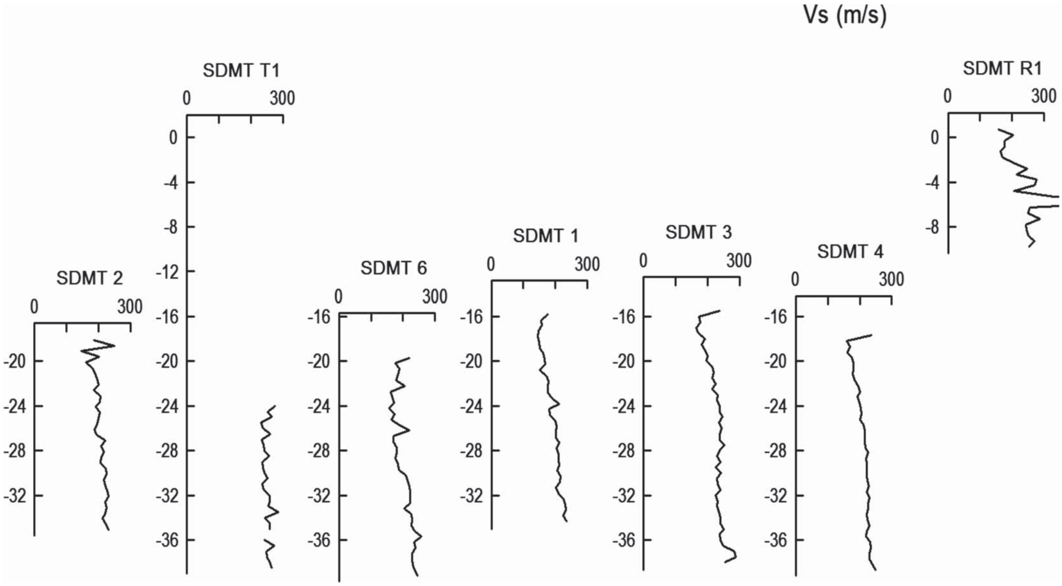

Figure 2. Shear wave velocity profiles obtained from SDMTs performed inland and offshore.

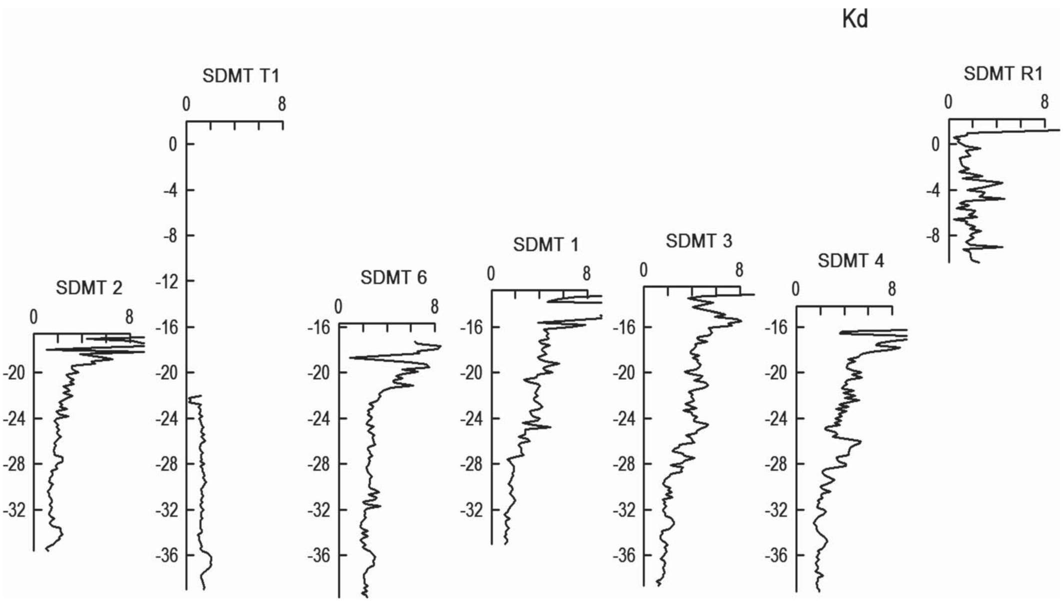

Figure 3. Horizontal stress index K_d profiles obtained from SDMTs performed inland and offshore.

ratio CSR is calculated by the following equation (Seed and Idriss 1971):

$$CSR = \tau_{av}/\sigma'_{vo} = 0.65\,(a_{max}/g)\,(\sigma_{vo}/\sigma'_{vo})r_d \quad (1)$$

where τ_{av} = average cyclic shear stress, a_{max} = peak horizontal acceleration at the ground surface generated by the earthquake, g = acceleration of gravity, σ_{vo} and σ'_{vo} = total and effective overburden stresses and r_d = stress reduction coefficient depending on depth. The r_d has been evaluated according to Liao and Whitman (1986). As regards the value of a_{max} = peak horizontal acceleration at the ground surface generated by the earthquake,

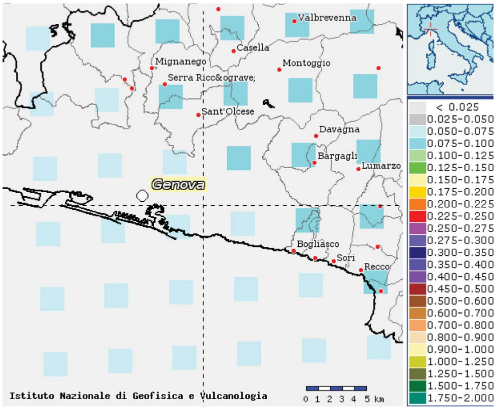

Figure 4. Interactive map of seismic hazard of Genova, Italy.

it has been fixed the value of $a_{max} = 0.075$ g from the Interactive Maps of Seismic Hazard of Italy (WebGis) (http://esse1-gis.mi.ingv.it/), as reported in Figure 4.

4 SHEAR WAVE VELOCITY FROM SDMT FOR EVALUATION OF CRR

The small strain ($\gamma \leq 0.001\%$) shear modulus, G_o, was determined from SDMT. Moreover it was attempted to assess G_o by means of laboratory test results. The Seismic Dilatometer Marchetti (SDMT) is an instrument resulting from the combination of the DMT blade (Marchetti 1980) with a seismic modulus measuring the shear wave velocity V_S.

Initially conceived for research, the SDMT is gradually entering into use in current site investigation practice.

The test is conceptually similar to the seismic cone (SCPT). First introduced by Hepton (1988), the SDMT was subsequently improved at Georgia Tech, Atlanta, USA (Martin & Mayne 1997, 1998; Mayne et al. 1999). A new SDMT system has been recently developed in Italy. The seismic modulus is a cylindrical instrumented tube, located above the DMT blade, housing two receivers at a distance of 0.50 m. The test configuration "two receivers"/"true interval" avoids the problem connected with the possible inaccurate determination of the "first arrival" time sometimes met with the "pseudo interval" configuration (just one receiver).

Moreover the pair of seismograms recorded by the two receivers at a given test depth corresponds to the same hammer blow and not to different blows in sequence, which are not necessarily identical.

The adoption of the "true interval" configuration considerably enhances the repeatability in the V_S measurement (observed repeatability $V_S \approx 1$–2%). V_S is obtained (Figure 2) as the ratio between the difference in distance between the source and the two receivers ($S2 - S1$) and the delay of the arrival of the impulse from the first to the second receiver (Δt). V_S measurements are obtained every 0.5 m of depth.

The shear wave source at the surface is a pendulum hammer (≈10 kg) which hits horizontally a steel rectangular base pressed vertically against the soil (by the weight of the truck) and oriented with its long axis parallel to the axis of the receivers, so that they can offer the highest sensitivity to the generated shear wave. In "Calata Bettolo" site the SDMT tests have been made by means a floating barge (Figure 1). The determination of the delay from SDMT seismograms, normally carried out using the cross-correlation algorithm, is generally well conditioned, being based on the two seismograms—in particular the initial waves—rather than being based on the first arrival time or specific marker points in the seismogram.

The profile of K_d (Figure 3) is similar in shape to the profile of the overconsolidation ratio OCR. $K_d = 2$ indicates in clays OCR = 1, $K_d > 2$ indicates overconsolidation.

It may be noticed that the repeatability of the V_S profile is very high, similar to the repeatability of the other DMT parameters.

The use of the shear wave velocity, V_S, as an index of liquefaction resistance has been illustrated by several authors (Tokimatsu & Uchida 1990; Kayen et al. 1992; Robertson et al. 1992; Lodge 1994; Andrus & Stokoe 1997, 2000; Robertson & Wride 1997; Andrus et al. 1999). The V_S based procedure for evaluating CRR has advanced significantly in recent years, and is included by the '96 and '98 NCEER workshops (Youd & Idriss 2001) in the list of the recommended methods for routine evaluation of liquefaction resistance. A comparison of some relationships between liquefaction resistance and overburden stress-corrected shear wave velocity for granular soils is reported in Figure 5.

The correlation between V_S and CRR given by Andrus and Stokoe (1997, 2000) is:

$$\mathrm{CRR} = a\left(\frac{V_{S1}}{100}\right)^2 + b\left(\frac{1}{(V^*_{S1} - V_{S1})} - \frac{1}{V^*_{S1}}\right) \tag{2}$$

where: V^*_{s1} = limiting upper value of V_{s1} for liquefaction occurrence; $V_{S1} = V_S\,(p_a/\sigma'_{vo})^{0.25}$ is corrected shear wave velocity for overburden-stress; a and b are curve fitting parameters.

This correlation has been improved by Andrus et al. (2004). CRR is plotted as a function of an overburden-stress corrected shear wave

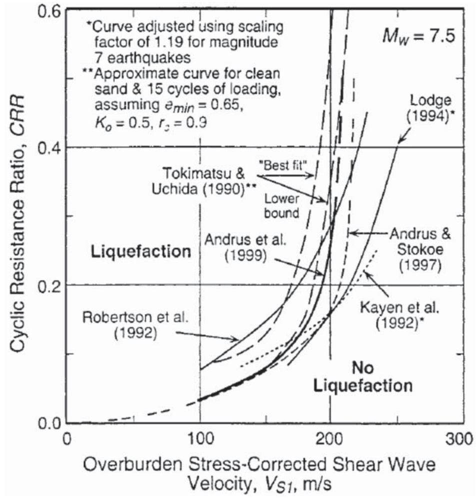

Figure 5. Comparison of some relationships between liquefaction resistance and overburden stress-corrected shear wave velocity for granular soils (Youd & Idriss 2001).

velocity $V_{S1} = V_S\,(p_a/\sigma'_{vo})^{0.25}$, where V_S = measured shear wave velocity, p_a = atmospheric pressure (≈100 kPa), σ'_{vo} = initial effective vertical stress in the same units as p_a.

The relationship CRR-V_{S1} is approximated by the equation for $M_w = 7.5$:

$$\mathrm{CRR}_{7.5} = \left[0.022\left(\frac{K_{a1}V_{S1}}{100}\right)^2 + 2.8\left(\frac{1}{V^*_{S1} - (K_{a1}V_{S1})} - \frac{1}{V^*_{S1}}\right)\right]K_{a2} \tag{3}$$

where V^*_{S1} = limiting upper value of V_{S1} for liquefaction occurrence, assumed to vary linearly from 200 m/s for soils with fines content of 35% to 215 m/s for soils with fines content of 5% or less. K_{a1} is a factor to correct for high V_{S1} values caused by aging, K_{a2} is a factor to correct for influence of age on CRR. Both K_{a1} and K_{a2} are 1.0 for uncemented soils of Holocene age.

For older soils the SPT-V_{S1} equations by Ohta & Goto (1978) and Rollins et al. (1998) suggest average K_{a1} values of 0.76 and 0.61, respectively, for Pleistocene soils (10,000 years to 1.8 million years). Lower-bound values of K_{a2} are based on the study by Arango et al. (2000).

The evaluation of CRR according to equation 2 (Andrus & Stokoe 2000) and equation 3 (Andrus et al. 2004), at "Calata Bettolo" site for V_s profile obtained by SDMT1 (Figure 2) is reported in Figure 6. From Figure 6 the CRR values given by equation 3 are lower than those given by equation 2, so therefore the evaluation given by equation 3 according to Andrus et al. 2004 is more conservative.

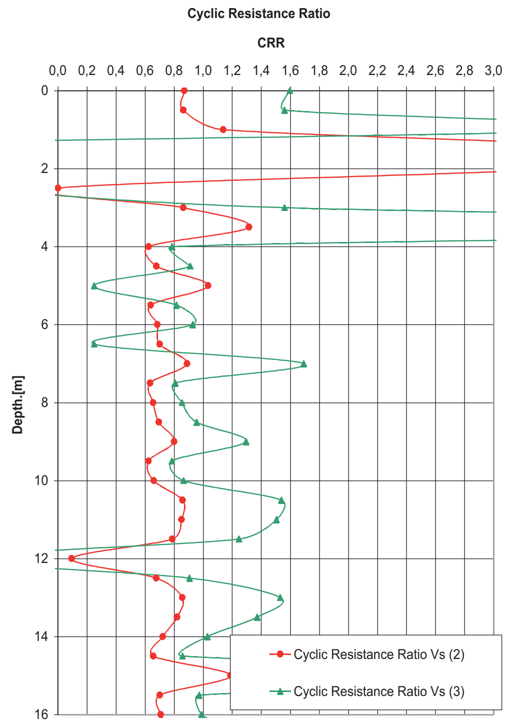

Figure 6. Evaluation of CRR at calata bettolo site using equation (2) and equation (3) and V_s profile obtained from SDMT1.

Figure 7 shows the evaluation of liquefaction potential index, P_L, according to Iwasaki et al. 1978, which shows that the liquefaction potential index, P_L, is low for V_s data using equation 3.

5 EVALUATION OF CRR FROM THE DMT HORIZONTAL STRESS INDEX K_d

Marchetti (1982) and later studies (Robertson & Campanella 1986; Reyna & Chameau 1991) suggested that the horizontal stress index K_d from DMT ($K_d = (p_o - u_o)/\sigma'_{vo}$) is a suitable parameter to evaluate the liquefaction resistance of sands.

Monaco et al. (2005) summarized the various correlations developed to estimate CRR from K_d, expressed in form of CRR-K_d boundary curves separating possible "liquefaction" and "no liquefaction" regions. Previous CRR-K_d curves were formulated by Marchetti (1982), Robertson & Campanella (1986) and Reyna & Chameau (1991).

A new tentative correlation for evaluating CRR from K_d, to be used according to the Seed & Idriss (1971) "simplified procedure", was formulated by Monaco et al. (2005) by combining previous CRR-K_d correlations with the vast experience

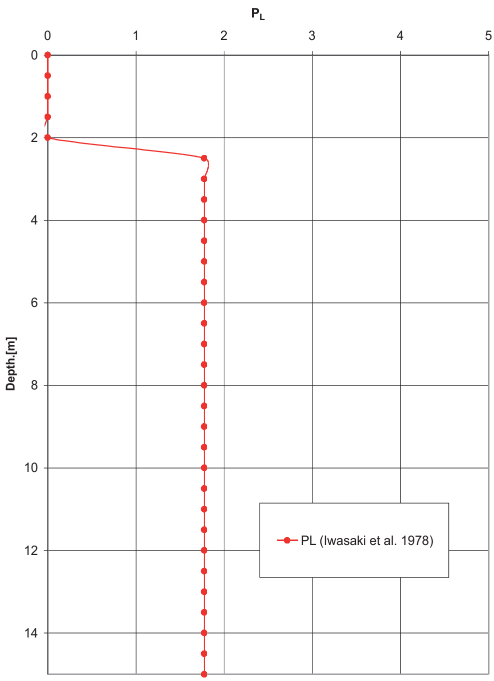

Figure 7. Evaluation of liquefaction potential index P_L from V_s data at calata bettolo site.

incorporated in current methods based on CPT and SPT (supported by extensive field performance databases), translated using the relative density D_R as intermediate parameter.

Additional CRR-K_d curves were derived by translating current CRR-CPT and CRR-SPT curves (namely the "Clean Sand Base Curves" recommended by the '96 and '98 NCEER workshops, Youd & Idriss 2001) into "equivalent" CRR-K_d curves via relative density. D_R values corresponding to the normalized penetration resistance in the CRR-CPT and CRR-SPT curves, evaluated using current correlations (D_R-q_c by Baldi et al. 1986 and Jamiolkowski et al. 1985, D_R-N_{SPT} by Gibbs & Holtz 1957), were converted into K_d values using the K_d-D_R correlation by Reyna & Chameau (1991).

The "equivalent" CRR-K_d curves derived in this way from CPT and SPT plot in a relatively narrow range, very close to the Reyna & Chameau (1991) curve.

The new tentative CRR-K_d curve is approximated by the equation:

$$\text{CRR} = 0.0107\ K_d^3 - 0.0741\ K_d^2 + 0.2169\ K_d - 0.1306 \quad (4)$$

was proposed by Monaco et al. (2005) as "conservative average" interpolation of the curves derived from CPT and SPT.

Additional CRR-K_d curves for San Giuseppe La Rena (Sicily) coastal plain area were derived by translating current CRR-CPT and CRR-SPT curves into "equivalent" CRR-K_d curves via relative density. D_R values, corresponding to the normalized penetration resistance in the CRR-CPT and CRR-SPT curves, evaluated using current correlations (D_R-q_c by Baldi et al. 1986 and Jamiolkowski et al. 1985, D_R-N_{SPT} by Gibbs & Holtz 1957), were converted into K_d values using the K_d-D_R correlation by Reyna and Chameau (1991). Some new tentative CRR-K_d curves approximated by the equations:

$$\text{CRR} = 0.0308\, e^{(0.6054 K_d)} \quad (5)$$

$$\text{CRR} = 0.0111\ K_d^{2.5307} \quad (6)$$

have been proposed by the authors (Grasso & Maugeri, 2006) as interpolation of the K_d curves derived from SPT and CPT.

The evaluation of CRR according to equations 4, 5 and 6 at "Calata Bettolo" site for K_d profile obtained by SDMT1 (Figure 3) is reported in Figure 8.

Figure 9 shows the evaluation of liquefaction potential index, P_L, according to Iwasaki et al. 1978, which shows that the liquefaction potential

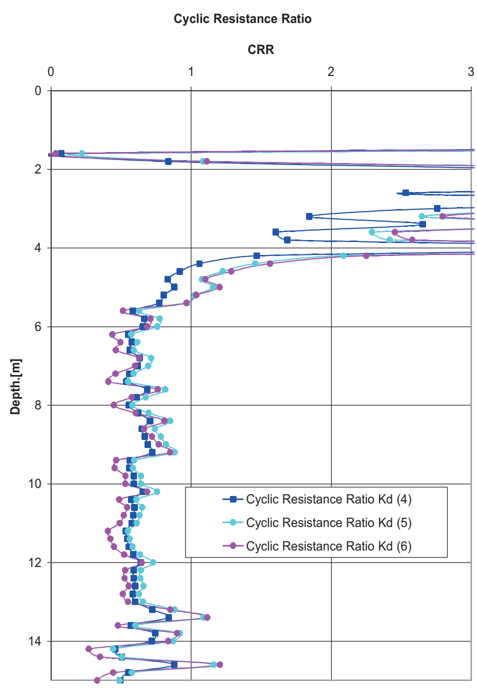

Figure 8. Evaluation of CRR at Calata Bettolo using equations (4), (5) and (6) and K_d profile obtained from SDMT1.

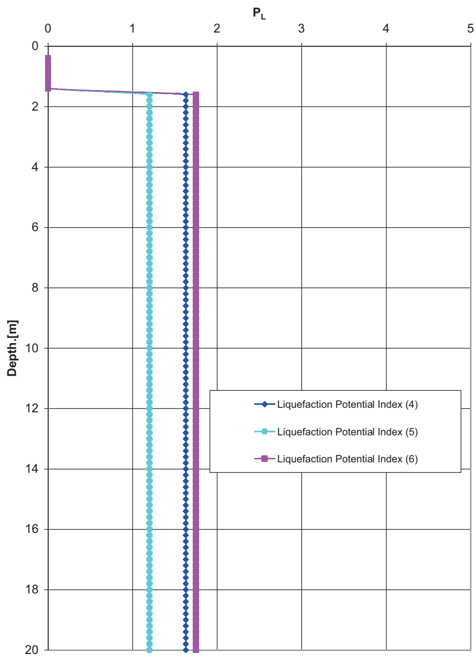

Figure 9. Evaluation of liquefaction potential index P_L from K_d SDMT1 data at Calata Bettolo site.

index, $P_{L,}$ is low for K_d SDMT1 data using equations 4, 5 and 6.

Similar results have been obtained for the other SDMT tests, so the liquefaction potential index of Calata Bettolo quay is low.

6 CONCLUSIONS

The use of SDMT allows to obtain the soil characterisation in three weeks and so in much less time than that needed with the traditional in-situ and lab tests.

For liquefaction potential index the SDMT gives the possibility to use two independent measurements V_s and K_d for evaluating it.

The liquefaction potential index evaluated by V_s shows low values of $P_L = 1.6$ which means low values of liquefaction hazard.

For the liquefaction potential index evaluated by K_d, three different equations have been used. The results show results comparable for all the equations. The maximum value of liquefaction potential index is $P_L = 1.75$, which still means low values of liquefaction hazard.

The values of liquefaction potential index by V_s and K_d are very near themselves, because of course at the bottom of the sea there is not the presence of the rigid crust.

The technique of performing SDMTs from a floating barge, allows to reduce considerably the cost at a fraction of about 15–20% of performing the tests from a self elevating platform.

The SDMT was operating from the floating barge continuously because inside the harbour the sea waves are very limited. Only when the waves exceed 8 centimetres the operability has to be stopped.

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