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Evaluation of dynamic cone penetration test for liquefaction assessment of gravels from case histories in Italy

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ABSTRACT: In North American practice, the Becker Penetration Test (BPT) has become the primary field test used to evaluate liquefaction resistance of gravelly soils. However, this test is expensive and uncertainties exist regarding correlations and corrections. As an alternative, the 74 mm diameter dynamic cone penetration test (DPT) developed in China has recently been correlated with liquefaction resistance based on field performance data from the Mw7.9 Wenchuan earthquake. In this study, liquefaction resistance was evaluated using DPT soundings with two hammer energies at two sites in Avasinis, Italy where gravelly soil liquefied in the 1976 Friuli, Italy earthquake. Gravel content ranged from 20 to 40% based on the 4.75mm criteria and The DPT liquefaction correlation correctly predicted liquefaction in all cases where liquefaction features were observed. These results indicate that the DPT can provide accurate liquefaction hazard evaluations for gravelly soils more economically than alternative procedures. Standard SPT energy corrections were found to be reasonable for the DPT.

1 INTRODUCTION

One of the most difficult problems in geotechnical engineering is characterizing the liquefaction resistance of gravelly soils in a reliable, cost-effective manner for routine projects. Even for large projects, such as dams, ports, and power-plant projects, assessing the liquefaction resistance of gravelly soils is still expensive and problematic. Nevertheless, liquefaction is known to have occurred in gravelly soils in nearly 20 earthquake events during the past century (Rollins et al. 2018). As a result, engineers and geologists are frequently called upon to assess the potential for liquefaction in gravels. Therefore, innovative methods for characterizing and assessing liquefaction hazards in gravels are an important objective in geotechnical engineering.

In gravelly soils, the, standard penetration test (SPT) and cone penetration test (CPT) are not generally useful because of interference from large-size particles relative to the diameter of the penetrometer. As the gravel content increases, the penetration resistance increases and may reach refusal even in cases when the soil is not particularly dense. This limitation often makes it difficult to obtain a consistent and reliable correlation between SPT or CPT penetration resistance and basic gravelly soil properties. To overcome this limitation, the Becker Penetration Test (BPT) has become the primary field test used to evaluate liquefaction resistance of gravelly soils in North American practice (Harder 1997). The Becker penetration test is performed by hammering a closed-end 168-mm diameter casing into the ground so that the penetration resistance is not much affected by particle size. However, this test is expensive and uncertainties exist regarding correlations with sand behavior and corrections for rod friction and chamber pressure etc. (Cao et al. 2013, Sy 1997). Although innovative instrumentation approaches, such as the iBPT promise to improve the reliability of energy assessment and skin friction losses for the BPT (De Jong et al. 2017), this approach does not reduce the cost and complexity of the test procedure. Furthermore, even after energy corrections, the BPT blowcount must be correlated with the SPT blowcount before liquefaction can be evaluated. This indirect approach increases the uncertainty in the method.

As an alternative, the penetration resistance from a dynamic cone penetration test (DPT) developed in China has recently been correlated with liquefaction resistance based on field performance data from the M_w 7.9 Wenchuan earthquake (Cao et al. 2013). The DPT consists of a 74 mm diameter cone tip continuously driven by a 120 kg hammer with a free fall height of 100 cm using a 60 mm drill rod to reduce friction. The DPT blowcount, N_{120}, value represents the number of hammer blows to drive the penetrometer 30 cm with a 120 kg hammer. Blowcounts are typically reported every 10 cm but are multiplied by three to get the equivalent N_{120}. Over the past 60 years, Chinese engineers have found that the DPT is effective in penetrating coarse or cobbly gravels and provides penetration data useful for liquefaction assessment (Cao et al. 2013). This test could provide an important new procedure for characterization of gravels and fill a gap in present geotechnical practice, but additional field performance data is necessary.

To verify the validity of the DPT-based triggering curves and provide additional field performance data to refine the triggering curves in the future, DPT soundings were performed at two sites where gravelly sands liquefied during the 1976 Friuli, Italy earthquake. DPT tests were performed with two hammer energies to evaluate the ability of SPT-based hammer energy correction methods to account for different hammer energies with the DPT.

2 EARTHQUAKE CHARACTERISTICS AND OBSERVED LIQUEFACTION EFFECTS

The main shock from the Friuli Earthquake on May 6, 1976 had a moment magnitude (M_w) of 6.4 and resulted in significant damage to 77 cities in northern Italy Maugeri (1976). The earthquake caused the deaths of nearly 1000 people and left over 40,000 people homeless for over six months. In addition, the earthquake produced widespread liquefaction within alluvial fan deposits in the town of Avasinis as shown in Figure 1. The stream running at the base of the alluvial fan maintains the water table relatively high in the lower portion of the permeable sediments of the fan, a factor playing a crucial role in the potential for liquefaction. The

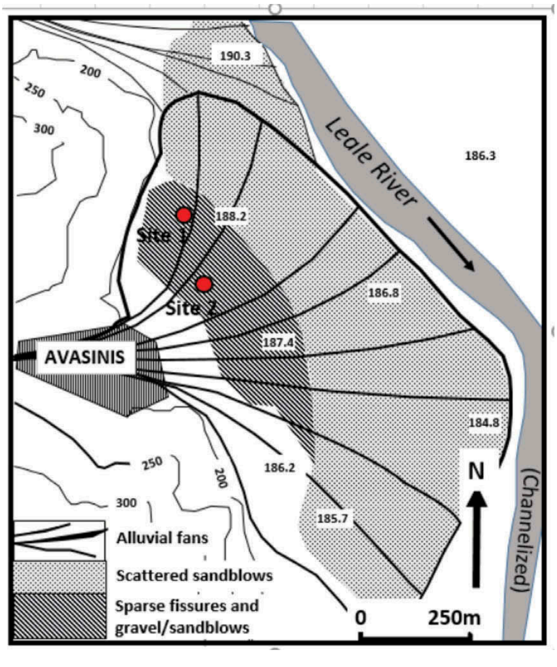

Figure 1. Site map showing locations of test sites with respect to topography and location of sand ejecta and sand/gravel ejecta (modified from Sirovich 1996a).

composite fan shows a clear lateral gradient in grain size with gravelly sediments near the apex of the fan and sandy sediments lower on the fan.

According to Sirovich (1996a), sand and gravel ejecta was observed in numerous boils and fissures in the north-western side of the fan, while sand ejecta was primarily observed in hundreds of sand blows in the south-eastern side of the fan as shown in Figure 1. Gravel and sand ejecta were observed at Sites 1and 2 during the main shock while liquefaction features were also observed at Site 1 during two subsequent aftershocks (M_w 6 and M_w 5.3). Peak ground accelerations were estimated to range from 0.47g to 0.25g for these three events (Bindi et al. 2011). At Site 1, liquefaction produced settlements of 60 to 70 cm. Therefore, these case histories can be particularly instructive in defining liquefaction triggering curves.

According to Sirovich (1996a), the sand and gravel in the upper 10 m of the profile are between 100 and 1000 years old. The gravelly soils in the upper part of the fan are composed of angular to semi-angular limestone particles within a weak structure that ranges from gravel clast-to-clast supported to matrix- supported (Sirovich, 1996a). Particle size distribution curves at the location of DPT 1 were provided by Sirovich (1996a). Although data is very limited, the gravel fraction appears to increase with depth while fines contents are typically around 10 to 15%. Based on the Unified Soil Classification System (USCS) where gravel size is defined as coarser than 4.75 mm, the gravel fraction is between 20 and 40%. However, many organizations throughout the world define gravel as coarser than 2 mm and based on this definition, the gravel content would be between 40 and 60% (AASHTO M 145 1995, EN ISO 14688-1 2018, BS 5930 2015).

3 DPT SOUNDINGS

DPT soundings were performed at two sites where gravelly sand ejecta was observed (Sites 1 and 2). The soil profile at Site 1, shown in Figure 2, is based on an SPT borehole previously drilled by Sirovich (1996b) on a roadway about 20 m west of the location of DPT 1. The soil profile was generally described as gravelly alluvium up to a depth of 30 m; however, the DPT 1 site has a weak surface layer about 1.5 m thick consisting of gravelly clayey silt. This surface layer appears to have been excavated and replaced with a denser gravel fill beneath the roadway. The N values obtained by Sirovich (1996b) from SPT tests have been corrected to $(N_1)_{60}$ values using procedures outlined by Youd et al. (2001) based on a measured SPT hammer energy of 42%. The resulting $(N_1)_{60}$ values are plotted in in Figure 2. The $(N_1)_{60}$ values typically range from 12 to 16 within the upper 12 m of the profile.

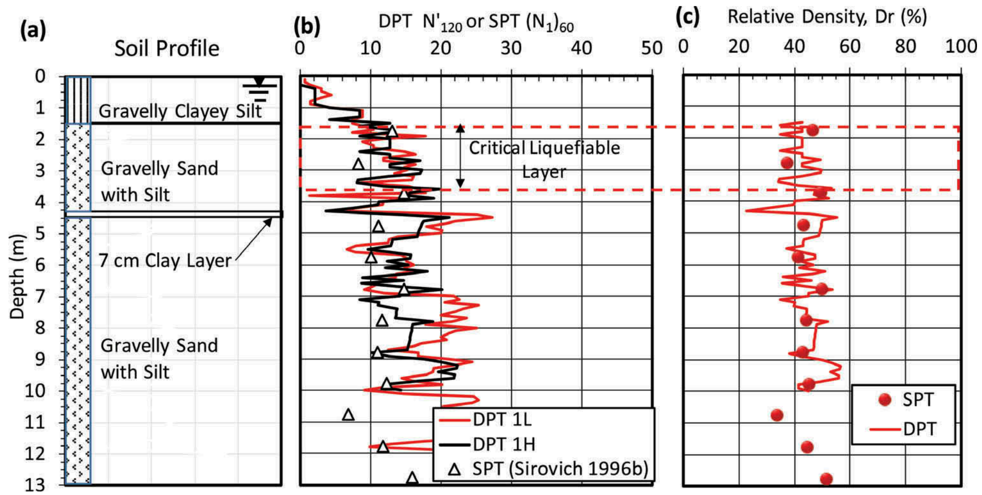

Figure 2. DPT 1 (a) soil profile, (b) DPT N'_{120} vs. depth for heavy (H) 120 kg and light (L) 64.6 kg hammer in comparison with SPT $(N_1)_{60}$ vs. depth (Sirovich 1996b) and (c) Relative density from SPT and DPT correlations.

The DPT soundings were performed using a drill rig with the capability of employing two different hammer energies. At each site, one DPT sounding was advanced using a free-fall SPT donut hammer with a weight of 63.6 kg dropped from a height of 0.76 m. A second sounding was then performed about a meter away using a 120 kg free-fall donut hammer with a drop height of 1.0 m as specified by the Chinese DPT standard (Cao et al 2013). Hammer energy measurements were made using an instrumented rod section and a Pile Driving Analyzer (PDA) device from PDI, Inc. These energy measurements indicate that the SPT and 120 kg hammers delivered averages of 65% and 75% of their theoretical free-fall energies, respectively, with standard deviation values of 5.6 and 5.9%. Based on 1200 hammer energy measurements, Cao et al. (2012) found that the Chinese DPT provided an average of 89% of the theoretical free-fall energy. Because the energy delivered by a given hammer (E_{Hammer}) may be less than the energy typically supplied by a Chinese DPT hammer ($E_{Chinese\ DPT}$), it may be necessary to correct the measured blow count downward. In this study, the correction was made using the simple linear reduction suggested by Seed et al. (1985) for SPT testing

$$N_{120}=N_{Hammer}(E_{Hammer}/E_{chinese\ DPT}) \tag{1}$$

where N_{Hammer} is the number of blows per 0.3 m of penetration obtained with a hammer delivering an energy of E_{Hammer}.

The ratio of hammer energy actually delivered divided by the energy delivered by the Chinese DPT hammer was 0.84 and 0.29 for the 120 kg and 63.6 kg hammers, respectively. In addition, Cao et al. (2013) recommend an overburden correction factor, C_n to obtain the normalized N'_{120} value using the equation

$$N'_{120}=N_{120}C_n \tag{2}$$

where

$$C_n=\left(100/\sigma'_o)^{0.5} \leq 1.7\right) \tag{3}$$

and σ'_o is the initial vertical effective stress in kN/m^2. A limiting value of 1.7 was added to be consistent with the C_n used to correct penetration resistance from other in-situ tests.

Plots of the energy corrected DPT N'_{120}versus depth profiles for the 63.6 and 120 kg hammers are provided in Figures 2 and 3 for DPT soundings 1 and 2, respectively. A comparison of these profiles indicates that the agreement obtained with the simple energy correction factor in Equation 2 is quite good. In addition to variations owing to differences in hammer energy, it should be recognized that differences in N'_{120} would also be expected between soundings with the same hammer energy owing to small differences in site elevation and soil stratigraphy even though the DPT soundings are relatively close together.

A comparison of the N'_{120} obtained from the lighter 63.6 kg hammer typically used with SPT testing and N'_{120} obtained with the heavier 120 kg hammer used with the Chinese DPT is provided in Figure 4 for all the DPT tests at Avasinis. The best-fit regression line for all the data points falls on the one-to-one line for perfect agreement indicating that the average DPT N'_{120} values are comparable after energy correction; however, there is scatter about the best-fit line and the correlation coefficient is only 63%.

In addition to the DPT N'_{120} values, conventional SPT $(N_1)_{60}$ values were previously conducted by Sirovich (1996b) near the locations for DPT 1 and DPT 2 and are plotted in Figures 2 and 3. Although the SPT $(N_1)_{60}$ values appear to exhibit similar trends with the DPT N'_{120}no useful correlation could be developed with the very small data set. Nevertheless, relative density (D_r) was computed based on the SPT $(N_1)_{60}$ values near DPT 1 and 2 using the equation

$$D_r = \left((N_1)_{60}/60\right)^{0.5} \tag{4}$$

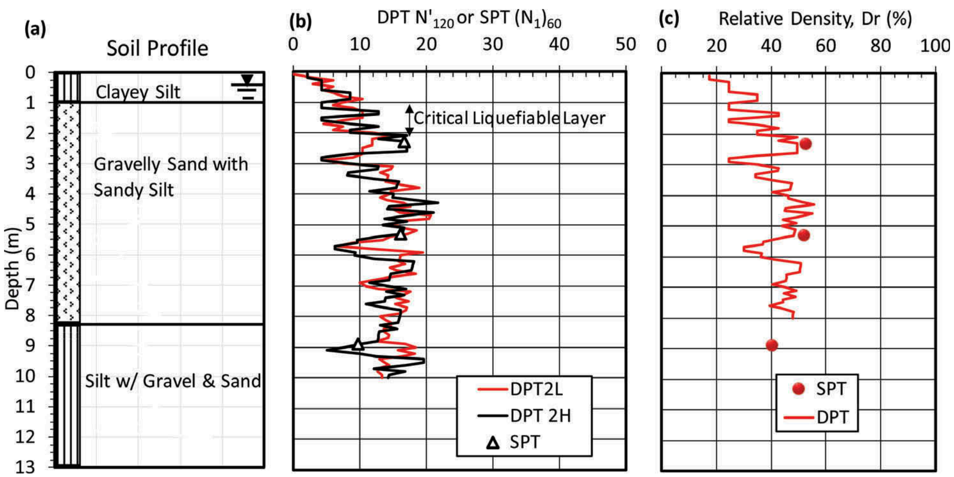

Figure 3. DPT 2 (a) soil profile, (b) DPT N'_{120} vs. depth for heavy (H) 120 kg and light (L) 64.6 kg hammer in comparison with SPT $(N_1)_{60}$ vs. depth (Sirovich 1996b) and (c) Relative density from SPT and DPT correlations.

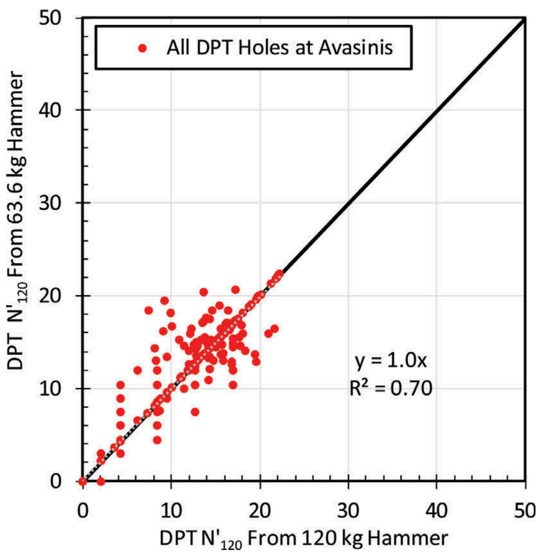

Figure 4. Comparison of DPT N'_{120} obtained from lighter 63.6 kg hammer typically used with SPT testing and DPT N'_{120} obtained heavier 120 kg hammer used with the Chinese DPT

proposed by Kulhawy and Mayne (1990) while D_r was computed for the DPT N'_{120} values at each site using the equation

$$D_r = \left(N'_{120}/70\right)^{0.5} \tag{5}$$

suggested by Chinese experience with the DPT

Relative density profiles are also plotted in Figures 2 and 3. Relative density values are typically between 40 and 50% at most sites. A comparison of the D_r from the DPT and SPT values for DPT 1 and 2 is provided in Figures 2c and 3c and the agreement between the two approaches appears to be relatively good for this limited data set.

Following the 2008 M_w 7.9 Wenchuan earthquake in China, 47 DPT soundings were made at 19 sites with observed liquefaction effects and 28 nearby sites without liquefaction effects. Most of these sites consisted of 2 to 4 m of clayey soils, which, in turn, were underlain by gravel beds up to 500 m thick. Looser upper layers within the gravel beds are the materials that liquefied during the Wenchuan earthquake. Because samples are not obtained with the DPT, boreholes were drilled about 2 m away from most DPT soundings with nearly continuous samples retrieved using core barrels. DPT soundings reached depths as great as 15 m, readily penetrating gravelly layers that liquefied as well as many layers that were too dense to liquefy.

Layers with the lowest DPT resistance in gravelly profiles were identified as the most liquefiable or critical liquefaction zones. At sites with surface effects of liquefaction these penetration resistances were generally lower than those at nearby DPT sites without liquefaction effects. Thus, low DPT resistance became a reliable identifier of liquefiable layers (Cao et al. 2011). At the center of each layer, the cyclic stress ratio (CSR) induced by the earthquake was computed using the simplified equation

$$\mathrm{CSR} = 0.65(\mathrm{a}_{\max}/\mathrm{g})(\sigma_{\mathrm{vo}}/\sigma_{\mathrm{vo}})\mathrm{r}_{\mathrm{d}} \tag{6}$$

where a_{max} is the peak ground acceleration, σ_{vo} is the initial vertical total vertical stress, σ'_{vo} is the initial vertical effective stress, and r_d is a depth reduction factor as defined by Youd et al. (2001).

Using DPT data, Cao et al. (2013) plotted the cyclic stress ratio causing liquefaction against DPT N'_{120} for the M_w 7.9 Wenchuan earthquake. Points where liquefaction occurred were shown as solid red dots, while sites without liquefaction were shown with open circles. Cao et al. (2013) also defined curves indicating 15, 30, 50, 70 and 85% probability of liquefaction based on logistical regression. To facilitate comparison with data points from other earthquakes, in this study the Cao et al. (2013) data points and triggering curves were shifted upward using the equation

$$\mathrm{CSR}_{\mathrm{Mw7.5}} = \mathrm{CSR}/\mathrm{MSF} \tag{7}$$

where the Magnitude Scaling Factor (MSF) is given by the equation

$$\mathrm{MSF} = 10^{2.24}/\mathrm{M_w}^{2.56} \tag{8}$$

proposed by Youd et al. (2001).

The two case histories with DPT test results provide an excellent opportunity to evaluate the ability of the DPT-based liquefaction triggering curves developed by Cao et al. (2013) to predict accurately liquefaction in gravelly soil. For the Friuli case histories, the geology, earthquake magnitude, and gravel layers are significantly different from those in the Chengdu plain of China and will provide a good test of the method. In addition, gravelly soils at Site 1 are reported to have liquefied in three separate earthquake events (Sirovich 1996a) so three separate data points can be generated for this site.

At each site, the critical layer for liquefaction was identified and these layers are shown for the DPT profiles in Figures 2 and 3. The critical layer has the lowest average N'_{120} below the water table and closest to the ground surface. It is the layer most likely to liquefy and produce the observed ejecta, although liquefaction might still occur in other layers. The average soil properties, vertical soil stresses, and r_d values for the critical layers at the two sites are summarized in Table 1. The peak ground acceleration (a_{max}) was estimated using ground motion prediction equations (GMPE) developed by Bindi et al. (2011). This GMPE law was calibrated using the Joyner-Boore distance (the closest distance from the site to the surface projection of the rupture fault plane) or hypocentral distance, and considers the style-of-faulting and site effects. The earthquake parameters are listed in Table 1 along with MSF values for each case along with the CSR, and CSR/MSF values.

The CSR and DPT N'_{120} values for the Friuli case histories are also plotted in Figure 5 for comparison with the triggering curves developed previously by Cao et al. (2013). The data

Table 1. Summary of average soil properties and earthquake parameters in critical liquefaction layer.

	Soil Properties							Earthquake Properties			
Sit-e	Avg. depth (m)	Avg. σ_o (kPa)	Avg. σ'_o (kPa)	Avg. N'_{120} (Blows per 0.3 m)	Avg. $(N_1)_{60}$ (Blows per 0.3 m)	Avg. $(N_1)_{60CS}$ (Blows per 0.3 m)	Avg. V_{S1} (m/s)	M_w 6.4 MSF=1.5 a_{max}=0.47 CSR/MSF	M_w 6.0 MSF=1.77 a_{max}=0.25 CSR/MSF	M_w 5.7 MSF=2.43 a_{max}=0.25 CSR/MSF	Depth Factor r_d
1	2.5	47.5	25.9	12.3	12.3	14.7	200	0.37	0.16	0.12	0.98
2	1.5	28.3	16.5	7.7	17	18.2	210	0.34	-	-	0.99

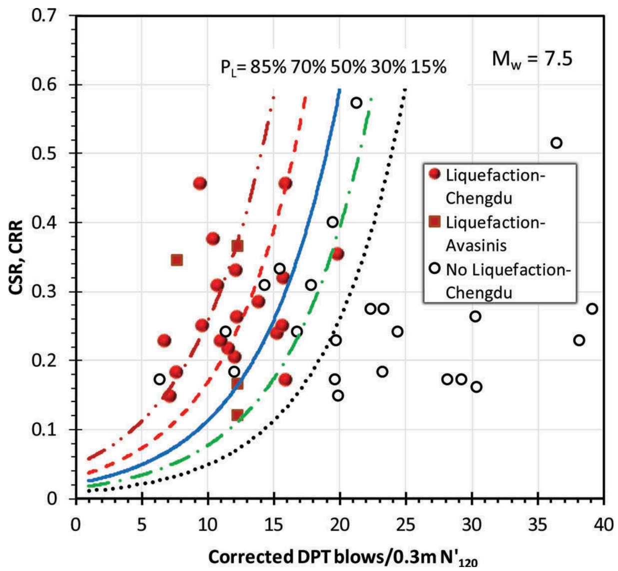

Figure 5. Probabilistic liquefaction triggering curves for gravelly soils based on DPT penetration resistance (Cao et al. 2013) after scaling to a M_w 7.5 earthquake. Liquefaction and no-liquefaction data points are from Chengdu, China and Avasinis, Italy.

points for DPTs 1 and 2 for the main shock (M_w 6.4) all fall on or above the 70% probability curve. However, the data points for the smaller aftershocks (M_w 6.0 and 5.3) at Site 1 where liquefaction was observed typically plot on or above the 30% probability curve. Thus, these data points correctly predict the occurrence of liquefaction surface features. Because these data points are some of the lowest of any in the overall data set, they could be particularly important in refining the shape of the triggering curve.

4 CONCLUSIONS

Based on investigations conducted using the Chinese Dynamic Cone Penetrometer (DPT) test at Avasinis, Italy the following conclusions have been developed:

1. The Chinese dynamic cone penetrometer could generally be driven through gravelly sand alluvium profiles with 20 to 40% gravel content using only the conventional SPT hammer energy despite the larger particle sizes.

2. Typical hammer energy correction factors provided a reasonable means for adjusting the blow count from the SPT hammer to give blow counts that would be obtained with the conventional Chinese DPT hammer energy. Ideally, the heavier hammer should be used for

liquefaction assessment where practical to avoid uncertainty associated with scatter in the energy correction correlations.

3. Liquefaction triggering correlations based on the DPT N'_{120}, correctly identified sites where liquefaction features were observed. These results suggest that the DPT can provide reasonable accuracy with a simpler, more economical method in comparison with alternative approaches.

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