Evaluation of the Dynamic Cone Penetration Test (DPT) for liquefaction triggering at gravel sites in Alaska and Italy

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ABSTRACT: The dynamic cone penetration test (DPT) developed in China has been correlated with liquefaction resistance in gravelly soil based on field performance data from the M_w7.9 Wenchuan earthquake in China. The DPT consists of a 74 mm diameter cone tip driven by a 120 kg hammer with a free fall height of 1 m. To expand the database, DPT soundings were performed at sites in Valdez, Alaska and L'Aquila, Italy where gravel did and did not liquefy during the 1964 Alaska and 2009 L'Aquila earthquakes, respectively. DPT testing was performed using an automatic hammer weight producing energy similar to the Chinese hammer. Based on the measured DPT N'_{120} and computed cyclic stress ratio (*CSR*), liquefaction triggering analyses were performed at six sites. The Chinese triggering curves were successful in predicting liquefaction, but also predicted liquefaction at some site where liquefaction did not occur. These results suggest the need for new regression analyses using the entire world-wide data set to produce an improved prediction method.

Keywords: Gravel liquefaction, Dynamic Cone Penetration Test (DPT), Cyclic Stress Ratio (CSR)

1. Introduction

Characterizing gravelly soils in a reliable, cost-effective manner for routine engineering projects is a major challenge in geotechnical engineering. Even for large projects, such as dams, ports, and power projects, characterization is still expensive and problematic. This difficulty is particularly important for cases where liquefaction may occur. Liquefaction is known to have occurred in gravelly soils at multiple sites during at least 16 earthquakes over the past 130 years as summarized in Table 1. As a result of these case histories, engineers and geologists must be prepared to assess the potential for liquefaction in gravels. In a number of cases, older dams were constructed on gravelly soil foundations before the potential for liquefaction in gravels was recognized by the profession. For these projects, assessing the potential for liquefaction and determining appropriate remedial measures are often multi-million dollar decisions. Therefore, innovative methods for characterizing and assessing liquefaction hazards in gravels are certainly an important objective in geotechnical engineering.

Because of the difficulty of obtaining meaningful results from Standard Penetration tests (SPT) and Cone Penetration tests (CPT) in gravelly soils, the large diameter Becker Penetration test (BPT) has been developed. Although the BPT-based approach has provided a reasonable method for liquefaction assessment of gravels, the method is expensive and involves empirical correlations between BPT and SPT penetration resistance that increase uncertainty. In addition, BPT testing equipment is not available throughout most of the world. Over the past 60 years, Chinese engineers have developed a Dynamic Cone Penetration Test (DPT) that is effective in penetrating coarse or even cobbly gravels and provides penetration data useful for liquefaction assessment [1]. At 74 mm, the DPT diameter is 50% larger than the SPT and 110% larger than a standard 10 cm^2 CPT and could be less affected by gravel size particles; however, it is still 55% smaller than the BPT.

Probabilistic liquefaction triggering curves have also been developed for gravelly soil based on DPT investigations on the Chengdu plain in China where gravel liquefaction took place during the Wenchuan earthquake [2]. This test provides an important new procedure for characterization of gravels that fills a void in present geotechnical practice and provides a simple, economic method for liquefaction hazard assessment of gravelly soil.

The objectives of this on-going research can be summarized as follows:

1. to evaluate the ability of existing DPT-based liquefaction triggering curves [2] to predict liquefaction based on DPT tests at sites around the world where gravel liquefaction took place during several major earthquakes.
2. to provide additional data points defining the liquefaction resistance as a function of DPT blow

count at sites throughout the world where gravels did or did not liquefy in past earthquakes.

3. to use the additional data points from these investigations to improve the probabilistic liquefaction triggering curves for the DPT for future evaluation of liquefaction hazard in gravelly soils.

Intuitively it is important to investigate those sites where gravelly soil has liquefied to understand the liquefaction characteristics under seismic loading. But, sites where gravels did not liquefy during significant earthquakes must also be investigated to provide a constraint on the liquefaction triggering curves at higher blow counts and cyclic stress ratios. Hence, to accomplish the objectives mentioned above, DPT tests were performed at two different locations, namely, Valdez, Alaska and L'Aquila, Italy, where there was evidence of both liquefaction and no-liquefaction during seismic events. This paper describes the test procedures used at these sites, the results that were obtained, and evaluates the performance of the DPT-based liquefaction triggering procedure given by Cao et al. [2] in predicting the liquefaction hazard for these sites.

Table 1. Case histories involving liquefaction of gravelly soil.

Earthquake	Year	M_w	Reference
Mino-Owari, Japan	1891	7.9	[3]
Fukui, Japan	1948	7.3	[4]
Alaska	1964	9.2	[5]
Haicheng, China	1975	7.3	[6]
Tangshan, China	1976	7.8	[6]
Friuli, Italy	1976	6.4	[7]
Miyagiken-Oki, Japan	1978	7.4	[3]
Borah Peak, Idaho	1985	6.9	[8]
Armenia	1988	6.8	[9]
Roermond, Netherlands	1992	5.8	[10]
Hokkaido, Japan	1993	7.8	[11]
Kobe, Japan Chi-Chi, Taiwan	1995 1999	7.2 7.8	[12] [13]
Wenchuan, China	2008	7.9	[2]
Cephalonia Is., Greece	2012	6.1	[14]
Muisne, Ecuador	2016	7.8	[15]

2. Limitations of current methods for characterizing gravels

Because of the difficulty of extracting undisturbed samples from gravelly soils, laboratory tests on undisturbed samples have not proven effective or reliable for measurement of shear strength or liquefaction resistance. Freezing of a gravel layer before sampling improves sample quality, but the cost is prohibitive for routine projects. Even when undisturbed samples can be extracted, changes in stress conditions between the field and laboratory can limit the usefulness of laboratory test results.

For sands and fine-grained soils, standard penetration tests (SPT) and cone penetration tests (CPT) are widely used to measure penetration resistance for applications in engineering design and for assessing liquefaction resistance. However, SPT and CPT are not generally useful in gravelly soils because of interference from large particles. Because of the large particles, the penetration resistance increases and may even reach refusal in cases when the soil is not particularly dense. This limitation often makes it very difficult to obtain a consistent and reliable correlation between SPT or CPT penetration resistance and basic gravelly soil properties. While liquefaction may still be predicted by SPT and CPT methods for loose gravelly soils, these techniques are more problematic for denser gravelly soils. In this case, it becomes very difficult to determine if the higher blow counts result from greater density or interference from gravel particles.

In North American practice, the Becker Penetration Test (BPT) has become the primary field test used to measure penetration resistance of gravelly soils. The BPT was developed in the late 1950s and consists of a 168-mm diameter, 3-m-long double-walled casing, whose resistance is defined as the number of blows required to drive the casing through a depth interval of 30 cm. For liquefaction resistance evaluations, closed-end casing is specified. To facilitate use of the BPT for liquefaction resistance calculations, Harder and Seed developed correlations between BPT and SPT blow counts in sand after correction for Becker bounce chamber pressure and atmospheric pressure at the elevation of testing [16, 17]. Because of its large diameter, the BPT is less affected by gravel size particles than any other in-situ test. However, the BPT has a high mobilization cost and is simply not available in most of the world. In addition, the method does not provide a direct correlation between liquefaction resistance and blow count because the BPT blow count must be correlated with the SPT

blow count which increases the uncertainty relative to a direction correlation. In addition, energy loss from skin friction on the sides of the BPT have been a concern [18]. However, Ghafghazi et al. [19] have developed more sophisticated instrumentation for determining the energy delivered to the base of the BPT, which should reduce the uncertainty associated with skin friction. The BPT blow counts adjusted with this procedure have subsequently been correlated with SPT blow counts at several sites [19].

3. Development of Dynamic Cone Penetration Test (DPT) for gravels

A dynamic cone penetration test (DPT) was developed in China in the early 1950s to measure penetration resistance of gravel for application in bearing capacity analyses. Based on their experience, standard test procedures and code provisions have been formulated [20]. Because gravelly deposits are widespread beneath the Chengdu plain, the DPT is become widely used in that region, particularly for the evaluation of liquefaction potential [21, 22].

DPT equipment is relatively simple, consisting of a 120-kg hammer, raised to a free fall height of 100 cm, then dropped onto an anvil attached to 60-mm diameter drill rods which in turn are attached to a solid steel cone tip with a diameter 74mm and a cone angle of 60º as shown in Fig. 1. The smaller diameter rod helps to reduce shaft friction on the rods behind the cone tip. The cone is driven continuously into the ground.

Prior to testing, the drill rods are marked at 10 cm intervals and the number of blows required to penetrate each 10 cm is recorded. The raw DPT blow count is defined as the number of hammer drops required to advance the cone tip 10 cm. A second penetration resistance measure, called N_{120}, is specified in Chinese code applications where N_{120} is the number of blows required to drive the cone tip 30 cm; however, N_{120} is calculated simply by multiplying the raw blow count by a factor of three, which preserves the detail of the raw blow count record.

As with the standard penetration test, a correction for overburden stress on the DPT blow count was applied using the equation

$$N'_{120} = N_{120}C_n;\ C_n = \left(\frac{100}{\sigma'_v}\right)^{0.5} \leq 1.7 \qquad (1)$$

where N'_{120} is the corrected DPT resistance in blows per 30 cm, 100 is atmospheric pressure in kN/m^2, and σ'_v is the vertical effective stress in kN/m^2 [2]. This is identical to the overburden correction factor applied in [23] which facilitates comparisons. A limiting value of 1.7 was added to be consistent with the C_n used for other in-situ tests.

Energy transfer measurements were made for about 1200 hammer drops with the DPT in China using the conventional pulley tripod and free-fall drop weight system [2]. These measurements indicate that on average 89% of the theoretical hammer energy was transferred to the drill rods with this system and the standard deviation of the energy transfer was about 9%. Rollins et al. [24] found that the energy correction factor developed for the SPT could also be used to correct DPT blow count.

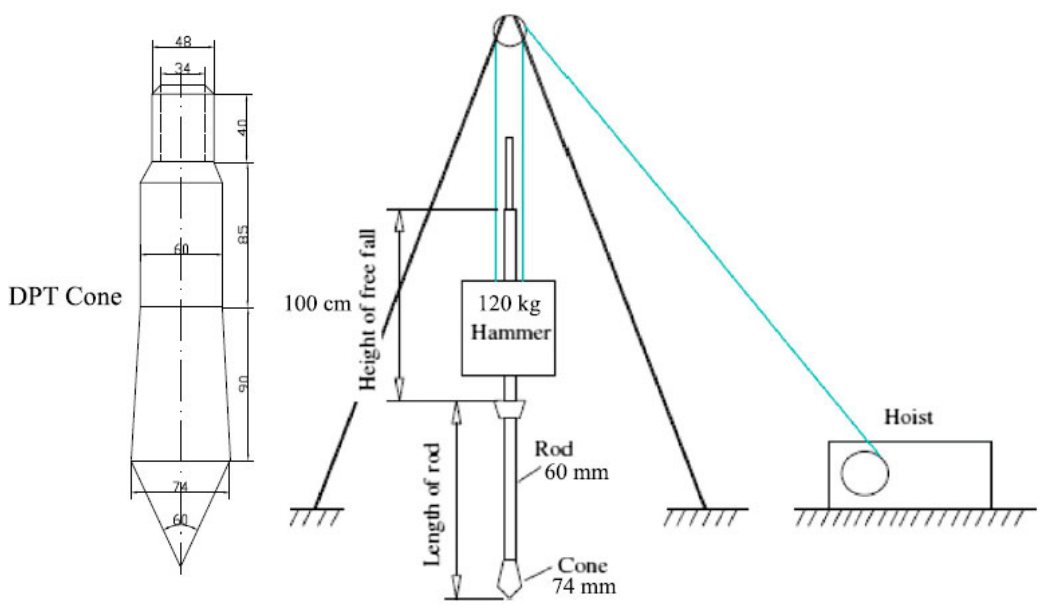


Figure 1. Component sketch of tripod and drop hammer setup for dynamic penetration tests (DPT) along with DPT cone tip. (After [2]).

4. Liquefaction resistance curve based on DPT penetration resistance

Following the 2008 M_w7.9 Wenchuan earthquake in China, 47 DPT soundings were made at 19 sites with observed liquefaction effects and 28 nearby sites without liquefaction effects. Each of these sites consisted of a 2- to 4-m thick surface clay layer, which, in turn, was underlain by gravel beds up to 500 m thick. The 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 90 to 100 mm diameter core barrels.

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 sites without liquefaction effects. Thus, low DPT resistance became a reliable identifier of liquefiable layers [21].

At the center of each layer, the cyclic stress ratio (CSR) induced by the earthquake was computed using the equation

$$\mathrm{CSR} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_{vo}}{\sigma'_{vo}}\right) r_d \quad (2)$$

where a_{max} is the peak ground acceleration, σ_{vo} is the initial total vertical stress, σ'_{vo} is the initial vertical effective stress, and r_d is a depth reduction factor as defined by [22].

Using DPT data, Cao et al. [2] plotted the cyclic stress ratio causing liquefaction against DPT N'_{120} for the M_w7.9 Wenchuan earthquake. Points where liquefaction occurred were shown as solid red dots, while sites without liquefaction were shown with open circles. Cao et al. [2] also define curves indicating 15, 30, 50, 70 and 85% probability of liquefaction based on logistical regression. Most other liquefaction triggering curves are calibrated for M_w7.5 earthquakes. To facilitate comparison with data points from other earthquakes, we have shifted the Cao et al. [2] data points and triggering curves in [2] upward to represent performance during a M_w7.5 earthquake using the equation

$$\mathrm{CSR}_{M_w 7.5} = \frac{\mathrm{CSR}}{\mathrm{MSF}} \quad (3)$$

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

$$\mathrm{MSF} = \frac{10^{2.24}}{M_w^{2.56}} \quad (4)$$

proposed by [22]. More recent magnitude scaling factor equations have been developed but they typically require an assessment of relative density or include SPT or CPT based parameters which makes their applicability questionable or problematic for gravel sites. For large magnitude earthquake events the differences in scaling factors are generally small. The data points and probabilistic triggering curves corrected for a M_w7.5 earthquakes are shown in Fig. 2.

The case histories in Valdez and L'Aquila with DPT test results provide an excellent opportunity to evaluate the ability of the DPT-based liquefaction triggering curves developed by Cao et al. [2] to predict accurately liquefaction in gravelly soil. For these case histories, the geology, earthquake magnitude, and stratigraphy are significantly different from those in the Chengdu plain of China and will provide a good test of the method.

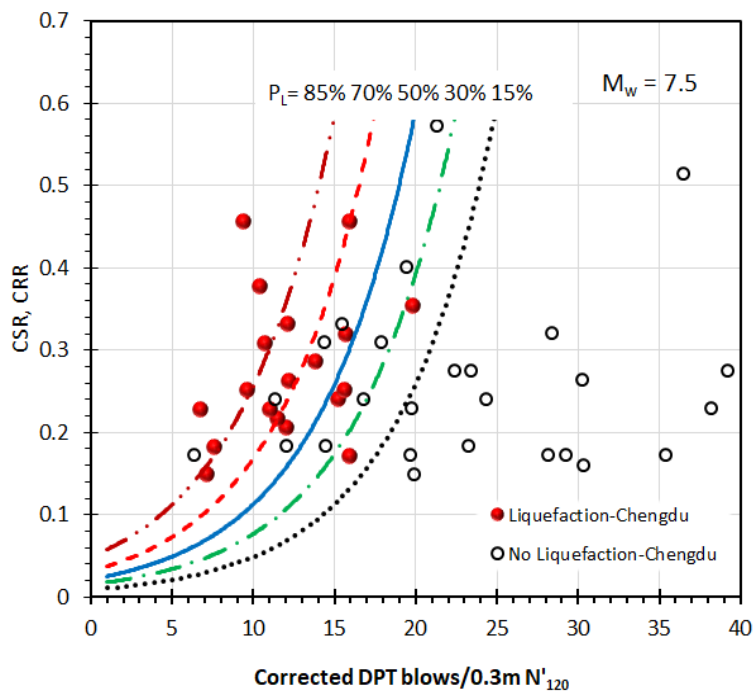


Figure 2. CRR vs. DPT N'_{120}triggering curves for various probabilities of liquefaction in gravelly soils developed by [Cao et al., 2013] adjusted for M_w7.5 earthquakes. Liquefaction/no liquefaction data points from sites on the Chengdu plain are also shown after adjustment to M_w7.5.

4.1 Liquefaction and site characterization at Valdez in Alaska

The geologic configuration at Port Valdez consists of one bedrock unit and three depositional complexes of unconsolidated sediments [25]. The Mineral Creek alluvial fan lying at the north west side of Old Valdez was deposited in an elongated depression between the main valley wall and a parallel outlying bedrock ridge. The fan has a slope of approximately 18 m above Mean Sea Level at the mountain front to sea level. Subsurface investigations performed by the US Geological Survey and Alaska Department of Highways show that this alluvial fan is underlain by more than 30 m of medium dense to very dense gravels with cobbles in a medium to coarse sand matrix. Because of the high tidal

range, broad tidal flats composed of silt, fine sand and organic muds are deposited at the seaward edge of the outwash delta and Mineral Creek.

4.2 Liquefaction effects

Liquefaction of gravelly soil occurred at the old port of Valdez following the M_w 9.4Alaska earthquake in 1964 [25]. Due to strong earthquake shaking, large fissures were observed to be opening and closing along the streets near the Valdez dock. Water saturated silt and sand ejected from many fissures. Above all, the most disastrous incident caused by the earthquake was a massive submarine landslide at the Valdez port near Mineral Creek. During the quake, the ship moored to the dock swung up and down violently up to a height of 6 to 9 m. Within seconds, the dock broke in two and the ground slid forward and vanished into the sea causing the loss of about 30 lives. Based on the investigation by USGS and the Alaska Highway Department [25], the fully saturated sandy gravel liquefied under the vibration of a critical intensity and duration. Besides the withdrawal of water due to low tide, the increased hydraulic gradient also reduced the effective stress at the toe of the slide. Thus, a number of factors combined together to cause the catastrophic failure. In addition to the massive landslide, liquefaction and lateral spread in the gravelly deposits further inland led to over 12 m of horizontal displacement. In contrast, the area situated far away on the other side of the sound from the old Valdez port had much denser gravel deposits [26] that did not show any sign of liquefaction manifestation during the earthquake. Thus, the location of the town and port facility was moved to new Valdez after the disaster took place at the old Valdez area.

The soil profile at the old Valdez port was subsequently investigated by the Alaska Department of Highways with rotary drilling. These borings extended to a depth of 14 to 15 m at all the locations. The soil profile was generally described as loose to medium dense sandy gravel up to a depth of 6 to 9 m, which is underlain by loose to medium dense gravelly sand containing thin lenses of silt.

4.3 DPT testing at Valdez

As part of this study, DPT soundings have been performed at two locations in old Valdez where gravel liquefied and two other sites in new Valdez where gravel did not liquefy. The location of these DPT soundings are shown in Fig. 3. The DPT tests were performed using a 154.4 kg automatic hammer with a drop height of 0.76 m. Hammer energy measurements were made using an instrumented rod section and a Pile Driving Analyzer (PDA) device from PDI, Inc.

Figure 3. Location of old Valdez (Site 1 and 2 towards right) and New Valdez points (Site 3 and 4 towards left). Google Earth© Base Map (2020)

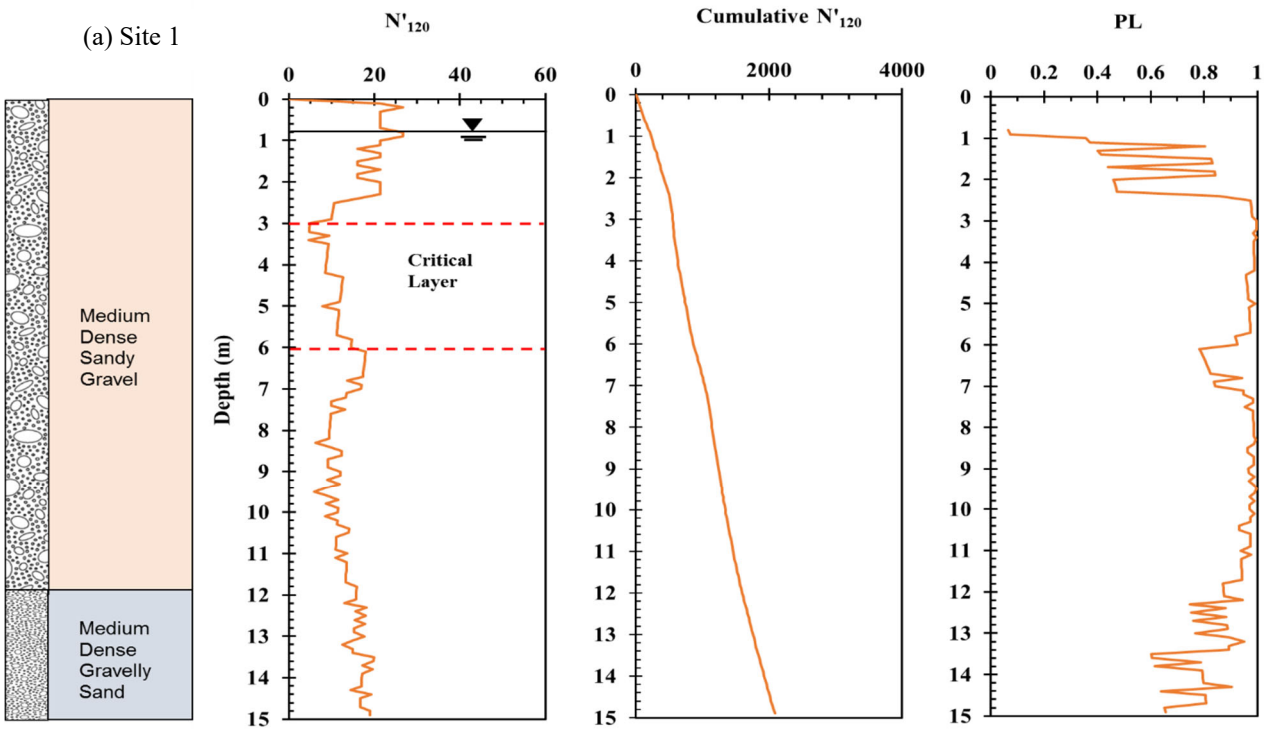
These energy measurements indicate that the hammer delivered 95% of the theoretical free-fall energy on average. Because the delivered energy was higher than the energy typically supplied by a Chinese DPT hammer, it was necessary to correct the measured blow count upward using the equation

$$N_{120} = N_{hammer}\left(\frac{E_{Delivered}}{E_{\text{Chinese DPT}}}\right) \quad (5)$$

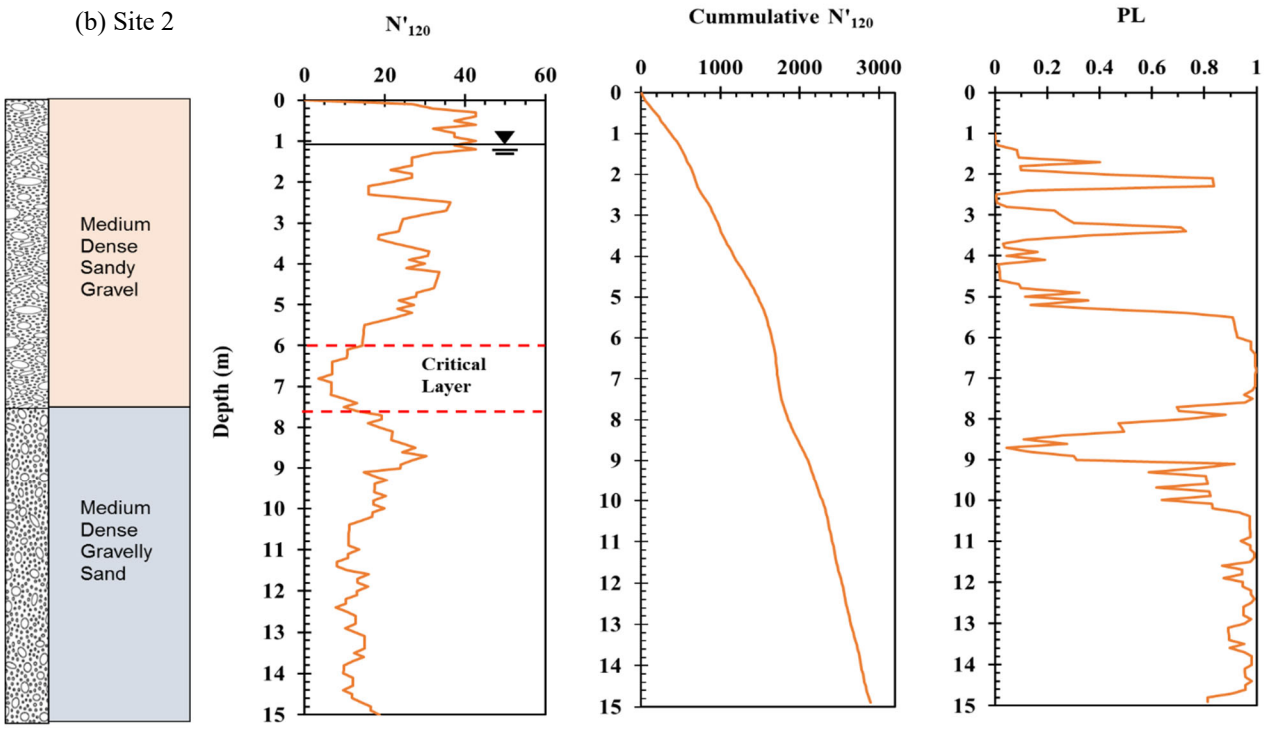
The ratio of energy actually delivered divided by the energy delivered by the Chinese DPT hammer was 1.04 for the 154.4 kg hammer. In addition, the overburden correction factor, C_n, (Eq. 1) was used to obtain the normalized N'_{120} value.

Plots of the energy corrected DPT N'_{120} versus depth for two sites in old Valdez and two sites in new Valdez along with the identified critical layer, cumulative N'_{120} and probability of liquefaction are provided in Fig. 4a through 4d. The probability of liquefaction has been obtained by using the equation given by Cao et al. [2].

(a) Site 1



(b) Site 2



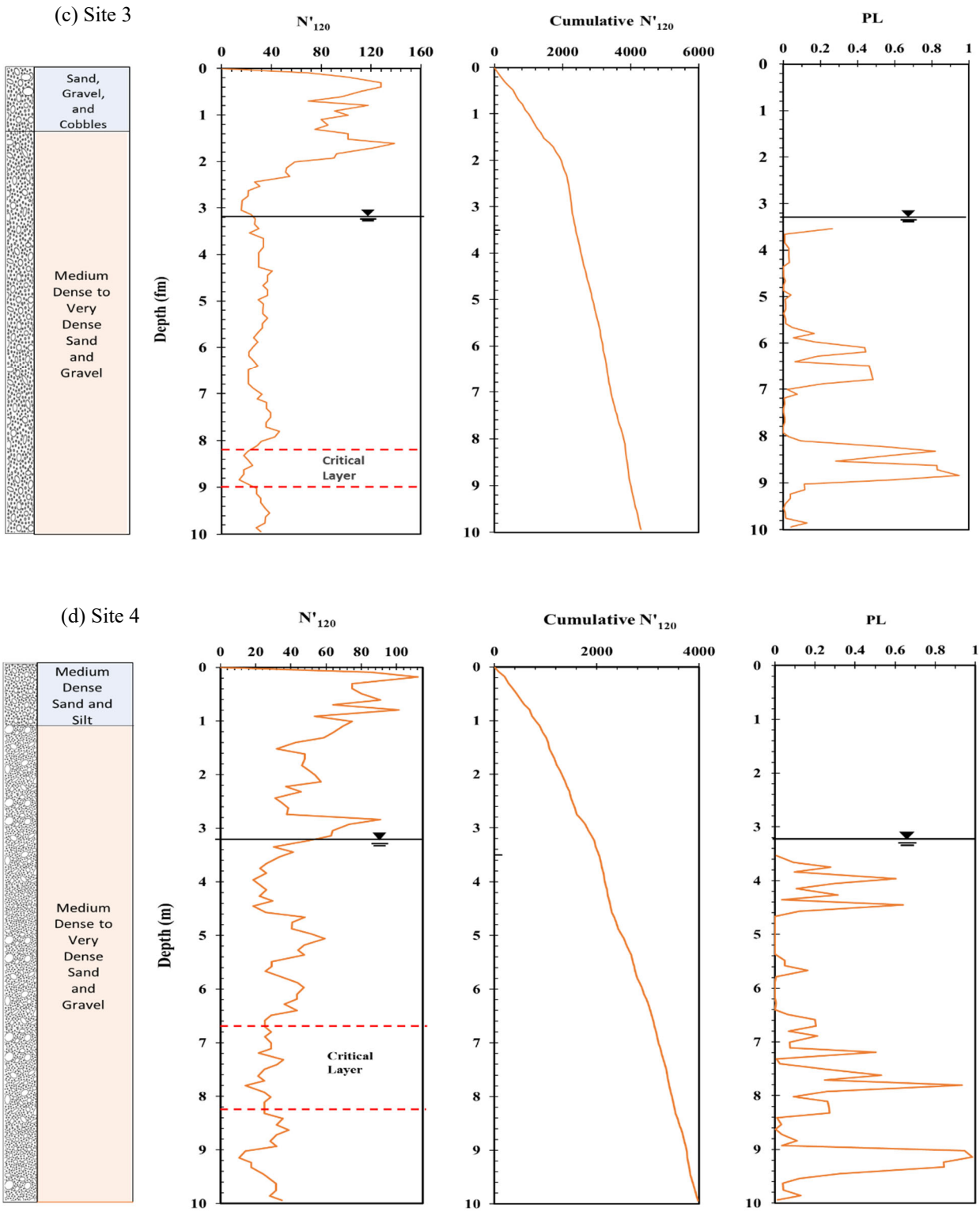


Figure 4. DPT blow counts, cummulative blowcounts and probability of liquefaction (PL) at (a) Site 1 and (b) Site 2 at old Valdez, and (c) Site 3 and (d) Site 4 at new Valdez, Alaska, USA

4.4 Liquefaction evaluations at Valdez

The DPT test profiles from Valdez provide an excellent opportunity to evaluate the ability of the DPT-based liquefaction triggering curves developed by Cao et al. [2] to predict liquefaction in gravelly soil. For each DPT sounding, we estimated the critical layer for liquefaction as illustrated in Fig. 4a through 4d. This zone was generally the loosest

average layer below the water table and closest to the surface or the layer. This layer typically has the highest probability of liquefaction an steepest slope on cumulative N'_{120} curve. The average DPT N'_{120} for each critical layer was plotted against the average *CSR* in the layer using a peak ground acceleration (*PGA*) of 0.44g and adjusted to a moment magnitude (M_w) of 7.5 using Eq. (4). The PGA was estimated from the USGS Shake map for the earthquake. Other soil and earthquake parameters for these sites e.g. σ'_{vo}, σ_{vo}, *CSR* and average DPT blow counts are shown in Table 2. The data pairs for each hole in old and new Valdez are plotted in Fig. 5 in comparison with the liquefaction triggering curves from [2] after magnitude scaling adjustments, which shifted the measured *CSR* values upward. The *CSR* values for these data points are the highest in the entire DPT liquefaction data set where data is particularly sparse. This fact makes them very important in constraining the shape of the probability curves at higher *CSR* and N'_{120} values. The two data pairs corresponding to the old Valdez area clearly plot above the 85% probability of liquefaction curve, which is consistent with the observed liquefaction effects. But the two points from new Valdez plot between the 30% and 50% probability curves which indicates a fair chance of liquefaction manifestation whereas no liquefaction effects were found in that area. Hence, this inconsistency for the new Valdez sites suggests that the triggering curves may need to shift to the left somewhat at these higher *CSR* values relative to the liquefaction triggering curves for gravelly soil developed so far.

Table 2. Soil and earthquake parameters for critical layer at old and new Valdez, Alaska sites.

Site	Avg. depth (m)	Avg. σ_o (kPa)	Avg. σ'_o (kPa)	Avg. N'_{120} (Blows per 0.3 m)	Avg. *CSR*
1	4.5	94.7	57.46	7.8	0.75
2	7	147.7	87.9	9	0.76
3	8.7	130	78.7	20.3	0.587
4	7.5	117	77	21.3	0.608

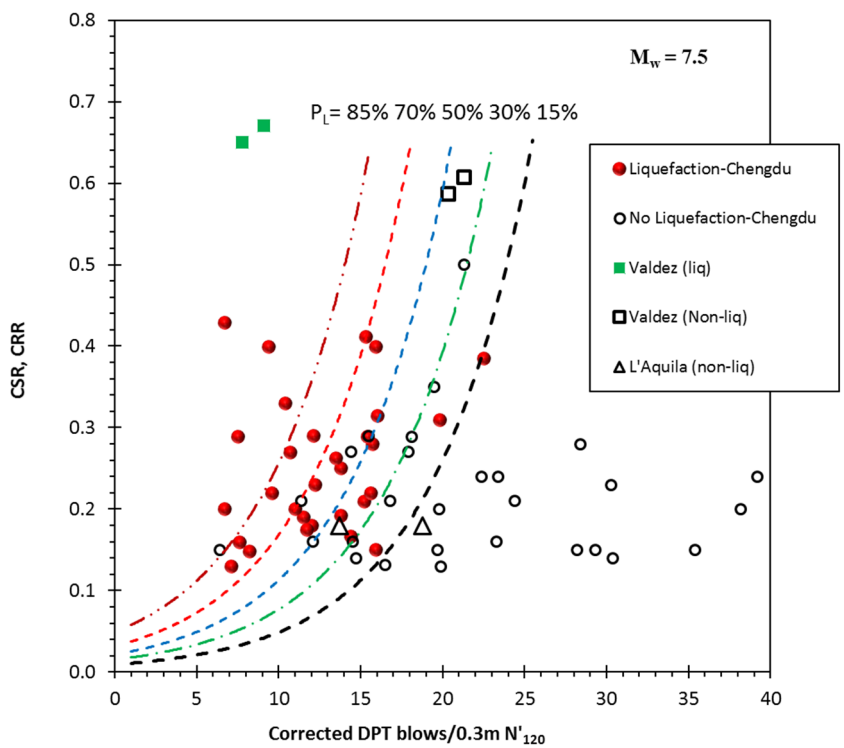


Figure 5. CRR vs. DPT N'_{120} curves for various probabilities of liquefaction in gravelly soils developed by Cao et al. [2] along with liquefaction/no liquefaction data points from Chengdu plain. Data points from old Valdez, new Valdez, Alaska and L'Aquila, Italy are also shown.

5. Liquefaction and site characterization at L'Aquila, Italy

The western L'Aquila Basin (WAB) is a typical Quaternary basin of the Central Apennines, extending in a WNW-ESE direction, between the structural units of the Gran Sasso and Ocre Mountains, along the Aterno-River valley. Continental sediment inside the WAB consists of lacustrine, fluvial and slope deposits characterized by a Pleistocene sedimentary sequence consisting of three main units namely: a clayey-sandy-gravel unit resting on bedrock, a gravelly-sandy-clay intermediate unit and the topmost

clayey-sand unit. The youngest deposit of the WAB corresponds to the alluvial unit of Holocene age, which represents the current stage of sedimentation in the Aterno River plain. The alluvial deposit consists of alternating layers of coarse gravels, sands and silty clays of fluvial and alluvial fan environments organized in lenticular bodies. More detail about the geological setting of L'Aquila can be found in [27].

In 2009, the L'Aquila basin was struck by a devastating earthquake sequence with a main shock having a moment magnitude (M_w) of 6.1 and two subsequent aftershocks. Although severe damage to infrastructure took place in the central town and surrounding villages due to the shaking, no sign of gravel liquefaction was observed either in the form of ejecta or significant settlement despite the presence of gravel layers throughout the valley [28, 29, 30]. Hence, this earthquake event provides several "no liquefaction" case histories for the gravelly deposits in the L'Aquila basin. Before the seismic network was deployed in 1993, both geotechnical and geophysical investigation was performed extensively on behalf of Servizio Sismico Nazionale to characterize the soil deposits [31]. Further, the geotechnical research group of the University of L'Aquila collected throughout the past years a preliminary selection of in situ and laboratory tests [32].

5.1 DPT testing at L'Aquila

As part of this study, DPT soundings were performed at two locations in L'Aquila. The location of these DPT boreholes are shown in Fig. 6. These sites are both located near the Aterno River. The DPT testing was performed using a 120 kg free-fall donut hammer with a drop height of 1.0 m. 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 120 kg hammer delivered 75% of the theoretical free-fall energy on average. The overburden and energy corrections were made using Eq. (1) and Eq. (5), respecitvely as explained previously. Plots of the corrected DPT N'_{120} versus depth profiles, along with the cumulative N'_{120} and probability of liquefaction profiles are given in Figs. 7a and 7b. The probability of liquefaction has been obtained using the equation given by Cao et al. [2] as in the case of Valdez.

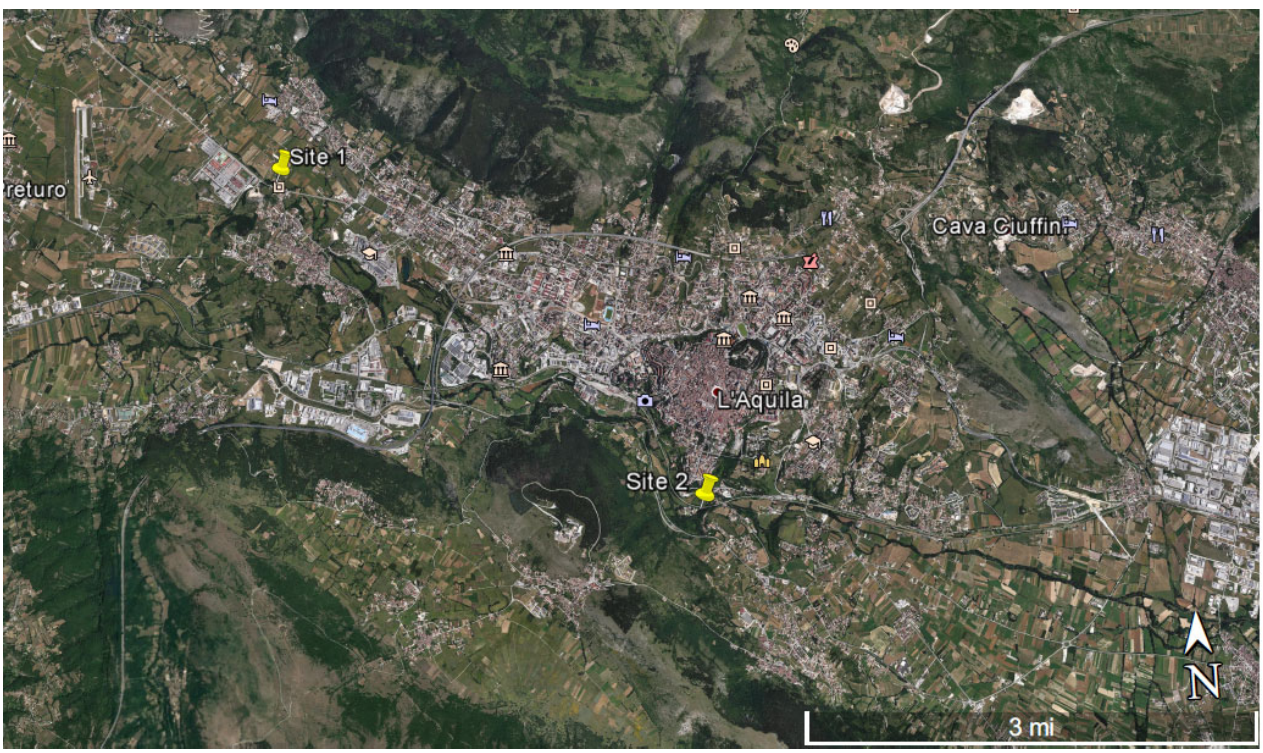


Figure 6. Location of L'Aquila Points (Site 1 and Site 2). Google Earth© Base Map (2020)

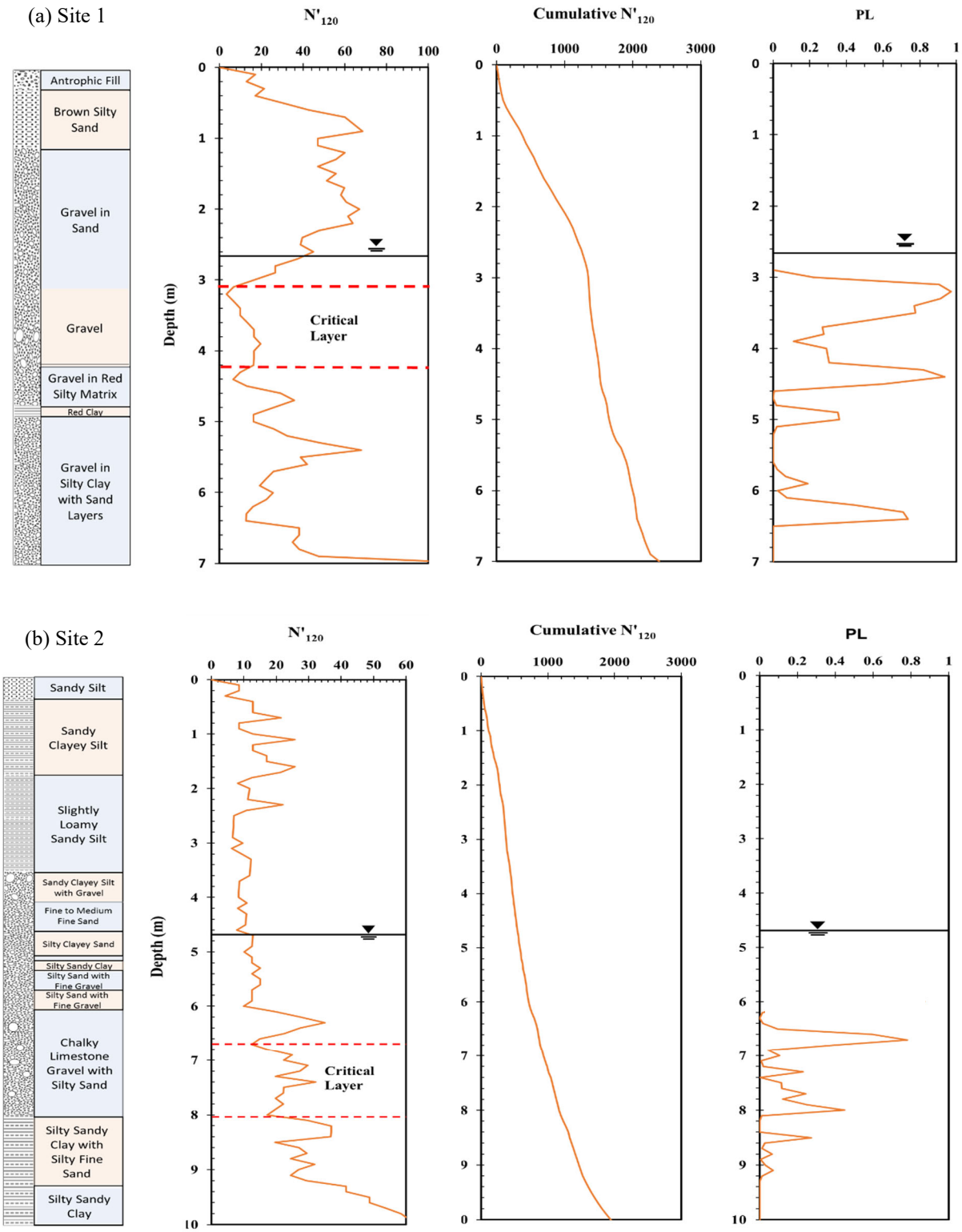


Figure 7. DPT blow counts, cummulative blowcounts, and probability of liquefaction at (a) Site 1 and (b) Site 2 [33] at L'Aquila (WWTP).

5.2 Liquefaction evaluations at L'Aquila

The DPT test profiles from L'Aquila provide another good opportunity to evaluate the ability of the DPT-based liquefaction triggering curves developed by Cao et al. [2] to evaluate liquefaction in gravelly soil. For each DPT

sounding, we estimated the critical layer for liquefaction as illustrated in Figs. 7a and 7b. This zone was generally the loosest average layer greater than 1 m below the water table and closest to the surface. Cohesive layers were not considered to liquefy. The average DPT N'_{120} for each critical layer was plotted against the average CSR in the layer using the *PGA* at the ground surface and adjusted to a moment magnitude M_w of 7.5 using Eq. (4). The *PGA* of 0.42g at site 1 was obtained from a seismograph station located about 30 m from the DPT sounding while the *PGA* of site 2 was taken from [34]. Other soil and earthquake parameters for the critical layers of both the sites e.g. total stress, effective stress, *CSR* and DPT blow counts are shown in Table 3. The data pairs for the two sites in the L'Aquila basin are plotted in Fig. 5 in comparison with the liquefaction triggering curves [2] after magnitude scaling adjustments. One of the data pairs (Site 2) clearly plots below the 15% probability of liquefaction curve, which is consistent with no observed liquefaction phenomena. But the other point (Site 1) plots between the 30% and 50% triggering curves which shows a moderate chance of liquefaction manifestation, whereas no liquefaction effect was found in that area. Hence, there is a certain discrepancy between the observed and predicted results that needs careful attention from researchers in imrpoving the probabilistic liquefaction triggering curves for gravelly soil.

Table 3. Soil and earthquake parameters for critical layer at L'Aquila, Italy sites.

Site	Avg. depth (m)	Avg. σ_o (kPa)	Avg. σ'_o (kPa)	Avg.N'_{120} (Blows per 0.3 m)	Avg. CSR
1	3.5	65.7	58.3	13.7	0.18
2	7.4	128.6	103.7	19.9	0.25

6. Conclusions

Based on the results of field investigations conducted in this study, the following conclusions have been developed:

1. The liquefaction triggering procedure developed by Cao et al. [2] correctly predicted the liquefaction of gravelly soil for two sites in old Valdez sites where liquefaction did actually occur during the 1964 Alaska earthquake. However, at two sites in new Valdez where liquefaction did not occur during the earthquake, the *CSR*-N'_{120} points lie between the 30% and 50% probabilistic triggering curves. Although the data points are below the 50% boundary, the probability of liquefaction is still higher than might be expected.
2. For the two sites in L'Aquila where liquefaction did not occur during the 2009 earthquake, the results were mixed. At one site, the data point clearly lies below the 15% probability of liquefaction curve, whereas the data point for the other site lies between the 30% and 50% triggering curve showing a moderate chance of liquefaction.
3. The data points collected in this study, particularly the "no liquefaction" points, provide important constraints on the DPT-based liquefaction triggering curves. This is particularly true for the data points in Valdez, Alaska where *CSR* values are high and data is sparse.
4. The test results suggest that new regressions using the DPT data points accumulating from around the world will be needed to reduce the uncertainty range between liquefaction and no liquefaction for gravelly soils.

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