Interpretation of CPT tests in liquefiable gravel

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ABSTRACT

Although gravelly soils have been observed to liquefy in 27 earthquakes in the past 120 years, many engineers believe that gravel cannot liquefy due to its high hydraulic conductivity. Gradations from gravel liquefaction case histories have shown these deposits typically contain 25 to 40% sand, reducing the hydraulic conductivity and enabling excess pore pressures to cause liquefaction. While cone penetrometers (CPT), typically used to evaluate liquefaction resistance in sand, may show increases in penetration resistance due to their small diameter relative to gravel particles, the CPT has successfully predicted gravel liquefaction for looser sandy gravels. Case histories in Wellington, New Zealand demonstrate the successful identification of gravel liquefaction hazards using CPT. Although some layers in the profile indicated high penetration resistance, most of the profile was correctly predicted to liquefy. The Soil Behavior Type (SBT) from the CPT did not consistently indicate a sandy gravel profile but was often classified as behaving like a sand or silty sand; likely influenced by higher sand percentages between gravel particles. To evaluate the ability of the CPT to characterize gravelly soils and their liquefaction potential, additional field case histories are desirable. This paper presents test results from two case histories, one in Wellington, New Zealand, and one in Petrinja, Croatia, where gravels have liquefied. In both cases, the CPT occasionally overestimated liquefaction resistance in gravel layers. The advantages of using a 74 mm diameter Dynamic Cone Penetrometer (DPT) are also highlighted with companion testing.

Keywords: Gravel liquefaction; Cone Penetration Test (CPT); Dynamic Cone Penetration Test (DPT); Petrinja, Croatia earthquake; Kaikoura, New Zealand earthquake.

1. Introduction

Although gravelly soils have been observed to liquefy in 25 earthquakes in the past 128 years (Rollins et al. 2021), many engineers believe that gravels are typically not liquefiable because of their high hydraulic conductivity or permeability. Early numerical studies investigating the generation and dissipation of excess pore pressures found that if the hydraulic conductivity of a gravel layer exceeds 0.004 m/sec, that excess pore pressures would dissipate as fast as they were generated (Seed et al, 1976). This finding has generally been confirmed in recent studies by Roy and Rollins (2022). Although clean gravels with low sand and fines contents may have permeabilities higher than 0.004 m/sec, this permeability can decrease dramatically as sand content increases (She et al 2006). When sand contents reach about 30%, the sand may occupy much of the void space and the permeability will be much lower than the 0.004 m/s limit depending on the grain size of the sand. For these sand contents, the permeability may be closer to that of sand than that of gravel as illustrated in Fig. 1 and the sandy gravel could develop excess pore pressure during an earthquake resulting in liquefaction.

Gradation curves for several gravel liquefaction case histories around the world are summarized in Fig. 2. While gravel may make up the majority of the coarse-grained fraction of the soil, the sand content is typically higher than 20% with fines contents between 1 and 23%. While the permeability coefficient of these liquefied sandy gravel mixtures has not typically been reported, estimates of the permeability of these soils have been made using the Kozeny-Carmen equation (Roy and Rollins 2022). Fig. 1 shows the permeability estimated for all the gravel sites that have liquefied plotted versus sand content. In all cases, the permeabilities are lower than 0.004 m/sec, explaining why they were susceptible to liquefaction.

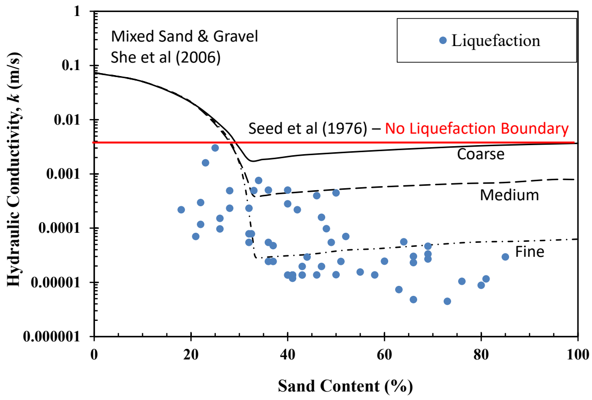


Figure 1. Grain-size distribution curve for several case histories where gravelly soils have been observed to liquefy.

Therefore, identification of liquefiable sand and gravel mixtures may require some understanding of the overall permeability of the mixture or the particle size distribution which may be controlled by the D_{10} size of the mixture (Kozeny, 1927). These factors produce uncertainty regarding the liquefaction evaluation of gravelly sites using these standard testing methods.

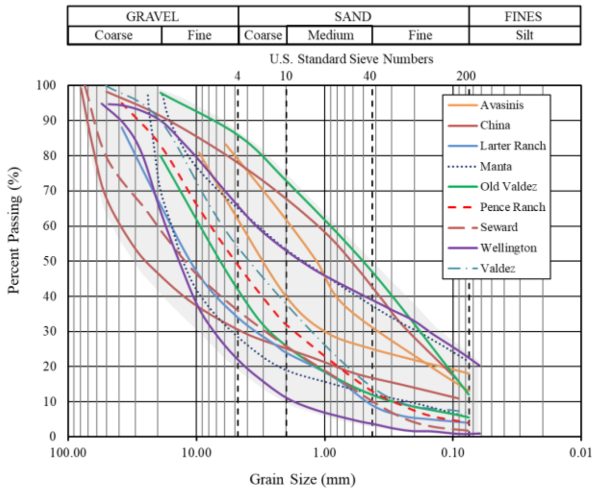


Figure 2. Grain-size distribution curve for several case histories where gravelly soils have been observed to liquefy.

Liquefaction potential in sands and silty sands is typically evaluated using the Cone Penetration Test (CPT). However, the CPT can become less reliable for gravelly soils e.g., reclaimed fills which often contain inhomogeneous soils with gravel or clean gravel layers (Tokimatsu 1998), due to interference of the penetrometer with large gravel particles. Based on Discrete Element Modeling (DEM), Iqbal et al. (2004) reported that the CPT is likely to reach refusal when the D_{50} size is greater than the size of the penetrometer and that interference effects start to artificially increase the cone resistance (q_c) when D_{50} is about one-third the size of the penetrometer. However, field testing indicates that conventional Standard Penetration Test (SPT) and the CPT may be able to correctly evaluate the liquefaction potential of loose gravelly layers when the penetration resistance is low (Andrus 1994, Kokusho and Yoshida 1997, Rhinehart 2016, and Dhakal et al. 2020a). In addition, the CPT may be successful in evaluating those gravelly deposits which are composed of gravel-sand-silt mixtures where the finer fractions (silt and sand) significantly influence the behavior of the entire soil layer (Cubrinovski et al. 2018, Dhakal et al. 2020b, and Roy and Rollins 2022).

Nevertheless, in medium dense to dense layers consisting of large gravel particles, the cone may not successfully penetrate, making it necessary to drill through a dense layer to continue advancing the cone through the remainder of the depth. Sometimes, when penetration resistance at a site increases, it becomes increasingly difficult to determine if the increased resistance is because of the increased density of the soil or because of interference with large particles. Penetration resistance may even reach refusal in some cases when the soil is not particularly dense (Cao et al. 2013).

To evaluate the ability of the CPT to characterize gravelly soils and the liquefaction potential of these deposits, additional field case histories are desirable. This paper presents test results from two case histories, one in Wellington, New Zealand, and one in Petrinja, Croatia, where gravels have liquefied. In both cases the CPT was generally capable of identifying the liquefaction hazard. However, the advantages of using a 74 mm diameter Dynamic Cone Penetrometer (DPT) were also highlighted with companion testing.

2. Geotechnical Testing at Gravel Liquefaction Sites in Petrinja, Croatia

The M_w6.4, Petrinja, Croatia earthquake produced liquefaction on alluvial plains along the Kupa, Sava, and Glina rivers. Sandy gravel ejecta was noted at six sites (Baize et al. 2022) in and around Petrinja with peak ground accelerations (PGA) between 0.4 and 0.5 g (USGS Shakemap, 2020). At each site, a borehole was drilled to define the soil profile and at Sites 1 and 5, a CPT was also performed.

2.1. CPT testing

The soil profile and CPT cone resistance for Sites 1 and 5 are plotted vs. depth in Fig. 3. The profiles show a sandy silt (ML) or sandy clay (CL) layer from near the ground surface to depths of 6.5 and 9.5 for Sites 1 and 5, respectively. Below the surface layer, the soil profile indicates layers of silty gravel with sand or gravelly sand that is likely the source of the ejecta identified by post-earthquake reconnaissance.

Based on the q_c and f_s values, the soil behavior type (SBT) index (I_c) was obtained using the following equations proposed by Robertson and Wride (1998).

$$I_c = \{[3.47 - \log(Q_{tn})]^2 + [1.22 + \log(F_r)]^2\}^{0.5} \quad (1)$$

where Q_{tn} and F_r are normalized cone resistance and sleeve friction ratios computed using the equations

$$Q_{tn} = \left(\frac{q_c-\sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma'_v}\right)^n \quad \text{and} \quad (2)$$

$$F_r = \left(\frac{f_s}{q_c-\sigma_v}\right).100\% \quad (3)$$

where σ_v is the vertical stress and P_a is atmospheric pressure (100 kPa). The Q_{tn} and F_r values within the gravelly soil layers at the two sites have been plotted relative to the Soil Behavior Type (SBTn) chart in Fig. 4. In the surface layers, the *SBT* type is typically 3 or 4 with an I_c value between 2.6 and 3.3. In the gravelly layers where gravel content varies from a low of 27% to a high of 56% and one might expect the data points to plot in Zone 7 for gravelly sand, the data points typically plot in Zones 5 or 6; typical of sand or sand mixtures. This suggests that sand is controlling the behavior of the sandy gravel mixture (Roy 2023, Chang et al. (2014).

2.2. DPT Testing

In addition to the CPT soundings, a companion DPT sounding was made at both sites (Amoroso et al. 2023)

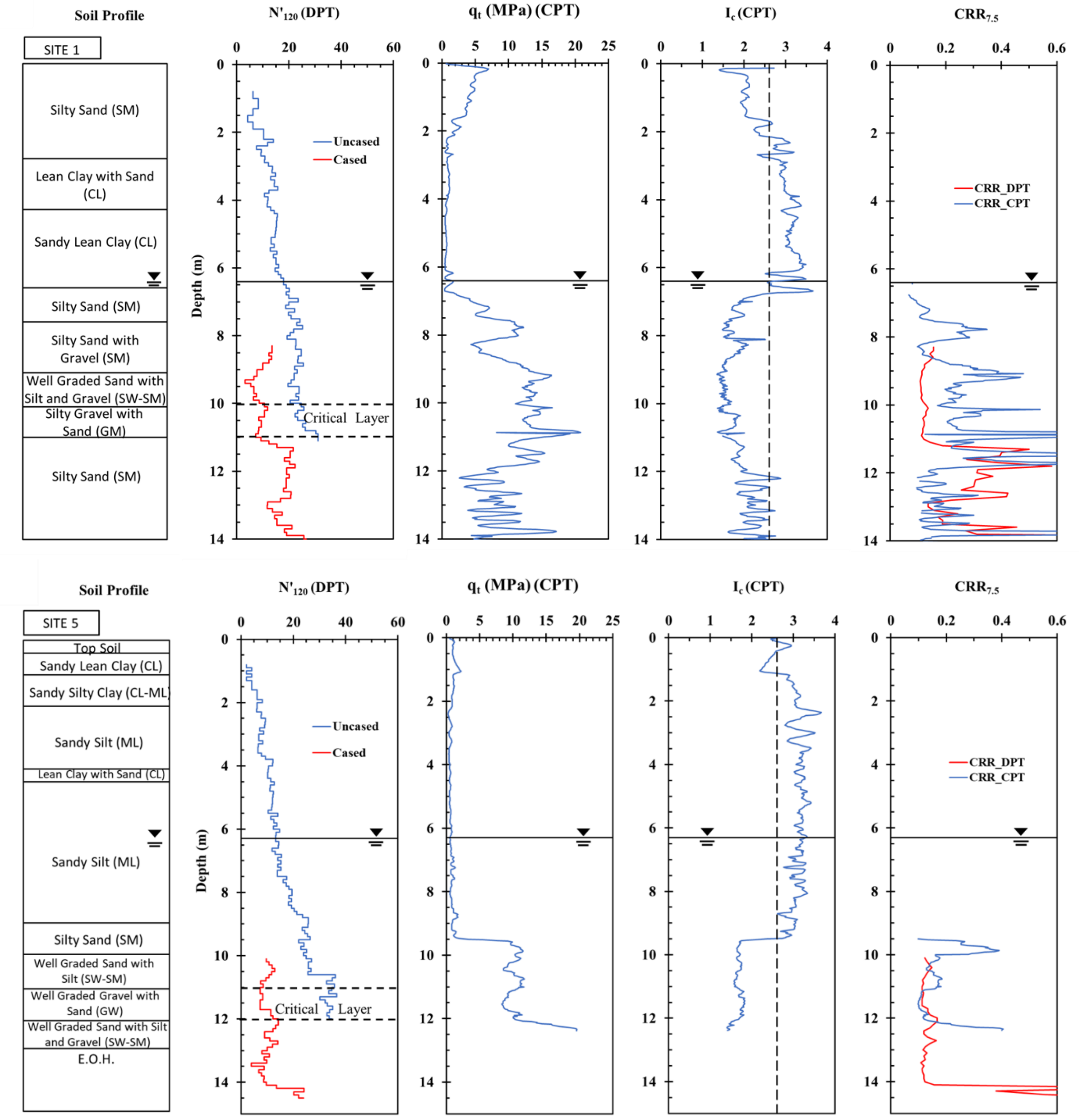


Figure 3. Corrected DPT blow counts, CPT cone resistance (qt), soil behavior type (I_c), and CRR based on DPT and CPT.

adjacent to the CPT sounding. The Dynamic Cone Penetration Test (DPT), developed in China to measure the penetration resistance of gravels, provides an alternative method for evaluating liquefaction in gravels (Cao et al. 2013). The DPT employs a relatively simple 60° cone penetrometer with a 74 mm diameter that is driven into the ground by a 120 kg hammer with a freefall height of 100 cm using a 60 mm drill rod that reduces skin friction on the rods. At 74 mm, the DPT diameter is 110% larger than a standard 10 cm^2 CPT which makes the equipment more effective in penetrating medium to coarse gravels to produce meaningful evaluations of soil resistance.

The blow count or penetration resistance (N_{120}) for the DPT is defined as the number of blows required to drive the penetrometer through 30 cm of penetration. As with the CPT, an overburden stress correction factor is applied using the equation:

$$N'_{120} = N_{120}C_N;\ C_N = \sqrt{100/\sigma'_v} \leq 1.7 \tag{4}$$

where N'_{120} is the overburden pressure 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. A limiting value of 1.7 was applied to be consistent with the C_N values used in other in-situ tests.

Plots of the DPT N'_{120} vs depth are also provided in Fig. 3 for Sites 1 and 5. Tests were originally performed without casing and the DPT N'_{120} increased with depth in the cohesive surface layer even though the CPT q_t remained essentially constant within this layer. This result strongly suggests that friction is developing on the drill rods during penetration and artificially increasing the blow count. To deal with this problem, a cased hole

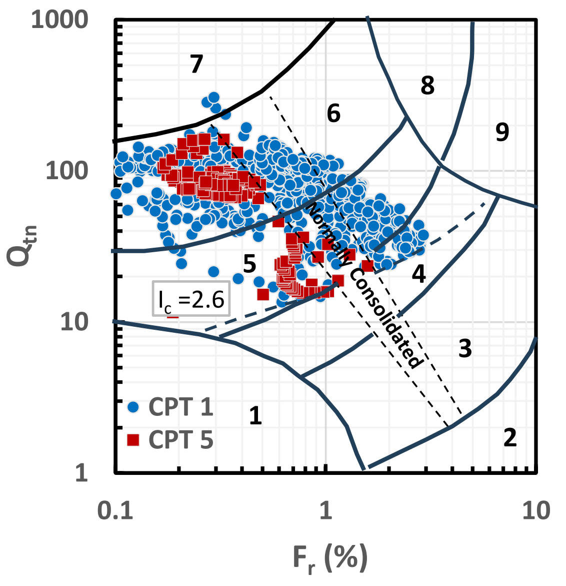


Figure 4. Distribution of SBT index (I_c) for gravel layers that liquefied in the Petrinja Earthquake.

was used to the base of the cohesive layer at both sites and the DPT was resumed below the casing to eliminate friction in the surface layer. The DPT N'_{120} vs depth curves for the cased hole show much lower blow counts relative to the uncased hole confirming that drill rod friction was developing.

2.3. Comparison of cyclic resistance ratio (CRR) from CPT and DPT testing.

Using the CPT test results, the cyclic resistance ratio (CRR) for a 15% probability of liquefaction was computed for Sites 1 and 5 using the Idriss and Boulanger (2008) approach. In addition, the results from the DPT testing were used to compute the CRR for a 15% probability of liquefaction at these two sites using the Rollins et al. (2022) approach. Fig. 3 shows plots of the CPT- and DPT-based CRR values versus depth in the sand and gravel layers below the cohesive surface layer that is not liquefiable (I_c>2.6) for comparison purposes. The agreement between the two tests is relatively good for Site 5. However, at Site 1 the CRR from the CPT is overestimating liquefaction resistance relative to the DPT CRR. This is likely due to interference with large gravel particles.

3. Geotechnical Testing at Gravel Liquefaction Sites in Wellington, New Zealand

The 2016 M_w7.4, Kaikoura earthquake in New Zealand produced significant settlement and lateral spreading at CentrePort in Wellington. Sand and gravel ejecta was pervasive throughout the port as reported by Cubrinovski et al. 2017 with recorded peak ground accelerations (PGA) of about 0.25g at nearby seismographs. The reclamation fills for the port consisted of loose sandy gravel that was simply end-dumped into the ocean without compaction. However, the top 3 m of the soil profile above the water table was densely compacted.

3.1. CPT testing at CentrePort in Wellington

Geotechnical testing at CentrePort in Wellington, New Zealand included 121 CPTs to characterize the soil in the reclaimed area of the port (Cubrinovski et al. 2018). The CPTs were performed with 10 cm^2 and 15 cm^2 cones. The top 3 m of the reclamation fill was very dense and it was necessary to predrill through this material. If early refusal was encountered during any CPT sounding at depths less than about 10 m, the cone was pulled up and drilling was performed using casing beyond the point of refusal until the soil became looser. Then, the CPT rig was brought back into position, and penetration was resumed below the casing. Out of 75 CPTs in the Thorndon Reclamation zone, there were 17 cases (23%) where refusal was encountered, and it was necessary to predrill to allow further penetration of the CPT (Roy and Rollins, 2023). In addition to CPTs, SPTs were performed to collect disturbed test samples. Particle size distribution curves from the SPTs and other investigations are shown in Fig. 5. Gravel content ranged from about 45 to 70%.

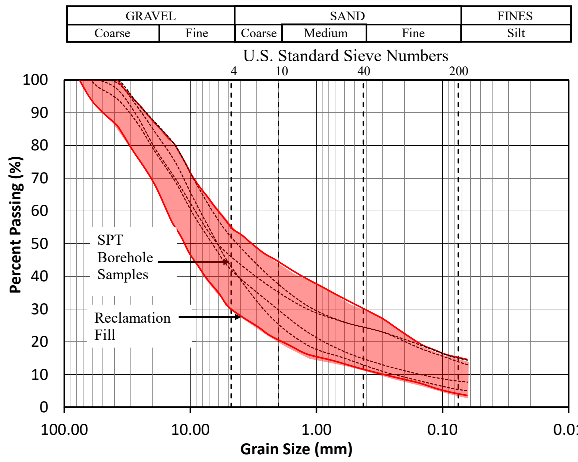
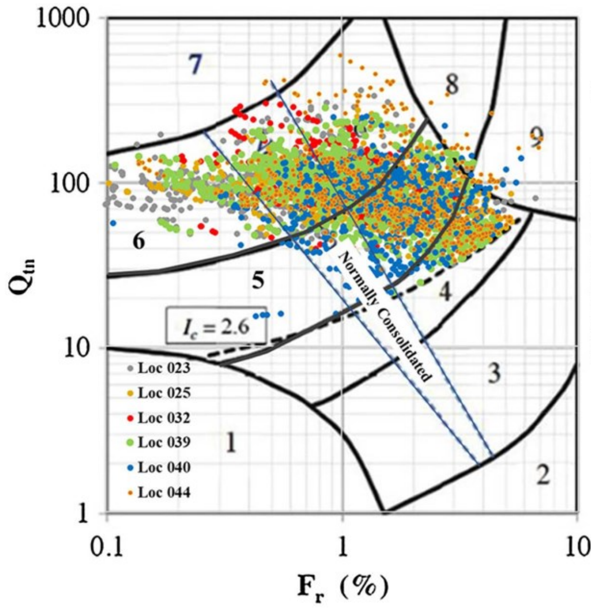


Figure 5. Grain-size distribution curve for reclamation fill at the port of Wellington, NZ.

The Q_{tn} and F_r values within the gravelly fill materials for six CPTs have been plotted relative to the Soil Behavior Type (SBTn) charts in Fig. 6. While plotting these data, all the layers having I_c>2.6 have been eliminated as those layers can be considered as non-liquefiable. Even though these deposits contained 45 to 70% gravel, nearly all the data points fall into the zones for sands, sandy mixtures and silt mixtures except for a few points along the border between the sand and gravelly sand zone. Hence, the SBT chart clearly indicates that the behavior of the fill material is governed by the sand and silt fraction although there is a significant gravel percentage in the soil matrix. Since the loose sandy gravel reclamation fill was not compacted it would be expected to be normally consolidated; however, a significant percentage of the data points plot above the normally consolidated wedge as shown in Fig. 6 indicating overconsolidation. Overconsolidation could have been produced by loading and unloading of cargo container stacks applying pressures as high as 40 kPa.



Soil Behavior Type (Robertson, 1998)

1 Sensitive fine-grained	6 Sand
2 Organic	7 Gravelly sand to sand
3 Clay	8 Very sitff sand to clayey Sand
4 Silt Mixtures	9 Very stiff fine-grained
5 Sand Mixtures	

Figure 6. Distribution of SBT index (I_c) for gravel layers that liquefied in the Kaikoura Earthquake.

The soil profile and CPT cone resistance is shown for location 023 in Fig. 7 and location 025 in Fig. 8. These logs also show the critical layer or the layer most likely to liquefy based on the CPT sounding. In both Figs. 7 and 8 the CPT reached refusal and it was necessary pull out the CPT, drill through the denser layer, and then reinsert the CPT to complete the sounding leaving gaps in the profile.

3.2. DPT testing at CentrePort in Wellington

To provide a comparison with the CPT soundings, DPT soundings were also performed to depths of about 14 m within 1 to 2 meters of existing CPT soundings at six locations at CentrePort. DPT blow counts vs. depth are shown adjacent to the CPT cone resistance in Fig. 7 and 8. While the CPT reached refusal once or twice at these locations, the DPT was able to penetrate these layers without difficulty. The critical layers for the DPT soundings are also shown in Figs. 7 and 8 and they are close to or coincident with the critical layer from the CPT.

3.3. Comparison of cyclic resistance ratio (CRR) from CPT and DPT testing at CentrePort in Wellington

Using the CPT test results, the cyclic resistance ratio (CRR) for a 15% probability of liquefaction was computed for locations 023 and 025 using the Idriss and Boulanger (2008) approach. In addition, the results from the DPT testing were used to compute the CRR for a 15% probability of liquefaction at these same two locations using the Rollins et al. (2022) approach. Figs. 7 and 8 provide a plot of the CPT- and DPT-based CRR values versus depth in the loose sandy gravel layers that comprise the reclamation fill. In general, Figs. 7 and 8 show that the DPT-based CRR profiles are closely aligned with the average CPT-based CRR profiles excluding the occasional spikes. However, the intermittent spikes observed in the CPT-based CRR profiles are absent in the DPT-based CRR profiles. This inconsistency between the DPT and CPT results can be largely explained by the fact that the DPT provides a larger penetrometer diameter to particle size diameter. This larger ratio helps make the DPT less susceptible to artificial increases in the blow count. Nevertheless, both the CPT and the DPT methods predict that liquefaction would occur in the loose sandy gravel at the port.

4. Observations and Conclusions

Based on the cone penetration testing (CPT) and dynamic cone penetration testing (DPT) conducted in gravelly soils impacted by the 2020 M_w6.4 Petrinja, Croatia earthquake and by the CPT and DPT testing at the CentrePort in Wellington impacted by the 2016 M_w7.8 Kaikoura, New Zealand earthquake the following observations and conclusions are presented:

1. The Q_{tn} and F_r pairs from the CPT soundings in loose liquefiable gravels from Croatia and New Zealand do not plot in Zone 7 ($I_c \approx 1.3$) which is thought to contain gravelly sands (Robertson e al. (1998). Instead, they plot in Zones 4, 5 and 6, for silt mixtures, sand mixtures, and sand, respectively. This result indicates that the percentage of sand and silt was sufficient to dominate the behavior of the gravel. The sand percentage lowered the permeability of the gravel so that excess pore pressure could develop and cause liquefaction during an earthquake.
2. The Cyclic Resistance Ratios (CRRs) computed using the DPT- and CPT-based gravel liquefaction assessment techniques indicate that the CPT can overestimate the liquefaction resistance in some gravelly soils. This is likely due to interference between the relatively small diameter of the CPT and large gravel particles. Additional studies considering the gravel content and maximum particles size would be desirable.
3. Improved methods are needed to better identify materials that might induce friction on the drill rods and artificially increase the DPT blow count. Likewise, methods to measure energy loss due to friction on DPT drill rods are needed to directly measure energy loss from friction.

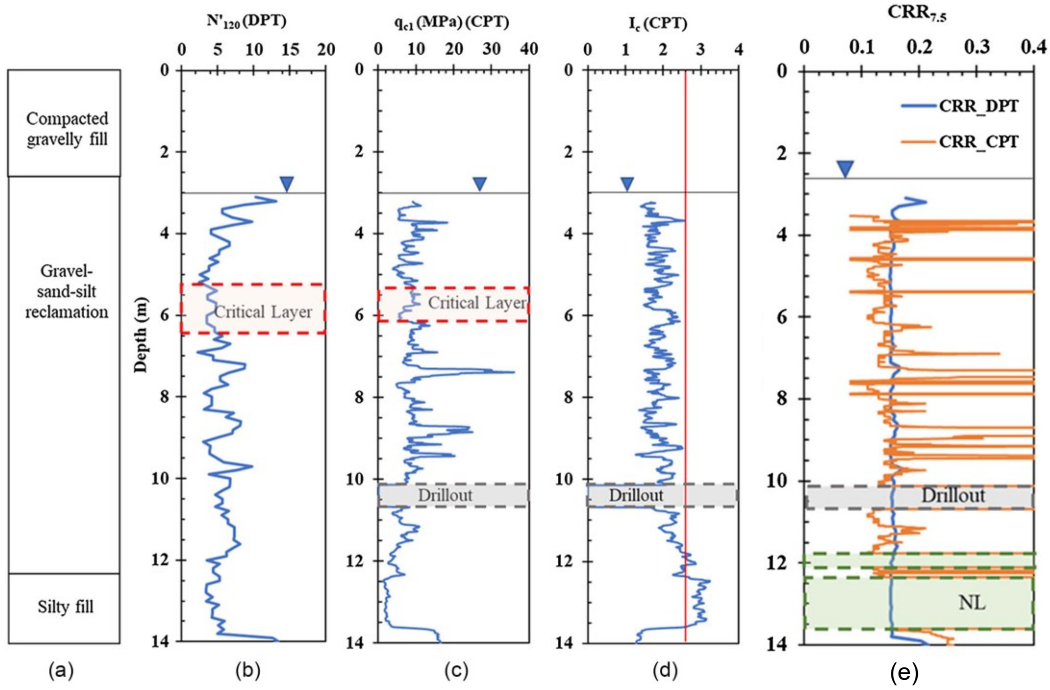


Figure 7. Plots of (a) soil profile, (b) DPT blow count, N'$_{120}$, and critical layer for liquefaction based on DPT. (c) CPT cone resistance, q_{c1} and critical layer for liquefaction based on CPT, and (d) soil behavior type, I_c , and (e) CRR from CPT and DPT at location 023. (Note: NL indicates "Non-Liquefiable based on I_c).

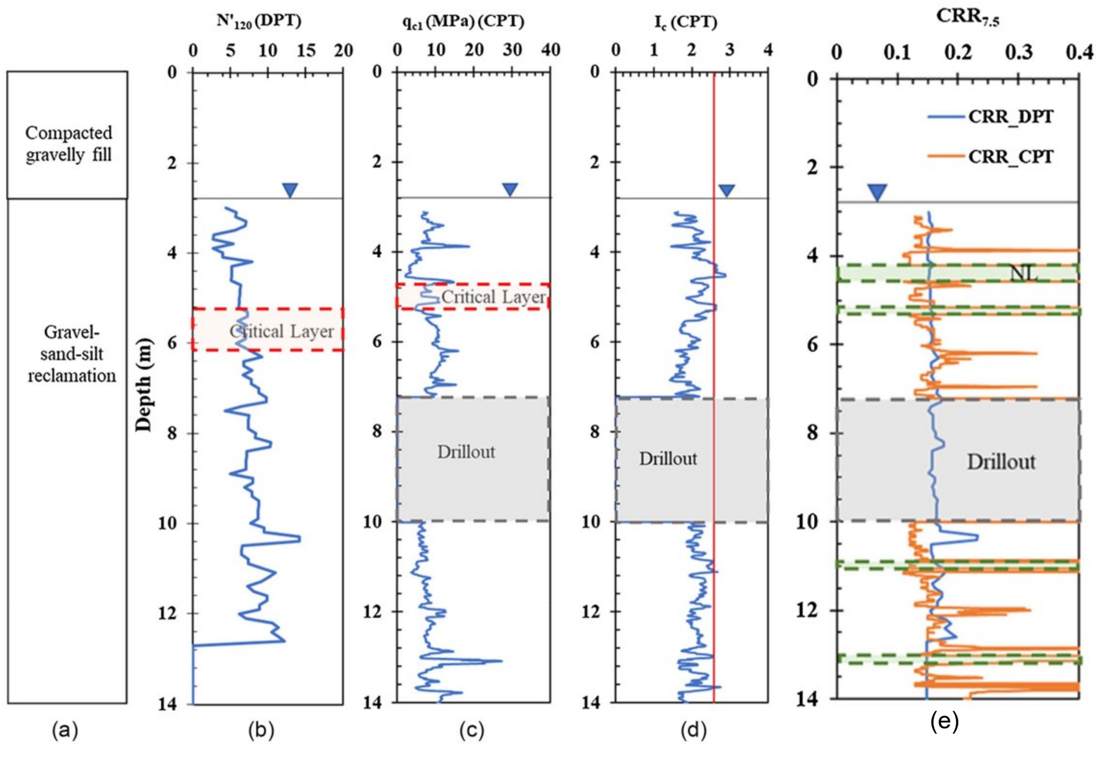


Figure 8. Plots of (a) soil profile, (b) DPT blow count, N'$_{120}$, and critical layer for liquefaction based on CPT. (c) CPT cone resistance, q_{c1} and critical layer for liquefaction based on CPT, and (d) soil behavior type, I_c and (e) CRR from CPT and DPT at location 025. (Note: NL indicates "Non-Liquefiable" based on I_c).

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