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Geotechnical Engineering Challenges to Meet Current and Emerging Needs of Society

Les Défis de la Géotechnique pour Répondre aux Besoins Actuels et Émergents de la Société

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Editors
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GEOTECHNICAL ENGINEERING CHALLENGES TO MEET CURRENT AND EMERGING NEEDS OF SOCIETY

'Geotechnical Engineering Challenges to Meet Current and Emerging Needs of Society' includes the papers presented at the XVIII European Conference on Soil Mechanics and Geotechnical Engineering (Lisbon, Portugal, August 26 to 30th, 2024). The papers aim to contribute to a better understanding of problems and solutions of geotechnical nature, as well as to a more adequate management of natural resources. Case studies are included to better disseminate the success and failure of Geotechnical Engineering practice. The peer-reviewed articles of these proceedings address the six main topics:

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Liquefaction resistance of carbonate sands from cone penetration tests

Résistance à la liquéfaction des sables carbonatés issue d'essais de pénétration au cône

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ABSTRACT: Liquefaction assessment is normally carried out using semi-empirical approaches based on the pioneering study of Seed and Idriss (1982). The methods relate a penetration resistance measured in situ (preferably from cone penetration tests, CPTs) to observed instances of liquefaction and non-liquefaction of sandy deposits. However, the applicability of a simplified approach is questionable for materials which fall outside the range of silica sands and slightly silty sands for which it has been developed and extensively verified against field data. For such cases a more rigorous method is needed. Carbonate sands are well known and documented to behave differently from silica sands: at the same relative density, penetration resistance is generally lower than in siliceous sands, while cyclic laboratory testing indicates higher cyclic resistance. These observations cannot rationally be combined into the commonly used simplified liquefaction procedures. An experimental approach, based on the measured mechanical properties of the carbonate sands, is essential. This paper provides an example of calibration of liquefaction resistance from CPTU for a fill made of dredged carbonate sand, based on centrifuge CPTUs and cyclic laboratory tests.

RÉSUMÉ: L'évaluation de la liquéfaction est normalement effectuée en utilisant des approches semi-empiriques basées sur l'étude pionnière de Seed et Idriss (1982). Les méthodes relient une résistance à la pénétration mesurée in situ (de préférence à partir de tests de pénétration au cône, CPTs) à des instances observées de liquéfaction et de non-liquéfaction de dépôts sablonneux. Cependant, l'applicabilité de l'approche simplifiée est discutable pour les matériaux qui sortent du cadre des sables de silice et des sables légèrement silteux pour lesquels elle a été développée et largement vérifiée contre des données de terrain. Pour de tels cas, une méthode plus rigoureuse est nécessaire. Les sables carbonatés sont bien connus et documentés pour se comporter différemment des sables de silice: à la même densité relative, la résistance à la pénétration est généralement plus basse que dans les sables siliceux, tandis que les tests de laboratoire cycliques indiquent une résistance cyclique plus élevée. Ces observations ne peuvent pas rationnellement être combinées dans les procédures simplifiées de liquéfaction couramment utilisées. Une approche expérimentale, basée sur les propriétés mécaniques mesurées des sables carbonatés, est essentielle. Cet article fournit un exemple de calibration de la résistance à la liquéfaction à partir de CPTU pour un remblai fait de sable carbonaté dragué, basé sur des CPTUs de centrifugeuse et des tests de laboratoire cycliques.

Keywords: Liquefaction; carbonate sand; CPT; centrifuge.

1 INTRODUCTION

There is very little guidance in the literature on seismic liquefaction assessment of carbonate sands. Available information suggests that these materials have crushable grains and can be significantly more compressible than silica sands at the same relative density, resulting in a penetration resistance generally lower, Marioka and Nicholson (2000). However, laboratory testing such as cyclic simple shear or triaxial testing, indicates that, at the same relative density, carbonate sands have a higher cyclic resistance (Hyodo et al. 1998, Marioka and Nicholson 2000, Brandes 2011, Sandoval et al. 2011). These observations cannot rationally be combined into the commonly used simplified liquefaction procedures. This paper provides an example of a liquefaction assessment for a carbonate sand through an uncoupled total stress approach. The assessment compares the resistance to seismic loading to the seismic demand in terms of a factor of safety, FS_{liq}. The pore water pressure generated by seismic loading is computed from the value of FS_{liq}. Referring to a case study of a hydraulic fill made of dredged carbonate sands, the steps of the assessment are the following:

- Determine seismic loading (cyclic stress ratio) based on site-specific ground response analysis;
- Determine in situ state parameter ψ ($\psi = e - e_{cs}$ where e = current void ratio; e_{cs} = void ratio on the critical state line at the same p', Been and Jefferies, 1985) based on the CPTU data;
- Determine cyclic strength of the sand, at the in situ state, based on laboratory cyclic shear strength tests; and,
- Compare cyclic resistance to the seismic loading to indicate the fill performance. Performance is measured in terms of whether liquefaction is likely or not, quantified by the factor of safety against liquefaction.

This engineering approach should be valid for any material that lies outside of the database of liquefaction case histories for which the NCEER (Youd and Idriss, 2001) approach is appropriate. This includes carbonate sands, as well as silts and tailings. It is also applicable for projects which are considered too complex for the simplified methods to apply.

2 SEISMIC LOADING

The reference case study is a 20 m thick hydraulic fill, made of a carbonate sand, dredged offshore, hydraulically placed and densified by vibrocompaction. The fill is located in the Persian Gulf.

The results of a regional seismic hazard analysis PSHA, indicating reverse and strike-slip earthquakes with moment magnitude of approximately M_w = 6, have been used as the basis for selecting and applying bedrock ground motions in the site response models. The number of cycles of loading, wich is mainly influenced by the magnitude, is N = 6, as defined by the PSHA.

The maximum cyclic stress ratio CSR through the depth of the fill has been determined through a series of site-specific dynamic soil response analyses using an uncopled total stress approach and a 1D nonlinear soil response model. Geotechnical and geophysical parameters measured for the calcareous sand fill and underlying geological materials have been used. It's worth noting that the 50 percentile value of the small strain shear modulus in the fill, derived from shear wave velocity measurements, was adopted in the analyses. This is considered appropriate since displacement compatibility is required and it is therefore the average stiffness that determines the cyclic strains. The non linear soil behaviour has been accounted for considering the experimental strain dependent stiffness and damping of the fill's sand.

The analyses output was a profile of maximum cyclic stress ratio, which was then multiplied by 0.65 to convert computed time histories of CSR containing transient peak amplitudes to an equivalent, uniform CSR profile, consistent with laboratory testing.

3 MECHANICAL PROPERTIES OF THE DREDGED CARBONATE SAND

3.1 ZS sand cyclic resistance

ZS sand, described in detail in Giretti et al. (2018a), is a biogenic carbonate sand with a carbonate content higher than 97%. The sand is composed of a major component referred to organigenic and bioclastic origin, clasts of micritic limestones, and minor component of sparry calcite. The prevalent mineral phases are calcite, aragonite and Mg-calcite. The grains are characterised by micro-porosity and fine borings due to action of endolithic algae are present in some cases.

Table 1 reports ZS grain size distribution parameters. Specific gravity, maximum and minimum density (as per ASTM D4254, ASTM D4253) and void ratio are summarised in Table 2. The ASTM procedure for maximum dry density was considered the more suitable to crushable carbonate sands, however the series of determinations carried out highlighted a significant range of variation, confirming the limited reliability of density as reference parameter to characterise carbonate sands.

The critical state line approximation of ZS in the e-$\log_{10}$(p') plane is shown in Figure 1.

To evaluate the cyclic resistance of ZS, many cyclic tests (both triaxial and simple shear) have been performed. The results have been interpreted to define relationships between the cyclic stress ratio CSR ($\Delta\sigma_a/2s'$, where $s' = (\sigma'_v+\sigma'_h)/2$, or τ/σ'_v) and the number of cycles N to cause liquefaction (defined as the condition when the double amplitude axial strain ε_{DA} equals 2.5% or the double amplitude shear strain γ_{DA} equals 5%), for different values of the state parameter ψ.

The cyclic resistance evaluated in triaxial condition has been converted to resistance in a simple shear condition using a multiplier factor (Ishihara et al., 1977, 1985) of 0.7. Data from triaxial tests corrected by this factor, for given values of the state parameter ψ, are compared in Figure 2 to data from cyclic simple shear tests.

Table 1. Grain size distribution parameters of ZS sand.

	D_{10} (mm)	D_{50} (mm)	D_{60} (mm)	U_C (-)	FC (%)	Gravel like (%)
ZS	0.085	0.339	0.474	5.57	7.9	0

Table 2. Index properties of ZS sand.

	γ_{min} (kN/m³)	γ_{max} (kN/m³)	e_{max} (-)	e_{min} (-)	G_S (-)
ZS	14.14	17.88	0.95	0.54	2.812

The relationships between the cyclic resistance CSR in simple shear conditions (τ/σ'_v) and N have been used to define relationships between CSR and the state parameter ψ at a given N, as shown in Figure 3. Equation 1, referred to the case of "failure" occurring on the 6th cycle of loading, is also reported in Figure 2.

$$\mathrm{CRR} = 0.129 - 0.308 \cdot \psi \qquad (1)$$

This equation has been used to evaluate the *in situ* cyclic resistance as a function of the values of ψ estimated from CPTUs (using a properly calibrated correlation with the cone resistance, see next section).

The results of the cyclic tests have been also used to define a relationship between FS_{liq} and the excess pore pressure build up. For a given applied stress ratio τ/σ'_v, the number of cycles necessary to induce an excess pore pressure $Ru = \Delta u/\sigma'_v$ ranging from 0.1 to 0.95 has been evaluated. The relationship between τ/σ'_v and N for a given Ru value has then been determined. Example of the τ/σ'_v – N – Ru relationships are given in Figures 4a and b.

The FS_{liq} values at N=6 can be computed as the ratio between τ/σ'_v at Ru = 0.95 and τ/σ'_v at lower Ru values. The Ru – FS_{liq} relationships are shown in Figure 5. A unique relationship Ru – FS_{liq} has been found as shown in Figure 5, which can be interpreted with an inverse power law function:

$$Ru = \left(\frac{1}{FS_{liq}}\right)^{6.6} \qquad (2)$$

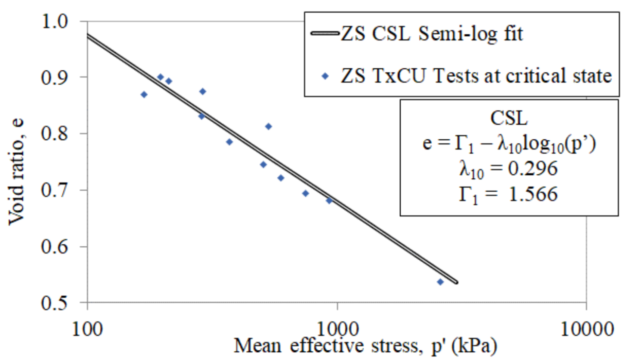

Figure 1. Critical state line of ZS Sand in the e - log10(p') plane.

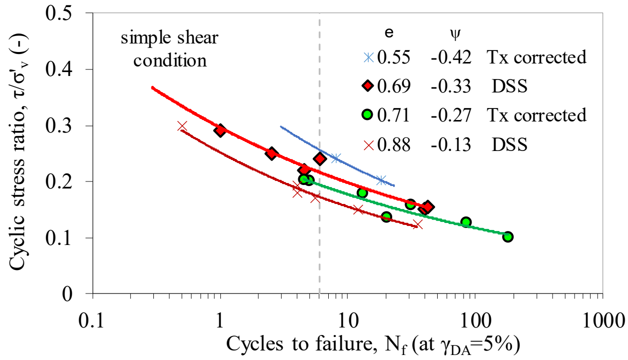

Figure 2. Cyclic Simple Shear Behaviour of ZS Sand.

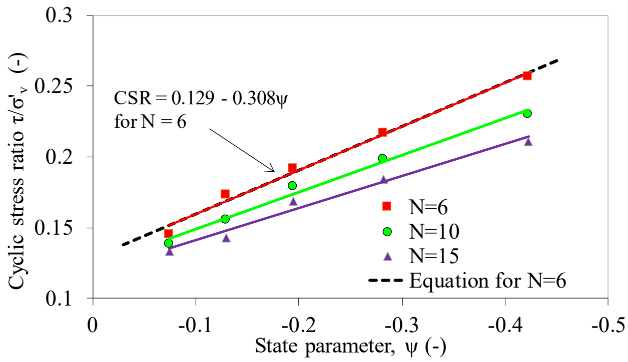

Figure 3. Cyclic stress ratio τ/σ' - number of cycles N relationships of ZS Sand.

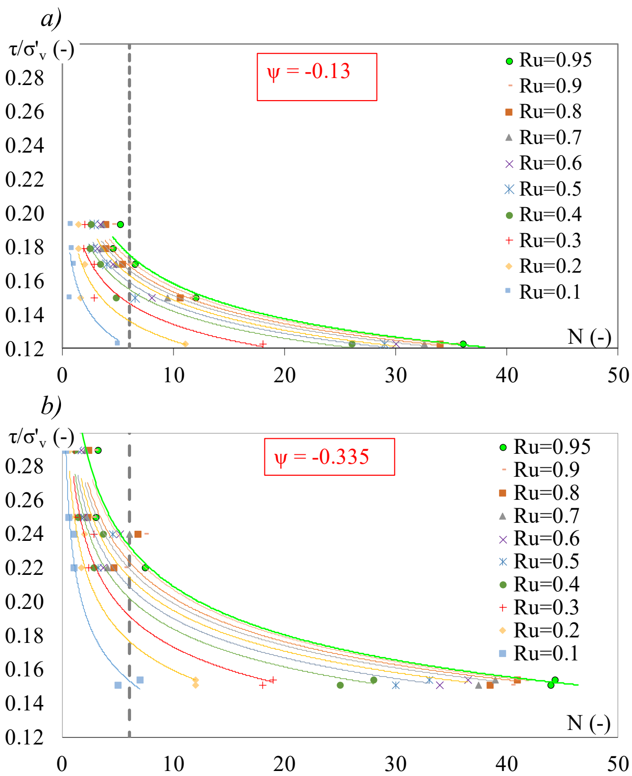

Figure 4. Cyclic stress ratio as a function of the number of cycles for Ru ranging from 0.3 to 0.95: a) ψ = -0.13; b) ψ = -0.335.

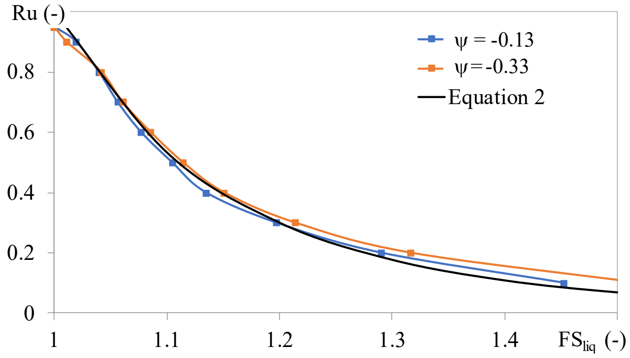

Figure 5. Ru - FS_{liq} relationships.

3.2 Cone penetration resistance in ZS

As described in detail in Giretti et al. (2018b), a significant number of calibration CPTU tests in a centrifuge and in a large calibration chamber have been performed to calibrate correlations between the cone resistance q_t and the state parameter ψ for ZS.

Figures 6 and 7 show typical centrifuge calibration test results of dry and saturated ZS sand over a range of average void ratios (mean value at the end of inflight consolidation). The tip resistance is plotted against the vertical effective stress σ'$_v$, computed accounting for the acceleration field distortion in the centrifuge. Saturated as opposed to dry conditions have a significant effect on the measured tip resistance for ZS sand. Figure 8 shows the same centrifuge data from Figs. 6 and 7 but processed as a normalised tip resistance Q_p vs. to the state parameter, ψ, in which Q_p is:

$$Q_p = \frac{q_t - p}{p'} \tag{3}$$

where p and p' are the total and affective mean stress.

The state parameter is calculated for each Q_p point based on the vertical effective stress, horizontal effective stress from $K_0 = 0.5$, void ratio estimated at that point and a linear critical state line approximation characterised by $\lambda_{10} = 0.296$ and $\Gamma_1 = 1.566$ (see Fig. 1). It's worth noting that in a small scale accelerated model, the vertical effective stress increases significantly from top to bottom of the sample, thus the void ratio also varies through the model. The void ratio profile in the sample can be estimated using the compression coefficients measured in oedometer tests, but maintaining the measured average void ratio (average at the end of the inflight consolidation induced by the augmented gravity fill). The lines of data for dry and saturated conditions in Figure 8 describe two similar relationships, highlighting the significant influence of ψ on tip resistance in sands. Following Jefferies and Been (2006) the relationships have the following Equation:

$$Q_p = k \cdot e^{-m\psi} \tag{4}$$

in which the parameters *k*, *m* are 42 and 5.1 for dry models and 35 and 5.1 for the saturated models.

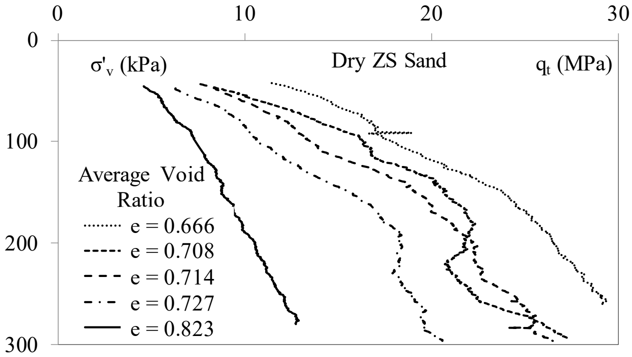

Figure 6. CPTUs in dry ZS.

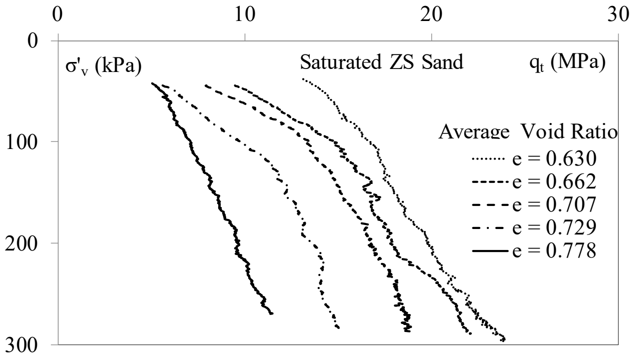

Figure 7. CPTUs in saturated ZS.

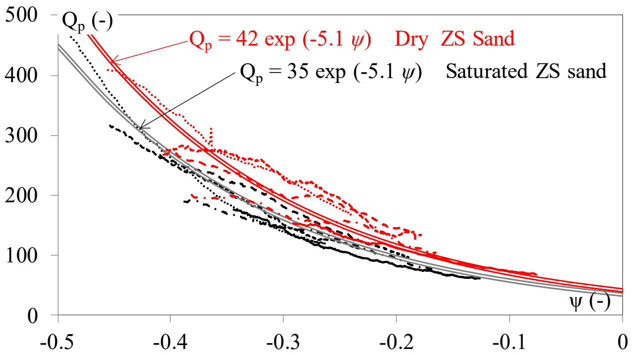

Figure 8. State parameter interpretation of centrifuge tests.

4 LIQUEFACTION ASSESSMENT

The first step of the liquefaction assessment consists in the determination of the in situ state of a sand fill through in situ CPTUs. The CPTU calibration summarised above provides the link between the measured tip resistance and the in situ state parameter. For a large volume of fill in which the in situ state is variable, the definition of a characteristic value for the liquefaction analysis is needed. For liquefaction to be a problem, a certain thickness of fill needs to be loose, and for this reason, 1 m thick layers or depth increments can be considered. However, only a relatively small portion of a fill needs to liquefy for the overall performance of the fill to be unsatisfactory. The analysis therefore may consider the 20 percentile value of the in situ state in each metre interval. A one metre depth interval provides a good vertical definition of a fill profile in relation to the likely size of foundations and depth of fill. From a liquefaction perspective, a greater depth interval is not desirable as that relatively thin liquefied layers can be problematic.

Figure 9a shows, as an example, a groups of 4 CPTUs carried out in a 25 m x 25 m box of a compacted hydraulic fill made of ZS, and the 20 percentile values in 1 m depth intervals derived from the group. The 20 percentile profile is used to compute the characteristic in situ state parameter by means of the inverse form of Eq. 4, as shown on Figure 9b. For the inversion, an average depth of the groundwater table of 6 m was assumed. Figure 9c presents the comparison between the cyclic strength CRR, derived from the 20 percentile state parameter profiles by means of Eq. 1, with the seismic loading obtained from the site-specific dynamic soil response analysis.

Equation 1 (see also Fig. 2) has been employed to calculate cyclic strength from state parameter for N = 6 cycles. The seismic loading on Figure 9c is 0.65 times the maximum CSR from ground response analysis. The cyclic strength of the compacted fill is clearly well above the cyclic loading, representing a very low risk of liquefaction (or high factor of safety).

The CRR profile in Figure 9c derives from the combination of Eqs. 1 and 4 and can be written as:

$$\mathrm{CRR} = 0.129 - 0.308 \cdot \left(-\frac{1}{m} \cdot ln\frac{Q_P}{k}\right) \quad (5)$$

The ratio between the CSR and CRR profiles in Figure 9c is the factor of safety against liquefaction FS_{liq}, from whose values the pore pressure ratio Ru can be calculated using Eq. 2. The calculated pore pressure profiles for 7 groups of 4 CPTUs are shown on Figure 10. In general, the pore pressure ratio is less than about 0.1, with two locations showing values up to 0.37 in some layers.

These values are low in terms of liquefaction - a range of 0.3 to 0.5 is normally considered to signify the onset of strain softening during seismic loading, while 0.9 is considered liquefaction.

5 CONCLUSIONS

This paper presented an example of a liquefaction assessment for a carbonate sand with reference to a case study of a hydraulic fill made of dredged calcareous, biogenic sands. The two principal ingredients of the assessment are the determination of:

- the in situ state based on CPTU data and a properly calibrated correlation for the fill sand between the cone resistance and the soil state;
- the cyclic strength of the sand, at the in situ state, based on laboratory cyclic shear strength tests.

This assessment procedure has been developed given that carbonate sands lies outside of the range of materials for which the NCEER approach is appropriately applicable.

Core of the procedure is the adoption of the state parameter ψ (Been and Jefferies, 1985) as proper indicator of the state of the fill's sand, given that density alone (void ratio or relative density), without stress level, is not sufficient for defining the state of the soil and quantifying its stress-strain-strength behavior. The procedure described allows to assess the satisfaction of performance requirement (such as minimum safety factor against liquefaction) for compacted hydraulic fill on the basis of a directly relevant physical parameter, not affected by i) testing method, ii) the applicability of empirical correlations between cone resistance and liquefaction resistance, iii) potential errors associated with the nature of carbonate sand (e.g. high and variable compressibility and susceptibility to particle crushing).

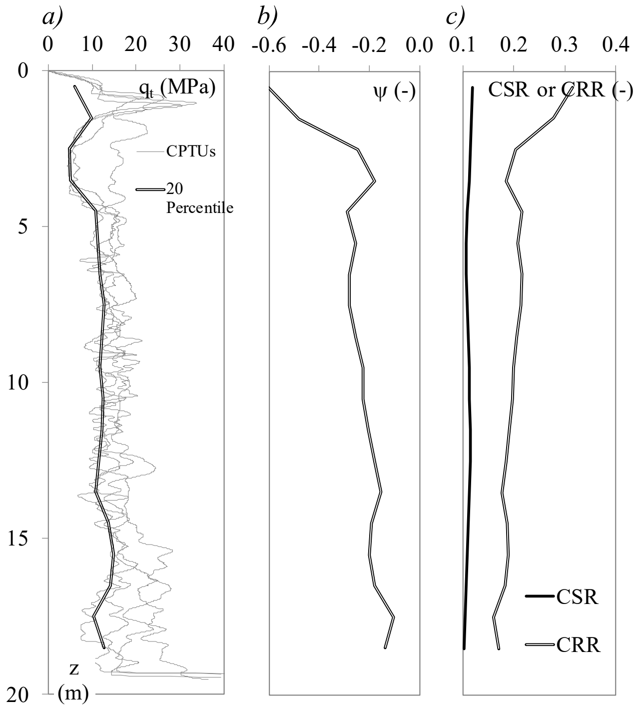

Figure 9. Liquefaction analysis for a group of 4 CPTUs: a) measured q_t profiles and corresponding 20 percentile value; b) characteristic in situ state parameter; c) cyclic resistance and cyclic stress ratio.

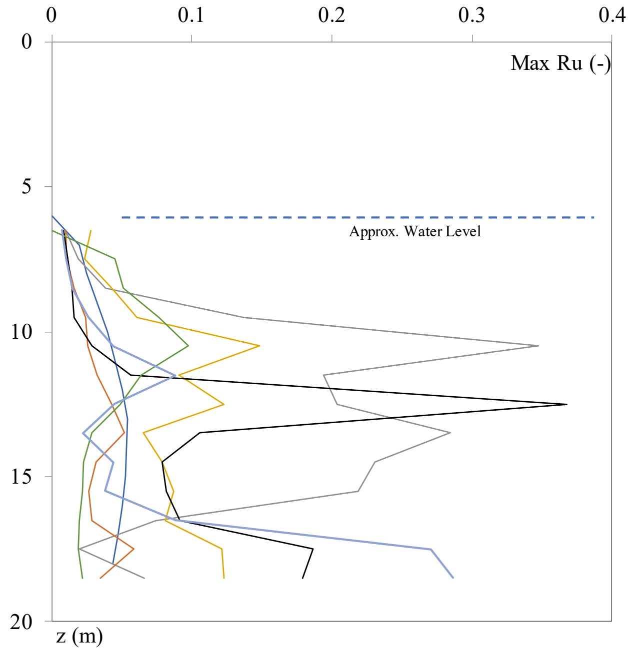

Figure 10. Ru from Uncoupled Liquefaction Analyses for 7 groups of 4 CPTUs.

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