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The Determination of Factor of Safety Against Liquefaction and Post-Liquefaction Settlement

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The Determination of Factor of Safety against Liquefaction and Post-Liquefaction Settlement

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ABSTRACT: This paper describes the liquefaction triggering relationships by NCEER (1997) and Youd et al. (2001, 2003), Idriss and Boulanger (2008) and Boulanger and Idriss (2014), used to assess the susceptibility of loose sands to liquefy. For the Idriss and Boulanger (2008) and Boulanger and Idriss (2014), the site-specific fines content correlation and carbonate content of the sands was incorporated in the analysis. The paper then demonstrates the use of CPTu and SCPTu soundings and shear wave velocity measurements to develop soil capacity for the site seismic input data. Factor of safety against liquefaction calculations are performed using the CPTu data and post-liquefaction settlements are calculated based on the normalized CPTu results and the factor of safety.

1 INTRODUCTION

As part of the design of the foundations and ground improvement works for a major industrial project a site specific seismic assessment was undertaken and in conjunction, a liquefaction assessment was also undertaken. The liquefaction assessment identified loose sands susceptible to liquefaction under the design seismic event. To determine the design requirements for foundations and ground improvement, the liquefaction assessment was extended to determine post-liquefaction settlements. This required calculation of the factor of safety against liquefaction, level of seismic strain and associated settlements.

The site-specific fines content correlation and carbonate content correction of the sands was incorporated in the analysis.

Post-liquefaction settlements were then calculated across the project site for the seismic inputs to identify the facilities impacted and the magnitude of post-liquefaction settlement. The results show that the post-liquefaction settlement varies from negligible in many areas to being significant in a number of facility locations. The results were used to assign the appropriate seismic site class and seismic design loads as well as appropriate foundations systems for the facilities, as determined based on the post-liquefaction settlement, as well as conventional bearing capacity and settlement basis.

2 SUBSURFACE CONDITIONS

Geotechnical investigations comprising more than 40 boreholes, 100 piezocone penetration tests (CPTu), 20 seismic piezocone penetration tests (SCPTu), 17 seismic dilatometer tests (SDMT), as well as geophysical surveys were undertaken for the design of the foundations.

The subsurface profile typically comprises compacted sandy fill over approximately 8 to 10m of natural poorly-graded silty sand, which is underlain by bedrock typically composed of calcarenite and sandstone and interbedded mudstone and gypsum to more than 70m depth.

CPTu soundings were initially terminated due to refusal in a very dense zone between 6 and 8.5m depth. Below this zone, loose sand was reported in the borehole and CPTu soundings that penetrated the very dense layer. Subsequent investigations were undertaken where the cone was withdrawn if refusal was achieved above 10m depth, and drilling was used to drill 0.5m below the depth of refusal. The CPTu was then continued until a depth of 10m was achieved.

The investigations were aimed at determining design parameters for piled foundations and ground im-

provement for large raft foundations, tall tank foundations, stockpile areas, reclaim tunnels, as well as a multitude of shallow raft and pad foundations.

The loose sand located from approximately 8 to 10m depth between the very dense layer and bedrock, was identified as potentially liquefiable.

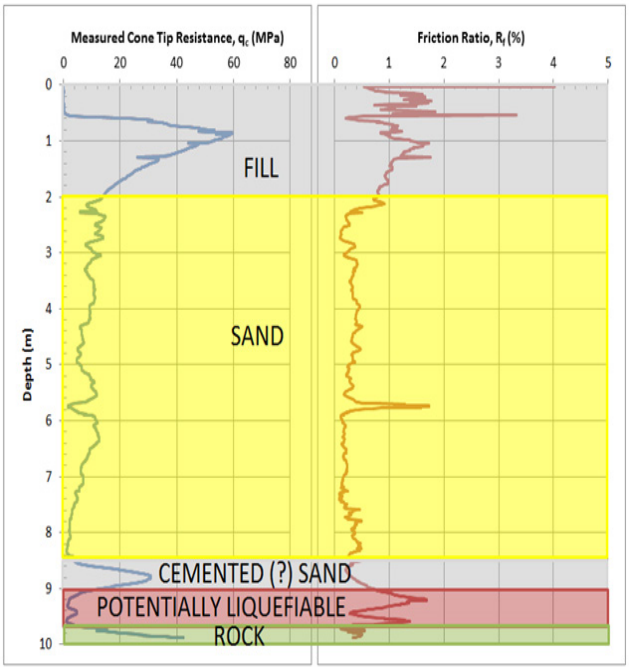

Figure 1. Subsurface profile showing typical soil conditions over the depth range of CPTu tests.

Figure 1 presents the measured cone tip resistance and friction ratio with stratigraphic interpretation of the subsurface profile and also illustrates the cemented zone, from approximately 8.5 to 9.0 metres depth, and the potentially liquefiable zone beneath from approximately 9.0 to 9.5 metres depth overlying the sedimentary rock strata.

3 SITE SPECIFIC CARBONATE CONTENT AND FINES CORRECTIONS

3.1 *Correcting Penetration Resistance for Carbonate Content*

The soils have a high carbonate content; typically, 82%. Soils with high carbonate content tend to be more compressible thereby requiring modification to typical correlations from penetration resistance to engineering parameters.

Due to the high carbonate content of the sands, a correction of penetration resistance was undertaken. Due to the carbonate nature of the sands, the generic fines content correlations do not perform well at this site. Therefore a site specific fines content correlation is constructed.

The process for use in liquefaction calculations is as follows:

- Calculate relative density using correlation for carbonate sands.
- Calculate equivalent penetration resistance for silica sand.
- Perform liquefaction calculation using input of equivalent silica sand penetration resistance.

Mayne (2014) provides a correlation from normalized cone tip resistance to relative density in carbonate sands as shown in Equation 1.

$$D_R = 0.87 q_{t1} \quad (1)$$

where Dr = relative density; q_{t1} = normalized cone resistance.

Mayne (2014) provides a correction factor to be used to convert from normalized tip resistance in calcareous-carbonate sands to an equivalent normalized tip resistance in silica-quartz sands as shown in Equation 2.

$$q_{t1}(silica - quartz) = q_{t1}(calcareous - carbonate) * CF. \quad (2)$$

where CF = correction factor for calcareous sands.

The process to correct for the calcareous nature of the sands for CPTs is as follows:

- Calculate relative density based on the field measured cone resistance.
- Calculate equivalent silica-quartz cone resistance.

The equivalent quartz penetration resistances are then used in the liquefaction calculations.

3.2 *Site Specific Fines Content Correlation*

Liquefaction calculations using CPT data typically correlate CPT data in the form of the soil behavior index, Ic, to a fines content or apparent fines content. The soil behavior index is calculated using Equation 3 from Idriss and Boulanger (2008) with a stress exponent of 0.5 adopted for the sandy soil type.

$$I_c = [(3.47 - \log(Q))^2 + (\log(F) + 1.22)^2]^{0.5} \quad (3)$$

where I_c = soil behavior index; Q = normalized cone tip resistance; F = normalized friction ratio determined as follows:

$$Q = \left[\frac{q_c - \sigma_{vo}}{P_a}\right]\left[\frac{P_a}{\sigma'_{vo}}\right]^n \quad (4)$$

$$F = \frac{f_s}{(q_c - \sigma_{vo})} * 100\% \quad (5)$$

where q_c = cone tip resistance; P_a = atmospheric pressure; σ_{vo} = vertical stress; σ'_{vo} = vertical effective stress; f_s = sleeve friction.

The site-specific correlation of I_c to apparent fines content was determined based on the laboratory testing of fines content and the corresponding soil behavior index calculated from the normalized cone and friction values. A power curve as shown in Equation 6 was used to calculate the apparent fines content from the soil behavior index in the calculations.

$$AFC = 2.8{I_c}^{3.2} \quad (6)$$

4 FACTOR OF SAFETY AGAINST LIQUEFACTION

The factor of safety against liquefaction is calculated using CPTu and shear wave velocity data using CPTu results is performed using the fines content correlations.

The factor of safety against liquefaction is calculated from Youd et al. (2001), as

$$FS = \left(\frac{CRR_{7.5}}{CSR}\right) * MSF * K_\sigma * K_\alpha \quad (7)$$

where FS = factor of safety against liquefaction; $CRR_{7.5}$= cyclic resistance ratio for an equivalent magnitude 7.5 event; CSR = cyclic stress ratio for a given magnitude; MSF = magnitude scaling factor; K_σ = overburden correction factor; K_α = correction factor for sloping ground, assumed to be equal to one for level ground.

Youd et al. (2001) provides several ways of calculating the magnitude scaling factor, MSF. The overburden correction factor, K_σ is determined from the relative density and overburden stress as follows:

$$K_\sigma = \left(\frac{\sigma'_v}{P_a}\right)^{(f-1)} \quad (8)$$

where σ'_v = vertical effective stress; P_a = atmospheric pressure; f = empirical exponent; for relative density, D_r, ≤ 40% f = 0.8; 40% < D_r < 80% f = 0.7; D_r ≥ 80% f = 0.6.

4.1 *Calculation of Cyclic Resistance Ratio (CRR) using CPT Data*

The CPT tip resistance is normalized for overburden as follows:

$$q_{c1N} = C_Q\left(\frac{q_c}{P_a}\right) \quad (9)$$

$$C_Q = \left(\frac{P_a}{\sigma'_{vo}}\right)^n \quad (10)$$

where q_{c1N} = cone resistance normalized for overburden; C_Q = normalizing factor for cone resistance; q_c = cone tip resistance; P_a = atmospheric pressure; σ'_{vo} = vertical effective stress; n = stress exponent.

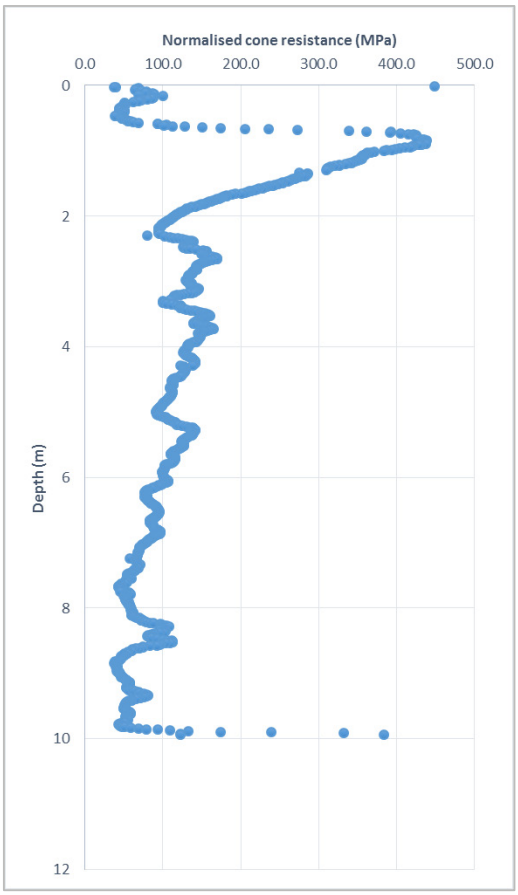

Figure 2. Normalised cone resistance.

Robertson (2009), provides a function for the stress exponent, n, as shown in Equation 11, which allows for convergence of liquefaction calculations.

$$n = 0.381(I_c) + 0.05\left(\frac{\sigma'_{vo}}{P_a}\right) - 0.15 \leq 1.0 \quad (11)$$

The $CRR_{7.5}$ is then calculated in Youd et al. (2001) dependent on the normalized cone resistance.
If $(q_{c1N})_{cs}$ < 50:

$$CRR_{7.5} = 0.833\left[\frac{(q_{c1N})_{cs}}{1000}\right] + 0.05 \quad (12)$$

If 50 < $(q_{c1N})_{cs}$ <160:

$$CRR_{7.5} = 93\left[\frac{(q_{c1N})_{cs}}{1000}\right]^3 + 0.08 \quad (13)$$

where $(q_{c1N})_{cs}$ = equivalent clean sand normalized penetration resistance; $CRR_{7.5}$ = cyclic resistance ratio for an equivalent magnitude 7.5 event.

4.2 *Calculation of Cyclic Stress Ratio (CSR) using CPT Data*

The cyclic stress ratio (CSR) is calculated using a site response analysis in which the input motion applied at rock level is propagated upward through the soil profile, as determined from the shear wave velocity profile, as shown in Figure 3.

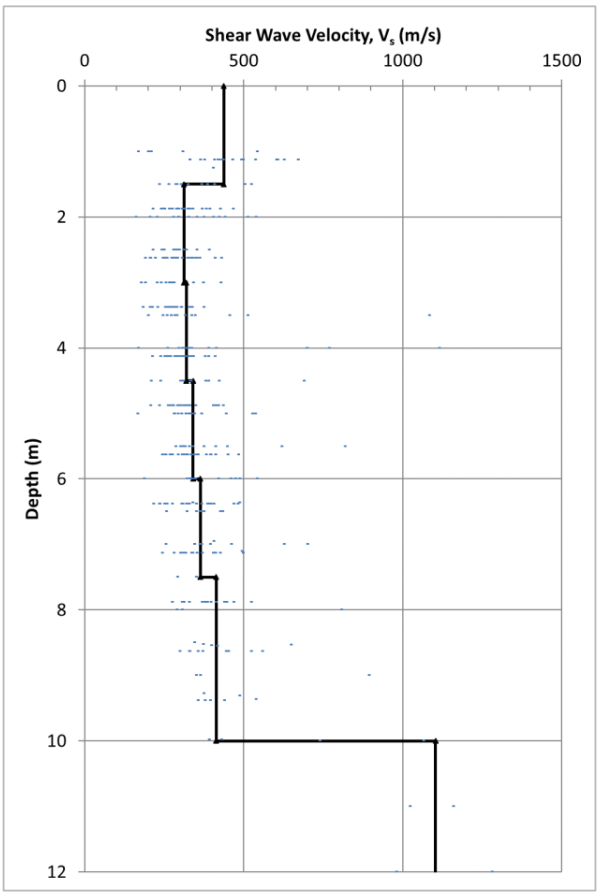

Figure 3. Profile of Shear Wave Velocities determined from SCPTu, SDMT and geophysical tests.

The input rock acceleration response spectra (ARS) and output ARS at grade are shown in Figure 4.

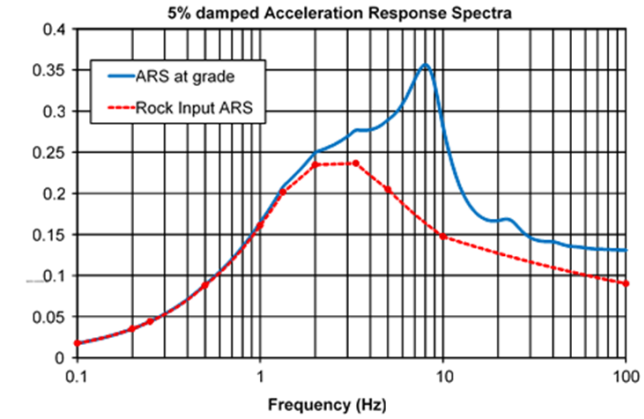

Figure 4. Acceleration response spectra at the rock level and the ground surface showing amplification in the soil profile.

The maximum shear stress is used to calculate the cyclic stress ratio (CSR) using Equation 2

$$CSR_{M,\sigma'_v} = 0.65\left(\frac{\sigma_v}{\sigma'_v}\right)\left(\frac{a_{max}}{g}\right)r_d \quad (14)$$

where $CSR_{M,\sigma'v}$ = cyclic stress ratio for a specific earthquake magnitude and in-situ vertical effective stress; σ_v = total vertical stress; σ'_v = vertical effective stress; a_{max} = peak horizontal ground acceleration; g = gravitational acceleration; r_d = stress reduction coefficient.

Based on the calculated cyclic resistance ratio and cyclic stress ratio the factor of safety against liquefaction is calculated for each CPTu profile. The calculated factor of safety is shown in Figure 5.

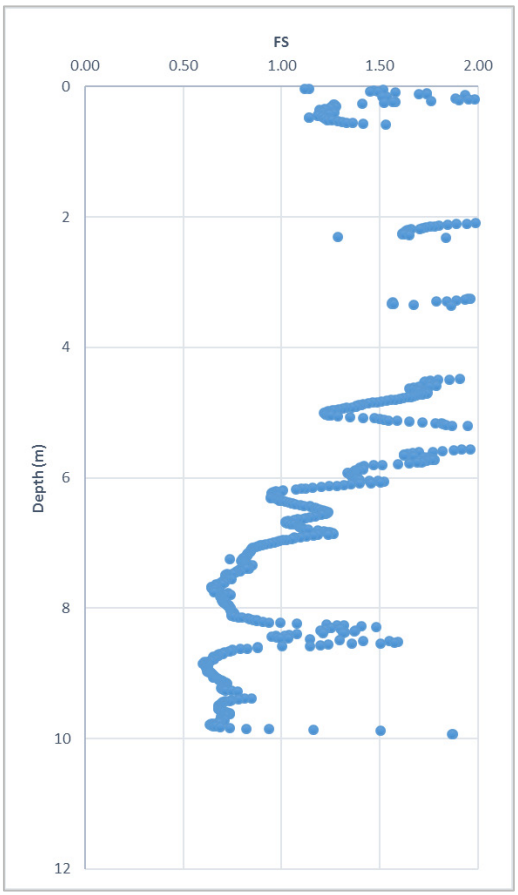

Figure 5. Calculated factor of safety against liquefaction.

5 POST-LIQUEFACTION SETTLEMENT

The calculation of post-liquefaction settlement is based on the limiting shear strain, the maximum shear strain, and the post-liquefaction strain.

The limiting shear strain is calculated from Idriss and Boulanger (2008), as follows:

$$\gamma_{lim} = 1.859(2.163 - 0.478(q_{t1Ncs})^{0.264})^3 \geq 0 \quad (15)$$

where γ_{lim} = limiting shear strain; q_{t1Ncs} = normalized clean sand tip resistance.

The post-liquefaction strain, F_α, is calculated from Idriss and Boulanger (2008), as follows:

$$F_\alpha = -11.74 + 8.34(q_{t1Ncs})^{0.264} - 1.371(q_{t1Ncs})^{0.528} \quad (16)$$

where F_α = post-liquefaction strain term; q_{c1Ncs} = normalized clean sand tip resistance.

The maximum shear strain is then calculated from Idriss and Boulanger (2008) as shown according to the criteria shown below and using Equation 17.

If FS ≥ 2 $\quad \gamma_{max} = 0$

If 2 > FS > F_α

$$\gamma_{max} = min\left(\gamma_{lim}, 0.035(2 - FS)\left(\frac{1-F_\alpha}{FS-F_\alpha}\right)\right) \quad (17)$$

If FS < F_α $\quad \gamma_{max} = \gamma_{lim}$

where FS = factor of safety against liquefaction; γ_{max} = maximum shear strain, γ_{lim} = limiting shear strain, F_α = post-liquefaction strain term.

The post-liquefaction volumetric strain is calculated from Idriss and Boulanger (2008) as follows:

$$\varepsilon_v = 1.5e^{(2.551-1.147(q_{t1Ncs})^{0.264})} * min(0.08, \gamma_{max}) \quad (18)$$

where ε_v = post-liquefaction volumetric strain; q_{t1Ncs} = normalized clean sand tip resistance, γ_{max} = maximum shear strain.

Settlement is then calculated from strain as:

$$S = t\varepsilon_v \quad (19)$$

where S = post-liquefaction settlement, t = thickness of layer, ε_v = post-liquefaction volumetric strain.

Figure 6 shows a calculated profile of incremental settlement calculated for each depth increment in the CPTu profile.

Total settlement is then calculated as the sum of each layer incremental settlement and the post-liquefaction settlement can be plotted across the project site. Post-liquefaction settlements can then be assessed for each facility to assign the appropriate seismic site class as well as appropriate foundation systems for the facilities in addition to the requirements determined based on conventional bearing capacity and settlement basis.

6 CONCLUSIONS

The calculation of factor of safety against liquefaction and post-liquefaction settlement has been determined using CPTu and shear wave velocity data readily obtained and recorded across the entire project site at centimeter depth increments and imported and calculated in spreadsheets to allow both profiling and contouring of the factor of safety and settlement.

This has allowed each facility to be assigned the appropriate seismic site class as well as appropriate foundation system based on conventional bearing capacity and settlement as well as liquefaction affects.

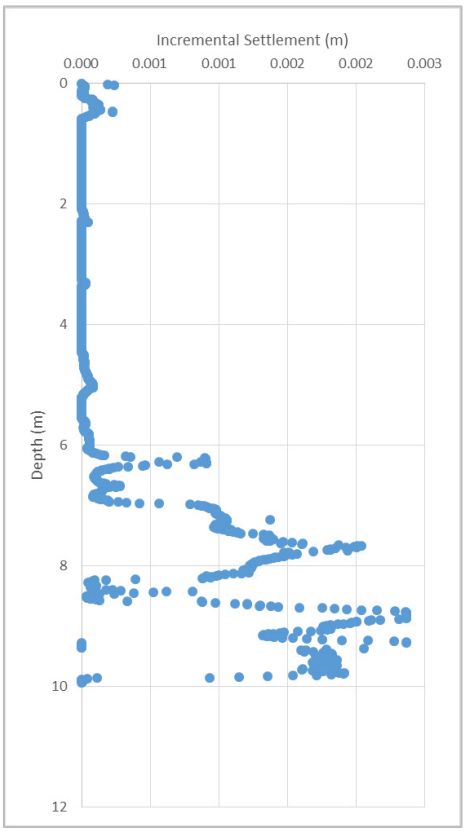

Figure 6. Incremental settlement calculated for each CPTu.

7 ACKNOWLEDGMENTS

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