Guidance for the CPT-Based Assessment of Earthquake-induced Liquefaction Potential in Australia, with a Focus on the Queensland Coast

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Abstract

The paper outlines a procedure to assess the potential for earthquake-induced liquefaction in Australia, with a focus on the Queensland coast. Ground input motions are derived from AS1170.4 and earthquake magnitudes referenced from Dismuke and Mote, 2011. The liquefaction potential is evaluated for a simplified ground profile of clean sand using the semi-empirical method first proposed by Seed and Idriss in 1971 and modified to permit a CPT-based probabilistic assessment after Moss et al., 2006. Liquefaction-threshold values for normalized CPT resistance are considered in relation to the annual probability of exceedance, groundwater depth, geography, and factor of safety.

Keywords: Liquefaction, Probabilistic, Australia, AS1170.4, CPT

1. INTRODUCTION

Australia's distance from the world's major tectonic boundaries makes the occurrence of large magnitude earthquakes within its borders relatively infrequent. There have been three documented cases of earthquake-induced liquefaction in Australia: the 1897 Ms 6.5 event near Beachport, South Australia; the 1903 Ml 5.3 event in Warrnambool, Victoria; and the 1968 Ms 6.8 event at Meckering, Western Australia. The 1989 event in Newcastle that killed 13 people – 12 due to structural failures, 1 due to shock – did not result in observed liquefaction, but it heightened awareness of seismic risk. More recent events in the seismically-active South Island of New Zealand have broadcasted to Australians the importance of seismic design, and specifically the consequences of liquefaction when not anticipated or prepared for.

The Australian Standard for seismic design AS1170.4 currently does not provide guidance for the evaluation of liquefaction potential. The standard provides a seismic hazard value (Z) from which design ground motions can be determined, but does not provide design magnitudes nor procedural recommendations. Dismuke and Mote in 2011 determined design moment magnitudes that are compatible with the available design ground motions of AS1170.4. The widely accepted semi-empirical approach first proposed by Seed and Idriss in 1971 for evaluating liquefaction potential using the cyclic stress ratio (CSR) can then be applied. It should be noted that the Australian National Committee on Large Dams – ANCOLD – does provide guidance for the evaluation of liquefaction potential in its 1998 Guidelines for Design of Dams for Earthquake. However, ground motions have been superseded by the hazard factors of AS1170.4 – 2007, and design magnitudes are presumed by the guidelines to be determined by site-specific investigations.

Absent from the original 1971 Seed and Idriss evaluation methodology is a treatment of probability. The methodology has sometimes been applied deterministically, such that if a point fell above the cyclic resistance ratio (CRR) curve, it would definitely liquefy if subjected to the design event. Articles and publications addressing the method or seeking to expand upon it have generally included data that did not comply with the curves. In 2006 Moss et al. compiled and back-analyzed CPT data associated with 188 case histories of world-wide liquefaction from 18 different earthquakes between 1964 and 1999. The abundance of information allowed for the incorporation of a probabilistic treatment of the liquefaction potential for specific ground motions and magnitudes. Where awareness of earthquakes and the potential hazards is low, a probabilistic treatment of risk is valuable. It empowers designers, managers and owners to make informed decisions. For example, where a construction project evaluation indicates that a particular ground profile is susceptible to liquefaction for a specific earthquake magnitude, knowing that based on empirical data the probability of liquefaction is only 15% could save significant resources.

The procedure outlined in this paper can be applied to all of Australia. The coastline and particularly the urban centres along the Queensland coastline will serve as the focus for the following reasons:

- the abundance of recent and unconsolidated deposits to be found on coasts;
- the concentration of Australia's population near/on the coast;
- a good variation in design ground motions and magnitudes along the Queensland coast, and
- the Queensland coast is currently host to massive infrastructure developments.

Naturally occurring loose sands and very soft soils tend to be recently deposited in the Quaternary or Holocene. As the product of erosion or weathering, these alluvial or estuarine sediments accumulate in rivers and along the coastline. According to the Australian Bureau of Statistics, in 2001 85% of Australians - including 88% of Queenslanders - lived within 50km of the coastline (ABS, 2004). While a relatively high volume of Quaternary and Holocene deposits is not to be expected 50km in land, the statistic underscores the extent of economic activity that is taking place directly on the coast. This in turn highlights the high relative proportion of artificial fill – controlled or uncontrolled - that has been placed historically to support the corresponding public and private infrastructure.

To simplify the analysis, a fictional ground profile comprising clean sand (fines content < 5%) will be considered. In summary, the procedure to be detailed below includes:

1. Assign an importance level of the structure in accordance with AS1170.0. Then assign a site-specific Z factor and appropriate probability and spectral shape factors from AS1170.4 (k_p and $C_h(T)$)
2. Assign an appropriate earthquake magnitude (M_{Ww}) from Dismuke and Mote, 2011.
3. Assign an appropriate magnitude weighting factor (M_f) in accordance with NCEER, 1998.
4. Define remaining inputs for the CSR-based evaluation:
    a. Total and effective overburden stress (σ_v and σ_v')
    b. The peak acceleration reduction factor (r_d)
    c. Cone tip resistance (q_c) and friction ratio (R_f) from CPT data
5. Normalise the CPT tip resistance
6. Plot CSR/q_c points on CRR curves from Moss et al. (2006) to assess the potential for liquefaction and corresponding probability.

2. SIMPLIFIED SOIL PROFILE

For the purpose of the discussion in this paper, the following 5m soil profile has been assumed (the water table is at 2m below ground surface):

Depth (m)	Soil	Unit Weight (kN/m3)	Unit Weight' (kN/m3)	q_c (MPa)	R_f (%)
1	Sand with fines < 6%	19	19	5	0.5
2		19	19	7	0.5
3		19	9.2	3	0.5
4		19	9.2	2	0.5
5		19	9.2	10	0.5

Table 1: Ground profile

The profile has been created following review of the Moss et al. (2006) database for liquefaction locations. This database indicated that critical liquefaction depths often exceed 3 to 4m and that corresponding groundwater tables are commonly 1 to 3m below the surface.

3. THE PROCEDURE

Step 1

AS1170.0 outlines the importance level of different structures, from 1 to 4. Select the importance level, and then in consideration of the design life, AS1170.4 provides the appropriate annual probability of exceedance for design. This in-turn establishes the probability factor (k_p). Selected factors are presented in Table 2 below.

Annual Probability of Exceedance	Probability Factor (k_p)
1/500	1.0
1/1000	1.3
1/2500	1.8

Table 2: Probability Factor (k_p) - extract from Table 3.1 of AS1170.4

Select the appropriate Hazard Factors (Z) for a specific site from AS1170.4. The Z value is equivalent to peak ground acceleration of bedrock during a 500 year seismic event.

A spectral shape factor (Ch(T)) specific to site class can be incorporated into the assessment to account for varying soil conditions. AS1170.4 classifies soil into five categories:

Class A: Strong rock
Class B: Rock
Class C: Shallow soil
Class D: Deep or soft soil
Class E: Very soft soil

Because liquefaction occurs most frequently in very loose sands, and these are excluded by the standard from Class C for a Shallow Soil site, Classes D and E are appropriate. The spectral shape factor (Ch(T)) for site classed D and E is 1.1 (see Table 6.4 of AS1170.4 and values in parentheses for Period = 0s).

Design Ground Motion (g) = $Z * k_p * Ch(T)$ (1)

Step 2

Dismuke and Mote (2011) have proposed design earthquake magnitudes – in units of moment magnitude - for Australia that are compatible with AS1170.4. Their de-aggregation model employs a weighted average of magnitudes that are likely to generate a ground motion in specific Australian seismic hazard zones. The weighting functions include: 1) the likelihood of earthquake magnitude given design ground motion, 2) spatial variability of magnitude, and 3) variability of ground motion intensity. The design earthquake magnitudes include 500, 1000, and 2500 year return periods. Design hazard factors and earthquake magnitudes for selected Queensland cities are presented in Table 3 below.

Location	Z_{500}	M_{Ww500}	Z_{1000}	M_{Ww1000}	Z_{2500}	M_{Ww2500}
Brisbane/Gold Coast	0.05	4.7	0.07	4.8	0.09	4.9
Bundaberg	0.11	5.8	0.14	5.9	0.2	6.1
Gladstone	0.09	5.8	0.12	5.9	0.16	6.0
Mackay/Townsville	0.07	4.8	0.09	4.9	0.13	5.0
Cairns	0.06	5.0	0.08	5.1	0.11	5.2

Table 3: Summary of M_{Ww} for Site Class B at urban concentrations of the Queensland Coast (AS1170.4, Dismuke and Mote, 2011).

Step 3

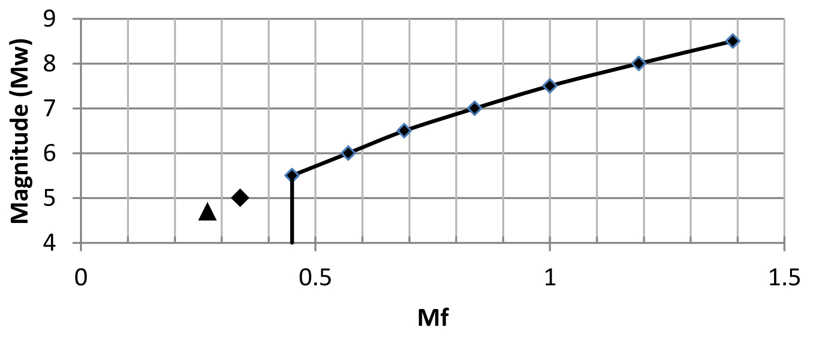
The semi-empirical Seed and Idriss methodology requires that design earthquake magnitudes be normalized to a moment magnitude of 7.5. This is done with the application of a magnitude factor M_f that varies as presented in Table 4.

Magnitude	M_f	No.Equivalent Cycles
5.5	0.45	2-3
6	0.57	5-6
6.5	0.69	
7	0.84	10
7.5	1	15
8	1.19	
8.5	1.39	26

Table 4: Variation of M_f with earthquake magnitude (NCEER, 1998)

In Queensland as in other states of Australia, most design magnitudes are below the 5.5 base (see Table 3). If extrapolated down to address Magnitude 5.0 and Magnitude 4.7 events, the corresponding M_f values would be approximately 0.34 and 0.27, respectively, as presented in Figure 1. The uncertainty in adopting such low values lies in understanding the stress accumulation per cycle for a specific earthquake event. Without investigating real time histories from small magnitude events, it is recommended that a floor be applied to the M_f value at 0.45. While perhaps conservative, this would effectively assume an upper bound value for the factor during lower magnitude events.

Figure 1: Normalisation to 7.5 (NCEER 1997)



It should be noted that Moss et al. (2006) present the following equation for the determination of the duration weighting factor (or the inverse of the magnitude factor), based on Cetin et al. (2004):

$$DWF_M = 17.84 * M_W{}^{-1.43} = \frac{1}{M_f} \quad (2)$$

The equation is valid for M_w between 5.5 and 8.5. For a lower bound assumption of M_w=5.5, the resulting magnitude factor (M_f) would be 0.64, or approximately 40% higher than the same factor using the NCEER recommendations. While the NCEER recommendations are adopted for this paper, it is recommended that consideration also be given to the Cetin et al. (2004) criteria in light of the variation observed at lower bound magnitude events.

Step 4

The semi-empirical methodology developed by Seed and Idriss relies on the following equation for the determination of the cyclic stress ratio:

$$CSR = \frac{\tau_{av}}{\sigma'_v} = 0.65 \frac{a_{\max}}{g} \frac{\sigma_v}{\sigma_v'} r_d M_f \tag{3}$$

Where σ_v is the total vertical stress, σ'_v is the total effective stress, a_{max} is the design ground acceleration, r_d is the peak acceleration reduction factor, and M_f is the normalization factor.

The average peak acceleration reduction factor can be determined from the below equation. The factor has the effect of reducing the CSR with depth based on confinement and observations that earthquake-induced ground acceleration will be highest at ground surface. The below equation has been proposed by Moss et al. (2006) for soil in the top 20m of a profile. The equation is included here as it is less conservative than the equation most commonly applied.

$$r_d = \frac{\left[1 + \frac{-9.147 - 4.173 * a_{max} + 0.652 * M_W}{10.567 + 0.089 * e^{0.089(-3.28 * d - 7.76 * a_{max} + 78.576)}}\right]}{\left[1 + \frac{-9.147 - 4.173 * a_{max} + 0.652 * M_W}{10.567 + 0.089 * e^{0.089(-7.76 * a_{max} + 78.576)}}\right]} \tag{4}$$

Where a_{max} is the maximum ground acceleration, M_w is the design earthquake magnitude, and d is depth in metres.

Cunderdin, Western Australia has the highest hazard factor of all locations specified in AS1170.4. For the following assumptions, a maximum theoretical value for CSR can be estimated for Australia. This then can act as an upper bound for the representation of CSR values.

return period = 2500 years, depth = 1m, water table at ground surface

$Z_{500} = 0.22$, $k_p = 1.8$, S = 1.1 for site class D or E

$a_{max} = 0.22 * 1.8 * 1.1 = 0.44g$

$M_{Ww} = 6.9$

$M_f = 0.82$

$r_d = 0.994$ at 1m

$$\frac{\sigma_v}{\sigma_v'} = \frac{19kN/m^3}{9kN/m^3} = 2.1$$

$$\text{CSR} = 0.65 * \frac{0.44\text{g}}{\text{g}} * 2.1 * 1.0 * 0.82 = 0.49$$

Step 5

Normalise the CPT tip resistance using the following equations:

$$q_{c,1=}C_q * q_c \tag{5}$$

$$C_q = \left(\frac{P_a}{\sigma'_V}\right)^c \tag{6}$$

Where P_a = reference stress of 101.325kPa, σ'_v = effective overburden stress, and c = normalization exponent. While many practitioners have used 0.5 for the value of c, Moss et al (2006) propose an iterative approach that has been demonstrated to better reflect the variability of soils. Their iterative equation and procedure for determining the c value are presented below.

$$c = 0.78 * q_c^{-0.33} * \left(\frac{R_f}{\text{abs}[\log(10+q_c)]^{1.21}}\right)^y \tag{7}$$

where $y = (0.32 * q_c^{-0.35} - 0.49)$ (8)

Procedure (best executed with a spreadsheet):

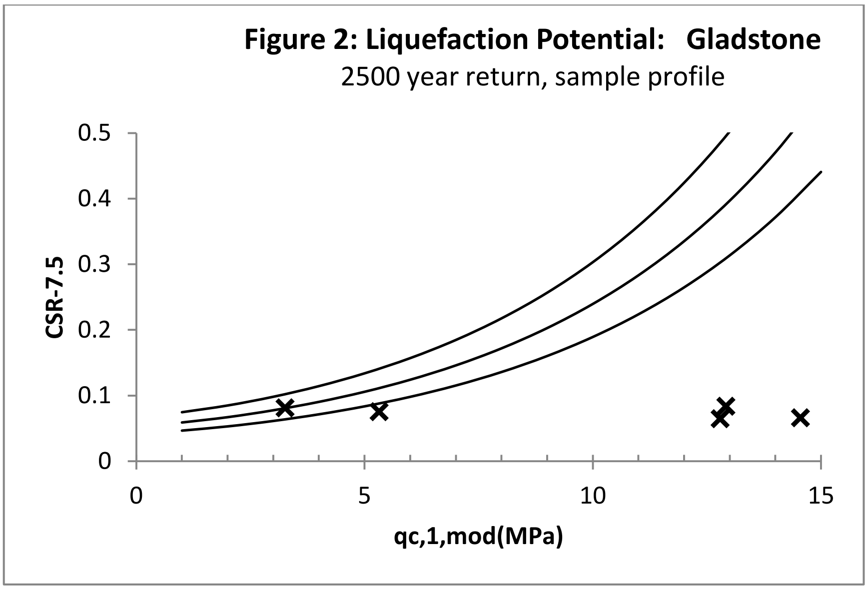
1. Using the raw tip measurements (q_c) and the friction ratio (R_f), make an initial estimate of the c value with equations (7) and (8) above.
2. Normalise the tip resistance with equation (6) followed by (5).
3. Using the normalized q_c value, calculate a revised c value with equations (7) and (8).
4. Repeat until convergence of the c value.

Step 6

After calculating the CSR value for a particular location, ground profile and depth in Step 4 and normalizing the CPT tip resistance in Step 5, the value should be compared to a relevant CRR curve. The curves presented in figures 2 to 5 have been generated using the probabilistic equation specified in Moss et al. (2006) with the following assumptions:

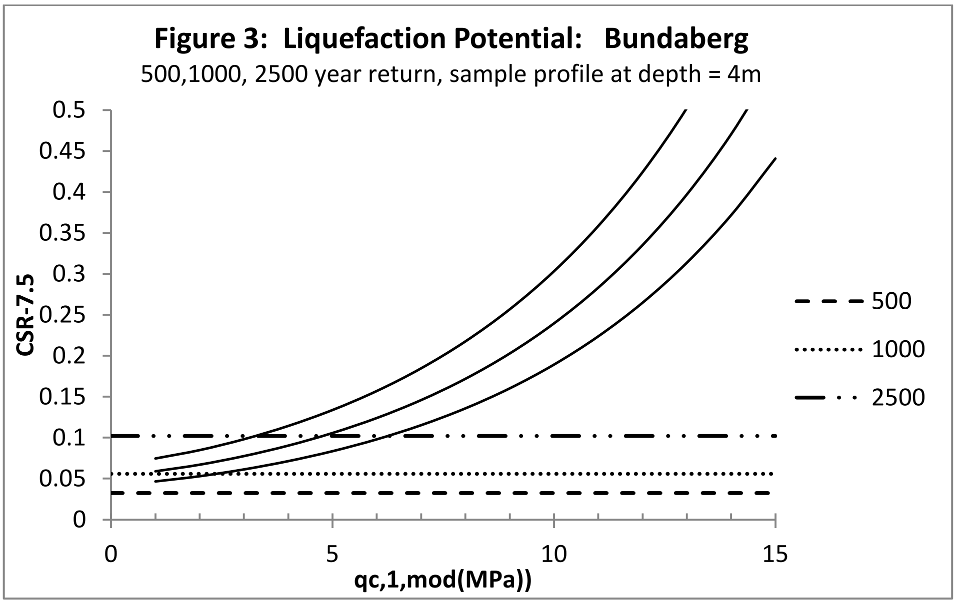
1. The CPT Friction Ratio is 0.5%, indicative of a sand with approximately <6% fines (Suzuki et al., 1995).
2. The normalization exponent c = 0.6.
3. M_w = 7.5
4. Probabilities include 15%, 50%, and 85%.

Figure 2 considers a structure in Gladstone designed to a 1/2500 annual probability of exceedance on the sample ground profile provided in Table 1. It is apparent that one point in the sample profile corresponding to depth = 4m has a greater than 50% probability of liquefaction during a 2500 year event. One additional point falls immediately below the threshold curve for a 15% probability of liquefaction. All remaining points reflect ground having sufficient strength to resist liquefaction.

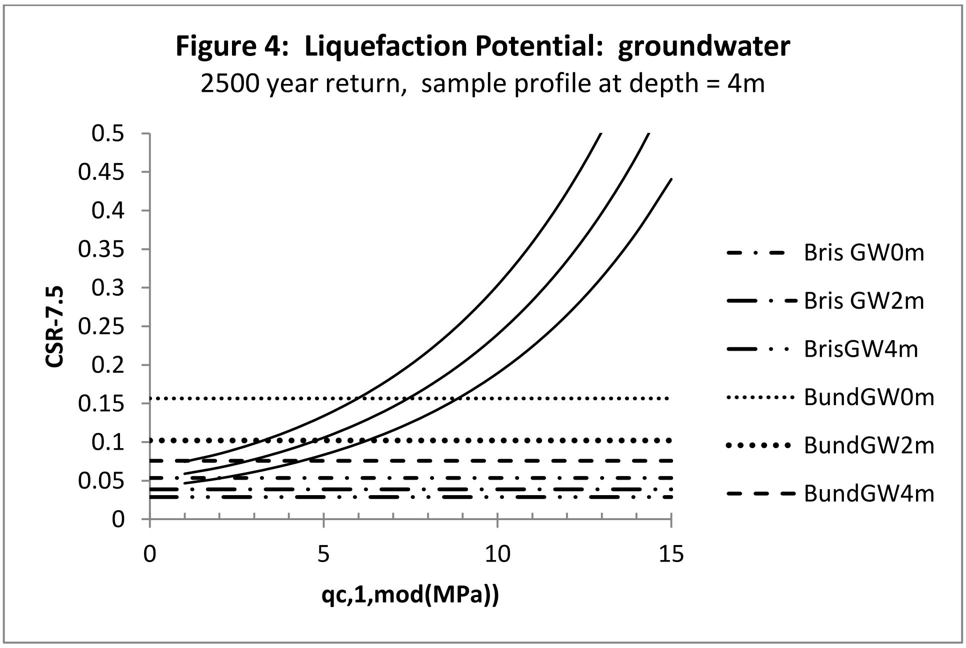


The steps above outline the process to carry-out an assessment of the potential for earthquake-induced liquefaction for any site in Australia. The below figures highlight the sensitivity of the curves to differing inputs. It should be noted that all curves have been generated in accordance with the Moss et al. (2006) equation using the following assumptions: R_f=0.5, σ' = 56kPa, Mw=7.5, PL = 15%, 50%, 85%

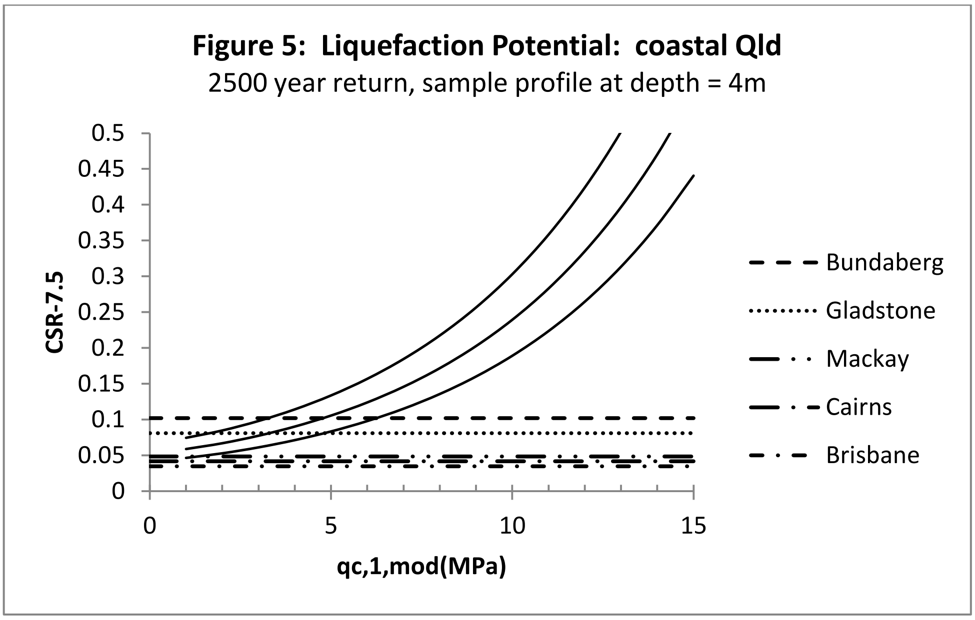
a) The annual probability of exceedance. Figure 3 reflects the CSR for a depth of 4m in the sample profile evaluated for differing annual probabilities of exceedance in Bundaberg. Evident is that regardless of CPT resistance, the 1/500 scenario falls below the 15% line of probability.



b) Depth to groundwater. Figure 4 reflects the influence of groundwater table on the assessment. For a 1/2500 annual probability of exceedance, three groundwater tables are assumed for Brisbane and Bundaberg and the CSR has been calculated for each at a depth of 4m. The influence of groundwater is significant and underscores the importance of reliable groundwater level data. As groundwater at surface level is the most onerous, it serves as an appropriate screening scenario for the possibility of liquefaction. The screening-level liquefaction Hazard maps for Australia created by Mote and Dismuke in 2011 appropriately adopt this assumption.



c) Geography. Figure 5 indicates the probability of liquefaction for the major urban concentrations along the Queensland Coast, assuming a 1/2500 annual probability of exceedance and the sample profile at depth = 4m. Reflective of the ground accelerations and magnitudes from Table 3, the difference in probability between the different cities in Queensland is apparent.



d) Factor of Safety. All of the above CSR figures have not accounted for any factor of safety. Factors of safety can be applied to the calculated CSR value as appropriate on a project-specific basis (FS = CRR/CSR). These normally lie between 1.0 and 1.4. However, given the application of a probabilistic treatment of liquefaction risk based on an earthquake ground acceleration and magnitude for a specific annual probability of exceedance, the incorporation of a factor of safety may be viewed as un-necessary.

4. CONCLUSION

A procedure has been summarized to carry-out a CPT-based assessment of the potential for liquefaction in Australia based on AS1170.4, the Dismuke and Mote (2011) magnitude values, and the Moss et al. (2006) probabilistic methodology. A sample clean sand profile has been created for the assessment of liquefaction potential along the Queensland coast in consideration of groundwater depth, annual probability of exceedance and geography.

Given the concentration of people and infrastructure along the Australian coastline, more effort is warranted to identify appropriate boundary parameters for assessment of liquefaction potential in this relatively thin slice of the country’s perimeter. Some of the equations presented above might be simplified in consideration of boundary parameters. Ultimately, some standardised guidance for the assessment of liquefaction potential should be incorporated into the Australian Standards.

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