Liquefaction Risk Potential of Road Foundation in the Gold Coast Region, Australia

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ABSTRACT

Soil liquefaction is a major concern for infrastructures constructed on saturated cohesionless soils in the event of an earthquake. This paper examines the liquefaction potential of the road foundations in twenty sites located in Gold Coast of South East Queensland. The preliminary seismic analysis was carried out in accordance with Seed and Idriss (1971) simplified procedure by using the in-situ SPT data. The factors of safety (F_S) was calculated for three different moment magnitudes of M_w = 3, 4 and 5, and the analysis was used to generate the Liquefaction Potential Index (I_L) and Liquefaction Risk Index (I_R) values for all sites. The calculated indexes are used to delineate the liquefaction resistance of the road foundations. The study shows that all cases were found to have low liquefaction failure potential for the moment magnitudes considered in the study.

KEYWORDS: liquefaction; SPT testing, Liquefaction Potential Index, Liquefaction Risk Index, failure

INTRODUCTION

Soil liquefaction has been one of the most active, complex, interesting and controversial topics in geotechnical earthquake engineering over the past 40 years. The term ‘liquefaction’ originally coined by Mogami and Kubo (1953), has historically been used in conjunction with various phenomena that involve soil deformations caused by monotonic, transient, or repeated disturbance of saturated cohesionless soils under undrained conditions (Kramer, 1996). The development of high pore water pressures due to the ground shaking and the upward flow of water may turn the sand into a liquefied condition with zero effective stress. Liquefaction research was accelerated in 1964 when the Good Friday Earthquake (M_w = 9.2) in Alaska was followed by the Niigata Earthquake (M_w = 7.5) in Japan, indicating spectacular damages including slope failures, flotation of buried structures, failures of bridges and several building foundations (Kramer, 1996).

Not all soil deposits are susceptible to liquefaction. In general, liquefaction requires three conditions: (i) loose to medium dense uniformly-graded, non-plastic, cohesionless soils, (ii) relatively shallow groundwater table to ensure full saturation of the deposit, and (iii) strong enough ground motion or cyclic loading (depending on amplitude and duration). Liquefaction can affect and damage many buildings, bridges, buried pipelines, and other constructed facilities in many different ways. In this regard, being able to evaluate accurate liquefaction potential of soils is the first step toward the mitigation of the damages caused by liquefaction. Several approaches by different researchers have been developed for evaluation of liquefaction potential of soils over the years (e.g., Seed and Idriss, 1971; Ishihara, 1996; Dobry et al., 1982; Iwasaki et al., 1982; Robertson and Wride 1997, 1998; Liao and Whitman 1986; Youd et al., 2001; Idriss and Boulanger 2004; Cetin et al. 2004, and others). Simplified methods using in-situ tests, originated by Seed and Idriss (1971), are widely used for this task. The in-situ tests commonly employed for liquefaction evaluation include the Standard Penetration Test (SPT), Cone Penetration Test (CPT) and Shear Wave Velocity Test (Vs). Though all these methods have their own advantages and disadvantages, they can be used for determination of the factors of safety (F_S) values as well as liquefaction potential evaluation.

The principal objective of the present study is to carry out a preliminary research on the soil liquefaction potential of the road foundations at twenty sites within the Gold Coast region. The preliminary seismic analysis was carried out in accordance with Seed and Idriss (1971) simplified procedure. To investigate the liquefaction risk and failure potential, a total of twenty SPT measurements at the road foundations are analysed and Liquefaction Potential Index (I_L) and Liquefaction Risk Index (I_R) values are calculated with respect to three different moment magnitudes of M_w = 3, 4 and 5.

SEISMICITY AND GEOLOGY

The Australian continent is far away from the boundary between the Australian and Pacific tectonic plates (pacific belt); therefore its historical earthquake record is quite short. However, strong earthquakes have occurred in Australia and more will occur. The largest earthquake on record a moment magnitude M_w = 7.3 event occurred in Meeberrie, Western Australia in 1941. Although harmful earthquakes are relatively uncommon in Australia, the high impact of individual events on the community makes them a costly natural hazard (Graner and Hayne, 2001).

The historical earthquake records of Queensland show that notable Queensland earthquakes include the 1918 “Bundaberg” earthquake sequence (M_w = 6.3) and also several earthquakes near Gayndah over the last 120 years. Although the overall earthquake hazard to South East Queensland is low, it can be high in many parts that are built on unconsolidated sediments or on Tertiary geological units. These ground conditions are expected to amplify the ground shaking from future earthquakes (Graner and Hayne, 2001). Therefore it is important that geotechnical engineers have the ability to predict and evaluate potentially liquefiable areas in order to mitigate the consequences. Once the potentially liquefiable areas are identified, then the consequences of the liquefaction can be investigated. In this regard, a detailed program research is necessary to be conducted on the liquefaction failure potential of the residual soils and mitigation methods should be considered.

Figure 1 shows the location of the study area in the city of Gold Coast. A total of twenty SPT boreholes were drilled while disturbed and undisturbed samples were taken. The depths of boreholes vary between 8m and 40 m. Gold Coast located in South East Queensland, possessing earthquake magnitude scale (M_w) of 3 to 5 and peak ground acceleration (a_{max}) of 0.05 to 1 (Figure 2). Therefore, in this study the liquefaction failure potential has been calculated for (a_{max}) =1 and three moment magnitudes of M_w = 3, 4 and 5, and used to define the Liquefaction Potential Index (I_L) and Liquefaction Risk Index (I_R) values.

Earthquake risk is not defined solely by the frequency and intensity of earthquakes. Some other factors include ground motion attenuation, site-specific soil conditions and the vulnerability of the building stock. Figure 3 (a & b) shows the geologic map of the city of Gold Coast. Geologically, the Gold Coast region is made up of three main groups of rocks and unconsolidated sediments. These are the Tertiary and older rock formations, fluvial (river) deposits and marine deposits. The rock formations make up the hilly and plateau landscapes of the hinterland, whereas fluvial deposits grade out into the coastal plains to merge with the marine deposits (Whitlow, 2000).

Figure 1: Location of the study area (Gold Coast)

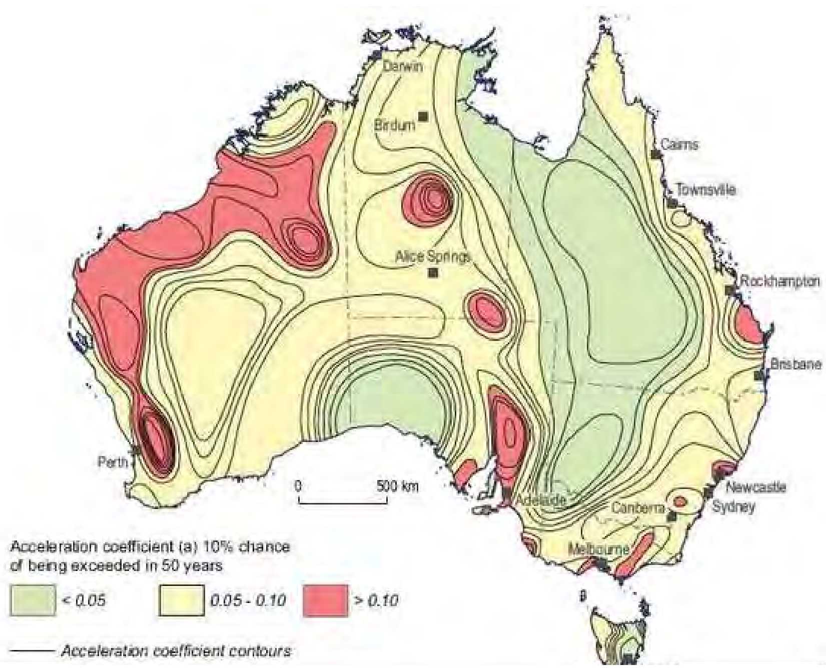
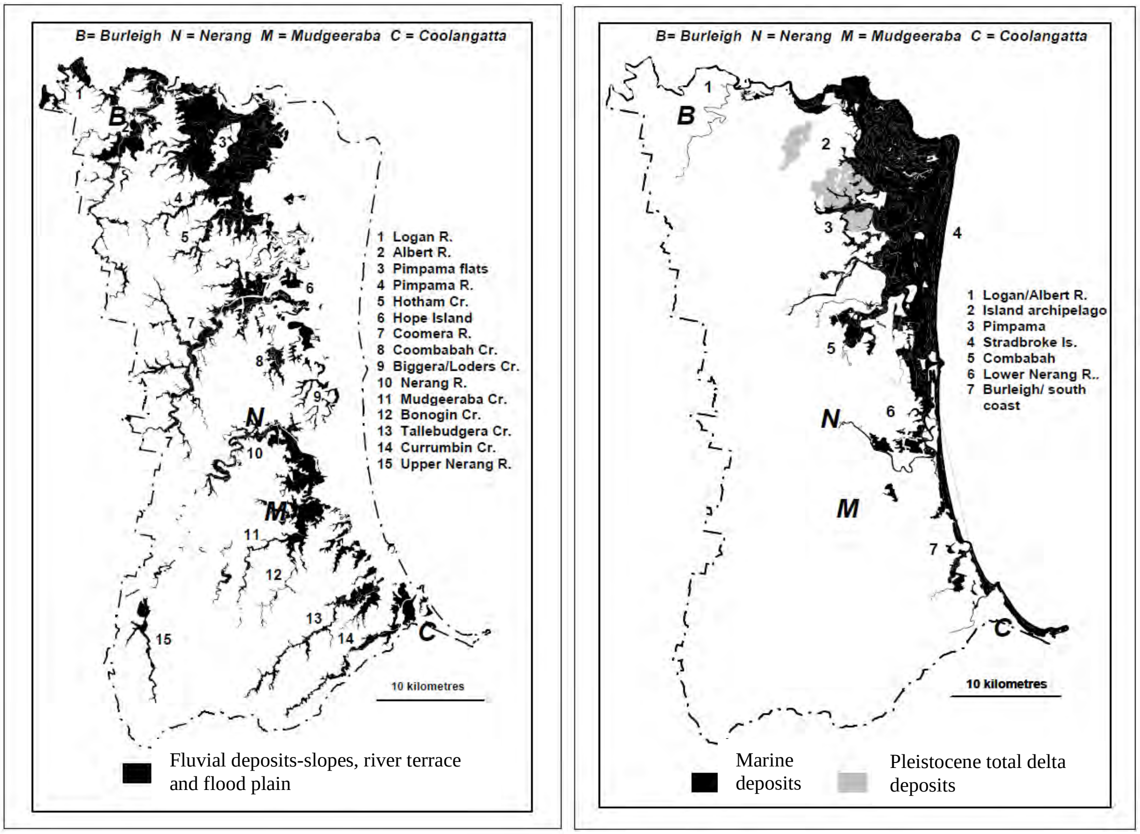


Figure 2: Earthquake hazard map of Australia



(a) (b)

Figure 3: Geologic map of Gold Coast: (a) Main Areas of Fluvial deposits, (b) Main areas of Marine and Pleistocene deposits, after (Whitlow, 2000)

SPT-BASED SIMPLIFIED METHODS

The Standard Penetration Test (SPT) has been widely used for the site specific evaluation of liquefaction potential of the soils, as well as for developing liquefaction resistance profiles. It is noted that the simplified method used in this study follows the general stress-based approach pioneered by Seed and Idriss (1971), requiring the determination of two variables, namely, cyclic stress ratio (CSR) and cyclic resistance ratio (CRR). In order to evaluate soil liquefaction potential of the soils, CSR and CRR values were taken into account for determination of the factor of safety (F_S) against liquefaction as well as Liquefaction Potential Index (I_L) and Liquefaction Risk Index (I_R) values.

EVALUATION OF CYCLIC STRESS RATIO (CSR)

In this paper, the equation for the earthquake loading (CSR) originally proposed by Seed and Idriss (1971) is adjusted to the benchmark earthquake (moment magnitude M_w = 7.5):

$$CSR_{7.5} = 0.65\left(\frac{a_{max}}{g}\right).\left(\frac{\sigma_v}{\sigma'_v}\right).(r_d) / MSF \qquad (1)$$

where a_{max} = the peak horizontal ground surface acceleration, g = the acceleration of gravity, σ_v = the vertical total stress of the soil at the depth studied, σ'_v = the vertical effective stress of the soil at the depth studied, r_d = the shear stress reduction coefficient (depth reduction factor) of the soil at the depth studied, and MSF = the magnitude scaling factor.

The variable r_d is calculated as follows (Liao et al., 1988):

$$rd = 1.0 - 0.00765z \ , \qquad z \leq 9.15 \text{ m} \tag{2a}$$

$$rd = 1.174 - 0.0267z \ , \qquad 9.15 \text{ m} < z \leq 23 \text{ m} \tag{2b}$$

The variable MSF is calculated as follows (Idriss, as cited in Youd et al., 2001):

$$MSF = 6.9 \exp(\frac{-M_w}{4}) - 0.058 \tag{3}$$

where M_w is the moment magnitude.

EVALUATION OF CYCLIC RESISTANCE RATIO (CRR)

Based on the corrected blow counts, the liquefaction resistance (CRR) was obtained from the following equation recommended by Rauch (Youd et al., 2001):

$$CRR_{7.5} = \frac{1}{34 - (N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10.(N_1)_{60} + 45]^2} - \frac{1}{200} \tag{4}$$

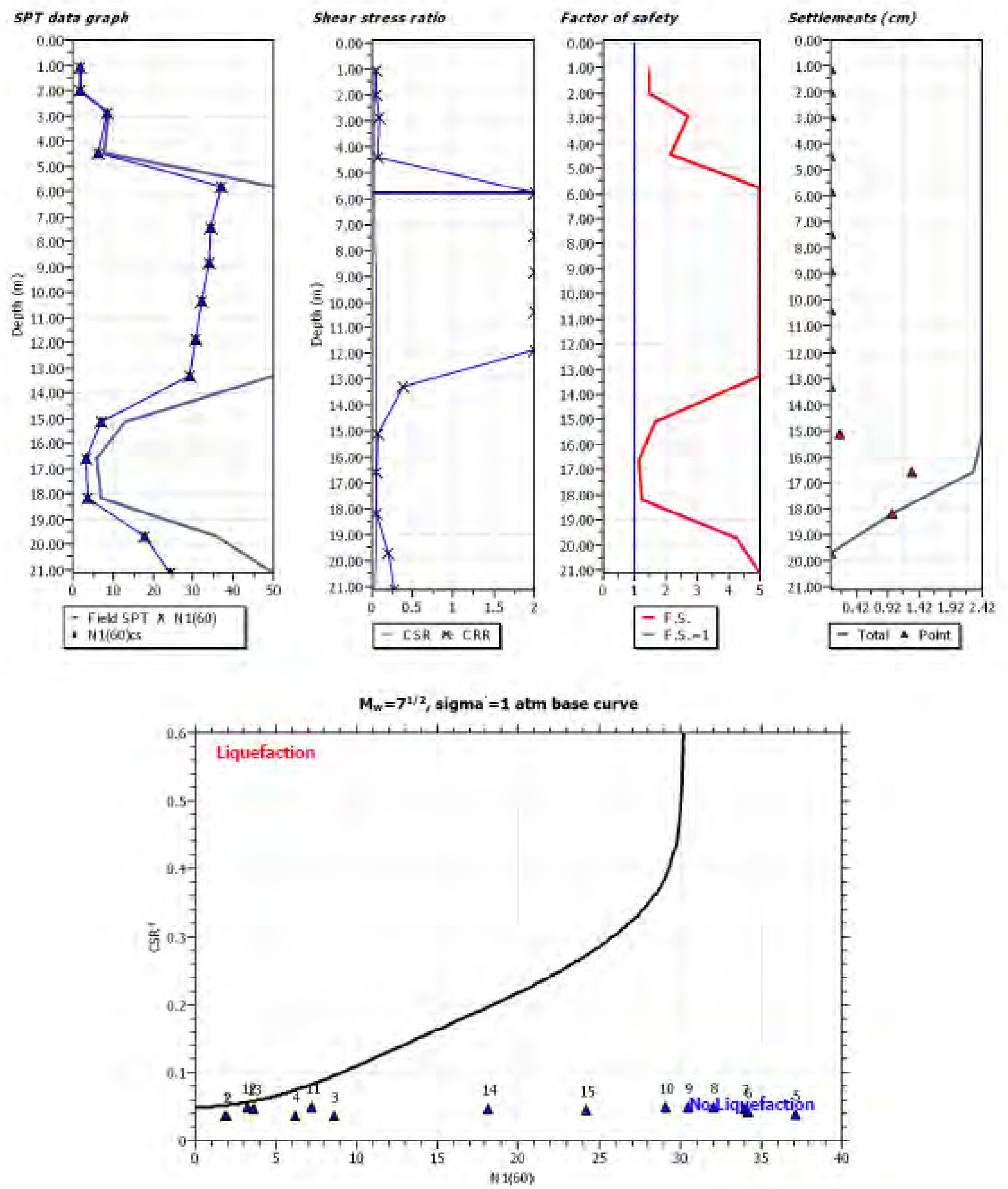
where $(N_1)_{60}$ = the SPT blow counts (N) normalized to an average overburden pressure of approximately 100 kPa and a hammer efficiency of 60%. This equation is valid for $(N_1)_{60} < 30$. For $(N_1)_{60} \geq 30$, clean granular soils are too dense to liquefy and are classed as non-liquefiable (Youd et al., 2001).

FACTOR OF SAFETY (FS)

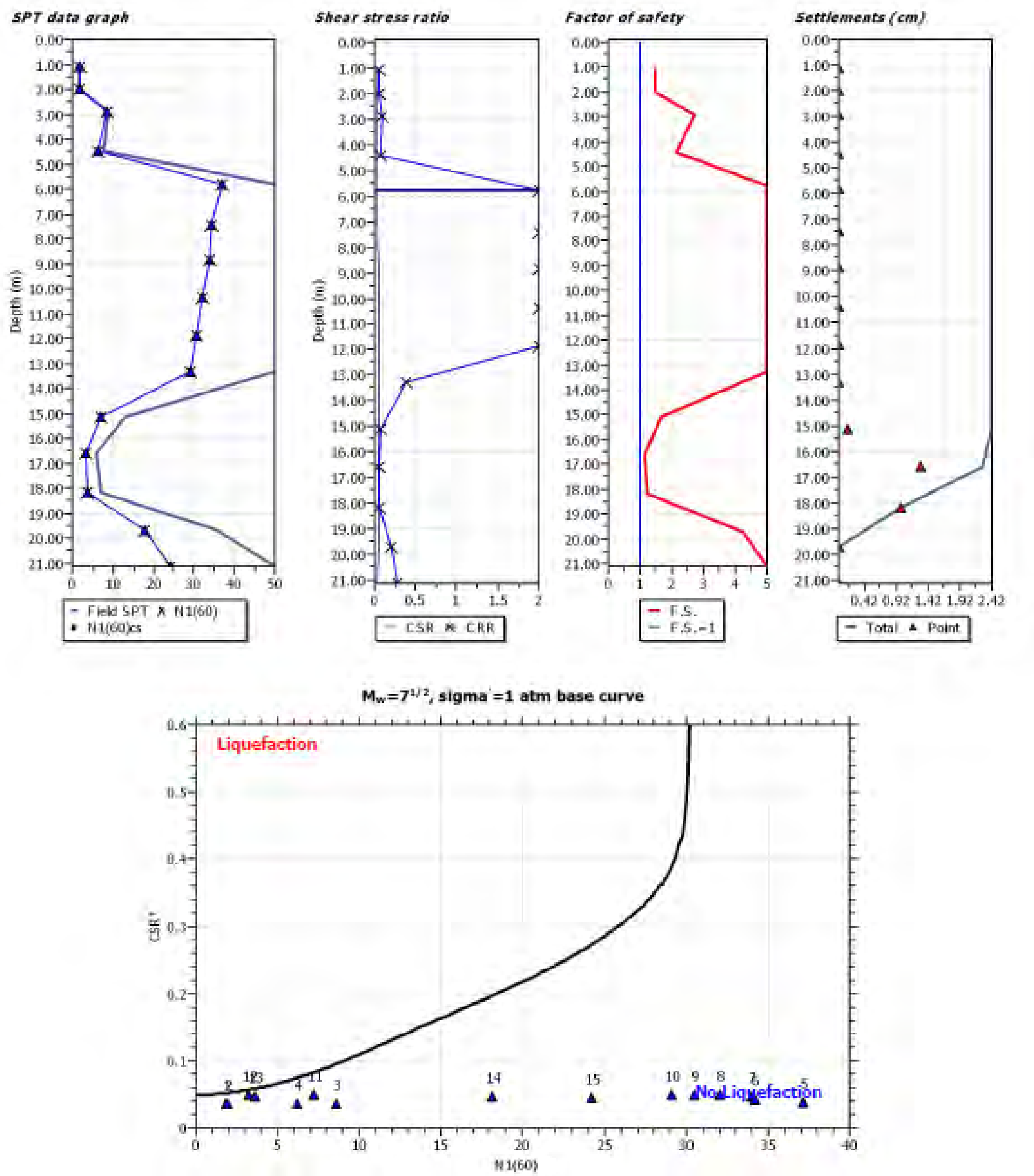
The potential for liquefaction can be evaluated by comparing the CSR calculated from eq. 1, with the $CRR_{7.5}$ determined from eq.4. This is usually expressed as a factor of safety against liquefaction by the following formula:

$$F_S = \frac{CRR_{7.5}}{CSR_{7.5}} \tag{5}$$

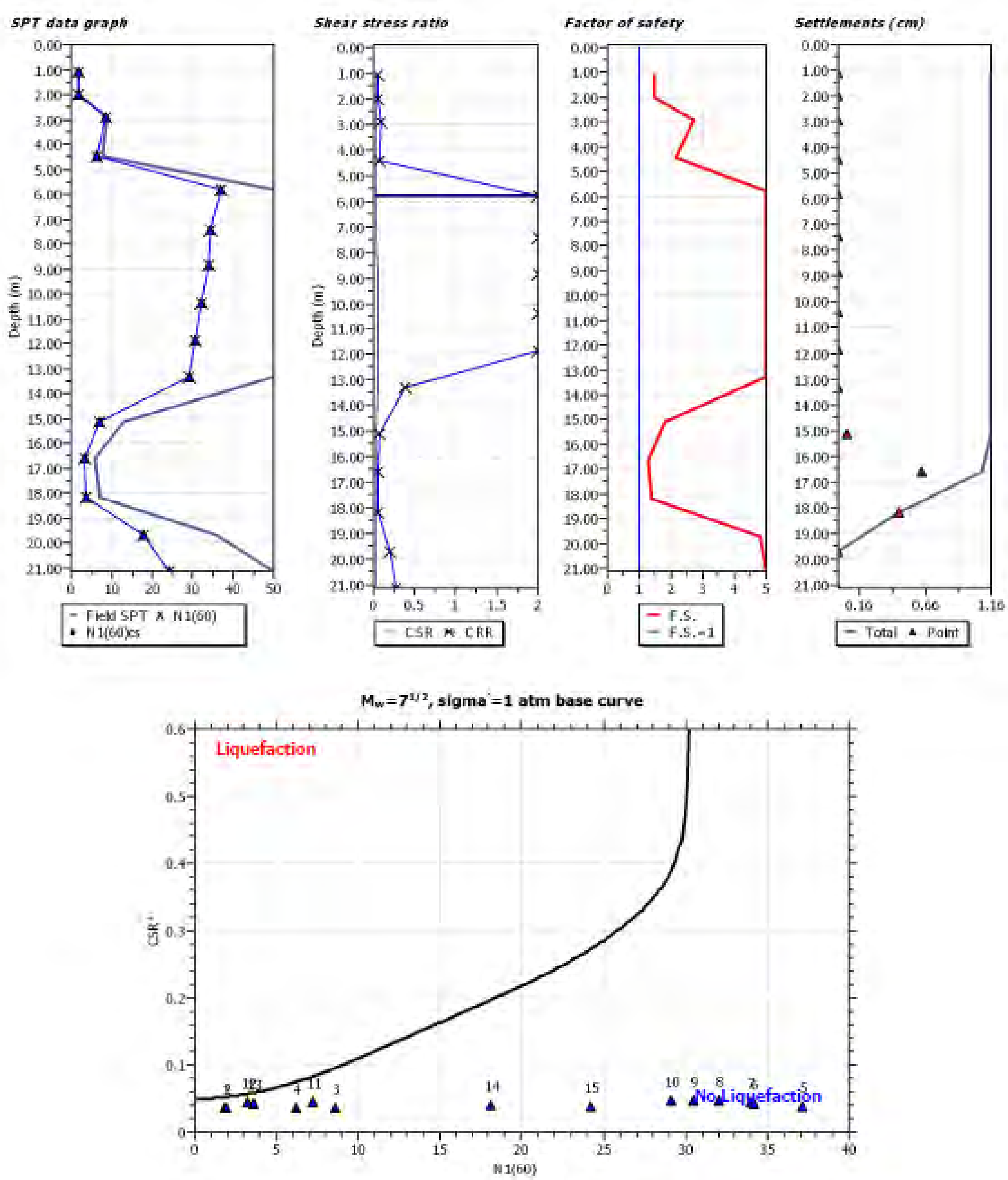
A typical site in GC-S2 located in Surfers Paradise, with SPT measurements is used to examine the F_S values against liquefaction for 3 moment magnitudes of M_w = 3, 4 and 5 with respect to a_{max}= 0.1 (Figure 4: a, b, and c). The soil in this SPT location mainly consists of sandy soils.


SPT data graph
Shear stress ratio
Factor of safety
Settlements (cm)
Depth (m)
Field SPT
N1(60)
N1(60)cs
CSR
CRR
F.S.
F.S.=1
Total
Point
Mw=7^1/2, sigma'=1 atm base curve
Liquefaction
No Liquefaction
CSR'
N1(60)

(a)


SPT data graph
Depth (m)
Field SPT
N1(60)
N1(60)cs
Shear stress ratio
Depth (m)
CSR
CRR
Factor of safety
F.S.
F.S.=1
Settlements (cm)
Total
Point
Mw=7^1/2, sigma'=1 atm base curve
Liquefaction
No Liquefaction
CSR*
N1(60)

(b)



(c)

Figure 4: Liquefaction analysis and Fs measurements via SPT testings in GC-S2, Gold Coast with respect to for 3 moment magnitudes of (a) $M_w = 3$, (b) $M_w = 4$ and (c) $M_w = 5$

The GC-S2 profile indicated very loose fine to medium sand at the depth of 2m and 4.5m as well as the loose sandy soils at 16m to 18m. The soil liquefaction analysis (Figure 4) indicated that with the M_w =3, 4 and 5, no liquefaction occurs at any depth and at all these depths; the F_S values are almost more than 1.

LIQUEFACTION FAILURE POTENTIAL

Liquefaction potential can be estimated from the simplified methods (Seed and Idriss, 1971) for a specific location and depth within the soil. Therefore, additional methods are required to quantify the liquefaction potential for an entire borehole. In this study, Liquefaction Potential Index (I_L) and Liquefaction Risk Index (I_R) are proposed to investigate liquefaction risk potential of the Gold Coast region.

LIQUEFACTION POTENTIAL INDEX (I_L)

Iwasaki et al. (1982) proposed the Liquefaction Potential Index (I_L) to evaluate the ground failure risk. The index I_L is defined as follows:

$$I_L = \sum_0^{20} F_1 W(z) dz \tag{6}$$

$$F_1 = 1 - F_S \ , \qquad F_S \leq 1 \tag{6a}$$

$$F_1 = 0 \ , \qquad F_S > 1 \tag{6b}$$

$$W_z = 10 - 0.5z \tag{6c}$$

where W(z) is a weight function of the depth and z is the depth from the ground surface in metres. W(z) is used to estimate the contribution of soil liquefaction at different depth to the failure of the ground. This method uses the factor of safety to determine the probability that liquefaction will affect the structures on the ground surface. Therefore, according to Iwasaki et al. (1982), damage caused by liquefaction can be severe if: 1) the liquefied layer is thick, 2) the liquefied layer is shallow and 3) the F_1 of the liquefied layer is less than 1.0. They provided the following liquefaction risk criteria, referred to herein as the Iwasaki Criteria:

$I_L = 0$, the liquefaction failure potential is extremely low;
$0 < I_L \leq 5$, the liquefaction failure potential is low;
$5 < I_L \leq 15$, the liquefaction failure potential is high;
$I_L > 15$, the liquefaction failure potential is extremely high

Table 1 shows the calculated I_L values with respect to three different M_w values via SPT testings for liquefaction resistance evaluation. It can be observed that all cases were found to have low liquefaction failure potential.

Table 1: Liquefaction Potential Index (I_L) at various sites in Gold Coast

SPT	**I_L (M_w=3)**	**Liquefaction failure potential**	**I_L (M_w=4)**	**Liquefaction failure potential**	**I_L (M_w=5**	**Liquefaction failure potential**
GC-S1	0	extremely low	0	extremely low	0	extremely low
GC-S2	0	extremely low	0	extremely low	0	extremely low
GC-S3	0	extremely low	0	extremely low	0	extremely low
GC-S4	0	extremely low	0	extremely low	0	extremely low
GC-S5	0	extremely low	0	extremely low	0	extremely low
GC-S6	0	extremely low	0	extremely low	0	extremely low
GC-S7	0	extremely low	0	extremely low	0	extremely low
GC-S8	0	extremely low	0	extremely low	0	extremely low
GC-S9	0	extremely low	0	extremely low	0	extremely low
GC-S10	0	extremely low	0	extremely low	0	extremely low
GC-S11	0	extremely low	0	extremely low	0.33	extremely low
GC-S12	0	extremely low	0	extremely low	0	extremely low
GC-S13	0	extremely low	0	extremely low	0	extremely low
GC-S14	0	extremely low	0	extremely low	0	extremely low
GC-S15	0	extremely low	0	extremely low	0	extremely low
GC-S16	0	extremely low	0	extremely low	0	extremely low
GC-S17	0	extremely low	0	extremely low	0	extremely low
GC-S18	0	extremely low	0	extremely low	0	extremely low
GC-S19	0	extremely low	0	extremely low	0	extremely low
GC-S20	0	extremely low	0	extremely low	0	extremely low

LIQUEFACTION RISK INDEX (IR)

The Liquefaction Potential Index I_L is defined with the profile of the factor of safety, and only those with $F_S < 1$ contribute to the index I_L. The liquefaction potential is not linearly proportional to the factor of safety; rather, it is linearly proportional to the probability of liquefaction (Lee et al., 2003). Thus, another index, called Liquefaction Risk Index (I_R) is defined by Lee et al., (2003) using the probability of liquefaction by the following formula:

$$I_R = \sum_0^{20} P_L W(z) dz \tag{7}$$

Where W(z) is the weight function as defined in eq. (6-b), and P_L is the probability of liquefaction obtained from the following equation by Chen and Juang (2000) by:

$$p_L = \frac{1}{(1+\frac{F_S}{1})^{3.5}} \tag{8}$$

Lee et al., (2003) provided the following liquefaction risk criteria as follows:
$I_R \leq 20$, the ground failure potential is low;
$20 < I_R \leq 30$, the ground failure potential is high;
$I_R \geq 30$, the ground failure potential is high;

Table 2 shows the calculated I_R values with respect to three different M_w values via SPT testings for liquefaction resistance evaluation. It can be observed that according to Lee et al., (2003) liquefaction risk criteria, all cases seem to have low risk of failure of liquefaction.

Table 2: Liquefaction Risk Index (I_R) at various sites in Gold Coast

SPT	I_R (M_w=3)	Ground failure potential	I_R (M_w=4)	Ground failure potential	I_R (M_w=5)	Ground failure potential
GC-S1	0.043	low	0.085	low	0.161	low
GC-S2	0.100	low	0.192	low	0.355	low
GC-S3	1.500	low	0.880	low	0.480	low
GC-S4	0.029	low	0.064	low	0.126	low
GC-S5	0.011	low	0.025	low	0.051	low
GC-S6	0.001	low	0.002	low	0.004	low
GC-S7	0.000	low	0.000	low	0.000	low
GC-S8	0.000	low	0.000	low	0.000	low
GC-S9	0.015	low	0.031	low	0.060	low
GC-S10	0.012	low	0.027	low	0.056	low
GC-S11	0.470	low	1.180	low	1.750	low
GC-S12	0.080	low	0.720	low	1.140	low
GC-S13	0.053	low	0.007	low	0.014	low
GC-S14	0.055	low	0.105	low	0.194	low
GC-S15	0.019	low	0.039	low	0.077	low
GC-S16	0.006	low	0.013	low	0.027	low
GC-S17	0.025	low	0.051	low	0.099	low
GC-S18	0.027	low	0.056	low	0.114	low
GC-S19	0.021	low	0.044	low	0.090	low
GC-S20	0.021	low	0.043	low	0.086	low

CONCLUSIONS

The seismic analysis of twenty road foundations in the Gold Coast region has been performed and presented in the paper. Based on the analysis of the SPT data, the following conclusions are reached in the preliminary study:

– When the SPT-based methods are integrated into the framework of the Liquefaction Potential Index (I_L) as defined by Iwasaki et al. (1982), the analysis showed that the foundation soils are mostly resistant to liquefaction and exhibit extremely low failure potential.

– The Liquefaction Risk Index (I_R) developed by Lee et al., 2003 showed an alternative method over the Liquefaction Potential Index (I_L) for predicting the liquefaction-induced failure potential. The term I_R is defined with the modified Chen and Juang (2000) method found to be less than 20. This means the ground failure potential is low for the Gold Coast region with respect to the three different moment magnitudes.

The simulation of the liquefaction resistance corresponding to three different moment magnitudes of M_w = 3, 4, and 5, may be used for future study of road foundations.

REFERENCES

1. Cetin, K. O., Seed, R. B., Der Kiureghian, A., Tokimatsu, K., Harder, L. F. Jr., Kayen, R. E., & Moss, R. E. S., (2004), "SPT-based probabilistic and deterministic assessment of seismic soil liquefaction potential.", Journal of Geotechnical and Geoenvironmental Engineering, ASCE 130 (12), pp. 1314-1340.

2. Chen, C. J., & Juang, C. H., (2000). ‘‘Calibration of SPT- and CPT based liquefaction evaluation methods.’’ Innovations applications in geotechnical site characterization, P. Mayne and R. Hryciw, eds., Geotechnical Special Publication No. 97, ASCE, New York, pp: 49–64.
3. Graner, K., and Hayne, M. 2001. Natural hazards and the risks they pose to south-east Queensland. AGSO-Geoscience Australia. ISBN: 0642467080
4. Idriss I. M., & Bouglanger, R. W. (2004). Semi-empirical procedures for evaluating liquefaction potential during earthquakes. Proceedings of the 11th ICSDEE & 3rd ICEGE. Berkeley, California, USA. pp. 32 – 56
5. Ishihara, K. (1996). Soil Behaviour in Earthquake Geotechnics. Oxford Engineering Science, 46, Tokyo University, New York
6. Dobry, R., Ladd, R. S., Yokel, F. Y., Chung, R. M., and Powell, D. (1982), “Prediction of Pore Water Pressure Buildup and Liquefaction of Sands During Earthquakes by the Cyclic Strain Method,” National Bureau of Standards Building Science Series 138, 150 pp.
7. Iwasaki, T., Arakawa, & T., Tokida, K., 1982. Simplified procedures for assessing soil liquefaction during earthquakes. Proceedings of the Conference on Soil Dynamics and Earthquake Engineering, Southampton, UK, pp. 925– 939.
8. Iwasaki, T., Tokida, K., Tatsuoka, F.,Watanabe, S., Yasuda, S., & Sato, H. (1982). “Microzonation for soil liquefaction potential using simplified methods.” Proc., 3rd Int. Conf. on Microzonation, Seattle, 1319–1330.
9. Kramer, S. T., (1996). “Geotechnical Earthquake Engineering”, Prentice Hall, Upper Saddle River, New Jersey
10. Lee DH, Ku CS, & Yuan H (2003). A study of the liquefaction risk potential at Yuanlin. Taiwan Eng Geol 71, pp: 97–117.
11. Liao, S.C.C., Veneziano, D., & Whitman, R.V., (1988). Regression models for evaluating liquefaction probability. Journal of Geotechnical Engineering, ASCE 114 (4), 389– 411
12. Liao, S. S. C., & Whitman, R. V. (1986). Catalogue of Liquefaction and Nonliquefaction Occurrences during Earthquakes, Res. Rep. Dept. of Civ. Eng., Massachusetts Institute of Technology, Cambridge, Mass. (cited in Youd et al, 2001).
13. Robertson, P. K. & Wride, C. E., (1997). Cyclic liquefaction and its evaluation based on the SPT and CPT. In Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils: Technical Report NCEER-97-0022: National Center for Earthquake Engineering Research, Buffalo, NY, pp. 41–87.
14. Robertson, P. K., & Wride, C. E. (1998). Evaluation Cyclic Liquefaction Potential Using the Cone Penetration Test. Canadian Geotechnique Journal, 35 (3): 442–59. doi:10.1139/t98-017
15. Seed, H.B.& Idriss, I.M., (1971). Simplified procedure for evaluating soil liquefaction potential. Journal of the Soil Mechanics and Foundation Division, ASCE 97 (9), 1249–1273
16. Whitlow, R, (2000). 'The Geology of the Gold Coast Region’. Geoscientific consultancy report. Gold Coast. Australia
17. Youd TL, Idriss IM, Andrus RD, Arango I, Castro G, & Christian JT (2001). ‘‘Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils.’’ J. Geotech. Geoenviron. Eng., 127 (10), 817–833.