DIFFICULTIES IN ASSESSING LIQUEFACTION POTENTIAL FROM CONVENTIONAL FIELD TESTING

Dr. Peter W. Mitchell[1] and Carly Moore[2]

1. BEng MEng PhD CPEng MIEAust, Senior Principal Geotechnical Engineer, URS Australia Pty Ltd, Adelaide, South Australia.

2. BEng/BA MIEAust, Structural Engineer, WorleyParsons Services Pty Ltd, Perth, Western Australia.

ABSTRACT

This paper discusses the difficulties associated with an assessment of the liquefaction potential of loose saturated sands at the site of a proposed large industrial complex in Adelaide, South Australia. An assessment of the potential for liquefaction was required, as a loss of foundation support would have considerable implications for footing and retention wall designs. The widely accepted Youd et al (2001) methods were used in determining a factor of safety against liquefaction for a design earthquake magnitude and ground surface acceleration. Questions in the assessment arose when the factor of safety against liquefaction determined from the results of standard penetration tests (SPT) differed significantly from those determined from cone penetration tests (CPT). Based on the SPT results, the site was predicted to be unstable against liquefaction for the design earthquake event, but the CPT assessment showed that the site had an adequate factor of safety. An investigation into the differences between results for a number of neighbouring sites revealed that the SPT *N* values were generally lower than values expected from published correlations with CPT profiles. A discussion of possible reasons for the disparities is presented within this paper and it is concluded that assessments based on SPT results alone are over-conservative for the ground conditions considered. Assessments using CPT results will provide a better assessment of liquefaction potential, as the CPT results are more reliable. The industrial site was therefore considered to have an adequate factor of safety against liquefaction for the design earthquake, resulting in considerable construction savings.

ACKOWLEDGEMENTS

The authors acknowledge the help and advice received from Dr Matthew Duthy, Brendan Scott, Leo Noicos and Dr Maria Pham of URS Australia Pty Ltd.

1. INTRODUCTION

Liquefaction is the temporary loss of shear strength in granular soils that results from increased porewater pressure during an earthquake. Accurately assessing the liquefaction potential of particular sites for the local 'design earthquake' can be important for a project, as liquefaction could have adverse consequences on foundation design and possibly lead to expensive protection measures.

A large industrial complex proposed for a site on the eastern side of LeFevre Peninsula in Adelaide SA is one such case. LeFevre is a peninsula of Holocene soil deposits (the St. Kilda Formation) which have been identified (Poulos et al 1996) as having a very high potential for liquefying during a sizeable earthquake event (Figures 1 & 2). This paper outlines the method used to assess the liquefaction potential and discusses the difficulties associated with conventional testing of the in situ soil conditions at the site.

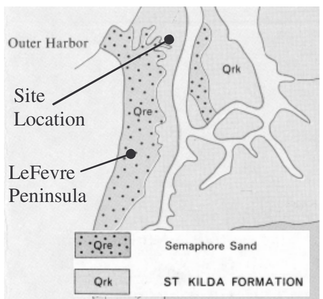

Figure 1 Geology of LeFevre Peninsula and vicinity (SA Dept. M & E 1989)

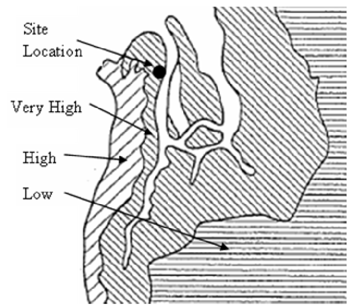

Figure 2 Liquefaction potential of Port Adelaide Area (Poulos et al 1996)

2. GEOLOGICAL PROFILE

Figure 1 (SA Dept. Mines & Energy 1989) shows that LeFevre Peninsula consists of Quaternary estuarine deposits, with coastal sand dunes on the western shore. From published data by Sheard & Bowman (1996), the geology of LeFevre Peninsula consists of the St Kilda Formation, overlying the Glanville Formation, underlain by the Hindmarsh Formation. The Hindmarsh Formation extends to about 90 m depth and overlies limestone, sandstone and dense sand of the Tertiary Dry Creek Sand Formation. Below about 130 m depth, the geological profile contains limestone, siltstone, dense sand and shale, with Proterozoic metamorphic rock types occurring below about 550 m.

Conventional field testing comprising standard penetration testing (SPT) and CPT electrical cone penetrometer testing (CPT) was conducted on the site in the upper soil layers, primarily for obtaining site-specific geotechnical parameters for design purposes. The SPT and CPT results were also used for the purpose of evaluating liquefaction potential. A typical soil profile for the site is presented in Figure 3. The water table is located at about 2 m depth, near the fill/natural soil interface. The St Kilda Formation, generally comprising sand and silty sand in a saturated and loose condition, has a typical grain size D_{50} = 0.15 mm and fines content of 5% to 10%. The Glanville Formation comprises calcareous (variably cemented) sand and limestone over stiff calcareous clay and is not prone to liquefaction (Seed 1996). The Hindmarsh Formation mainly comprises very stiff to hard silty clay. It is the loose, saturated sand and silty sand of the St Kilda Formation which is considered to be vulnerable to liquefaction.

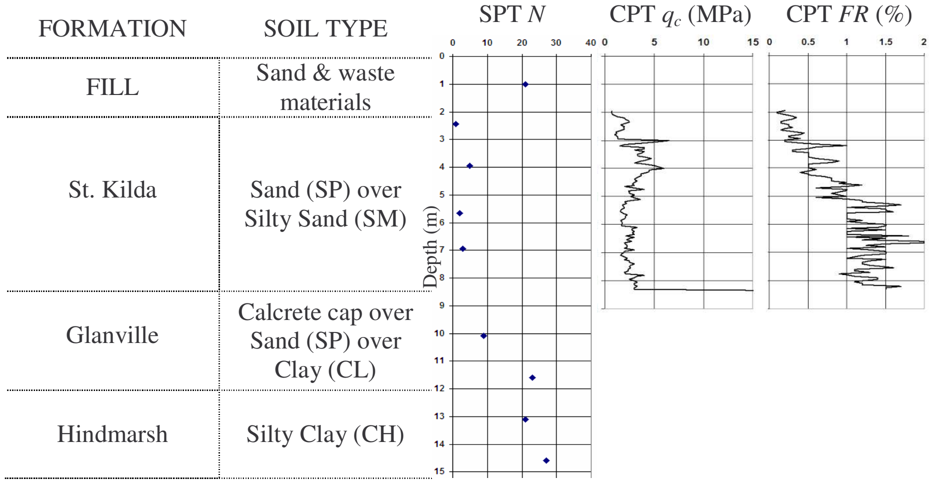

Figure 3 Generalised Soil Profile

3. LIQUEFACTION CONSIDERATIONS

Although liquefaction is a rare event in Australia, the two documented instances known to the authors (the 1897 Beachport, South Australia earthquake and the 1903 Warrnambool, Victoria earthquake) indicate that the effects of liquefaction on Australian soils can be as significant as those experienced in more seismically active countries (McCue 1978, Dyster 1996, Mitchell 2006). AS1170.4 (1993) requires the determination of earthquake loads based on a 10% probability of being exceeded in 50 years, corresponding to a return period of 475 years, which is the 'design earthquake' for which this liquefaction assessment was carried out.

4. PREDICTION OF LIQUEFACTION

In determining the potential for liquefaction, equivalent measures of earthquake loading and liquefaction resistance were compared, using the simplified method described by Youd et. al. (2001). The method involves a comparison between the seismic demand on a soil (cyclic stress ratio, *CSR*) and the liquefaction resistance capacity of the soil (cyclic resistance ratio, *CRR*). The procedure is based on empirical evaluations of field observations and both field and laboratory test data.

The *CSR* is a function of peak ground acceleration ($a_{\max}$), depth below ground and the ratio of total and effective vertical overburden stress σ_{vo}/σ'_{vo} (Equation 1).

$$CSR = f(a_{\max}, \sigma_{vo}/\sigma'_{vo}) \tag{1}$$

The resistance has been derived from observations of liquefaction occurring in a large number of magnitude 7.5 (moment magnitude, M_w) earthquakes. By plotting the *CSR* for each earthquake against soil strength, a curve defining the conditions of liquefaction was obtained. The curve gives the cyclic resistance ratio (*CRR*) as a function of normalised SPT or CPT test data and soil type (Equation 2). The SPT value $(N_1)_{60}$ is normalised to an overburden stress of 100 kPa and an energy ratio of 60%; the CPT value is the normalised, dimensionless value q_{c1N}. The *CRR* calculated from Equation (2) is adjusted by a magnitude scaling factor if the design earthquake magnitude, M differs from M_w=7.5.

$$CRR = f\left((N_1)_{60} \text{ or } q_{c1N}, \text{soil type}\right) \tag{2}$$

The prediction for liquefaction is then determined by a factor of safety (*FS*) by Equation (3), with *FS* greater than one indicating that liquefaction is not expected.

$$FS = CRR \ / \ CSR \tag{3}$$

It can be seen from the Youd et al (2001) method summarised above, that critical design parameters include the peak surface acceleration $a_{\max}$ and the earthquake magnitude M, which define the ‘design earthquake’.

5. VALUES OF M and $a_{\max}$ for THE DESIGN EARTHQUAKE

The magnitude recurrence plots for the Adelaide region by Love (1996a) and Gibson (1996) are shown in Figure 4. It can be seen that for a 475 year return period, an average value of the earthquake magnitude is M_L5.8 ($M_w = M_L$ at this magnitude).

The peak ground acceleration is the estimated rock acceleration corrected for site soil response. The value adopted for the site was $a_{\max} = 0.15$g for the following reasons:

1. AS1170.4 (1993) specifies for Adelaide an acceleration coefficient $a = 0.1g$, with an appropriate site factor $S = 1.5$, so that the equivalent $a_{\max} = aS = 0.15g$.
2. An analysis using Program SHAKE91 (Idriss and Sun 1992) using an accelerogram of a ‘thrust movement’ type earthquake and an ‘outcropping’ input ground motion of peak acceleration equal to 0.07g, appropriate for a rock profile in Adelaide, gives a peak ground acceleration for the site of $a_{\max} = 0.15g$ as summarised in Figure 5. It can be seen from Figure 5 that considerable amplification of the ground motion occurs in the St Kilda sands (labelled as ‘soft ground’ in Figure 5).

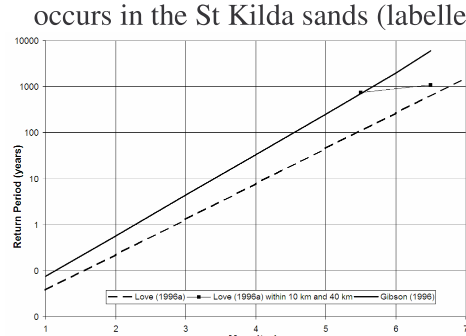

Figure 4 Earthquake magnitude recurrence relations for the Adelaide region.

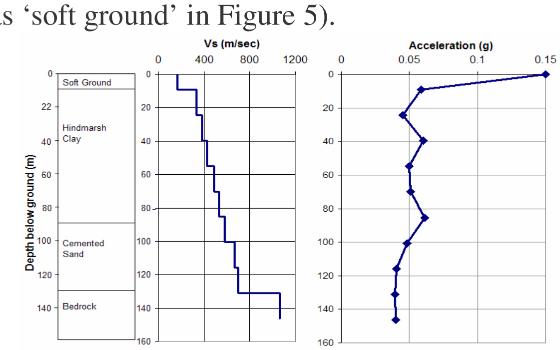

Figure 5 Soil Profile Adopted and Results of Program SHAKE91 for the site.

3. Analysis carried out by Love (1996b) of the intensity recurrence relation for Adelaide, indicated that for a 1 in 475 year earthquake, the intensity is MM6 to MM7. A relationship between peak ground acceleration ($a_{\max}$) and Modified Mercalli earthquake intensity was obtained by Wald et al (1999) from measurements from eight Californian earthquakes of magnitude 5.8 to 7.3. The relation is shown in Figure 6, and indicates that for an earthquake intensity of MM6 to MM7, the peak ground acceleration can be taken as $a_{\max}$=0.15g.
4. An approximate relationship between acceleration on soft soil sites and the acceleration on a nearby rock outcrop was given by Idriss (1990) (Figure 7). For $a = 0.07g$ in rock, appropriate for Adelaide, Figure 7 indicates that an acceleration of the order of 0.15g can be empirically adopted for a soft soil site.

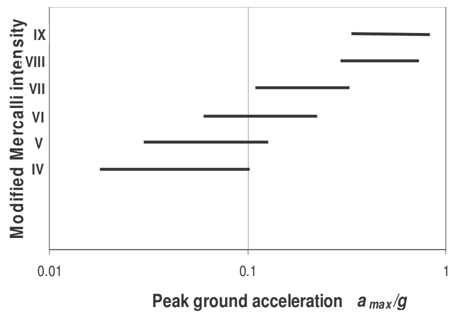

Figure 6 Earthquake intensity and ground acceleration (Wald et al 1999).

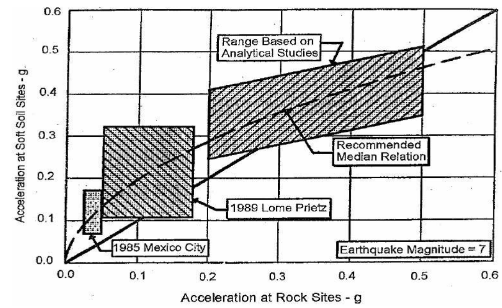

Figure 7 Accelerations on Soft Soil Sites vs. Rock Sites (Idriss 1990).

As the Idriss (1990) and Wald et al (1999) estimates were developed in areas of different seismic activity to that of Australia, caution must be used in adopting these methods. However, the estimations of peak ground acceleration of $a_{\max}$=0.15g by these methods are consistent with AS1170.4-1993 and the results of program SHAKE91.

It is acknowledged that the analysis by Poulos et al (1996) gave much higher peak ground accelerations for Adelaide than $a_{\max}$=0.15g used in this paper. However, as stated by Matuschka (1996), these higher accelerations are for higher earthquake intensities than associated with the 1 in 475 year design earthquake.

6. LIQUEFACTION ANALYSIS

The Youd et. al. (2001) method was used to assess the industrial site for liquefaction potential for the design earthquake, utilising both SPT and CPT results. Four locations across the site were assessed; each where a borehole was constructed with SPTs undertaken and a CPT was carried out adjacent to the borehole. For each location, a profile of factor of safety (*FS*) against liquefaction with depth was calculated from the CPT profile. The *FS* was also calculated at each discrete SPT test depth and plotted against the CPT *FS* profile, shown in Figure 8 for the St. Kilda Formation stratum.

As seen in Figure 8, it was noted that at each of the four locations, the *FS* calculated using SPT results was consistently and significantly lower than the *FS* calculated using CPT results. This is significant because if the assessment for liquefaction potential was carried out using SPT results alone, the site could be classified as being unstable against the design event. Conversely, the CPT analyses gave profiles of *FS* greater than 2 for the same stratum, suggesting it would be stable for the 1 in 475 year earthquake. To quantify, comparing the *FS* at each discrete SPT test depth with the CPT *FS* at the same depth, the SPT *FS* is on average 67 % less than the CPT *FS*, with the minimum difference being 47 % and the maximum 82 %.

The difference in *FS* values could have been a result of assumed input parameters, as the SPT method requires knowledge of the testing equipment (e.g. correction for hammer energy) and soil profile (fines content at each test depth). Based on the type of drill rig and SPT equipment, appropriate correction factors were applied to obtain normalised SPT *N* values, $(N_1)_{60}$. Particle size distribution (PSD) tests performed on five samples from two boreholes at the site showed the fines content to be 5 - 10 % within the St. Kilda Formation. A value of 10 % fines was used to calculate equivalent clean sand values $(N_1)_{60cs}$. The sensitivity of CRR to the hammer energy coefficient (C_E) and fines content was tested but showed that variations of +50 % to C_E and +20 % fines content did not increase the *FS* sufficiently to match the CPT value. It was therefore deduced that the difference was likely a result of the test data: either the SPT N values were lower than expected for the given soil profile, or the CPT q_c values higher.

Considering this theory, the same assessment was carried out for the design earthquake for boreholes with SPT and adjacent CPT across six other sites along the LeFevre Peninsula (14 assessments in total). It was found that in all cases, the *FS* calculated from SPT *N* values was lower than that calculated from CPT q_c at the same depth. The *FS* plot for one neighbouring site is shown in Figure 9 as an example. The accuracy of the test data was therefore considered to be the cause of the discrepancy between the SPT and CPT *FS* values. This was investigated further as discussed below.

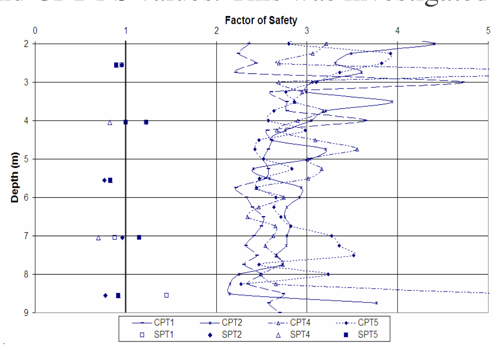

Figure 8 Factor of safety plots using SPT and CPT data for the industrial site

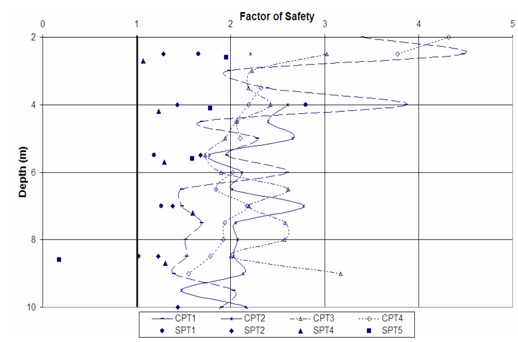

Figure 9 Factor of safety plots using SPT and CPT data for neighbouring site

7. REASONS FOR DIFFERENCES IN LIQUEFACTION ASSESSMENT

A number of published studies have been undertaken to relate the SPT *N* value to CPT cone resistance q_c, which can be used to check the consistency between the SPT and CPT results. Figure 10 shows the relationship between $(q_c/p_a)/N$ and grain size (with p_a being atmospheric pressure) from data summarised by Kulhawy and Mayne 1990 (as reported by Lunne et al 1997). For a grain size D_{50}≈0.15 mm for the St Kilda sand layer, $(q_c/p_a)/N$ lies within the range 2 to 6. Figure 11 shows the relationship between $(q_c/p_a)/N$ and fines content from other data summarised by Kulhawy and Mayne 1990 (as reported by Lunne et al 1997). For a fines content within the range 5 to 10 % (as for the St Kilda stratum) $(q_c/p_a)/N$ lies in the range 3 to 7.

For the St Kilda stratum, the measured values of *N* and q_c (shown in Figure 3) indicate $(q_c/p_a)/N$ ratios between 7 and 12, which are well outside the expected range from Figures 10 and 11. This indicates that either the measured SPT value is an under-estimation, or the CPT value is an overestimation of the in-situ strength of the sand.

To establish whether the electrical CPT readings recorded at the site overestimate the soil strength, results from the current project were compared with results of mechanical CPTs that were conducted on the same site for a previous project (PPK 1981). One such comparison is shown in Figure 12, which shows the current electrical CPT results are consistent with the previous mechanical CPT results. Based on the agreement between independent test results, it is considered that the CPT results from the current investigation are an accurate assessment of the soil strength at this site.

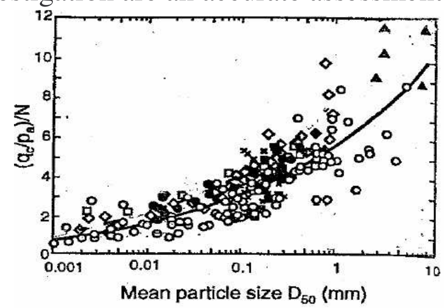

Figure 10 CPT-SPT Correlations with grain size (Lunne et al 1997).

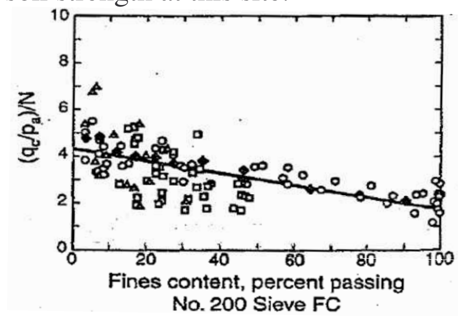

Figure 11 CPT-SPT Correlations with percent fines content (Lunne et al 1997).

To investigate the accuracy of the SPT results, geotechnical investigation reports for nearby industrial sites on LeFevre Peninsula were examined. One site was particularly revealing: two drillers each carried out a borehole with SPT in close proximity at different times. Results (Figure 13) show that driller 'B' gave unrealistically low SPT values at depth compared to driller 'A'. The difference increases as the depth below the water table increases, suggesting that the SPT can be largely operator dependant.

As the difference in SPT N values between the two drillers increases with depth below the water table, it is most likely that the difference in results is due to driller 'B' not maintaining an adequate fluid level in the borehole during drilling. An adequate head of drilling fluid must be carefully maintained to prevent the sand below the base of the drilling stem loosening (or even becoming 'quick') prior to the SPT being carried out. Disturbance and loosening of the sand below the base of the borehole prior to each SPT can lead to an underestimation of soil strength and is expected to be the reason for the abnormally high $(q_c/p_a)/N$ ratios. The assessment of liquefaction potential based on SPT results alone is therefore over-conservative for these sites and assessments using CPT results will provide a more appropriate estimation of liquefaction potential.

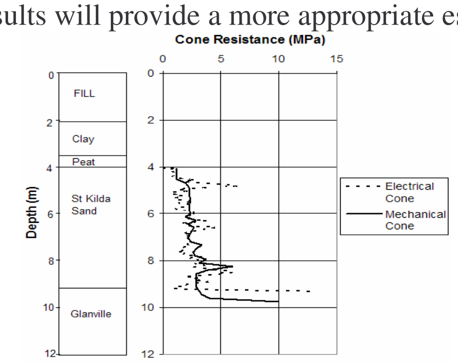

Figure 12 Comparison of electrical CPT and mechanical CPT at adjacent locations

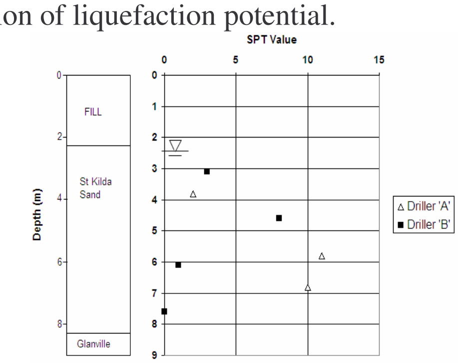

Figure 13 Comparison of SPT determined by different drillers at adjacent locations

8. CONCLUSIONS

An assessment of a site located on LeFevre Peninsula in Adelaide, SA was carried out for the design of a large industrial development. It was considered necessary to analyse the site for liquefaction potential, as the upper soil layer, the St. Kilda Formation, consists of loose saturated sands, which could be prone to liquefaction in an earthquake. The analyses by the Youd et. al. (2001) method, using both SPT and CPT test results, gave different factors of safety against liquefaction, suggesting a discrepancy in the in situ tests. Further investigation showed that SPTs on this and nearby sites typically under-predict the soil strength and that CPTs give a better estimation. Adopting the factor of safety based on CPT results resulted in significant construction savings.

9. REFERENCES

Dyster, T. (1996). Strong shock of earthquake. The story of the four greatest earthquakes in the history of South Australia. Dept. Mines & Energy South Australia. Report Book 95/47, September.

Gibson, G. (1996) Review of seismicity, MFP site, Adelaide. MFP Australia, Adelaide, vol. 5 section H4, Sept.

Idriss, I.M. (1990). Response of soft soil sites during earthquakes. Proceedings H. Bolton Seed Memorial Symposium, vol.2, BiTech Publishers Ltd., Vancouver, pp. 273-290.

Idriss, I.M. and Sun, J.I. (1992). Users Manual for SHAKE91. Centre for Geotechnical Modelling, Dept. Civil & Env. Eng., Univ. of California, Davis, California.

Love, D. (1996a). Seismic hazard and microzonation of the Adelaide metropolitan area. Department Mines & Energy South Australia, Ref. DME 7/91, Report Book 96/27, July.

Love, D. (1996b). Earthquake risk in Adelaide. MESA Journal vol. 3, pp. 22-25, October.

Lunne T., Robertson P.K. & Powell J.J.M. (1997). Cone Penetration Testing in Geotechnical Practice. E & FN Spon, London.

Matuschka, T. (1996). Seismicity and stability–general report. 7th Australia New Zealand Conference on Geomechanics. Adelaide, pp. 267 – 272. July.

McCue, K.F. (1978) The value of historical records – the Warrnambool earthquake of July 1903. Proc. Royal Society of Victoria. Vol. 90, pp. 245-250.

Mitchell, P.W. (2006). Pile design for liquefaction effects. Australian Geomechanics. Vol. 41, no.4, pp. 47-56, December.

Poulos, H.G., Love, D.N. & Grounds R.W. (1996) Seismic zonation of the Adelaide area. Proc. 7th Australia New Zealand Conference on Geomechanics. Adelaide, pp. 331 – 342. July.

PPK (1981). Report on Geotechnical Conditions at Osborne Site for Eglo Engineering. Pak-Poy & Kneebone Pty Ltd. Report ref 81A7704, July 1981, unpublished.

SA Dept. M & E (1989). Soil Association Map of the Adelaide Region, 2nd Ed.

Seed, R.B. (1996). Liquefaction hazard evaluation proposed MFP project site Adelaide, Australia. MFP Australia, Adelaide, vol. 5 section H5, September.

Sheard M.J. & Bowman G.M. (1996). Soils, stratigraphy and engineering geology of near surface materials of the Adelaide Plains. Rep. 94/9, DME 565/79, July.

Wald, D.J., Quitoriano, V., Heaton, T.H. & Kanamori, H. (1999). Relationships between peak ground acceleration, peak ground velocity, and Modified Mercalli Intensity in California. Earthquake Spectra. Vol. 15, no.3, pp. 557-564, August.

Youd, T.L. et. al. (2001). Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, vol. 127, no. 10, October, pp. 817 – 833.