# PILE DESIGN FOR LIQUEFACTION EFFECTS


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## ABSTRACT


Although liquefaction is a rare event in Australia, with only two documented instances, a pile design must still cater for liquefaction in a design earthquake. Liquefaction can lead to a loss of shaft resistance, additional horizontal displacements and bending moment under inertia loads, pile buckling and lateral spreading. This paper describes the four loading stages suffered by a pile before, during and after an earthquake when liquefaction may or may not occur. The method of Youd et al (2001) is used to predict the potential for liquefaction with depth. The important parameters for use in a 'design earthquake', namely peak ground acceleration and earthquake magnitude, depend on the seismic activity of the region and have been derived for the area of Adelaide, South Australia. If the soil profile has the potential to liquefy, the geotechnical capacity, the lateral behaviour and the buckling potential of the pile under the inertia loads must be determined for the loss of soil support. If lateral spreading can occur, a further lateral analysis is required. This method is used by the author in routine pile design and an example of the design process together with four case histories is given. The importance of continuous sampling and very careful logging of the soil profile during the geotechnical investigation is emphasised for an accurate liquefaction assessment.


## 1 INTRODUCTION

Australia does not have the same earthquake hazards as some other countries because it is located entirely within a tectonic plate rather than straddling a plate boundary. However, even though less frequently, Australia has experienced strong intra-plate earthquakes of magnitudes similar to those in seismically more active countries.

A geotechnical hazard associated with strong earthquakes is liquefaction. Liquefaction leads to settlement, loss of bearing capacity and lateral flow sufficient to cause significant damage to structures, including structures supported on piled footings as shown in Figures 1 and 2.

It has been stated (Seis, 2006) that an earthquake exceeding magnitude 7 occurs somewhere in Australia every 100 years or so. It would be unnecessary and financially unacceptable to design every piled footing in Australia based on this magnitude of earthquake. However, AS1170.4 (1993) requires the determination of earthquake loads based on mainly a 1 in 500 earthquake event at a particular site (i.e. 10% probability of occurring in 50 years – the so-called 'design earthquake'). It is therefore reasonable to expect geotechnical engineers to determine the likely extent of liquefaction for a 1 in 500 year earthquake event, and to make allowance for this in a pile design.

This paper outlines the procedure adopted by the author for the routine design of piled footings considering the possible effects of liquefaction, with examples of pile designs based on the seismicity of the Adelaide region.

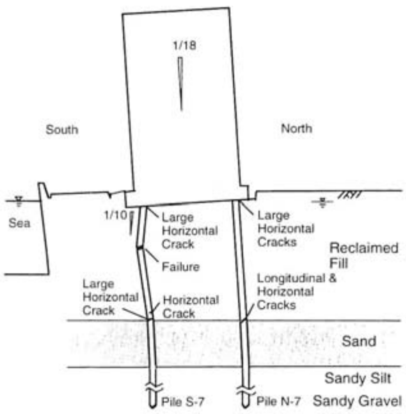


Figure 1: Pile failure by liquefaction in 1995 Kobe earthquake (Tokimatsu and Asaka, 1998).

Figure 2: Bridge failure by liquefaction in the 1964 Niigata earthquake (photo courtesy NISEE)

## 2 LIQUEFACTION IN AUSTRALIA

Liquefaction is the phenomenon occurring in predominately saturated cohesionless soil during or immediately after an earthquake when the pore pressure increases to a point where the effective stress becomes zero, resulting in a loss in shear strength. Sand boils, subsidence and lateral flow are the typical results of liquefaction.

To the author's knowledge, only two Australian earthquakes are known to have caused liquefaction, this statement remaining valid in the 10 years since this was made by Love (1996a). The two earthquakes are the 10 May 1897 Beachport (SA) earthquake and the 14 July 1903 Warrnambool (Vic.) earthquake. No seismograph record of either earthquake was made so that their magnitudes have been estimated using empirical methods. McCue (1975) estimated the 1897 Beachport earthquake to be magnitude $M_L$6.5, and McCue (1978) estimated the Warrnambool earthquake to be magnitude $M_L$5.3. This latter magnitude is surprisingly low, as earthquakes less than $M_L$5.8 (Williams 1988) or $M_L$5.5 (Love 1996a) are considered to be below the magnitude required to cause liquefaction. However liquefaction undoubtedly occurred during the Warrnambool earthquake. From mainly newspaper accounts of the Warrnambool earthquake, McCue (1978) stated that

'*…..the ejected material, seemingly a black silty sand and water, came from a depth of at least two metres, the depth of the fissures, and formed craters up to two metres across and half a metre high. The subsidence of sand embankments along both rivers was no doubt associated with liquefaction of the underlying sand layer.*'

Eye witness accounts of liquefaction during the Beachport earthquake are equally convincing. For example Goode (1942), recalling childhood experiences 45 years after the event writes

'*….we found a well, about 15 ft [4.6 m] deep, overflowing and flooding the garden. The water was stained yellow by a fine sand which we had never seen before.…......That low country on the Kingston-Robe road…was flooded on May 10, not by rain but by water that was squeezed up through the ground. The same thing happened with the rabbit burrows; the water rushed out of them, carrying with it the same fine yellow sand.*'

Dyster (1996) gives photographic evidence of the effects of liquefaction as a result of the 10 May 1897 Beachport earthquake as shown in Figures 3 & 4. These suggest lateral spreading, that can occur during and after an earthquake on slightly inclined land as a result of liquefaction.

If this evidence for liquefaction is accepted, the experiences with the two known cases of liquefaction in Australia indicate that although such an event is rare, the effects of liquefaction can be as significant as those experienced in more seismically active countries. Hence, the potential for liquefaction under a 'design earthquake' requires consideration during the design of a piled footing (or in any other type of geotechnical design).

In the design of a pile for liquefaction effects, it is important to examine the various loading stages on a pile during an earthquake event, as examined in the next section.

Figure 3: Slumping and lateral movement from the 1897 Beachport earthquake (Dyster, 1996).

Figure 4: 1897 Beachport earthquake effects on land adjacent to Lake Battye (Dyster, 1996).

## 3 LOADING STAGES ON PILE DURING LIQUEFACTION

Although more work is required, recent studies (e.g. Yasuda and Berrill 2000; Berrill *et al*., 2001; Bhattacharya *et al.*, 2003, 2004; Bhattacharya and Bolton, 2004; Liyanapathirana and Poulos 2005a, 2005b; Olson and Stark 2002; Tokimatsu *et al.*, 1996; Tokimatsu and Asaka, 1998) have made significant advances in the understanding of the complex mechanisms involved in the soil-pile interaction during liquefaction. The following is a simplified summary of the main loading stages of a pile before, during and after an earthquake event (adopted from Bhattacharya *et al.*, 2004). These stages are shown in Figure 5, and are as follows.

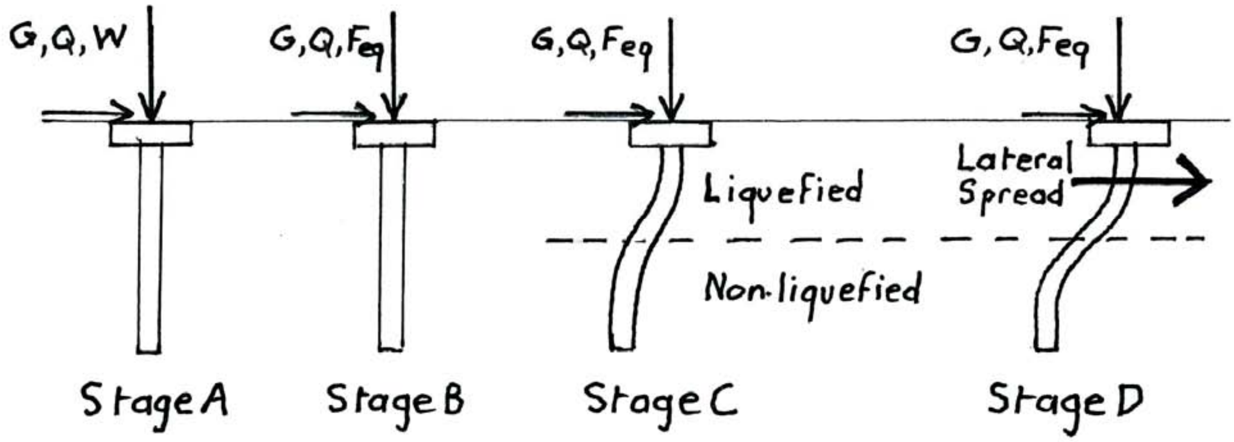


Figure 5: Loading stages for pile before and during liquefaction.

**Stage A:** Under no earthquake loadings, the pile will be subjected to the conventional design loads. For a building, these would normally comprise combinations of the dead load ($G$), the factored live load ($Q$) and the wind load ($W$).

**Stage B:** During an earthquake, the pile will be subjected to an earthquake load ($F_{eq}$) defined by AS 1170.4 (1993), as well as the dead load and factored live loads. The design wind load adopted in Stage A need not be considered as acting during the design earthquake. If no liquefication has resulted from the earthquake, then the soil surrounding the pile provides support to the pile.

**Stage C:** If the strength of the earthquake is sufficient to cause liquefaction over a certain depth, the soil support given to the pile decreases over the depth of liquefaction. Olson & Stark (2002) have shown that the shear strength of liquefied sand reduces to a small but finite amount. However for routine pile design the strength of liquefied soil is usually neglected. The loss of soil support leads to a loss of shaft resistance and hence larger pile settlement, additional horizontal displacements and bending moment under the lateral load, and the potential increases for the pile to buckle.

**Stage D:** If the site is located on even a slight incline or is located near a quay, lateral spreading of the liquefied soil imposes an additional horizontal load on the pile over the depth of liquefaction.

Bhattacharya et al. (2003; 2004) have indicated that pile buckling during Stage C loading is much more significant than previously thought. For a pile in a soil with a liquefied depth of $D_L$, and 'equivalent cantilever' length $D_L + D_F$ as shown in Figure 6, the critical buckling load $P_{cr}$ is given by equation (1).

$$P_{cr} = \frac{\pi^2 EI}{L_{ef}^2} \tag{1}$$

In equation (1), $EI$ is the effective pile stiffness and $L_{ef} = D_L + D_F$ for a restrained pile head as shown in Figure 6, and $L_{ef} = 2(D_L + D_F)$ for a free-head pile. The determination of the depth $D_F$ thus giving the 'equivalent cantilever' length can be determined from the lateral pile analysis that is undertaken to determine the effect of the horizontal load in Stage C. For concrete piles, the effective stiffness $EI$ would most probably be a post-cracking stiffness. The effective stiffness for a concrete pile can be derived using the Branson formula for section modulus by AS 3600 (2001) Section 8.5.3.1 (c). The dependence of the effective section modulus on the bending moment induced by the lateral load on the member requires an iterative solution.

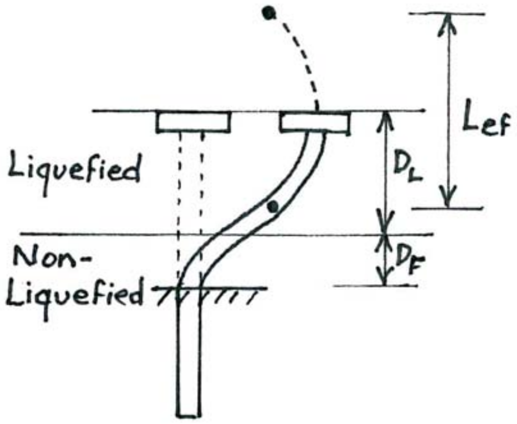


Figure 6: Buckling and the effective length of a pile with a restrained head.

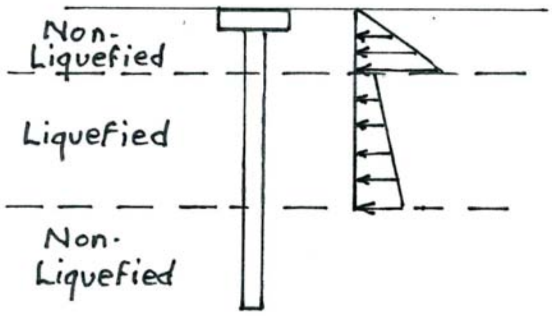


Figure 7: Applied pressures used by the Japanese Road Assoc. (Yasuda and Berrill, 2000).

A simple method of design for the soil forces associated with lateral spreading during Stage D in Figure 5 is the method developed by the Japanese Road Association as a result of the 1995 Kobe earthquake as shown in Figure 7 (Yasuda and Berrill, 2000; Fib, 2005). In the non-liquefied upper layer, the lateral pressure is the passive earth pressure, while in the liquefied zone the lateral pressure is 30% of the overburden.

It can be seen from Figure 5 that an important design decision is whether or not liquefaction will occur at a particular site under the design earthquake and, if liquefaction does have the potential to occur, then the depth of liquefaction needs to be determined. This is examined in the next section.

## 4 PREDICTION OF LIQUEFACTION

A common and convenient method of evaluation of the potential for liquefaction is by comparing equivalent measures of earthquake loading and liquefaction resistance by using the 'simplified' method described by Youd et al (2001). By this method, the earthquake loading is characterized by a cyclic shear stress ratio (*CSR*) with depth in the soil profile, with *CSR* being a function of the peak ground acceleration ($a_{max}$) and the ratio of the total and effective vertical overburden stress $\sigma_{vo}/\sigma'_{vo}$ by Equation (2). The value of $a_{max}$ is the estimated rock acceleration corrected for soil site response.

$$CSR = f(a_{max}, \sigma_{vo}/\sigma'_{vo}) \qquad \textbf{(2)}$$

The liquefaction resistance is characterized from observations on a large number of earthquakes of magnitude 7.5 (moment magnitude Mw) where liquefaction was either observed or not observed. By plotting the CSR value for each earthquake against the soil properties in terms of SPT or CPT values, a curve that bounds the conditions of liquefaction was obtained. The bounding curve gives the cyclic resistance ratio (CRR) as a function of SPT or CPT value and soil type (Equation 3). The SPT value is the normalized value N60 for an overburden stress of 100 kPa and for an energy ratio of 60%, and the CPT value is the normalized, dimensionless value $q_{c1N}$.

$$CRR = f(N_{60} \text{ or } q_{c1N}, \text{soil type}) \qquad \textbf{(3)}$$

The *CRR* calculated from Equation (3) is corrected for an earthquake magnitude *M* if the magnitude differs from $M_w$=7.5. For an earthquake of magnitude $M = M_L 6.0$ ($M_W$=$M_L$ at this magnitude), the correction factor can be taken as approximately 2.0.

The prediction for liquefaction is then determined from the factor of safety (*FoS*) by Equation (4), with a *FoS* greater than one indicating that liquefaction would not be expected.

$$FoS = CRR / CSR \qquad \textbf{(4)}$$

It can be seen from the Youd et al (2001) method summarised above, that critical design parameters include the appropriate values of the acceleration $a_{max}$ and the earthquake magnitude *M* defining the 'design earthquake'. These are dependent on the amount of seismic activity in the region of the particular site being examined. The appropriate values for the 'design earthquake' for Adelaide, South Australia are examined in the next section.

## 5 DESIGN EARTHQUAKE FOR ADELAIDE

As Adelaide has the highest earthquake hazard of any capital city in Australia (Geoscience Australia, 2004), many studies have been carried out into the seismicity of the Adelaide region (e.g. Greenhalgh and McDougall, 1990; Love 1996a; 1996b; McCue, 1975; Seed, 1996; Gibson, 1996; Poulos *et al*., 1996). The review and analysis carried out by Love (1996a, 1996b) indicated the intensity recurrence relation for Adelaide as shown in Figure 8 and the magnitude recurrence plot shown in Figure 9. It can be seen from Figures 8 and 9 that for a 1 in 500 year earthquake, the intensity is MM6 to MM7, and the magnitude is $M_L6$.

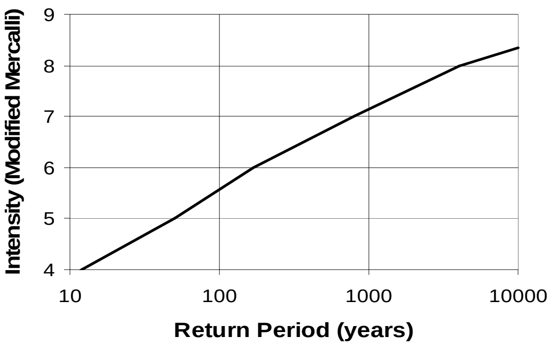


Figure 8: Earthquake intensity recurrence relation for Adelaide (Love, 1996b).

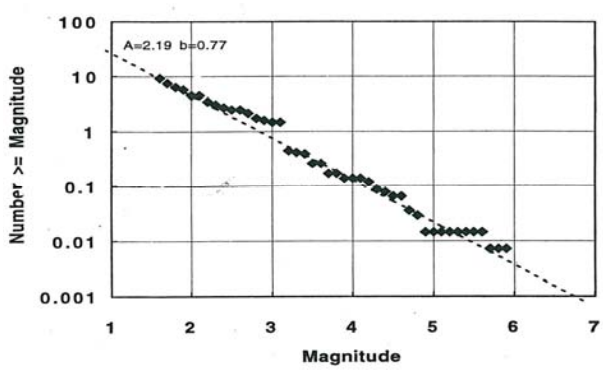


Figure 9: Earthquake magnitude recurrence relation for the Adelaide region (Love, 1996a).

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 10 and indicates that for an earthquake intensity of MM6 to MM7, the peak ground acceleration can be taken as $a_{max}$=0.12$g$, where $g$ is gravity. This is the value of peak ground acceleration adopted for Adelaide.

It is acknowledged that the analysis by Poulos et al. (1996) gave much higher peak ground accelerations for Adelaide than the 0.12$g$ used in this paper. However as stated by Matuschka (1996), these higher accelerations do not correspond to the magnitude of earthquake intensity used in the design earthquake, and so the Poulos et al. (1996) results are most likely to be an overestimate.

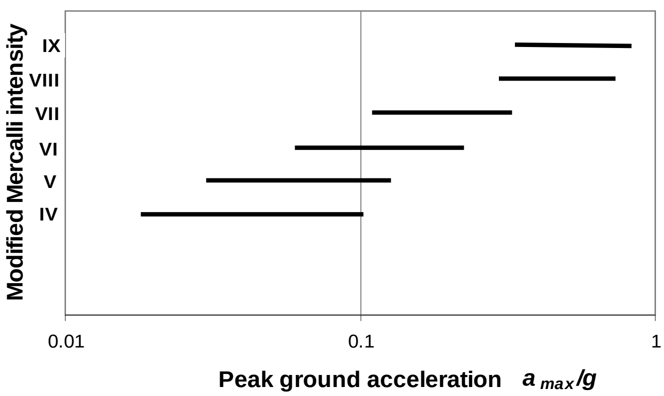


Figure 10: Earthquake intensity and ground acceleration in eight Californian earthquakes (Wald et al., 1999). The range of accelerations at each earthquake intensity is the range of the means of each earthquake.

For Adelaide, the earthquake parameters for the 'design earthquake' for use in the Youd et al. (2001) analysis are taken in this paper to be magnitude $M$=$M_L$=$M_W$=6, and peak ground acceleration $a_{max}$ = 0.12$g$. These parameters are consistent with those determined by Gibson (1996) for the Gillman area in Adelaide. Using $a_{max}$=0.12$g$ and $M$=6, the potential for liquefaction can be predicted by Youd et al. (2001) and if liquefaction occurs, the depth of liquefaction can be determined. The geotechnical and structural analysis can then be carried out to determine the pile capacities for the several load stages described in Section 3.

# 6 DESIGN EXAMPLE

The following example illustrates an application of the design process outlined above. Consider a concrete pile 400 mm square reinforced with 4-N20-2EF and restrained at its head, installed in Adelaide on the soil profile shown in Figure 11 ($N_m$ is the measured SPT value). The site is slightly inclined so that lateral spreading could occur. The earthquake load combinations acting on the pile are $S^*$=2500 kN vertical compression, and $F_{eq}$=50 kN acting laterally at the pile head. The pile is reinforced so that $\varphi Mu$ in bending under $S^*$=2500 kN is 263 kNm, cracking moment $M_{cr}$=52 kNm, uncracked $EI$=87 $MNm^2$ and cracked $EI$=12 $MNm^2$. The pile is founded in the very stiff clay at a depth of 18 m to develop the geotechnical strength required to support $S^*$=2500 kN.

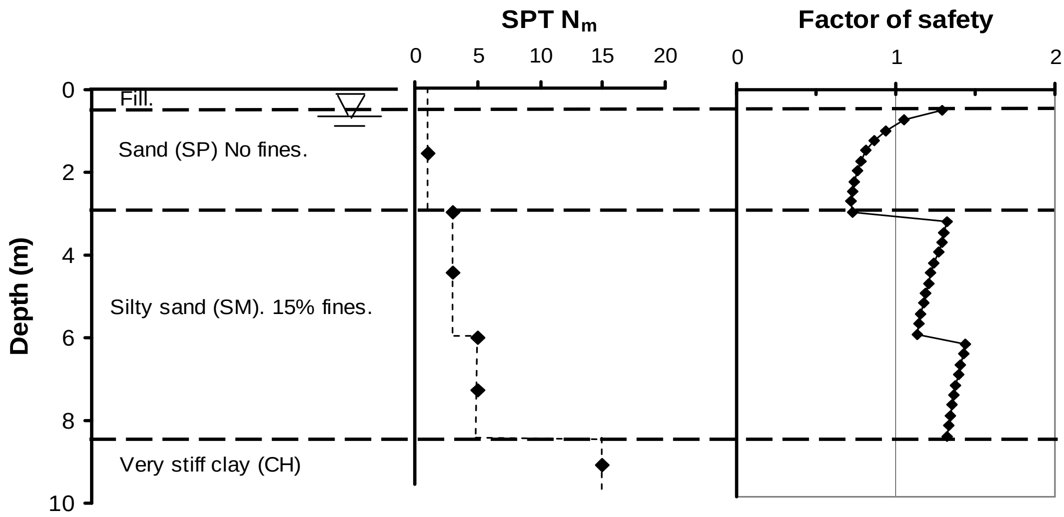


Figure 11: Soil profile and factor of safety against liquefaction for example pile.

Based on the SPT values, Figure 11 shows the results of the Youd et al. (2001) prediction of the factor of safety against liquefaction with depth, for a design earthquake of $M$=6.0 and $a_{max}$=0.12$g$. It can be seen that the sand from just below the water table to 3.0 m depth has the potential to liquefy. The overlying non-liquefied soil may offer some support to the pile during the earthquake, but this is neglected because it largely comprises fill with the possibility of sand boils during liquefaction of the underlying sand which would weaken the non-liquefied crust. The geotechnical capacity and the induced settlement of the pile needs to be checked for the loss of support over the upper 3.0 m depth.

A conventional lateral pile analysis for a horizontal load of 50 kN and the upper 3.0 m of the pile unsupported by the surrounding soil gives an induced bending moment of $M^*$=105 kN, which is within the pile capacity of 263 kNm. The effective pile stiffness corresponding to this bending moment is 20.8 $MNm^2$, and the deflected shape of the pile given by Figure 6 $L_{ef}$ = 4.2 m. The buckling load by Equation (1) is therefore 11,600 kN, which considerably exceeds the imposed load, so that buckling is not expected to be an issue.

During lateral spreading, an additional lateral load acts on the pile by the pressure distribution shown in Figure 7. The passive pressure on the pile in the non-liquefied upper soil, is $3\gamma' d\tan^2(45 + \varphi/2)$ where $\gamma' d$ is the effective overburden and $\varphi$ is the friction angle (eg Liyanapathirana and Poulos, 2005b). For the liquefied soil, the passive pressure is taken as $0.3\gamma' d$. A lateral pile analysis for these additional forces gives an induced bending moment $M^*$=170 kNm which does not exceed the pile capacity of 263 kNm. Note that a pile cap will attract additional load during lateral spreading.

The 400 mm square pile reinforced with 4-N20-2EF is therefore expected to cater for the effects of liquefaction during and after the design earthquake, although the effect of lateral spreading on the pile cap needs checking.

# 7 CASE HISTORIES

The author has used the above approach for routine pile design for several projects. The following case histories outline the application of the process for several projects in Adelaide, all of which were located in the zone classified by Poulos et al (1996) as having a very high potential for liquefaction.

### 7.1 CASE HISTORY 1 – APARTMENT BUILDING AT BRIGHTON

Figure 12 shows the soil profile and SPT results for a building at Brighton, Adelaide. Also shown in Figure 12 is the factor of safety against liquefaction using the Youd *et al*. (2001) analysis. It can be seen that the factor of safety is well in excess of 1.0, indicating that the pile design for this building need not consider the effects of liquefaction.

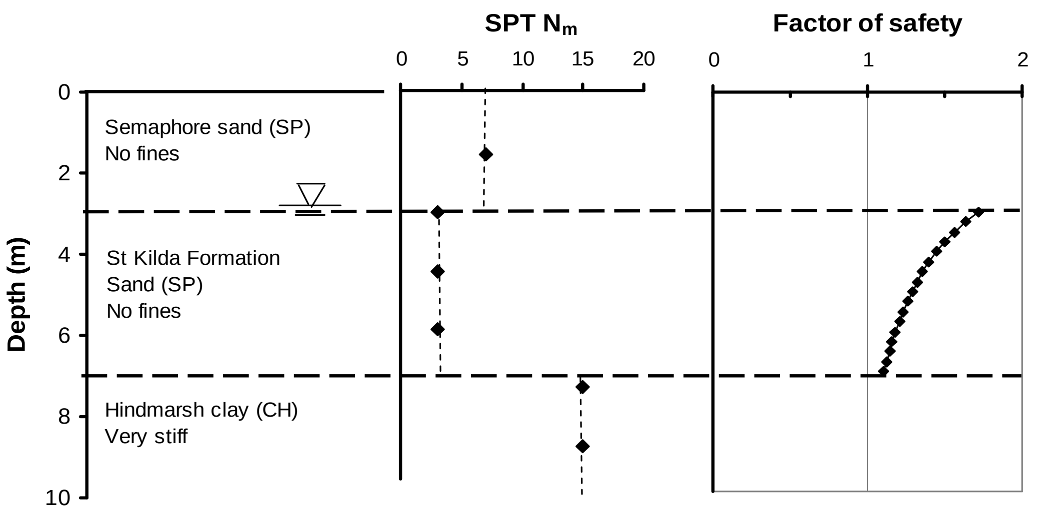


Figure 12: Soil profile and factor of safety against liquefaction for Case History 1.

### 7.2 CASE HISTORY 2 – PORT ADELAIDE BUILDING ON SILTY SAND.

Figure 13 shows the soil profile and SPT results for a building at Port Adelaide where the St Kilda Formation comprises silty sand with about 12% fines. Also shown in Figure 13 is the factor of safety against liquefaction using the Youd *et al.* (2001) analysis. It can be seen that the factor of safety does not extend below 1.0, indicating that the pile design for this building need not consider the effects of liquefaction.

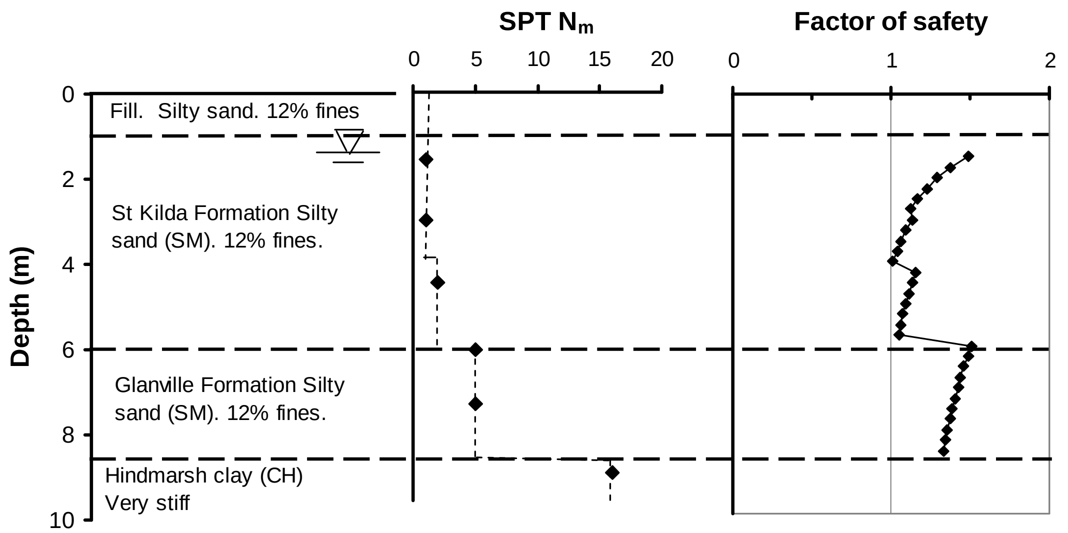


Figure 13: Soil profile and factor of safety against liquefaction for Case History 2.

### 7.3 CASE HISTORY 3 – PORT ADELAIDE BUILDING ON SAND.

Figure 14 shows the soil profile and SPT results for a building at Port Adelaide on a very similar soil profile to that of Case History 2, but where the St Kilda Formation is sand with no fines. Also shown in Figure 14 is the factor of safety against liquefaction using the Youd *et al.* (2001) analysis. It can be seen that the potential for liquefaction is high from about 3 m to 6 m depth where the factor of safety is below 1.0. The only difference between Case History 2 and 3 is the higher fines content in the St Kilda Formation in Case History 2. This illustrates the importance of continuous sampling and careful logging procedures being undertaken during the geotechnical investigation, because an overestimate of the fines content of sand can potentially lead to an under prediction of liquefaction. Even if a CPT was used to assess liquefaction potential, the Youd et al. (2001) procedure still requires boreholes for ground proving.

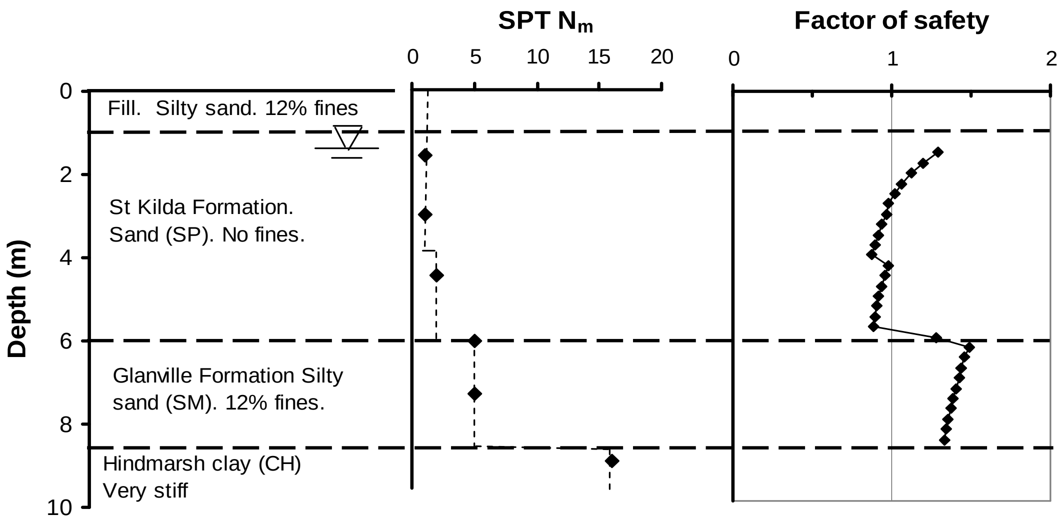


Figure 14: Soil profile and factor of safety against liquefaction for Case History 3.

### 7.4 CASE HISTORY 4 – PILE DESIGN ON A SLIGHTLY INCLINED SITE.

A geotechnical investigation and pile design considering liquefaction was carried out for a structure on a site that was slightly inclined towards the adjacent Port Adelaide River. The piles were bulbous base driven cast-*in-situ*. Careful procedures were carried out during the geotechnical investigation, involving continuous sampling of the soil profile, with independent checking of the borehole logs. The soil profile shown in Figure 15 was obtained. A Youd et al. (2001) analysis for the design earthquake indicated potential liquefaction in the upper 3.5 m depth. The critical design case for this project was the influence of load spreading, particularly because of the passive pressure from the upper 1.7 m non-liquefied crust.

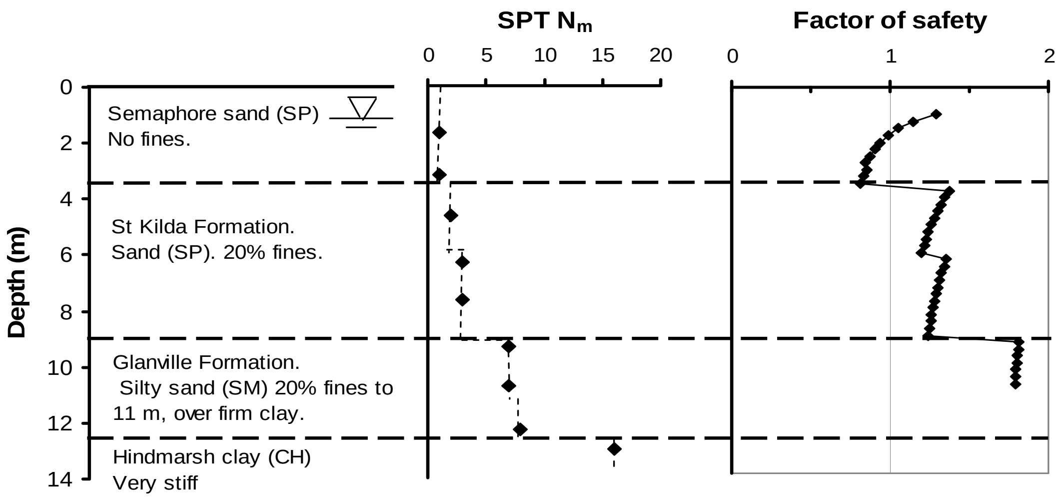


Figure 15: Soil profile and factor of safety against liquefaction for Case History 4.

## 8 CONCLUSIONS

Although earthquake induced liquefaction is a rare event in Australia, with only two documented instances (the 1897 Beachport and the 1903 Warrnambool earthquakes), a pile design must still consider the possible effects of liquefaction during a 'design earthquake'.

This paper outlines the approach used by the author for the routine design of piles considering the effects of liquefaction. The four loading stages on a pile before, during and after an earthquake that may or may not cause liquefaction are described. If liquefaction occurs, the effect of the loss of soil support on the bearing capacity and lateral behaviour of the pile under the earthquake inertia loads must be examined, together with an assessment of the potential for pile buckling. If lateral spreading can develop, a further lateral analysis must be carried out to quantify its effect.

The method of Youd *et al*. (2001) is used to determine the potential for liquefaction with depth. The parameters defining the 'design earthquake', namely the peak ground acceleration and earthquake magnitude depend on the seismicity of the region being examined. For Adelaide, these have been determined to be $a_{max}$=0.12$g$ and earthquake magnitude $M$=6.0. For the 'design earthquake' for Adelaide, an example of the design process is outlined and four case histories are described.

It is shown that continuous soil sampling and very careful logging during the geotechnical investigation is very important for an accurate assessment of liquefaction. This is because the liquefaction potential is dependent on the fines content of the soil, and an over-estimation of the fines content can lead to a non-conservative estimate of the liquefaction potential.

## 9 ACKOWLEDGEMENTS

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## 10 REFERENCES

AS1170.4 – 1993. Minimum design loads on structures, part 4: earthquake loads. Standards Australia.

AS 3600 (2001). Concrete structures. Standards Australia.

Berrill, J.B., Christensen, S.A., Keenan, R.P, Okada, W. and Pettinga, J.R. (2001). Case studies of lateral spreading forces on a piled foundation. Geotechnique. Vol. 51, no. 6, pp. 501-517, August.

Bhattacharya, S., Madabhushi, S.P.G. and Bolton, M.D. (2003). Pile instability during earthquake liquefaction. ASCE engineering mechanics conference (EM2003), Seattle. 16-18 July.

Bhattacharya, S., Madabhushi, S.P.G. and Bolton, M.D. (2004). An alternative mechanism of pile failure in liquefiable deposits during earthquakes. Geotechnique, vol. 54, no. 3, pp. 203-213.

Bhattacharya, S. and Bolton, M.D. (2004). Errors in design leading to pile failures during seismic liquefaction. Proceedings fifth international conference on case histories in geotechnical engineering. New York, N.Y., USA. 13-17 April, 2004.

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.

Fib, (2005). International Federation for Structural Concrete. Task group 7.4. 'Seismic design and assessment procedures for bridges'. Chapter 5, 'Foundation design'. Paris.

Geoscience Australia, (2004). Australian earthquakes. At www.ga.gov.au/urban/factsheets/earthquakes_australia. Accessed 24 May 2006.

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

Goode, E.W. (1942). Letter to the editor. The Adelaide Advertiser. 13 May 1942.

Greenhalgh, S.A. and McDougall R.M. (1990). Earthquake risk in South Australia. Civil Engineering Transactions, The Institution of Engineers Australia, no. 3, pp. 106-115, October.

Liyanapathirana, D.S. and Poulos, H.G. (2005a). Seismic lateral response of piles in liquefying soil. Journal Geotechnical and Geoenvironmental Engineering. ASCE. Vol. 131, no. 112, pp 1466, December.

Liyanapathirana, D.S. and Poulos, H.G. (2005b). Pseudostatic approach for seismic analysis of piles in liquefying soil. Journal Geotech. and Geoenvironmental Engineering. ASCE. Vol. 131, no. 12, pp 1480-1487, December.

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.

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

McCue, K.F. (1975). Seismicity and seismic risk in South Australia. The University of Adelaide Department of Physics, ADP137, 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.

Olson, S.M. and Stark, T.D. (2002). Liquefied strength ratio from liquefaction flow failure case histories. Canadian Geotechnical Journal, vol. 39, pp. 629 – 647.

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

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

Seis, (2006). Seismology Research Centre. Earthquakes in Australia. At www.seis.com.au. Accessed 24 May 2006.

Tokimatsu, K., Mizuno, H. and Kakurai, M. (1996). Building damage associated with geotechnical problems. Soils and Foundations, Special Issue on Geotechnical Aspects of the January 17, 1995 Hyogoken – Nambu Earthquake., pp. 219 – 234.

Tokimatsu, K. and Asaka, Y. (1998). Effects of liquefaction – induced ground displacement on pile performance in the 1995 Hyogoken – Nambu earthquake. Soils and Foundations, Special Issue on Geotechnical Aspects of the January 17, 1995 Hyogoken – Nambu Earthquake., no. 2, pp 163 – 177.

Wald, D.J., Quitoriano, V., Heaton, T.H. and 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.

Williams, D.J. (1988). Potential engineering risks in the earthquake hazard to the east coast of Queensland. Civil Engineering Transactions. The Institution of Engineers Australia, pp. 307 to 317.

Yasuda, S. and Berrill, J.B. (2000). Observations of the earthquake response of foundations in soil profiles containing saturated sands. Proceedings GeoEng 2000 - an international conference on geotechnical and geological engineering. Melbourne, Australia. pp. 1441 – 1470. November.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Liam Finn, W.D., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B. and Stokoe, K.H. (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, vol. 127, no. 10, October, pp. 817 – 833.