Liquefaction of Perth Sands

Rosemary Clough

School of Civil and Resource Engineering

Golder Associates

Abstract

Most people living in the city of Perth will never have experienced a major earthquake or strong ground motion. It is quite surprising therefore that the design of a great deal of construction in Perth is strongly influenced by the requirements for safety under earthquake conditions. These requirements are intended to minimise the risk of liquefaction of foundation soils, which can cause catastrophic damage. However, the methods used to measure this risk have been developed using data from overseas, predominantly Japan and the United State, and it is not known how appropriate they are for Perth soils. This study aims to address this problem by investigating the behaviour of soils from several different areas in the Perth region, and comparing the results obtained to those currently in use. This study is also interested in the prediction of liquefaction risk using Shear Wave Velocity testing, rather than the Standard Penetration Test or Cone Penetration testing which are currently more commonly employed.

1.0 Introduction

Liquefaction is " the transformation of a granular material from a solid to a liquefied state as a consequence of increased pore-water pressure and reduced effective stress" (Marcuson 1978, in Youd et al., 2001). In loose cohesionless soils, cyclic loading induces contraction as the sand densifies. In saturated soils this contraction leads to the development of positive pore-water pressure, and the increase in pore water pressure causes a corresponding decrease in effective stress. As the effective stress drops, the soil loses stiffness, and liquefaction occurs when the effective stress state falls to almost zero, causing the soil to fail catastrophically.

1.1 Predicting Liquefaction Risk

To determine the risk of liquefaction under a particular ground motion, one can compare the Cyclic Resistance Ratio (CRR) of a sand and the Cyclic Stress Ratio (CSR) expected to act upon it. Cyclic resistance ratio reflects the ability of a soil to resist liquefaction, and cyclic stress ratio is the seismic demand on a soil layer. If the CSR is greater than the CRR, liquefaction can be expected to occur. CSR is relatively straightforward to calculate, using the equation;

$$CSR = (\tau_{av}/\sigma_{vo}') = 0.65(a_{max}/g)\,(\sigma_{vo}/\,\sigma_{vo}')\,r_d$$

Where a_{max} = peak horizontal acceleration at ground surface
σ_{vo}= vertical overburden stresses
r_d = stress reduction coefficient.
Youd et al. (2001) give the estimates

$$r_d = 1.0 - 0.00765z \quad \text{where } z \leq 9.15\text{m}$$
$$= 1.174 - 0.0267z \quad \text{where } 9.15 \leq z \leq 23\text{m}$$

(Note that below 23 m depth the procedure is not verified.)

It is more difficult however to determine the CRR of a particular soil. Ideally it would be tested on undisturbed samples in the laboratory, however it is prohibitively expensive to conduct ground freezing to obtain such samples, and impossible to perfectly replicate the insitu stress conditions. To avoid these difficulties, the state of practice is to use In-situ sampling methods to predict the CRR. The most popular method is to use a modified version of the simplified procedure developed by Seed and Idriss in 1964. The original, mostly empirical method used SPT data and was relevant for earthquakes of a specific magnitude (magnitude 7.5). Since then, the procedure has been improved and adapted, and there now exist graphs and formulae of scaling factors for earthquakes of differing magnitudes, correction factors for criteria such as overburden pressure, differing composition (percentage fines) and other factors. Most importantly, the method has now been adapted for use with CPT and Vs data.

1.2 In-Situ Testing Methods

Three types of common in-situ tests have been mentioned as sources for the used in predicting liquefaction risk. Of these, the SPT is the oldest and most commonly used. The test essentially counts the number of hammer blows required to drive a sampling spoon a certain distance into the soil at the base of a borehole. It is relatively simple and cheap to do, however can be inaccurate if care is not taken. The CPT is somewhat more expensive, requiring more sophisticated equipment (a dedicated truck or rig to drive the cone into the ground, as well as the cone itself) and specialised operators, however it is more reliable and can deliver more information about soil profile than just its stiffness, such as pore water pressure profile and cone resistance (q_c). Vs testing basically involves the generation of a shear wave at a certain point in the soil which is picked up at another point. The distance travelled by the wave is measured and divided by the time taken for it to arrive to give the speed of the wave in the soil. Vs testing can be incorporated into CPTs by using an adapted rig, unlike the other two tests mentioned here it can also be performed in the laboratory using relatively small samples.

2.0 Methodology

2.1 Sands used in the study

It was initially intended to focus the study on a single type of sand, calcareous sand from the Kwinana area. However, once triaxial testing began it was decided to expand the study to include testing on two other sands, from Como and Bassendean, both siliceous, and to consider results of previous testing on sand from Shenton Park.

2.2 Testing

A large amount of testing was required for this study. The principal data was obtained from cyclic triaxial testing, however several index tests needed to be performed before this testing could be started, as the results from the preliminary tests were required to determine parameters for preparation of samples for the triaxial tests. Index testing was also used to accurately classify the sands. Following the completion of index testing, the triaxial testing was performed. In developing the testing schedule a range of variables were considered, with the aim of obtaining enough data to develop correlations between CSR, relative density, Vs and the number of cycles to failure.

2.2.1 Index Testing

Particle Size Distribution testing was performed on the three sands to help classify the soils and ensure that the fines content was minimal. Since the fines content and grading of a soil influence its tendency to liquefy, this set of tests was quite important. Minimum and maximum density testing was performed on the sands in order to enable determination of the relative density of each sample in the triaxial tests. Calcium carbonate testing assessed whether the composition of the sands matched what was anticipated from the locations they were obtained from. Specific gravity testing was necessary to determine the actual density and void ratio of the sand samples during testing.

2.2.2 Triaxial Testing

These tests were performed in the Soils laboratory at the University of Western Australia, using a modified triaxial machine with bender elements fitted to the top cap and base plate of the sample to measure Vs. Samples were prepared dry in a rubber membrane in a 72 mm split mould. The mould was tapped with a mallet, and in the case of higher densities, an extra mass was applied, until the desired density was obtained. Samples were flushed and saturated to a B value of at least 94% before being uniformly consolidated to 1MPa. Shear wave velocity was measured at this point, before the cyclic or monotonic test began. Cyclic testing was performed under an excess vertical load of 100 kPa, and stress loaded to the nominal CSR, although in practice the exact loads to obtain the requisite CSR were not reached in most cases. Loading was double amplitude cyclic loading, with a frequency of 0.1. The strain of the sample was monitored throughout the test, and testing was stopped after 100 cycles or when liquefaction was considered to have occurred. Liquefaction was judged to have occurred using the Japanese method of measuring the double amplitude strain. Generally a value of 5% strain is used, however on the advice of laboratory staff in this case 4% was used. Since strain was often much more pronounced in one direction, an average double amplitude strain of about 6% was also deemed to constitute liquefaction. A table of the tests successfully performed is given in Figure 1. The actual number of tests was in excess of 30, however, due to experimental problems such as membrane leaks, computer and apparatus problems and operator error, several tests did not provide usable data.

Sand	CSR	Dr	Sand	CSR	Dr
Kwinana	0.15	50	Como	0.175	15
Kwinana	0.15	30	Como	0.18	50
Kwinana	0.15	15	Como	0.18	25
Kwinana	0.175	15	Como	0.18	15
Kwinana	0.2	15	Como	0.2	70
Kwinana	0.225	15	Como	0.2	50
Kwinana	0.25	50	Bassendean	0.1	15
Kwinana	0.25	30	Bassendean	0.15	25
Kwinana	0.25	15	Bassendean	0.15	15
Kwinana	0.3	50	Bassendean	0.2	50
Kwinana	0.3	30	Bassendean	0.2	25
Como	0.15	25	Bassendean	0.25	30
Como	0.15	15	Bassendean	0.25	25

Figure 1 Cyclic Triaxial Testing Program

3.0 Results

3.1 Index Testing

3.1.1 Particle Size Distribution

The first test performed on the three sands was particle size distribution testing. All of the sands had a very small fines content, less than 1% in each case, hence no further testing of fines was considered necessary. The results of the Particle size density testing are presented in Figure 2 below, along with curves adapted from Shannon et al. ( ) showing an envelope of 19 soils which liquefied during earthquakes in Japan. All three soils sit reasonably well inside the envelope.

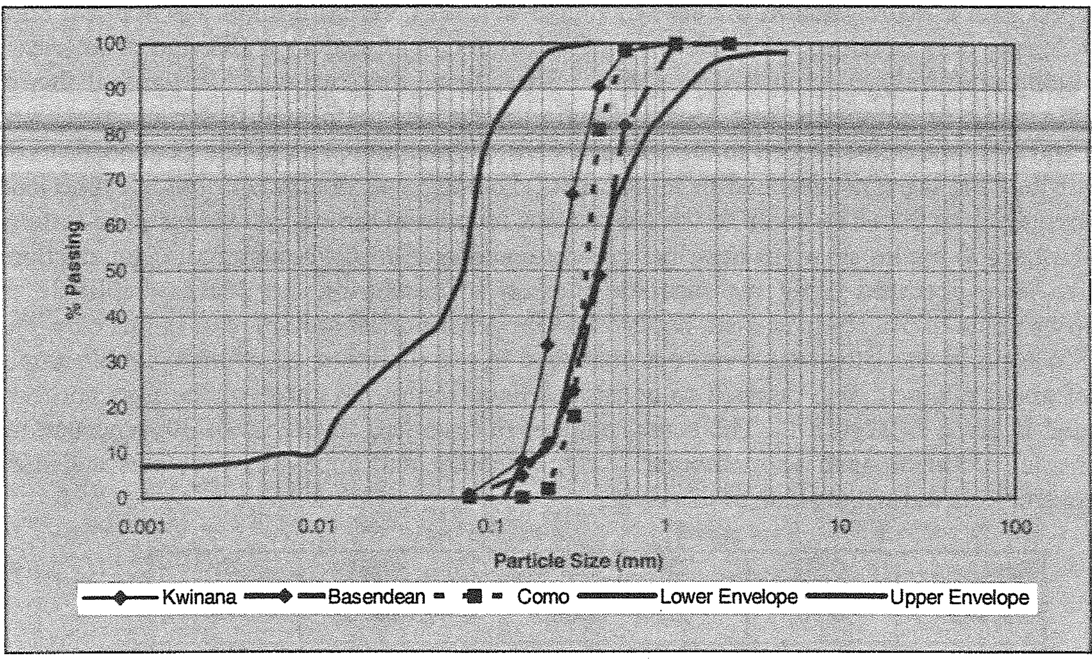

Figure 2 Particle Size Distribution results

3.1.2 Other Index Tests

Results of the remaining index tests are given in Figure 3. The majority of the tests were repeated three times, with error margins within acceptable limits, with the exception of the maximum density testing for Bassendean sand. This test was repeated by Golder Associates testing laboratory and the result they obtained was used.

	Kwinana	Bassendean	Como
Min Density (t/m^3)	1.03	1.58	1.53
Max Density (t/m^3)	1.34	1.88	1.72
CaCO3 content (%)	88.23	1.83	0.03
Sg	2.69	2.63	2.65

Figure 3 Results of Index Testing

3.2 Triaxial Testing

The results obtained from the cyclic triaxial tests were somewhat disappointing. Attempts were made to find equations for the correlations between CSR and number of cycles to failure (for different relative densities), cycles to failure and Vs (for differing CSR), cycles to failure and relative density (for different CSR) and relative density and Vs. Whilst trends were visible, and a few correlations were obtained, the reliability of the equations was doubtful, given the small amount of data for each trend. There were also issues with some data sets seeming to indicate trends opposite to what was expected, possibly due to outliers which may be from doubtful test results.

3.2.2 Shear Wave Velocity

The results of the Shear Wave velocity testing proved not to be very informative. There was very little variation in Vs values, even over a wide range of relative densities in each sand. As a result it seemed unfeasible to develop predictions for the CRR of the sands from the Vs readings. To demonstrate the ineffectiveness of the Vs data as a predictor, note that an accepted value for Vs_1* (the limiting upper value for the adjusted Vs for liquefaction occurrence) for sands with fines content <5% is 215 m/s (Youd and Idriss, 2001). All three sands studied should fit this category, however, many samples which did in fact fail in liquefaction recorded Vs values significantly higher than 215 m/s. We can also compare the calculated values of Cg for each sand. Cg is calculated using the formula;

$$C_g = \rho \, Vs^2 / F(e),$$

where

$$F(e) = (2.17 - e)/ (1 - e^2).$$

Cg should be a constant for any given sand, however it it clear in Figure 4 that there is considerable spread for each sand. This could be attributed to problems either in the Vs data or the density.

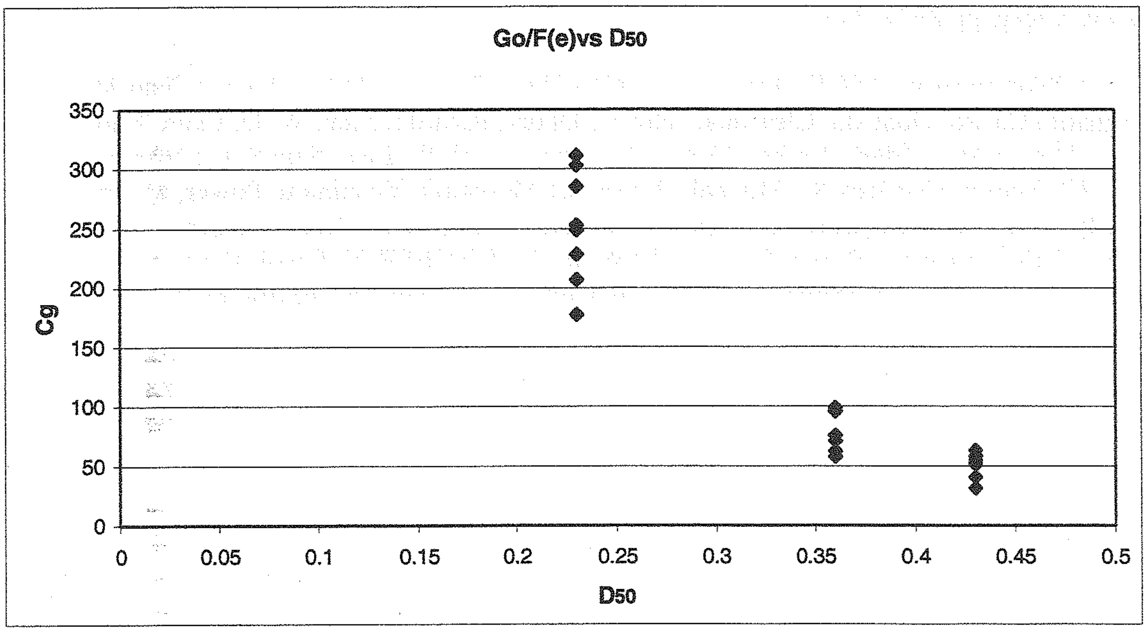

Figure 4 Cg values for Kwinana, Como and Bassendean Sand

4.0 Conclusions

The data obtained in this study was insufficient to meet its goal of developing more suitable predictions of liquefaction for the Perth region. In addition the sands tested did not perform as predicted by the state of practice methods, and as a result it can be assumed that engineers may not be able to optimise their foundation designs when working with these soils. It seems clear from these two points that there is a need for further work in this area. Further studies may particularly benefit from observing behaviour of a wider range of densities in their samples, which would hopefully give a better spread of shear wave velocities.

5.0 References

Andrus, R.D., and Stokoe, K. H. III, (1997) Liquefaction Resistance based on shear Wave Velocity. *Proc., NCEER Workshop on Evaluation of Liquefaction Resistance of Soils,* Nat. Ctr for Earthquake Engrg. Res., State Univ. Of New York at Buffalo, pp 89-128

Gaull, Brian. (September 2003) Seismic Characteristics of the Sediments of the Perth Basin, *Australian Geomechanics* **38** (3) – The Engineering Geology of Perth part 1. pp111 -122

Jamilowski, M., and Lo Presti, D. C. E. (1990). Correlation between liquefaction resistance and shear wave velocity, *Soils and Foundations,* Tokyo, **32**(2) pp 145-148.

Marcuson, W. F., III, (1978). Definition of terms related to liquefaction. *J. Geotech. Engrg. Div.*, ASCE, **104** (9), pp1197-1200.

Youd, T. L; Idriss, I. M., (April 2001) Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, *J. of Geotech. And Geoenv. Engrg,* pp 297 – 313.

Youd, T. L, (Chair, member ASCE); Idriss, I. M., (Co-Chair, Fellow, ASCE); Andrus, Ronald D., Arango, Ignatio; Castro, Gonzalo; Christian, John T.; Dobry, Ricardo; Finn, W. D. Liam; Harder, Leslie F. Jr.; Hynes, Mary Ellen; Ishihara, Kenji; Koester, Joseph P.; Liao, Sam S. C.; Marcuson, William F. III; Martin, Geoffrey R.; Mitchell, James K.; Moriwaki, Yoshiharu; Power, Maurice S.; Robertson, Peter K.; Seed, Raymond B.; Stokoe, Kenneth H. II. (October 2001) Liquefaction Resistance of Soils: Summary Report from the 1996 and 1998 NCEER/NSF workshops on Evaluation of Liquefaction Resistance of Soils, *J. of Geotech. and Geoenv. Engrg,* pp. 817.