PROBLEMS WITH LIQUEFACTION CRITERIA AND THEIR APPLICATION IN AUSTRALIA

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ABSTRACT

In many parts of the world, including Australia, the state of practice in assessing if liquefaction will occur is based on the recommendations of Youd *et al*. (2001) which arose from workshops convened in the United States by NCEER (now MCEER). In some regards, the final publication did not so much represent a consensus view as a compromise between differing opinions within the expert group. Since then, disagreements over key aspects of liquefaction assessment in North America have increased to the point of chaos (Youd, 2011). There is little awareness in Australia of this situation nor appreciation of the NCEER limitations in applying these recommendations. Poorly informed decisions are increasing costs and causing project delays. This paper presents no original research but is an attempt by a practising geotechnical engineer to point out some problematic aspect of the NCEER liquefaction criteria, and of current recommendations in the literature and in so doing to encourage other practitioners and regulators to consider reasonable adjustments or alternatives.

1 INTRODUCTION

In many parts of the world, including Australia, the state of practice in assessing if liquefaction will occur is based on recommendations given in a paper published in the United States by Youd *et al*. (2001). These recommendations initiated from a workshop convened by the US National Center for Earthquake Engineering Research or NCEER[1], which acronym commonly identifies the assessment methodology. The workshop was held in January 1996 and documented in a report in December of the following year authored by the chair and co-chair (Youd & Idriss, 1997). A second workshop was held in 1998 and publication of the recommendations in a forum open to discussion occurred three years later (Youd *et al., loc. cit*.). This activity was an initiative of Professor T.L.Youd of Brigham Young University who brought together 20 mainly research specialists in liquefaction, all but one from North America. While the NCEER recommendations are widely viewed as authoritative, their five year gestation period and two workshops reflected difficulty in arriving at agreement between the participants. The final publication in 2001 was in some regards a compromise, which can be discerned from comments in the literature (Seed *et al*., 2001). Since then, disagreements over key aspects of liquefaction assessment in North America have reached a fairly chaotic state (Youd, 2011). A new workshop process has been proposed to address the issues (O'Rourke, 2011). There appears to be little awareness in Australia of this disarray over liquefaction assessment. There is also a certain lack of appreciation of the NCEER limitations in applying these recommendations to Australian conditions. Poorly informed decisions are increasing costs and causing delays to large projects of significance to the national economy. This paper does not present original research but attempts to document some problematic aspect of the NCEER methodology and in so doing to encourage other practitioners and regulators to consider reasonable adjustments or alternatives.

2 BACKGROUND TO NCEER CRITERIA

2.1 ORIGINS

As a background to the problems it is perhaps appropriate to outline in broad terms the NCEER liquefaction assessment methodology and its origins. The approach is entirely empirical based on relating evidence of liquefaction to certain index parameters. These parameters have been measured *in situ* at liquefaction sites, supplemented in particular conditions by classification data. The methodology reflects early developments by the former Professor H.B Seed and his co-workers at the University of California at Berkeley (UC Berkeley). Professor Seed's primary collaborator at UC Berkeley was Professor I.M. Idriss (later UC Davis). Their research initiated from large earthquakes at Niigata, Japan and in Alaska in 1964 (Seed & Idriss, 1967). Ground failures in Alaska were slope instabilities and perhaps not surprising, while the Niigata failures were on essentially level ground which was quite unexpected. Professor Seed

[1] Now the Multidisciplinary Center for Earthquake Engineering Research, or MCEER.

travelled to Japan several times in the 1960s, where liquefaction was being intensively studied and instigated parallel development at UC Berkeley[2].

2.2 THE BERKELEY SCHOOL

The UC Berkeley focus for several years was the most susceptible type of natural soil, this being recently deposited, clean, uniformly graded sand of fluvial origin. Laboratory investigations were undertaken to understand the response of this sand to cyclic loading (Seed & Peacock, 1971). A "simplified procedure" was developed for characterising the shear stress (expressed as a cyclic stress ratio, or CSR, with the effective overburden stress) from a measurement or estimate of the peak ground acceleration at an earthquake shaken site. As in Japan (Koizumi, 1966), the sand state was initially characterised by relative density in order to link field and laboratory findings. For sites subject to earthquakes where incidences of liquefaction had occurred, a representative value of relative density (D_r) was selected based on the sand layer in the profile judged to be the most liquefiable. Seed & Peacock (1971) used results of laboratory testing to guide their interpretation of field liquefaction observations. They presented a plot of CSR versus relative density with data points representing sites with documented instances of liquefaction or no liquefaction and were able to draw a boundary curve for the critical CSR, termed the cyclic resistance ratio CRR, separating these conditions. Seed & Idriss (1971) formalised the "simplified procedure" using the boundary curve as a criterion for predicting if a site would experience liquefaction.

Seed (1979), citing Chinese practice in their earthquake code, later changed the site characteristic from relative density to the Standard Penetration Test N-value. This N-value was normalised (adjusted) to correspond to an effective overburden stress of one atmosphere in accordance with existing D_r - N relations. Seed showed that his clean sand curve was in close agreement with the independently derived relationship given in the Chinese earthquake code and the results of large scale shaking table tests in the UC Berkeley laboratory. With adjustments to the curve at extreme low and high N-values (where there were initially no field data), the Seed boundary curve developed from limited data later became the backbone of the NCEER recommendations. Numerous earthquake events since the boundary curve development have demonstrated that it is robust.

Subsequently, the UC Berkeley researchers acknowledged findings by workers in Japan and China indicating there was sand liquefaction behaviour which did not fit with their boundary curve (Seed *et al*., 1983). This new information came to the attention of the US researchers in two papers in 1981 at the first of several "recent advances" conference held at the University of Missouri, Rola. The electrical static CPT had been in use in China since the mid-1960s. Systematic investigations performed in 1977-78 following the 1976 Tangshan earthquake found quite different liquefaction criteria were necessary for clean, uniform sands and for silty sands (Zhou, 1981)[3]. The critical cone resistance to avoid liquefaction in soils with 60% fines was about half of the comparable value for clean sand. Tokimatsu & Yoshimi (1981, 1983) demonstrated a clear trend of increasing liquefaction resistance in finer grained granular soils based on data from the 1978 Miyagiken-oki earthquake which occurred on the east coast of Japan opposite Niigata. Their data may be characterised as indicating that silty sands with over 35% fines had critical CRRs about 85% greater than clean sands at a normalised N-value of 10. The findings in the Chinese and Japanese papers are broadly consistent. Seed responded in the 1983 paper (*loc.cit.*) by adding a less demanding boundary curve for silty sand to complement the existing clean sand curve in the CSR – N space. He also brought attention to Chinese experience contradicting the then-current view that more clayey soils are not vulnerable to liquefaction and introduced the "Chinese Criteria" of Wang (1979). These criteria for vulnerable clayey soils were based on classification parameters; namely, fines content, liquid limit and moisture content. Publications by Seed & Idriss (1982) and Seed *et al*. (1986) confirmed and refined this extension of the liquefaction assessment procedure. However, liquefaction criteria for soils other than clean sand remain in dispute to this day, which is unfortunate as assessment of such soils is often a project requirement.

2.3 IN SITU TESTING

Youd *et al.* (2001) placed primary emphasis on the SPT, which reflected historical practice, but also provided criteria for use with CPT and shear wave velocity (V_s) measurements. Today, the CPT dominates liquefaction practice. The NCEER workshop endorsed, with reservations, the procedure proposed by Robertson & Wride (1998) and this has subsequently become much used by the profession. Professor Robertson (then at the University of Alberta, Edmonton) has continually updated his procedures and the current version is found in Robertson (2012). Youd & Idriss (1997) noted that the NCEER workshop was unable to reach a consensus on CPT criteria. Professor Idriss took the view that the Robertson & Wride criteria were inadequately developed and that their soil behaviour type index I_c needed further verification. The workshop considered that CPT testing should be accompanied by soil sampling for validation of soil

[2] The 1968 Meckering earthquake had a corresponding effect in alerting engineers to earthquake hazards in Australia.

[3] Several Tangshan case history sites were recently reinvestigated to improve the data quality (Moss *et al*., 2011).

type. Criticism from the University of California (UC Berkeley and UC Davis) of Professor Robertson's method of recognising the effect on liquefaction potential of soil type and, by implication grain size, has since continued in the literature (Moss *et al.,* 2006; Idriss & Boulanger, 2008). There was also debate over the procedure for normalising (adjusting) the cone resistance to the one atmosphere standard, based on an alternate approach in a paper submitted to the NCEER workshop by R.S. Olson of the US Army Engineer Waterways Experimental Station (Olson, 1997). This debate has also continued in the literature (Moss, Seed & Olsen, 2006; Robertson, 2009a).

The NCEER workshop also endorsed use of shear wave propagation velocity based on Andrus & Stokoe (2000) but indicated preference for penetration testing considering the available databases and experience. Measuring V_s *in situ* was indicated as "fair" for detection of variability of soil deposits, whereas the SPT and CPT were rated as good or very good. A concern was that liquefaction involves larger strains than are associated with very low strain shear wave measurement. Shear wave velocity assessment of liquefaction potential was arguably the least advanced of these three methods (CPT, SPT, V_s) at the time of the NCEER workshops. As discussed later, there have been significant advances in V_s assessment since 2001 (e.g. Kayen *et al.,* 2013). Nonetheless, advocates of penetration-based liquefaction assessment in California continued to argue that V_s is not a good indicator of liquefaction resistance (Idriss & Boulanger, 2008).

2.4 ADDITIONAL CONSIDERATIONS

This paper primarily focuses on the effect of material type and origin on the triggering of liquefaction in level ground. There are several other key factors such as the effect of depth or confining stress of the potentially liquefiable material, the presence of static shear stress due to sloping ground or superimposed loading and post-liquefaction residual strength and ground movements. Currently, most are in dispute and contribute to the disarray indicated by Youd (2011).

The outline review above highlights the early leadership of Japanese and Chinese researchers in identifying and quantifying the influence of soil texture and type on liquefaction potential. Former Professor H.B. Seed, his presently active colleagues and their co-workers in California at UC Berkeley and UC Davis have unquestionably made a large contribution to the subject. Their lead has had a dominant influence in the United States where earthquake engineering and liquefaction is currently the largest single area of university research in the geotechnical field (Mitchell, 2012). However, major contributions, in addition to the ongoing work in Japan and China, have come from Professor Robertson, formerly at the University of Alberta, Edmonton in regard to the CPT and also from the area of critical state soil mechanics (CSSM) and constitutive modelling; for example Jeffries & Been (2006). In addition, laboratory based researchers provide a separate stream of information which is sometimes helpful. Unfortunately, there is a serious lack of agreement between these sources on liquefaction fundamentals and recommended assessment procedures. Most notably, UC Berkeley and UC Davis are in conflict (Youd, 2011), while both are highly critical of Robertson's procedures, viewing these as unconservative. In turn, Jeffries & Been (*loc.cit.*) indicate deficiencies in the empirical approach emanating from California and propose entirely different controlling variables for the liquefaction process. Also, the California penetration-based assessment schools criticise V_s as a poor indicator. Currently, a reasonable view of this field is that the application of liquefaction science is far from settled and that the approach to assessing materials other than clean uniform sand (formulated in the 1970s and 1980s) is in considerable disarray[4].

A detailed and well-informed discussion of the NCEER recommendations by Pyke (2003) identified several deficiencies. A major concern over the effect that the NCEER recommendations would have on liquefaction assessment practice was the inadequate qualifiers on the types of soil to which the recommendations could be said to apply. Aspects of Pyke's contribution are included in the discussion below, however Pyke's commentary is essential reading for those involved with liquefaction assessment. A response (published together with Pyke's discussion) attributed to the workshop participants generally agreed with Pyke's comments but nonetheless maintained the utility of the NCEER recommendations. A pivotal assertion in this regard is that application of the NCEER recommendations is more reliable than use of geological criteria. This assertion contradicted earlier recommendations of Kramer (1996), which are discussed later and also appears to negate restrictions on the site conditions to which the recommendations apply that were stated in the 2001 paper. These restrictions have largely been lost in geotechnical practice and the NCEER criteria are often viewed as definitive without qualification.

[4] Cetin *et al.* (2004) proposed a significant change to the SPT clean sand curve based on a revision of the liquefaction database at UC Berkeley. Assessment of their findings by Idriss & Boulanger (2012) identified a number of issues in the database revision and concluded that Professor H.B. Seed's SPT clean sand curve, adopted by NCEER, remains valid.

3 EFFECT OF FINES

There is conflicting information in the literature on the effect of fines content on liquefaction susceptibility. Broadly, the information may be categorised as field and laboratory based.

3.1 ORIGINAL FIELD DATABASE

The field information begins with the Chinese and Japanese investigations in the late 1970s, and Seed's formulation in the early 1980s discussed above which is essentially the 2001 NCEER chart shown in Figure 1. Seed formulated the SPT-based boundary curves for clean sand and for sands containing fines beneath level ground using a database of 125 case records from about a dozen earthquake events.

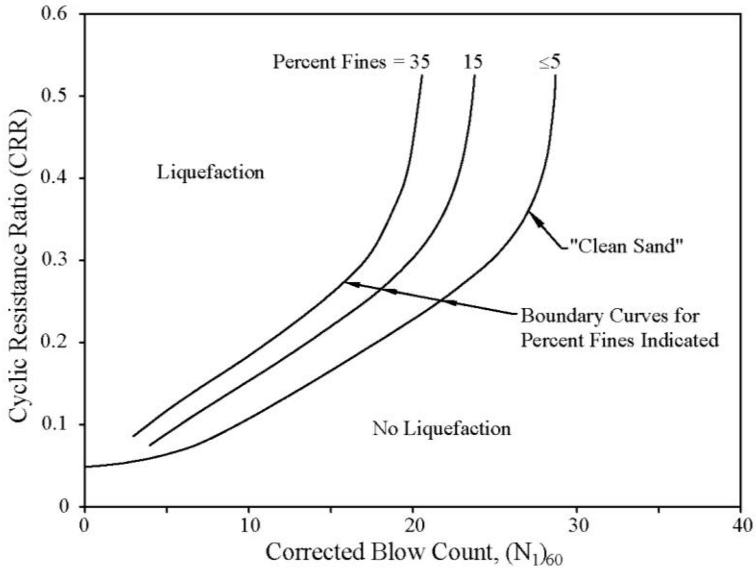

Figure 1: NCEER cyclic resistance ratio for magnitude 7.5 earthquakes ($CRR_{7.5}$) from SPT N - values (after Youd et al, 2001)

Fear & McRoberts (1995) published an independent review of this UC Berkeley catalogue and their readily available paper is recommended reading for those involved with liquefaction assessment. The review does not inspire confidence in the reliability of field based liquefaction assessment. As discussed later, this comment applies also to the current field liquefaction database. Clearly it is not difficult to retrospectively find fault with early studies in a new science and such criticism is not to belittle such pioneering endeavour. Equally, however, it is important to be aware of the limitations of recommendations that the profession is relying upon. Fear & McRoberts found a number of problems in the UC Berkeley catalogue including about one-quarter having no borehole log for SPT N-values and half the cases had the influence of sloping ground, a free surface, an embankment or a structure. This information had been used to develop recommendations for free-field level ground conditions which have no influence of driving shear stresses in the soil. In addition, Fear & McRoberts found no consistency in the selection of the characteristic N-value with these being, on a case by case basis, either the single lower or upper bound values or the average value. They note that a single low or high N-value can be a testing error. They found support in the database for a general observation that sands with fines are more resistant to liquefaction. However, their review did not find support for further discrimination based on the percentage of fines; that is, for discerning the difference in liquefaction resistance based on fines content indicated by the curves on Figure 1. Fear & McRoberts found a conservative bias in the original data interpretation and in their conclusions stated:

> *The original work has unfortunately evolved into a design method, which, if coupled with lower-bound site data, may well be overly conservative and either embody a poor use of resource allocation for risk reduction or unnecessary costs.*

While the foregoing relates to SPT N-values, the issue of averaging or lower bound characterisation in site characterisation also arises with the CPT, arguably to a greater extent given the fine detail provided by continuous profiling. This consideration is relevant to more recent development of liquefaction criteria from the current database.

3.2 CURRENT FIELD DATABASE

Since the original Seed work, a number of databases have been compiled based on CPT site characterisation of historical and more recent liquefaction events. There is a CRR – q_c chart corresponding to the original CRR – N chart in which q_c is the cone tip resistance. The newer databases have been used by their developers and others, to formulate liquefaction criteria in this form; notably by Shibata & Teparaksa (1988), Suzuki *et al*. (1995), Stark & Olson (1995), Olsen (1997) and by Robertson & Wride (1998), with the latter being the widely used NCEER procedure. The most recent and extensive of these data compilations based on the CPT is again now found at UC Berkeley and it can be downloaded using the link cited in the Moss (2003) reference at the end of this text. A considerable effort has clearly been made in assembling this catalogue which is well organised with a data sheet and profile information for each of the 185 case records. A classification of the data quality for each case record is provided.

Assessment criteria derived from the 2003 UC Berkeley catalogue have been presented in Seed[5] *et al*. (2003) and Moss *et al*. (2006). These papers from UC Berkeley essentially validate the existing NCEER (i.e. Robertson & Wride) boundary curve for "clean sand". There are however significant differences in their recommended boundary curves for soil containing fines as indicated by Figure 2 replotted from the UC Berkeley publications. The UC Berkeley criteria for these soils are based on the CPT friction ratio as derived from the database by regression analysis.

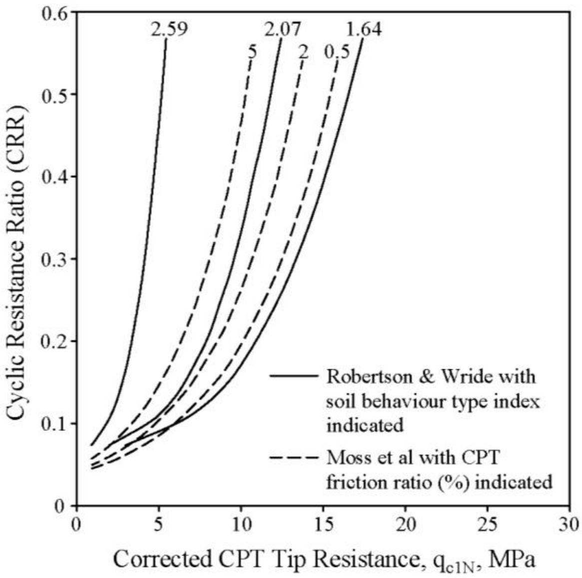

Figure 2: Comparison of Robertson & Wride (1998) and Moss et al (2006) liquefaction boundary curves for granular soils

A general comparison can be made with the Robertson & Wride (1998) recommendations as in Figure 2 as the variations in soil behaviour type index I_c and CPT friction ratio on Figure 2 nominally represent the same range of soils. A specific comparison can be made by considering a silty sand/sandy silt with a cone tip resistance of 10 MPa and a friction ratio of 0.5% at a depth corresponding to an effective overburden stress of 100 kPa. These parameters give an I_c close to 2.59 and it is apparent that q_{c1N} = 10 MPa is to the right of the relevant boundary curve and the soil is non-liquefiable by the Robertson & Wride (1998) criteria. In contrast, these cone resistance and friction ratio parameters indicate the soil is liquefiable at a cyclic stress ratio greater than about 0.2 by the Moss (2003) criteria.

The publications from UC Berkeley cited above criticise the Robertson & Wride criteria as unconservative on the basis of Figure 2. In turn, Robertson (2009b) has taken issue with the UC Berkeley CPT interpretation on two grounds. One is that their data includes the mechanical cone and cases where there is no friction sleeve data and values were inferred. Also, the profile interpretation in the Moss database can be challenged. In this regard, review of the current UC Berkeley catalogue (recommended for those involved in liquefaction assessment) indicates the inherent uncertainty in identifying and characterising the most susceptible zone in many of the case record profiles.

The UC Berkeley approach is to identify the most susceptible or critical zone in the profile and to adopt an average for this zone. One example CPT is given in Figure 3 in which the profile has been replotted for legibility. The q_c profile for this CPT varies from about 1.5 MPa to 15 MPa in the designated critical zone. A characteristic q_c of about 5 MPa was used for this case record in the catalogue. Clearly, there is scope for other interpretations of the critical zone and characteristic resistance associated with liquefaction in this profile. What is important is that the way in which a criterion of liquefaction is derived from the database information is also the way in which it is applied to assessment of liquefaction at project sites. Currently there is no standard method and there is considerable scope for bias being

5 R.B. Seed followed his father H.B. Seed as Professor of Civil Engineering at Berkeley with a specialization in earthquake engineering and liquefaction.

introduced in practical applications. If, for example, criteria developed from averaging of CPT data are applied on a point by point basis to a profile then liquefaction could well be incorrectly predicted. One method of reducing the potential for differences in subjective interpretation of fluctuating CPT profiles is to smooth the data by taking averages over an interval. Seed & de Alba (1986) used a 1-m interval in assessing the performance of the Heber Road site in the 1979 Imperial Valley earthquake while Boulanger *et al*. (1997) used a 0.6-m interval for the Moss Landing site in the 1989 Loma Prieta event.

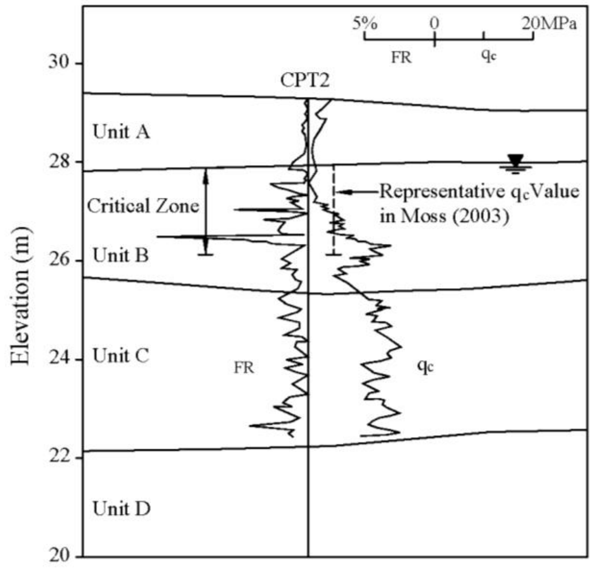

Figure 3: CPT interpretation (Moss, 2003) for Pence Ranch site, 1983 Borah Peak earthquake case record

Like its 1980s predecessor, the current UC Berkeley database includes case records with sloping ground and free face conditions. For one case record (Borah Peak, 1983) having a 7% slope, the static shear stress ratio (τ/σ_v') can be computed as 0.05 which is not insignificant when CRRs in the order of 0.2 apply to weak granular soils. Cases with sloping ground feature surface cracking and lateral spread in contrast to the sand boils that characterise liquefaction in level ground. Youd *et al*. (2009) have noted that a 0.3% slope is sufficient to induce lateral spread which is a stability failure mode. The Wildlife Park site in Southern California is perhaps the best documented, instrumented site in the database and it has experienced ground failure in multiple earthquake events. Curiously, this site has a free face condition which presumably should be a consideration in data interpretation. The influence of sloping ground/free face conditions on liquefaction potential needs to be dealt with in practice but this is a different situation requiring a different approach (Olson & Stark, 2003). To include this in a database that is not considered to address such conditions contaminates and biases criteria stated to be for level ground.

3.3 NATURE OF FINES

An example of the conflicting information on the effect of fines is found in Idriss & Boulanger (2008) and the CRR- q_c diagram in Figure 4. They compare the Robertson & Wride (1998) criteria for borderline silty sand to sandy silt with the cases in the Moss (2003) database that have the largest fines contents. Idriss & Boulanger indicate that the Robertson & Wride boundary curve is unconservative as it represents a fines content of 35% whereas the data points are for soils that are silts by grain size. This apparent conflict illustrates the confusing information in the literature. In this case, the linkage to fines content is nominal as the primary index to which the curve on Figure 4 relates is I_c, the indicator for soil behaviour type. Using combinations of cone tip and sleeve resistance to indicate soil type has been employed since the inception of the CPT. Capturing this in an index and using the index in liquefaction assessment has progressively replaced fines content in soil classification/liquefaction literature (Jefferies & Davies, 1993; Suzuki *et al*., 1997; Mayne, 2007). As discussed further below, fines content is but one aspect of soil type, with the latter being a major factor in determining liquefaction resistance.

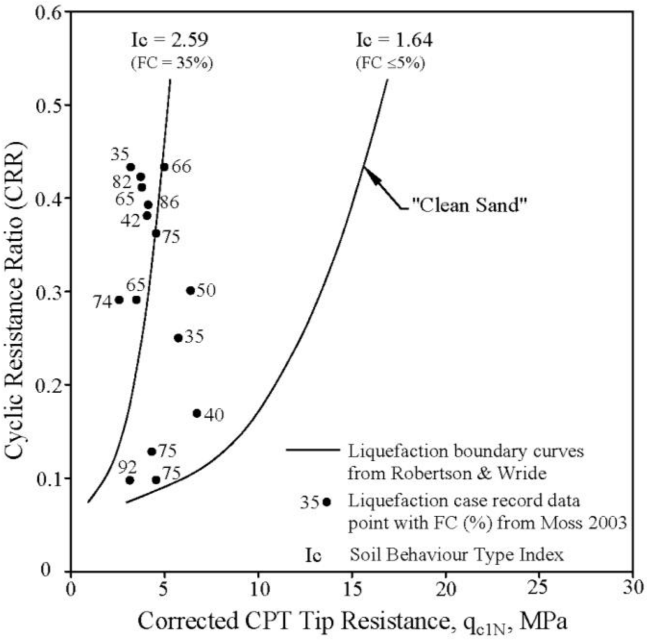

Figure 4: Comparison of case records of silty soils that liquefied with Robertson & Wride (1998) boundary curves (after Idriss and Boulanger, 2008)

3.3.1 Fines Content

For historical SPT based assessment, fines content was a convenient and natural way of differentiating different soil textures and their liquefaction susceptibility. Measuring fines content featured strongly in the NCEER recommendation of Youd *et al.* (2001). More recently, Idriss & Boulanger (2008) have recommended that the CPT be used to profile the site and identify locations for SPT or tube sampling. These authors emphasise the poor correlation between the soil behaviour type index I_c and fines content, implying that the latter is a truer measure of textural effects. Suzuki *et al*. (1995) among others appear to find good correlation between fines content and I_c but the considerable scatter in their supporting data is masked by use of double logarithmic scales.

The reality is that the effects of soil texture on liquefaction potential do not depend only on fines content. As noted by Andrews & Martin (2000), silt size particles can be viewed as very fine sand. The grain size boundary between sand and silt is commonly taken at 74 microns which is simply the smallest size that can be seen. There is no necessity for the liquefaction behaviour to change at this grain size. At the time when classification systems were being developed, Glossop & Skempton (1945) pointed out that key behavioural aspects of natural, uniformly graded soils changed in the region of 50 microns to 60 microns. Behavioural criteria included the effectiveness of gravity drainage, post construction settlement, frost heave and static liquefaction. Based on their own experience and referencing that of Karl Terzaghi, they placed the lower limit for sand-type behaviour in the coarse silt range. The lower limit corresponded to 80 to 85% finer than 74 microns with up to about 20% of the soil grading to medium silt (< 20 microns). Ishihara (1985) reported that Tsuchida (1970) placed the lower limit for potentially liquefiable soil in the medium to coarse silt range. As noted earlier, a review of the original 1980s UC Berkeley database by Fear & McRoberts (1995) did not find support for Seed's discrimination of liquefaction resistance based on the percentage of fines. It appears that fines content as a controlling variable should be viewed as an historical expedient that no longer stands scrutiny.

3.3.2 Plasticity

The significance of fines plasticity has been recognised from the outset in the 'Chinese criteria' discussed earlier. Quantifying the effect of plasticity (colloidal effects) is currently a major research focus and is quite controversial. A considerable part of the disagreement between UC Berkeley and UC Davis, leading to the chaos described by Youd (2011), relates to this issue. The situation is exacerbated by different streams of information, including laboratory investigations, each well substantiated and indicating contrasting effects. The literature on this topic is extensive. An early contribution from Tokyo University showed the laboratory measured CRR of sands containing fines was essentially unaffected by their plasticity up to PI = 10, but increased proportionally with PI thereafter (Ishihara, 1993). More recent work at Kyoto University indicated that laboratory liquefaction resistance initially reduced with fines plasticity, being a minimum at PI ≈ 4, then increased as PI increased (Gratchev *et al*., 2006). Carraro *et al*. (2003) and Park & Kim (2012) provide useful commentary on research findings from laboratory testing and note that the conflicting trends reported in the literature are in part due to different criteria being used to define comparable soil conditions. These can be global void ratio, intergranular void ratio or relative density, the latter being difficult to determine and therefore unreliable for soil other than clean sand.

Silt blends tested in triaxial compression at UC Davis by Romero (1995), reported by Boulanger & Idriss (2006), exhibited characteristics of sands at PI = 0 and of clays at PI ≥ 4. The characteristics were dilatancy and non-parallel virgin consolidation and critical state lines in e – log p' space. These results support the view of Boulanger & Idriss (2006, 2007) that there is a rapid transition from sand-like to clay-like behaviour within a range of relatively small PI values. Cyclic softening of soils exhibiting clay-like behaviour is assessed by considering undrained shear strength. Detailed field and laboratory investigations of fine grained soils which liquefied in the 1999 Kocaeli earthquake were undertaken by researchers at UC Berkeley (Bray & Sancio, 2006). These studies indicated that fine grained soils with moisture contents close to the Liquid Limit are susceptible to liquefaction especially if they are of low plasticity. PI was found to be an indicator for liquefaction potential rather than a criterion. Subsequently, UC Berkeley issued a report prepared by Seed (2010) criticising the Idriss-Boulanger recommendations as unconservative and a hazard to public safety. This uncertainty over procedures has not been resolved (e.g. Liao *et al*., 2010).

Zhu & Law (1988), Koester (1992, 1999) and Prakash & Puri (2003) have provided useful commentary on past findings on the effect of fines and plasticity on liquefaction susceptibility including assessment by field methods. Fines reduce penetration resistance and liquefaction resistance but not to the same extent, there being a disproportionately greater effect on liquefaction. Consequently, fines increase liquefaction resistance for a given penetration resistance. Apparently conflicting findings on the extent of the benefit are explained when the plasticity of the fines is taken into account. The NCEER CPT based criteria, based on Robertson & Wride (1998), indicated that materials with the soil behaviour type index $I_c > 2.6$ are most likely too clay rich or plastic to liquefy. Soils with $I_c > 2.4$ should be sampled and evaluated using the Chinese criteria. Subsequently, Robertson (2009b) revised his procedure and recommended different CPT evaluation criteria for soils with $I_c > 2.6$ (and curve fitted transitional criteria between I_c values of 2.4 and 2.6). Soft soils with $I_c > 2.6$ respond to cone penetration in an undrained manner and generate positive excess pore pressures. Robertson's revised evaluation criteria for plastic soils are based on undrained shear strength following Boulanger & Idriss (2006, 2007).

4 LABORATORY TESTING

This discussion has focused on appraisal of natural soils by field testing, in some situations supplemented by laboratory classification test data. Assessment of liquefaction by laboratory strength testing was initially adopted at UC Berkeley (Seed & Lee, 1966) but this later changed to reliance on field based criteria. This change in methodology followed critical appraisal of cyclic load testing and its results by Castro (1969, 1975) at Harvard University. Based on this work, Casagrande (1976) concluded that liquefaction assessment by laboratory testing was unrealistic due to non-uniformities induced by equipment boundary effects. Casagrande stated:

> *I cannot find a common denominator between the principal mechanisms that control the cyclic response of laboratory specimens and the response of an element in situ.*

The field of liquefaction science has since grown enormously since Casagrade's comment and has become increasingly specialised. Progressively more information on liquefaction is from researchers focusing on progressively narrower aspects of soil behaviour using the limited resources at their disposal. In addition to computers, the resources available in universities are often a laboratory and commercially available soils. Laboratory research findings are prominent in the liquefaction literature today and these encompass the range from very useful to highly questionable. Specific instances are not discussed here and it is certainly the case that excellent work has been done by knowledgeable investigators. However, it is advisable to have a good appreciation of the empirical evidence in regard to liquefaction in earthquake events before relying on laboratory findings. The conflicting findings from different studies in regard to the effects of fines of varying plasticity has already been mentioned. It is easy to gain an incorrect understanding of liquefaction behaviour from limited reading on narrowly based laboratory studies. In addition to Casagrande's findings, geologically controlled characteristics of natural soil deposits, such as soil fabric and age, are missing in laboratory processed soils.

There is a penalty in fragmentation of liquefaction research. It is disconcerting to see, as in Shuttle & Cunning (2008), the importance of geological knowledge being dismissed by researchers whose focus is the mechanics based approach to liquefaction assessment. Equally disconcerting is the presentation as "liquefaction criteria" of laboratory test results on mixtures of commercial sand and ground silica coupled with computer generated cone penetration testing as in Carraro *et al*., 2003. This is particularly so when the results contradict field evidence of soil behaviour. In appraising findings in this fragmented field it is important to keep in mind the factors that are likely to influence the behaviour of natural soils. As stated by Peck (1979) in reviewing liquefaction science:

> *In soil mechanics, no evidence can be considered reasonably adequate until there is sufficient field experience to determine whether the phenomena observed in the laboratory are indeed the same as those that operate in the field. It must also be determined whether predictions based on laboratory studies and theories are indeed fulfilled in the field.*

(Underlining added by the present writer.)

5 GEOLOGICAL CONSIDERATIONS

5.1 CONTROLLING FACTORS

Unlike artificial soils created in a laboratory, natural soils are subject to geologic processes which govern and constrain their characteristics. Accordingly, there are associations between characteristics which underlie empirical assessment methods. The utility of fines content (proportion finer than 74 microns) as an index to liquefaction susceptibility is the prime example. The potential limitation of such empirical criteria is that the link between the index and the behaviour may not manifest in the same way in different geological settings. Zhu & Law (1988) among others have noted that the Seed *et al.* (1983) fines corrections were developed from alluvial soils containing clay minerals. These fines corrections clearly do not apply equally well to silty sands, sandy silts and silts (termed transitional soils) that do not have plastic fines, as illustrated by Figure 4. The significance of plasticity (colloidal activity) was first recognised in China as reflected in the 'Chinese criteria' of Wang (1979). Koester (1992), referencing Chang (1987), indicates that the peneplain and subdued coastal regions of China produce sands containing more silt and clay than are present in the predominantly uniform sands emanating from the more rugged geomorphology of Japan. Kramer (1996) notes that well graded natural sands are generally less susceptible to liquefaction than uniformly graded sands and this is reflected in the overwhelming representation of uniformly graded sands in the liquefaction database. Clearly, natural soils having the same fines content can have quite different colloidal fractions, which impart characteristics that affect grain movement and liquefaction.

Consideration of the geological setting was urged by Pyke (2003) in commenting on the NCEER criteria. Pyke noted that the NCEER procedures are a cookbook approach formulated from experience of soils that have liquefied in earthquake events. Pyke emphasised that these soils are consistently young, uniformly graded clean sands and questioned the relevance of the NCEER recommendations to other soil types. In one respect is an odd thing that the NCEER recommendations can be so readily criticised for not emphasising geological control factors. The recommendations were formulated by a working group chaired by Professor T.L. Youd whose own research has done much to emphasise the link between liquefaction susceptibility and geologic setting (Youd & Hoose, 1977; Youd & Perkins, 1978). The susceptibility of sediments ranges from very high for post-Pleistocene river channel and delta deposits formed in the last few hundred years to very low for all pre-Pleistocene soils. In the textbook 'Geotechnical Earthquake Engineering', Kramer (1996) concludes the review of liquefaction assessment by stating:

> *Liquefaction susceptibility can be judged on the basis of historical, geologic, compositional, and state considerations. Geologic, compositional and state criteria must be met for the soil to be susceptible to liquefaction; if any of these criteria are not met, the soil is nonsusceptible to liquefaction.*

This is a widely informed and reasonable engineering approach that, to the writer's knowledge, is not often followed in Australia where adherence to the NCEER "one size fits all" method is commonly practiced.

Sedimentary deposits are never homogeneous. Stratification including fine intercalations can have a major effect on field liquefaction behaviour as noted by Dobry (1995), Kokusho (2003) and Kulasingam *et al*. (2004) among others. Casagrande (1976) observed that destructive movements at ground level may be caused by liquefaction of weak zones, while the behaviour is attributed to the surrounding stronger soil thereby distorting our understanding. This problem has been discussed above in connection with the UC Berkeley characterisation of liquefaction case records in Moss (2003). The presence of geological inhomogeneity is a reality which can only reduce the reliability of and confidence in our understanding of earthquake induced liquefaction. Engineers largely focus on what can be computed and tend to ignore factors which cannot readily be quantified, an example of which was given earlier. This weakness in approach can be mitigated by paying more, not less attention to geological factors.

5.2 FABRIC

The significant effect of fabric is well illustrated by the difference in liquefaction resistance of laboratory specimens at the same relative density when undisturbed and reconstituted or when reconstituted using different methods. This effect has been an issue in laboratory testing from the outset (Ladd, 1977). While fabric is a controlling factor in uniformly graded sands, its potential to affect liquefaction resistance increases with the addition of finer particles. The arrangement of particle contacts is a response to depositional factors and the subsequent stress regime as contacts adjust to carry load. Resistance to liquefaction is governed by the frequency of particle contacts and their robustness in response to the cyclic rotation of principal stresses. Santamarina (2001) and Mitchell & Soga (2005) and provide comprehensive discussions of fabric from the geotechnical engineering perspective.

Jeffries & Been (2006) have noted that fabric is of equal importance to the soil state (density and confining stress) in governing liquefaction resistance, which constitutes a significant limitation on the use of constitutive models in liquefaction assessment. While fabric has a critical influence it is also the most difficult property to assess and quantify in a manner that is feasible in engineering practice. Research on this daunting problem is ongoing but, considering the vast and in parts questionable literature on liquefaction being produced, it can be argued that a greater focus on fabric would be appropriate. A more robust fabric increasing the resistance of the soil structure to cyclic load. The small strain stiffness as measured by shear wave velocity would seem to hold promise as at least an indicator of this aspect of liquefaction resistance (Andrus *et al.*, 2004; Roy, 2008). Additionally, the effect of fabric is intrinsically captured in liquefaction assessment by considering induced strain rather than shear stress as argued by the proponents of cyclic strain theory (Dobry *et al*., 1982; Schneider & Moss, 2011; Dobry, 2012). Development of liquefaction is a medium to large strain process which is justification for relating it to large strain penetration resistance. However, the process cannot initiate without first overcoming the interparticle structure of the soil at small strain.

Mechanical changes to fabric occur over time and, in the absence of chemical effects, are the reason liquefaction resistance increases with the age of the soil. The mechanical adjustment of particle contacts with time can be seen in SEM images that show the way interparticle stress modifies the contacts (Michalowski & Nadukuru, 2012). The change in granular soil properties with time is well recognised in general geotechnical practice as increased stiffness and strength (Dramola, 1980; Mesri *et al*., 1990; Schmertmann, 1991; Mitchell & Soga, 2005). If the changes that occur over time are accepted in general there is good reason to consider these in liquefaction assessment.

As mentioned earlier, Boulanger & Idriss (2006, 2007) found a rapid transition in cyclic load behaviour over a small range of PI values. Behaviour changed from sand-like at PI = 3 to clay-like at PI = 8. At first glance it might be thought that the reason for the soil plasticity affect on liquefaction resistance is that the colloidal particles impart a cohesive component of strength to reinforce the soil skeleton. However, true cohesion due to colloidal forces is practically negligible (Mitchell & Soga, 2005; Schofield, 2005). A tentative explanation might be that plasticity infuences the formational soil fabric. PI can be a good index to specific surface area (Locat *et al.*, 1984) which empowers the particle contact level forces that affect depositional fabric (Santamarina, 2001).

5.3 AGE AND OVERCONSOLIDATION

Seed (1979) indicated that the factors which significantly affect liquefaction resistance are: (1) relative density, (2) fabric, (3) time, (4) K_o and (5) strain history. Today, soil density is considered in the context of confining stress, such that soil state is preferable to relative density. Strain history includes cyclic pre-shearing but in general the last two items are probably linked, with K_o being the dependent variable and both may be related to an overconsolidation effect. As indicated above, the arrangement of particle contacts, or fabric, changes with the passage of time due to micro-mechanical or chemical effects.

Seed (1979) reported the testing of identical reconstituted sand specimens at different times. The results indicated cyclic load resistance increased with time, this reaching 25% at 100 days. He extended the time range by testing undisturbed specimens of sands with known ages up to 10^6 days (≈ 3000 years) and found the liquefaction resistance increased by up to 75%.

Ishihara *et al.* (1978) reported on broadly similar testing on alluvial silty sands and sandy silts from three locations near Tokyo. They showed that modest overconsolidation ($1 \leq OCR \leq 2$) increased cyclic resistance and that this effect became more pronounced as the fines content increased. The increase in resistance from OCR = 1 to 2 was 40% for zero fines and 70% for 100% fines. In Vancouver, Campanella & Lim (1981) tested natural sandy and clayey silts and also found liquefaction resistance increased markedly with aging and overconsolidation. Kokusho *et al.* (2012) report on testing with a triaxial apparatus that incorporated a miniature cone to investigate more directly how cyclic load resistance varies with cone resistance. Specimens were lightly cemented with the purpose of simulating geological aging. They found that the effect of fines content in increasing liquefaction resistance at the same cone resistance increased with the degree of their simulated aging effect. That is, the liquefaction resistance of young soils was not as affected (improved) by a fines content as aged soils, both having the same penetration resistance.

Tronsco *et al*. (1988) performed cyclic load testing on undisturbed samples of tailings from the El Cobre mine site in Chile where the depositional history over 30 years is known. These silty sands had varying degrees of cementation resulting in the cyclic resistance increasing by 100% at one year and up to 250% at 30 years compared with freshly reconstituted specimens. Tailings are artificial soils and the El Cobre results are not directly relevant to natural soils. They do, however, indicate the effect that bonding of soil grain contacts has on liquefaction susceptibility. Bond development over time, albeit at a slower rate, is a natural consequence of aging and diagenetic effects in soils that lead ultimately to the formation of sedimentary rocks (Terzaghi *et al.*, 1996).

In the 1964 Niigata earthquake in Japan which initiated the current liquefaction assessment method, the soils that liquefied were alluvial sands and hydraulic fills placed after the late 19^{th} century. Much older deposits did not liquefy (Terzaghi *et al.*, 1996). In the 1976 Tangshan, PRC earthquakes, the effects of liquefaction were observed in a 20,000 km^2 area around the Ruan river southeast of Tangshan City. The most pervasive liquefaction occurred in the young alluvial deposits and became progressively less towards the older deltaic soils (Koester, 1999).

The case records used to develop the NCEER shear wave velocity based liquefaction criteria (Andrus & Stokoe, 2000) were for sands less than 3,000 years old. Subsequent studies have expanded the database and included case histories for sands up to several million years old (Hayati & Andrus, 2009). The results indicate that the liquefaction resistance, CRR, increases at just over 10% per log cycle of time. The database and these findings are noted to apply to sands having fines ≤ 35%. Baxter *et al*. (2008) postulate there may be a different relationship for silts based on laboratory test comparisons.

Lewis *et al*. (1999) report the results of detailed investigations, including those of Martin & Clough (1994) among others, of over 60 sites on the Charleston peninsula, South Carolina where liquefaction resulted from a large earthquake in 1886. Motivation and support for this substantial effort arose from the presence of nuclear reprocessing facilities also on the South Carolina Coastal Plain (SCCP) at Savannah River, Georgia which are discussed below. There was abundant relict evidence around Charleston of the extent of liquefaction in clean sands originating as beach features, and also evidence of a lack of liquefaction in similar but older sands. Data from 33 sites ranging in age from 85,000 to over 200,000 years indicated liquefaction resistances on average 1.5 to 2.5 times greater than obtained using the UC Berkeley CRR – N boundary curve (Seed *et al*., 1983) which essentially is the current NCEER recommendation.

Arango *et al*. (2000) describe studies undertaken to assess the need for foundation retrofitting of nuclear reprocessing facilities at the Savannah River Project. These facilities were supported by silty and clayey sands (fines content ≈ 10-20%) of Miocene age below the water table having SPT N-values varying between 3 and 15 with numerous values being below 5. Assessment based on the Seed *et al.* (1983) chart indicated a CRR of about one-half the design CSR. Concerns raised by the Regulatory Agency lead to careful sampling with efforts to ensure quality described as "enforced to the extreme". Cyclic load testing at UC Berkeley indicated CRR values 10% to 100% greater than the design CSR. Arango *et al*. (*loc. cit*.) combined these results with the Seed (1979) and the Lewis *et al.* (1999) information described above, plus performance data of a one million year old sand deposit in the 1994 Northridge earthquake, to define a relationship between liquefaction strength gain and time. This is shown as the upper curve on Figure 5. They report that the Regulatory Board permitted the ongoing operation of the facilities without foundation upgrading.

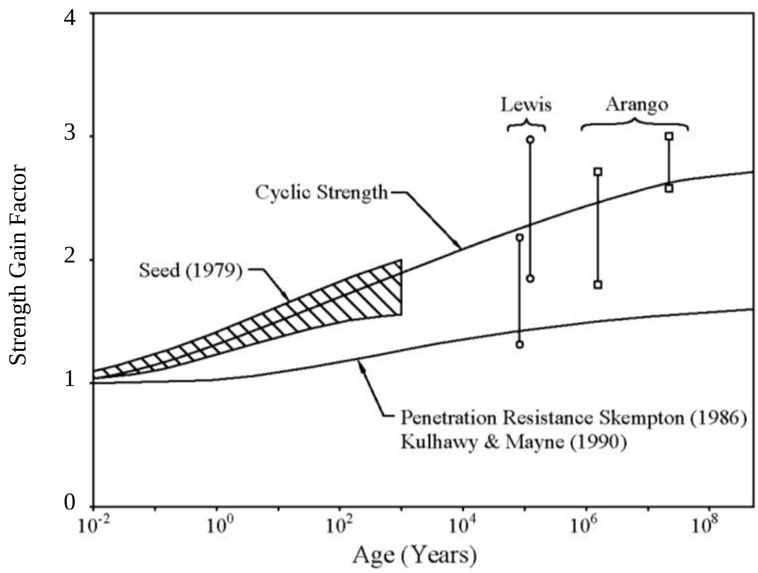

Figure 5: Cyclic strength and penetration resistance of aged sand deposits (after Arango et al, 2000)

The lower curve on Figure 5 shows the effect of the deposit age on penetration resistance based on recommendations of Skempton (1986) and Kulhawy & Mayne (1990). The effect of age on penetration resistance is less than its effect on liquefaction resistance. Jamiolkowski *et al.* (1985) reported that their test data showed the beneficial effect of mechanical overconsolidation and prestraining on liquefaction resistance to be about three times the corresponding effect on penetration resistance. At an early stage, Seed *et al*. (1977) articulated the view that penetration and liquefaction resistances were likely to be similarly affected by age, which had the effect of minimising its importance in empirical, field based liquefaction assessment. Unfortunately it was not indicated that, quantitatively, they increase at different rates. This is perhaps an example of the engineering proclivity to focus on what can readily be measured or computed while setting aside less tractable factors.

Leon *et al.* (2006) incorporated both curves on Figure 5 into a procedure for assessing the effect of deposit age on the CRR deduced from penetration resistance. They report investigations of different palaeo-liquefaction sites on the SCCP which support this age correction to the Seed liquefaction resistance criteria for Holocene sands. In a discussion of the Leon *et al.* paper, Monaco & Schmertmann (2007) noted:

> *A great merit of the paper is having explicitly highlighted to the geotechnical community the importance of aging when assessing liquefaction potential. The authors have shown that accounting for age is not a refinement but a necessity for economic design, because aging has a major influence on liquefaction behaviour.*

Moss *et al.* (2008) tested undisturbed and reconstituted specimens of a late Pliocene sand in California in a similar manner to the original work of Seed (1979). The estimated age of the sand deposit, 2.5M years, had been established by detailed geological mapping in 1994 as indicated in Moss *et al.* (*loc. cit.*). The liquefaction resistance increase factor was 2.2 which is reasonably compatible with the findings of the Lewis/Arango investigations described above. Moss *et al*. indicated that a programme of similar investigations at other sites of known age was being undertaken.

Ongoing research on the effect of sand age is being performed at field test sites (Geiger *et al.*, 2010; Saftner, 2011) The work includes induced field liquefaction as part of the US National Science Foundation programme 'Network for Earthquake Engineering Simulation' (Saftner, *loc.cit.*).

6 CONSTITUTIVE MODELS

Modelling the earthquake liquefaction process numerically is difficult and its developments are largely separate to the empirical field based approach widely used in practice. The static liquefaction of hydraulic fill is a different matter. Failures of artificial sand islands in the Canadian Beaufort Sea in the 1980s initiated studies leading to the critical state approach of Jefferies & Been (2006). Insights from critical state soil mechanics include the importance of the current soil state as defined by the combination of its density and effective confining stress (Wroth & Bassett, 1965) and considering this in relation to the critical state line in e – log p' space. Critical state theory also indicates the way in which different soil characteristics, particularly compressibility, can influence liquefaction resistance. Such insights are helpful in considering empirical evidence and can explain unusual or apparently conflicting experimental results such as the effect of silt content on liquefaction resistance. From a backdrop of critical state theory, Wroth (1988) emphasised that correlations developed between *in situ* test results and soil properties should be: (a) based on physical insight, (b) set against a theoretical background and (c) expressed in dimensionless form. These are intrinsically helpful observations and there are indications that they are influencing empirical field based liquefaction assessment. Examples are the normalisation of CPT results (Robertson, 1990) and use of the state concept in assessing high overburden stress effects on the cyclic resistance ratio (Boulanger, 2003).

Critical state theory indicates that compressibility, in both the elastic and plastic range, is a key property for liquefaction assessment. Sands are less compressible than clays and are more susceptible to cyclic loading. Transitional soil textures or types are intermediate in compressibility and liquefaction potential. This potentially powerful insight recasts the influence of fines content or behaviour type characterisations used in empirical procedures. Comparisons by Jefferies & Been (*loc. cit.*) indicate the soil compressibility λ correlates only weakly with fines content, which is perhaps not surprising given the uncertainty over the usefulness of fines content in liquefaction assessment. The correlation with soil behaviour type index I_c is good although there is a lack of data for transitional soils. The $\lambda – I_c$ relationship indicates a smooth transition from sands to clays. Boulanger & Idriss (2006, 2007) postulate a rapid transition in behaviour from sand-like to clay-like over a small range of PI between 3 and 8. With the Been & Jefferies critical state approach, a correspondingly rapid change in soil compressibility is implied and it is not clear that this is realistic.

One practical limitation on application of critical state theory is that it is formulated using mean effective stress, which is proper but inconvenient as this requires knowledge of K_o. In passing, it may be noted that cone penetration resistance is known to depend strongly on horizontal stress whereas effective overburden pressure is used for convenience in practice. Another limitation in regard to cyclic load behaviour is the key role of fabric, which is an independent variable not captured by the soil state. In the laboratory, two specimens of the same sand can be prepared at the same state (void ratio and consolidation pressure) by different methods. The samples will have similar ultimate shear stresses but different stress-strain behaviours and therefore propensity to liquefy. Fabric will affect several parameters of a constitutive model and it is difficult to separate these effects. Plastic stiffness in a work hardening model is controlled by a plastic modulus akin to Young's modulus in the elastic range. Shuttle (2006) demonstrated that differences in triaxial compression test behaviour resulting from fabric differences could be captured by varying the plastic modulus in a numerical model. However, test differences can also be captured by varying the initial soil state in the numerical model. Thus in matching a pressuremeter curve Shuttle (*loc. cit.*) was unable to arrive at a unique set of model parameters as more than one combination could produce a match. This is a fundamental difficulty for practical application of constitutive models to assessing liquefaction resistance. Replicating field behaviour using a multiple parameter model is one thing, predicting it quite another. This problem could be addressed by measuring additional soil characteristics with another type of test although this would increase the quantum of investigation. An alternative is to have more data streams from an *in situ* test. For example, Shuttle suggested adding conductivity logging to the pressuremeter in order to observe the changing void ratio as the test progresses.

7 APPLICATION IN AUSTRALIA

7.1 SOIL RESISTANCE

North American researchers have contributed a great deal to the international application of liquefaction science and have the dominant role in this field. Nonetheless, institutions in Japan and China have made pivotal contributions which are not always fully appreciated in the western world. The geological and seismic settings for liquefaction vary from place to place and this probably underlies different emphases in Chinese, Japanese and North American recommended practices. Countries that are earthquake prone have opportunities to investigate field liquefaction which are lacking in Australia. Therefore, it is probably worthwhile to consider where there are conditions similar to Australian settings and what has been learned there. For example, the Chinese seismic code (PRC, 2001) indicates that the critical penetration resistance to resist liquefaction in sand containing about 7% clay is one-half that of clean sand while 20% clay reduces the requirement to one-third of the clean sand value. These criteria are not universally true (Moss & Chen, 2008) but clearly are correct in Chinese experience (Zhou, 1987). An explanation could be that sediments derived from the peneplain coastal regions of China have sufficient active clay in their fines to affect their cyclic load resistance. The Chinese code also indicates that liquefaction need not be considered if the depth to groundwater is greater than about 7 m. Such considerations can have relevance to Australian settings.

In replying to Pyke (2003), the NCEER working group agreed that their recommendations would be conservative in many cases. They noted that knowledgeable engineers must understand the various nuances associated with liquefaction behaviour to correctly assess the hazard. Pyke expressed concern over the cookbook nature of the NCEER assessment procedure which came with equations to facilitate calculations. This concern has been validated in practice where the engineering graduate can readily develop a spreadsheet to calculate liquefaction safety factors without needing to understand the underlying issues. The writer believes that this simplistic practice dominates liquefaction assessment in Australia often with inappropriately conservative outcomes.

Burmister (1941) noted:

> *Once a formula has been printed, it takes on a more or less authoritative character, and the assumptions on which it is based and the limitations in its use tend to be forgotten or overlooked.*

But engineers have a natural affinity for calculations as indicated by the statement in the Jefferies & Been (2006) book Soil Liquefaction:

> *In effect, the approach in this book demands that the profession's view of liquefaction should pass what could be called a variation of the Turing test: "if it does not compute, then you have nothing."*

While a desirable goal, our computations (and theoretical models) are currently unable to capture all of liquefaction behaviour or predict it. Clearly Australia has coastal Holocene deposits that fit the NCEER prescription. It also has much older sediments and residual soils that are not represented in the databases from which the empirical NCEER procedures derive. The quantitatively different effects of age on penetration and liquefaction resistance illustrated by Figure 5 need to be taken into account. There is also a need for engineering geologists to be more involved in liquefaction assessment and, in the writer's view, for historical, geological and compositional screening (Kramer, 1996) to be used before applying a spreadsheet.

One other factor that stands out as needing to be better understood in practice is the use of averaging or point by point characterisation of the soil resistance profile. As discussed earlier, inspection of the current liquefaction database (Moss, 2003) shows that the case record soil profiles are characterised by the average resistance of the zone judged to be most critical. This has been the intent, if not always consistently implemented, from the inception of empirical field based liquefaction assessment. It is necessary that the same approach is adopted in applying the resulting liquefaction criteria, such as the NCEER recommendations. Specifically, applying the criteria to a site on a point by point basis is inappropriate as it introduces unintended conservatism. Unfortunately this practice is encouraged by computer based analysis of continuous CPT records.

7.2 GROUND MOTIONS

As pointed out by Liyanapathirana & Poulos (2001) most earthquakes occur at the boundaries of tectonic plates whereas Australia sits in the middle of one of the world's largest plates. As a consequence, Australia not only has relatively low earthquake activity but also a different type of earthquake ground motions. The major difference is that high acceleration levels last only a few seconds in Australian intraplate earthquakes while they may exist throughout the duration of a plate boundary (interplate) earthquake. Peak acceleration and the associated base shear force is of prime importance for the response of buildings and other structures but it is repetitive loading that matters for liquefaction. Structural earthquake engineering was established before the issue of liquefaction emerged in the 1960s so it was natural to follow this lead and to use the shear stress associated with peak acceleration for liquefaction assessment. Unfortunately, use of the peak acceleration stress coupled with the difference in intraplate and interplate earthquake motions creates another unhelpful problem in applying the NCEER-based liquefaction criteria in Australia.

Alternatives to the stress-based liquefaction assessment method have been formulated and one of these was adopted by Liyanapathirana & Poulos (*loc. cit.*) as being more suitable for Australian conditions. As earlier proposed by Kayen & Mitchell (1997), the Arias intensity has advantages in quantifying the earthquake demand with regard to cyclic load resistance of soil. Arias intensity, I_h, is a measure of the destructiveness of earthquake ground motions and is directly proportional to the area under the horizontal acceleration – time curve. Arias intensity is a function of, and therefore reflects, the amplitude variation, frequency content and duration of the earthquake motion. Kayen & Mitchell calculated I_h values for about 60 of the SPT based earthquake database clean sand sites having an approximately equal number of liquefied and non-liquefied conditions. The I_h – N data could be separated by a boundary curve similar to the original Seed CSR – N clean sand curve. There being no great difference in the outcome when using CSR or I_h is a result of all the causative earthquakes being of the same interplate type. For a common type of earthquake there is a constant relationship between I_h and CSR, as shown by Green & Mitchell (2003). However, if the nature of the ground motion differs significantly (e.g. the ratio of peak to average acceleration in the record) then the relationship between I_h and CSR will also be different. Considering an Australian intraplate earthquake, I_h is smaller than for an interplate event having the same CSR. The NCEER liquefaction recommendations are based on peak cyclic stress and empirical criteria from interplate events, which brings into question their relevance to Australian conditions because of the conservatism involved.

8 FUTURE DEVELOPMENT

It is worth restating that the reliability of assessing liquefaction resistance could be greatly improved by increased involvement of suitably informed engineering geologists. There is probably no other single change to the practice of evaluating soil resistance that would have such a beneficial effect. Longer term, the best prospect may be to also move away from empirical criteria towards a mechanics based approach underpinned by a theoretical framework. However, some input soil parameters to a computational scheme are strongly fabric dependent. An impediment to practical application of constitutive models is the inability to evaluate suitable values for the multiple input parameters, which is not a new dilemma. It leads directly to the need to increase the types of measurements that can be made using *in situ* site investigation tools in order to provide data redundancy. Conductivity logging (Shuttle, 2006) and use of radioisotopes (Shibata *et al*., 1994) can provide information on soil moisture content and density.

Improvement of the basic piezocone might also be helpful. There is disagreement over the utility of excess pore pressure (u_e) measurements and incorporating these into the soil behaviour type index I_c. Jefferies & Davies (1993) initially formulated the I_c index based on the effective cone resistance, $q_c – u_e$, originally proposed by Houlsby (1989) as an indicator of soil type. Roberston & Wride (1998) modified the definition by excluding excess pore pressure due to concern over measurement accuracy and reliability of the term $q_c – u_e$, and their simplified version is now in common use. The proponents of the original I_c argued that it unifies drained and undrained behaviour and that the Robertson & Wride version ignores valuable information. While more appealing in principle, the measurement accuracy of net cone resistance in weak soils is a practical issue and the matter is unresolved.

Soil fabric is of equal importance to state in determining liquefaction resistance but it is difficult to assess. Shear wave velocity, V_s, is a small strain measurement and is sensitive to soil fabric as well as state. Penetration resistance, q_c, is a large strain measurement and is sensitive to state but not to fabric. Correlation between q_c and V_s is weak (Roy, 2008) which may indicate the effect of fabric. Combining the two measurements by comparing the small strain shear modulus, G_0, to the cone resistance holds promise as an index for detecting stronger fabrics likely to be resistant to cyclic loading (Eslaamizaad & Robertson, 1996). An alternative approach that appears to have been pursued to a limited extent in the geo-engineering field is direct measurement of fabric (Mitchell & Soga, 2005). There is scope for further research on quantative assessment of fabric and relating this to liquefaction resistance. It should be noted that the CSIRO in Australia has long had a prominent role in the classification and measurement of soil fabric (Lafeber, 1966; Brewer & Sleeman, 1988).

It would be appropriate for the seismological community to consider the nature of ground motions in Australia and how these compare with earthquake events in the liquefaction database. Recommendations on appropriate adjustment of the seismic demand to better tailor the liquefaction assessment methodology to Australian conditions would be helpful. Current liquefaction practice can be made more relevant to Australia. There is perhaps a case for a comprehensive review to be undertaken by an expert panel under the auspices of an appropriate institution.

9 FINAL COMMENTS

Using the Moss (2003) database, Seed *et al*. (2003) proposed a complete overhaul of the *in situ* penetration based "simplified procedure" as found in NCEER. This revision recast liquefaction assessment on a probabilistic basis for use, together with demand uncertainty, in performanced based design[6]**.** Comparison with deterministic criteria indicate that use of the new UC Berkeley probabilistically based criteria would further increase the conservatism of the assessment procedure (Youd, 2011).

Idriss & Boulanger (2008) discuss the uncertainties in estimating liquefaction-related performance. These are present in site characterisation, liquefaction boundary curves, ground motions and the surface deformation caused by liquefaction. They note that conservative assumptions are made in each component and that these accumulate into a higher margin of safety than may actually be required, leading to more expensive conclusions than are necessary. There is little doubt that the degree of conservatism in current liquefaction practice is not understood by decision makers.

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[6] In the earliest consideration of stochastic uncertainty in liquefaction assessment, Donovan (1971) characterised the uncertainly in earthquake ground motions as about twice that of the soil resistance. This uncertainty has probably been reduced over time and should be less today.

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