UNDERSTANDING LIQUEFACTION TRIGGERING RISK - AN AUSTRALIAN GEOTECHNICAL DESIGN PERSPECTIVE

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ABSTRACT

Resilience is the ability to quickly recover during an adverse event. Following an earthquake the resilience of a community can be directly related to working infrastructure. Geotechnical resilience design must consider liquefaction from future large earthquakes.

Although Australia is considered a stable continental region with relatively low seismic hazard, earthquakes do occur and where susceptible geological conditions exist, liquefaction can occur. In fact, liquefaction has been documented in Australia on at least three occasions. In 1897, liquefaction was observed during a large (Ms 6.5) earthquake near Beachport, south-eastern South Australia (Collins et al., 2004); in the 1903 Warrnambool, Victoria (Ml 5.3) earthquake (Mitchell and Moore, 2007); and in 1968, numerous „sand blows" were observed following the Ms 6.8 earthquake at Meckering in Western Australia (Collins et al., 2004).

Liquefaction is a credible geohazard considered in current Australian geotechnical engineering practice, and infrastructure planning desk studies in Australia commonly identify liquefaction as a geohazard where susceptible soils exist within the project footprint. Further assessments are required in subsequent feasibility and detailed design phases. Accurately assessing the liquefaction triggering potential is an essential part of geotechnical design considerations.

The low seismicity of Australia creates a situation where liquefaction triggering is marginal at design hazard levels. This low level of seismic hazard makes the liquefaction trigger assessment very sensitive to the derivation of the seismic inputs. The lack of guidance on liquefaction from AS1170.4 requires interpretation of the basis seismic hazard inputs.

This paper explored the sensitivity to seismic inputs in low seismicity hazard Australia, to better understand liquefaction triggering risk in Australian geotechnical design. The components of the seismic hazard inputs are reviewed. A case study is presented showing that for liquefaction assessments in low seismicity regions, liquefaction triggering is sensitive to the selection of design magnitude and the calculation of the ground motions through the soil profile.

1 INTRODUCTION

Resilience is the ability to quickly recover during an adverse event. Lesson learned from recent earthquakes show that the resilience of a community can be directly related to the performance of its infrastructure. With transport, power and water functional communities are able to respond and recover quicker.

When considering the geotechnical resilience of infrastructure during earthquakes, liquefaction is a significant risk. Liquefaction is a soil behaviour in which saturated soil experiences a reduction in strength due to pore pressure increase during dynamic loading, such as earthquake ground shaking. Consequences of liquefaction include settlement, lateral displacement, loss of bearing capacity, and uplift of buried structures. The historic impact of liquefaction to society is well known from historic earthquakes including billions of dollars in damage from 1994 Northridge, 1995 Kobe, 1989 Loma Prieta (San Francisco) and most recently 2011 Christchurch.

Geotechnical earthquake engineering practice"s understanding of liquefaction is increasing with every real world example earthquakes from the early 1990"s to the current findings coming out of the 2011 Christchurch event. The nascence of understanding has meant that as a practice, liquefaction risk has been missed in design of many high importance/high consequence infrastructure, such as San Francisco"s Bay Area Rapid Transit Tunnel, the Massey Tunnel in Vancouver, San Pablo Dam in the East Bay of San Francisco and even the water and sewer network of the low lying suburbs of Christchurch. These significant structures considered earthquake ground shaking, but not liquefaction and had significant retro-fits or rebuilds.

Current building codes do not focus on earthquake resilience, but rather life safety. This means significant damage is allowed as long as the code objective is met. It is therefore not surprising that when a major earthquake strikes an urban region the losses are large and the general public is left to wonder why. The Christchurch earthquake in February 2011 is a prime example of this. As design focuses on resilience and beyond life safety, understanding risk is critical.

Although Australia is considered a stable continental region with relatively low seismic hazard compared to active tectonic areas of the world, earthquakes do occur and where susceptible geological conditions exit, liquefaction can trigger. In fact, liquefaction has been documented in Australia on at least three occasions. In 1897, liquefaction was observed during a large (Ms 6.5) earthquake near Beachport, south-eastern South Australia (Collins et al., 2004); in the 1903 Warrnambool, Victoria (Ml 5.3) earthquake (Mitchell and Moore, 2007); and in 1968, numerous „sand blows" were observed following the Ms 6.8 earthquake at Meckering in Western Australia (Collins et al., 2004). Note that liquefaction was not observed in the 1989 Newcastle Event, the colloquial reference design earthquake event for many Australian geotechnical practitioners.

Liquefaction is a credible geohazard considered in current Australian geotechnical engineering practice. Infrastructure planning desk studies in Australia commonly identify liquefaction as a geohazard where susceptible soils exist within the project footprint. Further assessments are required in subsequent feasibility and detailed design phases. Accurately assessing the liquefaction potential is an essential part of geotechnical design considerations.

The low seismicity of Australia creates a situation where liquefaction triggering is marginal at design hazard levels. This low level of seismic hazard makes the liquefaction trigger assessment very sensitive to the derivation of the seismic inputs. AS1170.4 (2007) – Earthquake Actions, does not provide liquefaction guidance or details from the seismic hazard analysis basis to support rigour in liquefaction assessments in Australia without an interpretation of the seismic inputs. In typical geotechnical engineering liquefaction assessments, focus is on characterising the ground conditions and characterising variation and material properties of ground conditions ultimately providing design advice against liquefaction risk. The seismic hazard inputs out of AS1170.4 are often taken for granted without understanding of application or implicit uncertainty in application of liquefaction assessment.

This paper explores the sensitivity to seismic inputs in low seismicity hazard Australia, to understand liquefaction triggering risk in Australian geotechnical design.

2 LIQUEFACTION TRIGGERING ANALYSIS METHODOLOGY

Liquefaction assessment methodology has been well established in earthquake engineering practice following Seed and Idriss (1971) and refinement over the last 40 years. The resistance to liquefaction depends on the relationship between the in-situ density of the soil with its critical state, as well as the behaviour of the soil under earthquake-induced cyclic loading.

Seed and Idriss (1971) proposed a simplified procedure for evaluation of liquefaction triggering that compare the soils" resistance to liquefaction, termed cyclic resistance ratio (CRR), with the cyclic stress caused by an earthquake, termed cyclic stress ratio (CSR), expressed as the factor of safety (FS_{liq}) against triggering liquefaction.

$$FS_{liq} = CRR/CSR$$

A FS_{liq} less than 1.0 implies that liquefaction triggering is likely.

The CRR for liquefaciton of in-situ deposits is evaluated from the penetration resistance using standard penetration tests (SPT) or cone penetration tests (CPT) or from laboratory testing on high quality samples. Empirical relationships have been produced by correlating the SPT N (Seed and Idriss 1971, Seed et al. 1985), CPT qc (Robertson and Wride, Suzuki et al. 1995; Moss et al. 2006), shear wave velocity (Andrus and Stokoe 1997; 2000) and seismic dilatometer paratmers (Marchetti et al. 2008) and the the estimates of the SCR of a number of sites which had or had not had liquefaciton during major earthquakes in the past (Youd et al. 2001; Seed et al. 2003) (Figure 1).

Discussion of state of the practice or uncertainty CRR is beyond the scope of this paper. Semple (2013) discussed problems with liquefaction criteria and their application in Australia and provided a thorough review of the current state of liquefaction susceptibility, including the relatively recent considerations such as low plasticity silts/clays from Bray & Sancio (2006) and as seen in Christchurch (Bray et al. 2014).

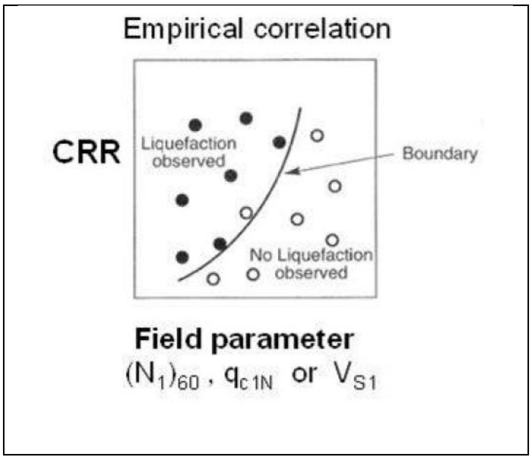

Figure 1: Liquefaction triggering boundary.

The CSR used in the simplified procedure for liquefaction triggering analysis is the ratio of average, or equivalent, shear stress induced by the earthquake to the in-situ effective vertical stress. Seed and Idriss (1971) proposed that the average equivalent CSR for liquefaction triggering assessment is about 0.65 times the peak shear stress, and may be estimated as:

$$CSR = 0.65 \cdot \frac{\sigma_v}{\sigma_v^{'}} \cdot A_{\max} \cdot r_d$$

Where σ_v is the total vertical stress, $\sigma_{v'}$ is the effective vertical stress, A_{max} is the maximum acceleration (taken as peak ground acceleration, PGA), and r_d is the non-linear shear stress reduction factor with depth.

While A_{max} defines the maximum ground acceleration it provides no information on the duration of shaking. The importance of design magnitude in liquefaction assessments is the provision of an indication of duration of shaking or the number of strong motion cycles.

The convention for assessing liquefaction triggering is to determine CSR normalized to the duration of a M7.5 earthquake, denoted $CSR_{7.5}$. This is achieved by modifying the CSR by a magnitude-duration weighting factor, DWF after Idriss and Boulanger (2008) and calculated as:

$$DWF = (6.9*EXP(-M / 4)-0.058)$$

The magnitude-duration weighted cyclic stress ratio, $CSR_{7.5}$, is calculated as:

$$CSR_{7.5} = CSR / DWF$$

To estimate a CSR, these assessments need design ground motions (PGA) and design earthquake magnitude. Groundwater is assumed to be near the surface unless subsurface information proves otherwise.

3 DESIGN CODE LIQUEFACTION GUIDANCE

In active seismic regions, most seismic design codes require liquefaction analysis and provide specific guidance on the assessment methodology. For example ASCE 7-10 requires a Geotechnical Investigation Report, for high risk structures, with an assessment of consequences of soil strength loss, including slope instability, liquefaction, total and differential settlement, surface displacing due to faulting or seismically induced lateral spreading or flow.

ASCE 41-04, seismic evaluation of existing buildings, requires that liquefaction susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 feet under the building for Life Safety and Immediate Occupancy. If these are encountered a detail study is required.

The NZ Transport Agency Research Report 553 – "The Development of Guidance for Bridges in New Zealand for Liquefaction and Lateral Spreading Effects", presents a clear set of available procedures for analysis and design of bridges based on most recent research findings. A recent development following the 2011 Christchurch Earthquake is the implementation of liquefaction settlement/lateral spread displacement tolerances in the building codes by NZ MiBE. These define vertical and lateral displacements to varying degrees under SLS and ULS events according to liquefaction risk maps that need to be addressed in the design. Structures in areas identified to have moderate to significant land damage from liquefaction require a site specific investigation.

In Australia, AS1170.4 does not consider the effect of a structure from related earthquake phenomena such as settlement, slides, subsidence, liquefaction of faulting. Seismic design for infrastructure (bridges, roads, wharfs, tanks, pipelines) is formally excluded from AS1170.4 (2007), but many infrastructure specific codes, such as AS5100.2 Bridge Code, refer back to AS1170.4 for the basis seismic hazard. AS5100.2 guidance on liquefaction is limited to stating that the possibility of soil liquefaction shall be investigated where saturated sandy and silty soils within 10m of the ground surface have SPT N <10.

Where Australian specific infrastructure seismic design codes do not exist, such as LNG Tanks, international codes are referenced. These often provide liquefaction guidance relying on the seismic hazard sourced from their country of origin, therefore AS1170.4 must be interpreted for application in these codes.

The application of seismic hazard from AS1170.4 (2007) in liquefaction triggering assessment requires significant interpretation which creates an uncertainty in design.

3 DESIGN GROUND MOTIONS CONSIDERATIONS

In Australian practice, liquefaction triggering assessments generally start with AS1170.4 (2007) to derive base ground motion levels by extracting a “Z” value for a site. The “Z” represents the PGA with a 1/500 annual probability of exceedance for bedrock ground conditions. Following the procedures in AS1170.4 the “Z” value is scaled by a probability factor (k_p) according to the importance level of the structure and by factors relating to ground conditions (site sub-soil class).

3.1 DESIGN LIFE, IMPORTANCE LEVEL, & DESIGN RETURN PERIOD CONSIDERATIONS

In most modern seismic codes, the design ground motion is determined from an acceptable risk of earthquake occurrence over the lifetime of a structure. For a design life of 50 years at a 90% confidence level (10% exceedance of design) the return period is 475 years or an annual probability rounded off to 1/500. This is the “normal” structure design level in most international codes.

Design for a more important structure might consider a 98% confidence level which represents 2% in 50 years and would have a 1/2500 annual probability of exceedance. Following the same logic, a structure with a design life of 100 years and a 10% chance of exceedance of design would have an annual probability of exceedance of 1/1000.

AS1170.4 defines the consequence importance level following AS1170.0 based on building type, occupancy, and purpose (emergency response) (Table 1). These are aligned with many international codes.

Table 1: Importance Level AS1170.0

Importance Level	Description
1	Minor structures (failure not likely to endanger human life)
2	Normal structures and structures not falling into other levels
3	Major structures (affecting crowds)
4	Post disaster structures
5	Exceptional structures

In deriving seismic hazard for design, the importance levels are assigned a probability factor (k_p) representing the annual probability of exceedance (return period). The greater the importance, the higher the risk, and the annual probability of exceedance for design.

In practice there is conflicting guidance in the derivation of design annual probability of exceedance that requires consideration and interpretation. This is exemplified in AS5100.2 (Bridge Code) where a proposed revision and the Austroads Bridge Design Guidelines for Earthquake (2012) provide conflicting advice. AS5100.2 generally follows the AS1170.4, annual probability of exceedance, except for the Importance Level 4 (Bridge Design Class 4) structures for essential emergency response, where 1/2000 is recommended compared to 1/2500 in AS1170.4.

Austroads suggests that all bridges be designed for the ultimate (damage control) limit state under a design (1/2000 year) ground motion to ensure that after the earthquake the bridge shall retain its structural integrity. Importance Level

4 will be designed for the serviceability limit state under the design (1/2000) year earthquake. Note that the basis for the 1/2000 annual probability of exceedance is a design life of 100 years and a 5% chance of exceedance of design.

To add to the confusion, AS1170.4 (from AS1170.0) suggests a 1/2500 annual probability of exceedance for an Importance Level 4 structures, yet the BCA (2014) suggests a 1/1500.

Table 2 is a summary of the different annual probability of exceedance

Table 2: Annual Probability of Exceedance for Importance Level Comparison

Importance Level	IL 2	IL 3	IL 4
Austroads	1/2000	1/2000	1/2000
A5100.2 (AS1170.4)	1/500	1/1000	1/2000
AS1170.4 (AS1170.0)	1/500	1/1000	1/2500
AS1170.4 (BCA)	1/500	1/1000	1/1500

Lesson learned from recent earthquakes show that the resilience of a community can be directly related to the functionality of its infrastructures. With transport, power and electricity and water functioning a community is able to respond and recover quicker. If a major earthquake occurs, it would be difficult to imagine society accepting that a modern major bridge is not functioning as it was not designed for the extreme event.

3.2 UPDATE TO AS1170.4 HAZARD MAP

The ground motions provided in AS1170.4 (2007) are based on the probabilistic seismic hazard analysis (PSHA) conducted by Gaull et al. (1990) with revision by McCue et al. (1993).

Geoscience Australia 2012 (Leonard 2013) have developed a revision to this map and recommended it as an update to the hazard map in AS1170.4. As shown in Figure 2, the GA 2012 generally shows lower hazard than AS1170.4, and the 1/2500 spectra is closer to the AS1170.4 (2007) 1/500 spectra. The GA 2012 model is more aligned with current site specific studies where the earthquake catalogue used in the PSHA includes the last 20 years of seismicity, compared to the two decide old AS1170.4 model. Review or comment on the application of the differences in hazard is beyond the scope of this paper, but geotechnical practitioners should be aware of the difference and the potential update to AS1170.4.

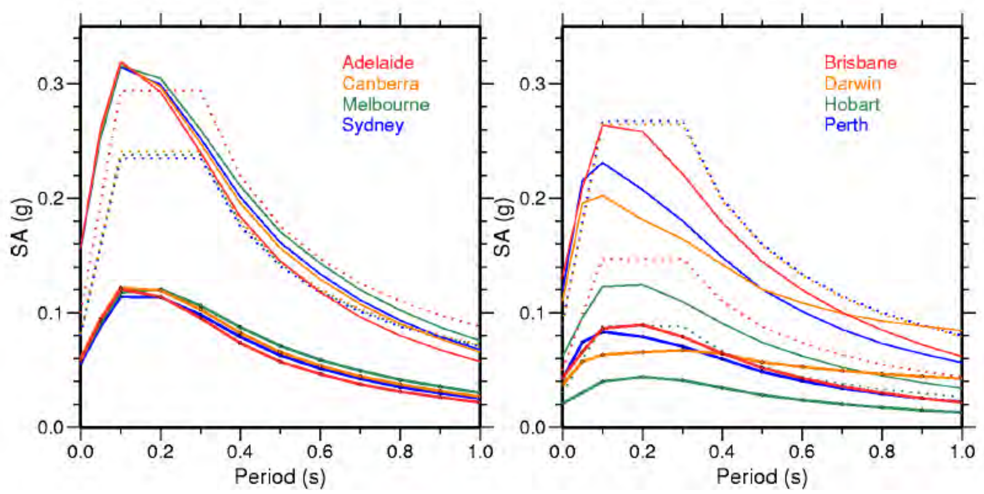

Figure 2: Geoscience Australia National Hazard Map (Leonard 2012) hazard spectra for 1/500 (thin lines), 1/2500 (thick lines) and AS1170.4 for 1/500 (dotted line).

3.3 SITE SOIL SUBCLASS AND SITE RESPONSE CONSIDERATIONS

Earthquake ground motion observed at soil sites can be substantially different from rock sites. The overlying soil deposits modify the earthquake induced bedrock motion as the motion is transmitted up through the soil profile to the ground surface. This modification is known as site response and is a function of the soil profile geometry, the soil properties and the characteristics of the earthquake excitation.

AS1170.4, and most other seismic codes, provide scaling factors to adjust the bedrock ground motions following site soil properties. The soil profile is classified into a Site Soil Sub-Class based on soil properties. The soil classes are defined by the natural frequency of the soil profile estimated by geotechnical investigation.

Using the simplified method, CSR is derived from the bedrock PGA, scaled for the desired return period, factored by the site soil sub-class, and scaled by the design magnitude to derive a design PGA.

Another method for calculation CSR directly in liquefaction analysis is a site response analysis. The site response analysis eliminates the need to scale for a site soil sub-class, by propagating an earthquake ground motion time-history, representing the bedrock ground motion, through the soil column to calculate the CSR at any location in the soil profile. Depending on the variability of the soil profile and depth, the site response analysis may present very different results to that of the site sub class scaling of AS1170.4.

Figure 3 presents the difference in CSR from a site response analysis compared to CSR derived using the simplified method for 2 different design magnitudes. Site response analysis isn‟t common in Geotechnical practice here in Australia, but can provide more precision for ground response compared to using the AS1170.4 site soil sub-classes.

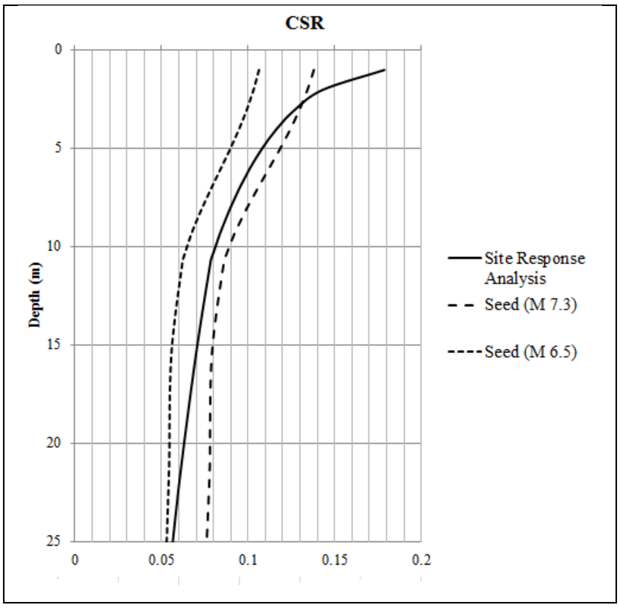

Figure 3: CSR derived from site response analysis and following Seed with M6.5 and M7.3

4 DESIGN MAGNITUDE

For liquefaction triggering assessments in Australia, a critical missing input parameter is the earthquake design magnitude. The design magnitude defines the duration weighting factor (representing the number of shaking cycles) and is included in the calculation of $r_{d,}$ the non-linear shear stress reduction factor with depth. In active seismic regions around the world, local seismic design code often provides specific guidance on the selection of appropriate design earthquake magnitudes to estimate a CSR. For example ASCE 7-10 recommends using the maximum considered earthquake (MCE) and post-Canterbury Earthquake practice in NZ following NZS1170.5 and MBIE 2012 guidelines specify M7.5 for all liquefaction calculations regardless of the importance level.

Where design magnitudes are not explicitly provided by code, the common method for selecting magnitude is to consider the probabilistic earthquake scenarios that contribute the greatest amount to the ground motion hazard (CalTrans 2012). This is done by examination of the magnitude deaggregation of the PSHA.

In Australian practice, AS1170.4 (2007) and Gaull et al. (1990) with revision by McCue et al. (1993) do not provide enough information to readily extract earthquake design magnitudes or to develop magnitude deaggregation plots. As a result, earthquake engineering practitioners in Australia have applied a number of different methodologies to assign earthquake design magnitude for site-specific studies. These methods range from estimating mean values from regional recurrence curves (Mitchell and Moore, 2007), using the maximum historic earthquake in Australia for a given region, consideration of a range of magnitudes (Yang and Wright, 2010) or choosing a conservative magnitude based on professional judgment.

Mote and So (2013) show in the low seismicity of Australia, liquefaction triggering is sensitive to earthquake design magnitude. Figure 4 shows that for a sand profile in a site soil sub-class D, for a typical "Z" values in Australia (e.g. 0.08g for Sydney) that the selection of a design magnitude above or below a M6.5 will determine whether liquefaction will trigger.

The lack of rigour in guidance on how to select design magnitude in Australia creates significant uncertainty in the liquefaction triggering assessments. The following sections explore a number of methods to select design magnitude in Australia.

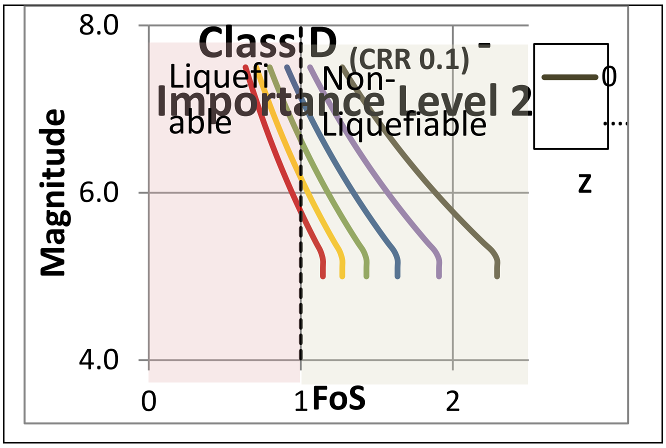

Figure 4: Sensitivity to Liquefaction (taken from Mote and So, 2013).

4.1 APPROXIMATE DEAGGREGATION OF AS1170.4

To understand application of the seismic hazard from AS1170.4 in liquefaction analysis, Dismuke and Mote (2012) developed an approximate magnitude distance deaggregation of the PSHA from AS1170-4 (2007).

The results showed that in the probabilistic modelling of scenario earthquakes, the very close and small earthquakes contribute greatest to the seismic hazard (Table 3). These values are surprising low and reflect the low seismicity nature of Australia. The magnitude of ~5 for the 1/2500 design event, questions the practical application of the magnitude-deaggregation method to derive design magnitudes in low seismicity regions as this design level is supposed to represent the rare event for essential and emergency structures and Australia is capable of much larger magnitude events.

Table 3: Mean Magnitude from Deaggregation of 1170.4 (Dismuke and Mote 2012)

Location	1/500	1/1000	1/2500
Sydney	4.9	5.1	5.2
Melbourne	5.0	5.1	5.3
Perth	4.4	4.5	4.6
Adelaide	5.1	5.2	5.3
Brisbane	4.7	4.8	4.9

4.2 MAXIMUM MAGNITUDES

An alternative magnitude selection method that matches international seismic design codes is using the maximum considered earthquake (MCE). Clark et al. 2012 developed maximum magnitudes estimates for Australia based on paleaoseismic research of past fault ruptures and lengths through the development of neotectonic domains. Figure 5 presents the maximum magnitudes for the 7 neotectonic domains across Australia.

These maximum magnitudes should be considered the upper bound magnitudes in liquefaction assessments.

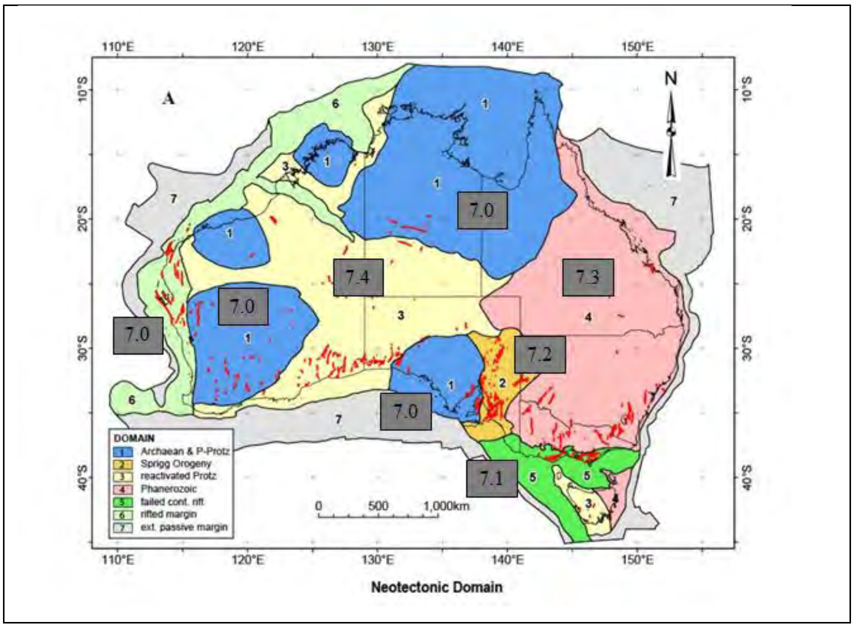

Figure 5: Neotectonic domains (D) and Maximum magnitudes (M_{max}) from Clark et al. (2010).

6 CASE STUDY

To explore the understanding of the seismic inputs to liquefaction assessment in Australia, a case study is presented for a soil profile in Sydney. This is followings the Geoscience Australia publication on Earthquake Ground Shaking Susceptibility of Botany Area, New South Wales (McPherson et al. 2013), which looked at the effect of ground shaking due to the soft soils. This publication did not considered liquefaction potential of the regolith, but the authors note that geotechnical data provide show the regolith is very soft with low shear wave velocities and SPT-N values. The soil profile (Class F from McPherson et al. 2013 in Figure 6) is assessed as a site sub-soil class C and in the upper 5m reports SPT N <10. Groundwater is assumed to be shallow.

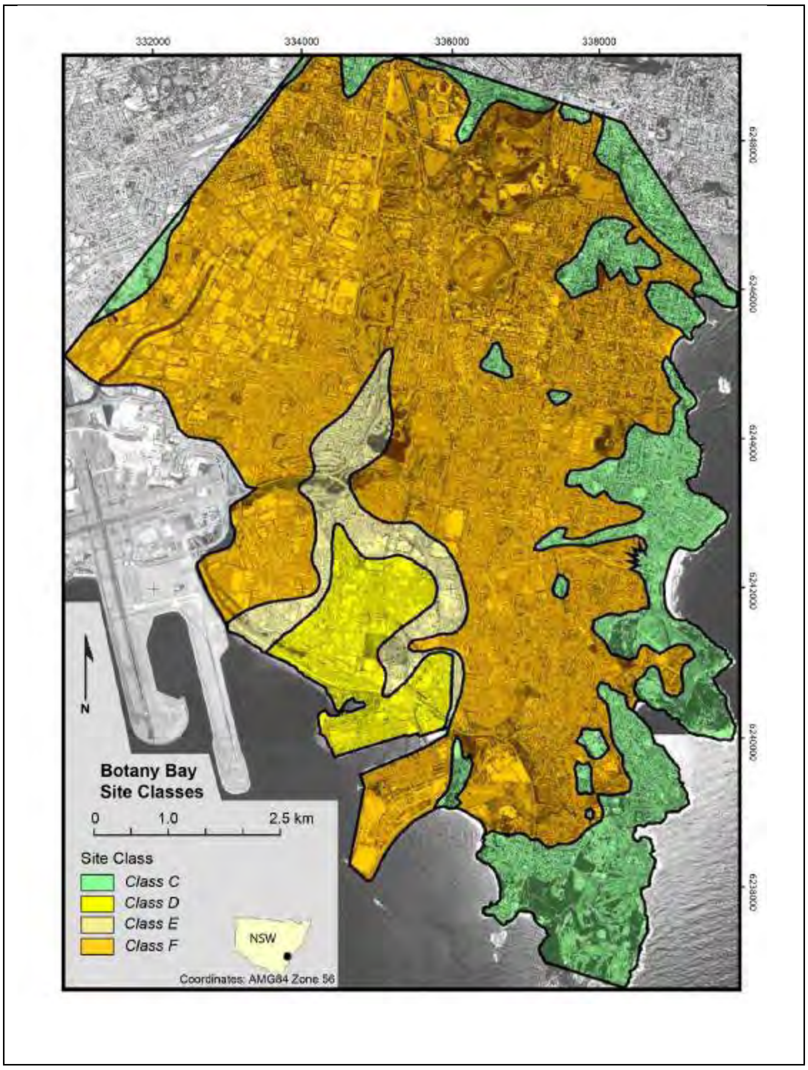

Figure 6: Class F ground conditions (taken from McPherson et al., 2013).

Using Seed et al. 2003 with inputs from AS1170.4 for a 1/1000 and 1/2500 annual probability of exceedance ground motions was considered. CSR was derived using a site response analysis and following the simplified method and using design magnitudes of M7.3 (maximum magnitude) and M6.5 (arbitrary magnitude). The Seed et al. 2003 methodology presents a probability of liquefaction where values greater than 50% are interpreted to have a high probability of liquefying.

The results shown on Figure 6 and Table 4 show for 1/1000 using the simplified method with PGA the design magnitude will determine whether it liquefies or not, but the site response analysis shows liquefaction is highly likely. For the 1/2500 the results show that profile would liquefy, but a lower design magnitude would give suggest liquefaction.

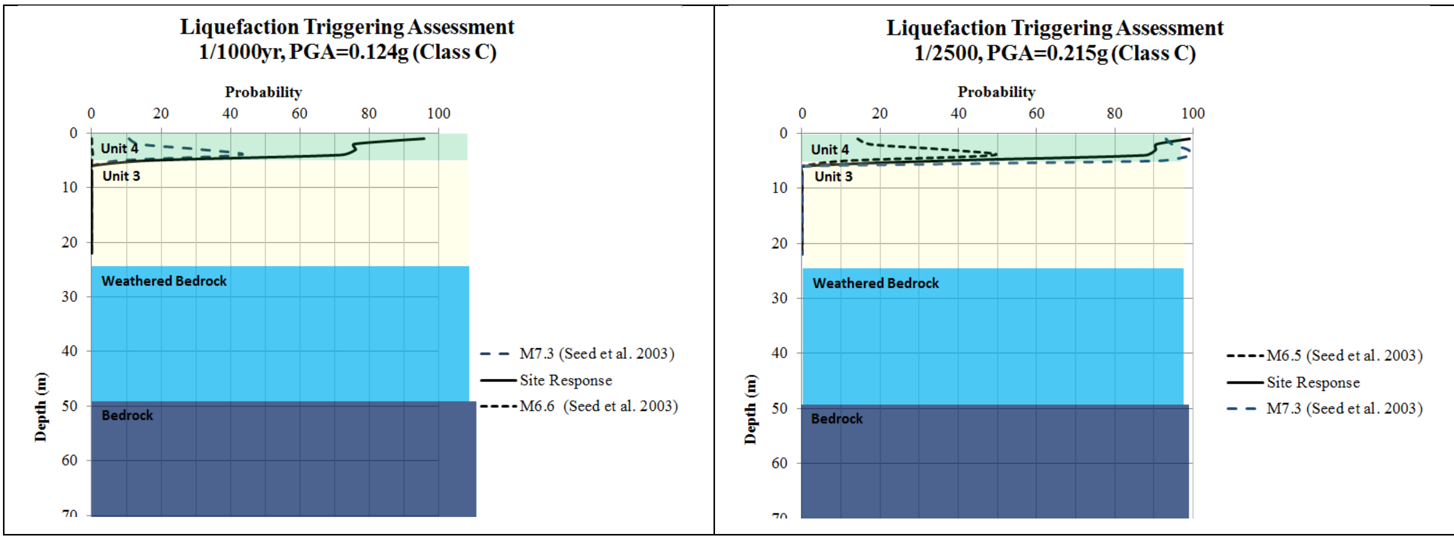

Figure 6: Liquefaction Triggering Probability for 1/1000 and 1/2500.

Table 4: Liquefaction Trigger Assessment Results

Model	Design Level Annual Probability (k_p)	PGA (g)	*Liquefaction*
M6.5	1/1000	0.124	No
M_{max} (M7.3)	1/1000	0.124	Marginal
Site Response	1/1000	NA	Yes
M6.5	1/2500	0.215	Marginal
M_{max} (M7.3)	1/2500	0.215	Yes
Site Response	1/2500	NA	Yes

6 CONCLUSIONS

Following an earthquake the resilience of a community, the ability to respond and recover, can be directly related to functioning infrastructures. Geotechnical resilience design must consideration of liquefaction from a future large earthquake. If a major earthquake occurs, it would be difficult to envision society accepting that a major bridge was not functioning. A key tenant of resilience is understanding risk.

The low seismicity nature of Australia creates a situation where liquefaction triggering is marginal, but very sensitive to the derivation of the seismic inputs. The lack of guidance from AS1170.4 requires interpretation of the seismic input for liquefaction assessments. This paper considered the sensitivity to seismic inputs in low seismicity hazard Australia.

A case study showed that liquefaction assessments in low seismicity regions, liquefaction triggering is sensitive to the selection of design magnitude. As a starting point one should consider using the M_{max} from Clark. The variability in soils and depth show that a performing a site response analysis can provide very a more accurate CSR.

The geotechnical practitioner should understand the design levels and consider a resilient approach, where performance based design is considered to get a community back up and running. Ultimately the infrastructure owners and developers will make design decision based on risk and the code, but it is up to geotechnical practitioners to understand the risk and communicate it.

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8 REFERENCES

Andrus and Stokoe 1997; Andrus, R. D., and Stokoe, K. H., II. (1997). „„Liquefaction resistance based on shear wave velocity."" Proc., NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Tech. Rep. NCEER-97-0022, T. L. Youd and I. M. Idriss, eds., National Center for Earthquake Engineering Research, Buffalo, 89–128.

ASCE 7-10. 2010. Minimum Design Loads for Buildings and Other Structures. American Society of Civil Engineers Standards.

ASCE 31-03. 2003. Seismic Evaluation of Existing Buildings, American Society of Civil Engineers.

AS1170.0. 2002. Structural design actions – General principles. Standards Australia.

AS1170.4. 2007. Structural design actions - Part 4: Earthquake actions in Australia. Standards Australia.

AS5100.2. 2014. DRAFT Bridge Design – Part 2 Design Loads. Standards Australia.

Austroads 2012. Bridge Design Guidelines for Earthquake AP-T200-12

Andrus and Stokoe 1997; Andrus, R. D., and Stokoe, K. H., II. (1997). „„Liquefaction resistance based on shear wave velocity."" Proc., NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Tech. Rep. NCEER-97-0022, T. L. Youd and I. M. Idriss, eds., National Center for Earthquake Engineering Research, Buffalo, 89–128.

Bray, JD & Sancio, RB 2006, 'Assessment of the Liquefaction Susceptibility of Fine-grained Soils', Journal of Geotechnical and Geoenvironmental Engineering, vol. 132, no. 9, pp. 1165-1177.Bray, JD, Cubrinovski, M, Zupan, J & Taylor, ML 2014, 'Liquefaction Effects on Buildings in the Central Business District of Christchurch', Earthquake Spectra, vol. 30, no. 1, pp. 85-109.

Caltrans 2012 Methods for Developing Design Response Spectrum for Use in Seismic Design Recommendations

Clark, D., McPherson, A., and Collins, C., 2010. Mmax estimates for the Australian stable continental region (SCR) derived from palaeoseismicity data. AEES 2010 Conference Proceedings

Collins C., P. Cummins & D. Clark M. Tuttle R. Van Arsdale, 2004, Paleoliquefaction studies in Australia to constrain earthquake hazard estimates. AEES 2004 Conference Proceedings.

Dismuke, J. D. and Mote, T. I. 2012, Approximate Deaggregation Method for Determination of Design Earthquake Magnitudes for Australia. ANZ 2012 Conference Proceedings.

Gaull, B.A., Michael-Leiba, M.O., and Rynn, J.M.W., 1990. Probabilistic earthquake risk maps of Australia. Australian Journal of Earth Sciences 37, 169-187.

Idriss, I.M., and Boulanger, R.W., 2008. "Soil Liquefaction During Earthquakes," monograph series, No. MNO-12, Earthquake Engineering Research Institute.

Kanai, K., 1961. An empirical formula for the spectrum of strong earthquake motions. Bulletin of the Earthquake Research Institute, University of Tokyo 39, 85-96.

Leonard, M., Burbidge, D., and Edwards, M. 2013. Atlas of Seismic Hazard Maps of Australia. Geoscience Australia Record 2013/2014 GeoCat 77399

Marchetti, S., Monaco, P., Totani, G., and Marchetti, D. (2008) In Situ Tests by Seismic Dilatometer (SDMT). From Research to Practice in Geotechnical Engineering: pp. 292-311.

McCue, K., (Compiler), Gibson, G., Michael-Leiba, M., Love, D., Cuthbertson, R., & Horoschun, G., 1993. Earthquake hazard map of Australia, 1991.

McPherson, A.A. and Hall, L.S. 2007. Development of the Australian National Regolith Site Classification Map. Geoscience Australia Record 2007/07, 37 p.

McPherson,A., Dhu, T., Jones J., and Neville M. 2013. Earthquake Ground Shaking Susceptibility of the Botany Area, New South Wales. Geoscience Australia Record 2013/26 GeoCat 75265

Mitchell P. W. and Moore C. 2007, Difficulties in assessing liquefaction potential from conventional field testing. AEES 2007

Moss, R.E.S., Seed, R.B., Kayen, R.E., Stewart, J.P., Der Kiureghian, A., and Cetin, K.O., 2006. CPT-based probabilistic and deterministic assessment of in situ seismic soil liquefaction potential, Journal of Geotechnical and Geoenvironmental Engineering, ASCE 132(8), 1032-051.

Mote, T. I. and So M. L. 2013. Sensitivity of Liquefaction Triggering Analysis to Earthquake Magnitude. AEES 2013 Conference Proceedings.

NZTA The NZ Transport Agency Research Report 553 – "the development of guidance for bridges in New Zealand for liquefaction and lateral spreading effects",

MiBE 2012. Guidelines for repairing and rebuilding houses affected by the Canterbury earthquake. NZ Ministry of Business, Innovation and Employment.

NZS1170.5 Structural Design Actions – Part 5: Earthquake actions – New Zealand

Robertson, PK & Wride, CE 1998, 'Evaluating cyclic liquefaction potential using the cone penetration test', Canadian Geotechnical Journal, vol. 35, pp. 442-459.Seed, H. B., and Idriss, I. M., 1971. Simplified procedure for evaluating soil liquefaction potential. Journal of the Geotechnical Engineering Division, ASCE, 97(9), 1249–1273.

Seed, H. B., and Idriss, I. M., 1971. Simplified procedure for evaluating soil liquefaction potential. Journal of the Geotechnical Engineering Division, ASCE, 97(9), 1249–1273.

Seed, R.B., K, O., Cetin, R.E.S., Moss, A., Kammerer, J., Wu, J.M., Pestana, M.F., Riemer, R.B., Sancio, J.D., Bray, R.E., Kayen, R.E. and Faris, A. (2003), "Recent advances in soil liquefaction engineering: a unified and consistent framework", Keynote Address, 26th Annual Geotechnical Spring Seminar, Los Angeles Section of the GeoInstitute, American Society of Civil Engineers, H.M.S. Queen Mary, Long Beach, California, USA.

Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. M. (1985). "Influence of SPT Procedures in soil liquefaction resistance evaluations." Journal of Geotechnical Engineering, ASCE, 111(12), 1425-1445.

Suzuki, Y., Tokimatsu, K., Koyamada, K., Taya, Y. and Kubota, Y. (1995), "Field correlation of soil liquefaction based on CPT data", Proceedings of the International Symposium on Cone Penetration Testing, Linkoping,Vol. 2, pp. 583-8.

Yang and Wright, 2010

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D., Harder, L.F., Hynes, M.E., Ishiara, 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. II (2001), "Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 127 No. 10, pp. 817-33.Semple, R. 2013 Problems with liquefaction criteria and their application in Australia, Australian Geomechanics Vol 48 No 3 September 2013