# Probabilistic Seismically-induced Liquefaction Evaluation

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Soil liquefaction is a major destructive phenomenon during earthquakes. Many simplified assessment methods for soil liquefaction based on in situ tests, such as the standard penetration test (SPT), the cone penetration test (CPT) and small-strain shear wave velocity (Vs) measurement, have been developed. These are widely used in engineering practice. The basic earthquake parameters that influence the potential for liquefaction are the level of shaking (i.e. peak ground acceleration) and the duration of shaking (i.e. earthquake magnitude). However, it is hard to choose the magnitude for a single “design earthquake” for liquefaction assessment as the level of shaking corresponding to a specified return period is obtained from an integration of all possible earthquake occurrences. Thus, as an alternative to the assessment using a single “design earthquake”, a probabilistic assessment of liquefaction potential may be performed using magnitude distribution. In this paper, the level of shaking and the magnitude distribution for a study site are estimated using probabilistic seismic hazard analysis. From the estimated earthquake parameters, together with soil properties, the probability of liquefaction is investigated.

**Keywords:** probabilistic, liquefaction evaluation

## 1. INTRODUCTION

It is well documented that certain loose saturated soil deposits tend to liquefy when they are subjected to earthquake shaking (Mogami and Kubo, 1953, Hwang et al. 2003 and Cubrinovski and Hughes, 2011). There are two common approaches available for assessment of the likelihood of liquefaction. One is laboratory testing of “undisturbed” samples. The other one is the use of empirical relationships based on correlation of observed field behaviour with various in-situ tests, such as the standard penetration test (SPT) and the cone penetration test (CPT). Generally, laboratory testing is difficult and expensive because of sampling “undisturbed” samples and performing high quality cyclic shear testing or cyclic triaxial testing. The use of empirical relationship is the dominant approach in common practice. The methodology used to evaluate liquefaction is mainly based on the equation for factor safety (FS) against liquefaction:

$$FS = \left(\frac{CRR_{7.5}}{CSR}\right) \times MSF \times K_{\sigma} \quad (1)$$

Where $CRR_{7.5}$ is the Cyclic Resistance Ratio determined for a magnitude 7.5 earthquake. It is estimated using the simplified base curve recommended for calculation of $CRR_{7.5}$ from SPT data developed by Seed *et al.* (1982). *MSF* is the magnitude scaling factor used to adjust the base curve for magnitudes other than 7.5. $K_{\sigma}$ is a correction factor for high overburden stress. *CSR* is the cyclic stress ratio derived from the simplified equation proposed by Seed and Idriss (1982):

$$CSR = 0.65 a_{\max} \frac{\sigma_{vo}}{\sigma'_{o}} r_{d} \quad (2)$$

Where $a_{\max}$ is the peak horizontal acceleration at ground surface which was generated by the earthquake; $\sigma_{vo}$ is the total vertical overburden stress; $\sigma'_{o}$ is the effective vertical overburden stress and $r_d$ is a stress reduction factor. A *FS* value larger than 1 implies the soil liquefaction is triggered. Alternatively, a *FS* value smaller than 1 indicates the soil liquefaction is unlikely. A detailed discussion of the above methodology and procedure may refer to the original documentations, such as Seed et al. (1982) and Youd *et al* (1998) .

Many researchers have contributed to the subject of probabilistic evaluation of liquefaction, such as Yegian and Whiteman (1978), Toprak et al. (1999), Juang et al. (2002) and Ku et al. (2012). The emphasis of most of those studies is placed on dealing with uncertainties that are associated with the simplified method and the database adopted for developing the empirical liquefaction evaluation model. However, to determine the risk of liquefaction, it is important to know not only if liquefaction is possible, but also the probability of its occurrence during the life of a facility. As shown in Equations (1) and (2), apart from insitu soil conditions, the occurrence of liquefaction depends basically on two earthquake ground motion parameters, the level of shaking (i.e. peak ground acceleration) and the duration of shaking (i.e. earthquake magnitude).

The level of shaking is estimated using probabilistic seismic hazard assessment (PSHA) and is obtained from an integration of all possible earthquake occurrences. In practice for general design purposes, a single dominant earthquake at a particular hazard level is selected on its basis of the contribution to hazard by magnitude and distance. In most cases, the single dominant earthquake obtained from deaggregation analysis is the event with a low to medium magnitude at short epicentral distance. However, in liquefaction assessment, a saturated soil deposit is particularly vulnerable to earthquakes with medium to large magnitude at long epicentral distance (i.e. long duration of shaking). Accordingly, using a single dominant earthquake might underestimate the liquefaction risk. In contrast, it might overestimate the liquefaction risk if the regional maximum credible earthquake is adopted in the assessment. Thus, as an alternative to the assessment using a single "design earthquake", a probabilistic assessment of liquefaction potential may be performed using magnitude distribution. Hence, it is suggested that probabilistic assessment of liquefaction potential could generally providea better sense of the risk and lead to better decisions. In this paper, a case study for probabilistic liquefaction assessment based on earthquake parameters is presented. The level of shaking and the magnitude distribution for a study site are estimated using probabilistic seismic hazard analysis. From the estimated earthquake parameters together with soil properties, the probability of liquefaction is investigated.

## 2. SEISMIC HAZARD ASSESSMENT

The study site is located approximately 100kmwest of Newman, Western Australia. It is located between the granite-greenstones of the Yilgarn and Pilbara Cratons in a sedimentary basin. Because of its distance from the relatively seismically active ancient zones of granitic complexes, its associated seismic hazard is considered to be relatively low. Based on the Geoscience Australia earthquake database, and within 300km of the study site there are 62 known events with magnitude larger than M3 that occurred between 1926 and 2013. Amongst these, 4 events occurred at less than 100km from the site.

Gaull *et al*. (1990) produced a seismotectonic model from regional patterns in seismicity together with local geology and tectonics. They identified the Pilbara Craton and Carnarvon Basin as two distinct seismic source zones (See SZ8 and SZ9 in Figure 1).The seismotectonic model AUS5 is provided by Brown and Gibson (2004). The identified seismic source zones located at the study site and surrounding area is presented in Figure 2. Their Pilbara source zone is consistent with Zone 8 of Gaull *et al*. (1990) model. Also their Gascoyne source zone in AUS5 is probably based on geological features despite McCue *et al.*'s (1998) comments that there seems to be little or no connection between Australian seismicity and geology or geophysical anomalies.

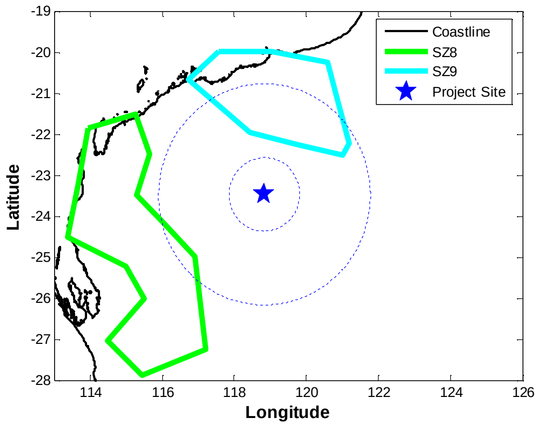



**Figure 1. Location of the study site and earthquake source zones defined by Gaull*et al.* (1990)**

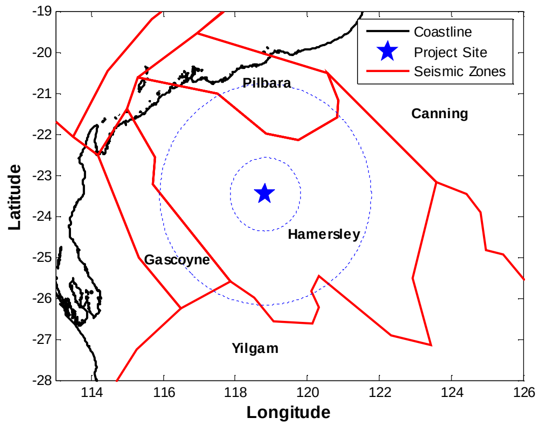



**Figure 2. Location of the study site and earthquake source zones defined by Brown and Gibson (2004).**

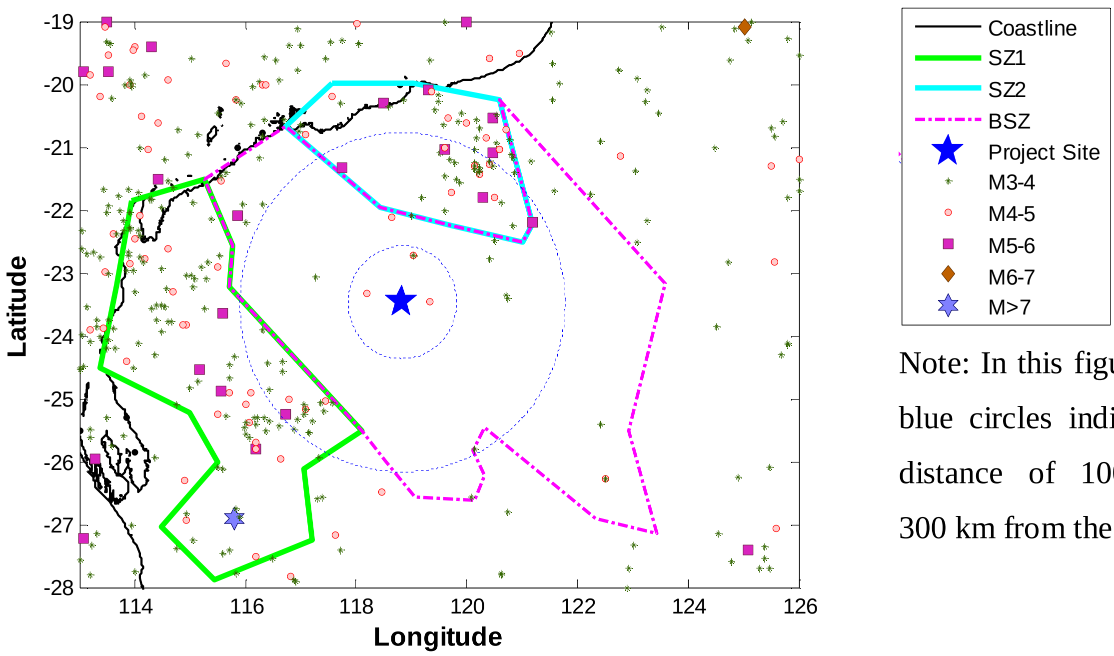
Note: In these figures the blue circles indicate distances of 100km and 300km from the site.

Taking into consideration the seismotectonic models and regional seismicity, some modifications were applied to Gaull *et al*. (1990)'s Zone 9 to include the recent seismic activity adjacent to the zone boundary. The modified Zone 9 and Zone 8 are adopted in this study. The updated seismic zonation is shown in Figure 3. It should be noted that the seismogenic source outside a radius of 300km from the study site are not expected to significantly influence the PGA and are not considered in this study. The Maximum likelihood method is used to obtain the Gutenberg-Richter earthquake recurrence parameters (A and b value) of SZ1 and SZ2. The A and b value for background zone (BSZ) proposed by Gaull *et al*. (1990) is adopted in this study. It is assumed that motion from an earthquake smaller than M4 would not have any effect on structures under consideration. Recent studies based on paleoseismological investigations (Clark et al., 2010 and Estrada, 2013) indicated that the maximum credible magnitude earthquake (Mmax) across the Stable Continental Regions (SCRs) of Australia can reach magnitudes range between Mw 7.0-7.5±0.2. Accordingly, the minimum and maximum magnitudes for each source zone are assumed to be M4.0 and M7.5, respectively. The adopted A and b value for each source zones are listed in Table 1.

**Table 1. A and b value for seismic source zones**

| SZ | A | b |
|---|---|---|
| SZ1 | 2.01 | 0.6 |
| SZ2 | 1.99 | 0.61 |
| BSZ | 1.78 | 1 |

Note: A value for BSZ corresponds to 10,000 square kilometres.




Note: In this figure the blue circles indicate a distance of 100 and 300 km from the site.

**Figure 3.Site Location and Historical Seismicity (1926-2013) with M≥3**

Two ground motion attenuation models (Gaull, 1988 and Liang *et al*., 2008) that were developed using limited southwestern Western Australia (SWWA) data are adopted in the assessment. As the SWWA models are based on limited available data, two models (Atkinson and Boore, 1995 and Toro *et al*., 1997) from central and eastern North America (CENA) are also used in the analysis. The CENA models are adopted because both CENA and SWWA are located in stable continental intraplate region. A weighting of 0.25 is assigned equally to the four ground motion attenuation models used in this study.

The rock site PGA seismic hazard curves for the study site are shown in Figure 4. The total expected PGA level at the site is determined by combining the seismic threats from the defined seismic zones. It is shown that the PGA source contribution for the study site is dominated by BSZ within which the site is located. By deaggregating the hazard by magnitude and distance as shown in Figure 5, the PGA hazard is dominated by the events with low to medium magnitude at short epicentral distance

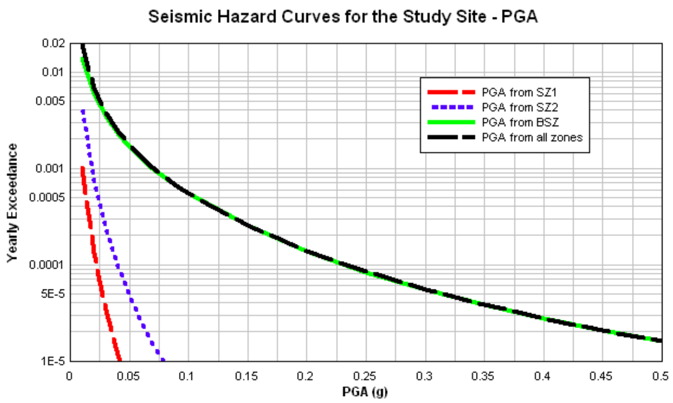



**Figure 4. Seismic Hazard Curve for Rock site PGA**

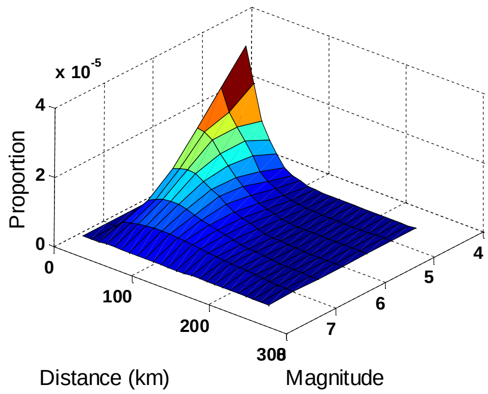



**Figure 5.Magnitude and Distance Contributions to the PGA Hazard at a 2500-year Return Period**

## 3. LIQUEFACTION SUSCEPTIBILITY ASSESSMENT

The site conditions comprise mainly Clayey SAND and Silty SAND. The soil shear wave velocity profile is shown on Figure 6. A site response evaluation was carried out on the basis of the subsurface information described and a site amplification factor of PGA for the study site was estimated to be approximately 1.45. Accordingly, a ground surface PGA of 0.18g corresponding to 2500-year return period was estimated based on the rock site PGA of 0.12g obtained from the PSHA analysis.

A loose sand layer is located approximately 1m beneath the ground surface and the ground water level at the site is close to the ground surface. Based on the in situ soil and ground water conditions, a simplified assessment method for soil liquefaction based on shear wave velocity (Andrus and Stokoe, 2000) is adopted in this study. The analysis indicated that the factor of safety against liquefaction for the saturated loose sand layer is less than 1 when it is subjected to the PGA corresponding to the 2500-year return period and earthquake magnitude greater than M6. Based on the calculated magnitude and distance contributions to the PGA hazard at a 2500-year return period, a curve for the probability of earthquakes of given magnitude or larger is plotted in Figure 7. It is shown that the probability for earthquakes of M6 or larger is approximately 30%, indicating that the probability of liquefaction will be 30% when the sand layer is subjected to seismic hazard corresponding to the 2500-year event.

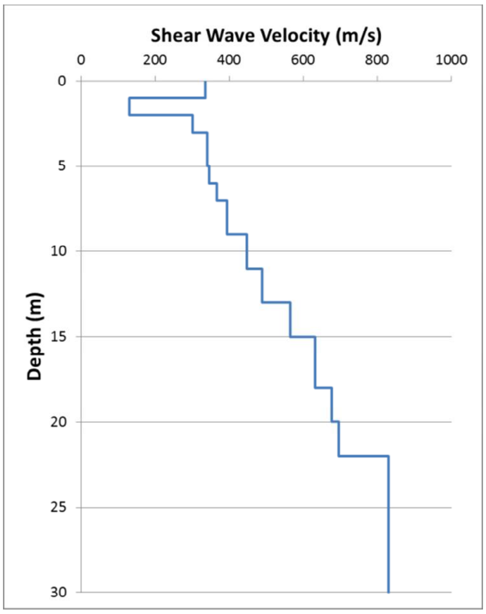



**Figure 6.Shear Wave Velocity Profile**

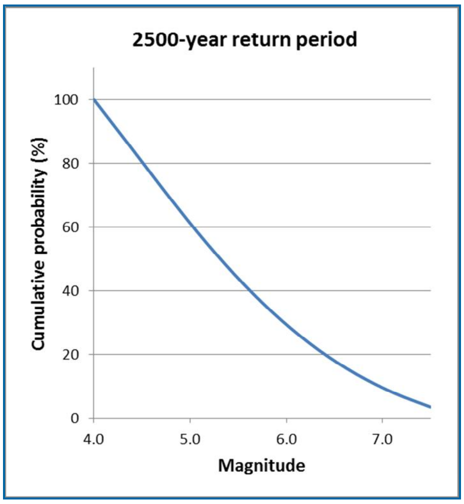



**Figure 7.Probability for Magnitude Exceedance for the PGA Hazard at a 2500-year Return Period**

## 4. CONCLUSION

A case study for probabilistic liquefaction assessment interpreted from PSHA results is presented in this paper. The results have shown the following:

a. It is considered that it is the preferred approach for estimating site specific probability of liquefaction. Using a single "design earthquake" method, either based on dominant earthquake or the maximum credible earthquake, might bias the level of risk of liquefaction.
b. The uncertainties that are associated with the simplified method and the database adopted for developing the empirical liquefaction evaluation model may also be taken into account to derive a reliable and unbiased estimation.

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