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Seismic assessment for industrial facility in Dunedin

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ABSTRACT: A detailed seismic assessment has been carried out for an industrial site in Dunedin. Initial geotechnical assessment, using published seismic data, suggested that the site was prone to liquefaction. As the site is near to the harbour, lateral spreading was also considered to be a risk. Mitigation measures for the lateral spreading were likely to be extremely expensive. A site-specific seismic assessment incorporating direct assessment of the liquefaction and lateral spreading risks was carried out to enable a more realistic assessment of these risks to be obtained, than is available from more commonly used procedures.

1 INTRODUCTION

Beca Carter Hollings & Ferner (Beca) were commissioned to carry out a site-specific seismic and liquefaction hazard assessment for an industrial site in Dunedin. The site is located approximately 40 m from the shore in the port area of the city. The purpose of the assessment was to determine the risk of liquefaction and lateral spreading occurring at this site as a result of seismic shaking.

Previously Beca had carried out a seismic and geotechnical assessment for this site. The previous assessment addressed the issue of the risk of liquefaction and lateral spreading using published seismic data and commonly used methods. The appropriate design philosophy (i.e. acceptable level of risk) for the facility was established using a procedure derived from Whittaker & Jury (2000). A 10% risk of exceedance was chosen. For a facility with a 50 year design life this implies a design return period of 478 years. The assessment concluded that both liquefaction and significant lateral spreading could occur during one in 478 year shaking. As the site contains tanks holding hazardous liquids it was considered important that the containment bund, which surrounds the site, should not be damaged in such an event.

Mitigation measures to address the level of lateral spreading, which was predicted (using a Newmark Sliding Block model) to be approximately 200-400mm, would be extremely expensive. It was considered likely that a site specific seismic study would allow a more rigorous assessment of lateral spreading to be carried out and potentially would allow a reduced level of mitigation. The client accepted this recommendation.

2 GROUND CONDITIONS

The ground conditions encountered in the investigation were as given in Table 1.1 below.

Table 1.1 Ground Profile Encountered in Investigation

Stratum	Description	Thickness	Typical SPT N	Typical CPT Tip Resistance (MPa)
A	Very loose to medium dense fine Sand with soft to firm silt layers	5 m	4	2-4
B	Soft dark grey SILT	2 to 3 m	2	0.5
C	Dense rounded coarse sandy Gravel	-	50+	N/A

Ground water level lies approximately 1 m below ground level.

Layer A appears to be reclamation material, which has been deposited on the original harbour bed (Layer B). Layer A was judged to be potentially liquefiable. Layer B & C were judged to be non-liquefiable.

For the purposes of the study, the portion of Stratum A which lies below the water table (i.e. the potentially liquefiable portion) has been divided into four 1 m thick sub-layers.

3 METHODOLOGY

3.1 Seismic Hazard Assessment

A desktop site-specific seismic hazard study was completed to assess the earthquake hazard at the site. This was achieved by:

1. Assessing the probabilistic hazard i.e. recurrence of Peak Ground Acceleration (PGA) at the site for different magnitude ranges, and
2. Assessing the expected levels of ground shaking at the site from possible earthquake scenario(s) i.e. from Maximum Credible Earthquake (MCE).

The input parameters for the assessment were developed for Beca Carter Hollings & Ferner Ltd (Beca) by the Institute of Geological and Nuclear Sciences Ltd (GNS) Stirling (2002). This included:

- A brief description of the geology and tectonics of the region, including locations of the known active faults.
- An assessment of the expected future seismicity i.e. expected earthquake recurrence rate for the region up to 150 km from the site.
- An assessments of candidates for the MCE for the site, and
- A recommendation for attenuation models for the assessment of ground shaking (PGA).

The seismic model i.e. recurrence of earthquakes in the region and the attenuation model were combined to assess expected recurrence of ground shaking, using HAZARD, Beca in-house developed software.

3.2 Liquefaction Potential

The liquefaction potential of Stratum A (sub layers 1 to 4) was assessed using the prediction relationship given in Technical Report NCEER-97-0022, 1997. The assessment was based on the Cone Penetration Test results and the methods of Robertson & Wride.

The peak ground acceleration (PGA) likely to cause liquefaction for various magnitude earthquakes was assessed, for each layer within the sand. The results of the site-specific seismic hazard assessment allowed an assessment of the return periods of these PGA values to be made. Assembling these return periods enables an overall return period for liquefaction to be assessed.

3.3 **Lateral Spreading Potential**

The lateral spreading potential of Layer A has been assessed using relationships from the Technical Report NCEER-92-0021 'Empirical Analysis of Horizontal Ground Displacement Generated by Liquefaction-Induced Lateral Spreads' by Bartlett & Youd. In this study recorded lateral deformations have been used to generate a generic empirical formula for prediction of lateral spreading. The formula is a function of earthquake magnitude, epicentral distance, thickness of the loose layer, fines content, mean grain size and the free face ratio (the ratio of the free face height to the distance to the free face). Utilising this relationship in combination with the seismic model provides an estimate of lateral spreading at the site. In effect this is an attenuation relationship, which directly relates earthquake magnitude and epicentral distance, for known other parameters as outlined above, to the anticipated lateral spreading which may occur at this site. The results of the seismic hazard assessment can therefore be used to directly predict the extent of lateral spreading for a particular return period.

4 **RESULTS OF STUDY**

4.1 **Seismic Hazard**

Results of the probabilistic site-specific seismic hazard study, in form of the recurrence of Peak Ground Shaking (PGA) for various magnitude ranges, are shown in Figure 4.1.

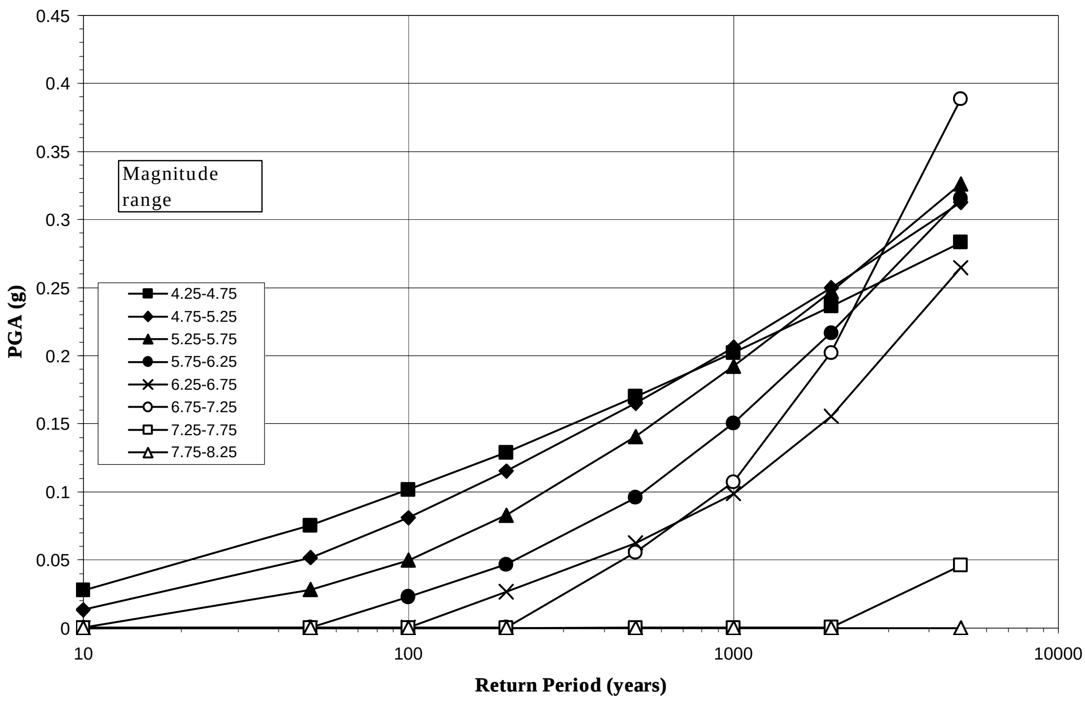

Figure 4.1. Recurrence of PGA for various magnitude ranges (Zhao et al, 1997 att. model, soil)

A comparison of the results of this study with the requirements of the current New Zealand Loadings Code (NZS 4203:1992) and the Draft Australian/New Zealand Loadings Code are shown in Figure 4.2.

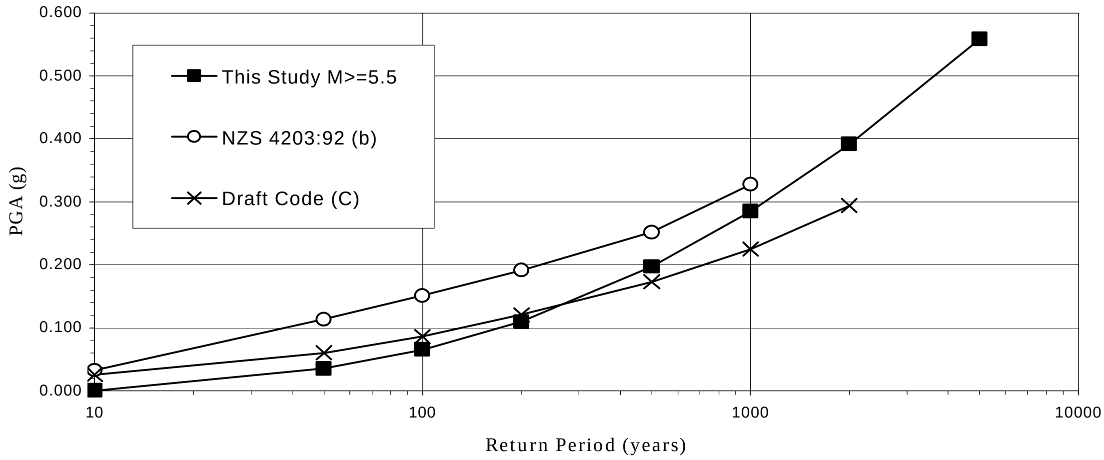

Figure 4.2: Recurrence of PGA This Study (Zhao et al, 1997 att. model, soil) versus Current and Draft Codes

This comparison indicates that the results of this study are in good agreement with draft code for return periods up to 500 years and exceed the recommendations of the code for return period greater than 500 years.

It also appears that for this site, and for the return periods which are commonly of interest in engineering design, the recommendations of the current code are conservative.

Predicted levels of ground shaking from the three candidates for MCE, as recommended by GNS, are summarised in Table 4.1.

Table 4.1. Expected levels of PGA for three candidates for MCE (Zhao et al, 1997 Attenuation model – Soil)

Scenario		Magnitude	Return Period (years)	Epicentral Distance (km)	Peak Ground Acceleration (g)		
					Median	Median + 1 s (16% exceedance probability)	Median + 1.5 s (7% exceedance probability)
1	Akatore Fault	7.1	3000	20	0.43	0.73	0.94
2	Hyde Fault	7	15000	50	0.15	0.26	0.34
3	Distributed Earthquake	5.5	3000	5	0.22	0.38	.49

It can be seen, from Table 4.1, that although Scenarios 1 and 2 represent events of a similar magnitude, predicted levels of PGA for Scenario 1 are higher due to the shorter epicentral distance. The assessed return period for Scenario 2 is significantly longer than for the other event. It is important to recognised that the return period of the event is not the same as the return period of the shaking that results from the event.

Scenario 3, although causing reasonable ground shaking at the site, is not likely to be critical for the assessment of susceptibility to liquefaction due to the short duration of shaking i.e., small number of cycles characteristic for this size of event.

Scenario 1 event, with a corresponding PGA of 0.43g (the median prediction) could be, therefore, considered as the Maximum Credible Earthquake. Although it would be possible to consider this scenario using values of PGA with the low exceedance probability i.e., median + 1.5 σ, we believe that the probability of this level of shaking from this event would be too low to be considered as a reasonably credible scenario event for this project.

4.2 Liquefaction

Susceptibility to liquefaction of the soils at the site was assessed using the following procedure (summarised in Table 4.2):

1. Soil stratum A (below the water table) was subdivided into four sub layers, each 1 m thick.
2. Peak Ground Acceleration (PGA) likely to cause liquefaction (NCEER-97-0022, 1997) for various magnitude ranges was assessed for each sub layer (Column A Table 4.2). Earthquakes of magnitude of less than 5.5 were excluded as they are unlikely to cause liquefaction due to the short duration of shaking caused by these events. However, if low magnitude events were included the result would not change significantly, as the frequency of liquefaction from these events would be very low.
3. The return period associated with the PGA to cause liquefaction for a particular magnitude was assessed from Figure 4.1 (Column B).
4. The Expected Annual Frequency was calculated by inverting the return period (Column C).
5. The return period for liquefaction for a particular layer is the inverse of the sum of the expected annual frequencies for each magnitude level.

The procedure and the results for the existing soils are shown in Table 4.2.

Table 4.2. Assessment of susceptibility to liquefaction for existing soil

Magnitude	Sub layer											
	1			2			3			4		
	A	B	C	A	B	C	A	B	C	A	B	C
M (+- 0.25)	PGA (g)	Return Period (years)	1/RP	PGA (g)	Return Period (years)	1/RP	PGA (g)	Return Period (years)	1/RP	PGA (g)	Return Period (years)	1/RP
5.5	0.375	5000	0.0002	0.275	3100	0.00032	0.225	1650	0.00061	0.275	3100	0.00032
6	0.3	4550	0.00022	0.22	2000	0.00050	0.18	1500	0.00067	0.22	2000	0.00050
6.5	0.22	3700	0.00027	0.16	2100	0.00048	0.13	1600	0.00063	0.16	2100	0.00048
7	0.19	1850	0.00054	0.137	1300	0.00077	0.11	1000	0.00100	0.137	1300	0.00077
7.5	0.15	100000	0.00001	0.11	100000	0.00001	0.09	100000	0.00001	0.11	100000	0.00001
SUM			0.00124			0.00208			0.00291			0.00208
Return Period of liquefaction (years)			806			481			344			481

It is apparent, from Table 4.2, that sub layer 3 is more susceptible to liquefaction than other three layers. The return period of occurrence of liquefaction for this layer is shorter than the acceptable value of 500 years. Layers 2 and 4 could probably be assessed as being on the margin of acceptability. Ground improvement in the form of vibro stone columns beneath structures and tanks was therefore recommended.

The reassessed minimum return periods of liquefaction in Layer A using the recommendations of Priebe, if the above improvement is completed, are shown in Table 4.3.

Table 4.3. Assessment of susceptibility to liquefaction for improved soil

Magnitude	Sub layer											
	1			2			3			4		
M (+- 0.25)	PGA (g)	Return Period (years)	1/RP	PGA (g)	Return Period (years)	1/RP	PGA (g)	Return Period (years)	1/RP	PGA (g)	Return Period (years)	1/RP
5.5	0.49	20000	0.00005	0.36	7000	0.00014	0.29	4000	0.00022	0.36	7000	0.00014
6	0.39	8000	0.00013	0.29	4250	0.00024	0.23	2500	0.00040	0.29	4250	0.00024
6.5	0.29	6000	0.00017	0.21	3600	0.00028	0.17	2300	0.00043	0.21	3600	0.00028
7	0.25	2750	0.00036	0.18	1800	0.00056	0.143	1350	0.00074	0.18	1800	0.00056
7.5	0.2	100000	0.00001	0.14	50000	0.00002	0.12	12000	0.00008	0.14	50000	0.00002
SUM			0.00072			0.00124			0.00185			0.00124
Return Period of liquefaction (years)			1398			806			540			806

Again, the critical layer is sub layer 3, which, however, after suggested improvement, satisfies the adopted criteria for minimum resistance to liquefaction.

References on the assessment of susceptibility to liquefaction published to date, such as the NCEER-97-0022 1997 used in this study, are not in a form that would allow for the refinement to determine the extent of liquefaction at the site at the stated return periods (e.g. localised sand boils or full liquefaction).

4.3 Lateral Spreading

Bartlett and Youd (1992) provide an empirical relationship for the assessment of the amount of lateral spreading as a function of earthquake magnitude, epicentral distance, thickness of the loose layer, fines content, mean grain size and the free face ratio (the ratio of the free face height to the distance to the free face). Utilising this relationship in combination with the seismic model, in a similar manner as in the assessment of recurrence of Peak Ground Acceleration, provided an estimate of recurrence of lateral spreading at the site. Results of the analysis, which incorporate the statistical uncertainty in the relationship, are shown in Table 4.4.

Table 4.4. Recurrence of Lateral Spreading (Bartlett and Youd, 1992 model)

	Return Period (years)							
	100	200	300	400	500	1000	2000	5000
Lateral spreading (mm)	0	0.4	5	25	44	178	412	1660

5 CONCLUSIONS

Conventional liquefaction assessment typically entails choosing an appropriate design return period, which relates to the design life of the structure and the acceptable level of risk. A Peak Ground Acceleration relating to the chosen return period is obtained from published sources. A representative earthquake magnitude is then assumed and the liquefaction risk at the site is assessed. This paper outlines a methodology, which allows a more rigorous assessment of the risk of liquefaction to be carried out by directly utilising the seismic data. For this assessment the predictions of liquefaction using the methodology outlined were not too dissimilar to those found from the more traditional approach.

As in all methods of assessment of liquefaction and lateral spreading the results must be used with care, fully recognising the likely accuracy.

Conventional methods of assessing the likely extent of lateral spreading such as Newmark Sliding Block Method, tend to provide a crude estimate at best. The relationship proposed by Bartlett & Youd used in conjunction with the hazard analysis as outlined above has the advantage of being based on the measurement of actual lateral spreading events. It also allows the direct use of site-specific seismic data and the adoption of a probabilistic approach.

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