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An application of liquefaction hazard evaluation in urban planning

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ABSTRACT: In planning for a new town at Pegasus Bay, 30 km north of Christchurch, the risk of seismic liquefaction was considered and evaluated. URS New Zealand Limited evaluated the likelihood of liquefaction within the soils beneath the areas proposed for residential development. The liquefaction assessment was developed using the results of a geological site investigation and a seismic hazard evaluation based on a Probabilistic Seismic Hazard Assessment.

This paper describes how the results of recent earthquake hazard research were incorporated into the analyses and an assessment of design criteria for liquefaction mitigation in the proposed residential subdivision.

The study demonstrates that liquefaction could occur at the site during the 50 year design life of the houses and commercial buildings, and that current New Zealand codes and standards require that countermeasures be provided during the subdivision design phase. It also demonstrates that the Alpine Fault is not the most critical earthquake source in terms of the liquefaction effects predicted for the site. An assessment of the mitigation options demonstrates that it is possible to construct the subdivision with a lower risk of liquefaction than comparable sites in eastern Canterbury.

1 INTRODUCTION

Pegasus Bay town is a proposed new 341 Ha residential development on land located immediately east of Woodend in North Canterbury (Figure 1). Liquefaction-susceptible materials have been identified in the area. Liquefaction is commonly observed during large earthquakes and occurred at nearby Kaiapoi following the magnitude 6.9 Cheviot earthquake in 1901. Many other instances have been recorded around New Zealand.

The study combined the use of a recognised Probabilistic Seismic Hazard Assessment (PSHA) (Stirling et al. 1999) with subsurface investigations to characterise the geotechnical properties of the soils underlying the site. The PSHA model is a comprehensive assessment of the seismic hazard in Canterbury. The results of this model were used in our study to predict the likely peak ground accelerations at Pegasus Bay Town and associated magnitude and epicentral distance for a range of earthquake return periods.

The site investigations comprised five cable tool boreholes (URS 1 to 5) drilled to depths of 15 to 25 metres with SPTs undertaken at 1 metre spacing and bag samples of the materials collected at 0.5 metre intervals for logging. Cone Penetrometer Tests (CPT) were undertaken at 16 locations around the site (CPTs 1 to 16). The depth of investigation by CPT varied from about six to thirteen metres. Figure 2 illustrates the drillhole and CPT locations.

This data served as inputs to the detailed liquefaction assessments. This included assessments of the magnitude of both total settlements and lateral spreading.

Liquefaction potential was assessed using both the Cyclic Stress Ratio (CSR) method (Youd & Idriss 1997) and the Davis and Berrill (1982) energy methods. The use of the two different methods served as a check on the robustness of the liquefaction prediction.

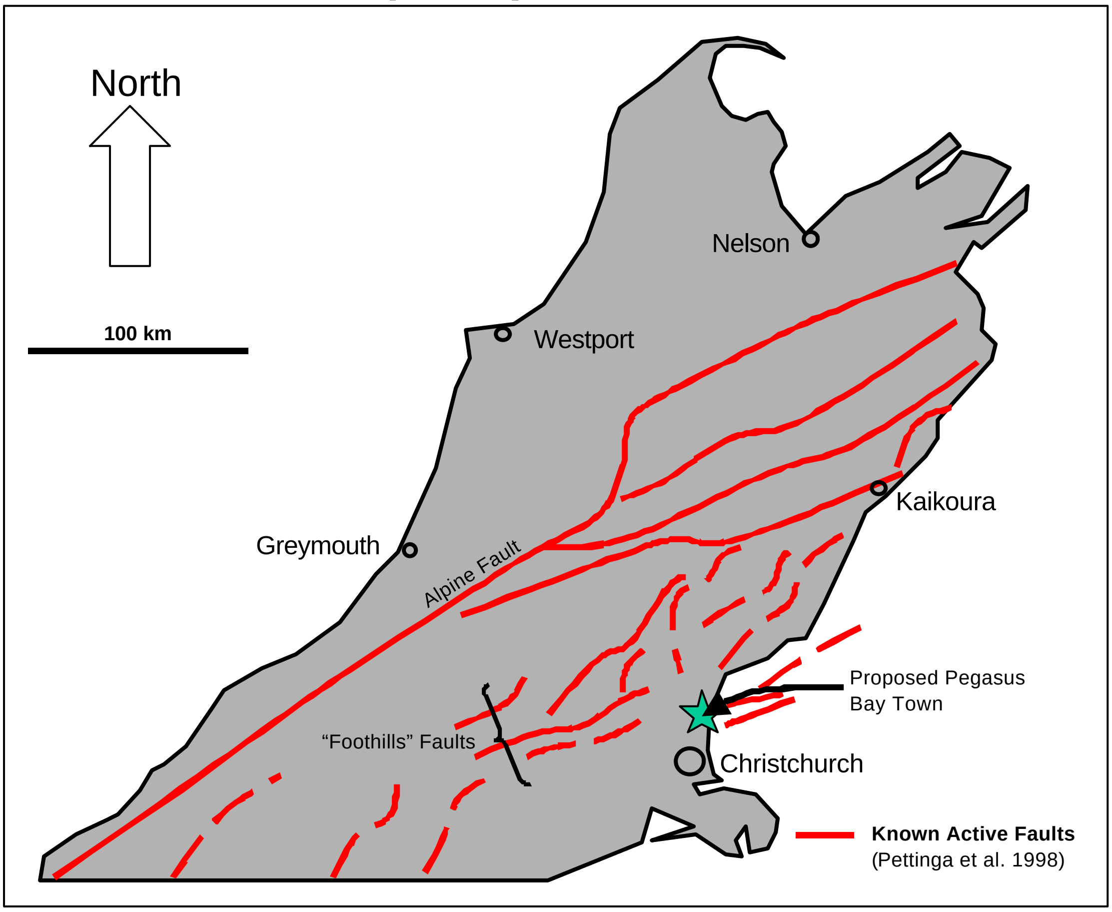

Figure 1. Map showing the location of the proposed Pegasus Bay town and known active faults.

2 SITE SETTING AND GEOLOGY

Pegasus Bay Town is located on coastal sediments within about 3 km of the existing coastline of Pegasus Bay. The northern end of the site is located about 3 km from the Ashley River. Sediments that reflect both the coastal marine and fluvial depositional environments underlie the site.

The upper 20 metres of the geological profile comprise Springston Formation and Christchurch Formation sediments consisting of sand and gravel dominated materials with minor amounts of silt and peat (Brown 1973) as shown in Figures 2 and 3. The Springston Formation comprises post-glacial fluvial sediments and the Christchurch Formation comprises post-glacial coastal marine sediments. Below about twenty metres depth a gravel stratum of about 30 metres thickness is present. This gravel stratum is referred to in Christchurch as the Riccarton Gravel.

A typical east-west cross-section through the site is shown on Figure 3. All bores and CPTs encountered sand and gravel dominated strata throughout the site. Minor silt and peat were also encountered. Sands are typically fine to medium grained with a typical D_{50} of 0.2 mm.

An arcuate scarp feature on the eastern margin of the dune sands is suggestive of lateral spreading. The geometry suggests that material may have flowed out to the east onto the lower ground. The model for lateral spreading predicted failure of this dune slope during shaking equivalent to that experienced during the 1901 Cheviot earthquake.

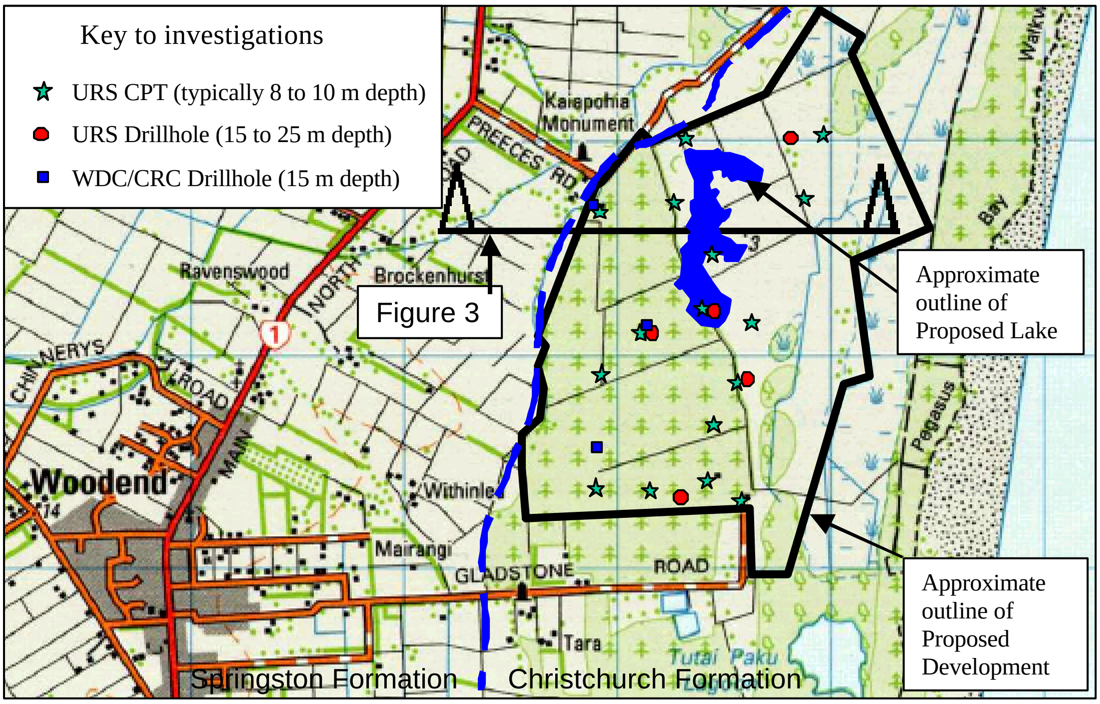

Figure 2. Layout of proposed Pegasus Bay town showing locations of investigations. Dashed blue line separates outcrop of Springston Formation (to left) from Christchurch Formation (to right) (after Brown 1973). Basemap is sheet M35 1:50 000 (Crown copyright reserved).

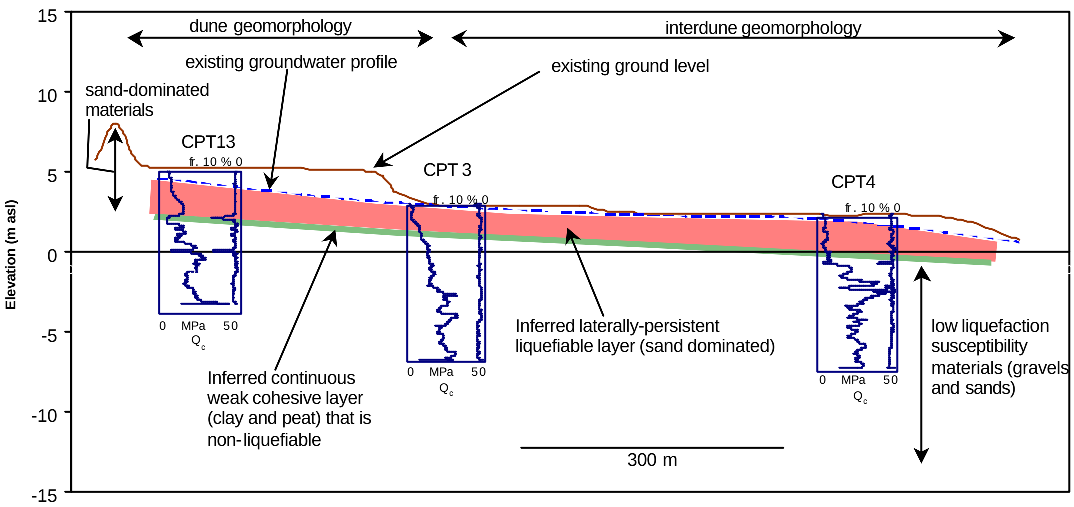

Figure 3. East-west cross section through site showing subsurface materials and inferred liquefiable layer. Alignment of cross-section shown on Figure 2.

3 REGIONAL SEISMIC HAZARD

The liquefaction prediction methods that we have used require inputs of peak ground acceleration, earthquake magnitude and epicentral distance for a given earthquake. Three earthquake scenarios have been chosen to represent a range of potentially damaging earthquakes predicted for the site (Table 1). The PSHA predicts increasing peak ground accelerations for corresponding increasing return periods as shown in Figure 4.

We have selected the 150 and 475 year return periods as two of the earthquake scenarios for our study. Stirling et al. (1999) predict peak ground acceleration of 0.28 g and 0.44 g respectively for these return periods. These accelerations have been predicted by “sampling” all known earthquake sources (faults) and “distributed seismicity” based on the historical earthquake record. For the 150 and 475 year return period earthquake scenarios we have selected a combination of earthquake magnitude and epicentral distance that fit the regional seismotectonic model. Known faults in the vicinity of the site (e.g. Porters Pass-Amberley Fault Zone) have earthquake magnitudes predicted by Stirling et al. (1999) in the range 6.5 to 7.2 and we have chosen M_w 7.2 for both the 150 and 475 year return period earthquake scenarios. The epicentral distances for these earthquake scenarios are shown in Table 1.

Table 1. Earthquake Scenarios used in Liquefaction Study

Earthquake Scenario	Magnitude (M_w)[1]	Epicentral Distance[2] (km)	Peak Ground Acceleration (g)
150 year return period earthquake	7.2	25	0.28
475 year return period earthquake	7.2	10	0.44
Alpine Fault Earthquake	8.0	110	0.18

1. M_w is moment magnitude.
2. Distance between earthquake source and Pegasus Bay Town

We have used the most recent seismic attenuation relationship for New Zealand (Stirling et al. 2000) to select appropriate values of earthquake magnitude and epicentral distance for the peak ground accelerations predicted by the PSHA.

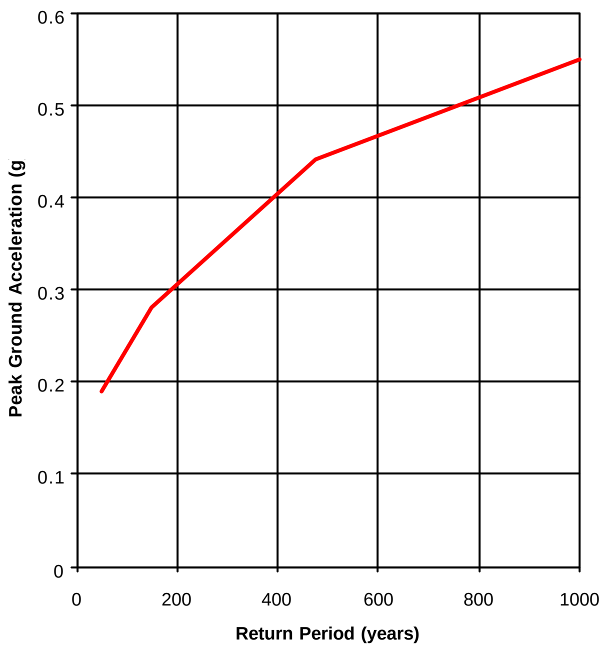

Figure 4. Peak ground acceleration versus return period curve predicted by Stirling et al. (1999) for Kaiapoi

The third earthquake scenario represents the special case of an Alpine Fault earthquake, which is recognised as the source of the largest earthquake (~M_w 8.0) likely to affect the site. The Alpine Fault is also a special case because of the high probability of an Alpine Fault earthquake occurring during the next 50 years (estimated to be 68%, Yetton et al., 1998). This fault may be nearing the end of its earthquake cycle, meaning that further movement is becoming increasingly likely as time passes. This cycle is estimated to last between 100 and 280 years, and the last known movement was around AD 1717 (Yetton et al., 1998).

We have estimated the peak ground acceleration for an Alpine Fault earthquake using Stirling et al. (2000), and have used the 84^{th} percentile value for added conservatism. Although the attenuation model is based on data in the Magnitude 5 to 7.5 range, this model can be reasonably extrapolated to a M_w 8 Alpine Fault earthquake (Stirling et al., 1999).

The peak ground accelerations predicted by Stirling et al. (2000) are for Class B sites. Pegasus Bay town site is believed to be a Class C site having a depth of gravels to bedrock of several hundred meters. However Stirling et al. (1999) reported that their peak ground acceleration values for Class B and C sites were similar. Therefore we have assumed for this study that their PGA values are applicable. Total site response analyses incorporating amplification of ground motion were not carried out to confirm this assumption.

4 PREDICTED LIQUEFACTION EFFECTS

The liquefaction assessment, using both prediction methods, demonstrated that it is possible that strong earthquake motions could affect the site during the design life (50 years) of the houses and commercial buildings and that it would be prudent to implement mitigation measures to prevent or minimise damage to these structures. Without mitigation measures, the most likely liquefaction effects would be:

- Lateral spreading of wide corridors adjacent to a proposed lake shoreline and eastern wetland margin affecting both buildings and buried services. Estimates of the lateral spreading without mitigation measures are shown in Table 2.
- Possible settlement induced damage to buried services and residential buildings. Predicted total settlements are given in Table 3 for the Energy Method.

Table 2. Estimates of Lateral Spreading without Mitigation Measures

Distance from lake shore (m)	**Estimated magnitude of lateral spreading for selected earthquake scenarios (m)**		
	Alpine fault	**150 year**	**475 year**
15	1.6	1.7	9.6
250	0.3	0.3	1.5

Table 3. Estimates of Total Settlement Ranges without Mitigation Measures (Energy Method)

	Earthquake Scenario		
	Alpine Fault	**150 year**	**475 year**
Settlement Range (mm)	6 - 68	6 - 80	27 – 135

Unmitigated, lateral spreading could result in collapse or severe damage to buildings in the 475 year event which would contravene the requirements of NZS 4203:1992 (the loadings code for structural design). While it is unlikely that all of the site area will experience lateral movement because of natural variations in soil properties it is impractical to confidently define those areas which will not be affected. Following the precautionary principle, mitigation of lateral spreading was recommended in all those areas adjacent to the proposed lake and eastern margin.

Settlement damage, in contrast, is a more benign source of liquefaction damage on the Pegasus Bay Town site. This is because the predicted total settlements are less than those observed to cause structural damage in major earthquakes (>100mm). When total settlements are less than 100 mm, damage to houses is generally very light or non-existent (Ishihara and Yoshimine, 1992).

5 LIQUEFACTION MITIGATION DESIGN CRITERIA

The liquefaction assessment demonstrated that there was a requirement for designers to consider the impact of liquefaction during design of the services and buildings needed to develop the subdivision. Specific criteria for design of the liquefaction mitigation measures were therefore required.

New Zealand does not have any explicit standards dealing solely with the design of liquefaction mitigation. Therefore, following a review of the New Zealand Codes and the California guideline Special Report 117 (DMG, 1997), the criteria given in Table 4 were selected as appropriate for design of the mitigation measures. They are believed to meet the requirements of The Building Act 1991 and the relevant New Zealand building standards. These criteria are believed to provide for a level of risk likely to be acceptable to all project stakeholders and provide for an adequate level of building performance during strong earthquake shaking.

6 DESIGN EARTHQUAKE RETURN PERIOD

The selected design earthquake return period is 150 years. This criterion was not based on the larger magnitude Alpine Fault event as the liquefaction assessment had shown that the impact of the closer Canterbury Foothills earthquakes was likely to have at least as great an impact in terms of liquefaction at Pegasus Bay as the more distant but larger Alpine Fault earthquake.

The use of a 150 year return period also reflects the criteria adopted for estimating the ultimate limit state design loads on New Zealand bridges in the 1980's prior to development of the current loadings code, NZS 4203:1992. At that time the use of a 150 year return period was adopted to reflect a balance between the risk of damage in moderate earthquakes and the low probability of loss of life if a bridge collapsed in a higher return period earthquake. This philosophy is also considered appropriate for a residential subdivision as the risk to life from liquefaction effects is typically low despite the financial damage that can result.

When NZS 4203:1992 was introduced the 475 year return period had been adopted as the ultimate limit state design criteria reflecting USA practice for multi-storey structures where the loss of life potential from collapse was much greater than for bridges. Use of this higher standard for mitigation design is not considered necessary because of the much lower threat to life from liquefaction in the proposed subdivision.

Further, it is likely that the mitigation measures designed for a 150 year return period event will minimise the extent of damage caused by the more severe 475 year return period. Consequently, the ultimate limit state requirements of NZS 4203:1992 i.e. preventing total structure collapse and loss of life, should also be met.

The serviceability limit state requirement of NZ 4230:1992 will also be exceeded, as the corresponding return period is only 10 years. Meeting the serviceability limit state criteria alone is not considered an appropriate level of protection against building damage. As the 10 year return period earthquake is unlikely to produce liquefaction it would effectively mean that no mitigation would be required.

7 SETTLEMENT AND LATERAL MOVEMENT CRITERIA

The criteria selected for analysis and design of the liquefaction mitigation measures for the large residential areas at Pegasus Bay Town are set out in Table 4. These criteria serve to indicate when mitigation measures should be considered based on analytical results, as well a target to be achieved through design for them. The design criteria for commercial or critical structures is not specifically covered in this table. Rather it was expected that designers would develop criteria specific to the purpose of these structures.

The maximum settlement criteria was set at 100 mm as being a practical value for residential building design in light of the observations by Isihara and Yoshimine (1992) that only minor damage normally occurs up to this level. The maximum lateral movement criteria was set at 250 mm as a practical level given that the design intention for mitigation measures in a residential subdivision should be to minimise the movement to a level where the risk of damage to services and structures was acceptable to the stakeholders without mitigation measures becoming financially onerous. This criteria recognises the need to obtain a balance between the cost of the mitigation measures and the benefit obtained.

The factor of safety criteria were adopted directly from Special Report 117 (DMG 1997), and reflect the different consequences of lateral spreading and settlement, the uncertainties in their predictions, and the expected differences in effects between loose to medium dense and dense soils. The Factor of Safety in terms of the CSR method is defined as CSR_{liq}/CSR_{EQ} where CSR_{liq} is the cyclic stress ratio required to generate liquefaction (i.e. the soil's resistance to liquefaction) and CSR_{EQ} is the cyclic stress ratio generated by the anticipated earthquake at the site. The factor of safety for the Energy Method is defined in a similar fashion.

It is anticipated that the final selection and application of these criteria will be dependent on the choice of mitigation option. For instance direct treatment of the liquefiable soils may readily achieve the factor of safety criteria whereas buttressing of an area of potential lateral spreading may allow some internal movement when the retained soils liquefy, even though the factor of safety against gross movement is greater than 1.0.

Table 4. Liquefaction Assessment Criteria to be Met for Residential Areas

Consequence of Liquefaction	**Maximum Permanent Ground Movements**[1,2]	**Factors of Safety**[3,4]	
		$(N_1)_{60}$ <15 (clean sand) **qc_{1cs} < 7 MPa**	**$(N_1)_{60}$ > 30 (clean sand)** **qc_{1cs} > 14 MPa**
Flows and Lateral Spreading (horizontal movement)	250 mm	1.3	1.0
Settlement (vertical movement)	100 mm	1.1	1.0

1. Predicted movement during design earthquake (150 year return period).
2. Differential settlements for design should be taken as one half of the maximum settlements.
3. Based on Special Report 117 (DMG 1997)
4. The $(N_1)_{60}$ and qc_{1cs} are standardised soil parameters determined from analysis of the Standard Penetration Test and Cone Penetration Test data respectively.

8 MITIGATION OPTIONS

Various options were identified that would prevent unacceptable levels of liquefaction damage to the subdivision in the event of a strong earthquake. Environment Canterbury and its engineering consultants reviewed these options and accepted the feasibility of implementing them during detailed design. It is expected that once the subdivision is constructed it will have a much lower risk of liquefaction damage than other subdivisions, which have been developed on similar soils in the eastern Waimakariri District, and in eastern Christchurch. Selection of the actual mitigation options will be carried out during detailed design of the subdivision.

Lateral spreading could be prevented by either an area wide treatment of the insitu soils to improve their liquefaction resistance e.g. vibro-compaction or by construction of a bund of specially strengthened ground near the proposed lake to act as a shear key. This shear key would be designed to resist the lateral spreading of the ground behind it by replacing the natural buttress removed as a result of lake excavation. This work will be carried out during the early stages of subdivision construction.

For most of the site the magnitude of liquefaction induced settlement is expected to be less than the level (100 mm) at which significant liquefaction damage begins to occur to houses. However several low-cost options are available to significantly reduce the low risk which exists. These include piling of the houses, excavating and recompacting the soils below the house and providing for additional reinforcement of the house foundations and concrete floors. These simple measures have proven effective at protecting private dwellings in liquefied areas in US earthquakes (Day 1999).

Lifeline services such as sewers and stormwater pipes are vulnerable to liquefaction. The risk of damage can be substantially reduced by ground improvement e.g. excavation and recompaction of loose soils around manholes or providing flexible connections between rigid structures and the connecting pipelines.

9 CONCLUSIONS

The following conclusions can be drawn from this study:

- The Pegasus Bay area contains liquefiable materials, and liquefaction has been observed during earthquakes in the past.
- The Alpine Fault is a major source of strong earthquake shaking but for the Pegasus area its expected liquefaction effects are less than those predicted for a 150 year return period event sourced in the Canterbury Foothills.
- Rational design criteria which meet the intent of current engineering design codes can be developed for the design of liquefaction remediation works for large scale residential areas.
- A number of feasible engineering options are available to reduce the risk of liquefaction damage in the developed subdivision to much lower levels than are likely to exist for other subdivisions founded on similar soils in the eastern Waimakariri District and Christchurch.

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