R E P O R T

Pegasus Town Infrastructure Geotechnical Investigations and Assessment Report

Prepared for

Infinity Investment Group

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Introduction

Pegasus Town Limited commissioned URS New Zealand Limited (URS) to undertake the geotechnical investigations of the proposed Pegasus Town site to provide sufficient geological and geotechnical information for design and construction.

The site is located 23 km north of Christchurch adjacent to SH1, 2 km east of Woodend (Figure 1), situated between Preeces and Gladstone Roads. Approximate dimensions of the site are 1.5 km (E-W) by 2.0 km (N-S), encompassing about 250 hectares.

The site includes two distinctly different areas. The eastern half of the area is currently planted in semi-mature pine trees. East of the plantation the site is typically lower-lying pasture and is used for cattle grazing.

Proposed Development

The proposed development has been designed by Beca and the current study assumes the development layout provided to URS 7 October 2005. The development includes both residential and commercial building with associated infrastructure.

The proposed development includes excavation of an artificial lake. The lake will be excavated to about RL-1.5 (approximately 4.5 m below existing grade for most of the lake area) and water level will be at about RL1.4. A wetland area to the east of the residential development (the Eastern Conservation Management Area) will include a large swale along the entire eastern border of the site. This will involve excavation of up to about 2 m below existing grade. Excavated material is to be used to raise the elevation of surrounding land above flood levels.

Access to the site will be from a roundabout on SH1, crossing the proposed Mapleham development. Investigations have been undertaken in this area to characterise the general subsurface conditions for road design.

Just beyond the northern boundary of the site is the Kaiapoi Pā, located north of the southern most corner of Preeces Road. It is recognised that settlement extended around the Pā site itself and numerous archaeological finds have been made within the area. Some of these finds are located within the site boundary. Consultation with the Runanga is outlined in Section 2.

Scope of Report

Previous investigations of the site have indicated a number of potential issues associated with these developments, mainly concerning liquefaction and lateral spreading due to earthquakes.

This report presents the findings of the current geotechnical investigation. This includes a description of the site geology, a summary of the investigations and laboratory testing undertaken. The report also summarises our interpretations of the collected data with respect to design of roads and underground services, suitability for residential building foundations, suitability as fill material, liquefaction and lateral spreading hazards and potential remedial measures to address those hazards.

Consultation with Upoko Runanga of Tuahuriri

Geotechnical investigations at the proposed site were undertaken with full knowledge that the area is one of cultural significance due to its proximity to the Kaiapoi Pā (Upoko Runanga of Tuahuriri).

During meetings with Runanga representatives URS indicated the proposed investigation methods and their locations. An open invitation was given to the Runanga or one of their representatives to observe all investigations.

Proposed investigation locations were checked against the plan of known archaeological sites in the area to ensure that these sites were not disrupted (Figure 2).

More than 24-hours notice was given to the Runanga prior to the commencement of investigations, as specified in Resource Consents issued by Environment Canterbury (ECan). Unfortunately in all cases the Runanga was unable to provide a representative.

Prior to commencement of subsurface investigations each location was checked for the presence of any surficial evidence of archaeological material. Numerous middens were noted around the site. No investigations were undertaken in these areas.

Hand-dug inspection pits were completed at all test locations to a depth of 1.0 m below ground level. No material of archaeological significance was recognised.

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3.1 Regional Geology

North Canterbury is generally characterised by a complex of thick alluvial fans that extend from the foothills of the Southern Alps to the coast. Pegasus Town is located between the fans produced by the Ashley and Waimakariri Rivers.

Throughout the coastal North Canterbury area, the upper ~20 m of the subsurface strata is dominated by sands and silts, with minor gravel layers. These deposits result from accumulation of either flood “overbank” alluvial sediments or dune, estuarine or beach marginal marine sediments. Based on logs of wells drilled in the area, Pegasus Town is underlain by a thickness of at least 200 m of gravel dominated alluvial fan deposits. Thin deposits of fine grained material occur within this sequence, resulting from deposition during periods of intermittent higher relative sea level (Brown and Weeber, 1992).

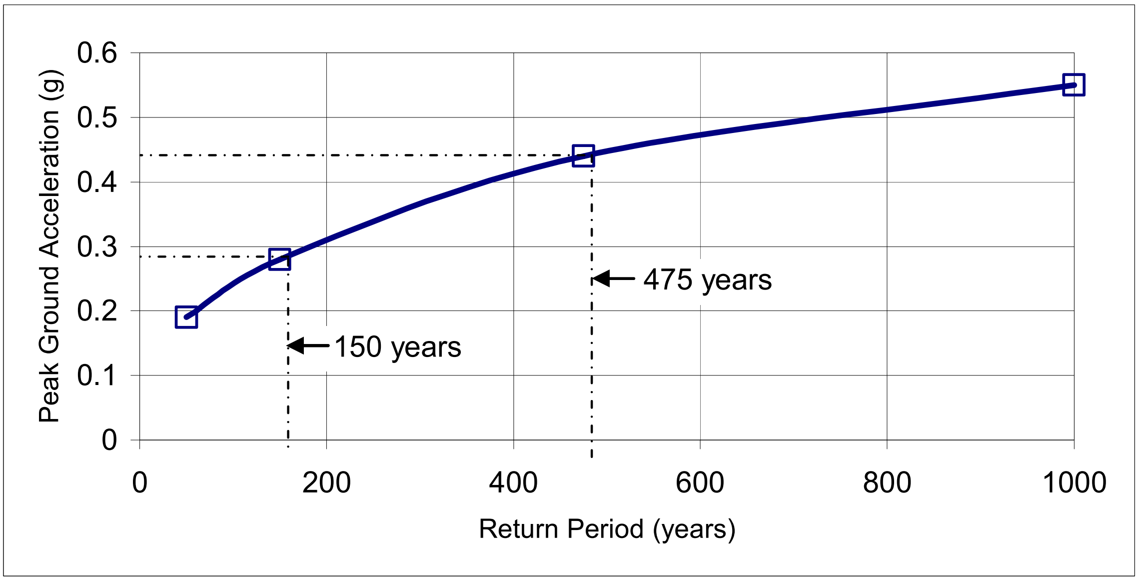
Pegasus Town is also located within 1 km of the modern coastline. The coastal geological environment is complex as a result of lateral movement of meandering drainage channels, coastline movement and areas of impeded drainage. During the period 14,000 years to 6,500 years before present (b.p.), sea levels rose steadily, but have remained stable since 6,500 years b.p.. Since then the coastline has migrated approximately 2 km eastward along much of the North Canterbury coast, depositing a strip of marine sediments (Schulmeister and Kirk, 1996).

3.2 Seismic Hazard

Canterbury is an area of relatively high seismic hazard owing to its close proximity to the boundary between the Australian and Pacific plates and related active faults in the foothills of the Southern Alps. The structure capable of generating the largest earthquakes in terms of energy release of importance to the town site is the Alpine Fault. This can generate earthquakes of M_{max} 8, and located at least 110 km from Pegasus Town. The closest fault with a recognised surface trace (the Springbank Fault - M_{max} 7.1) is approximately 16 km to the west of the site (Stirling et al. 2002). Additional faults have been recognised cutting the offshore sediments to the east of the site. Between these and the Alpine Fault a large number of additional faults with surface traces have been recognised in the foothills of the Alps and are thought capable of generating earthquakes in the range M_{max} 7 to 7.5.

The University of Canterbury and Institute of Geological and Nuclear Sciences have undertaken a comprehensive review of the seismic hazard in Canterbury in recent years. These studies have been published in two reports prepared for Environment Canterbury (Pettinga *et al.* 1998, and Stirling *et al.* 1999), and more recently as a seismic hazard study of New Zealand (Stirling *et al.* 2000, 2002). These reports include variations of a Probabilistic Seismic Hazard Assessment (PSHA) that predicts ground acceleration resulting for different time intervals for major urban centres in Canterbury. The results presented in Stirling *et al.* 1999 are shown in Figure 3 as a hazard curve.

Figure 3: Probabilistic Seismic Hazard Curve for Pegasus after Stirling *et al.* 1999



4.1 Geomorphology

4.1.1 Surficial Geology

Reference to Geological Map Sheet S76, Kaiapoi (Brown, 1973), indicates that the surficial geology over the site is separated into two distinct zones, separated by an approximately north-south boundary (Figure 4). To the west of the site, alluvial overbank and channel deposits (Springston Formation) dominate, while the eastern part of the site is underlain by dune and inter-dune marine deposits (Christchurch Formation).

The alluvial deposits to the west of the site are predominantly over-bank flood sediments, but also includes gravel-dominated channel fill deposits. Surface topography is relatively flat and generally falls toward the Pegasus site. Dunes at the western margin of the Pegasus site form a barrier to drainage (as discussed in Section 4.1.2).

The western area of the Pegasus site comprises fixed and semi-fixed dune deposits (Christchurch Formation (cs)). The dunes are up to about 5 m in height and elevations in this area range between 2.4 m and 7.1 m. At the southern boundary of the site these dune deposits measure over 1.0 km wide. At the northern boundary the deposits narrow to about 0.1 km wide. A prominent dune ridge forms the western boundary of the dune deposits. This area of the site is referred to in this report as the “high dunes” and is currently planted in pine trees (Photo 1).

The eastern part of the site is an area referred to as the “low dunes”. These deposits are also part of the Christchurch Formation (cs). The low dune deposits are widest at the north end of the site where they are approximately 0.8 km wide. At the southern end of the site the low dune deposits merge into the high dunes. These deposits are characterised by extensive areas of relatively planar surfaces mixed with irregular dunes, typically less than 1 m in height. This area is typically less than 2 m in elevation.

To the east of the low dunes is an area of drained inter-dune; coastal swamp and lagoonal deposits (Christchurch Formation (cp)). These are generally flat lying, bounded on their eastern side by active dunes parallel to the present-day Pegasus Bay coastline (Photo 2).

Combined width of the high and low dune areas is a generally about 1.0 km.

Photo 1 – Aerial View of High (Plantation) and Low Dune Areas

Photo 2 – View towards the South – Low Dunes Area.

4.1.2 Surface Drainage

Since occupation by European Settlers in the late 1800's significant changes to the flow of surface water have occurred.

Taranaki Stream originally drained along an abandoned channel of the Ashley River, located adjacent to the southwest boundary of the site. In the late 1800's the stream was diverted northward from its natural course and now drains into a natural wetland located within the western part of the site (Figure 4). The outlet level of the wetland has recently been artificially raised to increase water levels.

Geomorphology within the area of the wetland indicates the presence of numerous 'blind' channels leading into the wetland. These 'blind' channels may have originated from erosion adjacent to natural springs.

Beyond the wetland Taranaki Stream drains northward into the site of the Kaiapoi Pā along another section of artificial channel. Within the Pā site Taranaki Stream converges with a channel draining from along the western edge of the high dune deposits, along the intersection between alluvial and dune deposits. Water within the channel is spring fed from gravels beneath the overlying fine-grained overbank deposits, with 'blind' channels in the source area (Figure 4). The presence of higher permeability material just beneath the channel invert was confirmed during a discussion with the current land-owner, Mr J M Scott, who indicated that a shallow pit had been dug within the channel to increase water supply for irrigation. Higher permeability material had been encountered within a few metres beneath the channel invert.

In the eastern part of the site drainage has been significantly altered by construction of 2 m deep surface drains. Water levels within the surface drains appear to be high upstream of a section of piped culverts (Figure 4). This is likely to be a result of blockage within these pipes.

5.1 Subsurface Data from Previous Studies

In early 2000 Beca Carter Hollings & Ferner Ltd (BECA) completed twenty-six drillholes as part of a Liquefaction Study (August, 2000) undertaken for the Waimakariri District Council and Environment Canterbury (ECan). The study covered an area extending north from the Waimakariri River to just north of the Ashley River, bounded by the Pegasus Bay coastline on the eastern side and extending to a line 1 km west of and parallel to SH1.

Three detailed study regions were investigated including one at the Pegasus Town site. Four drillholes were completed along the high dune section of the site, in a north-south alignment. These four drillholes extended to 15 m depth below ground level with standard penetration tests (SPT) completed at 1.5 m intervals. Laboratory testing was completed on selected samples recovered from these drillholes.

A more detailed Seismic Liquefaction Study of Pegasus Town was completed by URS for Southern Capital Limited in July 2001 (URS, 2001). The investigations included five drillholes (URS-1 to URS-5) drilled to a maximum of 25 m below surface level and sixteen cone penetration tests (CPT-1 to CPT-16) generally to 15 metres below ground level. Laboratory testing was undertaken on selected samples from drillholes.

As part of the current investigation, information held within the Ecan Wells database was reviewed. Nine drillhole logs were obtained and although these drillholes have not been logged specifically for geotechnical purposes, the logs give an indication of deeper subsurface material distribution, useful for groundwater modelling.

The locations of previous investigations at the site are shown on Figure 5, and copies of logs are presented in Appendix A.

5.2 Current Investigations

The objective of the current investigation was to provide sufficient information for the design and construction of all infrastructure associated with the proposed development. This requires an evaluation of the distribution of the various geological materials and an assessment of the likely liquefaction susceptibility of materials, their distribution, and potential mitigation measures.

Previous investigations were completed to depths of 15 to 25 metres below ground level. These investigations indicated that the majority of geotechnical issues that could affect the development would relate to the nature and behaviour of the upper 10 metres of deposits. Accordingly the current field investigations were limited to a maximum depth of 10 metres.

All investigations were completed between late February and early May 2005. Test locations were surveyed using GPS and are shown in Figure 5.

Table 1 indicates the number of tests completed for each of the five investigation methods used. Specific details regarding investigations are shown in tabular form in Appendix B.

Some proposed investigations could not be completed as a result of site access issues. At the time of the current investigations the majority of the central "high dune" area of the site was covered in pine plantation. Trees in the northern half of the plantation were generally small, but densely planted, restricting access to many of the proposed test locations. Tracks were cut through the plantation to a number of the test locations using an excavator. However, due to the amount of clearing required some test locations were not accessed. The coverage of subsurface investigations is considered appropriate for the current stage of the project.

Table 1 – Investigations Completed During 2005 Investigation

Test Type	Completed
Cone Penetration Test (CPT)	97
Test Pits (TP)	49
Direct Push Dual Tube (PT)	13
Piezometer (PZ)	11
Infiltrometer (INFT)	9

5.2.1 Cone Penetration Test (CPT)

A total of 97 Cone Penetration Tests (CPTs) were undertaken during these investigations. The locations are shown in Figure 5. A rig provided by Site Investigation Ltd was utilised to collect detailed soil strength data from the near surface materials. The CPT is a truck-mounted rig that can quickly probe a soil profile in sands and fine-grained soils. Geological material type can also be inferred from the CPT and this information has been used to create the site geological model. The upper 1 m of each test was conducted in disturbed material where the hand dug inspection pit was undertaken to check for archaeological remains. Materials were observed during excavation of the inspection pit, but soil parameters were not measured.

Transportation of the CPT rig in the plantation area was difficult as the rig weighs 14 tonnes and is 2WD. A 20-ton excavator was required to pull the rig in and out of most test locations through the loose sand (Photo 3).

Investigations were undertaken using both standard CPT cone and the piezocone, which also measures pore water pressure in the soil. Piezocone tests were predominantly completed in the lake excavation area to increase detail of subsurface geological data collected. A number of piezocone dissipation tests were completed, in particular where low permeability materials were encountered.

An interpretation of the CPT data (both 2001 and 2005) was completed by Site Investigation Ltd using the Robertson and Campanella (1983) relationship between cone tip resistance (q_c) and friction ratio (f_R). This interpretation method outputs estimated material types in which tests were completed. No normalisation for overburden pressure was applied by Site Investigation Ltd during this interpretation. Test data and interpretations are shown in Appendix A (previous investigation data) and Appendix C (CPT data from current study).

Photo 3 – CPT rig in High Dunes Area

Calibration

CPT-063 (standard cone) was completed adjacent to CPT-15 (undertaken in 2001) to confirm equipment calibration. The two CPT results are very similar results (Figure 6).

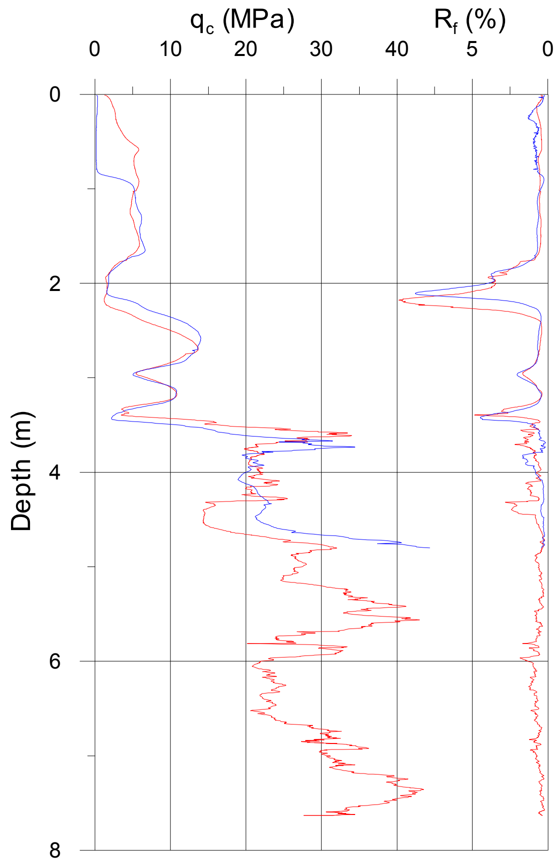
5.2.2 Test Pits (TP)

A total of 49 test pits (TP) were completed between 14 April to 19 April 2005. Locations are shown in Figure 5. These were dug using a wheeled (4WD) 10-ton excavator with a maximum depth reach of 4.5 m. Most test pits were abandoned, as a result of caving and collapse of the excavation walls, before the maximum depth was reached. Test pit logs are presented in Appendix D.

Two separate test pits (TP-100 and TP-101) were completed separately in the area of the proposed lake to observe excavatability, groundwater inflow and excavation stability. These test pits were not part of the geotechnical investigations contract, but were logged by an engineering geologist and logs are presented in Appendix D.

Figure 6: Comparison between CPT-15 (URS, 2001) and CPT-063 (2005)

[CPT-15 in red; CPT-063 in blue]



5.2.3 Direct Push Dual Tube (PT)

Direct Push Dual Tube (PT) holes were completed at 13 locations to sample soils and install piezometers to allow measurement of groundwater levels. Locations are shown in Figure 5. Details of the piezometer installations are presented in Appendix I.

5.2.4 Piezometer Installation (PZ)

Nine direct push dual tube locations (PT-004 to PT-013 inclusive) were completed to install piezometers (PZ-001 to PZ-009) but without soil sampling. Locations are shown in Figure 5. Details of the nine completed installations are presented in Appendix E.

5.2.5 Infiltrometer Tests (INFT)

Infiltrometer tests (INFT) were completed adjacent to the nine piezometers (PZ-001 to PZ-009) to measure the infiltration rate of water under very low head. This testing was performed to assess the likely rate of stormwater infiltration for stormwater disposal design. The test procedure and results are described in Appendix F.

5.3 Groundwater Conditions

Groundwater level measurements were made after the completion of CPT tests, when possible, and in test pits when groundwater was present. These levels are noted in Appendix B.

Standpipes and piezometers were installed at 22 locations during the current investigations. Thirteen were installed in direct push dual tube holes (PT-holes) completed across the alluvial plain with another three in the lake excavation area. Another nine piezometers were installed at the location of the proposed stormwater disposal sites (PZ-installations).

Five piezometers from previous investigations were located and measured during the duration of the investigations. Measurements made during the investigation period are noted in Appendix I.

Groundwater levels across the site range from 1.0 m to 5.6 m below ground level (RL 1.4 to 6.7 m), with a groundwater gradient to the east.

5.4 Laboratory Testing

5.4.1 Methodology

Samples for laboratory testing were taken from test pits and by direct push dual tube sampling methods. Results from previous investigations are included with the results from the current investigation.

Selected samples were sent to Central Testing Services (CTS) in Alexandra for analysis. Five test methods were used to categorise the materials at the site.

- Particle Size Distribution (PSD) (sieve)
- Particle Size Distribution (PSD) (hydrometer)
- Atterberg Limits
- Californian Bearing Ratio (CBR)
- NZ Standard Compaction

A full schedule of testing completed and laboratory results is presented in Appendix G.

5.4.2 Particle Size Distribution (PSD)

Particle size distribution is the main test characterising material properties. As part of the current investigations a total of 31 tests have been undertaken on samples collected from throughout the site. All samples were wet sieved and 15 had additional hydrometer analysis undertaken where greater than 10%

of the whole sample passed the finest sieve (63µm). All PSD results from previous investigations at the site (Beca, 2000 & URS, 2001) have also been reviewed and included in our material characterisation (Section 6).

5.4.3 Atterberg Limits

Liquid and plastic limits were assessed for soils that behave in a plastic manner. Almost all soils that were sampled were judged to behave as non-plastic materials. A total of 4 samples have been tested.

5.4.4 NZ Standard Compaction

Standard compaction tests evaluate the way the soils respond to compactive effort. Soils that have a significant fines content have an optimum moisture content, the moisture content at which the maximum density can be achieved for a standard compactive effort. A total of 18 samples have been submitted for the NZ Standard Compaction test.

5.4.5 Californian Bearing Ratio (CBR)

The CBR is a test characterising the strength of subgrade soils that is commonly used for designing road pavements. A total of 15 samples from the proposed road alignments have been submitted for the CBR test.

6.1 Method of Data Interpretation

Most investigations were completed at the site the CPT. This technique rapidly collects high quality geotechnical information but does not readily allow for the retrieval of subsurface samples.

Interpretation of the CPT data (2001 and 2005) was completed by Site Investigation Ltd using the Robertson and Campanella (1983) relationship between cone tip resistance (q_c) and friction ratio (f_R). Using this method of interpretation material types within which the tests are completed can be determined. No normalisation for overburden pressure was applied. Interpretation of the test data is shown in Appendix A (2001) and C (2005). Additional interpretation has been undertaken by URS to further classify the soil type and strength. The data interpretation is presented in Appendix C.

Figure 7 illustrates the distribution of materials in Sections A to M. The position of these sections is shown on Figure 5.

This information, combined with drillhole data, was used to create maps of the site indicating:

- CPT Refusal Depth – 2005 CPT Data Only (Figure 8 (RL); Figure 9 (BGL)),
- Reduced Level on Top of Sandy Gravel/Gravelly Sand Unit (Figure 10), and
- Reduced Level on Top of silty fine Sand/Silt/Clay/Peat Unit (Figure 11).
- Isopach Map Indicating Cumulative Thickness of silty fine Sand/Silt/Clay/Peat Unit (Figure 12).

6.2 Distribution of Subsurface Materials

The near surface geology has been divided into four principal units according to material properties and depositional environment. The units are:

a) Clean ***fine to medium sand*** is the most common unit, directly underlying most of the site. These materials are interpreted as mainly being dune or beach deposits.

b) ***Silt, clay and peat*** occurs as thin layers beneath the central and eastern parts of the site. These materials represent only a minor portion of the near surface geology. They are interpreted to be swamp or lagoonal deposits.

c) ***Interbedded silt and sand*** is a distinctive unit that underlies the alluvial plains west of the site. The interbedded silt and sand unit is interpreted to be an alluvial flood deposit. It is a weak unit characterised by low q_c values.

d) ***Sandy gravel and gravelly sand*** makes up a relatively minor proportion of the near-surface geology, but is more common throughout the site below about 5 m depth.

6.2.1 Fine to medium SAND

The properties of the fine to medium sand unit are summarised in Table 2.

Gradings confirm that this is a well sorted (uniformly graded) fine to medium sand, with typically less than 10 % silt or clay (Figure 13).

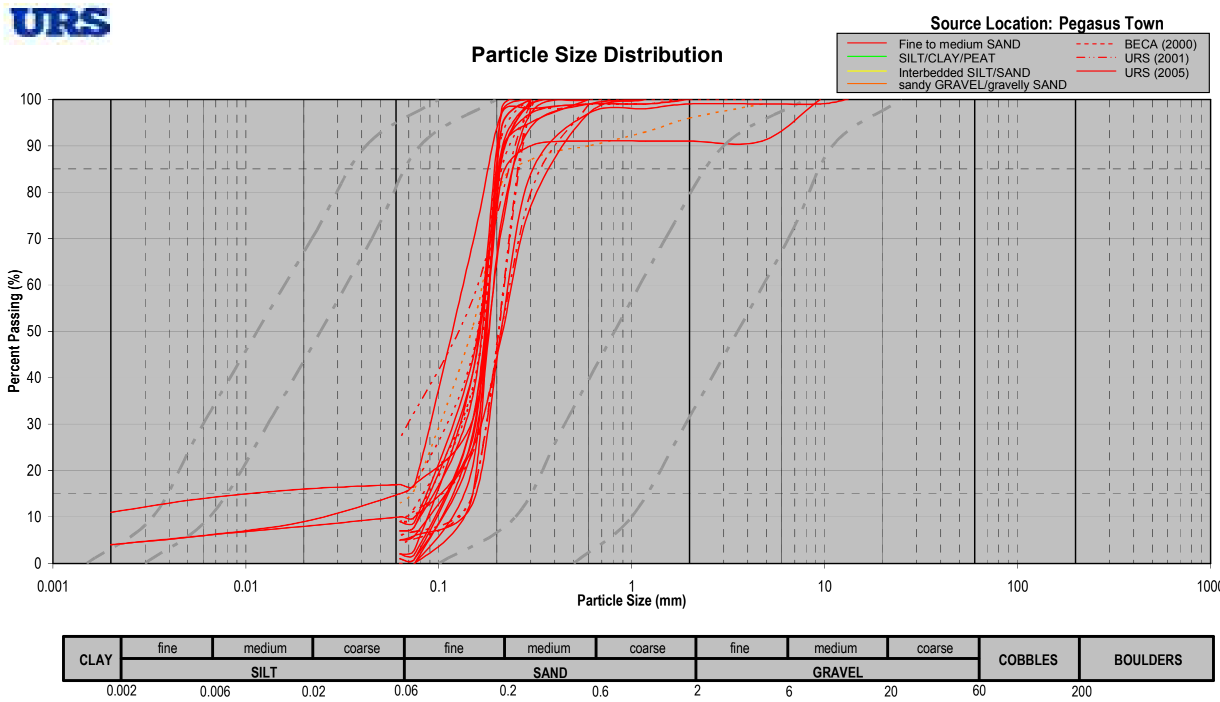
The fine to medium sands are typically loose to moderately dense (q_c values range from 5 to 25 MPa), suggesting relative density in the range 70 to 80 %.

The results suggest that the deposits have undergone varying degrees of consolidation. Normally consolidated deposits are encountered in much of the high dunes area, seen as a constant increase in q_c with depth. Normally consolidated sands underlain by over-consolidated sands are found in many CPTs completed in the low dunes area, indicating that these materials were formerly buried beneath a significant additional thickness of material (approximately 4-5 m), which has subsequently been removed by erosion.

Table 2: Summary of Material Properties for Fine to Medium Sand Unit

Test	No. Tests	Mean	Range	Comments
GRADING				Mean grain sizes (min-max range)
% gravel (> 2 mm)	24	1 %	0 – 9 %	D_{85} = 0.2 mm (0.17-0.37)
% sand (0.06-2 mm)		95 %	73 – 100 %	D_{50} = 0.18 mm (0.13-0.22)
% silt (0.002-0.06 mm)		3 %	0 – 10 %	D_{15} = 0.12 mm (0.01-0.16)
% clay (<0.002 mm)		1 %	0 – 10 %	
IN SITU PROPERTIES				
Moisture Content (%)	11	Above GWL 2.7%	0.2 – 5.1%	Average moisture content calculated for combined samples.
	8	Below GWL 29.2%	22.7 – 42.7%	
NZ STD COMPACTION (< 19mm)				
Maximum Dry Density (t/m^3)	8	1.60	1.55 – 1.65	-
Optimum Moisture Content (%)	8	17.8%	14.0 – 20.5%	
PERMEABILITY				
Prugh Method (CIRIA C515)	-	9.0 x 10^{-5}	1.5 x 10^{-4} – 5.0 x 10^{-5}	Derived from gradings.
CALIFORNIAN BEARING RATIO (CBR)				
CBR	6	19.0	3.5 – 25.0	Two tests completed using NZ Std Compaction and Optimum Moisture Content.
Maximum Dry Density (t/m^3)		1.58	1.48 – 1.62	One test completed had a CBR equal to 3.5 with a density of 1.48 t/m^3. It is assumed that this test is not representative of this material due to the low compaction achieved.

Figure 13: Particle Size Distribution for Fine to Medium Sand Unit (24 samples)



6.2.2 Silty fine SAND, SILT, CLAY and PEAT

Beneath the central and eastern part of the site, thin layers of cohesive soils were encountered in most CPTs, distributed within fine to medium sands. Laboratory testing shows that these cohesive materials are typically silty fine sands and fine sandy silts, with up to about 20% clay. Atterberg Limits indicate a low plasticity and CPT q_c values indicate a soft or firm consistency (average 2 to 4 MPa). Laboratory test results are summarised in Table 3 and gradings are presented in Figure 14. A plasticity index chart for this unit is presented as Figure 15.

These materials were not encountered in every CPT and the layers were not continuous across the boundary between the low and high dunes. To the east, the silt layers are at a lower RL, and are thinner than below the central high dunes area. Figure 11 indicates the distribution of these materials across this part of the site. The distribution of silt layers is further described below.

Thick peat layers were not encountered, however thin silt layers rich in organic material were encountered at some locations, particularly in the eastern part of the site.

High Dunes Area

Generally the top of the silt deposits occurs at between RL 1.5 to 3.7 m with higher elevations at the northern and southern ends of the high dunes area. Cumulative thickness of these deposits is generally between 0.5 and 1.5 m (Figure 12). Multiple silt layers were encountered by some CPTs.

Low Dunes Area

Generally the highest elevation of any significant silt layer is relatively consistent at between RL 0.5 and 1.0 m indicating a degree of lateral continuity. However, cumulative thickness is less than 0.5 m. Two zones in the northern part of this area indicate an absence of any low permeability material.

Table 3: Summary of Material Properties for silty fine Sand, Silt, Clay and Peat Unit

Test	No. Tests	Mean	Range	Comments
GRADING				Mean grain sizes (min-max range)
% gravel (> 2 mm)	6	0%	0%	D_{85} = 0.18 mm (0.1-0.2)
% sand (0.06-2 mm)		60%	20 – 77 %	D_{50} = 0.08 mm (0.009-0.17)
% silt (0.002-0.06 mm)		35%	19 – 60 %	D_{15} = 0.01 mm (0.001-0.012)
% clay (<0.002 mm)		5%	4 – 21 %	
IN SITU PROPERTIES Moisture Content (%)	1 6	Above GWL 5.2% Below GWL 30.3%	- 23.1 – 44.9%	Average moisture content calculated for combined samples.
ATTERBERG LIMITS (< 425 µm)				
Liquid Limit, LL (%)		31%		One sample tested
Plastic Limit, PL (%)	1	21%	-	⇒ CL
Plasticity Index, PI (%)		10%		
NZ STD COMPACTION (< 19mm)				
Maximum Dry Density (t/m^3)	3	1.69	1.54 – 1.82	-
Optimum Moisture Content (%)	3	18.0%	14.0 – 25.0%	
PERMEABILITY				
Typical value based on gradings.	-	5.0 x 10^{-7}	1.0 x 10^{-8} – 1.0 x 10^{-7}	Estimated from Gradings

Figure 14: Particle Size Distribution for silty fine Sand, Silt, Clay and Peat (5 samples)

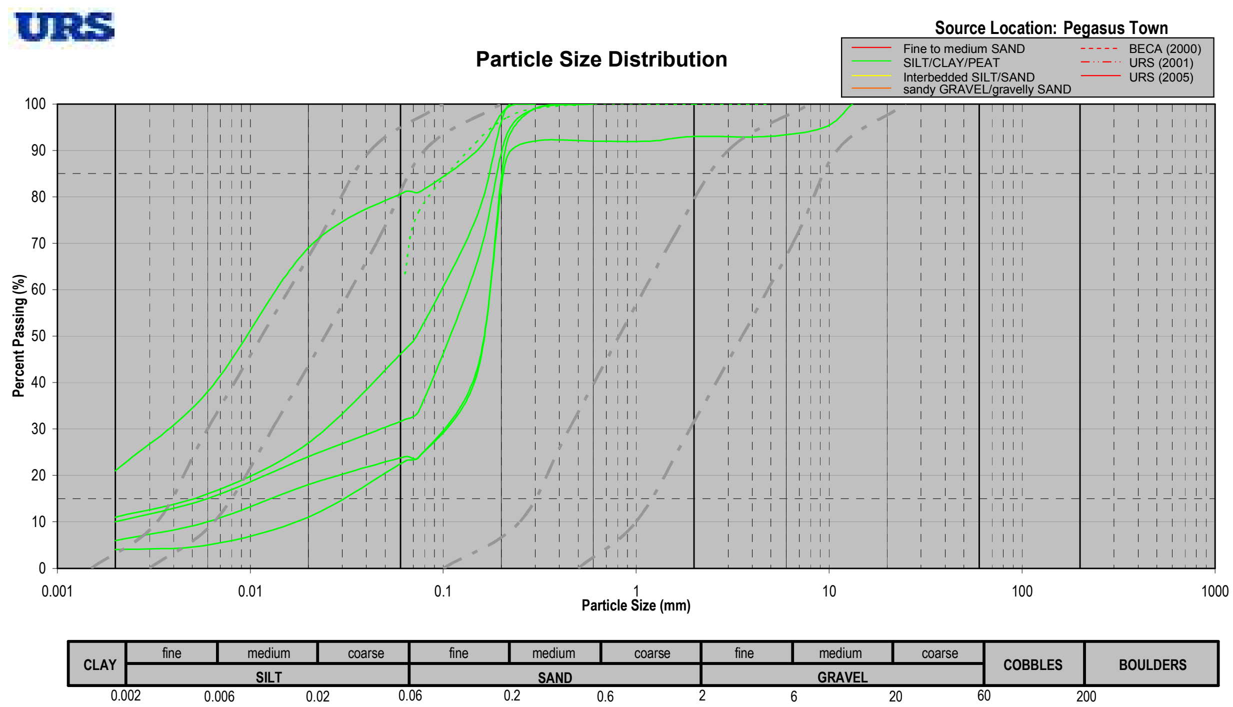
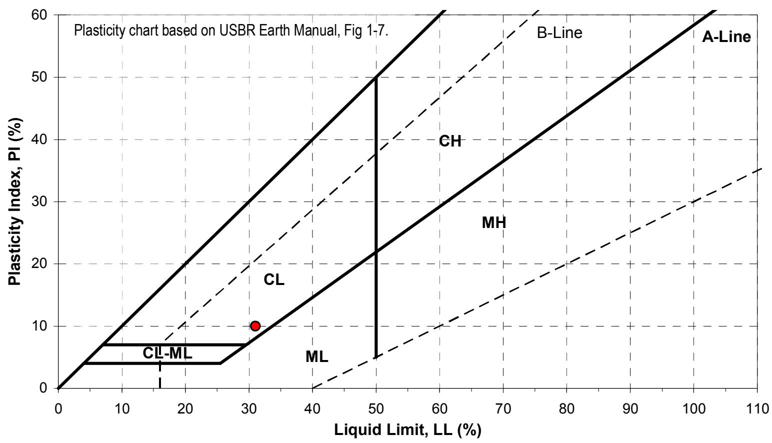


Figure 15: Plasticity Chart for Silt, Clay and Peat (1 sample)



6.2.3 Interbedded SILT and SAND

The Mapleham block is underlain by a thick sequence of silt-dominated sediments interpreted to represent alluvial overbank deposits (flood derived). CPT's indicate that this sequence typically has layer thicknesses ranging from 0.1 m to 1.0 m.

Total thickness of this unit ranges from 3.0 to 6.0 m over the majority of the area, appearing to become thinner toward the east. Thickness increases up to 12.0 m thick in the western corner of the site, at the location of the proposed intersection with SH1.

These materials have q_c values in the range 0.5 to 6 MPa indicating very loose or loose condition for non-cohesive layers and soft or firm condition for cohesive layers.

Gradings indicate that this unit predominantly comprises silt and fine sand, with 5 to 20% clay (Figure 16). Laboratory test results are summarised in Table 4. A plasticity index chart for this unit is presented as Figure 17.

Table 4: Summary of Material Properties for Interbedded Silt and Sand Unit

Test	No. Tests	Mean	Range	Comments
GRADING				Mean grain sizes (min-max range)
% gravel (> 2 mm)	7	0%	0 %	D_{85} = 0.12 mm (0.08-0.18)
% sand (0.06-2 mm)		42%	25 – 65 %	D_{50} = 0.06 mm (0.018-0.09)
% silt (0.002-0.06 mm)		45%	29 – 55 %	D_{15} = 0.003 mm (0.001-0.017)
% clay (<0.002 mm)		13%	6 – 19 %	
IN SITU PROPERTIES				
Moisture Content (%)	6	Above GWL 6.9%	3.0 – 12.1%	Average moisture content calculated for combined samples.
	4	Below GWL 25.0%	15.0 – 33.5%	
ATTERBERG LIMITS (< 425 µm)				
Liquid Limit, LL (%)		26%	24 – 27%	Three samples tested.
Plastic Limit, PL (%)	3	22%	20 – 24%	⇒ CL – ML
Plasticity Index, PI (%)		4%	0 – 7%	
NZ STD COMPACTION (< 19mm)				
Maximum Dry Density (t/m^3)	7	1.75	1.69 – 1.82	-
Optimum Moisture Content (%)	7	16.2%	15.5 – 18.0%	
PERMEABILITY				
Typical value based on gradings.	-	5.0×10^{-7}	1.0×10^{-8} – 1.0×10^{-7}	Estimated from Gradings
CALIFORNIAN BEARING RATIO (CBR)				Two tests completed using NZ Std Compaction and Optimum Moisture Content
CBR	6	9.0	6.0 – 14.0	
Maximum Dry Density (t/m^3)		1.76	1.70 – 1.82	

Figure 16: Particle Size Distribution for Interbedded Silt and Sand Unit (7 samples)

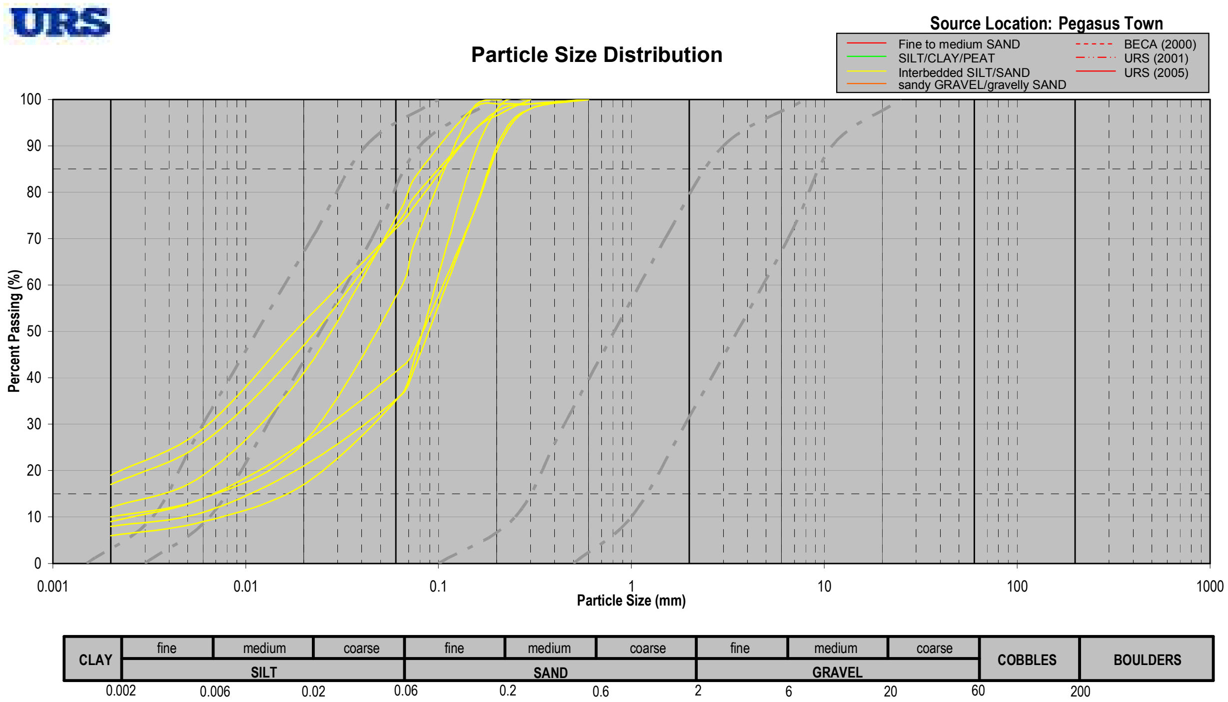
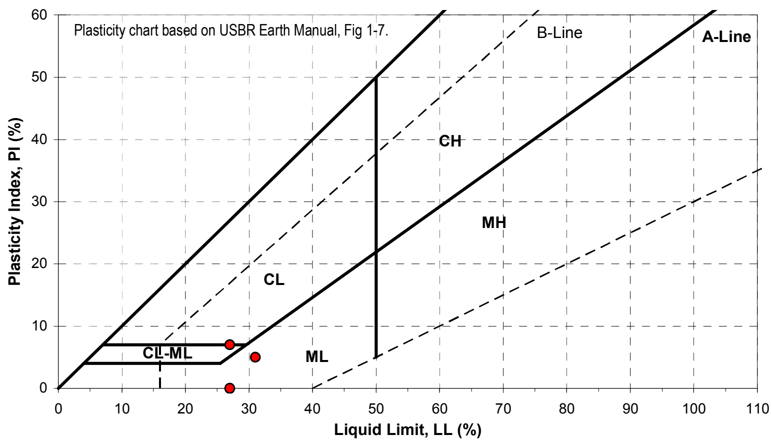


Figure 17: Plasticity Chart for Interbeddded Silt and Sand Unit (3 samples)



6.2.4 Sandy GRAVEL/gravelly SAND

Alluvial gravel-dominated deposits of the Springston Formation (syg) are inferred beneath much of the site. These materials include sandy fine to coarse gravels and gravelly sands. The upper contact of gravel deposits encountered within the upper 10 m is shown on Figure 10.

The gravel distribution is generally indicative of infilled channels with various orientations. Gravel was less common in the east of the site where marine depositional conditions prevail.

CPT's indicate that the gravel are typically at least medium dense (i.e. they have q_c values of 10 to >30 MPa). Gradings indicate that maximum clast sizes are 60 to 100 mm, and that silt typically represents less than 10% of the material. Gravel clasts are strong greywacke sandstone.

Beneath the high dunes depth to gravel deposits vary between RL 1.5 and 2.5 m in the south, RL 0.5 and 1.5 m in the central-western part, and RL -1.5 and -3.0 m in the north of the site. This probably reflects the morphology of buried channels.

In the low dunes area, gravel deposits generally occur between RL -1.5 and -3.0 m with the exception of a zone in the northern-most corner where gravels were encountered between RL 0.0 and 0.5 m.

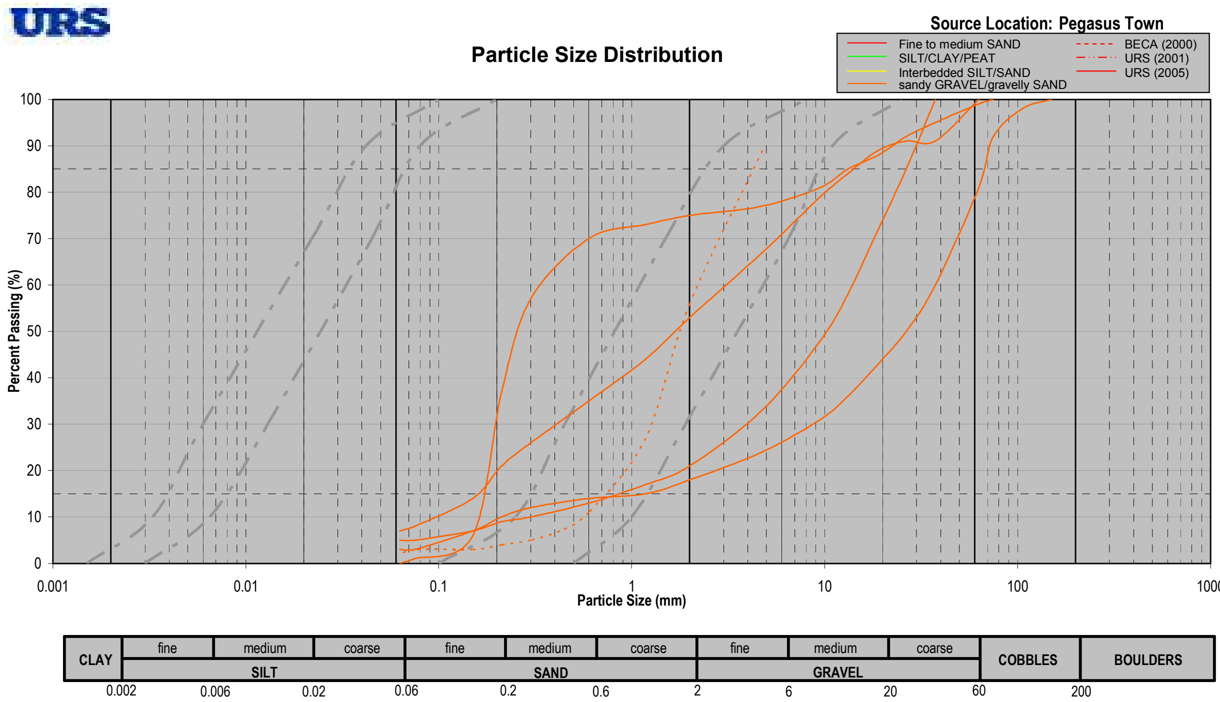
CPTs did not encounter gravelly deposits in the upper 10 m in about half of the low dunes area.

Material properties for the Gravel unit are presented in Table 5. Gradings results are presented in Figure 18.

Table 5: Summary of Material Properties for Sandy Gravel/gravelly Sand

Test	No. Tests	Mean	Range	Comments
GRADING				Mean grain sizes (min-max range)
% gravel (> 2 mm)	5	47%	25 - 82%	D_{85} = 16.0 mm (4.2-60.0)
% sand (0.06-2 mm)		46%	15 – 83 %	D_{50} = 1.7 mm (0.26-27.0)
% silt (0.002-0.06 mm)		7%	0 – 13 %	D_{15} = 0.3 mm (0.17-1.3)
% clay (<0.002 mm)		0%	0 %	
IN SITU PROPERTIES				
Moisture Content (%)	4	1.8%	0.4 – 3.6%	Moisture content should be saturated in these samples.
PERMEABILITY				
Typical value based on gradings	-	1.0 x 10^{-4}	1.0 x 10^{-3} – 1.0 x 10^{-5}	Estimated from Gradings
CALIFORNIAN BEARING RATIO (CBR)				Completed using NZ Std Compaction and Optimum Moisture Content
CBR	1	14	-	
Maximum Dry Density (t/m^3)		1.72		

Figure 18: Particle Size Distribution for Sandy Gravel/Gravelly Sand (6 samples)



6.3 Groundwater Modelling

A MODFLOW model was prepared to assess the likely groundwater levels in those areas of the site not assessed in detail. This model was then used to estimate the changes in water level as a result of construction of a lake as proposed for the Pegasus Town development.

The 5 km by 4 km study area was represented in the model by a 12 metre by 12 metre grid, resulting in approximately 140,000 cells. The geology of the model area was represented in two layers with surface elevations of the two geological units represented in the model based on the results of predominantly CPT testing at more than 120 locations across the study area. The model thickness was generally 20 metres. This represented the layers above the Riccarton Aquifer. The top layer represents the predominantly sand or silty sand layer with silt horizons and the lower layer represents the predominantly sandy gravel layer.

Model parameters were based on field observations and testing. Permeability (k) of the geological units was based on published values for the materials identified during the CPT testing and plotted in SURFER to be imported as a surface layer in MODFLOW. Permeability was grouped into 6 categories with values (horizontal) ranging from 0.00001 m/s to 0.005 m/s. The western portion of Layer 1 consisted of silts (assumed k of 0.00001 m/s), with sands dominant in the eastern portion of Layer 1 (k of 0.0005 m/s). The sandy gravel material of Layer 2 was with a k value of 0.0008 m/s.

Constant head boundary conditions were set on the western and eastern edges of the model based on known water levels at the western boundary, while the coastline was used as a constant head boundary on the eastern side.

Streams and drains in the model area were represented as river cells with stage and bed depths based on topographical survey information for the site. The bed material of the river cells was assumed to be the same as the underlying geology.

Water levels in the model were calibrated based on observed water levels within piezometers installed at the site. A good calibration was achieved with water levels in the model. The predicted groundwater level under existing conditions is shown in Figure 19.

To assess the affect of the lake on groundwater levels the lake was included in the model as a zone of river cells with a stage height of 1.4 metres and a bed level 3 metres below the stage height (-1.6m).

The modelled water levels with the lake are shown in Figure 20. The model showed the construction of a lake, as proposed, will result in a reduction in water level in proximity to the lake. A reduction of 1.0 metre is shown to occur within the immediate western edge of the lake, reducing to a 0.4 metre drop in water levels at a distance of 200-300 metres from the western margin of the lake and approximately 120 metres to the east of the lake. Within the western section of the model there appears to be a very small decrease in water levels as a result of the lake, while to the east there is no detectable change.

7.1 Liquefaction Mitigation Design

7.1.1 Liquefaction Effects

Previous investigations (URS 2001) have highlighted the liquefaction susceptibility of the proposed town site. The current investigations were designed to assess the extent and degree of this susceptibility and to recommend remedial measures against these effects.

A key feature of the proposed development is the inclusion of a lake excavation in the low dunes area to provide additional fill material to raise the surrounding ground level above the level of inundation due to flooding of the Ashley River. Excavation of this lake, combined with the liquefaction susceptibility, is likely to result in lateral spreading toward the lake during a significant earthquake event.

An initial assessment of the effects of lateral spreading were completed by URS (2001). Results from this assessment indicated that the majority of the proposed development would be affected by this process, affecting all infrastructure within the zone of displacement. Consequently planning rules were developed (see Section 7.1.2) to ensure that the potential for liquefaction was considered during design of the subdivision.

This section of the report is intended to firstly summarise the findings of the liquefaction and lateral spreading assessments and to summarise possible mitigation measures. Based on these results and recommendations an assessment of the foundation conditions of infrastructure within the proposed development is discussed, with possible foundation types recommended.

Foundation conditions at each site are assessed for both static and dynamic loads. Under normal operating conditions it is assumed that loads applied by the structure on to the foundation materials will be static. In the case of dynamic loading, resulting from earthquake induced shaking, both liquefaction susceptibility and lateral spreading are considered. However, whether lateral spreading occurs, and to what degree, is dependent on the mitigation measures utilised in the final design. In the structure assessments it is simply mentioned whether or not the proposed sites are within the area expected to be affected by lateral spreading.

7.1.2 Liquefaction Planning Rules

Waimakariri District Plan includes the following rules with respect to mitigating the effects of liquefaction:

1. *The layout, design and construction of any subdivision in the Residential 6 and 6A, Business 1 (at Pegasus Bay town) and Pegasus Bay Rural Zones shall take into account the potential for earthquake-induced liquefaction of the ground within these zones, and the potential effects of associated ground settlement and lateral spreading of the ground on structures and utility services. (rule 31.1.1.38)*

Liquefaction mitigation measures shall be designed and constructed to achieve the standards set out in Table 31.3 below:

Table 31.3 Liquefaction Mitigation Design Standards		
	Maximum Permanent Ground Movement	
Design Earthquake Return Period	***Settlement***	***Lateral Movement***
150 years	*100mm*	*250mm*

2. *Within the Residential 6, 6A and Business 1 Zones at Pegasus Bay town and the Pegasus Bay Rural Zone, all utilities shall be designed and constructed to ensure they will remain in service after a 150 year return period earthquake. This shall include taking into account the effects of earthquake-induced liquefaction of the ground (rule 29.1.1.11).*

7.1.3 Liquefaction Analysis

A liquefaction assessment has been undertaken of all CPT data collected to predict whether any of the strata underlying the site will liquefy during strong earthquake shaking. The following section summarises the results of a liquefaction assessment undertaken for the site. A more detailed description of the methodology is given in Appendix K.

Two different approaches have been used to assess the liquefaction potential: the Cyclic Stress Ratio (CSR) method (Seed et al. 2003), and the Davis and Berrill (1982) energy method. These take into account both the geology of the site and the estimated seismic hazard. The CSR method compares the stress induced within a soil by an earthquake to the resistance of the soil to withstand cyclic loading. The energy method uses an empirical relationship derived for the dissipation of seismic energy within the site soils and the liquefaction behaviour of those soils. The use of the two different methods serves as a check on the robustness of the liquefaction prediction.

Settlements have been predicted using an average volumetric strain of 3.7%, selected following the method of Ishihara and Yoshimine (1992). This volumetric strain is estimated to be the 84^{th} percentile value based on analysis of 6 of the CPT results.

In a probability-based assessment such as this, the mean groundwater level should be used to predict the degree of liquefaction that will occur in a given return period. As we do not have a statistically representative data set of groundwater measurements, we have used the highest groundwater levels measured on site during the site investigations. Based on our understanding of the site groundwater system, we believe that these levels are above average, but not unusually so.

7.1.4 Earthquake Scenarios

On the basis of a Probabilistic Seismic Hazard Assessment (PHSA) (Stirling et al., 1999) developed for the Canterbury Regional Council (Environment Canterbury), we have selected three earthquake scenarios to use as a basis for our study. Two of these earthquake scenarios represent the strongest shaking at the site predicted by the PSHA for 150 year and 475 year return periods. The third earthquake scenario represents the special case of the Alpine Fault, which is recognised as the generator of the largest earthquake (in terms of moment magnitude, M_W) likely to affect the site. Details of these three earthquake scenarios are given in Table 6.

In accordance with the Waimakariri District Plan the "design earthquake return period" for the subdivision design is 150 years.

Table 6: Earthquake Scenarios for Liquefaction Assessment

Earthquake Scenario	Magnitude (M_w)	Distance Between Earthquake Source and Pegasus Bay Town	Peak Ground Acceleration (g)
150 year return period earthquake	7.2	25 km	0.28
475 year return period earthquake	7.2	10 km	0.44
Alpine Fault Earthquake	8.0	110 km	0.18

7.1.5 Liquefaction Effects

Settlement

Post liquefaction settlement is directly proportional to the thickness of liquefied material at a particular site. The cumulative thickness of material predicted to liquefy during each of the three earthquake scenarios and the corresponding settlements are summarised in Table 7 and the spatial distribution is illustrated in Figures 21 to 22. As stated above settlements were calculated assuming that the liquefied layer experienced a 3.7% volumetric compressive strain after the excess pore water pressure has dissipated.

The results indicate that the CSR Method predicts that more of the soil column will liquefy than the Energy Method (the CSR Method predicts 2 to 3 times the cumulative liquefiable thickness compared to the Energy Method for a given earthquake event). The following discussion relates to the results predicted by the CSR method.

In the High and Low Dunes areas, up to about 3 m of cumulative liquefied thickness is predicted under the 150 year RP event, with the distribution shown in Figure 23. In the Mapleham area, approximately 30% of CPTs predict no liquefaction, and 30% predict more than 2 m of cumulative liquefied thickness. The greatest cumulative liquefied thickness is predicted for the Mapleham area is about 7 m under the 150 year RP event, and this is consistent with the findings of the WDC/CRC liquefaction study (Beca, 2000), which predicted some very extensive liquefaction in the Woodend area, to the immediate west of the Mapleham area.

Under the design earthquake, only 3 out of a total of 94 CPTs (~3%) predict greater than 100 mm settlement. The mean predicted settlement for the High and Low Dunes are is in the range 30 to 40 mm (Figure 24).

In the High and Low Dune areas the analysis predicts 30% greater cumulative liquefied thickness and settlement for the 475 year RP event compared to the design 150 year RP event. During the Alpine fault event, predicted cumulative liquefied thickness and settlement is about 80% of the design 150 year RP event. In the Mapleham site, cumulative liquefied thickness and settlement values are similar for all three earthquake scenarios.

Figure 23: Distribution of Predicted Cumulative Liquefiable Thickness (m)

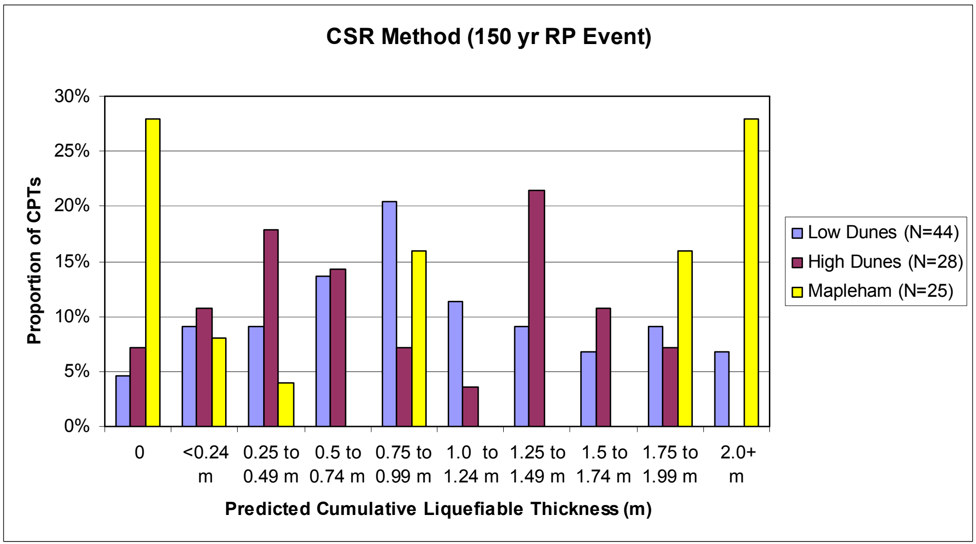


Figure 24: Distribution of Predicted Settlement (mm)

(all areas of site)

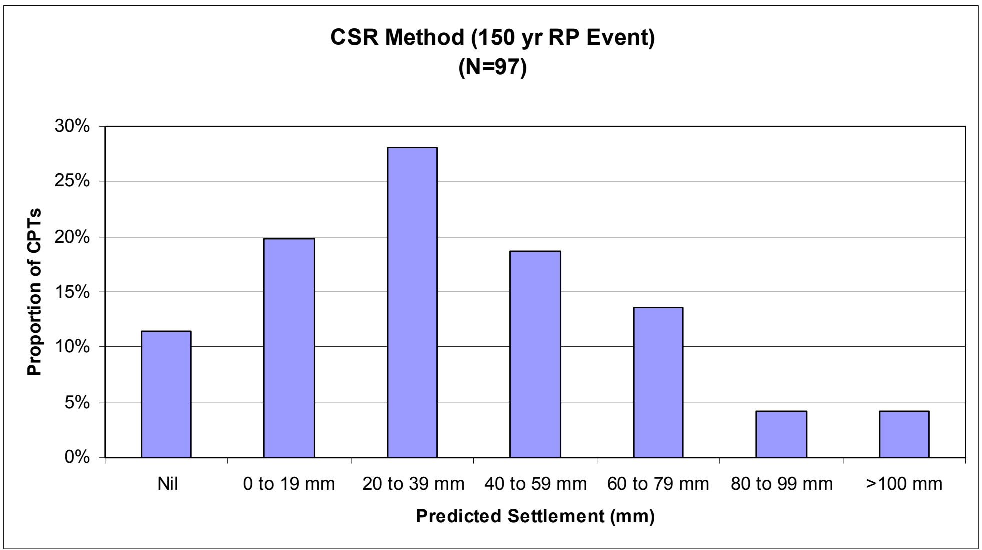


Table 7: Results of Liquefaction Assessment

Analytical Method		Cumulative Liquefiable Thickness (m)						Estimated Settlement (mm)					
		CSR Method			Energy method			CSR Method			Energy method		
EQ Scenario		150 yr RP	475 yr RP	Alpine Fault	150 yr RP	475 yr RP	Alpine Fault	150 yr RP	475 yr RP	Alpine Fault	150 yr RP	475 yr RP	Alpine Fault
High Dunes (28 CPTs)	Max	2.41	3.29	2.15	0.93	1.97	0.88	89	122	79	34	73	32
	Min	0	0	0	0	0	0	0	0	0	0	0	0
	Mean	0.88	1.15	0.71	0.24	0.81	0.21	32.61	42.50	26.18	8.75	30.07	7.61
Low Dunes (44 CPTs)	Max	2.93	3.34	2.68	1.52	2.55	1.4	109	124	99	56	94	52
	Min	0.17	0.23	0.13	0	0.2	0	6	8	5	0	7	0
	Mean	1.18	1.52	1.00	0.58	1.31	0.53	43.64	56.29	37.04	21.75	48.46	19.75
Mapleham Site (25 CPTs)	Max	7.1	7.24	6.98	2.34	4.24	2.26	263	268	258	86	157	84
	Min	0	0	0	0	0	0	0	0	0	0	0	0
	Mean	1.27	1.36	1.19	0.41	0.85	0.37	46.96	50.43	44.04	15.04	31.43	13.83

Lateral Spreading

Liquefaction induced lateral spreading has been assessed using both the Bartlett and Youd *free face model* and *sloping ground model* (1992) and the slope stability programme *Slide*, for Pegasus Bay Township. The *free face model* simulates the impact of excavating the sand to form the lake or eastern wetland, thereby creating a *"free face"* that the surrounding ground can move toward. The *sloping ground model* assumes that there is no free face and considers the impact on the natural ground slope on the ability of the ground to move laterally.

Bartlett and Youd's *free face model* (modelling the presence of the lake) predicted unacceptably large ground displacements around the immediate vicinity of the lakeshore and water feature in the Eastern Conservation Management Area, for the design 150-year return period earthquake. Lateral displacements of 250 mm are expected 220 metres from the lakeshore and 1000 mm movement, 30 metres back from the lake edge, subsequent to this design event. Figure 25 summarises the results of the lateral spreading assessment showing estimated zones inside which deformation of a certain magnitude is predicted by the Bartlett and Youd model. The modelling indicates that ground improvement will need to be undertaken to mitigate the lateral spreading hazard to buildings and infrastructure within the affected areas.

A liquefaction model of the site was created using the ground profile, subsurface layering of soils, groundwater depths, soil properties and the liquefaction assessment of CPT's completed. This model was inserted into *Slide* and loaded seismically with the design earthquake. Failure up to 44 metres from the lakeshore was observed when the model was exposed to the design earthquake shaking, prior to the liquefaction of the susceptible layers. The model was then analyzed after the earthquake had finished and the susceptible layers had liquefied, again resulting in failure of the ground.

7.1.6 Evaluation of Remedial Measures

Liquefaction in soils occurs primarily because sands and silts are in a loose state. Compacting these soils is a common way to improve liquefaction resistance in terms of both settlement and lateral movement.

The existing ground profile of the site consists of 'medium dense sands' which when saturated are very susceptible to liquefaction. One method to prevent liquefaction from occurring in these soils is to strengthen the sands by densification. By increasing the density of the saturated sand, the void ratio decreases, reducing the amount of settlement/densification and liquefaction susceptibility of the sand layer. Densification of sand layers can be done by one of the following ways.

1) Dynamic compaction, where a heavy weight is dropped from a significant height onto the ground surface to compact loose silts and sands. Dynamic Compaction can remove potential liquefaction layers to depths exceeding 6 metres. Below this depth the effect of any liquefiable layers at this site will be minimal and significant displacements eliminated.

2) Vibro floatation is the process where a vibrating probe is inserted into the ground to cause the compaction of sands.

3) Inserting stone columns into the ground is another technique used to strengthen weak soils and reduce the liquefaction potential of weak layers. This technique involves inserting a probe (similar to that used for vibro floatation) into the ground. The probe penetrates the ground using its own self weight and vibrating energy and is driven down to the stone columns specified depth. Once at this depth it is withdrawn in steps (lifts) of 1m and the hole is backfilled with gravel. The probe is then reinserted into the gravel backfill, compacting it into the surrounding soil and expanding the stone column diameter. The process is repeated across the site, in a grid type pattern, increasing the strength of the soil and removing potential layers of liquefaction.

Excavation and then re-compaction of loose sand is another way to create a denser soil structure. The sand will be re-compacted in layers to ensure all the sands are compacted to the desired density. The problems of using this method at Pegasus Bay is the relatively high water table that will create difficulties excavating to the depth of some of the loose and potentially liquefiable layers. This method is a lot cheaper than the dynamic compaction, however its effectiveness at depth is limited. In preparation of this report telephone discussions were held with Daniel Smith of Smith Crane & Construction Ltd about the likely cost competitiveness of area-wide treatments like vibro floatation or stone columns. His view was that these methods would not be competitive with the buttressing method proposed.

7.1.7 Buttress

This first option to prevent lateral spreading from the site at Pegasus Bay will involve dynamic compaction (or excavation and re-compaction) around the lakeshore. A compacted buttress of high density sand around the lake front will prevent liquefaction within the area compacted and contain/eliminate any lateral displacements of soils towards the free face of the lake. Using the slope stability model '*slide,*' the size of this buttress is recommended to be 10 metres wide and extend a future 20 metres into the lake. The depth of compaction is recommended to extend 3.5 to 5.5 m below the ground surface depending on the local depth variations to the lower liquefiable zone. Clearly this will be around 2.5 – 3.5 m below the water table, requiring a dewatering system to remove water that enters the excavations.

Excavating and then re-compacting 10 metre wide strips of ground around the lakeshore was investigated as a remedial measure to prevent lateral spreading during the design event. The strip was designed to extend to a depth of 4 metres. The dense strips would be placed below those streets oriented parallel or subparallel to the lake shore. The intention is that right at the start of construction the soils below subgrade down to the base of the uppermost liquefiable layer would be excavated, mixed and then simply recompacted to a dense configuration, removing their liquefaction susceptibility, while markedly improving their shear strength to resistance. This will also provide the additional benefits of supporting vital lifeline services e.g. sewers, water pipes, etc and improve road foundations along the streets where buttressing is employed.

The mitigation effect of the buttresses was analyzed in *Slide* resulting in an acceptable factor of safety of 2.3 and then applied to the second Bartlett and Youd approach where the free face is absent i.e. that predicted horizontal ground displacements of 0.125 metres. Although *Slide* does not produce lateral displacements of the ground, it does correspond to Bartlett and Youd's model and produced ground failure under the conditions expected during and after the 150-year return period earthquake.

Figure 25 illustrates those streets where it is suggested that the buttressing measure could be applied. Around much of the lake and along the margin of the ECMA it is proposed to locate houses between the lake shore and the nearest potential "street buttress" and it is strongly recommended that additional buttress or ground treatment be employed along the lake edge to minimize the potential for lateral movement at these locations. Figure 25 clearly illustrates that the locations of the greatest lateral movement are those closest to the lake. If buttressing is taken to 5.5m below the ground surface then excavations could be up to 30 m wide assuming 2H:1V excavation batters.

An alternative arrangement would be to create a single compacted buttress of sand around the lakeshore. This would tend to maximize residual lateral strains within the ground upslope of the lake.

7.1.8 Stone Columns

Stone columns could be used to prevent lateral spreading at Pegasus Bay. Although this method will not remove susceptible liquefaction layers, it will isolate these layers by very dense columns of gravel that will eliminate the potential for lateral spreading. The problems with this method will be the cost involved in treating such a large area in this way. Stone columns at this site will be an expensive remedial and this method is not recommended for Pegasus Bay.

7.2 Road Subgrade

7.2.1 State Highway 1 (SH1) Intersection

Site and Geology Description

Access to the site is proposed via a round-about constructed on SH1, at the location shown on Figure 26.

Two CPT and four test pits were completed at the proposed intersection. The investigations were split evenly between the northwest and southeast sides of SH1. CPT-079, on the northwest side of the road, was completed to 10 m depth through interbedded fine sandy silts and silty fine sands. No gravel deposits were encountered. CPT-080, on the southeast side of the road, was completed in identical materials, but was extended to contact gravels, encountered at about 11.0 m below ground level. Materials above the gravel deposits are generally loose to 1.5 m below ground level, becoming very loose below this depth.

Groundwater seepage was noted in all four test pits at about 2.5 m below ground level.

Static Assessment

CPT q_c values between ground level and 1.5 m below ground level appears to be relatively consistent with values between 2.5 to 3.0 MPa. These equate to equivalent Dynamic Cone Penetrometer (Scala) values between 1.4 and 1.7 blows/100 mm (60 to 72 mm/blow) giving *in situ* CBR values of 2.5 to 3.0. Below 1.5 m depth these values decrease to CBR values less than 2.

Laboratory CBR tests completed on materials from TP-031 and TP-033 gave values of 7 and 8 respectively. These tests were completed at NZ Standard Compaction (1.7 and 1.82 t/m^3) and Optimum Moisture Content (15.5 and 18.0 %).

Dynamic Assessment

Liquefaction is likely to affect this part of the site the greatest. During the 150 year event it is predicted that all of the saturated material will be susceptible (about 9 m thickness) with estimated settlements in the order of 300 mm.

Lateral spreading due to excavation of the proposed water features is unlikely to affect the State Highway or intersection. However, minor effects may be seen due to changes in the natural shallow slopes in this area.

Suggested Pavement Design CBR

The laboratory results demonstrate the benefit of compacting the *in situ* subgrade soils to improve their density enhance their design CBR values. The CBR value can be improved from around 3 to about 8 giving a reduced pavement structure thickness.

7.2.2 Access Road

Site and Geology Description

Access to the proposed development is via a road from SH1 across the proposed Mapleham development into the southwest corner of the residential and commercial zones (Figure 26).

The majority of the proposed road will be founded on materials derived from alluvial overbank flood deposits. These are comprised mainly of interbedded fine sandy silt and silty fine sand. Thicknesses of individual layers vary from about 0.1 to 1.0 m.

CPT-080 completed at the intersection with SH1 indicates that the subsurface geology is comprised of alternating silts and sands to a depth of 11.0 m below ground level before gravels are encountered. CPT-081 completed about 500 m south east, on the road alignment, refused on gravels at about 3.4 m. Between CPT-081 and the proposed development area gravels were encountered at depths between 4.0 and 5.5 m. CPT-054 at the entrance to the proposed development indicates that materials between ground level and

4.5 m are comprised of medium dense fine to medium sand with loose silt from 4.5 to 5.5 m and gravels below 5.5 m.

Densities of the overbank flood deposits ranged from loose to medium dense based on q_c values recorded by CPT tests.

Groundwater levels along the access road alignment vary from about 2.5 m at the SH1 intersection to about 5.0 m below ground level at the intersection between the alluvial plain and high dunes. The higher groundwater levels encountered near the SH1 intersection are likely to be due to downward migration of groundwater from the Taranaki Stream. Groundwater levels are likely to fluctuate due to seasonal variations.

Static Assessment

Equivalent Dynamic Cone Penetrometer (Scala) values derived from CPT-080 to CPT-083 indicate that they range from 1 to 8 blows/100 mm (12 to 90 mm/blow), giving CBR values between 2 and 25.

Laboratory CBR values were 6, 12 and 14 (TP-037, 039 and 040) with most tested completed at NZ Standard Compaction (1.70 to 1.82 t/m^3) and optimum moisture content (15.5 to 18.0 %).

Dynamic Assessment

Liquefaction is likely to affect the area near to and including the intersection with SH1 with lesser degrees of susceptibility occurring along the access road alignment.

Several shallow water features are planned near to the entrance to the proposed development and these excavations should be designed to account for possible lateral spreading during and after the design earthquake.

Suggested Pavement Design CBR

As for the State Highway 1 intersection the laboratory results demonstrate the benefit of compacting the *in situ* subgrade soils to improve their density enhance their design CBR values. The CBR value can be improved from around 2 to about 8 if the CBR design criteria are biased toward the lower part of the range of laboratory test values. Using a CBR of 8 will give a uniformity of pavement design along the access road from SH 1.

7.2.3 Residential Streets – High Dunes Area

As shown on Figure 26 the majority of the proposed development will occur within the high dune area of the site, in pine plantation at the time of writing this report.

Geology Description

Twenty eight CPTs were completed in the high dunes area during the current investigation. An additional eight CPTs were completed during previous investigations. Twelve test pits were completed with selected samples tested to give a representative CBR.

Materials within the upper 2 m of the deposits comprise fine to medium sand. Densities between ground level and 1.0 m below ground level range from very loose to medium dense (q_c = 1 to 5 MPa).

Groundwater levels vary across the site but generally occur at about 2.0 m below ground level.

Static Assessment

Due to the undetermined degree of surface recontouring that will occur within the area of the proposed development it is not appropriate to give subgrade CBR values at specific locations.

Table 8 presents an equation for the approximation of scala values at various depths for the calculation of *in situ* CBR values. This equation is derived from CPT q_c values from *in situ* tests. A correction factor of $q_c/1.8$ has been used to reach an approximate scala value (blows/100 mm).

It is important to note that use of this equation is only valid for materials within 1.5 m of the current ground surface. If the depth of foundations is deeper than 1.5 m, detailed assessment will be required as very loose materials may be encountered, greatly affecting the CBR value.

Table 8 – Estimate of Scala Values

Material	Depth Below Current Ground Level (m)	Scala Equivalent (Blows/100 mm)
In Situ	y = 0.0 to 1.5	$x_{average}$ = 2.3 (y) x_{min} = 1.1 (y) x_{max} = 3.7 (y)

Laboratory CBR completed on materials in this area indicate that CBR values of 17, 25 and 25 (TP-018, 019, and 023) are achievable at NZ Standard Compaction (1.57 to 1.82 t/m^3) and optimum moisture content (14 to 19 %).

Dynamic Assessment

Liquefaction is likely to occur within the high dunes area. The roading is within the area predicted to be affected by lateral spreading and settlements.

Suggested Pavement Design CBR

Following adequate compaction of the subgrade soils once the site contouring operations re complete, it appears as though design CBR's of 15 should be achievable.

7.2.4 Residential Streets – Low Dunes Area

Geology Description

Fourty one CPT tests completed in the area surrounding the proposed lake excavation indicated that the subsurface geology is predominantly comprised of fine to medium sand with some layers of silts.

Road foundations in this area are proposed to be formed on fill material derived from the lake excavation, to raise the area above flooding hazards.

The density of *in situ* materials between ground level and the groundwater table is generally loose.

Static Assessment

CBR values will be determined by the relative density of the compacted materials.

Laboratory CBR tests completed on materials within the proposed lake excavated area indicate CBR values of 14 and 19 (TP- 011 and 012) can be achieved. A low value of CBR 3.5 was reported for a sample from TP-024 but inspection of the test results sheet suggests that this sample was not well compacted in the laboratory and so it has not been included in the post compaction subgrade strength evaluation.

Dynamic Assessment

Liquefaction is likely to occur within this area. However, it is likely that the effect of this liquefaction will be minimal due to the flexible nature of the structure.

The roading is within the area is predicted to be affected by lateral spreading. The mitigation measures (See Section 7.1) used will determine the affect that this process will have on the roading.

Suggested Pavement Design CBR

Similarly to the High Dunes area pavement design CBR values of around 15 could be adopted.

7.3 Fill Source and Placement

Fill for the subdivision will be sourced predominantly from the lake excavation. The suitability of this material and its characteristics as a fill material have been assessed by excavating six test pits (TP 001 to 006) within the lake area and analysing their particle size distribution and compaction parameters in the laboratory. The compaction test used is NZ Standard Compaction - NZS 4402:1986, Test 4.1.1. The test results include the *in situ* moisture content. The results are presented in Table 9.

Table 9 – Summary of Laboratory Tests for Fill Material from Lake Excavation

Sample Source	*TP-001*	*TP-002*	*TP-003*	*TP-004*	*TP-005*	*TP-006*
Material Description (summary) See Appendix G for details	Fine to med sand with minor silt/clay	Fine to med sand with minor silt/clay	Fine to med sand with minor silt/clay	Silty fine to med sand with trace clay	Fine to med sand with trace silt/clay	Fine to med sand with trace silt/clay
Insitu M/C - %, (depth m)	2.7% (0.5 m)	5.1% (1m and 2.5 m combined)	27.9% (1m)	20.8% (1.1m)	23.2% (1m)	4.3% (1m)
	24.9% (1 m)		26.7% (2.9m)	26.2% (2.4m)	24.2% (2.4m)	31% (2.5m)
	53% (1.8 m)			22.4% (3.6m)	23.5% (2.8m)	32.3% (2.7m)
OMC(%)	20.5%	14%	17%	15%	18.5%	18.5%
Max Dry Density (t/m^3)	1.57	1.64	1.65	1.70	1.58	1.55

Examination of the Table 9 above and the detailed results in Appendix G leads to the following conclusions:

- The excavated materials to form the lake are suitable for use as a fill but are generally uniformly graded i.e. consist of predominantly similar size particles. This will tend to lead to lower material densities and will not forma as "tight" a structure as a similar well graded material.

- Material selected from below the water table is wet of Optimum Moisture Content (OMC) so this will need to be drained, possibly via use of temporary stockpiling immediately before placement. These saturated materials will be low strength and tend to flow immediately after excavation.

- Moisture control during fill placement will be important for producing a dense fill. Dry of optimum the compacted fill loses density less quickly than a corresponding incremental moisture content above OMC. Examination of the compaction curves in Appendix G illustrate this.

- At or close to the OMC several samples expelled free water during the testing, at about 10% MC. Compacted densities declined quickly under this condition as moisture content increased. Therefore this condition could serve as an observational warning to supervising staff during fill placement.

7.4 Underground Services

Geology Description

Assuming that all services are less than 1.5 m below ground level (assuming that ground level in the low dune area is raised to RL 3.5 m) all excavations will be completed in materials comprising fine to medium sand. No groundwater should be encountered in trenches.

Static Assessment

Design of trenches during the construction of these services should be based on the suggested design parameters outlined in Appendix H for unsaturated dry to moist fine to medium sand.

Based on the assumption that these services will be constructed within 1.5 m of the current ground level most of these services will be unaffected by loose silt and soft clay deposits under normal operational conditions. However, heavy pipes and thrust blocks may be affected by these materials and assessment on a case by case basis is necessary as the infrastructure design progresses.

Trench support will be required for safe excavations and where groundwater levels are high (especially in the low dunes area) dewatering systems to maintain dry trenches will be required.

Dynamic Assessment

As indicated above the area within which underground services will be constructed is affected by both liquefaction induced settlement and lateral spreading.

Design of the underground service reticulation in the main development areas (high and low dune areas) will need to be designed so as to minimise the effects of these processes; minimising disturbance due to differential settlement and lateral displacements.

Suggested Foundation Design Options

Underground services are clearly at most risk from area wide liquefaction effects which will induce shear stresses caused by differential movement, either laterally or vertically of the longitudinal pipelines. The most effective means of eliminating liquefaction related risk will be to prevent liquefaction along the service route using one of the methods described in Section 7.1. The provision of flexibility to the service line e.g. flexible joints will enable it to accommodate the differential movements.

The extent to which particular services are protected against liquefaction induced damage will need to be decided by the service designers taking into account their proposed hierarchy of service lifelines within the subdivision and the need to comply with legal requirements to protect these. For the remainder the need to implement specific measures will be determined by the need to comply with the Waimakariri District Council liquefaction planning rules for the town.

7.5 Pump Station Foundations

Foundation conditions at the six proposed pump station locations were completed with cone penetration tests; CPT-073 to CPT-078 inclusive.

Due to the lack of detailed designs at the time of writing of this report the assessments indicate whether the site is likely to require treatment for liquefaction.

A summary of each site is shown in Table 10.

Table 10 – Summary of Pump Station Sites

Pump Station Site	CPT	Liquefaction Susceptibility – CSR Method						Susceptible to Lateral Spreading
		Liquefiable Thickness (m)			Estimated Settlement (mm)			
		150 yr	475 yr	Alpine Fault	150 yr	475 yr	Alpine Fault	
1	CPT-073	0.50	0.66	0.44	19	24	16	Yes
2	CPT-074	1.89	2.81	1.64	70	104	61	Yes
3	CPT-075	1.40	2.06	0.99	52	76	37	Yes
4	CPT-076	1.83	2.39	1.51	68	89	56	Yes
5	CPT-077 (CPT-058)	0.71 (0.87)	1.27 (1.61)	0.51 (0.77)	26 (32)	47 (60)	19 (28)	Yes
6	CPT-078	0.45	0.57	0.35	17	21	13	Yes

Applicable to the design of all pump station with wet wells is the potential for floatation effects with high water tables and liquefaction of surrounding soils.

7.5.1 Pump Station 1 (CPT-073)

Site and Geology Description

Situated in the low dunes area, Pump Station 1 is located directly adjacent to the proposed lake excavation. It is intended that ground level at the site will be elevated to RL 3.5 m with the addition of fill material derived from the lake excavation. This is to raise the site above potential inundation due to flooding.

CPT-073 indicates that the *in situ* subsurface geology consists predominantly of fine to medium sand. The density of materials between the ground surface and 1.3 m (RL 1.1) are very loose to loose (q_c<4). Densities increase to medium dense between 1.3 m and 2.2 m (RL 0.2). Below 2.2 m materials are dense to very dense with a zone of gravelly sand from 2.2 m to 5.4 m (RL -3.0). Refusal occurred at 9.9 m (RL -7.5)

Groundwater level at the site is likely to fluctuate depending on the intensity of rainfall events and variations in the operational level of the proposed lake. The current permanent saturated water level is at about 1.0 m below ground level (RL 1.4) with the proposed lake level at RL 1.6.

Static Assessment

Foundation conditions at this site will be determined by the properties of the fill material to be placed and the preparation of the upper 1.3 m of *in situ* material. Assuming these material are compacted to obtain maximum densities, settlements during normal operation are likely to be minimal.

Dynamic Assessment

Liquefaction susceptibility is low due to the absence of silt layers and loose sand in this particular part of the low dunes area. Mitigation against damage due to lateral spreading, due to the sites close proximity to the lake excavation, will be dependent on the measures used in the final design.

Suggested Foundation Design

The most appropriate foundation design would appear to be one which founds the station structure within the dense sands which extend down from about 1 m below existing ground surface. Specific treatment for liquefaction does not appear warranted given the low level of predicted liquefaction effects i.e < 25 mm settlement.

7.5.2 Pump Station 2 (CPT-074)

Site and Geology Description

Pump Station 2 is sited in the northern part of the high dunes area.

A significant layer of silt occurs between 2.0 and 2.9 m below ground level (RL 1.7 to 2.6). Density of these materials is generally medium dense to dense with the exception of the silt layer which is very loose. The test reached its target depth, 10.0 m.

Groundwater level at the site post lake excavation is assumed to be approximately 2.5 m below the existing ground surface (RL 2.1) based on groundwater modelling.

Static Assessment

Assuming no ground treatment the layer of silt that occurs between 2.0 and 2.9 m deposit is likely to be located within the depth of influence of the structure.

Without removal and replacement of this material settlement of the structure may occur.

Dynamic Assessment

Liquefaction is likely to occur during the 150 year event with 1.89 m of material susceptible to liquefaction. The majority of this is inferred to occur within a zone of moderately dense silty sand between 4.5 and 5.2 m below ground level (RL 0.1 to -0.6). Settlement during this event is estimated at about 70 mm.

The site is within the zone potentially affected by lateral spreading.

Suggested Foundation Design

Treatment of the ground beneath the proposed pump station or piling to support it will be necessary to reduce the risk of settlement under normal operating conditions and seismic loading. Straight removal of the 0.9 m thick silt layer is likely to be the best means of mitigating the settlement risk.

Ground treatment methods include the use of stone columns or vibro-floatation to densify the loose materials at depth.

7.5.3 Pump Station 3 (CPT-075)

Site and Geology Description

Located approximately 400 m south of Pump Station 2, in the northern part of the high dunes area, is the proposed site for Pump Station 3.

CPT-075 indicates that the subsurface geology is composed predominantly of fine to medium sand with the exception of a 0.4 m thick layer of silt between 2.9 and 3.3 m below ground level (RL 1.7 to 2.1). *In situ* material density is generally medium dense (4 – 12 MPa) with the exception of the silt layer which is soft. Test refusal occurred at 9.9 m (RL -4.9).

Groundwater level post lake excavation is inferred to be approximately 2.7 m below ground level (RL 2.3) based on the groundwater model.

Static Assessment

Assuming no ground treatment the silt layer located between 2.9 and 3.3 m is likely to be within the zone of influence of the structure. Therefore, this layer will have an effect, if only small, on the settlement characteristics of the site.

Dynamic Assessment

Liquefaction assessment indicates a maximum susceptible thickness of 1.40 m during the 150 year event with settlement of about 52 mm. This occurs in loose sands close to the silt layer (RL 1.7 to 2.1) and also within moderately dense silty sand between 4.5 and 5.2 m below ground level (RL 0.5 to -0.2).

The site is within the zone potentially affected by lateral spreading.

Suggested Foundation Design

Two foundation options present themselves for this site:

- Local ground treatment using methods like stone columns or vibro floatation to densify the liquefiable soils, or
- Piles taken to below the liquefiable layers to support the pump station structure in the event of an earthquake.

Both options will minimise settlement risk for both the static and dynamic load cases.

7.5.4 Pump Station 4 (CPT-076)

Site and Geology Description

Pump Station 4 is located at the northwest corner of the proposed commercial development zone, in the central section of the high dunes area.

Data collected from CPT-076 indicates that the subsurface geology is quite variable at this site with interbedded layers of moderately dense silty sand and very loose clayey silt, characteristic of alluvial derived overbank deposits seen to the west of the high dunes area. The upper 2.5 m is likely to represent dune deposits. Testing reached the target depth of 10.0 m.

Based on data from the groundwater model it is inferred that water level at the site is at about 3.0 m below ground level.

Static Assessment

Settlement under normal operational loads is likely to be minor due to the depth of the very loose deposits. These are located at the bottom of the zone of influence.

Dynamic Assessment

Liquefaction is predominantly confined to the numerous very loose silt layers with the maximum liquefiable thickness, 1.83 m, occurring in response to the 150 year event. Based on this event settlement is estimated to be about 68 mm.

The site is within the zone potentially affected by lateral spreading.

Suggested Foundation Design

Treatment of the ground beneath the proposed pump station will be necessary to reduce the risk of settlement under seismic loading.

Two foundation options present themselves for this site:

- Local ground treatment using methods like stone columns or vibro floatation to densify the liquefiable soils, or
- Piles taken to below the liquefiable layers to support the pump station structure in the event of an earthquake.

7.5.5 Pump Station 5 (CPT-058 and CPT-077)

Site and Geology Description

The site is located approximately 250 m south of the proposed lake on the boundary between the high dunes and low dune/interdune areas. Due to the lack of accessibility to the exact location of the site, geology has been interpreted between CPT-058 and CPT-077.

CPT-058 indicates that the geology is predominantly composed of fine to medium sand with some significant thicknesses of sandy silt between 1.3 and 3.5 m (RL 0.8 to 3.0). Densities vary from medium dense to dense in sand deposits to loose to very loose in silt dominated materials. Refusal occurred at 9.2 m (RL -4.9) in very dense sand.

CPT-077 encountered 2.5 m of loose to medium dense sand overlying 0.5 m of silt/clay between RL 1.4 and 0.9 m. The remainder of the test was completed in medium dense to dense sand and gravelly sand to 9.9 m depth.

Groundwater level at the site is inferred to occur at about RL 1.5 m.

Static Assessment

Settlement at the site is likely to be affected by the presence of significant very loose layers of silt dominated material within the likely depth of influence of the structure.

Dynamic Assessment

Liquefaction susceptibility during the 150 year event indicates that CPT-058 will have about 0.87 m of material susceptible to liquefaction with an estimated settlement of 32 mm. CPT-077 is likely to have 0.71 m susceptible resulting in 26 mm of predicted settlement.

Lateral spreading is likely to occur at this site.

Suggested Foundation Design

Once access is able to be gained directly to this site it is recommended that a site specific CPT test be carried out at this location and the specific foundation designs based on those results.

7.5.6 Pump Station 6 (CPT-078)

Site and Geology Description

Pump station 6 is sited in the southern part of the high dunes area approximately 150 m west of the boundary between the high dune deposits and the interdune deposits to the east.

CPT-078 was completed to investigate the subsurface geology. The test encountered materials predominantly comprised of fine to medium sand with a layer of sandy silt between 2.7 and 3.4 m (RL 1.6 to 2.2). Densities within the deposits vary with loose sand between ground level and 1.2 m becoming medium dense to dense. The layer of sandy silt encountered between 2.7 and 3.4 m is very loose to loose. Refusal in very dense sand occurred at 5.8 m.

Groundwater level at this location is inferred to be about 2.1 m (RL 3.5).

Static Assessment

The presence of the sandy silt layer between 2.7 and 3.4 m (RL1.6 to 2.2) is likely to be near to the maximum limit of the depth of influence of the proposed structure and will therefore only have a small effect on the settlement.

Dynamic Assessment

Liquefaction assessment indicates that the sandy silt layer is susceptible to liquefy during an earthquake event. A maximum thickness of 0.45 m is susceptible during the 150 year event, causing an estimate 17 mm settlement.

Lateral spreading is likely to occur with movement towards the east. This is due to the topographic elevation difference between the high dunes area and the interdunes area to the east. It is unrelated to the lake excavation.

Suggested Foundation Design

Two foundation options present themselves for this site:

- Local ground treatment using methods like stone columns or vibro floatation to densify the liquefiable soils, or

- Piles taken to below the liquefiable layers to support the pump station structure in the event of an earthquake.

Alternatively it may be possible to simply excavate down to the base of the silty layer at 3.4 m and replace it with compacted sand material, then found the pump station in the dense sand materials. Excavation support and a dewatering system would be required below the water table.

7.6 Commercial Foundations

Site and Geology Description

The proposed commercial development zone is planned for an area within the central section of the high dunes area. Ten CPT tests were completed in the area zoned for commercial development.

Materials beneath the commercial area vary from location to location, but are predominantly comprised of sand, silt and gravelly sand deposits, generally in that order.

Dune sands form deposits between ground level and about RL 3.0 m. Between ground level and 0.5 m below ground level these materials are generally loose; becoming medium dense below this depth.

From about RL 3.0 to RL 1.0 m materials are generally comprised of silt, silty clay and some peat (CPT-063) interfingering with layers of sand. Deposits of these fine grain materials are between 0.2 and 0.5 m thick and are generally loose or soft.

Below about RL 1.0 m deposits are comprised of dense to very dense gravelly sand and sandy gravel.

Groundwater levels in the area are likely to be between RL 2.5 and 3.2 m post lake excavation.

Static Assessment

Due to many possible structures and various foundation solutions that may be constructed in the commercial zone it is not possible to estimate the amount of settlement. Specific structures should be assessed individually.

During assessment consideration of the compressibility of loose silts, soft clays and in some instances, peat should be taken into account where the depth of influence of foundations is greater than the depth of these deposits. This is likely to be the case in most situations.

Remedial measures may include removal and replacement of material, ground improvement techniques (i.e. vibro compaction or stone columns) or piled foundations.

Dynamic Assessment

Liquefaction susceptibility within the commercial development area is generally confined to the loose silts that generally occur below the groundwater level. Assessment of susceptibility indicates that the 475 year event results in the greatest thickness of liquefiable material, subsequently with the highest estimated settlement values.

Table 11 summaries the results of liquefaction modelling. During the 150 year event it can be seen that the total thickness of liquefiable material is generally between 0.3 and 1.4 m with estimated settlements between 10 and 50 mm. Lateral spreading is likely to be an issue in this area.

Table 11 – Summary of Commercial Zone

Location	Liquefaction Susceptibility – CSR Method						Susceptible to Lateral Spreading
	Liquefiable Thickness (m)			Estimated Settlement (mm)			
	150 yr	475 yr	Alpine F	150 yr	475 yr	Alpine F	
CPT-021	0.54	0.88	0.28	20	32	10	Yes
CPT-056	1.39	1.74	1.23	52	64	45	Yes
CPT-057	0.41	0.56	0.12	15	21	11	Yes
CPT-059	1.41	1.73	0.98	52	64	36	Yes
CPT-060	1.18	2.04	0.85	44	75	32	Yes
CPT-061	0.87	1.15	0.73	32	42	27	Yes
CPT-062	0.22	0.26	0.20	8	10	7	Yes
CPT-063	0.28	0.40	0.18	10	15	7	Yes
CPT-065	1.41	1.75	1.28	52	65	47	Yes
CPT-076	1.83	2.39	1.51	68	89	56	Yes

7.7 Residential Foundations

7.7.1 High Dune Area

Geology Description

Twenty eight CPT were completed in the high dunes area during the current investigation. An additional eight CPT were completed during previous investigations. Twelve test pits were completed.

Materials within the upper 2 m of the deposits are comprised of fine to medium sand. Densities between ground level and 1.0 m below ground level range from very loose to medium dense (q_c = 1 to 5 MPa).

Static Assessment

Structural loadings associated with most house designs are generally low. Settlement of foundation materials is likely to be minor. The main concern is the minimisation of differential settlements.

Dynamic Assessment

Both liquefaction and lateral spreading are likely to affect the majority of structures in this area. Measures will be required to mitigate against major damage.

Suggested Foundation Design

Ground treatment may be required in some instances to mitigate against settlement. This includes excavation of the near surface materials to say 1.5 – 2m depth and recompacting them to form a stiffened raft of soil. In many areas of the development, area-wide filling is proposed. Careful engineering of these fills could form part of the lateral spreading hazard mitigate measures.

Structural solutions include tying foundations together as a raft e.g slab on grade possibly with post-tensioning of the slabs to improve their action to minimise the risk of differential movement between adjacent foundation supports

Care will need to be taken with basement garages to mitigate against the impacts of stormwater disposal to ground and floatation effects induced by high water tables or liquefaction.

7.7.2 Low Dunes Area

Geology Description

Fourty one CPT tests completed in the area surrounding the proposed lake excavation indicated that the subsurface geology is predominantly comprised of fine to medium sand with some layers of silts.

The density if *in situ* materials between ground level and the groundwater table are generally loose.

Static Assessment

Static stability of structures will be dependent on the preparation of the existing ground and the properties of the placed material.

Dynamic Assessment

Both liquefaction and lateral spreading are likely to affect the majority of structures in this area. Measures will be required to mitigate against major damage.

Suggested Foundation Design

Similar foundation design measures as described for the high dunes area are applicable here.

7.8 Stormwater Disposal Investigations

Nine sites identified for the disposal of stormwater were investigated, with one remaining site inaccessible due to the amount of clearing required within the pine plantation.

Testing was undertaken using the Double Ring Infiltrometer method.

All tests were completed in *in situ* dune deposits composed of fine to medium sand, after removal of overlying topsoil and other organic material. Test results are shown in Appendix F.

These tests indicate that permeability of these materials range from 1.7×10^{-5} m/sec to 1.0×10^{-4} m/sec.

Permeabilities derived from gradings indicate similar values.

Due to the uniform nature of the materials forming these deposits it is likely that these variations are due to the density of these materials.

Based on the data collected during past and present investigations at the Proposed Pegasus Town site a complete geotechnical model has been formed. Models of existing and predicted groundwater, material distribution, predicted liquefaction susceptibility and lateral spreading have been incorporated into the geotechnical model.

This geotechnical model indicates that the site can be separated effectively into three distinct areas based on the present-day geomorphology of the site. These are the Mapleham development site across which the site access is proposed, High Dunes Area (in plantation) and Low Dunes area (to the east of the High Dunes).

Geotechnical properties and distribution of materials in the High and Low Dune Areas are similar, whereas those in the Mapleham development are more variable due to the depositional environment.

The following is a summary of the geotechnical findings in the vicinity of the proposed town site (High and Low Dunes Area):

- Materials at the site predominantly comprise fine to medium sand. Within the upper 1.0 to 1.5 m these materials are generally loose to medium dense.
- Silty fine sand, sandy silt, clay and peat deposits generally occur between 2.0 and 3.0 m below ground level in the High Dunes and 1.0 to 2.0 m in the Low Dunes area. These range in thickness between 0.3 to 1.5 m thick and are generally very loose to loose.
- Located beneath the fine grained materials are alluvial gravel deposits of undetermined thickness.
- Groundwater levels range between about 1.0 and 2.5 m below ground level over much of the proposed site.

The following points are the key geotechnical issues that need consideration in final engineering designs:

- Liquefaction is predicted to occur to varying degrees over the majority of the site.
- Liquefiable materials generally occur at depths of 2 to 3 m below ground level. Deeper liquefiable materials are present at some locations.
- Predicted settlements during the 150 year event indicate that the majority of the site is susceptible to less than 100 mm of settlement. These values will vary dependent on the loading of specific structures. Maximum settlements are predicted during the 475 year event.
- Lateral spreading is predicted to occur around the edges of the proposed lake excavation and along the margin of the proposed wetland development on the east of the site, with effects decreasing with distance from the excavation edges.

Assessment of the Mapleham area indicates that material distribution is more complex and variable in this area of the site. Liquefaction susceptibility varies between areas of no susceptibility to areas with about 9 m of susceptible materials and predicted settlements in the order of 300 mm (SH1 intersection).

Based on the results of this investigation it is recommended that measures are included in the final design to mitigate against the effects of lateral spreading and liquefaction.

Lateral spreading, due to the excavation of the proposed lake and wetland, is likely to have the greatest influence on the proposed development. Treatment of materials directly adjacent to the excavations will be necessary to control movements towards the lake in the event of a large earthquake, as outlined in Section 7.1. Without adequate treatment damage is likely to occur to life-line services.

Liquefaction is likely to occur over the majority of the site. Based on the detailed design of specific structures ground improvement techniques such as vibro-floatation, stone columns or removal and replacement of liquefiable material may be utilised.

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Figures