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Geotechnical Completion Report Prestons South Stage M & N

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Appendices

1. Executive Summary

CDL Land New Zealand Limited is developing Stages M and N of the Prestons South Subdivision, located on Prestons Road, Christchurch. As part of the this work, a geotechnical completion report is required to ascertain that the site works have been carried out to the required standard and provide recommendations for building developments. This report describes earthworks and ground improvement involved with Stages M and N of the Prestons South Subdivision.

The client's brief indicated that the land shall be developed using an impact compactor with gravel embankments along the stormwater basins as ground improvement to raise the land performance to TC1 equivalent. Aurecon's role was to monitor the ground improvement quality assurance testing, which included CPTs. Assessment of the results indicates the required ground improvement has been achieved.

In addition to impact compaction and gravel embankment construction, extensive earthworks including cutting and filling have occurred on the site. The quality assurance testing of the engineered earthfill indicates that the earthfill placed within Stages M and N area has achieved the compaction levels as per NZS4431:1989.

Following completion of the earthworks and topsoil placement throughout the subdivision, a series of CPT tests were carried out to confirm the ground conditions. The purpose of the CPTs was to allow an assessment of the future land performance during large earthquakes and to determine the equivalent technical category of the land. Assessments of these results indicate the liquefaction deformation limits fit within those of TC1 and therefore we consider the site is likely to perform to the level of TC1.

From the monitoring and testing undertaken as part of the development of Stages M and N the following is concluded:

Certificate of Compliance

<u>Standard of bulk earthworks generally meet the earthworks specification and the applicable codes, including NZS4431:1989.</u>

Land Performance

<u>In line with the subdivision consent soil test results and following the ground improvement carried out as part of the site development, the residential lots within Stages M and N are likely to perform to a level equivalent to TC1 as per MBIE (2012).</u>

Building Considerations

<u>As the residential lots are likely to perform to a level of TC1 and the lots are underlain by earthfill that has achieved the compaction as per NZS4431:1989, we consider NZS 3604:2011 type foundations are suitable for light weight timber or steel frame buildings.</u> Within Stages M and N a building setback is present along the western boundary. No residential structures can be constructed within this area.

This report shall be read as a whole and our limitations are at the back of this report.

2. Introduction

2.1 Geotechnical Completion

CDL Land New Zealand Limited is developing Stages M and N of the Prestons South Subdivision, located on Prestons Road, Christchurch. The site works on Stages M and N have included ground improvement and bulk earthworks. As part of the this work, a geotechnical completion report is required to certify the site works have been carried out to the required standard and provide recommendations for building developments.

This report has been prepared for CDL Land New Zealand Limited and issued to Christchurch City Council (CCC). It describes earthworks and ground improvement involved with Stages N and M of the Prestons South Subdivision (see Figure 1 in Appendix A).

The purpose of the geotechnical completion report is to present the following:

- Summarise previous investigation information carried out as part of the subdivision consent and detailed design;
- Summarise the ground conditions and liquefaction risk;
- Extent of ground improvement and quality assurance testing of the ground improvement;
- Extent of earthworks on the lots and compliance testing of bulk earthworks;
- Summary of the findings, land technical category and recommendations for building development.

This report has been prepared based upon geotechnical data from observations and compaction testing during and after earthworks construction and ground improvements. All references to cut-fill depths are based on the original (pre 2011) ground levels.

This report shall be read as a whole. Our limitations are presented in Section 11.

2.2 Site Description

The Prestons Road subdivision is located on the northern fringes of Christchurch City. The site is made up of a series of adjacent properties forming an irregular and elongated rectangle shape, orientated approximately north to south. The total area of the overall Prestons Subdivision site is approximately 190ha. The site can be separated into two distinct blocks. Prestons North runs from the Lower Styx Road in the north through to Prestons Road in the south. Prestons South continues from Prestons Road, through to Mairehau Road to the south.

The focus of the geotechnical completion report is on Stages M and N of the Prestons South Subdivision. Stages M and N incorporate the north western corner of the Prestons South subdivision (see Figure 1 in Appendix A).

As part of the site development stormwater channel has been constructed, which run along a north-south alignment through Prestons South Stages M and N.

3. Pre-Development Geotechnical Work

3.1 Geotechnical Testing

The subdivision consent and detailed geotechnical design for the subdivision included an extensive series of geotechnical investigations. These comprised cone penetration tests (CPT), test pits, groundwater measurements and laboratory testing.

The details of these investigations are presented in the following Aurecon reports:

- *"Prestons Road Subdivision, Geotechnical Assessment Report for Resource Consent"*, Revision 2 dated 5 March 2012
- *"Prestons Road Subdivision, Detailed Geotechnical Design Report"*, Revision 2 dated 12 July 2012
- *"Prestons South Subdivision, Resource Consent Geotechnical Report"*, Revision 1dated 6 June 2013

The investigation tests carried out within Stages M and N of the Prestons South area are presented in Figure 2 in Appendix A.

3.2 Ground Conditions

From the extensive geotechnical investigations the ground conditions within Stages M and N area were defined into various geological areas. The location of the geological area within Stages M and N is presented in Figure 2 in Appendix A. The typical ground conditions in the area are presented in Table 1. We note the geological areas numbering is the same as those used in the geotechnical reports above.

Table 1: Typical ground conditions within Geological Area 1

Depth to Top of Unit (m)	Depth to Base of Unit (m)	Soil Unit
0	0.2 to 0.75	TOPSOIL.
0.2 to 0.75	3	SAND, loose to medium dense, with silty PEAT layers up to 0.3m thick within the upper 3m.
3	15+	SAND, medium dense to dense, becoming very dense with depth. Trace PEAT and SILT layers at depths of 10m+.

Groundwater levels ranged from 0.5m to 1.5m below ground level. During the site earthworks the above soil profile and groundwater levels was typically encountered within the area of interest.

3.3 Liquefaction Potential

As part of the geotechnical assessment and detailed design a liquefaction assessment was carried out. The details of the liquefaction assessments were presented in the above reports. The land categorisation was based on the Ministry of Business, Innovation and Development (MBIE), formerly the Department of Building and Housing (DBH), Technical Categories deformation performance limits are set out in Table 2.

Table 2: Technical category definitions and foundation implications (MBIE, 2012)

Technical Category	Liquefaction Deformation Limits				Likely Implications for House Foundations (Subject to individual assessment)
	Vertical		Lateral Spread		
	SLS	ULS	SLS	ULS	
TC1	15mm	25mm	nil	nil	Standard 3604-like foundation with tied slabs
TC2	50mm	100mm	50mm	100mm	DBH Enhanced Foundation Solutions
TC3	>50mm	>100mm	>50mm	>100mm	Site Specific Measures – Piles or Ground Improvement

The results from the liquefaction assessment indicated that the Prestons Subdivision can be classified as Technical Category 1 (TC1) and Technical Category 2 (TC2). In addition the presence of the new stormwater retention ponds presents a 'minor to major' lateral spreading hazard that was mitigates as per the following section.

3.4 Liquefaction Mitigation Measures

The requirement from the client was to form TC1 equivalent land for the entire subdivision development. Therefore to address liquefaction and lateral spreading potential the following methodologies were utilised.

Liquefaction

Part of the site was identified as TC1 while part of the site was identified as TC2. On-site trials with the Landpac impact compactor indicated that the underlying sand layers in the upper 3m of the soil profile could be densified using an impact roller. Thus, by densifying the ground the liquefaction potential can be minimised.

A detailed discussion of the trial and results are presented in *"Prestons Road Subdivision, Detailed Geotechnical Design Report"*, Revision 2 dated 12 July 2012 and *"Prestons South Subdivision, Resource Consent Geotechnical Report"*, Revision 1dated 6 June 2013. Based on these results, ground improvement using the Landpac impact roller has been carried out where TC2 land has been identified. The area treated is shown in Figure 3 in Appendix A.

Lateral Spreading

The construction of the stormwater retention ponds was identified as being a potential cause of lateral spreading in a large seismic event, even with ground improvement using the impact roller. As the liquefiable layers are typically in the upper 2.5m to 3m depth of the soil profile, it was considered more feasible to remove the liquefiable layers and form a compacted gravel embankment.

Lateral spreading requires the need for a continuous liquefiable layer through to the free face. By removing this continuous liquefiable layer and reinstating with a compacted gravel (non-liquefiable) material the lateral spreading potential affecting land adjacent to the ponds can be eliminated. Depending on the depth of the stormwater pond and the extent of liquefaction near each pond the gravel embankments ranged in width and depth.

A stormwater channel is present within Stages M and N, where gravel embankments have been constructed to mitigate the risk of seismically induced lateral spreading. The gravel embankments are discussed further in the Section 5.

4. Ground Improvement

4.1 Introduction

As part of our brief was to raise the performance of the land to an equivalent TC1, ground improvement has been undertaken on any area identified as TC2, within the Stages M and N.

Field trials identified that a Landpac impact compactor sufficiently densified the upper soil layer to a depth of 3m to 3.5m. The soil layers susceptible to seismically triggered liquefaction were located within the upper 3m of the soil column and therefore it was considered that ground improvement carried by Landpac can reduce the liquefaction susceptibility of these soils.

In this section we discuss the impact compactor methodology and quality assurance process used to ensure that ground improvement to the required level was being achieved. The area that has undergone ground improvement is presented in Figure 3 in Appendix A.

4.2 Methodology

Our detailed geotechnical assessment summarised in Section 3 identified that ground improvement could be carried out and a TC1 performance level achieved. The methodology carried out for ground improvement for Stages M and N comprised of the following:

- Use a Landpac Standard 3-Sided dual drum impact compactor, with a total energy input of 250kJ/m^2.
- Carry out 40 passes over the required area, in a staged approach.
- Use a water cart to wet the compaction area, as required, to improve workability.

During the ground improvement works, Landpac monitored the soil response (discussed below) to ensure that maximum compaction force is being applied to the ground. Where the maximum compaction force was not being applied, then all soft soil was stripped and either a compacted gravel working layer up to 300mm deep was placed or alternatively the natural sand soil was compacted with a conventional compactor, provided it was appropriate as a subgrade.

Prior to any impact compaction, pre-compaction CPTs were carried out to confirm the pre-existing soil densities. Once the required 40 passes were completed post compaction CPTs were carried out to confirm the extent of the ground improvement. Details of these results are presented in the following sections.

4.3 Quality Assurance

Quality assurance testing of the ground improvement was carried out using continuous impact response (CIR) and pre/post compaction CPTs. Each of these is discussed below.

4.3.1 Continuous Impact Response

Continuous Impact Response (CIR) technology was used to measure the relative soil response to the dynamic loads induced by the impact drums. The recorded soil response measured in g-values (deceleration) is used to identify sub-surface weak materials and indicate relative soil stiffness across the compaction areas.

The recorded g-values (deceleration) and the locations are presented in a plot with the g-values categorised by colours representing low (Red), medium (Yellow), high (Green) and very high soil (Blue) responses.

This provided a good index tool to determine if maximum applied compaction force was applied to the ground. An initial 5 passes with impact compactor would be carried out to provide a soil response. If low soil responses were identified then the soft soils were over excavated and either a compacted gravel working layer up to 300mm deep placed or alternatively the natural sand soil was compacted with a conventional compactor, provided it was appropriate as a subgrade.

CIR plots that cover Stages M and N are presented in Appendix B. Initial CIR plots were high with some medium areas. Final CIR plots were high with localised very high areas. This indicates that the maximum compaction force was being applied during the impact compaction process.

4.3.2 CPT

Assessment of the ground improvement was carried out using CPT tests. Prior to any impact compaction, pre-compaction CPTs were carried out to confirm the pre-existing soil densities. Once the required 40 passes were completed post compaction CPTs were carried out near the pre-compaction CPTs, offset by 2m to 5m, to confirm the extent of the ground improvement.

As the depth of influence for the impact compactor is approximately 3m and MBIE Guidelines (2012) recommend technical categorisation should be based on the upper 10m of the soil profile the pre-compaction and post compaction CPTs were taken to a depth of 10m. Pre-compaction CPTs are presented in Appendix C and post compaction CPTs in Appendix D. CPT locations are shown in Figure 3.

Pre and post compaction CPTs were compared by two methods in assessing the ground improvement. The first method included a comparison of the cone resistance between the pre and post compaction to see if there is any overall soil density increase in the upper soil profile. The second method was to run a liquefaction assessment on the pre and post compaction to confirm the likely liquefaction induced settlements prior to impact compaction and those following impact compaction. Result of each of these is discussed below.

It is noted that pre-compaction CPTs were undertaken along the northern boundary of Stage M (CPT653, 654 and 655) but as the liquefaction analysis of these CPTs indicate that this area was TC1 equivalent, no impact compaction of subsequent CPTs were undertaken. The pre-compaction CPT analysis results have been provided to confirm the technical category along this boundary.

a) Cone Resistance Comparison

A comparison of the CPT cone resistance for each CPT, pre and post compaction, is presented in Appendix E. The results indicate that the cone resistance in the upper 3m have increased.

b) Liquefaction Reassessment

Introduction

As technical categories are derived by liquefaction induced deformation limits, liquefaction assessment on the pre and post compaction CPTs have been carried out to determine the extent of liquefaction and the induced settlements.

Earthquake Cases

Earthquake induced ground acceleration and sustained shaking, leading to sufficient load cycles, is a requirement and a potential trigger of liquefaction. For the assessment we have reviewed three levels of seismic shaking.

1. Serviceability Limit State (SLS) design level earthquake, as defined by MBIE.
2. Intermediate design level earthquake, as defined by the subdivision consent.
3. Ultimate Limit State (ULS) design level earthquake, as defined by MBIE.

Each of these earthquake cases is discussed in detail below:

<u>Serviceability Limit State (SLS) Earthquake</u>

From the MBIE Guidelines, we have derived a Peak Ground Acceleration (PGA) of 0.13g for a SLS event with a Magnitude 7.5 earthquake.

<u>Intermediate Level (Int) Earthquake</u>

Subdivision consent conditions indicate that liquefaction mitigation measures for the subdivision infrastructure shall be designed for a 1 in 150 years period of return under the serviceability limit state (SLS) and as defined by NZS1170.5:2004.

Based on NZS1170.5:2004 for an Importance Level 2 (IL2) structure, with an increased Z hazard factor of 0.3, we have derived a PGA of 0.2g for a 1 in 150 year period of return. A Magnitude 7.5 has been assumed.

We note that this PGA is equivalent to the assumed SLS design level earthquake used for the liquefaction analysis as part of our assessment for the subdivision consent and detailed geotechnical design.

<u>Ultimate Limit State (ULS) Earthquake</u>

The MBIE Guidelines (2012) recommend a PGA of 0.35g for residential buildings in Christchurch. We have adopted this PGA value with a magnitude 7.5 earthquake for our ULS assessment.

The liquefaction analysis as part of our assessment for the subdivision consent and detailed geotechnical design used a PGA of 0.34g for ULS, which was based on NZS1170.5:2002. This is slightly less than recommended guidelines and as the difference is 0.01g we consider that this will not alter our original assessment or recommendations. However, to be in in line with current MBIE Guidelines we have used a PGA of 0.35g.

Liquefaction Methodology

In assessing the liquefaction potential, two methods have been used to assess the potential settlement for each of the design level events. Previous experience indicates that some methods can over predict liquefaction induced settlements and the use of two settlement prediction methods will give us a range of results. The two settlement prediction methods are discussed below.

Idriss and Boulanger Method

The liquefaction assessment was carried out using the method developed by Idriss and Boulanger (2008), in accordance with the MBIE Guidelines (2012) for residential properties. The assessment was carried out using an excel spread sheet developed by Aurecon. At this stage the Boulanger and Idriss (2014) method has not been considered, to keep the liquefaction assessment consistent with the previous assessments that have been undertaken for the greater Prestons subdivision consistent. We note that a comparison of the Boulanger and Idriss (2014) method against Idriss and Boulanger (2008) indicates that the Idriss and Boulanger (2008) method gives slightly more conservative results for the soil types encountered at the Prestons Subdivision, hence the method used for this report is slightly conservative.

The method of Robertson and Wride (1998) with the modified fines content was used to assess the liquefaction potential from the CPT results. The method of Zhang et al (2002) was used for estimating the liquefaction induced settlements from CPT results.

NCEER Method

Liquefaction assessments were carried out using the National Centre for Earthquake Engineering Research (NCEER) method as outlined by Youd et al. (2001), and recommended in the NZGS (2010) Guidelines. The assessment was carried out using Version 5 of the CivilTech Corporation LiquefyPro computer programme.

The method of Robertson and Wride (1998) with the modified fines content was used to assess the liquefaction potential from the CPT results. The method of Ishihara and Yoshimine (1992) has been used for calculating potential liquefaction induced settlements for CPT results.

A groundwater depth of 0.5m below finished earthworks level has been allowed. Testing information throughout Stages M and N indicates the groundwater level is typically greater than 1m depth (more likely to be at depths of 1.5m or greater) therefore a conservative groundwater level has been used for the assessment.

Liquefaction Assessment Results

Based on the design earthquake levels and methodologies, the liquefaction induced settlements for pre and post compaction CPT to 10m depth are presented in Table 3.

Table 3: Liquefaction induced settlements for pre and post compaction CPT to 10m depth

Earthquake Magnitude 7.5, Water Depth 0.5m												
CPTs	**SLS Design Event (0.13g)**				**Intermediate Design Event (0.20g)**				**ULS Design Event (0.35g)**			
	Settlement (mm)				Settlement (mm)				Settlement (mm)			
	NCEER		*Idriss & Boulanger*		*NCEER*		*Idriss & Boulanger*		*NCEER*		*Idriss & Boulanger*	
	Pre	*Post*	*Pre*	*Post*	*Pre*	*Post*	*Pre*	*Post*	*Pre*	*Post*	*Pre*	*Post*
CPT506	0	**0**	5	**0**	5	**0**	15	**5**	5	**5**	25	**10**
CPT507	10	**0**	25	**0**	15	**0**	45	**5**	30	**0**	55	**15**
CPT508	0	**0**	5	**0**	10	**0**	25	**5**	30	**5**	55	**10**
CPT509	0	**0**	0	**0**	0	**0**	5	**0**	5	**5**	20	**15**
CPT532	0	**0**	10	**0**	5	**0**	35	**0**	15	**0**	45	**5**
CPT533	5	**5**	5	**10**	10	**5**	20	**10**	20	**10**	30	**20**
CPT534	0	**0**	0	**0**	0	**0**	10	**10**	10	**5**	30	**15**
CPT540	0	**0**	10	**0**	10	**0**	30	**0**	15	**0**	35	**5**
CPT601	0	**0**	5	**0**	5	**0**	15	**5**	10	**0**	25	**5**
CPT602	0	**0**	10	**5**	10	**5**	35	**15**	20	**15**	55	**25**
CPT603	0	**0**	5	**0**	0	**0**	25	**5**	15	**5**	40	**15**
CPT604	0	**0**	10	**0**	5	**0**	30	**0**	20	**0**	45	**0**
CPT605	0	**0**	10	**0**	10	**0**	30	**5**	20	**0**	35	**10**
CPT606	5	**0**	15	**0**	15	**0**	30	**5**	25	**0**	30	**5**
CPT607	0	**0**	5	**0**	5	**0**	20	**5**	10	**0**	35	**5**
CPT608	10	**0**	15	**0**	15	**0**	30	**5**	20	**0**	45	**5**
CPT609	0	**0**	0	**0**	0	**0**	10	**5**	5	**0**	25	**10**
CPT611	25	**0**	30	**0**	30	**5**	40	**5**	35	**5**	40	**10**
CPT644	0	**0**	0	**0**	0	**0**	0	**0**	0	**0**	10	**5**
CPT653	0	**-**	0	**-**	0	**-**	0	**-**	0	**-**	5	**-**
CPT654	0	**-**	0	**-**	0	**-**	0	**-**	0	**-**	5	**-**
CPT655	0	**-**	0	**-**	0	**-**	0	**-**	0	**-**	5	**-**

Note: The settlements presented above are to the nearest 5mm. There are inherent assumptions in the analysis methods used that may cause the actual site settlements to vary from those calculated.

Results indicate that there is a significant decrease in the potential liquefaction settlements for the method used and the various earthquake design levels.

To compare these results with current MBIE Guidelines we have considered the post compaction assessment on the CPTs. Based on these results the results fit within the liquefaction deformation limits of TC1.

5. Gravel Embankments

5.1 Introduction

The construction of the stormwater channel through Stages M and N was identified as being a potential cause of lateral spreading in a large seismic event, even with ground improvement with the impact roller. As the liquefiable layers are typically in the upper 2.5m to 3m depth of the soil profile, it was considered more feasible to remove the liquefiable layers and form a compacted gravel embankment to eliminate the potential hazard in its entirety.

Lateral spreading requires the need for a continuous liquefiable layer through to the free face. By removing this continuous liquefiable layer and reinstating with a compacted gravel (non-liquefiable) material we can eliminate lateral spreading affecting land adjacent to the ponds.

5.2 Gravel Embankment Details

The design of the gravel embankment was carried out by Aurecon. The overall design of the gravel embankments are discussed in *"Prestons Road Subdivision, Detailed Geotechnical Design Report"*, Revision 2 dated 12 July 2012 and *"Prestons South Subdivision, Resource Consent Geotechnical Report"*, Revision 1dated 6 June 2013. The purpose of the gravel embankment is to remove the liquefiable soils adjacent to the pond, as lateral spreading requires a continuous liquefiable layer extending through to the pond edge.

Depending on the depth of the stormwater channel and the extent of liquefiable layers near the channel, the gravel embankment size and depth varied. Each gravel embankment was designed so the bulk of the embankment comprises compacted gravel with an overlying layer of compacted sand. This optimisation of design ensured that the core of the embankment resisting lateral spreading comprised gravel while the upper embankment profile was compacted sand.

The details of the embankments are provided in Table 4.

Table 4: Typical embankment profile for stormwater channel within Stages M and N

Stormwater Basin	Base RL of Gravel Embankments	Depth of Embankment	Width of Embankment from Crest	Depth of Compacted Gravel	Depth of Overlying Compacted Sand
Open Channel	RL 10.1m	3m to 3.15m	8.5m	2.5m	0.5m to 0.65m

5.3 Gravel Embankment Construction

The gravel embankment design required that a well graded sandy gravel material was used for the bulk of the embankment. Material used on site comprised of imported, well graded sandy gravel (AP100). The gravel was topped with variable thickness of clean, engineered sand fill. The earthworks specifications required that 98% of MDD for both the gravel and the overlying sand was achieved, to ensure that the required embankment design parameters were attained.

Site observations by Aurecon geotechnical and civil engineers confirm the gravel embankments have been constructed with imported, well graded sandy gravel overlain by a layer of compacted sand. In addition, the compaction quality testing discussed in Section 6 indicates that the required level of compaction has been achieved on site with the sandy gravel fill material and the overlying sand.

A review of as built earthworks information provided by the civil engineers indicates that the required width and depth of the gravel embankment profile has been achieved. Asbuilt plans for the gravel embankments are provided in Appendix F.

A review of post earthworks CPT information and liquefaction analysis, discussed in Sections 7 and 8, indicate that the gravel embankments were founded into non liquefiable soils.

Based on our intended design and the gravel embankment construction, we consider that the gravel embankments have been constructed appropriately and lateral spreading adjacent to the stormwater basins is unlikely. From a lateral spreading perspective the site is likely to perform to the level of TC1 requirements where the fully designed gravel embankments have been constructed.

6. Subdivision Earthworks

6.1 General

Bulk earthworks for Stage 1 of Prestons were carried out in accordance with the requirements of NZS 4404:2010, "*Code of Practice for Urban Subdivision*" and NZS4431:1989 "*Code of Practice for Earthfill for Residential Development*". The works comprised regrading of the site contours for the residential lots by predominantly engineered filling with minor areas of cutting.

On those occasions when quality control testing did not meet the specification, the Contractor was required to rework the fill to achieve the required compaction.

6.2 Areas of Cut and Fill

Site earthworks within Stages M and N have been predominantly fill with areas of cut. The fill material comprises both gravel and sand overlying a natural sand subgrade. A layer of topsoil overlies the fill material. Extent of cutting and filling is shown in Figures 4 in Appendix A.

Within Stages M and N a 15m building setback is present along the western property boundary. The building setback was a planning requirement due to the presence of the neighbouring properties. As a building setback is present and no structures can be constructed within this area, non-engineered fill has been placed along a 5m width adjacent to the boundary.

6.3 Compaction Quality Control Testing

Independent testing of earthfill compaction was carried out by Coffey International Limited (Coffey). Testing was carried out using a Nuclear Densometer (NDM). The acceptance criterion was based on the Prestons Subdivision earthworks specification as follows:

- Compaction of fill is to be in accordance with NZS 4431: 1989.
- Compaction standard is 95% Maximum Dry Density (MDD) for all areas of bulk filling.
- The gravel embankments around the stormwater basins required a higher standard of 98% MDD.

Fill material comprised of predominantly site-won sand with some gravel fill. Compaction curves for each of the fill material are presented in Appendix G.

The MDD from the compaction curves were used to determine the level of compaction required for the fill material. The results of the NDM testing from Coffey are presented in Appendix G and the NDM testing locations are presented in Figure 5 in Appendix A. A summary of these NDM results are presented in Appendix H.

Coffey conducted the compaction tests at a test frequency of approximately 1 test per 1,000m^3.

6.4 Compaction Results

The results presented in Appendix F indicate that 95% MDD or greater compaction has been consistently achieved in the areas of bulk fill and that 98% MDD or greater compaction has been consistently achieved in the gravel embankment areas. From these results and our site observations we confirm that all the earthfill placed within Stages M and N area has achieved the required compaction.

6.5 Bulk Excavated Areas

As part of the site earthwork, bulk excavation was undertaken in two areas within Stage M and N to remove relatively thick peat and organic layers present at shallow depths. The extent of the excavated areas is shown on Figure 4 in Appendix A. The excavations were taken to depth of 1.5m to 1.8m below the original ground level. The bulk excavation was backfilled with sand which was then compacted using the impact compactor.

Quality assurance testing of the sand placement was undertaken using CPTs and DCP (dynamic cone penetration tests) undertaken by both Aurecon and the contractor. The testing indicated that the sand has been compacted to an appropriate level. In addition, a number of the post earthworks and verification CPTs were undertaken in these bulk excavated areas once the earthworks was completed to confirm site ground conditions and site performance, which are discussed in the following sections.

7. Post Earthworks CPT

7.1 Introduction

Following completion of the earthworks and topsoil placement throughout the subdivision, a series of CPT tests have been carried out to confirm the ground conditions. The CPTs have been carried out throughout Stages M and N of the Prestons South subdivision, whether it is within the ground improvement area or not.

The frequency of the CPT testing carried out was one test per hectare. For Stages M and N post earthworks assessment. The post filling CPTs are presented in Appendix I and the locations are shown in Figure 6 in Appendix A.

The purpose of the CPTs were to allow an assessment of the land technical category further to that already undertaken as part of the subdivision consent, detailed geotechnical design and ground improvement quality assurance testing.

7.2 Liquefaction Assessment

To allow an assessment of the land technical category, a liquefaction assessment has been carried out on the post filling CPTs. The liquefaction analysis methodologies and earthquake design cases used to assess these CPT results are the same as those detailed in Section 4.3.2. The CPT analysis has been done to a depth of 10m, as this is the required depth in the MBIE Guidelines for technical category assessment.

In addition to determining the liquefaction induced reconsolidation settlement we have assessed the potential for liquefaction induced ground damage based on the Liquefaction Severity Number (LSN), as defined by Tonkin and Taylor (2013). Other ground damage potential methods (such as Ishihara, 1985) were assessed but LSN was considered the more appropriate method. Tonkin & Taylor (T&T) developed the Liquefaction Severity Number (LSN) based on investigation data and observations made following major earthquake events in Christchurch. The LSN number is an index number which qualitatively assesses the effects of liquefaction on a site and on a shallow founded building. The LSN number is calculated by the equation below.

$$LSN = 1000 \int \frac{\varepsilon_v}{z} . dz$$

Where: $\varepsilon_v = \text{volumetric reconsolidation strain}$

$z = \text{depth of liquefaction below ground level}$

The LSN number is likely to be a better index of surface damage than reconsolidation settlement because the LSN number is affected more by shallow liquefaction and less by liquefaction at depth, which is less likely to affect the ground surface or shallow founded buildings. Reconsolidation settlement places the same weighting on deep liquefaction as shallow liquefaction, even though settlement will have less impact at the ground surface with increasing depth. LSN numbers have been correlated to observed liquefaction effects during recent earthquakes in Christchurch as shown in Table 8 below.

Table 5: LSN Ranges and Observed Effects (Tonkin and Taylor, 2013)

LSN Range	Predominant Performance
0-10	Little to no expression of liquefaction, minor effects
10-20	Minor expression of liquefaction, some sand boils
20-30	Moderate expression of liquefaction, with sand boils and some structural damage
30-40	Moderate to severe expression of liquefaction, settlement can cause structural damage
40-50	Major expression of liquefaction, undulations and damage to ground surface, severe total and differential settlement of structures
>50	Severe damage, extensive evidence of liquefaction at surface, severe total and differential settlements affecting structures, damage to services

When compared to the broad descriptions of expected land performance in TC1, TC2 and TC3, as outlined in Section 3.3, the LSN number can be approximately correlated to technical categories as follows:

- TC1 = $LSN_{(ULS)}$ < 10
- TC2 = $LSN_{(SLS)}$ < 20 and $LSN_{(ULS)}$ < 30
- TC3 = $LSN_{(SLS)}$ >20 or $LSN_{(ULS)}$ > 30

A groundwater depth of 0.5m below finished earthworks level has been allowed. Testing information throughout Stages M and N indicates the groundwater level is typically greater than 1m depth (more likely to be at depths of 1.5m or greater) therefore a conservative groundwater level has been used for the assessment.

The results for the liquefaction induced reconsolidation settlement are presented in Table 6. The results for the liquefaction induced ground damage potential (based on LSN numbers) are presented in Table 7.

The results indicate the liquefaction deformation limits fit within those of TC1 and therefore we consider the site is likely to perform to the level of TC1 requirements. The results indicate that no expression of liquefaction in the SLS case and little to no expression of liquefaction in the ULS case. This is consistent with the definition for TC1. CPTPF3 has LSN number slightly higher than what we consider is appropriate for TC1 however the likely liquefaction induced reconsolidation settlement is well within that required for TC1 and hence the ground performance is likely to be equivalent to TC1.

Table 6: Liquefaction induced settlements for post filling CPTs to 10m depth

Earthquake Magnitude 7.5, Water Depth 0.5m, 10m Analysis						
CPT	SLS Design Event (0.13g)		Intermediate Design Event (0.20g)		ULS Design Event (0.35g)	
	Settlement (mm)		Settlement (mm)		Settlement (mm)	
	NCEER	*Idriss & Boulanger*	*NCEER*	*Idriss & Boulanger*	*NCEER*	*Idriss & Boulanger*
CPTPF1	0	>5	>5	5	5	15
CPTPF2	0	0	0	>5	>5	5
CPTPF3	>5	5	>5	5	5	5
CPTPF4	5	10	10	10	15	20
CPTPF5	5	10	10	15	15	25
CPTPF6	>5	>5	>5	5	>5	10
CPTPF7	0	0	>5	>5	>5	10
CPTPF8	0	0	0	>5	>5	15
CPTPF9	0	0	0	>5	>5	15
CPTPF10	0	>5	>5	10	10	20
CPTPF11	0	0	0	>5	0	15
CPTPF12	0	0	0	0	0	0
CPTPF21	0	>5	>5	5	>5	15
CPTPF22	0	0	0	0	0	5
CPTPF23	0	0	0	>5	0	5
CPTPF26	0	0	>5	>5	5	10
CPTPF27	>5	>5	5	10	10	15
CPTPF28	0	0	0	<5	<5	5
CPTPF29	0	0	0	0	5	10
CPTPF30	0	0	0	6	5	10

Note: The settlements presented above are to the nearest 5mm. There are inherent assumptions in the analysis methods used that may cause the actual site settlements to vary from those calculated.

Table 7: LSN for post earthworks CPTs to 10m depth

Earthquake Magnitude 7.5, Water Depth 0.5m			
CPTs	SLS Design Event (0.13g)	Intermediate Design Event (0.20g)	ULS Design Event (0.35g)
	LSN	*LSN*	*LSN*
CPTPF1	1	4	6
CPTPF2	0	1	3
CPTPF3	2	4	17
CPTPF4	3	4	5
CPTPF5	1	2	3
CPTPF6	1	2	6
CPTPF7	0	0	1
CPTPF8	0	1	5
CPTPF9	0	0	3
CPTPF10	2	4	6
CPTPF11	0	2	5
CPTPF12	0	0	0
CPTPF21	1	3	6
CPTPF22	0	0	1
CPTPF23	0	1	3
CPTPF26	0	0	2
CPTPF27	4	10	14
CPTPF28	0	4	7
CPTPF29	0	0	1
CPTPF30	1	5	8

8. Verification CPT

8.1 Introduction

After at least one month following the post earthworks CPTs a series of verification CPTs were carried out throughout Stages M and N of the Prestons South subdivision, whether it is within the ground improvement area or not. These CPTs have been given the nomenclature CPTV - CPT Verification.

The purpose of the CPTs was to allow further assessment of the land technical category by testing areas previously not covered, as well as confirming whether there was strengthening of the ground over time following the ground improvement and site earthworks.

In total three CPTs were carried out within and adjacent to Stages M and N. The verification CPTs are presented in Appendix J and the locations are shown in Figure 6 in Appendix A.

8.2 Liquefaction Assessment

To allow an assessment of the land technical category and possible ground improvement we have carried out a liquefaction assessment on the verification CPTs. The liquefaction analysis methodologies and earthquake design cases used to assess these CPT results are the same as those detailed in Section 4.3.2 and 7.2. The CPT analysis has been done to a depth of 10m, as this is the required depth in the MBIE Guidelines for technical category assessment.

A groundwater depth of 0.5m below finished earthworks level has been allowed. Testing information throughout Stages M and N indicates the groundwater level is typically greater than 1m depth (more likely to be at depths of 1.5m or greater) therefore a conservative groundwater level has been used for the assessment.

The results for the liquefaction induced reconsolidation settlement are presented in Table 8. The results for the liquefaction induced ground damage potential (based on LSN numbers) are presented in Table 9. Based on the result we consider that overall there does not appear to be strengthening of the ground over time, however the liquefaction deformation limits typically still fit within those of TC1 and therefore we consider the site is likely to perform to the level of TC1 requirements. CPTV1 CPTV4 has LSN number slightly higher than what we consider is appropriate for TC1 however the likely liquefaction induced reconsolidation settlements are well within that required for TC1 even with conservative groundwater levels and hence the ground performance is likely to be equivalent to TC1.

Table 8: Liquefaction induced settlements for verification CPTs to 10m depth

Earthquake Magnitude 7.5, Water Depth 0.5m, 10m Analysis						
CPT	SLS Design Event (0.13g)		Intermediate Design Event (0.20g)		ULS Design Event (0.35g)	
	Settlement (mm)		Settlement (mm)		Settlement (mm)	
	Ishihara & Yoshimine	*Idriss & Boulanger*	*Ishihara & Yoshimine*	*Idriss & Boulanger*	*Ishihara & Yoshimine*	*Idriss & Boulanger*
CPTV1	0	5	5	10	10	25
CPTV2	0	0	0	0	0	0
CPTV3	0	5	5	10	10	15
CPTV4	0	0	0	5	5	20
CPTV5	0	0	0	0	0	0
CPTV6	0	0	0	5	5	10
CPTV7	0	0	0	5	5	10
CPTV8	0	5	5	15	15	25
CPTV9	0	0	0	5	5	10

Note: The settlements presented above are to the nearest 5mm. There are inherent assumptions in the analysis methods used that may cause the actual site settlements to vary from those calculated.

Table 9: LSN for verification CPTs to 10m depth

Earthquake Magnitude 7.5, Water Depth 0.5m			
CPTs	SLS Design Event (0.13g)	Intermediate Design Event (0.20g)	ULS Design Event (0.35g)
	LSN	*LSN*	*LSN*
CPTV1	4	9	15
CPTV2	0	0	0
CPTV3	2	4	6
CPTV4	0	5	14
CPTV5	0	0	1
CPTV6	0	2	3
CPTV7	1	3	4
CPTV8	2	6	9
CPTV9	0	0	2

9. Building Development

9.1 Technical Category

Extensive geotechnical testing has been carried out as part of the subdivision development. The testing indicates the lots within Stages M and N are likely to perform to the level equivalent to TC1.

9.2 Earthworks on Building Lots

The extent of earthfill on the lots in Stages M and N is shown on Figure 4 in Appendix A.

The fill areas have been constructed using materials and processes that have been randomly measured by independent testing. The testing shows that the placement of filling is generally in accordance with the specification and relevant standards.

9.3 Soil Suitability Criteria

Section 3 of New Zealand Standard NZS 3604:2011 "*Timber Framed Buildings not requiring specific Engineering Design*" provides several criteria for defining foundation soil suitability for lightweight timber or steel framed residential buildings.

Clauses 3.1.3 and 3.3 provide criteria for determining strength and suitability of founding soils.

Clauses 3.4.1 and 3.4.2 discuss depths to founding. For purposes of this report, we have interpreted these clauses as meaning that for sound bearing at depths of 200mm to 600mm, standard shallow type foundations can be utilised. For depths greater than this, specific foundation designs could be used or alternatively excavations can be backfilled to the required level with 10MPa site concrete or compacted hardfill. In line with the client's brief Aurecon undertook site specific investigations on each residential lot and we have prepared a site specific geotechnical report addressing the foundation requirement. The testing data for the lot specific investigations has been uploaded to the Canterbury Geotechnical Database.

9.4 Building Considerations

As the land is likely to perform to a level of TC1 and a number of the lots are underlain by earthfill that has achieved the required compaction, we consider NZS 3604:2011 type foundations are suitable.

We note that at the time of writing this report the location and structural form of the future dwelling on the lots are unknown and our recommendations relate to NZS3604:2011 type lightweight timber or steel framed residential buildings only.

9.5 Building Setback

Within Stages N and M a building setback is present along the western property boundary. No residential structures should be constructed within this area.

9.6 Future Earthworks

We do not anticipate that future earthworks will be required on the majority of the lots however should such work be required the following should be noted.

- All earthworks should be carried out in accordance with the Health and Safety and Employment Act 1992 and the Department of Labour approved Code of Practice for Safety in Excavations and Shafts for Foundations, 1995.
- Cuts that exceed 0.6m high around any of the house sites must be retained by a suitable retaining wall designed by a Chartered Professional Engineer.
- We recommend that no more than 450mm of fill is placed on the allotment without detailed engineering design.
- Fill should not be placed adjacent to any timber retaining wall.
- Any development where excavations greater than 1.2m in depth are proposed, must be subject to specific investigation and design to confirm these works will have no adverse effect on land stability, infrastructure and/or structures on adjacent lots. Excavations near sensitive structures or near boundaries may require geotechnical engineering input even if shallower than 1200mm.

9.7 Stormwater

All stormwater collected by impermeable surfaces (dwelling and pavement) and grassed areas shall be collected by lined channel drains and sumps etc. and be piped away from the lots to discharge into the Council vested services.

9.8 Construction Observations

The suitability of foundation conditions must be verified at the time of construction (refer Requirements of NZS 3604:2011). Foundation inspections by a Building Inspector or a Chartered Professional Engineer who are familiar with this report must be carried out to ensure the adequacy of the foundation subgrade prior to the placement of granular hardfill or the construction of foundations.

10. References

Boulanger R.W. and Idriss, I.M., 2014. *CPT and SPT based Liquefaction Triggering Procedures.* Center for Geotechnical Modelling Report No. UCD/CGM-14/01, Department of civil and Environmental Engineering, College of Engineering, University of California at Davis.

Christchurch City Council, 2010. *Infrastructure Design Standards - Part 4: Geotechnical Requirements*.

Idriss and Boulanger, 2008. *Soil Liquefaction during Earthquakes*. EERI Monograph Series MNO-12

Ishihara, 1985. *Stability of natural deposits during earthquakes*. Proceedings, 11th International Conference on Soil Mechanics and Foundation engineering, Vol 1, pp. 321-376.

Ishihara and Yoshimine, 1992. *Evaluation of settlements in sand deposits following liquefaction during earthquakes*. Soils and Foundations, Vol. 32, No. 1, pp. 173-188.

Ministry of Business Innovation and Employment (MBIE), 2012. *Repairing and rebuilding houses affected by the Canterbury earthquakes*.

NZGS, 2005. *Guidelines for the Classification and Field Description of Soils and Rocks in Engineering*. NZ Geotechnical Society Inc, Wellington, New Zealand.

NZGS, 2010. *Geotechnical earthquake engineering practice, Module 1 – Guideline for the identification, assessment and mitigation of liquefaction hazards*. NZ Geotechnical Society Inc, Wellington, New Zealand.

NZS1170.0:2002. *Australia/New Zealand Standard, Structural Design Actions, Part 0: General Principals.* Standards New Zealand, Wellington, New Zealand.

NZS1170.5:2002. *Australia/New Zealand Standard, Structural Design Actions, Part 5: Earthquake Actions – New Zealand.* Standards New Zealand, Wellington, New Zealand.

NZS 3604:2011. *Timber Framed Buildings*. Standards New Zealand, Wellington, New Zealand.

NZS 4404:2010. *Land development and subdivision infrastructure*. Standards New Zealand, Wellington, New Zealand.

NZS 4431:1989. *Code of practice for earth fill for residential development*. Standards New Zealand, Wellington, New Zealand.

Robertson and Wride, 1998. *Evaluating cyclic liquefaction potential using the cone penetration test*. Canadian Geotechnical Journal, Vol. 35, pp. 442 – 459.

Tonkin and Taylor (2013) *Liquefaction Vulnerability Study*, Tonkin and Taylor Report 52020.0200/v1.0. February 2013. 52 pages and 14 appendices.

Youd et. al., 2001. *Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshop on evaluation of liquefaction resistance of soils*. Journal of geotechnical and geoenviromental engineering. Volume 127, Issue 10, pp. 817-833.

Zhang, Robertson, and Brachman, 2002. *Estimating liquefaction-induced ground settlements from CPT for level ground.* Canadian Geotechnical Journal, Vol. 39, pp.1168 – 1180.

Zhang, Robertson and Brachman, 2004, *Estimating Liquefaction Induced Lateral Displacements using the SPT and CPT.* ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 130, No. 8, 861-871

11. Limitations

This report has been prepared for CDL Land New Zealand Limited. It may be made available to others but only in full. As noted above, it shall not be used by any person as a substitute for specific field observations and testing once house sites are confirmed.

This report has been prepared as part of the development of the Prestons South Stages M and N Subdivision. It has been prepared to provide the following information:

- To report on the management of the earthworks during construction, including compaction standards of fills.
- To report on the extent of ground improvement and the resulting land technical category.

This report does not remove the responsibility of the Owner / Builder / Building Certifier to satisfy themselves of foundation depth and suitability at the finally selected house location.

Subsurface conditions relevant to construction works should be assessed by experienced contractors and designers who can make their own interpretation of the factual data provided. They should perform any additional tests as necessary for their own purposes. Subsurface conditions, such as groundwater levels, can change over time. This should be borne in mind, particularly if the report is used after a protracted delay or in wet weather.

It is strongly recommended that any plans and specifications prepared by others and relating to the content of this report, or amendments to the original plans and specifications, are reviewed by Aurecon to verify that the intent of our recommendations is properly reflected in the design. During construction we request the opportunity to review our interpretations if the exposed site conditions are significantly different from those inferred in this report.

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