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Geotechnical and Civil Engineers

GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
CNR VICTORIA/VIVIAN/WILLIS STREETS
WELLINGTON

Prepared for:
Miro Street Developments Limited

OUR REF 12323
December 2019
REV A

GEOTECHNICAL INVESTIGATION
PROPOSED DEVELOPMENT
CNR VICTORIA/VIVIAN/WILLIS STREETS
WELLINGTON

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out on the block of land bounded by Victoria, Vivian and Willis Streets, Wellington. The purpose of the investigation was to assess the suitability of the site for a multi-unit development which will occupy the entire site.

The investigation has been undertaken at the request of Egmont Dixon Limited on behalf of Miro Street Developments Limited. A signed and returned copy of a Shortform Agreement sets out the terms of conditions and the planned scope of work that was described in ABUILD™'s proposal letter dated September 12, 2019.

To assist us with this study we were provided with the following information:

- Architectural plans A3.50, A3.52, A3.60 and A3.62 showing plans and sections through the proposed development.
- Architectural drawings showing a plan view and aerial view of the proposed development.
- Aurecon topographical survey plan of the site showing spot levels, ref drawing 507237-0001A.

2.0 INVESTIGATION

The investigation was initially commenced on September 15, 2019 and was subject to a significant delay whilst the location of an interceptor drain that dissected the site was confirmed by Wellington Water. The investigation was completed on October 24, 2019.

The investigation was carried out under the direction of a geotechnical engineer from our office and comprised the putting down of four boreholes to depths of between 12.45 and 15.94 metres below ground level using a track mounted rig provided by Griffiths Drilling Limited. The boreholes were advanced by wash drilling techniques with frequency in situ testing and sampling to the depths explored. Logs of the boreholes were compiled in accordance with NZ Geotechnical Society Field Description of Soils and Rocks (2005).

The locations of the boreholes are shown on the attached Investigation Location Plan drawing 12323-S1A presented in Appendix A, together with the borehole logs.

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An unnumbered architectural plan showing the layout of the various apartment blocks is presented in Appendix B.

3.0 SITE CONDITIONS

The site is presently in use as a carparking area managed by Wilson Parking with the south western part of the site is occupied by an existing building. The site is tarsealed and is accessed from both Willis Street and Victoria Street.

The site falls from Willis Street towards Victoria Street. The topographic plan shows the following spot levels:

Location	Level (m)
North west corner	19.45
South west corner	21.36
North east corner	17.12
South east corner	19.33

4.0 SUBSOIL CONDITIONS

Subsoil conditions have been identified in the four investigation boreholes and a relatively deep borehole put down for OPUS just off the north west corner of the site. This borehole extended to a depth of 35.5 metres below ground level.

Filling comprising variable medium dense to dense granular soils is inferred to exist to depths of between 2.4 and 3.5 metres in BH2 and BH3 respectively put down adjacent to Willis Street.

On the other side of the site BH1 and BH4 encountered inferred natural ground beneath the surfacings.

The general lithology comprises gravelly soil of inferred colluvial origin which is interbedded at various levels with clay/silt soils with sandy/discrete sand layers. The clay/silt soils range stiff to very stiff and hard with SPT N values in this material having a low SPT N value = 5 in a layer of clay at a depth of 11.9 metres in BH4 through to an SPT N = 13 in BH2 at a depth of 9.7 metres below ground level.

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The gravelly colluvial soils encountered in all boreholes comprise typically fine to medium gravel with numerous broken faces indicating that coarse angular gravels also exist throughout the gravel deposits. The gravel deposits are typically dense to very dense.

Groundwater was measured in all boreholes and ranged from 1.8 metres below ground level in BH1 to 3.0 metres depth in BH2 adjacent to Willis Street. The water levels are likely to have been influenced by drilling water and also the variability of subsoil conditions. The groundwater level measured in BH2 is in permeable gravels and likely to be a more accurate representation.

5.0 FAULTING

The closest active fault to the site and source of design earthquake shaking in the region is the northeast trending Wellington Fault, which is mapped as passing approximately 1.5 kilometres to the northeast of the site within a roughly 45 metre wide fault rupture hazard zone.

The Wellington Fault is a strike slip fault with a dextral sense (movement to the right), a characteristic magnitude of 7.6 (±0.3), and an average horizontal displacement of 3.5 to 5.0 metres, with a lesser and variable amount of vertical displacement (Begg & Mazengarb, 1996). GNS Science has described the activity of the fault as Class 1 with a recurrence interval of 790 - 900 years. There is an estimated elapsed time since the last movement of the fault of between 170 – 310 years, a perceived 11% probability of rupture in the next 100 years, and an average movement on the fault of 5.8 mm/year (Rhoades et. al., 2010).

6.0 SOIL STRENGTH PARAMETERS

The variability of subsoils across the site implied by the investigation boreholes precludes an accurate compilation of a soil model and soil strength parameters are appropriate for soil stiffnesses/densities encountered at various locations across the site:

Material Type	Soil Strength Parameters		
	Cohesion (c) (kPa)	Internal Friction (Φ) (°)	Soil Density (γ) (kN/m^3)
SILT very stiff	5 – 10	30 – 32	20
Silty CLAY	5 – 8	30	20
Medium dense GRAVEL	2 – 3	35 – 37	19 – 21
Dense GRAVEL	2 – 3	>40	21

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7.0 ENGINEERING DISCUSSION AND RECOMMENDATIONS

7.1 General

Opinions and recommendations contained herein are based on four (4) boreholes. Inferences about the nature of the subsoils away from the points explored are made, but it must be appreciated that actual ground conditions may vary from the assumed profile.

The proposed development comprises a total of nine apartment blocks between three to four storeys in height. Access to the complex will be gained off Willis Street and Victoria Street.

The apartment blocks are to be separated by east and west laneways and a shared laneway extending from Willis Street through to Victoria Street.

Finished floor levels will vary for each of the apartment blocks and will be achieved with both cutting and filling. Aspects of earthworks construction are discussed in the following sections.

Subgrade soils for each of the apartment blocks will likely expose variable soils having variable strengths/densities. Under static ground conditions all structural elements may be supported at shallow depth with possibly some subgrade pretreatment required to improve the strength/density and consistency of the subgrade soils across the various foundation alignments. We have assessed likely settlement under serviceability loads and also indicative differential movement for the various apartment blocks.

Loads generated from design earthquake shaking will likely impose increased tension and compressive forces that may be resisted by pile foundations. Ground anchors have also been considered to resist design uplift loads.

With regards to pile foundations that may be required to resist compression and tension, the actual depth of any piling may be influenced by the existing interceptor that dissects the site and the depth of the piles will also be governed by the magnitude of the design loads.

These aspects are considered in the following sections together with retaining walls, site liquefaction potential, and pavement design parameters.

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7.2 Site Development

The architectural drawings indicate that the apartment buildings will be at different levels, with finished floor levels (FFL) ranging from RL 21.45 metres to RL 17.8 metres. There are nine blocks of apartments, numbers F1 to F9 as shown on an unnumbered architectural plan.

The FFLs are achieved by both cutting and filling with F7 to F9 having split floor levels.

We have compared the FFL with spot levels as shown on Aurecon's topographic survey drawing to establish an average existing ground level for each of the apartment blocks. From this information we have calculated an average depth of cutting/filling for each of the apartment blocks as follows:

Block ID	Finished Floor Level (FFL) (RL)	Existing Ground Spot Levels (m)	Cutting (m)	Filling (m)
F1	21.45	20.7	-	0.7
F2	21.00	20.3	-	0.7
F3	19.30	19.8	0.5	-
F4	20.85	20.1	-	0.8
F5	20.25	19.8	-	0.5
F6	18.85	19.2	0.4	-
F7	19.93* 21.00	20.2	0.2	0.8
F8	21.10* 18.45	19.4 19.0	- 0.5	1.7 -
F9	18.45* 17.80	18.0 18.2	- 0.4	0.54 -

*split level apartment blocks

The depth of cutting and filling is indicative only and will depend on the need to remove unsuitable soils to expose a clean sound base on which to construct the filling.

7.3 Foundations Under Static Loading

7.3.1 Shallow Foundations

Foundation subgrade conditions will likely vary significantly across the site and beneath the footprint of each of the apartment blocks.

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Based on subsoil conditions across the site, and the depth of filling and cutting required to achieve design levels for each of the apartment blocks, we have established that there are broadly three subsoil conditions that would be expected to be encountered beneath the footprint of the apartment blocks:

1. **Apartment buildings F1, F4 and F7 (adjacent to Willis Street)**
   These apartment buildings will generally have between 0.7 and 0.8 metres of filling above the existing ground level with the exception of the southern part of F7 where cutting of 0.2 metres is proposed.

   Foundation walls/load bearing walls around these apartment buildings will likely derive support from within the gravel fill layer but subject to strength/density consistency across the foundation alignments.

2. **Apartment buildings F2, F5 and F8**
   The apartment buildings are expected to be situated over variable soils with F2 subsoils likely to be those encountered in BH4, whilst BH1 subsoils likely reflect conditions, with some variability, beneath F5 and F8.

   The FFL at F2 is 700 mm above the existing average ground level beneath the footprint of this building and the FFL at F5 is 0.5 metres above the existing ground level. F8 is split level with part of the building having an FFL approximately 1.7 metres above ground level and the eastern part of this building having a cut of 0.5 metres.

   The gravelly silt at BH1 is very stiff and the likely performance of F5 and F8 may be influenced by the presence of the wood fibre encountered between 0.9 to 1.2 metres depth. The thickness, depth and extent of this layer must be investigated.

3. **Apartment buildings F3, F6 and F9**
   These apartment buildings are adjacent to Victoria Street. Subsoil conditions are likely to be exposed following proposed cutting of between 0.4 to 0.5 metres, and have been based on BH1 subsoils. Subsoil conditions are likely to comprise very stiff silt with a zone of interbedded wood fibre. The very stiff silt represents a satisfactory subgrade on which to support foundations but subject to the thickness, depth and extent of the wood fibre beneath any foundations.

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The subsoils described above are capable of satisfactorily supporting all structural elements at shallow depth under static ground conditions following proof rolling of the building platforms.

Based on the borehole logs and the variability of the subsoils and founding levels for each of the apartment blocks, there is justification to present a unique ultimate bearing capacity (Q_{ULT}) for each of the blocks, however, for simplicity and design efficiency we recommend that foundations for each of the apartment blocks be designed using an ultimate bearing capacity Q_{ULT} = 450 kPa. This value must be factored appropriately.

All foundation subgrades must be tested to confirm subsoil consistency and strength of materials within the zone of stress influence beneath the underside of all foundations.

Testing will depend on the type of material exposed at subgrade level. For fine grained soils testing shall be by shear vane to assess the undrained shear strength of the soils. For coarse grained soils, ie sand/gravel, a penetrometer probe shall be used to measure the penetration resistance and hence density of the subsoils.

Target values for both tests are as follows:

- Undrained shear strength (as measured by a hand held shear vane)
  S_U = 150 kPa

- Penetrometer resistance
  10 blows/150 mm

Testing shall extend for a depth of not less than 2B (where B = width of foundations).

Fine grained soils are sensitive to remoulding and loss of strength if wetted, therefore all exposed subgrade soils shall be protected at all times. If fine grained soils become softened as a result of inclement weather then undercutting may be required and replacement with compacted hardfill as appropriate.

7.3.2 Settlement of Shallow Foundations

Under serviceability loading some settlement of foundations may be expected to occur.

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Foundations situated on a gravel/sand subgrade will likely settle elastically and by definition this settlement is expected to occur under application of load. Consolidation settlement associated with nominally consolidated clay/silt soils may occur over a period of time and is a function of soil stiffness. Settlement will depend on the type and stiffness/density of subsoils within the stressed zone of influence beneath any shallow foundation.

Estimates of settlement should be assessed when foundation geometry is known and when specific subsoil conditions are confirmed.

Differential settlement associated with total predicted settlement is expected to be relatively low for symmetrical buildings situated on consistent and competent subgrade soils.

7.3.3 Pile Foundations

Design loads, depending on their magnitude, could be supported on pile foundations that derive support from the underlying gravel deposits encountered in all boreholes. The gravel layer which is interbedded with fine grained soils, particularly in BH1 and BH2, extends to depths of between 2.4 to 9.7 metres in BH1, and to depths of between 4.6 and 15.5 metres in BH2.

All pile foundations would comprise prebored cast in place concrete piles that would socket into the gravel layer. An optimum pile bearing capacity is achieved when a pile has consistent dense material for a height of 4B above the pile base and by a depth of 2B below the pile base.

For bored piles that extend to a depth of nominally 6.0 metres below ground level in the vicinity of BH4 (ie apartment blocks F5, F6 and F9), we have assessed an ultimate pile bearing capacity of Q_{ULT} = 2,500 kPa. This value is based on any piles being embedded within the dense gravel layers with sufficient depth of gravels below any founding to prevent punching failure into the underlying weaker soils. This value must be factored appropriately.

If a piling option is preferred then some pretesting should be carried out to confirm or otherwise the presence of any weaker compressible layer within the zone of stress influence around the pile.

Any piling solution will require detailed design.

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7.3.4 Settlement of Pile Foundations

Settlement of prebored cast in place concrete piles is expected to be minimal for piles that socket into medium dense/dense granular soils.

Settlement of a single pile in an elastic medium may be estimated from the following equation:

$$S_1 = \frac{Q_a I_{PILE}}{L\ E_{V'}}$$

where:	Q_a	=	the applied load to the pile head
	$E_{V'}$	=	the average soil stiffness over the length of the shaft
	L	=	embedded length of pile
	I_{PILE}	=	the influence factor (a value of 1.8 is assumed for routine estimates of pile settlements)

The stiffness value $E_{V'}$ for the soil around the embedded length of pile is relatively high. Based on the density of the gravel layer encountered in BH4 at 6.0 metres depth, we have assessed a soil stiffness value of E_S = 70 MPa.

Elastic settlements will be influenced by the presence of any weaker soils within the stress zone beneath any piles and this aspect needs to be confirmed by additional testing.

The potential exists for differential movement between shallow foundations and pile foundations, and therefore composite foundations are not recommended beneath any of the apartment blocks.

7.3.5 Subfloor Filling

Filling is required beneath some of the apartment blocks to achieve design levels. The depth of filling varies between 0.5 metres (F5 and part of F9) to 1.7 metres depth over part of F8. As indicated previously the depth of filling relates to existing ground level and any undercutting that may be required to achieve a sound base to receive filling will affect fill volumes. We understand that based on existing cut/fill volumes some imported filling will be required.

All fill soils should comprise imported hardfill.

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The main engineering aspects associated with the placement and compaction of the subfloor filling are summarised below:

- Subgrade preparation shall comprise the removal of all unsuitable soils including topsoil and weak compressible soils.

- The exposed subgrade following removal of unsuitable shall comprise competent and low compressible soils and all subgrade shall be approved by an experienced engineer.

- Fill soils must be brought to the best practical water content and compacted in thin layers not exceeding 300 mm loose thickness using specific compaction machinery.

- The control of earthworks construction should be by:

  - air voids (but depending on material type)
  - dry density

  as determined by in situ density testing using a nuclear densometer.

  Routine acceptance testing can be by hand held shear vane (for fine grained soils <u>only</u>) and by scala probe (coarse grained soils).

- Compaction requirements shall be taken from the following but depending on material type:

1. Air voids - not exceeding 10%.

2. Relative compaction not less than 95%.

3. Scala probe - the number of blows required to drive the scala probe shall not be less than 10 blows/150 mm.

The contractor needs to demonstrate that his plant and compaction machinery are able to achieve the standard of compaction required and a specification, supervision and testing would be required for certification.

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7.3.6 Slab on Grade

The subfloor filling will likely be at least medium dense and have an equivalent CBR = 10.

A design chart for a concrete pavement/slab on grade is presented in the State Highway Pavement Design and Rehabilitation Manual and shows that for a given subgrade stiffness, as measured by the CBR, is partly dependent on the concrete modulus of rupture which may be estimated from $M_R = 0.09fc + 1.3$ (MPa).

The engineer shall satisfy themselves that the subgrade achieves a density of 10 blows/150 mm as recommended previously, completely over the footprint of the apartment building.

7.3.7 Retaining Walls

The design of foundation walls may be optimised on the basis of the fill soil strength parameters. The compacted filling is likely to have some cohesion c' and therefore some self-supporting capacity, however, for simplicity of design the cohesion is often ignored and the design of retaining walls may be based on the following soil strength parameters:

c'	=	0 kPa
Φ'	=	35°
ɣ	=	20 kN/m^3

The compaction of the subfloor filling behind any foundation wall will impose additional forces on any retaining wall and this aspect must be taken into account in the design of the retaining walls.

All retaining walls must be drained appropriately.

7.4 Foundations Under Earthquake Shaking

7.4.1 General

Under design earthquake shaking some buildings, depending on their aspect ratio may undergo rocking. The effect of the rocking will be that both compression and tension forces will be generated and these forces must be resisted by the foundations.

Shallow foundations are likely to have limited mass and passive pressure likely developed on the sides of shallow foundations is expected to be small, therefore shallow foundations are not expected to be able to resist design uplift loads.

Design loads may be resisted by pile foundations. Pile foundations may comprise prebored cast in place concrete piles or ground anchors.

These solutions are discussed in the following sections.

7.4.2 Bored Piles

Uplift resistance may be provided from skin friction developed on straight sided shaft piles. The unit skin friction (F_S) has been based on a correlation between SPT and CPT (Cone Penetration Test). Indicative values for unit skin friction have been assessed for a range of subsoils as follows:

Soil Type	SPT N Value Range	Unit Skin Friction (F_S) (kPa)
Sandy SILT	7 – 28	120 – 200+
SAND	8	100
Silty GRAVEL	30 – 45	150 – 230+

The above unit skin frictions are ultimate values and must be factored appropriately.

For overstrength design a GSRF of 0.8 must be applied to the ultimate value.

For design earthquake shaking a GSRF = 0.55 is appropriate.

It is common practice to ignore the top 2.0 metres with the uplift capacity generated below this level.

The order of magnitude of resistance to uplift is presented for both a 450 mm and 600 mm diameter pile for an 8.0 metre and 10.0 metres long pile.

Pile Diameter (m)	Pile Length (m)	Indicative Uplift Capacity (kN)
0.45	8.0 10.0	~ 850 ~1,200
0.60	8.0 10.0	~ 1,200 ~1,500

The values given are ultimate values and must be factored as recommended.

These values are based on an average unit skin friction over the pile shaft and are likely conservative. Optimum uplift capacities may be generated based on specific subsoil profiles as required.

7.4.3 Ground Anchors

Uplift forces may be resisted by ground anchors that derive support essentially from the medium dense/dense gravel layers. An appropriate ground anchor comprises an Ischebeck bar anchor system. The Ischebeck bar is a ribbed hollow steel bar that is spun into the ground using a lean mix grout. The lean mix grout is flushed out using a high strength grout over a predetermined fixed length of the anchor bar.

In order to optimise Ischebeck bar sizes the following capacities are given for a range of Ischebeck bars:

Ischebeck Bar Size (mm)	Characteristic Load Capacity (kN)	Yield Capacity (kN)
40/16	465	525
52/26	620	730
73/56	695	830
73/53	860	970
73/35	1,386	1,430

Uplift resistance would be dependent on drilling depth and would require specific design.

Depending on anchor size and drilling bit used, it is likely that the anchor hole may overbreak to say an average hole diameter of between 180 mm to 200 mm. The volume of grout used would be recorded so that the anchor hole diameter may be confirmed.

The capacity of any ground anchor must be proof tested using a procedure detailed in BS 8081 or similar document.

The performance of any ground anchor under earthquake shaking may be assessed by considering the theoretical elongation of the bar. This calculation is a function of E_S Young Modulus for the bar and varies depending on the bar size as indicated below:

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Ischebeck Bar Size	E_S (MPa)
73/53	151,000
73/35	156,000

Actual free length of the bar may be assessed following an anchor load test as recommended.

7.4.4 Pile Lateral Capacity

The capacity of a laterally loaded pile will depend in part on the rigidity of a pile.

A short pile will be governed by soil stiffness and a long pile will be governed by the yield moment in the pile. A laterally loaded pile may be designed on the basis of the soil stiffness which can be measured in terms of the coefficient of horizontal subgrade reaction K_h. K_h may be based on E_S where a multiplier of between 1 to 1.3 E_S is commonly used.

7.4.5 Base Shear Forces

We are not aware of design base shear forces that may be generated under design earthquake shaking.

Base shear forces may be resisted by passive pressure on ground beams, however, some movement may be expected to occur before passive resistance is generated.

Ground anchors have limited capacity to resist base shear forces and some capacity may be generated from raking piles in which the horizontal component may be considered.

Base shear forces may be generated from other structural elements and the magnitude of base resistance may be generated from $P_F = 2/3W\tan\varnothing$. $\varnothing$ may be conservatively taken as 30°.

7.4.6 Subsoil Category

We have referred to published information and a contour map of basement rock contours in Wellington CBD. The contour map indicates that basement greywacke rock is some 20 to 25 metres below ground surface. The classification for shallow soil sites may be based on the criteria detailed in Table 3.2 of NZS 1170.5:2004. We consider that the site may be designated Class C shallow soil site with the depth of strength soils not exceeding those given in the above table.

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7.4.7 Liquefaction Potential

Liquefaction is described as the progressive loss of shear strength under repeated loading due to the build-up of pore pressures between the soil particles. Repeated or cyclic loading which develops during earthquake activity produces rapidly increasing strains and under continued cyclic loading can lead to the development of large deformation within a soil mass, even at near-level sites.

Factors which affect the potential for liquefaction include soil type, relative soil density, initial confining pressures, intensity and duration of ground shaking. Soils that are particularly prone to liquefaction include loose sandy soils below the water table, and to a lesser extent silty sands.

The boreholes put down encountered typically very stiff fine grained soils and dense gravelly soils to the depth explored. These soils fall outside a grading envelope (Tsuchida) for liquefiable susceptible soils and we infer that the soils are at a perceived low risk of liquefaction having a likely trigger level above a ULS level of ground shaking.

8.0 CONCLUSIONS

The investigation has shown that:

- The site is suitable for the proposed apartment block development and is subject to the engineering conditions and constraints presented herein.

- Subsoil conditions are generally competent but vary significantly across the site with fine and coarse grained soils of inferred colluvial origin existing below a variable depth of dense granular filling in the boreholes put down adjacent to Willis Street. The boreholes put down adjacent to Victoria Street encountered similar soils comprising mainly coarse grained soil with some interbedded layers of fine grained soils.

- The various apartment blocks proposed across the site will be situated at various elevations with some earthworks comprising both cutting and filling required to achieve design floor levels.

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- Under static conditions foundations supporting all apartment blocks may be supported at shallow depth on strip or pad foundations. Some variability is to be expected at foundation subgrade level and in order to provide some consistency in terms of subgrade strength and density some compaction and/or minor subexcavation and replacement with compacted hardfill may be required. All foundation alignments and pad foundations must be tested to confirm the strength/density of the subgrade soils.

- For higher design loads, structural elements may be supported on prebored cast in place concrete piles that derive support from the dense gravel layers encountered at varying depths in all boreholes. Ultimate pile bearing capacities are given to allow optimisation of pile geometry.

- Elastic settlements are not expected to be high but should be confirmed when foundation geometry is confirmed.

- Pile foundations may be required to resist design earthquake shaking. Uplift capacity may be resisted by skin friction developed over straight sided shaft piles. Unit skin friction is provided for various sized piles and uplift capacities assessed.

- Site liquefaction is unlikely to occur under design earthquake shaking.

- Additional testing is required between the investigation boreholes to confirm subsoil consistency at each of the proposed apartment blocks. Additional testing will comprise putting down boreholes that target the strength/density of subsoil within the stressed zone between shallow and pile foundations.

9.0 LIMITATIONS

This report has been prepared solely for you as our client with respect to the brief provided. Data or opinions contained in this report may not be used in other contexts or for any other purpose without our prior review and agreement.

10.0 REFERENCES

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APPENDIX A

LOCATION PLAN
1:2000

SITE INVESTIGATION PLAN
1:500

KEY:

BH1 — BOREHOLE

NOTES:

1. DIMENSIONS AND LOCATIONS ARE APPROXIMATE ONLY.

2. AERIAL IMAGERY HAS BEEN OBTAINED FROM WCC

Rev	Date	By	Reason	Approval

Job: GEOTECHNICAL INVESTIGATION
Job Address: Victoria/Vivian Streets Wellington

Client: Egmont Dixon Lmited
Owner: Miro Street Developments Limited

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Drawn: GE
Checked
Traced
Scale: AS SHOWN AT A3
Date: 22/08/19

Sheet Title

SITE INVESTIGATION LOCATION PLAN

Job No: 12323
Sheet: S1
Rev: A

BORE HOLE LOG

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PROJECT:	Geotechnical Investigation
LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	~ 18.2
DATE:	15/11/2019
LOGGED BY:	[signature]

BOREHOLE #:	1		
SHEET 1 OF 3	JOB #:	12323	
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	GRIFFITHS DRILLING		
DIAMETER (mm):	100 ⌀		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa) 10 25 50 100 200	MOISTURE CONDITION
				0		TARSEAL GRAVEL/SAND loose, brown.			
						SILT, sl. gravelly, stiff, brown.			
		N=6	3/4/1/2/2/1	1		WOOD fibre			
						SILT, clayey, very stiff, friable, yellow brown.			
	15-9-19	N=30	5/7/7/8/7/8	2		GRAVEL (fine to med.) silty, med dense, light brown.			
		N=32	4/7/7/9/9/7	3		- fine to med. GRAVEL, sl. silty, dense, light brown.			
		N=8	6/5/2/2/2/2	4		SAND (med.) gravelly, med. dense, brown			
						CLAY, silty, very stiff, light brown.			

BORE HOLE LOG

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PROJECT:	Geotechnical Investigation
LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	
DATE:	15/11/2019
LOGGED BY:	

BOREHOLE #:	1		
SHEET 2 OF 3		JOB #:	12323
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	GRIFFITHS DRILLING		
DIAMETER (mm):	100φ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa)	MOISTURE CONDITION
						CLAY, silty v. stiff, brown.			
		N=32	5 / 7 / 7 / 9 / 8 / 8	5		GRAVEL (fine to med.) silty, dense, brown.			
		N=21	3 / 6 / 5 / 5 / 5 / 6	6		SILT, sl. sandy, very stiff to hard, light grey brown/orange brown.			
				7		GRAVEL, (fine to med.) silty. dense, brown.			
		N=45	5 / 8 / 8 / 12 / 15 / 10	8					
						SILT, sandy, hard, grey. ~friable			

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PROJECT:	Geotechnical Investigation		BOREHOLE #:	1
LOCATION:	Victoria/Vivian Streets WELLINGTON		SHEET 3 OF 3 JOB #:	12323
RL (m):			DRILL TYPE/METHOD:	WASH DRILLING
DATE:	16/09/2019		OPERATOR:	GRIFFITHS DRILLING
LOGGED BY:	RD		DIAMETER (mm):	100 ϕ

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa)	MOISTURE CONDITION
		N=28 3/5/5/6/7/10		9		SILT, sl. sandy, hard, grey.			
				10		10' SILT, clayey, stiff, grey			
		N=4 0/0/1/1/1/1							
				11					
						11.7 GRAVEL (fine to med.) sandy, very dense, grey brown.			
		N=50+ 2/7/10/15/11/14		12					
						BOREHOLE TERMINATED @ 12.45			
				13					

BORE HOLE LOG

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PROJECT:	Geotechnical Investigation	
LOCATION:	Victoria/Vivian Streets	WELLINGTON
RL (m):	~19.7	
DATE:	15/11/2019	
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BOREHOLE #:	2		
SHEET 1 OF 4		JOB #:	12323
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	GRIFFITHS DRILLING		
DIAMETER (mm):	100ϕ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa) 10 25 50 100 200	MOISTURE CONDITION
				0		TARSEAL			
						GRAVEL			
						CONCRETE			
				1		FILL, GRAVEL (fine & med.) sl. silty, dense, light brown.			
		N=34	4, 6, 8, 8, 8, 10						
	17-10-19			2					
		N=36	6, 8, 10, 8, 8, 10			GRAVEL (fine some med.) silty, sl. sandy, dense light brown.			
				3					
		N=40	8, 10, 10, 10, 11, 9			- gravel fragments (angular.), sl. sandy, dense, brown.			
				4		SILT, sl. clayey very stiff, brown.			M

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PROJECT:	Geotechnical Investigation
LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	
DATE:	15/11/2019
LOGGED BY:	[signature]

BOREHOLE #:	2		
SHEET 2 OF 4		JOB #:	12323
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	[illegible] DRILLING		
DIAMETER (mm):	100φ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa)	MOISTURE CONDITION
						SILT, sl. clayey, very stiff, brown.			M
						SILT, sandy, trace gravel very stiff, brown			
		N=27	3/6/7/7/6/7	5		GRAVEL (fine to med.) silty, med. dense to dense, brown.			
				6					
		N=36	5/6/8/8/10/10			- silty GRAVEL dense, light brown.			
				7					
						- angular fine some med. GRAVEL sl. silty / sand. dense to very dense, light brown			
		N=50	5/9/10/13/15/12	8					
				9					

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PROJECT:	Geotechnical Investigation
LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	
DATE:	15/11/2019
LOGGED BY:	[signature]

BOREHOLE #:	2		
SHEET 3 **OF** 4	**JOB #:**	12323	
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	GRIFFITHS DRILLING		
DIAMETER (mm):	100ϕ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa)	MOISTURE CONDITION
						GRAVEL (fine to med.) sl. sandy, dense light brown			
		N=13	3 3 2 3 4 5			SILT, clayey, very stiff, light grey			M
						-grades sandy			
				10					
						SILT/CLAY, very stiff, light grey-brown			
		N=9	1 1 1 2 5	11					
				12		GRAVEL, (fine to med.) very dense, light grey-brown.			
		N=50+	9 15 17 18 15						
				13					

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LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	
DATE:	15/11/2019
LOGGED BY:	[signature]

BOREHOLE #:	2		
SHEET 4 OF 4		JOB #:	12323
DRILL TYPE/METHOD:	WASH DRILLING.		
OPERATOR:	GRIFFITHS DRILLING.		
DIAMETER (mm):	100 ϕ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa) 10 25 50 100 200	MOISTURE CONDITION
		N=34 8 / 9 / 8 / 8 / 10 / 8		14		SILT, sl. sandy, minor gravel, hard, friable light brown.			
				15					
		N=50+ 12 / 10 / 12 / 13 / 16 / 9+				- sl. sandy SILT, hard, light brown.			
				16		BOREHOLE TERMINATED @15.94			

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LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	~20.6
DATE:	15/11/2019
LOGGED BY:	[illegible]

BOREHOLE #:	3
SHEET 1 OF 4	JOB #: 12323
DRILL TYPE/METHOD:	WASH DRILLING
OPERATOR:	[illegible] DRILLING
DIAMETER (mm):	100 ϕ

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa)	MOISTURE CONDITION
				0		TARSEAL			
						FILL, GRAVEL (fine to med.) sl. silty, loose to med. dense, light brown.			
				1					
		N=47	7, 9, 11, 11, 12, 13			– sl. silty, fine to med. GRAVEL, dense, light brown.			
	18-10-19			2					
		=46	10, 8, 13, 10, 11, 12						
				3					
		N=7	4, 2, 2, 2, 2, 1			SILT, sl. sandy, very stiff, light brown. – friable, pockets silty sand.			
				4					

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LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	
DATE:	15/11/2019
LOGGED BY:	[illegible]

BOREHOLE #:	3		
SHEET 2 **OF** 4		**JOB #:**	12323
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	GRIFFITHS DRILLING.		
DIAMETER (mm):	100φ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa)	MOISTURE CONDITION
		N=30	6 / 7 / 6 / 7 / 7 / 7	5		GRAVEL, (fine some med.) trace of silt, med. dense/dense, light brown.			
				6					
		N=37	6 / 8 / 9 / 8 / 10 / 10			— med. GRAVEL, sl. silty, dense, light brown.			
				7					
						SAND (med.) gravelly, dense light brown.			
		N=46	7 / 8 / 13 / 11 / 13 / 9	8		GRAVEL, (med. some fine) dense, light brown. — patches orange brown			
				9		SAND, gravelly, dense, brown.			

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PROJECT:	Geotechnical Investigation
LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	
DATE:	15/11/2019
LOGGED BY:	[signature]

BOREHOLE #:	3		
SHEET 3 **OF** 4		**JOB #:**	12323
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	GRIFFITHS DRILLING.		
DIAMETER (mm):	100 ϕ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa)	MOISTURE CONDITION
		N=45 7/8/15/13/10/7				GRAVEL (fine to med.) sandy, dense, grey-light grey.			
				10					
		N=42 6/7/17/6/9/10		11		- many angular broken faces. GRAVEL, silty, dense, grey			
				12		- fine to med. GRAVEL, sl. sandy, trace of silt, very dense, grey.			
		N=50+ 9/13/12/15/15/8+		13					

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PROJECT:	Geotechnical Investigation
LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	
DATE:	15/11/2019
LOGGED BY:	[signature]

BOREHOLE #:	3		
SHEET 4 OF 4		JOB #:	12323
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	GRIFFITHS DRILLING		
DIAMETER (mm):	100 ϕ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa)	MOISTURE CONDITION
		N=50+ 17/83/70 mm		14		GRAVEL (med.) angular fragments, very dense, light brown.			
				15					
		N=50+ 40/70				Med. GRAVEL, sl. silty, very dense, brown.			
						BOREHOLE TERMINATED @ 15.58 m.			
				16					

BORE HOLE LOG

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PROJECT:	Geotechnical Investigation
LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	~20.2
DATE:	15/11/2019
LOGGED BY:	[signature]

BOREHOLE #:	4		
SHEET	1 OF 4	JOB #:	12323
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	GRIFFITHS DRILLING		
DIAMETER (mm):	100ϕ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa) 10 25 50 100 200	MOISTURE CONDITION
						TARSEAL			
						CONCRETE			
						GRAVEL, (fine to coarse) loose to med. dense, brown.			
				1		SILT, sl. clayey, pockets silty clay, very stiff, light brown.			M
	▼	N=10	2/2/2/2/2/4						
				2		GRAVEL, (fine & med.) sl. silty, dense, light grey brown.			
		N=35	6/8/8/8/9/10						
				3					
		N=44	4/7/10/9/9/16			– fine to med. GRAVEL in a silt matrix. dense, light brown			
				4					
						SILT/CLAY very stiff brown.			M

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RL (m):	
DATE:	15/11/2019
LOGGED BY:	[signature]

BOREHOLE #:	4		
SHEET 2 OF 4	JOB #:	12323	
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	GRIFFITHS DRILLING		
DIAMETER (mm):	100 ϕ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa)	MOISTURE CONDITION
		N=12 0/1/1/1/3/7				CLAY, silty, minor gravel very stiff, light brown.			M
				5		GRAVEL (fine some med.) silty, med. dense, light brown.			
		N=28 2/3/5/8/7/8		6		- angular fine & med. GRAVEL, med. dense, brown.			
				7					
		N=50+ 14/20/20/15/11/4+				GRAVEL (fine) sandy, very dense, brown.			
				8					
				9					

BORE HOLE LOG

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PROJECT:	Geotechnical Investigation
LOCATION:	Victoria/Vivian Streets WELLINGTON
RL (m):	
DATE:	15/11/2019
LOGGED BY:	[signature]

BOREHOLE #:	4		
SHEET 3 **OF** 4		**JOB #:**	12323
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	[illegible] DRILLING		
DIAMETER (mm):	100 ϕ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa) 10 / 25 / 50 / 100 / 200	MOISTURE CONDITION
		N=33 5/7/8/8/7/10				GRAVEL, (fine to med.) sl. sandy, trace of silt, dense, light brown.			
				10		SILT, very stiff, grey.			M
		N=10 0/0/0/4/3/3							
						SILT, sl. gravelly, very stiff, grey.			
				11					
		N=5 0/1/1/1/1/2		12		CLAY, silty, very stiff, grey			M
				13		SILT, sl. gravelly, dense/hard, grey			

BORE HOLE LOG

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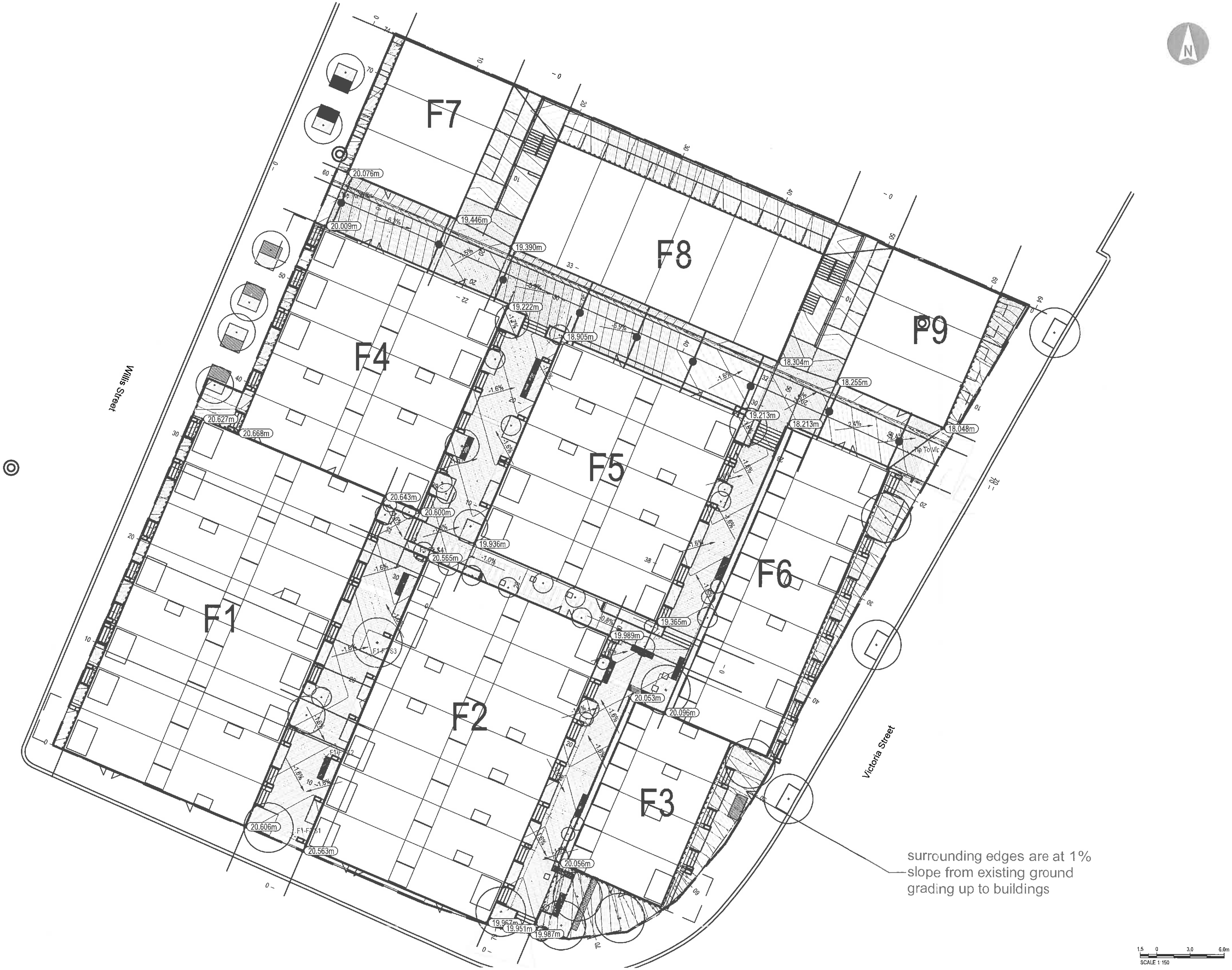
RL (m):	
DATE:	15/11/2019
LOGGED BY:	[signature]

BOREHOLE #:	4		
SHEET 4 OF 4		JOB #:	12323
DRILL TYPE/METHOD:	WASH DRILLING		
OPERATOR:	GRIFFITHS DRILLING.		
DIAMETER (mm):	100φ		

CASING	WATER	DEFECTS/SAMPLES & TESTS	SPT (N)	DEPTH (m)	GRAPHIC LOG	SOIL/ROCK DESCRIPTION	CORE RECOVERY (%)	SHEAR STRENGTH (kPa)	MOISTURE CONDITION
		N=29 6/6/6/8/5/7				GRAVEL, (fine & med.) sl. silty, dense, grey			
				14					
		N= 50+ 6/11/12/13/16/9		15		- fine to med. GRAVEL, silty, very dense, light brown.			
						BOREHOLE TERMINATED @ 15.45M			

APPENDIX B


F7
F8
F9
F4
F5
F6
F1
F2
F3
Willis Street
Victoria Street
surrounding edges are at 1%
slope from existing ground
grading up to buildings
SCALE 1:150