REPORT

Ministry of Education

Flat Bush School
Preliminary Geotechnical Investigation Report

Report prepared for:

MINISTRY OF EDUCATION

Report prepared by:

Tonkin & Taylor Ltd

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1 Introduction

Tonkin & Taylor Ltd (T&T) was engaged by the Ministry of Education to conduct preliminary geotechnical investigations as part of a risk assessment study for a proposed new school on Flat Bush School Road, Flat Bush. Written authority to proceed with this work was received from Frequency Project Management on 16 September 2014[1] following our offer of service of 3 September 2014[2].

The objective of this study was to evaluate the suitability of the site for a future school and to identify possible geotechnical constraints and opportunities for development.

The scope of the work carried out has consisted of the following:

- Review of all available geotechnical data including geological maps, historical aerial photographs, geotechnical data from T&T investigations;
- A site walk over survey of the site by a geotechnical engineer and geologist;
- Geotechnical investigations comprising ten hand augered boreholes and eight Cone Penetration Tests (CPTs);
- Evaluation of the site subsoil class for seismic design;
- Evaluation of the liquefaction risk to the site;
- Evaluation of the suitability of the surface soils for earthworks;
- Derivation of preliminary foundation design parameters
- Assessment of preliminary pavement design parameters;
- Preparation of this report.

The report presents the results of the preliminary geotechnical investigations undertaken and provides comment on the geotechnical suitability of the site for future school development and preliminary design recommendations based upon interpretation of the investigation data.

A separate ground contamination investigation and assessment has also been completed by Tonkin & Taylor. The results and conclusions of this investigation/assessment are presented under separate cover.

[1] Frequency Project Management (16 September 2014). Email to Tonking & Taylor. RE: Flat Bush School – Geotechnical investigations. Ref 30393.
[2] Tonkin & Taylor Ltd (3 September 2014). Letter to Ministry of Education. Flat Bush School, Proposal to provide geotechnical and ground contamination consultancy services: risk study assessment. Ref 30393.

2 Site description

2.1 Site location and layout

The Candor3 plan[3] provided by Frequency Project Management shows the proposed future school could be located part of the property located at 187 Flat Bush School Road, legally described as Lot 2 DP 48950 with a total area of 48hA. The school site is a 3.71ha parcel located on the southern side of Flat Bush School Road and is bounded by shallow gully features on both the western and eastern sides. The gully on the western side of the site is within the site boundary, and is typically 20m wide and less than 1.5m deep with a shallow gradient of 2% fall falling to the north. The gully to the east is outside the site boundary but is considerably deeper (approximately 8m deep) and more extensive (approximately 50m wide).

The site topography is flat to gently sloping, with ground surface elevations ranging between approximately RL 47m on the southern site boundary to RL 41m on the northern boundary. There is a shallow drain feature running in a west to east direction towards the eastern boundary.

The site is currently leased for farming purposes. Unsealed internal access tracks run in both a north to south and west to east direction across the site. Other than the access tracks and the shallow gully on the western boundary, the site is presently entirely covered in crops.

2.2 Proposed development

At the time of preparing this report, specific development plans for the future school were not available. The geotechnical investigations were therefore undertaken across the wider site in areas that could be readily accessed (i.e. not covered in crops). However, we would expect a typical school development to comprise the following as a minimum:

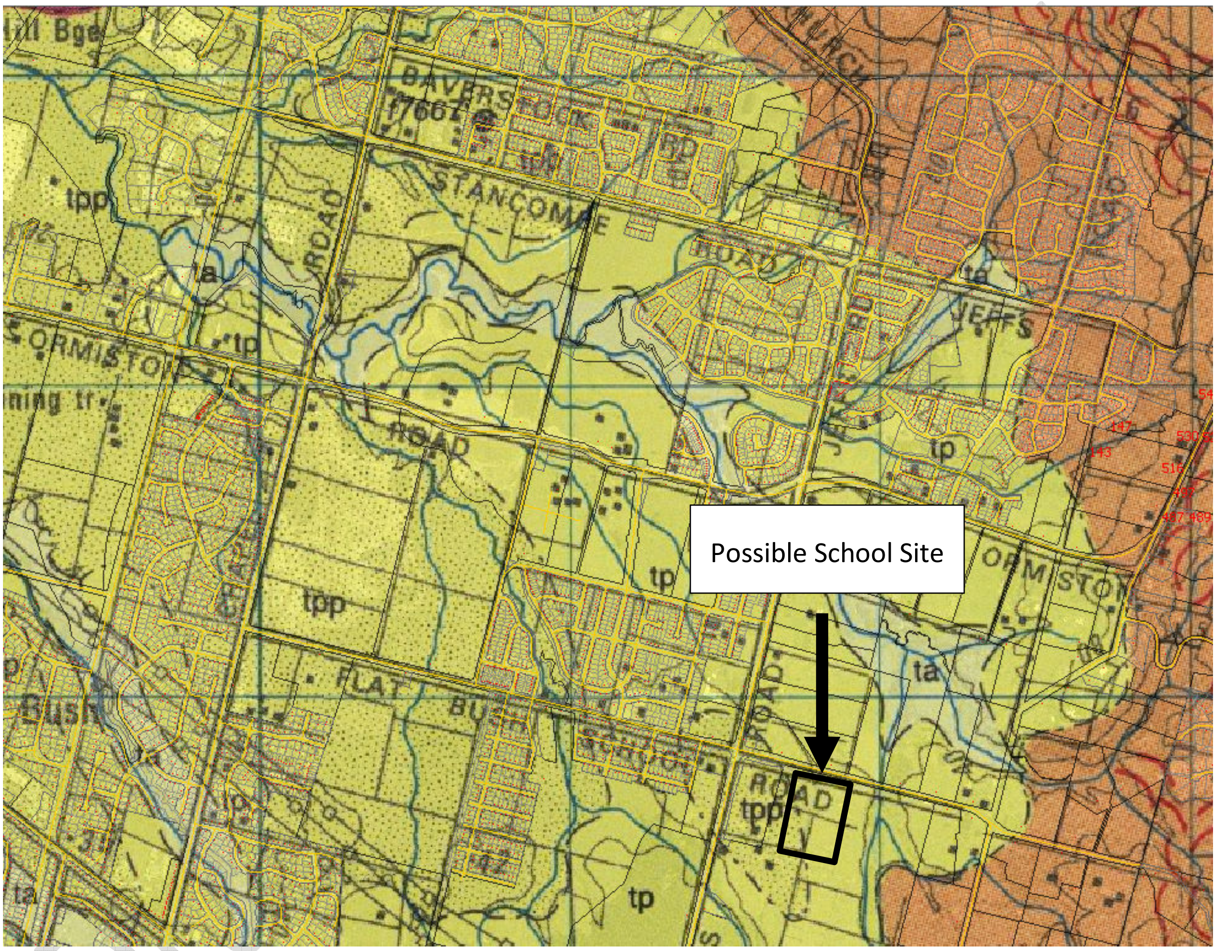
- Earthworks for regarding of the site and forming of building platforms
- Installation of civil infrastructure (stormwater, wastewater, water supply)
- Construction of new school buildings comprising administration blocks, hall, library, care taker facilities and multiple teaching spaces. The new buildings may be either single of two storey structures.
- New internal access roads and car parking
- Playing fields and court (pavement) areas
- Sporting facilities

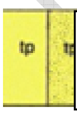
[3] Candor3 (November 2013). 187 Flat Bush School Road – Land Take Plan Sheet 2 of 3. Project no. 9202, Drawing SP001.

2.3 Published geology

The published geology of the area; Geology of the Auckland Area[4] (Scale 1:50,000), indicates that the site lies within an alluvial basin (area shown in yellow), substantially surrounded to the north, east and south by higher ground formed by the East Coast Bays Formation (ECBF – shown as orange). East Coast Bays Formation soils are present close to ground surface approximately 200 m to the east of the site, as shown below.

Figure A: ***Geological map of Auckland City, Source: Kermode L.O (1992)***





Puketoka Formation (tp) - light grey to orange brown, pumiceous mud, sand and gravel with black muddy peat and lignite

The alluvium is mapped in this area as Puketoka Formation, which is characterised by pumiceous muds and sands with some peat (tp) on rhyolite pumice sand (tpp).

[4] Kermode,L.O., 1992: Geology of the Auckland Urban Area. Scale 1:50,000. Institute of Geological & Nuclear Sciences Geological Map 2. 1 sheet + 63p. Institute of Geological and Nuclear Sciences Ltd, Lower Hutt, New Zealand.

3 Field Investigations

The geotechnical investigations undertaken by T&T for the proposed development were undertaken on 25 – 26 September and 9 October 2014. Two separate stages of investigation were undertaken due to access issues encountered around the present use of the site for horticultural purposes.

As noted in Section 2.1, the site was covered in crops at the time of investigation. Therefore, the geotechnical tests were positioned in areas that could be readily accessed without causing undue disturbance/damage to the crops.

The geotechnical investigations consisted of:

- Ten hand augered boreholes to depths varying from 5.1 m and 8.5m below ground level;
- Eight Cone Penetrometer Tests (CPTs) pushed to between 8m and 15m below ground level;

The hand auger boreholes were drilled by a qualified geologist from Geotechnics Ltd and all soils were logged on site. In situ strength testing was performed at 300 mm depth intervals using a hand Pilcon vane. The CPTs were conducted by DCN Drilling Ltd under the supervision of a geotechnical engineer from T&T. CPTs 3 and 9 were proposed but not ultimately completed due to site access constraints.

The locations of the geotechnical investigations are shown on the Geotechnical Investigation Plan presented in Appendix A. The logs for the boreholes and CPT traces are presented in Appendix B.

4 Subsurface conditions

4.1 General

The subsurface model presented in this report has been inferred using the results of hand auger boreholes and Cone Penetrometer Tests put down at point locations. It should be noted that the nature and continuity of the subsoils away from test locations is inferred and may vary from the assumed model.

4.2 Geological units

The proposed school site was found to be underlain by the following sequence of geological units:

- Puketoka Formation (Tauranga Group) alluvial deposits
- Residual East Coast Bays Formation (ECBF) soils
- ECBF weathered rock

A brief description of each of these geological units is presented below.

4.2.1 Puketoka Formation alluvial deposits

The Puketoka Formations soil are typically between 4m and 10m thick (generally thicker at the eastern side of the site and thinner at the western end).

The Puketoka Formation soils typically comprise very stiff to stiff clayey silts with undrained shear strengths in the range of 140kPa to 120kPa in the upper 3m, reducing to approximately 50kPa to 80kPa from 3m depth to the base of the unit (4 to 10 m below ground level). Peat was not encountered in any of the boreholes, however, minor organics were logged within the clays/silts.

CPT results show the upper soils have a cone resistance (q_c) of around 1.5MPa to 2.0MPa and the lower soils (from 3m depth) around 0.75MPa to 1MPa.

4.2.2 Residual East Coast Bays Formation Soils

The surface of the ECBF soils typically lies at an elevation of between RL 33m on the eastern side of the site and at RL 39m at the western side of the site (4 to 10 m deep).

The residual ECBF soil was encountered in four of the nine hand augered boreholes and comprises hard clayey silts and silty clays with undrained shear strengths exceeding 200kPa. The residual soil layer is very thin, typically 1.0m to 1.5m according to the CPT results. This grades quickly into a completely weathered rock.

4.2.3 East Coast Bays Formation Rock

ECBF rock level was inferred where the CPT reached refusal, typically where cone resistance, q_c values exceeded 15 MPa. The elevation of the top of highly weathered, extremely weak, ECBF rock varies between RL 32m on the eastern side and RL 38m on the western side of the site (8 to 13 m below ground level).

Whilst not confirmed by cored machine boreholes at this site, ECBF rock typically comprises interbedded layers of siltstone and sandstone with an unconfined compressive strength of 1 to 3 MPa.

4.3 Groundwater

Groundwater was encountered at depths of between 0.3m and 3m below ground level in the hand auger boreholes, one day after the drilling. The site investigation was undertaken after an extended period of wet weather and typical groundwater depths of 0.5m should be assumed at the lower (northern) end of the site, increasing to 1.0m to 1.5m depth at the southern boundary.

5 Geotechnical Considerations

5.1 General

The subject site is considered to be suitable for a school development. The upper soils are generally very stiff and are likely to be suitable for support of new buildings (up to two storeys high), bearing on shallow pad or strip footings. However, the following geotechnical issues will need to be considered for future development.

5.2 Earthworks

It is understood that the existing gully along the western boundary of the site will be infilled to maximise the developable area of the site. The following recommendations should be considered:

i. An ecological assessment of the gully/stream be undertaken to determine whether it is ephemeral, or intermittent as defined in the Auckland Unitary Plan. This may dictate consenting requirements for earthworks including the likely need for subsoil underdrains.

ii. Prior to infilling of the gully, it will be necessary to "muck-out' and remove all soft sediments that may be present at the gully invert. These may either need to be disposed off-site, or alternatively could be used for landscaping purposes. They are unlikely to be suitable for use as bulk (engineered) fill.

iii. Subsoil drainage will likely be required within the gully prior to backfilling of the gully. Recommendations for drainage are outlined in Section 5.3.

iv. Future buildings should be located away from the infilled gully, where possible to avoid differential settlement effects. Alternatively, a minimum period of approximately 2 years should be allowed prior to constructing over the infilled gully to allow for the primary consolidation process to effectively complete.

v. Where new fill, greater than 1 m in thickness, is to be placed in future building platform areas, a consolidation period of approximately 2 years should be allowed for the primary consolidation process to effectively complete.

vi. Cut/fill earthworks should be limited within building footprints to ensure that as much of the natural upper stiff crust (generally 3m thick) remains in-situ for support of new buildings and pavements.

vii. Site won fill taken from the surficial 2m of the site is likely to be suitable for use as bulk fill and should achieve satisfactory levels of compaction under the right conditions. Further testing of the soils will be required to confirm this as part of a detailed investigation.

5.3 Drainage

Due to the relatively high groundwater levels at the site, it may be necessary to install subsoil drainage beneath building platforms, pavements (including roads, sporting courts, car parking areas) and playing fields to ensure that groundwater levels are controlled, particularly during the winter months. Such drainage will be more critical in areas of cut, where the site elevations are lowered to form design levels and depth to the groundwater level is reduced.

Subsoil drains would typically comprise 110mm to 160mm diameter perforated Novacoil pipes installed within 300mm wide by 1m deep trenches and backfilled with drainage material (e.g. 25/7 scoria). The drain would be required to grade towards a drainage point (minimum grade of 1 in 100), and installed at approximately 25m centres, depending on the intended use of the area.

5.4 Seismic Considerations

5.4.1 Seismic subsoil class

From a review of the investigation data, we recommend that this site be designed using Site Class C ("Shallow soil site"). This is based on satisfaction of the criteria outlined in Section 3.1.3.4 (a) of NZS 1170.5[5].

5.4.2 Design peak ground acceleration

The recommend peak ground acceleration (PGA) for the Ultimate Limit State (ULS) and Serviceability Limit State (SLS) seismic events are provided in Table 3 below for an assumed Importance Level 3 structure. These were calculated based on NZS 1170.5:2004 and AS/NZS 1170.0: 2002[6] based on the following:

Importance Level 3 structures (school buildings):

- Design Life: Assumed 50 years
- Annual probability of exceedance: **1 in 1000 year, ULS event** (from Table 3.3, AS/NZS 1170.0:2002) **1 in 25 years, SLS event** (from Table 3.3, AS/NZS 1170.0:2002)
- Return period factor, R: 1.3 ULS, 1/1000 year event; 0.25 SLS, 1/25 year event
- Hazard Factor, Z: 0.13 - Auckland
- Spectral Shape Factor, $C_h(T)$: 1.33 based on 'Site subsoil class C'

Table 1: Design PGA (in-ground structures and liquefaction analyses)

Seismic Case	Importance Level 3 structures	
	Design PGA (g)	**Return Period (years)**
Ultimate Limit State (ULS)	0.22	1,000
Serviceability Limit State (SLS)	0.04	25

5.4.3 Liquefaction analysis

Preliminary liquefaction analyses have been undertaken to assess the potential for liquefaction to occur under the design ULS and SLS earthquakes, based on the results of each CPT sounding. The analyses give an indication of the depth and cumulative thickness of liquefiable materials.

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[5] NZS 1170.5: 2004. New Zealand Standard, Structural design actions- Part 5: Earthquake actions- New Zealand. Standards New Zealand

[6] AS/NZS 1170.0:2002. Australian/New Zealand Standard, Structural design actions- Part 0: General principles

5.4.3.1 Liquefaction triggering methodology

Liquefaction triggering analyses were completed using the simplified method outlined in Idriss & Boulanger (2014)[7] utilising the CPT data. The apparent fines content and the Soil Classification Index (I_c) was calculated using Robertson & Wride (1998)[14]. Where the I_c exceeds 2.6, the soil was assessed to be too "clay-like" to liquefy.

Liquefaction analyses were completed for both the ULS and SLS seismic events using the PGA values calculated in Table 1, i.e. 0.22 g and 0.04 g, respectively. A Magnitude 7.5 seismic event was assumed in conjunction with these PGAs.

The results of the liquefaction analyses completed for the eight CPT soundings are presented in Appendix C.

5.4.3.2 Liquefaction analysis results

Based on the results of the analyses, soils at this site are considered unlikely to liquefy under the SLS design seismic event (PGA 0.04 g, 1/25 year event).

The analyses indicate that the site may be susceptible to minor liquefaction under the ULS (PGA 0.22 g, 1/1000 year) design earthquake. The identified liquefaction trigger level PGA is approximately 0.16 g which corresponds to a return period event of around 300 to 400 years. However, the majority of the theoretical liquefaction appears to occur at a depth of between 3.1m and 7.5m below the ground surface and therefore should not present a significant risk to the stability of structures. However, it will be necessary to check the post earthworks "crust thickness" to ensure that a minimum 3.0 m thick layer of non-liquefiable soils are present in areas proposed for future buildings. Where cut earthworks result in removal of the surface crust in areas proposed for future buildings, it may be necessary to consider ground improvement works beneath building platforms or piled foundations for new structures extending to the ECBF rock. Further assessment is required at detailed investigation and design stages.

5.4.3.3 Liquefaction induced ground settlement

Liquefaction induced ground settlements were calculated based on the CPT data (see Table 2) using Zhang et al. (2002)[8].The calculated surface settlements range between 10 to 70 mm with an average of 40 mm, which implies that differential settlement may occur after a design earthquake event. Such settlement magnitudes (total and differential) are expected to be tolerable for new buildings and structures under ULS seismic conditions.

5.5 Foundations

5.5.1 General

The site is generally underlain by a very stiff surface "crust" over 3m in thickness. We consider that this crust should provide a suitable bearing stratum for support of new buildings bearing on shallow foundations to a limited size and loading (e.g. one to two storey structures).

Where larger buildings are considered (e.g. three storeys high), or where new buildings are proposed in areas of cut (increasing the risk of liquefaction related settlement and deformation – see Section 5.4.3.2) piled foundations may be required. The sections below summarise the design parameters for each of the foundation options.

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[7] Boulanger, R.W. & Idriss, I.M (2014) "CPT and SPT Based Liquefaction Triggering Procedures" UCD/CGM-14/01

[8] Zhang, G., Robertson, P.K. & Brachman, R.W.I. (2002). Estimating liquefaction-induced ground settlements from CPT for level ground, Canadian Geotechnical Journal.

5.5.2 Shallow Foundations

For typical school structures (one to two storeys high), shallow foundations pad and strip footings are expected to provide a suitable foundation solution. A preliminary geotechnical ultimate bearing capacity of 300kPa is recommended of footings founding in the stiff to very stiff Puketoka Formation soils. For comparison with Ultimate Limit State (ULS) design, a strength reduction factor ϕ_g=0.5 should be applied to this capacity.

Isolated pad and strip footing widths should be limited to 1.5m maximum or subject to specific design review. It is recommended that pads and footings be founded at a minim depth of 600mm below the adjacent finished ground level to reduce shrink/swell effect of the near surface soils.

Any existing fill materials should be removed prior to construction of the shallow foundations. Following excavation for the shallow foundations, we recommend an inspection be undertaken by a qualified engineer competent to assess the subgrade conditions and to validate the inferred subsurface profile. Local undercutting and replacement with compacted hardfill may be necessary if incompetent subsurface materials are encountered.

5.5.3 Deep Foundations

Piled foundations extending into the underlying ECBF rock may be used to support buildings with higher loads or where there is an identified risk of liquefaction. Suitable pile foundations types include driven steel Universal Columns (UCs) and bored cast reinforced concrete piles.

Piles should be extended into the ECBF rock which is estimated at a depth between 8 m and 13 m below existing ground level. Pile embedment depths into ECBF rock will depend on the pile section (for driven steel UCs) and required skin friction capacities for bored piles. A minimum pile embedment of 3xD (D=pile diameter) should be assumed for design purposes

For bored, reinforced concrete piles embedded into ECBF rock (from 8 to 13 m below ground level), the following preliminary design capacities may be assumed:

- Geotechnical ultimate end bearing capacity: 6 MPa
- Geotechnical ultimate skin friction capacity: 500 kPa

The above capacities should be factored by 0.5 for comparison with ULS design and 0.33 for working load design.

Provisional capacities for driven steel UC piles extending to ‘refusal’ into ECBF rock are presented in Table 2 below. These are based upon recent dynamic pile test results in the East Coast Bays Formation, the following maximum design loads are considered applicable for driven steel piles driven to virtual refusal with an appropriately sized hammer. Driven universal columns are likely to extend between 3 and 10 m into ECBF rock depending on the section size employed.

Table 2: Provisional Capacities for Steel UC piles driven to refusal in ECBF rock

Pile Size	Maximum geotechnical ultimate capacity (kN)	Maximum ultimate limit state capacity (kN)	
		No pile testing (ϕ_g =0.5)	Dynamic testing 10% of the piles (ϕ_g = 0.8)
200UC60	1,500	750	1,200
250UC73	1,800	900	1,440
250UC89	2,200	1,100	1,760
310UC97	2,400	1,200	1,920
310UC118	2,900	1,450	2,320
Note	Dynamic pile testing undertaken in accordance with AS 2159		

The driving of piles creates both noise and vibrations. The noise may be partly reduced by pre-auguring holes in which piles are driven. Vibrations tend to rapidly attenuate with distance from the point of driving. If driven piles are the preferred option and vibration sensitive dwellings or equipment are within 10 metres of the driven pile, it is recommended that vibration monitoring be undertaken either on the initial driven pile or a test pile. This is to assess ground attenuation characteristics and the level of vibration and to confirm that driven piles are suitable across the entire site, rather than just distant to the sensitive structures.

If more than a single pile is required to support the design load, piles should be no closer than “3×D” c/c to minimise group effects; where D= pile diameter.

5.6 Pavements

The upper stiff soils, where retained with a thickness of at least 1m, and/or new engineered fill placed across the site, will provide a competent bearing layer for support of new pavements (access roads, car parking, building slabs, sports courts etc).

A provisional subgrade CBR of 4% is recommended for the design of pavement structures where these are formed close to existing ground level. Proof rolling should be carried out on the subgrade prior to constructions and localised soft spots, if any, should be undercut and backfilled with suitable compacted hardfill (e.g. GAP 65).

6 Conclusion and Recommendations

Tonkin & Taylor was engaged by the Ministry of Education to conduct preliminary geotechnical investigations as part of a risk assessment study for a proposed new school on Flat Bush School Road. Geotechnical services were completed in accordance with our proposal dated 3 September, 2014.

A summary of the investigation results and geotechnical recommendations for the proposed development are outlined below:

- The published geology for the area indicates the site is underlain by undifferentiated recent alluvium overlying the East Coast Bays Formation soils and rock. This was confirmed by the geotechnical Investigations.
- Geotechnical investigations for the proposed school comprised 10 No. hand augered boreholes (HA1 – HA10) and 8 No CPTs (CPT1-2, CPT4 – 8 and CPT10). CPTs 3 and 9 were not completed due to site access constraints.
- The subsurface conditions of the site comprise:
  - Puketoka Formation alluvial deposits composed of very stiff to stiff clayey silts, 4m to 10m thick
  - East Coast Bays Formation residual soils (typically 1m to 1.5m thick) grading quickly into completely to highly weathered ECBF rock
- Groundwater levels were measured at 0.3m to 3.0m below ground level one day after the completion of the hand augered boreholes. However, fluctuations in groundwater levels are likely between the seasons. A preliminary design groundwater level of 0.5 to 1.5 m below present site levels is recommended.
- The gully feature on western boundary of the site should be “mucked-out” to remove soft alluvial soils prior to infilling. Drainage should be installed in the gully invert prior to completing this work.
- Site won soils (taken from the upper 2 m) are expected to be suitable for re-use as engineered fill, provided they are appropriate conditions and placed in appropriate weather conditions. Laboratory testing of the soils is recommended as part of detailed investigations.
- Given the relatively high groundwater levels encountered at the site, subsoil drainage is recommended beneath building platforms, roads, other pavement areas and playing fields.
- Based on the site investigation, it is recommended that the site be considered as Site Class C (shallow soil site), in accordance with NZS1150.5 for seismic design purposes.
- A preliminary liquefaction assessment has been undertaken using the results of CPTs. There is a low to moderate risk of liquefaction trigger occurring in soils greater than 3 to 7 m depth under peak ground accelerations exceeding 0.16g (i.e. return periods exceeding approximately 300-400 years). However the associated effects of this liquefaction on the proposed development are expected to be limited (ground surface settlements of 10 to 70 mm). Such settlements may either be accommodated in the building structure for ULS design cases or alternatively pile foundations extending to the ECBF could be considered.
- We consider that shallow foundations are likely to be suitable for support of one to two storey school buildings constructed on the stiff surface crust of Puketoka Formation soils (upper 3 m). Where earthworks remove much of this existing material, or where larger buildings are proposed, pile foundations extending into the ECBF rock (greater than 8 to 13 m depth) may be required.
- A subgrade CBR of 4% is recommended for preliminary pavement design. This assumes that future pavements are formed close to present ground level or on new engineered fill

(soils with an udnrained shear strength exceeding 100 kPa). Prior to construction, pavement areas should be proof rolled and localised soft spots, including topsoil, if any, should be sub-excavated and backfilled with compacted hardfill.

7 Applicability

This report has been prepared for the benefit of The Ministry of Education with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

This report is intended to provide preliminary geotechnical advice on the suitability of the site shown on Figure 1, 187 Flat Bush Road, for a future school development. It is essential that detailed geotechnical investigations and analyses be undertaken to support design and consenting stages of the school project, once development plans have been finalised.

This report should be read in conjunction with the T&T contamination report (T&T Ref 30393.001, dated October 2014).

Tonkin & Taylor LTD

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Appendix A: Figures

- **Site Investigation Plan**

Appendix B: Investigation Results

- Hand augered borehole logs
- CPT traces

Appendix C: Preliminary Liquefaction Analysis Results