# Document Control

<table>
<thead>
<tr>
<th>Date</th>
<th>Version</th>
<th>Description</th>
<th>Prepared by</th>
<th>Reviewed by</th>
<th>Authorised by</th>
</tr>
</thead>
<tbody>
<tr>
<td>March 2018</td>
<td>1</td>
<td>Draft Geotechnical Appraisal Report for client review and comment</td>
<td>Ana Anufe</td>
<td>Peter Millar</td>
<td>Peter Millar</td>
</tr>
</tbody>
</table>

**Distribution:**

- Murchison Group 1 copy
- Tonkin & Taylor Ltd (FILE) 1 copy
# Table of contents

1 Introduction 1  
   1.1 Scope of work 1  
   1.2 Proposed development 2  
2 Geology 3  
   2.1 Published Geology 3  
3 Ground and Groundwater Conditions 4  
   3.1 Previous geotechnical investigations 4  
      3.1.1 Opus Consultants Ltd (Opus) 4  
      3.1.2 Tonkin & Taylor Ltd (T+T) 5  
   3.2 Geotechnical model 6  
   3.3 Seismic assessment 7  
4 Geotechnical Issues 8  
   4.1 Foundation options 8  
      4.1.1 Shallow foundations 8  
      4.1.2 Piled foundations 8  
   4.2 Negative skin friction 8  
   4.3 Site contamination 8  
   4.4 Liquefaction potential 9  
5 Recommendations and Conclusions 10  
   5.1 Recommendations 10  
   5.2 Conclusions 10  
6 Applicability 11  

Appendix A: Herbst Maxey Metropolitan Architects Ltd Concept Design Drawings  
Appendix B: Previous ground investigations  
Appendix C: T+T Aerial Photograph Plan showing potential investigation location
1 Introduction

Tonkin & Taylor Ltd (T+T) has been engaged by Murchison Group to undertake a geotechnical appraisal of the underlying ground conditions at 423–427 Beach Road, Mairangi Bay, Auckland (henceforth referred to as the Site), in relation to a mixed use apartment and retail complex development. The location of the Site is shown on Figure 1 below.

![Figure 1: Site location indicated by blue square. Image source: Auckland Council Geo Maps, https://geomapspublic.aucklandcouncil.govt.nz/viewer/index.html, accessed 23/02/2018.](image)

The site is legally described as Lot 1 DP64377 and is currently occupied by a row of single-storey concrete buildings utilised by various retailers, with a large car park at the rear. The site is bounded on the north and south by commercial buildings, on the west by Beach Road, and on the east by car parks and the Mairangi Bay Village Green.

1.1 Scope of work

The scope of work was outlined in the T+T letter of engagement, titled Geotechnical Services for proposed redevelopment at 423-427 Beach Road, dated 28 August 2017 (T+T job number reference 473). The proposed development will include three (3) levels and is based on a ‘no basement level’ scenario as outlined in Section 1.2 below. The scope of works includes the following:

- Undertake a desk study, geotechnical appraisal report detailing the history of the site;
- Assess the site geology based on existing nearby ground investigation findings; and
- Provide a preliminary assessment of development implications and potential geotechnical issues.
1.2 Proposed development

T+T have been provided with a Herbst Maxcey Metropolitan Architects Ltd development drawing set, dated 29 November 2017. These include:

- Ground Floor Level Floor Plan. Drawing Sheet SK02, Revision C.
- Level 01 Floor Plan. Drawing Sheet SK03, Revision C.
- Level 02 Floor Plan. Drawing Sheet SK04, Revision C.
- Level 03 Floor Plan. Drawing Sheet SK05, Revision C.

The drawings show the ground level comprising 248m$^2$ of retail space, 20 car park spaces, two common areas totalling 152m$^2$, and three separate service areas totalling 57m$^2$.

Levels 1 and 2 each have a total ground floor area of 926m$^2$ and Level 3 has a total ground floor area of 856m$^2$. All three levels comprise common areas and apartments of various configurations ranging in area between 77m$^2$ and 187m$^2$.

The drawings listed above are provided in Appendix A.
2  Geology

2.1  Published Geology

The published geology beneath the site is described by Edbrooke\(^1\) as alluvial and/or colluvial deposits, estuarine deposits, lacustrine/swamp deposits and fan deposits belonging to the Tauranga Group (Q1a) overlying alternating sandstone and mudstone with variable volcanic content and interbedded volcaniclastic grit beds belonging to the East Coast Bays Formation (ECBF) of the Waitemata Group (Mwe). An extract of the geological map for Auckland is provided in Figure 2.

![Geological Map of Auckland](image)

**Figure 2: Extract of Geological Map for Auckland, Edbrooke\(^1\).**

The ECBF around Mairangi Bay is shown to generally be dipping 35 degrees south east. Edbrooke\(^1\) has mapped the nearest fault to the Site as a thrust fault, with associated anticlines approximately 7 km northwest (all of which are related to the Mangakahia Complex).

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3 Ground and Groundwater Conditions

3.1 Previous geotechnical investigations

Three (3) investigations have previously been carried out in close proximity to the Site. These are summarised below and logs are provided in Appendix B.

3.1.1 Opus Consultants Ltd (Opus)

Opus previously carried out three (3) boreholes (BH1 to BH3) along the right-of-way immediately east of the Site, spanning between Montrose Terrace and Sidmouth Street, in relation to a proposed gravity sewer line. The locations of the boreholes are shown on Figure 3 below.

![Figure 3: Location of boreholes BH1 to BH3 completed by Opus in May 2013 (Source: NZGD) relative to the Site.](image)

A summary of the borehole findings is provided in Table 1.

Table 1: Opus geotechnical investigation findings, May 2013.

<table>
<thead>
<tr>
<th>Geological Unit</th>
<th>BH1</th>
<th>BH2</th>
<th>BH3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>0.35</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Alluvium</td>
<td>1.75</td>
<td>6.0</td>
<td>8.4</td>
</tr>
<tr>
<td>Residual Waitemata Group</td>
<td>1.7</td>
<td>3.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Depth to top of rock (m)</td>
<td>3.7</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>End of BH</td>
<td>10.0</td>
<td>10.0</td>
<td>10.0</td>
</tr>
</tbody>
</table>

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2 Opus Consulting Ltd report, Title Unknown, Reference 1-C0212.98, borehole logs only sourced from NZGD.
3.1.2 Tonkin & Taylor Ltd (T+T)

T+T has undertaken numerous works at 8 Sidmouth Street, Mairangi Bay (see Figure 4). The investigations relevant to this development include a geotechnical investigation for the proposed apartment buildings (2000) and a geotechnical investigation in April 2001 to assess the environmental effects of dewatering for the basement excavation of the same apartment buildings.

![Figure 4: Location of 8 Sidmouth Street, relative to the Site. Image source: Auckland Council Geo Maps, https://geomapspublic.aucklandcouncil.govt.nz/viewer/index.html, accessed 23/02/2018.](image)

The T+T geotechnical investigation in 2000 comprised five (5) hand augers (BH1 to BH5) and two (2) machine boreholes (MH1 and MH2). A summary of the investigation findings are provided in Table 2.

The T+T ground investigation undertaken in April 2001 comprised one (1) environmental machine borehole (MH3) and three (3) geotechnical machine boreholes (MH4 to MH6). A summary of the investigation findings is provided in Table 3.

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Table 2: T+T geotechnical investigation findings, July 2000.

<table>
<thead>
<tr>
<th>Geological Unit</th>
<th>MB1</th>
<th>MB2</th>
<th>BH1</th>
<th>BH2</th>
<th>BH3</th>
<th>BH4</th>
<th>BH5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Topsoil/basecourse/ fill</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Alluvium</td>
<td>0.7</td>
<td>0.7</td>
<td>0.8</td>
<td>0.9</td>
<td>0.5</td>
<td>3.0</td>
<td>0.85</td>
</tr>
<tr>
<td>Dense alluvium</td>
<td>8.0</td>
<td>6.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Weathered ECBF</td>
<td>10.0</td>
<td>9.2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Cemented ECBF</td>
<td>10.5</td>
<td>10.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>End of investigation</td>
<td>13.5</td>
<td>15.0</td>
<td>4.6</td>
<td>5.5</td>
<td>3.3</td>
<td>3.8</td>
<td>3.8</td>
</tr>
<tr>
<td>Groundwater level</td>
<td>1.0</td>
<td>0.8</td>
<td>0.5</td>
<td>0.6</td>
<td>0.9</td>
<td>0.7</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Table 3: T+T geotechnical investigation findings, May 2001.

<table>
<thead>
<tr>
<th>Geological Unit</th>
<th>MB3 (wash driled)</th>
<th>MB4</th>
<th>MB5</th>
<th>MB6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Topsoil/basecourse/ fill</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>Alluvium</td>
<td>0.4</td>
<td>0.3</td>
<td>0.4</td>
<td>0.5</td>
</tr>
<tr>
<td>Dense alluvium</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Weathered ECBF</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Cemented ECBF</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>End of investigation</td>
<td>7.5</td>
<td>7.5</td>
<td>8.0</td>
<td>8.0</td>
</tr>
<tr>
<td>Groundwater level</td>
<td>Not measured</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.2 Geotechnical model

Based on the investigations discussed above and no site specific investigation, we have developed a preliminary geotechnical model comprising the East Coast Bays Formation, overlain by alluvium belonging to the Tauranga Group and mantled with fill of various thickness. The inferred geological units forming our geotechnical model are summarised in Table 4. The model is subject to confirmation by site specific investigations for detailed design.
Table 4: Inferred geotechnical model

<table>
<thead>
<tr>
<th>Geological unit</th>
<th>Thickness (metres)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Fill</td>
<td>0.35 - 3.0</td>
<td>Clayey sandy SILT. No strength information was obtained.</td>
</tr>
<tr>
<td>2. Alluvium (Firm to very stiff, and loose to medium dense)</td>
<td>5.8 – 7.2</td>
<td>Interbedded clayey SILT and silty SAND, light grey and brown. Moderate plasticity, with some inclusions of decayed roots. Corrected undrained shear strength values range between 22 and 100 kPa, SPT N values range between 0 and 9.</td>
</tr>
<tr>
<td>3. Alluvium (Very stiff to hard, and dense to very dense)</td>
<td>2.0 – 2.7</td>
<td>Interbedded clayey SILT and SAND, grey mottled orange. Non plastic. Corrected undrained shear strength values range between 101 and &gt;190 kPa, and SPT N values range between 5 and 14.</td>
</tr>
<tr>
<td>4. Residual / weathered ECBF</td>
<td>0.5 – 1.3</td>
<td>Weakly cemented SILT with corrected undrained shear strength values between 60 and 109 kPa, and SPT N values of 5 in both MH1 and MH2.</td>
</tr>
<tr>
<td>5. Cemented ECBF</td>
<td>&gt;5</td>
<td>Moderately to strongly cemented, alternating SILTSTONE and SANDSTONE. SPT N values are greater than 50.</td>
</tr>
<tr>
<td>Groundwater level</td>
<td>N/A</td>
<td>Inferred to be between 0.5 m and 2.0 m below ground level. Groundwater monitoring was not carried out near the Site.</td>
</tr>
</tbody>
</table>

3.3 Seismic assessment

Seismic accelerations to be resisted by structures are dependent on the stiffness of the soils and depth to underlying rock. In accordance with NZS1170.5:2004\(^5\), we consider the site to be ‘Class C - shallow soil site’.

The design peak ground acceleration (PGA) with respect to the importance level 2 structure is outlined in Table 5. The PGAs are derived based on the recommended return periods in Standard AS/NZS 1170.0:2002\(^6\).

Table 5: Design Peak Ground Acceleration

<table>
<thead>
<tr>
<th>Design Case</th>
<th>SLS 1 Event</th>
<th>ULS Event</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Return Period (T)</td>
<td>Design PGA</td>
</tr>
<tr>
<td>Importance Level 2</td>
<td>1 in 25 years</td>
<td>0.04g</td>
</tr>
</tbody>
</table>

\(^5\) New Zealand Standard (2004), Structural design actions Part 5: Earthquake actions – New Zealand. Standards New Zealand

\(^6\) Australian/New Zealand Standard (2002), Structural design actions Part 0: General principles. Standards New Zealand.
4 Geotechnical Issues

Based on the geotechnical model and our understanding of the proposed development, it is considered that the key geotechnical issues for the Site are:

a) Foundation options
b) Negative skin friction
c) Site contamination
d) Liquefaction potential

4.1 Foundation options

4.1.1 Shallow foundations

Shallow foundations (i.e. slab or strip footings) should be restricted to light weight structures. No such light weight structures are currently proposed for the development.

Should the development change from the building configuration that is shown on the Herbst Maxcey Metropolitan Architects Ltd drawings contained in Appendix A, we recommend a settlement analysis is undertaken due to the low undrained shear strength values obtained within the upper alluvium (Refer to Table 4, Geological unit 2: firm to stiff, and loose to medium dense).

4.1.2 Piled foundations

Piled foundations can be founded in the cemented ECBF, approximately 10 to 11 m below ground surface.

We understand that screw piles are the preferred method of installation and consider these to be appropriate for the development.

As an alternative to screw piles, driven steel piles may be utilised. However, this method of pile installation may lead to additional geotechnical issues such as increased potential for liquefaction of the upper alluvium layer due to vibrations, heightened noise and vibration, pile buckling and having a variable depth to the top of rock. All these factors will require detailed investigation and may result in increased construction costs.

4.2 Negative skin friction

An assessment of the effects of negative skin friction has not been carried out however it is anticipated that the effects will be negligible due to the lack of marine and highly compressible alluvium. As good practice, it is recommended that negative skin friction is still considered and an assessment of its effects should be carried out during detailed design.

4.3 Site contamination

An environmental assessment has not been carried out for this Site, however from our experience on nearby sites, it is not anticipated that significant contaminated material will be encountered during construction.
4.4 Liquefaction potential

Liquefaction is the phenomenon whereby soil loses shear resistance under cyclic seismic loading (cyclic shear strains). A number of conditions need to be satisfied for liquefaction to occur, including having a cohesionless, saturated, loose and poorly graded material. The clayey silt in the upper alluvium layer (Unit 2: Alluvium (Firm to very stiff, and loose to medium dense) are unlikely to liquefy due to being moderately plastic, however the thin sand layers within the unit has the potential to liquefy due to its loose packing. Despite this, it is anticipated that liquefaction of the upper alluvium material will have an insignificant effect on the proposed development since it is deposited in thin layers. It is also anticipated that settlement due to liquefaction will have an insignificant effect on the development where piled foundations are used provided the ground floor is detailed as a suspended structure. A site specific detailed assessment for liquefaction potential should be carried out during detailed design to confirm the extent and effects of liquefaction.
5 Recommendations and Conclusions

5.1 Recommendations

It is recommended that a site specific geotechnical investigation is undertaken in order to confirm the inferred ground conditions. A minimum scope should include at least two (2) machine boreholes to prove 5 m of cemented ECBF, and five (5) cone penetration tests (CPT). Potential investigation locations are shown in Appendix C. Provision should be allowed for environmental sampling and testing during the geotechnical investigation.

5.2 Conclusions

On the basis of our understanding of the proposed development and available geotechnical information from nearby Sites, we make the following conclusions:

- The Site is considered suitable for the proposed development, pending the findings of a Site specific ground investigation.
- It is anticipated that the Site is underlain by fill, alluvium and ECBF at approximately 10-15 m depth.
- Groundwater is inferred to be approximately 0.5 to 2.0 m below ground surface.
- The thin sand layers within the upper section of the alluvium has the potential to liquefy due to a high groundwater. Detailed assessment should be carried out at detailed design to confirm the extent and effects of liquefaction on the proposed development.
- Further geotechnical investigation is required to confirm the inferred ground conditions for detailed design.
6 Applicability

This report has been prepared for the exclusive use of our client Murchison Group, with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose, or by any person other than our client, without our prior written agreement.

The preliminary report is for concept development and resource consent, and the recommendations are subject to confirmation by site specific investigation for detailed design and observation during construction. Recommendations and opinions in this report are based on data from boreholes and hand augers at nearby sites. The nature and continuity of subsoil away from the boreholes and hand augers are inferred and it must be appreciated that actual conditions could vary from the assumed model.

During excavation and construction, the site should be examined by an engineer competent to judge whether the exposed subsoils are compatible with the inferred conditions on which the report has been based. We would be pleased to provide this service to you and believe your project would benefit from the continuity. However, it is important that we be contacted if there is any variation in subsoil conditions from those described in the report.

Tonkin & Taylor Ltd

Report prepared by: Authorised for Tonkin & Taylor Ltd by:

.......................................................... ..............................
Ana Anufe Peter Millar
Engineering Geologist Project Director

Ana Anufe / eanu
p:\1006183\issueddocuments\423-427 beach road geotechnical appraisal report.docx
Appendix A: Herbst Maxey Metropolitan Architects Ltd Concept Design Drawings

- Ground Floor Level Floor Plan. Drawing Sheet SK02, Revision C.
- Level 01 Floor Plan. Drawing Sheet SK03, Revision C.
- Level 02 Floor Plan. Drawing Sheet SK04, Revision C.
- Level 03 Floor Plan. Drawing Sheet SK05, Revision C.