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APPENDIX C

Beca Geotech Memo

Memorandum

To:	Jo Lester**Error! Unknown document property name.**	**Date:**	28 September 2020
From:	Sarah Barrett and Jasmine La	**Our Ref:**	3324206
Copy:			
Subject:	WIAL Master Grading Retaining Wall - Geotechnical Desktop Study (Revised)		

1 Background

Wellington International Airport Ltd (WIAL) released their 2040 Masterplan publicly in October 2019. The Masterplan focuses on development of the airfield and terminal in order to increase capacity up to 12 million passengers per annum (MPPA). The development includes moving international operations to a new terminal at the southern end of the existing terminal, and long-term expansion of apron areas east, into land previously occupied by the Miramar Golf Course.

Beca has developed a 3D Master Grading Model outlining the geometric design of the proposed layout of the proposed apron expansion. The model enables consideration of geometric requirements for finished surface geometries and outlines the anticipated earthworks depths and volumes including impacts on existing trunk services. The proposed eastern expansion requires a cut-volume of approximately 480,000m^3 to reach the finished surface level with an additional cut to of 110,000m^3 required to reach pavement foundation level. Concept drawings show the expansion extending to the toe of the hillslopes to the south-east of the existing airfield resulting in a 30-metre high cut-slope retained by a concrete retaining wall. The area of the designation, including indicative road alignment allowing for the realignment of Stewart Duff Drive is shown in Figure 1.

Beca has been commissioned by Wellington International Airport Ltd (WIAL) to conduct a desk-top study examining potential options for retaining the cut-slope. We understand from the concept drawings that the cut-slope could potentially be up to 30m high, and overall 500m long and consist of three segments: a 80m west-east aligned segment below the Moa Point access road crossing a ~20° northeast facing slope, a 260m southwest-northeast aligned segment along the toe of a northwest facing slope of about 20° to 30°, and a 160m south-north aligned segment. The proposed wall is within the western boundary of Lot 1 and 2, as indicated in Figure 2).

This memorandum outlines anticipated ground conditions, geologic hazards, and an overview of potential retaining solutions.

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Figure 1: Outline the designation including indicative road alignment allowing for the realignment of Stewart Duff Drive.

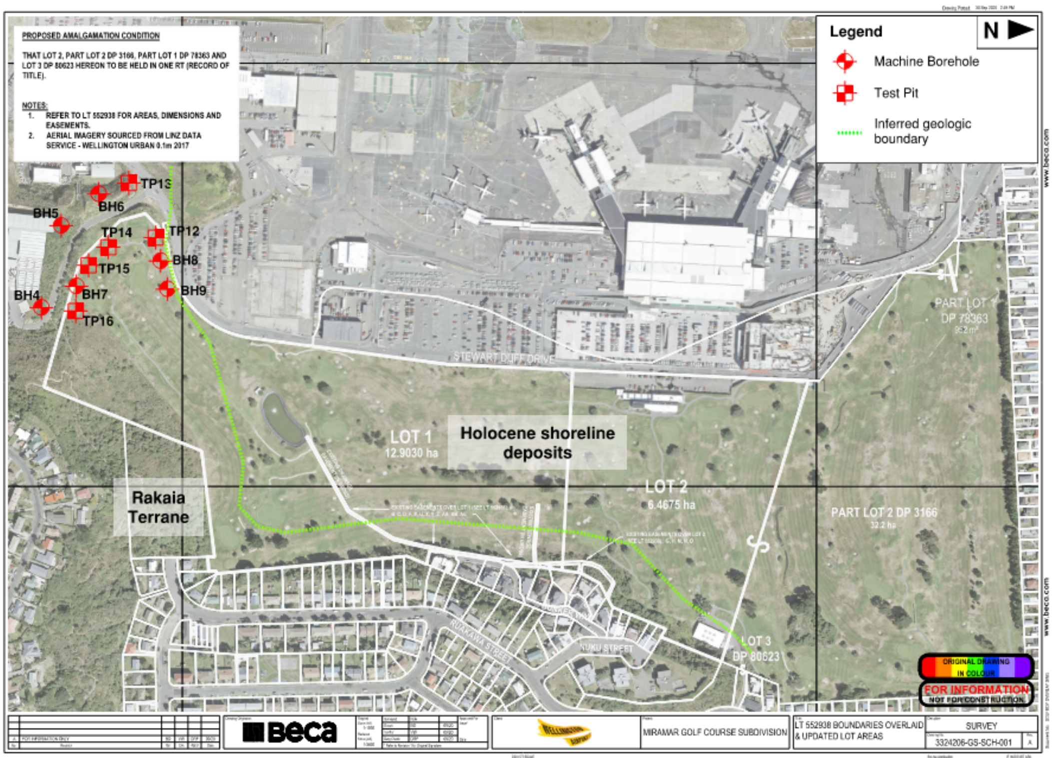

Figure 2: Alignment of proposed retaining wall shown in relation to mapped geologic units and previous geotechnical investigations.

2 Mapped Geology

The 1:250,000 published geological map of the Wellington region (Begg & Johnston, 2000) show the land as underlain by Holocene shoreline deposits of marine gravel, sand, mud, and beach ridges. The hillslopes immediately behind the golf course contain fine-grained sandstone interlayered with mudstone (argillite) consisting of greywacke of the Rakaia Terrane (Wellington Greywacke). Quaternary colluvium is present on the surface of the hillslope and colluvial wedges are present at the toe of the slopes. The location of beach deposits and greywacke with relation to the proposed cut-slope is uncertain due to the scale of the geologic map and masking by Quaternary colluvium.

3 Previous Ground Investigations

Previous ground investigations proximal to the proposed cut-slope were collated from a review of the New Zealand Geotechnical Database (NZGD), and internal Beca reports. Investigations were previously completed for the waste-water treatment facility immediately south-west of the site, as outlined in Beca Stevens (1990). Approximate locations of the investigations are shown in Figure 1. The proximity of the investigations to the proposed cut-slope and contact between Holocene shoreline deposits and Rakaia Terrane is not clear.

4 Anticipated Soil Profile

The investigations undertaken by Beca Stevens (1990) suggests that the south end of the golf course land is underlain by fill of varying thicknesses, and colluvium, underlain by g in-situ greywacke.

- Testing adjacent to the southern boundary of Lot 1 (BH7, TP15, and TP16; shown in Figure 2) revealed up to 4 m of fill consisting of sand with minor gravel, cobbles, and boulders, overlying up to 5 m of colluvium consisting of dense silty gravel, cobbles and boulders. The deposits are underlain by completely to moderately weathered greywacke at approximately 10 m depth.
- Testing in the golf course land (BH 8-9) indicates the area is underlain by up to 8 m of sand with organics and fine to medium gravel, and colluvium of sandy gravel to gravelly sand. BH08 encountered weathered greywacke at approximately 8m depth; BH09 terminated at 15m depth and did not encounter greywacke.
- Investigations at the south-east corner of Lot 1 (BH6 in Figure 1) revealed 3.0 m of sand underlain by 1.0 m of colluvium and completely to moderately weathered greywacke.

Investigations have not been performed across the remainder of the site. Underling deposits may consist of fill underlain by Holocene sands, colluvium, and/or greywacke. The thicknesses of these deposits and depth to greywacke cannot be determined without detailed geotechnical investigation.

5 Groundwater

Depth to groundwater was recorded from piezometers installed during the Beca Stevens (1990) investigations. Monitoring indicates that groundwater was encountered at 5m to 7m below ground level and approximately follows the ground contour (equating to RL 4m and RL 14m in terms of the Wellington 1953 datum).

6 Geotechnical Hazards

6.1 Fault Rupture

The Institute of Geological and Nuclear Sciences (GNS) Active Faults Database identifies the Evans Bay Fault as trending north-to-south through Evans Bay approximately 1 km west of the site. The fault is considered 'active' in that there is evidence for late Quaternary displacement (Begg and Mazengarb, 1996). Barnes et al (2018) estimate the fault has experienced one sea-floor rupturing earthquake in the past 10,000 years and is capable of generating magnitude (Mw) > 7 earthquakes. The Wellington Fault is located approximately 8 km west of the site.

It is anticipated that a rupture of the Evans Bay or Wellington Fault would result in very strong ground shaking at the site. The shaking intensities would need to be evaluated and considered in the design of retaining structures proposed for the cut-slope.

6.2 Liquefaction and Cyclic Softening

Liquefaction may occur in loose saturated sandy soils as earthquake-induced cyclic shearing causes pore-water pressures to increase and exceed confining pressures resulting in a loss of soil strength. Liquefied soils may be transported downslope towards riverbanks or coastlines resulting in large lateral displacements at the ground surface. Surface effects of liquefaction include differential settlements due to the densification of the affected sandy layers and loss of material to the surface.

Soft clayey soils may additionally cyclically soften during earthquake induced shaking resulting in a loss of soil strength. Cyclic softening typically results in a number of liquefaction-like consequences including differential settlements of the ground surface.

The Greater Wellington Regional Council (GWRC) Liquefaction Potential Map (Dellow et al., 2018) indicates that the Holocene beach deposits within the site have a moderate risk of liquefaction. The surrounding hillslopes are not considered susceptible to liquefaction. The risk of liquefaction to the cut-slope, and any proposed retaining structure, should be considered during design and is dependent on the materials encountered in the cut, depth to groundwater, and the material underlying the cut.

6.3 Tsunami

Tsunami have previously affected Wellington Harbour, including a 2.5 m high wave recorded at Lambton Quay following the 1855 Wairarapa earthquake (GeoEnvironmental Consultants, 2001). The Wellington Region Emergency Management (WREMO) Tsunami Evacuation Map shows the site as within the 'Yellow Self-Evacuation Zone'. The zone is modelled as being inundated under local source tsunamis with an Annual Exceedance Probability of 0.17, corresponding to 6000-year return period and a maximum travel time of 1 hour. Possible wave heights are in excess of 10.0m. Potential impact forces from a tsunami will need to be considered in the design of the cut slope.

6.4 Flooding

The Greater Wellington Regional Council flood hazard map indicates that the site and surrounding hillslopes are not in a designated flood hazard zone.

6.5 Slope Stability

The Greater Wellington Regional Council Earthquake Induced Slope Failure Hazard Map assigns the hillslope to the west of the golf course land a failure risk rating of 2-3 corresponding with low to moderate risk of failure. The risk is based on slope angles, local geology, and slope aspect. Changes in the slope profile from the cut will change the slope stability rating. A site-specific assessment considering impacts of the proposed cut-profile to the overall stability of the slope will be required prior to selection of any retaining structures, and during design of the selected solution.

7 Geotechnical Considerations

Expansion of Wellington International Airport to the east may necessitate a 30-meter high cut-slope in order to reduce the ground level to be consistent with the existing taxiway. If such a cut is required, Wellington International Airport will be required to ensure stability of the cut and avoid or mitigate potential impacts to residential properties upslope from the cut.

Geotechnical investigations will comprise a key component of the detailed design of any such retaining wall, and at this stage the likely materials that will be encountered in the cut are indicative only. Current concept designs of the proposed Masterplan show the cut being retained behind an approximately 500-meter long and 30-meter-high concrete retaining wall. This is a feasible engineering solution, but its final design and whether there could be other “softer” engineering techniques or alternatives will be highly dependent on the materials exposed in the cut.

Other factors that will also need to be considered as part of the detailed design is the susceptibility of the exposed and underlying materials to liquefaction; and/or the strength of any exposed rock. Stability of the cut will additionally be dictated by the angle of the cut, which is anticipated to controlled by the proposed realignment of Stewart Duff Drive and the geometric design of the proposed taxiway.

Potential retaining options that may be considered, depending on the encountered materials, are outlined below. A combination of these solutions may be considered if the soils vary along the length of the cut. The proposed solutions do not allow for provision of a roading bench for the realignment of Stewart Duff Drive.

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7.1 Benching Natural Cut Slope

The cut slope could be contoured so that the natural materials maybe retained. The feasibility of this option depends on the material encountered in the slope and the angle required to ensure the stability of the slope. Slope stability assessments incorporated geotechnical investigations would be required to assess the feasibility of this option given the limited space between the proposed cut and the overlying properties.

7.2 Mechanically Stabilised Earth (MSE) wall

An MSE wall may be considered if sand is encountered in the cut-slope and consists of pre-fabricated units of double twisted wire mesh (8×10 type) lined with an erosion control blanket and stiffened with a welded mesh panel. Construction requires excavation behind the proposed wall for the placement of geogrid and structural backfill. A 'green' option consisting of an angled front face and erosion control blanket are designed to facilitate the establishment of natural vegetation.

Potential factors impacting the feasibility of the option include:

- The available space between the retaining structure and overlying residential properties may mean the required slope angle cannot be achieved.
- Depth to rock may limit excavation behind the wall for placement of geo-grid reinforcement.
- Susceptibility of the underlying soils to liquefaction may impact the overall stability of the structure.

Figure 3 includes photographs of an example of a "green" type MSE wall.

Figure 3: Photographs of "green" MSE walls

7.3 Rock Stabilization

A retaining wall may not be required if greywacke is encountered in the cut and is assessed to be of sufficient strength and stability to mitigate against rockfall and/or slope failures. In this case, options may be considered to ensure the stability of the rock face and mitigate shedding of material. Options may include:

- Rock bolting or anchors to tie the slope together.
- Shotcreting the wall
- Meshing of the slope to mitigate against material being shedded from the slope and onto the taxiway.

7.4 Concrete Retaining Wall

A concrete retaining wall, such as that shown in the concept drawings, may be considered where the materials encountered in the slope are considered unsuitable for the above options, or where geometric designs dictate a slope profile is unable to be retained by other solutions.

7.5 Roading bench

Including provisions for a roading bench would change slope angles of the proposed cut which may be incorporated into a cut-bench solution. Selection of a suitable solutions would depend on the available space and encountered materials and may include a combination of the proposed solutions.

8 Recommendations

A ground investigation along the length of the retaining wall will be required to determine the nature of soils, depth to in-situ rock, and liquefaction susceptibility of the materials encountered in the cut-slope. The results of the investigations will be required for selection of an appropriate retaining solution and to inform detailed design of the other geotechnical options.

A slope stability assessment is additionally recommended for the area upslope of the retaining structure to mitigate impacts to the residential properties and minimize risks to downslope areas. The assessment will inform remedial design and may include a rockfall catchment fence to mitigate potential rockfall being dislodged onto the airport taxiway and/or rock anchors/soil nails to support the soil or rock above the proposed retaining wall.

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References

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