Before the Independent Commissioner Hearing Panel

Under	the Resource Management Act 1991 (**RMA**)
In the matter of	an application by **Dunedin City Council** to develop a landfill at Smooth Hill, Dunedin.

Statement of evidence of Samantha Webb

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Qualifications and experience

1 My name is **Samantha Webb**.

2 I am currently employed by Davis Ogilvie & Partners Ltd but was employed by GHD Limited during the ground investigation and reporting phase of the Smooth Hill resource consent application.

3 I am employed as an Engineering Geologist with a MSc Engineering Geology and BSc (Hons) Geological Sciences both from the University of Leeds, UK. I have worked in the field of geology and civil engineering for 31 years.

4 I have worked on the pre-characterisation of material sources for suitability as landfill lining material on a number of landfills in the UK and was involved in the Grey District Council landfill construction using the re-use of the Kaiata Formation. In the late 1990s I worked with the Southland Regional Council on looking for a suitable site for their new landfill, this included a detailed assessment of a site in the Hokonui Hills and a preliminary assessment of a second site at Ohau.

5 I have more recently worked on the Waimea Dam, Brightwater, looking at the stability of a temporary rock slope above the water intake structure.

6 I have read the Code of Conduct for Expert Witnesses in the Environment Court Practice Note 2014. This evidence has been prepared in accordance with it and I agree to comply with it. I have not omitted to consider material facts known to me that might alter or detract from the opinions expressed.

Scope of evidence

7 I have been asked to prepare evidence in relation to the interaction of the ground conditions of the proposed landfill site and the landfill design. This includes:

(a) explanation of the geotechnical ground investigation undertaken at the site;

(b) commentary on the results of the investigation including the laboratory testing results; and

(c) a description of how the geology and the landfill design interact, with an emphasis on the slope stability of the landfill and the stability of the earth fill toe bund under seismic load.

8 It should be noted that not all of the work undertaken to support the consent application was undertaken by myself but was a collaboration of a team of geo-professionals overseen by me.

Executive summary

9 As part of a team of geo-professionals I have overseen and reported on three phases of geotechnical investigation and two rounds of laboratory testing undertaken in and around the proposed landfill facility at Smooth Hill. The findings of the investigation are consistent with the published geology, in that the site is underlain by surficial loess soils overlying the Henley Breccia, which is a geological unit comprising interbedded siltstones, sandstones and coarse breccia. I agree with Tonkin &Taylor's s95 suggestion that additional investigation will be required during detailed design.

10 The laboratory testing confirmed the suitability of re-use of the weathered Henley Breccia as engineered fill in the construction of the slopes and toe bund. The testing confirmed the loess soils can be compacted to achieve a permeability of $x10^{-8}$ m/s but it also identified the loess is dispersive. Stabilisation of the loess to prevent dispersivity can be achieved with the addition of lime, however the testing was inconclusive on whether the addition of lime changes the deformation characteristics of the soil. If loess is to be used in the final liner system, this will need further consideration.

11 The stability of the more moderately to slightly weathered Henley Breccia is dominated by bedding. Taking a qualitative approach to the assessment of slope stability of the landform, the orientation of the bedding is favourable for two of the cut slopes and potentially unfavourable for the northwest facing slope above Stage 1. However, the proposed design angle of this permanent slope at 1 vertical : 4 horizontal is less than the mapped bedding, which indicates a kinematically stable slope. Further slope stability analysis will be part of detailed design, and this meets T&T’s s95 direction for additional analysis.

12 The seismicity of the site will be dominated by the Akatore Fault. Preliminary ground acceleration of 0.5 g (the ‘g’ units are units of acceleration, in that 1 g = 10 metres per second squared) has been provided by Professor Mark Stirling and this has been used in the most recent work. A site specific probabilistic seismic hazard assessment (SSPSHA) will be undertaken as part of the detailed design to confirm the design event earthquake. This will form a condition of this consent.

13 Proprietary slope modelling software Slope/W was used for modelling of the earthfill toe bund, against which the landfill waste is toe buttressed. This

modelling indicates the bund is stable (FoS >1.5) under static conditions. Using the provisional ground acceleration of 0.5 g for seismic modelling, the pseudo-static modelling produces expected factors of safety of less than 1 (<1). This indicates that during a seismic event, deformation of the bund will occur. Modelled deformation in accordance with the *Waka Kotahi New Zealand Transport Agency Bridge Manual* 3rd Edt Oct 2018 (NZBM) indicates that 2 mm of deformation may occur at the interface of the toe bund with the base of the landfill and 14 mm may occur on the visible downstream face of the bund. The former will require consideration as part of the liner design, the latter will require consideration during bund design, both during detailed design.

14 My evidence further includes, my discussion with Tonkin & Taylor as part of the s92 and s95 process; recognition and response to public submissions regarding specific areas of concern; and proposed wording for three specific geotechnical conditions.

Ground Investigation – Scope

15 The first stage in the ground investigation was to conduct a desk study to collate as much available published information about the local ground conditions. This drew reference from the following published GNS geological maps: Bishop (1994) at 1:50,000 scale; and Bishop and Turnbull (1996) at 1:250,000 scale.

16 Based on the desk study, a conceptual ground model was prepared that identified the site was likely to be underlain by rock of the Henley Breccia of Upper Cretaceous age covered by shallow loess soils. The nearby Titri and Akatore faults were identified.

17 Using the desk study data, a ground investigation technique called triple tube diamond core drilling was determined to be the best method to retrieve high quality core recovery. This would be supplemented by machine excavated test excavations (trial pits) to provide the opportunity to retrieve bulk samples of the shallow soils.

18 The distribution of investigation locations was set out to optimise coverage of the original landfill footprint so that a site-specific ground model could be developed, which confirmed the published data (Attachment A).

19 The number of investigation locations requires a balance between anticipated geological complexity, site access constraints, stage of design, programme and budget. In my opinion and experience, the number and spread of investigation locations undertaken at Smooth Hill meets the demands of the aforementioned balance for this project at this phase of

design. Additional holes of various depths will be required during detailed design to answer specific design queries.

20 It should be noted that with any planned investigation, the actual positions of the test locations are constrained by being able to physically and safely manoeuvre a drilling rig into position while also avoiding inappropriate ecological impacts.

21 The ground investigation was phased as set out below:

(a) Phase I was undertaken during May/June 2019 by McNeill Drilling (BH01 to BH10);

(b) Phase II was undertaken in October/November 2019 by Speight Drilling Ltd (BH201 to BH211); and

(c) Phase III was undertaken in August 2021 by Speight Drilling Ltd (BH301).

22 During drilling supervision of Phase I, the Engineering Geologist on site (Matt Fitzmaurice), geologically mapped the site and immediate environs including surface features associated with the overlying loess soils and small areas of rock exposure.

23 The second phase of drilling was used to fill in gaps identified after Phase 1. However, access to parts of the site were further restricted at this time due to ecological investigations and not all Phase II planned investigations were carried out (BH204 and BH210).

24 Phase III drilling was undertaken in response to Tonkin & Taylor's peer review and ORC's s92 request for further information. Phase III (BH301) was used to specifically answer a question in the ground model and hydrogeological model that had arisen during the concept design. Its location was possible because a section of previously inaccessible land had become available for investigation following the felling of a stand of trees.

25 With the reduction in landfill footprint from the original concept, a number of investigations fall outside the footprint of the landfill that is the subject of this application. However, the proximity of the data to the site means the data is still valid for contributing to the site ground model.

26 The ground investigation also included laboratory testing. Soil and rock testing was undertaken by International Accreditation New Zealand (IANZ) accredited Central Testing Services in Alexandra.

27 Shallow depth core samples and bulk disturbed samples from Phase 1 were selected for soil testing. The emphasis of this round of testing was to generally classify the surficial soils.

28 Bulk disturbed sampling from the Phase II test pits was to specifically retrieve samples of:

(a) loess soils; and

(b) completely weathered (CW) and highly weathered (HW) Henley Breccia.

29 The loess soil samples underwent pre-characterisation testing to specifically identify their suitability for re-use as a low permeability liner material.

30 The bulk samples of CW and HW Henley Breccia were tested to consider the suitability and strength of this material as engineered fill below the landfill liner, and construction of the toe bund.

31 To provide data for the hydrogeologists, standpipe piezometers were installed in a number of the boreholes. Details about these form part of Anthony Kirk's evidence and are not discussed further in my evidence.

Ground Investigation – Results - Geology

32 On completion of the investigation described in my earlier evidence, the data was collated and reported in the Geotechnical Factual Report (GFR) (Appendix 6 of the application). I was not the author of this report but oversaw the team working on it and signed off on the report as factually representing the data we had collected.

33 The GFR presents the desk study, borehole logs, test pit logs and laboratory results.

34 All logs were logged in accordance with the principles set down in *The Field Description of Soil and Rock – Guideline for the field classification of soil and rock for engineering purposes*, NZ Geotechnical Society Dec 2005. All subsequent descriptions of soils and logs use terms from this guide e.g. weathering grades, and strength.

35 Using the factual data, a Geotechnical Interpretive Report (GIR) was prepared (Appendix 5 of the application). I was not the sole author of this report but oversaw the team working on it and signed off on the report.

36 The borehole logs confirm the site is underlain by the Henley Breccia. The logs identify the Henley Breccia to be an interbedded sequence of primarily massively bedded siltstone, sandstone and coarse breccia with few defects. The degree of weathering ranges from shallow completely weathered (CW), to un-weathered (UW) with depth. The strength of the rock is less controlled by weathering grade and is more a function of the cementation of each unit. Cementation is the process in which sedimentary grains are held together by natural cements that are typically produced when water moves through rock and soil.

37 During aerial photo reconnaissance as part of the desk study phase, and confirmed during field mapping and test pitting, a number of discrete localised areas of shallow seated instability were identified. This is discussed later in my evidence.

Ground Investigation – Results – Laboratory

38 The laboratory testing focussed on two particular aspects:

(a) the suitability of the loess for re-use as a low permeability liner material; and

(b) the suitability of the weathered Henley Breccia for use as a structural engineered fill.

39 Because Loess can be a highly erosive soil, in addition to the Phase I standard soil classification tests (Particle Size Distribution, Atterbergs, Standard Compaction), the loess was also specifically tested for:

(a) Dispersivity (ASTM D4647-13e1);

(b) Crumb test (ASTM D6572-13e2 (Method B); and

(c) Triaxial permeability on re-compacted sample (ASTM D5084-16a).

40 The tests confirmed the loess:

(a) plots on the A-Line of the Casagrande plasticity chart – which means it has some plasticity;

(b) is dispersive – which means the soil loses its structure when it becomes saturated; and

(c) could achieve a compacted permeability (at optimum moisture content) in the range of 2.8 x 10^{--8} to 5.3 x 10^{-10} m/s – which means it can be described as being of low permeability.

41 Plasticity and low permeability are desirable properties for landfill liners; whereas dispersivity is an undesirable property.

42 The dispersivity of Loess soils can be treated by stabilising the soil with the additional of lime (or cement). However, there was concern that the addition of lime could make the loess lose its plasticity and become brittle in its deformation behaviour, which is undesirable.

43 Based on the Phase I lab test results, additional testing was scheduled on Phase II samples focussing on the effects of lime addition to permeability and plasticity of the Loess.

44 At the same time, it was decided to also test the effect of the addition of bentonite to loess as a stabiliser and to maintain its plasticity.

45 The bulk disturbed samples of Loess from Phase II were combined, mixed and sub-sampled to generate representative samples of how the material would be used on site.

46 The mixed loess samples were tested for:

(a) lime demand – to identify how much lime is required to stabilise the soil;

(b) Atterberg testing before and after lime stabilisation and bentonite stabilisation – to understand the plasticity behaviour;

(c) compaction of lime stabilised and bentonite stabilised soils; and

(d) shear strength of lime stabilised and bentonite stabilised soils.

47 The results show:

(a) improvement in dispersivity for lime stabilised samples (but not for bentonite); and

(b) nominal increase in plasticity for lime stabilised soils; and no change in plasticity for bentonite stabilised soils.

48 The results show that dispersivity of the loess soils can be mitigated by the addition of lime. However, the laboratory testing is inconclusive regarding the effect of either lime or bentonite on the plasticity of the loess.

49 The completed testing indicates that the loess may be suitable for use in the landfill lining system. In Mr Coombe's evidence he presents a design scenario which utilises loess as part of the landfill lining system. Further testing will be required during detailed liner design to better understand how

the loess can be used as part of the liner system and the option remains open to use other imported materials instead of the loess if they meet the WasteMINZ liner design criteria.

50 The suitability of the weathered Henley Breccia for use as a structural engineered fill was established by undertaking standard compaction testing of disturbed samples and unconfined compressive strength testing of re-compacted samples.

51 The strength tests of the compacted Henley Breccia have been used in the stability analysis of the landfill design which I discuss later in this evidence.

How the geology influences the landform part of the landfill design

52 The landfill design will comprise a bowl-like landform to be cut into the natural topography. This requires:

(a) the natural slopes to be cut to form stable engineered slopes;

(b) areas of engineered fill to be placed on the slopes to fill in natural hollows and create a uniform surface; and

(c) construction of an earthfill bund to toe buttress the waste at the bottom of the slopes.

53 I will discuss the stability of the slopes, followed by the earthfill bund.

54 Soil slopes have a natural angle of stability, and this is a function of two parameters; "phi" (the internal angle of friction) and “c” (cohesion). Slopes which exceed this natural angle will slump and slip.

55 Rock slopes are typically dominated by the friction between planes of weakness or fabric in the rock, and these may be bedding, discontinuities or joints. Depending on the orientation of bedding and/or discontinuities in relation to the orientation of slope, rock slopes are described as being either favourable or unfavourable. Very highly fractured rocks can behave very similar to soils when they fail.

56 For this site, the natural slopes comprise

(a) a shallow mantle of Loess (soil) a few metres thick; overlying

(b) CW to HW Henley Breccia, which has more soil-like properties; over

(c) moderately weathered (MW) to slightly weathered (SW) Henley Breccia, which behaves like a rock.

57 These will be discussed in the order a), c) and then b)

Surficial Soils - Stability

58 Discrete areas of shallow, localised instability of the covering soils were observed during the field mapping. These lobes of disturbed soil were mapped and confirmed that they appear to wholly reside in the surface soils, mostly loess and loess derived soils and possibly in the upper levels of the CW Henley Breccia.

59 Because of the known propensity to instability of loess soils, and because the loess is valuable as either lining, capping or daily cover material, all loess soils will be removed from the landfill footprint. This will remove this mechanism for slope instability.

60 Other unsuitable materials such as localised areas of fill associated with the former forestry skid sites, and shallow colluvium/alluvium in the base of the valley will also be wholly removed during construction. This will also remove any other areas of local instability beneath the landfill footprint.

MW to SW Henley Breccia – Stability

61 Logging of the recovered cores of the Henley Breccia indicate the bedding is typically *thick* (0.6 m to 2 m) to *very thick* (> 2 m); it can also be described as *massive*, i.e. with no discernible fabric observed within the rock units. In addition, very few discontinuities were recorded in the recovered core.

62 The strength of the rock appears to be dominated by the cementation of the grains and not by the weathering profile. Its strength is logged as ranging from very weak to strong.

63 The regional dip of the Henley Breccia on the published geology maps ranges from 15° – 30°, dipping generally to the northwest direction. The closest measured dip angle to the site is recorded with a dip of 18° at an estimated dip angle of 329° (measured off the published GNS map).

64 By way of explanation, the orientation of any plane in a rock mass is described by its dip (how many degrees from horizontal) and dip direction (the compass direction in which that maximum dip points to). This is graphically shown in Attachment C.

65 The massive nature of the rock, the lack of fabric and rare discontinuities means the rock slope performance will be dominated by bedding orientation.

66 My evidence focusses on the stability analysis of the current landform based on the updated May 21 proposed landfill design.

67 During the life of a landfill there are both temporary slopes and permanent slopes. Excavation of the landform occurs progressively as the stages of the landform are filled. Temporary slopes are cut at a steeper angle than permanent slopes. A permanent slope is the final angle of the slope and must be stable in the long term. A temporary slope, for example the slopes above Stage 1, may initially be cut steeper than the final slope angle because it does not have to meet the same long-term stability requirements. This is to minimise the quantum of earthworks in the early stages.

68 To simplify the slope stability analysis, the current engineered landform has been analysed as three cut slopes and an earthfill bund. (Attachment B) The orientations of the three cut slopes are

(a) a slope facing to the northwest;

(b) a northeast facing slope; and

(c) a slope facing to the southwest.

69 For the concept design, a qualitative statement has been made about the stability of the landform slopes with respect to the Henley Breccia. This qualitative assessment is based on the regional dip of the Henley Breccia and whether the bedding is "favourable" or "unfavourable" as set out below (this is graphically explained in Attachment C):

(a) "favourable" means the bedding is dipping into the slope, and the slope is typically stable; and

(b) "unfavourable" means the bedding is dipping out of the slope, and the slope could be unstable.

70 By geometry, the following statement can be made:

(a) the northeast facing slope is generally favourable;

(b) the southwest facing slope is generally favourable; and

(c) the northwest facing slope is <u>unfavourable.</u>

71 The maximum proposed angle of cut of the northwest facing slope is 1 vertical : 4 horizontal (~14°). This is a very shallow angle and is compatible with the mapped regional dip of 15°- 30°. This, together with the absence

of fabric and rare discontinuities, indicates that the cut slope should be stable.

72 The Phase III borehole was drilled specifically in the centre of this northwest facing slope to add an extra data point to the model. This borehole confirmed the same geology as my understanding of the site-wide setting.

73 In summary, from a qualitative stability assessment of the slopes, cutting the slopes at 1 vertical : 4 horizontal in the MW and SW Henley Breccia is assessed as stable.

74 The resulting permanent slopes have been designed to have an overall slope angle of 1 vertical : 5 horizontal with the steepest sections being the inter bench slopes of 1 vertical : 4 horizontal. For temporary slopes the angle will be a maximum of 1 vertical : 3 horizontal. These angles have been chosen to not exceed the stability angles of the natural geology.

CW to HW Henley Breccia

75 The core photos of the HW Henley Breccia, show relict rock fabric, i.e. the rock has not completely weathered to a soil. But, its likely failure mode will be more similar to a soil than a rock, and therefore for design purposes it has been assigned soil properties of “c” and "phi".

76 The strength of this material is logged as very stiff (soil like), to weak, to very weak (rock like).

77 Where the landform requires cutting into the slope, much of this CW to HW material will be removed, therefore its in-situ performance on the slopes has not been further assessed.

78 In the current design this material forms the foundation of the earthfill bund at the toe of the landfill. To assess the stability of the bund, this material is included in the model. For the model, all materials are assigned a strength. A conservative (low) phi of 30° and a cohesion 2 have been assigned to this material in the bund stability model. The results of the modelling are discussed later in my evidence.

79 Where remnants of this material are encountered (either on the slopes or the bund foundation) during construction, its suitability will be assessed by a geoprofessional on site and if necessary, undercut (i.e. removed) down to good soil/rock.

80 This material has been assessed for re-use as an engineered fill and this is discussed next.

Engineered Fill

81 To create a uniform slope and shape of the designed landform, both slope cutting and earth filling will be required.

82 The filling of hollows on the slopes and base, and the construction of an engineered earth fill bund at the toe of the slope using site-won weathered Henley Breccia is discussed next.

83 As mentioned earlier in my evidence, bulk samples of CW and HW Henley Breccia were tested in the laboratory for compaction performance and re-compacted strength.

84 All engineered fill requires the careful placement of material in a controlled, systematic manner. Thin layers, (typically no more than 300 mm thick), of soil are placed and rolled under compactive effort until the material achieves its maximum density at optimum moisture content. The next layer is then placed on top, rolled, tested and repeated. This ensures that a high degree of compaction of the material is achieved, and the resulting engineered fill is at its maximum strength.

85 Using this approach, the site-won CW to HW Henley Breccia will be used for one of two purposes:

(a) filling beneath the design level to create an even landform and slope; or

(b) construction of an earthfill bund at the toe of the landfill, which will buttress the waste.

86 The laboratory testing confirmed that the site-won weathered Henley Breccia could be successfully compacted to perform as a strong stable engineered fill. Using strength parameters derived from the laboratory testing, the landfill toe bund has been modelled to buttress the waste. Proprietary slope modelling software Slope/W (GeoStudio2021.3) using limit equilibrium analysis has been used by others under my direction to model the stability of the toe bund.

87 The toe bund model has been tested under both the static and seismic load cases. Before the results of this modelling are presented, I will provide a discussion on the site's seismicity setting.

Site Seismicity

88 The site seismicity is addressed in Professor Mark Stirling's written evidence. However, his expertise inputs into the design and some comment

is required in my evidence on how this translates into the slope stability assessment part of the design.

89 The block of land between SH1 and the sea, on which, the Smooth Hill site is located, is bounded by two NE/SW trending faults, the Titri Fault and the Akatore Fault. In Professor Mark Stirling's evidence he discusses that it is the Akatore Fault that is most significant for this site because of its recently confirmed recurrence interval of less than 5000 years.

90 In New Zealand we are required to consider the effect an earthquake may have on infrastructure. This means in addition to assessing how "structures" behave under the static load case, i.e. the everyday gravity loads, we also must consider the seismic load case when an earthquake occurs.

91 For slopes and embankments, seismic stability is assessed by applying a horizontal ground acceleration to a model. This is known as the pseudo-static approach. The ground acceleration applied is referred to as the peak ground acceleration (PGA). The key question is: which PGA to use?

92 PGA is expressed in terms of fractions of g (the standard acceleration due to Earth's gravity) as a decimal e.g. a PGA of 0.5 g is 0.5 x $9.81 m/s^2$ = $4.9 m/s^2$.

93 For structures, specific guidance on how to determine the design PGA is provided in *Structural Design Actions, Part 5 Earthquake Actions New Zealand* NZS1170.5: 2004 (1170.5).

94 For bridges and roads, guidance on how to determine the design PGA is provided by the NZBM.

95 Both of these documents consider the design life of the structure, where it is located in relation to known active faults, then they consider the likelihood of the fault moving within the chosen design return period and what is the likely magnitude and PGA of an event.

96 For landfills, there is no such specific New Zealand design guidance, in fact Section 1.1 of NZS1170.5 specifically excludes slopes.

97 In lieu of specific guidance, both NZS1170.5 and NZBM were originally considered early in the design process and a preliminary PGA for the ultimate limit state event was calculated as 0.31 g based on an Alpine Fault rupture.

98 However, during the investigation and reporting phase of this work GNS has published new data on the Akatore Fault, and on Prof. Stirling's advice, the design team have re-run the models using a higher PGA to represent

the action of the Akatore Fault. I will discuss these results in the next section.

99 Prof. Stirling's evidence refers to an indicative PGA for the Akatore Fault of 0.5 g.

100 I am also recommending that as part of detailed design, a site specific probabilistic seismic hazard assessment (SSPSHA) should be undertaken to determine the design magnitude and peak ground acceleration for the Smooth Hill site. In Prof. Stirling's evidence he refers to this as a Seismic Probabilistic Hazard Assessment (SPHA).

101 In Prof. Stirling's evidence he provides his opinion that the likelihood of direct fault displacement at the site is extremely low and he provides his justification for this conclusion. No modelling of ground rupture has therefore been undertaken to date.

Results of earthfill toe bund modelling

102 The earthfill toe bund will retain the landfill waste. The earthfill bund will be constructed using site-won weathered Henley Breccia placed as an engineered fill. In my earlier evidence I have discussed the results of the lab testing on the suitability of using this material.

103 Using Slope/W (GeoStudio2021.3) the stability of this earthfill bund has been modelled. In the following paragraphs I describe the analysis that has been undertaken. This is an update from the analysis presented in GHD's Interpretive Geotechnical Report submitted with the updated application (Appendix 5 of submission) and is based on Prof Stirling's latest advice. The models have considered

(a) the internal stability of the bund itself; and

(b) the sliding resistance of the bund interface with the natural ground when the landfill is at capacity i.e. when there is the maximum driving force on the earthfill toe bund.

104 All models were run for both the static and seismic load conditions.

105 To help explain the technical terms "user defined" and "entry/exit mode", which I refer to in the following paragraphs, I will refer to the model outputs presented in the attachments (Attachment C and C1 – C3).

106 Also, I provide an explanation of the term Factor of Safety (FoS). This is the ratio between the restoring forces (the forces trying to keep the slope stable) and the disturbing forces (the forces trying to cause instability). A

FoS of 1 therefore represents an equilibrium state between restoring and disturbing forces.

107 For the **static** load condition, *user-defined* slope surfaces were used to force the model to consider specific failure modes. The results were:

(a) the internal stability of the earth bund returned a FoS of 3.720; and

(b) the sliding resistance of the earth bund returned a FoS of 4.058.

108 This means that under everyday gravity loads, the bund is stable.

109 To provide a margin of safety in design to take into account natural variability of the ground, a slope with a FoS of ≥ 1.5 under static loading is typically required.

110 For the seismic load cases, initially the *entry/exit mode* was used to seek its own failure surfaces, either through the internal stability of the bund or the sliding resistance of the bund.

111 Two specific cases were explored:

(a) the model returning the lowest FoS; and

(b) the model returning a failure circle that extends into the waste – this model has been labelled “loss of confinement”.

112 The lowest FoS returned for the indicative PGA of 0.5 g produced shallow failures in the downstream face of the earthbund (Attachment C.1) i.e. localised internal instability. For a 0.5 g event: FoS 0.670 (<1).

113 A failure of this type and magnitude would be visibly obvious, repairable, and would not intersect with the liner. It would not cause leachate or landfill waste to escape from the site.

114 The “loss of confinement” FoS returned for the indicative PGA of 0.5 g produced failures that intersected with the interface between the upstream face of the bund and the waste, i.e. one that could potentially affect the liner or waste (Attachment C.2). For a 0.5 g event: FoS 0.764 (<1).

115 A failure of this type and magnitude would probably be visibly obvious, difficult to repair and would potentially intersect with the liner. It could cause landfill waste to spill over the bund. However, it is unlikely leachate would escape from the site because the depth of the failure surface will be above the leachate.

116 Because prevention of leachate escape caused by breach of the liner is critical to the design of the landfill, a further set of slope stability models were run. This time, the potential slip surfaces were *user- defined* to force a failure along the interface of the landfill earth bund and the ground, where leachate will be impounded. This model was labelled “global stability” (Attachment C3). For a 0.5g event: FoS 0.869 (<1)

117 For all of the above 0.5 g seismic load conditions, FoS of <1 are returned in nearly all models; but this was expected due to the way it was modelled.

118 A FoS of less than one (<1) means that deformation of the slope is considered likely to occur. So far, I have presented the results for the pseudo-static approach for modelling slopes under the seismic case.

119 Because the pseudo-static approach typically produces FoS <1, the engineering industry understands that it is not helpful for finding a solution for slope design under seismic load. To this end, the NZBM instead directs designers to consider the likely ground displacement of the slope under seismic load.

120 NZBM requires the evaluation of deformation by multiple methods using “*moderately conservative soil strengths*”. For this assessment, three methods have been used and the 50th percentile (mean) and 84th percentile (mean + 1 standard deviation) deformation in millimetres (mm) is reported for a 0.5 g event.

121 For the shallow failures (lowest FoS) in the downstream face of the earthfill bund, using a yield acceleration of 0.3 g and a design PGA of 0.5 g, the displacements are:

(a) 50th percentile range 4 - 14 mm; and

(b) 84th percentile range 13 - 38 mm.

122 For the global failure along the base of the landfill, using a yield acceleration of 0.4 g and a design PGA of 0.5 g, the displacements are:

(a) 50th percentile range 1 - 2 mm; and

(b) 84th percentile range 2 - 5 mm.

123 This quantum of movement is within the tolerable range of slope movement. During detailed design this movement will be used in assessing the tensile capacity of the liner materials to ensure the liner system does not tear.

124 NZBM (Section 6.3.2) instructs users to use the 50th percentile and "*upper bound values*". Therefore, for global stability, the design must be able to withstand 2 mm of movement under the design seismic load condition.

125 For shallow surface slips on the downstream face of the toe bund, 14 mm of deformation can be expected. This quantum of deformation is easily manageable in design.

ORC peer review s92 and s95

126 As part of the ORC section 92 (s92) request for further information process I had a number of telephone meetings with Andrew Stiles from Tonkin & Taylor (T&T) to discuss issues he had raised during his peer review. This led in particular to a number of refinements to how the overall landfill slope stability has been assessed. As a result of these discussions, there remain three outstanding items which appear in the T&T section 95 (s95) report under mitigation. These are:

(a) (para 39) Paucity of investigation;

(b) (para 40) Appropriate seismic design parameters; and

(c) (para 41) Cut and fill slope stability.

127 The following paragraphs describe the s92 discussion and how the outstanding three s95 items will be addressed.

128 Because the s92 review was carried out in two parts, the factual report and the interpretive report - my evidence below is split into those same two parts. Some topics have been combined for clarity e.g. all the loess questions are commented upon together.

ORC s92 Peer Review – Geotechnical Factual Report

129 There was discussion on the permeability of the loess and its suitability for use as a liner. Mr Stiles raised the issue of undertaking additional permeability testing using leachate as the permeant liquid to evaluate if this alters the permeability. As discussed earlier in my evidence, Loess has been considered as one option for use in the landfill liner construction and I agreed that further testing, including leachate permeability testing should be undertaken as part of detailed design.

130 Mr Stiles and I discussed the identification of nearby faults and whether the Alpine Fault was the primary driver. As discussed earlier in my evidence I have now incorporated Prof. Stirling's evidence based on the latest studies

into my analysis and including a SSPSHA as a condition of consent. This resolves the second of the three outstanding items in the T&T s95 report.

131 It was agreed that the extent and number of test locations was adequate for concept design, but that additional investigation is likely required for detailed design. During the peer review process an additional borehole (BH301) was completed to fill in the perceived “gap” in the testing footprint due to the previous presence of a remnant area of forest. The additional borehole partially resolves the s95 item regarding paucity of investigation.

132 A rock outcrop on site and logging of some of the core raised a discussion about whether the top conglomerate units were in fact the Taratu Formation, which unconformably overlies the Henley Breccia in the geological sequence in the wider region. Further research on the Henley Breccia concluded that rounded conglomerates are found within the Henley Breccia and all core recovered at the site has been re-labelled as Henley Breccia. It was agreed that the geotechnical properties of the two formations were sufficiently similar that their performance was the same and therefore differentiation between the two units was an academic exercise. This was also raised in the interpretive report review comments; this statement responds to both.

ORC Peer Review – Geotechnical Interpretive Report

133 The potential failure mode of the Henley Breccia was discussed and whether or not it could be driven by discontinuities in the rock. For the revised landfilll landform, this is how the slope stability analysis has been analysed in earlier paragraphs of my evidence. This begins to address the third mitigation item of the s95 report.

134 The recent addition of BH301 to the data set adds robustness to the current ground model and has confirmed the same massive nature of the rock. However, it is agreed in response to paragraph 39 of the s95 report, that additional investigations will need to be undertaken during detailed design to fill in any knowledge gaps in the ground model or where specific design items require more detail

135 Additional investigation will focus on the bedding orientation and inter-unit shear strength to add to the ground model and understanding of its performance. This will add robustness to the slope modelling and respond to paragraph 41 of the s95 report.

136 The issue of the presence of areas of shallow instability has been addressed by confirmation that these features will be fully excavated during construction of the landform during the removal of all unsuitable shallow

surficial soils (loess, topsoil, buried topsoil, alluvium, fill). We agree that the geotechnical hazards posed by shallow instability in the loess; compressible alluvial soils; and the potential presence of liquefaction prone alluvium, have all been adequately dismissed in my evidence.

137 In response to the peer review discussion, the whole site seismicity has been re-evaluated, as presented in my earlier evidence at paragraphs 87 to 125. The NZS1170.5 “Importance Level” of the landfill to determine its design life and PGA will be superseded by the SSPSHA. As stated previously, this will resolve the s95 item on seismicity.

138 The peer reviewer and I agree on the geotechnical design parameters used in the models. The design parameters for the Henley Breccia were discussed and re-evaluated. A combination of rock mass strength, uniaxial compressive strength and geological strength index based on field strength data and logging observations have been used to better define the different units within each weathering zone. These will be reviewed again at detailed design as part of the stability assessment.

139 The landfill slopes have been modelled with the shallow groundwater table below the slope. The strong downward gradient of the groundwater is anticipated to keep the slopes in a drained condition, which is favourable for stability.

140 The entire approach to how the landform and the bund has been modelled for stability has been updated during the peer review process and has been discussed and agreed with the peer reviewer. This has been described in my earlier evidence. This same approach will be used during detailed design to confirm the cut and fill slope stability.

Response to section 42A report

141 With reference to pages 15 and 16 of the ORC S42A submission report dated 20/4/2022 requesting consideration of […if a mechanism for amendment of the design was built into the consent conditions to provide more certainty regarding what is and isn’t being authorised by the consent.]

142 This request is in specific relation to design changes that could need to be made in response to the SSPSHA and slope stability analysis. In my opinion, based on the current design inputs (namely 0.5 g), changes are likely to fall within the envelope of the current design. The critical part of the design, captured by the draft Conditions of Consent, is that the slopes meet the specified factor of safety. Including a condition limiting the scope of changes that could be made to slope stability would be limiting for the landfill designer.

Response to matters raised in submissions

143 Matters raised in submissions received can be categorised into four topics:

(a) site seismicity and the effect an Akatore Fault movement could have on the site;

(b) choice of material for use as a liner;

(c) land stability; and

(d) extent of investigation – only 50 %.

144 Submissions were received, on the subject of site seismicity and the risk from the Akatore Fault, from

(a) Ōtokia Creek and Marsh Habitat Trust;

(b) James Molloy;

(c) Brighton Surf Lifesaving Club R Aburn;

(d) A Hutchison;

(e) S & B Judd;

(f) S Laing;

(g) Saddle Hill Community Board;

(h) K Schneider;

(i) South Coast Neighbourhood Society Inc; and

(j) M Sydor.

145 The recent updates on the Akatore Fault have been incorporated into my evidence and analysis and a SSPSHA will be undertaken to provide the seismic design parameters for the slope stability assessment at detailed design.

146 In regard to the potential for ground rupture at the site, Prof Stirling's evidence notes that direct fault displacement at the site is extremely low. He provides an overview of why he has reached this conclusion.

147 A submission was received on the choice of material for landfill liner from James Malloy.

148 The dispersive nature of the loess soil has been identified as a potential risk to the suitability of using this material as a liner material. The laboratory testing shows that the loess soils can be made to be non-dispersive by the addition of 3% lime. The liner design is part of detailed design and use of modified loess remains a preferred option. In the event that it is not found to be suitable an alternative and suitable material will need to be imported. Mr Coombe describes the design of the liner materials and how the loess may be incorporated into a multi-layer liner system.

149 A submission was received on the stability of the landfill slopes from the Mosgiel Taieri Community board.

150 The stability of the natural landform by the creation of cut slopes and placed engineered fill has been the subject of much of my evidence and has been peer reviewed to the satisfaction of the peer reviewer. As discussed, localised shallow slips within the loess and soils will be fully excavated during construction of the landform.

151 A submission was received on the coverage of ground investigation from the South Coast Neighbourhood Society Inc.

152 The nature and lack of complexity of the local geology means that whilst only 50% of the site has been investigated, the other nearby BHs do add value to the ground model. The addition of an extra borehole (BH301) does add another data point within the footprint. Therefore, I am confident that the current level of investigation is appropriate for this site and current stage of design, and note that more investigation will be carried out during detailed design.

Conditions

153 The following three conditions are recommended to be included as part of the consent. The wording of the proposed conditions below differs from that suggested by T&T in their s95 report and “matters arising” document, however, the intent is the same and addresses the same issues.

154 Geo condition A: The detailed design of the landfill will demonstrate the short term (construction and operation) and the long term (closure to post closure) stability of all cut and fill slopes of the landform. This will be achieved by undertaking quantitative limit equilibrium slope stability assessment of the designed landform and earthfill retaining bund to demonstrate a factor of safety for cut and fill slopes in the static load case of ≥ 1.5, and for slopes where the factor of safety is <1 in the pseudo-static seismic load case, the displacement method shall be considered as per Section 6.3.2 of the NZBM.

155 The change in wording from that originally proposed relates to changing from a solely pseudo static approach for the seismic load case to include a displacement approach.

156 Geo condition B: A Site Specific Probabilistic Seismic Hazard Assessment (SSPSHA) must be undertaken as part of Detailed Design of the landfill to ensure seismic risks are addressed. The landfill performance under seismic load must be consistent with an IL4 structure as defined in Table 3.2 NZS 1170.0.2004 Structural Design Actions - Part 0 General Principles ([… facilities containing hazardous materials capable of causing hazardous conditions that extend beyond the property boundaries.]) and Table 3.3 for appropriate annual probability of exceedances based on design life. The detailed design and construction of the landfill, in particular for permanent and temporary slopes, must be modified as necessary to incorporate any changes in seismic design parameters identified by the SSPSHA.

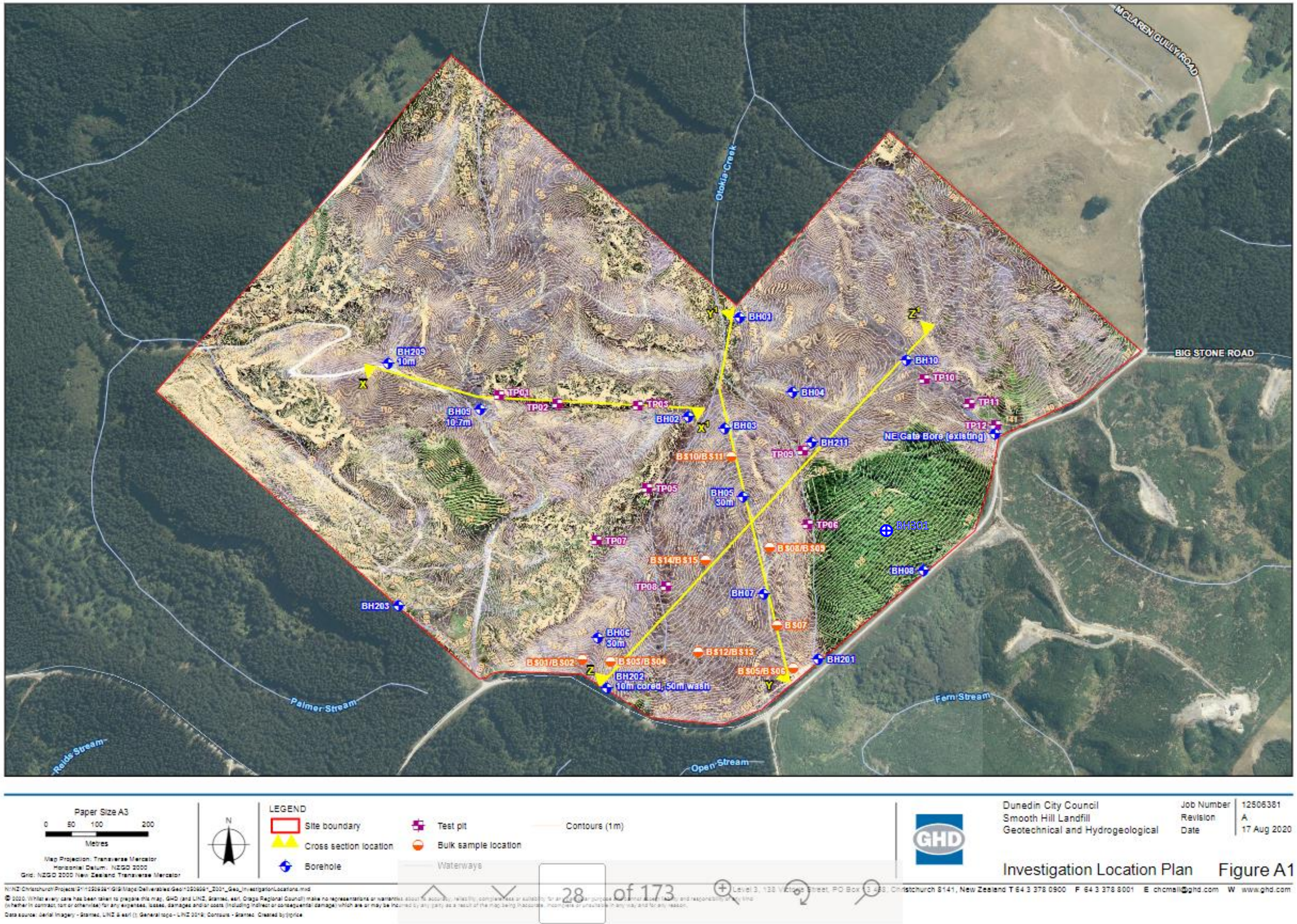
157 The wording differs from that suggested by T&T to put the emphasis on the SSPSHA and not on NZS1170.5:2004.

158 Geo condition C: Additional geotechnical investigations will be carried out as necessary as part of Detailed Design to generate a robust site-encompassing geotechnical ground model. The performance of the in-situ Henley Breccia is critical to the cut slope stability; further investigation shall include verification of the dip, and dip direction, of the Henley Breccia and strength assessment of the contacts between units. The location of investigation points will be determined during the initial stages of the detailed design process where specific confirmation is required.

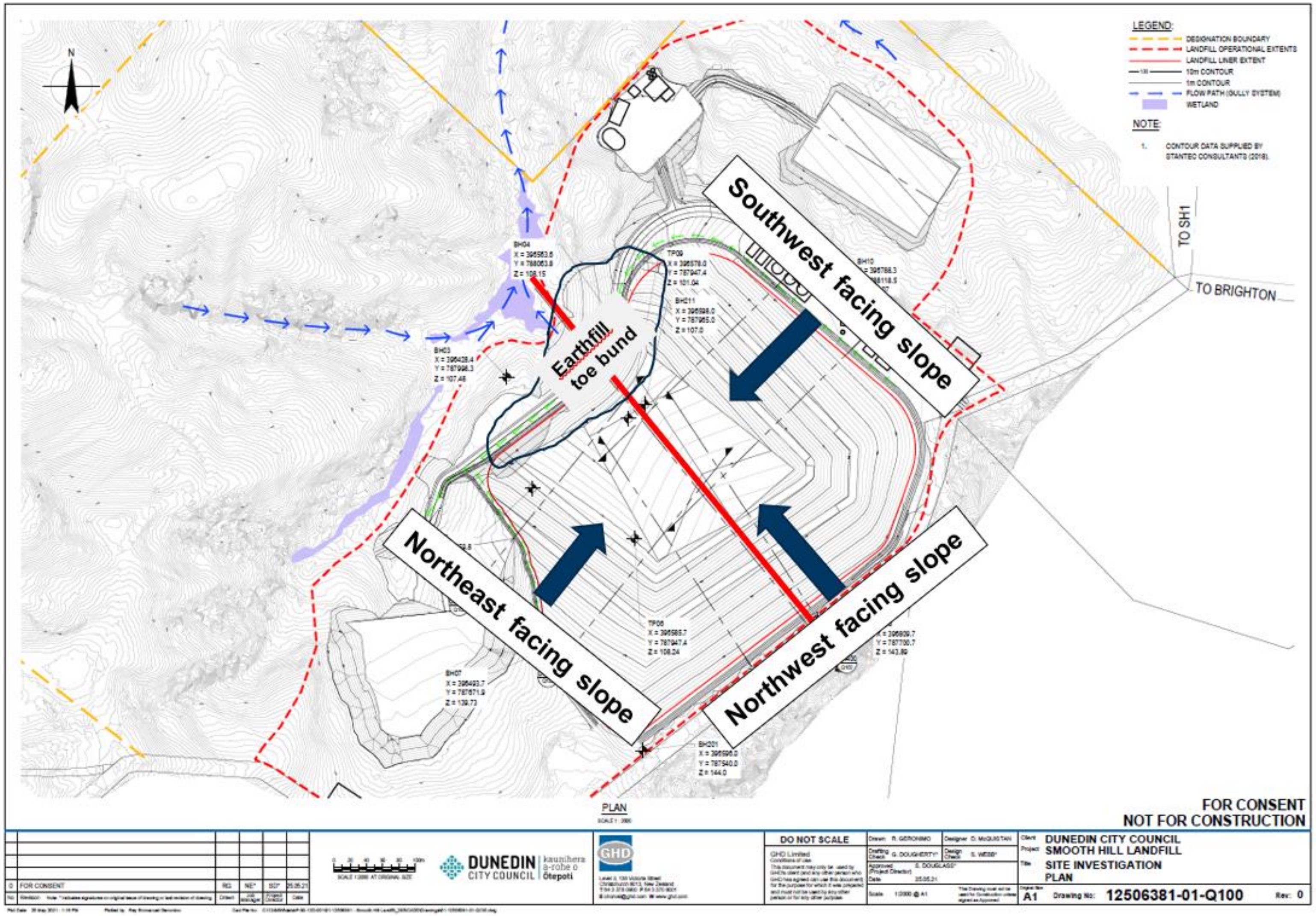
Samantha Webb

29 April 2022

Attachment A

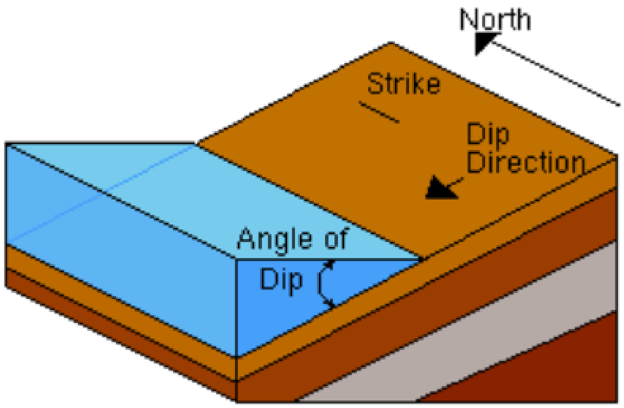


Attachment B

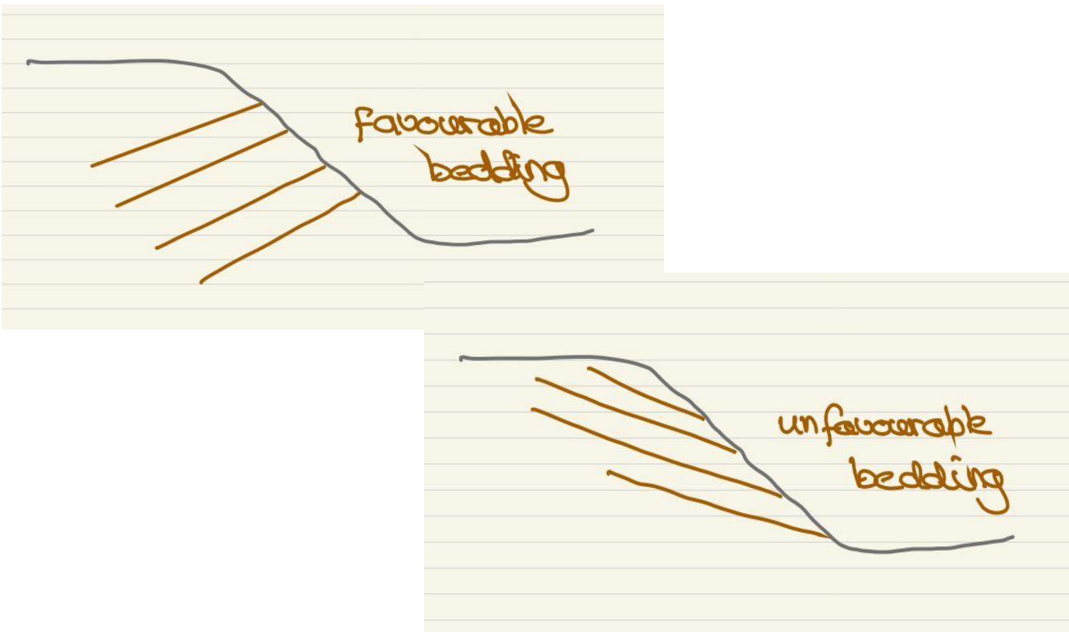


Attachment C

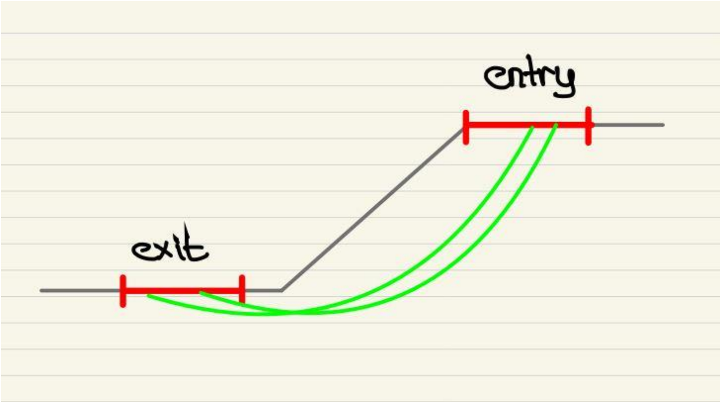
Dip and dip direction

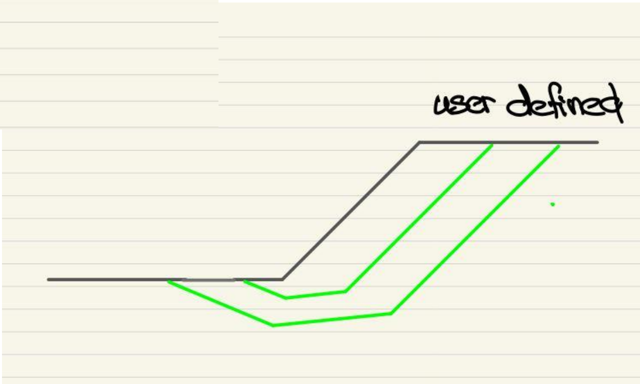


Favourable and unfavourable bedding



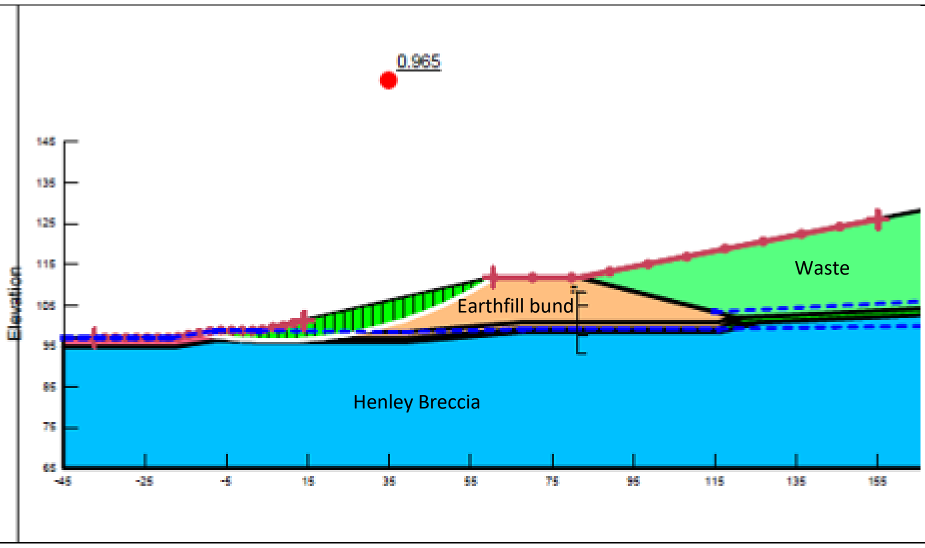
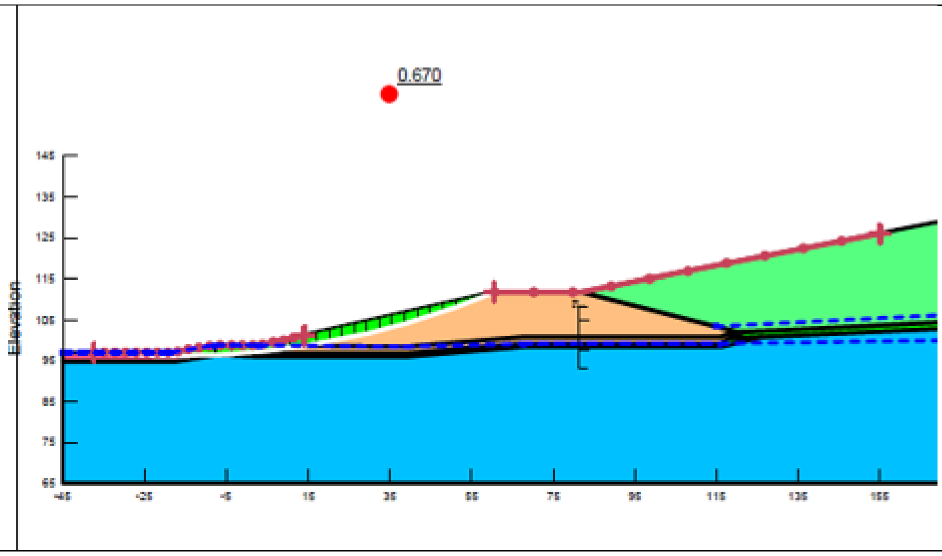
Entry-exit and User defined





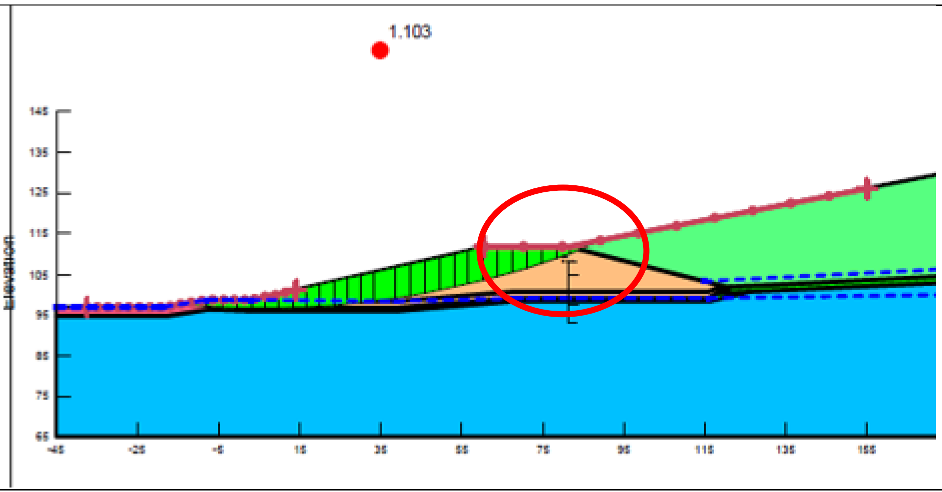
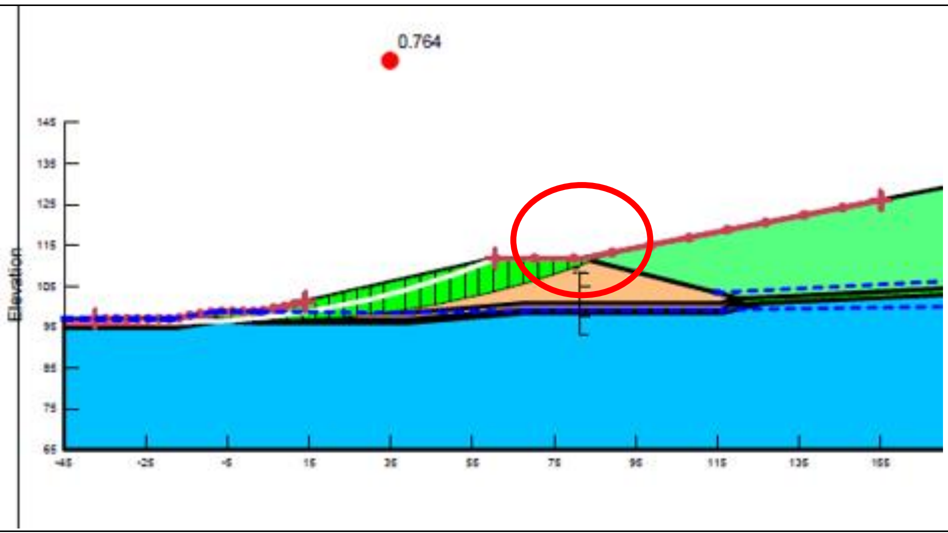
Attachment C.1

Entry/Exit mode – internal stability of the earthfill bund – lowest factor of safety

Horizontal Seismic Coef:	Model	FoS
0.31g		0.965
0.5g		0.670

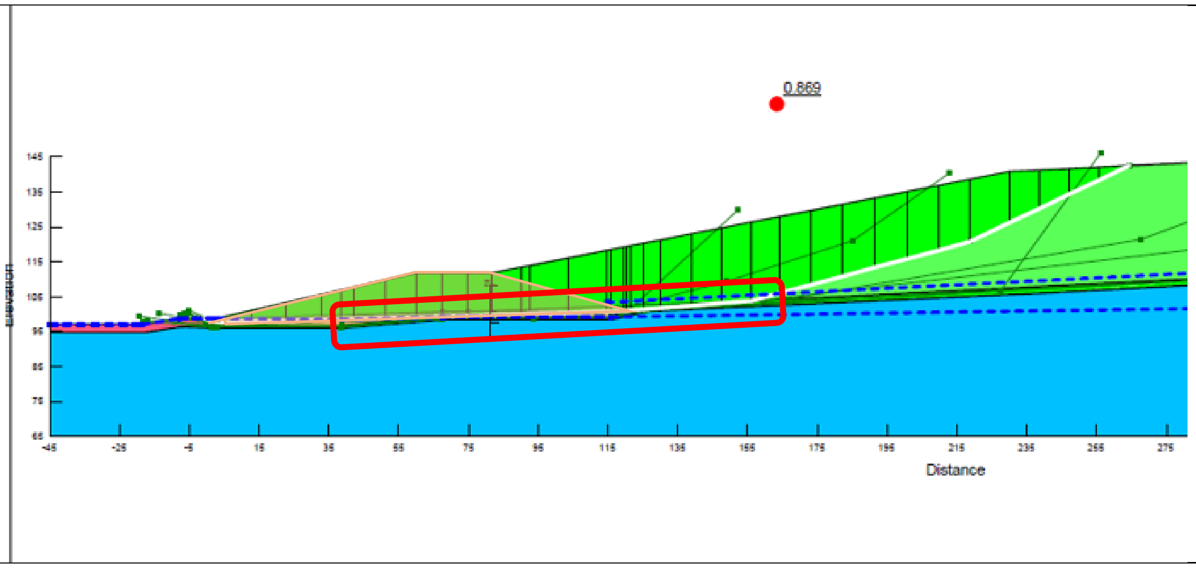
Attachment C.2

Entry/Exit mode – loss of confinement

Horizontal Seismic Coef:	Model	FoS
0.31g		1.103
0.5g		0.764

Attachment C3

User defined – Global stability

Horizontal Seismic Coef:	Model	FoS
0.5g		0.869