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Findings from the design and construction monitoring of an earthworks treatment at Flagstaff Hill, Dunedin

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ABSTRACT

Flagstaff Hill, also known as Observation Point, is a large hill situated in Port Chalmers, Dunedin. The hill is made up of Dunedin volcanic rock, characterised by Port Chalmers Breccia in the upper section and Dolerite in the lower section. The hill has been subject to several slip events since 1999, resulting in instability and rockfall towards the footpath and the road immediately below.

An earthworks solution has been designed and constructed in stages to stabilise the hill through excavations to flatten the steep slopes and remove areas of known instability. The solution has been adopted based on the geological investigations and mapping of the rock face and involves creating benches within Breccia in the upper section of the slope, followed by a uniform slope angle of approximately 45 degrees within Dolerite in the lower section. Construction of the second stage of the project was undertaken in 2019 and involved excavation of around 40,000 m^3 of slope materials within a residential setting over an approximately 4-month period.

This paper presents a summary of the site history and discusses the design approach adopted to stabilise the hill. The paper also discusses some of the findings and lessons learnt from the design as well as the construction of the most recent stage of the project.

1 INTRODUCTION

Flagstaff Hill, also known as Observation Point, is a large hill situated in Port Chalmers, Dunedin. The hill is in the order of 50m high, with slope angles ranging between typically 50 to 70 degrees. The hill is made up of Dunedin volcanic rocks, characterised by Port Chalmers Breccia in the upper section and Dolerite in the lower section.

The site has been subject to ongoing instability, particularly since major excavation in 1996. The instability has resulted in rockfall towards the footpath and the road immediately below. An earthwork solution has been designed and constructed in various stages between 1999 and 2019 to improve the kinematic stability of the hill by removing the over-steepened sections of the slope.

This paper highlights the site history and the remedial design approach adopted and discusses the key lessons learnt during the 2019 construction of the latest stage of the project.

2 SITE DESCRIPTION

2.1 Location

The Flagstaff Hill site (the 'site') is situated in the suburb of Port Chalmers, approximately 14.0 km to the north-east of Central Dunedin. The approximate location of the site in relation to Dunedin CBD is presented on Figure 1 below.

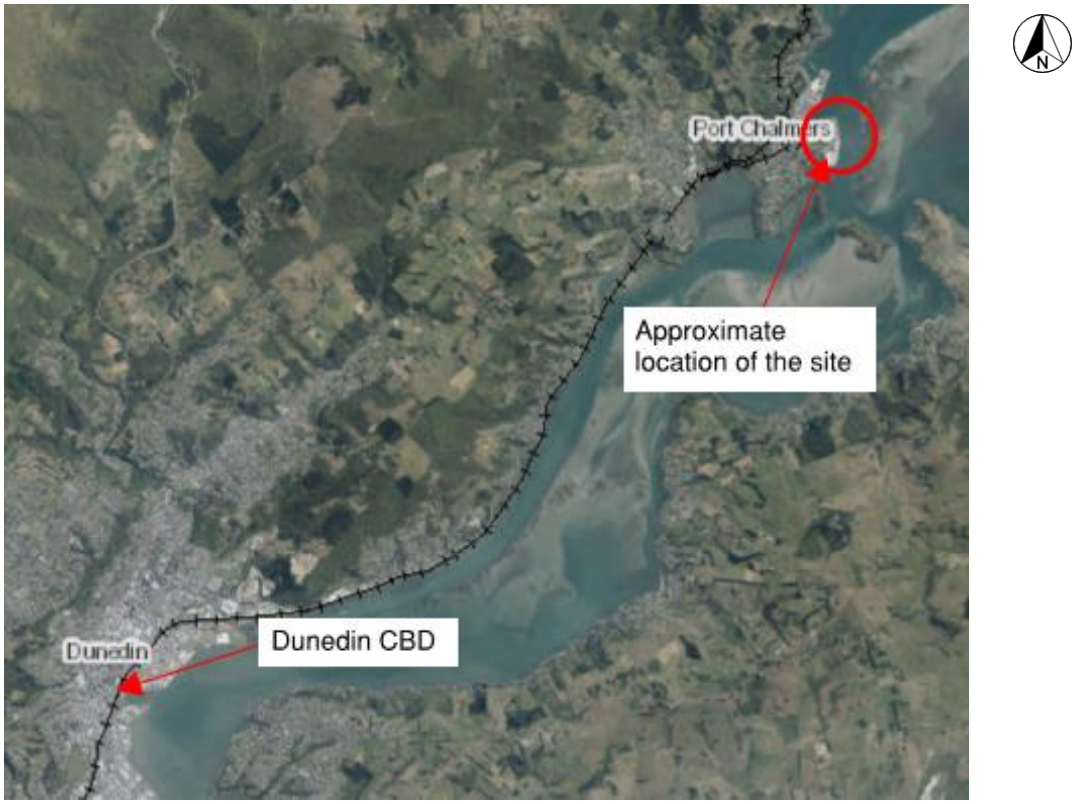

Figure 1: Approximate site location relative to Dunedin CBD (image courtesy of DCC's Web Map)

2.2 History

A major part of the Port Otago development works in the mid 1990's was the reclamation of the 'Back Beach' area which currently accommodates Port Otago's logging and dairy warehousing activities. This reclamation was primarily formed from the material sourced from Flagstaff Hill.

Practical completion of the reclamation work was achieved in December 1995, however ongoing and minor instability of the slope experienced from 1996 through to 1999 led to concern being established on the long-term stability of the hill. In September 1999, a significant slip on the excavated slope occurred following a heavy rainfall event. Run out from this slip crossed the Beach Street, with rock debris reaching the logging facility as shown on Figure 2 below.

Figure 2: Photographs of the site following 1999 event

In 2000, Opus International Consultants Limited (now WSP) undertook the geotechnical and geological investigations and developed a long-term solution to improve the stability of the cut batters that had been created during the reclamation borrow operation. The physical remedial work was staged to initially address mitigation of the most unstable western portion of the slope (referred to as Stage 1 works). A factor in this staging decision was the limited availability of spoil disposal sites at the time and their insufficient capacity to accommodate the total volume expected to be generated by the full remediation programme. The design included construction of benches within the upper section of the slope to allow for practical construction access from above together with the establishment of landscaping vegetation in this area.

With commitment to further spoil disposal arrangements and resource consenting in 2016, construction of Stage 2 of the project was undertaken by Fulton Hogan Ltd over an approximately 4-month period in 2019. This latest phase of work included extending the remedial earthworks solutions to an approximately 200 m long section of the hill immediately to the east of the Stage 1 works. The extended period of time between completion of stage 1 and commitment to stage 2 works enabled a review of the performance of the Stage 1 works relative to design expectations to be undertaken to confirm the suitability of the earlier design solution for the eastern flank.

3 GEOLOGICAL SETTING

The Published Geology Map by GNS Science indicates the site is underlain by Dunedin Volcanic Group trachytic rocks (Unit Mb). This unit predominantly consists of trachytic flows, pyroclastic deposits and subvolcanic intrusives of Miocene age. An extract from the Web Map is presented on Figure 3 below.

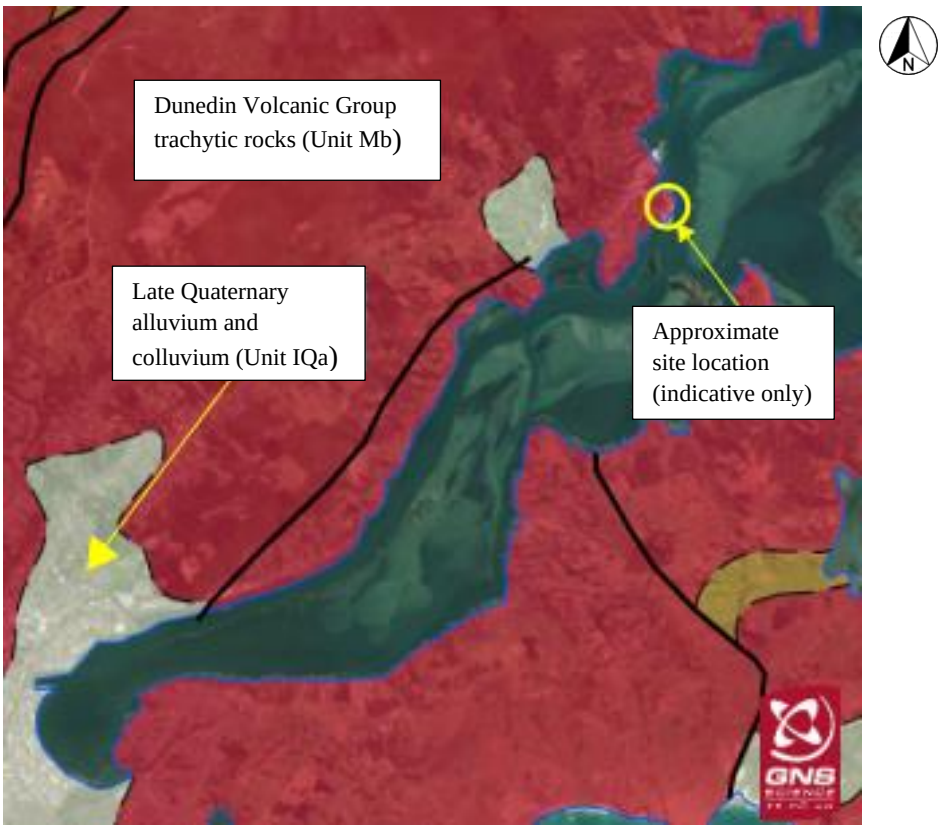

Figure 3: Site geology (image courtesy of the GNS Web Map)

The hill consists of Dolerite overlain by Port Chalmers Breccia. The sequence is associated with the first main eruptive sequence of the Dunedin Volcanic Field.

The Dolerite is likely the eroded remnant of a former plug intruded into the Port Chalmers vent. This is a hard medium to fine grained rock that contains many discontinuities. The Dolerite appears to be consistent across the site.

The Port Chalmers Breccia is carbonate cemented Breccia that consists of unsorted clasts of volcanics and minor schist, and acts as a vent filling material. The high energy environment in which it was deposited means there may be some differentiation between exposures. The Breccia is massive at this site; however, this differs in other areas around Port Chalmers.

4 PREVIOUS ASSESSMENT

As part of Stage 1 remedial design in 2000, detailed geological assessments were undertaken to investigate the stability of the slopes and develop a suitable stabilisation solution. As stated above, these earlier assessments and their associated failure mechanisms were reviewed and calibrated against the actual site performance experienced on both remediated and un-remediated sections to ensure the Stage 1 design solution was optimal for Stage 2 construction.

4.1 Geological

Geological mapping was undertaken by an experienced engineering geologist to facilitate assessment of the stability of the rock slopes. Scan lines were conducted using abseil methods supplemented with site observations of rock mass features such as discontinuity and persistence. Rock strength data was based on site descriptions supplemented with historical testing.

Given the hill forms a pronounced headland, it was possible to view the geology from slope exposures in various orientations to support development of the geological model.

The geological model highlighted that instability of a significant scale was driven by the condition of the Dolerite in the lower section of the slope. Shallow failures may be expected within the overlying breccia, however, the presence of a stable rock-mass below limited the size and depth of potential failures.

The Dolerite rock mass exhibited two persistent discontinues (J1 and J2) as well two less persistent but regular discontinuities (J3 and J4). As such, instability was defined by the structural control and assessed by kinematic stability. The assessment focused on more persistent joints as well as a mapped dike, as these had the potential to result in larger scale failures. A summary of the discontinuities and structural trends is presented on Figure 4.

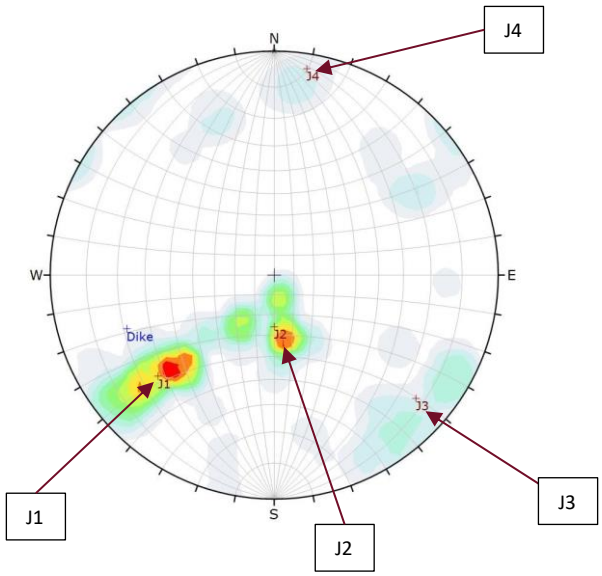

Figure 4: Stereonet showing the joint sets

4.2 Kinematic Stability Assessment

Detailed kinematic assessments were undertaken as part of Stage 1 and 2 remediation design to determine the stability of the slope profiles. These assessments considered the stability of both pre-remediated and post-remediated slopes. The headland topography led to the assessment of multiple slope orientations related to different portions of the headland.

Summary plots showing potential wedge failures for Stage 2 is presented on Figure 5. This shows both the north-eastern and south-eastern sections of the slope face with the old and newly cut slope face.

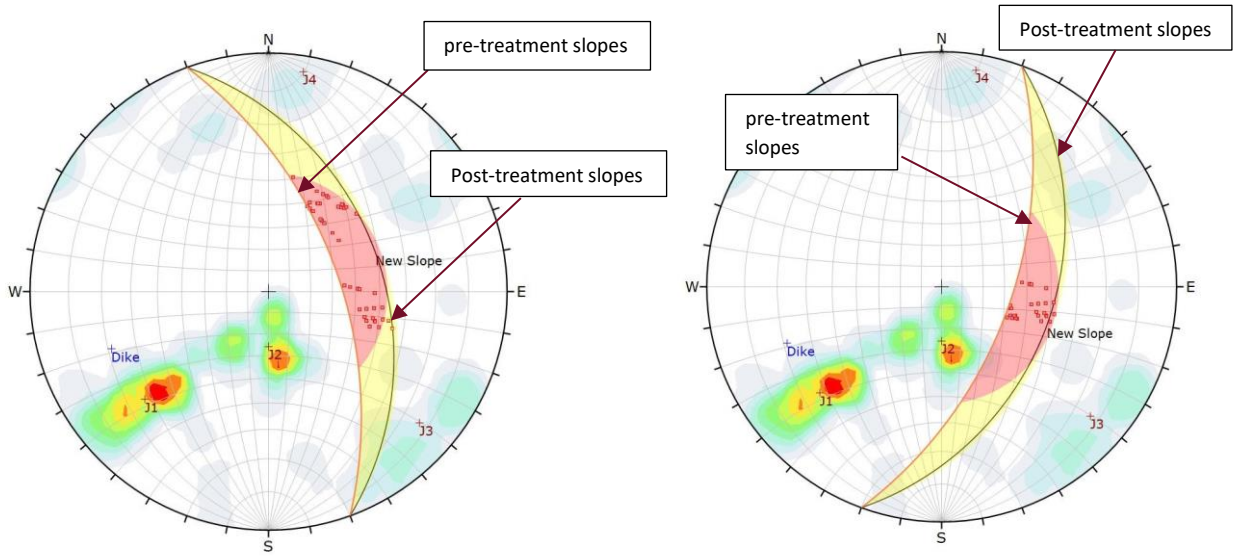

Figure 4: Potential wedge failures in Stage 2, the north-eastern (left) and south-eastern sections (right) of the slope face

The main findings from the kinematic assessments are as follows:

- The overall stability of the slope is governed by the lower Dolerite section. In order for large failures to occur, the failure plain must extend into the Dolerite. Stabilisation of the Dolerite section would therefore limit the instability within Breccia to shallow-seated failures.
- Due to the rock mass strength and presence of several discontinuities in the Dolerite, this material is likely to fail as a structurally controlled rock mass.
- The assessment showed that overall wedge failure limits the slope design. This is due to the combination of possible wedges given the slope orientation. As the slope transitions into north eastern facing slopes, wedge failure starts to become more prominent. There were a number of potential wedges and this limited the stable angle that these slopes can be cut. Toppling by comparison poses no problem along this section. The slopes are not steep enough to generate toppling failures.
- Although wedge failures tend to be the limiting failure mode, some potential planar failures may also be expected. These are likely to occur in the middle of the slope and limit the slope excavation angles.

The new proposed slopes were at 40 to 50 degrees and removed the potential failure mechanisms for large scale failures.

5 REMEDIAL SOLUTION

Based on the findings from the assessments, a practical solution was developed to concentrate on the global stability of the Dolerite section. Shallow and surficial instability within the Breccia was not expected to create large scale slips affecting the road below the site, and was therefore able to be addressed with local treatment methods.

The proposed earthworks solution included the following:

- Excavation of Dolerite in the lower section to form slope angles varying between 40 degrees to 50 degrees along slope to improve the kinematic stability and reduce the likelihood of wedge failure.

The variation in the slope angles was to accommodate different potential failure mechanisms that varied with the changes in the slope orientation transitioning from south-east to north-east.

- Formation of approximately 5m wide benches at approximately 5m height intervals within the Breccia in the upper section of the slope to contain any localised shallow-seated instability and fretting of this material. The adoption of benches in the upper slope also facilitated revegetation objectives
- Installation of an appropriate rockfall catch fence at the base of the slope to manage isolated small-scale failures.

An indicative sketch of the proposed remedial measures is shown on Figure 6 below.

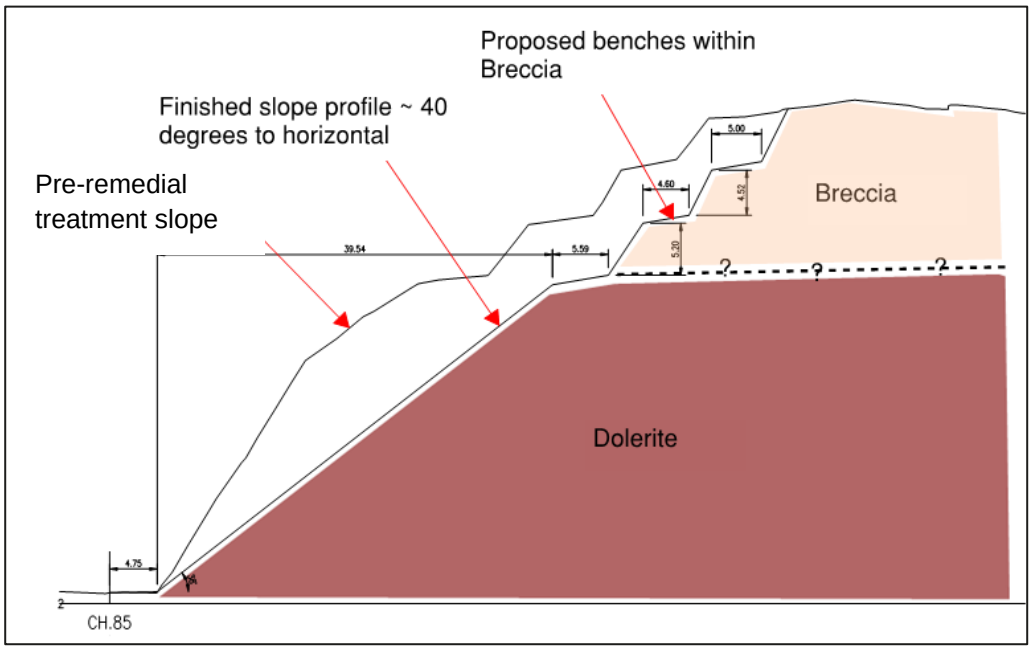

Figure 5: Indicative sketch of the proposed design solution

6 STAGE 2 DESIGN CONFIRMATION

In early 2019, WSP undertook the site inspections to review the performance of the Stage 1 design and the previously identified kinematic mechanisms within the un-remediated eastern slope to confirm the suitability of the original design for implementation in the Stage 2 works. The inspections revealed generally satisfactory performance of the Stage 1 works, with no signs of major instability of the hillside. Several rockfall debris accumulations were observed at localised zones the base of the slope and behind the catch fence, however the frequency and magnitude of these events were not able to be established. Wedge mechanisms in the eastern portion of the slope had not progressed beyond superficial fretting in recent years, confirming the earlier prioritisation decision.

A Safety in Design (SiD) register was developed prior to construction to address potential hazards and develop appropriate control measures during construction and maintenance. The main identified hazards included slope instability during construction, incidents with traffic (both vehicles and pedestrians) along Beach Street and sediment discharge.

7 STAGE 2 CONSTRUCTION

7.1 Methodology

Construction of Stage 2 of the project was undertaken by Fulton Hogan over an approximately 4-month period in 2019.

The methodology for bulk excavation included the use of excavators to form a track to the top of the excavation area. The site was then cleared of vegetation to allow for drone survey to confirm excavation quantities. Two excavators worked in tandem to bring the material from the top to the bottom. Once the stockpile of fill at the bottom of the slope was of sufficient size, one excavator moved to the opposite end of the face to continue excavation and the second excavator moved to the stockpile area.

The lower excavator was used to load materials into the truck and trailer units to be carted away. A safe working zone was delineated for the loading excavator and trucks. This was managed with spotters providing traffic management control and calling trucks in for loading when there was safe space.

A staged approach was developed to minimise the risk of instability affecting the construction workers. This meant that the excavation work was undertaken on a separate section of the site to where the loading of the trucks was undertaken.

Upon completion, the slope face was hydroseeded to minimise the risk of fretting of the slope face and sediment generation.

A photograph of site during construction is presented as Figure 7.

Figure 7: View of the site during construction

7.2 Observations

Geotechnical observations were made during the construction phase, including regular drone photography. The condition and strength of rock was generally consistent with the geological mapping findings during stage 1 and design expectations.

A photograph of the site during construction is presented on Figure 8.

Figure 8: View of the site and logging facility during construction

Following construction of the earthworks measure, a non-rated chain link mesh rockfall barrier was constructed at the base of the slope. A view of the site following construction is shown on Figure 9.

Figure 9: View of the finished site including the catch fence

8 CONCLUSIONS

The Flagstaff Hill has experienced occasional instability since 1996 following extensive reprofiling and over-steepening A practical remedial earthworks solution has been designed and constructed in stages to improve the kinematic stability of the hill. The design solution has allowed for the stabilisation of the hill by re-profiling to safe slope angles within Dolerite and creating benches to contain potential minor shallow-seated failures within the overlying Breccia and to facilitate landscaping and revegetation objectives. This has resulted in a considerable improvement in the stability of the hill and substantially reduced residual risk levels to the public including road and footpath users below.

Several factors have been instrumental in developing a practical design solution as well as the successful construction of Stage 2 works, as summarised below.

- A geotechnical assessment was primarily based on mapping of the exposed slope faces, minimising the need for expensive intrusive investigations, whilst providing a sufficient level of geological understanding. The assessment allowed for a pragmatic earthworks stabilisation measure to be developed to focus on improving the stability of the Dolerite section. The observations of rock condition and behaviour during construction were generally consistent with the investigation findings.
- The design solution aimed to remove the source of deep-seated instabilities by creating safer slope angles. This allowed a cost-effective non-rated rockfall fence to be adopted at the base of the slope to capture any fretting and shallow instability in the future. This highlights the importance of exploring and evaluating both active and passive stabilisation methods at early stages of the project.
- Finally, the site walkovers and review of site performance were key in ensuring the Stage 1 design solution was appropriate for the Stage 2 works. In the case of this project, the time gap between Stage 1 and Stage 2 works allowed the designers to review the effectiveness of Stage 1 remediation works and confirm the suitability of earlier proposals for stage 2 remediation in the lower risk zone.

9 ACKNOWLEDGEMENT

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