

Anzac Cliffs –Geotechnical aspects of cliff stabilisation works

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ABSTRACT

The Anzac Cliffs project involves two main elements: reshaping of the steep cliff, and realignment of the Manawatu River away from the cliff. The realignment of the river below Anzac Cliff was the last major development of the Lower Manawatu Scheme, City Reach Project. The project included the realignment of a 460m section of the Manawatu River opposite Anzac Park. The reshaping works were undertaken to stabilise the 50m high Anzac Cliffs adjacent to the river. The remedial works addressed the ongoing erosion of the unstable cliff by the Manawatu River.

Hazards associated with the project include direct river erosion undermining the lower cliff, and ongoing instability of cliff-face materials. The construction of foundations for a fill buttress within the active river channel and below the unstable cliff provided significant challenges, particularly due to the presence of liquefiable materials in its formation.

This paper provides a review of hazards associated with the project; sets out geotechnical investigations and analysis undertaken; and describes the development of the detailed design for the cliff stabilisation works. A review of construction and monitoring techniques for both the foundation treatment and compaction of granular and earth fill materials is presented, together with an outline of the management of numerous considerable technical and safety risks.

1 INTRODUCTION

The proposed Anzac Cliff residential subdivision is sited above the southern true left bank of the Manawatu River, some 3.5km south-east of Palmerston North City Centre (Figure 1). The north-eastern portion of the subdivision was bound by an approximately 50m high cliff that was subject to large-scale ongoing river erosion. The rapid erosion of this section of the cliff became a concern to various stakeholders with regard to the additional sediment load it added to the river, and the potential adverse effects that its altered course would have on the existing river protection works on the opposite northern bank. In April 2006 works to realign this section of the river back to its 1992 course were incorporated into the City Reach Project, a major works programme to upgrade the lower Manawatu River Scheme that protects Palmerston North from flooding.

Due to the unstable nature of the land along the site's north-eastern boundary, under the Palmerston North City District Plan, any building development within this portion of the proposed subdivision was a prohibited activity. It was identified that the proposed realignment of the river allowed for engineering works to be undertaken that would stabilise the land, making more of the site suitable for residential development. This led to a Plan Change that became operative in November 2011.

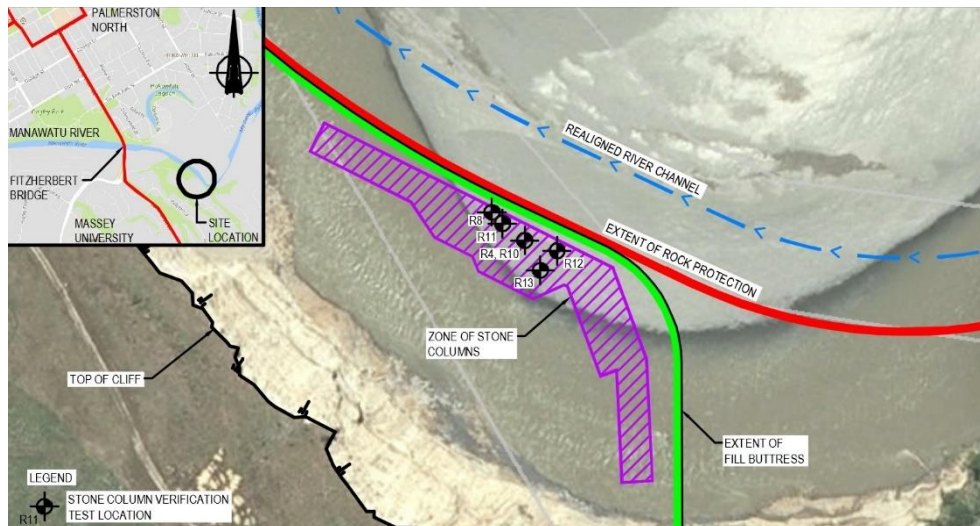


Figure 1: Location of machine holes for stone column verification testing, extent of stone columns, erosion protection and stabilisation works (additional borehole and test pit locations omitted for clarity)

The involvement of Riley Consultants Ltd (RILEY) with the project began in 2007 with the provision of a feasibility assessment of stabilisation works for the subdivision for HFH Properties Ltd (previously PBM Landco Ltd), and later with the development of a detailed design and definition of stable areas suitable for residential development, with Kevin O'Connor & Associates Ltd carrying out civil engineering, survey, and planning aspects. RILEY provided ongoing geotechnical consultancy input during construction, including monitoring of buttress filling.

The project consisted of the realignment of the river channel and construction of rock protection for the toe of the slope (undertaken by Horizons Regional Council (Horizons)). The buttressing and stabilisation of the cliff above the protection works was undertaken by HFH Properties Ltd. Stabilisation works included lowering the cliff crest by some 11m and construction of an approximately 38m high fill buttress along the north-eastern edge of the site. Foundation improvement works for the buttress fill included the installation of extensive stone columns and heavy compaction of the overlying ground. Earthworks consisted of cut to fill of approximately 300,000m³ over an area of 4.6ha. Fill material for the buttress was obtained from the excavation necessary to create the realigned river channel, and excavation of the top of the cliff.

2 INVESTIGATION

The initial feasibility study project undertaken by RILEY in 2007 was based on our knowledge of the area, visual assessment of the site and adjacent areas, and a review of available nearby subsurface information. The site had the advantage that the cliff materials were well-exposed over the majority of the cliff face (Figure 2). Subsurface investigation works at the base of the cliff were limited by access constraints to the gravel banks on the opposite side of the river to the cliff. In January 2008 two machine holes were drilled on the gravel banks adjacent to the true right northern side of the river, with a further two holes drilled along the gravel banks in August 2012 (Figure 2) in order to identify any constraints with respect to foundations of the proposed large fill buttress supporting the reshaped cliff.

Additional holes, drilled in August 2014, formed the basis for a verification trial for the stone columns prior to their construction. A series of test pits were also excavated in the gravel beach to assess the grading of material, along with four landward above the cliff to assess the suitability of the proposed fill materials. Samples recovered from standard penetration tests (SPT) split

spoon samples, and bulk samples taken from test pits, were logged, assessed for their intended purpose, and tested in the laboratory. Further machine holes and SPT tests were carried out to assess the effectiveness of foundation improvement works.



Figure 2: Composite of photographs taken from the opposite true right northern river bank during 2008 feasibility study

3 GEOLOGICAL MODEL

The Anzac Cliffs, along with the adjacent Waicola and Strand Cliffs, have formed as a result of down cutting of the Manawatu River into a coastal terrace formed during the previous interglacial period around 100,000 years ago (Lee & Begg 2002). These sand, silt, and interbedded sandy gravel horizons were visible in the cliff face and capped by loess. At the base of the cliff an erosion-resistant rock platform, interpreted to be of Pliocene Age, was exposed. The sandstone horizon is very weak in engineering terms, but is considered more erosion-resistant than the cliff materials above (Figure 3).

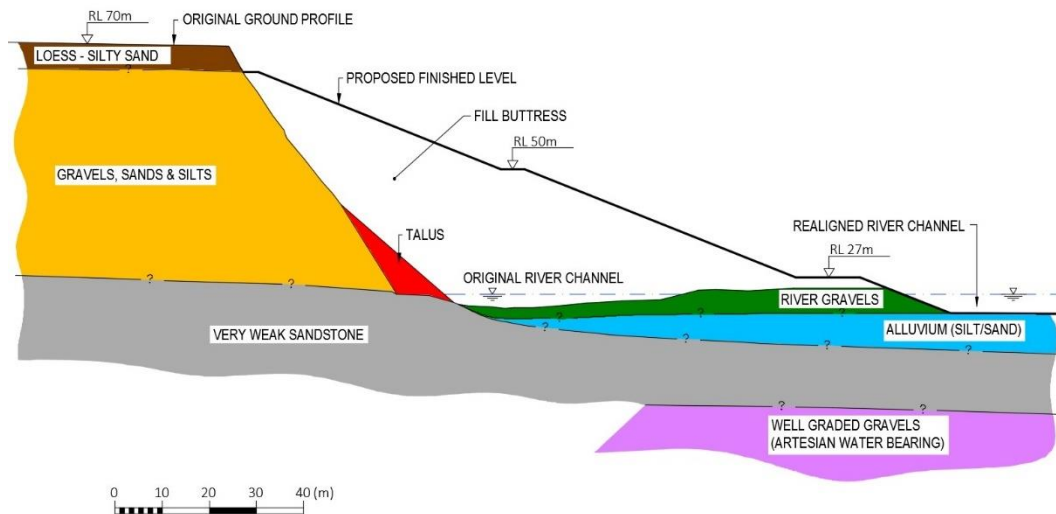


Figure 3: Simplified geological cross section showing original and finished ground profiles

4 BUTTRESS FILL DESIGN

4.1 Buttress fill foundation design

An assessment of the liquefaction risk to the project was carried out for the cliff materials, engineered fill, and foundations. Neither the in-situ cliff materials nor the engineered fill buttress were considered to be significant hazards. The cliff itself has a fundamentally low groundwater level, although minor seeps indicating perched water tables have been observed. Groundwater levels assumed for slope stability analysis purposes are shown in Figure 4. Buttress fill consists

of material compacted to engineered standards, with groundwater controlled by subsurface drainage. The more potentially hazardous location was assessed to be within the foundations of the fill buttress. Loss of support here due to widespread liquefaction could lead to large-scale lateral spreading, gross settlement and, in extreme cases, excess lateral movement of the entire cliff. Slope stability analysis was utilised for design of the fill slopes; this included an assessment and identification of where stability was assessed to be marginal below the buttress toe (Figure 4).

Modelling of the required ground improvement work was carried out utilising selected parameters and the slope computer program Slide, to meet the project criteria of no more than 500mm deformation in a 0.43g maximum design earthquake (MDE). The residual shear strengths for potentially liquefiable layers were selected based on correlations with SPT testing (Idriss and Boulanger 2008). Analysis indicating an 8m to 18m width of stone column treatment for different fill buttress heights could achieve the requirements of a static post liquefaction FoS of 1.2, from a FoS of less than 1.0 without stone columns. The finalised geometry of the stone column layout is based on these calculations, with extrapolation based on judgement. The design utilised a 15% replacement value of imported crushed gravels for the columns (Figures 1 and 4).

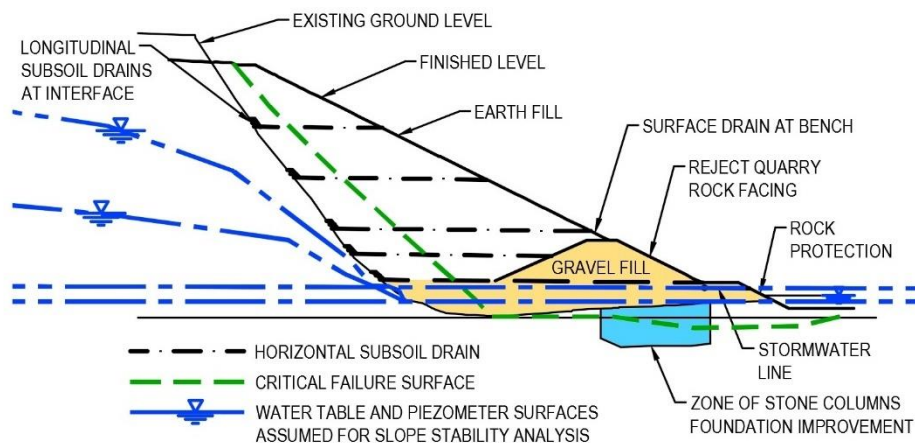


Figure 4: Cross section showing zone of stone columns, subsurface drainage, groundwater levels assumed for analysis, and critical failure surface from slope stability analysis

4.2 Buttress fill slope design

The buttress fill is a composite of gravel sourced from the river and finer largely cohesive earthfill material excavated from the cliff. The river gravels and terrace deposits have significantly different engineering properties. The base of the buttress within the area of potential flooding (below RL 27.0m) utilised compacted river-run gravel. The bulk of the fill buttress consists of silts, sands, and clayey soils.

It is an assumption of the design that in extreme rainfall and/or flooding of the toe by the Manawatu River, complete saturation over the full height of the buttress fill is an unrealistic scenario. The buttress fill may not be free-draining, and thus, a comprehensive system of subsurface drainage was incorporated into the design. A series of horizontal subsurface drains were constructed at four levels within the buttress (Figure 4). These form an important part of the design, providing the control of seepage and water pressures within the less permeable parts of the fill. They consist of horizontal trenched blanket drains along the interface with the in-situ cliff materials. These outlet by a number of horizontal strip drains to the face of the fill buttress. Drains were constructed using heavy-walled perforated polyethylene pipe surrounded by F/2 filter material, assessed to be filter compatible with the finest soils (Fell et al 2015).

Slope stability analysis was carried out using the computer program Slide and selected regression models to estimate permanent displacements in the MDE (Jibson, 2007). These displacements were calculated at the proposed building setback line for the subdivision. Slope stability analyses was carried out using parameters assessed to be attainable for the fill material, and parameters derived from back analysis of previous failures for the cliff materials.

5 RIVER REALIGNMENT WORKS

Works by Horizons to return the river to its 1992 alignment began towards the end of 2014 with approximately 250,000m³ of rock transported across the river to form the rock-lined gravel bank to protect the base of the slope from ongoing river erosion. This work was undertaken concurrently with the installation of the second production run of stone columns requiring a high degree of co-ordination between works within the shared area.

6 BUTTRESS CONSTRUCTION

6.1 Foundation improvement works

Construction of the stone columns was undertaken by Brian Perry Civil. It involved managing the challenges and mitigating the safety risks associated with working within an active river environment with constantly changing river levels, including flood events. The work, as with the majority of the construction, was restricted to favourable conditions during summer months.

A pre-trial for the stone columns was carried out over the period 13 May to 19 May 2014. A trial was necessary as the feasibility of the entire project depended on foundation improvement being demonstrated. Stone columns were installed utilising a wet top-feed method. Testing comprised split spoon SPTs undertaken at near continuous 0.5m intervals of depth within the areas of interest, along with the geotechnical logging of recovered samples. The installation of the stone columns and test drilling proved challenging due to rising water levels and the extremely permeable nature of the river gravels. For the pre-trial and main production runs a level platform, approximately 800mm below original ground level, was excavated to just above groundwater level. The initial trial comprised 12 stone columns centred on a borehole, chosen because it had encountered the most potentially liquefiable soils. SPT tests undertaken following installation indicated significant strength improvements, although at shallow depths from 3m to 4m limited improvement was noted. The test run was followed by the installation of 157 production columns, with an alteration in methodology to provide more vibration at the upper zone of the columns. These were installed, generally to the expected depth or slightly deeper, with no major difficulties experienced, and only one column unable to reach target depth, either due to a boulder or, more likely, a log or tree stump.

Further validation testing consisting of boreholes with SPT testing was carried out following installation of the columns with very significant strength improvements were recorded (Figures 1 and 5). As expected, there was a variability in the strength improvements, which is considered to be related to variability in material type, with the best response noted in the clean sands and gravels, and lesser improvements in silty sands. Figure 5 shows the calculated FoS of 1 against liquefaction based on correlations between relative density and liquefaction trigger values (Boulanger & Idriss, 2014). Due to the nature of the materials and significant scatter for the SPT results, the liquefiable layer was characterised based on the 66 percentile (i.e. 66% of values are greater than the design value) (Idriss and Boulanger, 2008).

The second stage of stone column installation commenced in December 2014 following works to divert the river (Figure 6). This involved the installation of a 3,899m total length of stone columns, with individual columns varying between 6.3m and 7.3m depth. River levels were a constant ongoing threat to progress, with a flood on 10 December raising water level about 2m

above the site of the column installation. Verification of the stone columns was completed with seven boreholes drilled that indicated satisfactory increases in SPT N values had been achieved.

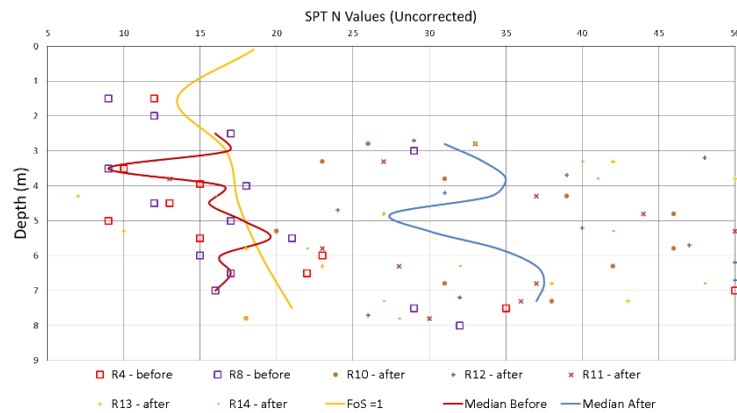


Figure 5: SPT N tests from boreholes measured before (plotted as squares), and after installation of stone columns installation (liquefaction trigger values (FoS=1), and median before and after SPT N lines shown)

6.2 Buttress fill slope construction

Earthworks for the buttress fill, undertaken by Goodmans Contractors Ltd, began in January 2015, and were completed in May 2017. This followed the completion of the realignment works by Horizons. Filling began with the placement of on-site river-run material above the rock buttress to RL 27.0m. Access to the fill area was initially limited to fording the river; later in the project access was created from the top of the cliff. The challenging constraints of the site provided significant hazards during construction. The contractor was required to develop construction methodologies to mitigate the risk of working beneath an unstable cliff, which included the removal of talus, excavation of the cliff, compaction of fill, installation of blanket and strip drains, and working in an active river bed.



Figure 6: Looking south towards stone column installation, (9 December 2014)

Figure 7: Looking north towards later filling operations (20 April 2016)

The design of the buttress fill assumed that any unsuitable materials within the now stranded river channel at the base of the cliff were excavated and the channel built-up with compacted river gravels. Although the high flow velocity of the river had prevented the deposition of fine-grained potentially liquefiable materials, compaction within the river channel posed specific challenges. Due to the very high permeability of both the natural river gravels and fill materials, the

groundwater level within the construction area reflected the current river level. The dewatering of sections of the channel excavation was problematic. These works were also being undertaken at the base of an unstable cliff. This meant that fill within the now stranded channel required a large thickness of loose granular material to be placed to a level above groundwater level, and compacted using heavy compaction equipment.

Above the foundation buttress, fill was sourced from material cut from the upper 11m of the cliff and terrace area, as well as the loose talus material at the cliff base. As the talus material was wet of the required moisture content for compaction, it was temporarily stockpiled on a high stand area, upstream of the buttress, until there was sufficient area to blend and condition this material. The material was then pushed from the upper terrace as part of the down-cutting of the cliff (Figure 7). Towards the upper portion of the buttress slope filling utilised materials with greater sand content, and care was taken by the contractor to stockpile finer-grained silt and clay material for the outside edge to help minimise surface erosion.

The variability of material encountered in the cliff meant that compaction test results needed to be closely monitored and testing criteria adjusted to reflect the materials. To ensure fill materials achieved the design criteria, ongoing standard compaction proctor tests to determine maximum dry density, were compared to materials tested in the field. Field tests consisted of Scala penetrometer, shear vane, and nuclear density meter (NDM) with oven-measured water contents. Testing of materials on the same site ranging from clay to sand highlighted the importance of selecting the correct targets for dry density, and field tests for different materials. The more sandy soils have typically shown a higher maximum dry density with a lower optimum water content, and the opposite for more clayey soils (Figure 8). Monitoring involved an ongoing review of field and laboratory test data and regular site visits, including documentation of fill material and ground conditions. Information and feedback shared between the laboratory, consultant, and contractor was required to ensure varying materials were conditioned to an acceptable water content.

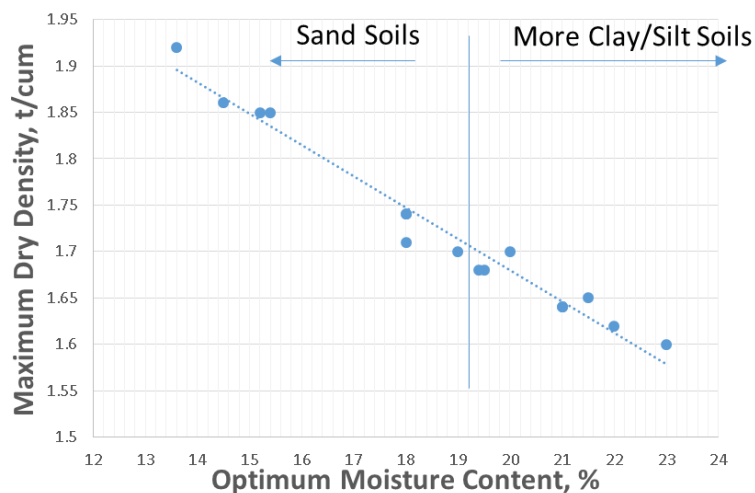


Figure 8: Range of maximum dry density vs optimum moisture content for buttress fill from laboratory proctor tests

Shallow-seated slumps were observed to occur on these steep cliff slopes where concentrations of surface water occurred. Surface collector drains were installed at the intermediate bench with a piped outlet to the cliff base (Figures 4 and 9). The reshaped slope is proposed to be vegetated to minimise surface water infiltration in the long term and also bind the surficial soils, reducing the risk of rill erosion on the face. To monitor the performance of the fill slope, a series of survey points were set up that are subject to ongoing monitoring for horizontal and vertical movement. To date the slope's overall performance is within expectations (Figure 9).



Figure 9: Photograph of completed realignment and cliff stabilisation works

7 CONCLUSIONS

The Anzac Cliffs project included detailed slope stability, liquefaction assessment, and modelling to facilitate the design of large-scale remedial works for the cliff, carried out within the active river channel of the Manawatu River. The works to realign the river, combined with the works to treat the buttress foundations and stabilise the slopes, have effectively stopped the erosion of the cliff, allowing the development of a proposed additional 36 residential lots.

Geotechnical assessment and modelling was used to identify the required scope of physical works to remediate the stability and liquefaction hazard, including defining a suitable building setback for the proposed residential subdivision. The initial model was developed further based on ongoing subsurface investigation work, testing, and the practical construction limitations encountered during construction.

The project demonstrates that large-scale works can be successfully undertaken in hazardous active environments provided ongoing monitoring, testing, and modelling manage the risk, and are combined to further develop the design as required.

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