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Geotechnical challenges during design of Aokautere Reservoir – Palmerston North

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ABSTRACT

Palmerston North City Council (PNCC) is currently constructing one of two planned new 2,500m^3 reservoirs and a new 1.2km long access road on a site at 291 Turitea Road, southeast of Palmerston North city. The reservoir site is located on a hill with slopes ranging from 15^0 to 30^0. The reservoir platform is to be created by cutting the hill top down by 7 m. The reservoir will be designed to be serviceable after a major earthquake event. The reservoir, and therefore its platform is designed as an Importance Level 4 (IL4) structure with special post-disaster function requirements. Site investigation and slope stability analysis was carried out for the reservoir platform to confirm the setback distance of the reservoir from the edge of the slope. The 1.2km access road alignment, which provides the alignment for the pumped rising main supplying the reservoirs, traverses steep valley sides with high sidling cuts. The challenges encountered during the design of the high cuts included restricted availability of information and limited scope for localised site investigation necessitating the extrapolation of surface geological mapping data. Conservative cut slope batters would have required large earthwork volumes at high cost. A risk based approach to the batter design was undertaken, accepting that steeper cut batter angles may result in failures along the access route, but that these failures could be managed for a lower overall cost. Lessons from the experience are drawn for application to similar situations in the future.

Keywords: Aokautere, reservoir, platform, slope stability, seismic design, analysis

1 INTRODUCTION

Palmerston North City Council (PNCC) is upgrading the water supply for the Summerhill, Aokautere and Linton areas. The building of two water storage reservoirs at a higher elevation than the City's current reservoirs is a project that was part of the Water Supply Development Plan, adopted in 1996. When completed, the new water supply will not only allow housing developments at higher elevations, but will improve the resilience of the City's water supply network following a major seismic event.

2 SITE LOCATION AND DESCRIPTION

The Aokautere area lies within the low-lying hills and raised terraces of the western side of the Tararua Range. This reservoir site is located on the gentler foothills of the Tararua range approximately 1 km northeast of Turitea Road. The access route to the reservoir site branches off from Turitea Road before travelling northeast up an existing farm track that flanks the southern side of the gully, and winds its way up to a saddle on a ridge. Most of the access route will be subject to potential slope instability. The reservoir site is located on the ridge covered in pasture. TheTproposal is to cut approximately 7.8m from the top of the ridge to create a platform for the reservoir. The northwest and southwest hillside slope of the reservoir platform is gentle, having slope gradient of no more than 13^0. The northeast and southern slopes are steeper, typically in the range of between 20^0 and 30^0. A critical section on the southeast slope is at 42^0.

3 DEVELOPMENT PLAN

The development plan for Stage One and Two of the Aokautere Reservoir project included the construction of an access road from Turitea Road to the reservoir platform completed by the end of May 2016. The total estimated project cost is $5.4m. Stage Three of the project includes the construction of Reservoir No.1, planned to be completed by June 2017. The second reservoir is planned for construction in approximately five years' time. Due to the hilly nature and steep slopes of the existing site significant earthworks were carried out to form the roads and the reservoir platform.

Refer to Figure 1 for the locations of the access road and the reservoir platform.

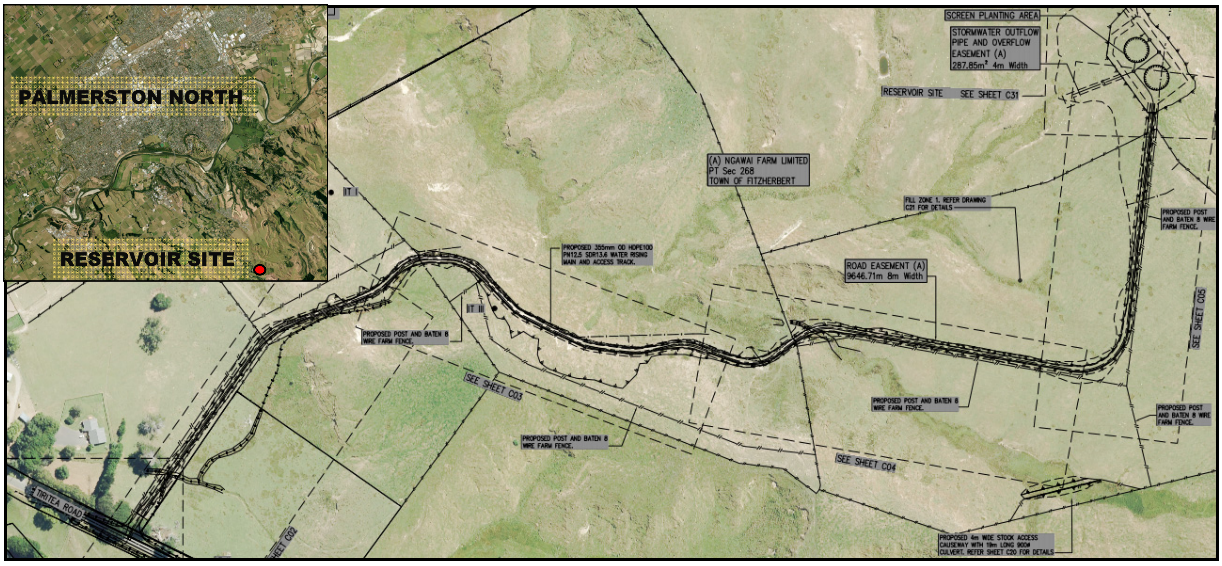

Figure 1. Site Layout Plan

4 GEOLOGIC SETTING

4.1 Geology of the Area

The geology of the area are indicated to be marine deposits of the Wanganui Series (Mid Pleistocene), which include well-weathered gravels, sands, and silts (DSIR, 1962). Wanganui Series deposits forms the highest marine terrace sequences of the Manawatu area and these unconformably overlie basement greywacke bedrock of the Tararua Range. IGNS (1994a) presented a more detailed geological map of the Palmerston North area that indicates the foothills near Turitea to be soft Plio-Pleistocene marine rock, deposited between 0.5 – 5 million years ago in shallow seas. Alluvial fan and colluvial debris flow deposits of Holocene Age, overlay the marine sediments on the foothills. A thin blanket of loess (up to 2m thick) also covers much of the area.

4.2 Seismicity

The Palmerston North region is one of the most earthquake prone in New Zealand. The active Wellington Fault lies approximately 9km southeast of the Turitea area. It presents the highest seismic hazard to the area, having a recurrence interval of between 500 and 770 years with a magnitude estimate of 7.6 ± 0.3 (Begg and Mazengarb, 1996).

4.2.1 Calculation of Peak Ground Acceleration

Based on the most conservative estimate of the SPT data (N value 30 – 50 and maximum depth of soil is 55 m) from boreholes, the site is consistent with a Class C (Shallow Soil Sites) classification. Therefore, the site is classified as Class C – Shallow Soil Site, in terms of the seismic design requirements of NZS1170.5:2004.

The client determined the proposed reservoir would be designated an Importance Level 4 (IL4) structure in accordance with NZS1170.0:2002 and will have a post-disaster function. The structure is to have a design working life of 50 years (for seismic considerations). Through application of Table 3.3 AS/NZS1170.0:2002, the earthquake annual probability of exceedances were established for the proposed reservoir. The design return periods (i.e. the inverse of the Annual Probability of Exceedance (APE)) are 25 and 500 years for SLS1 and SLS2 limit states, respectively.

NZS1170.5:2004 can be utilised to develop design magnitude-weighted PGA values (hereafter referred to as '(PGA)M=7.5') for use in the foundation design of the proposed reservoir. (PGA)M=7.5 values for the site are weighted to a magnitude 7.5 earthquake and are 0.13g and 0.51g for SLS1 and SLS2 limit states respectively. Earthquake actions for Importance Level 4 structure SLS2 limit state is avoidance of damage to structure to the extent that they can no longer remain operational after the SLS2 earthquake motions. The horizontal seismic coefficient K_h, was taken to be equal to peak ground acceleration calculated from the SLS2 earthquake motion, used in the slope stability analysis. A design FS of 1.0 was then targeted for the slope.

5 ASSESSMENT OF THE STABILITY OF RESERVOIR ACCESS TRACK

5.1 Geotechnical Investigations

A cut batter slope was proposed into the hillside slope to upgrade the farm track to permanent access track to allow for construction and maintenance of the reservoir. The existing natural slope of the hillside along the track route is between 25^0 and 55^0, and up to 14 m high. Observations on the site indicate the slope is only marginally stable. Erosion of silt and sandy silt was observed along the existing farm track. Observations also indicated mass movement at various locations. Geotechnical site investigations were carried out to assess the ground condition for slope stability analysis.

5.1.1 Field Work

Ten mechanically excavated trial pits were conducted at the site during June 2015. Of these, four trial pits were located on the proposed access track, and six trial pits were located on the top, face and bottom of the proposed track cut slope. Scala penetrometer tests were undertaken adjacent to the trial pits from ground level to a maximum depth of 3.0m or refusal. The number of blows for each 100mm penetration was recorded. Pilcon vane shear tests were also undertaken in the near surface soils during excavation of the trial pits.

5.1.2 Geotechnical Parameters

The soils encountered in the trial pits and parameters used in slope stability analysis are presented in Table 1, and have been derived from visual description of soils encountered, in-situ soil testing and published empirical correlations with in-situ test results

Table 1. Summary of Soil Properties

USCS Soil Classification	Description	Parameters used in SLOPE/W analysis		
		Unit Weight (kN/m^3)	Φ' (degrees)	c' (kPa)
CL	Silty CLAY	17	28	20
GM	Silty GRAVEL	20	36	2
ML	Sandy SILT	18	29	5
ML	Gravelly SILT	19	34	2
MH	Clayey SILT	16	30	10

5.2 Slope Stability Assessment

The geotechnical data was used to prepare three interpretive sub-surface ground models for the proposed cuts. Refer to Figure 2 for the presentation of the interpretive ground models, which formed the basis of the slope stability assessments. The slope stability assessments were conducted utilising numerical analysis using the computer programme SLOPE/W (GEO-SLOPE, 2012). In accordance with the standard engineering practice for slopes that provide integral support for or direct loading on a structure, target factors of safety (*FS*) against failure of 1.5 was established for static loading condition.The Morgenstern-Price method of slices was utilised to determine the *FS* for each failure surface. As no groundwater was encountered in any of the sub-surface investigations, the groundwater table was assumed to be below access track level for the analysis.

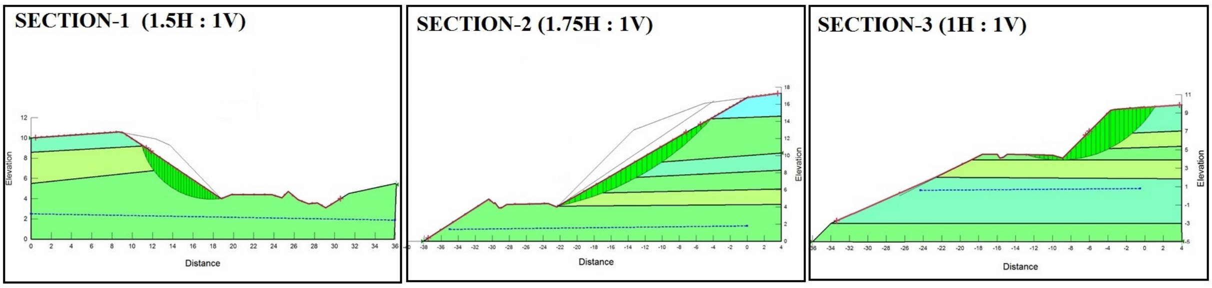

Figure 2. Access Track Analysis Models

5.3 Summary of Stability Analyses and Design of Slope Surface

The Factor of Safety for each section with different cut slopes to achieve has been briefly summarised below in Table 2.

Table 2. Summary of Analysis

Section Description	Analysis Model Chainage	Slope	Angle (^{0})	Factor of Safety (*FS*)
Section – 1	260 m to 340 m	1 H : 1 V	45	1.16
		1.5 H : 1 V	34	1.53
Section – 2	420 m to 580 m	1 H : 1 V	45	0.99
		1.5 H : 1 V	34	1.34
		1.75 H : 1 V	30	1.52
Section – 3	580 m to 660 m	1 H : 1 V	45	2.08

The cut slopes for the access road have been computed from the *FS* values returned from the slope stability analyses. All met or exceeded the target *FS* value of 1.5. Initial assessment of the cut slope for the access road using seismic case required large cuts, daylighting at the top of ridges with large earthwork quantities. After a value engineering process, the client decided to accept the risk of failure during major seismic event by assuming the access route could be reinstated quickly after the event.

6 ASSESSMENT OF THE STABILITY OF RESERVOIR PLATFORM

6.1 Geotechnical Investigations

The proposal for the reservoir platform was to cut approximately up to 7.8m from the top of the ridge to create a platform for the new reservoirs. Geotechnical site investigations, laboratory testing and slope stability analysis were carried out to confirm the setback distance from the edge of the slope to the water tank, stability of the reservoir platform and to check the bearing capacity and settlement of the ground beneath the reservoirs.

6.1.1 Field Work

Two boreholes and five static Cone Penetration Test (CPT) were conducted at the site during January 2016. Boreholes were drilled to 20m depth at the platform location before earthworks, with Standard Penetration Test (SPT) at 1.5m intervals. The core samples were logged in accordance with the Guidelines for the Field Description of Soil and Rock (NZGS, 2005). All CPT testing progressed until the cone meet refusal (was unable to penetrate the ground). CPTs meet refusal at a maximum depth of 5.3m, with further penetration retarded by dense gravel layers.

6.1.2 Geotechnical Parameters

The soils encountered in the boreholes and parameters used in slope stability analysis are presented in Table 3, and have been derived from visual description of soils encountered, in-situ soil testing and published empirical correlations with in-situ test results

Table 3. Summary of Soil Properties

USCS Soil Classification	Description	Parameters used in SLOPE/W analysis		
		Unit Weight (kN/m^3)	Φ' (degrees)	c' (kPa)
-	Fill	17	34	1
SM	Silty SAND	18	34	2
GM	Sandy / Silty GRAVEL	20	40	2
ML	SILT / Gravelly SILT	19	36	5

6.2 Slope Stability Assessment

The geotechnical data was used to prepare three interpretive sub-surface ground models for the proposed engineered cut/fill platform. Refer to Figure 3 for the presentation of the interpretive ground models, which formed the basis of the slope stability assessments. The slope stability assessments were conducted utilising numerical analysis using the computer programme SLOPE/W (GEO-SLOPE, 2012). In accordance with the standard engineering practice for slopes that provide integral support for or direct loading on a structure, target factors of safety (*FS*) against failure of 1.5, 1.0 were established for static and seismic loading conditions, respectively.

The Morgenstern-Price method of slices was utilised to determine the *FS* for each failure surface. As no groundwater was encountered in any of the sub-surface investigations, the groundwater table was assumed to be well below the reservoir platform level for the analysis.

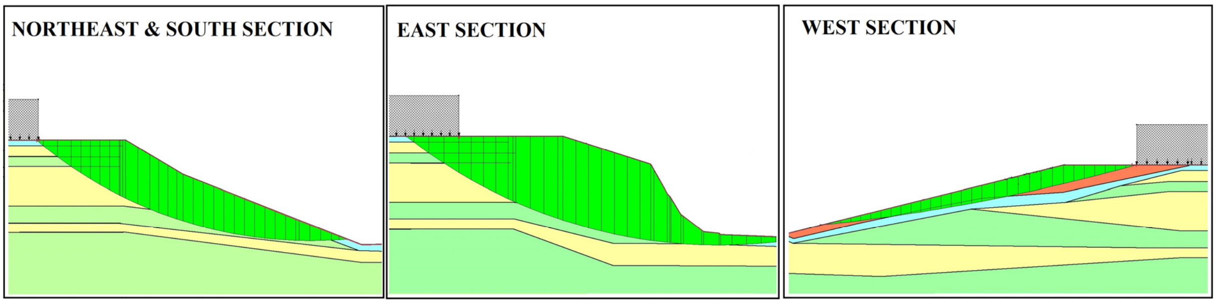

Figure 3. Reservoir Platform Analysis Models

6.3 Summary of Stability Analyses and Design of Slope Surface

The Factor of Safety for each section with different cut slopes to achieve has been briefly summarised below in Table 4.

Table 4. Summary of Analysis

Analysis Model	Analysis Type	Maximum Slope Angle	Factor of Safety (*FS*)	Distance of tank from edge of the slope[a]
Northeast & South	Static case	25^0	2.55	15 m
	Seismic case		0.87	
East	Static case	41^0	2.23	18 m
	Seismic case		0.93	
West	Static case	14^0	3.56	6 m
	Seismic case		0.92	

[a] Slip circles beyond this distance from edge of the tank are not considered in FOS calculation

Results from the stability analysis shows the slope stability of the critical section has a factor of safety of 2.23 under static condition. However, factor of safety under seismic condition is less than the recommended seismic factor of safety of one. This factor of safety is accepted considering the conservative value of K_h equal to PGA.

7 DESIGN OF RESERVOIR PLATFORM

The proposed setback distances for the reservoir could be achieved by increasing the footprint of the reservoir platform. The excavated material from the top of the ridge were used to construct engineered fill to increase the platform footprint. The development included removing a maximum thickness of approximately 7.8 m of materials from the site to form the proposed reservoir platform. This material was placed on the western side to increase the platform area to accommodate the northern reservoir. The predominant ground conditions found on the site were that of marine deposits consisting of sand and gravel interbedded with layers of silts. The marine deposits are considered suitable for re-use as engineering fill provided the water content is controlled, adequate compaction is achieved and unsuitable material such as loess/ residual soil were cut to waste. It was recommended that, in order to achieve a stable slope, all permanent slopes be constructed at a maximum gradient of 4H:1V (14^0) and that topsoil be spread out on any disturbed areas to an even depth, grade and finish.

Reservoir–1 (Southeast Reservoir) would be built fully on the cut platform. The depth of cut within the proposed reservoir footprint varies from 1m to 7m depth. Reservoir–2 (Northeast Reservoir) would be built across a cut and fill platform, which varies in depth from approximately 2.4m depth of cut to 2m thickness of fill. The whole Reservoir–2 footprint including the in-situ ground is to undercut by 2 metres minimum and recompacted with imported AP65 material to ensure a transition in the stiffness of foundation materials, to minimise differential movement.

8 SUMMARY AND CONCLUSIONS

On the basis of the terrain evaluation, ground investigations and analysis, the conclusions on access track cut and the reservoir platform are given as follows:

- The soils were likely to be erodible and care was required on the cut batter face to avoid rilling and surficial failures. Measures to stabilise the cut batter and exposed soils were to be incorporated into the works;
- Slope angles identified for the cut batter slope for different chainage were based on the factor of safety of 1.5 for static slope stability analysis given in Table 2;
- Global stability is considered satisfactory following the construction of the reservoirs. However, appropriately sized setbacks listed in Table 4 from the reservoir and crests of slopes were incorporated in the layout to mitigate seismic slope stability hazard;
- The reservoir site is underlain by marine deposits, which include weathered dense to very dense gravels, weathered firm to stiff silt, with a thin blanket of loess deposit. The marine deposits generally have a SPT N value greater than 40;
- Foundation bearing capacities are satisfactory, but differential settlement needed to be addressed. This is because Reservoir–2 is to be partially in cut and fill so that the net loading is non-uniform. Undercutting and replacing with granular fill was recommended to ensure uniformity of subgrade conditions; and
- It was recommended that, in order to achieve a stable slope, all permanent platform fill slopes be constructed at a maximum gradient of 4H:1V (14^0) and that topsoil be spread out on any disturbed areas to an even depth, grade and finish.

9 ACKNOWLEDGEMENTS

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