Seismic retro-fit of an historic earth dam using grouted stone columns

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ABSTRACT

An earth dam in Victoria, constructed in 1885 as part of Melbourne's water supply system, has been in continuous use and will continue to form an integral element of the system. A safety review between 2004 and 2006 identified remedial works requirements to address a number of issues including embankment stability. Analysis of geotechnical information for the site, in conjunction with geological modelling, concluded there were layers and zones of soils that exhibited the potential for liquefaction under earthquake loads, thus constituting a significant risk to the stability of the embankment and the water supply. Design parameters were agreed and three areas were specified for treatment. Ground improvement options were assessed and the solution adopted was to install grouted stone columns. The design was undertaken, reviewed and accepted and the work carried out between July and September 2011. Grouted stone columns were formed using the bottom feed vibro system, conventionally used for the construction of vibro stone columns. A cement-bentonite slurry mix formed the grout, which was added to the stone prior to its placement into the vibro rig's hopper. Several trial columns were carried out, with attention being paid to the stone size and grading; the grout components and proportions; and installation procedures. This paper presents the background to the solution adopted to retro-fit the dam against future seismic events and presents the results of testing, initial trials and the implementation of the ground improvement.

Keywords: ground improvement, earth dam, liquefaction, seismic risk, grouted stone columns, vibro

1. INTRODUCTION

Toorourrong Reservoir was constructed in 1885 to collect water from the Plenty River and the Wallaby Creek and Silver Creek Aqueduct systems at a point north of Whittlesea, Victoria, and to divert this water into the Clear Water Channel for transfer to Yan Yean Reservoir. The reservoir is located approximately 35km north-east of Melbourne city centre.

A safety review of the reservoir recommended remedial works be undertaken on the dam to address compliance-related shortfalls including embankment stability (SMEC 2006). As a result, Melbourne Water initiated the Toorourrong Reservoir Upgrade Project, which was undertaken by the Water Resources Alliance (WRA) comprising of Melbourne Water, Baulderstone, United Group, SKM, BECA and MWH. This paper addresses the design and implementation of remedial works associated with potentially liquefiable soils.

2. GEOLOGICAL SETTING AND SECTIONS OF CONCERN

The geotechnical conditions at the site generally comprise Quaternary age alluvium consisting of sand, silt, clay and gravel. These recent deposits are underlain by the Silurian to Lower Devonian age Humevale Formation, comprising several metres of weathered residual soil, grading to highly and less weathered massive to thinly bedded siltstone with minor sandstone. Figure 1 shows the site investigation locations and indicates the locations of sections of concern.

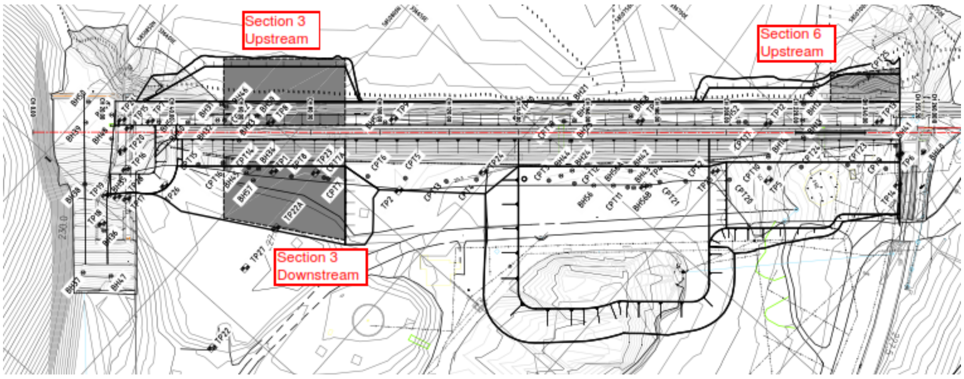

Fig.1 Layout Plan of Geotechnical Investigation Locations and Remedial Sections

Selected subsurface profile information for the site (WRA 2010a) are summarised in Table 1:

Table 1: Relevant Soil Units for the Project

Soil Unit	Description
T1	SILTY CLAYS (Former River Channel). Dominantly Silty CLAY (CL)/Clayey SILT (ML), occasional layers of Clayey SILT (ML).
T2	SANDS (Former River Channel). Dominantly Clayey SAND (SC). Also layers of Silty SAND (SM), Sandy SILT (ML), SAND (SP).
T3	SANDY CLAYS and SANDY SILTS (Former River Channel). Layers of Sandy CLAY (CL), Sandy SILT (ML).
T4	SILTY SANDS with clays and silts (Former River Channel). Silty SAND (SM), Sandy SILT (ML), Sandy CLAY (CL), Silty CLAY (CL)/Clayey SILT (ML).
T5	SILTY COARSE MICACEOUS SANDS with some GRAVELS (Former River Channel). Silty SAND (SM), minor deposits of Gravelly SILT (ML/GM) with siltstone fragments.
T7B	PARTLY FILL AND COLLUVIUM: CLAY and MUDSTONE FRAGMENTS: found on the right and left abutments to lower valley side slope. Typically CLAY (CI).
T8	SILTY CLAY (CH-CL). Typical range: Silty CLAY (CL), Silty CLAY (CI), Silty CLAY (CH).
T14	SILT (ML) (Channel Deposit)
T15	GRAVELS, BROKEN SILTSTONE AND MINOR INTERLAYERED RIVER DEPOSITS. Materials are a complex of layers and disturbed material. For example log descriptions include SILTSTONE (GM), Clayey SAND (SC), Silty CLAY (CL), Clayey SILT.

There are layers and zones of potentially liquefiable soils within some of the soil units. On the basis of the geological conditions and back-analysis of stability and long-term performance of the dam, a significant risk of liquefaction under earthquake loading and possible subsequent slope failure was identified at two cross-sections along the dam alignment. Three areas, as shown in Figure 1, were identified for remedial work to reduce the risk to an acceptable level.

Section 3 is located towards the west abutment and the spillway and is the location of the former course of the Plenty River before construction of the dam. The geometry of the foundation soils at Section 3 was idealised to four main layers (Units T1, T2, T4, T5) and a downstream lens T3. Units T2, T3 and T4 are the potentially liquefiable layers in this section.

Section 6 is located towards the eastern abutment of the dam adjacent to the Clear Water Channel. The geometry of the foundation soils at Section 6 was also idealised to four main layers, representing Units T7B, T8, T14 and T15, of which T8 and T15 are potentially liquefiable.

3. TREATMENT SELECTION

Several options were considered prior to the final selection of the adopted solution. Initial considerations ranged from the installation of sand columns, jet grouting and deep soil mixing, to the preferred option of vibro stone columns (Arulrajah et al 2009, Lawton 2004, Priebe 1998). After initially assessing the use of vibro stone columns, the solution finally adopted was grouted stone columns (GSCs), as untreated stone columns were considered to pose a significant risk to the migration of fines from the natural soils. Effectively constructing vibro stone columns with the void spaces filled with cement-bentonite slurry minimised the risk of developing preferential flow pathways through the untreated stone.

Factors of Safety (FOS) as low as 1.0 were computed for some slip surfaces in Sections 3 and 6 (WRA 2010b). The remedial zone was analysed as a soil block and the ground treatment was applied to achieve a minimum FOS of 1.2 for slope stability under post-liquefied ground conditions. The GSCs adopted for this project comprise 0.8m diameter columns constructed on a 1.7m triangular grid pattern. Column depths varied dependent on the depth of the weak soils and the shape of the failure surface. The diameter and spacing represents a replacement ratio of approximately 20%, which is within normal range for vibro (grouted) stone columns. The remedial work at Section 3 is shown in Figure 2.

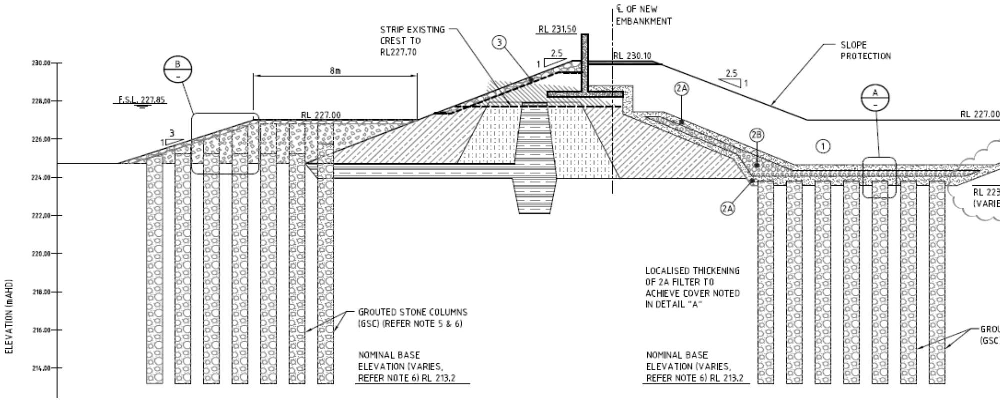

Fig. 2: Section 3 Treatment and Construction Cross-Section

4. DESIGN

4.1 Design Concept

The application of limit equilibrium (SlopeW, Morgenstern and Price 1965) to govern the improved soil block size width was considered too simplistic to address the ductility and deformation issues relating to grouted stone columns. The design concept therefore effectively assumed a redundancy or “sacrificial” width to address the edge effect issues.

The US Army Corps of Engineers method was adopted to define the treatment zone under a ‘conventional’ application, which extended it outside the perimeter by a distance of at least half the width of the treatment zone (see Figure 3).

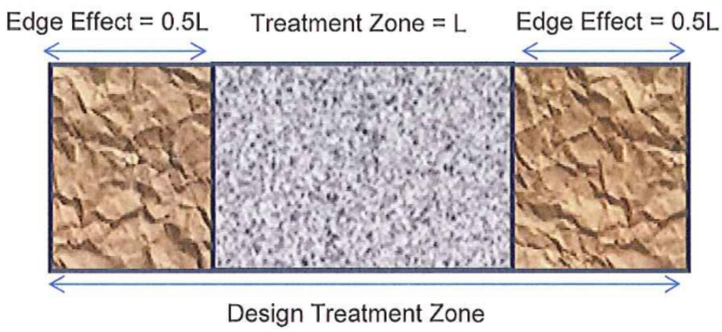

Fig. 3 – Treatment Block Width Adopted for Design

4.2 Design Parameters

The derived soil parameters described in the geotechnical design report provided (WRA 2010a) were incorporated into the Priebe Method (Priebe 1995) to produce the composite frictional angle (φ’c). This parameter was then input into SlopeW as design friction angle (φ’d) to analyse slope stability. The analyses were carried out based on the post-liquefied condition. Composite undrained shear strength (Su) was computed using the “Japanese method” (Goughnour et al, 1991) (see Table 2).

Table 2: Summary of Selected GRETA Output (Priebe Method) as Design Parameters

Section	Column Friction Angle (φ)	Composite Materials from GRETA Output			Unit Weight (kN/m^3)
		Friction Angle (φ)	Ave. Cohesion (kPa)	Constrained Modulus (MN/m^2)	
3 Upstream	32	30.7	11.2	14.6	19.0
	36	33.2	11.2	16.0	19.0
3 Downstream	32	30.7	6.7	17.5	19.0
	36	33.2	6.7	19.1	19.0
6 Upstream	32	32.0	7.9	21.7	19.0
	36	34.3	7.9	23.6	19.0

4.3 Design Analyses

A sensitivity check on a range of the design friction angle (φ'd) against slope stability FOS achieved from the limit equilibrium method are presented in Figure 4, demonstrating the adequacy of the proposed soil block dimensions. Different failure modes were assessed to ensure the selected models were within the safety zone for the case where the soils surrounding the columns within the soil block were not liquefied. The FOS obtained for composite internal friction angles of the soil block between 28° to 40° exceeded the required value of 1.2.

Concern was expressed that the introduction of bentonite into the grout mix could reduce the internal friction angle of the column significantly; this sensitivity check aimed to mitigate the design risk ensuring the proposed soil block was safe within the proposed range of design phi angle (φ'd).

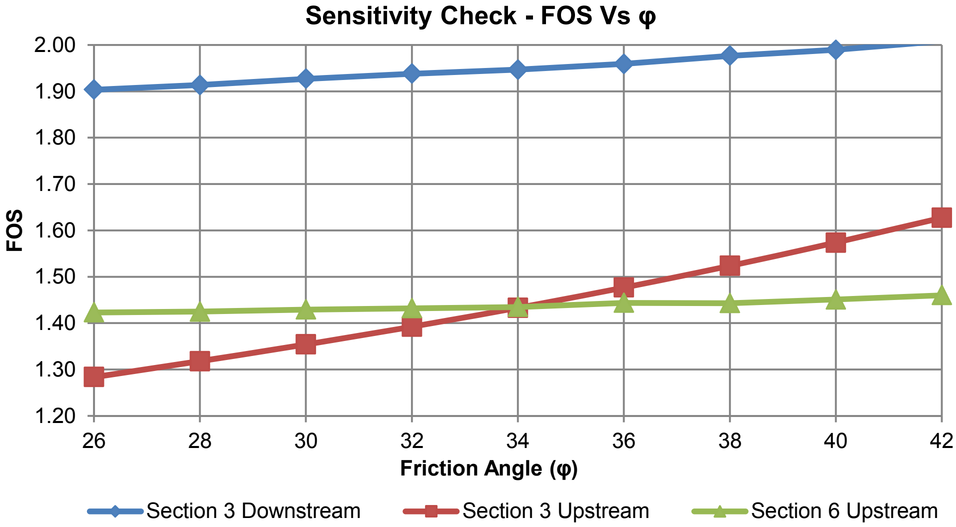

Fig. 4 – Sensitivity check of friction angle against factor of safety

As described above, the stress-strain effect or relationship was the critical element of the design, governing the failure mechanism of the soil block. Finite element analysis (FEA) was carried out to demonstrate the horizontal displacement at the top of the columns at different distances away from the edge of the soil block.

The results indicated that the columns, particularly the first two rows, were incapable of restraining such a large differential strain. The maximum calculated displacements of 55mm to 90mm represented failure strains of 6.9% to 10.9%. This confirmed the requirement for the extended treatment block width as shown in Figure 3.

Summary results from SlopeW (see Table 3) indicated that a minimum 10m width soil block was required to achieve the FOS ≥ 1.2 under a worst case scenario of soils surrounding the columns within the soil block being liquefied.

Table 3: Summary of SlopeW Results

Section	**F.O.S**	**Top of GSC (RL)**	**Bottom of GSC (RL)**	**Estimated Depth (m)**
Section 3 Downstream	1.23	224.8	213.2	11.6
Section 3 Upstream	1.26	224.8	213.2	11.6
Section 6 Upstream	1.29	225.8	219.2	6.6

5. MATERIALS

Stone obtained from local quarries was tested for particle size, natural angle of repose, specific gravity, bulk density and void ratio. The selected stone was a basalt rock, crushed and screened to deliver a produce of between 20mm and 40mm particle size, although the specification was subsequently modified to include finer particles down to 5mm. Laboratory tests were undertaken on dry and grouted compacted stone in a 300mm shear box (Queensland Main Roads Test Method Q181C), for which the stone had to be further crushed to be less than 20mm – Figure 6 shows a test set-up with grouted stone and Table 4 summarises the results.

The grout comprised a cement-bentonite mix, which delivered a 28-day UCS of between 600kPa and 1.5MPa.

Fig. 6: Shear Box Test on Grouted Stone

Table 4: Summary of Shear Box Test Results

		Single Grade 40mm – 20mm (crushed to 19mm – 16mm)			Graded 20mm – 5mm		
SG / bulk density		2.92 / 1.618 t/m^3			2.92 / 1.697 t/m^3		
	Normal Stress (kPa)	Peak Stress (kPa)	Friction Angle Peak	Friction Angle Residual	Peak Stress (kPa)	Friction Angle Peak	Friction Angle Residual
Dry stone	100	169	59.4		150	56.3	56.0
Stone + Grout (wet)	50	72	55.2		87	60.1	
	150	232	57.1		237	57.7	56.8
Stone + Grout (cured)	100	206	64.1	60.9	254	68.5	
	150				317	64.7	

As is evident, the friction angle of the stone was generally between 50^o and 60^o and slightly higher after curing of the grout. The results show little effect from the inclusion of bentonite in the slurry, a critical factor in the design analysis, where a friction angle of between 28^o to 40^o was calculated.

6. INSTALLATION

GSCs are formed using the bottom feed vibro system. The rig, hopper and skip are shown in Figure 7a, and the vibro probe is shown in Figure 7b.

Figure 7a: Installation Set-up

Figure 7b: Vibro Probe

Trial columns were installed in the area of Section 3 Downstream during the first phase of the three phases of work. These are shown in Figures 8a and 8b with their tops exposed.

Figure 8a: Trial columns

Figure 8b: Top of Typical GSC

7. CONCLUSION

Grouted stone columns were adopted and installed at an historic earth dam to provide protection from future seismic events. The ground improvement treatment was designed to address potentially liquefiable soils that could destabilise the water storage dam under earthquake loadings. The site works were completed during 2011.

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