

# HARRIS DAM SLURRY TRENCH : DESIGN AND CONSTRUCTION

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## SUMMARY

A slurry trench cutoff up to 27 metres in depth through residual granitic soils, has recently been constructed as part of the seepage control measures for the Harris Dam, Western Australia.

The slurry trench had to accommodate foundation settlements of about 500 mm and still remain relatively impermeable.

Extensive laboratory trials were carried out to develop a cement-bentonite slurry mix that satisfied these requirements. Close monitoring during construction confirmed that these design criteria were largely achieved in the finished product.

This paper describes the design and construction of the Harris Dam cutoff. A companion paper (1) describes the investigation and design of the slurry mix used in construction of the trench.

## INTRODUCTION

The Water Authority of Western Australia is constructing a dam across the Harris River in the Darling Ranges. This is at a point 10 km north of the town of Collie, in the State's south-west. The dam will supply the Great Southern Towns Water Supply system in which it will replace the existing Wellington Dam. Increasing salinity levels in the latter will restrict future use of its water to irrigation purposes.

Harris Dam is essentially an homogeneous earthfill embankment, 35m in height and 430m in crest length. A cross-section of the dam is shown on Figure 1. The dam is founded on in situ soils, generally 20 to 30m thick, formed by weathering of the underlying, predominantly granitic bedrock.

Preliminary conceptions of the site saw the foundation soils as being:

either i) relatively impermeable and likely to sustain

high pore pressures during embankment construction.

or ii) permeable and requiring a positive cutoff to limit underseepage. Detailed site investigations were carried out by Authority staff to resolve these and other design issues.

## SITE DESCRIPTION

The dam foundations comprise up to 30m depth of largely residual granitic soils, overlying granite bedrock dissected by dolerite dykes. Whilst completely and highly weathered materials predominate in the soil profile, less weathered materials exist locally either as occasional corestones or as thin zones above the slightly weathered to fresh bedrock. The bedrock is shallower at the abutments and outcrops on the left abutment to provide an unlined chute for the spillway. The residual soils are mostly silty and clayey sands and silts. Laterite is present as a thin mantle of slopewash gravels on the valley sides and as a bonded, ferruginous layer, 1 to 2m thick, just below the surface, mainly in the valley floor. A typical log and CPT trace of the soil profile are shown on Figure 2.

Measurements of soil permeability varied widely with the test method:

- i) laboratory constant head tests averaged  $1.9 \times 10^{-7}$  m/s
- ii) packer tests, typically on 1.5m long sections of borehole, averaged  $3.3 \times 10^{-6}$  m/s
- iii) 'pump out' tests gave average values of  $2.0 \times 10^{-5}$  to  $3.3 \times 10^{-5}$  m/s over aquifer depths of about 20m.

The results of the 'pump out' tests were considered to be the most reliable as the test is full scale and therefore includes macro soil fabric effects such as relict joints, fissures, cracks, root channels and quartz veins. Furthermore, 'drilling induced' smearing on the sides of the drillhole, has less of an effect in 'pump out' tests, compared to packer or 'pump in' tests.

Packer test measurements of permeability in the underlying bedrock varied considerably from the impermeable ( $<10^{-7}$  m/s), to greater than the capacity of the test equipment ( $>10^{-2}$  m/s). Average values however were of the order  $4 \times 10^{-6}$  m/s and fall within the 'groutable' category.

## SEEPAGE PROBLEM

A 2-dimensional seepage flow net analysis assuming a full reservoir condition and isotropic foundations, indicated that up to 3756 m<sup>3</sup>/day could be lost in underseepage. This is equivalent to 8% of the reservoir's annual yield or about \$1.6M in terms of the capital cost of the Harris Dam. Seepage control measures could obviously reduce this loss of

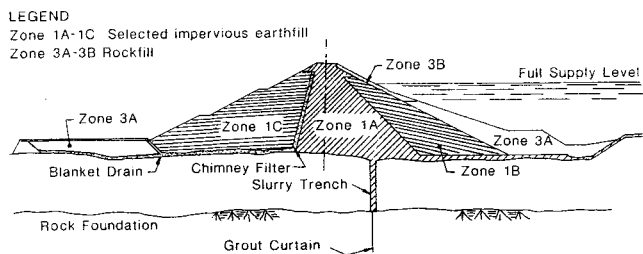


FIGURE 1 : Harris dam embankment cross section

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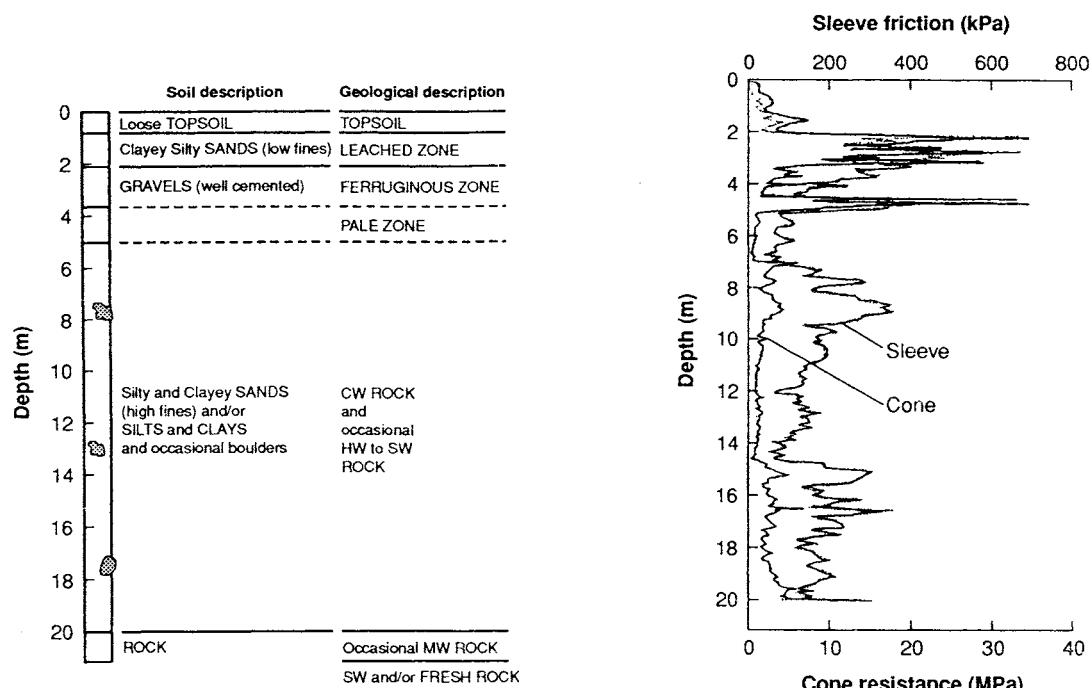


FIGURE 2: Typical soil profile in valley floor

water resource. Such measures would also permit major economies in the design of the downstream drainage blanket and/or embankment stability.

Various methods of control were considered and assessed as follows:

- a clay filled cutoff down to bedrock in opencut excavation was likely to be expensive with problems of river diversion, dewatering, slope instability, stockpiling and disposal of unsuitable material.
- a grout curtain through the residual soils would require the use of expensive chemical grouts and would not be as cost effective as more positive forms of cutoff.
- an upstream impervious blanket would be limited in its overall seepage reduction potential and would be too sensitive to the presence of permeable zones such as dykes or faults, extending under the dam.
- a slurry trench would provide a positive cutoff in the form of a continuous wall, extending some 25m down to bedrock. It would be constructed across the full width of the valley in a series of overlapping panels. Most of the trench could be excavated by grab with only the more resistant material, found locally, requiring first breaking up by chiselling. Costs were expected to be about \$250/m<sup>2</sup>.

The slurry trench cutoff was selected as being the most cost effective method of seepage control within the soil profile. The technique avoids the site disruption and risks associated with the large opencut excavation and provides a far more positive and effective cutoff than either soil grouting or an upstream impervious blanket. To complete the seepage control measures, the slurry trench was to be 'underpinned' by a cement grout curtain within the bedrock. This was

considered essential to prevent seepage bypassing the slurry cutoff through more permeable features in the underlying bedrock e.g. dykes and shear zones etc.

#### CUTOFF DESIGN

Whilst several slurry cutoffs have been constructed in Australia over the last twenty years, they are not common and of necessity their designs, are to some extent, site specific. (2)

There are two principal methods of slurry cutoff construction normally used. In one, bentonite slurry is employed to support the trench during excavation, following which it is displaced by a plastic concrete or soil bentonite slurry, tremied into the bottom of the trench. In the other, cement bentonite slurry, with or without retarders, is used throughout the operation, both for excavation support and final construction. The latter method was adopted for the Harris Dam as it is quicker and cheaper than the other method and avoids the risk of remnant bentonite contamination of the finished wall.

Efficiency of the slurry trench cutoff was examined by flow net analysis for various foundation and slurry permeabilities and trench widths. The results, which were later confirmed by finite element analysis, are given in Table 1 for an 800mm wide trench. For greater trench widths, the seepage will reduce proportionately.

The results clearly indicate that the key factor in reducing seepage is the permeability of the hardened slurry material. If a figure of  $1 \times 10^{-8}$  m/s could be achieved in the field, as some references suggest (3,4,5), then the seepage could be reduced by as much as 95%.

The trench width of 800mm was selected somewhat arbitrarily. It had to correspond to available clamshell grab sizes. In addition it had to limit hydraulic gradients across the section whilst providing some allowance for reduced width due to

Foundation permeability (m/s)	SEEPAGE (m <sup>3</sup> /day)		
	No cut-off	Plus slurry cut-off	
		Slurry permeability (m/s)	
		$k = 1 \times 10^{-7}$	$k = 1 \times 10^{-8}$
$2 \times 10^{-5}$	1832	773	129
$4 \times 10^{-5}$	3756	966	132
$1 \times 10^{-6}$	94	N/A	N/A

Table 1: Seepage estimates for 800 mm wide trench

misalignment at joints between panels.

The slurry trench was located about 20m upstream of the dam centre line to coincide with shallow bedrock. In this position the trench would still be subject to substantial embankment loads in the centre of the valley, where the seepage head conditions would be greatest. At the abutments, the trench was terminated on shallow rock ridges that extended above full supply level.

Throughout the length of the cutoff, the trench was intended to be keyed 500mm into underlying slightly weathered to fresh rock to provide a continuous impervious contact. A view of the trench and grout curtain in elevation is shown on Figure 3.

The grout curtain was to extend down some 10 to 20m below the base of the trench and was offset 1.2m upstream of the trench, rather than being on the same line. This was done to avoid subsequent drilling of grout holes through the trench disturbing the slurry or, if pregrouting, chisel operation at the bedrock contact, disturbing the set grout. Contact between the trench and grout curtain was to be sealed by TAM (tube a manchette) grouting over the bottom 2m of the soil profile, using the relatively fine, Type SA cement.

#### DESIGN REQUIREMENTS OF SLURRY

Although the target permeability for the slurry was known,

other properties of the hardened slurry needed to be defined. Clearly the material needed to be stable, erosion resistant and largely free of cracks. In addition however, the stress-strain characteristics of the slurry were important.

During construction of the 35m high dam embankment, the underlying foundation soils were expected to compress by up to about 500mm of largely immediate settlement. The slurry trench needed to have similar compression characteristics. If more compressible than the adjacent soils, the slurry material would yield, shedding overburden load onto the adjacent stiffer soils. In this situation, the trench material would be vulnerable to hydraulic fracture under reservoir head. Equally, if too stiff, the slurry trench would tend to attract load and could become over stressed, leading to cracking and associated leakage.

Ideally, therefore, the slurry material needed to have stress-strain characteristics identical to those of the surrounding soil and to remain relatively impermeable over the operating range of strains. In practice, as might be expected, the in situ soils at the Harris Dam site are not homogeneous like the man made slurry, but vary greatly in their compression characteristics. A perfect match between the strength properties of the slurry material and those of the in situ soils was therefore impractical. The best one could hope for, was to select a slurry modulus that minimised differential settlements between the slurry and adjacent soils, particularly in the upper, more critical sections of the trench. In addition, however, to cope with the local high shear stresses that would be generated at the slurry soil interface, the slurry material needed to be capable of deforming without cracking over a wide range of strain.

The following properties were finally selected as the design criteria for the slurry mix:

- i) laboratory permeability  $\leq 10^{-8}$  m/s
- ii) erosion resistance of ND1 in the pinhole test
- iii) durability - remain stable
- iv) elastic modulus of 40 MPa  $\pm$  10%

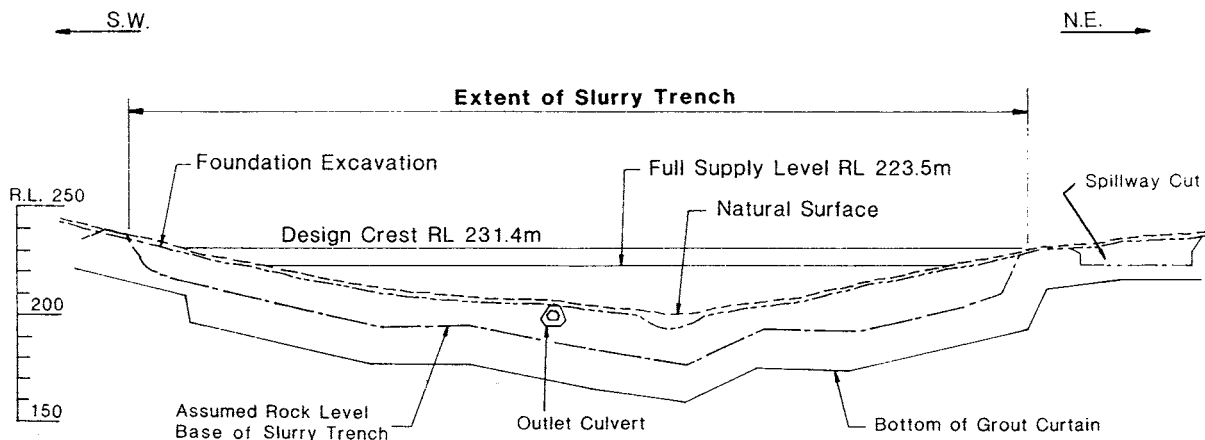


FIGURE 3 : Section along slurry trench

v) UCS  $\geq$  200 kPa

vi) deformability - elasto-plastic response up to a strain of 10% i.e. no cracking.

To be certain these criteria could be met in practice and to gain a better understanding of slurry properties and the effects of different cements, the Authority carried out a programme of trial mixes in the pre-contract design period. The results of this work, along with those of subsequent contract trial mixes, are discussed in the accompanying paper (1).

#### AWARD OF CONTRACT/PROGRAMME

Separate contracts were let for the cutoff construction including curtain grouting and for the embankment construction. Although both contracts started at the same time, in August 1988, the slurry trench/grouting works was programmed to be finished by April 1989, leaving the following summer for construction of the main embankment.

Five 'Schedule of Rates' tenders were received from a select list of contractors, at least three of whom had agreements with European specialist diaphragm wall contractors. A contract was awarded to the lowest tenderer, GFWA Pty Ltd. of Perth, for \$2.6M.

#### TRIAL MIXES

Since fluid properties of the slurry are critical to panel construction, the contractor was made responsible for mix design. The contract therefore, included for carrying out a series of trial mixes to determine a mix that satisfied both construction and design requirements. This work was carried out by a slurry specialist from SIF Bachy, GFWA's parent company with the assistance of staff from the Water Authority's Engineering Research Station (ERS).

The contractor's tender proposed a blend of 35% OPC and 65% crushed granulated blast furnace slag (BFSC) as the cementitious material for the slurry. Pre contract investigation of slurry mixes at ERS, using various cements, including OPC with and without blast furnace slag and fly ash, had already identified the strength advantages of blended cement. The contract trial mixes were therefore able to zero in on the final design mix without investigating a wide range of trial mixes.

From a construction standpoint, the following properties of the fluid slurry were important:

- Viscosity. This is largely determined by the bentonite content, to give a stable mix that prevents cement separation, limits bleeding and yet is pumpable and allows settling out of coarser excavated material.
- Density. This must be sufficient to support the sides of the trench.
- Bleed. Significant release of water from the slurry is undesirable.
- Setting Time. This must be sufficient, with or without retarders, to allow full excavation of the trench panel, including any keying into rock.
- Filter Losses. This is water lost from the slurry into

adjacent soils depending on their porosity. More significant at depth, it leads to the formation of a filter cake which increases the impermeability and strength of the outer skin of the cutoff wall.

The trial mixes indicated that increasing cement contents reduced the permeability of the slurry but tended to increase the brittle or non-plastic character of the material. Furthermore that soil 'take up' by the slurry, when placed in the ground, would probably lead to increased density, strength and modulus of the hardened slurry and a consequent decrease in deformability.

The design mix adopted contained the following:

- 225 kg/m<sup>3</sup> of blended cement
- 30 kg/m<sup>3</sup> of bentonite
- 913 l/m<sup>3</sup> of water
- 0.5 to 2.0% by weight of cement of retarder

#### CONSTRUCTION

Prior to any trenching, a series of boreholes were drilled at 5m centres using wash boring techniques to better define the bedrock profile. Any anomalies were then investigated by diamond coring and a clearer indication gained of both the likely trench termination depths and the presence of floaters or corestones.

The slurry trench was constructed in alternate primary and secondary panels, 2.7m long, corresponding to the dimensions of the grab. Primary panels were excavated first then secondary or infill panels, followed either the same day or 24 hours later, but in 3 or 4 cases, several weeks later. The secondary panels overlapped and excavated into adjacent primary panels by at least 350mm at the surface. The primary panels served as a guide or control for excavating the secondary panels. However for all panels, alignment and stability of the upper section of the trench was largely maintained by excavating between a steel guide frame, laid on the surface. Minimum overlap allowed between panels was 100mm longitudinally and 600mm laterally. A schematic view of panel construction is shown on Figure 4.

Excavation of the completely and highly weathered material which made up most of the panel, was carried out using a KL 800, 800mm wide by 2.7m long, 8 tonne, clamshell grab, twin cable operated by either an RB38 or RB61 crane. Excavation of rock to form the key into bedrock or to remove corestones or hard rock layers was carried out using a 9 tonne chisel to break up the material for later removal by the clamshell.

Construction of the first few panels quickly revealed that chiselling of the bedrock to establish a 500mm deep key was largely impractical. Not only was the operation very time consuming, it was adversely affecting the slurry mix. Repeated chiselling disturbed the trench sides causing increased loss of water and increased soil 'take up' by the slurry. In turn, workability of the slurry was greatly reduced, promoting difficulties in cleaning out the trench base. Density of the slurry was increased accordingly, with ultimately a reduction in deformability. The requirement for a key was therefore relaxed to one of establishing good contact with fresh rock over the width of the panel.

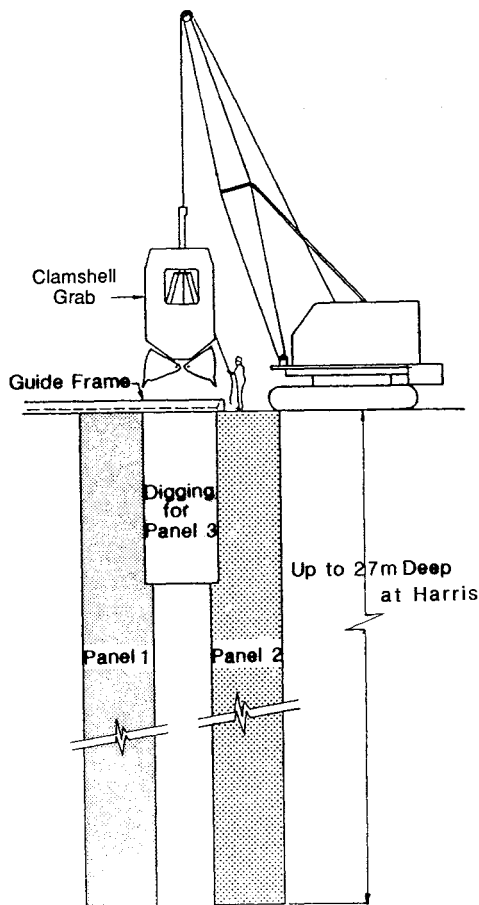


FIGURE 4: Sequence of slurry trench panel construction

Corestones or floaters of relatively unweathered rock were encountered at various levels in the completely weathered soil profile in about 20% of the panels. Most of these were satisfactorily dealt with by breaking through with the chisel. Where this was impractical, the trench was locally diverted around the obstacle.

At one location, high on the left abutment, extensive corestones could not be avoided and over 40kg of explosives were detonated in eighteen drillholes in an attempt to break up the offending rock. Although successful at shallow depth, the blasting made little impact on a deep seated stronger layer. The trench was therefore founded on this hard layer and the underlying 'window' of completely weathered material was later sealed off by TAM (tube-a-manchette) grouting with microfine cement.

The ground conditions and in particular, the floaters, presented some major headaches for the contractor when the clamshell grab became stuck or locked in the ground. This occurred several times throughout the job and techniques for freeing the grab ranged from the use of small explosive charges, special jacking equipment to a 6m long excavation shield. One of the grabs resisted all attempts to free it and, in the end, had to be abandoned.

Construction took longer than anticipated due in part to industrial stoppages and to delays in recovering grabs held

fast in the ground. Excavation commenced in November 1988, working uphill on the west bank of the river and finished up on the east abutment in June 1989. Production averaged one primary or two secondary panels per day and peaked at four panels per day in shallow working, high up on the abutments. Panel depths reached 27m in the riverbed area but averaged around 15m. A total of 177 panels were constructed which provided an overall surface area of 8100 m<sup>2</sup>.

## QUALITY CONTROL AND RECORD TESTING

The following parameters were tested daily as the principal means of quality control of the slurry: density, viscosity, pH, filter cake thickness, filter losses, bleed and sand content. Predictably the character of the bentonite 'master mud' (the bentonite: water mix before the addition of cement) and the slurry from the mixing plant remained relatively constant throughout the job. Slurry from the trench, however, was far more variable because of differences in the age of the samples, the ambient temperature and the amounts of in situ soil 'taken up' by the slurry.

In addition samples of both fluid and hardened slurry were taken periodically from the trench for checking against the design criteria.

Laboratory testing of these samples revealed the following:

- permeabilities close to target values
- wide variations in elastic modulus
- elasto-plastic behaviour generally up to 8-10% strains
- increased brittle behaviour with increased soil content

Typical values recorded of the design and construction properties of trench slurry are included in the Appendix.

## CONCLUSION

Efficiency of the slurry trench can only be really tested under full reservoir conditions which may be experienced this coming winter. Foundation piezometers either side of the slurry trench and downstream V-notch seepage measurements

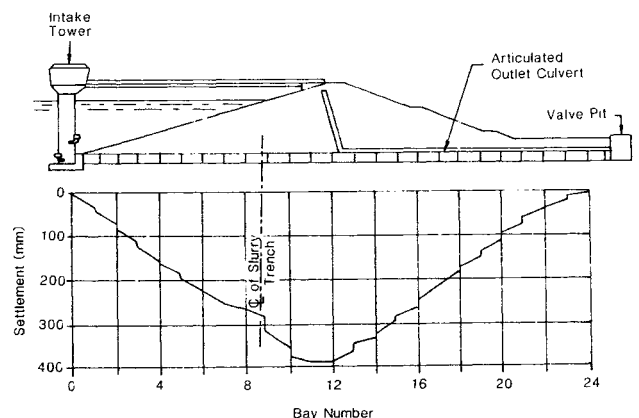


FIGURE 5: Culvert settlements on completion of embankment

should assist in analysing the situation.

In the meantime construction monitoring has produced some promising results:

- a) Measurements of settlement of the soil foundations due to embankment construction are remarkably close to predicted settlements, i.e. 475mm compared to 500mm.
- b) the relatively high modulus values of trench slurry samples suggest the slurry may be somewhat stiffer than the foundation soils, certainly at shallower depths.
- c) the relative stiffness of the slurry can also be seen in the settlement profiles of the articulated outlet culvert where the latter crosses over the slurry trench, in passing from upstream to downstream side of the dam. A noticeable kink in the otherwise uniform catenary shape of the settlement profile on Figure 5, marks the trench crossing. The result confirms that hydraulic fracture is an unlikely possibility.
- d) pore pressures in the foundation have risen only marginally in response to embankment construction. This was expected and confirms the permeable nature of the foundation soils and provides further evidence of the need for a slurry trench.

#### ACKNOWLEDGEMENTS

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The views expressed herein are those of the author and may not necessarily represent those of any organisation.

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#### APPENDIX

##### KEY DATA:

##### (a) Materials used:

Cement: Blended slag cement from Cockburn Cement (65% blast furnace slag cement, 35% OPC)

Bentonite: Aquagel (natural unpeptized bentonite) from Baroid Aus.

Admixture: Daratard combined plasticiser/retarder from WR Grace Aus.

##### (b) Slurry Mix used

<u>cement grout</u>	}	Slurry Mix:
cement 1190 kg/m <sup>3</sup>	)	Final Mix Proportions
water 598 l/m <sup>3</sup>	)	
<u>bentonite master mud</u>	)	cement 225 kg/m <sup>3</sup>
bentonite 37 kg/m <sup>3</sup>	)	bentonite 30 kg/m <sup>3</sup>
water 986 l/m <sup>3</sup>	)	
<u>Daratard</u>		water 913 l/m <sup>3</sup>
0 to 2% of cement weight		

##### (c) Typical Design Properties of Site Slurry:

Test carried out on 100mm diameter by 200mm long specimens. Samples taken of both hardened slurry from trench and fluid slurry from trench or feeder lines.

Lab permeability: Under hydraulic gradient of 45:1 and effective confining pressure of 200 kPa.

maximum  $8 \times 10^{-8}$  m/s  
 minimum  $0.2 \times 10^{-8}$  m/s  
 average  $2.5 \times 10^{-8}$  m/s

Elastic modulus: Taken as secant modulus at 50% of the yield stress on CID triaxial testing under 200 kPa effective confining pressure.

maximum 130 MPa  
 minimum 13 MPa  
 average 67 MPa

UCS: typically 300 to 600 kPa on batch plant samples 500 to 800 kPa on samples from slurry trench

Elasto-plastic behaviour commonly up to at least 5% strain, often 8 to 10% strain in CID triaxial testings as

above, under strain rate of 0.1% per minute.

##### (d) Typical Construction Properties of Site Slurry:

Refer to table 2 below:

Source of sample	Density g/mL	Viscosity sec	pH	Filter cake mm	Filtrate loss mL	Free water @2hrs	Sand content %vol
Bentonite master mud	1.02	31	8	1	17	-	-
Slurry mix at plant	1.17	41	13	-	-	2 mm	-
Slurry mix from trench	1.27	60	13	-	-	-	11

TABLE 2: Typical construction properties of site slurry