DESIGN OF PERMANENT INTRUDED PLUGS AT SOUTH DEEP GOLD MINE

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1 GENERAL

In the event of flooding of Randfontein No. 4 Shaft and recharge of the Gemsbokfontein West dolomitic groundwater compartment, five rough parallel-sided boundary plugs at 58 and 50 levels have been designed to resist safely hydrostatic heads of 1500 metres (15MPa) and 1250 metres (12.5MPa), respectively.

Given the unacceptable consequences of plug failure and the high water pressure at South Deep, all elements of the permanent plugs have been designed conservatively for a service life of 100 years. The specified performance requirements either meet or exceed the recommendations of the Chamber of Mines of South Africa "Code of practice for the construction of underground plugs and bulkhead doors using grout intrusion concrete" (1983).

Traditionally, designs of mortar intruded concrete plugs in South Africa have been based on successful precedent practice, dating in particular from technical papers by Garrett and Campbell Pitt (1958 and 1961). Essentially, the length of the plug is determined by assuming that the rock/concrete plug contact area resists punching by a uniformly distributed safe shear strength.

To the authors' knowledge, there has never been a structural failure of a high pressure mortar intruded concrete plug, but water leakage that is obvious and extensive has been encountered in service. As a consequence, particular attention has been paid to watertightness in the plug design at South Deep.

2 STRENGTH OF SURROUNDING ROCK

Three 58 level plugs at a depth of 1568m below collar were founded in Witwatersrand quartzite, whilst the two 50 level plugs at a depth of 1303m below collar were founded in Ventersdorp lava. Table 1 below illustrates that both rocks have very high strength and stiffness, although the lava has superior engineering properties.

Table 1 Typical engineering properties for Ventersdorp lava and Witwatersrand quartzite

Property	Ventersdorp lava	Witwatersrand quartzite
UCS (MPa)	295	175
Elastic modulus (GPa)	87	70
Angle of internal friction ($\varnothing^0$)	59	53
Cohesion (MPa)	31	32
Rock Mass Rating (after Bieniawski, 1973)	82	75

3 STRUCTURAL LENGTHS OF PLUGS

The 1983 code confirms the common use of a safe uniform shear stress of 0.83MPa that contains a large safety factor (not quantified in the code*) for a parallel-sided plug installed in Witwatersrand quartzite. This safe shear was adopted initially for the quartzite at South Deep and assumed conservatively for the Ventersdorp lava, as there are no published guidelines for this rock type.

> *Given the absence of any sign of incipient structural failure, back-analysis of test data on a rough parallel-sided experimental plug formed at a depth of 1216m in Witwatersrand quartzite at the West Dreifontein mine has indicated a test load factor of 7.2 when using a shear value of 0.83MPa [Garrett & Campbell Pitt (1958) for plug dimensions and pressures]. This factor is less than the ultimate load factor of safety.

At South Deep, the sectional dimensions of the parallel-sided haulages to be plugged extended typically up to 4.0m x 4.4m with a perimeter of 16.8m. For the 58 level plugs formed in Witwatersrand quartzite, the required structural length is the maximum punching force of 15MPa x 4m x 4.4m ÷ (16.8m x safe shear of 0.83MPa) = 18.9m. The equivalent structural length for the 50 level plugs in lava that may be subject to a head of up to 12.5MPa is 15.8m.

Tapered plugs were considered but discounted, bearing in mind the additional rock excavation inducing potentially further rock relaxation, the larger hydrostatic pressure on the wet face, extra intruded concrete and longer construction period.

4 STRENGTH OF MORTAR INTRUSION CONCRETE

Bearing in mind the high unconfined compressive strengths (UCS) quoted for the surrounding quartzite or lava, the weakest structural element in plug design at South Deep is the concrete due to its lower unconfined compressive strength and associated lower shear strength.

For mortar intruded concrete plugs, the 1983 Code recommends a minimum 28-day UCS of 17MPa for the mortar. This figure is based on the recommendation of Garrett & Campbell Pitt (1961). However, by reference to the American Standard ACI 322-72 for structural plain concrete, a shear stress of 0.83MPa requires a UCS of 25MPa. A further benefit of the higher strength is a more durable concrete. As a consequence, this higher value was specified as the minimum 28-day UCS at South Deep.

5 SHEAR RESISTANCE AT THE ROCK/PLUG INTERFACE

For parallel-sided plugs, a rough uneven (undulating) rock/concrete plug interface was considered essential, in order to

(i) provide friction and mechanical interlock at the rock/concrete plug interface, and thereby ensure shear failure in either the rock mass or the intruded concrete (as opposed to the rock/plug interface), and
(ii) increase the seepage path and thereby reduce the potential for leakage along the length of the plug.

Barring and scaling to sound rock was required and the rock surface roughness specified for plug design at South Deep was a clean surface containing no deleterious materials (e.g. loose debris, oil, grease, or slime) that could inhibit bonding of the intrusion mortar or plug tightening grout to the surrounding rock.

According to the 1983 Code, where a smooth surface is encountered over a large perimeter area, i.e. >10% of surface perimeter area of plug, the smooth surface of the exposed rock must be roughened by chipping. In the design at South Deep, the amplitude of the rough surface required was only 1-2mm, i.e. the same order as the maximum grain size of the sand in the intrusion mortar.

In parallel-sided plugs, mechanical interlock is desirable as a further component of the plug's resistance to shearing, since mechanical interlock can overcome shortcomings in the cleanliness and roughness of the perimeter surface of the rock. Mechanical interlock was achieved by ensuring that the undulations in cross-sectional dimensions of the roadway, prior to plug construction, were of the same order or greater than the maximum size of the pre-placed coarse aggregate, i.e. 300mm.

The design also required measurement of the longitudinal geometry of the plug site at 1m centres to enable the plug segment volume and shape to be determined and to ensure that a detrimental negative wedge shape was not created, i.e. where the cross-sectional dimensions of the plug increased from the wet face to the dry face.

6 HYDRAULIC GRADIENT

Once the minimum structural length of the plug had been determined on the basis of a uniform allowable shear stress, the watertightness of the plug contact and the surrounding rock mass was assessed.

Under the significant hydrostatic head of 1500 metres, the associated hydraulic gradient coupled with the permeability of the rock mass dictates groundwater percolation rates, and where a 100-year life is anticipated for the plugs, these parameters influence the potential dissolution of the grout at the rock/concrete plug contact and within the fissures of the surrounding rock mass.

At South Deep, it was considered advantageous to reduce the rate of groundwater flow past the plug as far as practicable by

(i) improving the watertightness of the rock/concrete plug interface and surrounding rock and
(ii) reducing the hydraulic gradient.

Grouting was employed to improve the former whilst extending the plug length was the simplest way to reduce the hydraulic gradient.

No guidance on hydraulic gradient limits for plugs is provided in the 1983 code. As a consequence, high pressure grouting to seal the rock/plug interface and the surrounding rock is employed routinely to address the potential problem of leakage in situ, although no maximum residual permeability is specified as a design performance requirement in the code.

An analysis of test results [produced by Garrett and Campbell Pitt (1958) on the experimental underground bulkhead subjected to temporary high pressures] indicated that where the rock/plug interface was ungrouted, the maximum hydraulic gradient (h/L) before leakage became obvious and extensive was about 50. In other words, for h = 1500m, a plug length (L) of 30m is required. The equivalent length for plugs at 50 level is 25m.

Although the hydraulic gradient can be raised (and the plug length reduced accordingly), as the rock/concrete plug interface and any preferential seepage paths in the surrounding rock mass are grouted, grouting at South Deep was specified as an important enhancement of watertightness, but not exploited to reduce plug length because grouting is known to be sensitive to quality of workmanship.

At South Deep, a residual watertightness ≤ 1 Lugeon (mass permeability ≤ 1×10^{-7}m/s) was specified, where 1 Lugeon is 1 litre/min/metre of hole at an excess head of 10 bars (1MPa). Depending on the thickness of the fractures and practical spacing of injection holes, it was accepted that it might be necessary to relax this figure to 3 Lugeons, if the grout treatment became more distant (e.g. > 5m) from the plug perimeter. These residual watertightness values are equivalent to those specified beneath large dams founded on rock where it is important to limit seepage.

Assuming effective grout tightening of the rock/concrete plug interface plus grouting of permeable features in the surrounding rock, it is anticipated that the factor of safety against excessive seepage[#] at the plugs is at least 15.

> [#]At South Deep, excessive seepage at a plug site was judged to be 200 litres/minute, or greater. By reference to Garrett & Campbell Pitt (1958), the recommended range of leakage factors of safety, i.e. 10 to 4, provides plug lengths of 6.6m to 16.5m. On the same basis, a 30m-long plug provides a leakage factor of 18.

7 PRECEDENT PRACTICE

Following a study of plug case histories world-wide, the following case is most relevant as it has a similar hydrostatic head to the 58 level plugs at South Deep. The key characteristics are compared below.

<u>Free State Geduld No. 2 Shaft (Head = 1551m)</u>
L = 30.48m W = 14.33m H = 3.35m
Adopted safe shear stress = 690kPa.
Hydraulic gradient = 1551m/30.48m = 51 (after Lancaster, 1964)

According to Leeman (1964), the Geduld No. 2 shaft plug was subjected to a short- term head of up to 2069m.

For comparison, the South Deep permanent plugs have the following characteristics.

<u>South Deep 58 Level</u>
L = 30m W = 4.4m H = 4.0m
Adopted safe shear stress = 524kPa.
Hydraulic gradient = 1500m/30m = 50.

<u>South Deep 50 Level</u>
L = 25m W = 4.4m H = 4.0m
Adopted safe shear stress = 524kPa.
Hydraulic gradient = 1250m/25m = 50.

In regard to long-term in-service conditions, both East Rand Proprietary Mines (ERPM) and Durban Roodepoort Deep (DRD) have mined below water bodies that have accumulated behind plugs in adjacent abandoned mines. In the case of ERPM, mining was below a plug on 58 level, i.e. 771m below sea level whilst the water level in the adjacent Rose Deep mine was 625m above sea level, i.e. 1396m above the plug. Water volume in the Rose Deep compartment and connecting mines is estimated to be 145000 Megalitres, compared with 29580 Megalitres for the Gemsbokfontein West Dolomitic groundwater compartment adjacent to South Deep, when fully impounded (Rison, 2000).

According to the latest available information, DRD and the neighbouring Rand Lease Mine have now reached equilibrium and the plugs are no longer subjected to a differential head of water.

Given adoption of a safe shear stress of 524kPa (c.f. the code value of 830kPa), a structural test load factor of 11.4 may be determined, i.e. [830/524] x 7.2 (Section 2), where a simple uniform shear distribution is assumed.

8 AGGRESSIVITY OF MINE WATER

Where a design life of 100 years is required, there is a need to consider the longevity of the intruded plug installation including the cement grout used to seal the rock/plug interface and the surrounding rock mass. In this regard, the aggressivity of the mine water at the plug sites, the hydraulic gradient of the water flowing past the plug and the permeability of the rock mass were considered.

At 58 level, the groundwater is highly acidic (pH = 1.8 to 2.8). At 50 level, the water is neutral.

To resist the aggressive mine water that could cause dissolution of the cementitious material in the intruded concrete, a low permeability inert bentonite impregnated geotextile sandwich (minimum thickness = 20mm) was specified for the dry face of the reinforced concrete retaining wall fronting all plugs. The bentonite expands on contact with water and the permeability of the seal is very low, i.e. 1×10^{-11} m/s.

In addition, 1000kg of lime [composition = 60% $Ca(OH)_2$ and 40% Na_2CO_3] was required to be deposited on the roadway in front of the plug to neutralise the acidic water in the immediate vicinity of the wet face of the plug.

Although the plugs at 50 level are not subjected to aggressive groundwater, the application of the bentonite geotextile was specified in view of its beneficial low permeability and relatively low cost.

The potential for long term dissolution of the cement grout infilling fractures in the rock mass immediately surrounding the plugs at 58 level was the subject of special study of the mechanisms of grout deterioration in the mine environment by Professor F P Glasser (Emeritus Professor of Inorganic Chemistry, University of Aberdeen, Scotland). Glasser (2002) predicted a conservative performance lifetime of over 4000 years.

9 CONSTITUENT MATERIALS OF MORTAR INTRUDED CONCRETE

9.1 Coarse Aggregate (Plums)

The coarse aggregate comprised bulky angular quartzite in the range 300mm down to 75mm that was both durable and chemically stable. The specified plum sizes reflect considerations related primarily to ease of handling and washing where the plums are finally carried within the plug site and placed manually. The minimum size was 75mm to ensure efficient permeation of the sand/cement mortar through the voids between the plums. For sand/cement mortars, the smallest pre-placed aggregate size can be reduced to 40mm (Littlejohn, 1984)

Rock plums were only permitted to be placed after a double-washing process and their satisfactory condition in situ was confirmed via regular inspections by the supervising engineer. Inadequate cleanliness of plums leads to a deterioration in the bond between the rock plum and the surrounding intruded mortar leading in turn to a reduction in strength and durability of the intruded concrete.

9.2 Fine Aggregate

The fine aggregate was Vaal River sand, supplied in accordance with South African Standard SABS 1083, with a grading of 1.18mm down and no more than 4% passing the 75-micron sieve. Within the 1983 Code, the 4% limit for sand applies to a 150-micron sieve. The finer sand was permitted at South Deep as it was verified to improve pumpability, exhibit low bleed, eliminate segregation and had no detrimental effect on mortar strength.

9.3 Cement

Cement was ordinary Portland cement (OPC) in accordance with SABS EN197.

9.4 Water

The water was potable Rand water (pH = 7.9-8.3).

10 MORTAR MIX DESIGN

To ensure appropriate fluid, stiffening and strength properties, the sand/cement ratio was specified as 1.0 (by weight) with a water/cement ratio not greater than 0.64 (by weight). The specified strength was not less than 25MPa at 28 days and a bleed not greater than 5% at 2 hours (bleed at 2 hours ≡ maximum bleed for mortar at South Deep).

For this mix, the maximum weight of cement per cubic metre of mortar is 752kg. Given a field test voidage of pre-placed coarse aggregate of 53%, the cement content reduces to 398kg per m^3 of intruded concrete. Temperature rises of up to 12^0C per 100kg OPC per m^3 of concrete can develop under adiabatic conditions, so a maximum temperature rise of 48^0C was estimated. The reduced cement content per m^3 reduces the risk of thermal cracking.

11 HIGH POINT INJECTION

Although, the specified maximum bleed for the mortar was low, injection of neat cement grout (water/cement ≤ 0.5) was specified at the high points in the hanging wall, in order to fill the minor pockets or lenses left as a result of residual bleed developed during the final two hours of mortar intrusion.

For high point injections, the locations of the outflow points were determined from observation of the barred and cleaned hanging wall profile during construction. One to three high point injection pipes were required for each segment of intruded concrete.

12 PIPES FOR MORTAR INTRUSION AND GROUTING

Given the passivating concrete environment, the type of embedded steel was not considered important from the point of view of longevity, if not in contact with minewater or the atmosphere. Mortar intrusion pipes comprised 25mm diameter sand blasted seamless tube. High point injection and grout tightening pipes comprised 50mm diameter sand blasted seamless tube. All steel pipes were required to conform to American Standard ASTM A106 Grade B.

Due to the number and locations of the pipes, it was required that plugs would be constructed in four segments, each 7.5m long at 58 level and 6.25m long at 50 level.

In accordance with the 1983 code, the general spacing of the outflow points of intrusion pipes was specified to be not greater than 2m horizontally and 1m vertically. Typically, 60 intrusion pipes were required for each segment.

Similarly, the spacing of outflow points of grout injection pipes was specified such that not more than $7m^2$ of rock/concrete interface would be covered by one pipe with an average of not more than $3.5m^2$ of interface per pipe. Typically, 40 and 32 injection pipes were required for 7.5m and 6.25m long plug segments, respectively, in order to accommodate the maximum haulage dimensions anticipated.

Figures 1 and 2 illustrate the typical mortar and grout injection pipe layouts for 7.5m and 6.25m long segments, respectively.

On completion of mortar intrusion, high point injection and grout tightening, sealing of all pipes and holes was required by filling with a stable neat OPC grout as thick as possible, e.g. water/cement ratio = 0.4 by weight.

13 INSTRUMENTATION

To monitor the development and dissipation of temperature with time and thereby optimise the start date for grout tightening, two thermocouples were specified to be placed within the heart of each plug segment and not closer than 1.5m to the rock perimeter to minimise any heat sink effect.

To monitor pore water pressures within the plug during service, when subjected to a hydrostatic head, two 50mm diameter stainless steel pipes were specified to be installed extending from the dry face to each cold joint (construction joint between concrete segments). Stainless steel pipe was required to conform to American Standard ASTM 316/L.

Each piezometer pipe was required to have a dip (inclination) of 1^0 to the horizontal from the dry to the wet end, in order to facilitate efficient sealing with grout infilling, if required.

14 GROUT FOR ROCK/PLUG INTERFACE AND ROCK MASS

The surrounding rock comprised very strong rock material with a very low permeability (kw < 1 x 10^{-10} m/sec). However, within the rock mass, permeable fractures existed of aperture equal to 0.1mm up to 10mm. Where the fractures were mining induced, e.g. within 1-2m of the plug perimeter, the fracture spacing was approximately 0.5m. For more remote geological fractures, the typical spacing increased to 5-10m.

Given these details, neat OPC grouts were specified for contact grouting at the rock/concrete plug interface and grouting of the fractures in the surrounding rock mass, in order to improve watertightness down to 1 Lugeon. Depending on the thickness of the fracture to be filled, water/cement ratios of 1.0 gradually thickening to 0.4 (by weight) were permitted.

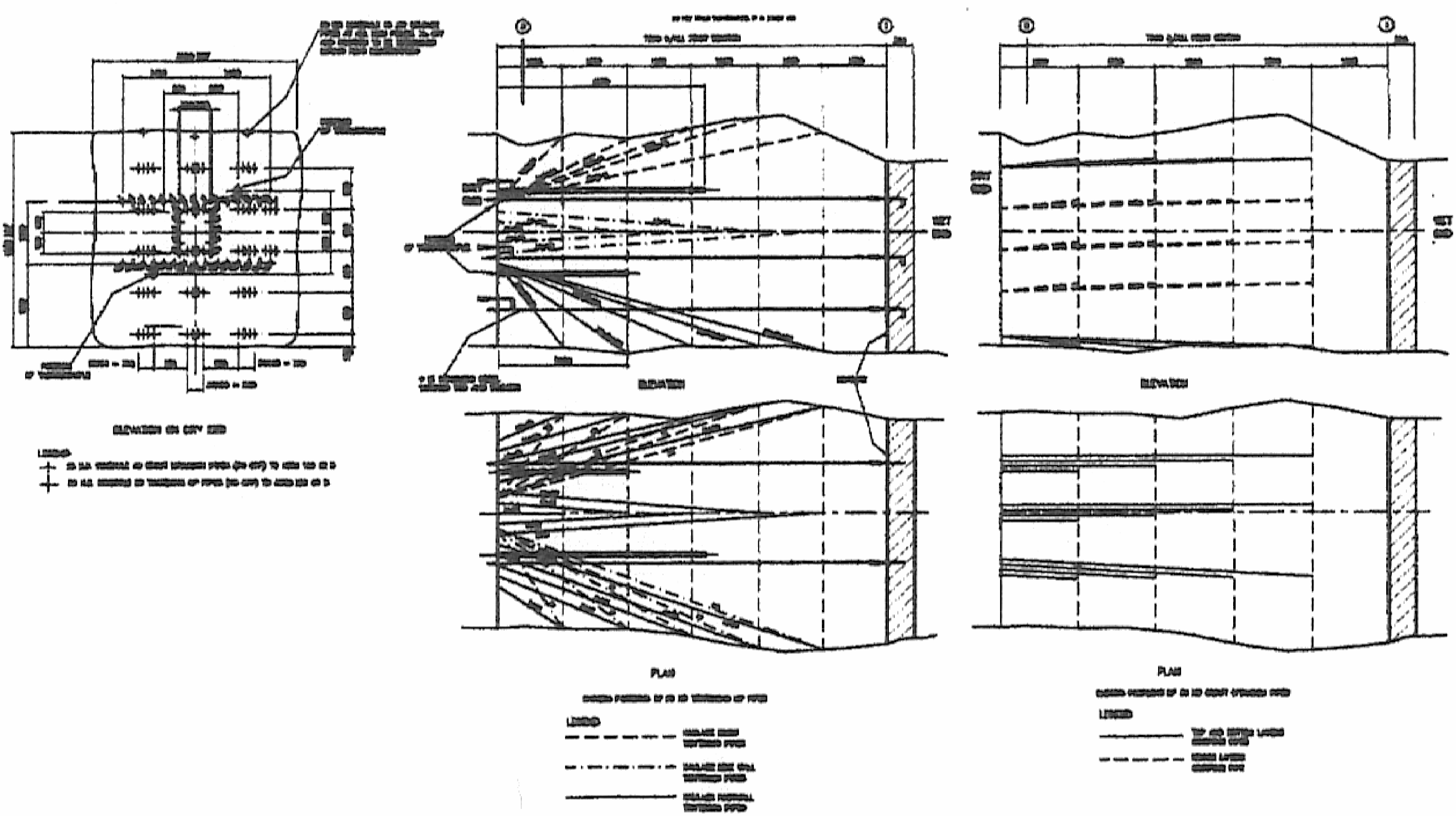

Figure 1 Typical layout of mortar intrusion and grout injection pipes in segment of 58 East Plug

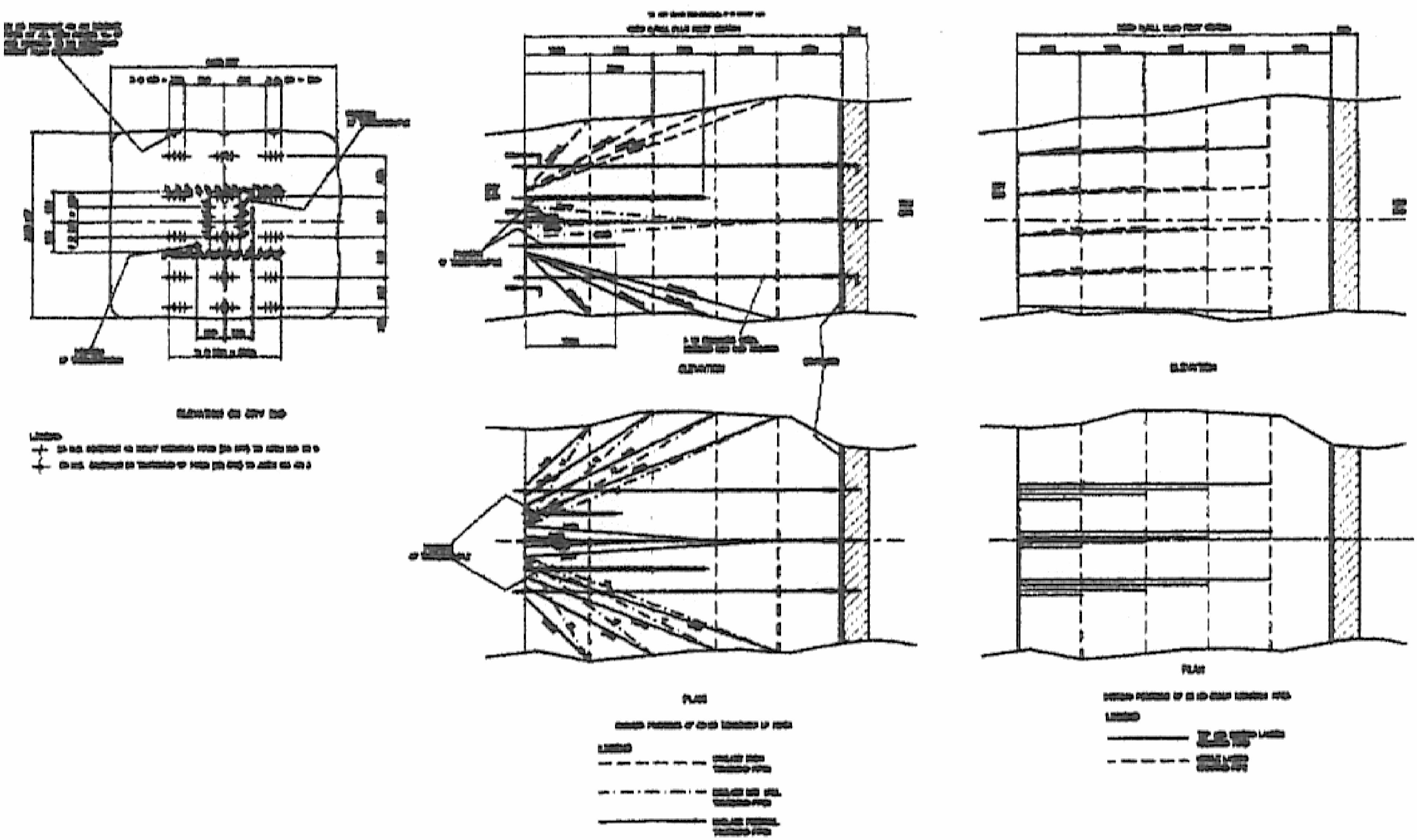

Figure 2 Typical layout of mortar intrusion and grout injection pipes in segment of 50 East Plug

15 PLUG TIGHTENING AND PRE-GROUTING

Where permeable features in the rock mass were located within 2m from the perimeter face of the plug, grouting of the rock was specified to be carried out in 0.5m stages via the inclined grout tightening pipes. High pressures up to 25MPa (250 bars) were permitted for injection, the specification requiring at least 1.5 x design hydrostatic pressure.

Based on the thermocouple results, plug tightening was not permitted to start until 28 days had elapsed after completion of mortar intrusion.

Where permeable features were located greater than 2m from the perimeter face, pre-grouting of these features was required to be carried out in advance of plug construction, as the remote discontinuities could not be grouted cost-effectively via the inclined plug tightening pipes.

The length and location of each hole was specified to be based on an engineering assessment of the local hydrogeology at the plug site. Thereafter, grouting proceeded in 3m stages with the objective of reducing the residual watertightness of the surrounding rock mass to 1 Lugeon, or less.

16 SPECIAL CIRCUMSTANCES

16.1 58 West 1 Plug

For 58 West 1 plug, two 150mm diameter S/S pipes (15.25mm wall thickness with bolted S/S flanges 62mm thick) were incorporated to permit controlled flooding up to 50 level and subsequent dewatering of REL4, in order to commission and carry out a full-scale assessment of the efficiency of this section of the boundary pillar as a water barrier. In cross-section, both pipes were located in a central position but offset 0.6m laterally from the plug centre line and spaced vertically 1m apart to avoid intrusion and injection pipes (Figure 3).

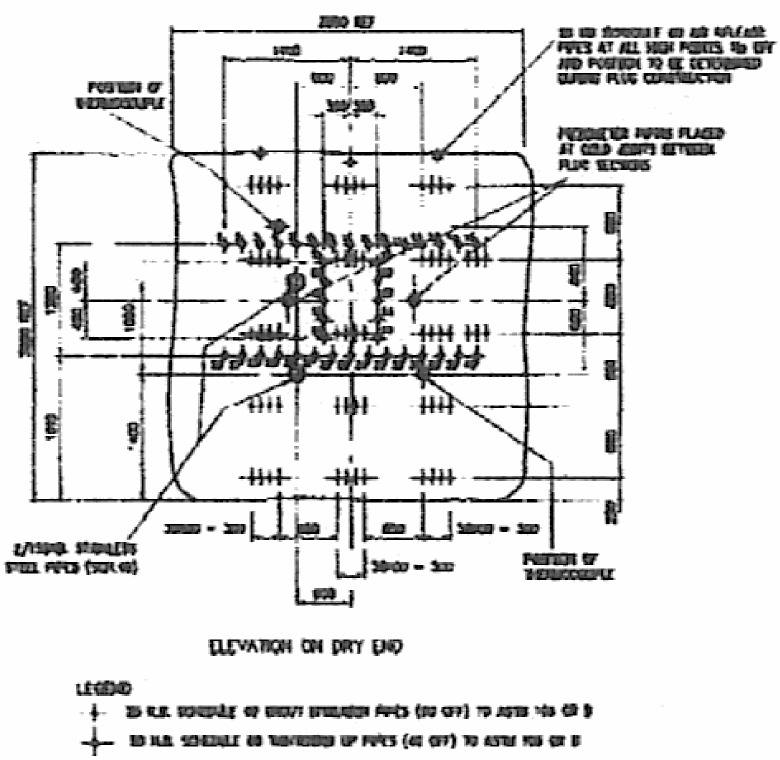

Figure 3 End elevation showing layout of 150mm nominal bore stainless steel pipes in 58 West 1 Plug

All S/S piping, valves and flanges were required to be supplied in material type 316/L in accordance with relevant ASTM and/or ASME Standards. Adequacy of the S/S pipe and flange designs for a head of 1500m was confirmed independently by Walker Ahier Holtzhausen Engineering Consultants CC.

Special tests were also carried out on the S/S pipe with flanges and associated valves to pressures of 53MPa for the 150mm nominal bore piping. These tests confirmed an acceptable load safety factor of 3.5 for the 150mm piping and associated flanges and valves. The S/S pipes and flanges are contained within intruded concrete plugs which adds a further constraint to any potential rupture mechanism by bursting pressures.

As a contingency against valve failure at the downstream end of a plug, a double valve arrangement was fitted. In the unlikely event of failure of two valves at one location, a replacement valve arrangement will be fitted, if practicable (depending on nature of damage), failing which the pipe can be grouted and sealed.

At the wet face, the fitting of 90^0 elbow sections and standpipes was specified to provide a high point for each pipe. This high point permits efficient grout infilling to seal each pipe, if required. To ensure no external preferential seepage path and provide some mechanical interlock, two S/S flanges were specified for each pipe within each plug segment.

16.2 58 West 2 Plug

The wet face of 58 West 2 plug abutted an existing concrete plug. Aside from acting as a passivating sacrificial barrier, no other reliance was placed on this existing plug. Under these circumstances it was not judged necessary to install a bentonite geotextile barrier.

16.3 58 East Plug

The design of the 58 East plug required particular assessment, because the concrete/steel element of an existing water door was located within the fourth segment.

Following study of a typical water door drawing, a solution incorporating the door was adopted, whereby the watertightness and integrity of the concrete surround was first confirmed by 2kg hammer sounding and water/pressure tests via cored holes. It was then specified that the steel lined section must be shot blasted to provide a sound rough surface and fitted with 50mm high steel angle shear connectors at one metre centres around the perimeter, in order to provide some mechanical interlock and eliminate preferred seepage paths. In addition, the steel surface was painted with a thin layer of cement paste to improve bond.

Thereafter, the water door was permitted to remain in place, provided that the existing concrete end was scabbled and the last intruded concrete segment extended 4m beyond the water door. The purpose of the extension was to ensure no preferential interface seepage paths beyond the water door and provide access for routine plug tightening of the rock/concrete plug interface and the surrounding rock mass.

16.4 50 West Plug

For the 50 West plug, eight 200mm nominal bore diameter S/S drainage pipes (12.2mm wall thickness with bolted S/S flanges 62mm thick) were incorporated to permit controlled water inflows from REL4, if required. In cross-section, these pipes were located centrally but spaced 950mm apart and two S/S flanges were specified for each pipe within each plug segment (Figure 4).

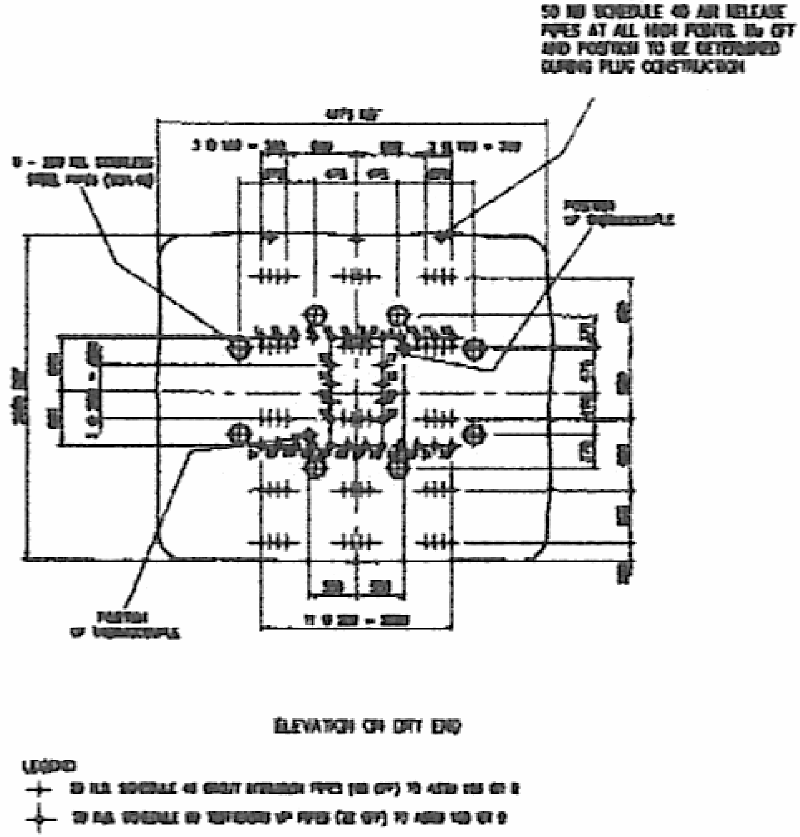

Figure 4 End elevation showing layout of 200mm nominal bore stainless steel pipes in 80 West Plug

The 200mm nominal bore diameter piping with 62mm thick flanges and separately a blank flange (68mm thick) were tested to 40MPa, when the CGI central spiral wound graphite gasket inserts in the pipe arrangements failed, thereby confirming an acceptable load safety factor of 3.2 for a maximum design working pressure of 12.5MPa.

17 SPECIAL TESTS RESULTS FOR PERMANENT PLUGS

17.1 General

In spite of the large number and variety of plugs that have been constructed in South African mines, there have been no significant developments in the design of intruded concrete plugs in rock over a period of 40 years. This is due probably to an absence of failures and an associated absence of in situ tests and instrumented plugs. As a result, there is a dearth of published information on

(i) strength and stiffness of surrounding rock mass,
(ii) strength and stiffness of intruded concrete,
(iii) strength and stiffness of production mortar
(iv) in situ structure, integrity and watertightness of production intruded concrete,
(v) in situ watertightness of rock/plug interface and surrounding rock, after grout tightening,
(vi) in situ contact stress at the rock/plug interface and minimum principal stress in the surrounding rock mass, and

(vii) shear strength of the rock/concrete plug interface.

Until more information is obtained on the in situ properties and service behaviour of intruded concrete plugs under high hydrostatic heads, it is not considered possible to carry out a critical review and advance significantly the current basis of plug design.

As a result of this situation, the international review panel commissioned special tests on some of the permanent plugs. It is considered that the results will assist future designs and at South Deep permit determination of the load safety factor against shear failure, as opposed to the traditional use of the 1983 code design example of a uniform "safe" shear stress of 0.83MPa for Witwatersrand quartzite.

In addition, the engineering parameters obtained can be used to permit more accurate and relevant mathematical modelling of confined intruded concrete plugs when subjected to high water pressures.

17.2 Strength and Stiffness of Witwatersrand Quartzite and Ventersdorp Lava

The results of uniaxial compression tests carried out by CSIR on NXCU cores are summarised in Table 2, where tan E, sec E, tan Poisson's Ratio and sec Poisson's Ratio are evaluated at 30%UCS.

Table 2 Strength and stiffness properties of Witwatersrand quartzite and Ventersdorp lava at the production plug sites

Rock type	Density (Mg/m^3)	UCS (MPa)	Tan E (GPa)	Sec E (GPa)	Tan Poisson's Ratio	Sec Poisson's Ratio
Quartzite	2.70	192	76	71	0.15	0.11
Quartzite	2.67	190	75	79	0.21	0.17
Quartzite	2.69	187	70	65	0.13	0.08
Lava	2.76	189	80	83	0.29	0.28
Lava	2.79	99	85	88	0.29	0.28
Lava	2.80	153	81	85	0.30	0.29

Based on these results a 'high strength' classification may be confirmed for both rock types at South Deep.

17.3 Strength and Stiffness of Intruded Concrete

Concrete core samples (300mm diameter) from a full-scale experimental plug, constructed prior to the permanent plugs, were tested independently by CSIR and 1-year UCS values ranged from 36.3 to 42.8MPa with a Young's modulus of 10.7 to 21.0GPa.

It should be noted that the assumption of an unconfined concrete strength is pessimistic because the intruded concrete that is confined within a strong stiff rock mass will be subject to compression and shear under an applied hydrostatic head.

17.4 Strength and Stiffness of Production Mortar

The unconfined compressive strength of production mortar was confirmed at an average value of 34MPa at 28 days and over 50MPa at 112 days from the routine quality control programme using 100mm cube samples.

The table below indicates further average properties for unconfined 100mm production mortar cubes (Sand/cement = 1.0; water/cement ≤ 0.64 by weight), where tan E, sec E, tan Poisson's Ratio and sec Poisson's Ratio are evaluated at 30%UCS.

Table 3 Average engineering properties for production mortar based on uniaxial compression tests on 100mm samples.

Density (Mg/m^3)	Tan E (GPa)	Sec E (GPa)	Tan Poisson's Ratio	Sec Poisson's Ratio
2.05	18.2	18.6	0.22	0.22

17.5 In situ Structure, Integrity and Watertightness of Intruded Concrete

Close examination of scabbled faces and cores from the intruded concrete indicated a homogeneous structure with excellent bonding of the mortar with the coarse aggregate (plums) and the surrounding rock.

Minor lenses 2-3mm in aperture and 20-30mm long were observed located at the underside of some plums, due to entrapped water or air. These were only occasional features e.g. three over the full plug face of some $17m^2$. These minor defects were not significant as they were not interconnected, as verified by water pressure tests in NXCU cored holes. There was no evidence of thermal cracking and the concrete was sensibly watertight.

17.6 In situ Watertightness of Rock/Plug Interface of Permanent Plugs and Surrounding Rock

Permeability tests were carried out on the 4th segments of three plugs to assess the watertightness of the rock/plug interface and the rock mass immediately surrounding the plugs. In essence, these tests were executed to check the effectiveness of the plug tightening, bearing in mind historical problems of leakages around plugs in service.

Ungrouted fractures were discovered occasionally in the rock surrounding 58 West 2 and water flows at low pressure in the rock surrounding 50 West (Report No. 320717/1 - SRK, 2003), together with water flows at low pressure at the rock/plug interface at 50 West and 58 West 1.

As a result of these test results, the watertightness of the rock/concrete interface around the inner ring of tightening holes in the 4th segment of all production plugs is required to be checked for Lugeon value by water pressure testing (Houlsby, 1976). This work is scheduled for July 2004.

17.7 Hydrofracture Tests to Estimate the In Situ Minimum Principal Stress (σ_3) in the Surrounding Rock Mass and the Normal Stress (σ_n) at the Rock/Plug Interface

Hydrofracturing is a stress measurement technique that uses fluid pressure to create and open fractures in rock. The pressure at which the fluid extends the fracture away from the hole to allow the fluid to penetrate the rock is accepted as being equal to the minimum principal stress in the rock.

Minimum principal stresses in the rock mass of 18-22MPa were recorded at the 50 West plug and 12.4MPa at the 58 East plug. While the former values are consistent with theory, the latter value was judged to be understated due to local fissuring and minor stoping. The theoretical value would be closer to 20MPa.

Hydrofracturing tests at the rock/concrete plug interface were carried out in special holes at the 4th segments of the 58 East and 50 West plugs, in order to determine the normal contact stress (σ_n). These holes were located within the body of the plug to avoid end effects, i.e. not at the dry end where greater relaxation of the rock is permitted.

Five values of the stress normal to the quartzite/plug interface at 58 East ranged from 4.6-6.0MPa. Three values at 50 West ranged from 4-10MPa. Based on these results, a conservative normal stress σ_n of 4MPa was adopted for the plugs at both 58 and 50 levels.

17.8 Shear Strength Parameters of Rock/Plug Interface

Considerable difficulties were encountered in obtaining NXCU cores of the rock/plug interface due primarily to the shallow angle that the coring intersected the interface. As a consequence, only three cores (two in lava at an experimental plug and one in quartzite at 58 West 2) were tested by CSIR in a direction parallel to the main axis of the plugs. The results of the shear tests are summarised below.

The peak (lava/concrete plug) shear strength for an intact contact comprised a cohesion of 0.56MPa and Ø of 32.6^0. The residual shear values for the same contact were zero cohesion and Ør of 32.1^0.

For an "open" lava/concrete plug contact, the equivalent peak shear values were a cohesion 0.12MPa and Ø of 37.1^0. The residual shear values reduced to zero cohesion and Ør of 31.3^0. An "open" contact means that the core sample was already in two pieces (one piece rock and one piece concrete) but brought together prior to testing.

The single "open" quartzite/concrete plug interface provided a zero residual cohesion and Ør of 38.3^0. Some peak values were determined but no sensible straight line could be fitted to them. For a normal stress σ_n of 4MPa, a shear strength of 3.16MPa may be calculated (4MPa x tan38.3^0). Assuming a uniform shear stress distribution, a load safety factor of 6.0 (3.16MPa/0.524MPa) may be estimated for the 58 level plugs. If the 1983 code safe average shear stress of 0.83MPa had been employed, a load safety factor of 3.8 would have been estimated.

If the lowest Ør of 31.3^0 is assumed for the lava/concrete plug interface, then for a normal stress σ_n of 4MPa, a shear strength of 2.43MPa may be calculated (4MPa x tan31.3^0). For a design shear strength of 0.524MPa, a load safety factor of 4.6 (2.43MPa/0.524MPa) may be estimated for the 50 level plugs.

17.9 Summary of Key Engineering Properties

Based on the tests carried out on the experimental and production plugs, the following engineering properties have been established, where tan E and tan Poisson's Ratio are evaluated at 30%UCS.

Table 4 Engineering properties of Ventersdorp lava, Witwatersrand quartzite, intruded concrete, sand/cement mortar and associated rock/concrete plug interfaces

Item	Density (Mg/m^3)	UCS (MPa)	Tan E (GPa)	Tan Poisson's Ratio	Cohesion (MPa)	Angle of internal friction (degrees)
Quartzite	2.67-2.70	187-192	70-76	0.13-0.21	21.9-33.8	50*
Lava	2.76-2.80	99-189	80-85	0.29-0.30	15.5-27.2	47*
Intruded concrete at 1 year	2.08-2.35	36.3-42.8	10.7-21.0	0.20-0.21	11.8-12.6	29.2-53.1
Mortar at 28 days	2.03-2.07	35.7-38.8	16.2-23.5	0.18-0.26	no test	no test
Mortar at 140 days	2.04-2.06	60.2-61.3	14.7-21.2	0.18-0.33	no test	no test
Quartzite /concrete interface (open)	n/a	n/a	n/a	n/a	no result 0.00 residual	no result 38.3 residual
Lava/ concrete interface (intact)	n/a	n/a	n/a	n/a	0.56 peak 0.00 residual	32.6 peak 32.1 residual
Lava/ concrete interface (open)	n/a	n/a	n/a	n/a	0.12 peak 0.00 residual	37.1 peak 31.3 residual

* Estimated using RocLab 1.0

It is considered that these properties may be used for plug design and mathematical modelling at South Deep

18 CONCLUSIONS

While many permanent boundary plugs have been installed in South Africa, it is considered timely to review design practice and in particular to assess the in-service condition and performance of existing plugs. For example, instrumentation (e.g. piezometers), should be incorporated at the construction joints of the plugs to measure internal pore water pressure gradients.

Once a boundary plug has been completed, including grout tightening of the rock/plug interface and surrounding rock mass, it is recommended that high quality coring should be carried out randomly through a completed segment, in order to confirm the watertightness of the plug concrete, rock/plug interface and the surrounding rock after grout tightening. These boreholes must be independent of the grout tightening holes so that the tests can verify the residual watertightness attained in the adjacent rock mass.

The cores at the rock/plug interface should also be tested to determine the shear strength parameters c and Ø.

Hydrofracture tests should be executed at the rock/plug interface and in the surrounding rock mass to determine the in situ contact stress and in situ minor principal stresses more remote from the plug. The modification of these stresses during plug tightening and when the plug is in service warrants investigation.

Such data, combined with shear strength parameters, can permit assessment of the

(i) shear strength of the rock/plug contact and thereby the load factor of safety against shear failure, and
(ii) clamping stress across discontinuities and thereby the risk of their opening during flooding.

The use of these new data will improve mathematical models and permit more relevant sensitivity studies, i.e. where the significance of variations in design parameters and environmental conditions is studied.

Wherever practicable, permanent boundary plugs should be commissioned, i.e. proof tested by controlled flooding, to provide field data on the watertightness of the water barriers incorporating the plugs. This approach would be similar to commissioning a dam by controlled reservoir impounding.

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