ROCK ENGINEERING ASPECTS
OF THE INGULA POWERHOUSE CAVERNS

Mr Gerhard J Keyter
Ms Monique Ridgway
Dr Paul M Varley

Braamhoek Consultants Joint Venture

Summary

The powerhouse caverns at the Ingula Pumped Storage Scheme are to be excavated in relatively weak mudrocks. The geology in the area of the powerhouse proved to be structurally complex given the presence of major dolerite intrusions with associated faulting and bedding parallel shearing. Careful consideration had to be given to aspects such as in situ stresses, mudrock durability, rockmass strength and deformation characteristics and the potential for long term creep affecting the behaviour of the cavern excavations.

Geotechnical investigations and in situ test work undertaken for rock engineering design of the powerhouse caverns of the Ingula Pumped Storage Scheme are described. On the basis of this work, cavern rock support is being designed, comprising the systematic installation of rockbolts, cable anchors and mesh reinforced shotcrete.

1 Introduction

The Ingula Pumped Storage Scheme is located about 23km north east of Van Reenen and straddles the Drakensberg escarpment between the Free State and KwaZulu Natal provinces as shown in Figure 1.

The scheme comprises two reservoirs interconnected by a tunnel system, with reversible pump/turbine units with a total rated generation capacity of 1332MW located in an underground powerhouse complex.

The escarpment edge, at 1770 masl elevation near the surge shafts, separates an elevated plateau from the foothills of the Drakensberg escarpment at lower altitude. The plateau comprises low flat koppies, spurs and rolling grassland, with strong local influence by the near-horizontal orientation of the underlying sedimentary rocks of
the Karoo Supergroup. Sandstone outcrops extensively along the upper reaches of the escarpment.

The high pressure waterways and powerhouse complex are aligned in plan with a prominent mountain ridge with steep slopes that forms part of the escarpment and which extends into KwaZulu Natal. Evidence of large scale landslides of up to 600m wide and of considerable age, are present on some of the steeper slopes in the area. Numerous, more recent, smaller slip scars and debris slides, reflecting movements within the upper few metres of the regolith, also occur on the escarpment slopes.

The area of the tailrace tunnel, located in the foothills of the Drakensberg escarpment, is characterised by rolling hills, small streams and erosion gullies. A massive dolerite sill outcrops at the lower reservoir site and numerous dolerite dykes occur in the area with prominent ridges evident due to preferential weathering of adjacent sedimentary rocks.

The distance between the upper and lower reservoirs is approximately 6km and the difference in elevation is some 470m. The upper reservoir is located on a headwater tributary to the Wilge River, which flows into the Vaal River System. The lower reservoir is located on a headwater tributary to the Klip River, which in turn flows into the Tugela River System. The main caverns of the powerhouse complex lie about halfway between the two reservoirs at a depth of almost 400m below ground level.

The scheme layout is shown in Figure 2 with the underground works comprising:

- An exploratory tunnel, main access tunnel and surge chamber access tunnel.
- Twin low-pressure headrace tunnels from the intake structure at the upper reservoir to twin surge shafts / surge risers near the escarpment edge.
- Twin high-pressure inclined shafts and tunnels leading from the surge shafts / surge risers to the machine hall.
- A transformer hall, connected to the machine hall by 4 busbar galleries and an extension of the main access tunnel.
- A bifurcation access adit, which also connects to anchor galleries.
- Four draft tube tunnels leading to two tailrace surge chambers.
- A dewatering shaft linking the main dewatering sump to a drainage and ventilation adit.
- Ventilation shafts linking the main powerhouse caverns to ground surface via a smoke extraction room.
- A single tailrace tunnel leading from the tailrace surge chambers to the outlet structure on the lower reservoir.
- Other miscellaneous tunnels, galleries and adits at the powerhouse complex.

Construction of the exploratory tunnel by Murray & Roberts (Concor) commenced in May 2005 and was completed in March 2007. Construction of access roads to the site commenced in November 2006 and is due for completion towards the end of 2008. The main access tunnel as well as the first spiral access tunnel off the end of the exploratory tunnel is currently being constructed by CMCM (a joint venture of CMC di Ravenna and P G Mavundla) with work having commenced in April 2007. Tenders for the dams, for the turbines and generators and for the main underground works are currently being evaluated, with the award of these contracts expected during the first half of 2008. Completion of the works and final commissioning of the power station is scheduled for the latter part of 2012.
A cross-section through the main powerhouse caverns is presented in Figure 3. The machine hall has an excavated span of about 26m at crane beam level. The transformer hall has an excavated span of just over 18m at the generator transformer enclosures.
Figure 3: Cross-section through Main Powerhouse Caverns

2 Geology

2.1 Geotechnical Exploration

Initial engineering geological / geotechnical investigations included a study carried out by Partridge Maud and Associates (1999), which involved drilling of 11 boreholes from surface, 4 of which are relevant to the underground works (BH4, BH6, BH7 and BH8) with BH8 a vertical hole near the position of the powerhouse.

The main phase of surface geotechnical exploration for the underground works was completed in 2005 (Braamhoek Consultants Joint Venture (BCJV), 2007) and involved drilling of a further 17 boreholes (BH101 to BH117), including 4 boreholes drilled into the area of the powerhouse complex using drill rigs airlifted onto the mountain ridge in a position directly above the powerhouse. Four additional surface boreholes were drilled along the main access tunnel alignment in 2006 (BH208 to BH211). A further 4 boreholes (ET301 to ET304) were drilled underground from the end of the exploratory tunnel in 2007, into the area of the powerhouse. A short vertical borehole (BH401) was completed in 2008 at the collar position of the vertical air inlet shaft to the underground powerhouse.

Core samples were taken at the drill rig shortly after the core was extracted from the borehole to prevent air slaking of samples and wrapped in plastic cling wrap and tin foil before being waxed into cut-to-length PVC pipes of appropriate size. Samples were then transported by road to rock testing laboratories in Gauteng, South Africa.
2.2 Regional Geology

The powerhouse and tailrace will be constructed in Volksrust Formation mudrocks of the upper Ecca Group, Karoo Supergroup, which is overlain by interbedded siltstone and sandstone horizons of the Normandien Formation belonging to the lower Beaufort Group. These sedimentary units have been intruded by dolerites of the Karoo Dolerite Suite, with the latter occurring in the form of dykes and sills. The sedimentary units are of Middle to Upper Permian age (260-250 myr) while the dolerites belong to the Lower Jurassic (183 myr).

The Volksrust Formation comprises silty, carbonaceous mudstones and siltstones. In the project area, rocks of the Volksrust Formation reach a thickness of almost 200m and include two main dolerite sills. The lower dolerite sill, with a thickness of approximately 40m, is located about 30m above the roof of the machine hall. The upper dolerite sill, with a thickness of some 17m, is found about 8m below the upper contact of the Volksrust Formation. Extended, prominent zones of light greyish brown, almost massive shale (or indurated mudrocks) are often found along the margins of these main dolerite sills.

These mudrocks are typical of many Karoo mudrocks in experiencing rapid disintegration on exposure to air, and oven-dried specimens exhibit free swells on prolonged immersion in water of less than 1.5 to 2% (low potential expansiveness). However, the mudrocks in the upper part of and immediately above the main powerhouse caverns appear less susceptible to such rapid disintegration and swelling, probably as a result of induration effects associated with the 40m thick, dolerite sill immediately above the powerhouse.

The Normandien Formation of the Beaufort Group makes up the bulk of the stratigraphic succession at the project site with almost the entire escarpment face comprising these rocks. The basal unit of the Normandien Formation is the Frankfort Member, which is some 130m thick, including an aggregate of about 35m of dolerite sills and comprises a succession of interbedded siltstone and sandstone layers, including frequent carbonaceous lenses, and often containing abundant mica along bedding planes. The entire succession is horizontally bedded. Above the Frankfort Member is a series of strata dominated by mudrocks, some markedly carbonaceous. About 280m in thickness, this unit contains two thin dolerite sills near its base and numerous horizons of sandstone.

The upper edge of the escarpment is formed by a prominent sandstone horizon known as the Rooinek Member. The sandstone is generally pale grey-white to olive in colour, and mainly comprised of quartz and quartzose rock fragments, with feldspar, a little mica, and a few clay clasts forming the remainder.

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1 The regional geology of the project area as described by Partridge et al (1998, 1999) is summarised here, together with some observations made during subsequent investigations and in mapping of mining excavation faces during construction of the exploratory and construction access tunnels.
These Beaufort Group mudrocks are typical of other Karoo mudrocks in their propensity to disintegrate on exposure to air, with some oven-dried specimens exhibiting free swells on immersion in water of up to 8% (medium to high potential expansiveness).

Numerous narrow dolerite dykes traverse the area. The most prominent direction of long dykes is south-east/north-west. Most vary from about 1 to 8m in width with very few being wider. Larger dolerite bodies invariably take the form of sills, such as the major dolerite sill above the machine hall crown. The fresh dolerite is often highly jointed, especially near its contacts with sedimentary rocks.

A geological long-section for the scheme is presented in Figure 4, which also shows the position of most of the geotechnical exploration boreholes that were drilled.

2.3 Structural Features and Seismicity

Although no known seismically active, structural features are present in the area, the geological structure is complex in terms of smaller features on a project area scale. The Tugela Fault, which in this region follows the boundary between the Kaapvaal Craton and younger rocks of the Namaqua Province, is located some 50km to the south of the scheme as shown in Figure 5.

An extensive Landsat study of tectonic lineaments, undertaken over the whole of the Tugela Basin as part of earlier investigations for the Drakensberg Pumped Storage Scheme (Partridge et al, 1999), did not reveal any major active faults, and the nearest large fault, along which evidence for recent activity has been proposed, is found near Matatiele, considerably further to the south. No photogeological or field evidence of recent fault activity was encountered during this earlier work by Partridge et al (1999). However, within an area extending from the south-eastern Free State to Swaziland, a number of significant seismic events have been recorded by the Council for Geoscience (previously the Geological Survey of South Africa).

A study by Kijko et al (2005) indicates a maximum regional earthquake magnitude in the region of the project site of 6.5 ± 0.38, with an annual probability of occurrence of a maximum possible horizontal peak ground acceleration (PGA) of 0.11g, occurring at a distance of 50km and closer to the site, estimated at approximately 1.3x10^{-5} per annum. The vertical-to-horizontal ground acceleration ratio was conservatively taken as 0.6 (after Ambraseys, 1995). Mean PGA values associated with the 1 in 200, 500, 1000 and 10 000 year return periods are given in Table 1 below.

Table 1: List of Mean PGA Values for Different Return Periods

<table>
<thead>
<tr>
<th>Return Period (years)</th>
<th>PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>0.01</td>
</tr>
<tr>
<td>500</td>
<td>0.018</td>
</tr>
<tr>
<td>1 000</td>
<td>0.025</td>
</tr>
<tr>
<td>10 000</td>
<td>0.075</td>
</tr>
</tbody>
</table>
Figure 4: Geological Long-section for the Scheme
Faults in the project area generally have small displacements (less than 50m) and most trend east-west or east-south-east/west-north-west. Some, near the foot of the escarpment, have downthrows to the south, show a branching pattern and appear to have displacements of up to 20m. Such faulting is inferred to be normal faulting and generally postdates the dolerite intrusions, although there is some evidence to suggest that these may have occurred contemporaneous on occasion.

Two other groups of small-displacement faults strike north west/south east and north east/south west, with normal faults belonging to the latter and dipping towards the powerhouse, intersected in the exploratory tunnel near the powerhouse. Large-scale slope instability of unknown (but geologically fairly recent) age has been associated with locations where faults from these two groups cross the face of the escarpment. It is not clear whether these mass movements have been the result of increased fracturing, weathering and groundwater ingress along fault planes, or of renewed movements along these faults. However, the former is considered more probable and it should be noted that the areas affected by such instability are, at their closest, at least 150m away from the projected surface position of the underground powerhouse. Reverse (or thrust) faulting has also been found in the upper end of the exploratory tunnel, near the upper contact of the main dolerite sill with the overlying Volksrust mudrocks. However, no reverse faulting has been found to date at the powerhouse and measured in situ stresses, other than those found in the main dolerite sill, have given no indication of high horizontal stresses that can be attributed to reverse faulting in the area.

Figure 5: Simplified Structural Geology Map of KwaZulu Natal

http://www.geology.ukzn.ac.za/GEM/kzngeol/kaapvaal.html
Dykes, presumably intruded along tension fractures, and other indeterminate lineaments (mostly joints) are more variably orientated. They conform in large measure to the pattern of lineaments interpreted from Landsat TM imagery (Partridge et al, 1999). Prominent among these are a number of north-south lineations, some of which are evident, in rather generalized form, on aeromagnetic maps. The northward flowing stream occupying the valley that will house the headrace intake structure, follows one of these north-south fractures. The headrace tunnel will be sited well away from this feature, as it is likely to serve as a conduit for substantial groundwater flows.

2.4 Geology of the Powerhouse Complex

The powerhouse complex, including the twin surge chambers, will be located entirely within the mudrocks of the Volksrust Formation. The machine hall crown will be constructed some 30m below a major dolerite sill with a thickness of approximately 40m. The contact zones to this dolerite comprise relatively massive shales (indurated mudstones) of approximately 4m thickness.

An interlayered siltstone / mudstone unit of approximately 12m thickness, with distinctive undulating pale grey bands, is located below the shale unit.

All the excavations at the powerhouse complex will be located within a mudstone unit which is situated below the siltstone / mudstone unit. This mudstone unit is approximately 135m thick.

A thinly laminated sandy mudstone unit underlies the mudstone unit and occurs well below the main drainage gallery invert. The sandy mudstone unit is underlain by a coarse grained sandstone unit.

These main lithological units and the elevations at which they occur, are listed in Table 2 and presented schematically in Figure 6 on a vertical section along the machine hall centre line.

Table 2: Main Lithological Units at the Powerhouse Complex

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Elevation</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>From (masl)</td>
<td>To (masl)</td>
</tr>
<tr>
<td>Dolerite</td>
<td>1284</td>
<td>1240</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Main dolerite sill, with bottom of sill 30m above the machine hall crown.</td>
</tr>
<tr>
<td>Shale</td>
<td>1240</td>
<td>1236</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Contact zone to dolerite sill, indurated mudstone, massive in appearance.</td>
</tr>
<tr>
<td>Siltstone / mudstone</td>
<td>1236</td>
<td>1224</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Coarser grained with distinct undulating pale grey bands.</td>
</tr>
<tr>
<td>Mudstone</td>
<td>1224</td>
<td>1090</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Massive in appearance, very thickly bedded.</td>
</tr>
<tr>
<td>Sandy mudstone</td>
<td>1090</td>
<td>1060</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sandy lenses present, thinly bedded.</td>
</tr>
<tr>
<td>Sandstone</td>
<td>Below 1060</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Coarse grained.</td>
</tr>
</tbody>
</table>
BEDDING SHEARS
FAULT ZONE IN EXPLORATORY TUNNEL
DIP / DIP DIR. : 52° / 342°

8.9 (7.7) = \( \sigma_{\text{min}} \) (\( \sigma_{\text{jack}} \))
4.3 - 4.7 (-) = \( \sigma_{\text{min}} \) (\( \sigma_{\text{jack}} \))

Figure 6: Powerhouse Geology, Vertical Section Along Machine Hall Centre Line
A number of bedding parallel features and faults have been identified within mudrocks in the powerhouse area, through mapping of access tunnel faces as well as from intersections in exploratory borehole core, as follows:

- A number of bedding shears has been identified, typically 15 to 50mm thick, forming continuous shear planes that can be traced across boreholes. As shown in Figure 7, these are generally characterized by an open, smooth or polished, planar joint wall contact on the one side, with a brecciated, soft rock infill comprising calcite stringers in a mylonitic matrix and a rough, undulating joint wall contact on the other side which is often intact. Mapping records show gradual thinning and thickening (pinch and swell) of the infill, with the shears reducing to open bedding planes with very little infill in places. These shears have been observed to roll gently on a macro scale (e.g. by about 0.5m in the plate bearing test adit, over an adit length of 30m) and have been offset elsewhere by faulting.

- Less significant bedding parallel features (bedding veins), typically less than 10mm thick, were also observed in borehole core. Unlike the bedding shears these do not show brecciation, have tightly healed contacts and infill that is mostly calcitic.

- A normal fault comprising a zone of slickensided and striated joints, infilled with calcite and/or mylonitic, healed amorphous calcite, with orientation 52°/342° (dip ranging from 44 to 60°) was found in the exploratory tunnel between chainages 945 and 970m. A bedding shear in this part of the tunnel showed a stepped throw across this fault zone, with a 10m total downthrow on the side of the powerhouse.

- A 2m wide, bedding parallel shear zone comprising thinly laminated, slickensided bedding planes without infill, was encountered between 220.9 and 222.8m depth in borehole ET303.

- In the area of the powerhouse, very fine grained, extremely hard rock stringers, generally in the order of 15cm thick, have been found in borehole core. The margins of these stringers still exhibit sedimentary structure and are considered to be a hornfels alteration product associated with thin basaltic intrusions along bedding planes.

A porphyritic dolerite sill was encountered some 5 to 10m below the invert level of the machine hall in boreholes BH107 and ET304. However, the sill appears to pinch out from the eastern end of the machine hall to the west as it was not encountered in boreholes BH106 and BH108.

A dolerite dyke was mapped on surface near the south western corner of the transformer hall and dips at approximately 80° towards the south-west, i.e. away from the transformer hall.

Other small off-sets in stratigraphic marker horizons, of the order of 5m or less, may be depositional in nature or are possibly due to host rock accommodation of the dolerite sills that pinch out below the machine hall, with associated small scale faulting in the form of part accommodation structures or reactivation of earlier fault structures to accommodate intrusion of these sills.
3 Groundwater

Lugeon testing was carried out in select boreholes in order to study the groundwater characteristics of the rockmass, with low lugeon values generally being recorded in mudrocks of the Volksrust Formation. Standing water levels were also recorded in surface exploration boreholes.

In excavation of the exploratory tunnel, the inflow of groundwater into the excavation was for the most part restricted to that part of the tunnel that is located above the main dolerite sill. Once the tunnel entered the sill, it became effectively dry and remained so for the rest of the tunnel. The main dolerite sill therefore acts as barrier to groundwater recharge to the area of the powerhouse. Nevertheless, groundwater occurrences in the underground works in general, are likely at dolerite contacts, particularly where these coincide with faults.

Jointing in the rockmass below the main dolerite sill is generally not throughgoing and only the mylonitic infill on bedding shears has shown signs of being moist in the area of the powerhouse. No other signs of groundwater seepage were evident on joints or faults encountered in this area when excavating the exploratory and construction access tunnels.
Due to this low permeability of the rockmass, only bedding shears and possibly some of the small fault zones in the area, are expected to play a noteworthy role in the longer term in acting as conduits for recharge to the area as well as in the dissipation of groundwater pressures in the rockmass around the main caverns.

Piezometer readings taken below the operating floor in the machine hall cavern, at approximately 1164 masl in borehole ET304, show a static groundwater head near the top of the main dolerite sill. However, during excavation of the main caverns, it is expected that dilation of the rockmass around the caverns will cause an immediate drop off in groundwater pressures around the excavations and it is considered unlikely that groundwater pressures or the build up thereof will, during construction or in the long term, play a significant role in the stability or otherwise of the excavations, because of:

- The low rate of groundwater recharge to this area.
- The low permeability of the rockmass.
- The powerhouse excavations acting as a large drain towards which groundwater will flow along pathways such as bedding shears, through holes drilled for installation of rock support during construction as well as by way of purposely drilled drainage holes.

4 Intact Rock

Values of uniaxial compressive strength ($\sigma_{ci}$) and Young’s modulus ($E_i$) as derived from laboratory tests on intact core, are presented in Figure 8 in relation to the position of the main powerhouse caverns. Also shown as traces on this Figure are $\sigma_{ci}$ and dynamic modulus values as obtained from wireline geophysics carried out in borehole BH108, as well as rockmass modulus values from Goodman jack testing carried out by Terra Monitoring in borehole ET301.

A generally decreasing trend in strength with depth below the main dolerite sill is evident in Figure 8, which can be attributed to induration effects given the proximity of the main dolerite sill above, however, changes in mudrock composition with depth also play a role. Also apparent in this Figure is the intrusion of the aforementioned porphyritic dolerite sill with associated induration effects in the mudrocks below the invert of the main powerhouse caverns.

The results are summarized in Table 3 in terms of the different lithological units found in the area of the powerhouse.

Significant strength anisotropy has been observed in the mudrocks, both in terms of point load testing carried out in the field as well as during laboratory testing. Based on available laboratory test data, a $\sigma_1$-$\sigma_3$ versus $\beta$ model that takes into account the weaker incipient bedding orientation (with $\sigma_1$ and $\sigma_3$ the major and minor principal stresses respectively and with $\beta$ the angle between $\sigma_1$ and the normal to the bedding plane), yielded Mohr-Coulomb strength parameters for the incipient bedding with a friction angle $\phi$ of 24° and a cohesion $c$ of 5 MPa.
Finally, between 15 and 20% of uniaxial compressive strength tests carried out on intact core specimens from the area of the powerhouse, failed on incipient joints at uniaxial compressive strengths ranging from 30 to 55 MPa. Such results were not included in Figure 8 or in the calculation of mean values as presented in Table 3. However, these incipient features need to be accounted for in ultimately deriving rockmass strengths for the mudrocks at powerhouse level.

5 Bedding Planes

Bedding planes within the mudrock units are generally oriented horizontally, however, some variation in bedding dip is present given smaller scale local deformations as well as the occurrence of cross-bedding.

The shale, siltstone / mudstone and mudstone units are typically thickly to very thickly bedded, while the sandy mudstone unit is thinly bedded. In the shale and siltstone / mudstone units, bedding plane partings are poorly developed, possibly as a result of induration effects due to thermal alteration of these units given their close proximity to the overlying main dolerite sill.
Table 3: Summary of Intact Rock Strengths and Stiffnesses at the Powerhouse

<table>
<thead>
<tr>
<th>Lithological Unit</th>
<th>Uniaxial Compressive Strength, $\sigma_{ci}$ (MPa)</th>
<th>Tangent Modulus at 50% of $\sigma_{ci}$, $E_i$ (GPa)</th>
<th>Poisson’s ratio, $\nu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dolerite</td>
<td>mean 283.9 std dev 59.3 (range 241.7 – 400.5)</td>
<td>mean 89.0 std dev 7.1 (range 78.3 – 100.3)</td>
<td>0.29</td>
</tr>
<tr>
<td>Shale</td>
<td>mean 147.5 std dev 13.4 (range 130.0 – 161.9)</td>
<td>mean 21.1 std dev - (range -)</td>
<td>0.28</td>
</tr>
<tr>
<td>Siltstone / mudstone</td>
<td>mean 134.9 std dev 9.0 (range 118.9 – 145.8)</td>
<td>mean 22.5 std dev 2.5 (range 19.0 – 25.4)</td>
<td>0.19</td>
</tr>
<tr>
<td>Mudstone</td>
<td>mean 89.6 std dev 21.3 (range 40.6 – 122.9)</td>
<td>mean 19.2 std dev 4.5 (range 12.1 – 38.8)</td>
<td>0.28</td>
</tr>
<tr>
<td>- Porphyritic dolerite in mudstone</td>
<td>mean 259.3 std dev 65.2 (range 185.4 – 318.3)</td>
<td>mean 80.2 std dev 8.4 (range 74.3 – 86.2)</td>
<td>0.29</td>
</tr>
<tr>
<td>- Shales in mudstone</td>
<td>mean 95.2 std dev 30.1 (range 45.0 – 129.8)</td>
<td>mean 18.8 std dev 4.2 (range 13.6 – 24.9)</td>
<td>0.19</td>
</tr>
<tr>
<td>Sandy mudstone</td>
<td>mean 54.1 std dev 16.9 (range 27.3 – 89.9)</td>
<td>mean 12.7 std dev 3.8 (range 7.5 – 21.2)</td>
<td>0.28</td>
</tr>
</tbody>
</table>

Note: std dev = standard deviation

Based on the density of recorded discontinuity traces in terms of the total area of exposed faces mapped in the exploratory tunnel in the area of the powerhouse, it is estimated that bedding planes in the mudstone unit are generally spaced very widely (2 to 6m) to extremely widely (> 6m). In the sandy mudstone unit, average bedding plane spacings varied between 1.7 and 3.3m in boreholes ET301 and ET304.

The persistence of bedding planes exposed in excavations completed to date in the area of the powerhouse, typically ranged from low (1 to 3m) to high (10 to 20m). Based on a statistical assessment of available data on bedding plane truncation (or censoring) and persistence as mapped on underground mining faces in this area, a best-fit lognormal distribution was derived for the persistence of bedding planes with a mean and standard deviation of 4 and 8m respectively.

Bedding planes in the mudrock units are generally characterized by smooth to slightly polished, planar surfaces without any infill.

Joint wall strengths of bedding planes were measured in exposures in underground mining faces and on intact N-size core specimens using a Schmidt hammer in accordance with ISRM guidelines (1978). Based on these measurements, an average joint wall strength of 48 MPa was obtained for bedding planes in the mudrocks.
Joint roughness coefficients JRC\(_0\) were measured on bedding planes and yielded a representative JRC\(_0\) value of 3 for a measured range of values of 2 to 4. Considering available mapping data, a typical in-situ block side length of 1.7m was derived.

Using these parameters as input to a Barton-Bandis model (1990), shear strengths for bedding planes in the mudrocks were empirically estimated. Such estimates were first derived on a laboratory scale, for comparison with results obtained during direct shear testing carried out in the laboratory on core samples of bedding planes. The resultant correlation is presented in Table 4 below, with estimates obtained using the Barton-Bandis model slightly higher than the results obtained in the laboratory.

The shear strength of bedding planes at the level of the powerhouse caverns was then estimated, the results are presented in Table 5.

### Table 4: Bedding Plane Shear Strength on a Laboratory Scale

<table>
<thead>
<tr>
<th>Description</th>
<th>Shear Strength, (\tau) (MPa)</th>
<th>Mohr-Coulomb Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Friction Angle, (\phi_j) (degrees)</td>
</tr>
<tr>
<td>Direct shear tests</td>
<td>1.10 (0.3 – 1.6)</td>
<td>24.2 (20.2 – 27.2)</td>
</tr>
<tr>
<td>Barton-Bandis model</td>
<td>1.18</td>
<td>24.2</td>
</tr>
</tbody>
</table>

Note: * Assumed equal to \(\phi_j\)-value derived from laboratory testing, for comparison of equivalent cohesion values.

### Table 5: In-Situ Bedding Plane Shear Strength at Cavern Level

<table>
<thead>
<tr>
<th>Description</th>
<th>Shear Strength, (\tau) (MPa)</th>
<th>Mohr-Coulomb Parameters*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barton-Bandis model</td>
<td>5.19</td>
<td>27.9</td>
</tr>
</tbody>
</table>

Note: * Equivalent Mohr-Coulomb parameters derived from tangent to Barton-Bandis model for a normal stress \(\sigma_n = 9.36\) MPa.

Furthermore, it was found that the Barton-Bandis model for bedding planes can be adequately approximated using a bi-linear Mohr-Coulomb model as follows:

- for \(\sigma_n \geq 1\) MPa: friction angle \(\phi_j = 28.5^\circ\) and cohesion \(c_j = 0.12\) MPa
- for \(\sigma_n < 1\) MPa: friction angle \(\phi_j = 31.0^\circ\) and cohesion \(c_j = 0.01\) MPa

A residual strength with friction angle \(\phi_r = 24.2^\circ\) and cohesion \(c_r = 0\) MPa was obtained from direct shear tests on bedding planes.

Values of initial shear stiffness \(k_{si}\) as determined from direct shear tests on bedding planes ranged from 2990 to 9990 MPa/m with a mean value of 6630 MPa/m, and can be compared to empirical estimates of initial shear stiffness obtained for bedding planes in the area of the powerhouse of 1340 MPa/m (after Barton, 1972) and of 7910 MPa/m with a failure ratio \(R_{ij}\) of 0.72 (after Kulhawy, 1975).
Values of normal stiffness $k_n$ as determined from direct shear tests on bedding planes ranged from 4100 to 11900 MPa/m with a mean value of 7650 MPa/m. This can be compared to empirical estimates of bedding plane normal stiffness obtained for this project after Barton (1972) and Bandis et al (1983) of 3550 and 3950 MPa/m respectively, with a maximum joint closure $V_m$ of 0.13mm and normal stiffness $k_n$ of 9750 MPa/m obtained from linear regression analysis of $\Delta V_j/\sigma_n$ versus $\Delta V_j$ plots of available laboratory data, with $\Delta V_j$ the measured joint closure as a function of normal stress $\sigma_n$.

It was furthermore noted that for any particular direct shear test, the normal stiffness $k_n$ remained more or less constant for different levels of normal stress $\sigma_n$ as applied during the test.

6 Jointing in Mudrocks

Joint orientations from face mapping carried out at in the last 70m of the exploratory tunnel (excluding any data on bedding) is presented on the stereonet in Figure 9, on the basis of which a number of joint sets (J1 to J8) has been defined as shown. The orientation of the long axes of the main powerhouse caverns is also shown in the same Figure.

The joint sets defined above only account for approximately 60% of the joints mapped in this area, with a significant number of randomly oriented joints evident.
As for bedding planes, an estimate of the spacing of joints was derived from the density of recorded discontinuity traces in terms of the total area of exposed faces mapped in the exploratory tunnel in the powerhouse area and it was found that joint spacings within the respective joint sets generally varied from very widely (2 to 6m) to extremely widely (>6m). Given such widely spaced joints, it follows that, during construction of the exploratory tunnel in the powerhouse area, generally only joints from a few of the above joint sets, together with bedding, were evident on a tunnel face being excavated.

The persistence of joints exposed in excavations in the powerhouse area typically ranged from low (1 to 3m) to medium (3 to 10m). Based on a statistical assessment of available data on joint truncation (or censoring) and persistence as mapped in this area, a best-fit lognormal distribution was derived for the persistence of joints with a mean and standard deviation of 2m.

Joints in the mudrocks are generally characterized by smooth, planar to slickensided, undulating joint surfaces without any infill. An average joint wall strength of 38 MPa was obtained for joints in the mudrocks by means of a Schmidt hammer used in accordance with ISRM guidelines (1978). Joint roughness coefficients JRC were measured on joints and yielded a representative JRC value of 4 for a measured range of values of 2 to 4. Considering available mapping data, a typical in-situ block side length of 1.7m was derived.

Shear strengths of joints in the mudrocks were empirically estimated using the Barton-Bandis model (1990). Such estimates were again first derived on a laboratory scale, for comparison with results obtained during direct shear testing carried out in the laboratory on core samples of joints. The resultant correlation is presented in Table 6 below, with estimates obtained using the Barton-Bandis model slightly higher than the results obtained in the laboratory.

The shear strength of joints at the level of the powerhouse was then estimated, the results presented in Table 7.

**Table 6: Joint Shear Strength on a Laboratory Scale**

<table>
<thead>
<tr>
<th>Description</th>
<th>Shear Strength, $\tau$ (MPa)</th>
<th>Mohr-Coulomb Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Friction Angle, $\phi_j$ (degrees)</td>
</tr>
<tr>
<td>Direct shear tests</td>
<td>0.94 (0.6 – 1.4)</td>
<td>23.1 (21.0 – 26.1)</td>
</tr>
<tr>
<td>typical (range)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Barton-Bandis model</td>
<td>1.13</td>
<td>23.1*</td>
</tr>
</tbody>
</table>

Note: * Assumed equal to $\phi_j$-value derived from laboratory testing, for comparison of equivalent cohesion values.
Table 7: Joint Shear Strength at Cavern Level

<table>
<thead>
<tr>
<th>Description</th>
<th>Shear Strength, τ (MPa)</th>
<th>Mohr-Coulomb Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Friction Angle, φ_j (degrees)</td>
</tr>
<tr>
<td>Barton-Bandis model</td>
<td>4.66</td>
<td>22.9</td>
</tr>
</tbody>
</table>

Note: * Equivalent Mohr-Coulomb parameters derived from tangent to Barton-Bandis model for σ_n = 10.3 MPa.

The above Barton-Bandis model for joints can again be approximated using a bi-linear Mohr-Coulomb model as follows:
- for σ_n ≥ 1 MPa: friction angle φ_j = 23.6° and cohesion c_j = 0.1 MPa
- for σ_n < 1 MPa: friction angle φ_j = 27.0° and cohesion c_j = 0.01 MPa

A residual strength with friction angle φ_r = 22.2° and cohesion c_r = 0 MPa was obtained from direct shear tests on joints.

Values of initial shear stiffness k_{si} as determined from direct shear tests on joints ranged from 5630 to 7910 MPa/m with a mean value of 6780 MPa/m, can be compared to empirical estimates of initial shear stiffness obtained for joints in the powerhouse area of 9230 MPa/m with a failure ratio R_{fj} of 0.73 (after Kulhawy, 1975).

Normal stiffness k_{n} values obtained from direct shear tests on joints ranged from 4730 to 9830 MPa/m with a mean value of 6020 MPa/m. An empirical estimate of joint normal stiffness of 2410 MPa/m was obtained for this project after Bandis et al (1983). Linear regression analysis of ΔV_j/σ_n versus ΔV_j plots of available laboratory data suggested a maximum joint closure V_m of 0.43mm and a normal stiffness k_{n} of 6490 MPa/m.

It was generally noted that for any particular direct shear test, the normal stiffness k_{n} remained more or less constant for normal stresses σ_n above 0.5 MPa as applied during the test.

7 Jointing in Dolerite

Joint orientations from face mapping in the part of the exploratory tunnel located within the main dolerite sill and from wireline geophysical measurements in boreholes BH107 and BH108 in the same dolerite sill, are presented on the stereonet in Figure 10. Four distinct joint sets have been defined for the dolerite sill as shown with some random joints also evident.

The jointing in the main dolerite sill is generally sub-vertical but with some random joints dipping at shallower angles. The strike of joint sets in the main dolerite sill correlates reasonably well with that of dykes and faults identified in the area.
Joints in the main dolerite sill are typically stained black, rough, undulating to planar surfaces, with some calcitic infill, which frequently has recemented the joint. Joint spacings in the main dolerite sill varied from closely (0.06 to 0.2m) to moderately jointed (0.2 to 0.6m) in general, to widely jointed (0.6 to 2m) in places.

Mohr-Coulomb strength parameters for joints in the dolerite with friction $\phi_j$ of 36° and cohesion $c_j$ of 0.61 MPa and with residual friction $\phi_r$ of 30° and cohesion $c_r$ of 0 MPa, have been used in the design.

8 Faulting and Bedding Shears

No specific information is available on the shear strength of the fault zone encountered at the lower end of the exploratory tunnel. In the absence of actual test data, design shear strength values for slickensided joints with $\phi_j = 15°$ and $c_j = 0$ MPa have been advised (Stacey et al, 1986). However, given the generally calcitic nature of the infill on joints comprising the fault zone, higher strength values with $\phi_j$ in the order of 20 to 25° and a $c_j$ of 0 MPa were considered more appropriate in characterising this fault zone. The normal stiffness $k_n$ and shear stiffness $k_s$ of these slickensided joints have been estimated at 4600 and 3750 MPa/m respectively, which are only slightly lower than that of bedding planes and joints in the mudrocks.

Direct shear testing of three 75mm diameter core samples, drilled through one of the bedding shears at chainage 900m in the exploratory tunnel, using a triple tube core barrel to minimize disturbance of the sample, yielded peak strengths with a friction angle $\phi_j$ of 27 ± 3° and a cohesion $c_j$ of 0.09 ± 0.05 MPa and residual strengths with $\phi_j$ of 23 ± 3° and $c_j$ of 0 MPa. All three specimens tested were described as having a JRC$_0$ in the range 1 to 3, with less than 1mm of light grey, silty infill material, i.e. the drillhole intersected the bedding shear where the infill has largely pinched out.
Considering the bedding shear infill as a Hoek-Brown material (Hoek et al, 2002), for a $\sigma_{ci} = 5$ MPa, an intact rock parameter $m_i = 19$ (for medium grained breccia) and a Geological Strength Index GSI = 10 (disintegrated, poorly interlocked, heavily broken rockmass), a friction angle $\phi_j$ of 17° and a cohesion $c_j$ of 0.27 MPa, is obtained for a 360m overburden as at machine hall crown level.

In comparison, Barton (1974) reported peak strengths of $\phi_j = 16^\circ$ and $c_j = 0.012$ MPa and residual strengths of $\phi_j = 11$ to 11.5° and $c_j = 0$ MPa for clay mylonite seams of 10 to 25mm thickness in coal measure rocks. Also, Hoek et al (1998) reported shear strengths of $\phi_j = 17 \pm 2^\circ$ and $c_j = 0.04 \pm 0.03$ MPa for foliated, laminated black shales encountered in underground excavations with shallow overburden, constructed as part of the Athens Metro Project.

Given the pinch and swell nature of the infill on these bedding shears, a shear strength halfway between that found in the laboratory and that reported by Barton (1974) and Hoek et al (1998) was used for design purposes, with a worst case strength considered similar to that reported by these authors.

The stiffness of the infill on these bedding shears was estimated empirically after Hoek et al (1998), using Equations 1 to 3, which suggested values of normal stiffness $k_n$ and shear stiffness $k_s$ of 4330 and 1670 MPa/m respectively.

Eq. 1: $E_0 = \sqrt{\frac{\sigma_{ci}}{100} \cdot 10^{\left(\frac{GSI-10}{40}\right)}}$ (in GPa), with $E_0$ the Young’s modulus of the infill.

Eq. 2: $k_n = \frac{E_0}{h}$ (in GPa/m), with infill thickness $h$ (in m).

Eq. 3: $k_s = \frac{G_0}{h}$ in (GPa/m), with infill shear modulus $G_0 = \frac{E_0}{2 \cdot (1 + \nu)}$ (in GPa)

No specific information is available for the calcite filled, bedding parallel veins encountered in boreholes. Peak frictional strengths of up to 36° have been reported for finely crushed calcitic fault gouge (USGS, 2007). Adopting a Hoek-Brown type approach, the peak strength of such an intact, healed calcite vein may be as high as $\phi_j = 43^\circ$ and $c_j = 3$ MPa, which is of the same order as that of the incipient bedding. The stiffness of these veins, both in shear and compression, was considered to be similar to that of the intact rock.

9 Rockmass Classifications

Rock mass classifications were carried out for the powerhouse area using NGI-Q (Barton et al, 1974; Barton et al, 1994; Barton, 2002) and rock mass ratings RMR$_{76}$ (Bieniawski, 1976) and RMR$_{89}$ (Bieniawski, 1989). It should be noted that ratings for the mudstone unit have not been downrated for the presence of bedding shears and faults as these structural features were modelled in both continuum and discontinuum type numerical models as discrete planes as and where encountered.
A summary of NGI-Q-ratings derived for the various lithological units is presented in Table 8.

A summary of RMR\textsubscript{76} values derived for the various lithological units is presented in Table 9, with empirical correlations using the above NGI Q-values in Equation 4 (Bieniawski, 1976) also shown.

Eq. 4: \[ RMR_{76} = 9 \cdot \ln(Q) + 44 \]

A summary of RMR\textsubscript{89} values derived for the various lithological units is similarly presented in Table 10.

Table 8: Summary of NGI-Q Ratings

<table>
<thead>
<tr>
<th>Lithological Unit</th>
<th>RQD (%)</th>
<th>J\textsubscript{n}</th>
<th>J\textsubscript{r}</th>
<th>J\textsubscript{a}</th>
<th>J\textsubscript{w}</th>
<th>SRF</th>
<th>Q typical (range)</th>
<th>Rock Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dolerite</td>
<td>80</td>
<td>6-12</td>
<td>1.5-3</td>
<td>2-4</td>
<td>1</td>
<td>1</td>
<td>8.4 (2.5-20)</td>
<td>fair</td>
</tr>
<tr>
<td>Shale</td>
<td>100</td>
<td>3</td>
<td>1-1.5</td>
<td>1-2</td>
<td>1</td>
<td>1</td>
<td>31.3 (16.7-50)</td>
<td>good (good-very good)</td>
</tr>
<tr>
<td>Mudstone/siltstone</td>
<td>98.5</td>
<td>3</td>
<td>1-1.5</td>
<td>1-2</td>
<td>1</td>
<td>2.5</td>
<td>12.3 (6.6-19.7)</td>
<td>good (fair-good)</td>
</tr>
<tr>
<td>Mudstone</td>
<td>95</td>
<td>3-6</td>
<td>1-1.5</td>
<td>1-2</td>
<td>1</td>
<td>2.5</td>
<td>8.9 (3.2-19)</td>
<td>fair (poor-good)</td>
</tr>
<tr>
<td>Sandy mudstone</td>
<td>95</td>
<td>6-9</td>
<td>1-1.5</td>
<td>1-2</td>
<td>1</td>
<td>5</td>
<td>4.5 (1.6-9.5)</td>
<td>fair (poor-fair)</td>
</tr>
</tbody>
</table>

Table 9: Summary of RMR\textsubscript{76} Classification

<table>
<thead>
<tr>
<th>Lithological Unit</th>
<th>C\textsubscript{ci}</th>
<th>RQD</th>
<th>Joint Spacing</th>
<th>Joint Condition</th>
<th>Groundwater</th>
<th>Joint Orientation</th>
<th>RMR\textsubscript{76}</th>
<th>Rock Mass Class</th>
<th>RMR\textsubscript{76} from Equation 4 average (range)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dolerite</td>
<td>15</td>
<td>17</td>
<td>18</td>
<td>20</td>
<td>10</td>
<td>-12</td>
<td>70</td>
<td>good</td>
<td>63 (52-71)</td>
</tr>
<tr>
<td>Shale</td>
<td>12</td>
<td>20</td>
<td>30</td>
<td>12</td>
<td>10</td>
<td>-10</td>
<td>74</td>
<td>good</td>
<td>75 (69-79)</td>
</tr>
<tr>
<td>Mudstone/siltstone</td>
<td>12</td>
<td>20</td>
<td>25</td>
<td>12</td>
<td>10</td>
<td>-10</td>
<td>74</td>
<td>good</td>
<td>67 (61-71)</td>
</tr>
<tr>
<td>Mudstone</td>
<td>7</td>
<td>20</td>
<td>25</td>
<td>12</td>
<td>10</td>
<td>-10</td>
<td>69</td>
<td>good</td>
<td>64 (54-70)</td>
</tr>
<tr>
<td>Sandy mudstone</td>
<td>7</td>
<td>20</td>
<td>20</td>
<td>12</td>
<td>10</td>
<td>-10</td>
<td>64</td>
<td>good</td>
<td>57 (48-64)</td>
</tr>
</tbody>
</table>
Table 10: Summary of RMR\textsubscript{89} Classifications

<table>
<thead>
<tr>
<th>Lithological Unit</th>
<th>σ\textsubscript{ci}</th>
<th>RQD</th>
<th>Joint Spacing</th>
<th>Joint Condition</th>
<th>Groundwater</th>
<th>Joint Orientation</th>
<th>RMR\textsubscript{89}</th>
<th>Rock Mass Class</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dolerite</td>
<td>15</td>
<td>17</td>
<td>11</td>
<td>25</td>
<td>15</td>
<td>-12</td>
<td>71 (50 – 70)</td>
<td>good</td>
</tr>
<tr>
<td>Shale</td>
<td>12</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>15</td>
<td>-5</td>
<td>82 (75 – 85)</td>
<td>good to very good</td>
</tr>
<tr>
<td>Mudstone / siltstone</td>
<td>12</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>15</td>
<td>-5</td>
<td>82 (75 – 85)</td>
<td>good to very good</td>
</tr>
<tr>
<td>Mudstone</td>
<td>7</td>
<td>20</td>
<td>20</td>
<td>20</td>
<td>15</td>
<td>-5</td>
<td>77 (60 – 85)</td>
<td>good</td>
</tr>
<tr>
<td>Sandy mudstone</td>
<td>7</td>
<td>20</td>
<td>15</td>
<td>20</td>
<td>15</td>
<td>-5</td>
<td>72 (55 – 75)</td>
<td>good</td>
</tr>
</tbody>
</table>

10 Rockmass Modulus

10.1 Empirical Estimates

The empirical approach developed by Hoek et al (2006) was used to derive values of rockmass modulus for the mudstone unit for a GSI ranging from 60 to 85.

For an undisturbed rockmass (i.e. with a blast disturbance factor D = 0), this approach produced rockmass modulus values ranging from 9.9 to 17.6 GPa. For the blast damaged zone around the excavation perimeter, using a D of 0.8, rockmass modulus values of between 3.2 and 9.1 GPa were calculated.

10.2 Goodman Jack Testing

In-situ (or true) modulus of deformation values (Heuze, 1984; Heuze et al, 1985), calculated from the results of Goodman Jack testing that was carried out by Terra Monitoring in borehole ET301 in accordance with ISRM (1996), are superimposed on static modulus values in Figure 8.

Between 1155 and 1200 masl (i.e. over the height of the powerhouse), the Goodman Jack yielded an average in-situ modulus of deformation for the rockmass of 15.8 GPa, which is approximately 80% of that derived from laboratory testing of intact rock core for the mudstone unit. The higher in-situ modulus of deformation values derived from Goodman Jack testing between 1130 and 1155 masl in borehole ET301, can be ascribed to the presence of the aforementioned porphyritic dolerite sill, which was found at this elevation in the area of borehole ET301.
10.3 Wireline Geophysics

In Figure 8, dynamic modulus of elasticity values as derived from down-hole wireline geophysics measurements in borehole BH108, are also superimposed on static modulus values derived from laboratory testing of intact rock core and Goodman Jack test work in borehole ET301.

Using down-hole geophysics, an average dynamic modulus of elasticity of approximately 32.4 GPa was obtained in the mudstone unit, giving a ratio of about 0.6 between static modulus values as derived from laboratory testing and dynamic modulus from down-hole geophysics. When compared to true modulus values derived from Goodman Jack test work, this ratio reduces to approximately 0.5.

10.4 Plate Bearing Testing

Four plate bearing tests, two set up horizontally and two set up vertically, are being undertaken by Solexperts at machine hall crown level in a test adit off one of the construction access tunnels. Each test comprises five cycles of equal loading increment to a maximum load of 12 MPa, with loads applied using hydraulic jacks and 400mm diameter, stiff plates acting on opposite walls of the adit, in accordance with ISRM (1981) and Ünal (1997).

Provisional results have been received for the two horizontal test setups, with work on the vertically arranged tests still in progress. From these provisional results, modulus values from initial loading stages ranged from approximately 1 GPa up to 10 GPa. However, seismic refraction work in the test adit has indicated a primary zone of blast damage around the adit which is between 0.1 and 0.4m deep and a secondary zone affected by blasting up to about 0.8 to 1m depth, with low modulus values from initial loading attributable to resultant bedding in of the test plates. Modulus values ranging from 5 to 21 GPa during reloading cycles and from 6 to 19 GPa during unloading cycles, are considered more representative of that of the undisturbed rockmass and correlate better with that obtained from empirical estimates and Goodman Jack test work as described above.

Also, from previous plate bearing test work carried out on Karoo mudrocks, higher rockmass modulus values were generally obtained from vertical test setups compared to that from horizontal test setups. A conclusion on the results from the plate bearing test work therefore remains to be finalised, pending the outcome of the vertical tests still to be completed.

10.5 Back Analysis from Convergence Monitoring Data

Convergence monitoring data from an array of multiple point borehole extensometers installed in the construction access tunnel at crown level at the eastern end of the machine hall, was back-analysed in order to obtain an estimate of rockmass modulus for the mudstone unit.
Two bedding shears were present in the face at the instrumented tunnel section, one at about 0.5m below the tunnel crown and the other just above the tunnel invert. An open bedding plane was also visible at about spring line level. The instrumented cross-section was modelled in Phase2, with the shears and bedding plane modelled as discrete planes, and with parameters assigned to these features similar to that presented earlier in this paper.

The rockmass between shears and bedding planes was modelled as a Hoek-Brown material using mean values for $\sigma_{ci}$ and $E_i$ as presented in Table 3. A blast damaged zone around the tunnel perimeter with a depth of 0.5m and $D = 0.8$, was included in the Phase2 model.

These analyses indicated that a GSI of 60 to 70, with rockmass modulus values in the order of 10 to 14 GPa, is required to emulate convergence monitoring data from the construction access tunnel.

11 Rockmass Strength

Hoek et al (2005) notes that the rockmass should be treated as intact for massive units of sandstones or siltstones where no significant bedding planes or discontinuities are present. The rockmass surrounding the Ingula powerhouse caverns clearly does not fall in this category, given the discussion above on the presence of bedding shears, small-scale faulting, bedding planes and joints, as well as the geotechnical character of these features.

Hoek et al (2005) furthermore presents GSI charts for rockmasses in which bedding planes or discontinuities are present, which can be used for estimating the strength of such rockmasses in accordance with the Hoek-Brown criterion. The first of these GSI charts (Hoek et al (2005), Figure 9) was developed for confined molasse and mainly for the field of tunnelling, with a suggested GSI range of between 55 and 95 for such tectonically undisturbed sedimentary rocks. The second GSI chart developed (Hoek et al (2005), Figure 10) is for fissile molasse and is to be used primarily for surface excavations, with a suggested GSI range of between 25 and 65, depending on the composition and structure of the rockmass on the one hand and surface condition of rockmass discontinuities.

In continuum type models such as Phase2 (where bedding shears and bedding planes were modelled as discrete planes) or FLAC (where a ubiquitous model was used to model regular bedding planes as well as bedding shears), GSI values in the range 60 to 75 were considered appropriate for characterizing the mudstone unit between these bedding shears and regular bedding planes, bearing in mind the following:

• The results of the back-analyses presented above.
• Evidence of tectonic disturbance of the rockmass (which is not of recent origin), albeit with a low level of deformation, as found during site investigations.
• The rockmass around the Ingula caverns, specifically in the machine hall crown and sidewalls, will be significantly less confined than what would be the case in a tunnel in the same rockmass.

Table 11 summarises the range of Hoek-Brown parameters used in deriving rockmass strength parameters $m$, $s$ and $a$ for the different lithological units, with values of $\sigma_{ci}$ and $E_i$ as listed in Table 3.
Table 11: Hoek-Brown Rockmass Strengths

<table>
<thead>
<tr>
<th>Rock Type</th>
<th>Input Parameters</th>
<th>Hoek-Brown rockmass parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>GSI</td>
</tr>
<tr>
<td>Dolerite typical</td>
<td>78</td>
<td>16</td>
</tr>
<tr>
<td>Shale</td>
<td>85</td>
<td>7</td>
</tr>
<tr>
<td>Siltstone / mudstone typical</td>
<td>85</td>
<td>7</td>
</tr>
<tr>
<td>Mudstone typical (range)</td>
<td>70</td>
<td>7 (60-75)</td>
</tr>
<tr>
<td>Sandy mudstone</td>
<td>60</td>
<td>7</td>
</tr>
<tr>
<td>Sandstone typical</td>
<td>60</td>
<td>17</td>
</tr>
</tbody>
</table>

In models developed using the distinct element code UDEC, a higher GSI value was adopted throughout for the Karoo mudrocks below the main dolerite sill as well as in the main sill itself, given that discontinuity sets were modelled as sets of discrete planes in such models. GSI values therefore adopted for ‘intact’ blocks in UDEC were determined by which joint sets have been excluded from the UDEC model as well as the need to account for random jointing and incipient joints in establishing a representative rockmass strength.

12 In Situ Stress

12.1 General

In situ and laboratory test work at the Drakensberg Pumped Storage Scheme had determined a horizontal:vertical in situ stress ratio of 2.5 at the powerhouse complex (Hoek, 2007) and it was initially assumed that this would also apply at the Ingula project. Initial stress measurements (Golder, 2006) were made by hydrofracture in five vertical boreholes, at three lithologically and topographically different sites, during the main site investigation for the Ingula scheme, as follows:

- Boreholes BH115 and BH116 are above the escarpment slope. The topography is relatively gentle here. Borehole BH115 is located at the surge shafts. The escarpment slope has a series of prominent sandstone beds from ground surface to approximately 200m depth.
- Boreholes BH106 and BH108 are on the escarpment slope, immediately above the powerhouse complex. Streams have cut into the slope and may have an influence on the ground stresses. Borehole BH106 is located close to a shear zone, which was intersected in borehole BH105 (Figure 4).
- BH117 is in the tailrace area, below the escarpment slope. The topography is relatively gentle here. Prominent dolerite sills occur below the escarpment and are present above and below the powerhouse complex.

The hydrofracture tests showed high minimum stresses ($\sigma_{\text{min}}$) in the dolerite sills, but much lower minimum:vertical in situ stress ratios ($\sigma_{\text{min}}/\sigma_v$) in the sedimentary rocks than had
been anticipated. The mean $\sigma_{\text{min}}/\sigma_v$ was 1.0 for the data as a whole, but the range was 0.4 to 4.4: the higher ratios being in the dolerite.

The vertical ground stress is assumed to derive from the unit weight of the rockmass and the depth to the test point, and is not measured during hydrofracture testing. The intermediate stress is estimated from the tensile strength of the rock material and/or the test parameters. This is an uncertain value and at Ingula, the two equations used to estimate the intermediate stress gave significantly different values.

An initial review of the data as a whole suggested that the lower bound value of $\sigma_{\text{min}}$ in the sedimentary rocks would be depth related. Projection of the regression line suggested that $\sigma_{\text{min}}$ would be zero between ground surface and 100m depth. One important consequence of this was that the length of the steel liner had to be increased to include virtually all of the low pressure headrace tunnel. Few data points were available at shallow depth, however, and further hydrofracture tests are being undertaken in three boreholes between the intake and surge shafts. Overcoring tests were also undertaken in a test adit off a construction access tunnel near the underground powerhouse to validate the hydrofracture tests in boreholes BH106 and BH108, and to provide data for the design of the power caverns. These showed significant differences to the stress data derived from the hydrofracture tests.

Further examination of the $\sigma_{\text{min}}$ data showed that above 200m the rate of stress reduction towards ground surface reduced significantly. This was supported by apparent hydrojacking during permeability tests in borehole BH116, which were made after the hydrofracture tests. For example, two regression lines were apparent from the Golder (2006) jacking pressures (Figure 11):
- Ground surface to 200m depth: $\sigma_j$ (or $P_j$) (MPa) = 0.01·depth (m)
- Below 200m depth: $\sigma_j$ (MPa) = 0.021·depth (m) – 2.7

The upper regression line has few data points and above a depth of 80m is purely a projection. Two of the boreholes in the current series of hydrofracture tests have been targeted to provide data between depths of 25 and 200m.

### 12.2 In Situ Stress in the Dolerite Sills

Four hydrofracture tests were performed in a 33 to 41m thick dolerite sill. This large sill crosses above the power caverns and is intersected in the downstream surge shafts, exploration and tailrace tunnels (Table 12).

The measured $\sigma_{\text{min}}$ values of 12.0 to 12.8 are very similar, even though the test depth ranges from 101.7 to 565.8m. This suggests that the sill has a locked in minimum stress of approximately 12 MPa, decreasing only slightly with the loss of 460m of overburden. The lower value (9.6 MPa) suggests that stress loss may start in the sill at approximately 100m depth.
Figure 11: Jacking Pressure vs. Depth
Table 12: Minimum Horizontal Stress and Stress Ratios in the Same Dolerite Sill

<table>
<thead>
<tr>
<th>Test</th>
<th>Depth (m)</th>
<th>$\sigma_v$ (MPa)*</th>
<th>$\sigma_{\text{min}}$ (MPa)</th>
<th>Jacking Pressure, $\sigma_j$ (MPa)</th>
<th>$\sigma_{\text{min}}/\sigma_v$ (MPa)</th>
<th>$\sigma_j/\sigma_v$ (MPa)</th>
<th>Dip of Hydrofracture (°)</th>
</tr>
</thead>
<tbody>
<tr>
<td>117/3</td>
<td>101.7</td>
<td>2.6</td>
<td>12.0</td>
<td>11.3</td>
<td>4.6</td>
<td>4.3</td>
<td>33, 39</td>
</tr>
<tr>
<td>117/4</td>
<td>106.3</td>
<td>2.7</td>
<td>9.6</td>
<td>10.0</td>
<td>3.6</td>
<td>3.7</td>
<td>15, 15, 15</td>
</tr>
<tr>
<td>108/2</td>
<td>322.1</td>
<td>8.1</td>
<td>12.3</td>
<td>10.9</td>
<td>1.5</td>
<td>1.3</td>
<td>12, 90</td>
</tr>
<tr>
<td>115/7</td>
<td>565.8</td>
<td>14.2</td>
<td>12.8</td>
<td>13.3</td>
<td>0.9</td>
<td>0.9</td>
<td>-</td>
</tr>
</tbody>
</table>

Note: * Based on the rock encountered in each borehole and average unit weights for each rock type.

The hydrofractures at approximately 100m depth had relatively shallow inclinations (15 to 39°), suggesting that $\sigma_{\text{min}}$ is steeply inclined here and may therefore not be the minimum horizontal stress $\sigma_h$. The measured stresses were much greater than the vertical stress estimated from the unit weight and depth of overburden, suggesting that the vertical stress is locked in. Assuming the sill had an original stress of 12.8 MPa, the excess stress ($\sigma_{\text{min}} - \sigma_v$) for test 117/3 is approximately equivalent to a cover depth of 400m, which is approximately the height of the escarpment above the sill. The locked in stresses are therefore probably a relic of the relatively recent erosion of the escarpment.

The measured minimum stresses ($\sigma_j$ and $\sigma_{\text{min}}$) in all four tests are of a similar order. This suggests that the propagated hydrofractures did not turn towards a direction of distinctly lower minimum stress. The 90° hydrofracture at 322.1m depth has a strike of 149°, which is approximately perpendicular to the direction of $\sigma_h$ for the hydrofracture data as a whole. Both vertical and shallow (12°) hydrofractures were created at this test depth. The sill may therefore have locked in minimum and maximum stresses of a similar magnitude.

The intermediate stress varies from being less than $\sigma_{\text{min}}$ to between 1.5 and 3.0 times larger than $\sigma_{\text{min}}$, depending on the depth and equation adopted. The higher values are not consistent with the intermediate stress values obtained from the overcoring tests. For those tests where the inclination of the hydrofractures suggests that $\sigma_{\text{min}}$ is not $\sigma_h$, the intermediate stress will not be the maximum horizontal stress $\sigma_{H}$.

A further hydrofracture test was made in a 6m thick dolerite sill below the power caverns. This gave $\sigma_{\text{min}}$ and $\sigma_j$ values of 8.8 and 7.9 MPa respectively at 489.3m depth. These values are equivalent to those in the adjacent mudstones at this depth and it is probable that these relatively thin sills do not retain locked in stresses. The hydrofracture is vertical and the estimated $\sigma_j$ stress of 12.3 MPa is significantly greater than the measured stress. It is therefore reasonable to assume that the measured stress ($\sigma_{\text{min}}$) is $\sigma_h$. 
12.3 Dolerite – Mudstone Contact Zone

Three hydrofracture tests were made in the contact zone adjacent to the large sill. The rock is altered by the intrusion of the sill and is significantly stronger and stiffer than is the unaltered host rock. The measured minimum stresses were similar to those in the mudstones at the same depth and much lower than those in the adjacent dolerite sills. The $\sigma_{\text{min}}/\sigma_v$ ratio ranged from 0.4 to 1.1. The hydrofractures had a moderately steep dip of 60 to 82°. The stresses locked in the dolerite sill appear to have dissipated at the contact with the host rock.

12.4 Mudrocks Below the Main Dolerite Sill

The powerhouse caverns will be excavated within mudrocks below the large dolerite sill. Thirteen hydrofracture tests were made in these mudrocks. The $\sigma_{\text{min}}$ values showed no relationship with depth below the sill, suggesting that the stress is not influenced by proximity to the sill.

The $\sigma_{\text{min}}/\sigma_v$ ratio ranged from 0.5 to 0.9 (mean 0.7) in eleven of the tests. The two remaining tests gave significantly higher ratios of 1.2 and 1.6. The locations of these two tests are coincident with a zone of horizontal bedding shears, in and below the lower cavern walls (Figure 6). A dolerite sill below the exploration tunnel pinches out towards the cavern at the same level as the shear zone. It is assumed that the intrusion of the sill caused the horizontal shears and the locally higher $\sigma_h$ values.

The $\sigma_{\text{min}}$ values showed two orientations, within a wide scatter of data. The orientations are similar to those of the faults and dykes mapped at ground surface. This suggests that $\sigma_h$ and $\sigma_{\text{H}}$ may have similar magnitudes.

12.5 Rock Above the Main Dolerite Sill

Eighteen hydrofracture tests were made in the sedimentary rocks above the main dolerite sill. The $\sigma_{\text{min}}/\sigma_v$ ratio was much lower (0.5) in the sandstones/siltstones than it was in the mudstones (0.8 to 1.4). The hydrofractures were steeply dipping in the sandstones/siltstones but tended to be shallow in the mudstones. This probably reflects a greater tendency to split along the bedding in the mudstones. The $\sigma_{\text{min}}$ values and orientations would therefore be expected to be of a different direction in the mudstones and sandstones/siltstones.

12.6 Overcoring Tests

In situ ground stresses (RMT, 2008) were measured with CSIRO hollow inclusion cells at approximately 10m from the sidewall of the plate bearing test adit (Table 13), at the level of the machine hall crown.
Table 13: Overcoring In Situ Stress Tests

<table>
<thead>
<tr>
<th>Measured Stress (MPa)</th>
<th>Resolved Stress (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stress</td>
<td>Bearing (°)</td>
</tr>
<tr>
<td>σ₁</td>
<td>5.8-6.3</td>
</tr>
<tr>
<td>σ₂</td>
<td>5.0-5.3</td>
</tr>
<tr>
<td>σ₃</td>
<td>4.3-4.8</td>
</tr>
</tbody>
</table>

The resolved σₗ/σᵥ stress ratio of 0.96 to 1.05 is higher than that estimated from the measured σₘ₃/assumed σᵥ stress ratio from the hydrofracture tests. The resolved σᵥ is much less than would be assumed from the unit weight of the ground and the depth below ground surface (8.2 MPa).

The test site is 100m to the east of borehole BH108. A hydrofracture test was made at a similar elevation (1197m) to that of the overcoring test (1206m). The σₘ₃ of 6.8 to 7.3 MPa is much higher than σ₃ or σ₉. When the difference in ground cover and the groundwater rest level in borehole BH108 and that measured in a borehole drilled underground from the exploration tunnel is taken into account, however, σₘ₃ becomes equal to σ₃ and the discrepancy is explained. The discrepancy in the resolved and estimated vertical stresses is not explained, however. The design of the underground works has therefore assumed a vertical stress derived from the unit weight and depth, a typical σₗ/σᵥ ratio K of 1 and a range in K of 0.7 to 1.2.

13 Potential Expansiveness and Durability

Unconfined, free swell tests were carried out on a number of intact core specimens from the powerhouse area, with free swell measured at right angles to the bedding. Tests were carried out on specimens conditioned as follows at the start of the test, prior to the specimen being submersed in water:

- Some specimens were oven dried at 105°C for 12 hours;
- Some were left unconditioned, i.e. specimen tested at the ‘as received’ moisture content;
- Some were brought to moisture content equilibrium at a relative humidity (RH) of 85%; and
- Some were brought to moisture content equilibrium at an RH of 97%.

Figure 12 shows the result of an intact rock core specimen subjected to various conditions. Generally, moisture content conditioning at an RH of 85 to 97% resulted in an increase in the moisture content of the specimen, while an RH of approximately 60% as maintained by the air conditioning system in the laboratory resulted in a net drying out of the specimen.

The results of the free swell tests after 24 hours of submersion in water are presented in Figure 13, on the basis of which the mudrocks at the level of the powerhouse can be described as having a low swelling potential (or potential expansiveness) with total free swells of less than 1.5 to 2.0% typically measured.
Time dependent expansion of these mudrocks was furthermore noted as is illustrated in Figure 14 which shows a typical free swell test result for a specimen from the area of the powerhouse, conditioned at an RH of 85% prior to submersion in water, with the test stopped after approximately 340 days. A number of free swell tests showed continued swelling strains over a period of about a year as shown in this Figure, i.e. swelling strains, albeit small, had not yet reached a constant level after a considerable period of submersion.
A few unconfined, free swell tests were also carried out on test specimens with free swell measured at both right angles to the bedding (i.e. vertical) and parallel to the bedding (i.e. two horizontal directions at right angles to each other) in order to examine swell anisotropy. A typical test result is presented in Figure 15 for a specimen at the ‘as received’ moisture content prior to submersion in water. The horizontal free swell (i.e. free swell parallel to bedding) is generally less than half the vertical free swell (i.e. free swell at right angles to the bedding) and appears less prone to continued expansion with time.

The durability of mudrocks at the powerhouse level has been categorised in terms of Olivier’s Geodurability Classification (1976, 1979), which indicates a mudrock durability ranging from good / excellent at and above the cavern crown level, reducing to moderately poor / fair at lower elevations.

Furthermore, no signs of rapid mudrock deterioration were evident in mining faces in the powerhouse area during construction of the exploratory tunnel. As an example, in one instance, the tunnel face stood unprotected and without any rock support for a period of about 2 weeks, with deterioration of the tunnel face limited to 10cm or less, most of which only came loose when barring the face before advancing the tunnel again.
14 Creep

A series of long term creep tests have been carried out at the CSIR on intact core specimens with height:diameter ratios in the order of 2:1. Final results on this work are still outstanding, however, an assessment of preliminary results indicate creep rates in the order of 1 to 10 microstrain/year for applied uniaxial stresses between 10 and 20 MPa, which increases to between 10 and 15 microstrain/year for applied uniaxial stresses of 50 MPa.

The fourth cycle of the first horizontal plate bearing test was extended for 15 days at a load of 9.6 MPa to investigate creep displacements parallel with the bedding. The load of 9.6 MPa is equivalent to the ground load at about loading bay level in the machine hall. Only minor creep was detected. The deepest extensometer anchor at 2.4m from the tunnel wall showed creep equivalent to 1.5mm/year. It is intended that the maximum load in the second vertical plate bearing test will be extended for 3 weeks to measure creep perpendicular to bedding, with the results of this work expected towards the middle of April 2008.

In comparison, distometer readings between points along the crane beams at the Drakensberg Pumped Storage Scheme (ESKOM, 1995) showed total ground closure of 0.9 to 3.9mm between 1983 and 1995. The rate of closure in the first 4 years was typically 0.75mm/year, but this then fell to 0.06 to 0.14mm/year. Over the same period, extensometers around the machine, transformer and valve halls showed total displacements of generally less than 2mm. The rate of continuing movement was generally less than 0.1mm/year in 1995.
15 Analysis and Design

The data presented in this paper is being used in finalising the rock support design of the main powerhouse caverns, with consideration being given to, amongst other things:

- Crown geometry.
- Stability considerations, particularly of the machine hall crown and sidewalls.
- Stability of the powerhouse caverns during seismic loading.
- Long term creep.
- Methods of excavation and support.
- Excavation and support sequencing.

Various numerical codes are being employed in this work, including Unwedge, Phase2, FLAC, UDEC and FLAC3D, with Itasca (South Africa) having assisted with the development of base models in UDEC and FLAC3D.

Dr Evert Hoek is carrying out a review of the work in his capacity as external reviewer to BCJV.

The detailed design was being finalised at the time of preparing this paper, with changes from that presented at the tender design stage, scheduled to be negotiated with the preferred bidder during the second quarter of 2008.

16 Acknowledgements

The authors would like to express their gratitude to ESKOM, specifically Mr Frans Louwinger, for granting permission to publish this work.

The efforts of BCJV site staff, notably that of Messrs Mike Neumann (supervisor of the works) and Giovanni Pradella (supervising geologist), are duly recognised.

Contributions made by Dr Evert Hoek in reviewing the design work being carried out are gratefully acknowledged.

Successful completion of the laboratory work at the CSIR would not have been possible without the labours and attention of Mr Uli Vogler, we stand indebted to his efforts.

The contributions made by many others who were involved in the project are also recognised:

- Contractors (construction): Murray & Roberts (Concor), CMCM
- Contractors (geotechnical): Solexperts, RMT, Terra Monitoring, Roelf Fourie Geotechnical Services and EEGS.
- Consultants: Golder & Associates, SEA Consult and Terralogix.
- Laboratories: Rocklab, CSIR and WITS.
- Suppliers: Minova, Duraset and DSI.

Finally, thank you to the Braamhoek Consultants Joint Venture partners and management for allowing preparation of this paper and presentation thereof.
References


