THE USE OF GEOTECHNICAL INSTRUMENTATION TO MONITOR GROUND DISPLACEMENTS DURING EXCAVATION OF THE INGULA POWER CAVERNS, FOR MODEL CALIBRATION AND DESIGN VERIFICATION PURPOSES

M. Kellaway, D. Taylor and G.J. Keyter
Braamhoek Consultants Joint Venture

Abstract

An extensive programme of geotechnical instrumentation and monitoring was carried out during construction of the Ingula hydro power caverns to validate design assumptions and monitor long term creep effects. This paper supports the use of geotechnical monitoring during construction and discusses the results compared to the predicted convergence and where the monitoring allowed for rapid assessment of problems encountered during construction. Differences between the predicted convergence and the monitoring results necessitated a review of the numerical models used for design, to ascertain the sensitivity of changes in the construction sequencing and geotechnical parameters encountered during construction on the models and derive a new set of predicted convergence.

Introduction

Construction of the Ingula pumped storage scheme commenced in 2004 and is due for completion in 2015. This paper focuses on construction of the main power caverns, specifically in relation to the use of geotechnical instrumentation to monitor ground displacements during excavation of the Ingula power caverns, for model calibration and design verification purposes.

The Ingula powerhouse complex (see Figure 1), comprising a machine hall with 26 m span, a transformer hall with 19 m span and appurtenant busbar tunnels with 11 m diameter, 5 m diameter high pressure penstocks, 9 m diameter main access tunnel and a series of smaller adits and shafts is located at a depth of almost 400 m below ground level, under a prominent mountain ridge off the Drakensberg escarpment between the Free State and KwaZulu Natal provinces, South Africa. The machine hall, 184 m long, has a double curvature profile with a relatively low span: height ratio of 2.5 and is up to 50 m deep in the turbine pits. The adjacent transformer hall, 21 m high, is 176 m long, with a cable and pipe gallery on one side running the length of the cavern and extending another 6 m below operating floor level.

Figure 1 also shows the location of main instrumentation arrays that were installed in the main caverns and adjacent tunnels as well as the intersection of a sheared dolerite dyke (green line) with the caverns at turbine floor.

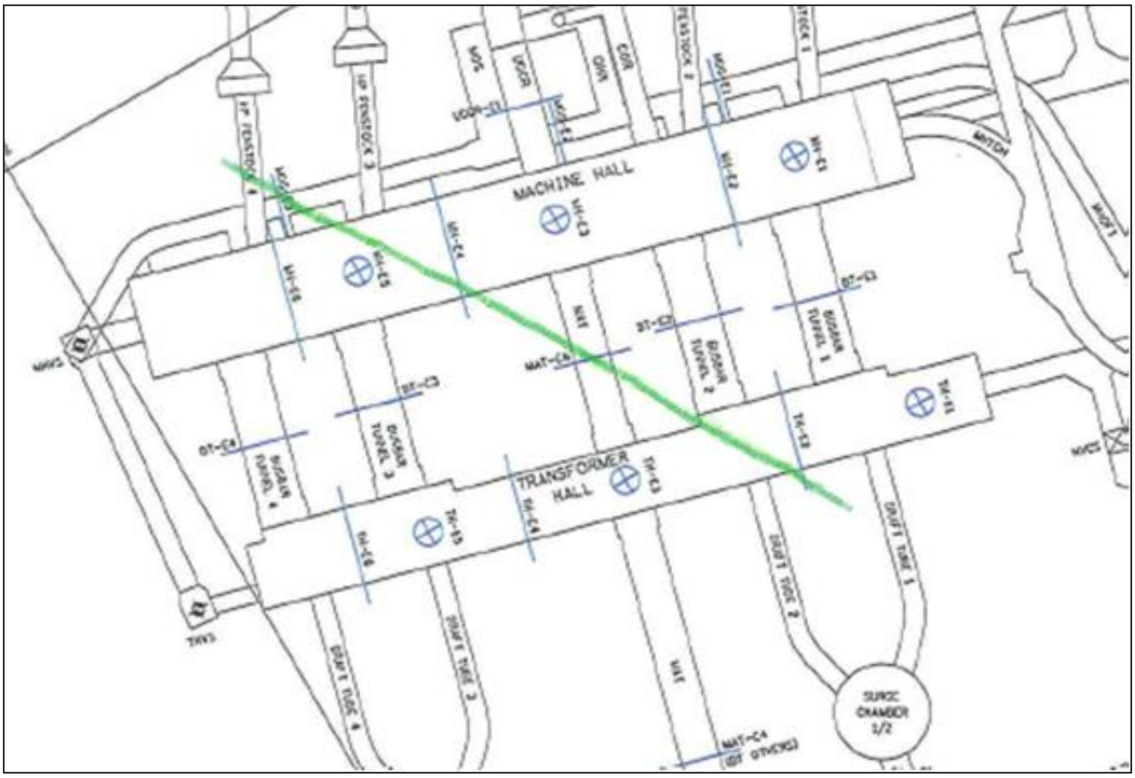


Figure 1. Plan View of Power Cavern Complex

Geology

The Ingula power caverns were constructed in horizontally bedded siltstones, mudstones and carbonaceous mudstones of the Volksrust Formation of the Ecca Group, Karoo Supergroup. The power caverns are located some 25 m below a 40 m thick dolerite sill. Intact rock properties derived from field and laboratory testing are presented schematically in Figure 2 in relation to the powerhouse. A decreasing trend in intact rock strength and stiffness with depth below the dolerite sill is evident. This can be attributed to induration effects given the proximity of the sill above as well as changes in mudrock composition with depth. The durability of mudstones at and above cavern roof level was classified [1] as 'good' to 'excellent' and at lower elevations, further away from the influence of the dolerite sill, as 'poor' to 'fair'.

Faults in the project area generally trend E-W and ESE-WNW with a further two sets of small displacement faults striking NW-SE and NE-SW. A sub-vertical, sheared and faulted dolerite dyke with strike orientation NNW-SSE intersects the power caverns and main access tunnel at an oblique angle (see Figure 1). A normal fault zone comprising slickensided, striated joints, infilled with calcite and mylonitic material, was intersected in access tunnels near the powerhouse with a few of these fault planes intersecting the far eastern end of the transformer hall. A number of bedding parallel shears have also been identified in boreholes and underground excavations in the powerhouse area and surrounds, three of which are located in and above the machine hall crown. However, there are no known seismically active faults in the immediate project area. The Tugela Fault, which follows the boundary between the Kaapvaal Craton and the Namaqua Province in this region, is located some 50 km to the south of the project area.

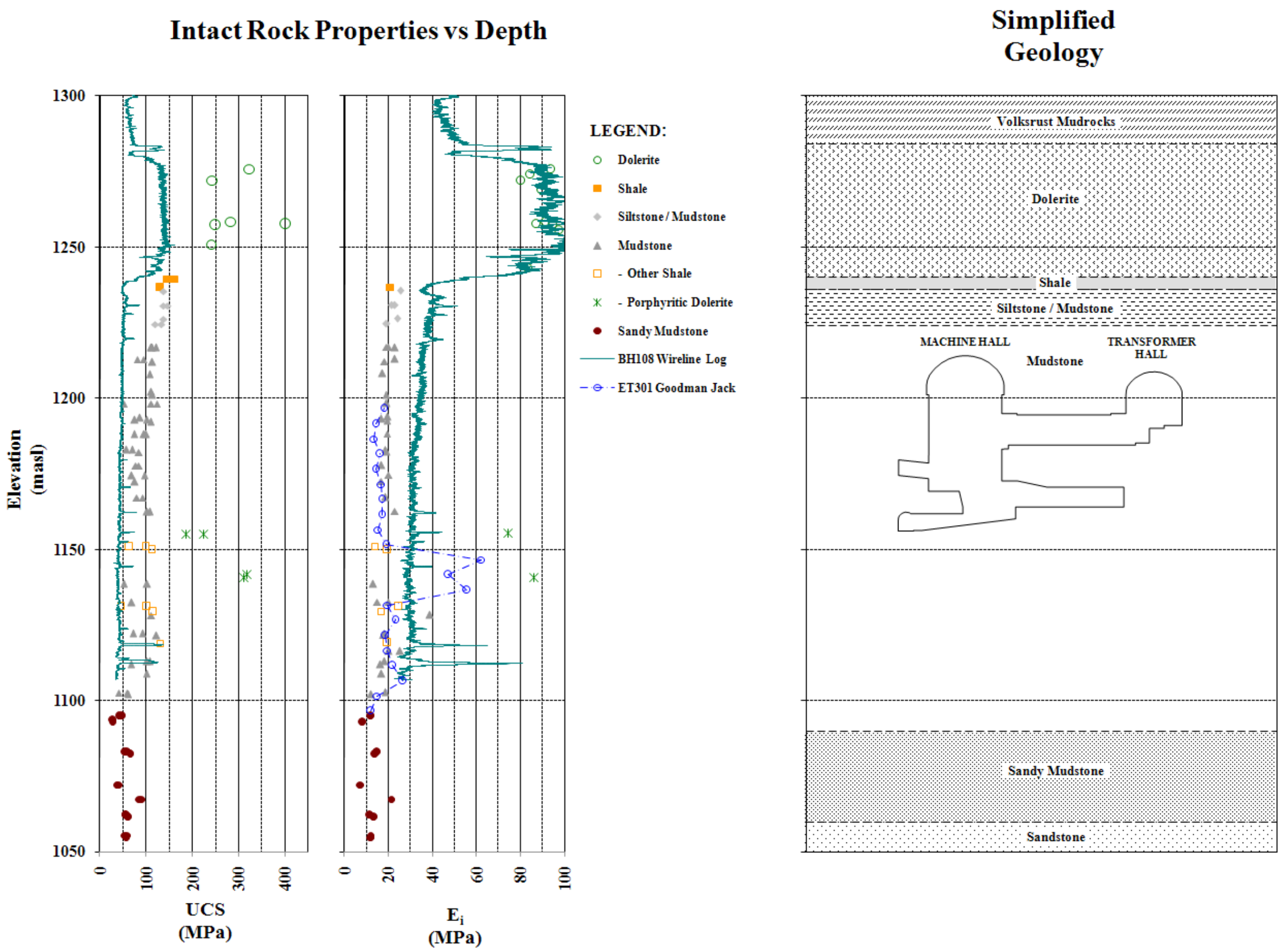


Figure 2. Rock Material Properties in relation to the Ingula Power Caverns

In situ ground stress was measured in hydrofracture tests in boreholes and in a small number of overcoring tests. The minimum horizontal stress is orientated NNW-SSE. The major horizontal stress is greater, and the minor horizontal stress slightly lower, than the estimated vertical overburden stress. Hydrofracture tests at cavern level gave a horizontal / vertical stress ratio (K-ratio) of between 0.5 and 0.9 while overcoring tests indicated a K-ratio of approximately 1.0 in the powerhouse area.

Time dependent deformation of the rock mass was noted during construction of the access tunnels to the powerhouse [4]. Taking into account scale effects given the size of the main power caverns compared to that of the tunnels constructed earlier, most of this time dependent deformation in the power caverns was expected to occur within 6 months to a year following excavation down to operating floor level and for about a year in the turbine pits after turbine floor level has been reached.

The rock mass at powerhouse level is characterized by closed joints with a resultant low rock mass permeability. Virtually no groundwater has been encountered at cavern level during excavation of the caverns.

Cavern support design

Initial estimates of the support required in the Ingula power caverns were based on precedent experience. The final support design was based on a detailed evaluation of all available geological and geotechnical information followed by numerical modelling using UDEC, Phase2 Version 7 and FLAC3D [5, 6]. The convergence of different points located on the crowns and sidewalls of the main power caverns was estimated on the basis of the results of this modelling work, for the anticipated excavation sequence as well as specified timing of support installation [2, 3].

Instrumentation and Monitoring Design

Instrumentation arrays were installed to monitor ground displacements as well as ground anchor loads during construction of the power caverns and adjacent excavations with some of this instrumentation being retained after construction to monitor longer term effects during scheme operation. The instrumentation was deemed critical for validation of design assumptions and analyses and to timely detect unanticipated cavern convergence and loads in ground anchorages.

Instrumentation was located based on the results of the modelling work, where areas of concern were perceived to exist. Instrument arrays comprising multiple point borehole extensometers (MPBX) shown in Figure 3 were located in each of the turbine pits, being the deepest part of the cavern complex but also between the pillars of the appurtenant 12 m diameter busbars. These arrays were complimented by rockbolt, cable bolt and cable anchor load cells and optical convergence targets. Given the cavern length and the geometry of the cavern crowns, an additional MPBX array and single point MPBX's were located in between the arrays and selected rock anchors were instrumented to monitor actual anchor loads to provide spatial coverage and comparison to the MPBX data.

Limits were set on cavern convergence and anchor loads in terms of so-called trigger levels and based on the anticipated construction sequence as follows, to allow a rapid but appropriate response to actual monitoring data:

- A **baseline level** on which the support design is based, with observed excavation convergence and anchor loads falling within expected limits in line with that predicted during design.
- A **warning level**, which is approached when convergence and anchor loads exceed the baseline level.
- An **alarm level**, which is approached when convergence and loads exceed the warning level.

Baseline convergence is satisfied where the measured increase in working load in an anchor is projected not to exceed 50% of the yield capacity of the anchor on cavern completion [5]. The warning level is reached where the anchor load is projected to exceed 50% of the yield capacity of the anchor and the alarm level when the anchor load is projected to surpass 62.5% of the anchor's yield capacity on cavern completion.

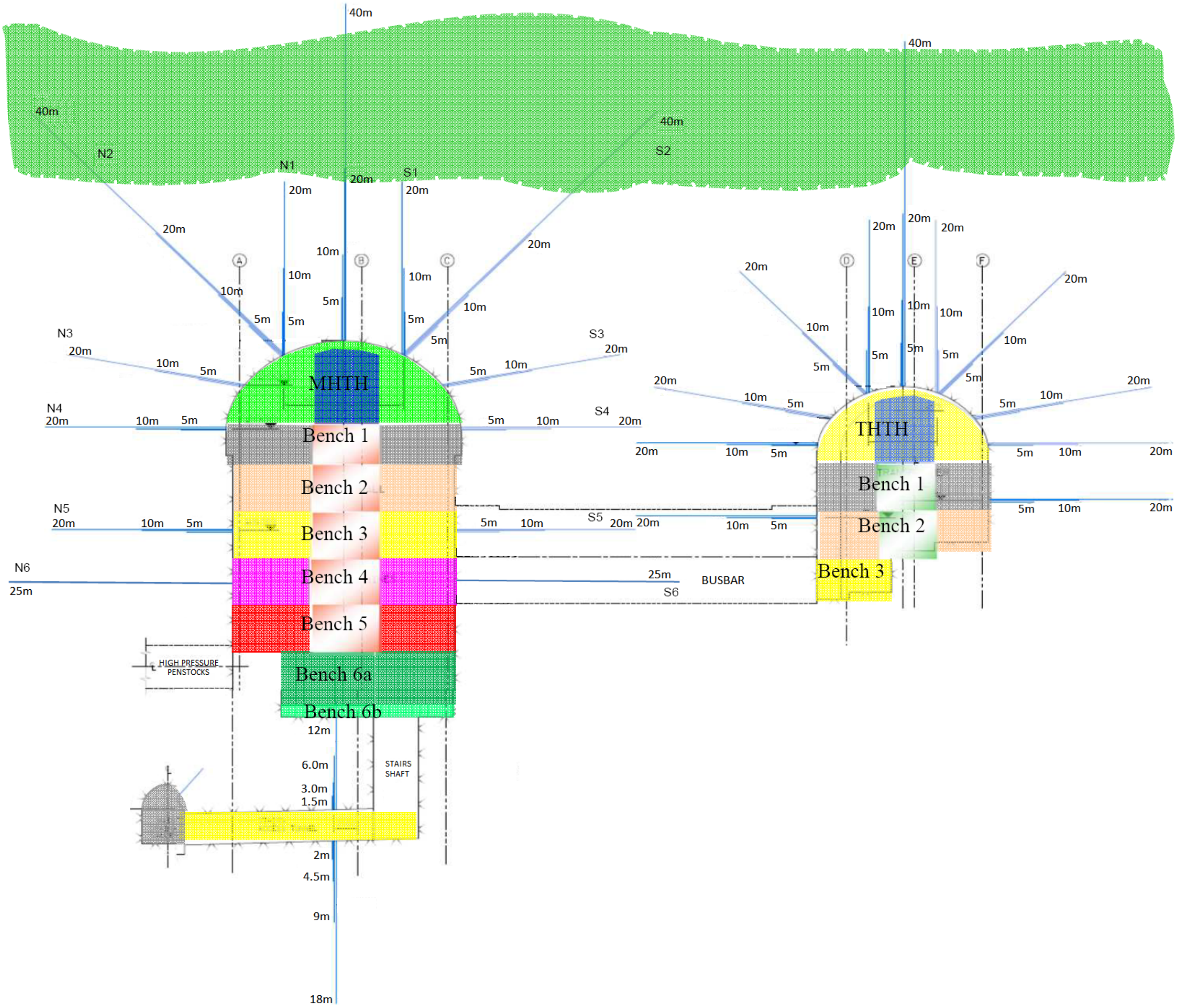


Figure 3. Multiple Point Borehole Extensometer Arrays in Power Caverns

Reading of Instrumentation during Construction

Following installation of a given instrument it was read after each blast advance until the rapid phase of convergence had passed. Thereafter the frequency and timing of readings was determined by the geotechnical engineer on site to provide readings at various stages of construction, for design validation.

Tabulated data and graphical plots were generated for each instrument to allow a rapid but detailed assessment to be carried out on a regular basis. However, given the number of instruments installed in the power caverns and adjacent excavations, a system to monitor the global trends was adopted to provide a high level, monthly summary whereby two flags were assigned to each instrument installation.

The flag status was indicated on an instrumentation plan of the powerhouse complex as follows:

- A first (or left) flag looking at 'total convergence' with a green flag designating convergence within the baseline; an orange flag convergence approaching the warning level; and a red flag indicating convergence approaching the alarm level.
- A second (or right) flag looking at 'rate of convergence' with a green flag indicating the excavation is stabilizing; an orange flag that creep deformation is noted; a magenta flag indicating blasting nearby; and a red flag accelerating convergence.

An extract from this summary with instrumentation flags is shown in Figure 4 for that part of the machine hall where the dolerite dyke was intersected. This system of flags allowed for a quick and easy, high level overview of cavern convergence and support performance in different parts of the cavern. For example, in Figure 4, all the instruments in the area of the dyke indicate total convergence within the baseline level (i.e. all the 'left flags' are green). However, note that all the instruments on the western side of the dyke are showing time dependent creep deformation (all the 'right flags' on this side of the dyke are orange) whereas instruments on the eastern side of the dyke are showing an excavation that is stabilizing (i.e. all the 'right flags' on this side of the dyke are green).

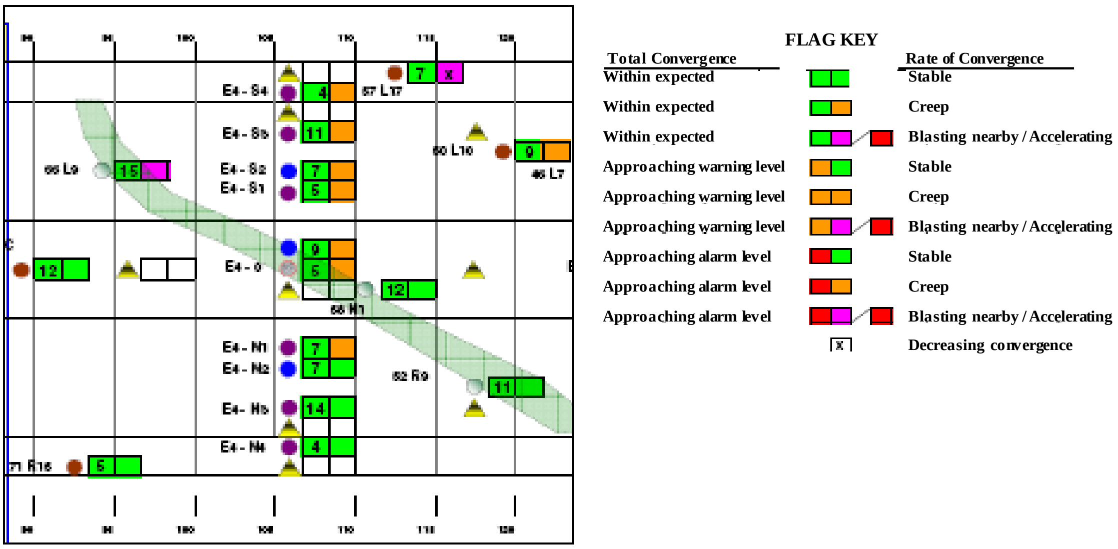


Figure 4. Instrumentation Flags in part of the Machine Hall Cavern

Actual Cavern Convergence and Support Performance

A high level summary of selected MPBX results are presented below to demonstrate the deformation characteristics of the machine hall (MH) and transformer hall (TH) caverns and appurtenant main drainage gallery (MDG) complex. Results of other instrumentation comprising load cells and optical convergence monitoring have not been included in this paper due to the level of detail and interpretation required to discuss the results.

No significant stability issues, apart from those discussed below, were encountered during cavern excavation. In comparison with the expected (i.e. modelled) deformations at design stage, the following general statements can be made with regard to observed displacements:

- The magnitude of measured displacements was less than that anticipated.
- Sidewall displacements were relatively greater in comparison to that observed in the crown.
- The rate of displacements dissipated more rapidly as the excavation front became more distant.

The total magnitude of measured displacements differed considerably at each instrument location (see Figure 5) and required review on a case-by-case basis, for detailed interpretation of cavern response to excavation and support sequences and activities.

Excavation of the Cavern Crowns

The cavern crowns were excavated by driving a central top heading, at which point the first MPBX's were installed. The side headings were then removed using a staggered approach to allow the primary cable bolt installation to take place at between 10 to 15 m behind the advancing excavation face. MPBX's were installed in the side headings as the staggered excavation faces reached the required position. Upon completion of the top heading, secondary support cable anchors were installed with weldmesh reinforced shotcrete.

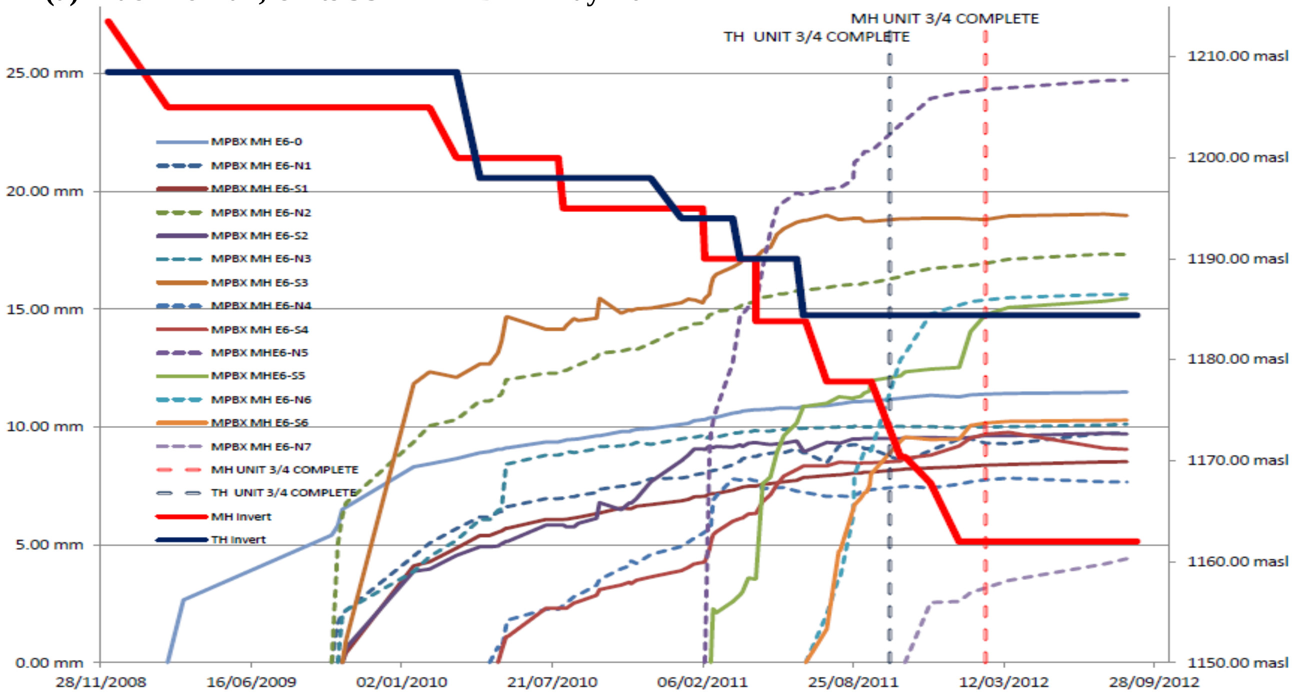
Between 50 and 90% of the total displacement recorded by MPBX's installed in the cavern crowns had occurred during the excavation increment in which they were installed and the immediately adjacent excavation increment as can be seen in Figure 6.

MPBX's installed in the MH crown generally recorded similar displacements during the side top heading excavation with some exceptions being explained by the presence of particular geological conditions.

Slightly higher displacements associated with MPBX E3 (almost double that of other crown MPBX's) can be attributed to the presence of the sheared dolerite dyke and associated shears and joints above the crown at this location.

Localised overbreak along a shear bedding plane developed rapidly whilst excavating one of the MH side headings [3]. This led locally to overbreak of more than 2 m beyond the theoretical excavation line. Increases in anchor loads were noted in load cells in the affected area and a jump in convergence was noted in nearby MPBX E6 in the cavern crown. By then, cracking of shotcrete developed in the 'brow' in the roof as shown in Figure 7. The displacement was successfully arrested by halting the excavation, reviewing instrumentation data, exploratory drilling to confirm the ground conditions above the crown, bringing forward the installation of secondary support and re-establishing the crown profile with backfill shotcrete. Excavation was restarted and the monitoring frequency was increased to check the adequacy or otherwise of the above remedial measures.

(a) Machine Hall, Units 3&4 – MPBX Array E6



(b) Transformer Hall, Units 1&2 – MPBX Array E2

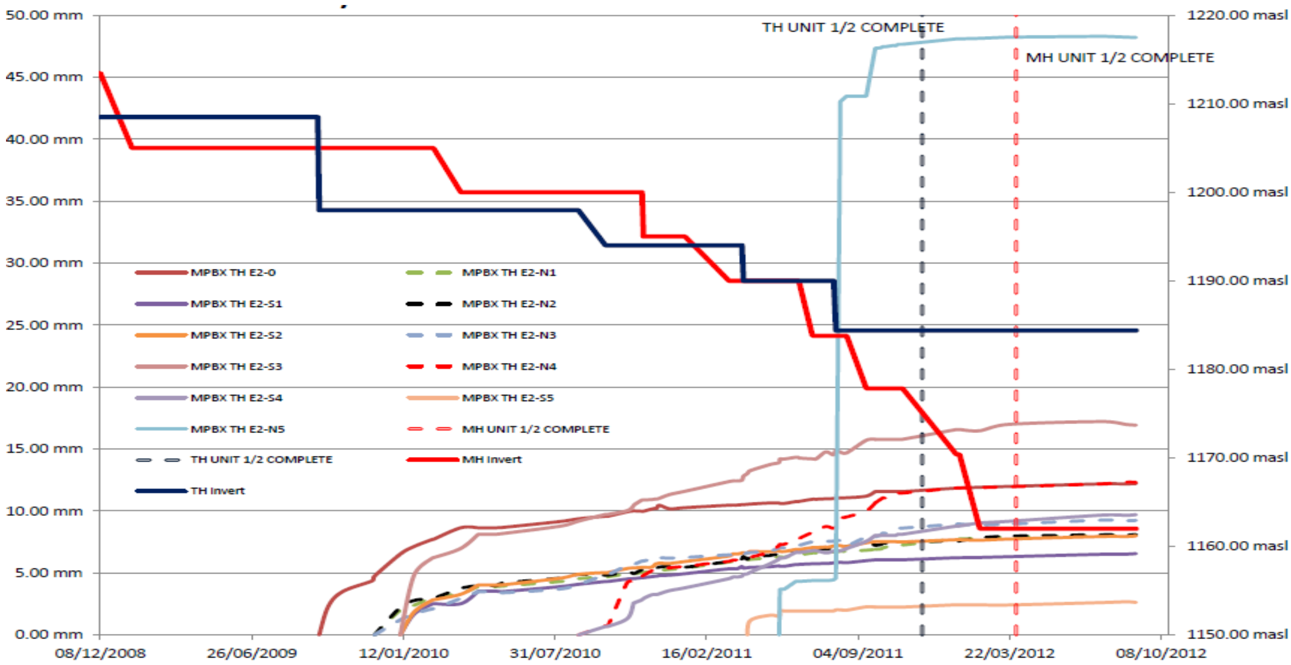
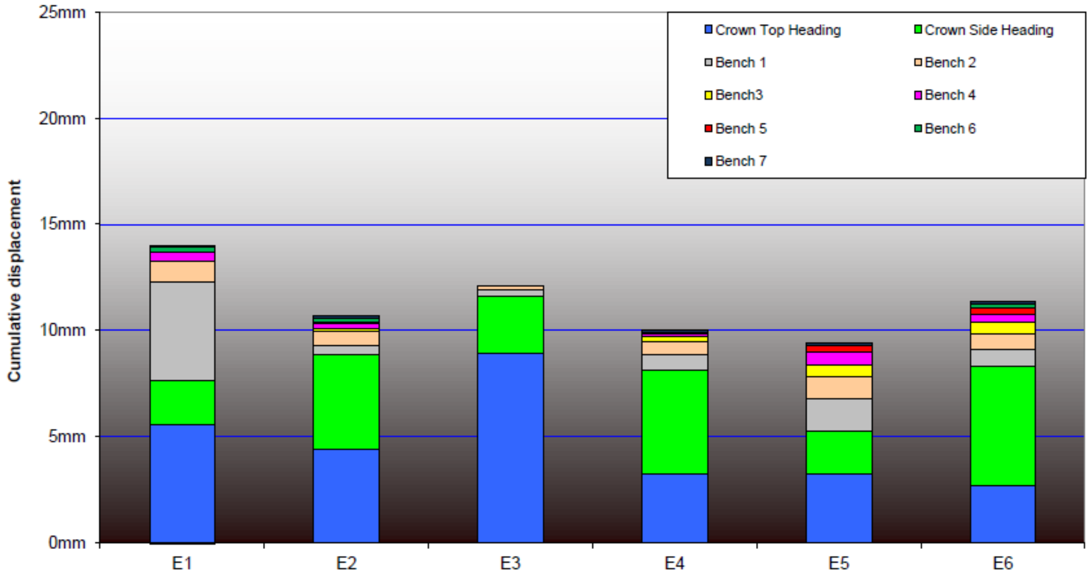
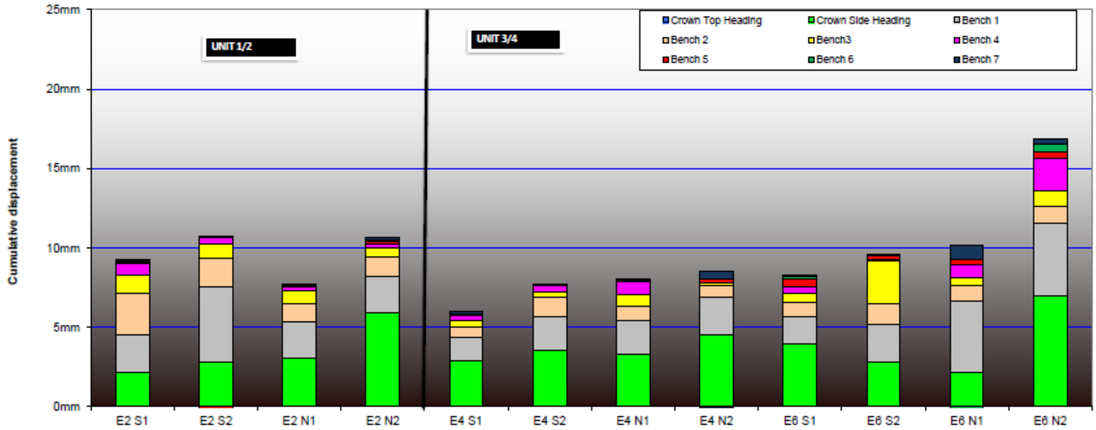


Figure 5. Measured Convergence versus Time

(a) Machine Hall Crown MPBX's, EL 1213.50 m



(b) Machine Hall Side Headings MPBX's, EL 1210.50 m



(c) Machine Hall Side Heading MPBX's, EL 1208.3 m

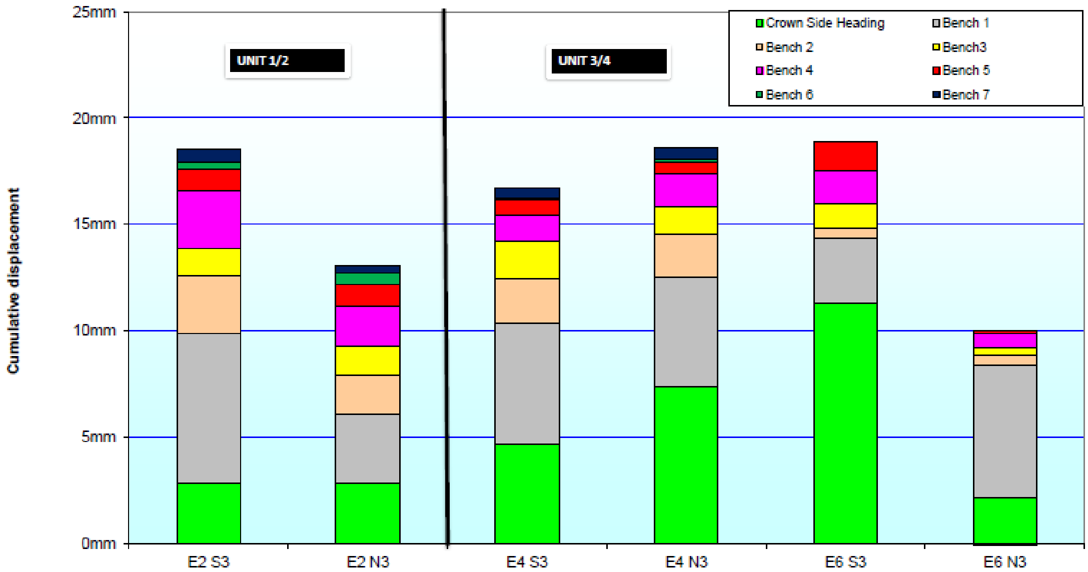


Figure 6: Cumulative Displacement of MPBX's installed in MH Crown

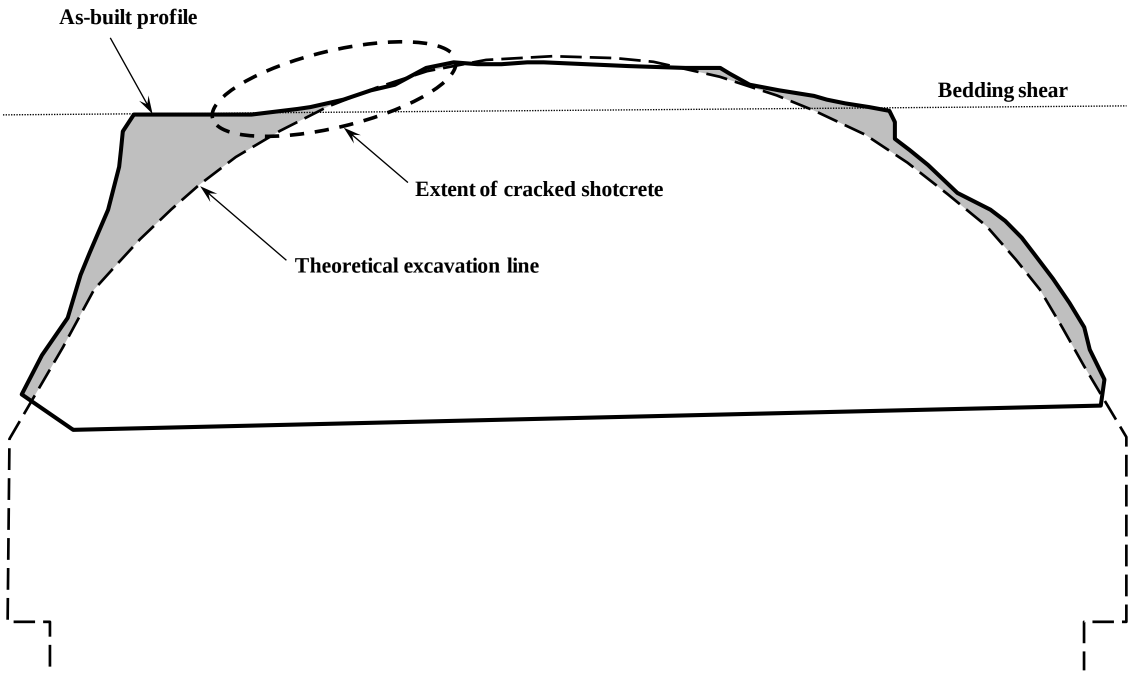


Figure 7: Localised Overbreak along Shear Bedding in MH Cavern Crown

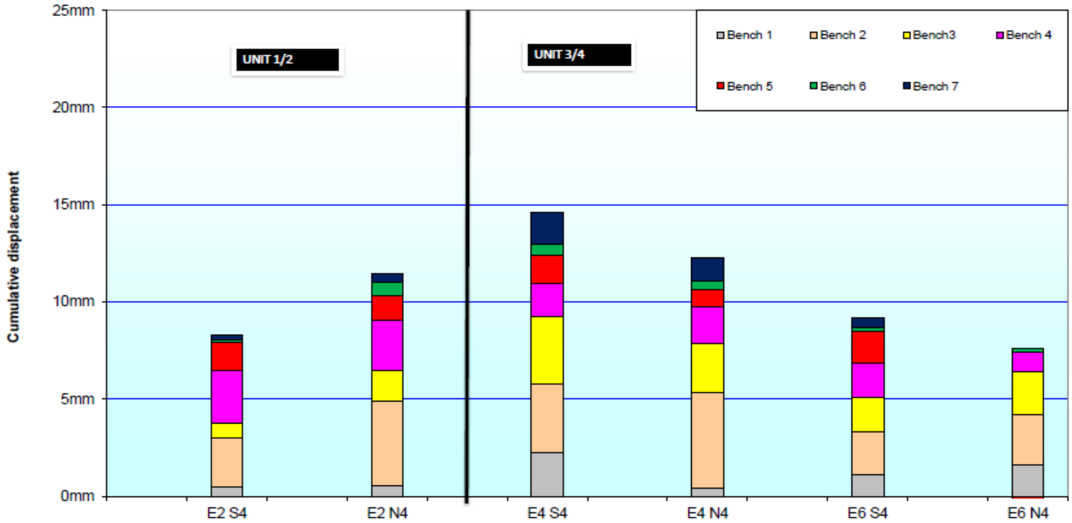
Excavation of the Cavern Benches

Following completion of the cavern crowns, further excavation of the power caverns was carried out bench-by-bench (see Figure 3) with vertical lifts not greater than 6 m and by excavating a central heading leaving a “rock buttress” against the sides prior to slashing out the sides. However, for Bench 3 of the MH and Bench 2 of the TH, the bench was excavated in two parts without a central heading. No significant stability issues were encountered during the excavation despite persistent over-excavation of the sidewall profile which subsequently had to be reinstated to the design profile by application of additional shotcrete.

The relatively larger displacement of MPBX MH E6-N5 is considered to be associated with poorer rock mass quality in the vicinity of the cross-cutting dolerite dyke (see Figure 1) and the removal of a large, blast damaged rock buttress between Valve Pits 3 and 4.

The dramatic increase in displacement in MPBX TH E2-N5 occurred as a result of exposing an unfavourable joint in the pillar between Busbars 1 and 2. Based on the instrumentation results and the occurrence of geological overbreak along this joint, the excavation was halted and the installation of primary support cable bolts was brought forward to stabilise the pillar. This sidewall wedge / block reacted to a lesser extent when the excavation recommenced but was completely stabilised by later installation of the secondary support cable anchors.

(a) Machine Hall Crane Beam MPBX's, EL 1203.3 m



(b) Machine Hall Sidewall MPBX's

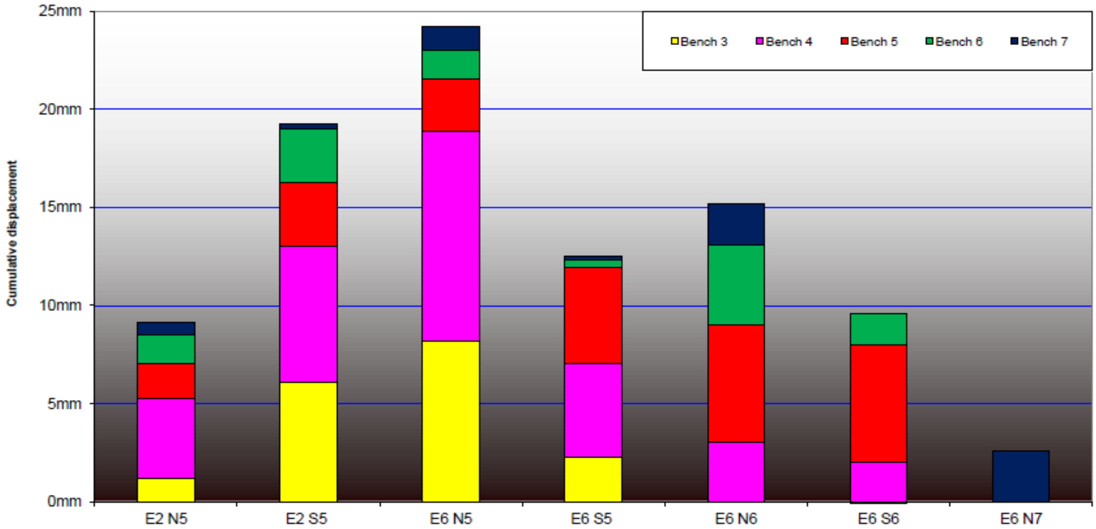


Figure 8. Cumulative Displacement of MPBX's installed in MH Sidewalls

Excavation of the Main Drainage Gallery Complex (MDG)

The bulk of the MDG complex (stairs access tunnels, main drainage sump, oil handling room and exhaust chambers) located below and immediately to the north of the machine hall was excavated prior to the excavation of the machine hall. The design of support for the MDG therefore had to take into consideration the effects of stress redistribution in the surrounding rock mass as the machine hall excavation proceeded downwards. Initial support of the mudstones exposed in these excavation only comprised a flash coat of shotcrete to seal the rock followed by installation of rockbolts and mesh; this was followed later, on cavern completion, by installation of additional rockbolts and a more rigid, shotcrete lining. Cracks were noted in the MDG shotcrete lining as a result of stress induced changes during excavation of the machine hall; however, the situation was exacerbated by installation of defective support which later had to be replaced.

MPBX's installed in the Stair Access Tunnels (SAT) immediately below the base of the machine hall exhibited rapid displacement as the invert of the machine hall heaved during the downward excavation; some reaching the limit of travel, others being destroyed when intercepted by the excavation as shown in Figure 9. However, relatively small displacements were seen in the other MDG complex tunnels that were not directly below the machine hall.

Interestingly, the invert MDG MPBX's exhibited much more reaction than the crown. Being the lowest tunnel in the powerhouse, the invert was frequently flooded and with the passing of heavy plant, the invert (MDG E2, Figure 9) exhibited rapid heave until it was protected by placement of blinding concrete. MDG E3 also exhibited relatively large shallow movements, again with some concerns due to the invert not being protected, but also clearly in reaction to the deepening of the machine hall.

Back-Analysis and Remodelling

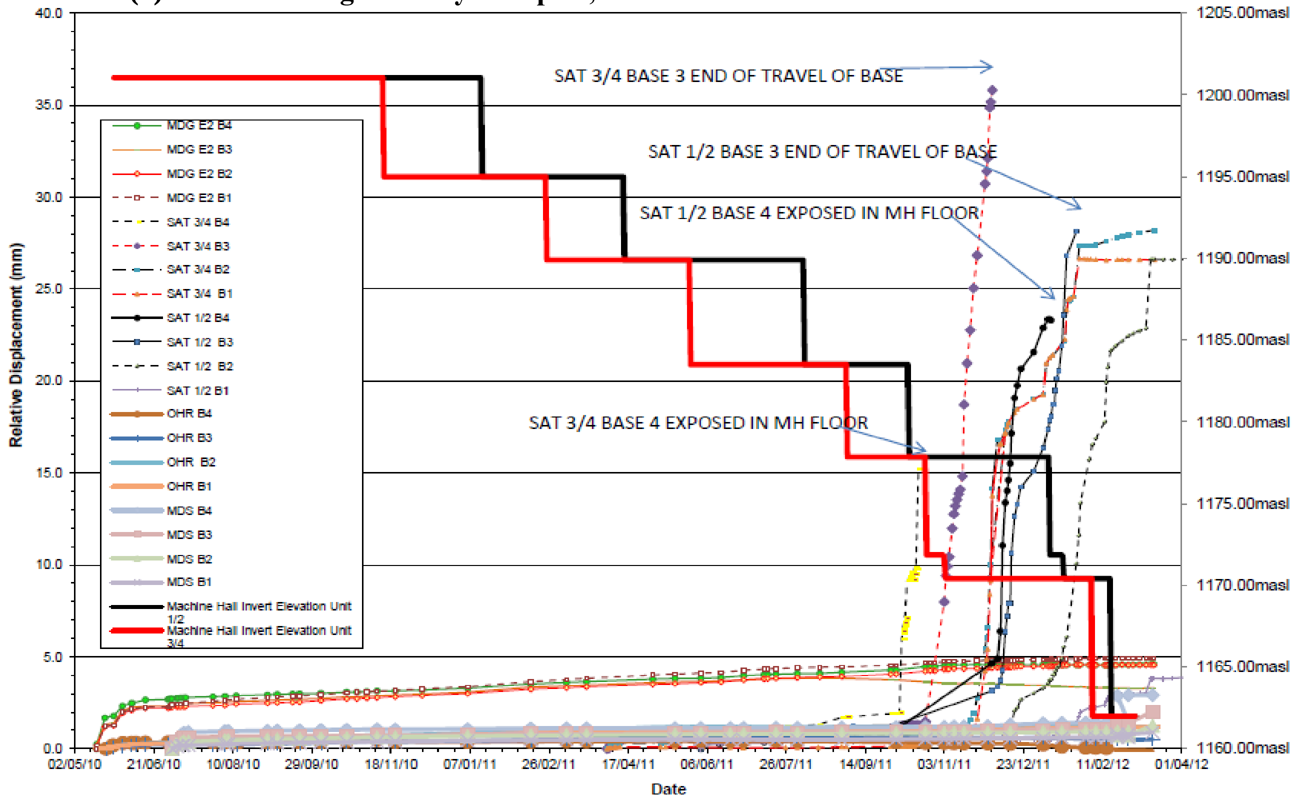
The Phase2 models set up during detailed design of the Ingula power cavern excavation and support were revised towards the end of excavation of the main power caverns to:

- Better reflect the actual geology encountered in Units 1&2 (Figure 10) and Units 3&4 along the installed instrumentation arrays, including modelling of the dyke with sheared contacts which intersects both Ingula power caverns at an oblique angle.
- The model staging (or excavation and support sequencing) of these Phase2 base models were revised to simulate the actual excavation and support installation sequence followed during cavern construction.
- Recalibration of the Phase2 models, taking into account available instrumentation and convergence monitoring data collected during cavern excavation.

The model calibration was carried out using the Phase2 model for the design section through Units 1&2 with a 'best fit' model convergence, compared to actual cavern convergence measured during construction, obtained by implementing the parameter changes listed in I. The MPBXs used for the purposes of this model calibration are shown schematically in Figure 10.

The changes in bedding and joint strengths for jointing in Karoo strata as outlined in Table I followed from a consideration of bedding and joint persistence based on available joint mapping information from the area of the Ingula power caverns. These open bedding planes and joints are not continuous in the direction of dip as can be seen in the diagram presented in Figure 10, with small 'rock bridges' comprising intact rock evident between the ends of adjacent discontinuity planes as well as undulations in bedding planes and joints, all of which will require shearing of intact rock during shear movements on these discontinuities.

(a) Main Drainage Gallery Complex, Crown MPBX's



(b) Main Drainage Gallery Complex, Invert MPBX's

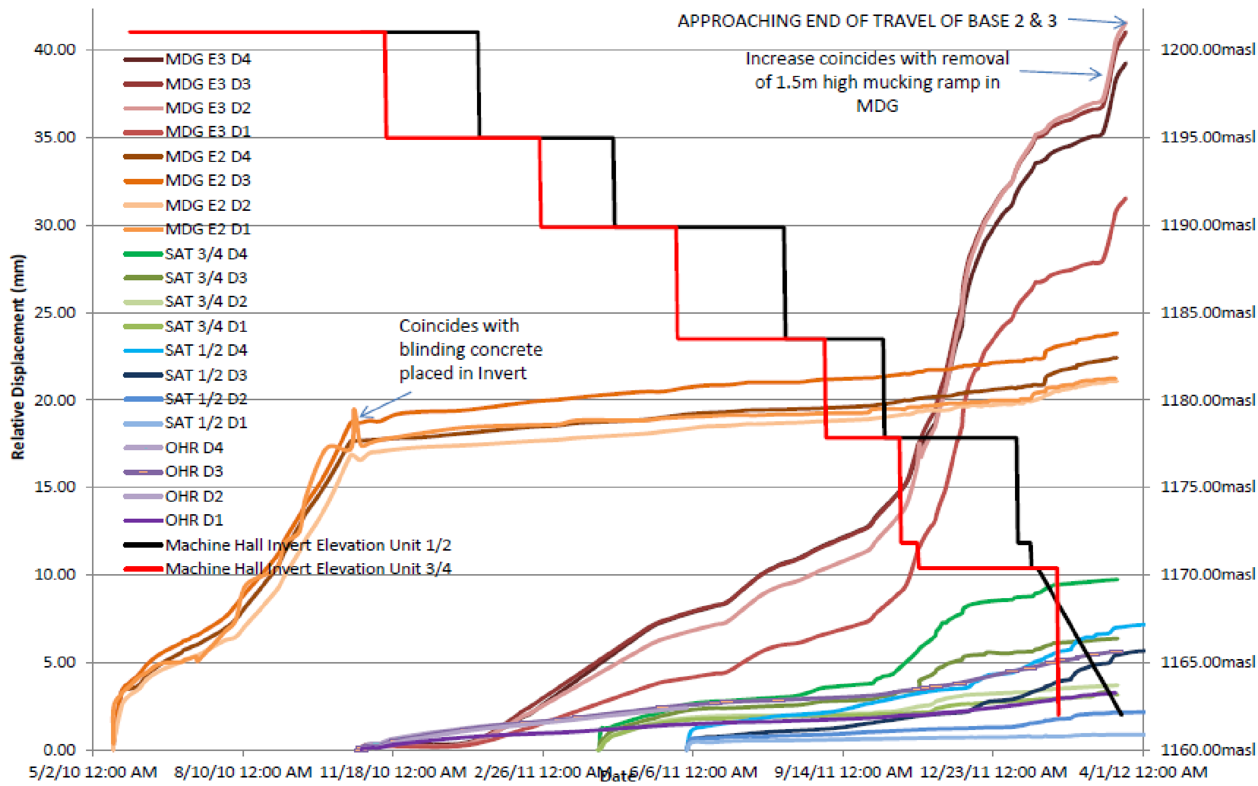


Figure 9. MDG Complex Displacement during MH Excavation

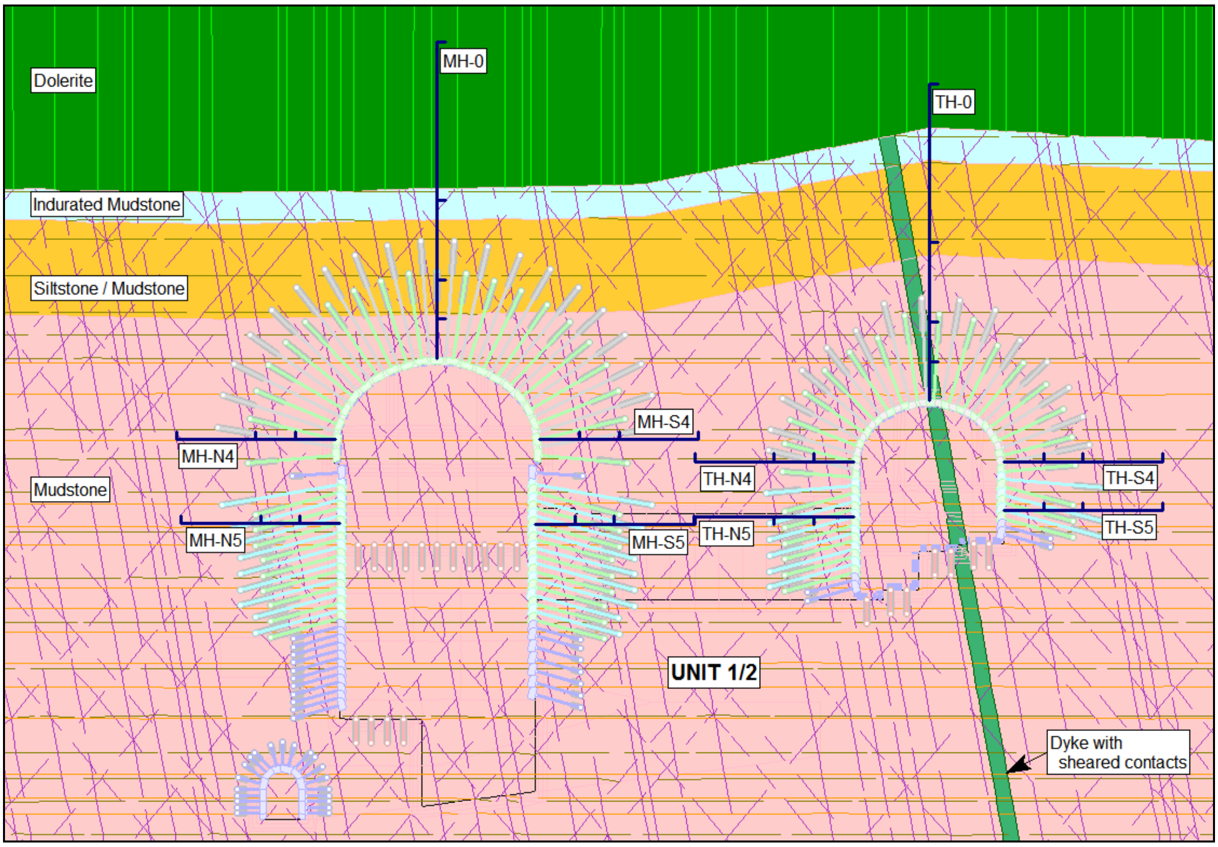


Figure 10: Geological Design Section for Units 1&2 (not to scale)

Table I. Parameter Calibration, Units 1&2

Model Parameter		**Original Design Value**		**Calibrated Model Value**
Geological Strength Index, GSI		70		100
Ratio of horizontal to vertical stress, K-ratio		1.0		0.7
Intact rock modulus, E_i	mudstone:	19.0 GPa		21.0 GPa
	siltstone / mudstone:	22.5 GPa		25.0 GPa
Joint shear stiffness, K_s	bedding:	1 330 MPa/m		2 500 MPa/m
	jointing in Karoo strata:	1 490 MPa/m		2 500 MPa/m
Joint normal stiffness, K_n	bedding:	9 750 MPa/m		15 000 MPa/m
	jointing in Karoo strata:	6 490 MPa/m		15 000 MPa/m
Bedding strengths	peak strength:	friction, ϕ	28.5 degrees	28.5 degrees
		cohesion, c	0.12 MPa	1.45 MPa
		tensile strength, σ_t	0 MPa	0.64 MPa
	residual strength:	friction, ϕ	24.2 degrees	28.5 degrees
		cohesion, c	0 MPa	0.12 MPa
		tensile strength, σ_t	0 MPa	0 MPa
Joint strengths	peak strength:	friction, ϕ	23.6 degrees	23.6 degrees
(for jointing in Karoo strata)		cohesion, c	0.1 MPa	4.5 MPa
		tensile strength, σ_t	0 MPa	1.93 MPa
	residual strength:	friction, ϕ	22.2 degrees	23.6 degrees
		cohesion, c	0 MPa	0.1 MPa
		tensile strength, σ_t	0 MPa	0 MPa

In the case of open bedding planes, such 'intact rock bridges' have been estimated to constitute approximately 5% of what would otherwise have been a throughgoing bedding plane. In case of joints which are less persistent than bedding planes, intact rock bridging in the plane increases to about 15%. For model calibration, it was assumed that the same percentage of rock bridges will also exist in the strike direction of bedding planes and joints.

The above adjustment in bedding and joint strengths countered the effect of infinitely long bedding planes and joints along strike in the two-dimensional (2D) Phase2 models. Without this adjustment, very large rock wedges formed / mobilised in the cavern crowns and sidewalls in the models. By including this adjustment, the size of rock wedges forming in the cavern crowns and sidewalls of the models reduced to that of typical rock wedges which were identified in the cavern crown ([3], also see discussion above) and sidewalls during construction.

The installation of the various MPBX's relative to the actual excavation and support sequence was also taken into account in the calibration process.

The calibrated parameters listed in Table I were then also used in the Phase2 model for the design section through Units 3&4.

Table II gives a brief comparison of actual crown and sidewall convergence measured up to the end of construction of the Ingula caverns, with that obtained from the calibrated Phase2 models. The colour scheme adopted in Table II gives an indication of the extent of deviation between actual measurements and modelled values. Red shaded values indicate that measured convergence exceeds modelled convergence at a location; blue shaded values indicate that modelled convergence exceeds measured convergence. The darker the shading, the larger the deviation. The modelled results are generally within 5 to 10 mm from the measured values, with deviations larger than this only noted at two MPBX locations as follows:

- Units 1&2, MPBX TH-S5: Jointing in the southern sidewall of the transformer hall formed a wedge in the Phase2 model whereas no such sidewall wedge was identified at this location in the transformer hall underground. Both Phase2 models appear to overestimate convergence of the southern sidewall in the transformer hall to some extent but generally not excessively so.
- Units 3&4, MPBX MH-N5: A large wedge which formed between the dolerite dyke and the northern sidewall of the machine hall resulted in measured convergence of approximately 25 mm at this location underground. The Phase2 model did not give the same amount of deformation for this location. The way the dolerite dyke was modelled in the calibrated Phase2 model in all likelihood does not accurately enough reflect the actual dyke geometry underground, resulting in a more stable sidewall in the Phase2 model.

Table II. Model versus Actual Crown and Sidewall Convergence

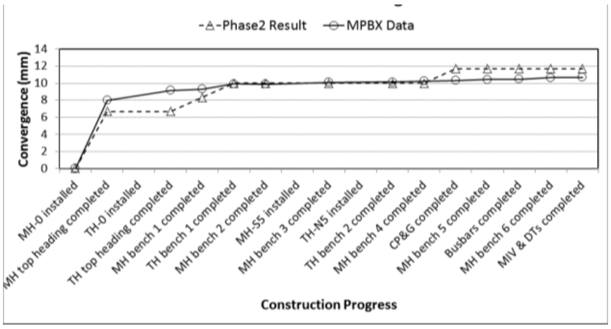
MPBX No.	MPBX Location	Convergence at Units 1&2		Convergence at Units 3&4	
		Actual (mm)	Modelled (mm)	Actual (mm)	Modelled (mm)
MH-0	MH crown, on centreline	11.1	11.7	11.5	11.7
MH-N4	Northern top heading of MH, just above crane beam level	11.7	10.0	7.7	18.4
MH-S4	Southern top heading of MH, just above crane beam level	8.6	5.0	9.1	3.3
MH-N5	A few metres above the operating floor level on northern MH sidewall	9.2	3.3	**24.7**	**1.6**
MH-S5	A few metres above the operating floor level on southern MH sidewall	19.9	10.0	15.5	5.0
TH-0	TH crown, on centreline	12.2	13.3	12.1	10.0
TH-N4	Northern sidewall of TH, at spring line level	12.3	20.0	7.1	11.7
TH-S4	Southern sidewall of TH, at spring line level	9.7	15.0	3.1	11.6
TH-N5	A few metres above the operating floor level on the northern TH sidewall	48.2	41.7	7.1	6.7
TH-S5	A few metres above the operating floor level on the southern TH sidewall	**2.6**	**23.3**	1.7	6.7

Actual MPBX measurements for the five MPBX installations highlighted by dark outline in Table II are compared in more detail in Figure 11, graphed against construction sequence.

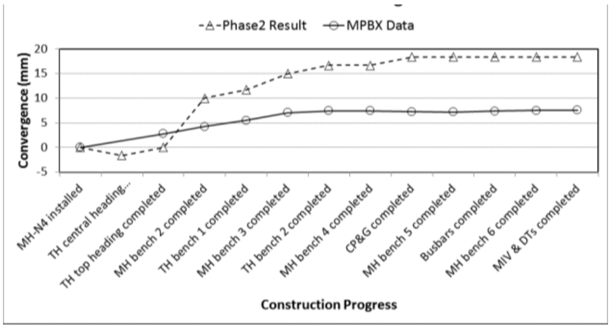
As can be seen from these graphs, some of the Phase2 model convergence results give an almost exact replica of actual convergence observed during construction. In other instances, the general trend of movements / convergence between the Phase2 model and what was observed underground is similar but the end values are different.

It is clear from this calibration exercise that the existence of local rock wedges in the cavern crown and / or sidewalls at the point of measurement – whether in the Phase2 model or in the actual cavern underground – have a significant effect on the total convergence noted at that location. Typically, where we have a rock wedge both underground and in the model, the comparison is good. The same can be said where a rock wedge is absent in both cases. However, where we have a rock wedge in one but not in the other, the difference in convergence is significant, with the convergence in the one with a rock wedge typically about twice that of the one without the rock wedge.

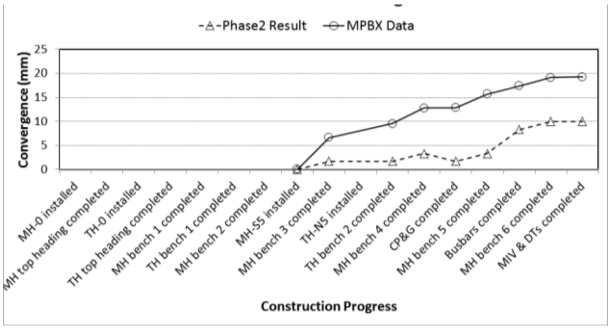
(a) MH Crown Centre Line, Units 1&2 – MPBX MH-0



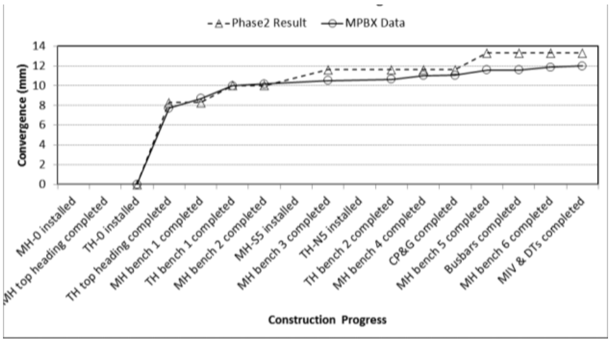
(b) MH Side Heading Above Crane Beam Level, Units 3&4 – MPBX MH-N4



(c) MH Sidewall Above Operating Floor Level, Units 1&2 – MPBX MH-S5



(d) TH Crown Centre Line, Units 1&2 – MPBX TH-0



(e) TH Sidewall above Operating Floor Level, Units 1&2 – MPBX TH-N5

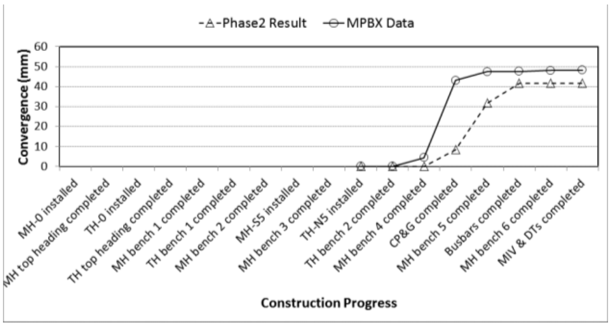


Figure 11: Actual versus Remodelled Convergence for select MPBX's

With reference to Table II both Phase2 models appear to somewhat underestimate sidewall convergence in the machine hall (more red shading) while generally overestimating sidewall convergence in the transformer hall (more blue shading). Crown convergence on centreline in both caverns is well reproduced by the Phase2 models.

Nonetheless, despite these differences, the above calibration was considered sufficiently representative of actual convergence measured in the Ingula caverns to allow construction defects to be modelled and the impact of such defects on long term cavern convergence and stability to be assessed.

Construction Defects [8]

Defect 1: Invert Floor Dowels

Galvanised invert floor dowels were installed during construction instead of double corrosion protected invert floor dowels as was specified in the cavern design. Potential floor heave long term in case of significant corrosion of these floor dowels was a concern given this construction deviation [7].

The invert floor dowels were included in the original design to counter significant floor heave during construction as well as to reinforce these excavation inverts during construction against traffic loading effects which may result in loosening of the rock mass below the excavated invert prior to casting final concrete floor slabs.

The potential effects of this construction defect were investigated by removing all the floor dowels installed in the erection bay floor and MIV pit floor in the machine hall as well as all the floor dowels in the transformer hall at the end of construction using the calibrated Phase2 models. The results of this modelling work is summarised in Table III.

Table III. Floor Heave in Phase2 Models on Invert Dowel Removal

Location	Additional Vertical Displacement (mm)	
	Units 1&2	**Units 3&4**
MH operating floor / erection bay floor	< 0.8 mm	no change
MH MIV pit floor	no change	no change
TH operating floor	< 0.8 mm	no change
TH cable & pipe gallery invert	< 0.8 mm	< 0.8 mm

The modelling results showed that these invert floor dowels have very little effect long term given that floor heave generally will already have occurred by the time these floor dowels have corroded away - either initially, during construction, or later on as a result of longer term creep deformations which are generally expected to only last about 6 months to one year after excavation of the turbine pits have been completed [4].

The installation of galvanised floor dowels in the Ingula power cavern inverts instead of double corrosion protected floor dowels as was specified in the design, will therefore have no significant impact on the long term stability of structural foundations and / or relative settlement (or heave) of final concrete floors at erection bay level in the machine hall or at operating floor level in the transformer hall.

Defect 2: Cable Anchor Hole Deviation

Significant drill hole deviations were noted during construction of the Ingula caverns, specifically during excavation and support of the northern sidewall of the machine hall, with cable bolt holes having deflected into the penstock tunnels immediately upstream of the cavern [7]. Survey of a subsequent check hole suggested that such deviation was in all likelihood a widespread occurrence during general installation of long anchors in the Ingula caverns.

Drill hole deviations noted generally occurred as a result of: (a) incorrect setting-up resulting in holes being drilled at incorrect angles to start off with; and (b) deviation from the start inclination due to poor drilling control and / or anchorage of drill rigs.

Potential damage to double corrosion protection provided in the end anchorages of permanent anchors may have resulted due to the following:

- Anchor holes being drilled deviating into end anchorages of adjacent cable anchors already installed, damaging corrosion protection and, possibly, cables in end anchorages.
- Inadequate corrosion protection due to fixed length tendons not being located centrally in the grout filled hole.
- Damage to plastic sheathing as a result of anchors being pushed down spiralling holes, etc.

These construction deviations may result in long term corrosion of individual anchorages with potential instability of the cavern crown or sidewall being cause for concern given this construction deviation [7].

All permanent cable anchors installed in the Ingula power caverns passed the specified acceptance tests in accordance with the requirements of BS 8081 [9], [7]. However, these acceptance tests were carried out on horizontal rows of installed cable anchors before drilling of the next horizontal row of anchors to be installed below the anchor row being tested. A successful acceptance test as such therefore does not provide a complete guarantee that an anchorage is still in good working order given that drilling damage to that anchorage, where it has occurred, will have happened after acceptance testing of the anchor. Successful acceptance test results do however indicate that, where cable anchors have not been damaged as a result of drill holes deviating into adjacent anchorages, the load capacity of such anchorages have been proven beyond any doubt.

Installing cable anchors at angles with the horizontal steeper than what was specified in the design will have resulted in a reduction in the overall support system capacity. The risk in this case being that larger key blocks and sidewall wedges will not be as well retained as would have been the case had the design been executed accurately. Effective fixed anchorage lengths beyond the critical failure plane will be reduced in affected permanent cable anchors, resulting in reduced factors of safety for anchorages thus affected. The extent to which these overall factors of safety have been reduced, cannot be quantified given a lack of relevant construction data. Mitigation against the increased residual risk profile in these cavern excavations can only be achieved by maintaining cavern instrumentation and monitoring installations for the design life of the scheme, to continue monitoring the stability of the caverns, with appropriate remedial support to be installed should cavern crown or sidewall instability in whatever form present itself in the long term.

Damaged anchorages were modelled in the calibrated Phase2 model by staged removal of cable anchors, to assess the potential impact this may have on excavation behaviour. In the Phase2 model for Units 1&2, staged removal of anchors resulted in a steady increase in convergence, with most convergence occurring in the transformer sidewalls as well as at crane beam level in the southern sidewall of the machine hall. However, removal of a single permanent cable anchor in the northern sidewall of the transformer hall in the Phase2 model for Units 3&4 resulted in mobilisation of a large sidewall wedge, also causing two adjacent cable anchors retaining the wedge to fail in tension. The Phase2 model run did not converge for this stage – that is, the transformer hall excavation was thereafter not stable anymore.

The Phase2 modelling results confirm what is to be expected, namely that:

- Long term yield failure of some damaged cable anchors may have little impact on the stability of the cavern excavation, with rock loads being shed to adjacent anchors as damaged anchors yield and fail.
- However, long term yield failure of a damaged anchor installed at a critical location, may result in mobilisation of a large key block or wedge in the cavern roof or sidewall, while working loads in adjacent anchors may already be so high that the shedding of additional rock loads to such adjacent anchors may simply result in progressive failure of these already overloaded anchors.

As in the case of anchor holes which have been drilled at incorrect angles, the extent to which overall factors of safety will reduce long term as damaged anchorages yield or fail, either (1) as a result of corrosion due to damage to plastic sheaths and / or (2) overloading or fatigue of damaged cable strands, cannot be quantified given a lack of relevant construction data. Mitigation against the increased residual risk profile in these cavern excavations can only be achieved by maintaining cavern instrumentation and monitoring installations for the design life of the scheme, to continue monitoring the stability of the caverns, with appropriate remedial support to be installed should cavern crown or sidewall instability in whatever form present itself in the long term.

Any such remedial work and associated costs which may be required long term will in all likelihood impact on operation of the Ingula scheme and on associated financial revenues.

As in the case of anchor holes which have been drilled at incorrect angles, the extent to which overall factors of safety will reduce long term as damaged anchorages yield or fail, either (1) as a result of corrosion due to damage to plastic sheaths and / or (2) overloading or fatigue of damaged cable strands, cannot be quantified given a lack of relevant construction data.

Mitigation against the increased residual risk profile in these cavern excavations can only be achieved by maintaining cavern instrumentation and monitoring installations for the design life of the scheme, to continue monitoring the stability of the caverns. Appropriate methods of remediation, if required long term, are still being investigated to further mitigate any risks in this regard.

Conclusions

An extensive programme of geotechnical instrumentation and monitoring was carried out during construction of the Ingula hydro power caverns, to validate design assumptions and monitor long term creep effects.

Key aspects of instrumentation and monitoring work carried out during excavation of the Ingula caverns have been presented and actual convergence data and support performance discussed. Regular reading of monitoring instruments during construction allowed for rapid assessment of problems encountered during cavern excavation.

Differences between predicted convergence at design stage and monitoring results and measurements during construction necessitated a recalibration of numerical models used for design, to assess the sensitivity of changes in the construction sequencing and geotechnical parameters encountered during construction on the models and derive a new set of predicted convergence.

Finally, these calibrated numerical models were used to model specific construction defects, to allow an assessment of the impact of such defects on long term cavern convergence and stability.

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PRESENTER'S DETAILS

Full Name:	GERHARDUS JOHANNES KEYTER
Company:	*GeoStable SA*
Designation:	Geotechnical Engineer
Qualifications:	B Eng (Civil), University of Pretoria, South Africa
	MSc DIC (Soil Mechanics), Imperial College of Science Technology and Medicine, London, UK

SHORT HISTORY OF PREVIOUS EXPERIENCE:

Gerhard Keyter has specialised in geotechnical engineering in civil engineering and mining. He has been involved in the design of open pit mines and other surface excavations as well as underground mining over the past 18 years and has spent 6 years working on the design and construction of the underground works of ESKOM's Ingula Pumped Storage Scheme. He is co-founder of the firm *GeoStable SA* and has been working as an independent consultant in the civil geotechnical and mining engineering fields since mid-2009.