Brisbane Airport Link Project – Design and Construction of the 26 m Span Kidron Caverns: Comparing Monitoring Data with the Design

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ABSTRACT
The Brisbane Airport Link Project (AirportlinkM7) includes the design and construction of more than 12.5 km of tunnels, with the Kidron interchange including four ramp tunnels that intersect with mainline tunnels and result in caverns with spans of up to 26 m. These caverns are in highly variable ground conditions with less than 35 m of cover underlying residential neighbourhoods and commercial properties. The Kidron caverns were constructed using the sequential excavation method (SEM). Owing to complex geometry and excavation sequence, the caverns were designed based on numerical modelling methods, including both two- and three-dimensional models. These numerical methods were used to develop a suite of support types with excavation sequences for the expected range of ground conditions. A combination of methods that included in-tunnel monitoring (convergence monitoring and extensometers), probe holes, endoscope probes, face mapping and ground settlement measurements were used to verify performance of the tunnel support systems and to select and refine the appropriate support system for each section of the caverns. The construction monitoring data indicated that numerical modelling can provide a good basis for predicting the performance of large-span underground openings in highly variable ground conditions. This paper presents a comparison of the performance of the caverns predicted by state-of-the-art numerical models and the actual measured performance.

PROJECT DESCRIPTION
The Airport Link Project (AirportlinkM7) is a 6.7 km long motorway (toll road) in the northern suburbs of Brisbane, Australia. This project, Australia’s largest road infrastructure project to date, was developed by the Queensland Government via the special purpose company vehicle, City North Infrastructure Pty Ltd (CNI), through a public private partnership (PPP). Parsons Brinckerhoff (PB) and Arup in joint venture (PBAJV) were engaged to deliver the detailed design for this PPP design-and-contract project. The motorway connects with the northern outlet of the Clem Jones Tunnel (Clem7), which crosses beneath the Brisbane River and the western end of the East–West Arterial Road, which leads to the Brisbane Airport. Airportlink M7 completes the motorway infrastructure for north–south travel through Brisbane (Figure 1). The project includes 12.5 km of tunnelling works, consisting of 5.1 km of twin mainline tunnels, access ramps, connecting caverns and cross passages. Much of the tunnel alignment underlies residential neighbourhoods and light commercial properties with an overburden of less than 40 m. The excavation of the tunnels was accomplished using tunnel boring machine (TBM), roadheader excavation and drill-and-blast methods. The $5.6 B project was delivered in four years and opened to traffic in July 2012.

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Four on-and-off-access ramps were constructed underground at the Kedron interchange, connecting to three- and four-lane sections of the motorway, and these sections required large caverns. Two parallel caverns accommodate the east-bound on-ramp and the west-bound off-ramp, which are part of the Kedron connections with Lutwyche Road (see Figure 1). These caverns were excavated by roadheaders using the sequential excavation method (SEM). This paper focuses on the design and construction of three of the four east-bound Kedron caverns associated with the east-bound on-ramp – namely KEC-01, KEC-02 and KEC-03. The locations of the different sections of the Kedron caverns are shown in the insert of Figure 1.

GEOLOGY AND GROUND CHARACTERISATION

The geology along the east-bound Kedron caverns is characterised by subhorizontally bedded volcanic and sedimentary sequences consisting of lower and upper Brisbane tuff, conglomerate, sandstone and siltstone/mudstone, and overlying residual soils and alluvial deposits. Thin layers of ash, which have weathered to clay, are also present within the siltstone/mudstone. Figure 2 shows the longitudinal geological profile for the east-bound Kedron caverns. The minimum ground cover over the caverns is approximately 30 m. The groundwater level is about 5 to 10 m below the ground surface (Figure 2). Artesian water within the lower tuff was expected but not encountered during
construction. The predominant features of the geological units encountered along the Kedron cavern alignment are characterised as follows (Parsons Brinckerhoff and Arup joint venture (PBAJV), 2009, 2010):

- Lower and Upper Brisbane tuff (tuff) consists of highly fractured to massive, moderately weathered to fresh, and medium to very high strength (according to International Society for Rock Mechanics (ISRM), 1981) porphyritic welded tuff. The tuff contains predominantly three joint sets (one subhorizontal and two subvertical) with joint spacing ranging from 0.2 to 3.0 m. Thin (10–20 mm thick), near-horizontal layers of clay are present at various depths in the upper tuff unit.

- Conglomerate/sandstone (CD) consists of massive to highly fractured, highly weathered to fresh, and low to high strength (according to ISRM, 1981) well-cemented sedimentary rock. The rock mass discontinuities include one subhorizontal bedding discontinuity and two predominantly subvertical joint sets with spacing ranging from 0.1–2.0 m. Siltstone layers 2–3 m thick occur on both the upper and lower interfaces of this unit.

- Siltstone dominant sedimentary rock (SD) consists of laminated/very thin to medium bedded, weathered to fresh, and very low to medium strength (according to ISRM, 1981) rock. Bedding planes with subhorizontal dips and occasional clay filling (10–20 mm thick) are the major defects in this unit. The siltstones also contain a number of micro-faults with offsets from the beds ranging from a few millimetres up to 1 m.

- Overlying residual soils consisting of silty, sandy, or gravelly clay and alluvial deposits consisting of silty sand and clayey gravel are typically between 15 and 20 m thick and come within 10–15 m of the crown of the caverns.

Ground characterisation was carried out to define rock mass classes (RMCs) for each of the geological units anticipated to be encountered within the caverns and to estimate design parameters for each of the RMCs based on an evaluation of the available geotechnical data (PBAJV, 2009). Each geological unit was divided into four or five RMCs based on predominant lithology, fracture frequency and condition, rock strength, and degree of weathering.

Design rock mass parameters for each RMC were derived according to Hoek, Carranza-Torres and Corkum (2002); and Marinos, Marinos and Hoek (2005) using geological strength index (GSI) and intact rock strength. As rock mass parameters are scale-dependent, two sets of rock mass parameters were developed to cater for two different scales: one set was for the tunnel scale used in continuum analyses, and the other for an approximately 1 m² scale used in discontinuum analyses. For design purposes, two subsets of parameters were derived for many of the RMCs representing the expected rock mass characteristics: design case (DC) and the lower-bound (LB) rock mass characteristics. The rock mass and soil properties used in the design are presented in Table 1.

### KEY CONSIDERATIONS IN DESIGN AND CONSTRUCTION

The east-bound Kedron caverns described herein have three different profiles, ranging in width from 19 to 26 m and varying in height from 11 to 14 m. The ground cover over the east-bound caverns ranges from 30 to 35 m, with only 15 m of the cover consisting of rock. A south-bound TBM tunnel was excavated adjacent to the east-bound Kedron caverns following the excavation and temporary initial support of these caverns. The rock separation between the caverns and the TBM tunnel is about 15 m. Given the low rock cover relative

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**TABLE 1**

Rock mass and soil properties.

<table>
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<tr>
<th>Rock mass class</th>
<th>Constitutive model</th>
<th>Assumed ground conditions</th>
<th>UCS (MPa)</th>
<th>GSI</th>
<th>Deformation modulus (MPa)</th>
<th>Poisson’s ratio</th>
<th>Friction angle (°)</th>
<th>Cohesion (kPa)</th>
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<td>25</td>
<td>10</td>
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<td>N/A</td>
<td>50</td>
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<td>35</td>
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</table>

Notes: DC – design case; LB – lower bound; N/A – not applicable; UCS – uniaxial compressive strength; GSI – geological strength index.
to the span of the caverns, existing surface infrastructure above the tunnel, and impact of the adjacent TBM tunnel passage, ground settlements were a key consideration in the development of the design for the tunnel excavation sequence and temporary initial support.

As shown in Figure 2, highly variable geological conditions, both in the cavern face and above the tunnel, were expected to be encountered along the caverns. The different rock units were expected to exhibit different behaviours during excavation, requiring a support solution that could accommodate these varying behaviours as well as controlling surface settlements.

Three design cross-sections, one for each of the three cavern profiles, were selected to determine the excavation sequence and temporary initial support requirements for the expected range of ground conditions. These geological conditions and discreet, site-specific features (such as zones of increased joint frequency in the upper tuff unit present above the crown and weak clay layers in the siltstone unit at the springline) were considered in the design. Numerical modelling showed that these features would influence tunnel behaviour and support performance. Figure 3 shows a typical geological cross-section used for KEC-01.

**Design methodology**

Owing to variable and weak ground conditions, low ground cover, existing surface infrastructure, and complex geometry and excavation sequence, the Kedron caverns were designed using numerical methods, including both two- and three-dimensional models. Empirical methods based on Q and rock mass rating (RMR) systems (Barton, Lien and Lunde, 1974; Bieniawski, 1989) were not considered applicable for the anticipated ground conditions and sizes of the caverns. Empirical methods could not specifically account for the effects of structural features (such as weak clay layers, joints and micro-faults). Numerical models, however, were used to evaluate the effects of complex lithological stratigraphy and geological features, staged construction, and three-dimensional tunnel geometries.

Results from numerical analyses – using a range of rock mass parameters and various excavation sequences and support types – were the basis for defining the requirements of excavation sequence and temporary support types for the Kedron caverns.

**Excavation sequence and support types**

Given the size of the caverns and the expected variability, and in some cases poor ground conditions, the caverns were designed to be excavated in multiple stages. First, the top heading would be excavated to full span using two to three stages (drifts), followed by the bench/invert excavation. The excavation sequence and temporary support requirements for the three caverns varied because of the differences in ground conditions and cavern geometries. The main difference in excavation sequence among the three caverns was how the top heading was removed. For the KEC-01 cavern, the top heading was divided into three drifts (two side drifts and a central pillar). The main reason for the three drifts was to mitigate potential instability associated with the cavern's large span (26 m) and with the anticipated poor ground conditions. For the KEC-02 cavern, which was slightly smaller in size and where better ground conditions were expected, the top heading was split into two drifts. Similarly, because of the smaller size and better expected ground conditions in the area of the KEC-03 cavern, a full face top heading was the basis for the design.

Temporary support was designed to ensure stability at all stages of construction and to minimise potential risk of building damage caused by excavation-induced ground settlements. The main elements of the temporary support system consisted of steel fibre reinforced shotcrete, lattice girders and rock and/or cable bolts. Spiles and face dowels were additional support measures and installed as needed, based on the ground conditions encountered.
The shotcrete, with a 28-day design strength of 32 MPa, was designed to act in composite action with the rock reinforcement system. Rock bolts with an ultimate capacity ranging from 310 kN to 550 kN and cable bolts with an ultimate capacity of 550 kN were used. Bolt lengths ranged from 3.7 m to 8.0 m. Rigid rock bolts were used for anchor lengths up to 6 m, and cable bolts were used for anchor lengths in excess of 6 m. In the larger KEC-01 cavern profile, shorter rock bolts were installed during the initial phase of excavation (i.e., during excavation of the two side drifts) and longer cable bolts were installed prior to widening out of the cavern excavation (i.e., removal of the central pillar). The staging of the bolting in KEC-01, short then long bolts, was preferred by the contractor as it simplified construction in the relatively small/tight side drifts and reduced cycle times at the face to enable quicker access to the subsequent smaller KEC-02 cavern profile (see Figure 4).

Two main support types (STs) for each section of cavern were developed to address the range of ground conditions at various sections along the alignment (see Figure 2). Support type 3 was required for the DC conditions, while Support type 4 was required for the LB conditions (Table 2). Actual support types installed during construction were chosen based on the observed ground conditions and the selection criteria established during the design. Additional support types were developed during construction based on the ground conditions actually encountered and the results of the mapping and monitoring performed during excavation, as discussed below. Figure 5 illustrates a typical cross-section of the KEC-01 cavern, showing the excavation sequence and support type. Both STs, type 3 and type 4, included additional tool box items, such as face dowels or spilling that were required if specific geological conditions or behaviours were observed during excavation.

**Cross-cut for schedule benefit**

Excavation of the Kedron caverns was on the critical path of the construction program. Because of a tight schedule and the complex ground conditions, use of a robust excavation sequence and temporary support system was key to minimising the risk of production delays during excavation of the caverns. To achieve an efficient construction sequence

<table>
<thead>
<tr>
<th>Design section</th>
<th>Top heading</th>
<th>Bench</th>
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<td>200 (Support type 3)</td>
</tr>
<tr>
<td></td>
<td>450 (Support type 4)</td>
<td>300 (Support type 4)</td>
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<tr>
<td>KEC-02</td>
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<tr>
<td></td>
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</tr>
</tbody>
</table>

**Fig. 4** — Eastbound Kedron cavern cross-cut (plan view).

**Fig. 5** — Typical excavation sequence and support type for KEC-01.
that met the goals of the contractor, a temporary cross-cut was excavated at the interface of KEC-01 and KEC-02 (see Figure 5). The location of the cross-cut was determined by the contractor to facilitate simultaneous construction of the top headings of both KEC-01 and KEC-02. The cross-cut was about 28 m long, 9 m wide and 15 m high. In order to provide a stable roof for advancing the KEC-02 drifts and excavation of the remaining KEC-01 top heading drifts, the shape of the cross-cut was designed to form two arches, one longitudinal (parallel to the cross-cut) and the other transverse (parallel to the caverns). The crown of the arch, ie the height, was designed to be just above the crown of the excavation line for the larger cavern KEC-01. This ensured that the initial support of the cross-cut would not have to be removed during the break-out of KEC-01’s remaining drifts and also eliminated the risk of the cross-cut support interfering with installation of the final lining.

**EVALUATION OF CAVERN STABILITY AND TEMPORARY SUPPORT DESIGN**

Overall stability of the Kedron caverns was evaluated using FLAC (Version 5.0, Itasca International Inc, 2005) and UDEC (Version 4.0, Itasca International Inc, 2006a) in two-dimensional analyses and using FLAC3D (Version 3.1, Itasca International Inc, 2006b) in three-dimensional analyses using both sets of DC and LB rock mass parameters. The key aspects of the temporary support design that were investigated by these analyses include: ground reaction curves (GRC); loads versus capacity for all temporary support elements; wedge support and face stability; surface settlements; excavation sequence and support requirements for the cross-cut excavation; and impact of the adjacent TBM tunnel excavation.

In these analyses, the ground was modelled using Hoek-Brown elasto-plastic constitutive models, Hoek-Brown strain-softening constitutive models, where applicable, and Mohr-Coulomb elasto-plastic models. Higher quality rock conditions were modelled using the Hoek-Brown (Hoek, Carranza-Torres and Corkum, 2002) strength parameters. For the weaker (UCS <10 MPa) or poorer quality (GSI <30) RMCs, the design parameters were assigned Mohr-Coulomb strength parameters. The Mohr-Coulomb strength parameters were derived by calculating a Hoek-Brown curve using intact strength and GSI, then fitting the Mohr-Coulomb curve at a confining stress corresponding to the depth at which the RMC occurs. Shotcrete was modelled by means of beam elements with relevant axial and bending stiffness properties. The shotcrete strength and stiffness used in the models were modified according to the stage of excavation considering both the strength versus time relationship and the rate of excavation advance. The stiffness properties of the shotcrete were adjusted at the appropriate stage in the numerical models to account for the hardening of the shotcrete relative to the advance rate. Fully grouted rock dowels were modelled as cable elements with specified stiffness, bond strength, yield capacity, and defined length and spacing.

The load factors used for the design of the shotcrete lining were 1.35 and 1.0 for the DC and LB rock mass parameters, respectively. The allowable capacity of the rock bolts was assessed using 60 per cent of the ultimate tensile strength. A bond strength of 0.5 MPa was adopted for the bond capacity between the grout and the ground. This value represented a lower-bound for the various ground conditions along the caverns.

**Ground reaction curve and ground relaxation at tunnel face**

Ground reaction curves were generated for the two sets of rock mass parameters applicable to each of the three design sections to assess the ground response to tunnel excavation, as well as the appropriate two-dimensional stress relaxation factors that represent the three-dimensional stress redistribution occurring ahead of an advancing tunnel face. Figure 6 shows the GRCs for three design sections for both DC and LB conditions. Assuming that 30 per cent of the total

![Fig 6 - Ground reaction curves for KEC-01, KEC-02 and KEC-03 sections.](image-url)
elastic deformation occurs at the tunnel face (Panet, 1995; Kielbassa and Duddeck, 1991), the corresponding relaxation of in situ stresses was estimated to vary from approximately 30 to 40 per cent based on the GRCs generated for the caverns (see Figure 6). Therefore, temporary support was conservatively designed assuming that a stress relaxation of 30 per cent occurred prior to the support installation.

**Evaluation of performance of temporary support**

The loads in the shortcrete lining were evaluated in order to validate the design shortcrete thickness for each of the three sections and for both DC and LB ground conditions. The forces (thrust, bending moment and shear) developed in the lining at various stages of excavation were checked against the lining’s design capacity defined by a moment-thrust interaction diagram for the section (see Figure 7). These forces were calculated based on the full moment of inertia of the lining section of interest. In cases and locations where high moments (as indicated by the points falling outside the capacity envelope in Figure 7) were predicted by the models, an adjustment was made by reducing the moment of inertia of the shortcrete lining section by 50 per cent to account for the effect of cracking of the section. The forces developed in the shortcrete lining were then recalculated after this adjustment and in all cases the resulting forces were within the capacity envelope (see Figure 7). This approach effectively limited the tensile stress that would develop in the shortcrete lining and permitted plastic rotations and deformations to develop. Lining rotations were also investigated for critical cases and checked to ensure rotations did not exceed the allowable value which was defined by the shortcrete properties and the thrust acting in the lining at the section of interest. Where steel fibre reinforced shortcrete was used, the design was carried out in accordance with both the International Union of Laboratories and Experts in Construction Materials, Systems and Structures (RILEM) and the German Society for Concrete and Construction Technology (DBV) (RILEM TC 162-TDF, 2003; DBV, 2001, 2001). The design of conventional reinforced shortcrete was performed in accordance with Eurocode 2 (ENV 1992-1-1:1992). Results of the evaluation indicate that the shortcrete lining design for various support types would be adequate for the expected range of loading conditions at all stages of construction.

Rock bolt performance was validated in three different ways. First, maximum rock bolt loads at each excavation stage were compared to the bolt capacities. The bolt loads at all excavation stages were predicted to be less than the applicable bolt capacities (Figure 8). Second, for the side top heading excavations, analyses were performed considering that the bolts alone would provide the ground support. Bolt loads during these stages of tunnel excavation were also predicted to be lower than the bolt capacities confirming the proposed design. Third, the impact of the shear strains, especially those occurring in weak layers (clay seams), were investigated to assess their impact on the cable bolts. The failure mode of cable bolts in this condition would be tension failure. Although maximum shear strains of up to three per cent were predicted in weak layers, the proposed cable bolts have adequate shear capacity (Stjern, 1995).

**Surface settlements**

Settlement analyses were carried out for each design section. These settlement analyses evaluated the stress redistribution that occurs in response to tunnelling and did not assess the settlements that may occur because of compression or consolidation in response to groundwater drawdown. The settlements due to consolidation were expected to be small compared to those induced by tunnelling due to minimal depressurisation in overlying soils. The predicted settlement profiles over KEC-01 for the DC conditions are presented in Figure 9. These results also include the additional settlements that occur following the excavation of the adjacent TBM tunnel, indicating that the impact of TBM passage would be minimal.

**Effect of cross-cut excavation**

Three-dimensional FLAC analyses were performed to verify the stability of the cross-cut during excavation. These analyses accounted for the advance length, shortcrete strength as a function of age, and shortcrete thickness buildup sequence with the tunnel advance. The modelled excavation sequence
(stages) for KEC-01 and KEC-02 is illustrated in Figure 10. A photograph showing the nearly completed KEC-01 is shown in Figure 11. The modresults indicate that the proposed temporary support consisting of 450 mm thick shotcrete and 8 m long cable bolts installed in the cross-cut had adequate capacity to maintain the cross-cut stability during all stages of excavation.

CONSTRUCTION MONITORING AND CAVERN PERFORMANCE

During design, geological criteria (ie support selection criteria) were established to define corresponding support types for different combinations of RMCs in the invert, sides, crown and overburden as it was anticipated that the conditions would vary significantly along the caverns. Site geologists
The performance of the excavated cavern and installed support was monitored by geotechnical instruments installed during construction and observations recorded by site engineers/geologists. In-tunnel convergence was measured using survey arrays installed within the cavern; extensometers (installed from the ground surface and from within the tunnel) were used to monitor ground movements in the rock above the caverns; and surface settlements were measured using survey equipment. Figure 12 shows the typical locations and layout of surface and in-tunnel monitoring.

Support selection was confirmed by comparing measured support performance to that predicted by the numerical modelling through the use of trigger levels calculated during design. The trigger levels were used to establish a response...
protocol for mitigating excessive displacements prior to their impacting the stability of the caverns or the integrity of buildings and infrastructure at the surface. Two trigger levels were used: amber and red, reflecting increasing convergence and necessitating increased mitigation techniques.

In general, the measured cavern convergence was below trigger levels, especially the vertical displacements. In locations where the amber trigger levels were exceeded, the monitoring frequency was increased to determine if the deformation measurements were erroneous or if the tunnel deformations were stabilising. In the first (left) drift of KEC-01, deformations were higher than anticipated, requiring a review of the trigger levels and encountered ground conditions. The higher than expected ground deformations required the installation of additional cable bolts to stitch together a localised zone of faulted material. The convergence measurements stabilised after mitigation and the support in KEC-01 performed as predicted when the second drift and centre pillar were subsequently excavated.

Figure 13 shows the convergence monitoring data for instruments from a typical monitoring array at Chaineage 4932 in KEC-01 in comparison with trigger levels established for this section. These data also show the date of commencement of the various stages of excavation. The monitoring data show a total vertical convergence of approximately 20 mm following the completion of the first two stages of top heading excavation (see Figure 4), with the maximum measured vertical convergence increasing to approximately 30 mm following the third stage of heading excavation. The maximum measured horizontal convergence was approximately 12 mm following the completion of the first two stages of the top heading, and the maximum measured horizontal convergence increased to approximately 18 mm following excavation of the third stage of heading excavation. No significant additional movement occurred during the advance of KEC-02. The additional ground movements measured during the bench excavation were also negligible. These observed trends are very similar to the deformation patterns that were predicted by the numerical models.

The extensometers were used to corroborate the convergence monitoring and measure the amount of deformation occurring ahead of the cavern face. The maximum measured surface settlement was approximately 20 mm, and the majority of settlements were measured to be between 10 and 15 mm. There was no change in surface settlements during and after the bench excavation. Surface settlements never exceeded any trigger levels, and no impact to surface structures or infrastructure was observed.

The combination of observed geological conditions and support performance that were as good as or better than predicted allowed an optimisation of the temporary support design. The as-encountered geological conditions were used to develop revised numerical models which, in turn, were used to develop a third support type for KEC-03 along with associated deformation criteria and trigger levels. The following modifications were made to the support types developed prior to construction of KEC-01 and KEC-02: a reduction of welded wire mesh gauge; substitution of welded wire mesh with steel fibre reinforced shotcrete; and/or increased advance lengths. As the new and modified support types and excavation sequences were used, the support performance was continuously monitored to ensure that the deformation criteria were not exceeded. This approach provided a support solution that fulfilled the deformation requirements while providing the most cost and time effective solution possible.

CONCLUSIONS
The design of the temporary support for the east-bound Kedron caverns (profiles KEC-01 to KEC-03) required the application of two- and three-dimensional numerical analyses to address variable and weak ground conditions, low ground cover, and a complex excavation geometry and sequence. These analyses were used to develop a suite of support types and associated excavation sequences with support type selection criteria for the expected range of ground conditions. The appropriate support type installed in each section of the
caverns was selected based on the specified selection criteria and the observations from a probing, mapping and monitoring program that included conventional probe holes, endoscope probes, face mapping, tunnel convergence monitoring and ground settlement measurements. In all cases the measured performance was as good as or better than predicted. Based on the favourable performance and using the as-encountered geological conditions, the design of the temporary support was optimised. The experience gained from the design and construction of the Kedron caverns indicates that numerical modelling can provide a good basis for determining support requirements and predicting the performance of large-span underground openings in highly variable ground conditions. However, the design and construction of the east-bound Kedron caverns also highlights the importance of the observational approach during the construction of large span underground openings in variable ground conditions. The geological data gathered from strategic probing and face mapping and the measured performance of the excavations allowed the targeting of local support measures and the optimisation of the temporary support system for two sections of the east-bound Kedron caverns.

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