IMPLEMENTING OBSERVATIONAL METHOD IN TUNNELING

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INTRODUCTION

Brisbane's Airport Link Project (APL) is the biggest infrastructure project ever built in Australia. This 6.7km long electronic toll road has been constructed mainly underground between the city's airport and Central Business District (CBD) to reduce surface traffic and journey times significantly by avoiding 18 sets of traffic lights. It further connects the northern arterials to the CBD main network: the Clem 7 tunnel to the South, the Inner City Bypass to the East and West and local road networks, eases congestion and improves the quality of life in the overlying suburbs.

The first 2.5km of the twin 2 lane mainline tunnels from the airport were constructed using two 12.5m diameter TBM up to the Lutwyche caverns, which span up to 27 m in width, representing the widest road tunnel span in Australia. From the Lutwyche Caverns to Bowen Hills the remaining tunnel section was constructed by conventional mining techniques using roadheaders, rock bolts and sprayed concrete. Figure 1 shows the layout of the project.

The on- and off-ramps in the Kedron area provide a connection between the northern suburbs and the mainline tunnels approximately halfway through the journey in the form of an underground directional T-interchange. The caverns and the ramps were also constructed by conventional tunneling methods, necessitating in some areas. canopy tubes or spiled bars roof support due to the complex geology.

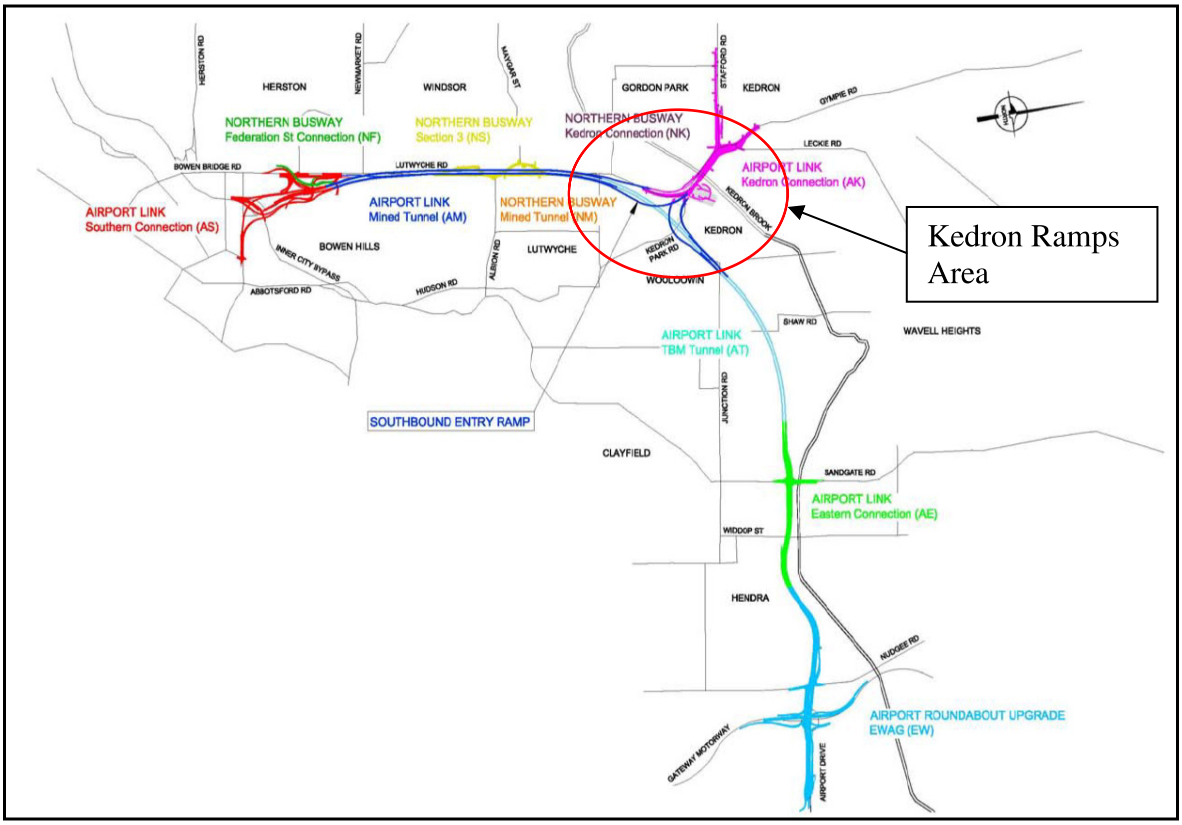


Figure 1: Layout of the APL project

GEOLOGICAL CONDITIONS

The contract award was followed by extensive geological investigation which established a solid basis for the tunnel design. The southern section of the tunnels lay in the Brisbane Tuff and the Neranleigh-Fernvale formations which were relatively well known from Brisbane's previous large scale Clem 7 tunneling project. These formations are high strength slightly weathered volcanic rocks, mainly metasandstone, phyllite, breccia tuff and ignimbrite. Highly weathered transition zones exist between the lithological boundaries but are not considered as major risks to tunnelling. The TBM tunnels on the eastern part were driven through siltstones with different strength overlaid by a thick layer of alluvium consisting of clay, gravels and sand. Due to the robust geology in the west and the well controllable and safe tunneling method by EPBM shields in the east the majority of the new boreholes were drilled in the Kedron area where the major geological formations provided challenging mixed face conditions for the Eastbound and Southbound ramps as well as the caverns.

Structures built in the Kedron area are located within a basin where two masses of Brisbane tuff (BT), the Lower Tuff and the Upper Tuff, are separated by a series of 'Inter-Tuff' sedimentary layers. The Lower Tuff typically comprises of a fine grained, widely jointed, high to very high strength, fresh, welded porphyritic tuff. It is overlain by a bedded sedimentary rock unit which consists mainly of siltstones (SD) and conglomerate (CD). The Upper Tuff comprises both stratified and porphyritic welded Tuff, which varies greatly in strength and degree of weathering and is often weathered to the point where it becomes residual soil, silty and sandy clay. The resulting very complex lithological boundaries for Kedron are shown on Figure 2.

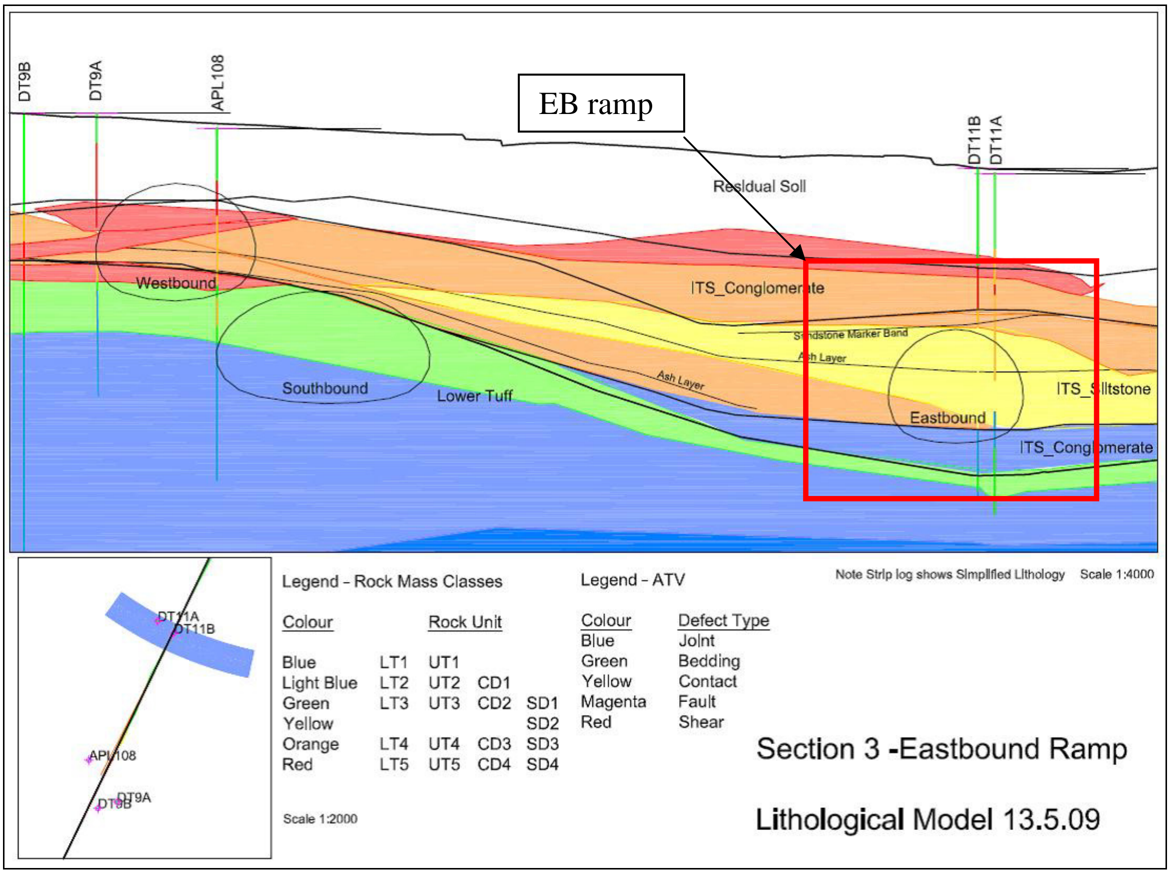


Figure 2: Lithological boundaries in Kedron

The current paper focuses on the design and construction of the Eastbound (EB) off-ramp within this intricate geological profile. Some core samples from the borehole drilling along the tunnel alignment are shown on Figure 3.

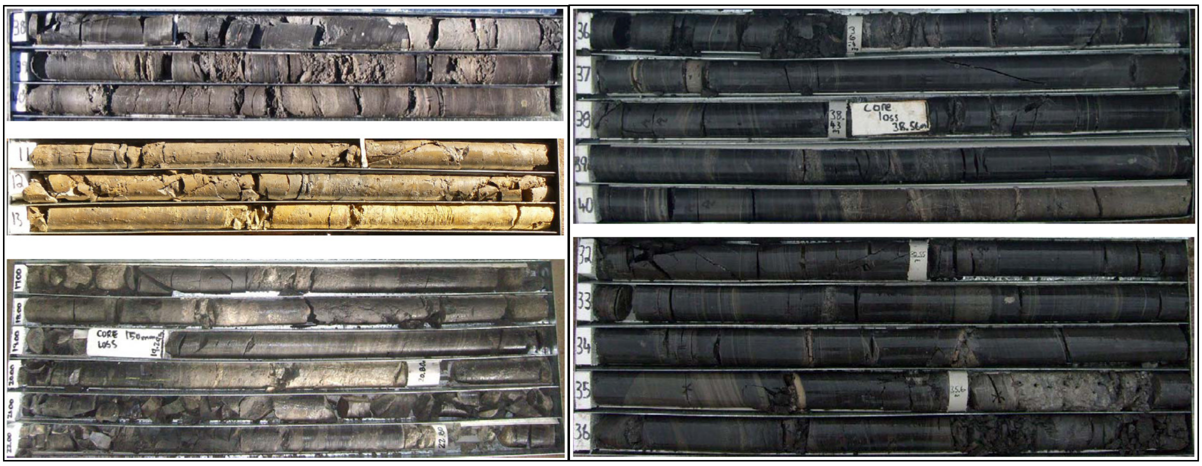


Figure 3: Weathered, low strength siltstone found in the crown area of the tunnel (left) and bedded, middle to high strength siltstone from the face of the tunnel (right)

The new information provided by the additional investigation provided a further understanding of the defect systems of the sedimentary rock unit: a sub-horizontal bedding system with two orthogonal joint sets with a well-developed lithological layering. The spacing of the bedding plane varies from laminated to thinly bedded. Bedding plane partings where they occur are typically planar, smooth and in places contain a thin, usually 1mm to 2mm clay infill. Dip angles vary across the area ranging from 0° to 35° while dip directions also vary typically ranging between North to East to Southeast. Occasional, 10-150 mm thick clay seams and 300mm-500mm extremely low strength siltstone defects (ash band) were also encountered in these rock units. Face mapping shows thick clay-filled bedding in the siltstones on Figure 4.

In the conglomerate, jointing does not appear to be systematically developed. It is anticipated to be discontinuous with rough surfaces. In places the rock unit can be weathered with locally clay filled joints.

Figure 4: Tunnel face with thick bedding through the excavation

ROCK MASS CLASSIFICATION AND GEOTECHNICAL PARAMETERS

The intact rock strength of the siltstone in which the EB ramp was constructed is between 3Mpa and 10Mpa. It is thinly laminated and contains horizontal bedding planes (minor defects) and thick stiff clay infills, extremely low strength ash or siltstone bands (major defects). The discrepancy between the intact rock strength and the weak beddings was incorporated in the design parameters and the numerical modelling approach.

Normally GSI and Hoek-Brown criteria are not used for rock masses with GSI<30 and UCS<15MPa, but for the sedimentary rock masses it was decided to use these approaches. The intact rock strength properties were derived from laboratory test data by using Geological Strength Index (GSI) approach. The equivalent Mohr-Coulomb parameters (c′ and φ′) for the stress range of 0 to approximately 1.2MPa (50m depth) were provided based on Hoek-Brown parameters (mb, s, a). The disturbance factor (D) for the Hoek-Brown approach was set to 0 as the tunnel was excavated by road header in undisturbed rock. In order to better represent the strength of the rock mass around the tunnels a scaling factor related to a 15m wide tunnel was applied to account for the lamination and bedded structure of the siltstone.

The numerical modelling was defined by a hybrid approach of using the 15m scale properties and explicitly modelling major discontinuities in a continuum analysis. Major defects not just simply weaken the rock mass but govern the overall behaviour of the tunnel and therefore strongly influence the support type selection. The application of discontinuum analyses in such rock conditions would have led to an unnecessary overdesigned support system. Typical geological section is shown on Figure 5.

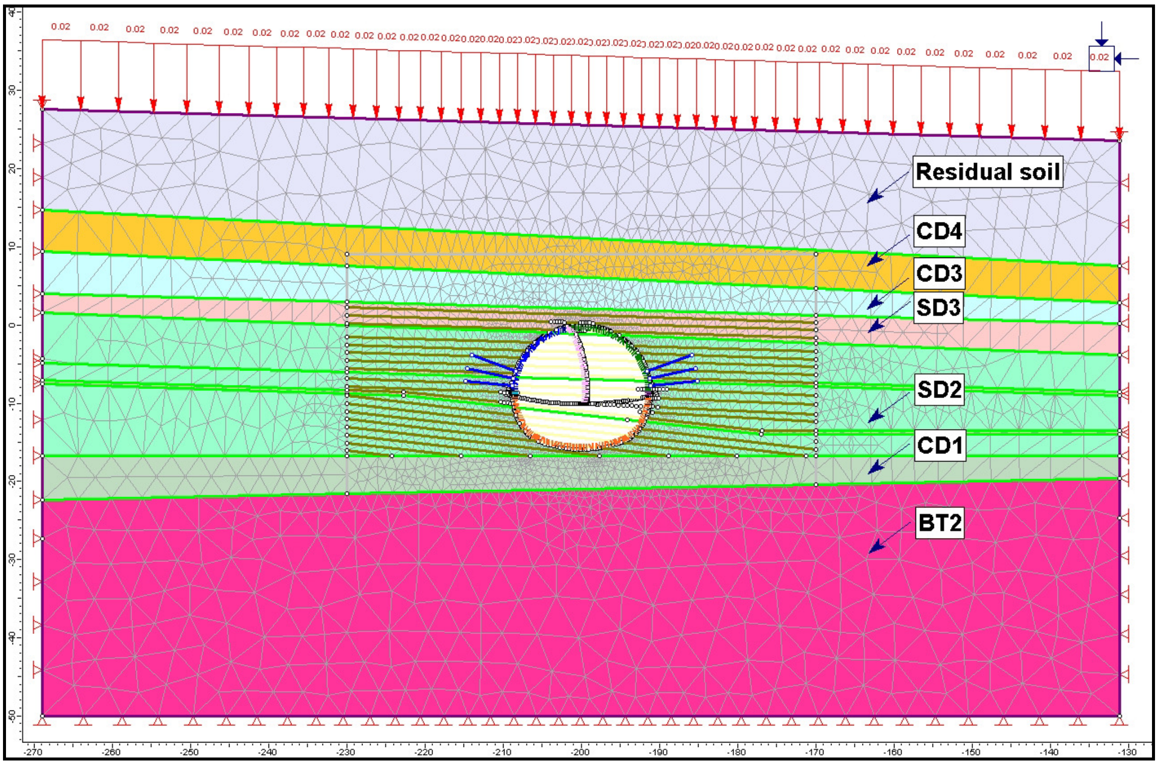


Figure 5: Geological cross-section

Intact rock properties and derived geotechnical parameters can be found in Table 1 and Table 2 respectively. Design data for discontinuities is presented in the Table 3 and Table 4.

Table 1: Intact rock sample properties

Parameters	CD Class 4	CD Class 3	CD Class 1	SD Class 3	SD Class 2	BT Class 2
UCS (MPa)	2.5	5	35	3	10	50
E_{intact} (MPa)	500	2500	10000	1500	3000	15000
m_i	13	13	13	13	13	13

Table 2: Geotechnical parameters

Parameter	Residual soils	CD Class 4	CD Class 3	CD Class 1	SD Class 3	SD Class 2	BT Class 2
GSI	NA	15-30	30-50	65-80	20-30	30-40	60-80
E_{mass} (MPa)	40	150	600	5000	150	300	7500
ϕ'(°)	20	22	35	55	27	30	55
C′ (kPa)	10	90	200	500	90	200	750
Poisson's ratio, ν	0.4	0.3	0.2	0.2	0.3	0.2	0.2
Density γ (kN/m^3)	19	22	22	22	22	22	24
k0 values	0.6	0.8	0.8	0.8	0.8	0.8	1.2

Table 3: Defect data

Rock formation	Defect type	Thickness (mm)	Infill type	Angle of friction	Normal Stiffness (MPa/m)	Shear Stiffness (MPa/m)
Conglomerates and sandstones	Bedding or jointing	Tight	Clean	35°	10000	1000
		1-5	Firm clay	35°	6000	600
Siltstones	Bedding or jointing	Tight	Clean	26°	10000	1000
		1-5	Firm clay	12°-25°	6000	600
		5-10	Firm clay	12°-25°	800	80

Table 4: Properties of ash band

Rock formation	Defect type	Thickness (mm)	Infill type	E (MPa)	φ' (°)	c' (kPa)
Ash band	Bedding	300-500	Extremely low strength rock	60	25	35

NUMERICAL MODELLING AND SUPPORT DESIGN

In the Kedron area the combination of the geology with the relatively large diameter of the ramps and the close vicinity of other structures proved to be a major challenge. The EB tunnel with an excavated width of 18m and a top heading height of 8m was launched from an approximately 30m deep cut and cover box (CC210) constructed by secant pile retaining walls supported by grouted anchors. CC210 with the EB ramp in FLAC 3D and the key plan of the area is shown on Figure 6.

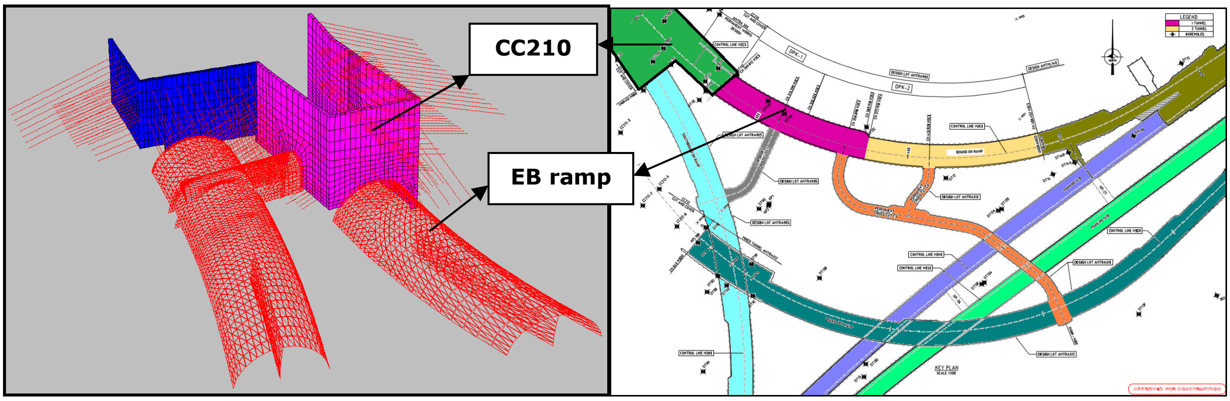
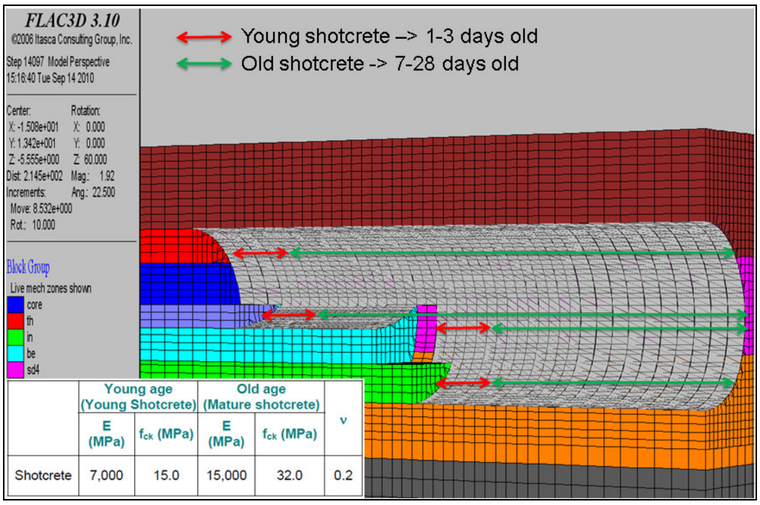


Figure 6: Key structures in Kedron

After assessing the overall stability of the key structures in Kedron using FLAC 3D the detailed design process of the EB ramp consisted of two major steps. First detailed 3D analyses were carried out to understand the stress relaxation in front of the tunnel face, to assess face stability and to follow load development in support measures behind the excavation as shown on Figure 7. Time dependant shotcrete properties such as stiffness and strength were included in the calculation to capture the 4th dimension of tunnel construction – time. Then the 3D tunnelling process was translated into subsequent 2D modelling steps to carry out detailed analysis of the key design sections.



	Young age (Young Shotcrete)		Old age (Mature shotcrete)		
	E (MPa)	f_{ck} (MPa)	E (MPa)	f_{ck} (MPa)	ν
Shotcrete	7,000	15.0	15,000	32.0	0.2

Figure 7: FLAC 3D analysis of single tunnel

The modeling results indicated that the temporary support was not required to take the full rock load and the tunnel support should be designed assuming a pre-relaxation of approximately 25-30% prior to the support installation. This stress relaxation pattern gathered from the 3D analyses was confirmed by using elastic estimate by Panet (1995) and producing ground reaction curves (GRC) as proposed by Hoek et al (2008). Figure 8 shows the normalized tunnel convergence against the excavation and support installation along the tunnel's longitudinal axis.

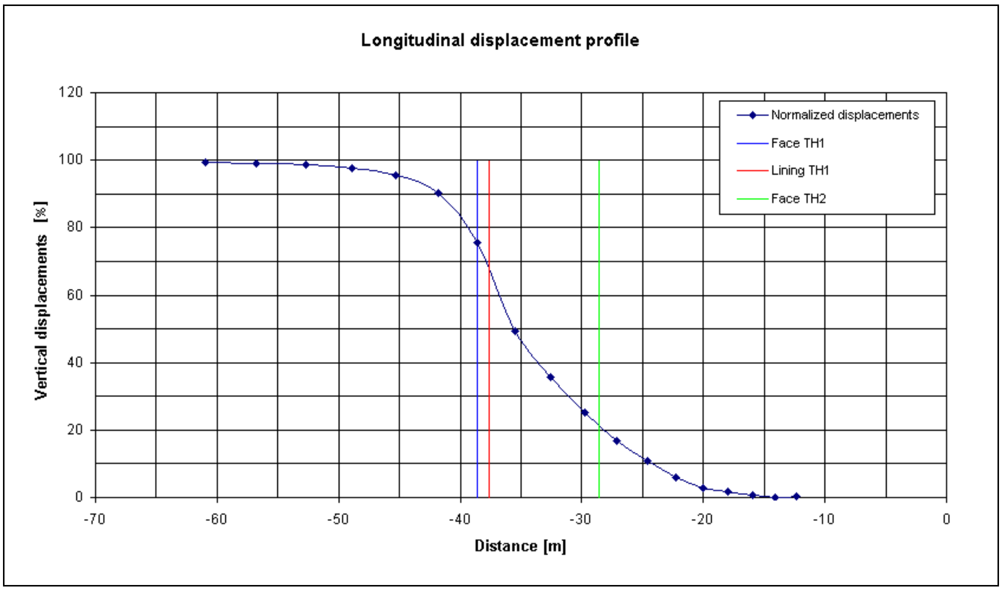


Figure 8: Longitudinal displacements along the tunnel axis

Two dimensional finite difference and finite element analyses using Flac (V6.0) and Phase2 (V7.0) were carried out considering tunnel geometry (shape, span and height), the excavation sequence, temporary support requirements, adjacent structures, applicable geotechnical conditions and taking into account constructability issues and equipment. Development of shotcrete strength and stiffness with age was incorporated based on estimated excavation advance rates provided by the contractor. The stiffness was set according to the age of the lining and included an allowance for creep. The sprayed concrete lining was modelled by one-dimensional linear elastic beam elements attached to the periphery of the excavated grid via interface elements. A full moment connection between adjacent beams was introduced.

Ground was modelled using an elastic-perfectly plastic model with Mohr-Coulomb failure mode that also took the peak and residual design parameters for different rock mass into account. Fully drained behaviour was analysed for the temporary support design and weep holes were drilled into the lining to avoid water pressure build up behind the tunnel lining.

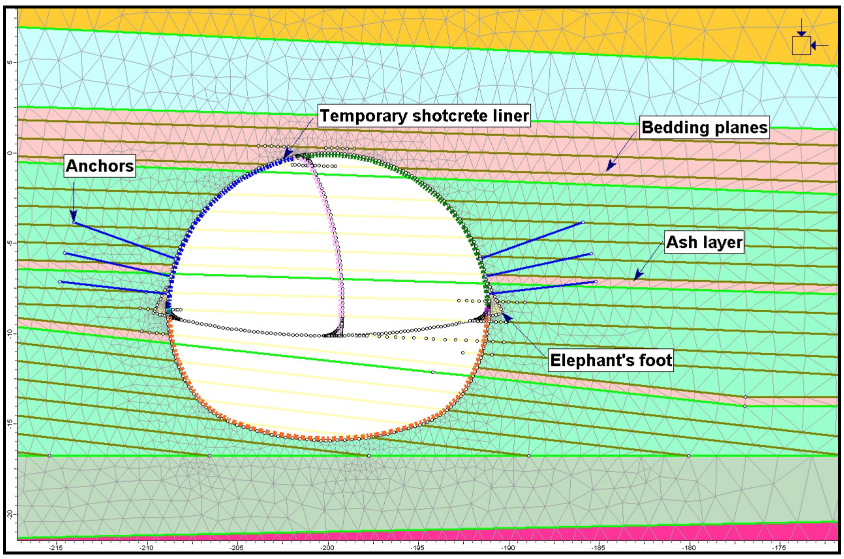


Figure 9: 2D design section in Phase 2

Rock bolts were also included in the models to provide horizontal stability for the footings in the temporary construction phase while the beneficial effect of the canopy tubes and spiled bars were not considered in the 2D analyses.

Numerical models tend to predict extremely high bending moments and shear forces at sharp corners (e.g. at the connection of the top heading and center wall) that generally are not observed during construction. In order to prevent this numerical discrepancy modelling of the sharp corners was avoided and the theoretical excavation profile was followed as accurately as possible.

Face stability was assessed by analysing rock wedges which can potentially slide along an existing joint plane and can cause instability at the face. The full top heading of the tunnel was taken into account and joint orientation data (dip direction and dip angle) was plotted to assess potential wedges at the face. Unwedge (by Rocscience) was also used to calculate wedge size and possible distribution scenarios based on data collected during site investigation and face mapping. The resisting force was derived considering the frictional shear resistance on the sliding plane and the two side planes of the wedge while the contribution of cohesion to the shear resistance was neglected. The face stability was analysed using the Terzaghi's silo theory and assessed in the detailed FLAC 3D model as well. Fibre glass rockbolts were installed with a sealing shotcrete layer of 50mm to stabilize potentially dangerous wedges. It was found that even the intact, high strength siltstone would lose most of its strength along the horizontal lamination when becoming dry.

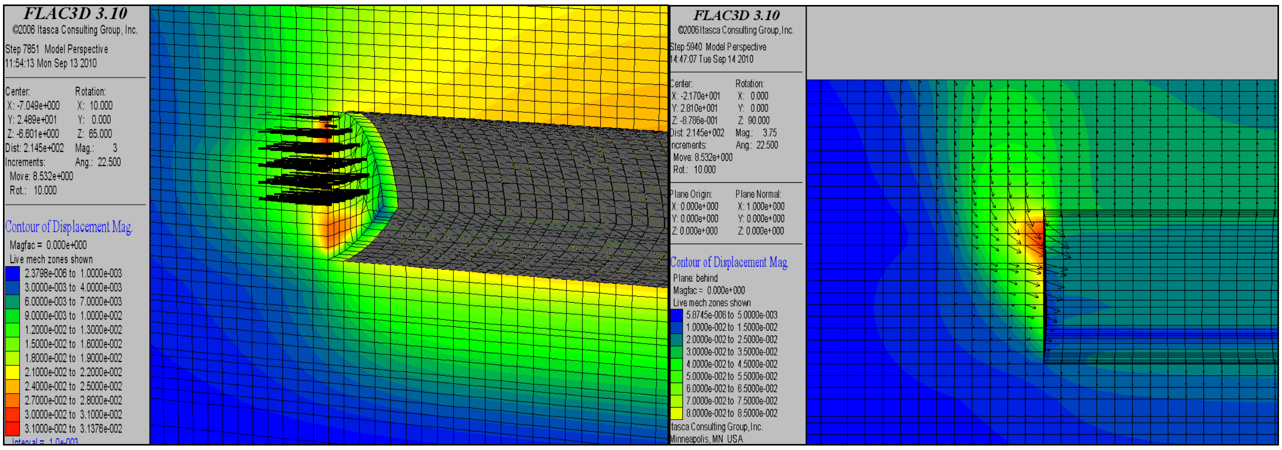


Figure 10: Face stability assessment using FLAC 3D

Wedges around the tunnel opening were analysed by the same principle of plotting possible wedge scenarios and using Unwedge to give an estimate of the rock blocks. The rock mass around EB on ramp showed an increase in strength and a decrease in weathering with depth.

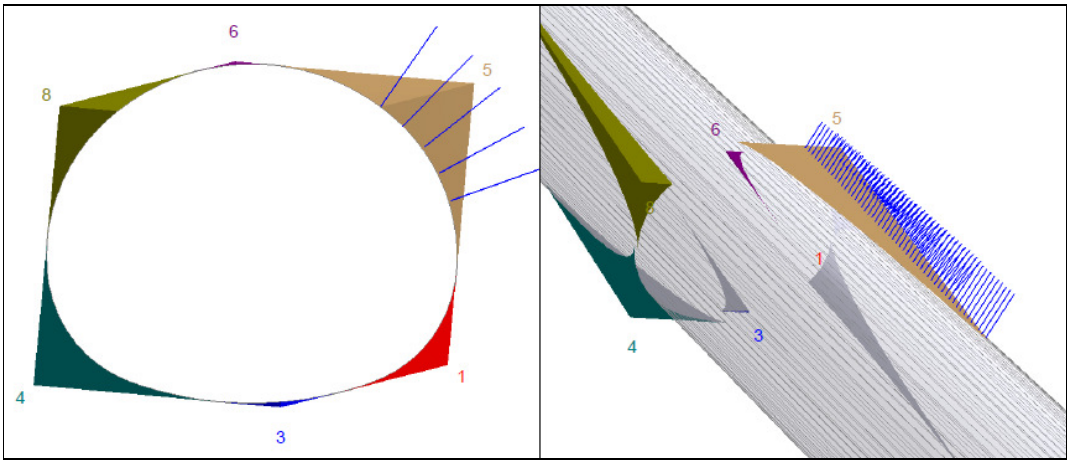


Figure 11: Wedges around completed ramp visualized in Unwedge

Along the crown of the EB tunnel the cover and the quality of the sedimentary rock mass was poor. In order to avoid excessive failure of the roof structure and to provide a safe environment for the construction works canopy tubes or later spile bars were installed. Canopy tubes also helped to preserve the confining pressure for the grouted anchors stabilizing CC210 keeping the ash layer intact.

Arrays of 3D monitoring prisms and extensometers were placed along the tunnel to follow tunnel convergence and movements in the rock as well as surface settlements. The measurement data from site was collected and reviewed by the surveyor team in a timely manner to allow review of the support measure behaviour. Monitoring data provided crucial input for back calculation of the rock mass properties and to reassess support measure requirements.

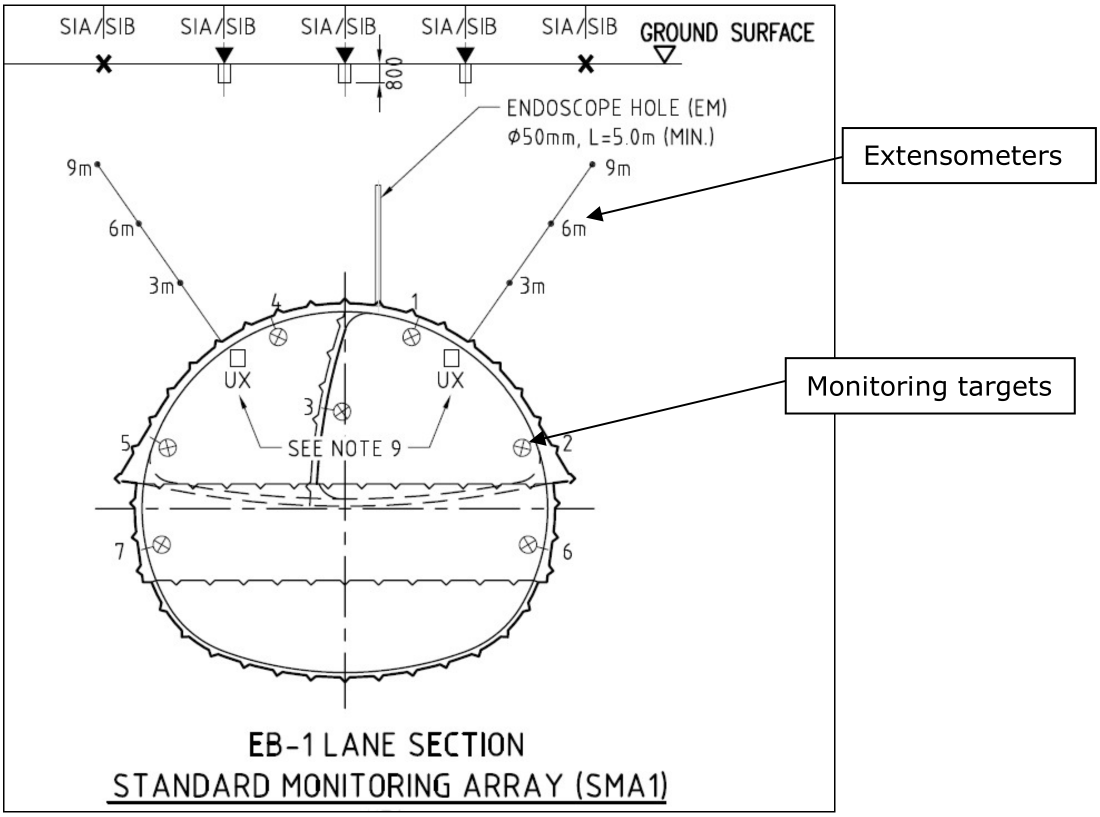


Figure 12: Typical monitoring section

SUPPORT OPTIMIZATION

Due to the lack of previous experience with the various sedimentary rock mass around Brisbane, the designers adopted the ‘observational method’ to define support classes for the EB ramp. During mechanized tunnelling or exceptionally difficult geological conditions (e.g. swelling rock) it is hard to implement design changes because they can lead to cost or time overrun whereas the NATM toolbar can be modified during the construction phase with a higher flexibility.

Eurocode 7 describes the observational method as follows:

‘(1) When prediction of geotechnical behaviour is difficult, it can be appropriate to apply the approach known as "the observational method", in which the design is reviewed during construction.

(2) The following requirements shall be met before construction is started:

- acceptable limits of behaviour shall be established;

- the range of possible behaviour shall be assessed and it shall be shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits;
- a plan of monitoring shall be devised, which will reveal whether the actual behaviour lies within the acceptable limits. The monitoring shall make this clear at a sufficiently early stage, and with sufficiently short intervals to allow contingency actions to be undertaken successfully;
- the response time of the instruments and the procedures for analysing the results shall be sufficiently rapid in relation to the possible evolution of the system;
- a plan of contingency actions shall be devised, which may be adopted if the monitoring reveals behaviour outside acceptable limits.'

As shown on Figure 6 the first section of the EB ramp (EB1) is the biggest cross section with the lowest proper rock cover and the first 40m lies in the influence area of the CC210 shaft and temporary cross cut C2. Here extra support measures were implemented to avoid the destabilization of these structures. After 100m the excavated width and height reduces to approximately 15m and 8m respectively (EB2).

Acceptable limits of behaviour were established via a series of sensitivity analyses with different geotechnical parameters, stress state, relaxation parameter and support measures. This extensive modelling work and more importantly monitoring results from the site during the construction of EB1 enabled the design team to conclude their findings in four support types for EB2 from ST4 to ST6b as shown on Figure 11 and in Table 5.

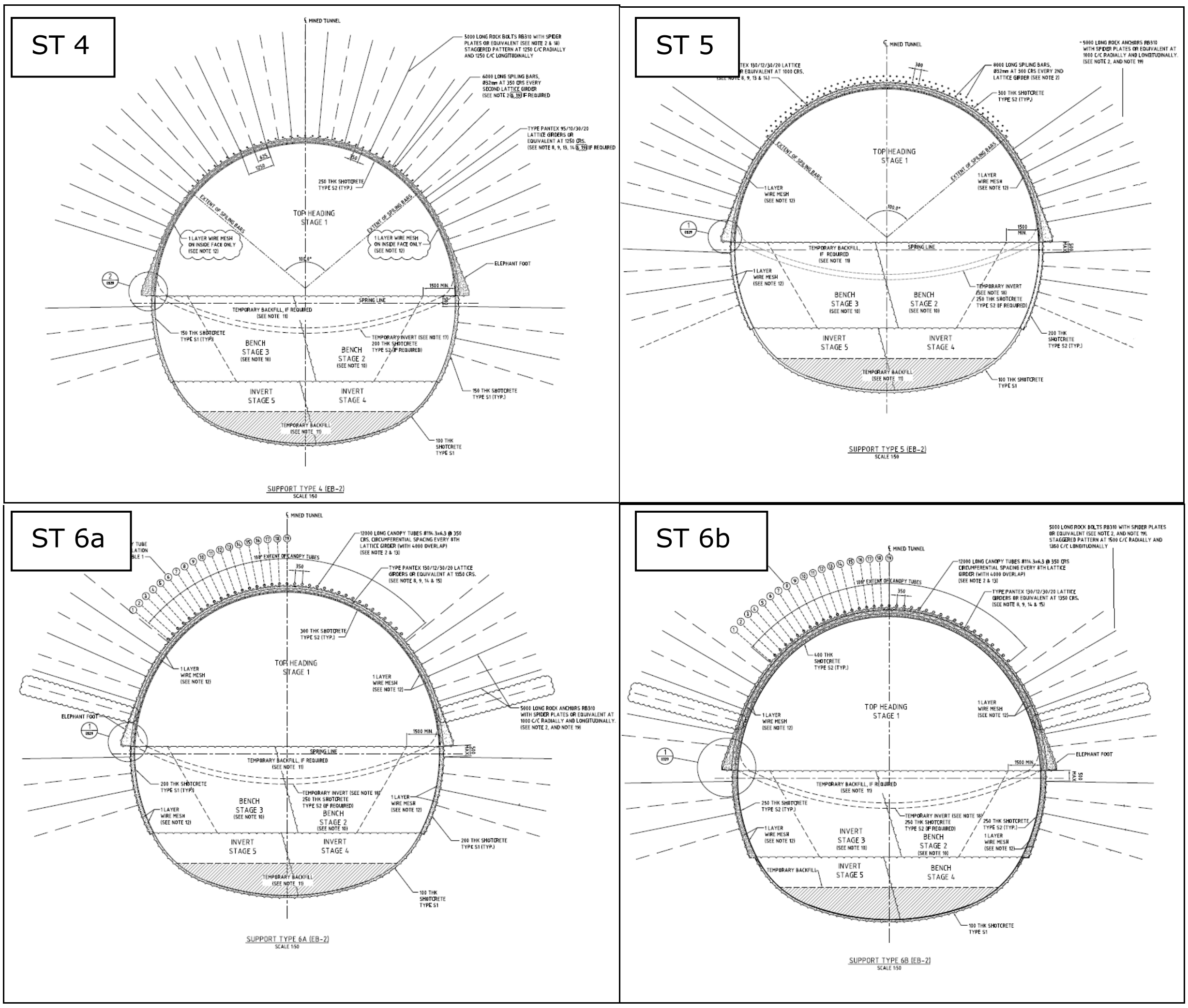


Figure 13: Support types for EB 2

Table 5: Support measures for EB2

Support type	Shotcrete thickness[1] (mm)	Shotcrete type[1,2]	Roof support[3]	Wedge support[3]	Optional
ST4	250/150/100	S2/S1/S1	RB310, 5m, 1.25x1.25	RB310, 4m, 1.5x1.25m	lattice girders with spiled bars, micropiles, closed invert
ST5	300/200/100	S2/S2/S1	Φ32mm spiled bars, 8m	RB310, 5m 1.25x1.0m	micropiles, closed invert
ST6a	300/200/100	S2/S1/S1	Φ114mmx6.3mm canopy tubes 12m	RB310, 5m 1.25x1.0m	micropiles, closed invert
ST6b	400/250/100	S2/S2/S1	Φ114mmx6.3mm canopy tubes 12m	RB310, 5m 1.0x1.0m	micropiles, closed invert

Notes:
[1] Top heading/bench/invert
[2] Type S1: steel fibre reinforced shotcrete and Type S2: sprayed concrete with wire mesh
[3] Support properties: diameter and/or type of support, length, pattern (radially and longitudinally)

The top of the siltstone was determined as highly variable and inconsistent in quality and sensitivity calculations proved that one of the governing factors of support selection is the quality and height of proper (class 3 or better) sedimentary rock cover above the crown. Therefore geological conditions were assessed on site by inclined probe drillings from the face and vertical endoscope test in the excavated round. If a minimum of 5m adequate quality rock cover was encountered ST4 could be installed otherwise ST 5, 6a or 6b had to be constructed.

The thickness and quality of the sedimentary rock mass around the tunnel substantially influenced the stresses in the lining and ST6a or 6b had to be installed if the proper rock cover fell under 3m. With such a low cover the arching effect of the rock above the tunnel became the critical issue. To address roof stability problems and to help reserving confining stress canopy tubes were installed. These grouted steel pipes transferred the load longitudinally as they had been drilled into the rock face and supported by lattice girders embedded in shotcrete behind the face.

Optional footing support measures were defined for all support types if poor quality siltstone layers (ash bands) or thick clay infills were encountered. It was the site geologist job to verify the conditions before each excavation round and with the support of the site engineer to decide which additional support measure (micropiles, closed invert) should be installed. Furthermore if real time monitoring of the footing revealed high displacements, contingency measures including additional temporary invert, rock bolting, ground anchors or partial backfill of the tunnel were specified.

Following real time monitoring data from the roof, extensometers showed the correlation between the installed roof stabilization and the encountered geology. It provided valuable information for site personnel to have a better input for design optimization and to properly interpret additional ground investigation findings.

Monitoring plan for each design section at every 40m along the tunnel alignment was submitted with the instruments shown on Figure 10. Every 5m a monitoring section containing only the prisms and endoscope hole were installed. The frequency of reading was based on the monitoring section's distance from the face. Continuous monitoring allowed real time comparison of prism movements against pre-defined alert and alarm levels. These displacement levels were specified for each target at each excavation stage.

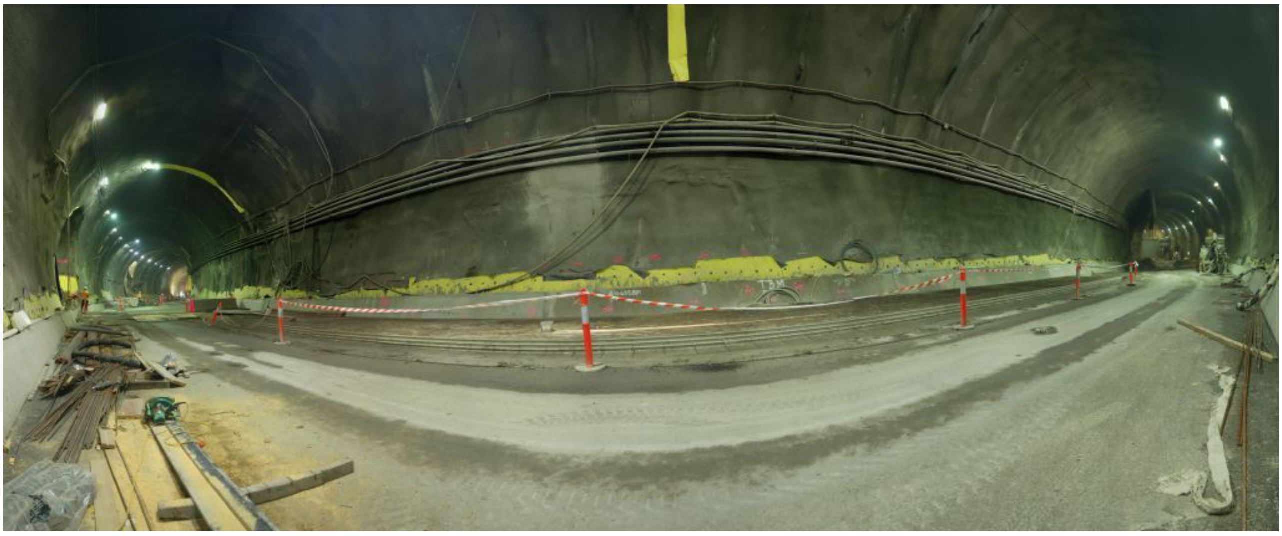


Figure 12: EB Onramp after completion

CONCLUSION

The paper presents a case study where extensive ground investigation, thorough design and continuous communication between the site and the design office enabled the designer to adjust its design assumptions. All of these three phases of a project proved to be equally important during the whole duration of the project. The construction of the EB ramp on APL project showed that large diameter tunnels can be constructed safely in complex geologies involving poor rock conditions without overestimating the necessary support measures neither giving up high safety standards.

The lack of a well-defined and well-implemented monitoring plan can jeopardize safety and can lead to financial loss. Furthermore a well-setup, real time, online monitoring system provides a quicker and easier way to share information and help the cooperation between the parties involved in design optimization.

The observational method cannot be applied successfully without highly skilled site personnel who have a good understanding of both the encountered geology and the design principles applied. Additional, low cost ground investigation can provide vital information in their decision making process which can lead directly to cost and time saving at the same level of safety.

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