PENCHALA Tunnel: example of an efficient combination of empirical, analytical, and observational approaches for design

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ABSTRACT

The 710m-long Penchala tunnel is the first underground civil structure of the Kuala Lumpur highway system. It is a twin-bore tunnel and each bore has a horse-shoe shaped section with an excavated area of 118-144m². The ground conditions vary from residual soils to fresh, strong granites and the overburden from a few meters to a maximum of 115m. Given the importance of this tunnel and the relatively limited experience of Malaysia in constructing tunnels of this dimension, the construction contract was procured through international tendering, admitting proposals with alternative solutions for design and construction. This paper illustrates how an efficient combination of empirical, analytical, and observational approaches, allows to develop a flexible Construction Design and to speed the execution. Moreover, it gives a quick overview of the optimization introduced by the winning, Alternative Solution proposed by the Designer Geodata together with the Sub-Contractor CMC, who excavated the tunnel, over the original Tender Design. The tunnel excavation and support were completed in May 2003. The example represents certainly another successful case of technology transfer from Italy to abroad.

1. INTRODUCTION

The Penchala tunnel is the first underground civil structure of the Western Kuala Lumpur Traffic Dispersal Scheme. The 710m-long, twin bore tunnel was realized between 2002-2003 by the Joint Venture CMC (Italy) and CM Indah (Malaysia), on the basis of the Construction Design and follow-up of GEODATA (Turin, Italy). The main features of the Penchala Tunnel are given in Table 1.

Table 1. Characteristics of Penchala tunnel

Characteristic	Eastbound tube (LHS)	Westbound tube (RHS)
Length along central axis of the natural tunnel	709.26m	691.70m
Minimum radius of curvature along central axis	455.25m	604.75m
Grade	+3.5%/-2.5%	+4.0%/-2.5%
Overburden	average 60m, maximum 115m	
Pillar width	8m at the portals up to a maximum of 14m	
Cross passages	No.3, 4.7m width and 4.4m height	
Geology	Granites with various degrees of weathering and fracturing	
Excavation section area	Average 140m²	
Excavation method	Drill & Blast and hydraulic machinery	
Working faces	No.4 (n.1 from each portal)	

The bid foresaw a "lump sum" contract with a fixed time of 21 months to finish the work, with the possibility to present Alternative Design solution with respect to the Tender design. The Engineer was part of the Organization who studied the preliminary design and prepared the bid, while the General Contractor, who constructed all the stretch, was also part of the Manager who had the license for the Western Kuala Lumpur Traffic Dispersal Scheme.

The twin bore tunnel was on the critical path since it was the most complicated and, in the same way, unforeseeable civil work of all the stretch for Malaysian Contractors. In order to satisfy the targets of both the Owner and the Joint Venture in terms of budget and specially time, the construction design, starting from the Tender, has been developed defining very flexible excavation section types (which could guarantee to cover all the foreseeable underground conditions) to be managed on site directly by the Design Representative according to the criteria of the observational approach.

Moreover some optimizations with respect to the Tender solution were introduced in order to speed the execution, to lower the cost and increase the overall safety of the works.

2. DEVELOPMENT OF THE CONSTRUCTION DESIGN

2.1 Summary of the key elements of the technical approach

The construction of the tunnel, starting from the first design stage to the last work on site is regulated by the concept expressed in Figure 1, where the need for the Designer to be involved for the entire duration of the works is implicit.

Moreover, having in mind the concept that the optimal design involves empirical, analytical and observational methods (Bieniawski, 1984), the intent was to define a very flexible design wherein the various excavation section types are integrated in order to allow a smooth variation and an easy definition of their application directly at the excavation face, optimizing both the advance rate and the installation of stabilization measures.

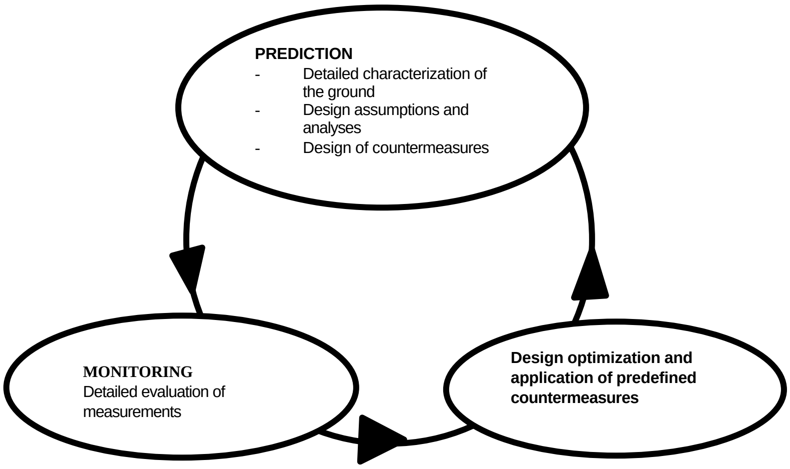

Figure 1. Basic design approach

The first step, in order to define both the excavation sections and the rules for their right application is a linear correlation between the geomechanical context and the excavation works and thus the construction design has been based on the successive determination along the tunnel alignment of (Russo et al., 1998):

- *geomechanical groups*, representing the different rock masses with well-defined intrinsic geomechanical properties;

- *behavioral categories*, which express the deformation responses of the geomechanical groups at tunnel level;
- *technical classes*, which are directly associated with the different project solutions, in terms of typical sections of excavation and support.

2.2 Definition of the geomechanical groups

The tunnel is entirely excavated in the so-called Kuala Lumpur Granite, that is typically a medium to coarse grained granite, with various grades of fracturing and weathering. The whole tunnel is below groundwater table, which follows the slope surface (located at the transition from the top soil to the bedrock); a stream flows on the surface, at a distance of about 20m from the tunnel alignment. For investigation purpose, 12 boreholes were drilled along the tunnel alignment for the preliminary design, for an accumulated length of 865m. In situ tests involved also geophisical surveys, 28 permeability tests and an adequate number of laboratory tests.

The basic features of the geomechanical study include:

- the variability and residual uncertainties on the geotechnical properties are analysed with statistical methods and incorporated in the tunnel design by means of a probabilistic approach;
- in addition to the Q-System (Barton, 1974; Grimstad and Barton, 1993) used in Tender design, the GSI (Geological Strength Index, Hoek et al., 1995) is applied for evaluating the geomechanical quality of the rock masses. This addresses the requirement of referring to a "pure" classification system, which reflects intrinsic rock mass properties and can be directly correlated with principal geomechanical parameters.
- the weathering grade of the granite is classified in accordance with the proposal of the Geological Society of London (GSL,1995), referring to the condition of uniform (intact) material.

The analysis of the available data suggested to distinguish 5 major Geomechanical Groups (G.G.) along the tunnel alignment; the typical description of these groups is given in Table 2.

Table 2: Typical characteristics of the geomechanical groups

G.G.	Intact rock material		Rock mass	
	Weathering[1]	**Strength[2]**	**Fracturing[2]**	**Quality[3]**
G1	>>W1/(W2)	high	low	Good to very good
G2	>>(W1)/W2 -W4/W5 in shear bands	-high -low in shear bands	low to medium	Globally poor to fair
G3	>>W2/W3 -W4/W5 in shear bands	-moderate -low in shear bands	high	Globally very poor
G4	>>W3/W5	low	high to very high	Very to Extremely poor
G5	>>W4/W5	low to very low	very high	Extremely poor

Notes: [1]GSL (1995); [2]ISRM (1981); [3]Barton, 1974.

The statistical analysis of the Q-System input parameters for each geomechanical group allows to transform frequency distributions into density functions, which are then used as input for deriving probabilistically representative Q and GSI values, as well as the principal parameters of rock masses. In particular, the MonteCarlo method (Metropolis and Ulam, 1949) is employed with Latin Hypercube sampling. The principal results of statistical and probabilistic analysis (500 simulations) are shown in Table 3.

Table 3: Typical properties of the geomechanical groups (G.G.) from statistical and probabilistic analysis

G.G.	**Intact rock properties**		**Rock mass quality Index**	
	Co(MPa)(1)	**m_i**	**Q(2)**	**GSI**
G1	80-160 [31-161]	20-30	17-132 [5.9-165.8]	70-90
G2	35-110 [25-160]	20-30	0.9-11.3 [0.51-15]	50-70
G3	20-40	10-15	0.1-0.6 [0.048-1.6]	35-50
G4	10-20	10-15	0.01-0.07 [0.006-0.14]	20-35
G5	1-10	8-12	0.004-0.02 [0.002-0.02]	<20

(1)10°-90° percentiles and [min, max]; (2)typical values and in parenthesis possible extended ranges (derived from GSI with proper adjustment to take into consideration potential Jw and SRF values).

Note that even though the GSI is the reference classification index to identify a geomechanical group and its behaviour to the excavation, the Q index is used both to orient the dimensioning of primary support in the favourable geomechanical context and to manage the variability of the section types (see Table 5 and Section 4).

2.3 Definition of the behavioral categories

After the geomechanical groups have been defined, it is possible to analyze their relative behavior (or response) to the excavation (without stabilization measures), taking into account the existing stress conditions at tunnel levels.

The analysis is performed by combining the "Convergence-Confinement" method (solution of Brown et al., 1983) and the probabilistic approach, using as input the parameter values shown in Table 2. In addition, geo-structural analyses are performed for evaluating the possibility of instability of rock wedges.
The results of the simulations are classified on the basis of deformation indexes of the face and of the cavity (Russo et al., 1998), distinguishing six possible categories of behavior: from the best ("a" class, characterized by elastic behavior) to the worst condition ("f" associated with conditions of immediate collapse of the tunnel face). Table 4 gives the results of the probabilistic analyses carried out for three characteristic conditions of overburden.

Table 4. Behavior of the excavation (500 simulations)

G.G.	**Overburden (H)**		
	50m	75m	115m (max)
G1	a/b	a/b	a/b
G2	b	b	b
G3	b(~35%); c(~65%)	c	c
G4	c	c	c(~45%); d(~55%)
G5	(e)/f	(e)/f	(e)/f

2.4 The Design solution - typical sections of excavation and support

The typological choice of the stabilization measures is a function of the behavioral category. Each section type is composed by a primary support system and a final/permanent un-reinforced or reinforced concrete lining cast in place. Tables 5 and 6 give indication about the composition of the section types and the possibility to change the rock-bolt density as a function of the rock-mass condition.

Table 5. Main characteristics of the primary support

Section type	Behavioral category (G.G.)	SFRS thickness [mm]	Swellex Mn16 type bolts	Steel ribs type and spacing [m]
1	a/b	50	Spot L=4.5m	-
2	b(G2) b(G3) H<75m	100	2.0m x 2.0±0.5m L=4.5m	-
3a	c(G3) H>75m	150	1.5m x 1.5±0.5m L=4.5m	-
3b	c(G4)	200	No.2 for each side (1) every 1.25±0.25m L=4.5m	2 – 200x100 every 1.25±0.25m
4	d	200	1.25m x 1.25±0.25m L=4.5m No.4+No.2 optional L=6.0m	2 – 200x100 every 1.25±0.25m
5	f	250	No.4 in external side and No.3 in the internal side (1) L=6m	2 – 200x100 every 0.75m (1)

Note: (1) Pre-supported with Umbrella arch

Table 6: Criteria for managing the variability of the section types

Section type		Section type		
		Heavier ($^{+}$)	Ordinary	Lighter($^{-}$)
1	GSI	70-90		
	Q	<10	10-40	>40
2	GSI	50-70		
	Q	<1	1-4	>4
3	GSI	25-50 (H<75m)		
	Q	<0.1	0.1-0.4	>0.4
4	GSI	25-50 (H>75m)		
	Q	<0.01	0.01-0.04	>0.04
5	GSI	<25		
	Q	(not variability applied)		

Note: The combined use of the GSI index and the Q-System allows the possibility of taking into account the effect of groundwater condition too.

The choice of the single elements (type and variability) composing the excavation section and their verification has been made on the basis of empirical and analytical method, depending on the rock mass quality:

- for G1, G2 and G3 empirical method based on the Q-chart plus geo-structural analysis for the wedge fall;
- G.G. G4 and G5 analytical method based on Convergence-Confinement, plus structural analysis of the primary and final lining.

Table 4 demonstrated the smooth variation of the solution and a clear link of the section types to the behavioral categories and, hence, to the rock mass characteristics through the geomechanical classification. This link and the rules for applying heavier and lighter derived sections have been the basis for a successful follow up of the works (see Section 4).

3. OVERVIEW OF THE OPTIMIZATIONS FORESEEN IN THE ALTERNATIVE DESIGN

The success of the work was also due to the introduction of some optimizations to the Tender Design regarding some major features. Table 7 illustrates the main differences between the two designs (Tender and Alternative) whereas the corresponding advantages of the alternative solution are depicted in Table 8.

Table 7. Comparison between Tender Solution and Alternative Solution

Main Design Features	Tender Solution	Alternative Solution
Tunnel alignment	The complete section rotates around the tunnel axis, positioned at one side of the carriageways, to follow the various curves	Only the carriageways rotate around the tunnel axis, which has been moved in the middle of the tunnel section
Support design	Six typical sections	• Five typical sections
	Not foreseen	- Systematic probing in advance with a double function of site investigation and drainage.
	Final concrete lining only for limited extent (support class V)	• Continuous plain concrete lining, 40cm-thick • Continuous steel reinforced concrete lining, 50cm-thick at the portals
	Non continuous drainage	Systematic waterproofing
Advancement Scheme	Multi-drift sequential excavation	Heading and Benching or full face using controlled blasting

Table 8 Major advantages of the optimizations introduced by the Alternative Solution

Advantages of Alternative Solution	Implications
Tunnel alignment	
The foundation of the support base is always at the same level	no need to define for each advance different levels for the foundation of the support
Support Design	
Less problems during construction for installing support elements with a certified durability over one hundred year, since the concrete lining will guarantee for the long term durability	Increase the rate of advance, especially in problematic areas.
Advancement scheme	
The influence of the excavation to the surrounding rock mass is significantly reduced	In the same geomechanical conditions the support structure can be lighter
Excavation cycle is significantly simplified and site organization is a lot easier, with sufficient space for implementing an efficient ventilation system	Requires carefully designed, and executed drilling and blasting scheme to minimize over-break
Reduced construction time due to increased advance rate	Special equipment is required for obtaining high advance rates
Simplified support installation cycle; the critical operation of extending the legs of steel ribs is eliminated	Requires a better control of the deformation and stability of the excavation face
Both the footwalls of the final lining and the final invert can be cast as close to the face as needed	Requires a more rigorous geotechnical data collecting as well as good supervision from the designer during construction

4. CONSTRUCTION OF THE TUNNEL

Since the quite large tunnels sections have been excavated in rock, the most suitable excavation method was the Drill&Blast, implemented with the smooth-blasting technique to reduce over-excavation on the perimeter. Moreover, the two tunnel faces, advancing from the same portal, maintained a minimum distance of about 40m to minimize any possible negative interferences, especially in the portal areas where the inter-axis was minimum. The typical drilling pattern was a V cut round of 4.0m with about 250 blast holes (50 only for the perimeter) with a specific powder consumption of 1.5kg/m^3. Among the overall typical tunnel equipment used for D&B; the most interesting were two semi-automatic jumbos which allow to reach a very good drilling control and indeed a reduction of the over-break (about 10cm) and also of the production holes number.

The construction supervision procedure together with the geomechanical monitoring (convergence measures, piezometers, extensometers and blasting vibrating controllers) have been the basis for the right application of the design on site, with respect to managing the section type variability (see Table 5 and 6).

The cited procedure to achieve this target consisted in:

- executing systematic probe hole at the face for geological investigation and drainage;
- mapping the face using the GSI and Q index at each round;
- collecting geomechanical monitoring data;
- participating at the daily meeting between the Designer the Contractor and the Resident Engineer (Client) to agree the tunnel advance methodology and the right primary support type, based upon the analysis of all the available filed data collected.

Table 9 shows the actual application of the section types compared with the foreseen one, which demonstrates as the on site follow up allowed to reduce the overall installed measures and to adapt the pre-defined section types to the encountered rock mass conditions.

Table 9: Comparison between the foreseen and the actual application length of the section types

Section type	1	2 (1)	3a	3b	4	5
Foreseen application (%)	30-35	30-35	10-15	5-10	5-10	2-4
Actual application (%)	65	23	4	4	0	4

Note: (1) the 70% has been of the lighter type

As derivable from Table 9, the larger application of the section 1 than foreseen reflects more favourable geomechanical conditions than predicted; about this topic, it is interesting to observe that:

- the (%) foreseen application- which revealed rather "pessimistic"- was essentially based on statistical analysis of geomechanical classification index of the boreholes logs obtained from a fixed portion of three tunnel diameters centred at tunnel level;
- in the Tender design, a further "pessimistic" hypothesis was done, probably based on the analysis of the entire columns of the boreholes;
- on the contrary, back-analysis indicates that the (%) actual application is in very good agreement with the statistical analysis performed at tunnel level (i.e. considering only one diameter).

5. CONCLUSIONS

On the basis of the information presented and discussed in the previous sections of this paper and in view of the fact that the construction of the Penchala Tunnel was completed ahead of schedule (19 months instead of the 21 months foreseen in the Tender documents), and within budget the following statements can be made:

- the optimum solution can only be found through close and proactive collaboration among the Owner, the Contractor, the Designer and the Resident Engineer at all stages;
- furthermore, it is essential to use a combination of empirical, analytical and observational approaches to fulfill the project objectives – on time, within budget and in quality; in particular, in the described project:
  - empirical method has been applied for dimensioning the primary support in the more favorable geomechanical context and, more in general, for managing the variability of the section types;
  - analytical method has been used for evaluating the ground response to the excavation and for the structural design of the support and lining;
  - observational method permitted, in agreement with the concept of flexible design, the application of the adequate technical solution for each geomechanical condition encountered during the advancement of the excavation: the introduction of the variability of the section types and precise rules of application further improved the fitting between required and applied stabilization measures, allowing the possibility of selecting among a good number of 13 combinations of support.

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