Evaluation of Construction Methods for the Mae Ngad – Mae Kuang Project

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ABSTRACT

During the planning of the Mae Ngad-Mae Kuang project a comprehensive geological/geotechnical investigation program led to an optimization of the alignment with respect to technical and environmental risks. Different options for the construction of the project were evaluated, including NATM and open, as well as shielded TBM excavations, with a maximum of 5 headings from two access tunnels and one portal. After establishing the geological model and characterization of the rock mass, the rock mass reaction to the excavation was analysed along the entire alignment, which for that purpose was divided into sections of 50m each. Depending on the ground behaviour excavation and support methods, as well as auxiliary measures were selected. Time and costs were assigned to each activity.
In addition to the variation in construction methods, the geological risk was assessed by using expected, optimistic and pessimistic rock mass parameters. This evaluation showed a variation in excavation time of up to 8 months depending on the construction methods chosen. A comparison of construction costs revealed that due to the relatively low labour costs in Thailand the option with the greatest share in drill and blast excavation is the most economical one.

1. INTRODUCTION

The purpose of the project near Chiang Mai in northern Thailand is to link the Mae Ngad reservoir to the Mae Kuang reservoir, in order to better utilize the capacity of both reservoirs in case of locally different precipitation. In particular the Mae Kuang reservoir has shown a deficit in water supply in years of lower rainfall. Alignment studies based on hydraulic requirements and geotechnical conditions eventually led to the selection of a tunnel of a length of approximately 23 km, with an overburden thickness up to 550m. Logistic and topographical conditions allow excavation from two adits and the outlet portal, while at the inlet due to difficult access an excavation should be avoided. The internal diameter of the tunnel was determined with 4,20 m. Hydraulic considerations led to the requirement of a smooth lining of the tunnel.

2. GEOLOGICAL CONDITIONS

2.1 Geological setup

The lithology of the project area includes sedimentary rocks belonging to the Lower and Upper Palaeozoic, the Tertiary and the Quaternary. Igneous rocks include Triassic granitoid intrusives and felsic volcanic rocks, which are generally considered to be Permo – Triassic and inferred to have formed in a magmatic arc.

The oldest rocks in the project area comprise the Thung Song Group, which belongs to the later part of the Ordovician, possibly extending into the Silurian. The rocks of this group are mainly limestone with some shale horizons.

Through the Silurian deeper water conditions prevailed. The area lay on an E-facing passive continental margin where pelagic sedimentation spread and continued into the Devonian. The prevailing rock is sandstone, in parts developed as sedimentary orthoquartzite (or slightly metamorphic sandstone). Shale (siltstone) and chert occur in lesser quantities. Occasionally limestone deposited in a shelf environment occurs.

A thick sequence of poorly fossiliferous, mainly clastic marine sedimentary rocks (in parts slightly metamorphic), known as the Thong Pha Phum Group, conformably overlies the Thung Song Group. It is essentially Siluro – Devonian, extending up into the Carboniferous in places. These sediments are broadly interpreted as having been deposited in shelf to back-arc settings. The rocks of this group include mainly sandstone, shale (siltstone) and chert. Occasionally limestone and agglomerate are found. Volcanic activity was marked in the Upper Devonian – Middle Carboniferous time interval.

Overlying the Thong Pha Phum Group, generally without any marked discordance, are various Carboniferous – Permian sequences, including turbidites (Mae Hong Son/ Phrae Formations; Dan Lan Hoi Group, Ratburi Group). Conditions became essentially shallow-water by the Middle Carboniferous. Reef limestone was deposited in the Late Carboniferous. By the Upper Carboniferous period much of the area may have been emergent, although generally with low relief. There are some red beds in the uppermost Carboniferous and mafic volcanic activity (basalt), with the characteristics of an extensional continental setting. This continued into the Permian. Extensive shelf seas developed in the Permian with an abundance of shallow – water carbonatic rocks. Typical rocks of this sequence are sandstone, shale (siltstone) and limestone of mainly Lower – Middle Permian age, which forms striking karst landscapes wherever it crops out. Occasionally greywacke, chert and agglomerate occur. By the end of the Permian there had been general emergence with only local basins of sedimentation remaining.

Tertiary sediments fill graben, more than 2,000 m thick in places. There is a scarcity of coarse sediments in many of the basins, suggesting a landscape of no great relief for much of the Tertiary.

In the Pliocene to early Pleistocene large areas were uplifted to form highlands. The erosion that accompanied and followed this uplift flooded low-lying areas with the Pleistocene continental deposits. Weathering processes and the formation of slope debris and river deposits developed into the Holocene.

The tectonic setting in the project area is characterized by thrust sequences. A fold-thrust belt developed during late Palaeozoic – early Mesozoic events. This mobile belt (Yunnan – Malay belt) results from the convergence and collision of the Shan Thai block in the West and the Indosinia block in the East, with the closure of an intervening ocean basin that was preceded and accompanied by eastward subduction and underthrusting.

The Triassic collision was associated by granitic intrusions, which was followed by general uplift. Thereafter, relatively stable conditions prevailed. According to Mitchell (1992) there was again overthrusting further West in the Late Triassic and Early Jurassic. Continuing or renewed convergence in the extreme W of Thailand caused folding affecting the Mesozoic molasse formation. The severity of this folding has decreased towards the East.

Late Cenozoic tectonic events are generally attributed to the India – Eurasia collision. As a result of this collision, Southeast Asia was pushed south-eastwards and rotated. Strike-slip faults oriented NW-SE caused displacements of tens of kilometers. NNE-SSW-trending strike-slip faults are also important. In conjunction with these strike-slip dislocations, a series of extensional basins was opened up, extending N-S through the central part of Thailand. This extension tectonics may have begun in the Oligocene in the S (gulf area), and continued into the Miocene towards the N. There were basaltic eruptions in the early Quaternary, related to the prevailing extensional tectonics.

2.2 Geological conditions along the alignment

From the inlet to approx. km 6.7 the alignment traverses mainly highly fractured and locally faulted quartzitic sandstone with shale and siltstone intercalations. Depending on the orientation of the tunnel alignment, the moderately SW dipping bedding planes of the sandstone sequence are oriented parallel to the tunnel axis (approx. km 1.0 – 5.0). Due to the tunnel alignment curve between approx. km 5.0 and 6.7 the bedding planes are oriented with angles of approx. 40° to 50° to the tunnel axis. Between the inlet portal and approx. km 1.0 the dip orientation and angle of bedding planes is expected to be highly variable due to the influence of tectonic faulting and folding on the rock mass structure. Most faults are expected to run more or less perpendicular to the tunnel alignment. In combination with the fault zones concentrated inflow of groundwater to the tunnel has to be expected.

After crossing a NW-SE striking boundary fault the most crucial alignment section, located in a thrust fault of considerable thickness, is reached. The thrust fault is remarkably heterogeneous. It consists of limestone blocks, varying in sizes from hundreds of meters to decimeters, which are embedded in sheared shale, chert and sandstone. The limestone blocks usually show karst features such as corroded joints, pipes as well as caves. The karst cavities are likely to be filled partially with residual soils and boulders of collapsed rock mass. Perennial artesian springs from limestone blocks, located in this area, indicate a certain transmissivity through the faulted rock mass, most likely linked to individual fault/fracture zones. Peak inflow rates from such fault zones, exceeding several 10 l/s and slow decrease of the inflow have to be expected. At the present stage the probability of encountering limestone blocks at the level of the tunnel is considered to be low.

The highly tectonically disturbed boundary zone, which continues to approx. km 11.5, is followed by quartzitic sandstone with shale intercalations. From approx. km 12.5 to the tunnel outlet the moderately NE dipping bedding planes of the sandstone sequence are oriented parallel to the tunnel axis. According to the study of aerial photographs, highly fractured rock mass and faults are limited to sections under passing gullies running mainly NE – SW as well as from km 15.3 to approx. km 17.0. Most faults are expected to run almost perpendicular to the tunnel alignment, except a few NW – SE trending faults, identified by aerial photo analysis. In addition, sheared- and faulted shale intercalations parallel to the bedding have to be expected within this section. In combination with the fault zones high concentrated inflow of groundwater to the tunnel has to be expected.

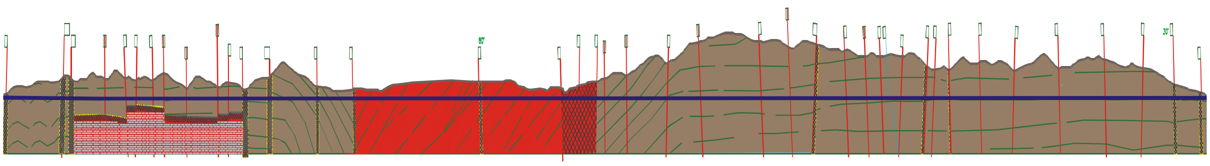

Figure 1.Simplified longitudinal section along the tunnel

2.2 Investigations

During several design phases field investigations were conducted. Besides the field work subsurface investigations were carried out. The subsurface investigations, including the drillings for the feasibility study, performed until now, consist of:

- 28 core drillings in total (boreholes T 7/1 – T 7/6 drilled for the feasibility study, boreholes DHW-1 to DHW-19 drilled for the detailed design study, DHI-1 and DHI-2 at new inlet - portal location, DHW-6 “New” close to the new alignment section)
- water pressure tests carried out in selected boreholes
- refraction seismic profiling and resistivity sounding
- laboratory tests (point load tests, uniaxial compressive strength tests, triaxial tests, tensile strength tests (Brazilian Test), direct shear test on discontinuities, Cerchar abrasivity tests, thin sections on outcrop and core samples), qualitative clay mineral analysis using X-Ray Diffractometry

Based on the results of the field, subsurface, and laboratory investigations the alignment of the tunnel was selected in a way to minimize excavation in poor rock masses.

3. ROCK MASS CHARACTERIZATION

3.1 Determination of Rock Mass Types

Based on the Austrian Guideline for the geotechnical design of underground structures (OeGG, 2001) the information gained from the field investigation, drilling and laboratory testing was used to define Rock Mass Types. Rock Mass Types are defined as groups of rock masses with similar geotechnical properties.

For this project 17 Rock Mass Types have been determined. Basis for the classification were following parameters: Rock type, Bedding thickness, Joint persistence, Fracturing, Joint roughness, Karstification, and Intact rock strength

For each Rock Mass Type rock mass properties were determined. As a basis the intact rock properties served, and for the upscaling in the rock mass scale the procedure proposed by Hoek (2002) was used with some minor modifications.

Both the intact rock properties and the rock mass properties in the project area vary in a wide range. Sandstones have been tested with a strength up to 250 MPa, while weaker fault rocks have a strength of a few MPa only. The rock mass strength properties accordingly vary in the range of around 1 MPa to a maximum of 100 MPa.

3.2 Distribution of Rock Mass Types along the alignment

Based on the investigation results the defined rock mass types were allocated to the alignment of the tunnel (see figure 2). For the purpose of further processing and comparison of the various construction methods investigated, the alignment was divided into sections of 50m length.

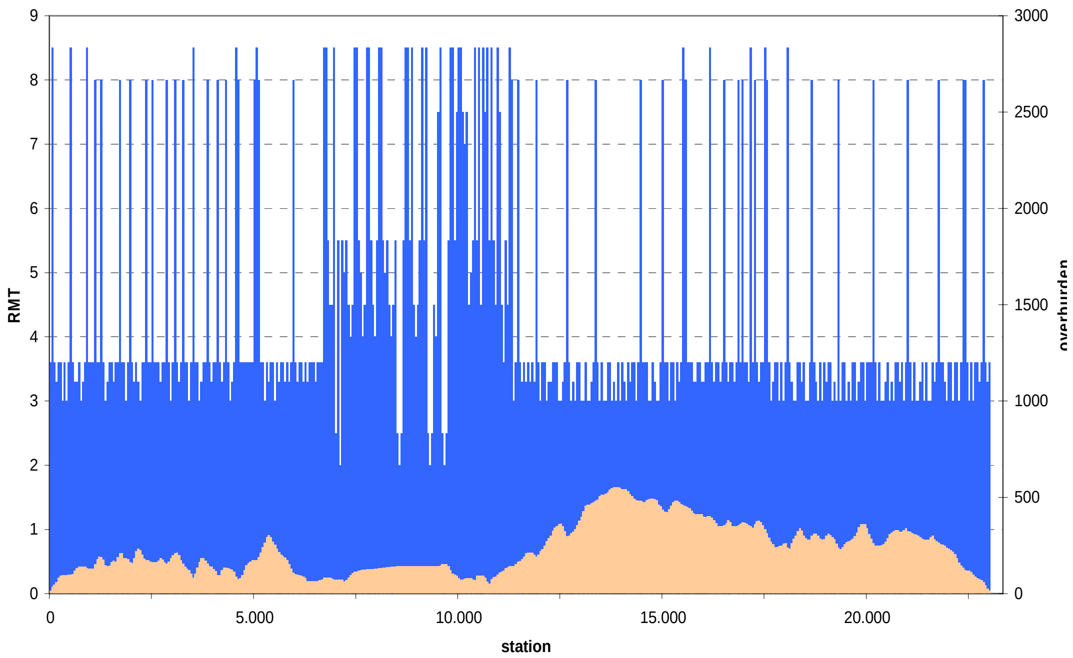

Figure 2. Distribution of the Rock Mass Types along the alignment. Shown in the diagram is also a longitudinal section along the tunnel

4. DETERMINATION OF ROCK MASS BEHAVIOUR

According to the Austrian Guideline the Rock Mass Behaviour, defined as the ground reaction to the excavation without support has to be evaluated prior to allocating construction methods. This requires the evaluation of failure modes under consideration of rock mass structure and quality and influencing factors, such as primary stresses, orientation of dominating structural features, and ground water.

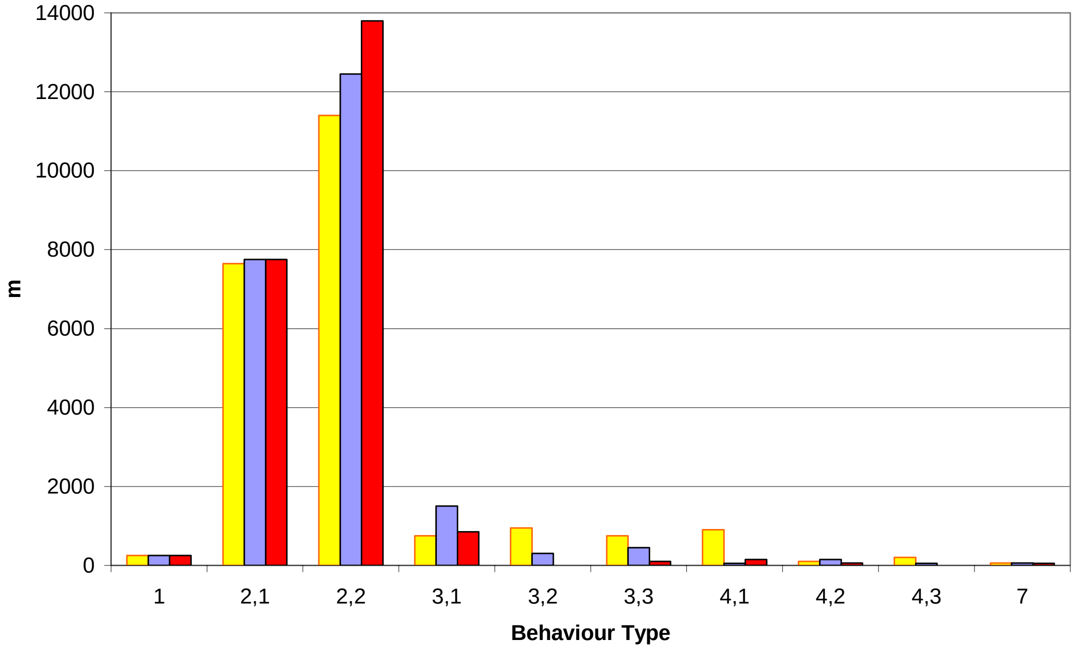

Figure 3. Distribution of Behaviour Types for the entire tunnel (1=stable, 2=discontinuity controlled overbreak, 3=shallow stress induced failure, 4= deep stress induced failure, 7= chimney type day lighting failure)

Analytical functions are used to evaluate stresses, displacements, and other values. Delimiting criteria are used to distinguish between the single failure modes. In a hierarchical way the rock mass behaviour is checked for occurrence of one of more failure modes. If no failure is detected the tunnel is supposed to be stable.

For three different sets of rock mass parameters this evaluation was done for the whole alignment. As can be seen from figure 3, the dominating Behaviour Type for all conditions evaluated is within category 2, which represents discontinuity controlled overbreak. For a minor length also Behaviour Type categories 3 (shallow stress induced failure) and 4 (deep seated stress induced failure) was identified. Due to the intense fracturing of the rock mass, stable conditions without support is expected on a length of a few hundred meters only. Subcategories have been defined for the categories 2, 3 and 4 for an appropriate allocation of construction measures to the single behaviours.

5. INVESTIGATED OPTIONS

Three options for construction have been studied. With option 1 the major part of the tunnel is excavated by drill and blast with conventional shotcrete rock bolt support, while from the outlet a TBM excavation on a length of approximately 11 km was foreseen. For this option both adits are required for construction. With option 2 a second TBM was foreseen to excavate from adit 6 towards the intake, while drill and blast was considered from adit 6 towards the outlet. With this option, adit 5 is required for depletion of the tunnel only. Option 3 finally consisted of two TBM excavations, one fron adit 5 towards the outlet, and the other one from the outlet towards the intake. With this option the adit 6 would not be required.

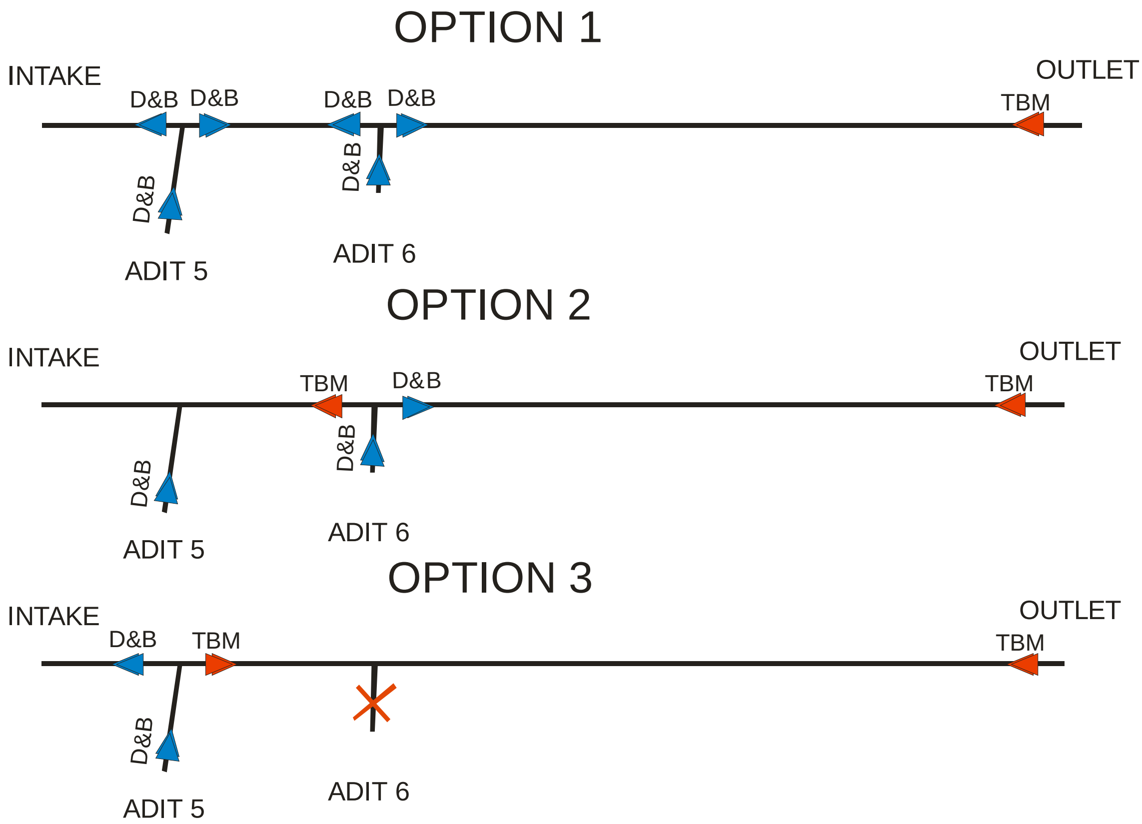

Figure 4. Options for construction methods studied.

Due to the requirement of a smooth inner surface for the TBM sections a continuous segment lining was foreseen. A double shield TBM was chosen for excavation. For the drill and blast sections a final concrete lining with a minimum thickness of 25cm was chosen.

Geotechnical criteria are used to assign excavation and support type to each behaviour type. Four excavation and support classes have been designed for the drill & blast excavation, and two segment types have been allocated to the TBM excavation. In the drill & blast sections the supports consist of a roof protection by shotcrete and spot bolting to prevent blocks from falling into the opening for the better rock masses, while a continuous shotcrete lining and systematic bolting, as well as steel arches are provided for the low quality rock masses. The difference between the two types of segments is in the concrete quality and amount of reinforcement.

After assignment of construction methods, the compatibility of the supports with the requirements is checked, and in cases of insufficient agreement the support modified.

Once excavation and support methods have been fixed for the whole tunnel, construction time and costs are evaluated. Construction time includes cycle times for the different excavation and support classes, as well as delays due to treatment of water and other activities, like probing ahead, etc. This again can be linked to the evaluated ground behaviour types, respectively rock mass types.

In terms of construction time for option 1 the time evaluated for the main civil works (excavation and support and inner lining) was around 40 months after award of contract (figure 5). For option 2 the required time for the main works would be very similar to that of option 1, while option 3 showed to be the fastest with a time requirement for the main works of around 30 months.

It shows that the total construction time with varying rock mass strength does not change significantly. The reason is that with higher rock mass strength the penetration rate of the TBM decreases, while the more favourable rock mass conditions allow a faster drill and blast progress due to a higher share of “better” excavation classes. This slightly changes the breakthrough location, but has a minor influence on the total construction time.

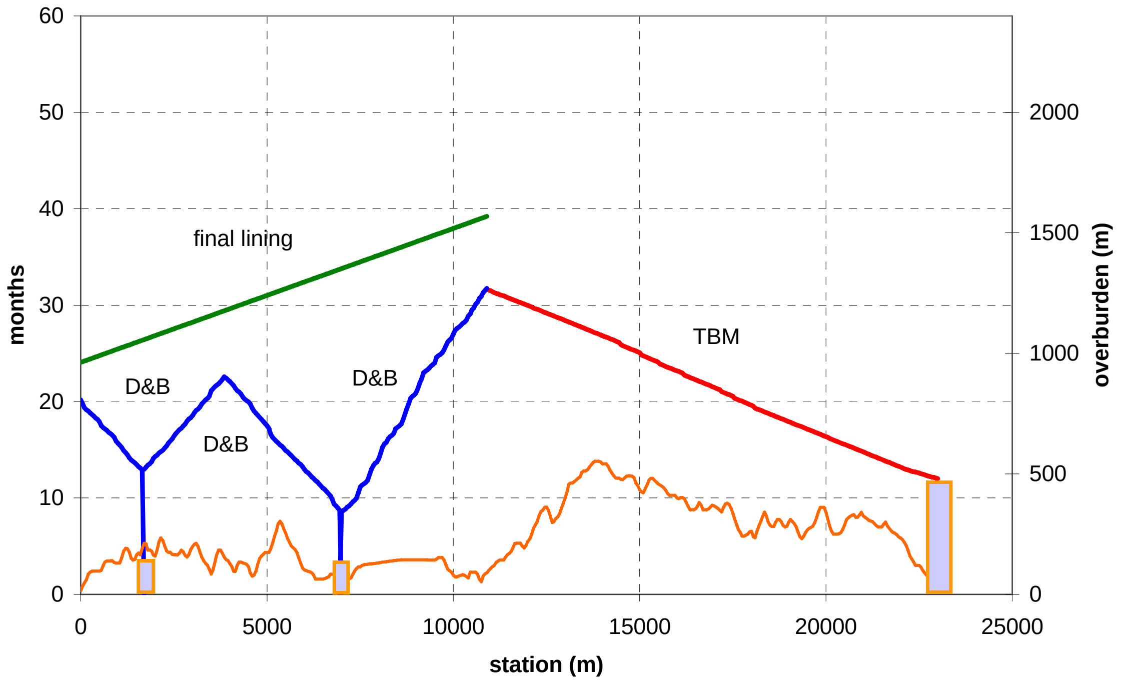

Figure 5. Rough construction time schedule for option 1 with medium rock mass quality.

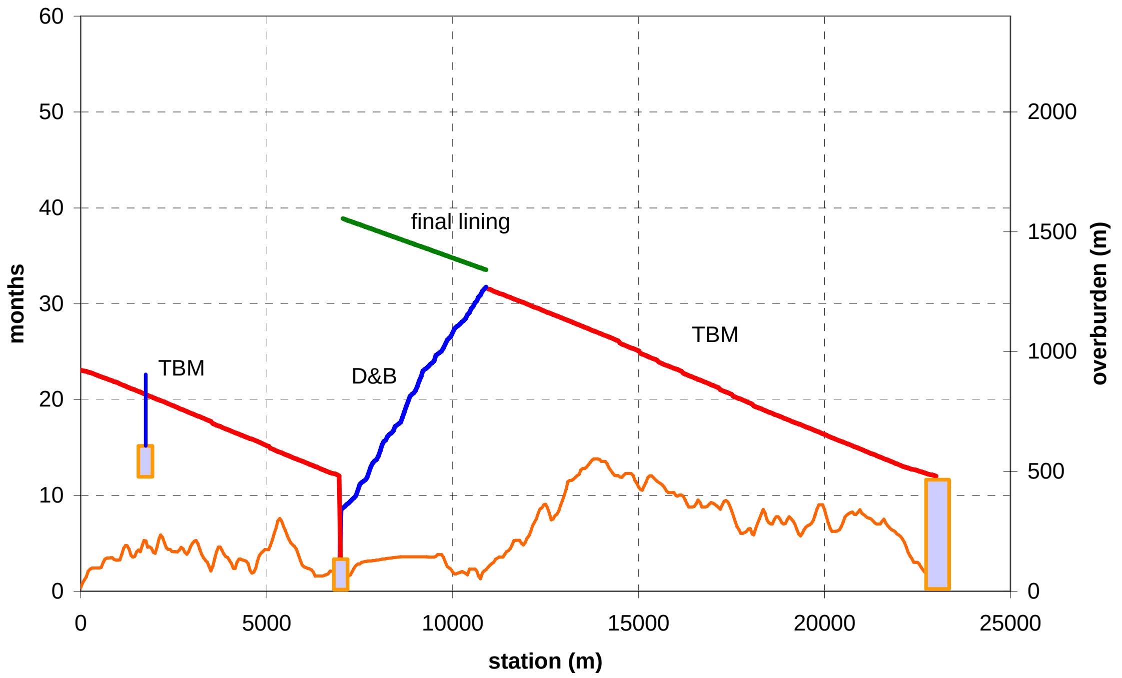

Figure 6. Rough construction time schedule for option 2 with medium rock mass quality.

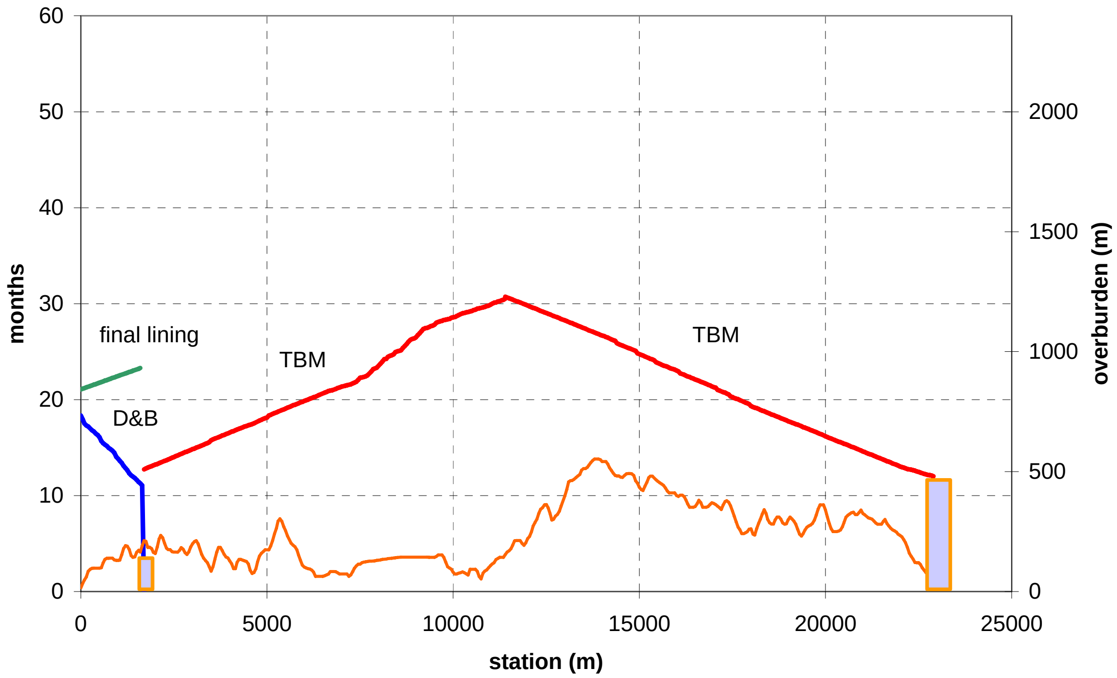

Figure 7. Rough construction time schedule for option 3 with medium rock mass quality.

A preliminary evaluation of the construction costs due to the relatively low labour costs in Thailand showed that option 1 with a higher share in labour intensive drill and blast would be the most economical one (figure 8). The difference in tunnelling costs between the most expensive option 2 with low rock mass quality to option 1 is in between 12% to 17%. The evaluation also clearly shows that the rock mass quality does not influence the costs of a TBM excavation considerably. This is attributed to the low degree of variation in support in relation to the rock mass quality. With the drill and blast method better rock mass quality reflects in lower costs, as progress rate increases, while support demand decreases.

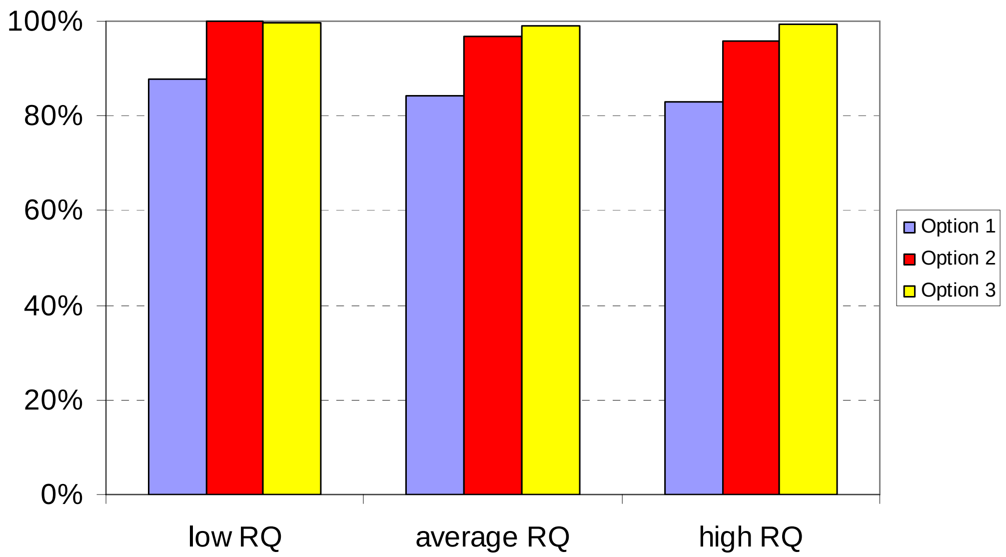

Figure 8. Relative costs of the options studied

6. CONCLUSION

When comparing several options in terms of construction method for a project, a consistent procedure is required to arrive at an unbiased result. In the case of the Mae Ngad – Mae Kuang water conveyance tunnel, a comprehensive rock mass characterization was followed by an evaluation of the ground behaviour. Assigning construction methods and auxiliary measures to the different ground behaviour types allows an objective evaluation of construction costs and time for different excavation options. The consideration of the spread in rock mass parameters allows evaluating the spread in construction time and costs.

Even with the in-depth evaluation of different alignments and construction methods, residual risks with respect to construction costs and time remain. This particularly is attributed to the complex geological conditions, and the uncertainty in the evaluation of realistic ground water conditions. The study therefore has considered delays due to effects of the ground water on the excavation on the safe side. A big advantage of the chosen option with total 5 headings is the fact, that in case of a problem in one of the headings requiring a longer stop, the other heading can proceed further, thus reducing the impact on the total construction time.

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