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Tunnel design in hilly area of Taiwan High Speed Rail project

Design du Tunnel traversant une zone vallonnée pour le Projet de Train a grande vitesse de Taiwan

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ABSTRACT

The Taiwan High Speed Rail (THSR) runs through several hilly areas in the northwestern island of Taiwan and there are therefore quite many tunnels constructed along the alignment. Special considerations are required for the design of these tunnels due to the face of soil-like rock strata, low overburden and underpassing the existing freeway with cover as thin as 9 m. The frequently used numerical-statistical classification methods like Bienawski's RMR-system or NGI's Q-system are not applicable for these tunnels. Considerations are also given to auxiliary measures, special portal design criteria for THSR, and effects of asymmetrical loads and earthquake loads. Various monitoring systems were then adopted to verify the initial design parameters and assumptions. The results reveal that the deformation and convergence of tunnel linings are significantly affected by the asymmetrical loads due to inclined topograpy, unfavorable bedding conditions and the poor strength of rocks.

RÉSUMÉ

La ligne de train a grande vitesse de Taiwan (THSR – pour Taiwan High Speed Railways) traverse dans le Nord du pays un zone de collines impliquant de nombreux ouvrages souterrains. L'environnement spécifique, en particulier une lithologie en succession de couches de roches tendres se aux propriétés de sol, de très faibles couvertures et le passage 9m en-dessous de la chaussée de l'autoroute No.2, qui est une des artères principales du pays, a demande une approche conceptuelle particulière. Les outils de design traditionnels tels que le « Rock mass rating » de Beniawski ou le Q-rating ne s'appliquent pas à ce genre de configuration. Le design doit en effet prendre en compte des paramètres supplémentaires, tels que les différentes caractéristiques de la lithologie rencontrée, le niveau de la nappe phreatique, la séquence d'excavation, la gamme de supports provisoires à la disposition de l'entreprise, les spécifications particulières aux entrées de tunnel pour trains a grande vitesse, le type de revêtement permanent, ainsi que l'influence de chargements dissymétriques et bien sur des tremblements de terre. Une panoplie de mesures en cours d'excavation a été mise en place pour vérifier les hypothèses et les valeurs de paramètres utilisées dans le design. Ce suivi a permis de vérifier l'influence déterminante sur la de formée et la convergence de l'excavation du tunnel des chargements dissymétriques générés par la topographie, de l'orientation des couches de terrain ainsi que des faibles caractéristiques géotechniques des roches tendres rencontrées.

1 INTRODUCTION

The Taiwan High Speed Rail (THSR) project, with an estimated cost of US$15 billion, is considered to be one of the largest BOT (Build-Operate-Transfer) projects in terms of scale. The rail runs along the western coast of Taiwan, where the majority of the Taiwan population resides. The majority length of the Rail runs on a relatively flat alluvial plane while there are several hilly areas along the northern segment where many tunnels were constructed. Discussed herein are the design of 11 tunnels in Contract C220. The lengths of these tunnels vary from 121m to 830 m. These tunnels are buried in weak sedimentary rocks and conglomerate, with limited covers above crowns. One of them underpasses the existing Second Freeway at a distance of only 9m.

The tunnels are approximately 15m in diameters with sectional areas of excavation ranging from 140 m^2 to 170 m^2. They are mined by using the Sequential Excavation and Support SES construction method.

2 SITE DESCRIPTION

2.1 *Ground Conditions*

The tunnels in the Contract C220 (Section Hsin-Chu) are located within the Western Foothills in the northwestern Taiwan, which are formed by theNeogene to Quaternary clastic sediments. These sediments are mostly uncemented to poorly cemented and are extremely weak to weak. The sediments comprise the Pliocene (Cholan Formation: sandstone and shale) and Pleistocene Formations (the Yangmei Formation: sandstone, mudstone, shale, interbedded with conglomerates; the Tamaopu and Toukoshan conglomerates; Tientzuhu Formation: laterite, gravel, sand, intercalated with sand/silt lentils). The Yangmei sediments were deposited in a deltaic environment, whereas the Tamaopu Conglomerate was laid down by braided streams. Generally, these coarse sediments were deposited in the beginning of the orogenic movements (sudden uplift of the Central Range) in western Taiwan.

2.2 *Design Requirements*

In accordance with specifications (THSRC 2000), all the components and materials with permanent functions shall be designed for a design life of 100 years. All the tunnels shall be lined with cast-in-place concrete and structural invert shall be provided. To mitigate aerodynamic effects, especially the so-called sonic boom, portals shall have inclinations flatter than 45°.

In the analysis for the permanent lining of the tunnels, earthquake effects are not necessary to be considered. However, for portals and for structures with covers less than 15 m or with cohesionless cover, seismic loads shall be included in the design. Furthermore, for all the tunnels constructed by using the cut and cover methods, seismic loads shall also be considered.

The cross-sectional areas of the tunnels are specified by the Employer. Design of the cross-sections shall account for:

- Aerodynamic effects
- Dynamic envelope and clearances
- Maintenance considerations
- Service requirements
- Safety requirements

A typical cross section of tunnels is shown in Fig. 1.

For a maximum speed of 300 km/hr of trains, a minimum cross sectional area of 90 m^2 is required to satisfy aerodynamic requirements.

3 DESIGN CONCERNS

During the design stage, emphasis was given to the following facts:

a) Very and extremely weak rocks are highly susceptible to softening once soaked leading to face stability problems.
b) Unfavorable dipping of rock beds may result in large horizontal displacement and even failure.
c) Shear failure at crown and chimney-type collapse should be avoided all the times.
d) Surface settlements should be strictly limited to avoid disruption to the operation of the existing roads and highway above crowns.

4 GEOTECHNICAL MODEL

4.1 *Methodology of Rock Classification*

The rocks encountered at the faces are predominantly soft rocks with poorly developed joints and residual soils with water-bearing strata. Characteristics of rocks, such as the properties of joints, which govern the design of rock tunnels are therefore not essential in such a case. As a result, the commonly used numerical-statistical classification methods such as Bienawski's RMR-system or NGI's Q-system are not applicable to these tunnels. However, existing bedding planes, which may cause some an-isotropic behavior of the rock mass, still have to be considered.

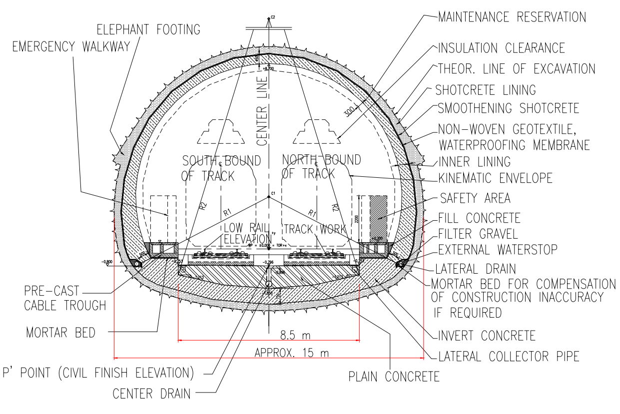

Figure 1. Typical Cross-section.

4.2 *Rock Mass Types*

The rock mass characterization includes characteristics of the lithological sequence and the results of mechanical testing. The definition of Rock Mass Types is as shown in Table 1 and 2.

The rocks encountered in Contract C220 are predominantly Types RM2, RM3 and RM4 rocks. At some locations, Type RM5 rocks are present.

Table 1. Rock Mass Types RM2 to RM5

Description	Rock mass type	Rock mass type		
Strength range of intact rock (CEDD, 1994)	-	Weak rock	Very weak rock	Extremely weak rock
Uniaxial compressive strength of intact rock	-	1.25MPa ≤UCS≤ 5MPa	0.5MPa ≤UCS≤ 1.25MPa	UCS≤ 0.5MPa
Index	-	a	b	c
Sandstone	RM2	RM2a	RM2b	RM2c
Mudstone	RM3	RM3a	RM3b	RM3c
Alternation of sandstone and mudstone, thickness of different layers in the range of decimeters	RM4	RM4a	RM4a	RM4c
Alternation of sandstone and mudstone, thickness of different layers in the range of centimeters to millimeters High potential of slickensides	RM5	RM5a	RM5b	RM5c

Table 2. Rock mass type RM1, RM6~RM9

Description	Rock mass type	
Moderately weak sandstone, sandstone intercalated with mudstone, intact sandstone is well cemented	RM1	Uniaxial compressive strength of intact rock 5MPa ≤ UCS ≤ 12.5MPa
Fault zones or highly fractured zones	RM6	
Lateritic terrace deposits	RM7a & RM7b	Lateritic gravel (RM7a) and lateritic clay (RM7b)
River deposits	RM8	Well graded gravel with cobbles and boulders
In-situ soil	RM9	Highly weathered sandstone (RM9a) and highly weathered mudstone (RM9b)

5 DESIGN AND ANALYSIS OF TUNNELS

5.1 *Excavation Sequence*

In order to minimize ground settlements and to maintain face stability, excavation is usually carried out in sections and it is essential to select an adequate construction sequence for excavating large size tunnels in soft rock with shallow covers. It is also often required to lower groundwater to below the invert level in advance.

In general, the construction sequences showing in Figures 2, 3 are followed. Excavation of the top heading will advance for, at least 30 to 40 m, in order to provide sufficient space for maneuvering the necessary construction equipment. The section may have to be closed by providing temporary arch at the invert if ground conditions are poor. Decision is to be made at the site based on the results of analyses and field observations.

For the tunnel under-passing the Second Freeway, the tunnel section is subdivided into two "side wall galleries" and a center pillar as shown in Fig. 4. One side-wall gallery was excavated first for a distance of 10m to 30m and lined by shotcreting before the second side gallery was advanced. The removal of the center pillar was in pace with the lining for the second gallery with a pre-specified distance in-between.

5.2 *Support types*

Tunnel supports were analyzed by using analytical and/or numerical procedures with due considerations given to the construction sequences. The adequacy of the construction sequence and the tunnel supports was verified by observations during tunnel driving, and if necessary, the construction sequences and tunnel supports were modified based on back-analysis.

Because of the poor ground conditions, only Type III (short time stable), Type IV (Unstable), Type V (Slightly to fairly squeezing) and Type P (Portal class) are adopted. Type I (stable condition without support) and II (stable condition with little support) supports are not applicable herein.

5.3 *Portal Design*

The basic principle is to reduce the disturbance to the slopes as little as possible. The following are the major issues in the design of portals:

a) Minimizing the volume and height of cuts.
b) Using mainly nails (fully grouted steel rods installed in boreholes drilled with casing using air flush or self-drilling injection bolts) and shotcrete as temporary retaining measure.
c) Increasing face stability by dewatering (drainage drillings/weephole) to avoid seepage forces.
d) In order to minimize the height of portal cut, the tunnels have to be driven within low cover of 3 to 10 meters. For such shallow overburden, auxiliary supporting measures such as pipe roof may become necessary.
e) Application of cement to stabilize soil embankments in case of unfavorable portal geometry.

Soil nailing is applied mainly for temporary slopes. However, in some special cases, soil nailing is allowed to be used for permanent stage. In such cases, the nails have to have sufficient protection against corrosion in order to meet the specification that permanent components must be designed for a design life of 100 years.

5.4 *Analysis*

The tunnel structure consists of the primary shotcrete lining and the secondary cast-in-situ inner lining.

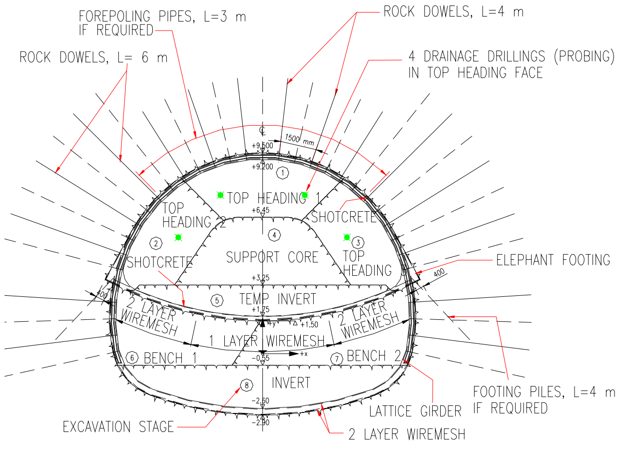

Figure 2. Typical Support Type.

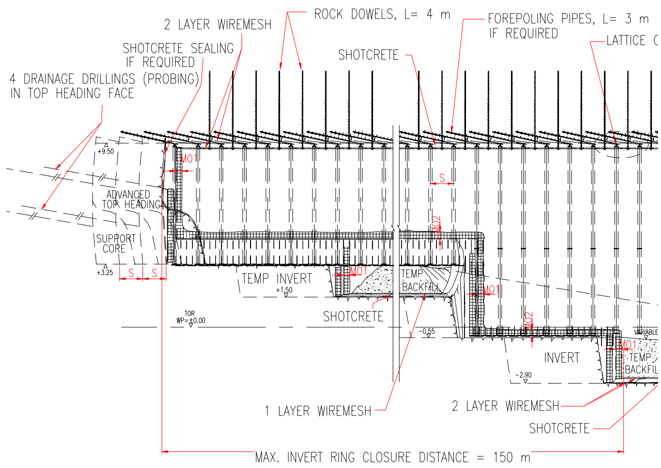

Figure 3. Typical Excavation Sequence.

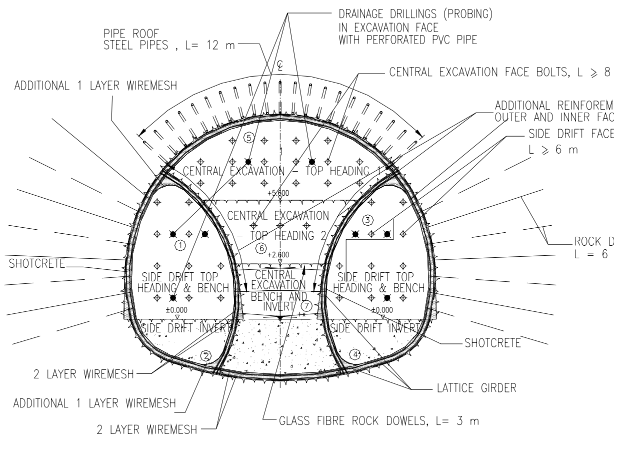

Figure 4. Typical Section of Side Wall Galleries Tunneling (for the underpass of the Second Freeway).

Shotcrete lining is designed as initial ground support considering soil overburden, ground water condition and nominal surcharge load. It was typically achieved with an array of rock bolts from the footing of the bench to the crown, with lattice girders spaced at 0.8m to 1.8m, and shotcrete with a thickness ranging from 200mm to 300mm.

The cast in-situ concrete secondary lining is designed for partial earth load and maximum expected water pressure.

For both the primary lining and the secondary lining, the Limit State Design Concept is adopted with considerations given to the ultimate failure conditions and service life.

The analysis of the shotcrete lining was performed using the finite difference method (FDM) using FLAC code (1999) or finite element method (FEM) using Z_soil. For cross-check, analytical approaches such as closed-form analysis (Erdmann 1982 & 1983) and silo theory (Terzaghi 1943) are used.

Face stability is evaluated by using analytical approach of a simple wedge model and 3-dimensional silo theory (Terzaghi 1943).

The analysis of the secondary lining is performed by means of a plane frame analysis model using SAP2000 (1995) or finite element method (FEM) using Z_soil with the primary lining and the secondary lining coupled by using inter-face elements.

Earthquake effects are simulated by applying horizontal earth pressure on only one side of the structure. This additional earth pressure is called dynamic soil pressure increment and is distributed constantly over the entire height of the structure. In addition, raking analysis was performed by applying pre-specified deformations in the design of secondary lining.

6 DESIGN VERIFICATION

During construction, the design was constantly verified based on the field observations. The instrument readings obtained were compared with the related specified control limits, which are presented in the working specifications.

The monitoring included

- Surface movements in 3 directions
- Deformation of tunnel lining in three directions
- Variation of ground movements with depth
- Lateral ground movements at portals

The following has been observed (DCK JV et. al 2001, Daiho Corp. et al. 2002 & 2003):

a) The rates of settlements were 1.3 mm/day to 4 mm/day within the first 4 days and became minimal once the face was 20m to 40 m away.

b) At some places, initial readings were not established in time. To be conservative, it was assumed that ground had settled by 2mm/day to 4mm/day prior to the establishment of the initial readings based on the above observation.

c) The total settlements were in the range of 12 mm to 51 mm after the completion of the top heading and increased by 5mm to 21mm during the excavation for bench.

d) During the excavation of some tunnels where the bedding dip angles are in the range of 22 degree to 30 degree with strike almost parallel to the axis of tunnel, large deformations occurred and supplementary rock dowels had to be applied for maintaining safety. Such unfavorable dipping of rock beds caused some anisotropic ground behavior. In some cases, significant lateral down-hill movements were observed.

e) Joints and beddings with dip angles less than 14 degrees do not affect significantly the stability of tunneling and the patterns of ground movements.

f) For tunnels with shallow covers, there is no indication that there exist high horizontal active pressures if the rock beds dip at angles less than 14 degree.

g) The deformation and convergence are significantly affected by the asymmetrical load due to inclined topograpy, unfavorable bedding condition and the poor rock strength.

7 CONCLUSIONS

This paper describes the methodology and rock classification adopted on the tunnel design of THSR in Contract C220, in which the classification methods such as Bienawski's RMR-system or NGI's Q-system are not applicable due to soft rock condition.

All the tunnels with shallow covers were successfully completed and a tunnel underpassing a busy freeway was completed without disturbing traffic on the freeway.

Observations reveal that ground movments and convergence of tunnel lining are significantly affected by the asymmetrical load due to inclined topograpy, unfavorable bedding condition and the poor rock strength.

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