ABSTRACT:

The Taiwan High Speed Rail Project includes the Design + Build Contract C220 between Chu Pei and Pao Shan which comprises 12 tunnels with total length of some 3,960 m. The most interesting and challenging mined tunnel of this contract is the 830 m long Hsinchu Second Freeway Tunnel. The southern part of the tunnel underpasses the existing Second Freeway for a length of some 108 m with the theoretical excavation area of 150 m^2. The vertical clearance between the freeway pavement and the tunnel crown is about 9 m only. The contract specifications have two particular requirements as firstly the tunnel section below the freeway shall be designed as an undrained watertight tunnel, and secondly that the settlements under the freeway will not exceed 20mm. Full operation of the freeway is required during the tunnel construction works.

The existing natural ground consists mainly of weak to extremely weak sandstone with alternations of weak to extremely weak mudstone. Slickensides and old slip-planes from past landslides had to be considered. The freeway embankment consists of compacted soil originated from excavated extremely weak sandstone and mudstone. The fill material shows SPT N300 values between 16 and 60 blows. The groundwater is located at tunnel invert. Locally some perched water exists in sand layers at higher elevations.

Several construction options have been investigated. Finally the NATM method has been selected - applying the particular excavation sequence with two side galleries followed by central enlargement with double pipe roof, face bolting and short ring closure. Integral part of the design is extensive above-ground monitoring comprising automatic “on-line” devices (with measurement interval of few minutes) as 3D-surface deformation markers, automatic settlement sensors embedded in the freeway fill, horizontal extensometers between pavement and tunnel crown along the entire underpass section, vertical extensometers and vertical inclinometers. In addition observation wells and piezometers are foreseen which have to be monitored manually. Below ground monitoring consists of the optical 3D-monitoring of the shotcrete lining deformations in absolute co-ordinates. Control limits and related action plans have been elaborated in order to take correct timely actions in case of unexpected geotechnical behavior.

The construction works for the underpass has been commenced end of September 2002 and all related outer lining works have been completed successfully mid of May 2003. The inner lining of the entire tunnel has been completed end of 2003 including cut and cover structures. The biggest challenge was that the embankment material might be very heterogeneous with local cohesion-less zones. In practice it turned out that the fill was quite uniform in terms of strength but the extent was quite variable. Below the north bound carriageways the tunnel excavation was located full-face in fill material associated with larger lining deformations. The extensive monitoring program provided a satisfying tool to control “on-line” the safety conditions of the tunnel construction works.

Taiwan HSR C220 – The Underpass of the Second Freeway

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1 INTRODUCTION

The Design + Build Contract C220 of the Taiwan High Speed Rail Project is located in northern Taiwan in the larger area of the city Hsinchu. The Contractor is Daiho Corporation (Japan), main design Consultant is Moh and Associates (Taiwan) with United Geotech. (Taiwan) and iC Consulenten (Austria) as sub-consultants for tunnel design. Daiho Corporation employed additionally iC as adviser. The contract documents allocate the ground risk to the Contractor and require a self-certification system employed by the Contractor to supervise the construction works.

The Figure 1 shows the layout of the underpass site of the Hsinchu Second Freeway Tunnel. The vertical clearance between the freeway pavement and the tunnel crown is about 9 m only. A prerequisite for the tunnel construction is the full operation of the freeway without any lane diversions.

2 GEOTECHNICAL CONDITIONS

The tunnel runs through the lower member of the Toukoshan formation which consists of interbedded sandstone, siltstone and mudstone. The intact rock strength is weak (1.25MPa ≤ UCS ≤ 5 MPa; UCS = unconfined compressive strength) to extremely weak (UCS ≤ 0.5 MPa). In fact the extremely weak rock can be just soil with very low cohesion - especially the sands in combination with groundwater can be very vulnerable to erosion. The formation shows distinct bedding planes. At the underpass area the bedding is dipping some 18 degrees to the south towards the Kantzuki Syncline – which is located approximately 100 m south and parallel to the freeway. Natural shear planes with slickensided surface have been found near the South portal dipping 45 degrees south.

The tunnel is partly located within the freeway embankment – which consists of compacted soil originated from excavated sandstone and mudstone. The fill material shows SPT values between 16 and 60 blows. The high fine content of the fill material makes improvement by grouting practically impossible. In the year 1994 a large-scale landslide occurred during freeway embankment construction. The tunnel is crossing the old slip plane below the southern crest of the freeway embankment. The groundwater at the underpass section is located at tunnel invert. Locally some perched water exists in sand layers at higher elevations.

3 DESIGN APPROACH FOR EXCAVATION AND MONITORING

3.1 *Excavation, Support and Over-all Construction Sequence*

With respect to the large tunnel cross-sectional area and the rather short tunnel length - the NATM method has been selected for all mined tunnels of C220. For the particular underpass section several construction options have been investigated. Finally the NATM method has been chosen - applying the particular support type "S" with two side galleries (SD1, SD2) followed by central enlargement (CE). The tunnel drive direction is chosen from north to south in order to minimize the risk of face slides due to slickensided bedding/shear planes. The shotcrete lining is 300mm thick with 2 layers of wire mesh – also for the inner side-walls. The excavation sequence is designed to achieve a short ring closure – e.g. 7 m for the CE. Additional support elements are grouted fore poling (PUIF method, L = 3 m) for SD1 and SD2, double pipe roof (AGF method, L = 15.9 m) for CE. Glass fiber face bolting (Small-pipe method, L = 12 m for SD; L = 16m for CE) is mandatory for all faces in fill material and for the short excavation from South portal towards north (Small-pipe method, L = 15 m & 17 m). In order to improve the ductility of the lining - systematic application of rock dowels is foreseen at all side walls. The over-all excavation sequence is designed in detail as follows:

- Tunnel construction from North portal until 78+300
- Installation of special geotechnical monitoring devices at Second Freeway area
- Further tunnel excavation from north portal until 78+365.3 and installation of domed shotcrete head wall
- Commence tunnel construction Type S - side drifts SD1 and SD2 - between 78+365.3 and 78+472.9. The faces of SD1 and SD2 are staggered between 10 m and 20 m
- Excavation from South portal until 78+472.9 using common support type with additional heavy face bolting from south to north; install domed shotcrete head wall at 78+472.9 including face bolts
- Break-through of side drift faces SD1 and SD2 at southern head wall 78+472.9
- Repair of SD1/SD2 shotcrete lining with respect to inspection results
- Execution of central TH excavation and B/I between 78+365.3 and 78+472.9 from north to south with short full ring closure, break-through at 78+472.9
- Installation of inner lining as soon as possible

3.2 *Monitoring Arrangement and Devices*

The monitoring arrangement for the underpass excavation is shown in Figure 1. The arrangement consists mainly of 3 main monitoring sections along the freeway, 3 special settlement sections along the High Speed Rail tunnel and regular monitoring sections for tunnel lining deformations with regular spacing of 8 m. Surface points, settlement cells, inclinometers and extensometers are measured automatically with continues data transfer to the monitoring station and further internet distribution to all involved parties.

Surface deformation markers are installed along the freeway crests and freeway centre line shallow below ground level. Those markers are monitored by automatic optical survey methods to determine 3D-displacements. The points consist of solid steel bars, which are founded by use of concrete. A survey target is fixed at the top of the steel bars.

Long horizontal boreholes with diameter 150mm are arranged some 5m above the side drift tunnels and the central enlargement. Those 3 boreholes harbor in-place inclinometers which consist of a series of interconnected hinged rods – with a tilt sensor mounted on each rod. The horizontal in-place inclinometers are connected to the data acquisition unit. The accuracy is specified as ± 2,5 mm per 30 m of casing.

The system for the Deep Subsurface Settlement Indicator (Settlement Sensor) consists of 15 "liquid settlement cells" with electric transducers. The accuracy is in the range of ±5 mm. The cells are also installed in the horizontal boreholes for the horizontal inclinometers.

Vertical inclinometers are foreseen at 4 locations at the freeway embankment crest adjacent to the tunnel. The scope of those inclinometers is in particular to detect at early stage any critical movements of potentially deep seated slickensided shear planes. The same system is applied – as for the horizontal in-place inclinometers. Vertical multiple extensometer are installed adjacent to the vertical inclinometers with electronic monitoring sensors at depth of 4, 8, 14, 20 and 30m.

Piezometers (observation wells) are the only above ground monitoring devices which required manual data collection. The installation locations are the embankment crests adjacent to the tunnel.

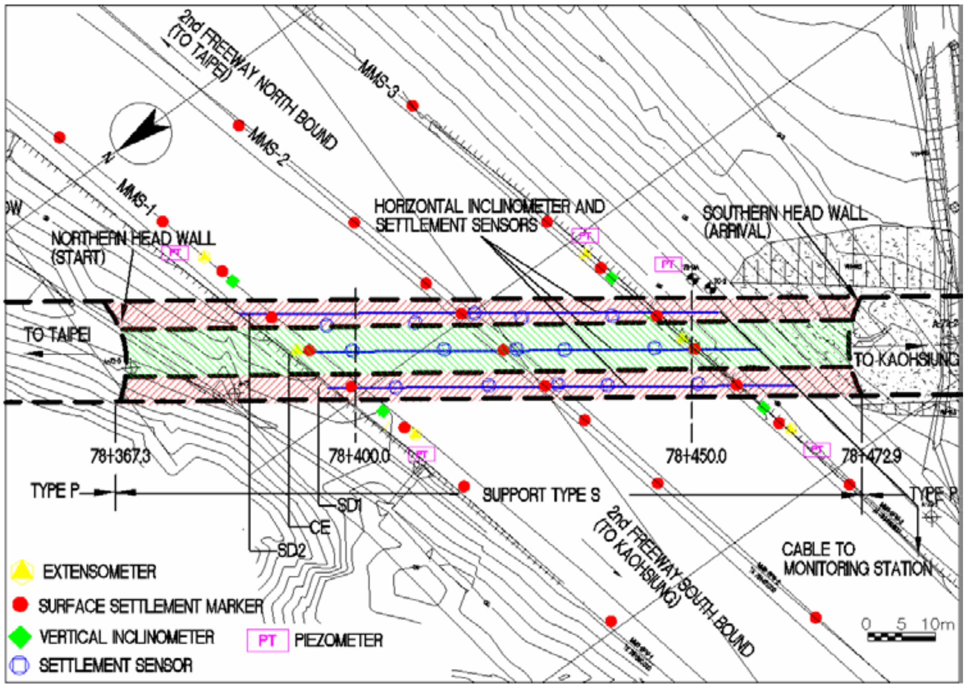

Figure 1: Layout of underpass site with monitoring arrangement

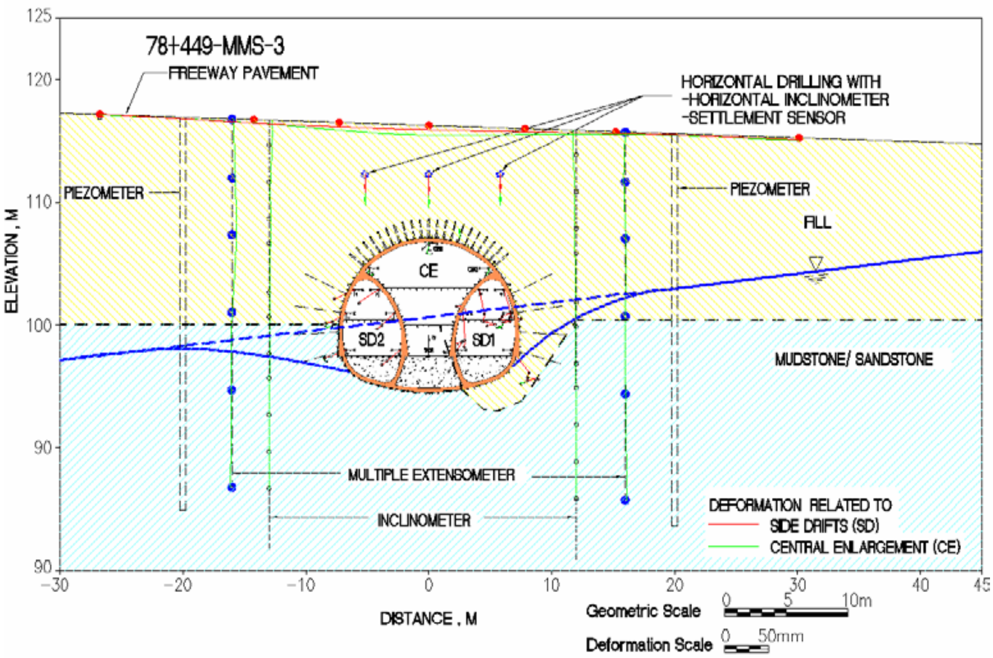

Figure 2: Main monitoring section 78+449 with monitoring results

Optical deformation monitoring of targets with reflectors is used to monitor deformations of the tunnel excavations in the three-dimensional co-ordinate system. The measurements are carried out with a high precision optoelectronic theodolite with integrated coaxial electronic distance meter.

3.3 *Control Values and Implementation of Monitoring Results*

Trigger values relate to 50% of the expected design values. The conditions are considered as normal if the monitoring values are within the trigger values. Increased attention is taken if the monitoring values are exceeding the trigger values. Contingency measures are prepared.

Design values relate to the upper bound of expected values based on design calculations. Critical conditions are anticipated if the monitoring values exceed the design values – thus contingency measures are carried out.

Allowable values are defined as 1.25 times the Design Values. Allowable values are the absolute limits which shall not be exceeded.

The defined control limits for the shotcrete lining deformations and convergences depend on the age of the shotcrete and are specified for defined construction zones. The "Excavation Zone" defined between excavation face and closed shotcrete ring. Design values are in the range of up to 30mm. The "Young Shotcrete Zone" applies between ring closure and one tunnel diameter behind. The age of the shotcrete is less than 1 week. Design values range up to 25mm. The "Hard Shotcrete Zone" is further behind the "young shotcrete" zone. The age of the shotcrete is more than 1 week. Related design values are up to 20mm. The deformation velocity within the "Hard Shotcrete Zone" - in a distance of about 3-times the excavation diameter from the face - shall decrease down to less than 1 mm/month for stable lining conditions. Displacement acceleration shall be below zero in order to confirm a stabilizing trend.

The design value for surface settlement is 40mm and the related maximum inclination of the settlement trough is 1:350. Design values for vertical inclinometer and extensometer are less than 50mm. Design values for pore pressures relate to hydraulic head at spring line elevation.

Even if the system behavior seems to be more favorable than predicted no cost reduction by adjustment of the design is appropriate due to the high-risk area underneath the freeway.

4 CONSTRUCTION AND MONITORING RESULTS

The construction works for the underpass have been commenced end of September 2002 and the side drifts have been completed in February 2003 – the progress is about 1.2 m/calendar day. The encountered ground conditions were within the expectations, the monitoring results predominately within the trigger values. Figure 2 shows the most southern main monitoring section 78+449 with the measured deformations. In particular the surface settlements were less than 10mm. Tangential strains of the shotcrete lining are estimated in the range of 0.1% based on the evaluation of the monitored lining deformations – with local exception of 0.27% in the full-face fill section.

Some adverse exception was the shotcrete lining displacements for some monitoring sections (78+432 to 78+448) below the north bound carriageway near the South portal. At those monitoring sections the SD1 excavation was located full-face in fill material. The deformations did not cease – thus additional grouted glass fiber bolts and drainage drillings had been installed at the side-walls and invert of both side drifts. This measure could stop the deformations quite quickly.

The central excavation started in February 2003 and all related outer lining works have been completed successfully mid of May 2003 – the progress is about 1.3 m/calendar day. The occurred further deformations were generally smaller than experienced during side drift excavation. The surface settlements increased to some 15mm in maximum. For the monitoring sections between 78+432 and 78+448 – again additional glass fiber bolts and drainage drillings were required to cease the lining deformations. Increase of tangential strains of the shotcrete lining is in general in the range of 0.05% - locally up to 0.10%.

The inner lining has been installed between June 2003 and August 2003. The construction works for the underpass could be completed successfully without any traffic disturbance. The inner lining deformations are measured in three monitoring sections. The results show practically no deformations.

Long-term monitoring is foreseen – as part of THSRC's entire long-term monitoring program - for certain installed devices as the vertical inclinometer and extensometer, the surface monitoring targets, the three monitoring sections for the inner lining.

ACKNOWLEDGEMENT

The authors acknowledge gratefully the Taiwan High Speed Rail Corporation (THSRC) for the permission to present this paper.

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