Design and Construction Challenges in Urban Settings for the NATM Tunnels' Line 2 of the Riyadh Metro

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Abstract: The Riyadh Metro Project entails the construction of six new lines. ArRiyadh Development Authority, the executive arm of the High Commission for the Development of Arriyadh contracted BACS consortium to design and build Lines 1 and 2; Gall Zeidler Consultants designed mined tunnels which are sections of the tunnels for the Lines 1 and 2 including emergency egress shafts and connection adits.

This paper describes design and construction of Line 2 mined tunnels in an urban settings including challenges encountered during excavation in close proximity to the building foundations, piers and methods to control inflow of groundwater in ground conditions varying from disturbed brecciated to fresh intact limestone.

INTRODUCTION

The Riyadh Metro is a rapid transit system which will be serving the city of Riyadh that consists of 6 metro lines with an overall length of 176 kilometers including 85 stations. The project is envisioned to meet the transportation needs of Riyadh's growing population which is estimated to increase by more than 8.3 million by 2030. The network will connect the King Khalid International Airport and King Abdullah Financial District with the main universities, downtown and the public transport center. The project will also reduce traffic congestion and improve air quality. The total estimated project cost is $23 billion. The metro project is owned and operated by ArRiyadh Development Authority (ADA). ADA awarded the design and construction contracts of the Riyadh metro to the ANM, FAST and BACS consortiums in October 2013. The BACS consortium for the design and construction of the Metro Lines 1 and 2 with an overall contract value of $10 billion includes Bechtel, Almabani General Contractors, Consolidated Contractors Company and Siemens and is led by Bechtel.

Contracted by the BACS consortium, Gall Zeidler Consultants (GZ) provided technical guidelines and requirements for the design of all structures associated with the permanent and temporary works for the construction of the Line 2 running tunnels and Lines 1 and 2 emergency egress shafts. This paper describes the design and construction of the Line 2 running tunnels that were excavated using the sequential excavation method (SEM).

Project Description

The Metro Line 2 runs mostly at grade from east to west along King Abdullah Road, and will extend more than 25km with 14 stations including 2 transfer stations. The running tunnels, shown in Figure 1, are part of the Line 2 tunnel. The total length of the running tunnels is approximately 2km, with a 1-km section driven westbound of the Olaya Station that connects to the cut-and-cover tunnels adjacent to the 2B1 Station, and a 1-km section driven eastbound of Olaya Station that ends at the 2B4 Station. The alignment is generally straight in a SW-NE direction and has a curvature around the Olaya Station to accommodate the transitions in and out of the station. The running tunnels were excavated from a temporary shaft built within the footprint of the 2B2 Olaya transfer Station. The tunnel is located at depths varying from 5m to 17m below ground surface. The vertical alignment starts at an approximate elevation of 612 m to the east of station 2B1, deepens to an elevation of 605m and then slightly rises to a level of 607m at the Olaya Station. East of the Olaya Station to the end of the mined section, the alignment ascends to an approximate elevation of 612m.

GEOLOGY AND GROUND CONDITIONS

The regional geology within the project area comprises thick sedimentary successions of Jurassic-Cretaceous limestone and anhydrite which follow the general structural setting of the region and gently dip from SW to NE. The general stratigraphy along the Line 2 consists of a thin layer of fill material and

Quaternary alluvial deposits comprising interbedded layers of silty sand and gravel overlying anhydrite and limestone rock members of Cretaceous and Jurassic age. Three different rock formations namely the Sulaiy Formation, Arab Formation and Jubaila Formation are encountered from top to bottom respectively. The Cretacious Sulaiy Formation consists predominantly of limestone with calcarenite beds and outcrops along East Riyadh. The Jurassic Arab Formation comprises limestone and brecciated limestone. The Jurassic Jubaila Formation comprises limestone with some calcrenite beds and mostly outcrops along West Riyadh.

The running tunnel alignment cuts through the carbonate members of the Arab Formation and encroaches locally into the Sulaiy Formation at the east close to Station 2B4. Thin layers of fill and Quaternary deposits comprising mostly silty sand and silty clay with limestone fragments and cemented granular materials overlie the Arab Formation. The Arab Formation is a contorted and brecciated alternation of limestone and thin beds of anhydrite consisting of four stacked carbonate-evaporite cycles, named Upper Limestone Breccia, Arab-C Disturbed Bedded Limestone, Lower Limestone Breccia and Arab-D Undisturbed Bedded Limestone. The mined tunnel plan and profile view with geology is shown in Figure 2.

The Upper Limestone Breccia member represents cemented breccia of collapsed blocks of the overlying Sulaiy formation mixed with limestone blocks from Arab formation. This member originates from the dissolution of the Hith Anhydrite member initially present between the Sulaiy and Arab Formations. The breccia is generally matrix-supported and comprises clasts ranging from coarse

Figure 1. Line 2 Tunnel alignment (red) Running Tunnel and its proximity to the other structures

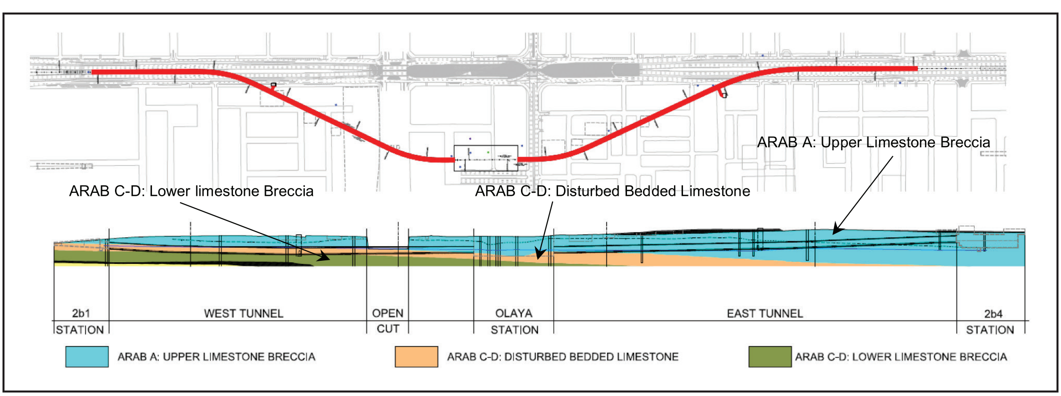

Figure 2. Mined tunnel plan and profile view with geology

gravel to very large sized boulders. The Arab-C member comprises very thin to thinly bedded calcareous claystones, calcareous siltstones and reddish brown partially dolomitised calcarenite and fine-grained siliceous limestone. This member is similar to the one encountered in the Upper Limestone Breccia originating from the dissolution of breccia beds. The Lower Limestone Breccia comprises moderately weathered, moderately to well-cemented, moderately strong, creamy white to yellowish brown, matrix-supported clasts of limestone breccia. Some larger clasts have been dolomitised and are strong to very strong. The matrix comprises yellowish brown, mottled white silt, some clay, and minor quantity of fine sand. The Arab-D member comprises grey to bluish limestone and some calcarenite beds. The limestone is slightly weathered, thick to very thin sub-horizontally bedded and undisturbed.

The west tunnel is mostly within the Disturbed Bedded Limestone with tunnel roof reaching the Upper Limestone Breccia at higher elevations. The east tunnel is mostly within the Upper Limestone Breccia while the invert reaches the lower Disturbed Bedded Limestone near the Olaya Station. Both of these members are highly heterogeneous and display varying behavior based on the intactness of the rock. Weathered profiles characterize the shallow strata especially at the eastern part of the alignment, however localized seams of deteriorated material were expected along deeper level. Sub-horizontal and sub-vertical discontinuities and vugs characterize the entire Arab-C Formation and local failures were expected during excavation.

Dissolution cavities were expected in all limestone formations. These cavities originated from dissolution of limestone and anhydrite beds by acidic groundwater along fractures and discontinuities, which is a typical karstification process in carbonate rocks. The upper members of the Arab formation are most prone to such cavities.

The carbonate rocks encountered along the alignment of the Line 2 mined tunnels comprise the main regional aquifer in the Riyadh area. The groundwater level is highly variable in the city of Riyadh as it is influenced by infiltration from sewage and is hydraulically connected to the shallower aquifers developed in the gravel in surrounding areas. Therefore, groundwater levels displayed a significant seasonal variation along with very high permeability of the limestone formation led to significant groundwater inflow in the tunnel.

TUNNEL DESIGN AND ANALYSIS

The cross section has maximum height of 9m and width of 10.9m at the tunnel springline (Figure 3). The excavation and support design followed the principles of SEM which is also referred to as New Austrian Tunneling Method (NATM). The excavation and support method was selected to preserve the load-bearing capacity of the surrounding rock mass which mobilizes the strength of the ground and allows implementation of lighter support. The selected excavation and support method also considered the timing of the support installation to optimize support requirements. The design also provided toolbox item such as rebar spiles as local measure to provide additional support when soft, highly fractured rock or soil was encountered in the crown.

The primary lining comprised 250mm of steel fiber-reinforced shotcrete lining and systematic rock dowels. The rock dowels were designed with a length of 3m spaced in a 2m × 1.5m staggered pattern. The dashed line on Figure 3 indicate those rock dowels within which were omitted during construction due to the prevailing good ground conditions. The excavation and support sequence typically included a 1.5m long top heading round followed by installation of rock dowels and shotcrete lining. During construction, the 1.5m round length was gradually increased to a maximum of 3m upon encountering of ground with relatively good rock mass quality. The cycle was repeated twice followed by staggered excavation and support of bench/invert with a round length of 3m. The secondary lining comprised 300mm of cast-in-place steel fiber-reinforced concrete. A full-round waterproofing membrane was placed between the primary and secondary linings to prevent water inflow into the tunnel.

The running tunnels were also connected to the two Emergency Egress Shafts by means of smaller connecting adit tunnels. The adits were constructed using SEM by breaking out from the Line 2 SEM running tunnels. The tunnel design was in accordance with BS EN 1990, BS EN 1991, BS EN 1992, and BS EN 1997. Structural loading predicted in the numerical models was in accordance with BS EN 1997, Design Approach 2. The primary lining was assumed to degrade over the design life, and was designed to provide temporary support prior to installation of the secondary lining.

The sprayed primary lining concrete mix was designed to a compressive strength of class C20/25 and early age strength gain per J1 curve. A performance-based requirement was followed for the steel fibers which were specified to provide a strength of class D3/S2 as defined by BS EN 14487-1: 2005. The secondary lining mix was specified to have a compressive strength of C30/37 class along with residual flexural strength of 2Mpa at mid-span deflection corresponding to a crack mouth opening displacement (CMOD) of 3.5mm. The forces imposed on the lining were determined using a 2D and 3D finite element model (Figure 4). The three-dimensional finite element analyses were performed

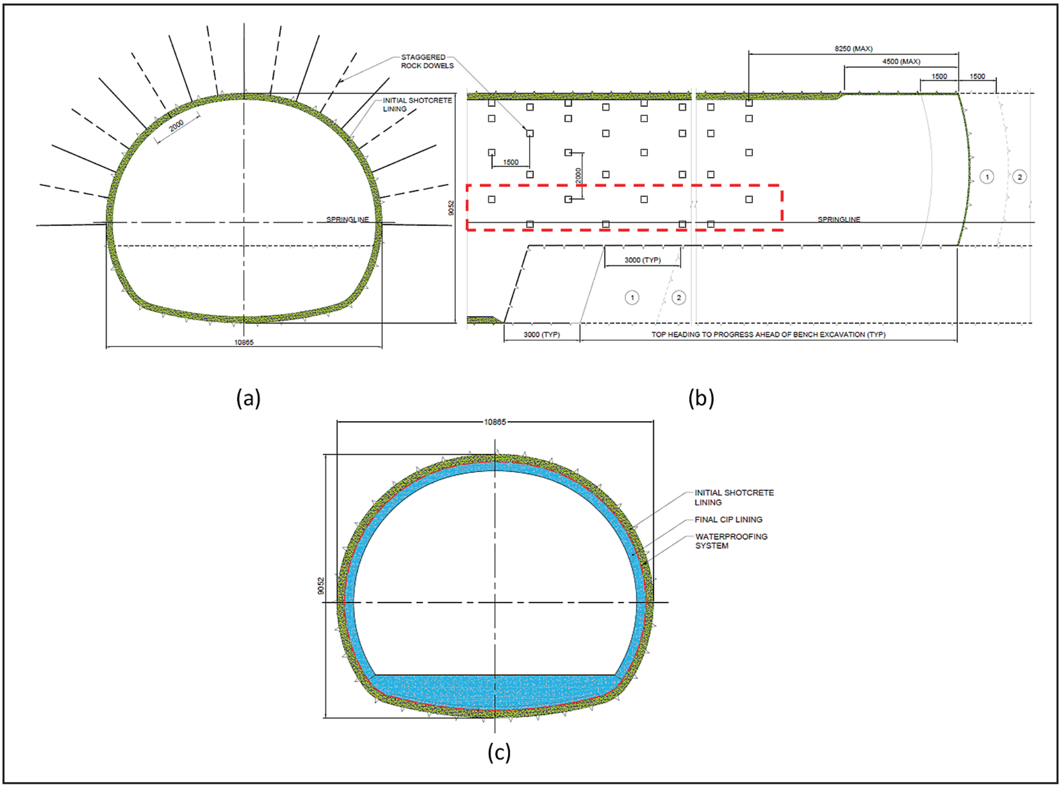

Figure 3. Typical tunnel geometry: (a) primary lining, (b) typical excavated sequence with rock dowels, and (c) secondary lining

with the software Rocscience Phase 2 for 2D analysis and Midas GTS NX for 3D analysis. Ground elements were modelled with 4-noded tetrahedral elements. Sprayed concrete lining elements were simulated with two-dimensional three-noded plate elements, and rock dowels were modelled with one-dimensional embedded truss elements.

Green (fresh) sprayed concrete is subjected to creep during installation due to deformations caused by ground loadings. Based on experience, a stiffness of 7.5GPa was used for green shotcrete to account for creep during installation of the first layers of sprayed concrete. A stiffness 15GPa was used to model the stiffness of the hardened sprayed concrete for the primary lining. The designed ground parameters are shown in Table 1.

A waterproofing system was required to provide long-term protection of the running tunnels against moisture and water inflow. In order to ensure durability of the tunnels, the design specified installation of fully tanked sheet PVC waterproofing membrane between the primary and secondary linings to act as a permanent water barrier all around the mined section.

The tunnel lining was designed for fire exposure in accordance with the Riyadh Metro project requirements. The concrete mix was designed for a 3 hours of minimum fire resistance (Figure 5). The fire design curve ISO-834-1 with temperature curved extended to a minimum of 180 minutes was considered in the design. The ISO 834-1 for 180 minutes (HC and RWS curves are commonly used for road tunnels due to the higher fire HRR). For improved fire resistance of the secondary lining a microfilament Polypropylene (PPE) fibers were added to the concrete mix. Fire tests were conducted to determine PPE fiber dosage and the test results indicated that a minimum of 1kg/m^3 of the PPE fibers is sufficient to achieve the required fire resistance. Further, the PPE fibers were only added to the secondary lining mix for the arch portion of the tunnel and no PPE fibers were added for the invert mix since there was a second stage concrete to cover the entire invert.

Ground Movement

During design development, calculations were performed to estimate the surface settlements induced by the tunnel excavation. Representative cross

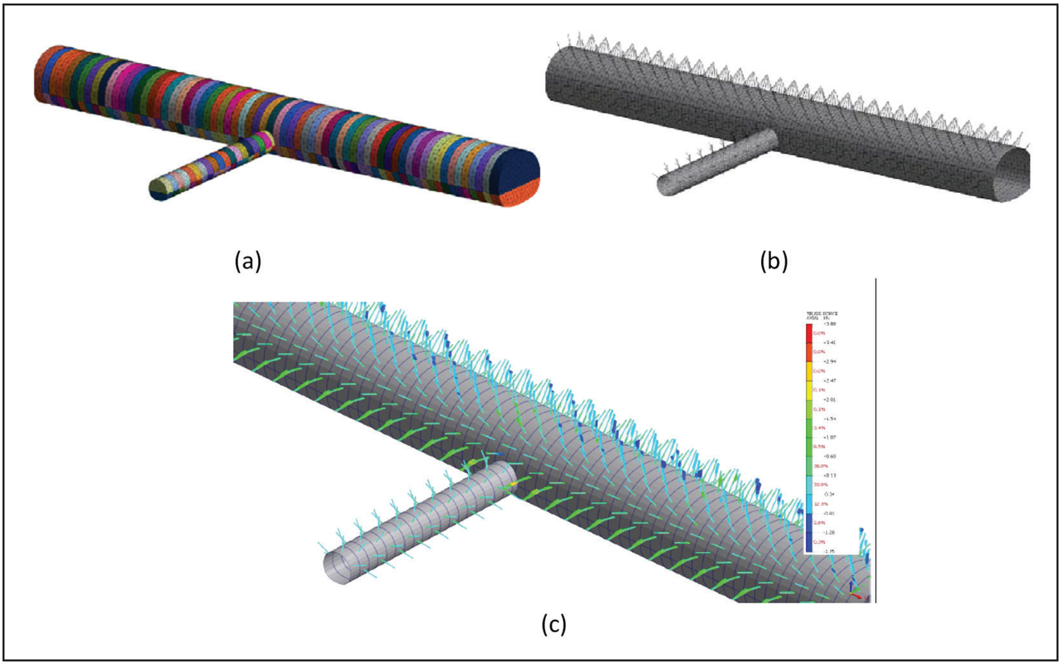

Figure 4. 3D Finite element model for the mined tunnel with connecting Adits: (a) model construction stages, (b) stage after support installation, and (c) axial forces developed in the rock dowels

Table 1. Ground parameters used in the design

Strata	Unit Weight (kN/m^3)	Young's Modulus of Elasticity (MPa)	Poisson's Ratio	Friction Angle	Cohesion (MPa)	Coefficient of Lateral Earth Pressure at Rest
Fill/Soil	18	35	0.30	35	10	0.40
Arab Formation—Upper Limestone Breccia (Highly Weathered)	24	1100	0.34	42	110	0.52
Arab Formation—Upper Limestone Breccia (Moderate Weathered to Fresh)	24	4800	0.34	49	190	0.52
Arab Formation (Arab C)—Disturbed Bedded Limestone	25	8000	0.34	56	470	0.52
Arab Formation—Lower Limestone Breccia	25	7000	0.30	49	220	0.52

sections in both 2D and 3D models were developed taking into account construction sequence, geology, and proximity to existing structures, tunnel geometry, and the construction of break-outs. Based on the results, settlement contour lines were derived along the entire Line 2 alignment. The maximum surface settlement above the tunnel centerline was predicted to be slightly over 2mm with virtually no adverse impacts on buildings and infrastructure during construction of the running tunnels. To measure ground movement around the tunnel opening during excavation, the design required installation of in-tunnel convergence monitoring cross-sections at predetermined tunnel sections and surface settlement monitoring points. Each in-tunnel monitoring section consisted of 5 convergence monitoring points (3D optical targets) around the tunnel periphery, while surface settlement monitoring was carried out using automated total stations and optical targets arranged along the alignment.

CONSTRUCTION

The Line 2 running tunnels were constructed following the SEM principles. Two Alpine roadheaders were used for the excavation (Figure 6). The tunnel excavation commenced from temporary shafts located within the footprint of the Olaya station (east and west sides), which was still to be built. The running tunnel excavated westbound transitioned to an open-cut excavation while the eastbound tunnel transitioned into the 2B4 Station.

The tunnel construction started in November 2014 and was completed in December 2015. Excavation and support was a 24-hour operation of two 12-hour shifts for 6 days per week. During construction, the excavation sequence was adjusted depending on the ground conditions. In better ground, the top heading round length was increased to a maximum of 3m. Similarly, the timing and round length of bench/invert excavation was modified accordingly since no immediate ring closure was required in good competent rock. The bench/invert excavation and support installation was usually implemented after excavation and full support of 10-15 rounds of top heading. Systematic probing was performed to allow investigation of deteriorated ground and/or groundwater ahead of the advancing tunnel face. Typical probing comprised 17-meter long probe holes with minimum overlapping of 5m. The tunnel geometry was locally enlarged when soft, highly fractured rock or soil was encountered in the tunnel crown to facilitate installation of the grouted steel pipes to provide additional support and minimize ground instability.

Throughout the project, two Senior SEM Engineers were available at all times on site to ensure that the design was adhered to throughout the construction process. This was achieved by leading the daily Shift Review Group meeting (SRG) and preparing the Required Excavation Support Sheet (RESS) in agreement with the tunnel construction team which was represented by the BACS Construction

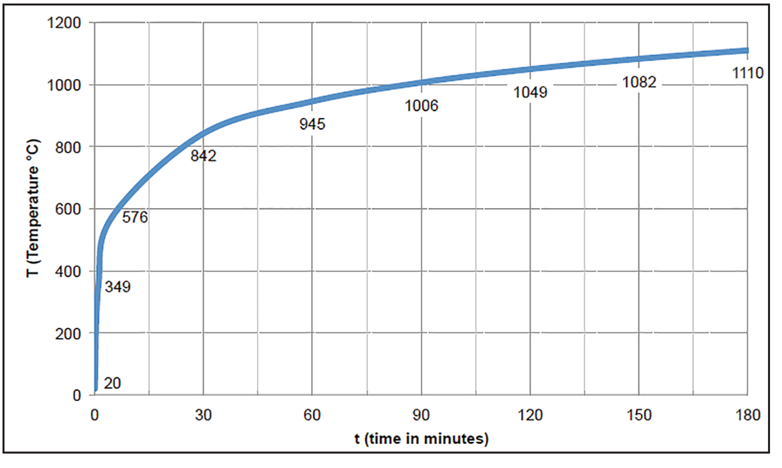

Figure 5. Standard ISO 834 temperature-time curve for a 3-hour fire duration

Figure 6. Tunnel excavation with roadheader (left) and secondary lining formwork (right)

Figure 7. Running tunnel (dashed red line) and its proximity to above ground buildings and viaduct

Manager, Head Geologist, Head Surveyor, Health and Safety Officer and Quality Manager. Adaptations to cater for geological and hydrological conditions as well as to construction and logistic requirements were made in agreement between all SRG members during the daily meetings. During the daily meetings, monitoring data, observations and any unexpected occurrence as well as specific issues such as quality of workmanship were reviewed and adaptations or corrective measures agreed upon. No tunnel excavation was permitted without a RESS being in effect and signed by all involved.

Challenging Urban Settings

The tunnel passed under building foundations and underneath the viaduct pier and therefore it was crucial to minimize ground movement during excavation to avoid impact to the overlying structures. Figures 1 and 7 also illustrates the critical structures along the tunnel alignment. The round length of the tunnel heading advance was reduced at these locations. Similarly, attention was given during construction for any possible future construction and tunnel support design was modified accordingly to make such future construction easier. For instance, GRP rock dowels were used in lieu of steel dowels at locations of future breakouts and at the end of the tunnels at 2B4 stations where new structures had to be constructed directly above the tunnel.

No discernible settlement or ground movement was observed and none of the above ground structures was impacted during construction.

A section of the west tunnel passed below an existing open cut excavation with the tunnel crown daylighting into the open cut. This open cut had been excavated prior to the tunnel construction, for a large commercial building foundation and was later backfilled.

Tunneling in such setting of minimal cover was very challenging and required a careful and well-thought out design. Mined tunnel construction under the open cut area was made possible by the installation of a temporary, roof-shaped slab ('turtle back') alorng the open cut section above the tunnel roof. The existing backfill material was used as earth-form. A plastic sheet on top of the fill material was used as separation layer. The 'turtle back' was nominally reinforced with wire mesh. The concrete roof slab was finally cast with the finished structure forming an arched-type lid above the tunnel section. The mined tunnel was excavated under this canopy in 3-meter top heading rounds. Shotcrete primary lining was placed against all exposed rock surfaces in the sidewalls. The two ends of the concrete canopy lid at the interface with the existing rock were reinforced with spiles to avoid overbreak at the interfaces, prevent movement and maintain tunnel profile. Figure 8 illustrates a section of the concrete canopy lid during construction. In the same area, Figures 8c and 8d also illustrates the close proximity of the tunnel crown with respect to the foundations of residential buildings and close proximity to the above ground bridge viaduct highlights the tunneling complexity in such urban settings.

Challenging Geology

One of the anticipated primary challenges during construction was the presence of fractures and dissolution features within the limestone. The aquifers typically developed in carbonate rocks have high permeability due to increased secondary porosity associated with karstic or other dissolution features such as vugs. These vugs were observed in borehole cores recovered during the geotechnical exploration program. These dissolution features developed along the sub-horizontal and sub-vertical joints and discontinuities, encountered in the Arab Formation. Therefore, the presence of undetected voids and cavities presented significant construction risk. Very thin interbeds of dark brown very weak laminated marls were also identified which represented weak seams within the rock. The fractured and less cemented layers within the limestone and breccia were also prone to develop rock slabs for fall out. In general, though, the actual ground conditions encountered during excavation were mostly favorable with competent rock except for few instances, mostly along the west alignment, where cavities and rock slabs were noted within the disturbed bedded limestone.

As part of the risk mitigation, systematic probing was performed from within the tunnel to identify geology ahead of the advancing tunnel face. The probing provided information regarding the presence of weak or fractured rock as well as the presence of any cavities or voids. In case that such adverse

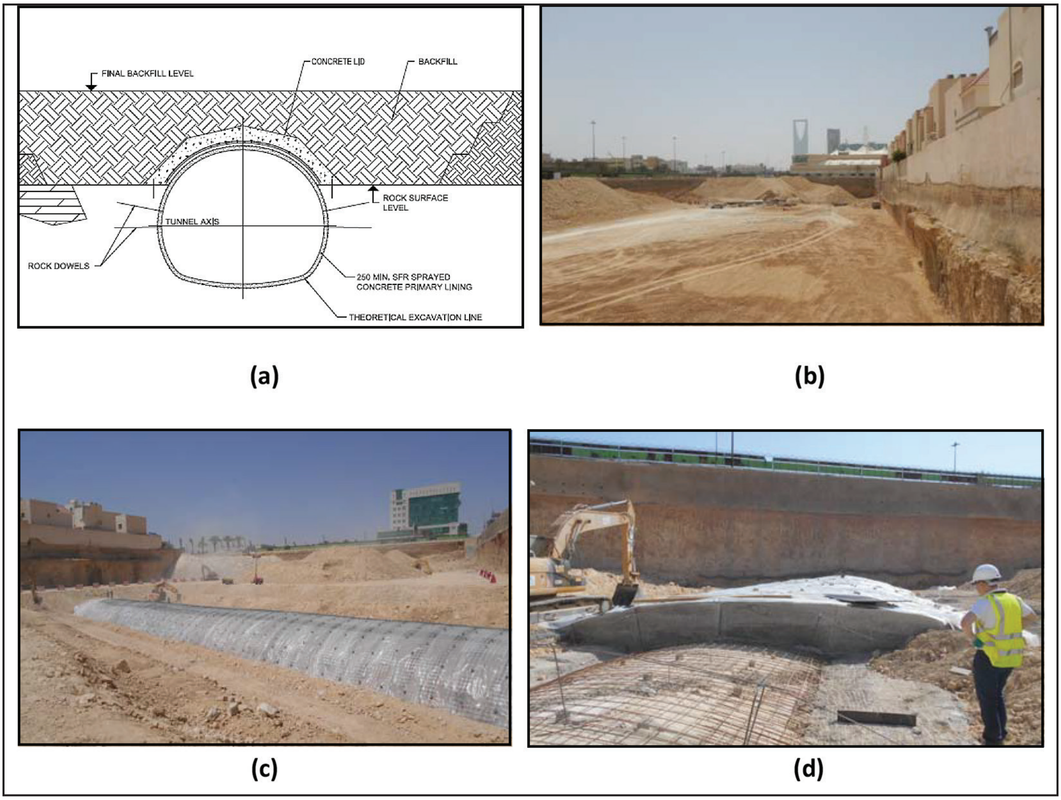

Figure 8. Construction of west tunnel section within the open cut area: (a) tunnel configuration with respect to backfill and canopy lid, (b) open cut area before tunnel excavation, (c) backfill with PVC above the tunnel section, and (d) tunnel concrete canopy

ground conditions were anticipated, mitigation measures such as reduction in round length and filling of the voids were adopted along with other local support measures (tool box items such as spiling) prescribed by the design. Rebar spiles were particularly required to be installed at the break-in and break-out locations and under shallow cover close to the tunnel portals to provide additional support, limit overbreak and minimize ground settlement. Overall, the carefully designed robust systematic support system allowed completion of the tunnel excavation and support on schedule avoiding delays despite the challenging ground conditions at due to dissolution features and fractured rock.

Groundwater Management

Groundwater control was an additional challenge during construction. The presence of dissolution features and pores increases the permeability in bedded limestone and breccia and provides preferred groundwater flow path which increases groundwater inflow. The groundwater level was varying over the length of the tunnel, but significantly above crown level along most part of the westbound tunnel and at invert level or below along the eastbound tunnel and therefore significant groundwater inflows were expected during excavation at locations of the westbound tunnel. Furthermore, probe holes were converted into dewatering holes when required to allow gravity drainage. The groundwater discharge rate through the probe holes was noted. It was planned install drain holes around the excavation periphery, if pre-determined values were exceeded, to inject grout and control the groundwater inflow. Such measures were not required to be implemented during the construction. Drain mats and flexible drain hoses encased in shotcrete were used to collect localized water inflow and allow shotcrete installation while channeling the water into small temporary sumps located near the invert. As the tunnel excavation progressed, a trench was excavated along the invert centerline and a 12-inch perforated pipe was installed along the trench to divert all collected groundwater along the tunnel to a temporary pump sump (Figure 9). The temporary sump was located

Figure 9. In-tunnel groundwater control: (a) trench with inlet and outlet piles, (b) temporary sump

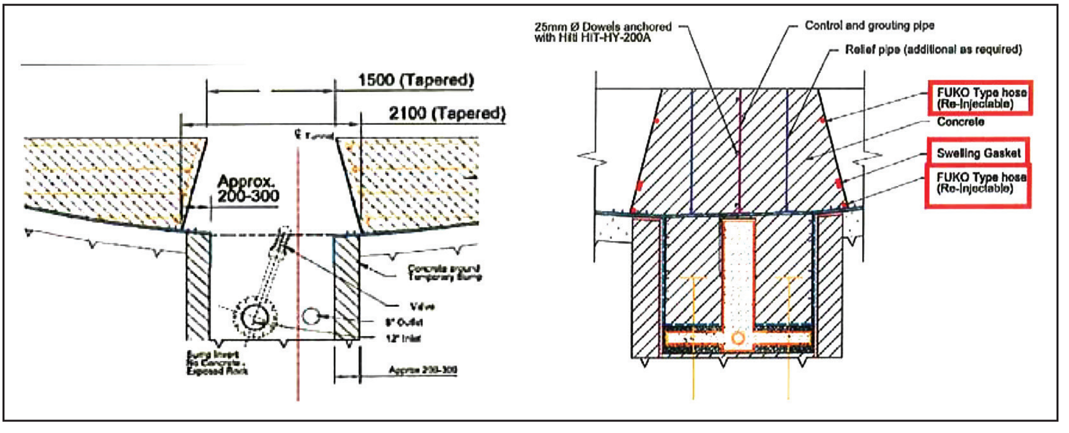

Figure 10. Front view of the temporary sump: before and after completion of final lining

in the middle of the west tunnel at its lowest point and contained three 6-inch dewatering pumps. In an urban environment, discharge of groundwater poses an additional challenge on the construction team. The invert-trench between the temporary sump and the open excavation for the Olaya Station also accommodated two solid 8' pipes, which were used for groundwater discharge back into a pump sump at Olaya Station. From there the water was pumped to the surface and discharged. The pumps were in continuous operation until completion of the final lining due to the high groundwater recharge rate.

The high inflows resulted in increased potential for flooding of the tunnels in case of failure of the dewatering pumps and pumping redundancy was in place to mitigate the risk. At one instance, groundwater discharge of as much as 1500cm^3/min had to be pumped from the west tunnel to provide safe working conditions. The east tunnel was relatively dry, not requiring major groundwater management measures.

Closure of the sump for installation of the invert final lining was a significant construction challenge as the pump had to be in operation at all times. A temporary closure of the pump could easily flood the tunnel due to the large water inflows and would cause invert heave in the tunnel section where the arch was not completed. Therefore, a carefully designed plan was executed to finally close the temporary sump pit and complete the secondary lining installation. Before closing of the sump pit, the secondary lining including the invert was completed along the tunnel alignment. For the closure of the sump pit, at first, the feeding pipes into the sump pit were cut flush to the sump walls to provide a flat substrate for installation of PVC membrane and then ground anchors along with blinding and gravel layer were installed on the base of the sump. A PVC membrane along with drainage mats were installed over the sump faces. A perforated T-shaped drainage pipe was then installed to keep the dewatering system running and prevent build-up of groundwater pressure during concrete curing.

A 4-inch drainage layer comprising of clean crushed stone was placed above the sump pit to allow the filtered water reach into the perforated T-pipe. A PVC membrane was installed on top of the drainage layer and connected to the PVC membrane of the sump walls. The sump pit was then filled with concrete under continued pumping to avoid groundwater pressure building during curing. Additional relief pipes were attached to the top of the finished concrete extending all the way to the top surface of the finished invert secondary lining (Figure 10). If water discharge was observed through the pressure relief tubes, a pumping was implemented to reduce the water pressure and ensure depressurized conditions for the invert pour above the sump.

CONCLUSION

The Line 2 running tunnels were successfully constructed without any delays or impact to the utilities and structures located above the tunnel alignment. A systematic and robust design along with ground probing and presence of experienced on-site SEM personnel were key for the successful execution of the project. The presence of dissolution features, including cavities, groundwater, and fractured rock presented significant construction challenges which were addressed with consideration of the geology and careful interpretation of ground probing ahead of the tunnel face. Identification of adverse ground conditions ahead of the tunnel face including the presence of groundwater allowed mitigation of such risks by either reducing the round length or implementing grouting program. Although the running tunnels were excavated in an urban setting in close proximity to building foundations and viaduct piers, no adverse impact to the existing structures was observed as a result of a carefully designed and executed tunnel excavation and support. Adherence to the design was ensured by experienced SEM Engineers through the daily SRG meeting where the excavation and support requirements were discussed and agreed for implementation.