Journal of Rock Mechanics and Tunnelling Technology (JRMTT)
30 (1) 2024 pp 19-38

Available online at www.isrmtt.com

Challenges and Remedies during Construction of the North-south Corridor of Pune Metro through Deccan Basaltic Rocks - A Case Study

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ABSTRACT

The continuously growing urbanization of the metro cities needs a fast and well-organized transportation system. Underground transportation system often creates an ideal result in terms of effectiveness and little disturbance to the environment. Urban areas are in very high demand of underground metro rail due to growing population and having limited land availability. Hence, Pune metro is expected to catch growing traffic congestion due to the increasing population and rapid urbanization and provide comfortable and convenient journey in the city. Pune metro rail will perform as the back bone to the public transport system.

Development of underground space like metro rails in challenging environmental conditions needs careful planning & design and use of advanced construction techniques. This paper covers the constraints encountered like ground vibrations induced by construction activities, TBM tunnelling below Mutha River, heavy ground and water seepage under low rock cover and their remedial measures during construction of the underground section between range hill depot and Swargate station (an ancient heritage structure). These adverse conditions may seriously affect the project's time schedule, cost and public image.

Keywords: Building condition survey; NATM; Mutha River; piling; TBM; drill and blast; instrumentation and monitoring

1. INTRODUCTION

The Pune Metro Rail Corporation (PMRC) is constructing line-1 and line-2 rail metro (Fig. 1). The line-1 (PCMC to Swargate) of Pune metro consists of 11.57km elevated section and 5.019km underground corridor. The underground section of line-1 is divided in to two packages: UGC-01 (Range Hill Depot to Budhwarpeth station) and UGC-02 (Budhwarpeth to Swargate station). The construction of these two underground packages UGC-01 and UGC-02 has been allotted to Gulermak-Tata projects limited JV. The underground alignment of this corridor is passing through the major Mutha River and highly build-up area in the confined city.

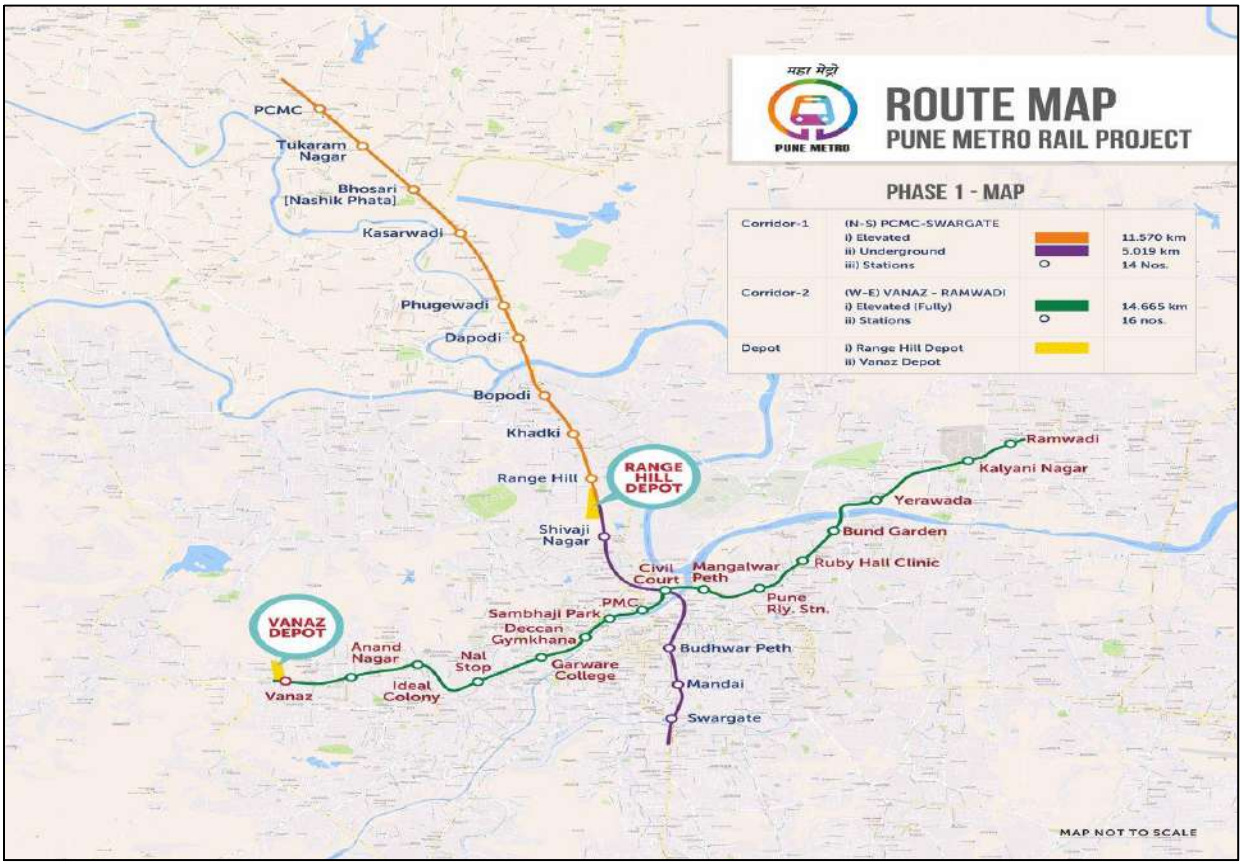

Figure 1 - Route map of line-1 and 2 of Pune metro

The part of the proposed alignment of underground section is identified for the ramp/cut and cover, TBM tunnel, NATM cross over, underground stations and a tunnel which is set in a relatively low dipping area at a mean elevation of 550m to 578m. The ground surface is assumed to be leveled and the top layer of soil corresponds to manmade ground conditions. The soil underlying this manmade ground is thin exhibiting residual character which quickly grades into variably weathered to fresh basement rocks identified as the deccan basalt.

However, it's important to note that conventional underground metro stations have traditionally been designed using the cut and cover methodology, which can be land-intensive during construction. Controlling the space constraint problems of urban areas is a major challenge for the construction industry (Huang and Luo, 2009; Jurado et al., 2012; Atallah et al., 2015; Ehrlich and Silva, 2015; Font-Capo et al., 2015).

2. SALIENT FEATURES OF UNDERGROUND SECTION OF PUNE METRO

Maharashtra Metro Rail Corporation Ltd., a Joint venture of Government of Maharashtra and Government of India, is executing the design and construction of 5 underground stations viz. Shivajinagar, civil court, Budhwarpeth, Mandai and Swargate and associated tunnels between Range Hill Depot and Swargate station of the Pune metro rail project.

In this project, 5 underground stations are planned to be constructed using cut and cover construction method. The total length of the tunnel is 8370.48m and it is being constructed using EPB (earth pressure balance) TBM. This TBMs is used due to very low rock cover & mixed geology at tunnel face. Two crossover tunnels having length of 280m and two stabling tunnels each of length 185m are being constructed using NATM (New Austrian Tunnelling method). The stabling tunnels are constructed for use of stabling and reversing of trains. At the end of the

Swargate station, the train reversal facilities have been proposed in form of stabling tunnel also the cross-overs at the rear end.

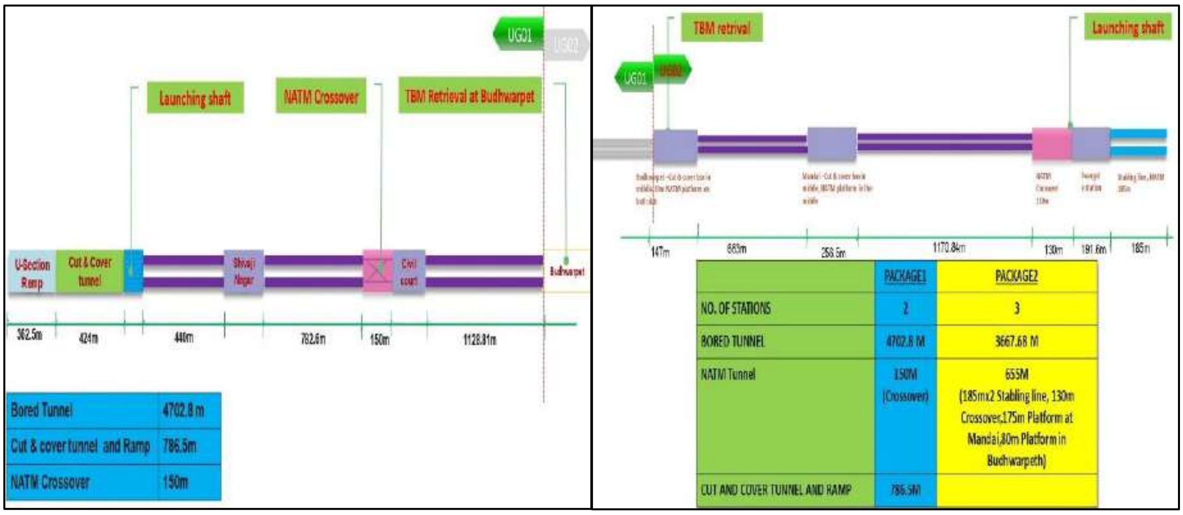

Figure 2 - Various components and construction methods adopted in the project

3. GEOLOGY OF THE PROJECT AREA

The area of Pune city is underlain by deccan volcanic basalts of upper cretaceous to Eocene age (Table 1). The project alignment is set in the stratigraphic domain of about 250m thick Pahoehoe flow, Karla formation. Karla formation is present in the three directions of Pune city towards Central to North-west, South-east and South-western part covering adjoining areas and overlies the Indrayani formation. The individual flows are separated from each other by red bole, which varies in thickness from a few centimeters to more than a metre. These are essentially ferruginous clayey horizons and are useful as marker horizons for flow separation.

Table 1 - showing stratigraphic succession of the area

Group	Subgroup	Formation	Characteristic feature
Western Deccan Volcanic Province	Wai	Purandhargad	Simple and AA type flows
		Diveghat	Aphyric AA type flows
	Lonavala	Karla	Compound pahoehoe flows
		Indrayani	Simple flows of columnar jointed and aphyric types
		Ratangad	Compound flows of phyric type

Presence of red boles at critical places such as bottom of station areas and the top of tunnel crown could have a significant impact on the design and construction of the underground structures. Underground water, if present, may saturate the rock cover and may be a serious threat for construction activities, if it exists in form of an aquifer. A red bole horizon as marked in Figure 3 was observed 12m below ground level (BGL) on the top of crown of cross over tunnel in the Swargate station.

To know the subsurface condition of soil and rock, a detailed geotechnical investigation work has been carried out. Field investigations in the area revealed the following occurrences of soil and bed rock based on approx. 100 no. of bore holes (50 nos. in each package) at various locations along the project alignment.

Figure 3 - showing the soil/rock samples of bore hole no. ABH-22 near cross over at Swargate

3.1 Complex Soily Strata

Based on the drilling information from the boreholes, soil present can be broadly grouped in to two parts in the project area; (i) overburden material upto 2.0m from the ground level. It is highly heterogeneous in nature, as it comprises of variety of materials, ranging from stones to significant amount of gravel. (ii) below the layer of overburden material, there is residual soil stratum primarily composed of sand, clay and silt with presence of gravel at depth of 3.0-7.0m. Most of these soils appear to be in a state of transition to the in-situ decomposed bedrock (Fig. 4).

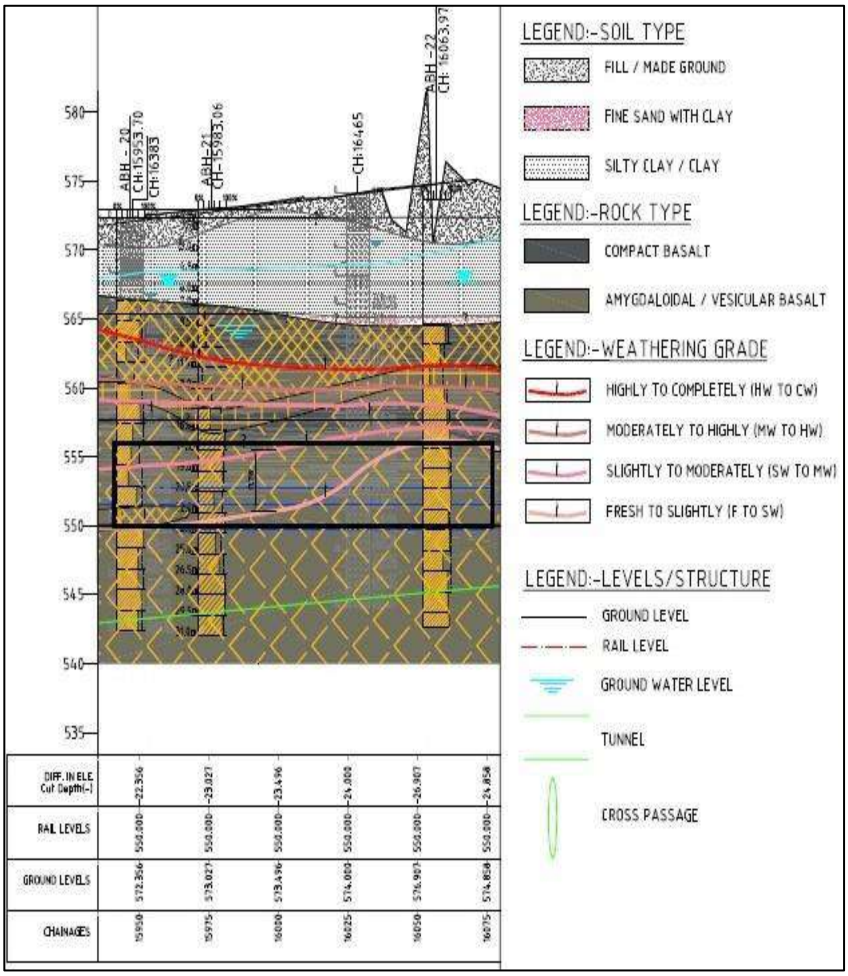

Figure 4 - showing the geological L-section of crossover tunnel, Swargate

3.2 Bed Rock

The bed rock identified with the Deccan basalt flows which is known to exhibit variability in their physical and chemical attributes and the degree of fracturing (Fig. 4).

The bed rock starts below residual soil between 7.0m and 15.0m BGL. The chemical weathering alters the mineralogical properties of the parent bedrock. Its effect reduces with depth. The categories of basalts identified from the boreholes, are (i) vesicular basalt (VB), (ii) amygdaloidal basalts (AB), and (iii) massive basalts (MB).

The variations in basic composition of intact basalts can have implications on their strength and mechanical attributes. This basic feature is further accentuated by the occurrence and density of jointing/ fracturing in the rock mass (Fig. 4).

No major geological structure like folds, faults, large shear zones, and dykes are observed. However, review of hydrogeological and lineament map suggests the occurrence of a lineament just North-West of confluence of Mula-Mutha rivers not far from the alignment of underground package-1 and 2. The basaltic melt flows are largely understood to be horizontal, and hence the margins separating them can be weak, often identified distinctly where occurrences of red bole layers are observed. The individual layers are expected to be jointed, by tensile forces induced during cooling, and some possibly by shear forces during the elapsed geological time. Such joints could be steeply inclined and required to be monitored and recorded in mappings for analysis of stability of excavations during construction.

4. HYDROGEOLOGICAL CONDITIONS

The records of ground water levels published for city suggest its depth in range of 2.5 to 7.5m from ground level measured during the monsoonal months of June, July and August. This could drop down to about another 2.5m during pre-monsoon seasons. The permeability of rock mass is a function of fracture density, whereas in case of soil, it depends on its grain size distribution. For the variably fractured rock mass in the project area, the coefficient of permeability (k) is found to vary between 10^{-5} m/s in more persistent and open jointed conditions and 10^{-7} m/s in massive rock. It must be however noted that the permeability / water ingress from local - persistent open joints or fracture/ weak zones can be substantially high, e.g. red bole layers especially below the river and deeply weathered and fractured basalt.

5. CONSTRUCTION METHODOLOGY

For safe excavation main tunnels, stations, cross over and stabling tunnels are excavated using TBM, cut & cover technique and NATM respectively.

5.1 Using Tunnel Boring Machine (TBM)

Prior to the advent of pressurized face TBMs, urban tunnelling was confined to ground conditions with sufficient stand-up time to allow the tunnel to be advanced and support erected. Tunnelling below the groundwater table was limited to strata that could be stabilized by the application of compressed air to control seepage through the face. The twin bored tunnels were driven using EPB TBM in different modes i.e., in semi-closed, closed modes and in open mode (Babendererde et al., 2005) as per the different ground conditions (Fig. 5). Most of the part of tunnel was excavated in open mode tunnelling but at the locations where heavy seepage and weak rock encountered the tunnelling was done in semi close or close mode to restrict the ground settlement. It was observed that 10.5m BGL the rock is slightly weathered (SW) to fresh condition exhibiting grade-1 and II rock with UCS 40MPa, tensile strength 0.46MPa, friction angle 64^0 showing very good rock condition (Table-2).

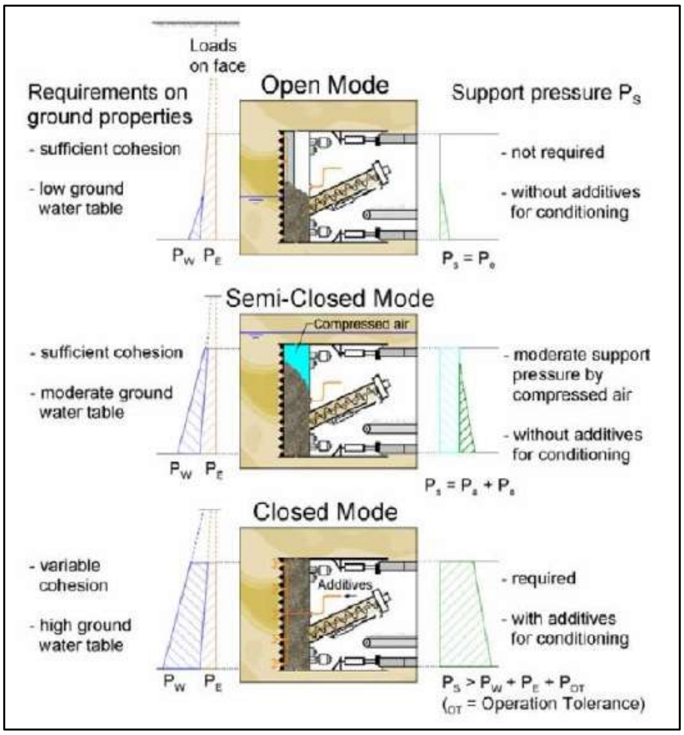

Figure 5 - showing the different modes of EPB TBMs

Table 2 - showing the design parameter of rock for the TBM tunnels

Depth below GL	Strata type	GSI	Intact Modulus Rock (E)	UCS of Intact Rock	Tensile strength of rock mass	Poisson's ratio (υ)	Cohesion	Friction angle	Permeability (k)
(m)			[MPa]	[MPa]	[MPa]		[MPa]	[°]	[m/s]
6.0 - 7.5	HW	20	5000	20	0.003	0.3	0.08	48°	$1x10^{-6}$
7.5 - 10.5	MW	45	9000	30	0.03	0.3	0.21	59°	$5x10^{-7}$
>10.5	SW to Fresh	80	14000	40	0.46	0.3	1.4	64°	$1x10^{-7}$

5.2 Cut and Cover Method

The use of cut-and-cover construction will further impact traffic, require utility relocation and/or support in place, affect businesses of nearby shops and expose the public to noise, dust and vibration and impact the people quality of life during construction. The station box of metro projects has always been constructed by cut and cover techniques. This method is of two types, i.e., (i) top-down and (ii) bottom-up. In top-down method, the roof slab of the station is built first and which involves staged excavation. The station walls and floor are then constructed as the excavation under the roof slab continues. In bottom-up method, base slab of the station is built first.

Here for construction of underground stations the bottom-up technique was used in Pune metro due to presence of rocky strata (Fig. 6). The length-width-height of excavated zone of civil court station was 150x26x31m. In this method, first the top soil is protected by secant pile which is a ground improvement technique which is very useful for improvement of soil stand up time during excavation. These piles are temporary structures only to facilitate the safe excavation for station box and terminated in to hard rock having socketing of 2-D of pile diameter. Then after the soil inside the station box is removed systematically and exposed piles are protected by rock bolts and Waller beam. When hard rock starts, a berm of 1 m width is provided which will provide the toe stability of secant pile. Then further the remaining excavation carried out by drilling and blasting technique and rock mass characterized based on GSI system.

The geological strength index (GSI) is a system of rock mass characterization that was developed, by Hoek (1994) and Hoek et al. (1995), to link the failure criterion to engineering geology observations in the field (Hoek-Brown Failure Criterion and GSI – 2018). The rock face after the excavation is protected by rock bolts, wire mesh and shotcrete. Total excavated volume of muck was approx. 1,20,000 m3 which was achieved in 15 months. The temporary rock support as per design drawing below the berm level was systematic rock bolt 25mm dia., 6m long, 2x2m c/c spaced, wire mesh (150x150x6mm) and shotcrete 100mm thickness was to be provided at the exposed rock face/walls. But during the excavation based on the Rock Mass Rating value (80-85) (After Bieniawski 1989), GSI value (75-80) and instrumentation and monitoring results, the rock support have been optimized. The total wall height below berm level was approx. 25m in which only 10m below berm level the temporary rock support as per design drawing was provided. Remaining 15m wall height no temporary support has been provided due to hard and competent basalt rock. Due to these modifications in installation of temporary rock support have significantly reduced the time and cost of completion of civil court metro station by 3 months (Fig. 6).

Figure 6 - Progress of civil court station by cut and cover method of tunnelling

5.3 New Austrian Tunnelling Method (NATM)

The new Austrian tunnelling method (NATM) is one of the most popular observational method of tunnelling. this method is based on the idea to stabilize the tunnel itself by using the surrounding rock mass strength.

However, these challenges are addressed very carefully by designed tunnelling method, the use of the latest technologies in tunnelling, the use of careful excavation and support sequencing, implementation of ground improvement and a robust instrumentation and monitoring program. With a risk mitigated approach during the design phase, and the use of the latest TBM technologies, tunnelling has proven successful in complex urban settings (Fig.7). Cross over tunnels at civil court and Swargate stations, tunnels at Budhwarpeth and Mandai stations, 06 nos. of cross passages of Pune metro have been constructed by using NATM tunnelling technique.

Figure 7 – Excavation using NATM in cross over tunnel at Swargate station

6. MAJOR ENGINEERING CHALLENGES DURING CONSTRUCTION STAGE

Pune metro underground tunnelling shows a number of unique challenges. The proposed alignment of the underground section is passing below major roadways, deep underpasses, existing foundations and buried structures, large intricate networks of utilities, very old building structures, river and canals. Additionally, space constraints in urban settings magnify the challenge of implementing tunnelling in such a manner as to avoid inducing displacements damaging to adjacent facilities, structures and utilities. Some of the challenges faced during the construction of Pune metro are discussed as follows.

6.1 Excavation in Vicinity of Heritage Structures

Many structures in the area are quite old and constructed with spread footing and load bearing walls with tin shed and many old wooden structures. The major modifications and additions in these structures have taken place. These modifications have been carried out without following any standard code of engineering practice. However, a detailed survey has been carried out for the structures along the alignment in the influence zone of construction for the purpose of assessment of the health of these structures (Fig. 8). The zone of influence measured 50m from center line of twin tunnel and twice the depth of cut & cover excavation from the periphery of permanent wall structure. This assessment includes the settlement analysis and preparation of contours along the alignment. Based on the types and severity of defects and damages, the building condition has been categorized and given in Table 3 and Figure 9 (Mair et al., 1996). More than 104 number of buildings are identified as severe category along the proposed alignment.

Figure 8 - Severe category of buildings in the influence zone of proposed alignment

Table 3 - showing the damage category of buildings

Section	No. of buildings in influence zone	Damage category of buildings/structures				
		Negligible (B0)	Very Slight (B1)	Slight (B2)	Moderate (B3)	Severe (B4)
Range hill depot to Swargate	1382	220	337	459	262	104

Appropriate remedial measures were taken during the construction stage in form of i) TBM tunnel excavation done in semi close to close mode. ii) NATM tunnels done by controlled blasting practices. iii) Proper instrumentation work done in buildings, tunnels and on ground to know the settlement and deformation. iv) Severe buildings/structures after the structural survey will be protected by using the propping/shoring (Fig. 10). The structural survey of buildings in metro projects will always be carried out before starting the excavation work and then after installation of instruments for monitoring the building settlement will be installed.

Suitable instrumentation along with vibration monitoring done in the Pataleshwar cave temple during TBM tunnelling which is a monument and located in the influence zone of proposed tunnel alignment between Shivaji Nagar to civil court station. It is a rock-cut cave temple, carved out in the 8^{th} century in the Rashtrakuta period by Kannadiga kings. All the vibrations during construction activity are recorded (<1mm/s) below the permissible limit (5mm/s).

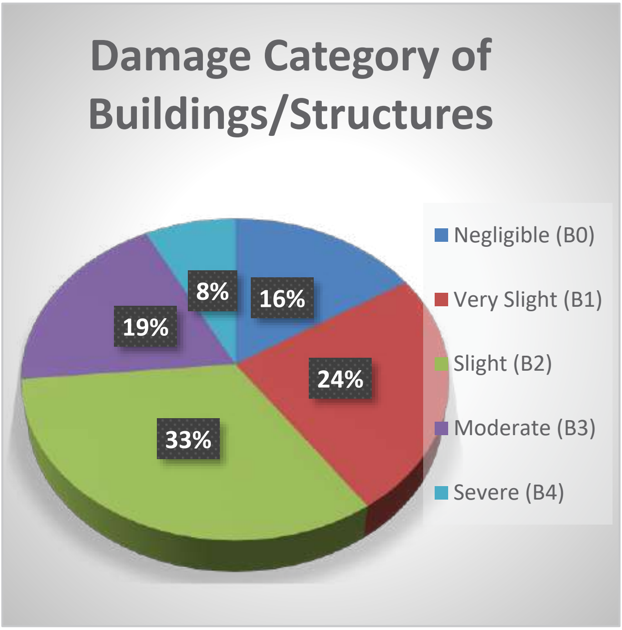

Figure 9 - Damage category of buildings

Figure 10 - Hutment wall protected by Shoring at civil court station

6.2 Excavation Below Major Roads

The layout of this section of Pune metro is passing through many major roads and one deep underpass near the Swargate NATM crossover and another subway near NATM crossover at civil court station. The cross over NATM tunnel (22m width and 9.6m height) at swargate is passing through a deep underpass from which the huge traffic is running and rock cover between tunnel crown and bottom of subway is 9.96m (Fig. 11). The ground level is 574.25m and depth of underpass up to bottom level with reference to ground level is 6.736m.

The rock type is amygdaloidal/compact basalt. The GSI value ranging between 45-65. Heavy ground water seepage (>100 l/min) was observed especially from the crown and spring level. One shear seam also observed at the tunnel crown. Due to this shear zone (5-10mm thick) heavy ground water seepage as well as many collapses in form of over break occurred in the crown. The primary support design is performed with the FEM analysis. The main variables considered in the analysis are the overburden, ground types parameters.

For stability of tunnel crown and ground structures, immediate remedial measures taken as described below:

i. The full face of NATM crossover is divided in 4 parts i.e. top heading 1, 2, 3 and invert (Fig. 12) for Sequential excavation. But below the underpass and critical ground structure, all the top headings again divided in 3 parts and invert in 2 parts at the site for 1m face advancement.

ii. The allowable ground vibration limit is 5mm/s which is successfully achieved by applying the sequential excavation method. To control the peak particle velocity (PPV) value, line drilling and light charging of holes are also followed.

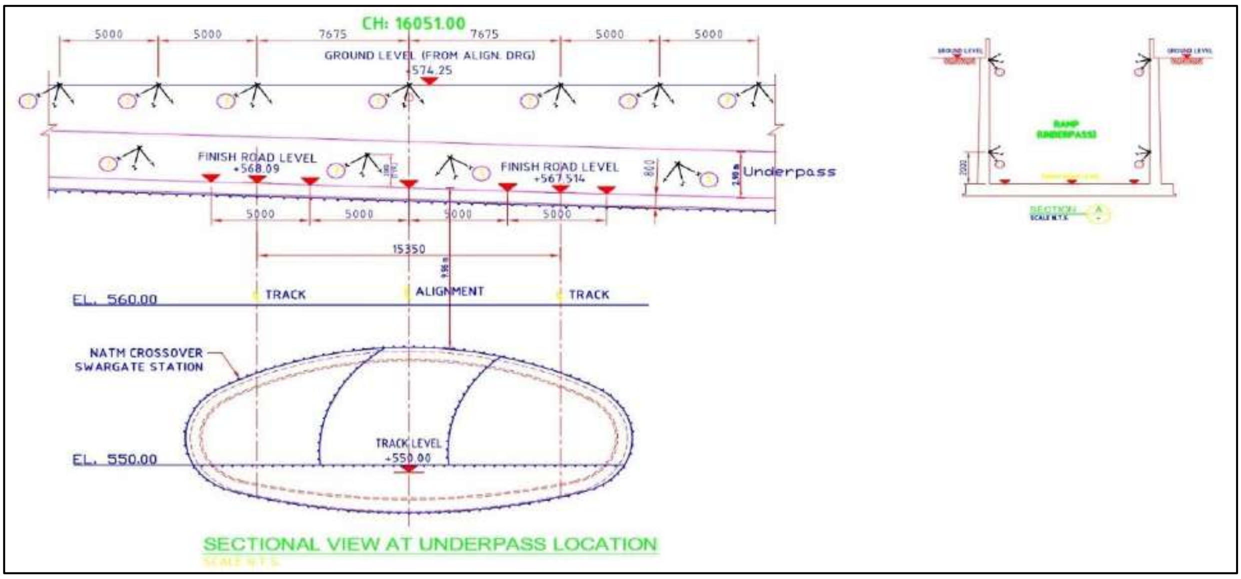

Figure 11 - Cross-section of NATM tunnel Vs underpass at Swargate

iii. Maintained a minimum lag distance of 14m between top headings 1, 2 and 3.

iv. Round length of 1m was set for the first top heading-1 and sealing concrete of 50 mm was applied on the face

v. First layer of wire mesh (150x150x6mm) was installed.

vi. Installation of 4m log SN rock bolts with 25mm diameter were installed at 2 x 1m c/c following staggered pattern.

vii. Application of 100mm thick shotcrete.

viii. Installation of wire mesh 150x150x6mm second layer and spraying of 75 mm thick shotcrete.

ix. Weep holes of 50mm dia., 1m long have been installed where the ground water seepage observed.

x. This cycle is repeated after each 1m face advancement.

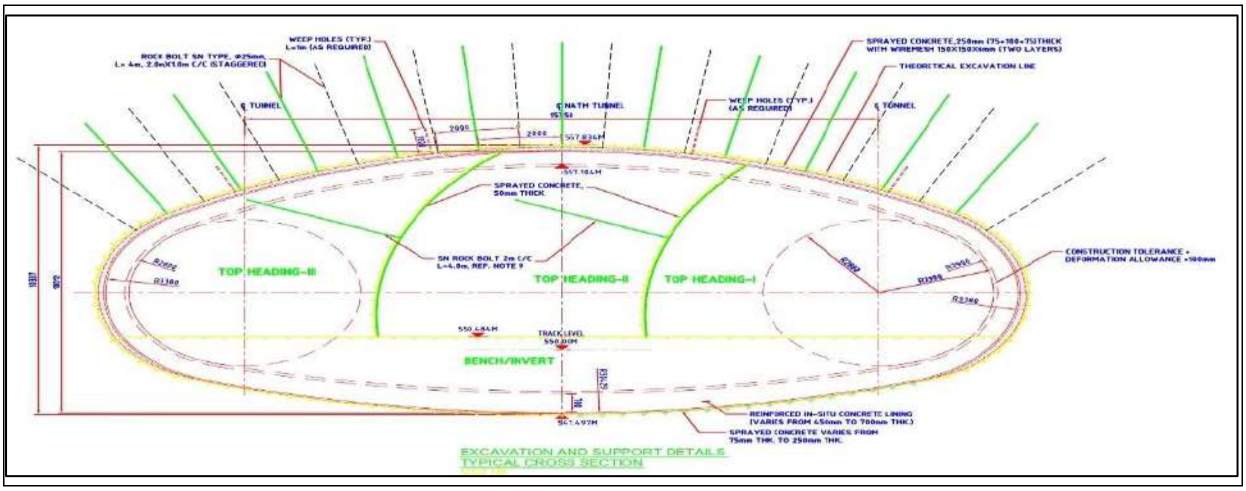

Figure 12 - Temporary support system for NATM cross over tunnel Swargate

The geotechnical instrumentation plays a vital role in evaluating the structural performance of an underground structure. All the instruments show negligible movement during the excavation of cross over except stand pipe piezometers which shows a dropdown of 8m. Ground settlement markers (GSM) installed on pavement (Fig. 11) show that the maximum surface settlement is within alert level value.

The 3D bi-reflex target installed on the underpass at Level-I and II (Fig. 11) show that maximum displacements are within the alert level value, except some BRT has breached alert & action level. These displacements seem to be due to the road construction activities & disturbances. However, these displacements are stable and are being correlated with the monitored surface settlements data which shows no significant changes in surface settlement values.

6.3 Excavation Below Existing Deep Foundations

Many existing deep foundations are found at the proposed tunnel alignment i.e., Pump house at Kasaba Peth area (Figure 13), and flyover at Swargate (Figure 14).

6.3.1 Pump house structure at Kasaba Peth

The TBM tunnel alignment is passing below a pump house approx. Ch. 13150m downline near Kasaba Peth area. As shown in Figure 13, the pump house building is directly located above the downline tunnel and clear distance of 1.37m is available from the TBM shield outer diameter to bottom level of rock anchors installed below raft foundation of the building.

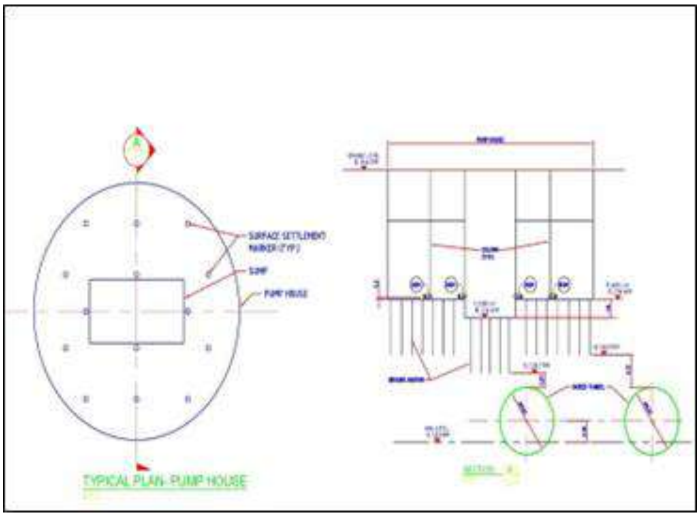

Figure 13- Plan and Section of Pump House and TBM tunnel

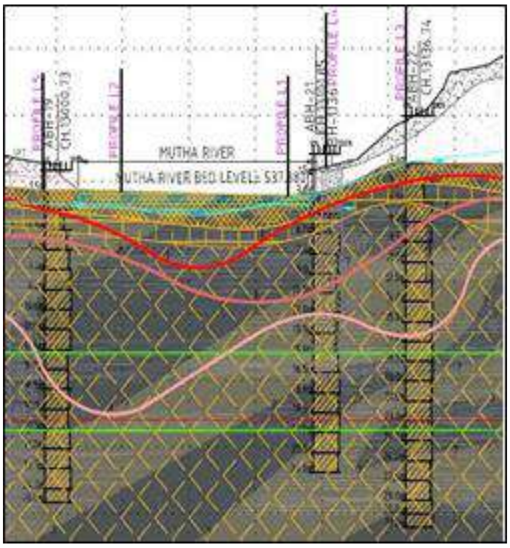

Figure 14 - Geological L-Section

Since the building (pump house) falling in the influence zone of tunnel alignment, pre-construction building condition survey (BCS) has been carried out. The survey includes the identification of defects, damages and their representation. Based on the types and severity of defects and damages, the existing building condition has been categorized under “2-Slight” category (Mair et al., 1996). Building load calculated from the structural details of the building (outer walls, columns, beams, slabs and sump), the total building surcharge load at the bottom level of sump was estimated to be approximately 66kPa.

For ground conditions, **t**wo number of boreholes (ABH-21 and ABH-22) were drilled near pump house building location. UCS (uniaxial compressive strength) of intact rock was determined to be in the range of 35-100MPa. The field permeability test results suggested a reduced water intake with depth. The tunnelling depth is approximately 20m below ground surface level at the building location, and the permeability of rock mass was in between 10 to 30 Lugeon.

To minimize the risk of damage of the ground surface structures, following measures were taken:

- To mitigate the risk of higher settlements (face instability) due to high water ingress in presence of prominent open joints, the BM was operated in closed mode below pump house building. For the closed mode TBM operation, the recommended EPB face pressure to be applied at the tunnel axis level was 150kPa (the recommended face pressure is equivalent to the prevailing hydrostatic pressure value). The fully closed mode TBM operation controls the water inflow into tunnel and stabilizes the face in unstable ground conditions if any.
- Excavation was carried out with a constant rate and interruptions have to be minimized for the necessary maintenance operations. The tunnel face was supported with the help of bentonite slurry which acts as stabilizing pressure to the tunnel face.
- Tail void grouting was carried out immediately after segments (ring) installation. The instantaneous filling of the "annulus", which is created behind the segmental lining at the end of the shield tail, is an operation of paramount importance. Its main goal was to minimize surface settlements due to unavoidable over-excavation generated by the passage of the TBM.
- During TBM drive below the building zone, 24-hour continuous monitoring of the proposed instrumentation scheme of the building was carried out as a risk mitigation measure.

6.3.2 Flyover above the cross over tunnel, Swargate

The Swargate flyover pier foundation and crossover tunnel have also very low cover as the continuous traffic is moving on this road. The distance between toe of pile (P30) and (P29) from

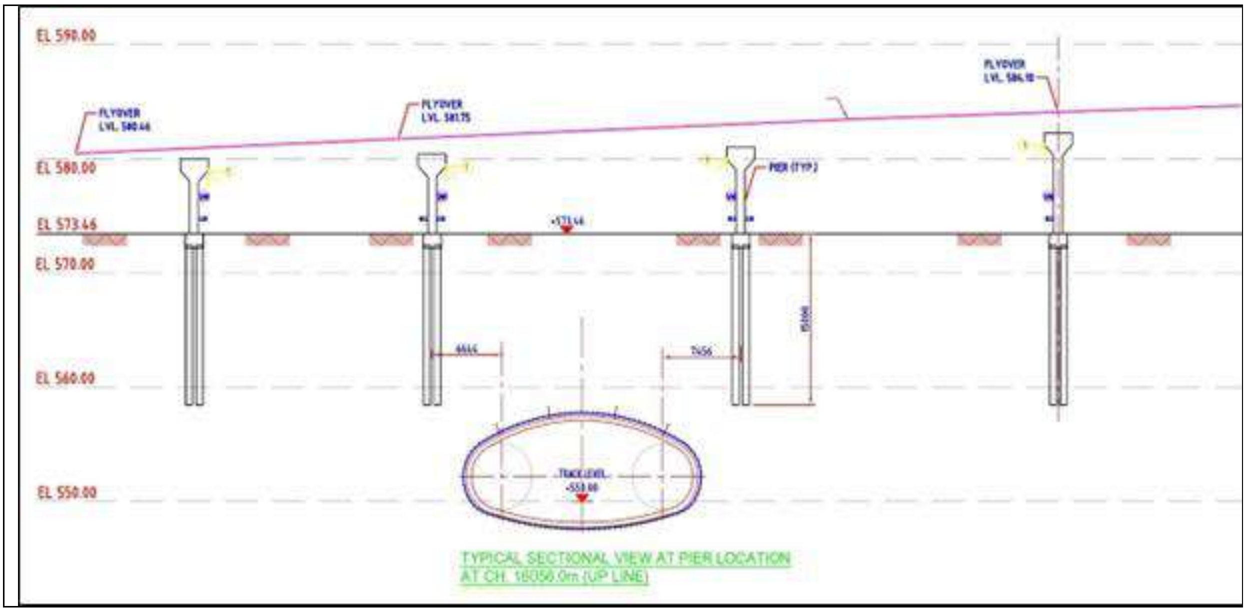

Figure 15- Cross section of crossover tunnel and piers of flyover

tunnel periphery was 4.6m and 6.5m respectively (Fig. 15). The excavation of NATM tunnel at Swargate below the critical structure (Flyover and Underpass) was done by adopting all the controlled blasting techniques. The NATM tunnel full profile was excavated in 10 parts to control the ground vibration in the permissible limit (<5mm/s). Real time monitoring of the pillars of flyover was also done to know the deformation. If any settlement breaches the design limit due to construction activity immediate action is taken in form of review the Drill and Blast pattern, installation of temporary rock support, seepage controlled by grouting, strengthening work of structures.

6.4 TBM Tunnelling Below Mutha River

The TBM tunnel is passing through the Mutha riverbed at upline chainage Ch.13000m to Ch.13100m approximately. The river width at this location is approx. 100m. To identify the subsurface condition below the river, 04 nos. of bore holes have been drilled (Fig. 16). Additional seismic refraction test (SRT) also performed along and across the river at proposed tunnel alignment. The results from SRT report correlating with nearest boreholes and from the investigation boreholes in the Mutha river, have been confirmed the twin TBM tunnel will be driven in moderately weathered to fresh basalt rock mass.

Figure 16 - showing the geotechnical investigation work at tunnel alignment.

During the TBM drive under Mutha river moderately weathered amygdaloidal basalt encountered having infilling of zeolite minerals in the veins. Due to some open joints very high-water ingress from tunnel face through geologically weak zone conduction water that makes the face susceptible to collapse which leads to difficult working conditions. To control the water ingress, TBM was set to proceed in closed mode. For the closed mode TBM operation, the EPB face pressure applied at the tunnel axis level was 150kPa (the recommended face pressure is equivalent to the prevailing hydrostatic pressure value).

Excavation should proceed with a constant rate and interruptions have to be minimized for the necessary maintenance operations. Tail void grouting shall be carried out immediately after segments (ring) installation. The applied injection pressures and theoretical volume of tail void grout must be maintained and controlled during the injection process as TBM advances.

To monitor the seasonal variation of ground water table at the Mutha river section, 04 nos. of standpipe piezometers installed on either bank of the river. The monitoring data of stand pipe piezometer shows a groundwater drawdown of 1m (more than the alarm value as per contract). However, at this location to monitor ground surface settlement and 3-D targets installed on buildings to check any effect on ground settlement and buildings due to ground water drawdown. During TBM drive at Mutha river section, increased monitoring frequency of the proposed instrumentation inside the tunnel and at surface shall be carried out. No significant change in surface settlement and in buildings is observed due to groundwater drawdown.

7. INSTRUMENTATION AND MONITORING

Instrumentation and monitoring are critical for detecting ground movement and implementing corrective measures. All underground excavation causes stress redistribution in the ground, which leads to ground deformation. Mitigating the associated risk is an essential factor during the design process. Instrumentation monitoring of existing structures was done as per the design drawings which was based on the settlement analysis and construction impact assessment of the structures in the influence zone by the BRTs, Tilt pate, crack meter, building settlement markers, ground settlement markers, piezometers, MPBX, Load cells, strain gauge, pressure cell and real time monitoring. During construction, collected ground deformation data is reviewed against predicted values (Table-4). This may result in previously performed analyses being modified and TBM operations parameters and drilling blasting excavation methodology being adjusted.

Table 4- showing the monitoring results of different instruments

Sl. no.	Geotechnical instruments installed for monitoring	Maximum recorded settlement/ horizontal displacement at site	Designed value		
			Alert	Action	Alarm
1	Ground settlement marker (GSM)	2mm	5 mm	8mm	10mm
2	Building settlement marker (BSM)	2mm	5 mm	8mm	10mm
3	Inclinometer	5mm	10mm	15mm	20mm
4	Load cell	2.1 tonne	8.25 tonne	13.2tonne	16.5tonne
5	Crack meter	0mm	3mm	4mm	5mm
6	Stand pipe peizometer	6m	1m	1.5m	2m
7	Tilt meter	0.005°	0.05°	0.05°	0.1°
8	Peak particle velocity (PPV)	7 mm/s	12 mm/s		

The ground settlement markers (GSM) installed in the vicinity of station excavation show a maximum surface settlement of 2mm which is within the designed value. The building settlement points (BSM) installed on the buildings show a maximum building settlement of 2mm. The monitoring data of inclinometers shows a maximum horizontal displacement of 5mm which is well within the designed value as mentioned in the monitoring report (Fig. 19). The load cells installed on the rock bolts in the secant pile supporting system shows 2.1 tonne change in load which is insignificant (Fig. 21).

The buildings near civil court station are very close to its construction boundary approx. 3-5m away. The crack meters installed on the buildings show no change in crack width (0mm) which is within the design level and also the tilt-plates installed on the buildings show very minor tilt within the allowable design limit (Fig. 20).

The monitoring data of the standpipe piezometer SP-02 (Fig. 18) shows a groundwater drawdown more than 2m (breached the designed value as per contract) from the lowest recorded groundwater table during station excavation. The groundwater drawdown is correlated with the monitored surface settlements; the monitoring data shows no significant changes in surface settlement is observed due to groundwater drawdown. However, at this location it is recommended to monitor the ground surface settlements on daily basis to check any effect on ground settlement due to groundwater drawdown.

The peak particle velocity data always taken in the nearby structures of civil court station box (Fig. 17). The maximum allowable limit for Peak Particle Velocity during drilling and blasting activity was 25mm/s and 15mm/sec for Framed structures and domestic houses respectively (Table-5). But designer decided and taken a safer value of 12mm/s for the structures near civil court station. But during the excavation of station box and crossover NATM tunnel at civil court station all the vibrations are restricted below 5mm/s. only few are recorded up to 6-7mm/s. All necessary precautions to reduce the PPV value have been taken i.e. line drilling, controlled blasting, use of delay detonator, Powder factor, Maximum charge per delay. Below Table-5 showing the allowable PPV values for different structures.

Table 5- Permissible PPV at the foundation level of structures in mining areas in mm/s (DGMS (Tech.) S&T, 1997)

Type of structure	Dominant excitation Frequency, Hz		
	<8 Hz	8-25 Hz	>25 Hz
(A) Buildings/structures not belong to the owner			
(i) Domestic houses/structures(kuchha brick & cement)	5	10	15
(ii) Industrial Buildings (RCC & Framed structures)	10	20	25
(iii)Objects of historical importance & sensitive structures	2	5	10
(B) Buildings belonging to owner with limited span of life			
(i) Domestic houses/structures(kuchha, brick & cement)	10	15	25
(ii) Industrial buildings (RCC & framed structures)	15	25	50

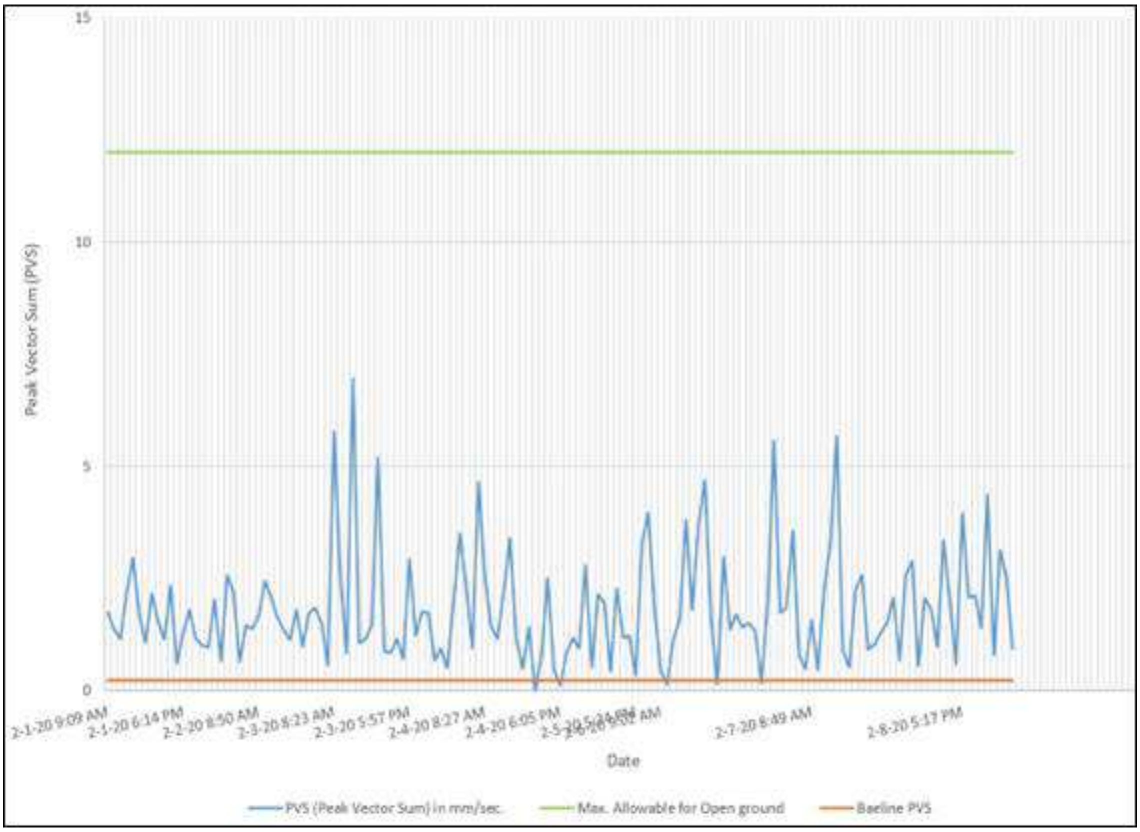

Figure 17 - PPV values observed during the blasting work

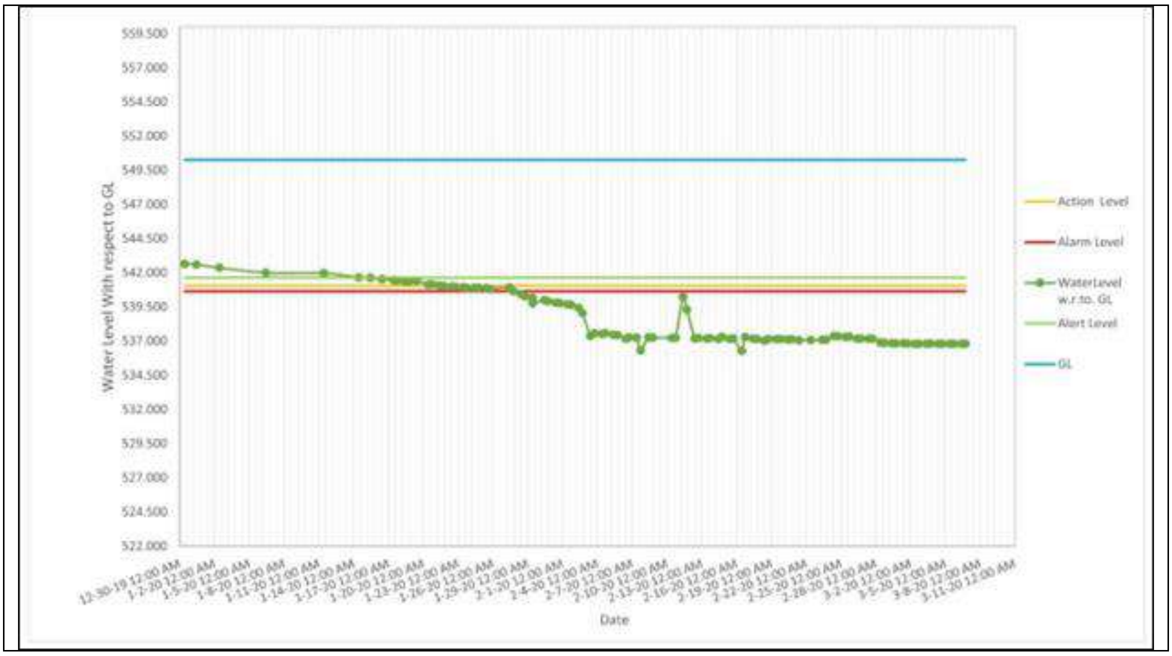

Figure 18 - Drawdown in ground water level during excavation of civil court station

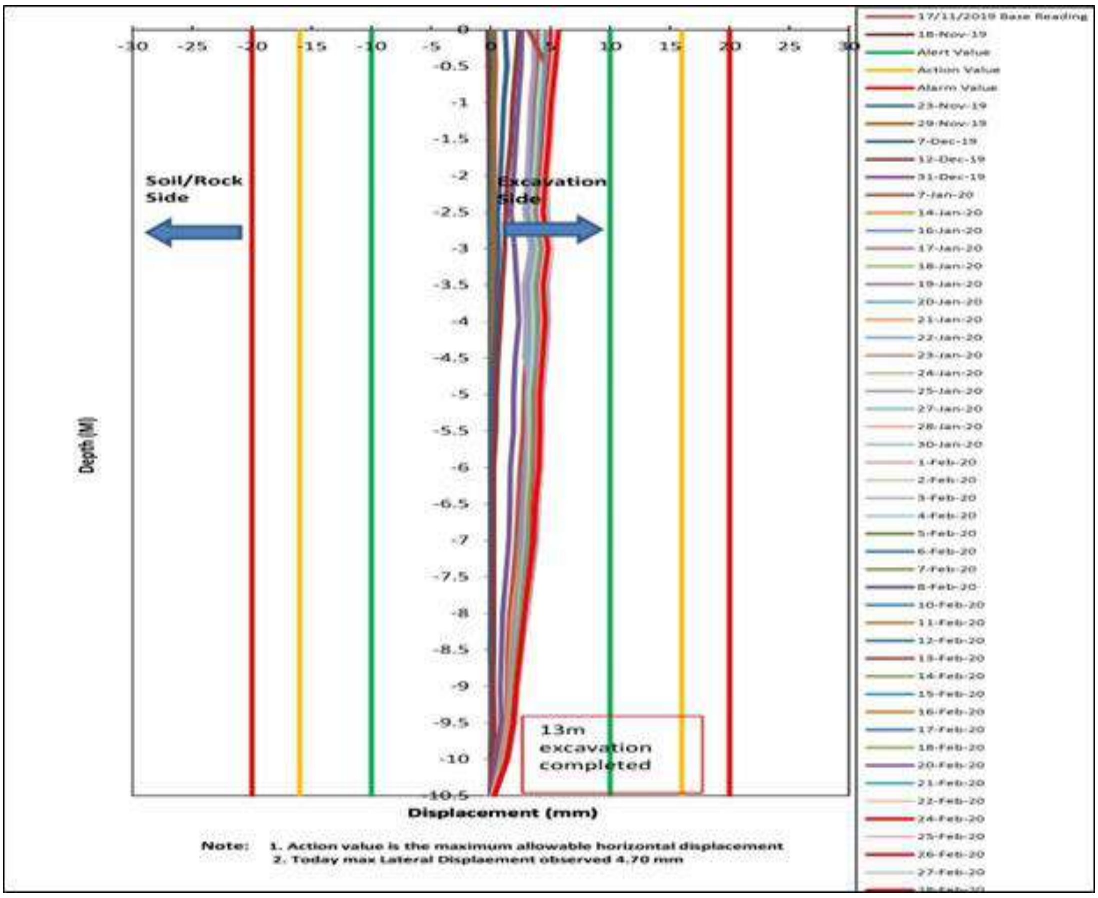

Figure 19 - Horizontal displacement in inclinometer installed in the piles of civil court station

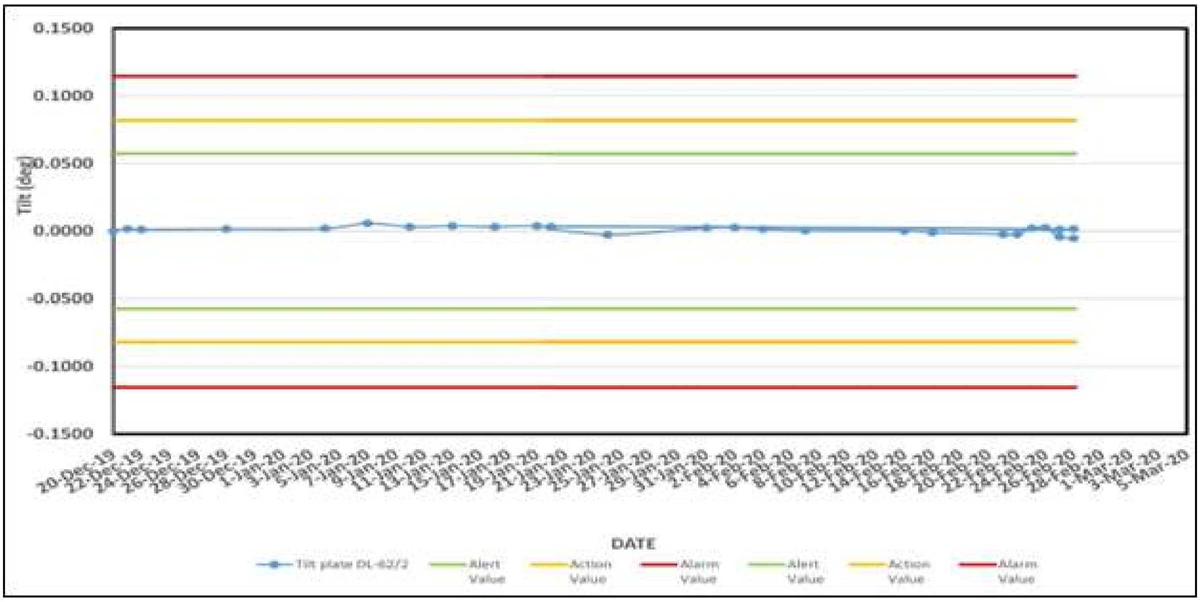

Figure 20 - Variation in tilt plate data installed in family court premises during the excavation

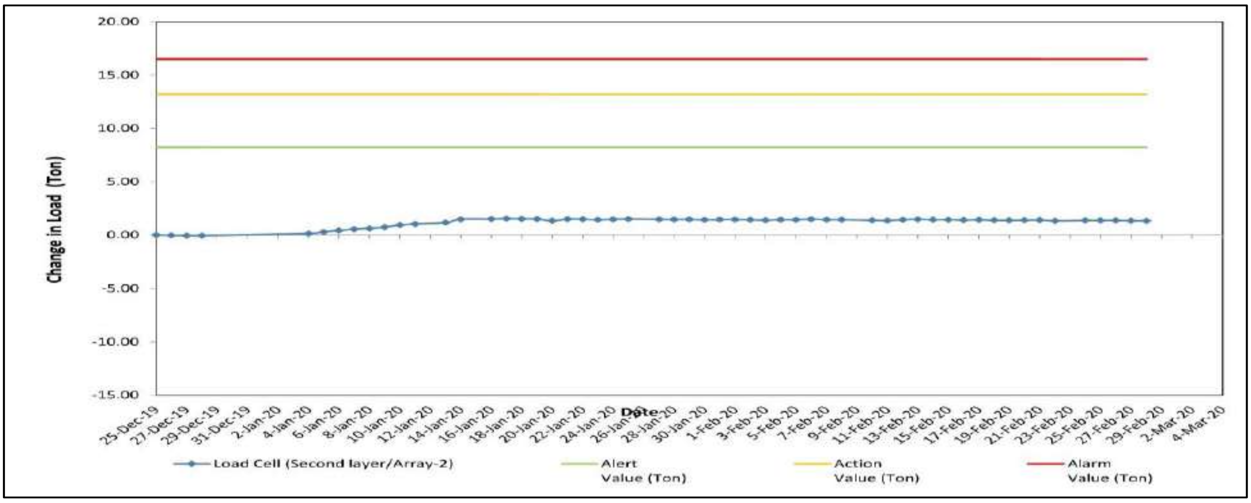

Figure 21 – Variation in load cell readings as the excavation of civil court station deepens

8. CONCLUSIONS

The ground improvement measures improve soil standup time during excavation and allow the installation of optimized initial support while providing safe excavation. Ground improvements also serve to control ground water, reduce ground loss and potential surface settlements and minimize the tunnel deformations during excavation. The variety of ground improvement techniques was used that include secant piling, dewatering, cementitious grouting, walers, struts, cable anchors, rock bolts, shotcrete with wire mesh etc. Proper instrumentation and monitoring of existing structures was done on the basis of the settlement analysis and construction impact assessment of the structures in the influence zone by various types of instruments. During construction, collected ground deformation data is reviewed against predicted values. This may result in previously performed analyses being modified and during execution the parameters being adjusted. The temporary rock support based on design drawing was also reviewed during excavation and optimized which significantly reduced the time and cost for completion of station box.

Acknowledgements

Authors are thankful to the management of MAHA Metro and M/s. Gulermak-TATA Projects Ltd. for providing necessary support to carry out the work.

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