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Soft ground shield tunneling in Bangladesh-application, construction challenges and moving forward

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ABSTRACT: Constructing tunnels using tunnelling shield is an almost 200 years old concept. However, tunnelling itself and its application in soft ground using slurry hydro-shield type tunnel boring machine (TBM), is a new construction methodology in Bangladesh. At 2450-meter length and 10.8meter in internal diameter twin tube road tunnel under the Karnaphuli River, is the first tunnel to be constructed in Bangladesh. There are several challenges for slurry pressure balance TBM tunnelling in soft soils – a tunnel built in soft ground such as clay, silt, sand or mud requires special technique compared to hard rock, to compensate for the shifting nature of the soil. This paper presents the relevant features, construction techniques, challenges in soft ground slurry shield tunnelling and solution for the construction challenges of the Karnaphuli Tunnel.

1 INTRODUCTION

The Multi-Lane Road Tunnel under the River Karnaphuli or Karnaphuli Tunnel Project (KTP) is designed as a single-deck two-way four-lane tunnel with the East part and the West part of the tunnel constructed in two separate drives under the river or the tunnel is bored in two separate tubes. Utility corridors and escape routes are arranged under the upper deck, and three cross passages are arranged between the two tunnel tubes. In this report, construction difficulties and solutions of building the Karnaphuli Tunnel is discussed and a brief description of Shield Tunnelling Methodology is also included. It all starts at the working shaft, where complex geological condition was encountered and made the construction sluggish. Coupled with a prolonged rainy season, the underground water percolates with the river water and sea water. At the time of TBM launch, there was a risk that the saturated sand layer was prone to water and sand burst – resulting in seal failure. To mitigate these risks, prior to tunnelling, there was a need to strengthen and stabilize the ground where the tunnel boring machine (TBM) will break-in. So, a "sliding ring steel + freeze" method was adopted, where a tank-like steel sleeve will support the TBM breaking-in. Also, tri-axial mixing pile and Jet Grouting Pile (JGP) injection system was used in front of the working shaft to stabilize the ground forming a stabilized area. Finally, a ground freezing technique was utilized to reinforce the launching section.

2 OVERVIEW OF THE KARNAPHULI TUNNEL PROJECT (KTP)

The Karnaphuli River divides Chittagong City into two –first piece is the area where main city and the seaport sits, the other piece is for the industrial areas. The part of the river that traverse the port, is heavily congested with ships and other vessels. The existing two bridges over the Karnaphuli River are already saturated with traffic movement. The Karnaphuli Tunnel Project will be used as a bypass route for traffic coming from Dhaka headed to Cox's Bazar and the Karnaphuli Tunnel Project will reduce the traffic congestion of Chittagong Seaport as the trucks and lorry headed to go Cox's Bazar from Chittagong sea port can bypass Chittagong City altogether.

For Karnaphuli Tunnel Project (KTP),the tunnel is bored with 10.8-meter internal diameter and 11.8-meter external diameter, the ring width is 2000mm and ring thickness is 500mm. Precast reinforcement concrete common tapered segments are assembled with staggered joint. The lining rings consist of 8 segments where 5 standard segments (B), 2 adjacent segments (L), and 1 key segments (F). The ring segments uses three reinforcement type, designated as R1, R2 and R3 and adopting a C60 concrete strength with P10 impermeability.

The tunnel is bored or mined using a pressurized slurry hydro-shield type Tunnel Boring Machine (TBM) over a length of 2450meter ground, starting on the West Bank near Patenga and ending in the East Bank near the Anowara. The two tunnels will be connected by three traversal cross passages at 700-meter interval with clear height of 2.6 meters for each cross passage. Refer to Figures 1 to 3.

Two cut and cover and open cut sections are constructed at both ends of the tunnels. For retaining the earth in these structures, different solutions have been chosen. In the deepest part, diaphragm walls are used and in the shallow part steel sheet pile are used prior to excavation. To ensure safety from flooding on both banks during construction, temporary cofferdams was installed that can withstand a 20-year flood amount. Also, two flood gates would be installed at permanently at both ends of the tunnels. Total length of KTP including Tunnel section, cut and cover at both sides, open cut and approach roads is 9265 meters. The Karnaphuli River where it crossed by the tunnel is about 1240 meter wide. Commencement date of the KTP Project was 5th December 2017 and target completion on 4th December 2022.

Figure 1. Site location of Karnaphuli tunnel project (Patenga, Chittagong, Bangladesh).

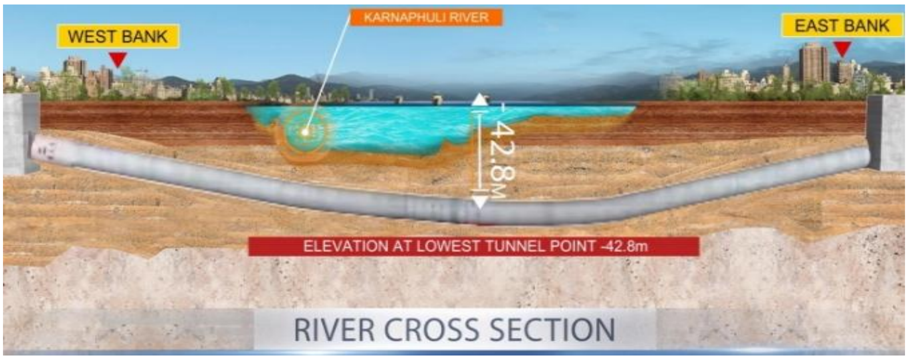


Figure 2. Longitudinal section of left north tunnel.

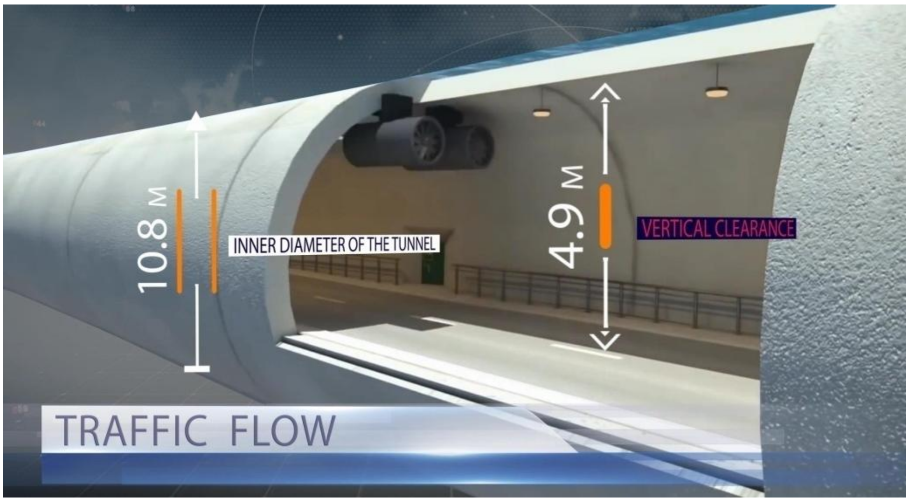


Figure 3. Cross section of Karnaphuli tunnel.

3 TUNNEL CONSTRUCTION

3.1 *TBM Launching*

Priorto the launching of the tunnel boring machine (TBM), ground treatment using a triple pipe jet grouting method was adopted with 15meter distance from the tunnel portal. Also, at the starting position of shield, in order to reduce the risk of instability at the portal concrete wall and ensure the solidity of the tunnel face and to reduce the possibility of water leakage infiltration, freezing reinforcement wall within 1.5m was made at the shield starting end.

The TBM Launching shaft was constructed using diaphragm wall and each launching shafts was a reinforced concrete thrust structure located at the back wall of the shaft to act as a reaction frame for advancing the TBM. Steel cradles and steel sleeve were installed to form a sealed chamber for retaining slurry during the start of the excavation.

3.2 *Face Support Pressure*

During tunnel construction, soil is removed from the tunnel face. The soil layer in front and above the tunnel face exerts active earth pressure. The presence of infrastructures or surcharge also contributes as additional earth pressure. For the tunnel alignment below the groundwater table, water pressure is another significant component of pressure acting at the tunnel face. Establishing, and then maintaining the correct face support pressure (face pressure) for the ground and groundwater conditions is critical to the safe operation of slurry TBM. If inadequate face pressure is applied, this will lead to excessive ground movement, and may result in the collapse of the tunnel face. During the boring of the left line Karnaphuli Tunnel, the face pressure applied varies from approximately 0.5 bar to 5.5 bar.

3.3 *Annual Grouting*

The annular space exists between lining segment and natural soil, and this space is filled up by primary grouting, thereby forming a peripheral waterproof layer. If it is determined to be insufficient, secondary grouting and subsequent compensation grouting should be done if necessary. The single-fluid grouting is used for primary grouting; cement paste is used for secondary or compensation grouting. Water glass or sodium silicate is used to treat temporary leakage and emergency treatment, to improve the durability of grout. To reduce shrinkage of grout, all grouting materials is mixed with micro-expansion agent. The grouting materials should possess good workability and dispersibility for water resistance as well as the appropriate gel time and strength, and its proportion of the mixture is determined by trial mixes.

The grouting pressure is generally higher than the tunnelling hydraulic and face pressure by 1-2 bars, however, it should be adjusted according to the geological stratum and hydraulic pressure of the ground. In principle, the pressure should: (1) not be more than hydraulic and face pressure of excavated surface, (2) not cause uplift on the ground by more than 10mm, or further settlement of more than 30mm, (3) not dislocate or deform the segment from local pressure, (4) not allow slurry to leak frequently or in large quantity from the annulus between the segment clearance or shield machine and the segment. Lastly, proper measures were adopted to control the quality of grouting and the timeliness of grouting.

3.3.1 *Grouting pressure control*

There are six slurry injection points at the tail of the shield. The annular grouting pressure of the tail is different due to the position of the slurry injection point. The construction should also be adjusted according to the actual conditions to achieve the balance between grouting pressure and the surrounding pressure. The pressure is roughly chosen to be equal to the sum of stratum resistance (pressure) plus 0.1 to 0.2 MPa. In addition, compared with the injected pressure earlier on, the post-injection pressure is 0.05-0.1 MPa larger than the injected pressure earlier on, and is used as a pressure management benchmark. The so-called resistance strength is the intrinsic value of the stratum, which is the minimum value of the pressure that the grout can be injected into the shield tail gap. The injection pressure is usually selected to be equal to the sum of the stratum resistance strength and the additional items determined by the injection conditions (grout properties, discharge amount, and injection method). Choose the proper grouting pressure which is beneficial to control stratum subsidence. In addition, in the injection pressure management, due to different soil quality, slurry, injection method and construction conditions, the required grouting pressure tends to fluctuate greatly. At this time, the grout injection amount must be managed at the same time.

3.3.2 *Grouting volume control*

Theoretical volume of the construction gap between the cutter-head and the segment is:

$$V = \frac{\pi}{4}\ x\ (D^2 - d^2)\ x\ L \qquad (1)$$

$$V = \left(\frac{\pi}{4}\right) x\ (12.16^2 - 11.80^2)\ x\ 2 = 13.55\ \frac{m^2}{ring} \qquad (2)$$

Where the injection volume, Q of the backfilling grout can be estimated using the following formula:

$$Q = V\alpha \qquad (3)$$

V=theoretical volume
α =injection rates

For Karnaphuli Tunnel Project, annular grouting adopts the filling coefficient as mentioned in the Chinese Code GB 50446-2017, depending on the ground condition uses a factor and multiple by the theoretical volume of $13.55m^3$. For example, for mucky silty clay and silty clay, it is 1.1 to 1.3 (110% to 130%), or grouting volume of $14.9m^3$ to $17.6m^3$. In silty-fine sand, it is 1.3 to 2.0 (130% to 200%), or grouting volume of $17.6m^3$ to $27.1m^3$.

During the construction, the two parameters, the pressure and the grouting volume are controlled to ensure the filling quality and quantity. If the volume of grout injection continues to increase during the construction process, factors such as over-excavation and leakage must be checked. When the injection volume is smaller than the predetermined value, the reasons that injection ratio, the injection period, the advancing rate, or the failure may be considered, and increasing grouting pressure or conducting secondary grouting are generally adopted.

3.3.3 *Grouting time and speed*

Simultaneous grouting is carried out during TBM advance, and the speed of grouting was match with the advancing rate. Tunnelling and grouting end at the same time.

3.3.4 *Grouting sequence*

Annular grouting of the shield uses 6 grouting holes at the same time, and a pressure detector is arranged at the discharge of each grouting hole to detect and control the grouting pressure and the grouting volume of each grouting, thereby achieving a symmetrical uniform injection of the back side of the segment.

3.3.5 *Secondary grouting or compensation grouting*

Despite the best efforts of the procedure for annular grouting, there is a possibility that there are still voids behind the tunnel lining. To fill these voids, secondary grouting or subsequent compensation grouting was carried out from the backup gantries behind the TBM machine. The grout flow rate will start from 10L/min and then increases gradually which is dependent on the grouting pressure. Once the desired pressure is reached, the flow rate will be reduced in step by 10L/min. The grouting will then stop and move to next grouting injection hole.

4 SLURRY TREATMENT PLANT

Pressurised slurry is the bentonite suspension used to support the tunnel face during TBM advance. At the Slurry Treatment Plant, a separation plant was used to recover slurry and re-used by separating the spoil from the slurry. The proper demanding and desilting by the plant was to ensure that the slurry densities and the slurry rheological properties will maintain safe for TBM advance.

The separation system of slurry equipment was divided into three processes, including rough selection process, primary separation with dehydration, and secondary separation with dehydration. Soil samples with particle size of 4mm to 250mm were screened out after rough selection. Soil samples with particle size of 74mm to 4mm were screened out after primary separation with dehydration. Soil samples with particle size of 74mm to 2mm were screened out after secondary separation with dehydration.

Slurry's key performance index or KPIs, namely– density, viscosity, pH, and sand content were tested two times during each ring excavation. The slurry is delivered to the muddy water tank by P1.1 pump via the slurry pipeline, the slurry seeps into the tunnel face stratum under the action of air cushion chamber pressure to form dense and stable mud film. The mud film cut by the cutterhead drops into the slurry pond bottom by gravity, the mixed slurry is delivered to the slurry treatment station by P2.1 pump via the slurry pipe.

The slurry delivered from the tunnel boring machine (TBM) is subject to soil and slurry separation by the slurry separation equipment; the slurry separated reaches the applicable standard after index conditioning, and then delivered to the TBM slurry pond by P1.1 pump. The overflow of the Slurry Separation System enters the slurring and adjusting system. After being adjusted, the overflow enters TBM for recycling. Refer to Figure 4 below.

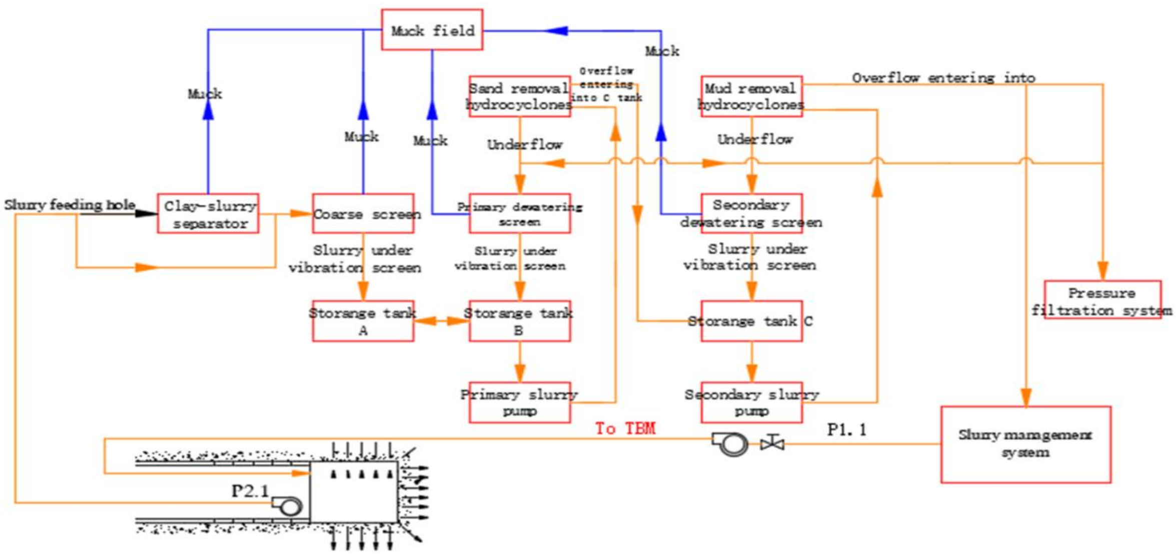


Figure 4. Slurry circulation process.

5 EXCAVATION MANAGEMENT CONTROL SYSTEM

During the TBM advance, excavation quantity is of utmost important. It is inherent to tunnelling that over-excavation or under-excavation is prevented especially at highly built areas, where over-excavation or under-excavation can cause catastrophic consequences. These catastrophes are preventable, by machine advancing and excavating soil by the right amount or between the target excavation volumes.

By measuring the excavation flow rate and the density of the slurry, the net dry weight of the solids removed during excavation is calculated and compared with the theoretical excavated volume. This provides the basis for assessing whether there has been significant over-excavation or under-excavation at the face of the machine. The measurement is done by placing a flow meter and a density meter on both the feed and discharge lines. The dry weight of the solids pumped into the tunnel is subtracted from the dry weight of the solids pumped out. The net dry weight of material removed is then compared with the theoretical dry weight, to determine if there has been potential over- or under-excavation.

6 CONSTRUCTION CHALLENGES AND MOVING FORWARD

During the construction of Left Tunnel, are many difficult situations face such as stepping between concrete segment rings, TBM shield tail deformation and leakages at tail brush location and subsequent replacement of brushes.

6.1 *Steps between Rings*

During the initial stage, the steel sleeve installation and prior TBM trial tunnelling were completed successfully and quickly followed by TBM launch and formal tunnelling advance. After almost 1-1/2 years of tunnelling, 1,200rings over 1,225 rings required were completed. However, these segmental rings were subjected to uplifting forces causing it to float after coming out of the TBM tail shield. These phenomena continued for several more rings. So, tofix these and bring the machine back to the design axis, adjustments were made.

First, the TBM stance or orientations were modified with respect to the design tunnel axis. Secondly, the grout mix was modified, and sodium silicate was used. However, the TBM tunnel segments continued to uplift.

For Ring No. 54, the segmental lining evidently moved upwards. At this time, TBM continue mining but stance was driven downwards and segmental Ring No. 56, showed a downward trend. After that, the boring machine continued to move downward, however, with the segmental linings has continued deflecting upwards from the design axis. The segment lining exceeded the maximum allowable acceptance deviation of 150mm. Between Ring No. 60 and Ring No. 68,actions were taken to mitigate and correct the TBM stance, however TBM continued to move downward, and difficulty countering the uplift forces.

After improving and controlling the TBM stance for Ring No. 69 to Ring No. 71, the downward trend of the TBM continued to slow down, the shield tail is stabilized, and the overall segment uplifting was controlled and slightly decreased. However, the TBM continued to move downwards, and the end of the TBM machine moving the opposite direction.

Due to the uplifting of the tunnel, the adjacent ring stepping was high, Ring No. 66 has maximum step of 35mm which exceeded the maximum allowable acceptance for segment lining stepping is ±17mm according to the Chinese Code GB 50446-2017 for this tunnel project.

6.1.1 *On-site solution to reduce segment steps*

According to the construction record, to control the tunnel floating, different grout mix ratios and different grouting volumes are used during the on-site construction.

6.1.2 *Grout mix ratio*

During the initial tunnel drive from Ring No. 1 to Ring No. 80, three different grout mix ratios were used. Refer to Table 1 below.

Table 1. Grout usage for synchronous grouting.

Section	Grout No.	Grout mix ratio (cement: fly ash: sand: water: bentonite)	Grout density (kg/m³)	Setting time (hours)
Rings01 to 26	1	154:288:840:494:78	1839	9.0
Rings 27 to 60	2	179:263:840:494:78	1886	8.0
Rings 61 to 80	3	204:238:840:78:494	1896	6.5

The setting time for Grout No. 1, No. 2and No. 3 decreases after the change, so the buoyancy of the grout on the TBM tail shield segments in the three sections (Rings 01 to 26, 27 to 60, and 61 to 80) is reduced in turn. It follows that shortening the setting time of the grout can help restrain the segment floating. At Ring 60, it was found that the segment uplifted, so the grout mix ratio was adjusted to shorten the setting time to 6.5 hours, but the segment floating did not change.

6.2 *TBM Tail Shield Deformation and Rectification*

At the start of January 2020,at Ring 614, the TBM tail shield was significantly deformed due to external pressure on TBM machine. TBM shield tail roundness has deviated approximately 67mm (refer to Figure 5),where the maximum allowable deviation is 30mm. After this deformation, the cutterhead face pressure was modified and the TBM machine stance with respect to the tunnel alignment deviated. At this point, TBM advancing works stopped and rectification of the tail shield started.

6.2.1 *Possible causes of tail shield deformation*

- excessive external pressure
- TBM operation - steering and grouting
- excessive grout volume (from Ring 490) of more than 150% of theoretical volume

6.2.2 *Action taken for tail shield deformation rectification*

Plan A (which consists of “A Thrust-Jack system in combination with a steel beam frame”). Steel beam frame and 500-tonne jack to apply thrust on shield. Measured total deformation from the applied pressure was approximately 7mm. However, after releasing pressure (applied by jack) the tail Shield deformed back by approximately 5mm, that is remaining plastic deformation of approximately 2mm. Plan A failed. Then experts suggested to make some pressure release hole on the TBM tail Shield wall to remove soil (See Table 2). Total 7 nos. Pressure Release Holes (RH01- RH07) were installed and around 3.8m^3 sand removed and trust applied by jack. Applying this technique 19mm plastic deformation achieved. For further rectification contractor applied plan B. Please refer Figures 5 to 9 for details.

Plan B (“Circular steel Plate frame” plus a thrust-jack system). Installation of the circular steel Frame inside the TBM tail shield. Pushing by the 6 jacks and installing four stiffener rings. Repeatedly applied thrust on 6 jacks and deviation was controlled within limited range of 30mm.

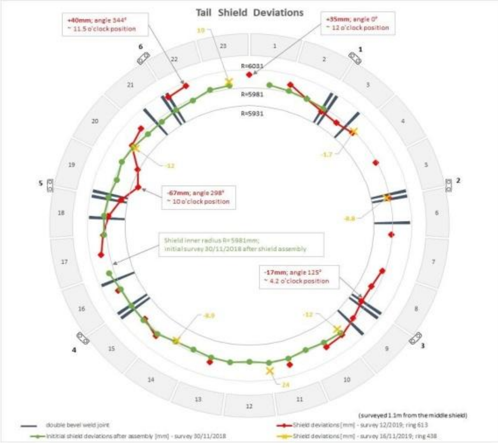


Figure 5. Sketch showing maximum deformation at shield tail.

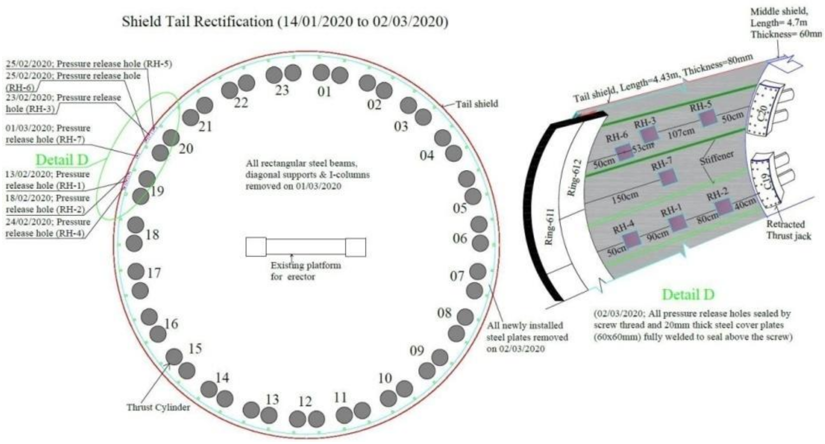


Figure 6. Pressure release holes RH-1 –RH-7.

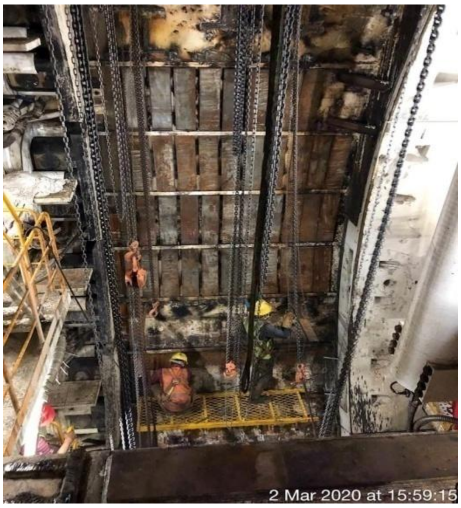


Figure 7. TBM tail shield rectification.

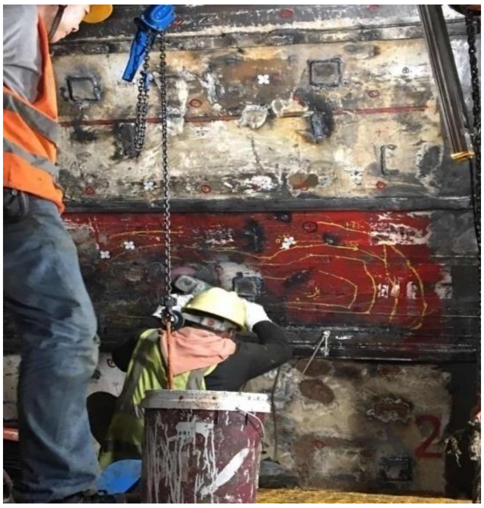


Figure 8. TBM tail shield rectification.

Figure 9. TBM tail shield rectification.

6.2.3 *Replacement of tail brushes*

During TBM excavation, tail seal grease sometimes leaked through the brushes which are located at the lower portion of the tail shield. The leaks became more frequent when the TBM machine continued advancing.

At this situation TBM advance stopped and replace the damaged tail brushes to ensure shield water tightness and safety of the tunnel. In total, the tail brushes were changed three times to complete left north tunnel of Karnaphuli Tunnel Project.

Table 2. Soil removed from behind shield.

Date	Location	Daily Volume (m^3)	Cumulative Volume (m^3)
16/02/2020	RH-01	1.00	1.00
17/02/2020	RH-01	1.00	2.00
24/02/2020	RH-02	0.80	2.80
26/02/2020	RH-01 & RH-04	1.00	3.80

7 EXPERIMENTAL STUDY ON STRUCTURAL SAFETY ASSESSMENT

7.1 *Objectives*

- Understand the safety of circumferential joint under different intensity of steps.
- Strength, stiffness, and the failure model of circumferential joint.
- Similarities and differences between joints connected by different links.
- Understand the amount of shear force distributed on each link and the corresponding failure sequence of each component.

Note: Refer to Figures 10-15, for more details.

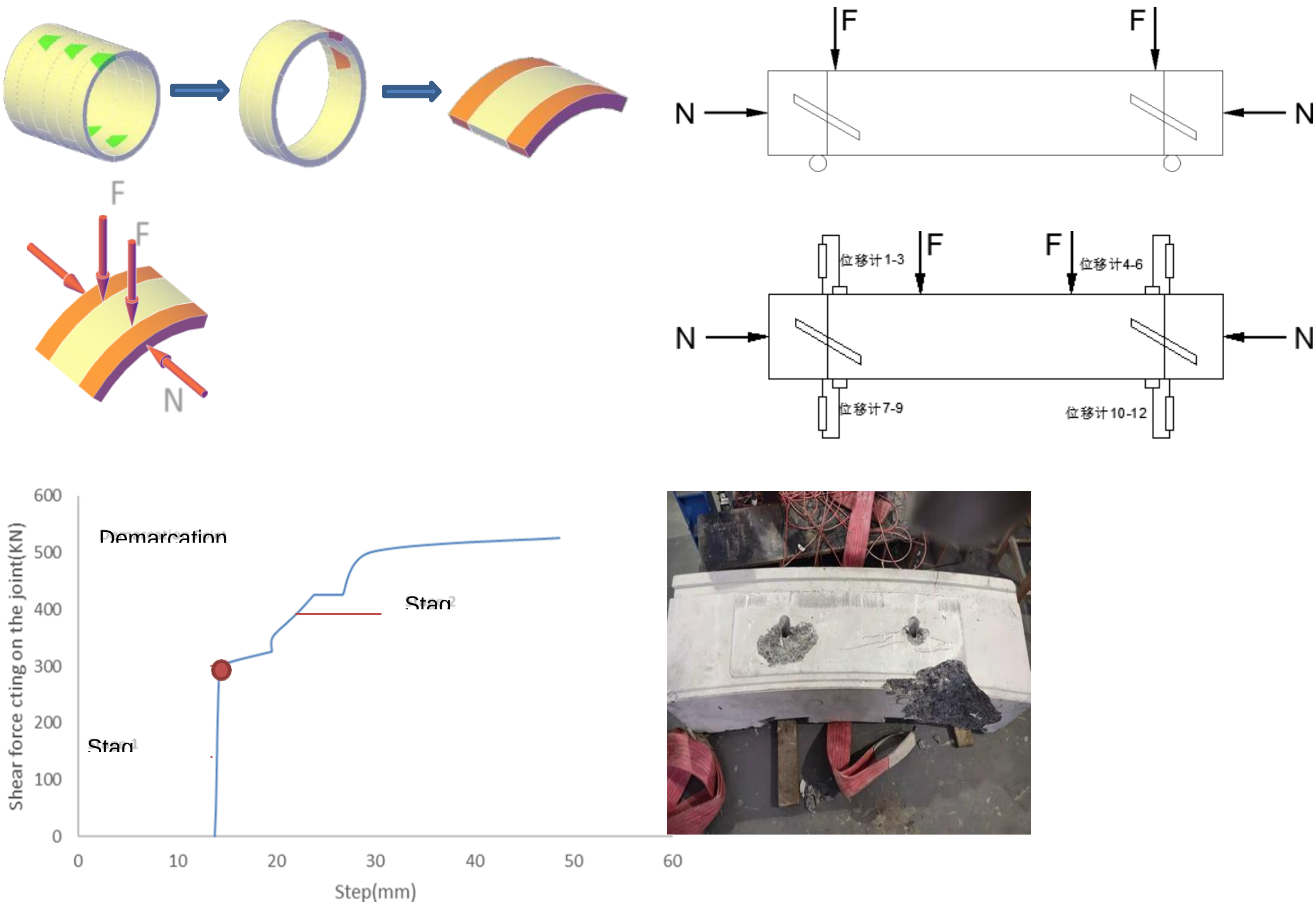


Figure 10. Test on segments to find the relationship between strain of the bolts and step.

7.2 *Bolts Tensile Test*

Figure 11. Testing of specimen at failure mode

Figure 12. Specimen showing actual failure

7.2.1 *Failure mode and damage process of specimen*

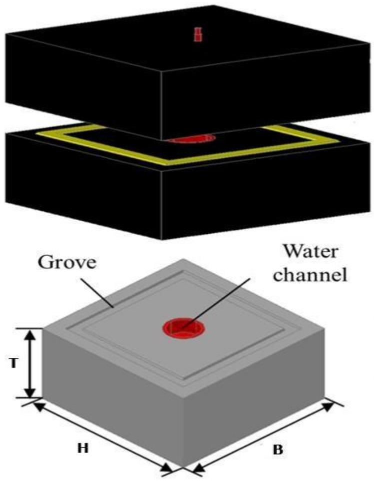
- The damage process of the specimen can be divided into two stages.
- No shear force acting on the bolts; shear force balanced by friction.
- The increment of shear force acts on the bolts until segment's failure.
- Punching shear failure happens on the concrete beneath the bolt.
- The strain is the maximum strain measured, which is the worst case.
- The maximum tensile strain of the bolt is 945με, when the step of the joint is 35mm, which is still inelastic.

7.2.2 *Bolts tensile test results*

- The bolts are in elastic state when the step reaches35mm.
- Punching shear failure happens on the concrete beneath the bolts for all the tests.
- The gasket reduces the stiffness of the circumferential joint before shear force acting on the bolts and reduces the bearing capacity of the circumferential joint.
- The initial stiffness of the specimen with shear pin increases significantly, while the shear pins have minor contribution to the ultimate bearing capacity of the specimen.
- The initial stiffness has no relationship with longitudinal force when the shear force does not exceed the friction.

7.3 *Gasket Waterproofing Test*

(a)



(b)

Figure 13. Gasket waterproofing test (b). Repair work (applying adequate glue) for the detached gasket (a).

Figure 14. Gasket waterproofing test.

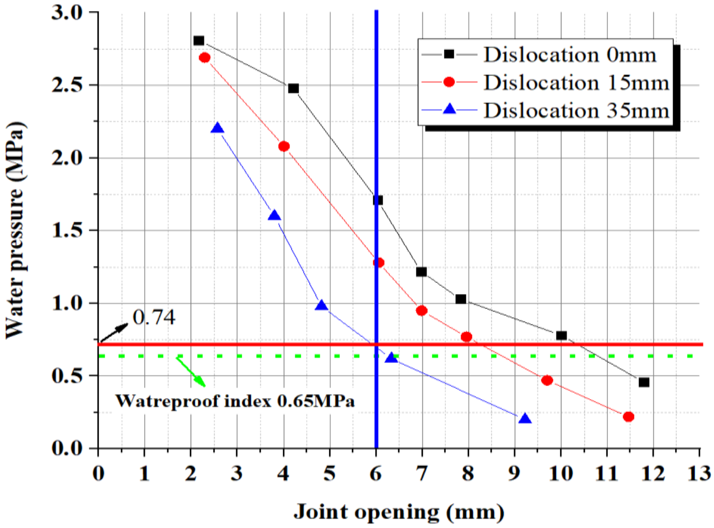


Figure 15. Relationship between water pressure with joint opening.

- If no dislocation, the waterproofing capacity of the joint is 1.71MPa, when the opening is 6mm
- If 15mm dislocation, the waterproofing capacity of the joint is 1.28MPa, when the opening is 6mm
- If 35mm dislocation, the waterproofing capacity of the joint is 0.74MPa, when the opening is 6mm,and 0.9MPa (5mm opening), 1.5MPa (4mm opening)

8 CONCLUSIONS

Although considerable amount of tunnelling experience gained managing this large tunnel boring machine (TBM) through complex ground under the Karnaphuli River, there were some difficulties encountered. But with proper steering, face pressure control in difficult geology, deforming shield, and uplift issues, overall, it was overcome with team effort. The continues monitoring and supervision produced a high-quality segment lining with minimal defects and leaks. With knowledge gained from large TBM excavation in adverse conditions, a great deal of confidence was attained and would be applied to similar projects in the future.

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