Underpinning and tunnelling undercrossing Keppel Viaduct using specialized cutter tools to cut the piles

Bryan Chng
Land Transport Authority, Singapore

ABSTRACT: This paper presents the design and construction challenges of underpinning Keppel Viaduct and the successful application of specialized cutter tools on a 6.6m diameter Earth Pressure Balance (EPB) Tunnel Boring Machine (TBM) to cut through its piles in Contract C882 Keppel Station and Tunnels of Circle Line 6 (CCL6). The existing Keppel Viaduct piers FP9 and P51, located near 43 Keppel Road, was undercrossed by the section of CCL6 outer bound tunnel, that runs from Keppel Station to Cantonment Station. Two bored piles of Pier FP9 and six micropiles of Pier P51 were within the face of the proposed outer bound bored tunnel alignment. Underpinning works to the structures with micropiles were proposed and following load transfer, the existing affected piles were cut to disconnect them from the pier and subsequently bored through during the tunnelling stage.

1 INTRODUCTION

Given the highly urbanized environment of Singapore, undercrossing structures for the construction of new tunnels is increasingly inevitable. In order to minimize the impact on surface activities, engineering solutions should ensure that the surface structures remain functional throughout the duration of construction. In the case of undercrossing a vehicular viaduct, the roads will have to remain open while works are carried out to underpin and eventually tunnel through the detached piles. Contract C882, Keppel Station and Tunnels of Circle Line 6, was faced with the challenge of underpinning and undercrossing two separate viaduct piers in close proximity to each other. The construction works had to be precise to accomplish this and extensive instrumentation was put in place to monitor the movements of the structures at all times. C882 adopted innovative engineering solutions to increase the productivity and safety of the undercrossing by utilizing specialized tungsten carbide cutter tools to cut through the detached piles. This removed the need for risky and laborious cutterhead interventions (CHIs) that were required to manually remove the defunct piles that were within the tunnel face, saving an estimated 30 days from the construction program. This paper seeks to document the challenges of underpinning a live vehicular viaduct and highlight the successful application of specialized cutter tools to cut through piles during undercrossing, serving as a reference for future projects encountering similar technical challenges.

1.1 Scope of works

As part of CCL6, Contract C882 managed by the Land Transport Authority (LTA) was awarded to CSCEC-Nishimatsu JV. The contract comprises of the construction of Keppel Station and 4 tunnel drives (two tunnel drives towards HarbourFront Station and two tunnel drives towards Cantonment Station, as shown in Figure 1) using two 6.6m diameter EPB tunnel boring machines. In the alignment of the outer bound tunnel towards Cantonment Station, the TBM undercrossed two Keppel Viaduct piers, piers FP9 and P51. The viaduct piers were underpinned with a transfer beam supported on micropiles to fully support the load of the original piles. Load transfer onto the new foundation was carried out using sacrificial hydraulic jacks that were cast in once the works were completed. The original piles that no longer bear the pier’s load were disconnected from the pier via wire saw cutting to allow the TBM to tunnel through those piles safely. Using specialized tungsten carbide cutter tools, the team was able to successfully cut through the piles within the tunnel face. Real-time instrumentation monitoring was deployed to ensure that the pier movements were within predefined acceptable limits and the viaduct remained safe to use throughout. Since the design and construction methodologies for pier FP9 and P51 are similar, this paper focuses on the underpinning and undercrossing works for pier FP9 to remain concise and reduce repetition.

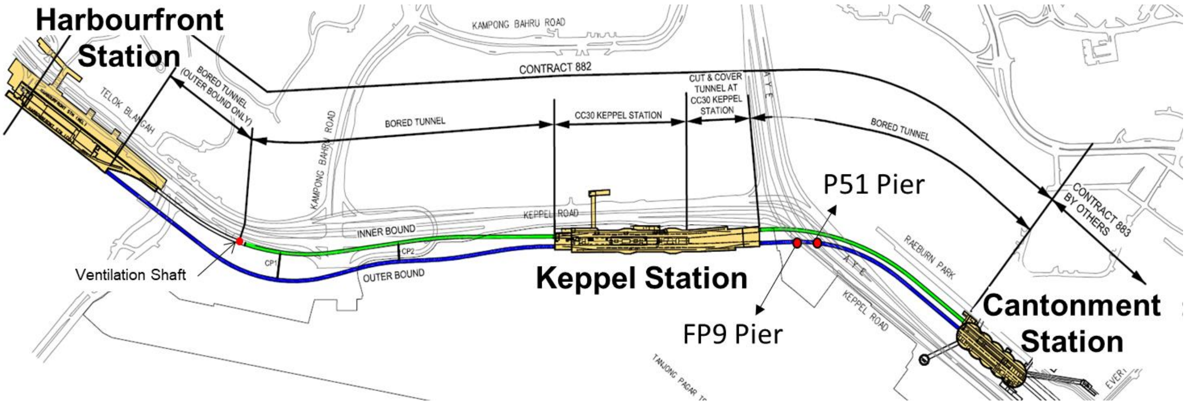

Figure 1. Overview of contract C882 Keppel Station & Tunnels

1.2 Location of proposed undercrossing

Figure 2 shows a plan view of the tunnel drives from Keppel Station to Cantonment Station and the proposed undercrossing locations at the outer bound tunnel. Pier FP9 is a solitary pier supporting the Keppel Viaduct ramp, while pier P51 supports an extension of the Keppel Viaduct. Figure 3 shows a 3D rendering of the abovementioned piers.

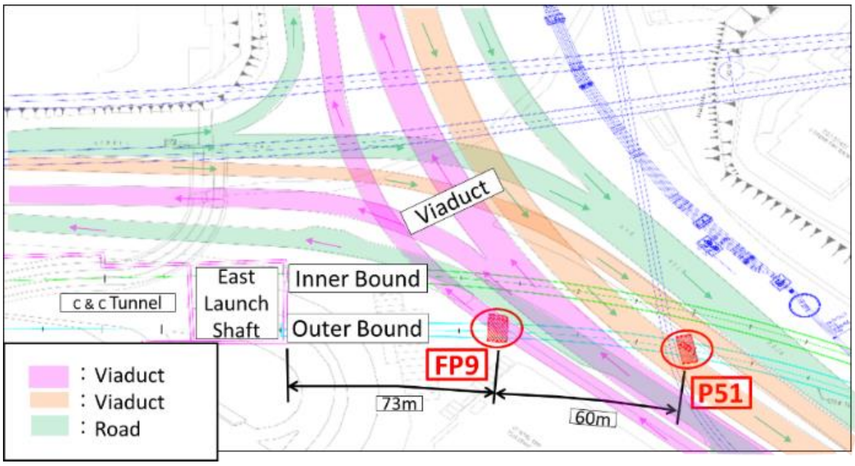

Figure 2. Plan view of the tunnel drives from Keppel Station to Cantonment Station, specifying the piers' location

Figure 3. 3D rendering of the two affected piers

1.3 Geology

Jurong formation consists of a wide variety of sedimentary rocks, which include beds of siltstone, mudstone, sandstone, conglomerate and limestone that underlies the Southwest area of Singapore. The formation was formed in the late Triassic to middle Jurassic geological periods. The geology at the underpinning works comprised of Jurong Formation residual soils (SVI) and completely weathered rock (SV). Figure 4. depicts the geological profile at piers FP9 and P51.

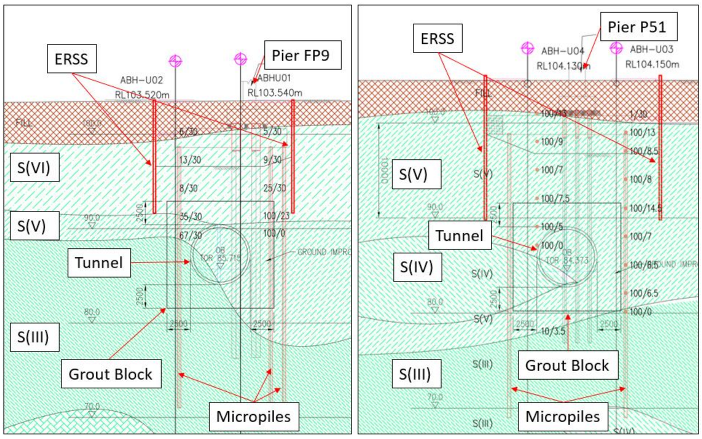

Figure 4. Geological profile at pier FP9 (left) and pier P51 (right)

1.4 Ground improvement

CHI was planned at the underpinning areas to facilitate the change to the specialized cutter tools. The roller cutters were replaced with the specialized tungsten carbide cutter tools for the purpose of pile cutting and swapped back after the pile cutting. As the geological formation at the tunnel horizon is S(IV) and S(V) with SPT values ranging from 35 to 100, the ground is competent and only treated for permeability. Double packer Tube a Manchette (TAM) grouting to an extent of 2.5m surrounding the tunnel cross-section was performed in advance to reduce the permeability in the area. The grout block is represented by the square surrounding the tunnel section shown in Figure 4. After completing the ground improvement, the permeability of the ground was tested using the falling head permeability test at two different locations and at three depths each. The treated ground is required to have a permeability K value of not greater than $1 \text{x} 10^{-7}$ m/s and the permeability test results are shown in Table 1.

Table 1. Coefficient of permeability recorded for the falling head permeability test

S/No.	Location	Coefficient of Permeability, K (m/s)			
		Testing Levels (mRL)			Average K, Ground Improvement Block
	BH Ref.	91.0~ 90.0	87.7 ~ 86.4	84.5 ~ 83.5	
1	FPT3	5.80E-08	1.10E-07	5.50E-08	7.43E-08
2	FPT2	3.30E-08	8.10E-08	3.50E-08	4.97E-08

The improved ground condition minimizes the water ingress, allowing the CHI to be performed in free air condition, subjected to the approved step-down procedure. This improves the productivity and safety of the CHI and is especially important if manual cutting the piles are required, as working under compressed air condition would significantly reduce the productivity.

2 UNDERPINNING WORKS AT PIER FP9

2.1 Underpinning construction sequence

The construction sequence are as follows:

1. Install the micropiles to the design toe level
2. Install the ERSS for the localized trench excavation
3. Excavate 1m below the strut level and install the strut
4. Repeat until the bottom of proposed transfer beam level
5. Cast 100mm thick lean concrete

6. Cast transfer beam below the existing pile cap with existing piles debonded
7. Jack the existing pile cap against the transfer beam to transfer the load onto the transfer beam
8. Using the wire saw, cut the original piles to disconnect it from the pile cap
9. Monitor the pier movements throughout the works
10. When the instrumentation readings have shown a stable trend with no signs of further settlement, cast the gap between the existing pile cap and transfer beam with concrete
11. Backfill the trench in stages, up to the ground level
12. Extract ERSS

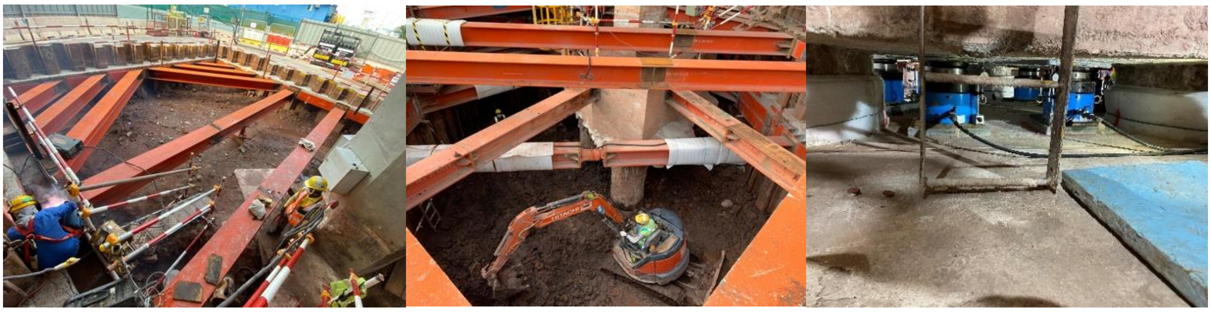

Figure 5. Images of underpinning works at pier FP9

2.2 Existing Foundation

Keppel Viaduct pier FP9 was designed for a total working load of 7000kN and is supported by three 900mm diameter bored piles of approximately 24.7m in length, each containing nine T25 reinforcement bars. The schematic of the piles' location and tunnel alignment is show in Figure 6. One bored pile lies in the direct alignment of the tunnel drive and has a section of 5.77m length of pile within the tunnel face, a second pile is at the edge of the alignment.

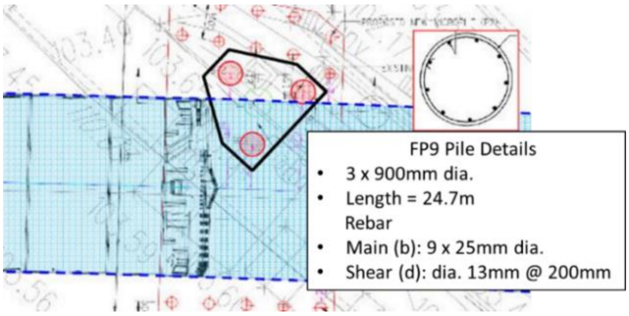

Figure 6. Schematic of the affected piles from plan view

2.3 Micropile construction

After completing the ground improvement works, 15 micropiles were installed. To overcome the low headroom restrictions of 4.5m under the viaduct, a low headroom pilling machine was deployed and the micropiles were installed in 3m sections. In compliance with the design requirements, a section of each micropile was debonded. The micropiles are 406mm in diameter with 5H40 reinforcements and installed to a length ranging from 44.5m to 47m. Figure 7 shows the layout plan of the micropiles at the transfer beam of pier FP9.

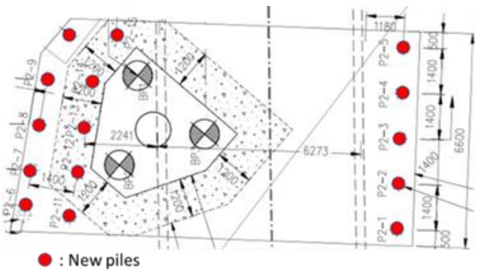

Figure 7. Schematic of the micropiles' location at the transfer beam of pier FP9

2.4 Transfer beam for underpinning

A transfer beam of dimensions 12.385m (Length) x 6.6m (Width) x 2m (Depth), supported on 15 newly constructed micropiles, was constructed to carry the load from pier FP9. To construct the transfer beam, an Earth Retaining Stabilising Structure (ERSS) using sheet piles was installed to facilitate the excavation up to 7m below the ground level. Three layers of struts were installed to prop the ERSS during the excavation down to the transfer beam level. As seen from Figure 8, the transfer beam (shaded blue) is supported by the 15 micropiles (shaded yellow) and the existing three bored piles were debonded from the transfer beam. Hydraulic jacks were used to jack the existing pile cap against the transfer beam to transfer the load onto the new micropiles.

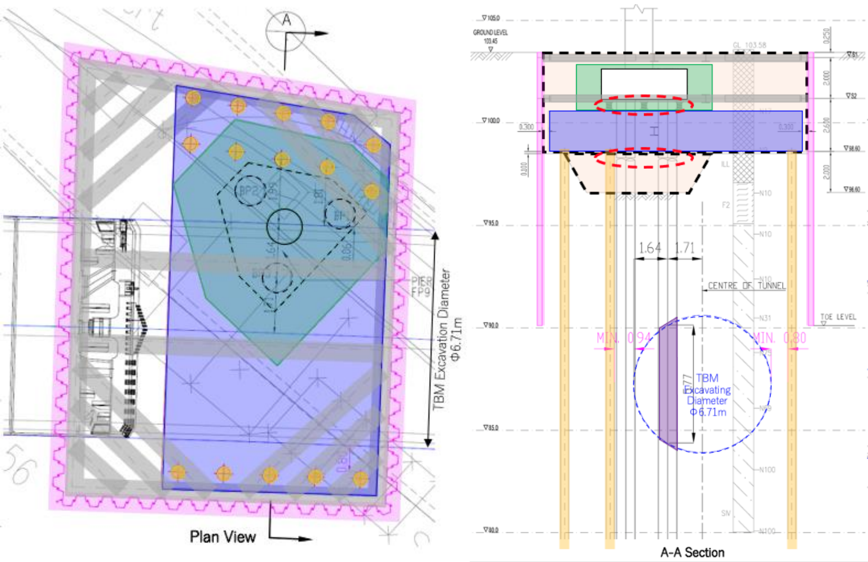

Figure 8. Schematic of the underpinning works in plan and section view

2.5 Instrumentation for the underpinning works

The instrumentation used to monitor the pier during the works were installed in the layout shown in Figure 9. Following the design, tiltmeters were installed in perpendicular planes on the pier to monitor the tilting in both axes and optical prisms were installed to monitor the horizontal and vertical displacement of the pier. Building settlement markers were installed on four corners of the existing pile cap and on the transfer beam to record any absolute and relative displacement. Surface settlement markers were installed on the ground surrounding the ERSS to observe for any surface settlement.

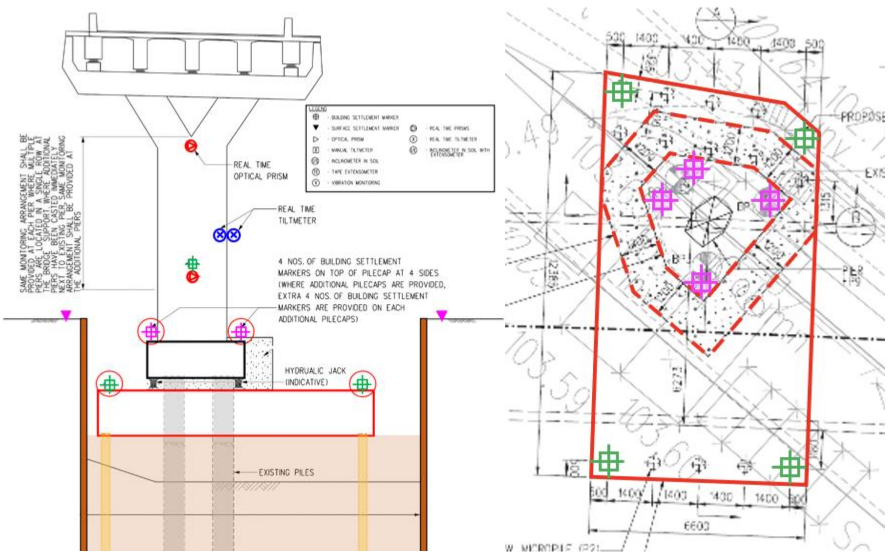

Figure 9. Schematic of the instrumentation layout on the pier in section (left) and plan (right) view

2.6 *Pier Jacking for load transfer*

The pier jacking procedure transfers the pier's load onto the new foundation. Hydraulic jacks installed between the pile cap and transfer beam were extended against both surfaces to transfer the load from the pile cap onto the transfer beam and new micropiles. This operation was conducted meticulously with real-time monitoring of the pier's movements to ensure that the settlement is controlled, and the deflection of the transfer beam is within allowable limits.

2.6.1 *Hydraulic Jack requirements*

There are three primary requirements for the jacks used in the pier jacking. Firstly, the hydraulic jacks must have sufficient capacity to support the load. Secondly, it must have a threaded piston and lock nut to allow the jacks to be locked in place. Thirdly, the jack's hydraulic system must be able to be drained and replaced with non-shrink grout. These jacks are sacrificial and will be cast with the transfer beam once the jacking works are completed.

The design requirement for total hydraulic jack capacity is three times the design total working load of the pier. The design permanent action for pier FP9 is 5000kN and its design variable action is 2000kN, therefore the total working load is 7000kN and the total hydraulic jack capacity required for this operation is 21000kN. These values are summarized in Table 2. According to the design, a minimum of four hydraulic jacks are required for this operation to ensure that the load is distributed. For pier FP9, six jacks of 3500kN capacity each, providing a total of 21000kN, were used. However, the actual required jacking load applied for the works is only 90% of the design permanent action of the pier, which amounts to 4500kN.

Table 2. Hydraulic jacks' design capacity requirements for pier FP9

Total Working Load of Pier FP9	Permanent action	5000kN
	Variable action	2000kN
Required Hydraulic Jack Capacity	3 x total working load	3 x (5000 + 2000) = 21000kN
Actual Jacking Load Applied	90% of permanent action	0.9 x 5000 = 4500kN

2.6.2 *Instrumentation monitoring of the pier during jacking works*

As the building settlement markers can only monitor up to an accuracy of 1mm, it is not precise enough for the purpose of settlement monitoring during the jackings works. As such, dial gauges that have an accuracy of 0.01mm were deployed as the primary instrument used to monitor the movements of the pier and transfer beam. The layout of the dial gauges is shown in Figure 10. The dial gauges (PD1 and PD2) were deployed on the pile cap to monitor the movement of the pile cap itself. Dial gauges (TD3, TD4, TD5, TD6, TD7 and TD8) were deployed on the transfer beam to monitor its settlement and its deflection in the center relative to the edges.

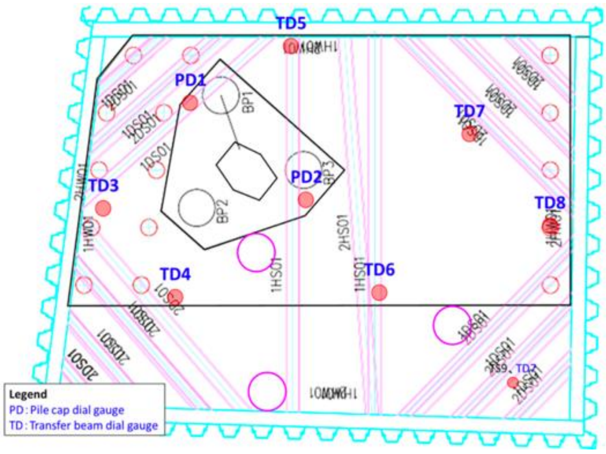

Figure 10. Schematic showing the location of the dial gauges at pier FP9

2.6.3 *Pier jacking sequence*

The pier jacking process was carried out in 10% load increments and stops when it reaches the required load of 4500kN, or when the existing pile cap is uplifted by 2mm. After each step, a monitoring period of 20 minutes is held to allow for any movements in the structure to stabilize and for an accurate instrumentation reading to be taken for assessment. Once the instrumentation readings are confirmed to be within the design tolerance, the next increment of the jacking sequence can be carried out. For pier FP9, the design tolerance for the pier movement is 5mm settlement and 2mm uplift with a maximum transfer beam deflection of 2mm. Once the target jacking load is achieved, the structure is further monitored on an hourly basis for 12 hours to ensure that the movements have stabilized and are within acceptable limits. The above jacking procedure can be visualized in the flowchart shown in Figure 11.

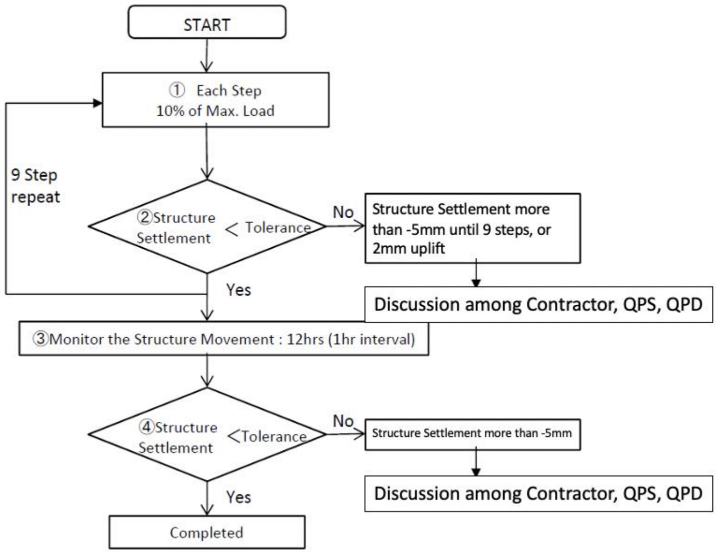

Figure 11. Flowchart depicting the pier jacking procedure

2.6.4 *Pier movements recorded during jacking*

All the dial gauges were calibrated before the pier jacking works. The dial gauges' readings were recorded 20 mins after each jacking sequence. At the time when the final jacking sequence was completed, the pile cap had uplifted by 0.60mm and the deflection of the transfer beam was less than the 2mm design limit.

2.7 *Pile cutting to disconnect defunct piles from pier*

After the pile jacking works had been completed, the original piles should no longer be supporting load from the pier. These piles will have to be disconnected from the pier to facilitate the process of tunnelling through them, as the TBM would likely cause movement to the pier if they remain connected. A wire saw was used to cut a 200mm section from the piles to fully disconnect them from the pier. This was carried out by excavating beneath the transfer beam as shown in Figure 12 and cutting the piles one at a time. The excavated area was subsequently backfilled once this process was completed. Figure 13 shows the actual piles during and after pile cutting.

Figure 12. Schematic demonstrating the pile cutting procedure with plan (left) and section (right) views

Figure 13. Images of the piles during (left) and after (right) the pile cutting

3 TUNNELLING THROUGH THE PILES

3.1 Specialized cutter tools

Specialized cutter tools using tungsten carbide inserts were designed and manufactured to be used specifically for the purpose of cutting through the piles. The cutting bits with tungsten carbide inserts were procured from a specialist company in Japan. In order to ensure compatibility with the TBM, these cutting bits were then shipped to China where the TBM manufacturer, China Railway Engineering Equipment Group Co., (CREG), is located to install on a cutter tool housing that fits the TBM precisely. For the purpose of the pile cutting, 35 roller disc cutters were replaced with the specialized cutter tools. There are three types of specialized cutter tools used as shown in Figure 14. They are the centre cutters, face cutters and gauge cutters.

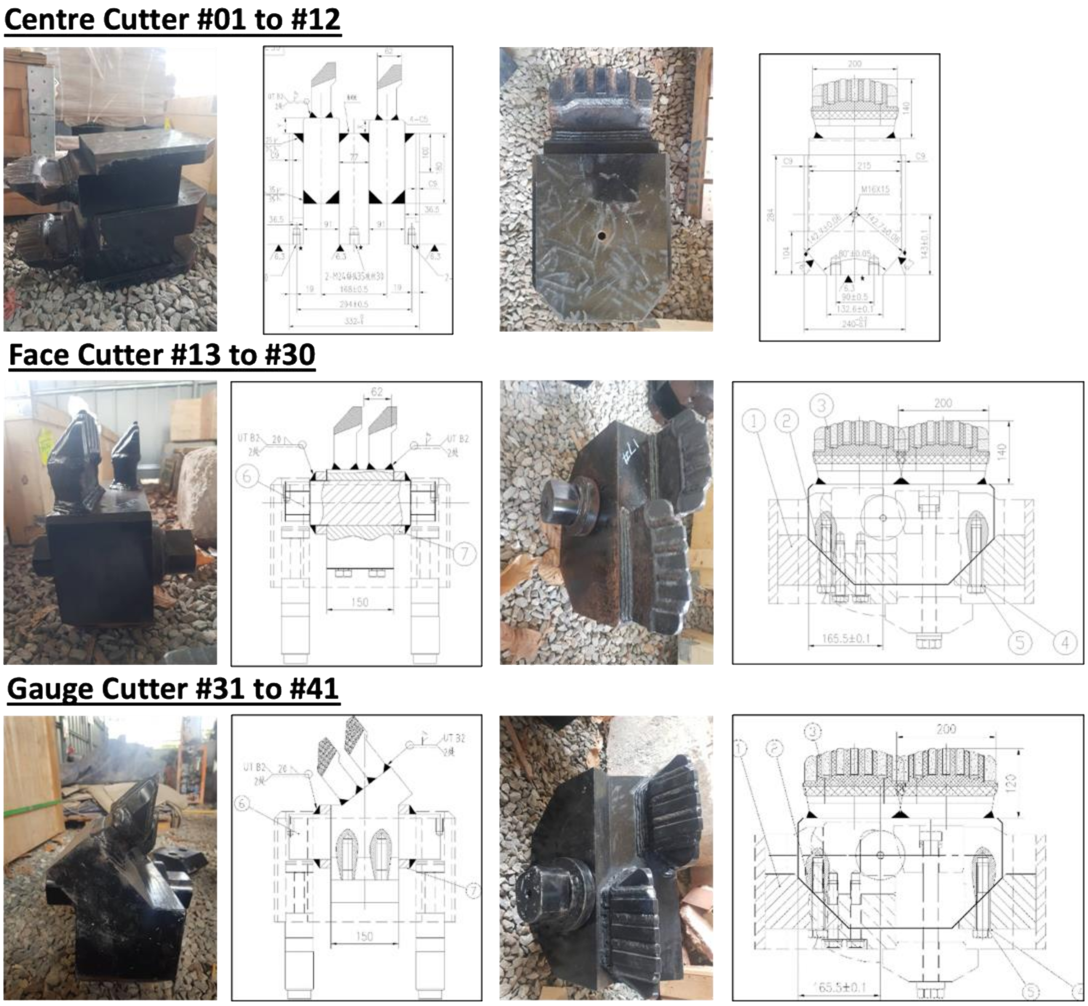

Figure 14. Specialized cutter tools used for pile cutting

The location where these cutter tools are installed is shown in Figure 15. The centre cutters will be replacing the roller disc cutters shaded in blue, the face cutters will replace the roller disc cutters shaded in purple and the gauge cutters will replace the roller disc cutters shaded in orange. The specialized cutter tools were designed such that after installation, the cutter tool tracks create an angular profile that protrudes at the center of the cutterhead. Figure 16 shows the change in profile of the cutterhead before (left) where the profile is flat and after (right) changing the cutter tools where the profile is protruding at the center.

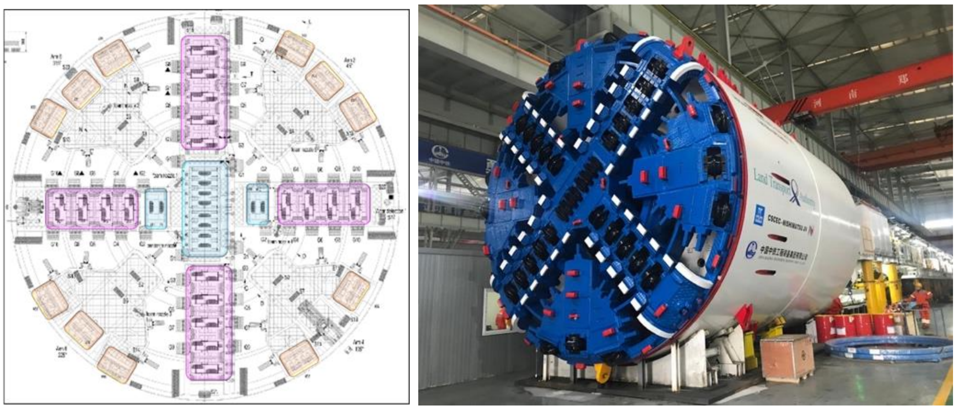

Figure 15. Schematic of the cutterhead (left) and photo of the actual cutterhead (right)

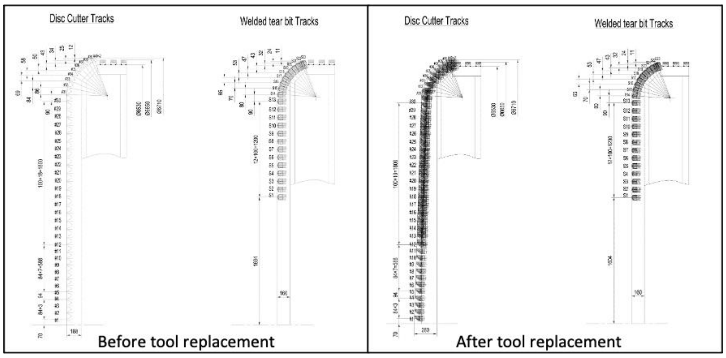

Figure 16. Profile of cutter tracks before (left) and after (right) cutter tools replacement

The angular profile of the specialized cutter tools enables the TBM to progressively excavate into the reinforced concrete (RC) pile, instead of a flat head-on approach where a high torque will likely be observed. Figure 17 demonstrates the intended excavation sequence into the RC pile starting from the center of the cutterhead where the specialized cutter tools protrudes the most. This results in the progressive disintegration of the pile.

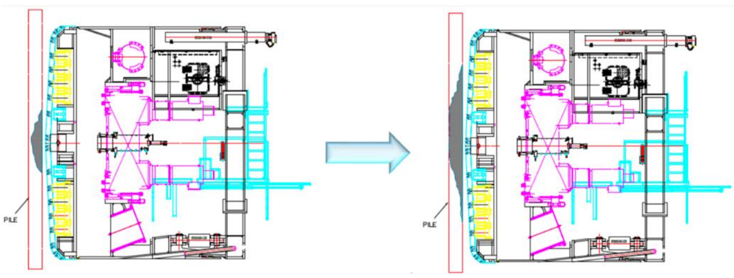

Figure 17. Schematic of the intended approach for the TBM excavating into the piles

3.2 TBM's operational parameters

Although the specialized cutter tools assist in the pile cutting operation, its success depends heavily on how the TBM's operational parameters are managed and monitored. The target TBM operational parameters were set to prioritize the pile cutting operation and maximize the specialized cutter tools' ability in cutting the piles.

The target operational parameters are as follows:

- Advance speed = 1-2 mm/min
- Cutter rotation = 1.5-2.0 rpm
- Cutter torque ≤ 3500 kNm
- Thrust force ≤ 15000 kN

As the target advance speed was significantly lower than regular tunnelling advance speed, the auxiliary hydraulic pump was connected to the thrust jacks to provide the TBM operator with a more precise control over its rate of extension. Thus, allowing the advance speed to be maintained between 1-2mm/min. The TBM advances in a slow and controlled manner to allow the grinding action of the specialized cutter tools to cut the rebars within the piles.

3.3 Parametric analysis

The TBM's parameters were analyzed after successfully cutting through the piles and compared against the parameters during the regular tunnelling in soil. The pile cutting works at pier FP9 took 8.5 hours and the excavation management system reported values that were within the acceptable limits. Figure 18 plots four important TBM operational parameters; advance speed, cutter speed, cutter torque and thrust force during pile cutting on the left and during regular tunneling in soil on the right.

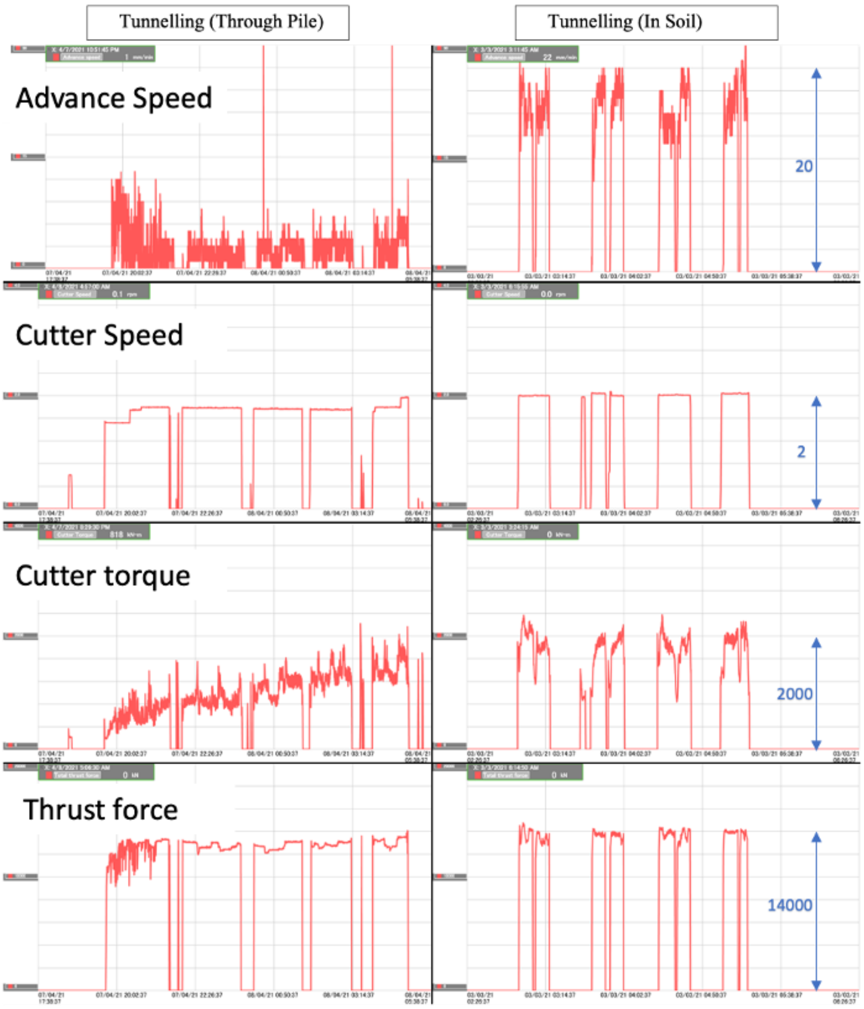

Figure 18. Comparison between the TBM advance speed, cutter speed, cutter torque and thrust force whilst cutting through the pile (left) vs regular tunneling in soil (right)

The following observations can be drawn from comparing the parameters:

- The advance speed averaged 1.5mm/min which is significantly lower than the 20mm/min average during the regular tunnelling in soil. This slow advance speed was intended, as it provided time for the piles to be progressively cut.

- The cutter speed was 1.8rpm compared to 2rpm for regular tunnelling in soil. The lowered cutter speed facilitates the grinding motion of the specialized cutter tools against the pile, protecting the tools from impact damage.
- The cutter torque is lower than the values of regular tunnelling in soil as the Clay Shock in the excavation chamber was progressively replaced with excavated materials.
- The thrust force remained relatively constant throughout and is comparable to the values observed in the regular tunnelling in soil.

3.4 Instrumentation readings during pile cutting

The instrumentation installed on the pier were monitored closely during the pile cutting operation to ensure that there were no sudden movements of the pier and it remained within allowable limits. The readings from the real-time optical prisms and tiltmeters remained stable throughout the duration of pile cutting. Figure 19 shows the elevations readings from the real-time optical prisms on the left and the readings from the real-time tiltmeters on the right.

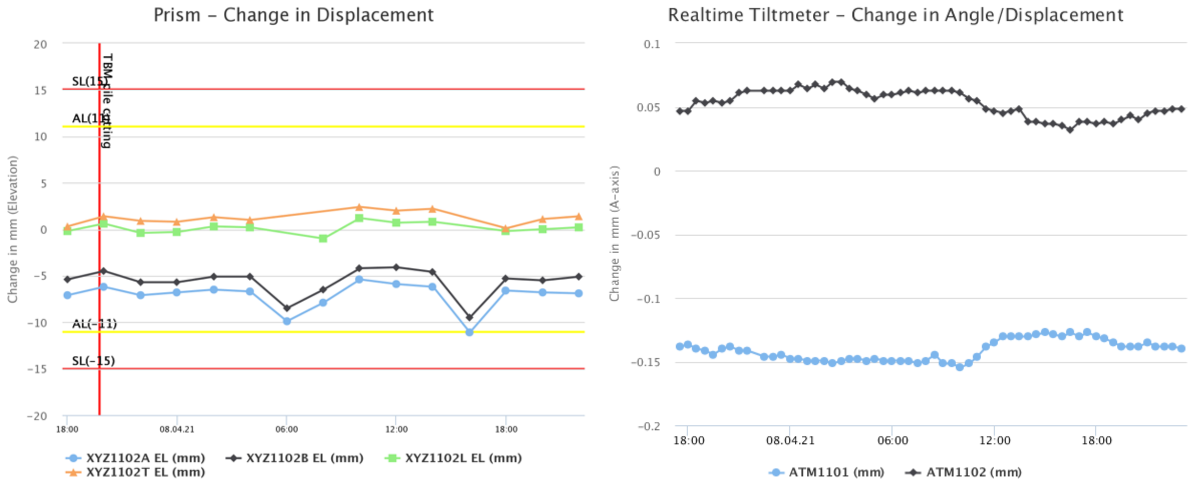

Figure 19. Plots of the real-time optical prisms (left) and tiltmeters (right) during pile cutting

3.5 Post pile cutting

After successfully cutting through the pile, CHI was carried out to replace the specialized cutter tools back to the original roller disc cutters. All the rebar fragments were cleared from the excavation chamber and examined to gauge the effectiveness of the tools in cutting the rebars into small segments. The removed specialized cutter tools were also examined for wear and refurbished where necessary. The refurbishment was done by the specialist welder who removed the damaged cutter bit and welded a new cutter bit onto the cutter tool’s housing.

3.5.1 Extracted rebars

Large number of rebar segments were accumulated at the bottom of the excavation chamber and they were removed manually during CHI to prevent the possibility of it subsequently jamming the screw conveyor. This accumulation of rebars segments in the excavation chamber was expected, as during pile cutting, the screw conveyor operated at a very low speed to maintain the face pressure. Analyzing the rebar segments’ cutting marks and the average length per segment concludes that the specialized cutter tools were effective in cutting the rebars into small segments as intended. On average, the rebar segments were less than 300mm in length. The rebars shown in Figure 20 demonstrated shearing patterns on its edges, indicating that the specialized cutter tools were effective in cutting and performed as intended. The longer rebars found were possibly caused by the cutting of the rebar ends before it could be further cut into smaller segments.

Figure 20. Photos of the rebars retrieved from the excavation chamber after cutting through the piles

3.5.2 Documenting the cutter wear

Upon removing the specialized cutter tools, it was observed that some of the tungsten carbide inserts were worn as shown in Figure 21.

Figure 21. Images of the worn off cutter tools after cutting through the piles

Replacement cutting bits shown in Figure 22, were ordered in advance as the wear in the cutting bits was anticipated. The specialized cutter tools were refurbished by replacing the worn-out cutting bits with new cutting bits by a specialist welder. This is to ensure that the subsequent pile cutting operation at pier P51 would remain as effective. The refurbishment was done after cutting through the piles of pier FP9, while was tunnelling towards the piles of pier P51.

Figure 22. Replacement specialized cutting bits

4 CONCLUSION

Following the works at pier FP9, the TBM managed to successful cut through six 350mm diameter (5T32 reinforcements) micropiles within the tunnel face at pier P51, the pile cutting operation took 3 days. The successful cutting of the piles at pier FP9 and P51 is attributed to the careful management of the TBM operational parameters and close coordination between the parties involved, coupled with the use of specialized cutter tools, therefore allowing the TBM to cut the piles effectively. The monitoring instrumentation data indicated that ground movement was kept within acceptable limits and the TBM parameters suggested that the TBM was capable of cutting the piles without much difficulty. The successfully implementation of this method of undercrossing compared to the manual method of pile hacking where workers have to enter the chamber and hack the piles manually, saved approximately 30 days off the construction program. This paper seeks to document the process of underpinning a live vehicular viaduct and highlight the promising results of employing specialized cutter tools in TBM pile cutting operations.