Tunnelling underneath a large diameter water main in Shanghai

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ABSTRACT
This paper presents a successful case of tunnelling underneath an existing 3.0 m diameter water main using an earth pressure balance (EPB) tunnel boring machine (TBM) in Shanghai, China. The water main was located 10 m below the ground surface with a clearance of 4.1 m between its invent and two new tunnels. In order to ensure the safety of this water main, a tight tunnelling construction control was implemented by maintaining the face stability and controlling spoil extraction and grouting operation. The construction was divided into four stages with different control measures implemented based on the relative position of the TBM to the water main. The water main was kept operational and safe during underneath tunnelling due to implementation of these measures.

RÉSUMÉ
Ce document présente un cas réussi du perçage d'un tunnel sous une force d'eau existante de diamètre de 3.0 m utilisant une aléseuse de tunnel de l'équilibre de pression de la terre (EPB) (TBM) à Changhaï, Chine. La force d'eau était des 10m localisés au-dessous de la surface au sol avec un dégagement de 4.1 m entre le son inventent et deux nouveaux tunnels. Afin d'assurer la sûreté de cette force d'eau, une commande serrée de construction de perçage d'un tunnel a été mise en application en maintenant la stabilité de visage et en commandant l'extraction de corrompre et en jointoyant l'opération. La construction a été divisée en quatre étapes avec différentes mesures de contrôle mises en application basées sur la position relative du TBM à la force d'eau. La force d'eau a été maintenue opérationnelle et coffre-fort pendant sous le perçage d'un tunnel dû à l'exécution de ces mesures.

1 INTRODUCTION

There is an increasing demand for tunnelling in cities for transportation, water conveyance, etc. It is challenging to ensure the safety of adjacent structures and utilities during tunnelling in these heavily congested urban areas.

A few studies have been presented in literature related to the protection of underground pipes during adjacent tunnelling (O'Rourke & Trautmann 1982; Attewell et al. 1986; Owen 1987; Vorster et al. 2005; Liao et al. 2009). The responses of underground pipes are influenced by many factors, such as tunnelling methods, their relative positions, and structural and ground conditions. Since each project has its unique features, there are currently no specific rules in dealing with these kinds of challenges. The normal protection methods for shallow pipelines include the exposure and backfill method, underpinning, grouting, etc.

This paper presents a successful case of protecting a large diameter water main while tunnelling underneath it in Shanghai. Due to the deep depth of this water main, a tight construction control method was used in this project to ensure its safety during tunnelling.

2 ENGINEERING BACKGROUND

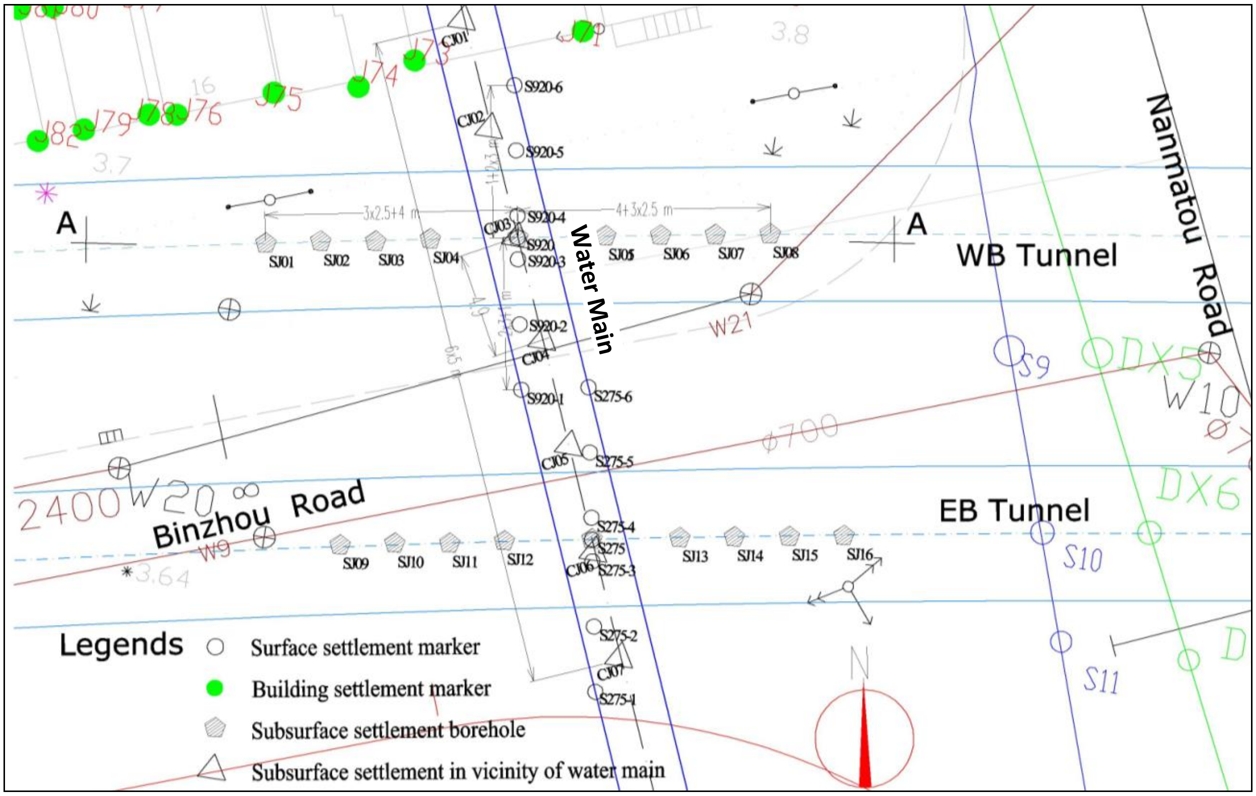
This project was for the construction of twin tunnels for Shanghai Metro Line No. 7 in Shanghai, China. The new twin tunnels were constructed using an earth pressure balance (EPB) tunnel boring machine (TBM). Each precast concrete segmental liner has an internal diameter of 5.7 m, a width of 1.2 m and a wall thickness of 0.35 m.

There is a 3.0 m diameter steel water main with a wall thickness of 24 mm located between Dongmin Road Station and South Yanggao Road Station. The water main is skewed approximately 85^{o} with twin tunnels. It is embedded approximately 10 m below the ground surface with its invert at approximately 4.1 m above the crowns of new twin tunnels. The plan and sectional view of the existing water main and new tunnels are shown in Fig. 1.

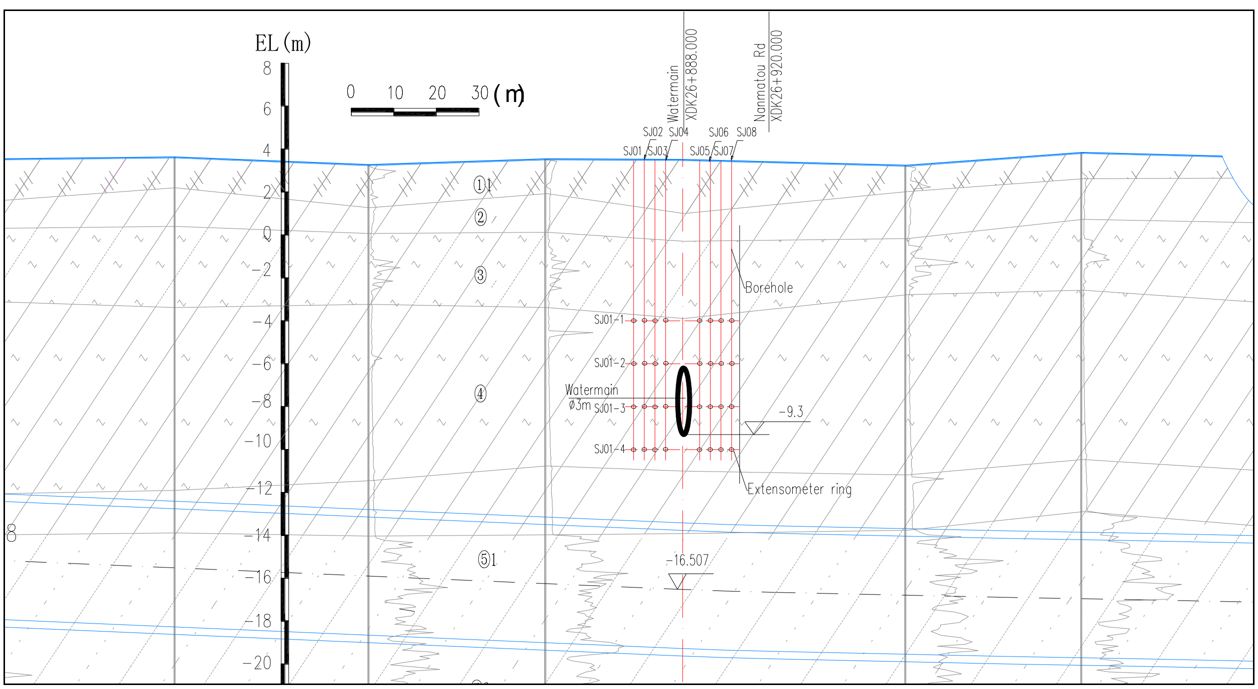
The soil at this site is the typical Shanghai soft clay. It is composed of alluvial deposits with saturated, flow to highly plastic soft clay with high compressibility and low shear strength. It consists of five main soil strata within 20 m depth from the ground surface. Layer 1 is approximately 2 m deep fill followed by Layer 2, which is 3 m deep silty clay. Layer 3 is approximately 5 m deep very soft clay overlying Layer 4, about 12 m deep clay interbedded with silts, and followed by Layer 5, more than 12 m deep silty clay or clayey silt. The groundwater is located approximately 1 m below the ground surface with an annual fluctuation of 0.5 m. Typical physical and geotechnical properties of soils at the site are shown in Table 1. This paper focuses only on the construction of western-bound (WB) tunnel drive.

3 WATER MAIN PROTECTION PLAN

An intensive monitoring program was developed in this project to ensure the safety and normal operation of this water main during tunnel construction.



a) Plan view



b) Sectional view

Figure 1. The relation between new twin tunnels and existing water main

3.1 Field Instrumentation and Monitoring

Field measurement included ground surface settlement, borehole extensometers for deep soil movement, and building settlement, as shown in Fig. 1. The surface settlement targets were installed on two sections perpendicular to the tunnel alignments in the vicinity of the water main. A total of sixteen boreholes were drilled along two tunnel alignment. The extensometer rings were embedded at depths from 8 m to 14 m below the ground surface. There were also seven borehole extensometers directly above the water main. These extensometer readings, as noted CJ01 to CJ07 in Fig. 1, were installed and managed by the water main company.

The area covering 10 m from both sides of the water main was designated as a protection zone. The initial readings were set-up for soil and the water main by averaging two readings of each instrument taken before tunnelling. When the TBM was within the protection zone, the monitoring frequency was six times per day, which could be increased to once every hour in case the settlement rate had reached to ±3 mm per day. After the TBM pass-by, monitoring was gradually reduced from six times per day to 2 times per day until the settlement rate reduced to 1 mm per day.

Table 1. Physical and geotechnical properties of soil at the site

Layer	Soil type & description	ω %	γ kN/m^3	e -	I_P -	K_V cm/s	K_H cm/s	c (kPa)	ϕ (deg)	E (MPa)
③	Muddy clay, interbedded w silt	38.3	17.9	1.07	15.7	5.0E-6	4.2E-5	13.3	10.5	27.5
④	Muddy clay	49.7	16.9	1.39	21.1	1.0E-7	1.8E-7	16.2	11.0	9.8
⑤$_1$	Silty clay, gray coloured	45.2	17.3	1.27	20.6	1.6E-7	2.3E-7	8.1	26.5	13.9
⑤$_2$	Clayey Silt	32.7	18.1	0.95	--	7.1E-5	1.2E-4	15.3	17.0	41.4
⑤$_3$	Clay	34.8	18.0	1.02	17.5	--	--	--	--	--

Note: ω-water content, γ-unit weight, e-void ratio, I_P-plasticity index, K_V-permeability in vertical direction, K_H- permeability in horizontal direction, c-cohesion, ϕ-angle of friction, E-Young's modulus

3.2 Tunnelling Construction Control

Tunnelling was controlled mainly by adjusting two construction parameters: the face support pressure and grouting. Tunnelling in the vicinity of the water main was performed in four stages: Stage 1- the cutter head moved from 20 rings away from to directly under water main, Stage 2-The TBM shield under water main to shield tail moved out of water main, Stage 3-The shield tail moved away from the water main to 5 rings away, Stage 4-The shield tail moved more than 5 rings away from water main. The details of control parameters and related ring numbers are also listed in Table 2.

3.2.1 Face support pressure control

The face pressure was controlled approximately at 380 kPa, which was based on the lateral earth pressure at-rest condition, *Ko*, in order to maintain the face stability and minimize the influence on the water main. Since the TBM would drive through two sub-layers: ⑤$_1$ – gray coloured silty clay and ⑤$_2$ – silt, silt would create high cutter resistance due to its relatively higher shear strength. At the same time, its high permeability would easily cause soil loss and then create instability in front of the TBM, which could jeopardize the safety of the water main. In order to improve tunnelling operation and provide a smooth soil extraction, foam conditioning was used to provide a better face support pressure control and enhance the face stability. The foam was used at a quantity of approximately 18~25 litres/ring during tunnelling within the protection zone.

Table 2. TBM control stages related to the water main position

Stage	Relative Position of TBM w. Water Main	Ring No.	TBM Control Parameters
1	Cutter head moved from 20 rings away from to directly under water main	EB Drive： 267～287 WB Drive：741～761	Face pressure and spoil extraction
2	Shield under water main to shield tail moved out of water main	EB Drive：287～294 WB Drive：761～768	Face pressure, spoil extraction, and annular grouting
3	Shield tail moved away from water main to 5 rings away	EB Drive：294～299 WB Drive：768～773	Annular grouting
4	Shield tail more than 5 rings away from water main	EB Drive: After Ring No. 299 WB Drive: After Ring No. 773	Secondary grouting

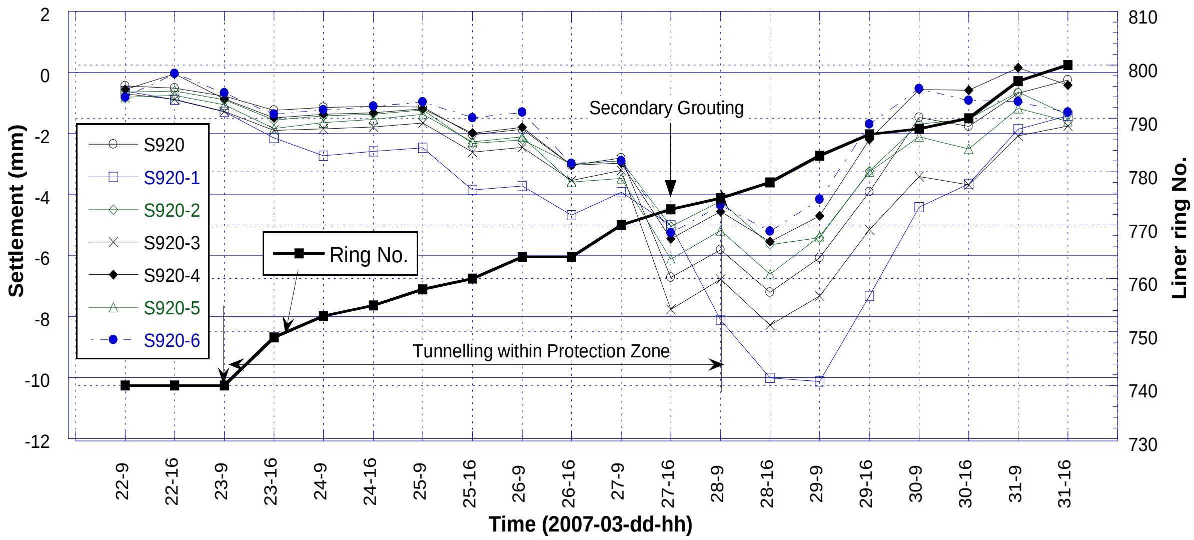


Figure 2. Surface settlement during western-bound tunnelling

3.2.2 TBM driving speed and spoil extraction

In order to minimize the disturbance due to tunnelling, an advance rate of 1~1.5 cm/min was used within the protection zone. This speed was at the lower range of the speeds measured from a testing drive. Both tunnels had a horizontal curvature of 1000 m in plan and a grade of +8.6‰ near the water main. The TBM route was controlled tightly to avoid overcutting within this area. The volume for excavation was calculated to be 37.86 m^3 for each ring construction based on the TBM face area of 31.55 m^2 and a ring width of 1.2 m. In order to provide a good face support, the soil extraction was controlled slightly less than this theoretical excavation volume. In case of the decrease of the face support pressure, the screw conveyor for soil extraction was rolled backward to compensate for the pressure loss. Within the protection zone, a higher construction speed was also implemented in addition to less thrusts being released during ring erection.

3.2.3 Annular grouting and second grouting

The time and amount of annular grouting were adjusted based on the settlement measurement. The grouting pressure was controlled at approximately 0.3 MPa with an amount of 3.3~3.6 m^3 per ring, which was about 200~220% the annulus void between the mined diameter and the external diameter for each ring. After the shield tail pass-by, the secondary grouting was conducted inside the tunnel at every three to five rings to seal water and reduce ground settlement. The chemical grout ingredients were cement: water: sodium silicate mixed at a 1.0: 0.5: 0.03 ratio by weight to provide a quick hardening and a low shrinkage.

4 GROUND MOVEMENT AROUND THE WATER MAIN DURING TUNNELLING

4.1 Surface Settlement during Tunnelling

There were seven surface settlement targets at the water main location, as shown in Fig. 1. The settlements of these targets are shown in Fig. 2 along with the construction activities. It can be found that the surface settlement accelerated significantly after the TBM pass-by. These settlements were reduced by the injection of secondary grouting.

The surface settlement curve on the cross section perpendicular to the tunnel alignment is shown in Fig. 3. An asymmetrical settlement trough was developed at the ground surface with higher settlements were measured on the south side closer the eastern-bound (EB) tunnel. It is believed that the soil disturbance due to the earlier drive of EB tunnel contributed to this asymmetrical settlement trough. The maximum ground surface was measured before the start of secondary grouting. The secondary grouting significantly reduced the ground surface settlement, as shown by two heavy lines before and after secondary grouting. An upward ground movement of 8 mm was measured due to this secondary grouting.

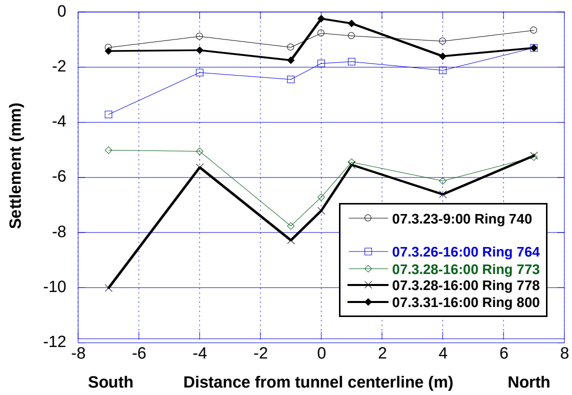
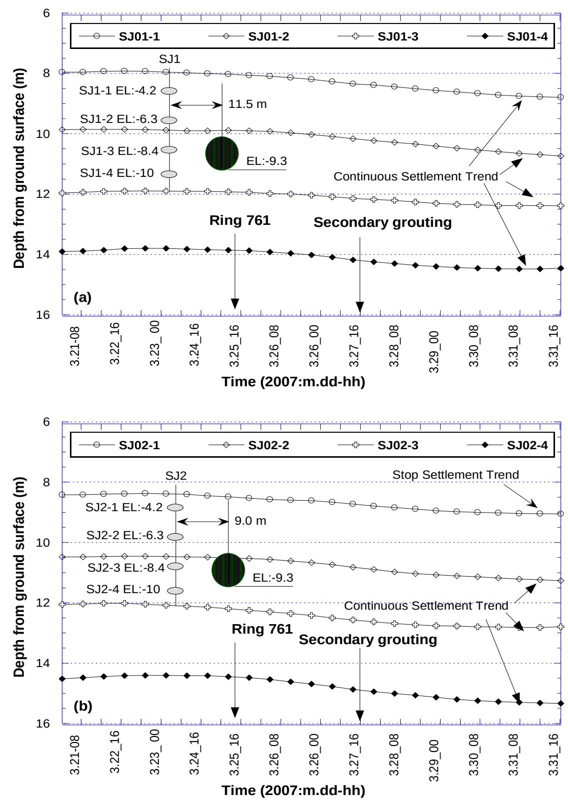


Figure 3. Surface settlement at the water main location

4.2 Subsurface Soil Settlement during Tunnelling

There were eight borehole extensometers, named from SJ01 to SJ08, installed along the WB tunnel alignment near the water main. The plan and sectional views of these boreholes are schematically shown in Fig. 1, where the extensometer rings were embedded at depths of approximately 8, 10, 12, and 14 m from the ground surface. Two boreholes, SJ07 and SJ08, were damaged during construction and only the rest six boreholes provided readings during tunnelling.

The deep soil movement are shown in Fig. 4, where the magnitudes of ground movements are multiplied by 10 to show clearly their trends. It can be found that the rates of settlement in two boreholes immediately adjacent to the water main, SJ04 and SJ05, were reduced after secondary grouting, especially the two depths above the water main elevation. i.e., the readings at two depths of approximately 8 m and 10 m showed that the settlements were stopped due to secondary grouting, while other locations did not show this upward movement trend, but most cases the settlement rates were reduced after introduction of secondary grouting. The reasons for these different responses are believed to be the poor soil compaction during trench backfilling for water main installation. In addition, the secondary grouting has more effect on soil with lower overburden pressures.



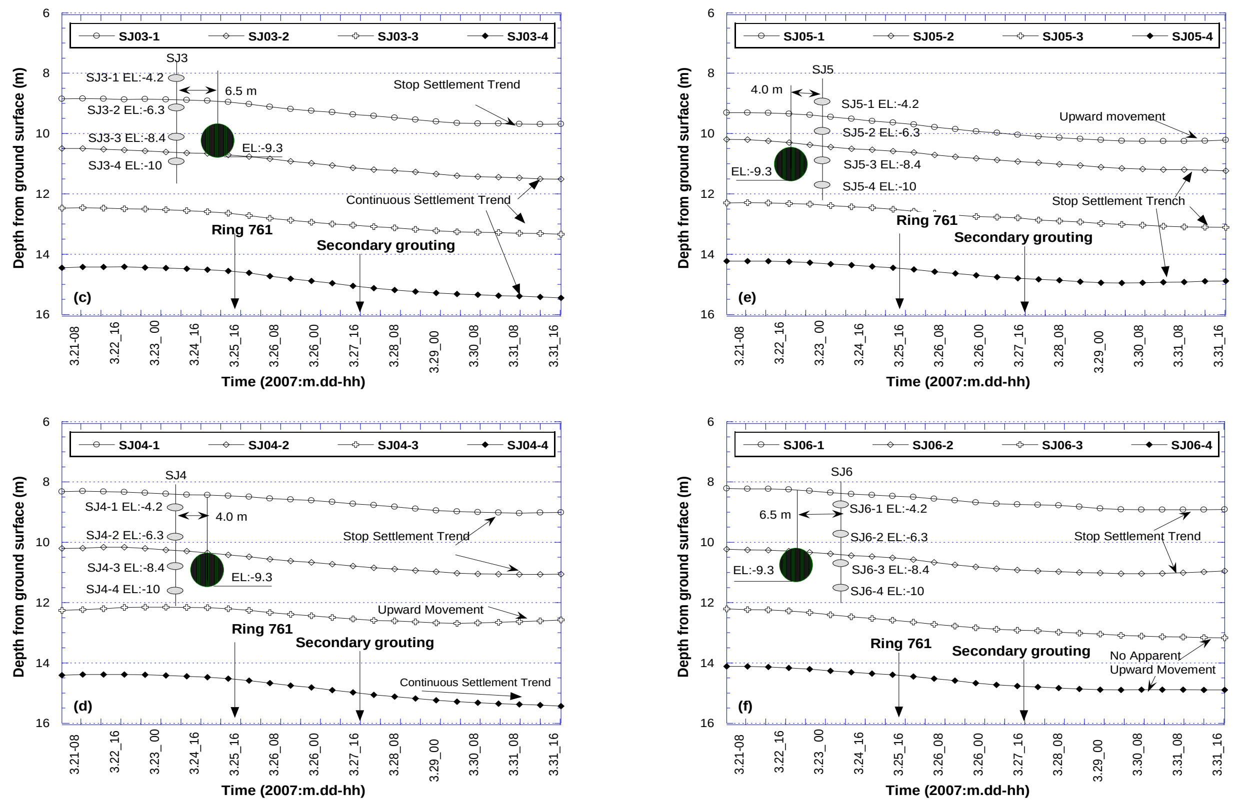


Figure 4. Subsurface soil settlement around the water main during tunnelling (Note: The magnitudes of settlement are multiplied by 10 in the figure)

4.3 Settlement of Water Main during Tunnelling

There were only limited readings available on soil movement immediately above the water main due to the private ownership of these readings. These soil settlements reflect approximately the settlement of water main itself. These settlements are shown in Fig. 5 after pass-by of TBM (Lu at el. 2007). It can be found that the settlement of the water main was approximately within 4 mm based on these measurements. The water main settled gradually after TBM pass-by due to dissipation of excess pore pressure generated mainly due to grouting.

During tunnelling, the water main was kept optional and there was no interruption reported. It can be concluded that the water main was protected successfully in this project.

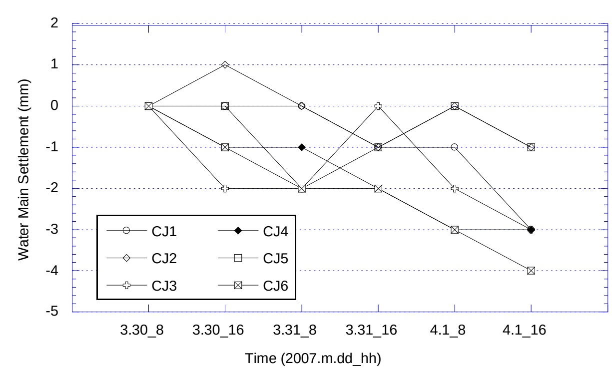


Figure 5. Soil movement in the vicinity of the water main

5 CONCLUSIONS AND DISCUSSIONS

This paper presents a successful case of an EPB tunnelling underneath an existing 3.0 m diameter water main in Shanghai. The clearance was only 4.1 m between the water main and tunnels. In order to ensure the safety of this water main, a tight tunnel construction control was implemented: In addition to using earth pressure at-rest, soil extraction was kept slightly smaller than theoretical excavation volume; the annular grouting was applied simultaneously with tunnelling with the grout volume doubling the theoretical volume of the annular void; the secondary grouting was applied after the TBM pass-by. With these measures, the water main was kept operational during underneath tunnelling. This project demonstrated that the construction control method is an efficient method for property protection during urban tunnelling.

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REFERENCES

Attewell, P. B., Yeates, J., Selby, A. R., 1986. *Soil Movements Induced by Tunnelling and Their Effects on Pipelines and Structures,* Blackie and Son Ltd., London.

Liao, S.M., Liu, J.H., Wang, R.L. Li, Z.M., 2009. Shield tunneling and environment protection in Shanghai soft ground. *Tunnelling and Underground Space Technology*, 24, 454–465.

Lu, M.J., Du, S.J., Wen, Y.J., Qiu, Z.H. 2007. Soil movement around a water main during underneath TBM tunnelling, *Proceeding of 3rd Shanghai International Symposium on Underground Construction and Risk Control*. 364-372.

O'Rourke, T. D., Trautmann, C. H., 1982. Buried pipeline response to tunnel ground movements. *Europipe 82* Conf.. Basel, Switzerland, Paper 1.

Owen, R. C. 1987. Observations of the effects of shallow tunnels on buried services. *Ground Engineering*, 20(2), 21–28.

Vorster, T. E. B., Klar, A., Soga, K., Mair, R. J., 2005. Estimating the effects of tunneling on existing pipelines. ASCE, *J. of Geotechnical and Geoenvironmental Engineering*, 131(11), 1399-1410.