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# Repairing a shield tunnel damaged by secondary grouting

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ABSTRACT

This paper reports on a repair work which has recently been conducted for a metro tunnel in Hefei city, China. The tunnel has been originally constructed using shield method where synchronous grouting was used to fill the gaps between the tunnel segments and soil. Following a regular maintenance inspection of the tunnel, several leakage issues were identified between three stations. Secondary grouting was adopted as a solution to block the tunnel leakage, however, shortly after the start of grouting work, the track and track bed were found to be unevenly uplifted with significant cracks in the tunnel's segments. The paper describes and discusses key aspects of this case study including ground conditions, leakages patterns of the tunnel, recorded volumes and injection pressure of the secondary grouting, as well as survey data of track displacement and segment cracks. The investigation confirmed that the situation was caused by an inappropriate implementation of the secondary grouting, particularly by high grouting pressure (significantly higher than the geostatic pressure), large volumes of injected grout, and poor selection of grouting locations. Ground Penetrating Radar (GPR) was conducted to inspect the tunnel conditions before commencing the structural repair work, which revealed that there were no voids under the track bed of the affected zone. The study presents simplified strategies used to repair the damage while maintaining minimum disturbance to the affected segments.

## 1. Introduction

As a safe, fast, efficient and environmentally friendly form of transportation, underground trains (metro) have quickly become the first choice for many large cities around the world to solve their traffic congestion. In China, more than 40 cities have either carried out or are planning to construct metro tunnels. The length of urban metros is expected to reach 6000 km by 2020 (He et al., 2015), covering the major cities of China.

Over the past decades, the tunnelling industry has developed a series of construction methods addressing various geological, hydrological, technical, and economic challenges. These methods have been developed from the traditional single open-cut and cut-and-cover to more advanced techniques such as the shield method (Guo and Wan, 2004). The shield method has been widely used in urban areas because it has little disturbance to the surrounding environment, ensures rapid construction and is better adapted to water-rich soft stratum. In the process of shield tunnelling, concrete segments are gradually assembled while leaving the shield tail. However, as the shield shell inner diameter is greater than the outer diameter of the segment lining, gaps inevitably emerge between the segments and soil. If these gaps are not filled in time, stress release occurs in the soil around the segments leading to ground surface subsidence. For any tunnelling technique, it is essential to avoid ground subsidence and water leakage during and after construction. In shield tunnels, these risks are managed by grouting (Tang et al., 2016).

Grouting is generally subdivided into different types depending on its function, implementation time and location. If cement slurry is injected from a grouting pipe at the shield tail to the outer wall of the segment, it is called “grouting at shield tail” (Ye, 2007). Alternatively, cement slurry might be injected from grouting holes within segments, into the outer wall after the shield has advanced some distance; this is known as “grouting through segments” (Ye, 2007). The two types of grouting, explained above, are synchronous with shield tunnelling thus are categorised as “synchronous grouting”. Synchronous grouting fills the gaps between soil and segment, which can help to alleviate soil stratum deformation, ensure uniform stress of the lining, improve the impermeability of shield tunnel lining, fix the position of the segment lining, and transfer the loads of the tunnel and the other auxiliary facilities to the foundation soil (Ye et al., 2015).

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According to the Chinse technical code (Ministry of Construction of the PRC, 1999), the volume of synchronous grouting should be 130–180% of the volume of gap generated between soil and segments during shield tunnelling. However, in reality, ground subsidence can not be restrained completely even if synchronous grouting volume reached 180% of the gap size. This is because synchronous grouting slurry cannot completely, fill the gap between the segments and soil and some of the slurries are infiltrated and lost if the soil has a large coefficient of permeability. In addition, synchronous grouting experiences 1.4% volume shrinkage during solidification (Zheng, 2015). Therefore, a gap or void would still exist between segments and soil after synchronous grouting work. As a consequence, water seepage may occur in the tunnel through the voids and crack zone, or further settlement of ground may take place due to the existence of voids (Wu et al., 2013; Chi, 2015; Li et al., 2016). Thus, there is a need to fill the remaining voids by further grouting. Such grouting is carried out after the completion of construction and is known as "secondary grouting".

Previous research conducted on grouting of shield tunnels (Komiya et al., 2001; Kasper and Meschke, 2006a; Stille and Gustafson, 2010; Butrón et al., 2010; Yong and Breitenbücher, 2014) has established that the selection and control of grouting pressure are key for effective and successful implementation of almost all grouting types. If the grouting pressure is significantly large in comparison with in-situ ground stresses, segments can be uplifted, dislocated, cracked, crushed or destroyed, and grouting slurry might flow inside the tunnel. If the grouting pressure is too small, a gap or void may remain unfilled and thus ground surface subsidence can occur. Therefore, grouting pressure must be chosen properly during shield tunnelling (Li et al., 2006; Gou, 2013; Kasper and Meschke, 2006b).

This paper reports on the unsuccessful application of secondary grouting which was used in a shield tunnel in Hefei city, capital of Anhui province in China in December 2015. The secondary grouting was selected to treat leakage issues. However, the grouting work led to track, and track bed uplift with several cracks appearing on tunnel segments. The paper describes and discusses key aspects of this case study including ground conditions, leakages patterns of the tunnel, records of injection pressure and volume of the secondary grouting, as well as survey data of track displacement and segment cracks. The investigation revealed that injection pressure and volume of the grouting were too high and applied asymmetrically, which induced uneven uplift of the track and track bed as well as significant cracks in the tunnel segments. Ground Penetrating Radar (GPR) was conducted to inspect the tunnel conditions to inform the repair work. The study presents a simplified approach to repair the structural damage and leakage, with minimum disturbance to the affected segments. The outcome of this work can be used as a case study by the tunnel engineers who are involved with similar maintenance work for shield tunnels.

## 2. General description of the tunnel construction

Metro line 1 of Hefei city is a key route connecting the north and south of the city, with a total length of 29.06 km and 26 stations. Part of this line, the focus of this study, extends over three stations: Yungu road station, Nanning road station and Guiyang road station. The line between these stations consists of two parallel circular tunnels with a spacing of 15.0 m, as shown in Fig. 1, and was built using the shield construction method.

This tunnel was excavated by a ZTE6250 Earth Pressure Balance (EPB) shield machine, with a diameter of cutter head of 6280 mm, an opening rate of cutter head about 45%, and a driving torque 5700 kN·m. The whole length and weight of the EPB shield machine are about 85 m and 450 tonnes respectively. The maximum total thrust of the propulsion system is 42,575 kN, and the cylinder stroke is 2100 mm. The speed of shield tunnelling is kept as constant as possible at about 15–20 mm/min. The shield lining is stagger-jointed assembled with six blocks of C50 reinforced concrete segments, including three standard blocks, two adjacent blocks and a seal roof block in each ring. The segment has an inner diameter of 5400 mm, outer diameter 6000 mm, thickness 300 mm and width of 1.50 m. Segments are connected with each other by bolts, using 16 ring joint bolts of M27 (i.e. 27 mm diameter) and 12 longitudinal connecting bolts (M27) in each ring. The tunnel section is shown in Fig. 2, and a photograph of the tunnel is shown in Fig. 3.

In order to ensure quality implementation of synchronous grouting during the shield tunnelling, the contractor carried out standard testing on the slurry used for the grouting work. The testing included slurry composition, mixture proportion, slurry mixing process, and slurry properties. The slurry components and mixture proportion of the synchronous grouting are presented in Table 1, as extracted from as-built documents of the tunnel. During the slurry mixing, the following are recorded: mixture ratio, slurry consistency and slurry volume used for each ring of the segments.

## 3. Geological and geotechnical conditions

The existing geological information indicates there is an engineering fill at the ground surface, underlain by Quaternary sediments and Cretaceous sandy mudstone extending to the maximum depth of investigation (40 m below ground surface). The working face of the shield is mainly located in a clay layer within the Quaternary sediments; the clay is coded as "3" (Fig. 4). This clay is described as pale yellow or brown, hard plastic, medium compressibility, smooth and glossy cut surface, high dry strength containing iron-manganese concretions, It has a bearing capacity of 240 kPa according to the engineering geological survey report. The physical and mechanical parameters for each soil layer (extracted from the report) are summarised in Table 2, where $\gamma$ is the unit weight of soil (kN/m$^3$), $E_{s1-2}$ is the compression modulus (MPa), $C_u$ is the undrained cohesion (kPa), $K_0$ is the lateral pressure coefficient of soil at rest, $k_v$ and $k_h$ are the vertical and horizontal coefficient of permeability (m/s), respectively.

Expansive soil is one of the characteristics of ground conditions in Hefei city. This type of soil exhibits a significant volume change (shrinking and swelling) when subjected to change in water content. In the proximity of tunnels, this volumetric change can produce excessive stresses on underground structures leading to cracks. In the process of shield tunnelling, the excavated soil is expected to lose some of its water content at locations exposed to the construction working face, while water content increases in the soil located behind the tunnel segments due to water dissipated from grouting material. An expansive soil would shrink and repeatedly swell during shield tunnelling, which impacts shield segments (Li, 2014). Therefore, it is necessary to determine the relevant engineering properties of such soils to assess the effect of swell-shrink behaviour on the tunnel. The expansiveness indices of each soil layer are presented in Table 3, where$\delta_{ef}$ is the free swelling ratio (%), $\delta_{e50}$ is the swelling ratio (%), $\lambda_s$ is coefficient of shrinkage, $P_e$ is the swelling force (kPa).

During the ground investigation, perched groundwater was detected within the first layer, with a maximum depth of 5.3 m below ground surface. The perched water is mainly recharged by atmospheric precipitation and paddy field irrigation and discharged by evaporation at the surface. Although this groundwater is located above the shield working face, it is believed to have no direct influence on the shield tunnelling construction. This is because the coefficient of permeability of the ground is very small (see Table 2), which can prevent any significant water seepage into the working face within the short time-scale of the construction. However, after the completion of tunnel construction, long-term seepage might continue toward voids around the tunnel, increasing the water pressure with time and thus causing the reported leakage problem. The existence of voids was confirmed by the pre-repair inspection work carried out for the tunnel as presented in Section 7.