

Geotechnical Infrastructure in Taipei Metropolis

Infrastructure géotechnique à Taipei Metropolis

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ABSTRACT: Rapid development of infrastructure in the past 50 years or so has drastically improved the quality of life in Taipei metropolis. Construction with significant geotechnical contents started with the sewage system and 2 open type shield machines were used for the first time in 1976, followed by the Railway Underground Project, Metro System, High Speed Rail, and lastly the Taoyuan International Airport Access MRT. The metro system is of particular interest to geotechnical engineers because the excavations were very deep and the presence of a highly permeable water-rich gravel layer frequently led to failures. The soft nature of the young deposits also presented serious challenges. Described in this paper are a few innovative construction techniques to deal with difficult situations.

RÉSUMÉ (in French) - Le développement rapide de l'infrastructure au cours des 50 dernières années a considérablement amélioré la qualité de vie dans la métropole de Taipei. Les travaux de construction avec un contenu géotechnique important ont commencé avec le système d'égout et 2 écrans ouverts ont été utilisés pour la première fois en 1976, suivis du projet ferroviaire souterrain, du système de métro, du train à grande vitesse et enfin de l'aéroport international de Taoyuan. Le métro est particulièrement intéressant pour les ingénieurs géotechniciens car les fouilles étaient très profondes et la présence d'une couche de gravier très perméable et riche en eau a souvent conduit à des échecs. La douceur des jeunes dépôts pose également de sérieux défis. Décrit dans cet article sont quelques techniques de construction innovantes pour faire face à des situations difficiles.

KEYWORDS: Taipei Basin, Underground Construction, Deep Excavation, Dewatering, ESA Tunneling, DOT Shield Machine, NATM

1 INTRODUCTION

Taipei Metropolis is located at the northern tip of the Taiwan Island as depicted in **Figure 1**. It includes Taipei City, New Taipei City and Jilong City with a total area of 2,457 km² and a population of 7 million. Taipei City is the capital of the country and as such it is the administration and business centre. Major construction with significant geotechnical contents in the last 50 years included:

- (1) Sewage system: shield tunneling (open-type shield machines were used for the first time in 1976)
- (2) Freeways: stability of cut slope, ground anchors and piles, cofferdam for constructing bridge piers
- (3) Underpass below airport runway and taxiway: Endless-Self-Advancing (ESA) tunneling
- (4) Railway Underground Project: deep excavations
- (5) Metro System: deep excavations and dewatering, shield tunneling, NATM tunneling in soft ground
- (6) High Speed Rail: deep excavations and rock tunneling
- (7) Taoyuan International Airport Access MRT: Double-O-Tube (DOT) tunneling under river bed, underpinning the pier of a viaduct
- (8) Taipei Port: reclamation and ground improvement

The metro system is of particular interest because the excavations were very deep (up to 40m in depth) and the presence of a highly permeable water-rich gravel layer frequently led to failures. Discussed herein are some issues important to geotechnical engineers and a few benchmark events of underground construction.

2 GEOLOGY OF THE TAIPEI BASIN

Located at the centre of the metropolis is the Taipei Basin which is densely populated with infrastructure well developed. The Taipei Basin was formed as a result of ground subsidence due to tectonic movements along the Shanjiao Fault along the western perimeter of the basin in the Middle Pleistocene Epoch. As can be noted from the schematic profile shown in **Figure 2**,

the young sediments in the basin include, from top to bottom, the Songshan Formation, Jingmei Formation, Wugu Formation and the Banqiao Formation (Teng, et al. 1999). The deepest depth to the base formation, i.e., the deformed Tertiary Strata, revealed by the Central Geology Survey is 679m.



Figure 1. Satellite Image of Taipei Metropolis

What is of interest to geotechnical engineers is the fact that the Jingmei Formation is a very permeable water-rich gravelly stratum. Excessive pumping back to the 1970's to provide sufficient water for domestic use drastically lowered the piezometric level in the Jingmei Formation, leading to the reduction of porewater pressures in the Songshan Formation. Underground constructions in the early days were thus

benefited by the lower groundwater pressures and increases in soil strengths in the Songshan Formation as a result of consolidation. However, pumping was banned in late 1970's and the piezometric level in the Jingmei Formation recovered quickly. Quite a few disastrous failures occurred in the 1990's as the water from the Jingmei Formation suddenly discharged into pits during excavations in the Stage 1 construction of the metro system. Such accidents occurred most frequently at the junctions of station boxes and tunnels as openings were made on the station boxes for the shield machines to pass through. Learning from these failures, the designers and contractors became more cautious and no significant events occurred in the Phase 2 construction in the last 15 years.

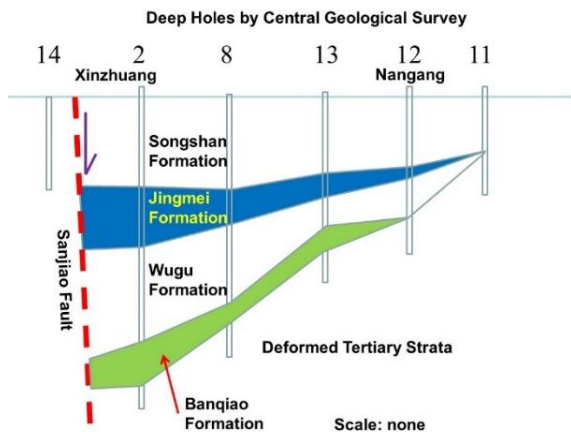


Figure 2. Schematic geological profile of the Taipei Basin

3 DEEPEST EXCAVATION

Figure 3 shows the deepest excavation carried out in the Taipei Basin. It was carried out to a depth of 40m below ground surface at the west bank of the Tamsui River for constructing a turnout in the Zhonghe-Xinliu Line (Orange Line) in Phase 2 metro construction. The pit was retained by diaphragm walls of 1.5m in thickness and 63m in length.

Because of the absence of a continuous clayey layer below the formation level, it was necessary to form an impervious slab at the toe level of the diaphragm walls to seal off seepage from the underlying Jingmei Formation and to resist water pressures at the bottom. This slab, 5m in thickness, was installed by using high-pressure jet grouting. Even with this grouted slab, pumping was still required to lower the piezometric pressures in the Jingmei Gravel by 13m or so for maintaining a sufficient factor of safety against uplift.

The entire turnout is 126m long and was partitioned into 5 sections by internal diaphragm walls. Excavations were carried out, from the east toward the west, section by section. This

reduced the maximum pumping rate to 3000 m³/hour (DORTS 2010).

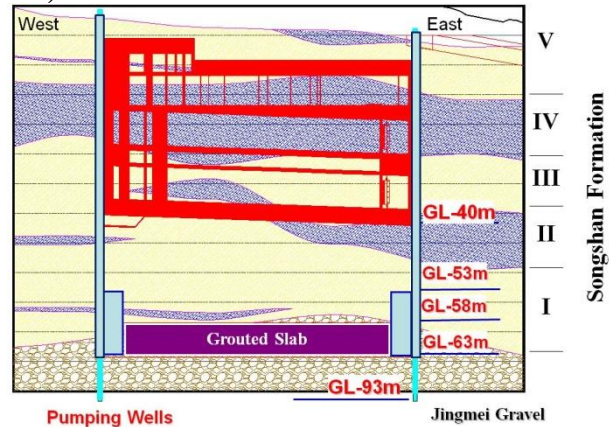


Figure 3. Excavation carried out at the Turnout

4 LARGEST DEWATERING SCHEME

Taoyuan International Airport Access (TIAA) MRT System, refer to Figure 4, connects Taoyuan International Airport to Taipei Station Special District which serving as a transportation hub providing transfer services for Taiwan Railways, Taipei Metro, High Speed Rail, TIAA MRT and city buses. As such, A1 Station of TIAA MRT is a gateway to Taipei Metropolis. The station is a 4-level underground complex with two skyscrapers, i.e., C1 Tower (56 stories for a height of 241.5m) and D1 Tower (76 stories for a height of 320.7m) on top. For C1 Tower, the excavation was carried out to EL. -23.2m (or a depth of 27.2m below ground level) and pumping was necessary to lower the piezometric level in the Jingmei Gravels to EL. -16m (or a drop of 13m) to maintain the stability of the base of the excavation against piping and uplift. For D1 Tower, although excavation was carried out to the same depth as C1 Tower, it was necessary to lower the piezometric level to EL. -22m (or a drop of 19m) because the overlying aquitard was thinner. As excavation was carried out at its bottom depth, 18 wells were used and the maximum pumping rate reached 7,000 m³/hr as depicted in Figures 5 (Yang et al. 2010; Moh and Hwang 2015). This pumping rate was unprecedented and is still being held as the record high in the Taipei Basin. It could well be the record on the entire island.

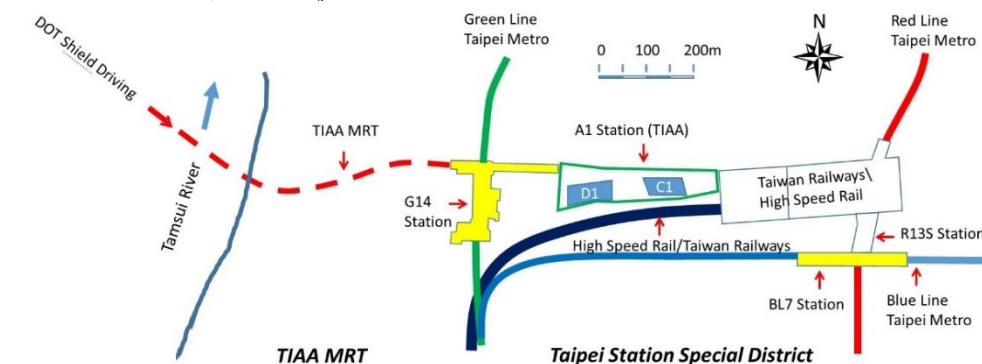


Figure 4. DOT shield tunnelling for TIAA MRT and Taipei Station Special District (Moh and Hwang, 2015)

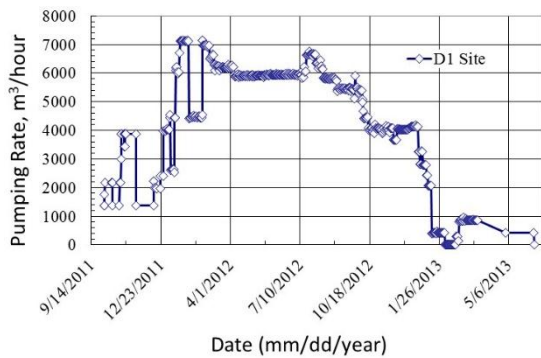


Figure 5. Number of pumping wells operated at D1 site

5 NATM TUNNELLING IN SOFT GROUND

A 222m section the Xindian Line (Green Line), as depicted in Figure 6, was driven by using the so-called NATM tunnelling method (or the shotcrete method), as a pilot project to investigate the feasibility of the method in soft ground (Yang et al., 1997). Figure 7 shows a typical cross section of the tunnels. The tunnels were bored through the Sungshan Formation, which is composed of soft silty clay and loose silty sand with N values, typically, varying from 5 to 12. The crowns of the tunnels lie at depths of 14m to 17m below ground surface (Yang et al. 1997; Moh et al. 2002).

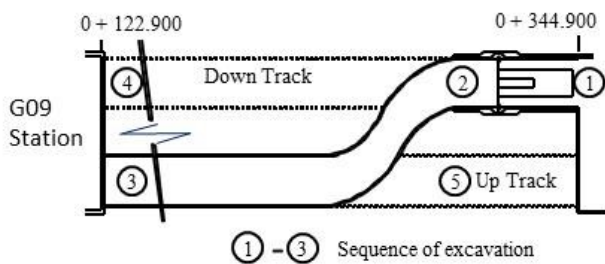


Figure 6. Plan of NATM tunnels in Contract CH221

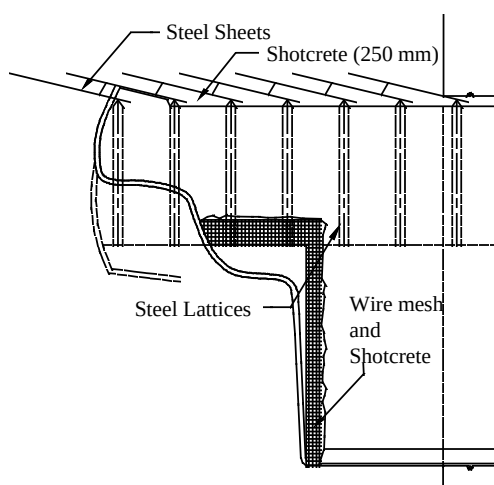


Figure 7. Profile for NATM tunnels in Contract CH221

Excavation was carried out in two headings in each tunnel drive. The upper heading was kept at a distance of 2 m to 4 m ahead of the lower heading. Lattice girders were installed at 1m

intervals and the tunnels were protected by shotcrete, 250 mm in thickness, and wire mesh as primary lining. For maintaining the stability of the headings, steel lagging sheets, 6 mm in thickness, 200 mm to 300 mm in width and 2 m in length, were closely spaced to make a canopy. The tunnels were finally lined by 350mm reinforced concrete as permanent lining.

The excavation itself was successful. However, a scary incident occurred as a transformer malfunctioned and disrupted the electrical supply. Air pressure dropped to 0.2 bar in 12 hours before the transformer was replaced and the electrical supply was restored. At that time, refer to Figure 7, Stage 3 excavations had already been completed and the heading was at Ch 0+180 of the Down-Track tunnel. In other words, there was a length of 352m of tunnel already been driven. Although the primary lining was designed to hold the tunnels even without compressed air, the face did rely on compressed air to stand up. A 2000 mm diameter water main runs across the two tunnels supplies water to the entire Taipei City. Should it rupture, the consequence would be disastrous. Fortunately, the face had been stabilized by grouting for reducing the loss of compressed air. The contractor was able to replace the transformer promptly and the crisis was resolved without even minor damages.

The soft ground called for the use of compressed air to a maximum pressure of 1.35 bar. Construction was carried out in such a way that the two tunnels were inter-connected, as shown in Figure 6, by a cross drift so that both tunnels were able to be pressurized by using a single set of compressed air facility. Also shown in the figure is the sequence of excavation. Excavation was carried out in five stages. Phase 1 excavation was carried out in free air for providing a space to house the compressed air plant. The rest of excavation was carried out in compressed air. Air pressure was not released till both tunnels were fully excavated and primary lining was completed.

6 DOUBLE-O-TUBE SHIELD TUNNELLING

The construction of the section between the first two stations, i.e., Taipei Main Station (A1 Station) and Sanchung Station (A2 Station) of TIAA MRT had to pass under the Tamsui River, refer to Figure 8. To avoid the high risk of constructing crosspassages at shallow depth under the river, a Double-O-Tube (DOT) shield machine, as shown in Figure 9, was adopted for the very first time in Taiwan. Tunneling started from the launching shaft located on the west bank of the Tamsui River and terminated at the western end of the cut-and-cover tunnel box which was integrated into G14 Station of Taipei Metro, refer to Figure 4, and constructed as a part of the station box (Moh and Hwang 2015).



Figure 8. Tamsui River Crossing, Taoyuan Airport Access MRT

Within a short distance from the launching shaft, the alignment swings from the west side of Huan Ho Nan Road (Riverside South Expressway) to the east side of the expressway and the shield machine had to go underneath one of the piers, i.e., Pier P64, supporting the viaduct of the expressway. The 9 bored cast-in-place piles, 1.5m in diameter and 60m in length, supporting the pier had to be removed to make way for the machine to pass and the pier had to be underpinned as depicted in Figure 10.



Figure 9. Double-O-Tube shield machine and concrete linings



Figure 10 Underpinning P64 in progress

7 TUNNELLING BENEATH RUNWAY AND TAXIWAY

To ease the traffic congestion in the northern part of the Taipei City, an underpass was constructed underneath the field of the Taipei International Airport which is a busy airport serving both civilian flights and military planes. As shown in Figure 11, the underpass has to pass underneath the runway with a very thin cover of only 5.6m in thickness above its roof.

As shown in Figure 12, the underpass has 2 lanes in each direction and the twin-cell box is 22.20m in width and 7.80m in height. The requirement that the air traffic shall be maintained all the time eliminates the possibility of using the cut-and-cover construction method in the sections where the taxiway and the runway are present. In fact, because construction activities above the surface are limited to the period between 11 pm to 5 am within the entire boundary of the airport, the cut-and-cover method is used only for constructing five working shafts which are absolutely necessary for the work to be done.

To meet the stringent settlement requirements, steel piles were installed from working shafts to provide a shelter to enclose the soil to be removed. Reinforced concrete segments were cast in the launching shaft and were driven one by one toward the receiving shaft by using the so-called Endless Self Advancing (ESA) Method (Hsiung 1997). The cutter was partitioned into 24 compartment to maintain the face stability and the soil at the

face was excavated from these compartments. Jacks were provided between segments to provide thrust to push the segments forward. Each time only one segment was moved so the thrust required was minimal. The underpass was successfully completed without interrupting air traffic (Hsiung 1997; Moh et al. 1999).

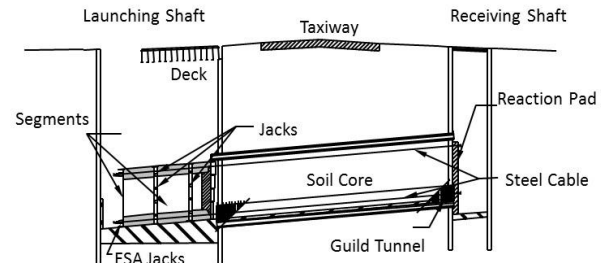


Figure 11. ESA Method of tunnel driving

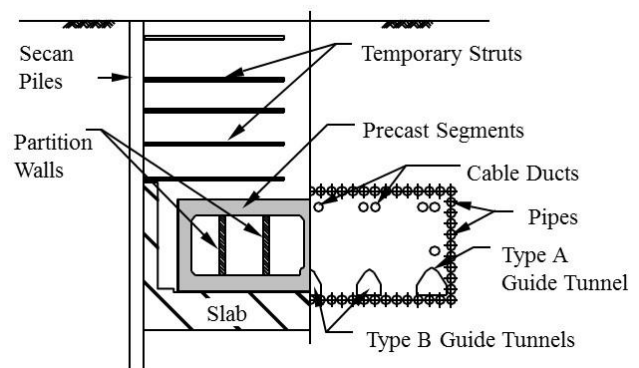


Figure 12. Section of the launching shaft

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St. Isaac's Cathedral: Behavior of a Historical Monument in St. Petersburg (Russia)

La Cathédrale St Isaac: Comportement d'un Monument Historique a St. Petersburg (Russie)

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ABSTRACT: St Isaac's Cathedral in St. Petersburg was completed in 1858 after 40 years of construction; it is today the fourth largest domed Cathedral in Europe. The soil is a relatively soft saturated sediment and carries this 3138 MN structure which is 100 meters high with an imprint of 92 by 102 m. It is founded on a 7.5 m thick mat of granite and limestone blocks resting on relatively short timber piles of different lengths. The Cathedral has progressively experienced significant deformation including differential settlement causing cracks in the pillars and tilting of the porticoes. The paper summarizes the geotechnical engineering aspects of the soil on which the Cathedral is built as well as classical issues such as foundation ultimate capacity and settlement analysis through simple calculations. The paper concludes by presenting the measurements made over time on the deformation of the Cathedral.

RÉSUMÉ : La Cathédrale Saint-Isaac à Saint-Petersbourg a été achevée en 1858 après 40 ans de construction; C'est aujourd'hui la quatrième plus grande cathédrale en dôme en Europe. Le sol est un sédiment saturé relativement mou et porte cette structure de 3138 MN qui est de 100 mètres de haut avec une empreinte au sol de 92 par 102 m. Elle est fondée sur un radier de 7,5 m d'épaisseur fait de blocs de granit et de blocs de calcaire reposant sur des pieux en bois relativement courts et de différentes longueurs. La Cathédrale a progressivement connu des déformations importantes, y compris des tassements différentiels causant des fissures dans les piliers et l'inclinaison des portiques. L'article résume les aspects d'ingénierie géotechnique du sol sur lequel la cathédrale est construite aussi bien que des questions classiques telles que la capacité de portance ultime de la fondation et l'analyse de tassement par des calculs simples. L'article conclut en présentant les mesures faites au fil du temps sur la déformation de la cathédrale.

KEYWORDS: Monuments, St. Isaac's Cathedral, Engineering Properties of Soil, Settlement Analysis, Bearing Capacity.

1 INTRODUCTION

The historical monuments in downtown St. Petersburg including St. Isaac's Cathedral are listed as World Cultural Heritage and have been protected by UNESCO since 1990. The current St. Isaac's Cathedral is the fourth church at this site dedicated to St. Isaac. It was built in 1858 and has a Greek cross ground plan with a large central dome and four smaller domes at each corner. The final engineer in charge was August de Montferrand.

The purpose of this paper is to analyze the behavior of St. Isaac's Cathedral based on existing historical documents, on site investigations performed in 1954 and 2009 including in-situ and laboratory tests, on settlement calculations and observations.

2. ST. ISAAC'S CATHEDRAL CONSTRUCTION, DIMENSIONS, AND LOADINGS

The construction of the current cathedral started in 1818 by Montferrand. The new design preserved the foundation of then previous cathedral under the western section. Construction of the Cathedral was completed in 1841, but the internal decoration took another 17 years. St. Isaac Cathedral was consecrated in 1858.

The foundation of the Cathedral posed serious problems, as the soil was very soft and difficult to build upon. Construction of the new foundation started by a 5-m deep excavation. The profile of the structure is shown in the figure 1. Then pine timber piles 0.26 m in diameter and 6.4 m to 10.7 m in length were driven. Montferrand decided to use the 13000 existing piles from the previous Cathedral built by Rinaldi which were

0.26 m in diameter, 10.5 m long under the columns and 8.4 m long elsewhere. The total number of piles used for the construction of St. Isaac Cathedral was about 24000. The bottom of the piles is in the saturated postglacial marine and lake sand and silt as well as the glacial lake clay but the piles act as friction piles as the end bearing is small compared to the friction capacity.

The pile cap consisted of a full size mat made of massive granite blocks under the most heavily loaded elements, with limestone masonry for the remainder of the foundation. The mat was 7.5 m thick and rose 2.5 m above the ground level. In the 7.5 m thick mat, 2.5 m by 2.5 m galleries were constructed. The total height of St. Isaac Cathedral is 100 m. The overall length of the Cathedral with porticoes is 102 m, and its width is 92 m. The total weight including the 980 MN for the massive foundation is 3138 MN, and makes St. Isaac Cathedral the heaviest building in Saint Petersburg. The area of the mat is 9,384 m². The average pressure under the mat is therefore 341 kPa. The weight of soil removed for the 5 m deep excavation using a unit weight of soil equals to 18 kN/m³ is 751 MN and the corresponding pressure is 90 kPa. Therefore, the net pressure for the Cathedral is 251 kPa.

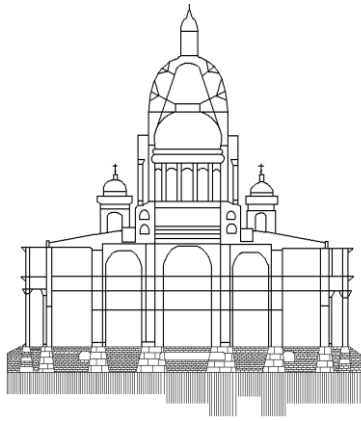


Figure 1. Profile of St. Isaac's Cathedral structure.

3. SOIL PROPERTIES AND GEOLOGICAL-ENGINEERING PROCESSES

3.1 Soil properties profiles

Soil properties were obtained in 1954 and 2009 and some properties are combined in the figures presented. The natural water content for one of the boring is presented in Fig. 3a. It indicates that there is no clear trend in the evolution of the water content with time and that the largest water contents occur in the layer of silty sand with peat at a depth about 3-5 meters and in the varved clay at the depth of 15-20 meters. Fig. 3b shows the plasticity index.

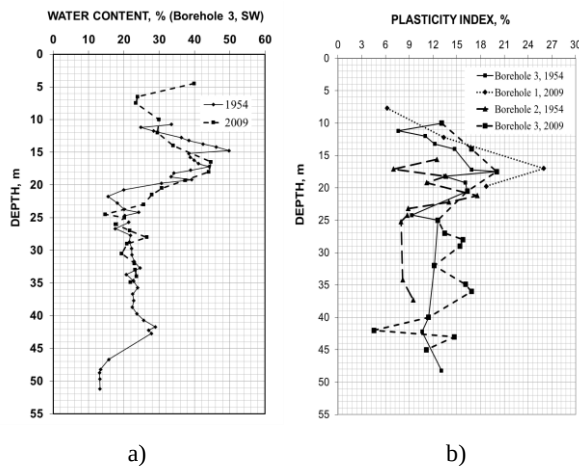


Figure 3. Profile of the natural water content versus depth (3a) and profile of plasticity index versus depth (3b)

Falling head permeameter tests were performed in 2009 on the silty sand samples and gave relatively low hydraulic conductivity varying from 1.1×10^{-5} to 6.8×10^{-5} cm/sec. Permeability tests were also performed on the silty sand after oven drying it at 105°C ; the permeability of the dry silty sand increases to 1×10^{-3} to 1×10^{-4} cm/sec. This likely occurred because the drying process burnt some of the organic matter thereby increasing the size of the pores.

3.2 Soil strength and modulus

In 2009, unconsolidated undrained triaxial tests were performed. The undrained shear strength s_u of the very soft varved glacial lake clay underlying the pile foundation has an s_u values between 18 and 20 kPa. The glacial clayey soil below the

varved clay is a bit stronger with values in the range of 25 to 72 kPa. The Vendian clay below that glacial soil has s_u values which vary with depth from 18 kPa at the top of the clay to 125 kPa. A modulus of deformation of the soil was calculated from the triaxial test results. The modulus E reaches 5000 kPa in the top layer of the lacustrine marine soil, becomes much lower in the glacial lake clayey soil with values between 290 to 1000 kPa, and then increases slightly in the moraine from 415 to 2000 kPa.

4. ULTIMATE BEARING CAPACITY OF THE FOUNDATION

The ultimate bearing capacity p_u of a square shallow foundation on clayey soil under undrained conditions can be calculated using Skempton's equation

$$p_u = N_c s_u + \gamma D \quad (1)$$

Where N_c = bearing capacity factor depending on the foundation shape and the relative embedment depth; s_u = undrained shear strength; γ = total unit weight of the soil within the embedment depth; and D = embedment depth. The relative embedment depth for the mat is $5/92=0.054$ and the corresponding value of $N_c = 6.3$ (Skempton, 1951). The overburden pressure γD is taken as $18 \times 5 = 90$ kPa. With an s_u value of 30 kPa for the clayey lacustrine marine soil the ultimate bearing capacity is 279 kPa.

Note that the pressure immediately under the Cathedral foundation is 341 kPa and the net bearing pressure or the increase in stress on the soil is 251 kPa as mentioned earlier. So the soil beneath the cathedral is near failure.

5. SETTLEMENT CALCULATION

Settlement calculations start by defining the depth of influence H_c which is calculated using two approaches. The approach in the United States is to use the depth at which the increase in total vertical stress due to the weight of the structure is equal to 10% of the pressure under the foundation (Briaud, 2013). The depth of influence according to the Russian approach is the depth H_c at which the increase in vertical total stress due to the weight of the structure ($\Delta\sigma_{zp}$) is equal to 20% of the total vertical stress in the soil prior to construction (σ_{zg}) (Fig. 4). If the calculated depth of influence reaches a layer within which the modulus of deformation E is less than 5 MPa, the criterion is changed to 10%. Calculations of $\Delta\sigma_{zp}$ and σ_{zg} show that the criteria $\sigma_{zp} = 0.2 \sigma_{zg}$ is in the hard Vendian clay at a depth of about 45 m. With the assumption that the hard clay is stiff enough and the modulus is above 5 MPa, the depth of influence is 40 m below the foundation.

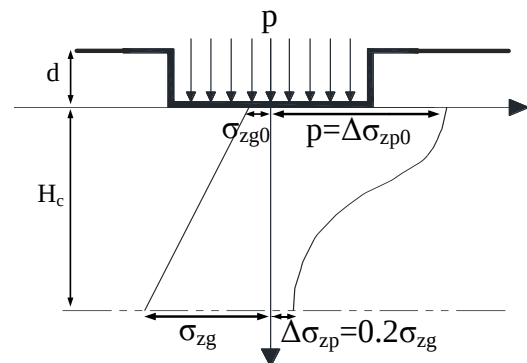


Figure 4. Vertical stress curves to obtain the depth of influence by the Russian practice

In Russian practice, the settlement analysis continues by finding out where the net pressure under the structure fits on the pressure versus settlement curve. Two thresholds are identified: a pressure p_1 called the first critical pressure and a pressure p_2 called the second critical pressure. The first critical pressure p_1 corresponds to a state at which the soil behavior changes from linear to non linear and plastic deformations take place. Note that the pressure can increase beyond p_2 as larger settlement takes place. For soils having an angle of internal friction less than 5-7 degrees, the pressure p_1 is calculated by the equation (Dashko, 1984).

$$p_1 = 3.14s_u + \gamma D \quad (2)$$

and is found to be 184 kPa in his case. The second pressure p_2 was calculated as the ultimate pressure or 279 kPa. The pressure under the cathedral foundation mat is 341 kPa and the net pressure at the mat level is $p = 341 - 90 = 251$ kPa; therefore the net pressure is within the intermediate range of pressure between p_1 and p_2 . As such the settlement is in the nonlinear range and plastic deformation plays an important role. In this case, the settlement calculation should be undertaken using Rosenfeld solution (1977) which uses a soil modulus depending on the stress level (Dashko, 1984).

To calculate the settlement, some assumptions are made: 1) the calculated depth of influence (2 times the width of the foundation) is more than the depth to the top of the hard Vendian clay which is assumed to be comparatively non-compressible; therefore the depth of influence H_c is limited to 40 m; 2) the modulus is constant for this portion of the pressure vs. settlement curve; 3) the modulus is taken as the average value within the depth of influence. The settlement within the linear portion of the curve (S) is given by (Construction Guidelines of Russia, 2011):

$$S_L = \frac{p_c B K_c}{K_m} \sum_{i=1}^n \frac{K_i - K_{i-1}}{E_i} \quad (3)$$

Where p_c = the net pressure at the foundation level = 341-90 = 251 kPa, B = the width of the foundation = 92 meters, K_c = a coefficient depending on $2H_c/B$, $K_c = 1.4$, H_c is the depth of influence = 40 m, K_m = a coefficient depending on the modulus and the width of the foundation, $K_m = 1.5$, K_i = coefficient depending on the relative depth $2z/B$, $K_1 = 0.025$ and $E_1 = 5000$ kPa for the first layer (lacustrine marine soil), $K_2 = 0.075$ and $E_2 = 1000$ kPa for the second layer (glacial lake clayey soil), $K_3 = 0.15$ and $E_3 = 2000$ kPa for the third layer (glacial clayey soil).

Settlement calculations for this kind of case history are usually done by using the results of consolidation tests or by using the modulus of drained triaxial tests as the settlement has taken place over the last 150 years. None of this data was available to the authors; instead the modulus obtained in unconsolidated undrained triaxial tests was available. While using an undrained modulus and an elastic equation in such an instance to obtain the total settlement is not an appropriate approach, it was used nonetheless to gain some insight on the magnitude of the settlement for the Cathedral.

The settlement calculation using Eq. 3 gives $S = 1.78$ m for the total foundation pressure and 1.3 m when using the net foundation pressure. Note that these settlement calculations are done for the linear strain model in Russian practice. The settlement for the non-linear strain model makes use of Rosenfeld solution (1977) and gives a settlement equal to 2 m for the total foundation pressure and 1.46 m for the net foundation pressure (Dashko and Shidlovskaya 2013).

6. SETTLEMENT OBSERVATIONS, ANALYSES, AND COMPARISONS

The Cathedral started to experience differential settlements during construction in 1841. The tilting continued to increase

over time and in 1927 it was observed that all columns of the porticoes tilted towards the center of the Cathedral with a maximum tilt of 15 cm (Nikitin 1939). The tilt of the floor of the Cathedral was measured and gave tilt values (slopes) averaging 0.004. In 1939 also, the difference in settlement between the East and West sides of the dome underneath the cross had reached 0.27 m (Nikitin, 1939). In 1950, the deterioration of the bearing structures was documented including cracks in the pilones, the arches, and the columns (Florin 1954). In 1983, a survey of the ground surface near the South-West end of the Cathedral was performed and indicated a maximum difference in elevation around the South-West part of the Cathedral of 0.85 m (Sotnikov, 1986).

The survey of Sotnikov (1986) gave the elevation contours of the ground surface West of the Cathedral. The differential movement is estimated to be about 25 cm according to the measurements. The value of 25 cm comes from using the elevation of the borings. On the West side, the elevations are 2.62 m and 2.71 m for an average of 2.665 m while on the East side they are 2.79 m and 3.04 m for an average of 2.915 m leading to a difference of 25 cm. The differential movement can also be estimated using a value of the tilt measured in 1984 by LISI to be 0.004 and the distance of 78 m between the two points (Sotnikov 1986). Using this approach, the differential movement of the cathedral is about 31 cm.

8. CONCLUSION

St Isaac's Cathedral is a remarkable historical monument in St Petersburg, Russia founded on soft soils. The engineering properties of these soils including stiffness, strength, and permeability are given in this paper. The ultimate bearing capacity calculations are compared to the estimated pressure under the Cathedral. The comparison shows that the soil under the Cathedral is near failure. The settlement estimates all indicate a very large movement and confirm that the Cathedral is near failure of the soil. Unlike the Tower of Pisa which has a high height to width ratio, the St Isaac Cathedral has a very low height to width ratio. Therefore it is in no danger of overturning but it is likely that the creep settlement will continue for centuries. The differential movement over the 92 m width of the structure is comparatively limited to about 25 cm. The uneven distribution of the piles below the foundation and the dishing effect of the mat combined with the long term creep of the soft soil lead to continuous cracking of the structure and of the columns supporting it. Future monitoring of the structure is necessary to better assess the situation and develop an action plan to avoid further deterioration.

9. ACKNOWLEDGEMENTS

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Case History of Pre-Supported Tunnelling Method in Weak Weathered Rock

Historique de la méthode de tunnel pré-supportée dans la roche usée et usée

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ABSTRACT: Construction of underground tunnel in the densely populated area is great challenge for geotechnical engineers. Sometimes a wide section tunnel passes through the area of partially weathered rock with a thin cover soils. The stability of superstructures are all the time problem during the construction of tunnel like this geological situation. In this paper, Korean pre-supported tunnelling method (KPST) in corporate with pre-nailing as well as pilot tunnelling technique is presented to improve the tunnel stability and reduce the probable tunnel deformation by grand arching effect of the reinforcement. Finally one case history of 321m-long tunnel construction in the completely weathered and fractured dark shale by using KPST method is presented with detail construction sequences.

RÉSUMÉ : La construction de tunnels souterrains dans la zone densément peuplée est un grand défi pour les ingénieurs géotechniciens. Parfois, un tunnel de section large traverse la zone de roche partiellement dégradée avec des sols de couverture mince. La stabilité des superstructures sont tout le problème du temps pendant la construction du tunnel comme cette situation géologique. Dans cet article, la méthode coréenne de pré-support de tunnelisation (KPST) en entreprise avec pré-clouage ainsi que la technique de tunnelisation pilote est présentée afin d'améliorer la stabilité du tunnel et de réduire la déformation du tunnel probable par effet de grand arc du renforcement. Enfin, un cas d'une construction de tunnel de 321 m de long dans le schiste sombre complètement altéré et fracturé par la méthode KPST est présenté avec des séquences de construction de détail.

KEYWORDS: pre-supported tunnel, KPST, pre-nailing, pilot tunnel

1 INTRODUCTION

Recently more than 31 cities in the world are called Mega City which has a population more than 10 million people. The urbanization of world population is rapidly increased since year 2000. The installation of underground infrastructures in urban area is great challenge for geotechnical engineers. The instability of tunnel during construction work has been reported by Shin et al.(2017) and Hoek(2001) due to poor rock quality in the field. Oreste and Dias(2012) reported the stabilization of the excavation face in shallow tunnels using fiberglass dowels. Atkinson and Potts(1977) studied the stability of a shallow circular tunnel in cohesionless soil. Seo et al.(2014) investigated the stability of shallow soft-ground tunnel with pre-nail supported system through experimental and numerical parametric study of design factors. In general NATM technique is normally applied in the tunnel construction with the good quality rock condition. It is sometimes difficult to use NATM technique in the weak weathered rock condition. However Korean pre-support tunneling method(KPST) can overcome instability problem even in the weak weathered rock condition. Because it can provide the pre-supported system in the soft rock with utilizing the soil nailing. The soil nailing reinforcement of the soft rock can be done with the construction of pilot tunnel prior to the main large scale excavation work. The KPST method is much safer in stability and gives high efficiency to construct the tunnel in the shallow cover soil and also in the weak weathered rock.

2 BASIC THEORY OF PLASTIC ZONE AROUND TUNNEL AND SEQUENCE OF KPST METHOD

Bray(1967) proposed rheoretical model to describe the failure mechanism for weak weathered rock during the construction of the tunnel according to the Mohr-Coulomb theory. Figure 1 shows the elastic-plastic zone given by Bray's theory during the construction of the tunnel. The shear strength of fracture rock in the plastic zone with the radius of R is defined as

$$\tau_p = S_j + \sigma \tan \phi_j$$

Where, τ_p is the sheer strength of fractured rock, S_j is the apparent cohesion, σ is the normal stress, and ϕ_j is the internal friction angle of the fractured rock in the plastic zone. Bray assumed the failure patterns in the plastic zone are log spirals with the inclination of δ degree with respect to the radial direction of tunnel. P_i in Figure 1 is the internal pressure in the tunnel provided by the support. The radial and tangential stresses in the plastic and elastic zones are well defined by Bray's solution and Goodman(1989).

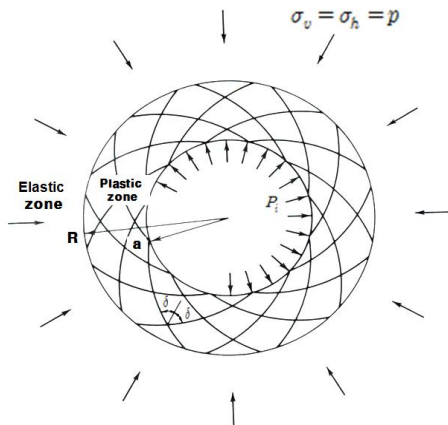


Figure 1. Assumed elastic and plastic zones around tunnel (Bray, 1967)

The construction of tunnel in the weak weathered rock has a high collapse potential as shown in Figure 2 (Seo et al., 2014). The collapse potential can be much reduced with the reinforcement of upper layer around tunnel. The reinforcement methods of weak weathered rock are either combined rock nailing with pressurized grouting or rock bolt. The pipe roof method with combination of pressurized grouting is also applied around the tunnel where the soil condition is very weak. The tunnel construction project in urban area where the weak weathered rock is existed has several problems such as superstructures above ground surface, heavy traffic, road, limited working space, and pedestrian road. The ground reinforcement with combination of pre-nailing and pressurized grouting would be the solution to overcome fore-mentioned problems. In the construction work, residential house, office building, and so on are existed above ground surface. Thus, pre-nailing work can not be proceeded from the ground surface.

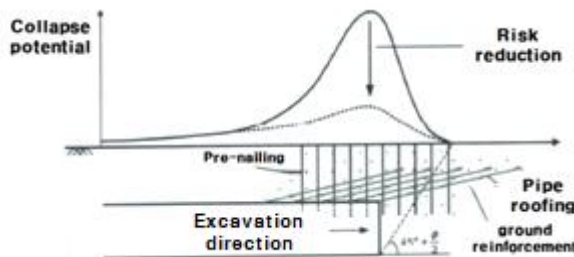


Figure 2. Potential tunnel collapse during tunnel construction work (Seo et al., 2014)

Therefore only way to do the pre-nailing in this case is underground working with the excavation of pilot tunnel as shown in Figure 3 (a). The economical point of view, this method is much cheaper than that of pipe roofing method. The application of TBM method is also difficult because there is a shallow cover soil and also the weak fractured rock with ground water table could be existed. The second step of KPST method is to drill the hole for installation of pre-nailing with grouting pipe in inside of pilot tunnel Figure 3 (b). The pressurized grouting work is performed with using the enlarged packer to increase the grouting efficiency in the radial direction for the peripheral zone where the excavation will not be proceeded Figure 3(c). The upper half part of main tunnel is excavated by using the ordinary excavation equipment for soil or adoption of blasting method for the rock Figure 3 (d). The nails are clamped with bearing plate which is connected on the steel rib. The lower half part of main tunnel is removed by using either the excavation equipment or rock blast method Figure 3(e). The remaining part of un-nailing zone at the lower part of tunnel is reinforced with installation of rock bolts. In the final stage of KPST method in Figure 3(f) is combining pre-nailing with the steel-rib and shotcrete lining with the thickness of 25cm.

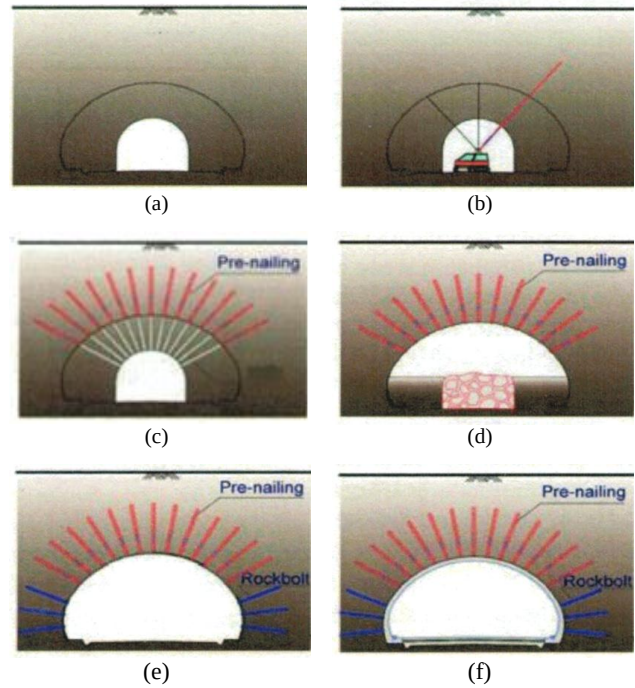
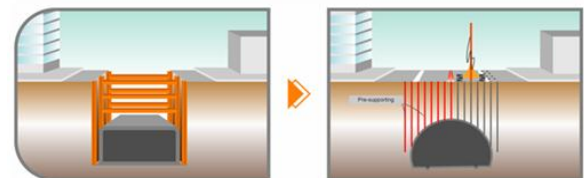


Figure 3. Construction sequences of tunnel by using KPST method

3 CASE HISTORIES OF KPST METHOD

There are two types of reinforcement methods around tunnel in the weak weathered rock, that is, external method and internal method. The external method is normally used where the cover soil above the tunnel is weak and shallow, and there is no superstructure on the ground surface. Figure 4(a) shows a road on the ground surface and need a underground tunnel. The KPST-external method would be suitable because the construction of tunnel would be possible without interruption of work due to not having an open-cut procedure. The weak weathered rock layer above the tunnel can be reinforced in advance by utilizing external soil nailing with pressurized grouting work from the ground surface to the tunnel section. Once the weak weathered rock is reinforced, a single large tunnel can be constructed instead of twin tunnels in one direction as shown in Figure 4(b). An adopting this method can reduce the construction cost and shortening the construction period. The safety of tunnel is also improved and reduced the maintenance cost of tunnel. The internal method of KPST is already described in Figure 3.



(a) Conversion from open cut and cover tunnel to pre-supported tunnel



(b) Conversion from twin tunnels to one single tunnel

Figure 4. Application of external reinforcement method for shallow tunnel construction project

The location of Daeri Tunnel is situated 300km South of Seoul in suburb of Daegu City, Kyungsangbuk province. The Daeri Tunnel is necessary to access the industrial complex easily from the major highway called “Kyungbu Expressway”. The original design concept of this tunnel was adopted NATM technique. However the construction of tunneling method was changed from the NATM technique to KPST method because the geological characteristic of this site is consisted of completely weathered and fractural dark shale as shown in Figure 5. The total length of tunnel is 321m long with width of 21.6m(4 lanes +side walk) and the height of 10.8m. There are several reasons to adopt the KPST method. The risk of tunnel collapse during construction work can be significantly reduced. The reinforcement of the weak rock layer by pre-supported nail can prevent the deformation of tunnel at the top and side after tunnel excavation work. The inlet and outlet of tunnel have required steel pipe grouting for the case of using NATM technology because these parts of a tunnel have severe weathered rock zones. For the case of KPST, the inlet and outlet of a tunnel can be reinforced by using external nailing technique with pressurized grouting as shown in Figure 6. The advantage of using KPST as shown in Figure 7(a) is to reduce the excavation stages comparing with NATM which is shown in Figure 7(b). The view of pilot tunnel construction and completion of Daeri Tunnel are shown in Figure 8.

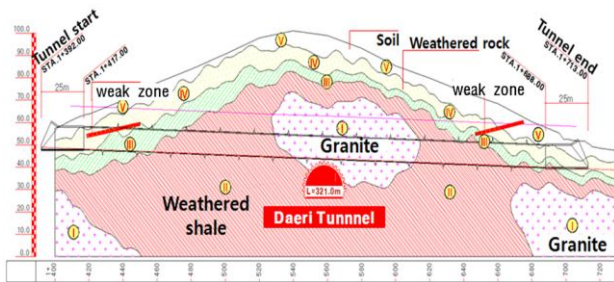
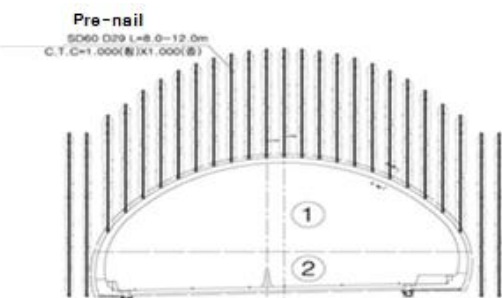


Figure 5. Geological profile for Daeri Tunnel site

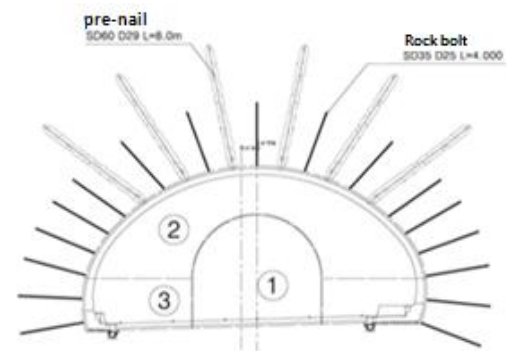


(a) External pre-supported tunnel method

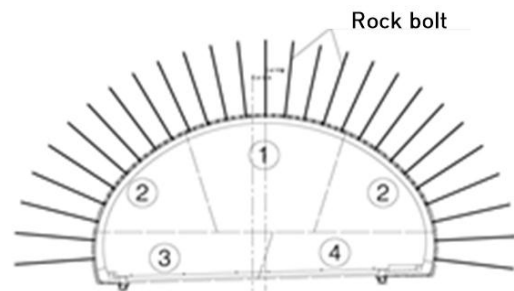


(b) Packing and pressurized grouting

Figure 6. External soil reinforcement of tunnel by soil nailing and pressurized grouting

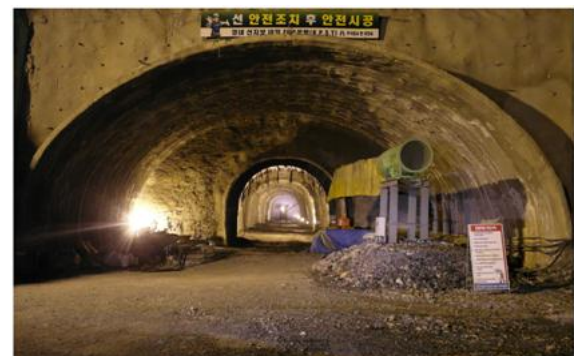


(a) KPST-internal method



(b) Original design by NATM

Figure 7. Internal soil reinforcement and excavation method for KPST and NATM



(a) Pilot tunnel completion



(b) Tunnel completion with concrete lining

Figure 8. View of pilot tunnel completion and final views of Daeri Tunnel

4 CONCLUDING REMARKS

Recently the development of underground space has been a lot of attention to obtain the marginal space for social infrastructures such as tunnels for highway and railway, caverns for storage of natural gas, and agricultural products, nuclear waste disposal facility, hydraulic power house, parking area and shopping center as well as recreation area. There are a number of methods to construct the tunnel like TBM, shield, and NATM. The selection of tunneling method is depended on the characteristics of geological formation of soil and rock, the quality of rock, the location of tunnel construction, and finally concerned the cost of tunnel construction. With considerations of various factors, KPST method would be the best choice to construct the tunnel with elimination of tunnel collapsing problem as well as cost of tunnel construction in the weak weathered rock condition. This method is also applicable in the densely populated urban area in the soft ground because it can reinforce the soft soil and weak weathered rock from the underground by utilizing the pilot tunnel and pre-supported nailing with pressurized grouting technique.

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Geotechnical challenges shaping the infrastructure for Megacities -(Practical aspects and case studies)

Défis géotechniques pour l'infrastructure des mégapoles
- (Aspects pratiques et études de cas)

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ABSTRACT: Current market conditions creating new future challenges for the construction industry in combination with the decreasing demand for construction at least in well-developed regions, it becomes more and more important to gain competitive advantages through innovative solutions in order to deal with the continuing pressure on costs, schedule and quality also. For geotechnical engineers the main future challenges are to be familiar with theoretical as well as practical geotechnical applications to identify the optimal combination of technologies including the best design approach to be competitive for a specific project.

RÉSUMÉ: Les conditions actuelles du marché créent de nouveaux défis pour l'industrie de la construction en combinaison avec la demande décroissante de construction au moins dans les régions bien développées, il devient de plus en plus important d'acquiescer des avantages concurrentiels grâce à des solutions innovantes pour faire face à la pression continue Sur les coûts, le calendrier et la qualité également. Pour les ingénieurs géotechniciens, les principaux défis futurs sont de se familiariser avec les applications géotechniques théoriques et pratiques pour identifier la combinaison optimale de technologies, y compris la meilleure approche de conception pour être compétitif pour un projet spécifique.

KEYWORDS: Innovation, GeoBIM, cased deep mixing

1 Current market conditions and trends

The ongoing financial crisis in the western world and the efforts not only to consolidate national budgets, but also to rescue certain banks, have not left the building industry unaffected either. Some drastic cutbacks to public budgets, along with the increasingly restrictive granting of credit for the financing of construction projects, have created enormous over-capacities in the construction sector, with purely negative impacts. As a consequence of these negative developments, many companies are experiencing the pressure to restructure more quickly and to reduce costs, but at the same time to keep the consequences of all activities, both externally and internally, within limits acceptable to employees, owners and finance institutions and last but not least costumers. The construction industry is not only undergoing substantial changes because of these basic alterations in the global conditions as during the coming years and decades additional transformations will influence the development:

- the **acceleration in globalisation** must be reckoned, with the economic output of developing countries growing about 3 times faster than those of the industrial nations,
- the **mega centres** of the world will continue to attract more workers and infrastructure problems will accompany the corresponding development.
- **Energy requirements** will increase dramatically and require a more efficient and stable infrastructure.
- The **acquisition of capital** will increasingly move towards private financing, because of the public deficits, which can then act as a driver for further growth.

These future changes in the basic conditions will also force the construction industry to change in order to remain competitive. Attempts to work under these conditions both competitively and cost-efficiently are limited and, in many cases, result in a pure competition on costs. Apart from the cost factor, competitiveness is in addition determined by the following parameters:

- **Costs:** determined by effectiveness and better efficiency as well as industrial safety, technology and competency
- **Quality:** experience and authority of the employees, training and further training as well as management abilities

- **Time:** influenced by productivity and technology
- **Innovation:** in the form of product and/or process innovations

A business can cope with the transformations and changes in progress only by improving its own competitiveness through innovative approaches. The influencing factor of **innovation** offers the largest possibilities for increasing competitiveness and attaining competition advantage through new, innovative ideas.

2 Future challenges in construction

The environment described above clearly shows the always and ever changing nature of tasks, particularly in the geotechnical sector. The significant challenges in the geotechnical world can be summarized as:

- a. Construction projects are becoming more and more complex:

The increasing complexity of projects will lead to significantly more attention being paid to experience and competences in such tasks as planning, designing, cost estimation and execution. Only by integrating all these competences into the construction process complexity will be manageable.

- b. Economic optimization requirements will promote global co-operation and increasing global competition:

In the future, due to more and more global competition from low cost regions, optimizations for economic improvements will create substantial cost pressure and will force cooperation. These developments will create increasing international competition and with increasing internationalization of the market conditions intensified contractual conditions and clauses. To cope with this challenge, extended knowledge of contract and risk management will be essential, alongside the increasing requirement for technical capabilities.

- c. Increasing demand for automation and the automatic monitoring of building processes:

With increasing complexity and increasing pressure on efficiency, automation and the automatic monitoring of building processes will become increasingly important to maintain the required standard of quality. A significant task for the future will be to minimize risks of faults, to bridge as many gaps in experience as necessary and to avoid negative consequences. But automation will create new requirements, on device

manufacturers and suppliers in particular, to directly and promptly optimize productivity and quality using improved, process-oriented monitoring in the construction process.

d. Clients and consultants will expect a more efficient documentation of the processes and quality:

Increasing requirements for documentation of construction processes will lead inevitably to improvements in existing quality and risk management systems, to be developed in parallel to the requirements to automation. An even closer cooperation between users and manufacturers of construction equipment will in most cases be adapted as counteraction to these future tasks for process cycle documentation by using measuring and sensor technology.

e. Environmentally-friendly technologies are becoming more attractive:

The development and advancement of resource-saving technologies and building processes will become a significant challenge for the entire construction industry, but particularly for geo-technologies. Alongside the reduction in the consumption of materials and of emissions (noise, dust, vibration...), of CO₂ emissions in parallel to the reduction of energy consumption and also the improvement of the logistics process will be at the forefront.

Considering all these future challenges as general threats to the different construction processes, innovative solutions as combination of alternative products, alternative designs, alternative materials and alternative execution procedures could show a way out of this problematic issue.

3 Innovation and building practice

Innovations, whether as improvements to products or procedures, are subject naturally likewise to the technical codes and standards which generally and particularly must and should be fulfilled. On top an innovation usually involves an increase in complexity and thus to higher risk levels with warranty entitlements increasing at the same time. In addition, innovations require more investments in crosschecks and quality assessment, in addition to ever faster implementation cycles (time to market). If the competitiveness is to be maintained over a longer period, constant innovation and the improvement of existing products are the basic requirement. In addition, each innovation must to all above deliver an increase in value for the customer in terms of costs, quality and/or time. Only if an innovation fulfils this complementary condition of creating an added value for the customer, it can also sustain in the competition.

In consideration of these economic basic parameters, the construction process usually has also to manage the following technical requirements and framework conditions as a challenge:

- restricted construction areas close to some neighbouring buildings which must be integrated
- short building phases with partially overlapping process cycles and restricted and limited infrastructure and accessibility of the construction area
- high requirements for minimum deformations and settlement/subsidence as well as for minimised emissions (noise, dust, smell, visual, vibrations...)
- high requirements for resource-protecting building methods and for health and safety protection

This combination of economic and technical requests inevitably require a modification of the current technologies and processes in the construction process as demand led innovation. Contrary to these demand led innovations, through customer/market requirements, new technologies are also brought to the market via innovations promoted by suppliers (supply pushed). The following projects show how innovative solutions can

positively influence competitiveness in terms of costs, quality and execution time.

4 Innovative solutions

With the increasing demand in reuse of inner city areas and the urgent need for deeper foundation pits in constricted areas, adjacent structures close to and high ground water level, the well-known established technologies are often too time consuming and occasionally not economic to install. The alternatives for these specific cases could be the soil mixing technology (Topolnicki, 2013).

The selected projects should show in the first case how innovative solutions can be used to increase value for clients and tenderers in terms of risk mitigation (Wehr, Topolnicki, Sondermann 2013), whereby the second example shows the utilization of an innovation to the benefit of the project and the client.

4.1 Modernisation of the Victoria Station in London

The upgrade of the Victoria Station in the centre of London, where the Victoria and District & Circle lines cross, involved construction of a new ticket hall on the north side of Victoria Street, extension and modernization of the existing South ticket hall and building of eight elevators providing step-free access from street to platform levels and an underground walkway to the mainline station. Many of these new structures were built at the level of the interface between the London clay and water bearing terrace gravels, and construction took place next to or beneath the foundation of fragile listed buildings. The crowns of the District & Circle line tunnels are just 2.5 m below street level in some areas, and the crown of the Victoria line is about 14 m below ground. These challenges dictated that the new tunnels, ranging from 4.5 to 9 m in diameter, were constructed at shallow depths, with an axis approximately 10 m below ground surface and clearances of less than 10 to 30 cm from essential LU assets in places.

To enable safe tunneling in the challenging ground and site conditions, about 2,500 interlocking jet grouting (Soilcrete) columns ranging in diameter from 1.4 to 1.8 m were installed to depths up to 14 m to provide a watertight annulus around the planned tunnels with a minimum thickness of 2 m. Their execution required continuous refinement and improvement, including the 'leap of faith' needed to apply BIM to a complex ground treatment process on an unprecedented scale. Spatial visualization of the planned arrangement of Soilcrete columns along the underground structures of the Victoria Station is shown in Figure 1.

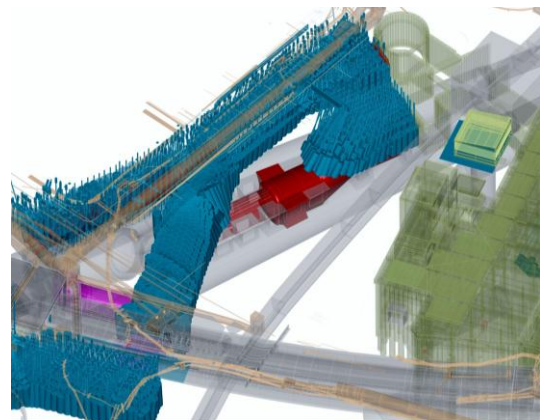


Fig. 1. Visualization of planned jet grouting works at the Victoria Station (initial model prepared by Taylor Woodrow BAM Nuttall JV and the designer Mott MacDonald).

The requirements regarding the design and orientation of Soilcrete columns as well as sequencing of jet grouting works were driven by a risk review of each individual column, taking into account criteria such as soil types, column installation and proximity to sensitive structures and numerous underground utilities (Fig. 2).

To identify, survey, and model these utilities and to plan the orientation of Soilcrete columns to ensure sufficient coverage and safe tunneling environment, a comprehensive GeoBIM 4D (the fourth dimension was time) model was created and implemented. Figure 3 presents example section of the model and shows the alignment of Soilcrete columns adjusted to the location of underground installations and ground conditions. One of the most complex execution tasks was the careful measurement of the orientation of the drill string using GPS sensors at the top and bottom of the masts in order to achieve the required 50 mm tolerance. In addition, column diameter was monitored by the thermal degradation technique.

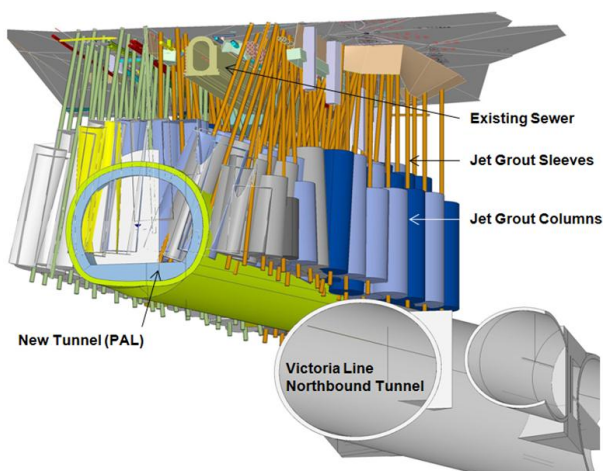


Fig. 2. Arrangement of Soilcrete columns, modelled with the GeoBIM.

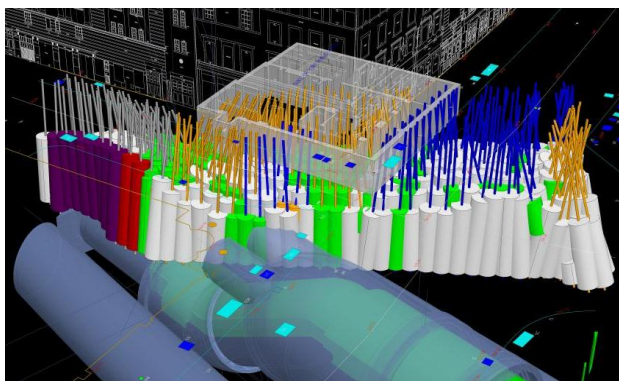


Fig. 3. GeoBIM enables review of Soilcrete columns

Thanks to accurate as-built data and the enhanced model functionality, the adopted GeoBIM could be also used to analyse the safety of the excavation works under the protective cover of the constructed Soilcrete block. For this purpose a preceding simulation of tunnelling works conducted with the NATM method was performed to detect potential gaps inside the Soilcrete block which could pose a risk of groundwater inflow and/or loss of tunnel crest stability. Figure 4 shows an example simulation made for the pedestrian tunnel, which illustrates the as-built jet grout annulus and the gaps identified in subsequent cross-sections of the tunnel.

The described GeoBIM, which represents part of a more comprehensive BIM used for the entire project, was developed for the needs of a large and complex construction project, and represents a leading example of a mature model comprising a broad scope of information.

The full BIM embraced 18 separate design specialties, and enabled direct exchange of information between them. According to the user, London Underground, implementation of BIM into the project allowed them to:

- shorten the time of design works and preparation of more accurate documentation, eliminating errors and collisions to a greater extent,
- quick implementation of design changes, with a possibility to repeat the calculations, correct measurements and update the executive drawings with a minimum associated workload,
- improved control of the works' planning and performance, quality and compliance with technical specifications.

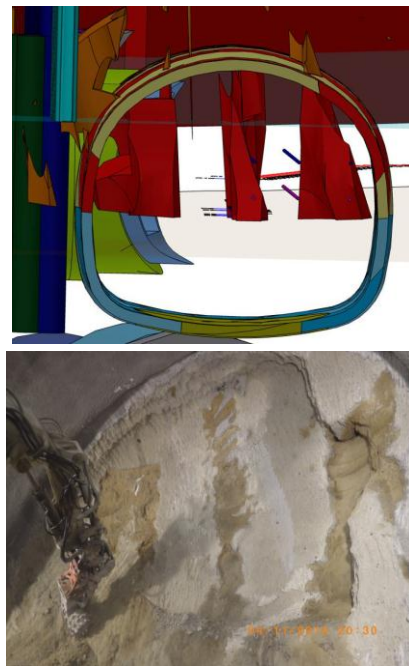


Fig. 4. Simulation of tunneling works with the GeoBIM and location of potential gaps inside the Soilcrete block. (a) Forecasted location of main gaps.(b) Exposed tunnel cross-section and leakages.

Furthermore, it is expected that when all construction works are completed, the established BIM, comprising full technical documentation and a virtual model of the whole project, will provide all users a greater ability to manage station resources

4.2 Cased deep mixing supported by jet grouting

The Tubular Soil Mixing system (TSM) as a combination of a cased mechanical mixing with integrated support of hydraulic cutting by jet grouting (Fig. 5). The patented Tubular Soil Mixing process (TSM) can be used to produce a water tide lateral foundation pit support in an inner city environment (Kirsch, Sondermann, 2003).

The boring and mixing tool consists of a auger, which is equipped with removable teeth and suspension nozzles at the tip. The auger is guided along its entire length in a counter-rotating casing. The tool is driven by a dual rotary head which is guided on a guide unit. The combination of hydraulic and mechanical

mixing ensures a very homogeneous mixing of in-situ soils and produces a high and homogenous firmness in the cubature and/or columns. The combined use of auger and casing allows obstacles like masonry or unreinforced concrete to be penetrated and intersecting columns to be produced. Furthermore, the casing gives a high accuracy in verticality and an absolutely uniform diameter of the columns over the entire casing depth. Immediately after penetrating and mixing from bottom up, a steel girder can be inserted into the fresh material of the column (Sondermann, 2012).

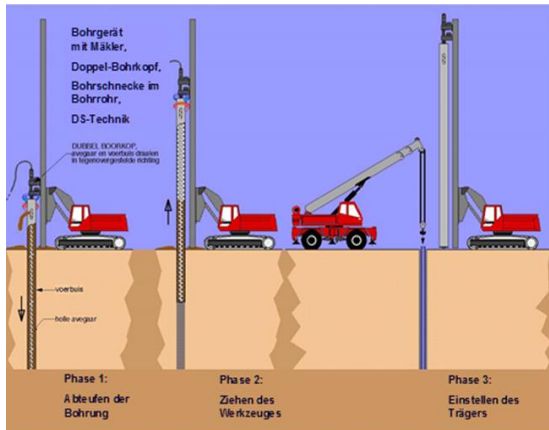


Fig. 5 The Tubular Soil Mixing system and process

The structural system of a shoring wall created using the TSM process is demonstrated in figure 6. Because – unlike the MiP process – when using the TSM process the jet grout technology is used to support the mixing process, the general structural design requirements for the jet grouting elements can be used for the analysis of the cubature manufactured in the TSM process.

Regarding waterproofing and appearance, a temporary pit system made with intersecting TSM columns is comparable to a secant bored pile wall. The points of intersection produced by a superstructure, like evenness, tolerance, mounting possibilities for the insulation and requirements of the formwork also correspond to those of a bored pile wall.

With this innovative technology even complex deep lateral support systems can be managed (flexibility, adaptability) and the whole process is automatically recorded and documented.

5 Conclusions

Innovative solutions in use of new or improved technologies as well as the changing and adjusting of existing technologies to suit the specific requirements of the project can contribute to generating an increase in value for the owner. These approaches can also play their part in meeting the future challenges to reduce complexity, to increase quality through automation and documentation as well as to strengthen the global competitiveness.

For geotechnical engineers the main future challenges are to be familiar with theoretical as well as practical applications of advanced technologies to identify for a specific project the optimal combination of technologies including the best design approach to stay competitive.

To cope with this more general and conceptual formulation of the task the below summarized capabilities and competences of geotechnical engineers are required:



Fig. 6 Boring/mixing tool for the TSM process, finished TSM wall with inserted H beam

- deep knowledge of technologies and processes including the limits of application of the different technics
- excellent judgement on subsoil characteristics and stress strain relationships of soil materials
- advanced design capabilities including judgement regarding application of design model and approach
- understanding the execution process and the impact of the process on adjacent structures and the environment
- fair judgement on technical requirements versus products involved including alternatives

To cope with these requirements in addition to the technical skills geotechnical engineers should show capabilities in:

- Planning and organizing
- Analysing and structuring
- Accuracy and reliability
- Team work and relationship with colleagues

Those geotechnical engineers developing a sound and perfect combination of the technical expertise and the soft competences will be fit for future challenges. As a geotechnical engineer is should be the goal not only to sharpen the geotechnical knowledge but also acquire and shape interpersonal skills.

6 Literature / Norms / Permits / Patents

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Application of X-ray CT in geotechnical engineering

Application de la tomographie par rayons X à l'ingénierie géotechnique

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ABSTRACT: An X-ray CT was developed as a diagnosis method on human body and recently, this apparatus has been used in science and engineering. The author started to study on the application of X-ray CT in geotechnical engineering in 1996. In this paper, a brief introduction of X-ray CT with its fundamental issues is summarized at first. Then, since the session theme is “Geotechnical Infrastructure for Megacities and New Capitals” under TC305, one of the application studies which is the problem on tunneling is discussed. Here, the mechanism of face failures of tunneling which are both active and passive types are presented in details. Finally, the salient future on the application of X-ray CT in geotechnical engineering is stated.

RÉSUMÉ : La tomographie par rayons X a été développée comme outils d'aide au diagnostic sur le corps humain et plus récemment, cet outil a été utilisée dans le domaine des sciences et de l'ingénierie. L'auteur a commencé à étudier l'application de la tomographie par rayons X à l'ingénierie géotechnique en 1996. Dans cet article, une brève introduction de la tomographie par rayons X et de ses principes fondamentaux est présentée. Par la suite, et puisque le thème de la session porte sur les "Infrastructures géotechniques pour les mégapoles et nouvelles capitales" sous l'égide du TC305, le cas d'étude des tunnels est évoqué ici. Les mécanismes de rupture de front de taille dans le creusement de tunnel sont présentés en détail, ceux-ci mobilisent le sol de manière à la fois active et passive. Pour finir, on évoquera l'avenir de l'application de la tomographie par rayons X en ingénierie géotechnique.

KEYWORDS: X-ray CT, X-ray Physics, Tunnel face failure

1 INTRODUCTION

X-ray Computed Tomography (CT) is a non-destructive technique that allows imaging and quantification of internal features of an object in three dimensions (3D) and it is one of the novel prize in 1979. Since then, there are wide variety of applications in the field of medical diagnosis. Although this apparatus was originally developed for medical imaging, recently, it has been widely used in material sciences and proved its interest in various domains of geosciences and geotechnical engineering within last two decades. Followings are one of consecutive conferences related to this topic; Otani and Obara 2003(a), Desrues et al. 2006, Alshibli and Reed 2010, Cnudde and Bernard 2013, and Long and Francus 2015. In fact, a new international society called “IntACT (<http://www.intact-tomo.org/>)” was established in 2014 under those activities. Although there are many conferences and symposia, the application in the engineering field is limited. On the other hand, in 1996, an industrial X-ray CT scanner was installed at the Department of Civil and Environmental Engineering in Kumamoto University, Japan. And that of MicroFocus type was also installed in 2010. Under those facilities, the author's research team has been done many of application studies in geotechnical engineering (i.e. Otani et al. 2000, Otani 2003(b), Otani et al. 2006, Mukunoki et al. 2009, Chevalier et al. 2011, Eskisar et al. 2012(a), Watanabe et al. 2012(b) and Takano et al. 2015).

The objective of this paper is to summarize fundamental issues for X-ray CT at first and then, one of the application study which is the evaluation of tunnel face failure is introduced. Finally, the salient future of this study in geotechnical engineering will be stated.

2 FUNDAMENTALS OF X-RAY CT

2.1 X-ray CT

X-ray Computed Tomography (CT) is a technique that allows non-destructive imaging and quantification of internal features

of an object in three dimensions (3D). The method reveals differences in density and atomic composition. The principle of the measurement consists in two steps: first, recording x-ray radiographies of a specimen at different angular positions with a full angle of at least 180° and then, reconstructing virtual slices from these different projections, using appropriate algorithms which are either algebraic or based on a back projection principle. Stacking several sequential slices allows of reconstructing a 3D image of the object, in which one of the examples is shown in Fig. 1 for the case of soils. Reconstruction of the slices is based on the spatial distribution of the linear attenuation coefficient of x-ray, which is determined by using the well-known attenuation Beer-Lambert's law:

$$I = I_0 \exp(-\mu x) \quad (1)$$

where I and I_0 are the x-ray intensity measured after passing through the object and its initial one, respectively; x is the path length of the x-rays through the object; and μ is the linear attenuation coefficient. This coefficient depends on the energy of the x-rays as well as on the density and the atomic composition of the material. For most materials and commonly used values of energy, the linear attenuation coefficient is generally given by the simplified equation (Van Geet 2001):

$$\mu = \rho(a Z^{3.8}/E^{3.2} + b) \quad (2)$$

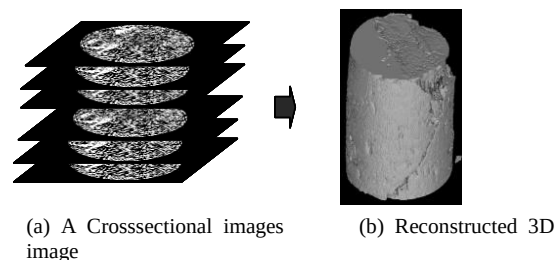


Figure 1. X-ray CT principle (Otani et al. 2000).

where ρ and Z are the density and the atomic number of the object, respectively and E is the energy of the x-rays. The quantities, a and b appearing in Eq. (2) are instrument dependent parameters (Curry et al. 1990). Most of the time, a CT value, which is also called Hounsfield value and is defined by the linear attenuation coefficient with respect to water, is used and defined as :

$$CT \text{ value} = \alpha + \kappa(\mu - \mu_w)/\mu_w \quad (3)$$

where μ : the x-ray attenuation coefficient of an object, μ_w : the x-ray attenuation coefficient with respect to water, and κ and α : some constants. It should be noted that the values of 1000 for κ and 0 for α are usually selected. Thus, the CT value of air should be -1000 because the coefficient of absorption for air is zero. Likewise, the CT value for water is 0. Thus, Eq.(3) can be rewritten by following equation:

$$CT \text{ value} = 1000(\mu - \mu_w)/\mu_w \quad (4)$$

When slices are visualized, they are classical pictures composed of a matrix of pixels in two dimensions (2D). Actually, a slice represents a finite thickness of the object and is composed of voxels which is the extension to 3D of a pixel. Figure 2 shows a schematic of a slice composed by $n \times n$ voxels. One slice represents a volume defined by an area of pixels in the 2D image, which is represented by a bold square in Fig. 2, times the thickness of the slice. The size of the voxel which means the spatial resolution is "pixel length" times "pixel length" times "beam thickness". Thus, the value of a voxel is a volumetric value and the final parameter visualised i.e. the voxel value in the reconstructed image, is linked to the mean of the linear attenuation coefficients of all the components in the volume of one voxel. The authors recommend the references (Kak and Slaney 1988, Baruchel et al. 2000) for further information about this technique.

2.2 X-ray CT images

Figure 3 shows the constructing process of the CT image. Fig.3(a) shows the frequency of the CT values for total of 5 cross sections of a soil sample and Fig.3(b) shows two different cross sectional images for this sample in which the range of CT value for the case (1) is $-1000 < CT \text{ value} < 500$ while that of case (2) is $0 < CT \text{ value} < 500$ as shown in Fig.3(a). As easily realized from those images, different images are obtained although the material is the same.

Thus, the image should be constructed along the target range of the density and it is important to note that the range of the CT images has to be obtained after conducting statistical analysis as shown in Fig.3(a). In our study, all the original images are constructed by the way of case (2) in Fig.3(b), in which the range of existing CT values is checked for each material and testing.

2.3 X-ray CT apparatus

Different types of x-ray source and detector exist and two ways to generate the measurement at different angles of the x-ray attenuation are possible, which are rotating the specimen or the pair source/detector. There are four scanning generations and

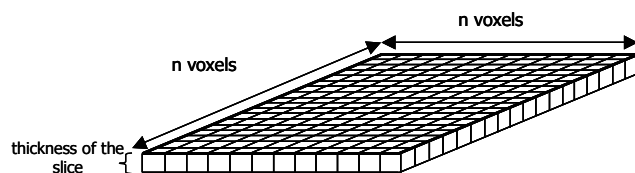
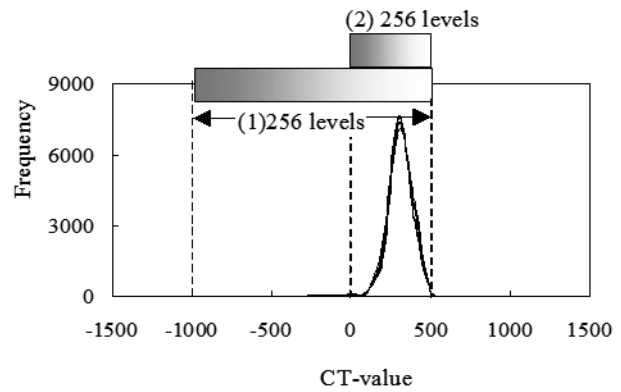
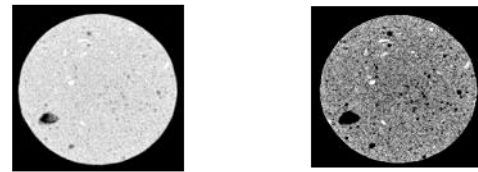


Figure 2. Schematic of a slice (composed of $n \times n$ voxels).



(a) Frequency of CT-values



(1) $-1000 < CT\text{-value} < 500$ (2) $0 < CT\text{-value} < 500$

(b) CT images

Figure 3. Voxel and the concept of X-ray CT image.

	(a) Translate-Rotate 1	(b) Translate-Rotate 2	(c) Rotate-Rotate	(d) Stationary-Rotate
Scan type	Source of x-ray detector	Source of x-ray detectors	Source of x-ray detectors	Source of x-ray detectors
Shape of x-ray	Pencil beam	Narrow fan beam	Wide fan beam	Wide fan beam

Figure 4. Generations of X-ray CT scanner.

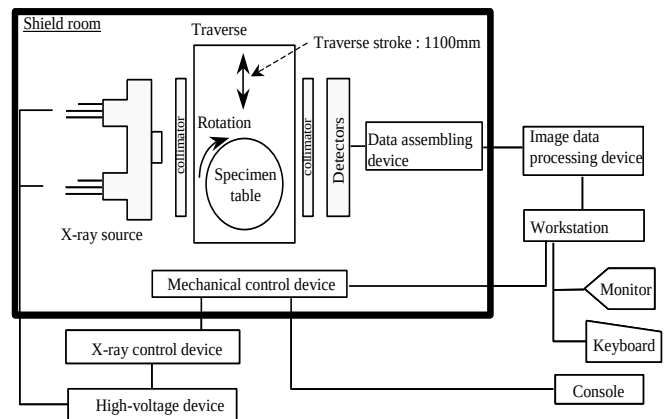


Figure 5. General organization of X-ray CT scanner (Industrial X-ray CT scanner at Kumamoto University).

these are shown in Fig. 4 in which those are dependent on number of data and time of scanning. Figure 5 shows one of example of CT scanner which is an industrial one at Kumamoto University. The method used for reconstructing the slices is a simple back projection. The measured parameter is the CT value determined by Eq.(4). Obviously, Eq.(4) is theoretical and thus, at the beginning of the scanning in practice, the values for

air and water are close to the theoretical ones. For each scan, the value for air is measured during the translate move in order to calibrate the obtained CT values. To improve the capacities of the scanner, hardware and software elements have recently been changed in order of improving the image quality, and this makes more wide range of application studies possible. For further information about the scanner, see Otani et al. 2000.

3 TUNNEL FACE FAILURE

Figure 6 shows the developed model loading test apparatus. The soil tank was scanned after pulling out the rod from the ground. Model ground was made by Toyoura sand. Figure 7 shows the CT images at the center of the tunnel cross section for 0 mm (initial condition) and 2 mm of pull-out length with the CT value distributions at the location of showing two arrows. There is a low density zone in front of the tunnel face in Fig. 7 (b). This can be considered a shear band due to strain localization and the area enclosed by shear band can be thought a failure zone. The failure zone in this study is defined as an area within the voxel on the edge of the lower density zone than its initial state correspond to the point B in Fig. 7(c) and the tunnel face correspond to the point A in Fig. 7 (c). Figure 8 shows a series of horizontal slices through the X-ray tomographic images

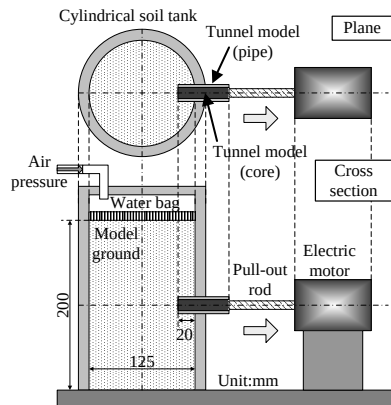


Figure 6. Schematic view of tunnel pull-out model test system

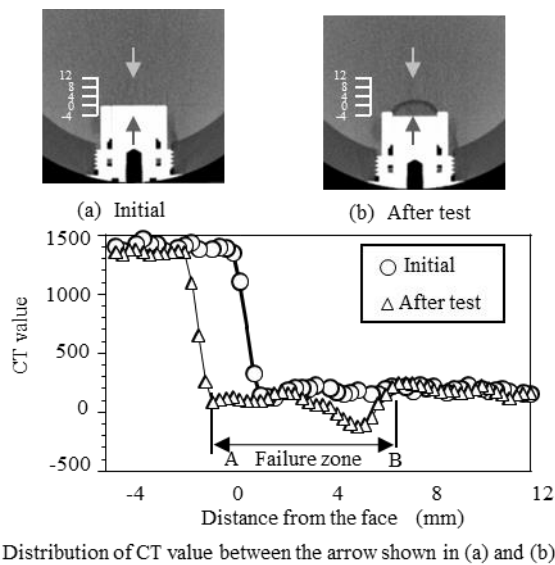


Figure 7. CT image before and after experiment and the definition of failure zone

around the tunnel model at two different positions which are at tunnel center and crown. It can be observed that the failure zone with shape of semi-circular line in front of tunnel face after 1 mm of pull-out length without significant change of density within the failure zone. Displacement of the tunnel face follows an inflow of soil mass into the tunnel inside, although the shape of failure zone in front of the tunnel face has not expanded toward the direction of tunnel axis. Figure 9 shows three dimensional images of failure zone segmented from volume image through tomography. Figure 10 shows the colored tomographic image of vertical cross section with the center of tunnel model to the direction of pulling out. Failure zone is developed from the bottom of the tunnel model to the ground surface and it is closed due to arching effect of the soil around the crown at 1 mm and 2 mm pull-out length. Moreover, there is no density change at the inside failure zone. It is considered that this zone move toward the direction of tunnel inside as a rigid body. As the pull-out length of the tunnel model increases, the failure zone is developed vertically toward the ground surface. Figure 11 shows the comparison of the result of X-ray CT and that of centrifuge test for 3D tunnel face failure. The pull-out length of centrifuge test and X-ray CT test (XRCT) is 0.3 D and 0.25 D, respectively. The failure zone reaches to ground surface in both cases. These show the similar angle of slip surface and

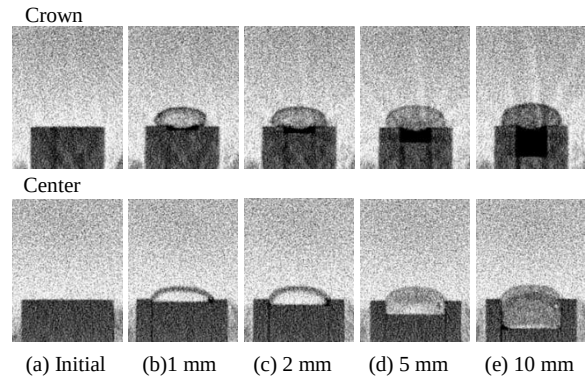


Figure 8. Horizontal XRCT images from (a) initial to (e) 10 mm of displacement at the tunnel center and crown.

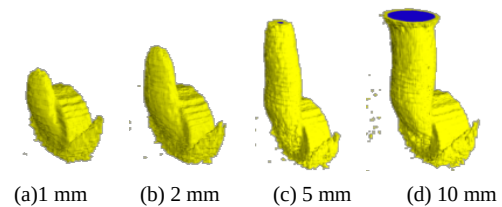


Figure 9. 3-D images of active failure zone.

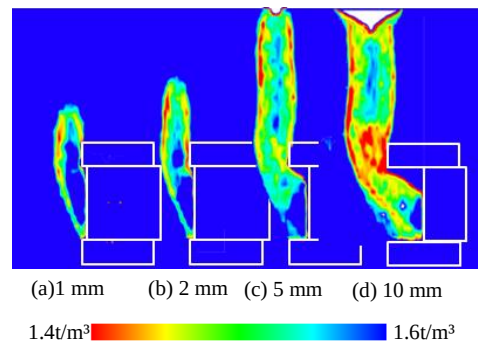


Figure 10. Vertical slices through ground density.

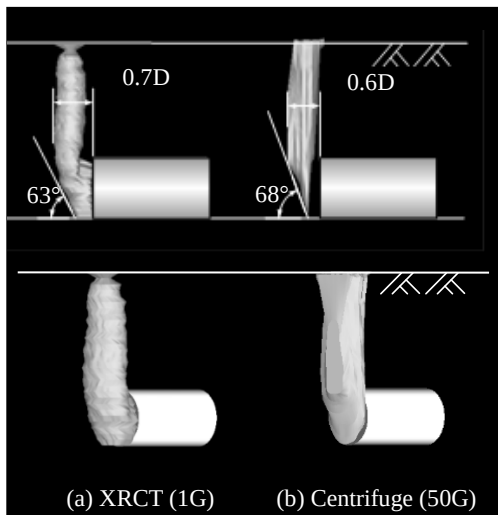


Figure 11. Comparison 3D images through (a) XRCT and (b) centrifuge.

width of failure zone, 63° and 0.7 D for XRCT and 68° and 0.6 D for centrifuge, respectively. The active failure zone occurred in the centrifuge test shows close similarity to the one occurred in the XRCT test. It is said that the 1g model test can simulate active failure at tunnel face without the effect of model size and the boundary condition of soil tank. More details of this study, please refer the paper, Takano et al. 2009.

4 CONCLUSIONS AND FUTURE

The basic idea of X-ray CT was reviewed including x-ray physics, and one of the application studies which is the evaluation of tunnel face failure was introduced. Since the author have done a large number of application studies, the salient future on this topic is summarized:

(1) About X-ray CT apparatus:

There are different type of CT apparatus including synchrotron and recently, higher resolution has been expected. However, basically, it is true that the more expected high resolution is, the smaller the size of specimen is. This means that it is difficult to do 1g model test with CT scanning. That is why there are many studies on the micro behavior of soil specimen including deformation, failure and its seepage. A key point of using X-ray CT is the challenge of the behavior from macro to micro, so that it is important to know that the suitable apparatus have to be chosen and it depends on what the researcher wants to do with CT scanning.

(2) Application topics:

There are wide variety of application studies on the use of X-ray CT in geotechnical engineering, which are from the behavior of soil element to that of physical modellings of the ground or soil-structure interaction behaviors. An image analysis called Digital Image Correlation (DIC) method has been widely used for the quantitative discussion such as displacements and strains in the soils in three dimensions, so that when the images with relatively high resolution can be obtained, the expected displacements and strains will be able to be analyzed. Unsaturated soils is one of the ongoing studies which is three phases (solid, air, and liquid) problem and a new finding on this topic will be expected soon.

Finally, it is concluded that X-ray CT is useful tool in geotechnical engineering and it is expected that current design or construction technique is improved by those studies.

5 ACKNOWLEDGEMENTS

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Experience in Construction and Design Justification of the Technology of Underground Structures in Moscow

L'expérience dans la construction et la technologie étude de conception OMS des structures souterraines à Moscou

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ABSTRACT: The report examines the impact of the technology of erecting an underground structure on its final stress-strain state. As examples, are given real construction objects. Also are described advantages of the developed approach – «technological mechanics of soils».

RÉSUMÉ : Le rapport examine l'impact de la technologie de construction de la construction souterraine dans son état de contrainte-déformation finale. Les exemples sont les objets réels de la construction. Donne un aperçu des avantages de l'approche - «le processus de la mécanique des sols»

KEYWORDS: Technological Mechanics of Soils, Construction and Reconstruction of Underground Structures, Accounting for the Stage of Erection of Underground Structures

The large-sized structure, while it is erected from the very beginning, interacts with the ground and complicates its constructive scheme, gradually reaching the final state of the structure according to the design. A direct experiment shows the need to determine the effort precisely in the process of erecting the structure.

Example, concreting the lower foundation plate of the turbo-unit ($l = 75$ m, $b = 25$ m, $h = 4$ m) with a capacity of 1200000 kW. The measured contact stresses under the basement not only did not coincide in value with the traditional calculation of the plate for a load of its own weight. Moreover, the bending moments had the opposite sign: the stretched fibers were on the top face of the plate, and not on the bottom, as calculated. This prompted the development of a methodology for calculating the «growing beam» by developing the classical theory of beams on an elastic foundation. In this case, accounting of the technology of concreting led to the identification of a significant margin of strength of the lower foundation plate. It turned out that construction technology always affects the final efforts in the design.

The simplest example in Fig. 1 shows the deformations of the foundation pit and the bending moments in it when excavating the soil and the installation of wall lining in two versions: in the form of overlapping stiffeners and in the form of prestressed anchors. In one of the zones along the height of the wall, bending moments in two calculations have opposite signs.

This approach has been conventionally called «technological mechanics of soils», its main features:

a) taking account in the calculation of the destruction of the natural (or initial) stress-strain state of the soil in the process of the pit excavating;

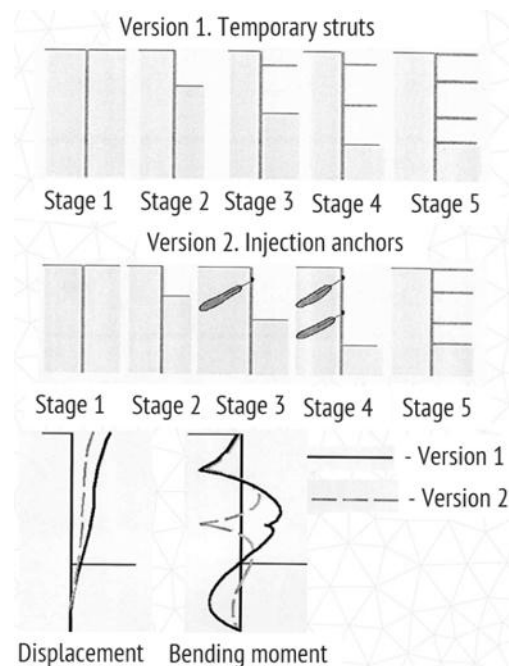


Figure 1. Deformations of the foundation pit and the bending moments

b) the design schemes describe the formation of a new stress state in the construction of a structure when a new element is added to the elements that have been created and have already taken the load, but it has already completed the previous structural design and will be included in the work. And so on for cycles.

c) the calculation can be performed both in a linear setting (the effect of technology is always manifested), and in the nonlinear, which is typical of all soil models

The foregoing calculation methodology was applied in the construction of the first in Moscow underground shopping and recreational complex Okhotny Ryad in 1996, Fig. 2 a, b, where 8 cycles of the construction process were taken into account.

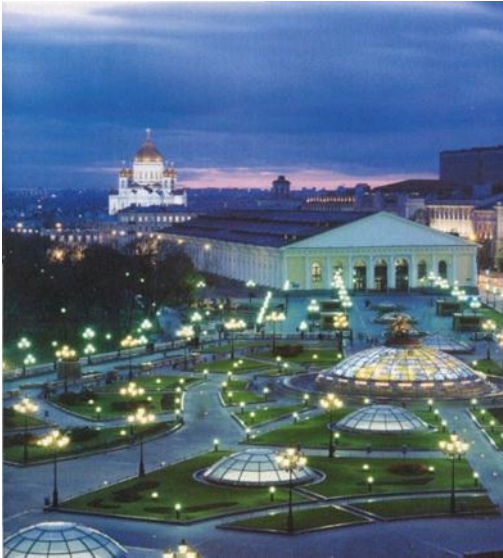


Figure 2a. Moscow underground shopping and recreational complex Okhotny Ryad

Similarly, the reconstruction of the underground part of the Bolshoi Theater in Moscow was carried out at all stages: the construction of a wall in the ground outside the outline of the theater, the concreting of the ceiling at zero level with the support on the wall in the ground, the piling of steel piles and the transfer of loads to them from the historic walls of the building, excavation And concreting four more overlaps from the top to the bottom, concreting the foundation slab and building up the frame from the bottom to the top, cutting off the pressed piles and transferring the load to the framed frame. One of the fragments of the calculation scheme is shown in Fig. 3.

One of the last large facilities, calculated by the technique of «technological mechanics of soils» was the Alabian-Baltic tunnel. Its peculiarity is: a) the tunnel is buried 20 m and all is in water-saturated sands, b) the tunnel passes under the active highway and two tunnels of the metro, for which the permissible draft is 4 mm, Fig. 4. Between the tunnels under construction and the operating transport tunnels, was constructed a buffer zone in which a ground mixture can be added or removed to compensate for the deformations of the existing tunnels. In total, were performed more than 250 stages of calculation, corresponding to the design technology of installation of tunnel, which is especially important for installation of its top overlapping under existing tunnels without stopping traffic on them.

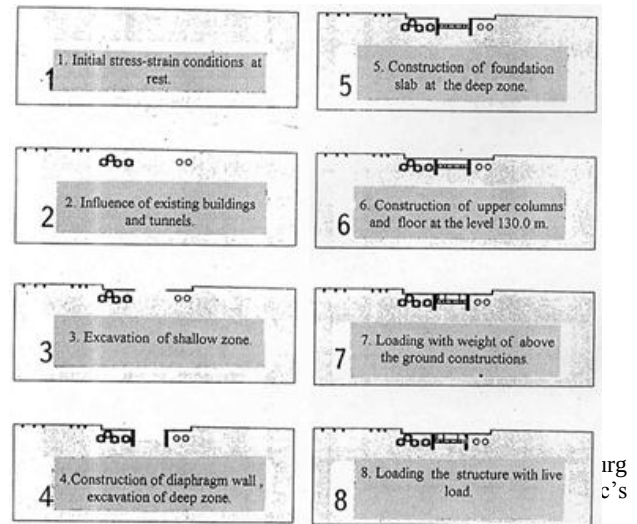


Figure 2b. 8 cycles of the construction process

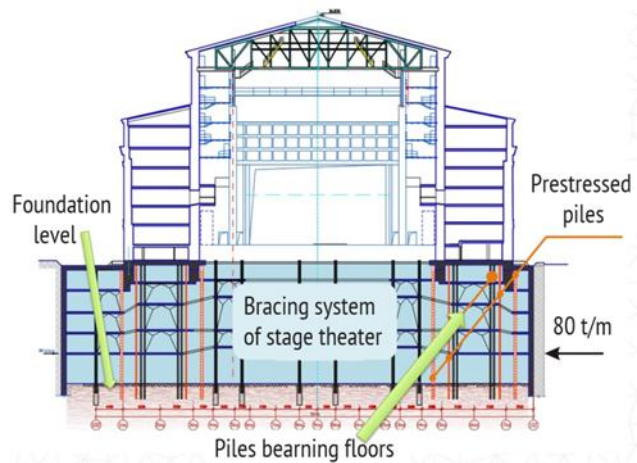


Figure 3 Fragment of the calculation scheme

«Technological mechanics of soils» makes it possible to have at any time a mathematical model of the underground structure under construction, which gives the following advantages:

- systematically monitor the stress-strain state of the object's structures;
- reliably interpret the readings of the sensors of the monitoring system at the facility;
- variant calculations "forward" to identify possible adverse situations. These advantages ensure the safety of construction of complex unique underground facilities. In addition, the «technological mechanics of soils» takes into account the variability of the stress-strain state of the construction under construction depending on the technology and thereby opens the possibility for optimizing the construction technology.

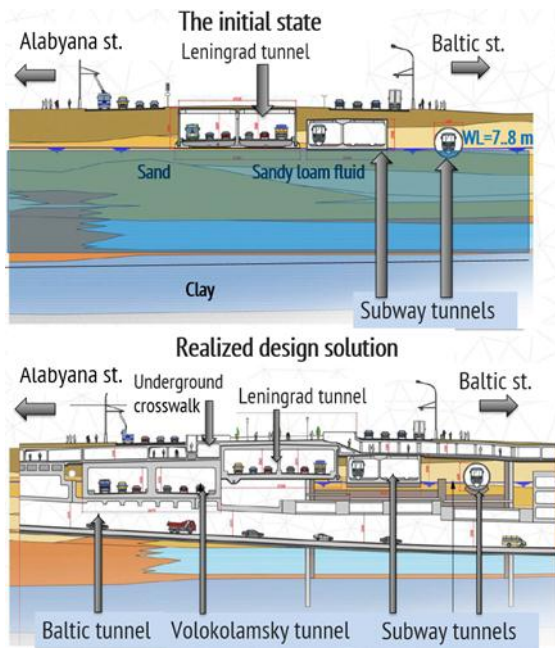


Figure 4. Alabian-Baltic tunnel

In Russia there is a sufficient number of organizations specializing in underground construction. In Moscow, for example, the program of construction of more than 70 new metro stations is being implemented.

The above-described experience in design and construction is used in the construction of building standards and is used in the design of metro stations.

lake sand and silt as well as the glacial lake clay but the piles act as friction piles as the end bearing is small compared to the friction capacity.