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ACCUMULATION OF RAINFALL IN THE PERMEABLE FILL BEHIND A SOIL NAIL WALL

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ABSTRACT

TOOBA deep excavation project was conducted in a densely developed area in the North West of Tehran, capital of Iran, to provide space for 4 basement levels for multiple buildings around the already functional TOOBA tower. Besides excavating on 3 sides of TOOBA tower, this project involved excavation to the depths varying from 9 to 28 meters depending on the sloping ground condition. A 5 story school building and a 2 story residential building abut the excavation boundary.

Previous experience of constructed soil nail walls in cemented soils of Tehran indicated relatively small wall deformations. Therefore, except for the retaining system of the TOOBA tower, a soil nail wall system was seen appropriate for supporting excavation faces. Wall and ground deformations were monitored during and after construction and the ground around the site was regularly checked for tension cracks.

Geotechnical explorations indicated the presence of a disturbed fill about 5 meters thick overlaying the intact cemented soil layer. In the 29th of August 2011 following a three-day rainfall a tension crack suddenly occurred on the eastern side of the excavation site. The maximum width of the crack at the surface of the road was 3 cm. This paper summarizes the information on the forensic study which concluded that the rainfall was confined to the fill layer. Therefore drainage system which was located in the cemented soil layer with lower permeability could not function properly. Limit equilibrium analysis correctly predicted the location of the tension crack and the unstable block.

INTRODUCTION

The deep excavation project for TOOBA commercial complex is located in 35° 45’ 58.41” N, 51° 22’ 12.58” E in a densely developed urban environment in North West of Tehran, capital of Iran. This deep excavation project was aimed at providing space for 4 basement levels for hypermarket and parking spaces of multiple buildings around the already functioning TOOBA tower. The office building block, referred to as TOOBA tower in this paper, is located in the northern leg of the excavation boundary in a way that the northern side of the building abuts the excavation edge. Hence, this project involved excavation on 3 sides of the roughly rectangular plan of TOOBA tower to the depth of 16.5 meters below its foundation level. Other sides of the excavation boundary, on the other hand, were excavated to the depths varying from 9 to 28 meters depending on the sloping ground condition. Fig. 1 and Fig. 2 show the plan of the site and the location of the adjacent structures. A 5 story school building and a 2 story residential building adjoin the excavation boundary from south and west, respectively.

The geometry of the excavation site and the need for space for the movement of heavy equipment in the excavation ruled out the possibility of using a strutting system as retaining wall.
On the other hand, previous experience in cemented soils of Tehran indicated relatively small wall and ground deformations in soil nailed walls. Soil nail walls are passive retaining systems in which tensile forces develop in nails as the result of ground deformation. It appears that in cemented soil of Tehran, nails develop tensile forces with relatively small deformations in the retained soil. This could be as the

result of very stiff interface between nail and grout and grout and cemented soil. Soil nailed walls have been successfully used in Tehran for depths up to 50 meters. Therefore, except for the retaining system of the tower, a soil nail wall system was deemed appropriate for other excavation faces.

The necessity for constraining the deformations of the TOOBA tower commended the construction of contiguous bored concrete piles around the building supported at 4 different levels with tiebacks and wailing. This section of the excavation project has been fully covered in a separate paper by the authors of this paper (Haeri et al., 2012). A monitoring program for measuring the deformations of the tower and supporting system was also enforced during and after excavation.

Fig. 1.Arial photo of the site (Source: "Tehran." 35° 45' 58.41" N and 51° 22' 12.58" E. ***Google Earth.*** *June 30, 2009. September 30, 2012.*

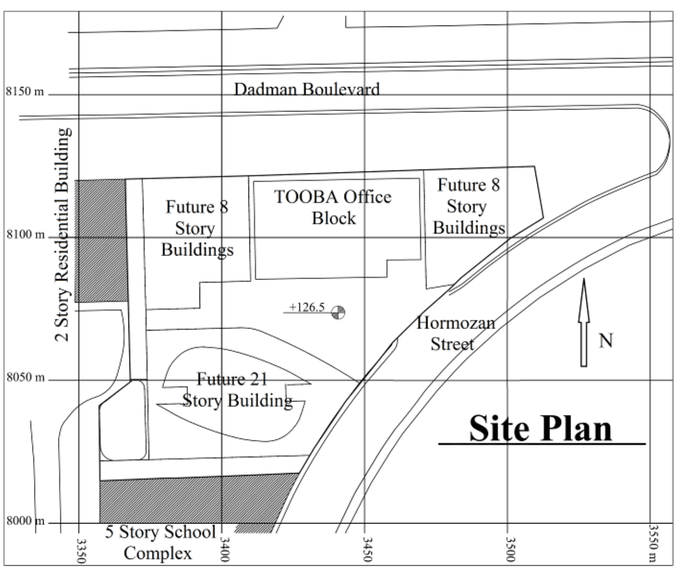


Fig. 2. Plan view of the location of the site, the adjacent and future buildings, and the neighboring streets

The future structures designed for the site incorporated permanent retaining walls in their basement levels. Hence, the soil nail system was designed to be a short term retaining structure (service life of about 4 month). Therefore, hypothetical situations such as occurrence of precipitation with a long return period or earthquake loading were not considered in the design of the soil nailed wall. However, the construction hit unexpected delay due to legal problems between investors and construction process was halted for a period of a year.

As far as the TOOBA tower is concerned, the retaining structure performed well during the entire construction process. The excavation procedure was ensued with no excessive deformations occurring in the building during or after the excavation. The performance of the soil nail wall system, on the other hand, is discussed in this paper.
In August 29, 2011 following a three-day rainfall a tension crack suddenly opened on the surface of the road at the eastern side of the excavation site. Tension cracks could happen in passive earth retaining structures such as soil nail walls; but, an opening to the width of 3 cm very close to the top of the wall over the period of one night after 3 days of continuous raining was interpreted as a signal that a shallow moving block has formed. This paper summarizes the information on the forensic study which concluded that the rainfall was confined to the fill layer. Therefore drainage system which was located in the cemented soil layer with lower permeability could not function properly. Limit equilibrium analysis correctly predicted the location of the tension crack and the disturbed block.

GROUND CONDITIONS

4 years before commencing of the excavation 10 exploratory borings to the depth of 75 meters were performed to provide SPT N-values, obtain soil samples and observe groundwater table. In addition to that 3 observation wells to the depth of 11 meters were used to carry out PLT tests on the soil. Borings were conducted using rotatory augers with wash-boring method and continuous core sampling.

For more careful geotechnical explorations 2 more borings to the depths of 80 meters and 5 more observation wells to the depth of 25 meters were made to complete the available geotechnical information on the site. In-situ density test were performed in addition to PLT tests in the observation wells. The later borings were more resent (after the decision for a deep excavation project for the site was taken). Therefore, these tests were conducted with the deep excavation in mind. Soil samples in the later borings were obtained using core barrel method.

The samples from the borings of both series of explorations were used for laboratory tests including Atterberg limits, sieve test, density, and direct shear tests. Fig. 3 shows the location of test pits and bore holes in addition to the location of the adjacent and future buildings.

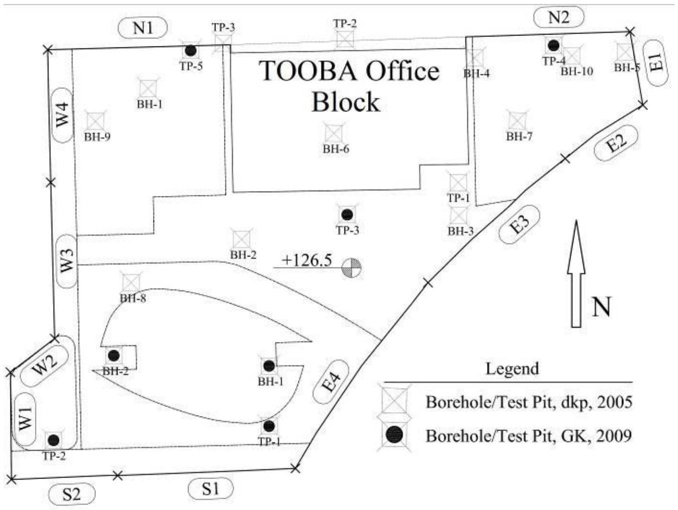
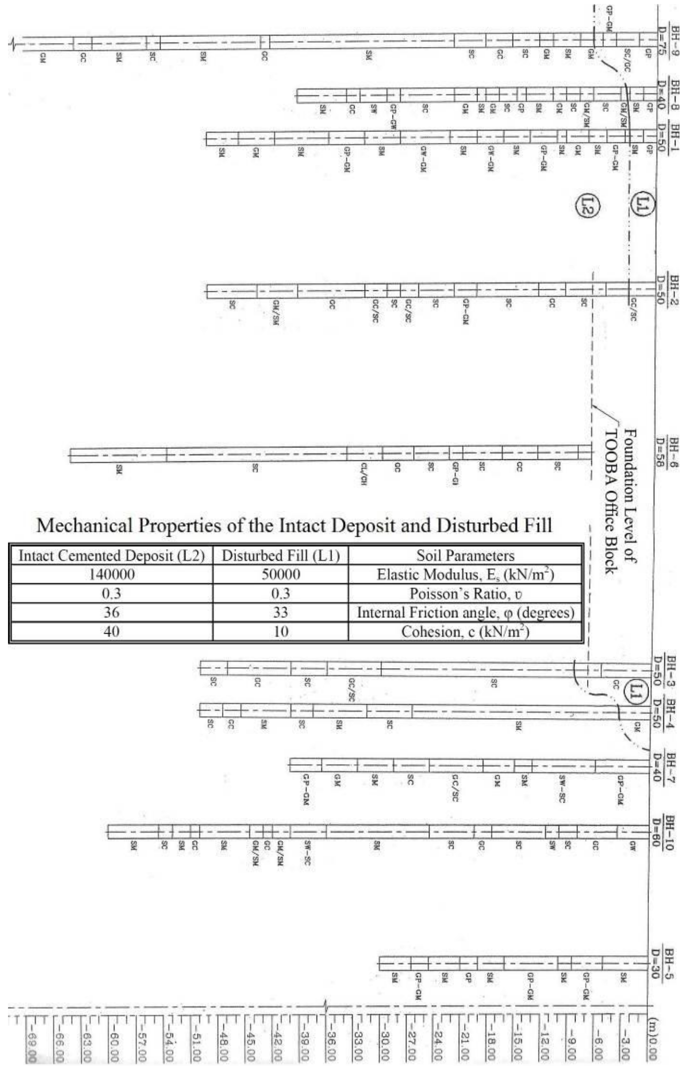


Fig. 3. Location of test pits, bore holes, and the location of the soil nail walls on the excavation boundary

Geotechnical explorations indicate that the site consists of altering layers of silty and clayey sand and gravel (SC, SM, SW, GC, GM, and GP) along with boulders and cementation. Soil to the East of the site is generally sandy, whereas the west of the site tends to contain more gravel. Soil is generally brown or light brown and rarely light green or grey. Fig. 4 shows an example of the sizable boulders encountered in the site.

Fig. 4. The site consists of silty and clayey sand and gravel along with boulders up to few meters in diameter

Past experience of the site shows that the soil is cemented and that its mechanical behavior is heavily affected by the cementation bonds between soil particles. Considerably lower shear strength parameters of remolded and disturbed specimens in direct shear test at laboratory was also an indicator of cementation in the soil. As a result, soil parameters were estimated by interpreting the in-situ test results. Detailed information on the characteristics of the cemented soil of Tehran has been published by a number of researchers (Haeri and Hamidi [2009], Hamidi and Haeri [2008, 2005], Haeri et. al. [2006, 2005a, 2005b, 2004, 2003], Asghari et. al. [2004, 2003])



Mechanical Properties of the Intact Deposit and Disturbed Fill

Intact Cemented Deposit (L2)	Disturbed Fill (L1)	Soil Parameters
140000	50000	Elastic Modulus, E_s (kN/m^2)
0.3	0.3	Poisson's Ratio, υ
36	33	Internal Friction angle, φ (degrees)
40	10	Cohesion, c (kN/m^2)

Fig. 5. The disturbed layer (L1), the intact deposit (L2) and the alternating sand and gravel sub layers from the bore holes

Based on the SPT test results the soil to the west and south of the site is very dense and has corrected SPT value of higher than 50. However, a loose fill material was detected in the vicinity of the TOOBA tower to the depth of 8.5 meters below the ground surface. The foundation of the tower is located 8.5 meters below the ground surface on the dense soils. This could indicate that the pit for tower construction was excavated using slopes which were subsequently filled with the in-situ soil and BH8 and BH9 bore holes are probably located within this trench. Fill material was also present on the east side of the site to a depth of 7 meters.

To the authors' knowledge, this could be as the result leveling the geological folds in the area before development. It is well known that the site, which is located on a formation of geological folds consisting of shallow synclines and anticlines, has been leveled before the area turns into the heavily developed urban environment. In other words, the soil on the higher ground was cut and filled in the shallower areas without sufficient compaction effort.

Aerial photos of the site from 1969 corroborate our knowledge about the geological formation of the site. Aerial photos from 1969 of the area show the area before any construction development occur. Unfortunately, these pictures cannot be reproduced here for copyright issues. But, a syncline on the east of the site might explain why the bore hole BH9 indicate that disturbed fill is present at this part of the site to an approximate depth of 7 meters.

As discussed before, the site consists of alternating sub layers of sand and gravel with isolated patches of very hard clay encountered occasionally (for example a hard clay layer (CL/CH) was encountered at the depths of 23 to 27 meters below the TOOBA tower). Despite the fact that the site is very heterogeneous, but, from the mechanical point of view the site stratigraphy can be divided into two distinct layers, both consisting of sandy and gravelly sub layers.

The mechanical properties of these 2 layers are shown in Fig. 5 (the disturbed layer (L1) and the intact deposit (L2)). These two layers can be described as follows: A very dense cemented layer which is intact. And, a disturbed fill layer which has the same grain size distribution of the underlying soil but the broken cement bonds and lower compaction in this layer gives the soil lower elastic modulus, lower shear resistance and probably higher permeability. Therefore, it can be described as a medium dense soil.

The mechanical properties shown in Fig. 5 have been obtained based on the results of the field and laboratory tests and engineering judgment, and were used in some phases of the design. The presumed boundary of the disturbed layer has been sketched with a dash double-dot line based on the results of geotechnical explorations. Fig. 5 also shows the alternating sand and gravel sub layers discovered from the bore holes.

GROUND WATERTABLE

During the geotechnical investigations a number of abandoned drainage wells (drains) were discovered to the north and east of the TOOBA tower. The water level in the borings and observation wells were extremely different in each boring and well. It is thought that the water level in borings is affected by penetration of water from abandoned drains and flumes. For example test pit TP4 (See Fig. 3) was engulfed by water when it reached sand lenses at 9m depth. The rate of water entering the well was so high that efforts for pumping the water out for continuing the boring of the well were abandoned.

Hence, initial design considered the possibility of localized groundwater intrusion into the pit during excavation. Drains were also installed near the bottom of the excavation. Drains were also designed to conduct the waste water from TOOBA tower to the drainage network and avoid accumulation behind the retaining structure.

No permeability tests were conducted on the soil but the formation of the shallow sliding block as the result of raining shows that the crushed nature of this fill has given it higher permeability as well.

Fig. 6. Construction sequence: Excavation to a stable depth

Fig. 7. Construction sequence: Installation of soil nails - Drilling

CONSTRUCTION SEQUENCE

Soil nailed wall technique utilizes reinforcing elements to stabilize cuts and slopes and is usually constructed using top-down excavation method. The usual building sequence consists of excavating the cut to a stable depth (Fig. 6), installing the tensile elements or soil nails (Fig. 7) and applying a temporary mesh and shotcrete facing (Fig. 11 and Fig. 12). The same procedure is then continued until reaching the final excavation grade. The construction sequence and technique for these walls is well described in the literature (i.e.

Byrne et al 1998, Lazarte et al 2003). When there is ground water intrusion in to the excavation pit or a danger of instability exists sometimes steel mesh and shotcrete facing can be applied before nail installation.

Soil nail installation process consists of following steps:

- Drilling a hole 90 mm in diameter with the specified inclination and length (Fig. 7)
- Preparation of the steel bars by fitting the centralizers (Fig. 8), injection pipes (Fig. 8), and threading the nail head for installation of heavy duty nuts (Fig. 9)
- Grouting (Fig. 10)

Plastic centralizers were used to assure that a minimum grout cover around the nail bar is achieved. Soil nails were made using steel bars with 32 millimeter in diameter and 4000 kg/cm2 ultimate tensile strength. Nail head was sealed before cement grout get injected under the pressure of 2 bars.

Fig. 8. The centralizer and the injection pipe attached to the nail

Fig. 9. Steel bars are threaded so that a heavy duty nut can be fitted to the nail head

Wall and ground deformations were monitored during and after construction and the ground around the site was regularly checked for tension cracks. Lack of visible tension cracks after the excavation was over confirmed negligible wall deformations that are typical of soil nail walls in the cemented soil of Tehran.

As shown in Fig. 6, excavation for soil nail walls was usually completed in lifts; however, near displacement sensitive structures and whenever there was a danger of water intrusion into the pit, the excavation was done in blocks of 6 to 10 meters wide. Fig. 13 shows one such occasion.

Fig. 10. Construction sequence: Installation of soil nails – insertion of nails into the drilled holes and preparing for injecting the grout

Fig. 11. Construction sequence: Temporary facing using steel mesh and shotcrete

GENERAL CHARACTERISTICS OF THE SOIL NAIL WALLS

Contractors familiar with soil conditions in an area sometimes give very good initial estimate of the cost of soil nail wall construction based on nail density parameter. Nail density is defined as the sum of the length of the soil nails divided by the surface of the wall. In other words, nail density is the average length of nails used per square meter of the finished excavation face. Nevertheless, nail density for the soil nail walls of this project differed from 0.8 m/m^2 for W1, the 8 meter tall soil nail wall on the west of the site, to 3.4 m/m^2 for the 28 meter tall N2 soil nail wall on the north.

Fig. 12. Construction sequence: Temporary facing using steel mesh and shotcrete

Fig. 13. Excavation for the soil nail walls in blocks rather than lifts at some locations for increased stability

Nails are usually spaced 2.5 meter vertically and have horizontal spacing of 1.5 and 2 meter depending on the wall. Fig. 3 shows the location of soil nail walls. Each soil nail wall is named with a letter and a number indicating its position. Fig. 14 shows a schematic cross section and front view of the W1 soil nail wall to give the reader an idea of soil nail pattern on the walls. Fig. 15 , on the other hand, depicts nail head configuration designed to be used on some of the nails in this project.

The site is excavated to a constant level throughout the excavation site. Fig. 16 shows W3, W4 and N1 soil nail walls before excavation reaches its final grade. On the other hand, S1, S2, E4 and W1 soil nail walls and W1, W2 and W3 soil nail walls are shown in Fig. 17 and Fig. 18 respectively. The School complex on the southern leg of the excavation boundary is depicted in Fig. 17 and Fig. 19.

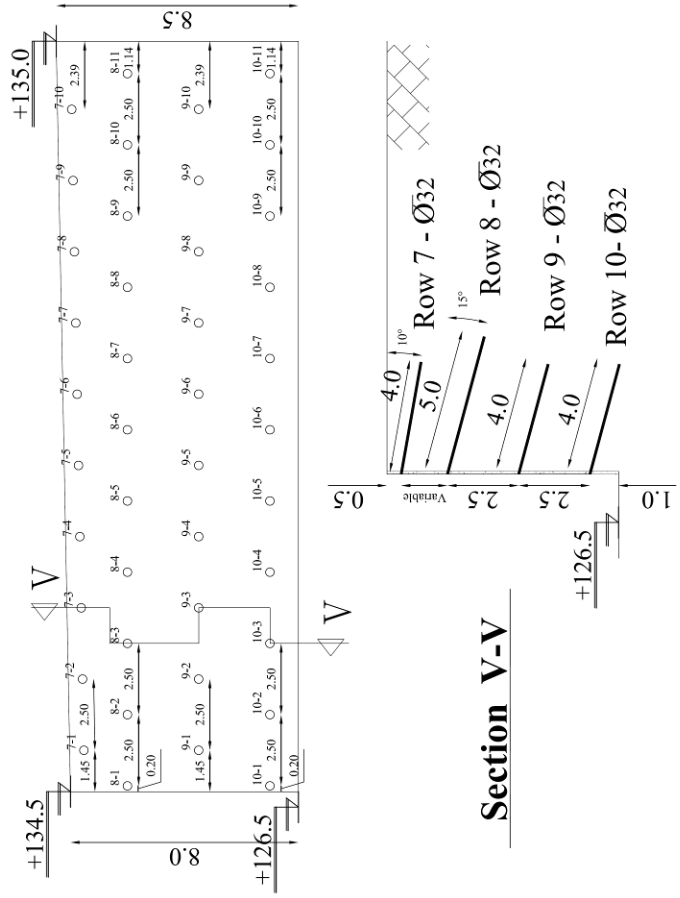


Fig. 14. Cross section and front view of the W1 (dimensions are in meters)

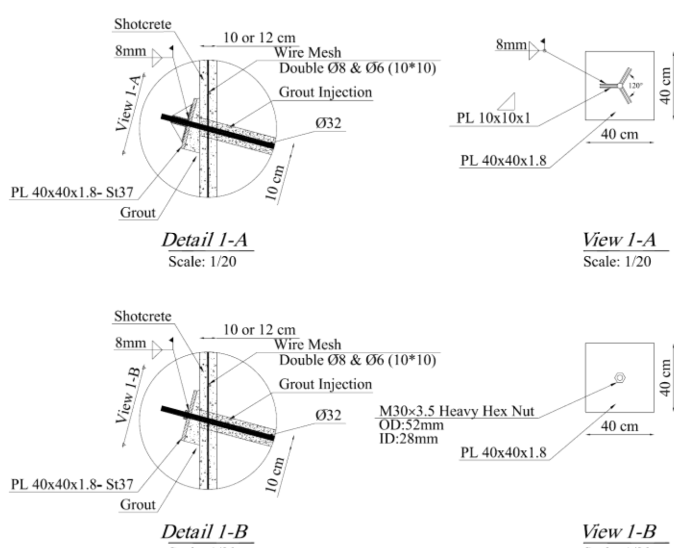


Fig. 15. Design details of the nail head configuration

Careful and economical design of soil nail walls required that the exact value of bond strength between soil nails and soil to be known. Therefore, nail pull out tests were conducted using the specifications proposed in FHWA-SA-96-069R (Byrne et al, 1998).

Fig. 16. W3, W4 and N1 soil nail walls during the excavation process

Fig. 17. S1, S2, E4, and W1 soil nail walls after excavation to the full depth

Fig. 18. W1, W2, W3 soil nail walls after excavation to the full depth

Fig. 20 and Fig. 21 show a typical nail pull out test apparatus and the test apparatus setup at the site, respectively. Pull out tests were conducted on 2.5 and 3.5 meter nails. Drilled hole diameter was 90 mm and the grout was 7 days old when the tests were conducted. None of the tested nails failed by pull out. Fig. 22 shows the results of pull out tests on 3 nails. This chart indicates a stiffness of about 58200 kN/m which can be considered a very stiff load deformation behavior for the soil nail. Prolonged loading of 15 tons showed no creep.


Fig. 19. A view of the southern part of the excavation pit captured from the roof top of the TOOBA tower

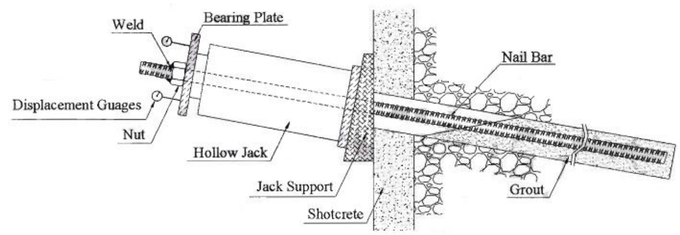


Fig. 20. Schematic diagram of soil nail pull out test apparatus


Fig. 21. Pull out test apparatus at the site

INSTRUMENTATION & MONITORING

Compared to the extensive monitoring program for the TOOBA tower, a less sophisticated monitoring scheme was employed for the soil nail walls. In addition to monitoring the deformations of the soil nailed walls, the ground surface and particularly the surface of the surrounding streets were checked regularly for possible tension cracks.

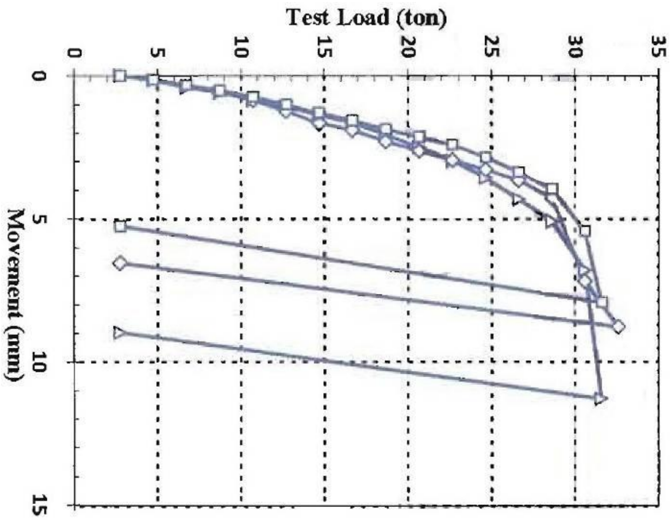


Fig. 22. Pull out test results

Fig. 23. Targets installed on the shotcrete facing and nail heads

Fig. 23 and Fig. 24 show examples of the targets installed on the shotcrete facing and nail heads. These targets were monitored regularly. Deformations of the southern soil nail wall (near the school building) and western soil nail wall (near the residential building) were within acceptable limits. No signs of cracking were observed in the school and residential buildings. Deformation pattern at these wall sections were compliant with the deformation of the soil nail walls published in the literature and numerical modeling of the wall with PLAXIS (maximum horizontal deformation at the end of excavation occurred at the middle third of the height of the soil nail wall). The main purpose of this paper is discussion of the practical aspects of this project; therefore, a complete discussion of the numerical modeling with PLAXIS is beyond the contents of this paper. Deformations after each excavation lift remained constant therefore no evidence of creep deformations was observed. On the other sections of soil nail walls, however, the measured vertical and horizontal deformations were negligible and were very close to the tolerance of the reading device. In other words, deformation recordings from these sections did not produce meaningful results.

Fig. 24. Target installed on the shotcrete facing

Recording of the deformations of the soil nail walls was continued until 14th of June 2010 well after the excavation process was completed.

APPEARANCE OF WATER MARKS ON THE SURFACE OF SHOTCRETE

When the wet season began in the region and the amount of precipitation increased measures had to be taken to remove the water accumulation inside the pit. Fig. 25 shows the pond in the site where the water was pumped out after accumulation.

Fig. 25. Accumulation of precipitation water in an excavated pond

Moreover, the initial concern that there might be localized intrusion of ground water into the excavation pit proved to be correct and water marks began to form on the surface of the shotcrete on some of the soil nailed walls.

At this point the drains incorporated just above the bottom row of soil nails did not seem to function as expected. It appears that the lower permeability of the intact cemented soil and existence of pockets of permeable materials conducts the water towards the shotcrete facing without allowing water pressure dissipation through the lower drains. Fig. 26 shows water marks formed on the surface of the E4 soil nail wall.

Fig. 26. Water marks on the surface of the E4 soil nail wall

The quicker relief of water trapped behind the shotcrete facing was accomplished by scratching the surface of the shotcrete at the location of the large water marks for drains to be installed. Fig. 27 and Fig. 28 show the parts of the shotcrete that were scratched and the finished temporary surface with the drains installed.

Fig. 27. Shotcrete scratched at the location of the water marks

TENSION CRACK AFTER PROLONGED RAINING

In August 29, 2011 following a three-day rainfall a tension crack suddenly opened on the surface of the road at the eastern side of the excavation site, just behind the E3 soil nail wall. The maximum width of the crack at the surface of the tarmac was 3 cm. The speculation was that the actual crack width beneath the tarmac could be wider. Later reports show that the width of the crack increased to 7 cm during the next day. Tension cracks could happen in passive earth retaining structures such as soil nail walls; but, an opening to the width of 3 cm very close to the top of the wall over the period of one night after 3 days of continuous raining was interpreted as a signal that a shallow moving block has formed.

Fig. 28. Finished surface after installing the drains

As discussed previously, the monitoring of the soil nail walls was ended at 14th of June 2010; by this time the excavation procedure had been completed and the next phase which was the construction of the buildings had already begun. Therefore, no record of the deformations of the soil nail wall is available immediately before appearance of the tension cracks. Deformation recordings up to this time, as discussed before, show very small deformations in the eastern soil nail walls.

Fig. 29. Frozen water at the tip of a drain pipe installed on the soil nail wall (the drain was installed at this location after the water marks appeared on the shotcrete facing)

<u>Important Infrastructure at the Vicinity of the E3 Soil Nail Wall</u>

The study of subsurface infrastructures near the excavation pit indicated that there is a high pressure gas pipe to the south east of the TOOBA tower which supplies the tower. Fig. 30 shows the position of the gas pipe the crack and their relative positions to the E3 soil nail wall and the TOOBA tower. The gas pipe has held in place using a strut system since the beginning of the excavation as shown in Fig. 30. In this picture, water marks are also evident on the shotcrete surface of the E3 soil nail wall.

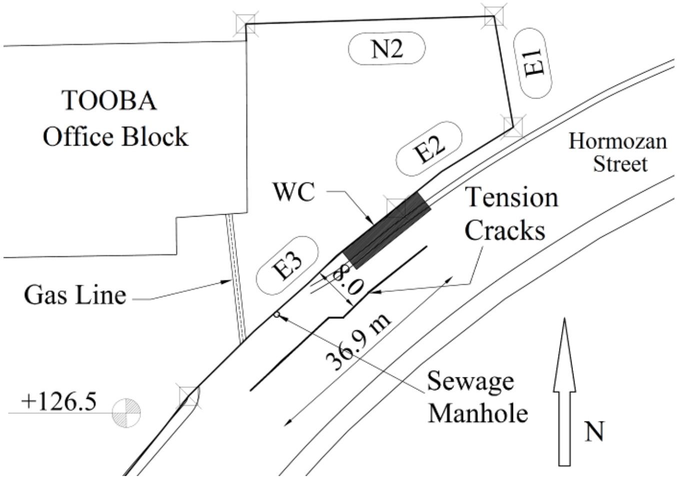


Fig. 30. Positions of the subsurface infrastructures near the excavation pit

Fig. 32 and Fig. 33 show schematic cross section and front view of the E3 soil nail wall and depict the location of the underground infrastructures.

Fig. 31. Strut holding the gas pipe between the E3 soil nail wall and the TOOBA tower

Short Term Remedial Measures

All the cracks on the surface of the road were closed with proper combination of tarmac to avoid further intrusion of water into the tension crack.

The scratching of the shotcrete at the location of watermarks alone allowed that some of the accumulated water behind the wall to dissipate quickly. However, 6 meter long handmade drains which consisted of a grooved PVC pipe wrapped inside a geotextile cover were installed to help dissipate the rest of the water possibly trapped inside permeable deposits behind the wall. Fig. 34 shows the strut from a different angle while workers are installing the drains on the locations were water marks have appeared on the surface of the shotcrete.

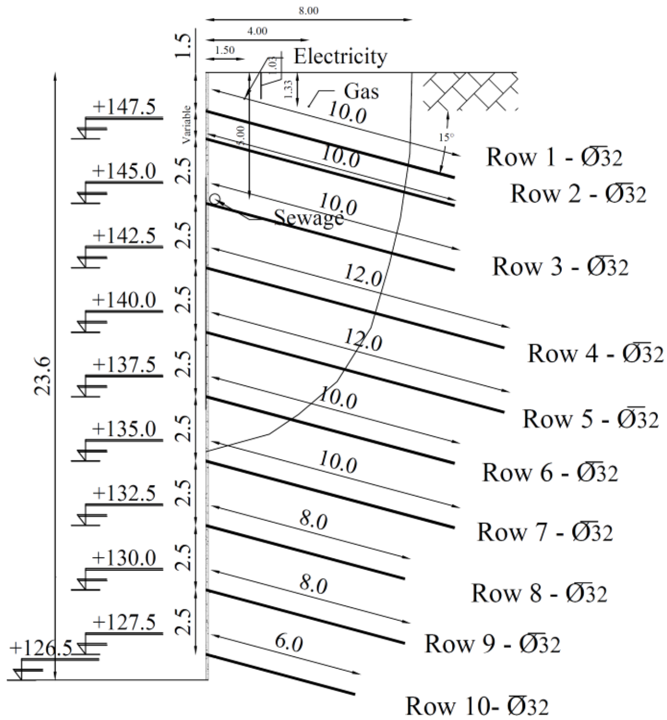


Fig. 32. Cross section of E3 soil nail wall (view XII-XII)

A new monitoring scheme was developed to monitor the further movements of the wall and the strut with prism targets. The quality of the water escaping from the drains showed that the waste water from the water closets shown in Fig. 30 might have been leaking therefore escalating the situation of the E3 soil nail wall. Therefore, these water closets which were made for the construction force at the vicinity of the wall were relocated. The sewage pipe shown in Fig. 33 was used to carry the waste water from the tower to the main sewage system of the area. Construction crow assigned with the task of preparing the shotcrete for the temporary facing and the grout for injection sometimes used the manhole shown in Fig. 30 and Fig. 33 to discharge their excess water/cement mix to the sewage pipe. Although care was taken for the water/cement mix to be very lean when discharged, it seemed possible that the presence of waste water in the ground water behind the wall could have been due to the clogging of the pipe by the discharged grout. Therefore, the pipe was abandoned and the waste water from the tower was carried through a different route.

The temporary shotcrete facing of the soil nail wall was carefully investigated for any signs that could indicate that a moving block of soil has been formed. Based on the fact that the crack occurred 8 meters from the excavation boundary, it was expected that the toe of the block be located at 14 to 15 meters below the top of the wall. Since no manifestations of the crack was observed on the wall surface, the premise that soil block is moving was abandoned. Further monitoring of the E3 soil nail wall did not indicate progressive deformations.

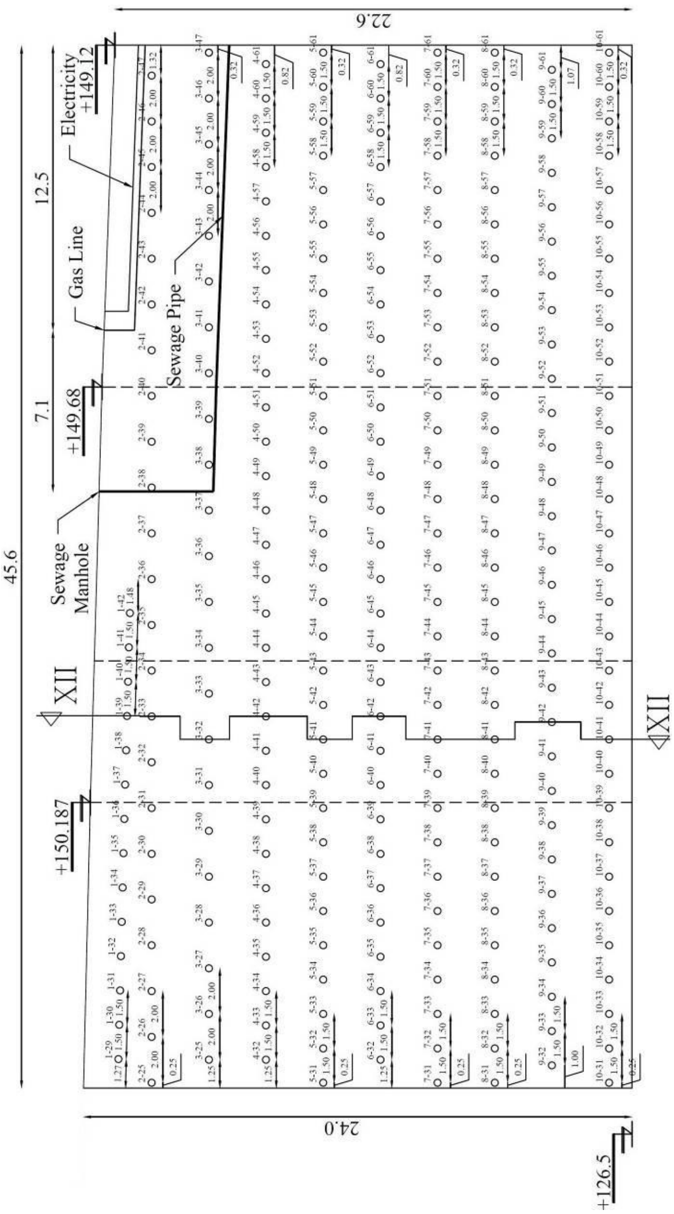


Fig. 33 . Front view of E3 soil nail wall

SUMMARY & CONCLUSION

This contribution documented a deep excavation work in north-west of Tehran which included excavating very close to 3 sides of a 21 story functioning office block and two other structures. A monitoring program for measuring the deformations of the retaining systems of the excavation faces and the adjacent buildings was enforced during and after excavation.

The purpose of the deep excavation is to accommodate enough space for 4 basement levels for a 21 story building in the south of the site and multiple 8 story buildings on the rest of the site. The project site consists of layers of silty and clayey sand and gravel with boulders. The difference between the in-situ and laboratory tests indicated that soil is highly cemented. As a result soil parameters were estimated by interpreting the in-situ test results.

Fig. 34. Workers installing the drains on the locations were water marks have appeared on the surface of the shotcrete

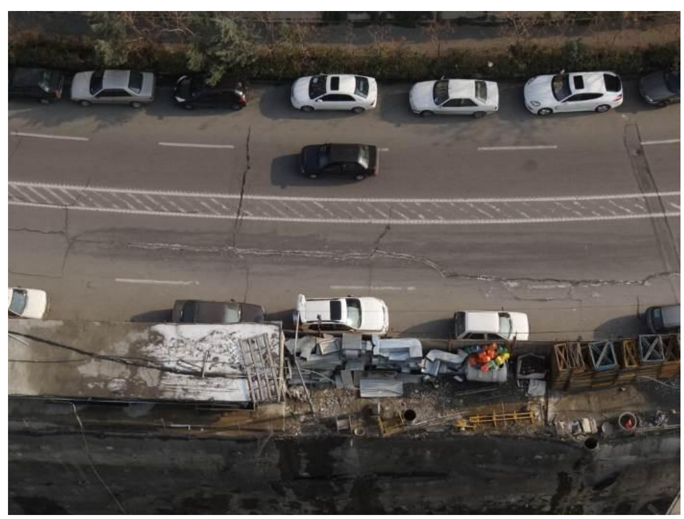


Fig. 35. View of the crack at the surface of the tarmac captured from the roof top of the TOOBA tower

From geological point of view, the site was located on a formation of geological folds consisting of shallow synclines and anticlines. Aerial photos from 1969 of the area which show the area before any construction development occur corroborate our knowledge about the geological formation of the site. More recent aerial photos of the site show that the site was leveled before the area turns into the heavily developed urban environment of today. In other words, the soil on the higher ground was cut and filled in the shallower areas without sufficient compaction effort. Therefore, a disturbed fill layer which has the same grain size distribution of the underlying soil but the broken cement bonds and lower compaction in this layer gives the soil lower elastic modulus, lower shear resistance and probably higher permeability was present in some locations of the site.

The retaining structure was supposed to be a short term one designed for a 4 month period. However, the construction hit unexpected delay due to legal problems between investors and construction was halted for a period of a year. Whereas the retaining structure for the tower performed well during the entire construction process, the performance of the soil nail walls has been documented in this paper.

Previous experience with soil nail walls and anchored retaining walls in the cemented soil of Tehran indicates the injected grout forms a very strong and very stiff interface between the installed tensile elements and the cemented soil. This well documented project is an example of performance based design in geotechnical engineering.

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