



A Comparison between the Lateral Deformation Monitoring Data and the Output of the FE Predicting Models of Soil Nail Walls Based on Case Study

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ABSTRACT

Very often, predictions of the deformation behaviour of a soil-nailed structure through FEM are required to ensure that displacement limits set by the construction requirements are not exceeded. Besides, in-situ monitoring of deformations can be used in verifying design parameters and providing early warning of impending failure. The results of the performance of soil nailed walls in Baran project are compiled in this study. The in-situ monitoring data is compared with the output of the predictive FE numerical analyses. The lateral displacements based on the in-situ surveying data generally agree with the FE modeling data.

RÉSUMÉ

Très souvent, prédiction de comportement de la déformation du structure d'un sol cloué par FEM est nécessaire afin d'assurer que le déplacement des limites fixées par les prescriptions de construction ne soient pas dépassées. En outre, la surveillance in situ des déformations peut être appliqué pour vérifier les paramètres de conception et de l'avis précoce en cas de défaillance imminente. Les résultats de la performance des murs en sol cloué du projet Baran est enlevé dans cette étude. La surveillance in situ de données est comparée à la sortie d'extrapolation des analyses des numériques FE. Les déplacements latéraux sur la base des enquêtes in-situ de données sont généralement en accord avec la modélisation des données FE.

1 INTRODUCTION

In the past forty years, one of the major trends in earth retaining structure design has been toward reinforcement methods which improve the internal strengths of soil masses sufficiently to make them self supporting. These methods include the New Austrian Tunneling Method (NATM), Reinforced Earth and soil nailing, as examples. In recent years, due to the demand for the construction of high-rise structures, and due to the need for obtaining enough parking spaces, soil nailed walls are extensively employed within the city of Tehran as temporary retaining walls to allow great number of basements below these tower structures. Soil nailing is an in-situ earth reinforcement method which enables an earth mass to achieve a state of self-support through the introduction of driven or grouted steel bars (called nails) into the mass during excavation and exposure. Slope stability is increased by the mobilization of shear stresses at the soil-nail interface in the resisting zone below (or beyond) the critical slip surface. The excavation face is supported by a weather-resistant facing, most commonly shotcrete.

Pazhoohesh Omran Rahvar Consulting Engineers has been engaged with numerous deep urban excavation Engineering-Procurement-Construction (EPC) projects through the City of Tehran, whose vertical cut heights vary between 12 to 36 metres.

Soil nailing is a very versatile excavation retaining system for deep excavations in urban areas surrounded by major structures and other infrastructure provided that limiting lateral displacements are not exceeded. Excessive displacements are normally followed by the formation of visually noticeable tension cracks in adjacent structures, causing concern. Due to project constraints imposed by the Municipality of Tehran and other local authorities to limit the impact to the adjacent structures and environment of the site so as not to impede the public transportation flow in urban commuter routes and not to induce panic in occupants of the nearby structures, precise monitoring and applying numerical FE based design approach for risk assessment has taken on a great importance. The latter will be highlighted in this paper. Neither minor single lane closures, temporary evacuation of the adjacent residential areas, nor interruption of daily irrigation of the green areas beside the excavation line are permissible within the course of soil nail wall construction in any project.

The results of the performance of temporary soil nailed walls in the Baran Commercial Tower EPC project, of cut height reaching to about 29 meters at 90° to the horizontal is compiled in this study. The applied steel nail bars are of 28 to 32 mm in diameter and 6 to 14 m in length. The vertical and horizontal spacing varies between 1.25 to 2.5 metres, while the predrilled holes have diameter of 76 mm and an inclination of 10° to 25°.

depending on the distance from the surface, width of the soil benches (along the excavated walls), and the ground water level. A special algorithm of pressurized grouting is applied. The grouting pressure and the grout ingredients tightly depend on the soil properties and the environmental conditions. The applied construction method, cutting panels' dimensions, soil geotechnical properties (specially the shear strength properties, grain size distribution, plasticity, and the permeability properties), underground water condition, available surcharges in vicinity of the project line, soil nail density, drainage system configuration including the inclined drains and the laid down geotextile strips, dewatering techniques such as pumping from the open sumps, grout mixture's specifications (i.e. cement mixing quantity, w/c ratio, ...) and the facing characteristics are briefly discussed.

The performance of walls is monitored by Laser Total Station recordings taken at certain time intervals in parallel to the excavation process at the perimeter of the project line. The next sections present a comparison between the FE modeling outputs and the achieved results from the in-site persistent detailed monitoring.

2 PROJECT GENERAL SPECIFICATIONS

Baran project site is located along Shariati Street in the north of Tehran. The complex with about 322,048 m² building space will contain about 30 floor levels above the ground and 7 to 9 levels of basement.

Conventional excavation support systems were originally studied to support these cuts. However, given the large potential spans, an internally braced system was found not to be feasible. Therefore, a soil nail system was considered. The soil nail and shotcrete system consisted of approximately 2800 soil nails, and 10584 m² of exposed shotcrete wall surface. The total volume of excavation is about 220000 bank cubic metres. Figure 1 illustrates the East view of the project site.



Figure 1. The east view of the project site

The site is surrounded by 3 to 6 floor residential buildings to the north, south and east. The crowded Shariati Street abuts the project site to the west. Adjacent structures are located the approximate distances of 5 to

15 metres from the excavation line. The original site topography slopes at 2.2% from the NW to the NE, 2.9% from SW to SE, 1% from SW to NW, and 1.4% from SE to NE of the property. The top elevation (EL) of the NW, NE, SE and SW corners is 104.04, 102.70, 100.81 and 102.61 respectively. However the excavation was performed to EL 76.65. At the deepest cut point, the proposed structure cuts into the original slope, resulting in a total cut of 28 m at the NW corner to reach the lowest 9 basement level. Figure 2 shows the topography of the project site.

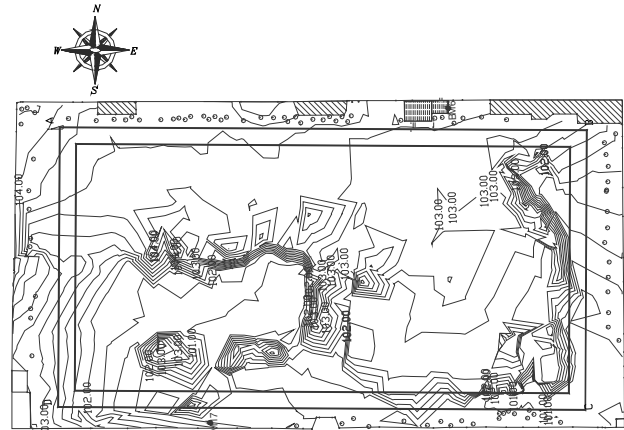


Figure 2. The topography of the project site

2.1 Geotechnical Conditions

Comprehensive geotechnical investigations were performed to provide required data for the design process. A total number of 10 boreholes and 6 test pits were drilled within the project site. The investigations were tailored to characterize the soil condition. SPT, CPT, Pressuremeter, sand-cone density test, PLT, in-situ permeability test, the patented test of 'Loading at the edge of the trench' (Asghari, 2002) and other routine soil property tests were conducted. It is important to note that as typical with most projects the geotechnical investigation was tailored to the subject site and did not extend outside of the property line. Subsequently, no information related to the stratigraphy was available within the zone of influence of the soil nails. In general, water levels observed within the test pits/boreholes revealed a fluctuating pattern that was generally perched on top of the existing clayey soil layer.

According to the obtained geotechnical data, four separate geological sections are considered within the Support of Excavation (SOE) design process. Type A is attributed to the west wall and parts of the north wall (an alignment of 40 meters in length from the west wall). Type B is attributed to the middle alignment of the north wall (of about 40 meters in length), while Type C is attributed to the rest of the north wall (an alignment of about 40 meters in length between Type B and the east wall). Type D applies to the east and south walls. The design of the system was accomplished for the four types in such a way that an appropriate wall height along the relative

zone is considered in each case. The crest elevations in Types A, B, C and D are 103.8, 103.4, 102.7 and 101.1 respectively. Since types have been arranged in such a way to ease the design process, the variation of crest elevations across the project line are disregarded within the models. The average elevation of the ground water table at the location of Types B and C is about 91.2. The average ground water level at the location of Types A and D are about 85.8 and 85.6, respectively. However, by applying special dewatering/drainage techniques, the ground water level can be considered at EL. 85.8 for all types. The stratigraphy, thickness of each layer, and soil properties for each typical section are presented in Table 1. Within the table, 'E' stands for the vertical modulus of elasticity, 'ν' stands for the Poisson's ratio, 'C' stands for the cohesion, 'Φ' stands for the internal friction angle and 'γ' stands for the moist density.

Table 1. Stratification and soil properties¹

Type A.						
Soil Type	Thickness	Φ (°)	C	γ	E	ν
Fill	1-m	15	5	17	100	0.4
SC-SM	3-m	28	20	18	250	0.3
Alluvium	5-m	28	25	18	250	0.35
SC-SM2	9-m	34	20	18	350	0.3
CL	3-m	25	35	19	350	0.4
SC-GC	8-m	30	25	19	500	0.3
GC-GM	To the end	36	20	19	600	0.3
Type B.						
Fill	1-m	15	5	17	100	0.4
SC-SM	4.1-m	28	20	18	250	0.3
Alluvium	3.7-m	28	25	18	250	0.35
SC-SM2	15.6-m	34	20	18	350	0.3
CL	2.2-m	25	35	19	350	0.4
SC-GC	2-m	30	25	19	500	0.3
GC-GM	To the end	36	20	19	600	0.3
Type C.						
Fill	1.5-m	15	5	17	100	0.4
Alluvium	6-m	28	25	18	250	0.35
SC-SM2	18.4-m	34	20	18	350	0.3
SC-GC	2-m	30	25	19	500	0.3
GC-GM	To the end	36	20	19	600	0.3
Type D.						
Fill	1-m	15	5	17	100	0.4
Alluvium	5-m	28	25	18	250	0.35
GP-GC	5-m	37	15	18	350	0.3
SC-SM	3.5-m	28	20	18	250	0.3
SC-SM2	4.1-m	34	20	18	350	0.3
CL	8-m	25	35	19	350	0.4
SC-GC	To the end	30	25	19	500	0.3

¹ C (Kpa), γ (kN/m³), E (Kg/cm²)

The permeability values were applied within the FE PLAXIS numerical seepage analyses in order to design the length and inclination of the required drains and the required geotextile strips' configurations. Analyses were conducted on two critical Types of A and C, in order to design the drainage system, with consideration of the natural direction of the ground water flow. The main purpose of determining Types A and C as the critical types is the lower ground water level in Type A (with

respect to the other regions through the project site) and the north-to-south natural path of the underground water flow. The vertical and horizontal coefficients of permeability were assessed by summarizing the laboratory and in-situ Lufran permeability test results. The obtained coefficient of permeability from the constant head and falling head Lufran tests are in the range of 5.4E-5 to 9.8E-7 cm/s and 2.9E-7 to 1.4E-4 cm/s respectively. Furthermore, the obtained coefficient of permeability from the laboratory constant and falling head permeability tests are in the range of 2.9E-4 to 5.3E-4 cm/s and 5.6E-7 to 6.0E-6 cm/s respectively. The values are summarized in Table 2.

Table 2. Permeability properties of the soil layers

Soil Type	K _{HOR} (Cm/s)	K _{VER} (Cm/s)
Type A.		
SC-SM2	4.2E-5	4.2E-5
CL	1.9E-5	3.28E-6
SC-GC	1.31E-5	4.36E-4
GC-GM	3.6E-7	3.6E-7
Type C.		
SC-SM2	3.18E-5	3.18E-5
SC-GC	1.31E-5	4.36E-4
GC-GM	3.6E-7	3.6E-7

2.2 Adjacent structures

An old 6 storey administrative building is located at the north west corner of the project site, a distance of 12 metres from the project line. Likewise, a number of 3 to 4 storey residential and administrative buildings, back yards, and green areas are located across the north wall within 10 to 20 metres from the project line. The west project line is about 10 metres from Shariati Street, which is at least 20 metres wide. A number of 3 to 4 storey residential buildings are located within a distance of about 11 metres from the southern project line. The narrow Dashtestan Alley is located within 6 metres of the southern line before the residential buildings. At last, a 2 storey house and a private garden are located within a distance of 5 metres from the eastern project line.

Loads are assigned to the models with respect to the dead and live loads of the adjacent structures and their distance from the project line. However, it is essential to consider a working load of about 2 to 5 Ton/m² in the immediate vicinity of the project line to cover the design requirements for crane/cement silo/stores installation. The width of this loaded area depends on the predicted space for the construction facilities.

3 DESIGN AND ANALYSES

3.1 Support of excavation system

The soil nail system was designed conventionally based on the Federal Highway Administration (FHWA, 2003) guidelines and the LE Slope/W 2004 software.

The critical design profile consisted of an outer cut from the top elevation to 94.0 (7.1 to 9.8-m), the first two-metre wide horizontal bench, then the 9 metres height middle cut down to the EL 87.0, the second two-metre wide horizontal bench, and then the 10.3 metres height inner cut down to EL 76.65.

The excavation had to proceed in a staged pattern. Each lift had a height of about 2 m and a width of 6 m. The free face was immediately covered with a 10 cm thick layer of temporary facing. Temporary facing consisted of shotcrete and cold-drawn steel WWM sections (and additional waler bars around the nail heads at the lower elevations to resist the punching forces).

Given that the excavation was surrounded by water from the approximate elevation of 86.0, the potential for high water inflows was a major concern. The potential for high water pressures developing on the facing defining potential sliding wedges was also a concern. Further, due to the lack of working space, the application of curtain grouting was not possible. Thus, to prevent water pressure from developing behind the wall facing, inclined four inch drains are accompanied with vertical geocomposite strips of 30 cm width were installed from the EL 87.0. Drains, in length of 4 to 6 m, were implemented at 2.5 metre horizontal spacing's with an inclination of 10° to the horizontal. The typical vertical spacing was 3 metres. The drainage system also included a footing drain and weepholes to convey collected drainage water away from the wall face. A number of temporary open sumps and trenches were excavated during the construction procedure which included five submerged water pumps.

3.1.1 North and west walls

For the north and west walls, the critical design profiles are Types A, B and C.

The soil nail wall design for the north and west upper cut consisted of 4 rows of nails on a 2.0 by 2.5 m nail pattern. The installation elevations of the upper cut nails in a top to down configuration were 101.0, 99.0, 97.0, and 95.0. All nails were inclined at 15 degree angles, except where the nails were inclined at 10 degrees within the first row to avoid the urban underground utilities. Within the first two rows 12 m length nails are applied, while for the next two rows 14 m length nails are used. The applied bar sizes are 28 mm in diameter (#28 AIII bar) with the yield stress of 4000 kg/cm².

The soil nail wall design for the north and west middle cut consisted of 3 rows of nails on a 2.5 by 2.5 m nail pattern. The installation elevations of the inner cut nails in a top to down configuration were 93.0, 90.5, and 88.0. The distance of each nail from the upper/lower bench is about 1 m. All nails were inclined at 15 degree angles, except the nails installed in EL 88.0 which were inclined at 25 degrees to meet the particular requirements of specific grouting system. Fourteen metre length 32 mm diameter (#32 AIII bars) nails are applied for the three rows in the middle cut.

The soil nail wall design for the north and west inner cut consisted of 6 rows of nails. The installation elevations of the inner cut nails in a top to down configuration were 86.0, 83.5, 82.0, 80.5, 79.0 and 77.5.

Likewise, the nail lengths in a top to down figuration is 12, 12, 10, 8, 8 and 6 m. Except for the first two rows (those of which inclined at 25 degrees) and the next row (which is inclined at 20 degrees), nails are inclined at 15 degrees angles. Except for the range of 30 m from the west for the north wall and 30 m from the north for the west wall, the horizontal spacing of the soil nails is in the order of 2.5 m, while for the mentioned range, the last 4 rows have a horizontal spacing of 2.0 m. All nails have a diameter of 32 mm (AIII #32 bars).

The 25 degrees inclined nails were under a comprehensive grouting programme which was implemented to control groundwater inflows into the excavation area.

3.1.2 East and south walls

For the east and south walls, the critical design profile is Type D.

The soil nail wall design for the east and south upper cut consists of 3 rows of nails on a 2.0 by 2.5 m nail pattern. The installation elevations of the upper cut nails in a top to down configuration were 98.5, 97.0 and 95.0. All nails are inclined at 15 degree angles, except where the nails are inclined at 10 degrees within the first row to avoid urban underground utilities. Within the first two rows 12 m length nails are applied, while for the third row 14 m length nail is used. The applied bar sizes are 28 mm in diameter (#28 AIII bar) with the yield stress of 4000 kg/cm².

The soil nail wall design for the middle cut consists of 3 rows of nails on a 2.5 by 2.5 m nail pattern. The installation elevations of the inner cut nails in a top to down configuration were 93.0, 90.5, and 88.0. The distance of each nail from the upper/lower bench is about 1 m. All nails are inclined at 15 degrees angles, except the nails which are installed at EL 88.0 which were inclined at 25 degrees to meet the particular requirements of the specific grouting system. Fourteen metre length 32 mm in diameter (#32 AIII bars) nails are applied for the middle cut three rows.

The soil nail wall design for the inner cut consists of 6 rows of nails. The installation elevations of the inner cut nails in a top to down configuration were 86.0, 83.5, 82.0, 80.5, 79.0 and 77.5. The nail lengths in a top to down configuration were 12, 12, 10, 8, 8 and 6 m. Except for the first two rows (those of which inclined at 25) and the next row (which is inclined at 20 degrees), nails are inclined at 15 degree angles. The horizontal spacing of the soil nails is 2.5 m. All nails are of diameter of 32 mm (AIII #32 bars).

3.2 Grouting Algorithm

By the end of the drilling procedure, the reinforcing steel bar was positioned in the centre axis of the borehole. Two tremie pipes were attached to the bar, one of which to be used as the grout path and the other to be applied as the air channel. Moreover centralizers were placed around the nails prior to insertion to help maintain alignment within the hole and allow sufficient protective grout coverage over the nail bar. The bore head was then packed by appropriate material. Afterwards, the drillhole

was filled with the neat cement grout (water/cement ratio of 0.5) through the grouting tremie pipe under the 2 bar initial pressure. Pressure grouting enhances the pullout capacity of the soil nail. Whether the injected cement weight reaches to the maximum value of 100 Kg/m (or the minimum value of 20 Kg/m), and whether the grout could be seen at the borehole head, the injection procedure could be ended. Stiffer sand-cement grout were used at special conditions (i.e., whenever the grout could not be observed at the borehole head and the cement weight exceeds the 100 Kg/m limit). Furthermore, admixtures were used for the boreholes below the ground water level.

Pullout tests were conducted on at least two nails per row in accordance with the FHWA methodology. The nails were loaded with up to 1.25 times the working load to check the quality of the injection procedure. Figure 3 shows a picture of the testing process (Lazarte et al. 2003).



Figure 3. Pullout test

3.3 Predictive Finite Element PLAXIS models

3.3.1 Input parameters

The entire soil nail wall system (Types A, B, C, and D) was numerically simulated using a two-dimensional finite element code PLAXIS, mainly to provide information for the performance of the soil nail wall. Simulation of the entire soil nail wall construction process was carried out in a sequence of construction stages. In each construction stage a sufficient number of calculation steps were used to obtain an equilibrium state.

The elastic perfectly-plastic Mohr-Coulomb (M-C) constitutive law was used to model the native soil, and for the nails and facing elements an elastic model was used. The problem with over prediction of dilatancy was overcome by introducing plastic potential function (g) in addition to the yield function (f), denoted as non-associated plasticity (Brinkgreve et al. 1998). Thus, dilatancy parameter is introduced to the models, describing the volume expansion of soils under plastic flow. The dilatancy angle is related to the volumetric strain and axial strain in plastic conditions (Aryal, 2006). The tension cut-off procedure is used to zero the

allowable tensile stress (Brinkgreve et al. 1998). Furthermore, the stiffness has been increased with depth within each layer by defining the increment option from the advanced parameter PLAXIS tab. The plane strain mesh was defined by 15-node triangular elements with 12 Gaussian points. Pore pressure distributions were generated based on phreatic level and the steady-state groundwater calculation. Shotcrete wall of 10 cm thickness with reinforced wire mesh, was modeled as beam elements of $EA=3 \times 10^6$ kN/m and $EI=2500$ kN/m²/m. Soil nails were modeled as geotextile elements of $E=2.1 \times 10^8$ kN/m² (Gysi et al. 2002). However, scaling was applied in input parameter definitions in order to average the effect of three-dimensional problem to a two-dimensional problem (Itasca. 1999). Interface elements were used to model the stick-slip interaction between the soil nails and the native soil. An elastic-plastic model was used to describe the behaviour of interfaces for the modelling of soil-structure interaction. Each data set had an associated strength reduction factor for interfaces (R_{inter}) of 0.8 to 1.0. Actually the interface properties are calculated from the soil properties in the associated data set and the strength reduction factor by multiplying the c_{soil} , $\tan \phi_{soil}$ and the tensile strength of the soil (σ_t) into R_{inter} (Brinkgreve et al. 1998). This technique is considered conservative because the soil nails slide easier through the retained soil and thus to increase conservatively the predicted wall deflections (Finno et al. 2007).

3.3.2 FE outputs

3.3.2.1 Post-construction condition

The quality of post-construction prediction is presented within this section. Figures 4 to 7 show the predicted horizontal displacement shadings for Types A, B, C, and D. As shown in the figures, the maximum horizontal displacements take place at the location of the second bench rather than the crest. The reason lies in the fact that considerable irrelevant displacements were imposed to the mesh at levels lower than the bed elevation within each calculation phase; while the mesh deformation and the incremental displacements occurring at the lower elevations prior to the excavation are almost meaningless. On the other hand, the upward trend of the trapezoidal-shaped lateral earth pressure envelope against depth leads to an increase in the lateral displacements at the deeper portions of the soil nail wall. Alternatively the drastic lateral displacements at lower elevations can be regarded as a reflection of global rotation. Besides, the presence of the ground water table and therefore the local concentration of plastic points at EL 85.8 might stand for the intense horizontal displacements at the elevation of the second bench. The value of the extreme horizontal displacement at the location of the installed reference points (EL. +97) for types A, B, C and D are 42.30E-3 m, 42.74E-3 m, 71.64E-3 m, and 33.15E-3 m respectively.

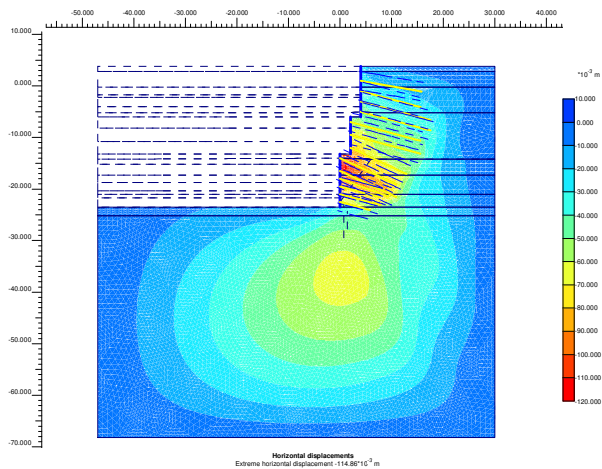


Figure 4. The horizontal displacement shadings after construction in Type A

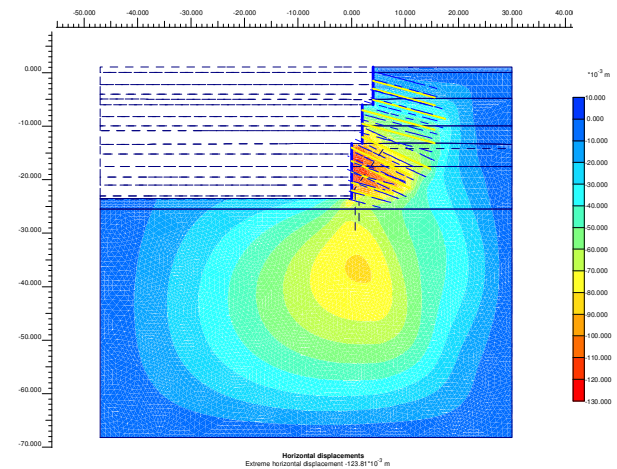


Figure 7. The horizontal displacement shadings after construction in Type D

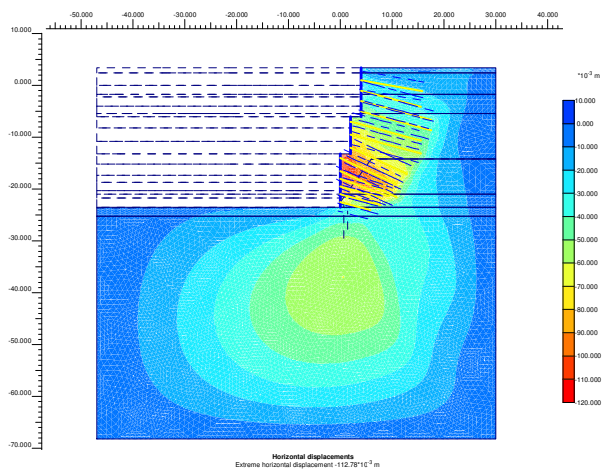


Figure 5. The horizontal displacement shadings after construction in Type B

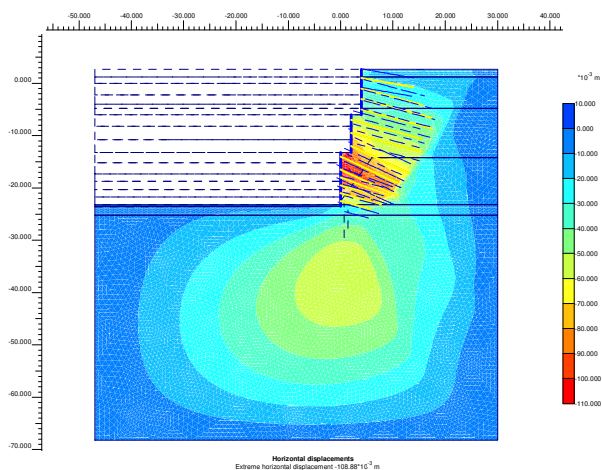


Figure 6. The horizontal displacement shadings after construction in Type C

The horizontal displacements are highest for Type C. The maximum horizontal displacement of 7.2 cm was estimated at the location of the reference points in Type C. The value declines to 4.3 cm in Type B. The value in Types A and D were equal to 4.2 and 3.3 cm, respectively. These values accompanied with the predicted maximum horizontal displacements were adopted into the Contract Documents as limiting values for wall movement.

3.3.2.2 Staged condition

Figure 8 illustrates the variation of the predicted horizontal displacements at the location of the reference points, against the excavated height for all types.

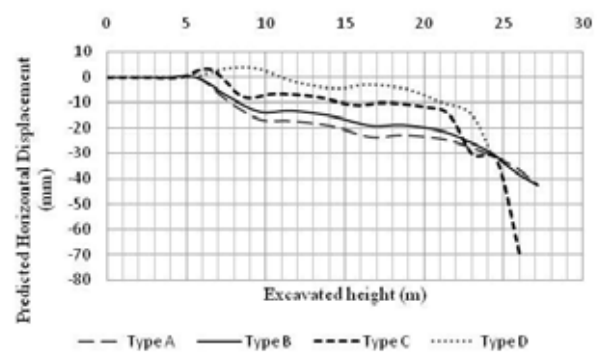


Figure 8. The variation of the predicted horizontal displacements vs. excavated height

The movements contain an initial cantilever pattern of deformation, as one would expect for a soil nailed wall. The figures show a partially regular upsurge trend in all types; however, sporadic leaps in the horizontal displacements can be observed once the excavation reached the elevation of the soil benches. Leaps can be

attributed to the change in the cut's geometry. The passive behaviour at the initial stages is due to the inevitable overprediction of dilatency, the stiffness of the facing (beam element) and its elastic behaviour, and the concentration of the plastic points at the ground water level. The lateral displacements increased slightly with depth up to the ground water level. For greater depths towards the final excavation bed, the horizontal displacement increases drastically in all cases. The drastic increase can be attributed to the presence of the pore water pressure and the wall height.

4 IN-SITU PERFORMANCE MONITORING

An instrumentation programme was established to monitor the performance of the soil nail system and to provide early detection of deflections that could potentially damage the nearby structures. The contractor was responsible for installation of instrumentation while all measurements and interpretation were the responsibility of the engineer. Forty displacement reference points (reflectors) were placed at various locations at the EL. +97 to measure the horizontal movements. The location of the reflectors is illustrated in Figure 9. The reference points were monitored using a Laser Total Station.

The displacement reference points were generally read five times a month during the excavation.

In addition to the instrumentation, daily visual surveys were performed during the course of the excavation to check for cracking of shotcrete and the inevitable cracking of the adjacent structures.

The displacements recorded at four sides of the excavation line namely west, east, north, and south, are plotted in Figures 10-a to 10-d. The mean value of the predicted horizontal deformations in Types A to C is expected to match the north wall measurements. Moreover the condition of west and south walls should be consistent with Types A and D, respectively. The mean value of the predicted horizontal deformations in Types C and D is expected to match the east wall measurements.

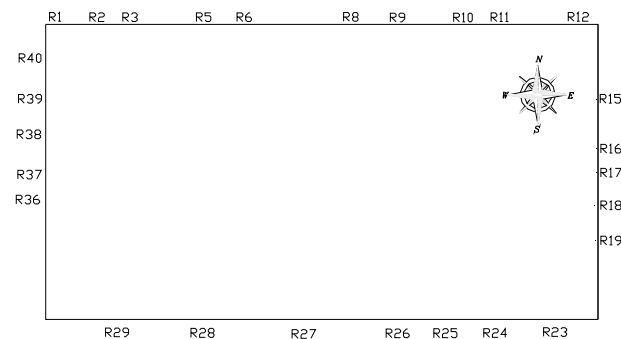


Figure 9. The location of the reference points

The maximum measured value of the average cumulative horizontal displacement on the project occurred along the north wall, where the measured average horizontal movement was approximately 99.5 mm.

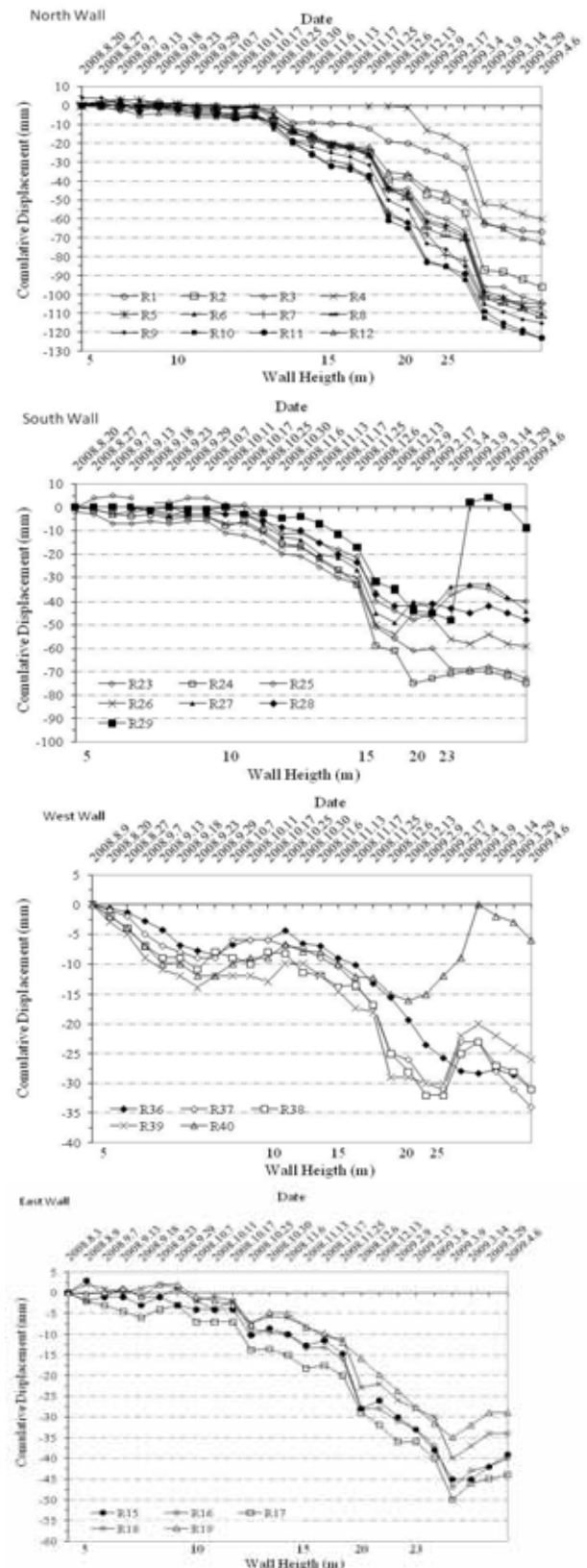


Figure 10. Horizontal displacement trend of the soil-nailed wall in post-construction condition

The mean cumulative horizontal displacement for the south, east, and west walls are 49.7, 37.2 and 25.6 mm respectively. As evident in all figures, significant displacements were recorded after the commencement of underwater excavation.

The data presented in the figures indicates that the maximum measured rate of movement in the north, west, south and east walls is in the order of 6.0, 1.8, 1.29 and 2.0 mm/day respectively. The overall mean rate of movement during the course of excavation in north, west, south and east walls is on the order of 0.62, 0.24, 0.43 and 0.31 mm/day, respectively. Figure 11 shows the trend of maximum measured rate of displacements against the reference points

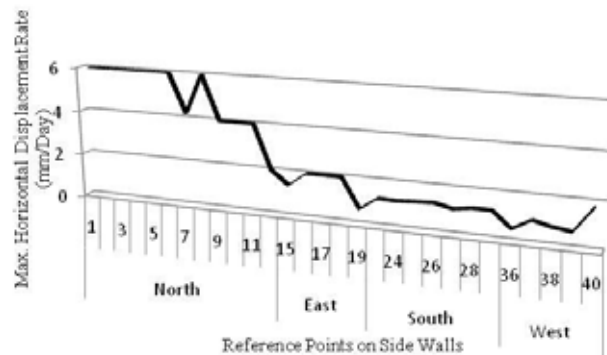


Figure 11. The maximum measured rate of the horizontal displacements

5 THE RELATION BETWEEN PREDICTED AND MEASURED DEFLECTIONS

As shown in Figure 8 and Figure 10, the predicted/measured horizontal displacements show a partial plateau with slight volatility indicating the natural initial cantilever behaviour of the soil nail wall. The trend experienced a gradual rise with sporadic leaps at the bench levels where an alteration in the slice weights took place. Displacements increased dramatically over the stages next to the ground water level as the plastic points formed and markedly concentrated below the water level. The horizontal displacements bottomed out when the excavation bed level was reached, towards the end of construction. However, the latter is inferred from the in-situ monitoring data. The lateral displacement pattern of the predicting models generally is consistent with the outcomes of the monitoring programme.

As mentioned in section 4, in order to compare the in-situ measurements with the predicting numerical modelling outcomes, one should precisely evaluate the conformity of the modelling types with their counterpart reference points. The variation of ultimate predicted values against ultimate measured values in four side walls of the project site (at the elevation of the reference points) is illustrated in Figure 12. Furthermore, Figure 13 illustrates polynomial regression of predicted and measured values of horizontal displacements at the level of the reference points for the side walls. It can be inferred that the ultimate predicted values generally agree with the ultimate measured values with a variance of ± 1.5

cm. However, for the north wall, the measured displacements were significantly larger than the predicted values. The substantial difference could be accounted by the extended width of lifts in lower levels, the intense inflow of water with an unpredicted high rate in a North to South direction, the significant number of active underground aqueducts, construction difficulties especially in installation of geo-composite strips on the wet excavated walls, those of which impeded the expected construction rate.

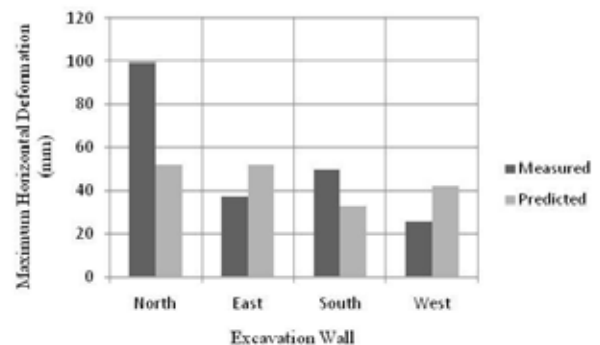


Figure 12. Maximum measured horizontal displacements vs. Maximum predicted horizontal displacements

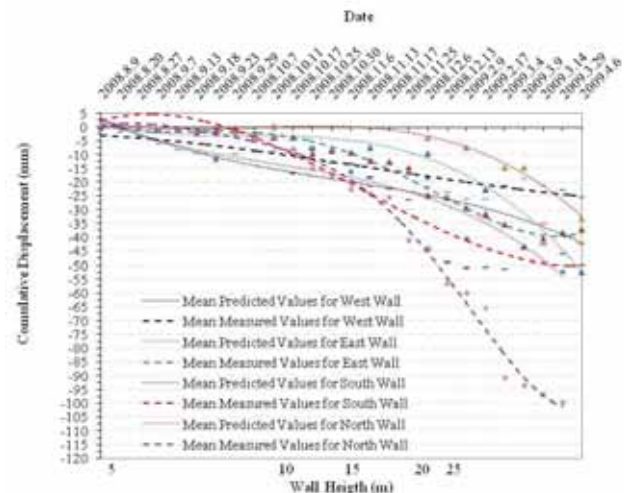


Figure 13. The polynomial trend of the mean predicted and measured horizontal displacements against the construction stage

6 CONCLUSION

The excavation for the Baran Commercial Tower involved numerous challenges during both design and construction. Using conventional methods of design, including the Federal Highway Administration (FHWA) and previously developed charts for estimating lateral displacements or performance ratio may be misleading in deep soil nailing applications. Alternatively, monitoring and numerical FE analyses were applied. The lateral

displacements based on the in-situ surveying data generally agree with the FE modeling data.

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REFERENCES

- Aryal, K.P. 2006. *Slope Stability Evaluations by Limit Equilibrium and Finite Element Methods*, PhD thesis, Norwegian University of Science and Technology.
- Asghari E. 2002. *Effect of Cementation on Shear Strength and Deformation Parameters of Coarse Grained Soils with Reference to Tehran Alluvium*, PhD thesis, Tarbiat Modares University, Tehran, Iran.
- Brinkgreve R.B.J., Vermeer P.A., Bakker K.J., Bonnier P.G., Brand P.J.W., and Burd H.J. 1998. *PLAXIS Version 7 – The General Information*, A.A.Balkema, Rotterdam, Netherlands
- Finno, R.J., Langousis, M., Winter D.G., Smith, M.W., and Chin, K.H. 2007. Real time monitoring at the Olive 8 excavation, *7th International Symposium on Field Measurements in Geomechanics*, Boston, Massachusetts: 1-12.
- Gysi H.J., Morri C., and Gysi Leoni Mader AG. 2002. *Würenlingen Practice*, Bulletin of the PLAXIS No. 12, Netherlands: 14-16.
- Itasca Consulting Group. 1999. *FLAC Manual : Theory and Background*, Minneapolis, Minnesota.
- Lazarte C.A., Elias V., Espinoza R.D., and Sabatini P.J. 2003. *Geotechnical Engineering Circular No.7 – Soil Nail Wall*, FHWA0-IF-03-017, Maryland, Columbia.