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Flexible Earth Retaining Structures - Soil Nailing

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ABSTRACT

Soil nailing as a permanent retaining system for excavations and slopes is a widely accepted method in countries where the seismic activity is the main concern. Considering the increasing demand for deep excavations in connection with extensive land utilization within densely populated areas and as a result of increasing value of land, the economy and construction speed of soil nailing has made the method an outstanding alternative for both temporary and permanent excavations in İstanbul. In this paper, the current practice for the encountered basic lithological unit is outlined. Furthermore a case study of soil nailing application for a failed rigid tie-back retaining wall is presented.

1. INTRODUCTION

With the growing demand for deep excavations in connection with extensive land utilization in densely populated areas, soil nailing has found an increasing application area in Turkey in the last five years. The economy and construction speed of soil nailing has made the method an outstanding alternative among other retaining systems for excavations. Since its early applications in seventies as a flexible retaining structure, soil nailing has been primarily implemented to retain slopes, excavations and tunnel portals.

The main application of the system in İstanbul is to retain slopes in deep excavations. It has been observed from recent nailing applications in İstanbul, that soil nailing performs well for the main lithological unit, alternating sandstone, siltstone and shales known as greywacke formation, as well as the overconsolidated clays in the western part of the city known locally as Gürpınar formation. Such performances are monitored with proper geotechnical instrumentation during the construction process and the monitoring results are processed in the performance of the retaining structure.

Davis Method (Shen et. al., 1981) is utilized in soil nailing design in current practice in Turkey. As other conventional soil nailing design methods this method is based on limit equilibrium analysis in which a parabolic sliding surface passing through the toe of the wall is examined. In this method, horizontal and vertical components of the resisting force of each nail on the sliding surface are included in the stability calculations and the overall stability of the slope is determined using classical method of slices. The resisting force of each nail is taken as the minimum of the tensile strength and pullout resistance.

2. Soil Formation and Nailing Practice in İstanbul

The main lithological unit in İstanbul is greywacke formation composed of alternating sandstone, siltstone and shale layers. Greywacke is often dark gray or dark brown colored, containing many coarse angular grains of quartz and feldspar which have been sedimented with little sorting together with mica and small rock fragments and fine matrix material. Greywacke is formed in the seas adjacent to rapidly uplifted landmasses. Many greywackes especially the ones in İstanbul vicinity show graded bedding, in which the sediments passes from coarser to finer particles from the bottom of a bed upwards; this structure is produced by settlement of sand and mud mixture in water. The formation has extensive structural discontinuities as a result of tectonic movements.

Table 1. Geotechnical Properties of Greywacke

Property	Impact In Situ Condition	Extreme Condition Weathered
c (kPa)	500	0.0
ϕ	35-40	14-18
E (MPa)	2000	100-400
ν	0.22	0.35
γ_n (kN/m^3)	25	20

c : cohesion
ϕ : internal friction angle
E : modulus of elasticity
ν : Poisson ratio
γ_n : natural unit weight

The greywacke formation of İstanbul region named Trakya formation does not present bearing capacity problems, but on the other hand have serious stability problems on vertical excavation surfaces of deep excavations, especially along structural discontinuities. The range of values for geotechnical properties for greywacke formation encountered in İstanbul are listed in Table 1.

In deep excavations many factors including the local subsoil parameters, groundwater condition, the duration of excavation and foundation of neighboring super and infra-structures should be evaluated for the choice and design of the suitable retaining structure. It should be noted from the table that strength parameters show a drastical drop from the peak values in case of extreme weathering. The effects of air and water are the main reasons of weathering. Therefore the soil surface exposed during excavation steps should be immediately covered with proper facing and drainage measures should be installed to avoid water accumulation behind the wall. Therefore, shotcreting used as facing elements immediately after the excavation of each step and drainage means in soil nailing system should be taken against such problems.

In the current practice, soil nailing is primarily implemented to retain foundation excavations of buildings with various number of basements in densely populated areas in İstanbul. The economy and construction speed of soil nailing has made the method an outstanding alternative for both temporary and permanent excavations. It has been demonstrated from recent nailing applications that soil nailing performs extremely well for the greywacke formation in İstanbul. Such performances are monitored with proper geotechnical instrumentation during the construction process and the monitoring results are

used in the performance evaluation of the retaining system. Recent soil nailing applications of Zetaş Zemin Teknolojisi A.Ş.* are summarized in Table 2.

Table 2. Summary of Soil Nailing Applications in İstanbul

Project	Year	Formation	Wall Height (m)	Wall Area (m^2)	Notes
Emlak Bankası Housing Dev. - Ataşehir	1992 1993	weathered arkose	9.3 14.0	627 890	slope failure remediation temporary structure
Toyota Plaza, Ortaköy	1993	greywacke	17.5	880	permanent structure
Ağaçkakan Visual Arts Center, Esentepe	1994	greywacke	15.5	1290	temporary structure
Halk Sigorta General Directorate, Zincirlikuyu	1995	greywacke	14.0	730	temporary structure
Hayat Hotel Plaza, Sefaköy	1995	overconsolidated clay	15.0	1900	implemented as a remedial project for a failed rigid tie back ret. wall with piles
Ayhan Bermek Business Center, Kağıthane	1995	greywacke	14.8	1560	temporary structure
Tepe Housing Development, Beykoz	1996	greywacke and tuff	12.0 17.5	2820 880	temporary structure temporary structure
GenPa General Directorate, Etiler	1997	greywacke	13.5	1180	temporary structure

Most of the above listed applications are implemented as retaining systems at the foundation excavations of buildings with basements, located at the center of the city where in most cases, structures adjacent to excavation were present. In Ataşehir a failed excavation slope remediation is implemented in 1992. On the other hand in 1993 a retaining structure near the main water line (in diameter 1800 mm) for the city is constructed. In the Ortaköy Totyta Plaza first application of the permanent wall has been performed. In Hayat Hotel Plaza in 1995, soil nailing has been implemented for the remediation of a rigid retaining system failure and the case has been presented in the following sections. The application of the system where conventional rigid retaining system has resulted in a severe failure presents main advantages of soil nailing.

3. Application of Soil Nailing for the Remediation of a Retaining Structure Failure

A sudden failure has occurred in a tieback retaining wall during the deep excavation of a high rise hotel building in Sefaköy. The depth of the excavation base is planned to be 20.5m, whereas the sudden failure took place when the excavation was at 16.0m depth. The sliding block covered the entire wall with a length of 80 meters, the main scarp extending to 15 meters behind the wall. A plan of the excavation area and initial retaining system is shown in Figure 1.

The failure mechanism has been investigated and design considerations have been evaluated in terms of stability and safety. A back analysis has been performed using the available geotechnical parameters on a typical cross section a d it has been determined

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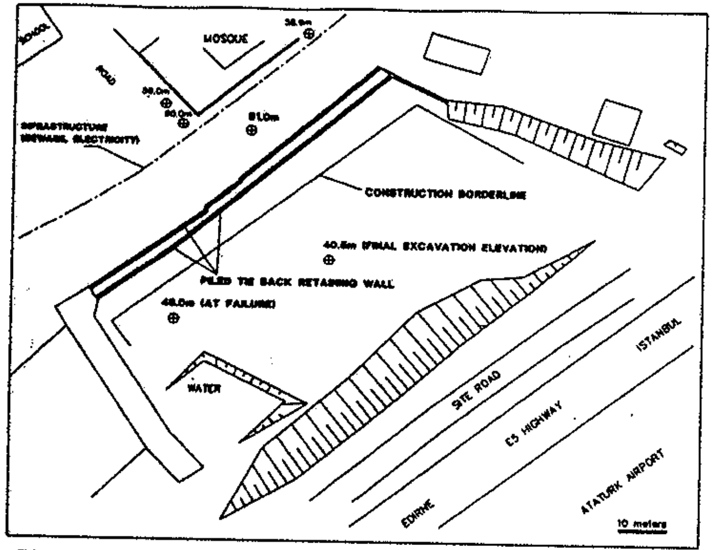

Figure 1. Plan of the Tie-back Retaining Wall and Construction Area

that maximum capacity of the elements of the tie back retaining wall, i.e. prestressed anchors has been exceeded with the drop of the peak shear strength parameters to residual values (Olgun and Durgunoğlu, 1996).

A proper soil nailed retaining system has been designed according to the evaluations of the geotechnical conditions and failure geometry. Stability analyses have been performed in the design of the nailed wall using residual shear strength parameters. Nail lengths have been optimized by evaluating the location of critical sliding surfaces along the wall. Lateral deflections have been monitored with inclinometers during construction and the proposed system has been applied with safety.

3.1. Initial Retaining System and Investigations

The governing strength parameters and the design considerations of the initial project have been evaluated to determine the failure mechanism, taking into consideration that the failure geometry exhibited a systematic pattern, covering the entire retaining wall. The present soil formation consists of sedimentary deposits. Erratic layers of clay, marn and sand are present at the top; limestone overlying overconsolidated clay is present below this covering surface formation. Geotechnical parameters that were proposed in the design are summarized in Table 3.

Table 3. Geotechnical Parameters (Toğrol, 1993)

Soil	c' (kPa)	ϕ' (deg)	γ_n (kN/m^3)
clay, marn, sand layers	0	26.5	18
marn, limestone	10	37.5	19
overconsolidated clay	40	22.0	18

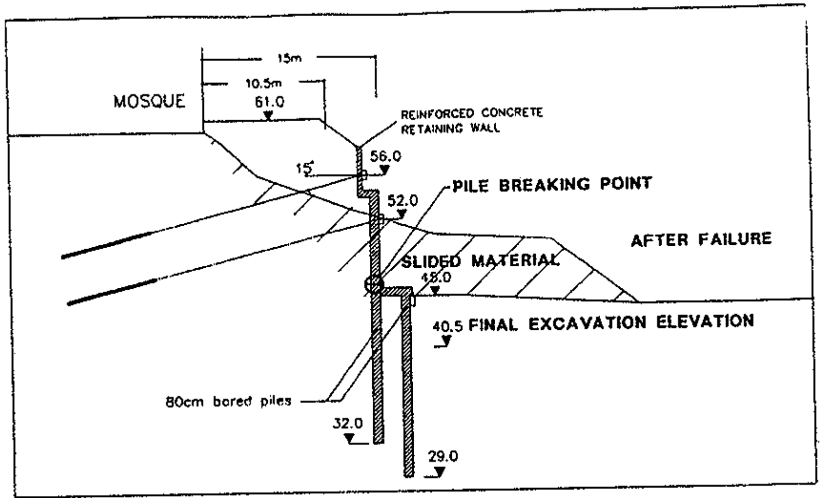

Figure 2. Typical Cross Section of Tieback Retaining Wall and Failure Geometry

The tieback wall was designed as the retaining system for the foundation excavation of this hotel. The retaining system consists of intersecting two rows of bored piles of ϕ 80 cm diameter, tied back with two rows of 20.0m long prestressed anchors. A rigid reinforced concrete retaining wall was located above the first row of piles, tied back with the aid of first row of anchors. The initial ground elevation prior to excavation was 61.0m and the elevation of the final excavation base is 40.5m, creating an excavation depth of 20.5m. A typical cross section and failure geometry is shown in Figure 2.

A sudden failure has occurred in the rigid retaining wall described above when the excavation was at elevation 45.0m, corresponding to 16.0m depth. The failure covered the entire wall length of 80 meters, extending to 15.0m behind the wall. All of the anchors supporting the piled wall have failed, some pulled out from the soil, some broken by the tendons. As a result some damage to neighboring structures and with complete damage of sewage pipes and electricity cables, whole material behind the wall in this zone slided into the excavation area.

The forces expected to develop on the elements of the tieback wall have been investigated by performing a back analysis on a typical cross section, using the available parameters. The backcalculated forces have been evaluated by comparing the forces expected to develop on the retaining wall with maximum capacities. Possible scenarios of failure have been checked to determine the failure mechanism. The results of the analyses are summarized below in Table 4

It has been determined from the performed analyses that, even though the geotechnical parameters used in the design were valid, forces exceeding the allowable capacities are expected to develop on the elements of the tieback wall. Finally it has been speculated that the possible intrusion of water and decomposition resulted an initial sliding movement. With this initializing movement, the shear strength of overconsolidated clay dropped from peak values to residual values and the stresses in the anchors increased to values exceeding the maximum capacity, causing the resulting sudden failure (Durgunoğlu, 1995).

Table 4. Element Forces Back-calculated Using Available Parameters

Element	Calculated Force-Moment	Allowable-Maximum Values
1st row anchorage	653 kN/anchor	450 kN-650 kN (all.-max.)
2nd row anchorage	550 kN/anchor	450 kN-650 kN (all.-max.)
Toe support system - bored piles	400 kN/m	3000 kN/m
Pile section moment	257 kNm/pile	365 kNm - 512 kNm (all.-max.)
Support beam moment	199 kNm	not critical

3.2. Remedial Project

After the studies performed to determine the failure mechanism and with the evaluations of the geometrical configuration of the slide, a soil nailed wall starting behind the slided slope as seen in Figure 3, has been designed for remediation works. The nailed wall consists of two stages extending from elevation 45.0m to 60.0m. The nails are located with 2.0m horizontal spacings. The final excavation depth which is below the base of the nailed wall is designed to be achieved by using the unbroken piles remaining from the initial project.

Stability analyses have been performed both for the nailed wall and overall deep sliding. The optimum nail length is determined with the evaluation of possible slip surfaces along the wall. Two critical sliding surfaces have been checked using the residual parameters assuming no cohesion in residual condition. One of these surfaces is the shallow sliding surface intersecting the toe of the slope. The nail lengths have been optimized with the evaluation of this surface. Alternatively the deep sliding surface passing under the piles has been checked and it has been determined that it was not critical in terms of statical and earthquake conditions.

The building construction started following the completion of the excavation supported by soil nailing. At the first stage, the part of the hotel building 5.0m wide has been constructed to enable the excavation area to be backfilled so that the neighboring road could be opened to traffic. The top of the piled wall has been tied back with additional anchors to withstand the overturning moments due to the backfill. Moreover, micro piles of ϕ 15cm diameter and 20.0m length with 1.0m spacing have been installed to the base of this 5.0m wide building section to support the building for the overturning moments, as shown in Figure 4.

3.3. Instrumentation

It was determined from the performed analyses that the shear strength parameters of the subsoil had dropped to residual values due to sliding. The deformations during the excavation were expected to rise to values critical for neighboring structures, due to the presence of a sliding surface and the drop in soil shear strength. Two inclinometers were installed at 52.5m elevation at the berm of the nailed wall to monitor the lateral movements during excavation. The location of the inclinometers along the wall are shown in the plan showing soil nailing application. The second stage of the nailed wall that has been monitored is completed in five excavation stages. The lateral deflections measured with two inclinometers for each excavation step are shown in Figure 5.

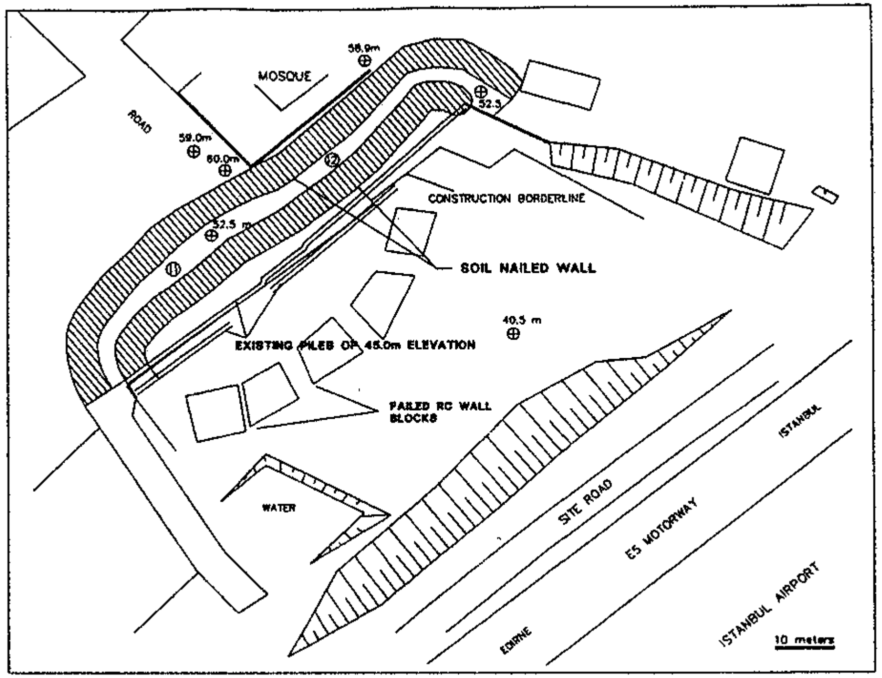

Figure 3. Plan of the Soil Nailed Retaining System

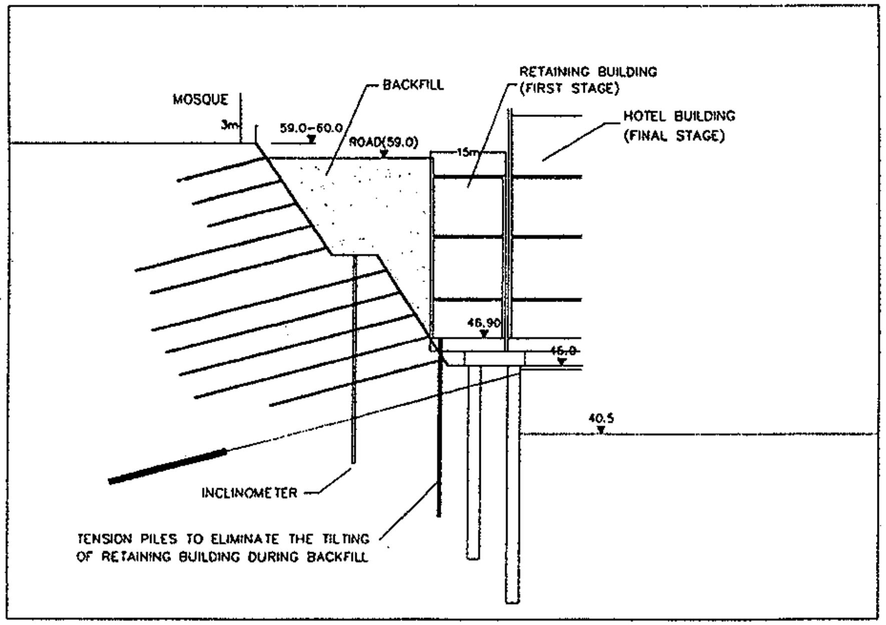

Figure 4. Cross Section of the Building after Remediation

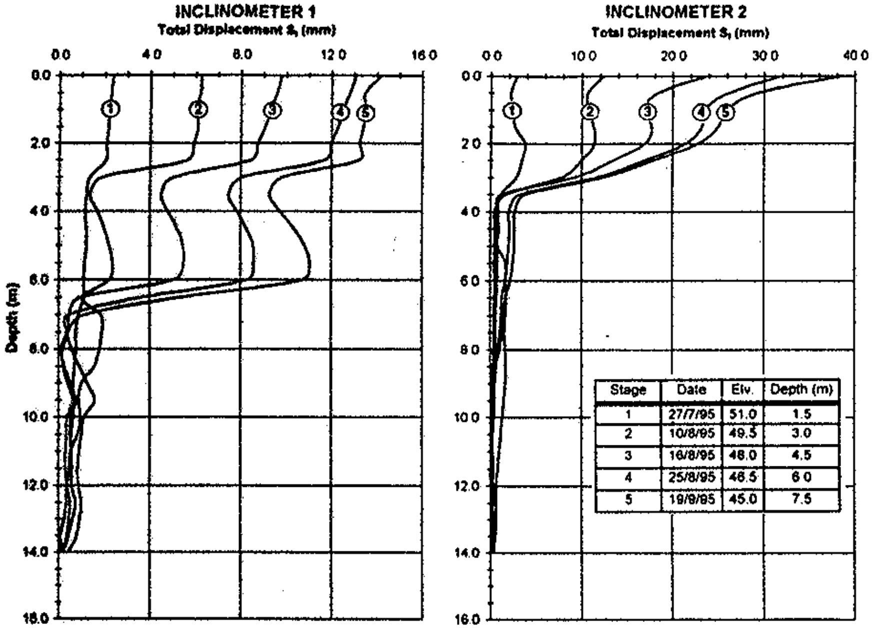

Figure 5. Lateral Displacements Measured During Excavation

Several sliding surfaces have been detected during the excavation and construction processes. It is seen from both inclinometer readings that, movement has started 3.0-3.5m below the berm at the first excavation stage, but this movement has stopped at location of inclinometer 1, in the proceeding excavation steps. In the following excavation stages, a surface located at 7.0 m depth has started to move at location of inclinometer 1. The deflections at inclinometer 2 location have continued to increase at depth 3.5 m to values measured larger than the first inclinometer. The measured lateral displacements have shown to be in good agreement with the sliding surfaces proposed in the design procedure of the nailed wall.

The ratio of lateral displacements measured during excavations to corresponding wall heights are summarized in Table 5. The lateral displacements measured are in ranges between 0.2 to 0.5 percent of wall height. These values are in good agreement with the ones proposed in the literature (Elias and Juran, 1989) for deflections required for full mobilization of nail forces and measurements at recent soil nailing applications in İstanbul (Özsoy, 1996).

Table 5. Excavation Stages and Lateral Wall Movements

Excavation Stage	Elevation (m)	Wall Height - H (m)	Lateral Movement - D (mm)		D / H (x 1000)	
			I1	I2	I1	I2
1	51.0	1.5	2.4	2.8	1.6	1.9
2	49.5	3.0	6.3	12.3	2.1	4.1
3	48.0	4.5	9.8	23.6	2.2	5.2
4	46.5	6.0	13.1	31.8	2.2	5.3
5	45.0	7.5	15.4	38.4	2.1	5.3

4. CONCLUSIONS

Soil nailing as a flexible earth retaining structure uses a passive earth reinforcement technique to strengthen the soil. and its behavior is quite different from the rigid systems that retain soil behind a vertical cut such as tied back piled wall. The main components of the system are the ground itself, soil nails and facing elements. The main advantage of the system is the low cost due to relatively rapid installation of the nails and a relatively thin shotcrete or concrete facing. In addition flexibility of the system offers greater advantage under seismic conditions.

The remediation works after a retaining structure failure are outlined above to present the application of flexible retaining system where conventional rigid systems have shown to be unsuccessful. Analyses have been performed to determine the failure mechanism and it has been shown that failure occurred since the anchor forces were exceeded with the drop of strength parameters from peak to residual values after initial movement as a result of increase in porewater pressures.

A retaining system with soil nailing has been designed and constructed as a remediation measure. Soil nailing as a flexible retaining system can conform to the surrounding ground and withstand greater total and differential ground movement. The lateral movements of the nailed wall has been monitored with inclinometers during the excavation and construction process. The measured deflections have shown to be in the ranges of the ones proposed in the literature for similar soil conditions.

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FOUNDATION PROBLEMS OF KONYA ALAEDDİN MOSQUE AND IMPLICATION OF REMEDIAL MEASURES

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ABSTRACT

Severe structural cracks in the structural system of the historical Konya Alaeddin Mosque have been investigated and a remedial design has been performed to eliminate this problem resulting from the settlements of the foundation subsoil and resulting differential settlements of column and wall foundations. Subsoil conditions have been investigated and a geotechnical model has been developed to identify the related displacements and structural damage mechanism. Micro piles have been constructed beneath the structure to transfer the loads to deeper soil layers and cement grouting has been implemented to improve the subsoil conditions. Extensive instrumentation has been implemented and the relevant parameters have been monitored during the application of the remedial measures.

1. INTRODUCTION

Konya Alaeddin Mosque was built in the twelfth century by the Seljuks, on Alaeddin Hill known to be a man-made artificial fill. The monument, has taken its final form in time with various revisions and extensions along the eastern and western sides. It is observed that remnant pieces have been used in construction and the structural system is formed of masonry arched resting on stone columns. A plan of the mosque showing the locations of the masonry walls and the stone columns is given in Figure 1.

Although the problems related to the foundation system and subsoil have been encountered long before and several attempts have been made for solution of the

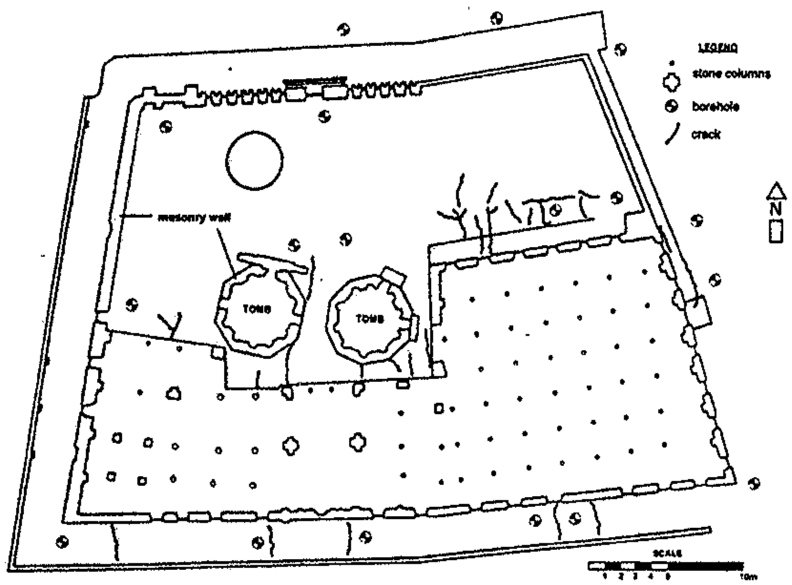

Figure 1. Konya Alaeddin Mosque, location of structural elements

problem, the situation became worse and the damage in the structural system has expanded to the other parts of the structure due to improper recognition of the cause of problem at early stages. Especially in the last 25-30 years, elements of the structural system have been subjected to severe cracks and distortions. Various investigators have agreed that this was a result of deformations and settlements in the subsoil. Although several remedial measures including structural reinforcement of the foundations and ceiling have been conducted during the period of 1967-1972, the problems related with the structural elements have not been overcome. Finally in the past with accelerating crack propagation and structural damage, to avoid any structural collapse, the structural elements have been supported with a frame system and the mosque has been closed to the public.

At this stage the governmental agency have consulted the prime author, describing the past problems and have asked to evaluate the problems towards the technical solutions. Consequently, integrated analyses have been performed in the assessment of the problem and implementation of a engineering solutions. Various studies have been made at the initial stage with the evaluation of available data. Additional investigations have been performed to determine the subsoil conditions and the assessment of the deformation pattern. The problem has been modeled using various numerical techniques with the implementation of instrumentation data. It is determined that the foundation soil is responsible for the observed

structural damage as a result of leaching with water. Remedial design has involved the implementation of cement grout for subsoil improvement and micro piles beneath the structural columns and walls. The behavior of the structure has been monitored during implementation by means of instrumentation.

2. SUBSOIL PROPERTIES AND PROBLEM DEFINITION

The deformations in the subsoil are expected to be the main cause in the generation of the observed cracks. Identification of subsoil behavior and problem definition is one of the primary factors in the choice of proper remedial measures and soil improvement technique. For this purpose, detailed and integrated subsoil investigations have been initiated in 1983 and the first attempt has been made for a model test to monitor the results of remediation [1]. It has been concluded with the evaluation of the performed subsoil investigations and available data, that the cracks in the structural elements were caused by the differential settlement of the foundation. The fill material that forms the foundation subsoil is identified to be the main source of the differential settlements. Seepage as a result of leakage from a water reservoir near the mosque, piping system and seasonal precipitation have created solution cavities and channels in the foundation subsoil which ultimately caused local collapses within the structure. Therefore it has been decided that the foundation subsoil conditions have to be improved to prevent any additional deformations as a result of soil structure collapse.

Furthermore, following the identification of the problem, trial cement groutings have been performed for the choice of proper soil improvement method, and it has been observed that cement-water mixture grouting could be properly utilized for soil improvement purposes, without causing any further damage to structural members [2]. Finally the remedial project consisting of subsoil grouting and micro-pile construction beneath load carrying walls and columns has been implemented.

A monitoring system has been developed in the mosque to monitor the soil behavior and structural system performance during and after remediation. Series of pressuremeter testing has been conducted prior to and after the improvement to evaluate the achievement of the remediation with respect to mechanical properties of the subsoil.

3. STRUCTURAL AND GEOTECHNICAL MODELING

The structure and the foundation subsoil have been modeled for the identification of the deformation mechanism and evaluation of various soil improvement scenarios. In these studies, the cracking mechanism, the explicit indication of deformations has been examined and several factors contributing to the lateral and vertical deformations have been separately analyzed. Governing subsoil behavior and crack formation pattern is identified based on the results of these modeling studies.

3.1. Structural Cracks

Crack monitoring and analyses of structural behavior are important measures of control necessary during the design and application phases of the remedial project. For this purpose numerous extensometers have been installed to monitor the propagation of cracks and ground movements.

It has been determined from the evaluation of performed subsoil investigations and study of crack locations that the foundation of the structure has been subjected to differential settlements. The foundation subsoil has been observed to be highly decomposed due to leakage from the water reservoir located below the southern face and eastern corner of the mosque, which has been used for long years as municipal water storage. Finite element analysis has been performed to evaluate and model the deformation pattern expected to occur within the subsoil located below the mosque. The settlement mechanism and the resulting crack pattern along the northwest-southeast wall are shown in Figure 2.

3.2. Lateral Deformations

It has been determined from overall slope stability analyses, no stability problems or lateral deformations are expected to occur at the Alaeddin Hill where the mosque is located. However the cracks in the structural elements have revealed that the mosque has been subjected to lateral deformations. The mechanism could be explained in a way that the vertical deformations in the foundation subsoil have

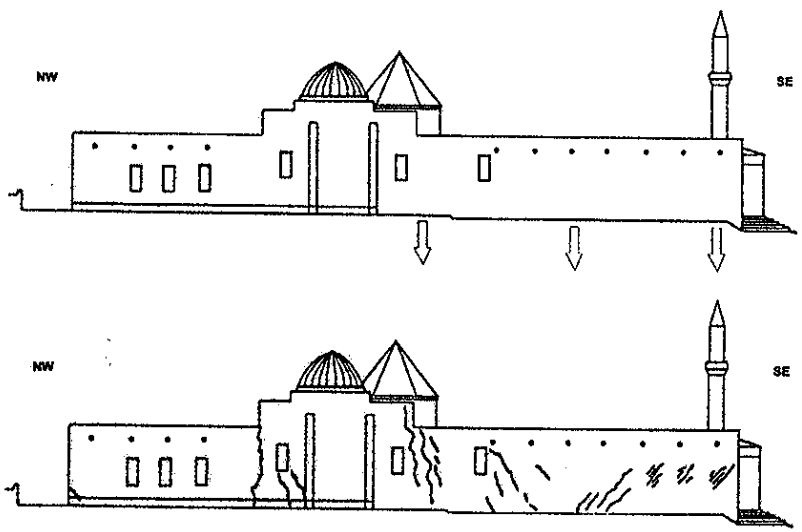

Figure 2. Settlement mechanism and resulting crack pattern along SW-NE wall

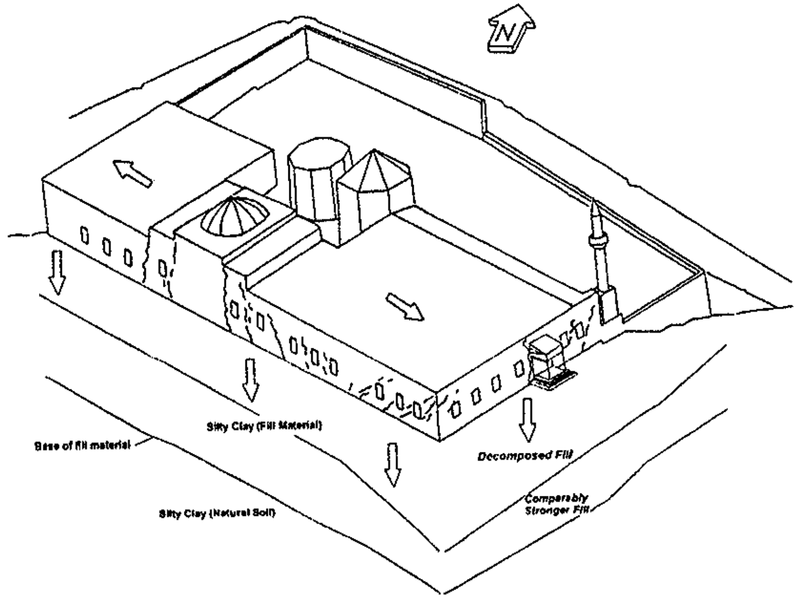

Figure 3. Schematical representation of three dimensional deformation pattern

resulted in lateral deformations in the structure due to the behavior of the structural system. In one of the remediation attempts in the past the altar has been connected with tie beams and the roof at these locations has been replaced with reinforced concrete slab. This solution has prevented a possible local failure of the structure and however at the same time it has led the problem to spread to other sections/parts of the mosque. This situation has changed the behavior of the structure drastically. Due to the connections made along structural elements differential settlements in the foundation have resulted in lateral deformations in the structure. A schematical representation of the three dimensional deformation pattern is given in Figure 3.

4. SOIL IMPROVEMENT STUDIES

The method implemented in the improvement of subsoil conditions involves pressure grouting within the fill having solution cavities and channels as a result of leaching with seepage water which is identified to be the main cause of the settlement problem and underpinning of load carrying elements with micro piles. The deformations in the subsoil and structural elements have been continuously monitored during the implementation of the project. The degree of improvement in the mechanical properties of the subsoil has been assessed with pressuremeter testing performed before and after grouting.

Two lines of cement grouting have been performed along the outer walls and these have been followed by grouts along the inner walls facing the courtyard and around the tombs. Finally the courtyard interior part of the mosque has been grouted, with grout locations forming a grid along the plan. The groutings have been produced to penetrate the nearly 25.0 m thick fill completely and socketed 2.0 m to the natural alluvial subsoil.

After the completion of the grouting application, micro piles have been constructed beneath all load carrying members including masonry walls and columns to transfer the loads to deeper soil layers. The micro piles have been drilled to penetrate the alluvial subsoil located below the fill. Upon completion of the micro piles, all structural load bearing elements, i.e. walls, stone columns, were connected to the micro piles with special elements, in order to be able to transfer any additional load from the superstructure to the newly constructed micro piles. Vertical capacity of the micro piles have been assessed by means of load tests.

5. INSTRUMENTATION AND MONITORING PROGRAM

Deformations in the subsoil and propagation of present cracks in the masonry walls and columns during grouting have been monitored with a precise instrumentation. For this purpose, crackmeters have been installed at cracks present in the load carrying members, such as masonry walls and columns. The settlements and deformations in the foundation soil during grouting have been monitored with several borehole extensometers. Additionally, the general pattern of structural movements have been monitored with extensometers installed between column spacings and exterior wall corners [3]. The safety of the remediation has been realized with the evaluation of the monitoring data.

A total of 56 crackmeters with 0.01 mm precision have been installed at crack locations and the propagation has been continuously monitored. It is seen in general that the movements in the cracks have terminated at the time just after the implementation of the remedial measures.

Additionally borehole extensometers have been installed within the subsoil to monitor the deformations and settlements in the subsoil during remediation works. Such deformations are important measures of the differential settlements that create the crack formation mechanism. The extensometers are installed at 5.0 m and 10.0 m depths from the ground surface to monitor the deformations at these locations. A total of 58 extensometers have been installed for this purpose.

A database system has been developed for the treatment of instrumentation data in the evaluation of the subsoil movements and structural behavior.

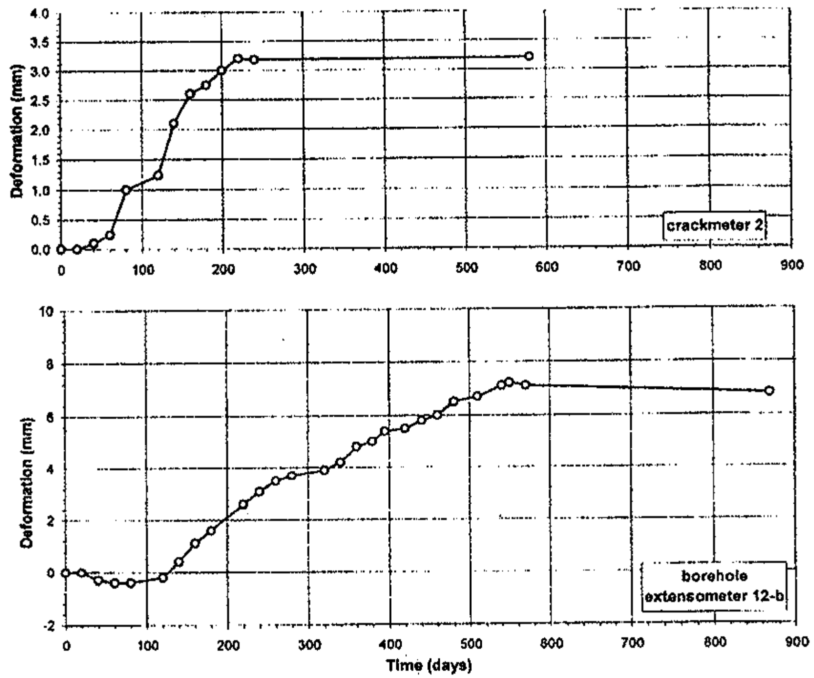

Figure 4. Deformation measurements from crackmeter and borehole extensometer

Deformation data obtained from a crackmeter and a borehole extensometer are given in Figure 4. The deformations observed both by crackmeters and extensometers are observed to diminish within time with the implementation of remediation works.

6. ASSESSMENT OF SOIL IMPROVEMENT BY MEANS OF IN-SITU TESTING

In-situ testing in the form of pressuremeter readings has been implemented to determine the degree of improvement in the mechanical properties of the subsoil. Pressuremeter testing is performed to evaluate the improvement in the bearing capacity and compressibility of the subsoil as a result of grouting. Pressuremeter testing is performed in 17 boreholes with 1.5 m intervals, extending up to 25 m depth before and after implication of soil improvement measures and therefore the degree of soil improvement has been assessed.

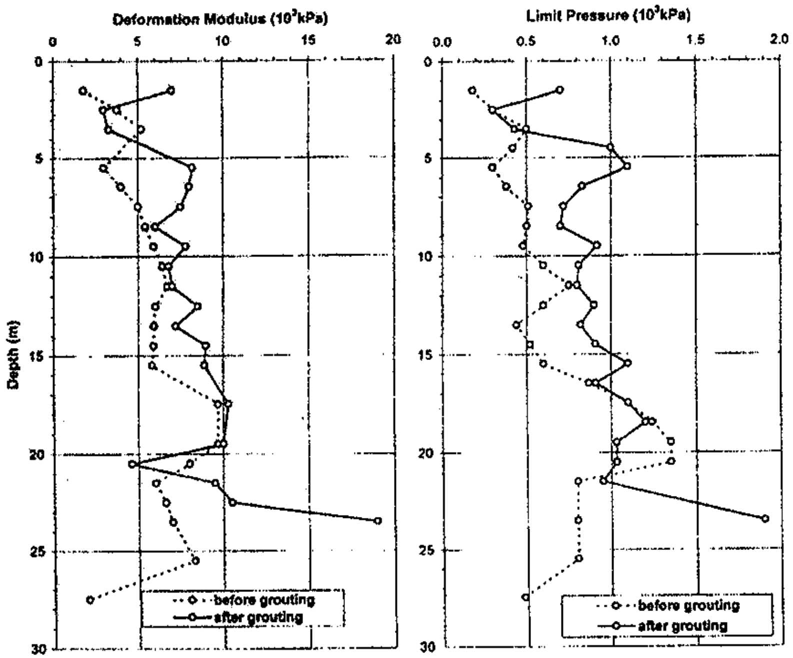

Figure 5. Assessment of degree of soil improvement from comparison of deformation modulus and limit presure obtained from pressuremeter testing.

The results of pressuremeter tests are presented in Figure 5. The change of limit pressure and deformation modulus before and after improvement are shown in the figure. The increase in limit pressure is a measure of the increase in shear strength and the bearing capacity and the increase in the deformation modulus is a measure of decrease in the subsoil compressibility.

Therefore, safety of the structure during implementation of remedial measures has been satisfied with the integrated approach of instrumentation and monitoring during application, and it has been possible to justify and implement an engineering solution by monitoring the efficiency of the soil improvement. Both the results of the displacement monitoring and pressuremeter testing indicate that the mechanism that results in structural cracks have been eliminated.

7. CONCLUSION

The causes of the severe structural cracks which occurred in the historical Konya Alaeddin Mosque have been investigated and a remedial design has been implemented to eliminate the problem. The remedial project consists of the improvement of the subsoil conditions and foundation systems of the structural walls and columns.

Integrated studies have been performed including subsoil investigations and geotechnical modeling to identify the related deformation and structural damage mechanism. The load carrying structural elements have been underpinned with micro piles and cement grouting is implemented to improve the subsoil conditions. Instrumentation has been conducted and the relevant parameters have been monitored with various geotechnical instruments during the implementation of the project.

As a result of this integrated remediation, the settlement mechanism that generated the problem has been eliminated and the mosque has been furnished for further structural restoration.

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