ANALYSIS OF THE PERFORMANCE OF RETAINING SYSTEMS IN DEEP EXCAVATIONS IN GREYWACKES

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Dedicated to my wife, Balca

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ABSTRACT

ANALYSIS OF THE PERFORMANCE OF RETAINING SYSTEMS IN DEEP EXCAVATIONS IN GREYWACKES

Due to the economic growth, the need for high-rise buildings and shopping malls with multiple basement levels increased in Istanbul and construction of retaining structures to support deep excavations reaching 25 to 40 meters became compulsory. The encountered subsoil formation in the deep excavations is soft rock greywacke locally known as Trakya formation, lithologically alternating sandstone, siltstone and claystones with various degrees of weathering and fracturing.

This thesis covers an investigation about the retaining systems widely used in deep excavations in Istanbul. Since Istanbul is in a seismically very active region and obviously at risk of being hit by a major earthquake, flexible retaining walls should be preferred in deep excavations carried out in the city. Therefore special emphasis is given in soil nailing as a flexible retaining system. Six major case studies of deep soil nailing walls having total surface area of 63,000 m^2 are thoroughly examined and the results of the performance of walls with different heights are compiled. As a result the values of performance ratio for soil nailed walls together with nail length and density in typical greywacke formation of Istanbul are developed based on these extensive case studies as a guideline for future applications. Finite element back analysis is utilized to verify stiffness parameters of greywacke formation resulting the measured lateral displacements and to come across a relation between the actual lateral displacements and the stiffness parameters, as well.

Additionally, a significant case study is presented in this study for the utilization of various retaining systems for different subsoil and groundwater conditions encountered within a given site. Displacement data and experience obtained from this case study is provided as a valuable source of data and example for future applications in similar conditions within the city.

ÖZET

GROVAKLARDA YAPILAN DERİN KAZILARIN İKSA SİSTEMLERİNİN PERFORMANS ANALİZİ

Ekonomik büyümeden dolayı İstanbul'da yüksek yapılara ve çok katlı bodrumları olan alışveriş merkezlerine artan talep nedeniyle derin kazılar yapılmakta ve bu kazıları desteklemek üzere 25 – 40 m yüksekliğindeki iksa sistemlerine gerek duyulmaktadır. Yapılan bu derin kazılarda karşılaşılan zemin yapısı genellikle grovak olup Trakya formasyonu olarak bilinen bu formasyon farklı derecelerde aşınmış ve kırılmış kumtaşı, silttaşı ve kiltaşı tabakalarından oluşmaktadır.

Bu tez çalışması, İstanbul'daki derin kazılarda sıkça kullanılan iksa sistemlerinin araştırılmasını içermektedir. İstanbul sismik olarak aktif bir bölgededir ve büyük bir depreme maruz kalma riskini açıkça taşımaktadır, bu nedenle esnek iksa sistemlerinin kullanımı tercih edilmelidir. Bu çalışmada bir esnek iksa sistemi olan zemin çivili duvarlar üzerinde yoğunlaşılmıştır. Toplam 63,000 m^2 derin zemin çivili duvar uygulamasına sahip altı büyük proje incelenmiş ve değişik yüksekliklerdeki duvar performanslarının sonuçları derlenmiştir. Sonuç olarak, geniş çaplı vaka çalışmalarına ve izlenen iksa performanslarına dayanarak, İstanbul'da gelecekte yapılacak zemin çivili duvar uygulamalarına rehber olması amacıyla tipik grovak formasyonu için temel tasarım ve performans prensipleri oluşturulmuştur. Ayrıca ölçülen yanal deplasman değerlerini sağlayan grovak formasyonunun sıkılık parametrelerini doğrulamak ve yanal deplasman değerleri ile sıkılık parametreleri arasında bağıntı kurmak amacıyla sonlu eleman analizleri uygulanmıştır.

Bunlara ek olarak, bu çalışmada, aynı saha içerisinde farklı iksa sistemlerinin değişik zemin yapısı ve yeraltı su seviyesi durumlarında uygulanmasını içeren bir vaka çalışması detaylı olarak incelenmiştir. Bu çalışmada elde edilen deneyim ve deplasman verileri, kent içinde benzer koşullar altında yapılacak olan gelecekteki uygulamalar için degerli bir veri kaynağı ve örnek teşkil etmesi amacıyla sunulmuştur.

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A_c	Cross-sectional area of grout covering
A_{st}	Cross-sectional area of steel tendon
B_r	Bond ratio, DxL/S
c	Cohesion
D	Nail hole diameter
d	Equivalent thickness
d_i	Depth of the subsurface layer
EA	Normal stiffness
E_c	Elasticity modulus of concrete
EI	Flexural rigidity
E_{st}	Elasticity modulus of steel
E_0	Dynamic elasticity modulus
G_0	Dynamic shear modulus
H_{max}	Maximum soil nailed wall height
H	Excavation height
K_a	Active earth pressure
K_p	Passive earth pressure
K_0	Earth pressure at rest
L	Average nail length
LL	Liquid limit
L_r	Length ratio, L/H
M_D	Disturbing moment
M_{RS}	Restoring moment due to shear strength of soil
M_{RR}	Restoring moment due to presence of soil nails
PI	Plasticity index
PL	Plastic limit
P_r	Performance ratio, δ_h/H
R_{inter}	Interface reduction factor
r_u	Pore pressure ratio
S	S_h x S_v, area per nail

S_r	Strength ratio, D^2/S
v_p	Pressure wave velocity
v_s	Shear wave velocity
w_n	Natural water content
α	Batter angle
β	Slope angle
γ	Total unit weight of the subsoil unit
γ_{unsat}	Soil unit weight above phreatic line
δ_h	Horizontal displacement at top of the soil nail wall
δ_v	Vertical displacement at top of the soil nail wall
η	Nail density, average nail length per area, L/S
ξ	Correction factor for number of pullout tests
ν	Poisson's ratio
Ø'	Internal friction angle
ψ	Dilatancy angle
ω	Nail orienation
CPT	Cone penetration test
DMT	Plate dilatometer test
FHWA	Federal Highway Administration
PMT	Pressuremeter test
SPT	Standard penetration test
UCS	Uniaxial compressive strength
VST	Vane shear test

1. INTRODUCTION

During the last decade, the city of Istanbul has performed significant growth in economy. Becoming the biggest metropolitan city of the region, the need for high-rise residential and office buildings and shopping malls with multiple basement levels increased noticeably considering the raised value of the land, which became a major part of the cost in construction of buildings. In order to build great number of basement levels, especially to obtain parking space and room for shopping and entertainment facilities, deep excavations and construction of retaining structures became compulsory. The depths of the excavations commonly reach to 25 to 40 meters below the ground surface. Most of these tower structures are constructed mostly in the west and the north–west part of the city and especially along the newly developed longitudinal axis, Buyukdere Avenue, having similar subsoil conditions and seismicity.

The city of Istanbul is in very seismically active region. Marmara Fault System is very close to the city, which is the western end of the North Anatolian Fault, NAF, and located at the south of Istanbul. 1999 Kocaeli (M_w=7.4) and Düzce (M_w=7.2) earthquakes occurred on NAF within the Marmara Region in approximately 100-150 kilometers from the city of Istanbul. According to the studies carried out after 1999 Kocaeli and Düzce earthquakes, the probability for the occurrence of a M_w>7.0 earthquake effecting Istanbul within the next 30 years due to the existence of potential seismic gaps is about 65 per cent (Parsons *et al*., 2000).

Based on the probabilistic earthquake hazard analysis using a function relationship of Boore *et al*. (1997), peak ground acceleration for 10 per cent probability of exceedance in 50 years is found to be 0.24 g for the encountered soft/firm rock conditions.

The faulting system of the main Marmara fault indicates the worst-case scenario earthquake is an event, which would involve the two segments immediately south of Istanbul. The length of rupture is 110 kilometers producing an event of M_w=7.5. The distance to the fault from the tower sites is about 25 kilometers, and the focal depth is 12 kilometers.

The encountered subsoil formation mostly in the deep excavations of high-rise buildings of Istanbul is soft rock greywacke locally known as Trakya formation. The term greywacke is used for dark gray, firmly indurated, coarse grained, lithologically alternating sandstone, siltstone and claystones, that consist of poorly sorted, angular to subangular grains of quartz and feldspar, with a variety of rock and mineral fragments embedded in a compact clayey matrix and containing an abundance of very fine grained illite, sericite and chlorite minerals (Eroskay, 1985). They are typically dark grey-green or greyish-brown due to weathering. Sandstone is the most abundant rock type in this formation, and limestone and conglomerate interbeds or lenses are found between layers. The thickness of the Trakya formation varies between 600-1700 meters (Eroskay, 1985).

The Trakya formation is very intensely folded, faulted, fractured, and is also weathered which is well developed along discontinuities. The major structural features of the area are NW-SE and NE-SW trending faults. Dense crack surface developments are observed and rock masses are completely fractured from place to place. Usually, three or four clearly defined major sets of joints are found. Minor sets or random joints also occur in study area. The strike directions of joints are almost NW-SE and NE-SW (Tugrul and Undul, 2006).

At location of tower structures, the main lithological unit present is greywacke formation with alternating layers of sandstone, siltstone and claystones with various degrees of weathering and fracturing. As a result, it became essential for the geotechnical engineers to obtain the geotechnical modeling of subsoils, both to a depth of excavation to employ in the design of retaining structures and well below the foundation levels, to be utilized in foundation design and seismic analysis of these structures. Often, soil investigations were required to be performed to a depth of as great as 50+ meters below the ground surface.

Obviously, the extend of weathering and fracturing of greywacke formation controls the mechanical properties and in fact geological observations do well agree with the results of measurements reflecting mechanical properties of the formation. The geotechnical modeling of formation, weathered zones, extend of fracturing and compressibility modulus of formation are usually obtained by means of integrated seismic survey and Menard pressuremeter testings performed within the boreholes at various locations and depths (Durgunoglu and Yilmaz, 2007). “Geodynamical-seismic” model is driven by integrated shallow seismic surveys below the ground surface. The seismic model is defined by three sets of parameters; geometry of the various subsurface layers, soil-bedrock interface, and the P-wave and S-wave velocities (v_p and v_s) of the layers themselves such as proposed by Yilmaz (2006). In general, the greywacke is being fractured sandstone, siltstone and claystone formation having some degree of weathering close to surface, they are classified as soft rocks having average shear wave velocities in the range of v_s^*=400 – 800 m/sec depending on the extent of fracturing for the top 30 meters.

The range of small strain modulus values for greywacke formation, G_0, 300 to 1200 MPa, determined based on v_s^*, 400 to 800 m/s, values are great. The modulus degradation values have been estimated for the soils in the past by various authors. However, almost no data exists for the rock conditions. Based on the recent study by Durgunoglu and Yilmaz (2007), the modulus ratios for the greywacke formation closer to ground surface, 0 – 30 m, for pertinent soft rock conditions the ratio is about 30 to 50, on the other hand, for deeper layers, 30+ m, pertinent firm rock conditions the ratio is about 50 to 200.

The greywackes, which constitute the basement rock formation of the city of Istanbul, are classified as critical as far as their stability due to excavation is concerned. They contain very complex discontinuity surfaces and sliding and rock falls along these discontinuities are very common at unsupported excavations made in this rock formation. The greywacke beds, which possess rather high shear strength and uniaxial compressive strength, are very poor from the stability viewpoint due to relatively low residual shear strength they develop when exposed to water and atmosphere upon excavations. Therefore, the deep excavations to be utilized in Istanbul greywackes should be supported by various means (Saglamer, 1986).

Considering the seismicity of the city of Istanbul, one of the principal concerns while selecting appropriate retaining system whether it is flexible or rigid. Other factors affecting the predetermination of the retaining system are depth of cut, location of neighboring structures and infrastructures, time schedule, budget constraints, access to the site and the available techniques that can be readily employed by the geotechnical contractor. The retaining systems widely used in the deep excavations in the city of Istanbul are tied-back cast in-situ piles, tied-back micro piles, tied-back cast in-situ reinforced concrete walls (manual caisson, segmented or diaphragm walls), strutted or shored support walls, soil nailing or combinations of these.

Since Istanbul is in very seismically active region and obviously at risk of being hit by major earthquake, flexible retaining walls especially for permanent applications should be preferred in deep excavations carried out in the city. Based on the previous positive records of flexible earth retaining structures during earthquakes in Turkey by Mitchell *et al*. (2000) and Durgunoglu *et al*. (2003a), soil nailed walls in such excavations performed within the city offer great advantage especially for the encountered subsoil and seismic conditions. Therefore soil nailing as a flexible retaining system deeply emphasized and thoroughly examined in this study.

Soil nailing is a technique where either natural soil or existing fill material is reinforced by the insertion of slender tension-carrying elements called soil nails. Soil nails are made of metallic or polymeric material and may be installed into pre-drilled hole then grouted, drilled and grouted simultaneously, or inserted using a displacement technique. While the terms “soil nail wall” and “soil nailing” are broadly applied to soil systems, the technique is also applicable to excavations in soil-like materials as soft rock or weathered rock, such as greywacke.

The basic idea of soil nailing consists of the reinforcing the ground by closely spaced passive, that means not pre-stressed as for ground anchors, inclusions to built up a coherent gravity structure by increasing shear strength of in-situ soil and restraining its movement as some other type of soil reinforcement techniques. Soil nailing is typically used to stabilize existing slopes or excavations where top-to-bottom construction is advantageous compared to other retaining wall systems. For certain conditions, soil nailing offers a viable alternative from the viewpoint of technical feasibility, low construction costs, and short construction duration when compared to utilizing pre-stressed ground anchor walls, which is another top-to bottom retaining system.

Although several thousand soil nail structures have been constructed worldwide, only a limited number have been instrumented to provide performance data to support design procedures and ensure adequate performance (FHWA, 2003). Performance monitoring instrumentation for such walls should include inclinometers, top of wall survey points, load cells, and strain gauges. The most powerful method of monitoring a soil nailed structure is to observe its horizontal displacements because in literature most of the failures have reported being due to the horizontal movement of nails by starting from the top. Therefore, a special concentration should be paid to the horizontal displacement reflecting performance of the retaining structure and in order to observe these movements, inclinometers are the best solution.

During the last ten years soil nailed walls have been extensively constructed within the city of Istanbul as temporary and permanent retaining walls to support the basement excavations of various structures. According to recent compilation by Zetas (2006) about 160,000 m^2 of wall had been constructed in 60 different projects and the performances of some of these soil nailed wall structures have been reported previously by Ozsoy (1996), Durgunoglu *et al*. (1997) and Yilmaz (2000). In this study six major case study of deep soil nailing walls having total surface area of 63,000 m^2 of which BJK Fulya Complex, Istinye Park Complex, Kanyon Complex, Mashattan Residence, Tepe Shopping Mall and Besler Warehouse that constructed in Istanbul in locally well-known Trakya formation – greywacke are examined.

The results of the performance of walls with different heights in various sites having the similar greywacke subsoil formation are compiled. The performances of walls are monitored by inclinometer recordings taken at certain time intervals in parallel to the excavation at various locations. The displacement and normalized displacement, i.e. performance ratio, P_r, data are presented together with some basic parameters of soil nailed walls such as, height of wall (H), area per nail (S), average nail length (L), nail density (η=L/S), length ratio (L_r), bond ratio (B_r) and strength ratio (S_r). As a result the values of performance ratio for soil nailed walls together with nail length and density in typical greywacke formation of the city of Istanbul are developed based on these extensive case studies as a guideline for future applications.

The main shortcoming of the limit equilibrium design methods is that they do not give a prediction of displacements. They also do not consider the displacement required to mobilize the resisting forces in the soil and soil nails. These methods cannot therefore provide a thorough description of the contribution of each soil nail to overall stability based on the pattern of deformation behind the slope or wall (FHWA, 2003). Displacements can be predicted approximately using empirical correlations and while these are appropriate in many cases they have limitations, especially for deep excavations (Durgunoglu *et al*., 2003b)

In situations where more confidence is required, a higher level of analysis should be adopted using numerical modelling such as finite element and/or finite difference methods. The accuracy of numerical modelling depends on the quality of data acquired, the estimation of in-situ stress and soil stiffness and the availability of good case histories to calibrate the numerical models. The stiffness parameters in these models should be adjusted to match the values obtained from actual site monitoring results at an early stage.

Observations of displacements during construction are essential and cannot be replaced by numerical modelling. Even using numerical modelling, it is still relatively difficult to predict displacements for layered soil stratigraphies and nonhomogeneous weathered rocks with discontinuity surfaces such as seen in greywacke formation. The accurate modelling of the grout/soil interface is also difficult, as mobilisation of tension forces is often not directly proportional to facing displacements and/or construction stages (Phear *et al*., 2005).

In this study finite element computer program 'PLAXIS' was deployed to simulate the excavation sequence and installation of nails and to carry out back analysis for the six case studies in Istanbul of which performance analysis is achieved by means of inclinometer monitoring of lateral displacements. The finite element analyses were aimed to verify stiffness parameters of subsoil greywacke formation resulting the measured lateral displacements and to come across a relation between the actual lateral displacements and the stiffness parameters.

Detailed numerical analysis was performed on a specific section which thorough examination on the excavation steps and variation of the lateral displacement with depth and time was made. Stiffness parameters were adjusted on every excavation stage in order to match the actual lateral displacements and thus actual subsoil profile in terms of stiffness were achieved. In this comprehensive finite element back analysis, the importance of verification and improvement of design during the construction stage through close observation and monitoring techniques is exposed. In this fashion it is possible to recognize and feed back potential problems and also make potential cost savings.

Additionally, an interesting case study is presented in this study for the utilization of various retaining systems for different subsoil and groundwater conditions encountered within a given site, considering the output of optimization of the cost as well. The project is known as "BJK Fulya Complex" consisting of high-rise residential twin towers, a hospital building and a hotel building covering approximately 160,000 m2 floor area including a hypermarket, a technomarket, a cultural center, entertainment facilities and underground parking area. The project is located at a very prestigious district of the city, therefore maximum underground space gain were desired. As a result nearly 20 m of excavation was performed partly under groundwater.

The project site has a very rugged topography having about 25 m difference in elevation in perpendicular direction to the covered old creek located at the bottom of the valley along the main street. Due to unique topography and geology, subsoil and groundwater conditions at various faces of the excavation differ considerably. Furthermore, again due to unique topography, at the hillside in addition to 18.5 m of temporary retaining structure, permanent retaining structure of about 15-20 m high had to be constructed over the temporary wall leading to a retaining structure as high as 36 meters.

Due to complicated geology and the high seismicity of this site, it was necessary to employ extensive soil investigations to identify the limits of various lithological units and the ground water conditions. As a result, various types of retaining structures were employed having both flexible and rigid retaining systems at various locations within the site of “BJK Fulya Complex”. Various forms of retaining structures that have been utilized at the site include temporary soil nailing, permanent soil nailing, temporary and permanent soil nailing along with the permanent tied-back cast in-situ reinforced concrete caisson wall and temporary tied-back diaphragm wall consist of soldier cast in-situ piles with jet grout columns in between.

The performances of various systems at the subject site are closely monitored by means of inclinometer recordings taken at certain time intervals in parallel to the staged excavation. Readings from sixteen inclinometers at different locations were recorded throughout the construction. The performances of various retaining systems employed are critically evaluated.

Displacement data and experience obtained from this case study together with previous experience (Durgunoglu *et al.*, 2007a) serves an excellent source of data and example for future applications in similar conditions within the city.

2. THE SEISMICITY AND THE SUBSOIL CONDITIONS

2.1. Introduction

Istanbul is located at a seismically active region. Marmara Fault System is very close to the city, which is the western end of the North Anatolian Fault, NAF, and located at the south of Istanbul. 1999 Kocaeli (M_w=7.4) and Düzce (M_w=7.2) earthquakes occurred on NAF within the Marmara Region in approximately 100-150 kilometers from the city of Istanbul. According to the studies carried out after 1999 Kocaeli and Düzce earthquakes, the probability for the occurrence of a M_w>7.0 effecting Istanbul within the next 30 years due to the existence of potential seismic gaps is about 65% (Parsons *et al*., 2000).

Within the deep excavations of high-rise residential and office buildings and shopping malls, the main lithological unit encountered is soft rock greywacke locally known as Trakya formation with alternating layers of sandstone, siltstone and claystones with various degrees of weathering and fracturing. The extent of weathering and fracturing controls the mechanical properties. The geotechnical modeling of formation, weathered zones, extend of fracturing and compressibility modulus of formation are usually obtained by means of integrated seismic survey and Menard pressuremeter testings performed within the boreholes at various locations and depths (Durgunoglu and Yilmaz, 2007).

The greywackes, which constitute the basement rock formation of the city of Istanbul, are classified as critical as far as their stability due to excavation is concerned. They contain very complex discontinuity surfaces and sliding and rock falls along these discontinuities are very common at unsupported excavations made in this rock formation. The greywacke beds, which possess rather high shear strength and uniaxial compressive strength, are very poor from the stability viewpoint due to relatively low residual shear strength they develop when exposed to water and atmosphere upon excavations. Consequently, it became essential for the geotechnical engineers to obtain the geotechnical modeling of subsoils, both to a depth of excavation to employ in the design of retaining structures and well below the foundation levels, to be utilized in foundation design and seismic analysis of these structures.

2.2. The Seismicity of Istanbul

Turkey is located at the boundary area where the Arabian Plate and African Plate are moving north towards the Eurasian Plate, as shown in Figure 2.1. From east to west, a large-scale fault line called North Anatolian Fault is formed more than 1000 km long in the northern part of Turkey and many strong earthquakes have occurred along this fault line historically. Istanbul city, which is located in the western part of Turkey, lies on an active seismic zone ranging from Java–Myanmar–Himalaya–Iran–Turkey and Greece, where many large earthquakes occurred in the past. The North Anatolian Fault, which is a right-lateral strike-slip tectonic fault, draws the boundary between Black Sea microplate and Anatolian microplate (Kilic *et al*., 2006).

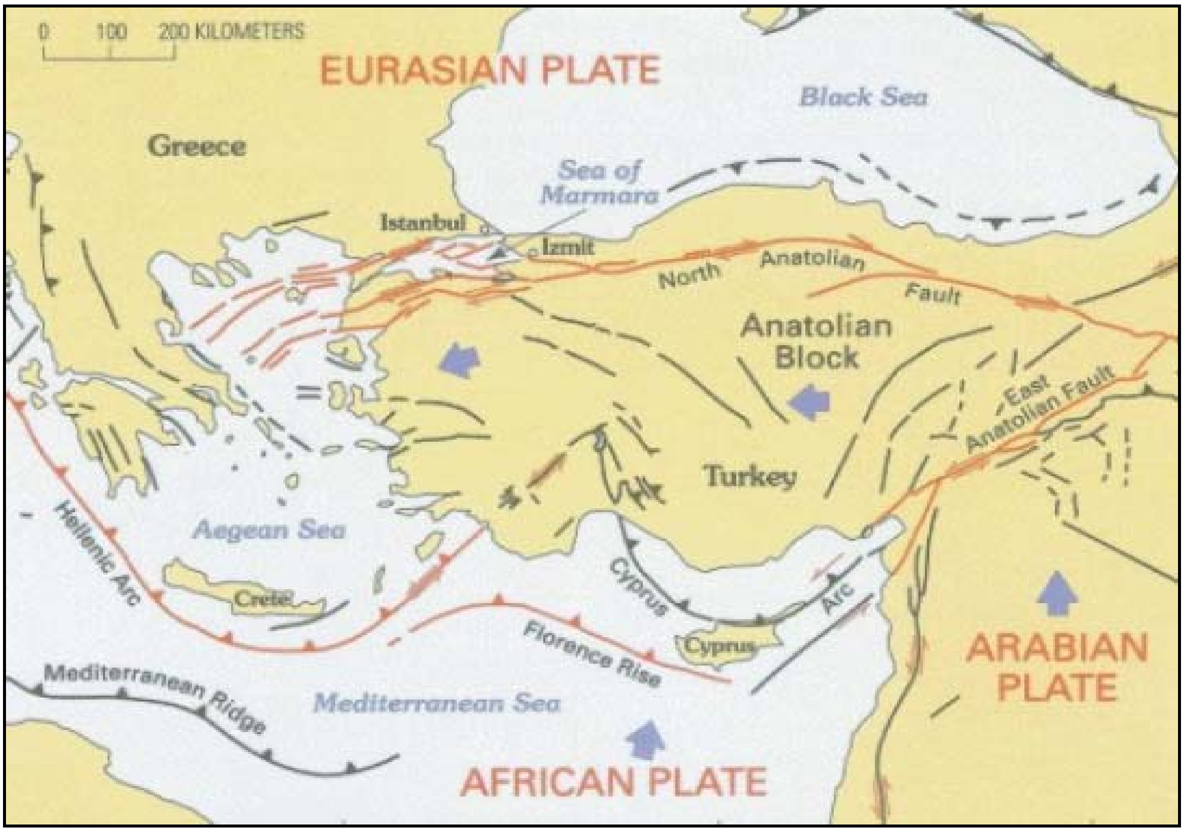


Figure 2.1. Tectonic map of Turkey (Barka, 1992; Undul and Tugrul, 2006)

Since North Anatolian Fault Zone is one of the most active fault zones on earth, several researchers have concentrated on the earthquakes that have taken place along on the during last century, especially during 1939–1999. During this period, seven major earthquakes with surface rupture and with magnitudes in excess of 7.0 had occurred. These earthquakes had ruptured a total segment length of more than 800 km along the fault zone (Kilic *et al*., 2006).

The city of Istanbul is in very seismically active region. It is potentially under the influence area of the Marmara Fault System, located at the south, in the Marmara Sea, which is the western end of the North Anatolian Fault. Marmara region in Northwestern Turkey has been the site of numerous destructive earthquakes in history and the most of the historical documentation is related to damages suffered in Istanbul. Istanbul has been the crossroad between east and west for more than two thousand years. Being a continuously populated region and having as its center Istanbul, the capital of both Byzantine and Ottoman empires, the historical seismicity record is continuous and relatively complete (Erdik *et al*., 2004). The long-term seismicity of the Marmara region is illustrated in Figure 2.2.

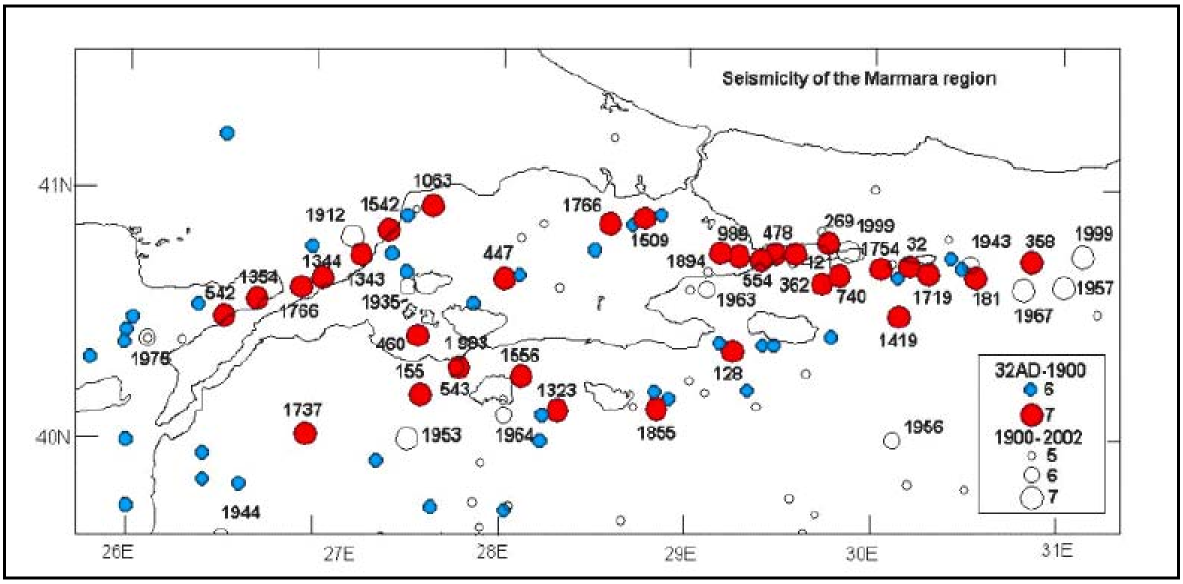


Figure 2.2. Seismicity of the Marmara Region (Ambraseys and Finkel, 1991)

Earthquake records spanning almost two thousand years indicate that, on average, at least one medium-intensity earthquake has affected Istanbul in every 50 years and the average return period for high intensity events has been 300 years (Erdik *et al*., 2004). There has not been any earthquake that ruptured the entire length of the Main Marmara

Fault from Gulf of Izmit to Gulf of Saros; the seismicity accounts for all of the expected 2.2 cm/year slip and; there is a time dependence of seismic activity that should be accounted in earthquake hazard assessments (Ambraseys, 2002)

Recently August 17, 1999, Kocaeli (M_w=7.4) and November 11, 1999, Düzce (M_w=7.2) earthquakes occurred on North Anatolian Fault within the Marmara Region in approximately 100-150 kilometers east of the city of Istanbul. In these two catastrophic earthquakes 18,000 people were killed, 15,400 buildings were destroyed, and 10 to 25 billion U.S. dollars damage was occurred. But the Kocaeli earthquake is only the most recent in a largely westward progression of seven large earthquakes along the North Anatolian fault since 1939 (Parsons *et al*., 2000).

After these two catastrophic earthquakes further worldwide scientific interest has been given to the structure of North Anatolian Fault System, especially under the Marmara Sea. Le Pichon *et al*. (1999) developed a fault model based on the data collected in 1997 by the ship "MTA Sismik-1". Data obtained during the recent high-resolution bathymetric survey of the Ifremer RV Le Suroit vessel, Figure 2.3, indicates that a single, thoroughgoing strike-slip fault system cuts the Marmara Sea from east to west joining the 1999 Kocaeli earthquake fault with the 1912 Sarkoy-Murefte earthquake fault.

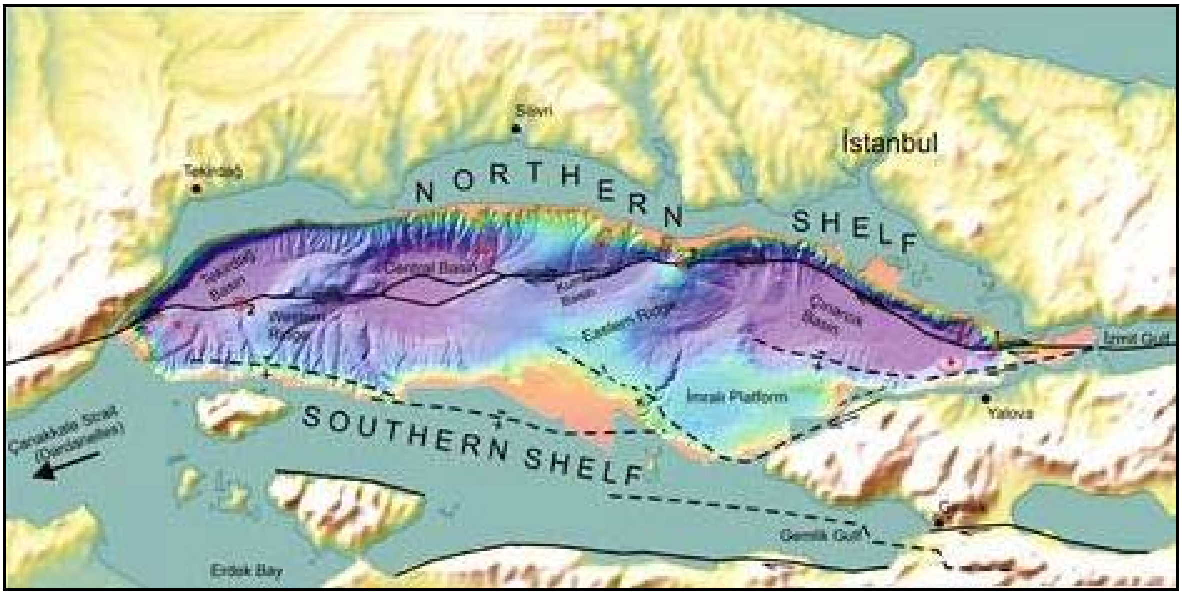


Figure 2.3. Marmara Fault System (Le Pichon *et al*., 1999)

The Kocaeli earthquake, which was caused by the rupture of a 110-km segment in the western extension of the North Anatolian Fault, resulted in damage in some districts in Istanbul. The North Anatolian Fault Zone is forked to the west of the gulf of Izmit, the northern branch being the major seismic source for Istanbul. A strong movement along this segment would result in a major earthquake to hit the areas along the north coast of the Marmara Sea. It is obvious that Istanbul is at risk of being hit by a major earthquake (Kilic *et al*., 2006). According to the studies carried out after 1999 Kocaeli and Düzce earthquakes, the probability for the occurrence of a M_w>7.0 earthquake effecting Istanbul within the next 30 years due to the existence of potential seismic gaps is about 65% as indicated in Figure 2.4. (Parsons *et al*., 2000).

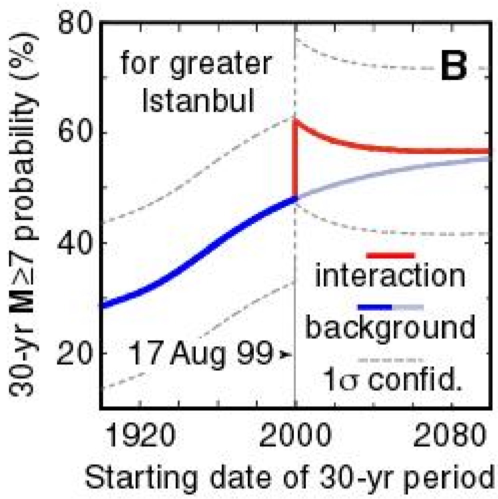


Figure 2.4. Probability for the occurrence of an M_w>7.0 earthquake affecting Istanbul for the next 30 years (Parsons *et al*., 2000)

According to the earthquake risk map of Istanbul, presented in Figure 2.5, the southern parts of Istanbul are under higher earthquake risk. From the south to the north, earthquake risk gradually decreases, because the active faults are in the Marmara Sea.

The most of the high-rise residential and office buildings are being built on the “second degree risk” earthquake zone in Istanbul according to the Specification for Structures to be built in Disaster Areas by the Ministry of Public Works and Settlement of the Republic of Turkey effective from 01/01/1998. The effective ground acceleration coefficient, A_0 is given as 0.30. Given the soil/rock conditions, being greywacke formation, the sites have a local site class Z1. According to the 1998 Earthquake Code, spectrum characteristic periods are given as T_A=0.10 sec and T_B=0.30 sec for Local Soil Class “Z1”.

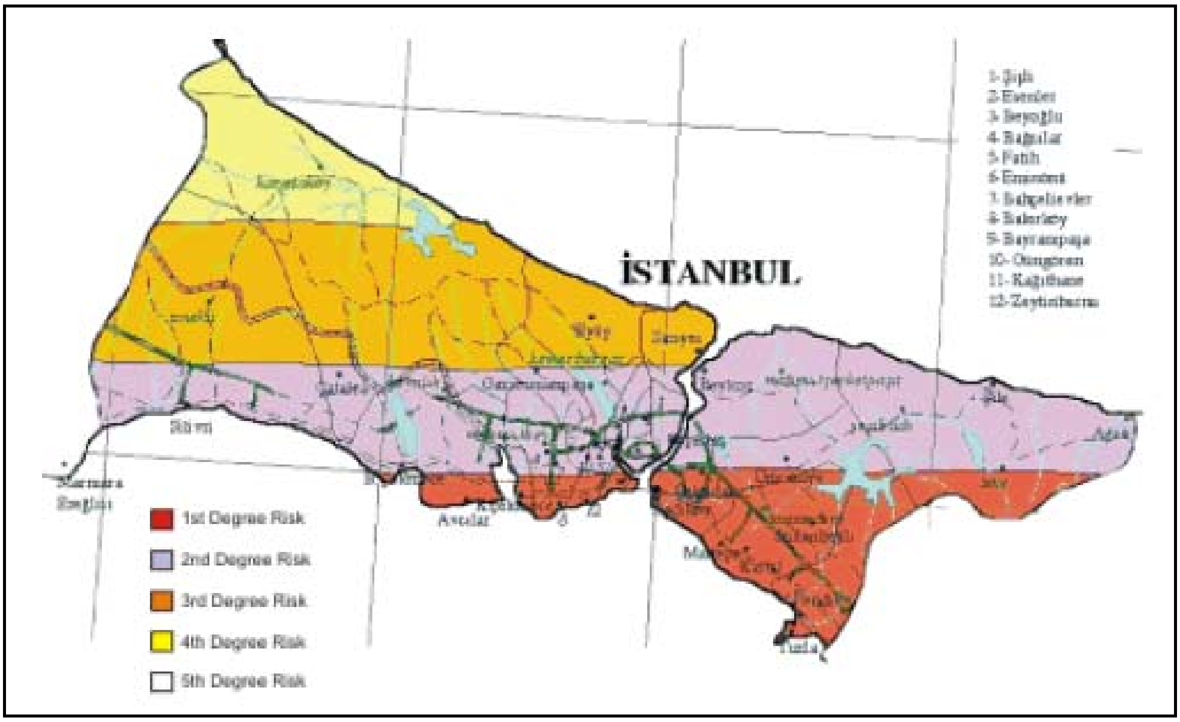


Figure 2.5. Earthquake zoning map of Istanbul

Based on the probabilistic earthquake hazard analysis using a function relationship of Boore *et al*. (1997), Peak Ground Acceleration for 10% Probability of exceedance in 50 years is found to be 0.24g for the encountered soft/firm rock conditions represented by greywacke formation.

The faulting system of the main Marmara fault indicates the worst-case scenario earthquake is an event, which would involve the two segments immediately south of Istanbul. The length of rupture is 110 kilometers producing an event of M_w=7.5. The distance to the fault from the tower sites is about 25 kilometers, and the focal depth is 12 kilometers.

2.3. The Geology of Istanbul

Geological units within Istanbul peninsula start with Early Paleozoic and continue conformably from Silurian through lower Carboniferous. This sequence is overlaid by the Triassic sedimentary rocks uncomformably. Paleozoic aged units generally comprise detrital, carbonaceous rocks of Dolayoba, Kartal, Baltalimani and Trakya Formations (Ternek *et al*., 1987).

The Dolayoba Formation, aged Silurian, is the oldest unit of the peninsula and consist of cemented limestone, quartzite sandstones. The limestone mainly made up of reef. Then, the Lower-Middle Devonian Kartal Formation, Middle-Upper Devonian Tuzla Formation, Lower Carboniferous Baltalimani Formation and Trakya Formation deposited conformably (Zabci *et al*., 2003).

Devonian aged Kartal formation rock consists of coarse limestone, limy shale, graywacke and fossilious limestone with clay interlayer. Tuzla Formation made up of fossilious limestone, calcareous shale and siliceous-bedded rock (Kaya, 1973).

Carboniferous aged rock unit, which are located at upper levels of Paleozoic Basement and occurred mostly at the western European side of the Bosphorous. Carboniferous sequence starts with its bottom unit of Baltalimani Formation over Tuzla Formation. Baltalimani Formation consists of lydites and siliceous shale bounded by nodular limestone and/or graywacke at the base and partly calcareous shale at the top. This slightly thin unit outcrops at a narrow belt. The typical composite section is derived from Baltalimani Creek. The lydites are dark gray to black, thinly laminated and form sedimentation units average 4 cm in thickness. The unit includes a prolific radiolarian micrafauna (Kaya, 1973). Then Trakya formation deposited on Baltalimani formation conformably. Trakya Formation is a succession of shale, siltstone, greywacke, sandstone subordinate conglomerate and carbonates, bounded by lydites at the base and limestone at the top. A regional tectonism and related regression accompanies to erosion of Paleozoic sequence. Hercynian orogeny of Late Paleozoic resulted in regional uplift and erosion of Paleozoic rocks. General geological map of Istanbul is shown in Figure 2.6.

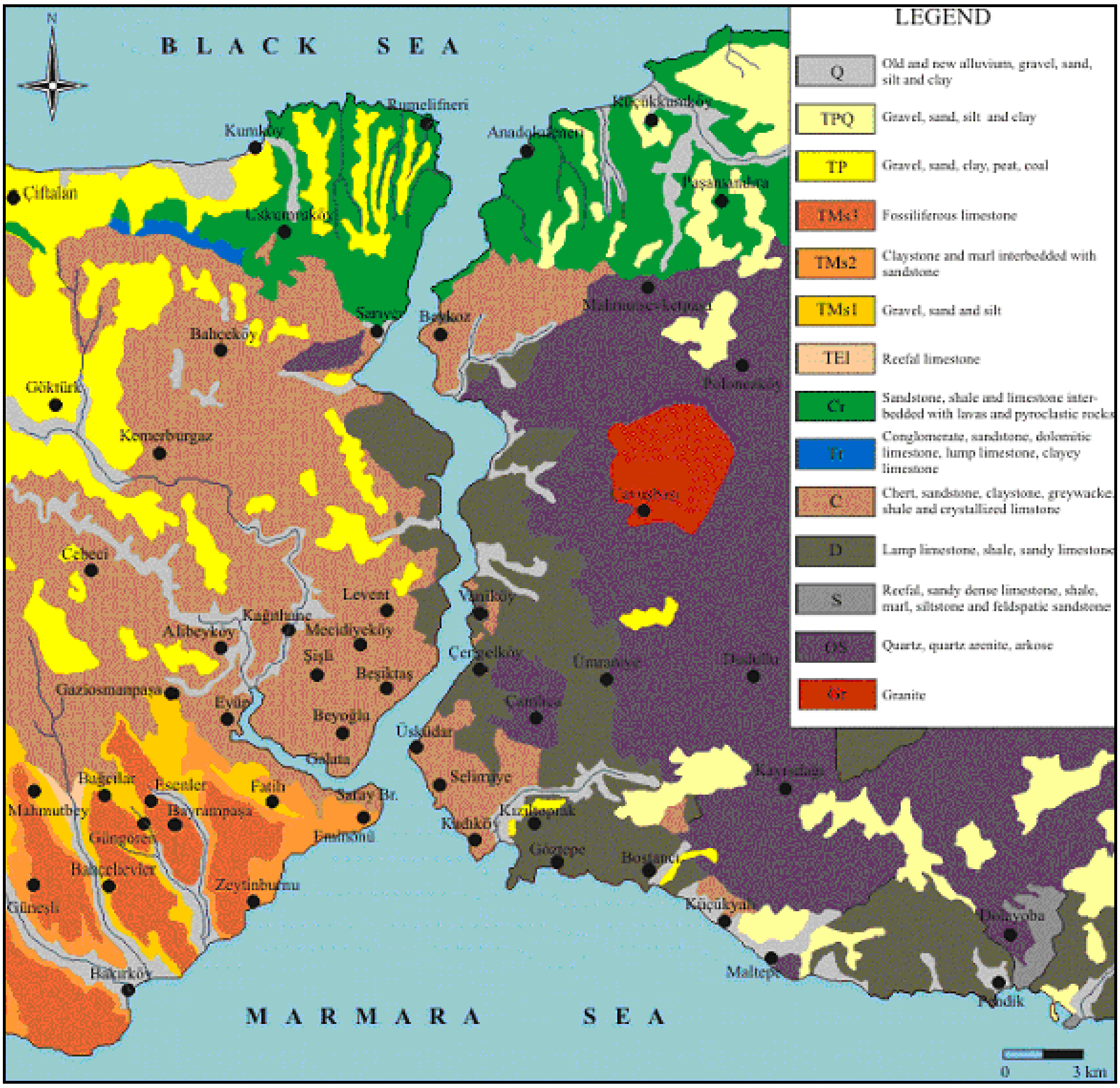


Figure 2.6. Geological map of Istanbul (Ketin, 1991; Undul and Tugrul, 2006)

Tertiary sediments lying with angular unconformably on Paleozoic formations start with Kirklareli formation, the deposition of which continued from Middle Eocene through Early Oligocene. The sequence starts transgressively with a basal conglomerate and levels containing clay and coal, which are fallowed by cream limy claystone and karstic reef limestone. The sequence ends with lithologies such as argillaceous limestone, marl and limy sandstone. On top of this unit, Gurpinar, Camurluhan, Cukurcesme, Gungoren and Bakirkoy Formations, a new phase of deposition, commencing again with layers of gravel, sand and coal, took place till the end of Miocene (Yildirim and Savaskan, 2003).

The sand and gravel layers often encountered in the bottom levels of Gurpinar are gradually followed by overconcolidated green clay which is frequently interbedded with sand.

Camurluhan Formation conformably overlies Gurpinar formation in a very restricted area, especially north of Bayrampasa, south of Atisalani and southwest of Gaziosmanpasa. Camurluhan Formation consists of gravel, sand, clay with sand interlayer, marl, weak sandstone.

Cukurcesme Formation overlies conformably Gurpinar clay but unconformably Trakya Formation. It is made up of gray, grayish white fossilius sand and gravel with greenish brown clay and marl interlayers.

Gungoren Formation is situated as gradually passing up from Cukurcesme unit of the sedimentary bottom and passes up to overlying Bakirkoy limestone. Gungoren Formation is dominantly grayish green colored having silt and fine sand bands or chalky limestone interbedings, locally contains carbonate lumps. It’s thickness change between 10 and 20 meters.

Bakirkoy Formation comprises white, gray limestone with green, thin clay layer. Clay, limestone interbedding can be observed at the lowermost levels of the sequence.

The Pliocene aged Belgrad and Samandira Formation deposits lie unconformably on older units. It is made up reddish yellow clay, silt, grayish silt and fine sand.

Quaternary represented by stream sediments consisting of gravel, sand and clay. It is well developed in Istanbul Peninsula covering basements of valleys from small rivers to large depression areas. Figure 2.7 represents the general stratigraphy of Istanbul.

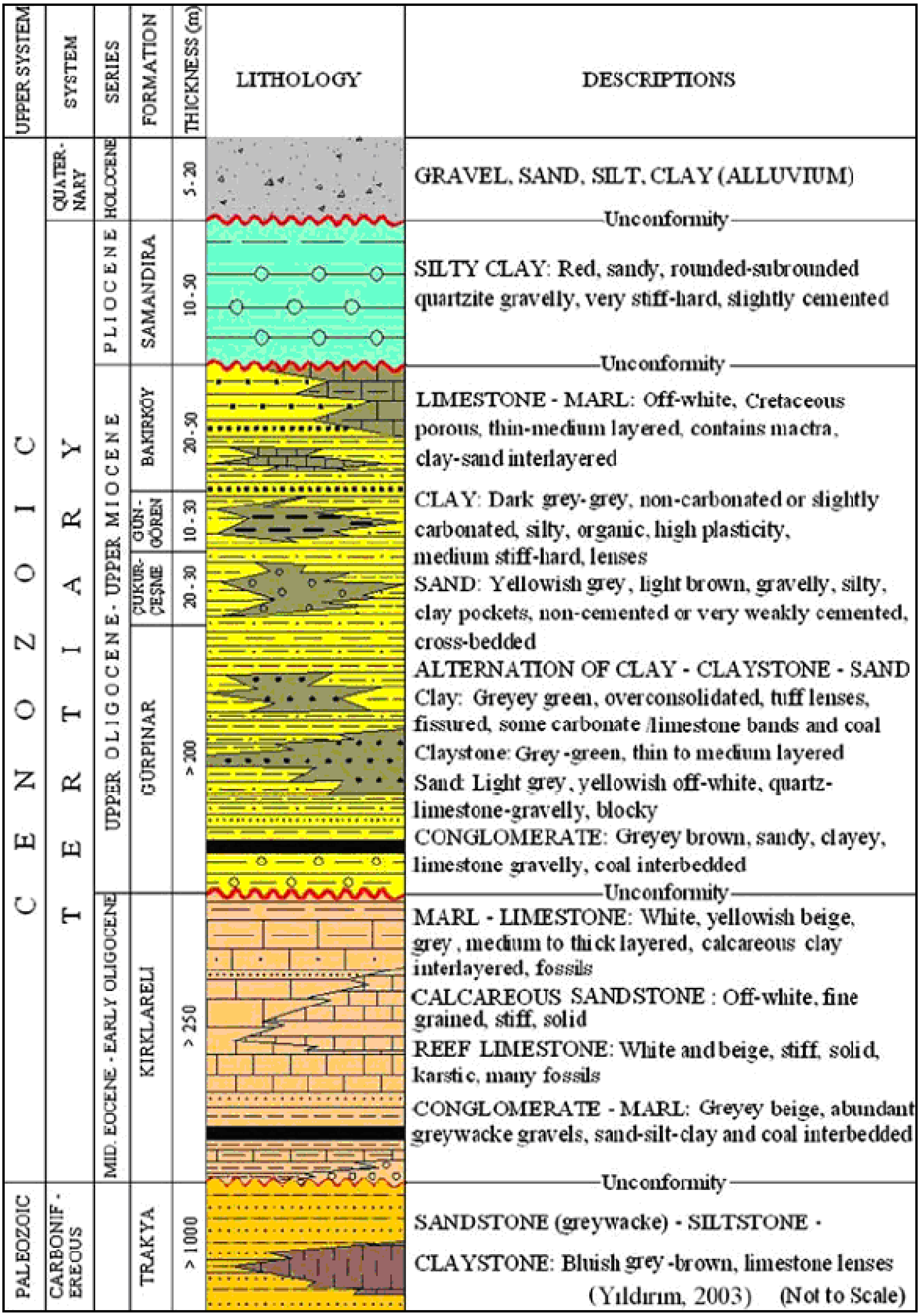


Figure 2.7. General stratigraphy of Istanbul (Kilic *et al*., 2006)

2.4. The Engineering Geological Characteristics of Istanbul Greywackes

Throughout the deep excavations of high-rise buildings located along the axis of Buyukdere Avenue in the city of Istanbul, typically soft rock greywacke, locally known as Trakya formation, is encountered. The Trakya formation is commonly exposed in the European side of Istanbul, especially in Ikitelli, Cebecikoy, Sisli, Besiktas, Levent and Gaziosmanpasa. General location of greywacke formation in Istanbul is given in Figure 2.8.

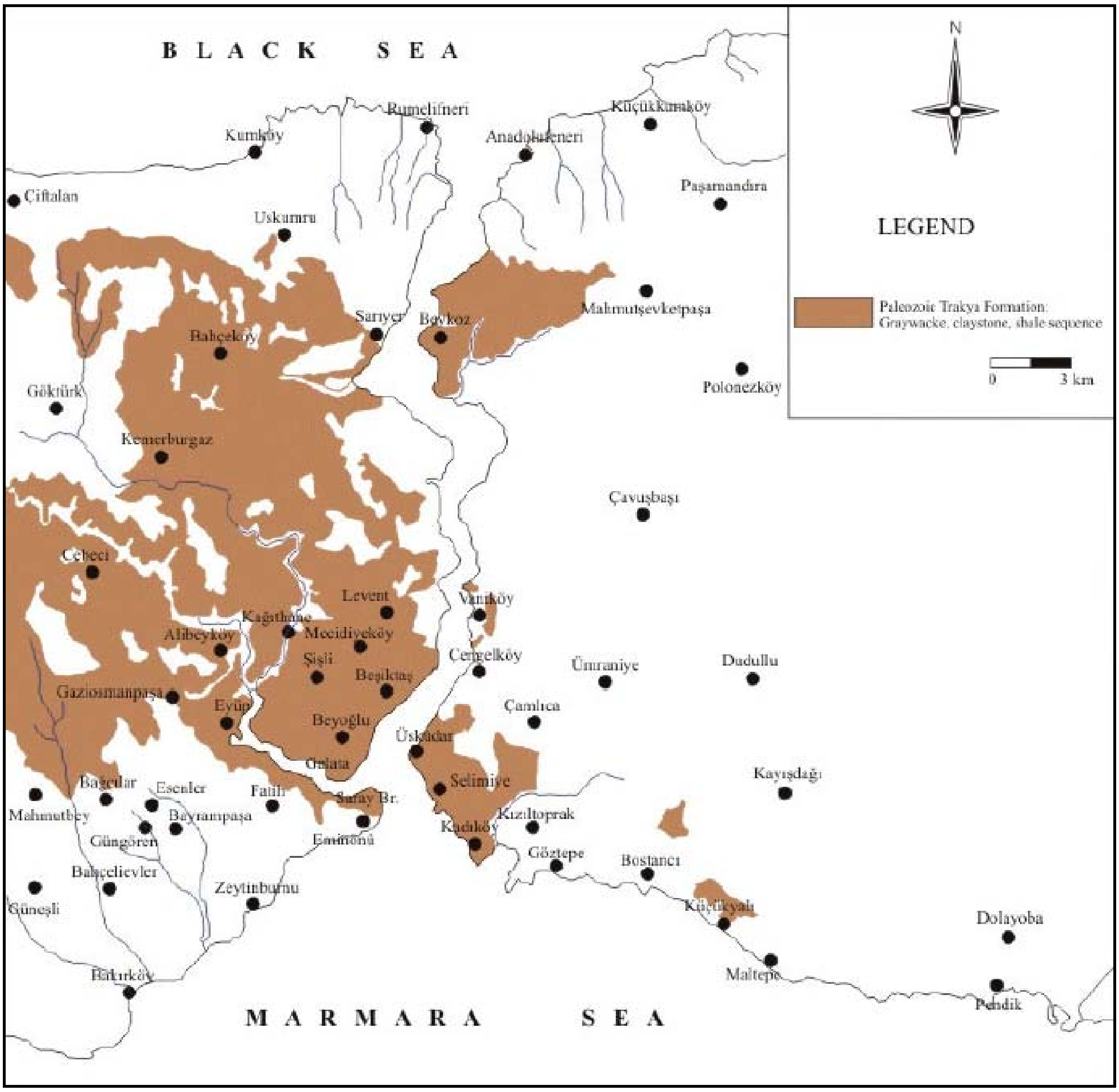


Figure 2.8. The location of greywackes in Istanbul (Ketin, 1991; Tugrul and Undul, 2006)

The term greywacke is used for dark gray, firmly indurated, coarse grained, lithologically alternating sandstone, siltstone and claystones, that consist of poorly sorted, angular to subangular grains of quartz and feldspar, with a variety of rock and mineral fragments embedded in a compact clayey matrix and containing an abundance of very fine grained illite, sericite and chlorite minerals (Eroskay, 1985). Few examples of greywackes that are met in the deep excavations in the city are given in Figure 2.9.

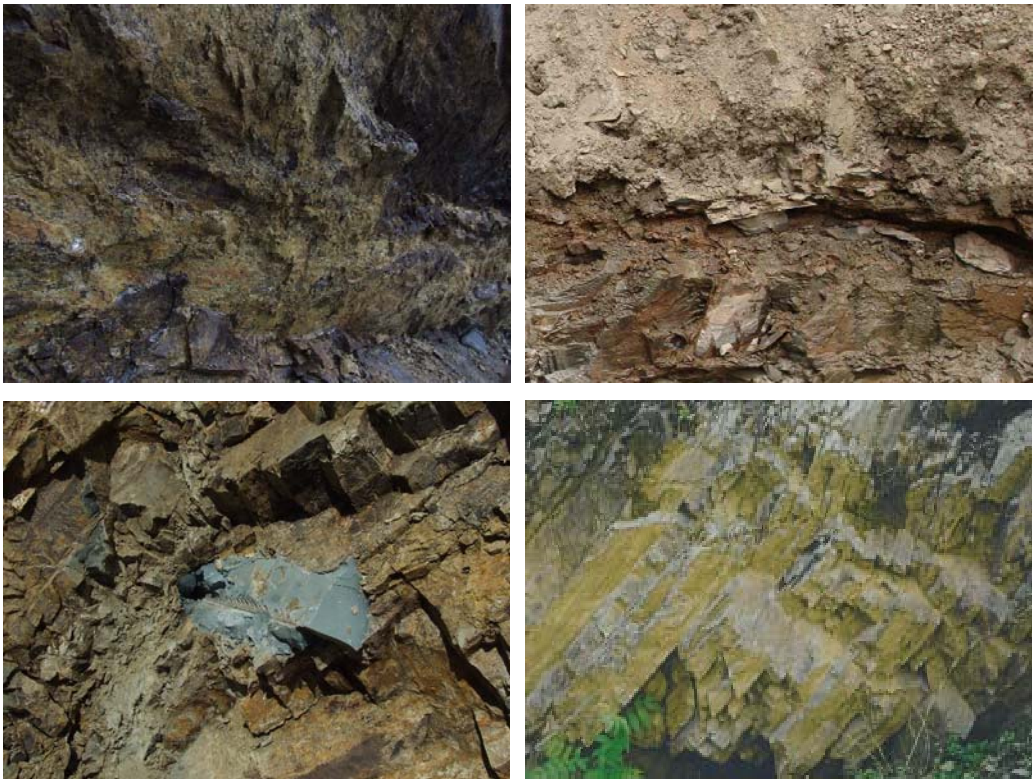


Figure 2.9. Features of typical greywacke formation in Istanbul

The interrelationships of primary textural characteristics such as grain size, shape (form, roundness, surface texture), and fabric (grain orientation and grain-to-grain relations) control other properties such as density, porosity, etc. (Boggs, 1987). The relationship between petrographical, mineralogical, chemical and geomechanical properties of this type of rock has been studied by many investigators. They reported significant correlations between geological and geomechanical characteristics of different types of sandstones.

The Trakya Formation, greywackes, consists of intercalated sequences of sandstone, shale, siltstone and claystone. They are typically dark grey-green or greyish-brown due to weathering. Sandstone is the most abundant rock type in this formation, and limestone and conglomerate interbeds or lenses are found between layers. The thickness of the Trakya formation varies between 600-1700 metres (Eroskay, 1985). Greywackes are generally of marine origin and believed to have been deposited by submarine turbidity currents (Pettijohn, 1972). Greenish brown coloured andesite dykes, up to 10 m thick, are common in the city and generally follow a NW-SE direction. When they were fresh, they could be easily identified in the field. But in the highly weathered conditions, dykes which were yellowish brown coloured, could only be differentiated with difficulty from the greywackes (Eroskay, 1985).

The Trakya formation is very intensely folded, faulted, fractured, and is also weathered which is well developed along discontinuities. The major structural features of the area are NW-SE and NE-SW trending faults. Dense crack surface developments are observed and rock masses are completely fractured from place to place. Usually, three or four clearly defined major sets of joints are found. Minor sets or random joints also occur in study area. The strike directions of joints are almost NW-SE and NE-SW (Tugrul and Undul, 2006).

Petrographical and mineralogical analyses have been used to document the fabric and mineralogical properties of both unweathered and weathered greywackes. Generally coarse and medium grained greywacke sandstones are dominated by angular and subangular grains of quartz, and a lesser amount of feldspars (orthoclase and plagioclase), muscovite, chlorite and hematite with a variety of dark rock and mineral fragments. The quartz grains range from 60 to 75%. The rock fragments are between 8-15%; the muscovite is more than 3%. The cemented materials consisted mainly of clay (sericite, chlorite and illite) and sometimes hematite. They are generally poorly cemented, but have been firmly lithified by compaction and close packing of the sand-size grains (Tugrul and Undul, 2006). The above petrographic characteristics classify the rock as lithic wacke according to Dott (1964). Based on the Wentworth (1922) scale size classes, the greywackes range from very coarse to coarse sandstone.

The extend of weathering and fracturing controls the physical and mechanical properties of greywacke formations. The weathering of the greywackes decreases gradually and progressively with increasing depth. Physical properties of Istanbul greywacke are given in Table 2.1. Both porosity and water absorption increase and dry unit weight decreases with increasing weathering grade.

Table 2.1. Physical properties of greywackes in different locations in Istanbul.

Weathering	Location	Dry unit weight γ_d (kN/m^3)	Water absorption (by weight) w_a(%)	Total porosity (%)	References
Weathered	Levent (Deep Excavation)	25.6-26.1	2.23-4.12	-	Tugrul and Undul (2006)
	Eminonu (Boreholes)	25.8-26.6	3.14-3.73	8.47-10.23	Zarif and Tugrul (2002)
	Eyup Tunnel	26.0	2.8	8	Erguvanli (1985)
	Halic Tunnel	25.9	1.9	-	Erguvanli (1985)
	Average	**26.0**	**3.0**	**8.9**	
Unweathered	Levent (Deep Excavation)	26.4-27.3	0.22-1.06	-	Tugrul and Undul (2006)
	Eminonu (Boreholes)	26.5-27.7	0.39-1.49	0.95-3.66	Zarif and Tugrul (2002)
	Eyup Tunnel	27.2	0.22	2	Erguvanli (1985)
	Dolmabahce-Baltalimani	26.5	0.97	-	Erguvanli *et al.* (1987)
	Average	**26.9**	**0.7**	**2.2**	

Pore size distribution could be determined using the mercury intrusion technique as the mercury is forced in the specimen and its volume is determined from the displaced fluid volume. During the test, the mercury fills up the large pores under low pressure and the small ones under high pressure. The results obtained from porosimeter tests are demonstrated in Figure 2.10 (Tugrul and Undul, 2006). The effective porosity values of weathered samples are generally higher than unweathered samples. Changes in pore geometry are caused by the dissolution of some minerals and increase in microfracture density by progression of weathering and new mineral formation (Tugrul, 1995). The strength of the unweathered and weathered greywackes could be determined on sample cores using the uniaxial compression test. Results of laboratory tests at different locations in Istanbul are given in Table 2.2. The mechanical properties of the greywackes, i.e. strength and modulus, decrease considerably with weathering.

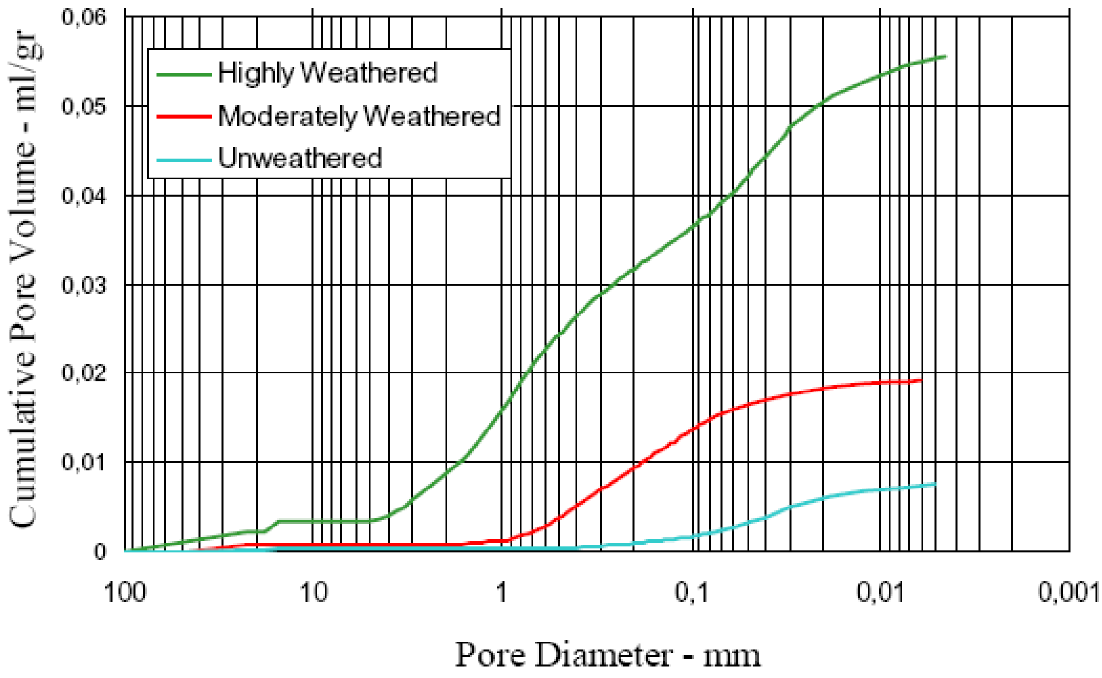


Figure 2.10. Porosity change with weathering of greywackes (Tugrul and Undul, 2006)

Table 2.2. Mechanical properties of greywackes in different locations in Istanbul.

Weathering	Location	Uniaxial Comp. Strength UCS, (MPa)	Compression Modulus, E (10^3 MPa)	Poisson Ratio ν	References
Weathered	Levent (Deep Excavation)	38-62	4-36	-	Tugrul and Undul (2006)
	Eminonu (Boreholes)	14.1-43.3	33.9-47.3	0.16-0.33	Zarif and Tugrul (2002)
	Eyup Tunnel	35-60.5	4.1-9.6	0.22-0.24	Erguvanli (1985)
	Dolmabahce-Baltaliman	36-62.1	1.6-18	-	Erguvanli *et al.* (1987)
	Average	**43.9**	**19.3**	**0.24**	
Unweathered	Levent (Deep Excavation)	58-64.7	10.8-48.2	-	Tugrul and Undul (2006)
	Eminonu (Boreholes)	51.3-66.2	13.9-96.3	0.20-0.26	Zarif and Tugrul (2002)
	Fatih Tunnel	61-77.5	34.7-40.6	0.19-0.23	Erguvanli (1985)
	Halic Tunnel	66.9	37.5	0.19-0.21	Erguvanli (1985)
	Dolmabahce-Baltaliman	118-121	22-31	-	Erguvanli *et al.* (1987)
	Average	**76.1**	**37.2**	**0.21**	

2.5. Characterization of Main Lithological Formation of Greywackes

At location of tower structures, the main lithological unit present is greywacke formation with alternating layers of sandstone, siltstone and claystones with various degrees of weathering and fracturing. As a result, it became essential for the geotechnical engineers to obtain the geotechnical modeling of subsoils, both to a depth of excavation to employ in the design of retaining structures and well below the foundation levels, to be utilized in foundation design and seismic analysis of these structures. Often, soil investigations were required to be performed to a depth of as great as 50+ meters below the ground surface.

Obviously, the extend of weathering and fracturing of greywacke formation controls the mechanical properties and in fact geological observations do well agree with the results of measurements reflecting mechanical properties of the formation. The geotechnical modeling of formation, weathered zones, extend of fracturing and compressibility modulus of formation are usually obtained by means of integrated seismic survey and Menard pressuremeter testings performed within the boreholes at various locations and depths. (Durgunoglu and Yilmaz, 2007). "Geodynamical-seismic" model is driven by integrated shallow seismic surveys below the ground surface. The seismic model is defined by three sets of parameters; geometry of the various subsurface layers, soil-bedrock interface, and the P-wave and S-wave velocities (v_p and v_s) of the layers themselves such as proposed by Yilmaz *et al*. (2006).

A typical P-wave velocity-depth model at a planned tower site located at Zincirlikuyu as an example is presented in Figure 2.11. The near-surface at the comprises three main units starting from the ground level: (1) top soil and/or fill with vp velocities varying between 500-1,000m/s and thickness varying between 3-18m; v_s=200-600m/s (2) heterogeneous layer with velocities varying between 1,000-2,500m/s in most parts of the site and thickness varying between 10-35m; v_s =600-1200m/s (3) homogeneous layer with velocities varying between 2,500-3,500m/s, mostly in the vertical direction, and thickness varying between 10-20m. Below the near-surface layers is the geological bedrock with velocities exceeding 3,500m/s. P-wave velocities generally are 10-20% higher in the NS direction than the velocities in the EW direction, particularly within the third layer and bedrock. Such directional difference in velocities may be attributed to seismic anisotropy caused by fracture surfaces in the EW direction that may be present in the third layer and bedrock. The depth of the near-surface-bedrock interface varies between 20-45m.

Some dikes and faults can also be inferred from the structural interpretation of the P-wave velocity-depth models based on velocity contrast. The variation of v_s with depth obtained at four stations, S4, S3, S2 and S6 is shown in Figure 2.12.

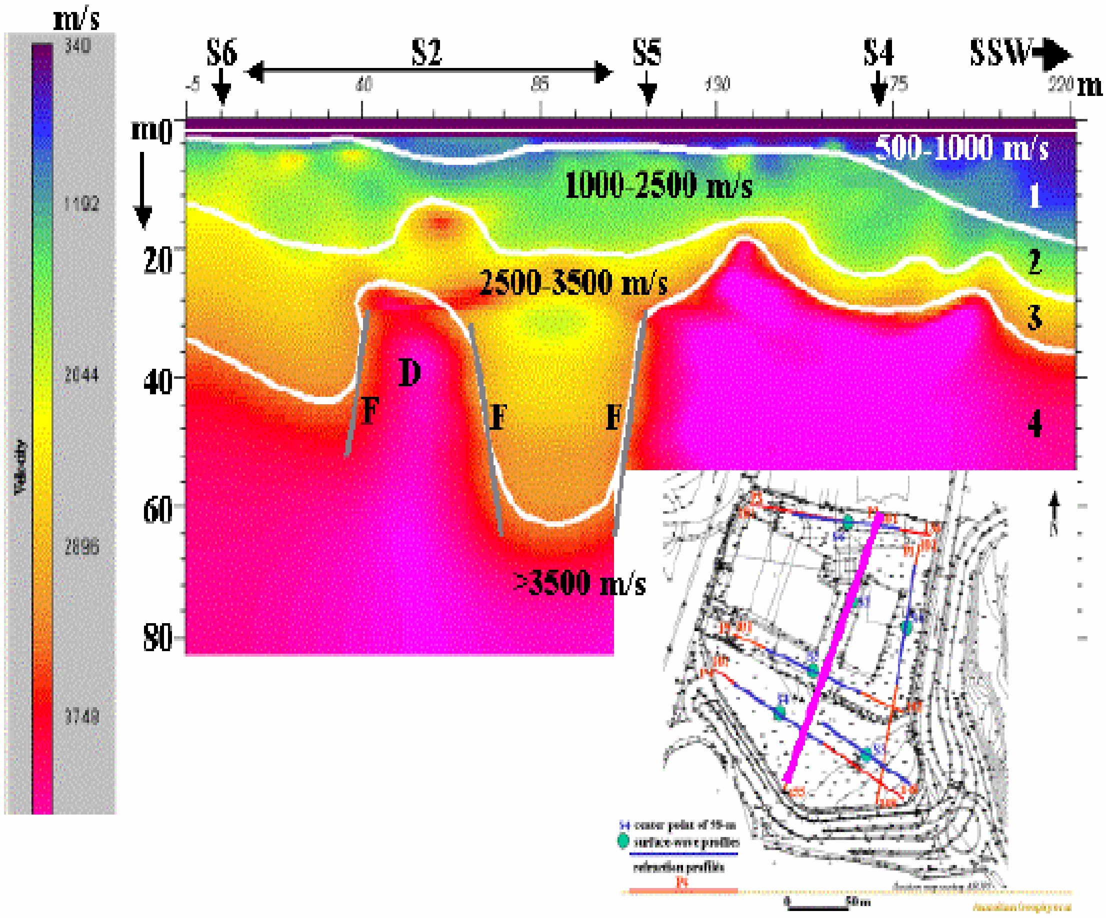


Figure 2.11. An example of structural interpretation of the P-wave velocity-depth model at a tower site (Durgunoglu and Yilmaz, 2007)

It may be seen that v_s values are typically in the range of 200-1200m/s depending on the depth of the greywacke formation.

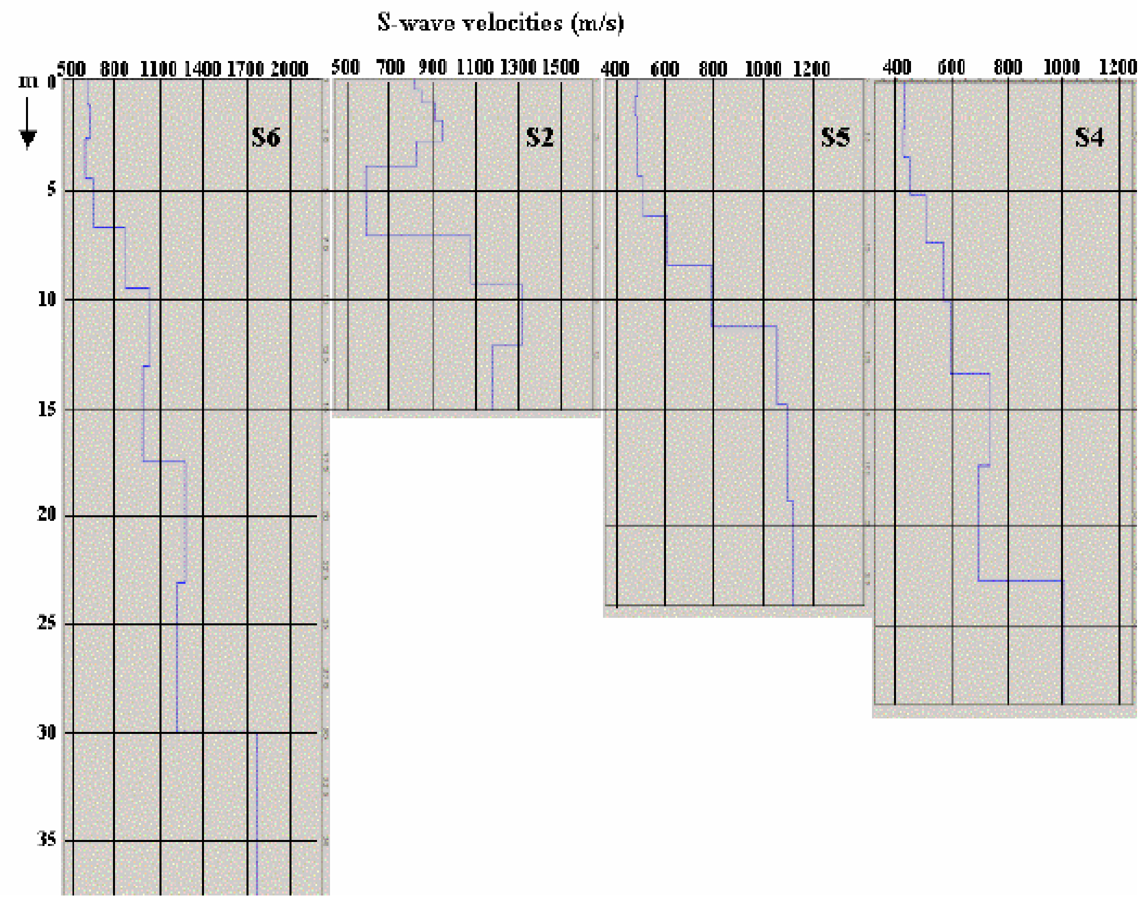


Figure 2.12. An example of the S-wave velocity-depth profiles from a tower site (Durgunoglu and Yilmaz, 2007)

The v_s values for many strong-motion recording sites used in various ground-shaking regression attenuation relationships is poorly established. Usually, the mean shear wave velocity in the upper 30 meters v_s^* m/s is used. However, appropriate v_s^* m/s values for various subjective site descriptions currently are being debated as shown at the Table 2.3.

Table 2.3. Definition of soil formation on v_s^* values (Durgunoglu and Yilmaz, 2007)

Reference	Vs^* (m/s)				
	Weak Soil	Firm Soil	Soft Rock	Firm Rock	Hard Rock
Borcherdt, 1994	150	290	540	1050	1620
Boore et al., 1997	<180	180-360	360-750	>750	-
BSSC, 1998	<180	180-360	360-760	760-1500	>1500
Willes and et al., 2000	-	289	372	724	-
Frankelet et al., 2000	-	-	-	760	-
Campbell and Bozovgnia, 2003	163	301	372	718	-

In general, the greywacke is alternating fractured sandstone, siltstone and claystone formation having some degree of weathering close to surface. They are classified as soft rocks having average shear wave velocities in the range of v_s^*=400-800 m/sec depending on the extent of fracturing for the top 30 meters.

The range of shear modulus values for greywacke formation for very low strains, G_0, 300 to 1200 MPa, determined based on v_s^*, 400 to 800 m/s, values are great. Typical G_0 values, $G_0=\rho v_s^2$ for various soil types as comparison is presented in Table 2.4.

Table 2.4. Typical range of G_0 for various soil types (FHWA, 2002)

SOIL TYPE	**G_0 (MPa)**
Soft Clays	3 – 14
Hard Clays	7 – 35
Silty Sands	30 – 140
Dense Sand and Gravel	70 – 350
Soft Rocks (Durgunoglu and Yilmaz, 2007)	300 – 1200

It is well known that, the strain levels develop as a result of deep excavations are much higher than the levels resulted in geophysical measurements. Further, it is also known that, the modulus value is strongly dependent on the strain level. Increase in strain cause significant decrease in modulus i.e. modulus degradation depending on the nature of the lithological unit. Therefore, the modulus degradation values to be utilized under various strain levels have to be determined.

The modulus degradation values have been estimated for the soils in the past by various authors. However, almost no data exists for the rock conditions. The modulus ratios for the greywacke formation have been extensively studied by Durgunoglu and Yilmaz (2007). The variation reported for three tower sites in Istanbul are presented in Figure 2.13. It may be seen that closer to ground surface 0-30m for pertinent soft rock conditions the ratio is about 30 to 50, on the other hand, for deeper layers, 30+ m, pertinent firm rock conditions the ratio is about 50 to 200.

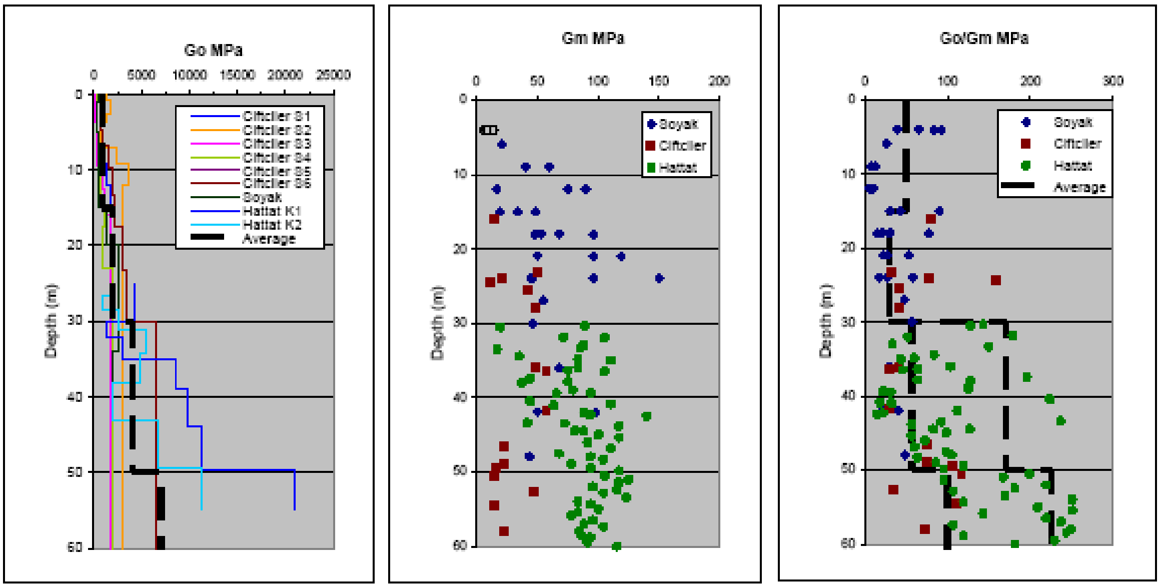


Figure 2.13. The variation of shear modulus with depth of typical greywacke in Istanbul (Durgunoglu and Yilmaz, 2007)

It is seen that the degradation ratios for greywacke formation to obtain G_m to be utilized in prediction of lateral displacement of various retaining structures are large as well. Therefore the modulus value of greywackes of Istanbul to be utilized in numerical models such as finite element computer program, PLAXIS, to predict the displacements do vary in a very big range. Consequently, the displacements of retaining structures based on selected modulus values are only indicative.

3. RETAINING SYSTEMS USED IN DEEP EXCAVATIONS

3.1. Introduction

During the last decade, the city of Istanbul has performed significant growth in economy. Becoming the biggest metropolitan city of the region, the need for high-rise residential and office buildings and shopping malls with multiple basement levels increased noticeably considering the raised value of the land, which became a major part of the cost in construction of buildings. In order to build great number of basement levels, especially to obtain parking space and room for shopping and entertainment facilities, deep excavations and construction of retaining structures became compulsory.

The greywackes, which constitute the basement rock formation of the city of Istanbul, are classified as critical as far as their slope stability is concerned. They contain very complex discontinuity surfaces and sliding and rock falls along these discontinuities are very common at unsupported deep excavations made in this rock formation. The greywacke beds, which possess rather high shear strength and uniaxial compressive strength, are very poor from the slope stability viewpoint due to relatively low residual shear parameter they show when exposed to water and atmosphere. As a result, the deep excavations to be utilized in Istanbul greywackes should be supported by various means (Saglamer, 1986).

3.2. Determining the Type of Retaining Systems

Throughout the selection of the system and the design of the supporting structures in retaining of the deep excavations, a comprehensive study should be performed focusing on the following matters (Saglamer, 1986):

- Subsoil profile and engineering characteristics of soil and/or rock formation
- Duration of the excavation will be kept open, i.e. service life
- Groundwater profile

- Structural characteristics such as depth and size of the foundation of the neighboring structures and the location of infrastructural utilities.
- Limitations on the deformation of the retaining structures and estimation of the lateral displacements.

Since the city of Istanbul is in very seismically active region, seismicity is one of the principal concerns while selecting appropriate retaining system whether it is flexible or rigid. Other factors affecting the predetermination of the retaining system are time schedule, budget constraints and the available techniques that can be readily employed by the geotechnical contractor. The retaining systems widely used in the deep excavations in the city of Istanbul are tied back cast in-situ piles, tied-back micro piles, tied back cast in-situ reinforced concrete walls (manual caisson, segmented or diaphragm walls), strutted or shored support walls, soil nailing or combinations of these.

3.3. Tied Back Cast In-situ Bored Piles

Tied back cast in-situ bored piles as a retaining system is very common type of supporting deep excavations in Istanbul. Bored piles are constructed by excavating the ground to obtain a shaft (hole) and then placing the reinforcement and concrete (Candogan, 2007). Typical diameters of the piles are in the range of 45 cm to 120 cm.. Jet grout columns could be utilized behind or between the piles to prevent ingress of water into the excavation in the presence of high groundwater. Several rows of pre-stressed anchors together with horizontal reinforcing beam depending on the stability of the subsoil formation, excavation height, engineering characteristics of the soil/rock, etc. are utilized as tie backs.

Some of the advantages of the tied back cast in-situ bored piles are suitability for installation under groundwater table, installation to great depths and with large diameters enabling the design to large lateral and vertical loads, installation in any type of ground conditions as drilling tools can penetrate, easy adjustment of length according to the encountered ground conditions. Main drawbacks of the system are relative rigidity under seismic condition, high installation time and cost. Typical applications of such system are presented in Figure 3.1 through Figure 3.4.

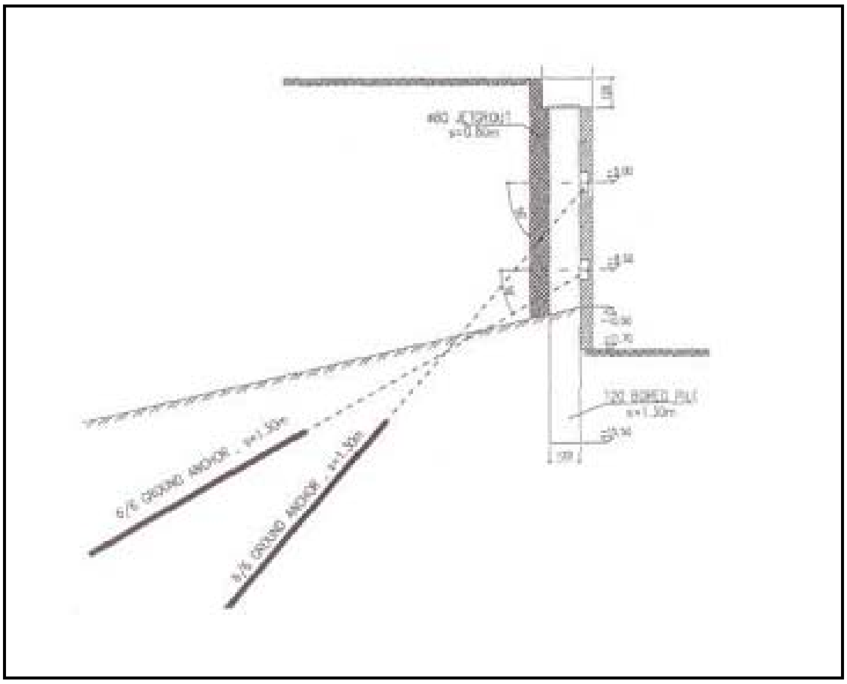


Figure 3.1. An application of a tied back cast in-situ bored pile with jet grout columns behind for a hotel building in Istanbul (Saglamer *et al*., 2007)

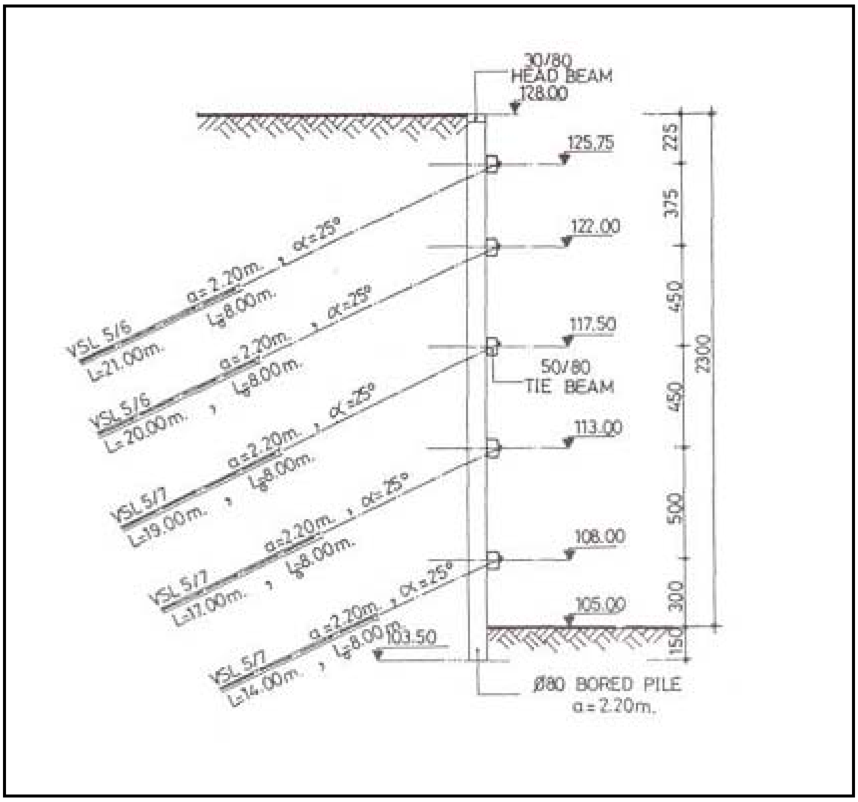


Figure 3.2. Tied back cast in-situ bored pile application on the deep excavation of Sabanci Towers (Candogan and Saglamer, 1991)

Figure 3.3. Tied back cast in-situ bored pile application on the deep excavation of Mashattan Residence (Zetas, 2006)

Figure 3.4. Tied back cast in-situ bored pile application on the another section from the deep excavation of Mashattan Residence (Zetas, 2006)

3.4. Tied Back Cast In-situ Micro Piles

Similar to the bored piles, micro piles with pre-stressed anchors are used to support deep excavations in a relatively shallow depths and better subsoil conditions. Having rather small diameters, micro piles are preferred to bored piles in stiffer and stronger subsoil conditions such as in greywackes for limited depth of excavations. Another benefit using micro piles is installation equipment of the micro piles are lighter hydraulic drilling rigs, while bored piles are installed with large mechanical or hydraulic rigs. Typical diameters of the micro piles are in between 10 cm and 35 cm. Again pre-stresses anchors together with horizontal reinforced beams are used to support lateral loads as in the bored piles. In deep excavations micro piles are utilized at various stages by providing horizontal berms in between. Typical applications of such system are given in Figures 3.5 and 3.6. thorough Figure 3.7.

Some benefits of the tied back cast in-situ micro piles are; relatively easy installation in limited working space, capability to install near buildings and roads rather than bored piles, installation in nearly all types of ground condition, easy mobilization and access. Main disadvantage of the system is having low flexural strength due to small cross-section.

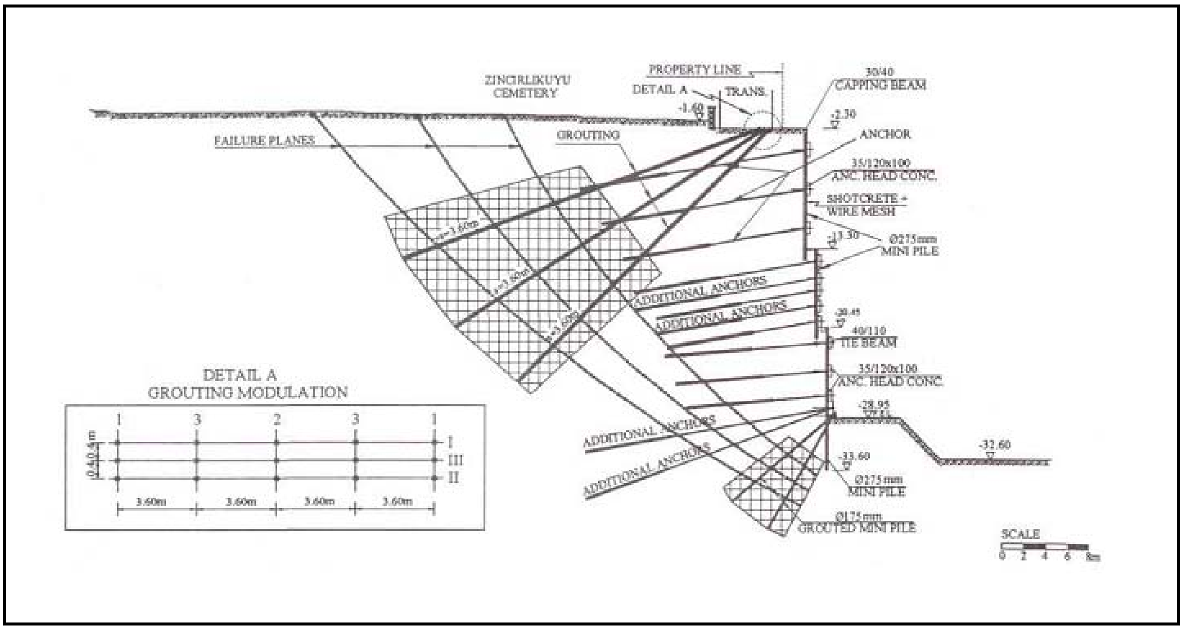


Figure 3.5. Micro piles with pre-stressed anchors and jet grouting behind applied for the deep excavation of Metrocity Center (Candogan and Duzceer, 2001)

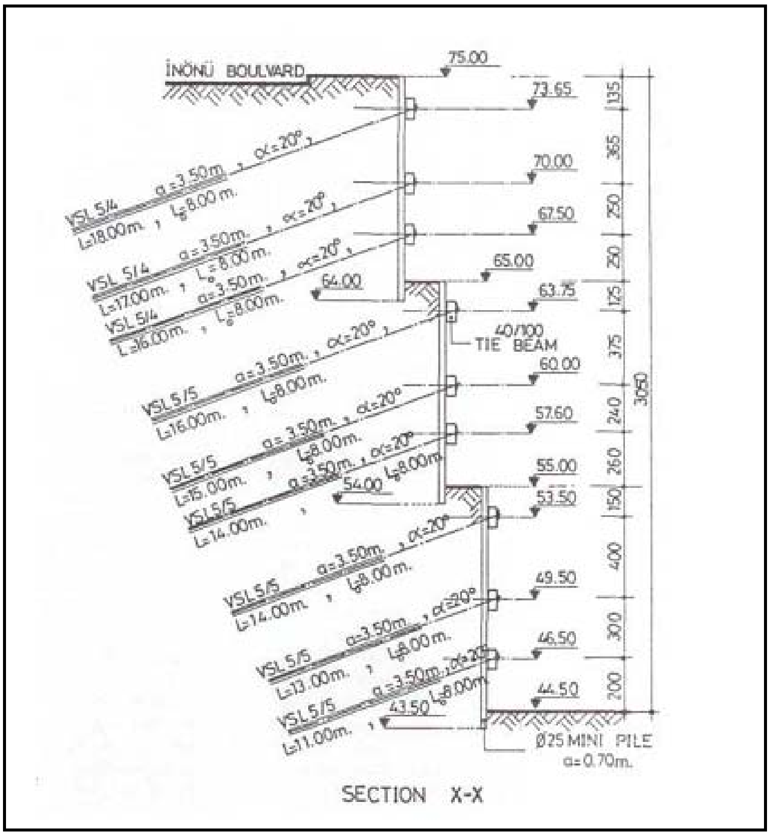


Figure 3.6. Tied back cast in-situ micro pile application on the deep excavation of Park Hotel (Candogan and Saglamer, 1991)

Figure 3.7. Micro piles with pre-stressed anchors applied for the excavation of a building in Istanbul (Zetas, 2006)

3.5. Tied Back Cast In-situ Reinforced Concrete Walls

Another type of retaining systems used in deep excavations in the city of Istanbul are tied back cast in-situ reinforced concrete walls. In this type of retaining system reinforced concrete walls are used instead of bored piles or micro piles. Reinforced concrete walls constructed using bentonite slurry technique is preferred when groundwater table is comparatively high and leakage of water is needed to be limited. The types of reinforced concrete walls are manual caisson walls, segmented reinforced concrete walls, where there is no groundwater table and the diaphragm walls under groundwater table. Manual caisson walls are preferred when there is lack of space for construction equipment.

Main advantages of the reinforced concrete walls are; manual installation can be made by excavating with hand tools when the access of the machinery to site is restricted. Typical applications of the system are given in Figures 3.8 thorough 3.10.

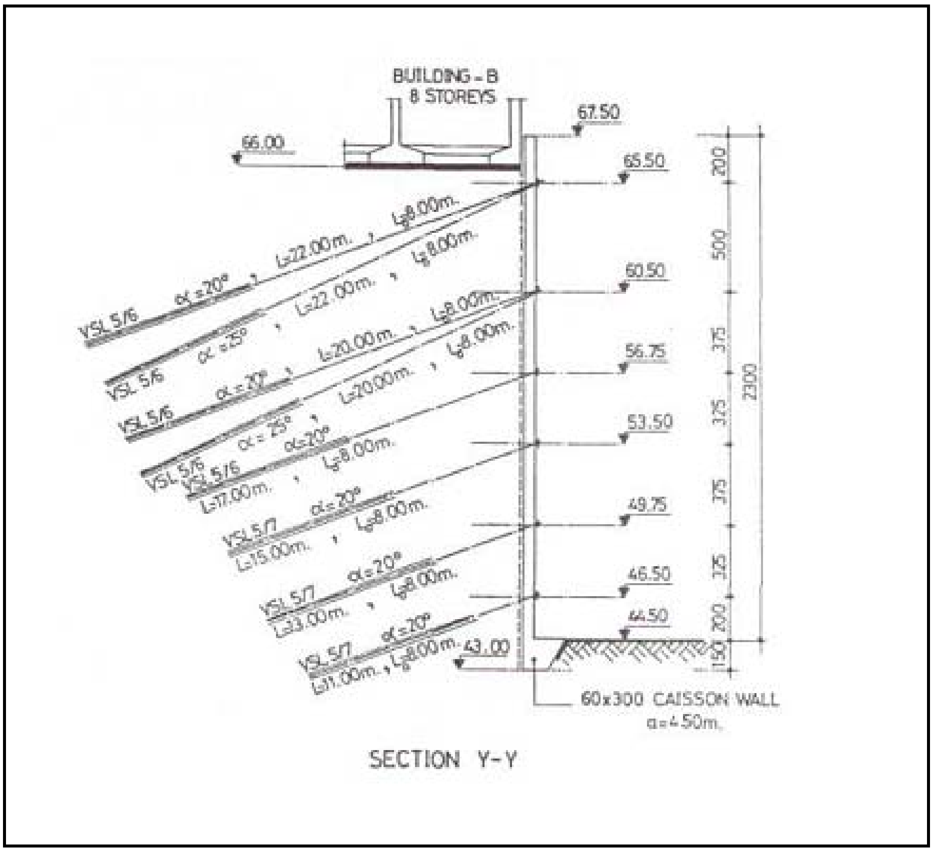


Figure 3.8. Cast in-situ manually dug caisson wall with pre-stressed anchor application for the deep excavation of Park Hotel (Candogan and Saglamer, 1991)

Figure 3.9. Segmented reinforced concrete wall with pre-stressed anchor application for the deep excavation of a hotel building in Istanbul (Zetas, 2006)

Figure 3.10. Cast in-situ manually dug caisson wall with pre-stressed anchor application for the excavation of a hospital building (Zetas, 2006)

3.6. Strutted or Shored Support Walls

Struts or shores (rakers) are used in deep excavations to support retaining systems laterally whether piling or reinforced concrete walls are to be constructed when the width of excavation is limited. Struts are horizontal compression elements put two opposed walls or slope while shores are diagonal elements placed from a fixed point on the ground to the retaining wall. Struts are used in deep excavations where one dimension is narrow or used diagonally on the corners of the excavation. Struts or shores can be used as supplementary systems to pre-stressed anchored pile or wall system.

Struts or shores, which can be used multiple times, are installed especially where pre-stressed anchorages are not feasible for subsoil conditions or there is no enough space to install anchors. Struts are not practical for spans longer than 20 m and shores are not feasible for walls higher than 10 m. Two pictures from typical strutted support wall system are given in Figure 3.11 and Figure 3.12

Figure 3.11. Struts used together with pre-stressed anchors for the excavation of a hotel building (Zetas, 2006)

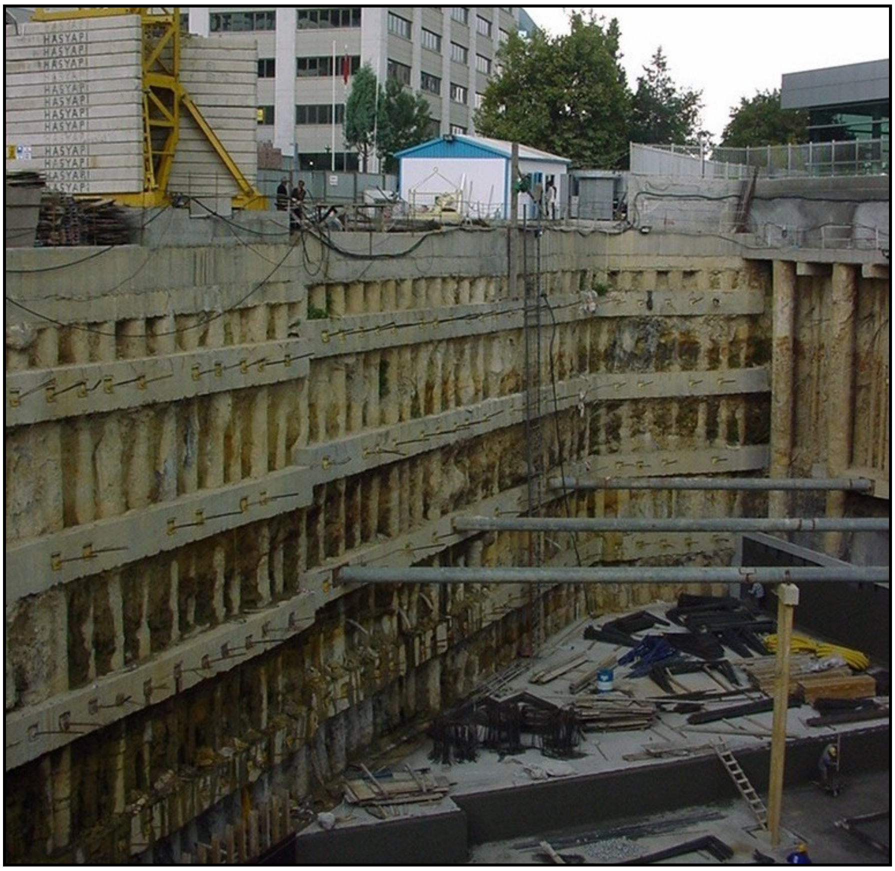


Figure 3.12. Struts used instead of pre-stressed anchors for the one face of a deep excavation in Istanbul (Zetas, 2006)

3.7. Soil Nailing

Soil nailing is used as both temporary and permanent retaining structures for deep excavations. In a typical soil nail designed wall, the soil is excavated in lifts and then near horizontal inclusions are placed into the soil at regular intervals to increase the shear strength of the soil and make it as a block (Macnab, 2002). Soil nailing is a flexible earth retaining system, since in order to mobilize the system lateral displacements are needed to occur. As soil nailing is used as a separate retaining system, various combinations with another systems in the same cross-section can be utilized. The most common type of combination is their employment with pre-stressed ground anchors to prevent excessive lateral displacements. Figure 3.13. and Figure 3.14 present soil nailing used as separate retaining system, while Figure 3.15 shows soil nailing system used with pre-stressed ground anchors to prevent lateral displacements.

Figure 3.13. Soil nailing used for all faces of the deep excavation of Kanyon Complex (Zetas, 2006)

Figure 3.14. Separate soil nailing application for a deep excavation in Istanbul (Zetas, 2006)

Figure 3.15. Additional one row of pre-stressed anchor at the top to prevent displacement soil nailing in BJK Fulya Complex (Zetas, 2006)

Soil nailing could be used as a reinforcement of old existing retaining walls. An example is presented in Figure 3.16.

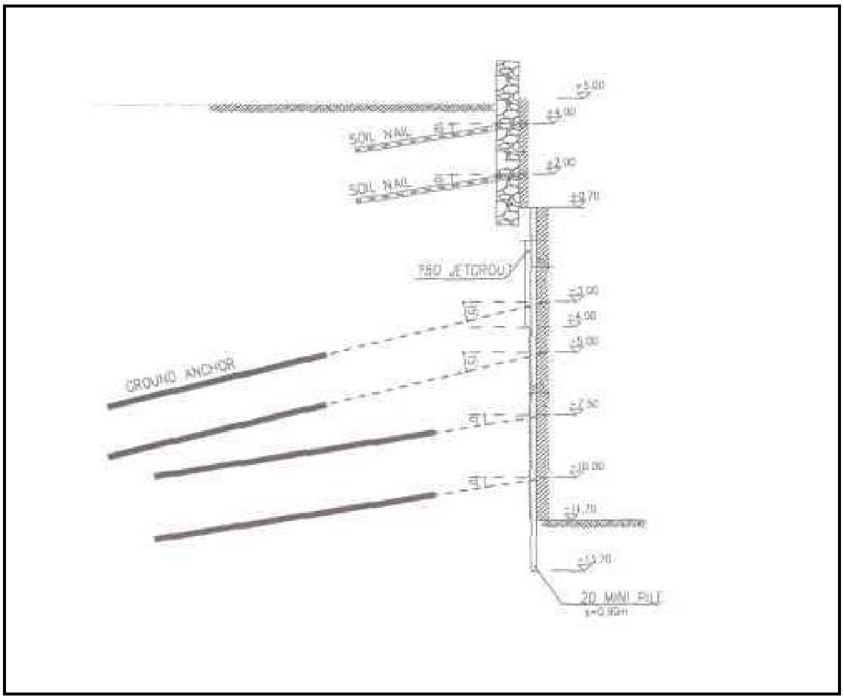


Figure 3.16. An application of soil nailing as a reinforcement of an old wall in Istanbul (Saglamer *et al*., 2007)

The advantages of the system mainly are; economy, construction speed, easy access of equipment used. The flexibility offers great advantage especially at seismically active areas similar to Istanbul. The two main drawback of the system are that they cannot be used on soils which could not stand unsupported even in a short period of time and when strict limited lateral displacements are required near existing structures. Soil nailing is also used in combination with other systems such as presented in Figure 3.17 through 3.20.

Figure 3.17. Combination of soil nailing with tied-back bored pile application for the deep excavation of Mashattan Residence (Zetas, 2006)

Figure 3.18. Combination of soil nailing with tied-back bored pile application for a deep excavation in Istanbul (Zetas, 2006)

Figure 3.19. Combination of soil nailing with tied-back bored pile application for the deep excavation of BJK Fulya Complex (Zetas, 2006)

Figure 3.20. Combination of soil nailing with tied-back bored pile application for the deep excavation of Istinye Park Project (Zetas, 2006)

Considering the seismicity of the city of Istanbul, flexible type of retaining systems should be used especially in deep excavations. Soil nailing, together with offering economical solutions and the speed of construction, provides flexibility needed and feasible in the frequently encountered subsoil, greywacke. Therefore soil nailing as a flexible retaining system deeply emphasized and thoroughly examined in this study.

4. SOIL NAILING AS A FLEXIBLE RETAINING SYSTEM

4.1. Introduction

Soil nailing is a technique where either natural soil or existing fill material is reinforced by the insertion of slender tension-carrying elements called soil nails. Soil nails are made of metallic or polymeric material and may be installed into pre-drilled hole then grouted, drilled and grouted simultaneously, or inserted using a displacement technique. They are usually installed at a slight downward inclination to the horizontal. A hard, flexible or soft facing may be used at the surface of the slope. While the terms “soil nail wall” and “soil nailing” are broadly applied to soil systems, the technique is also applicable to excavations in soil-like materials as soft rock or weathered rock, such as greywackes.

The basic idea of soil nailing consists of the reinforcing the ground by closely spaced passive, that means not pre-stressed ground anchors, inclusions to built up a coherent gravity structure by increasing shear strength of in-situ soil and restraining its movement as some other type of soil reinforcement techniques. Soil nailing is typically used to stabilize existing slopes or excavations where top-to-bottom construction is advantageous compared to other retaining wall systems. For certain conditions, soil nailing offers a viable alternative from the viewpoint of technical feasibility, construction costs, and construction duration when compared to ground anchor walls, which is another system utilized top to bottom retaining wall construction.

The technique gained a reputation as being flexible and cost-effective and it has been used for applications ranging from the stabilization of new and existing cutting and embankment slopes to the remediation of distressed retaining walls.

4.2. Development of Soil Nailing

Soil nailing originated partly from techniques developed for rock-bolting and multi-anchorage systems, and partly from reinforced soil techniques (Clouterre, 1991).

Retaining walls using anchored bars grouted into the bedrock and faced with reinforced concrete are constructed in schists for stabilization of slopes for overflow spillway at Notre Dame de Commier Dam in France in 1961 (Bonazzi and Colombet, 1984). A cross-section is given in Figure 4.1.

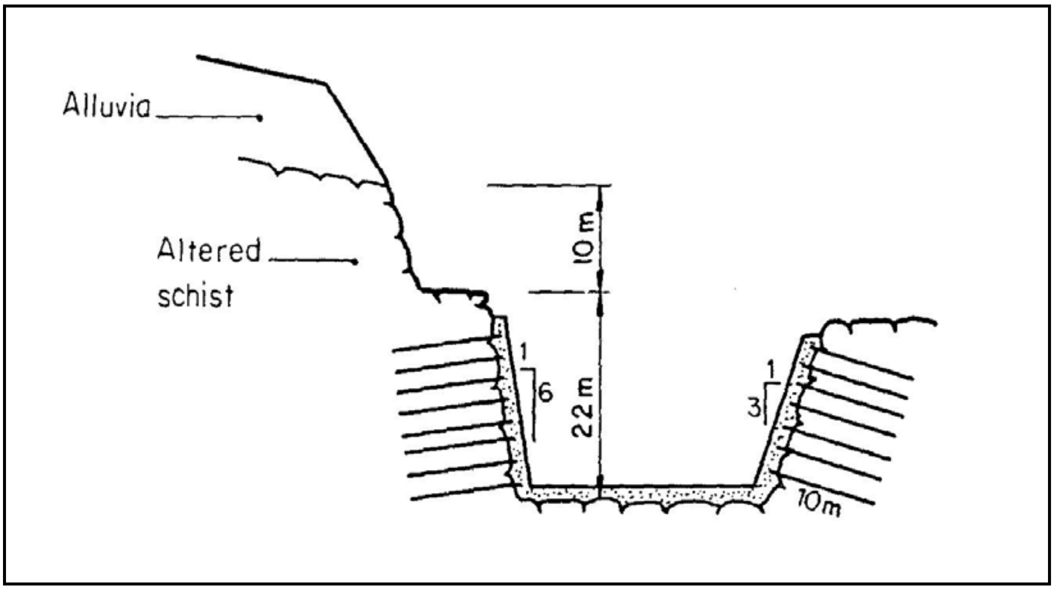


Figure 4.1. Overflow spillway of Notre Dame de Commier Dam, France, 1961 (Bonazzi and Colombet, 1984)

A similar technique is the New Austrian Tunneling Method (NATM), which was developed by Rabcewicz in 1964 for supporting underground galleries and tunnels. The technique involves reinforcing the ground using anchored bars (bolts) all around the gallery immediately after the face has been excavated, thus allowing a appreciable reduction in the amount of final lining required. The nails used are usually between 3 and 6 meters long. A typical cross-section is shown in Figure 4.2.

From experience with NATM in France, it was realized that similar techniques could be used for the temporary support of soft rock and soil slopes. In 1972 the first reported soil nailed wall was built at Versailles in France for the widening of a railway cutting (Rabejac and Toudic, 1974). This was an 18-m high temporary wall being built using a high density of short nails 4 m or 6 m long anchored with grout in dense, uniformly graded Fontainebleau sand, as shown in Figure 4.3.

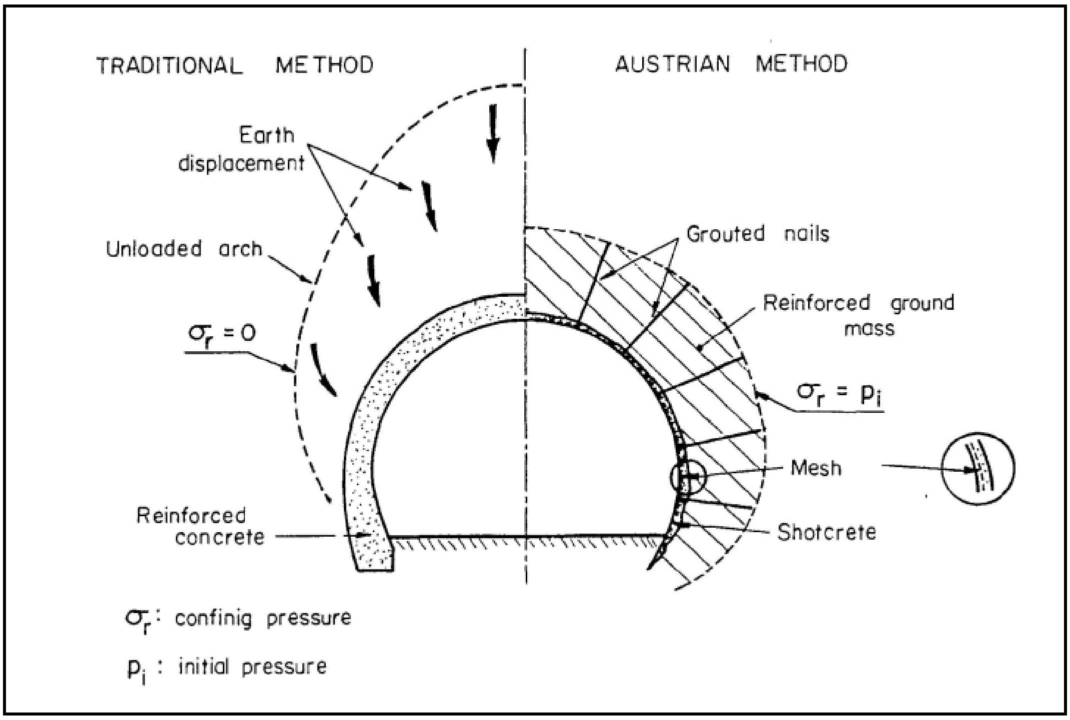


Figure 4.2. Comparison of the New Austrian Tunnelling Method and a traditional method of supporting an underground gallery (Bruce and Jewell, 1986)

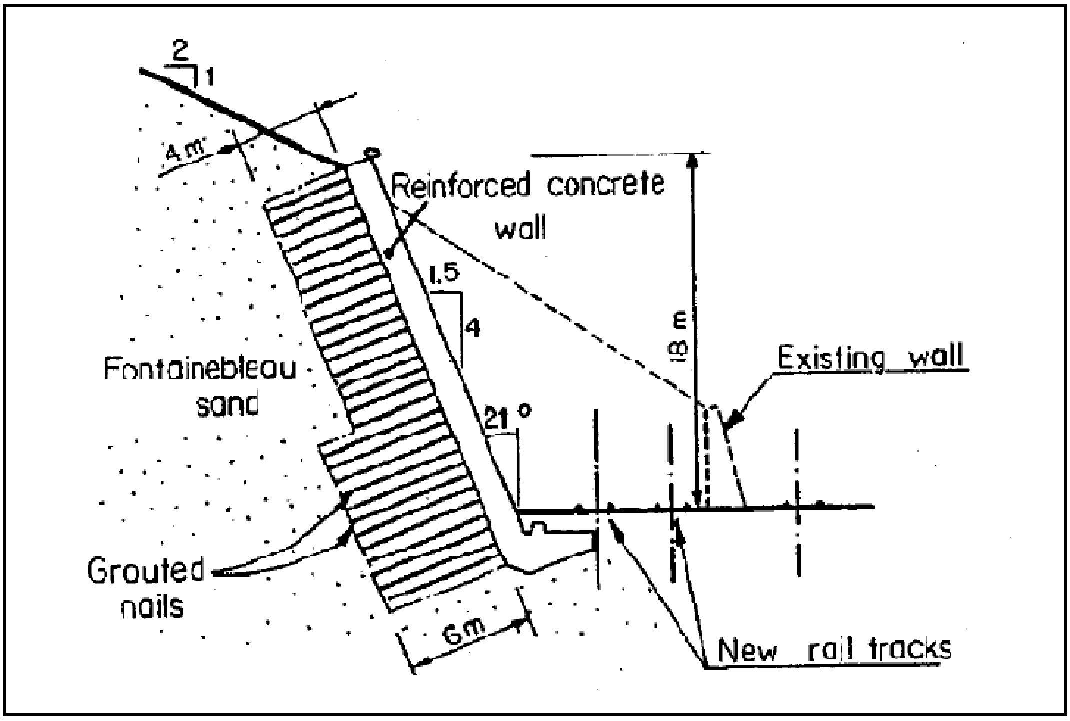


Figure 4.3. Cross-section of the first soil nailed wall at Versailles, France, 1972/73 (Rabejac and Toudic, 1974)

The first major research program on soil nail walls was carried out in Germany from 1975 through 1981 by the University of Karlsruhe (Gässler and Gudehus, 1981; Schlosser and Unterreiner, 1991). In France, the Clouterre research program, involving private and public participants, was initiated in 1986. The results of this research were published in 1991 and form the basis of the soil nailing design approach used in France and adopted in other countries.

During the last 20 years very rapid development has taken place in the techniques of reinforcing in-situ soils by using nails. Many very deep cuts are realized using soil nailing in Turkey as well (Durgunoglu *et al*., 2003b; Durgunoglu *et al*., 1997). Soil nailed walls are technically feasible and, in many cases, a cost-effective alternative to conventional retaining walls used in excavations in temporary and permanent applications. Its economy is just one of the major attractions of the method. Additionally construction flexibility, its ability to make use of small construction equipment which is particularly suited for use in urban environments and its overall adaptability for special applications have led soil nailing gain a distinct identity in both, the theory and practice in recent years.

4.3. Components of a Soil Nail Wall

Although components of a soil nail wall are different for different soil nail installation techniques, the most common practice for soil nailing consists of drilled soil nails, in which a steel bar is placed in a pre-drilled hole and then grouted. A typical cross-section of a “drilled and grouted” soil nail wall is presented in Figure 4.4. The main components of a soil nail wall are soil nails, drainage system and facing. Typical soil nail is composed of steel reinforcing bars, grout, nail head, hex nut, washer and bearing plate, centralizers and corrosion protection elements for permanent applications. Figure 4.5. shows detailed cross-section for a typical soil nail.

4.3.1. Nail Bars

The solid steel reinforcing bars are the main component of the soil nail wall system. These elements are placed in pre-drilled drill holes and grouted in place. Tensile stress is

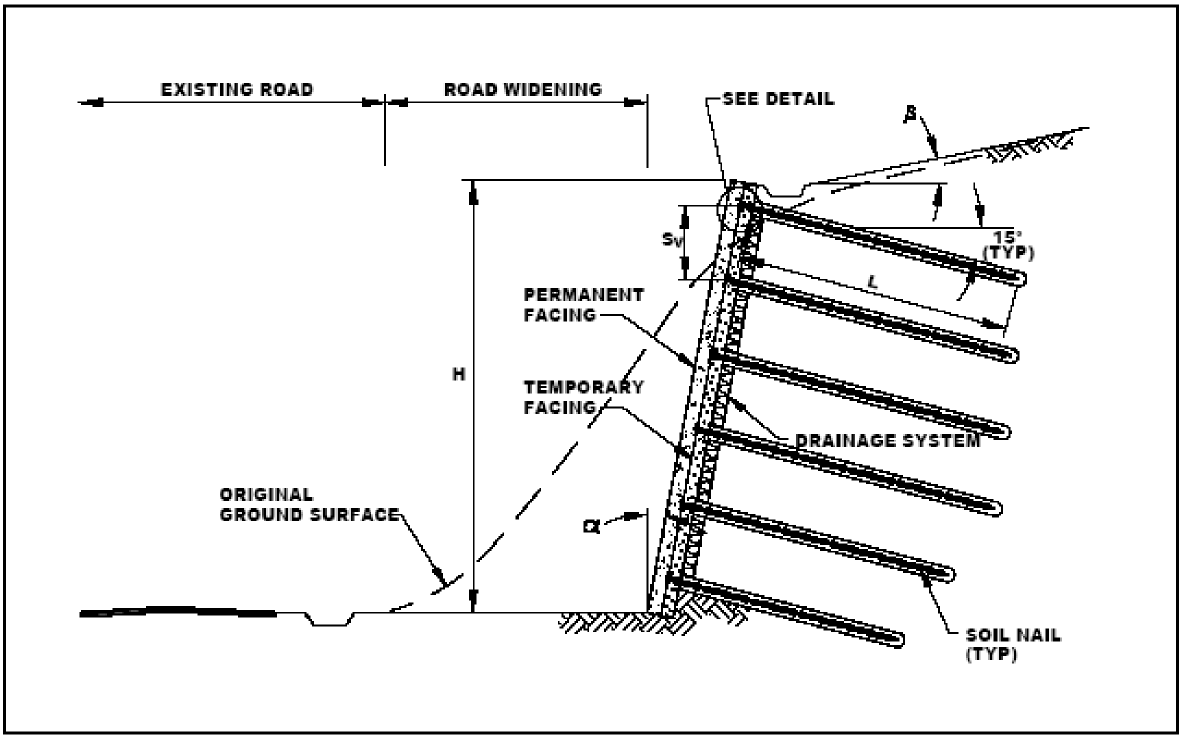


Figure 4.4. Typical cross-section of a soil nail wall (FHWA, 2003)

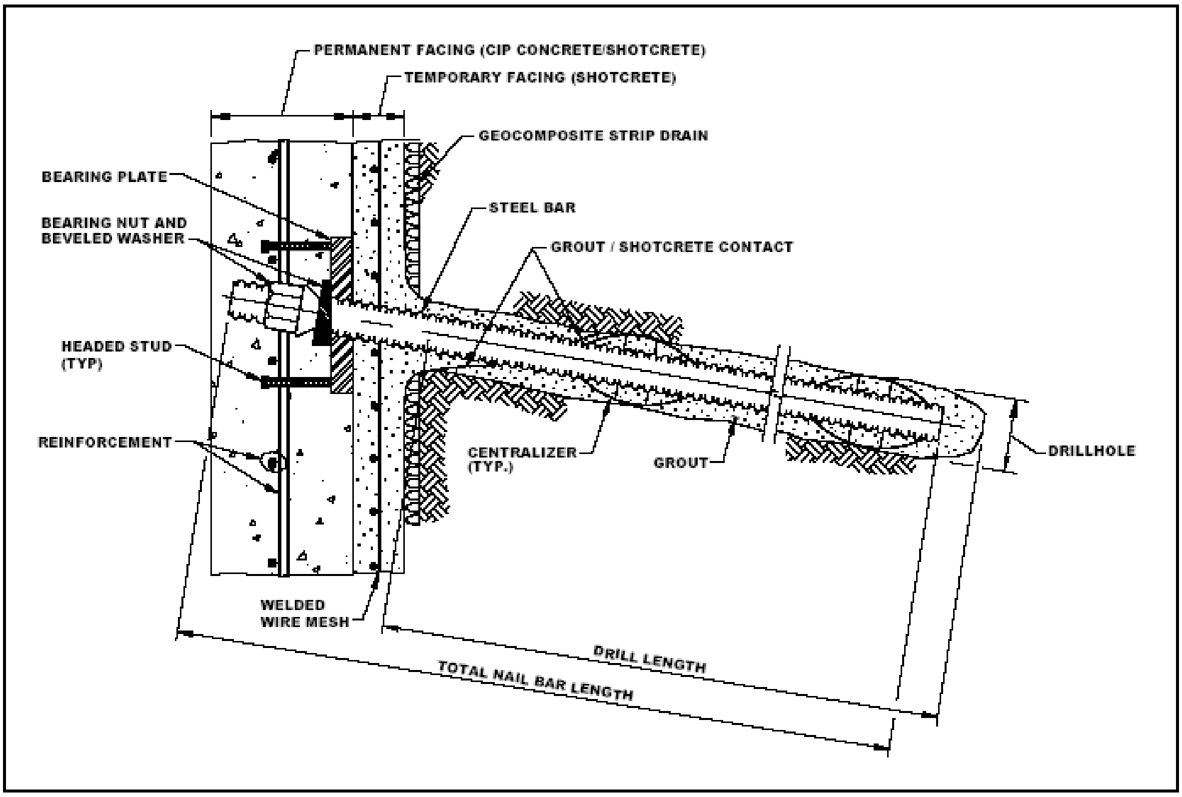


Figure 4.5. A detailed cross-section of a soil nail (Porterfield *et al*., 1994)

applied passively to the nails in response to the deformation of the retained materials during subsequent excavation activities. Bars having lower tensile strengths are preferred because they are more ductile, less susceptible to corrosion, and available (FHWA, 2003).

4.3.2. Nail Head

The nail head is the threaded end of the soil nail that protrudes from the wall facing. The nail head comprises two main components, the bearing-plate, hex nut, and washers; and the headed-stud, which are shown in Figure 4.5. The purpose of the bearing plate is to distribute the force at the nail end to the temporary shotcrete facing and the ground behind the facing. Beveled washers are then placed and the nail bar is secured with a hex nut or with a spherical seat nut (Porterfield *et al*., 1994).

4.3.3. Grout

Grout is placed in the pre-drilled borehole after the nail is placed. The grout serves the primary function of transferring stress from the ground to the nail. The grout also provides a level of corrosion protection to the soil nail. Grout for soil nails is commonly a neat cement grout, which fills the annular space between the nail bar and the surrounding ground (FHWA, 2003). Occasionally, the stiff consistency of the grout may cause difficulties with the installation of the centralizers. In this case, the grout itself may provide sufficient support to centralize the nail bar within the drill hole. The characteristics of the grout have a strong influence on the ultimate bond strength at grout-ground interface. Grout is pumped shortly after the nail bar is placed in the drillhole to reduce the potential for hole squeezing or caving. Grout injection must be conducted smoothly and continuously in such away that the space between the drillhole and the nail bar is filled completely, with no voids or gaps (Porterfield *et al*., 1994).

4.3.4. Centralizers

Centralizers are devices made of PVC or other synthetic materials that are installed at various locations along the length of each nail bar to ensure that a minimum thickness of grout completely covers the nail bar. They are installed at regular intervals, typically not exceeding 2.5 m, along the length of the nail and at a distance of about 0.5 m from each end of the nail (FHWA, 2003).

4.3.5. Corrosion Protection Elements

In addition to the cement grout, which provides both physical and chemical protection to the nail bars, other devices are typically used to provide additional corrosion protection for permanent soil nailing applications. Protective sheathings made of corrugated synthetic material as HDPE (High Density Polyethylene) or PVC tube surrounding the nail bar are usually used to provide additional corrosion protection. The internal annulus between the sheathing and the nail bar is pre-filled with grout. This system is commonly referred to as corrosion protection by encapsulation (FHWA, 2003).

4.3.6. Wall Facing

The major role of the facing is to stabilize the surface of the ground between the soil nails. It also provides erosion protection. Nails are connected at the excavation surface or slope face to a facing system, which most commonly consists of a first-stage, temporary facing of shotcrete during construction, which typically reinforced with wire-mesh. Purpose of the permanent facing is to provide connection among nails, a more resistant erosion protection, and an aesthetic finish. There are three commonly used facing types as soft facings, flexible structural facings and hard structural facings (FHWA, 2003).

4.3.7. Drainage System

To prevent water pressure from developing behind the wall facing, vertical geocomposite strip drains are usually installed between the temporary facing and the excavation. The drainage system also includes a footing drain and weep holes to convey collected drainage water away from the wall face. In some cases, conventional horizontal drains are also installed (FHWA, 1998).

4.4. Construction Sequence of a Soil Nail Wall

The typical sequence to construct a soil nail wall using solid steel nail bars is described below and shown schematically in Figure 4.6 (FHWA, 2003).

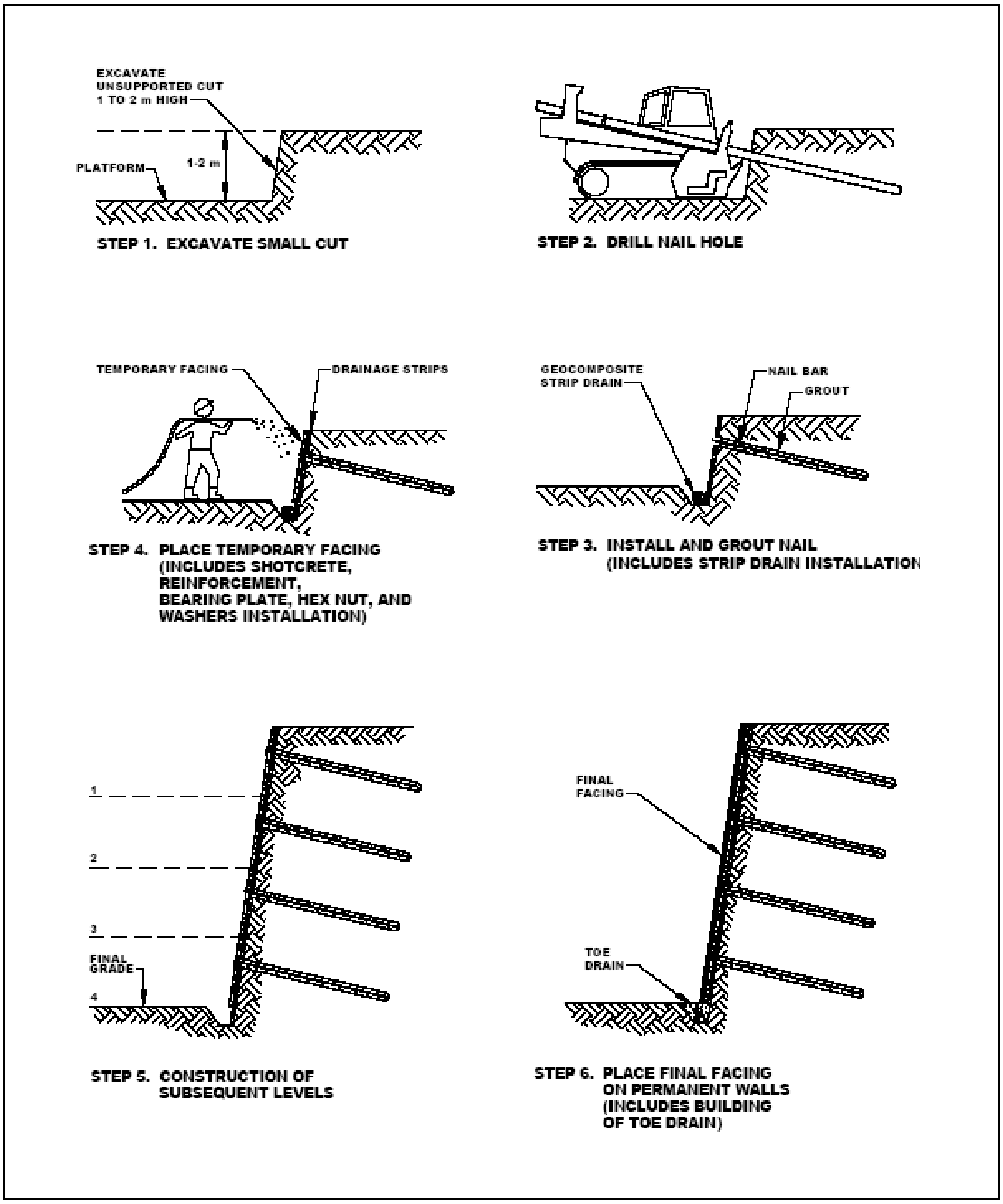


Figure 4.6. A typical construction sequence for a steep soil nail wall (FHWA, 2003)

Step 1 is excavating the initial cut. Initial excavation is carried out to a depth for which the face of the excavation has the ability to remain unsupported for a short period of time, typically on the order of 24 to 48 hours. The depth of the excavation lift is 1.5 m. The width of the excavated platform or bench must be sufficient to provide access to the installation equipment.

Step 2 is drilling nail holes. Drillholes are drilled to a specified length , diameter, inclination, and horizontal spacing from this excavated platform. Drilling method include both uncased methods for more competent materials, rotary or rotary percussive methods using air flush and dry auger methods, and cased methods for less stable ground, single tube and duplex rotary methods with air or water flush, and hollow stem auger methods.

Step 3 is nail installation and grouting. Nail bars are placed in the pre-drilled hole. Centralizers are placed around the nails prior to insertion to help maintain alignment within the hole and allow sufficient protective grout coverage over the nail bar. A grout pipe, tremie, is also inserted in the drillhole at this time. When corrosion protection requirements are high, corrugated plastic sheathing can also be used to provide an additional level of corrosion protection. The drillhole is then filled with cement grout through the tremie pipe. The grout is commonly placed under gravity or low pressure. Geocomposite drainage strips are installed on the excavation face approximately midway between each set of adjacent nails. The drainage strips are then unrolled to the next wall lift (FHWA, 2003).

Step 4 is construction of temporary shotcrete facing. A temporary facing system is then constructed to support the open-cut soil section before the next lift of soil is excavated. The most typical temporary facing consists of a lightly reinforced shotcrete layer commonly 100 mm thick. The reinforcement typically consists of welded wire mesh, which is placed at approximately the middle of the facing thickness. Before proceeding with subsequent excavation lifts, the shotcrete must have cured for at least 72 hours or have attained at least the specified 3-day compressive strength which is typically 10.5 MPa (FHWA, 2003).

Step 5 is construction of subsequent levels. Steps 1 through 4 are repeated for the remaining excavation lifts. At each excavation lift, the vertical drainage strip is unrolled downward to the subsequent lift. A new panel of welded wire mesh is then placed overlapping at least one full mesh cell. The temporary shotcrete is continued with a cold joint with the previous shotcrete lift. At the bottom of the excavation, the drainage strip is tied to a collecting toe drain (FHWA, 2003).

Step 6 is construction of a final, permanent facing. After the bottom of the excavation is reached and nails are installed, a final facing may be constructed. Final facing may consist of cast-in-place reinforced concrete, reinforced shotcrete, or prefabricated panels.

4.5. Applications of Soil Nailing

Soil nail walls are particularly well suited to excavation applications for ground conditions that require vertical or near-vertical cuts. They have been used successfully in stabilization and reconstruction of existing retaining structures and stabilization of unstable existing slopes. Soil nail walls have also been shown to be particularly well suited for temporary and permanent excavations in urban environments (Durgunoglu *et al.*, 2007b).

4.5.1. Stabilization of Existing Retaining Walls

Soil nailing has proved a useful technique for stabilizing existing retaining walls. The soil nails are installed directly through the retaining structure. In these applications, which represent a departure from the original concept of soil nailing, the ground deformations required to mobilize the reinforcing resistance are not derived from removal of lateral support during excavation but from ongoing movements associated with the distressed structure (Phear *et al.*, 2005). Relevant applications include masonry or reinforced concrete retaining walls that exhibit structural deterioration or excessive deformations, often related to loose or weak backfill, or poor foundation performance; and mechanically stabilized earth walls that have deteriorated because of reinforcement corrosion or poor quality backfill. Figure 4.7 shows a sketch of an old masonary gravity wall stabilized by means of soil nailing in Germany which is mention by Schwing in 1990.

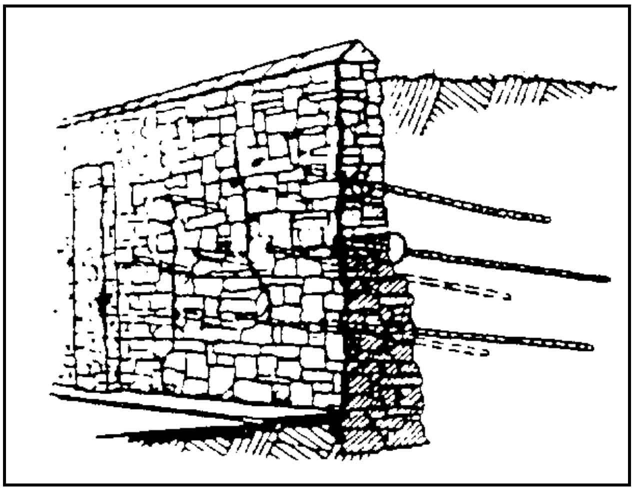


Figure 4.7. Stabilization of old masonry gravity walls using soil nailing (Schwing, 1990)

4.5.2. Stabilization of Existing Slopes

Soil nailing can be used to stabilize natural slopes, railway embankments, highway embankments and cuts, coastal and quarried cliffs and embankment dams. Soil nailing can be a good solution where access is difficult, because drilling equipment mounted on long-reach excavators can sit at the toe of the embankment slope and temporary works are minimal because no scaffolding or access platforms are needed. An example at Dolywern, north Wales, in 1986 a 10 m-high slope was stabilized using seven rows of soil nails, as shown in Figure 4.8 (Barley, 1992).

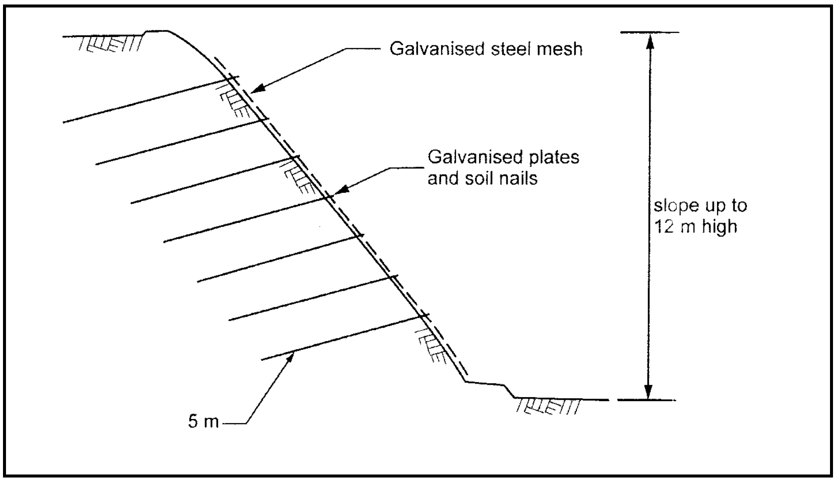


Figure 4.8. A natural slope in Wales, stabilized with soil nails (Barley, 1992)

4.5.3. Construction of New Steep Slopes

Soil nailing can be used as a retaining structures in widening of highways and railway cuttings. Figure 4.9 shows examples of the use of soil nail walls in temporary and permanent cut applications in highway widening (FHWA, 2003).

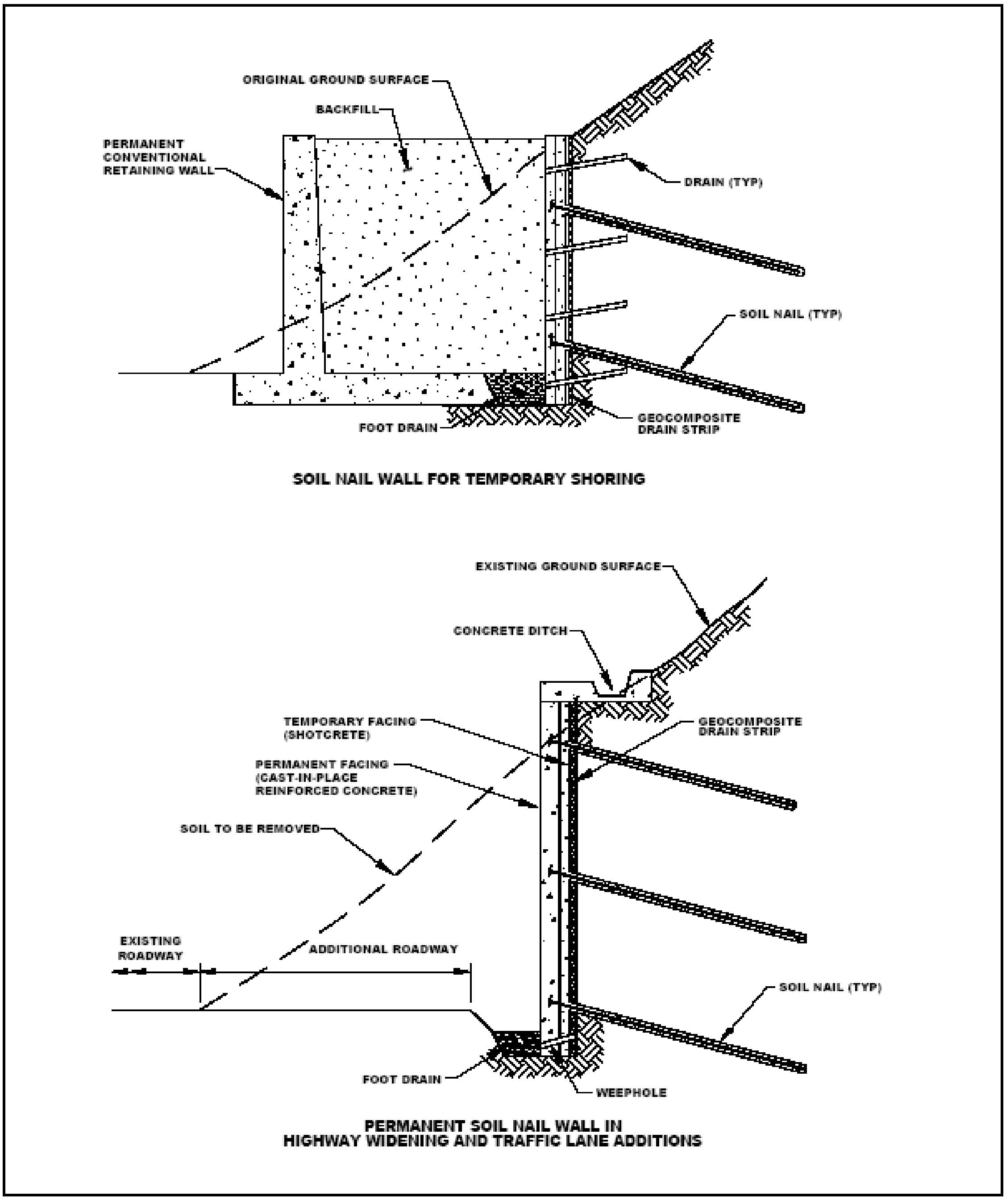


Figure 4.9. Soil nail walls for temporary and permanent cut slopes (FHWA, 2003)

4.5.4. Construction of Retaining Structure Under Existing Bridge Abutments

Soil nail walls can be advantageous for underpass widening by removal of an existing bridge abutment end slope, as shown in Figure 4.10, when compared to conventional ground anchor supported walls. Soil nail walls can be installed at comparable costs; however, the installation of soil nail walls does not require that bridge traffic be interrupted (FHWA, 2003).

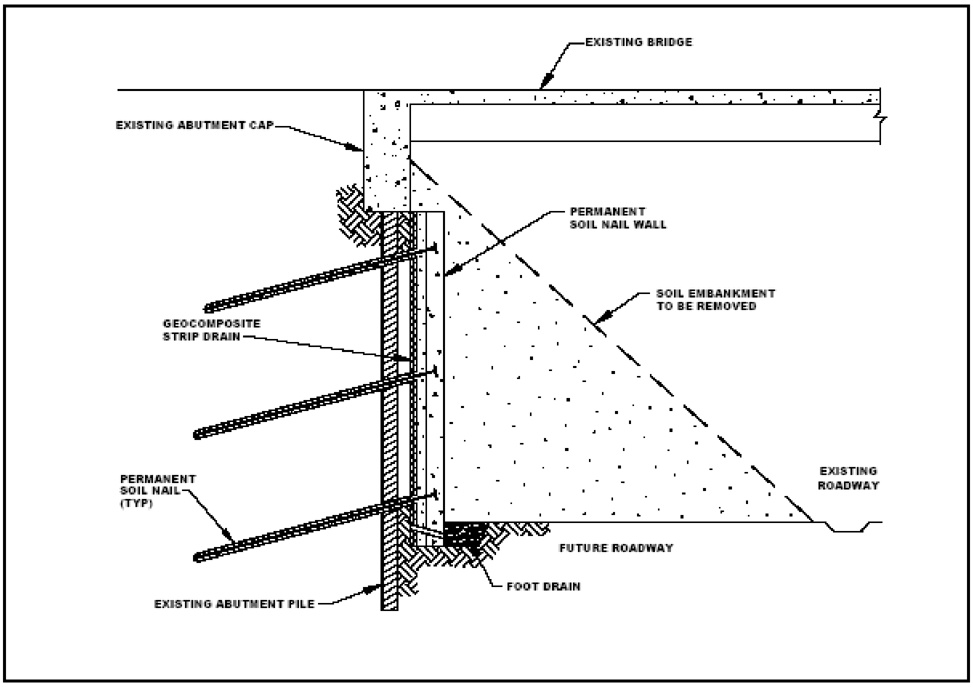


Figure 4.10. A typical construction of road widening under existing bridge (FHWA, 2003)

4.5.5. Construction of Retaining Walls

Soil nailing can be used to construct new temporary and permanent retaining walls in urban environments as they add stabilizing resistance in situations where other retaining structures like anchor walls are commonly used and where ground conditions are suitable. The relatively wide range of available facing systems allows for various aesthetic requirements to be addressed. In this application, soil nailing is attractive because it tends to minimize excavation, requires reasonable right-of-way (ROW) and clearing limits, and hence, minimizes environmental impacts.

4.6. Feasibility Evaluation of Soil Nailing

The feasibility evaluation of a soil nail wall should encompass technical and economical considerations and include an evaluation of the prevailing ground conditions, an assessment of the advantages and disadvantages of a soil nail wall for the particular application being considered and evaluation of costs.

4.6.1. Favorable Soil Conditions for Soil Nailing

Soil nail walls have been constructed successfully in various types of soils. Construction difficulties and long-term complications can generally be avoided when specific favorable soil conditions prevail. Soil nailing has proven economically attractive and technically feasible when:

- the soil in which the excavation is constructed is able to stand unsupported in a 1-2 m high vertical or nearly vertical cut for one to two days,
- all soil nails within a cross section are located above the groundwater table,
- if the soil nails are below the groundwater table, the groundwater does not adversely affect the face of the excavation, the bond strength of the interface between the grout and the surrounding ground, or the long-term integrity of the soil nails (FHWA, 2003).

It is advantageous that the ground conditions allow drillholes to be advanced without the use of drill casings and for the drillhole to be unsupported for a few hours until the nail bars are installed and the drillhole is grouted. Alternatively, soil nails have been installed with success using the hollow-stem drilling method in fully and temporarily cased drillholes (FHWA, 1998).

Based on the general criteria for favorable conditions noted above, the following ground types are generally considered well suited for soil nailing applications.

- Stiff to hard fine-grained soils. Fine-grained soils can be tentatively classified as stiff if they have SPT N-values of at least 9. To minimize potential long-term lateral displacements of the soil nail wall, fine-grained soils should be of relatively low plasticity, in general, plasticity index PI < 15.

- Dense to very dense granular soils with some apparent cohesion. These soils include sand and gravel with SPT N-values larger than 30, and with some fines typically no more than about 10 to 15 percent or with weak natural cementation that provide cohesion. Capillary forces in moist fine sands may also provide an apparent cohesion. In general, the apparent cohesion for these soils should be greater than 5 kPa to assure reasonable stand-up times.
- Weathered rock with no weakness planes. Weathered rock may provide a suitable supporting material for soil nails as long as weakness planes are not unfavorably oriented such as dipping into the excavation. It is also desirable that the degree of weathering be approximately uniform throughout the rock so that only one drilling and installation method will be required. The greywacke formation studied within thesis is included in this group.
- Glacial soils. Glacial outwash and glacial till materials are typically suitable for soil nailing applications as these soils are typically dense, well-graded granular materials with a limited amount of fines.

Because of the construction flexibility of the method, soil nailing is also well suited to mixed-face conditions including competent ground containing cobbles and boulders in which soldier beam installation is very difficult and expensive, providing each of the different materials is individually suited to soil nailing in accordance with the above (FHWA, 1998).

4.6.2. Unfavorable of Difficult Soil Conditions for Soil Nailing

As with most construction methods, soil nailing is not universally applicable and its limitations must be clearly understood. These limitations can be technically solved by appropriate design or construction provisions, but this often results in the method no longer being cost-effective. The following soil types or ground conditions are not considered well suited to soil nailing or limit it's application (FHWA, 2003):

- Dry, poorly graded cohesionless soils with a uniformity coefficient of less than 2, unless in a very dense condition. During the construction, these soil types will tend to ravel when exposed due to a lack of apparent cohesion.
- Soils with high groundwater. Perched groundwater occurring behind the proposed soil nail wall will require significant drainage, which is necessary to stabilize the mass of soil in this location.
- Soils with cobbles and boulders. A large proportion of cobbles and boulders present in the soil may cause excessive difficulties for drilling and may lead to significant construction costs and delays.
- Soft to very soft fine-grained soils. These soils typically have SPT N-values less than 4 and are unfavorable for soil nailing because they develop relatively low bond strengths at the nail-grout-soil interface.
- Organic soils. Some organic soils such as organic silts, organic clays and peat typically exhibit very low shear strengths and thereby low bond strengths, which causes uneconomical nail lengths.
- Weathered rock with unfavorable weakness planes and karst. Weathered rock with prevalent unfavorable weakness planes such as joints, fractures, shears, faults, bedding, schistosity, or cleavage may affect the drillhole stability and make grouting difficult. In addition, the presence of these discontinuities may cause the formation of potentially unstable blocks in the retained mass behind the wall during excavation.
- Loess. When it is dry, loess may exhibit acceptable strengths that would allow economical installation of soil nails. However, when sizable amounts of water ingress behind the proposed soil nail wall, the structure of the loess may collapse and a significant loss of soil strength may take place.

In addition to the difficulties described above, other aspects related to soil conditions must be considered when assessing the feasibility of soil nail walls (FHWA, 1998):

- The prolonged exposure to ambient freezing temperatures may cause frost action in saturated, granular soils and silt; as a result, increased pressures will be applied to the temporary and permanent facings.

- Repeated freeze-and-thaw cycles taking place in the soil retained by the soil nail wall may reduce the bond strength at the soil nail grout-ground interface and the adhesion between the shotcrete and the soil. To minimize these detrimental effects, a suitable protection against frost penetration and an appropriate shotcrete mix must be provided.
- Granular soils that are very loose ($N \leq 4$) and loose ($4 < N \leq 10$) may undergo excessive settlement due to vibrations caused by construction equipment and traffic.
- Loose and very loose saturated granular soil can be susceptible to liquefaction in seismically exposed regions. Several ground modification techniques may be utilized to densify granular soils and thereby minimize these damaging effects.

4.6.3. Intermediate Soil Conditions for Soil Nailing

There exists some soil conditions that are intermediate to the two conditions described previously. Although, the engineering properties are less favorable, soil nail walls have been installed successfully and cost-efficiently in certain intermediate soil conditions. Examples of intermediate soil conditions are presented below (FHWA, 2003):

- Engineered fill. Soil nails can be installed in engineered fill if it is a mixture of well-graded granular material, approximately 90 percent of the mix or more, and fine-grained soil with low plasticity, $PI < 15$.
- Residual soils. Those soils created from the in-place weathering of the parent rock material may be an acceptable material for soil nailing. For these types of soil, specific consideration should be given to the soil spatial variability and its ability to drain.

4.6.4. Advantages of Soil Nailing

Soil nailed structures have several advantages when compared to pre-stressed ground anchors and alternative top down construction techniques. These advantages can be grouped into three main categories as construction, performance and financial advantages (FHWA, 2003).

Some of the constructional advantages of soil nailing are presented below.

- Soil nailing is advantageous at sites with difficult or remote access because of the relatively small size and the mobility of the required construction equipment.
- Soil nailing is well suited to urban construction where noise, vibration and access can create problems.
- Soil nailing is a flexible technique, rapidly accommodating variations in soil conditions and work programs as excavation proceeds.
- There is no need to embed any structural element below the bottom of excavation as with soldier beams used in ground anchor walls.
- Overhead construction requirements are smaller than those for ground anchor walls because soil nail walls do not require the installation of soldier beams; this is particularly important when construction occurs under a bridge.
- Requires smaller right-of-way than ground anchors as soil nails are typically shorter,
- Since significantly more soil nails are used than ground anchors, adjustments to the design layout of the soil nails are more easily accomplished in the field without compromising the level of safety;
- Easy adjustments of nail inclination and location can be made when obstructions are encountered as cobbles or boulders, piles or underground utilities.
- There is no need for a high capacity structural facing since the maximum earth pressure support loads are not transferred to the excavation face.
- Soil nailing is well suited to special applications such as the rehabilitation of distressed retaining structures.

Advantages with regard to the performance of the soil nail walls are as follows.

- Soil nailing is relatively flexible and can accommodate relatively large total and differential settlements.
- Since there are large number of nails, failure of any one may not detrimentally affect the stability of the system, as would be the case for conventional tie-back system.
- Surface deflections can be controlled by the installation of additional nails.

- Overall movements required to mobilize the reinforcement forces are relatively small.
- Soil nail walls have performed well during seismic events owing to overall system flexibility.

Financial advantages of soil nailing are summarized below.

- Soil nailing is more economical than conventional concrete gravity walls when conventional soil nailing construction procedures are used.
- Soil nailing is typically equivalent in cost or more cost-effective than ground anchor walls.
- Shotcrete facing is typically less costly than the structural facing required for other wall systems.

4.6.5. Disadvantages of Soil Nailing

Some of the potential disadvantages of soil nail walls are listed below (FHWA, 1998):

- For new steep cuts, soil nail construction requires the formation of cuts generally 1-2 m high in the soil. These then have to stand unsupported for at least a few hours before placement of sprayed concrete and nailing. The soil therefore has to have a degree of natural cohesion or cementing.
- Soil nailing is not well suited for applications where strict deformation control is required for structures and utilities located behind the proposed wall, as the system requires some soil deformation to mobilize resistance.
- In urban areas, the occurrence of utilities may place restrictions on the location, inclination, and length of soil nails in the upper rows.
- For soil nailing, dewatered face is desirable. If the groundwater percolates through the face the unreinforced soil will slump locally on initial excavation, making it impossible to establish a satisfactory sprayed concrete face.
- Permanent soil nail walls require permanent underground easements.

- Soil nail capacity will not be economically developed in cohesive soils subject to creep, even at relatively low load levels.
- Excavations in soft clay are unsuited to soil nailing because the low frictional resistance of the soft clay would require a very high density of soil nail reinforcement of considerable length to ensure adequate levels of stability.

4.6.6. Construction Cost Evaluation

Soil nailing is considered as a unique cut retaining technique due to its technical and economical advantages over more conventional techniques. If installed in ground conditions well suited for soil nail wall construction, soil nailing has proven to be a very economical method of constructing retaining walls. The system has reported lower cost due to the following factors (FHWA, 2003):

- The expediency of construction.
- Relatively rapid installation of the unstressed inclusions which are considerably shorter than earth anchors.
- Light construction equipment required to install nails.
- Simple grouting equipment that is easy to mobilize.
- Relatively thin shotcrete or concrete facing.
- Structural benefits of distributing the face loads over large number of nails.

In Europe, it is reported that soil nailing costs in ground suitable to soil nailing are, in general, 20 percent lower than comparable tieback structures. For cut retention, experience on U.S. highway projects indicates soil nail walls when used in ground well suited to soil nailing, can provide 10 to 30 percent cost savings versus permanent tieback walls or conventional cast-in-place walls with temporary shoring (FHWA, 1998). The cost savings of the applications of soil nailing in deep excavations in greywackes are reported as high as 35 per cent in Istanbul.

Costs for soil nail walls are a function of several factors, including ground conditions, site accessibility, wall size, facing type, corrosion protection requirements, temporary or permanent application, availability of contractors specialized in soil nailing and shotcrete, and regional conditions as seismic and frost susceptibility.

4.7. Soil Nailing Mechanisms and Behavior

It is important to understand the fundamental mechanisms of behavior associated with soil nailing before starting to design a soil nailed system. The soil nailing system, development of load in soil nails, role of bending and shear resistance, role of the facing, deformation and redundancy in soil nailing systems are briefly covered together with limit state design and general requirements and factors of safety discussions herein.

4.7.1. The Soil Nailing System

Soil nailing is an in-situ ground reinforcement technique that creates a reinforced soil block resisting destabilising external loads such as earth pressure, surcharges and live load and internal nail forces such as those due to relative movements and local material failures.

Stability is satisfied by the mobilisation of shear stresses at the soil/nail interface. Since the system satisfies equilibrium by mobilising resisting forces directly in response to destabilising forces, soil nails are often described as "passive" (Phear *et al*., 2005).

The internal stability of a soil-nailed system is often considered in terms of two zones divided idealistically by a potential failure plane which are shown in Figure 4.11. The first is the resistant “passive” zone behind the potential failure plane contains the distal end of the nails with sufficient bond length to prevent the reinforcement being pulled out and the second zone is the “active zone” in front of the potential failure plane contains the proximal end of the nails. The direction of interface bond stress is different in the active and resistant zones, as shown in Figure 4.12.

Internal stability failure can occur in one of three ways: (i) The nail tendon reaches tensile capacity, or in extreme cases tensile and bending capacity, and ruptures, (ii) the grout/nail tendon interface bond capacity is reached and (iii) the bond capacity of the soil/nail interface is reached, either in the resistant or active zone.

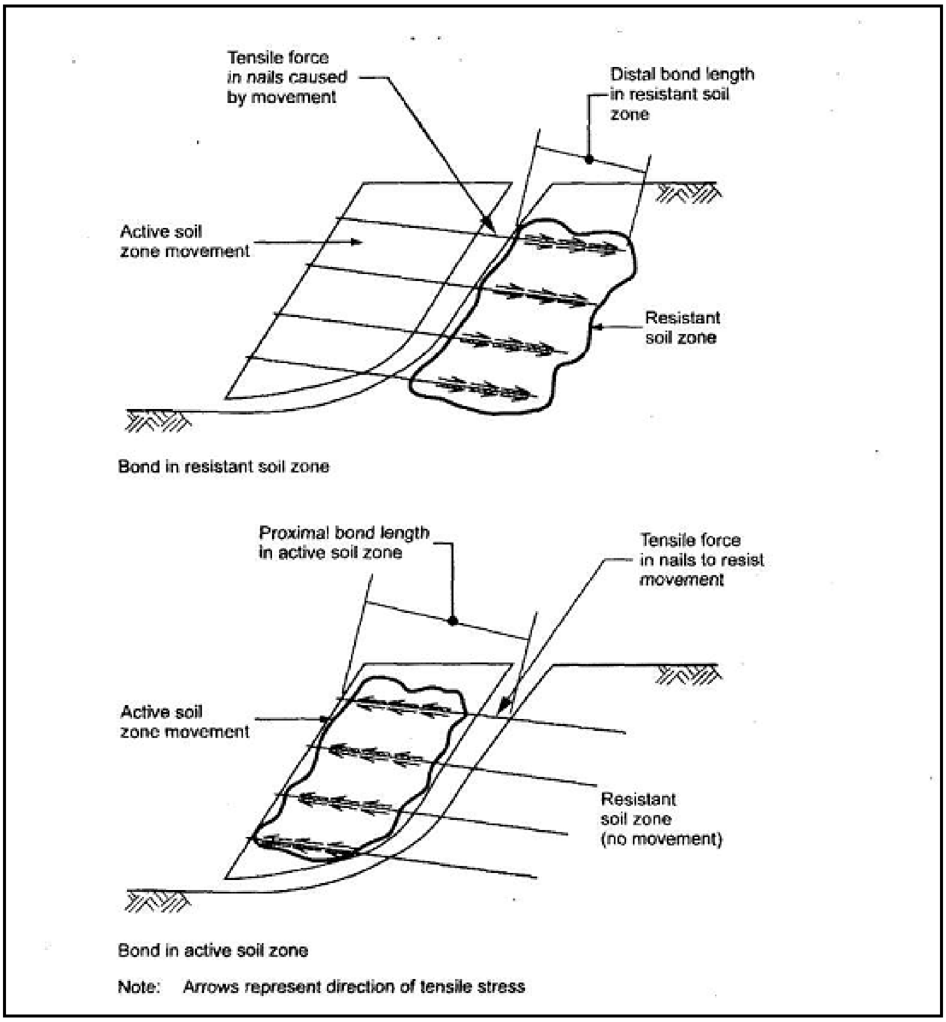


Figure 4.11. Schematic diagram of a soil-nailed slope (Barley *et al*., 1997a)

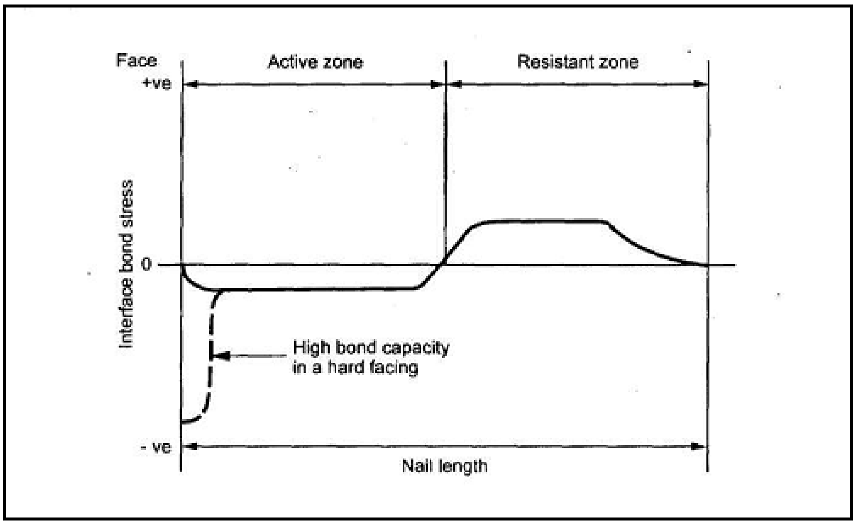


Figure 4.12. Probable distribution of bond stress at soil/nail interface in a soil nail (Barley *et al*., 1997a)

4.7.2. Development of Loads in Soil Nails

Movement of the slope face needs to occur to induce load on the soil nails. The ways in which such movements can develop are discussed below for slopes or walls constructed by excavation and for stabilisation of existing slopes. The main difference between these two cases is the way that the soil nails are loaded (Phear *et al*., 2005):

- for steep slopes and walls constructed by excavation, the nails are progressively loaded as each stage of excavation is performed
- for existing slopes stabilised using soil nails, the nails will remain untensioned, therefore will have little stabilising effect, until relative movement of the active and resistant zones of the slope occurs. This may take some years, or may never happen.

Soil nailing frequently involves staged construction and progressive excavation to form the final soil nailed wall. The bond stress distribution at the soil/nail interface changes as excavation proceeds and the face deforms, as shown in Figure 4.13.

When the first row of nails is installed there is almost no load on the nails because the slope has only experienced a small lateral and downward movement and the active zone is very small. When the next level of excavation is made, the slope deforms sufficiently for the active zone to enlarge. Only the proximal end of the nail is initially actively stressed by this movement. In this initial active zone the straining soil probably distributes a reasonably uniform stress along the nail (Barley *et al*., 1997a). As excavation proceeds to lower levels, the extent of the active zone increases as the slope deforms further and a greater volume of soil undergoes strain.

The stress distribution in working soil nails has been examined in several laboratory-scale investigations using centrifuges (Gassier and Gudehus, 1981 and Davies *et al*., 1997) and in the field.

For existing slopes stabilised by soil nailing the bond stress distribution at the soil/nail interface also only develops as the slope face deforms and strains occur within the soil, as shown in Figure 4.14.

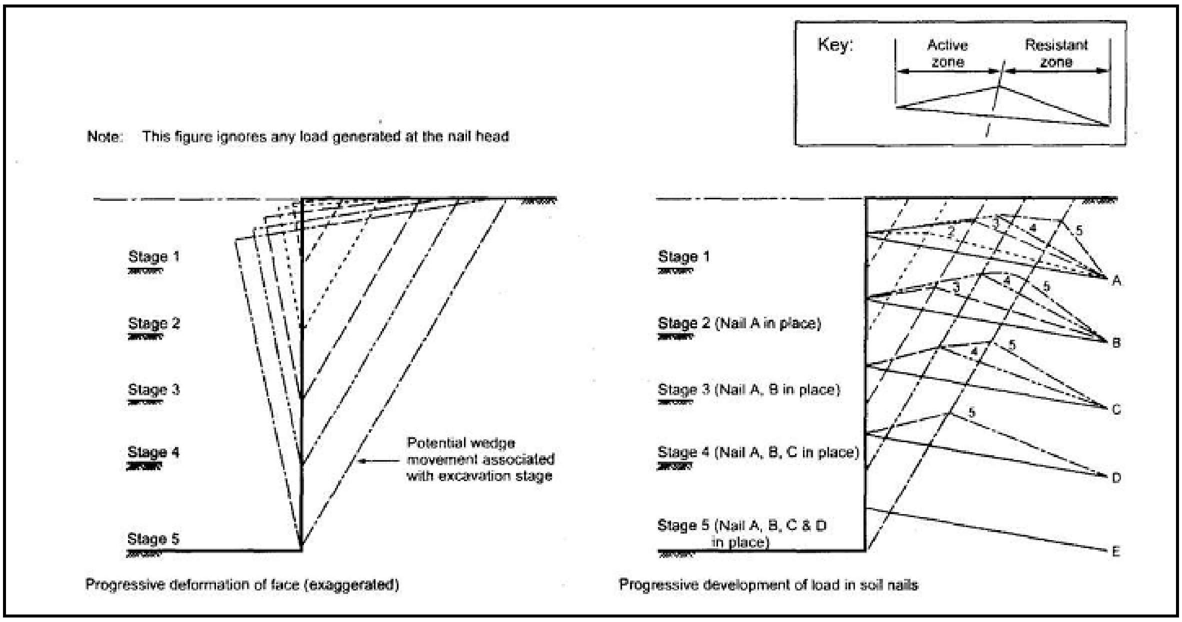


Figure 4.13. Development of deformation and tensile load in soil nails during the excavation in stages of a steep slope (Pedley *et al.*, 1990)

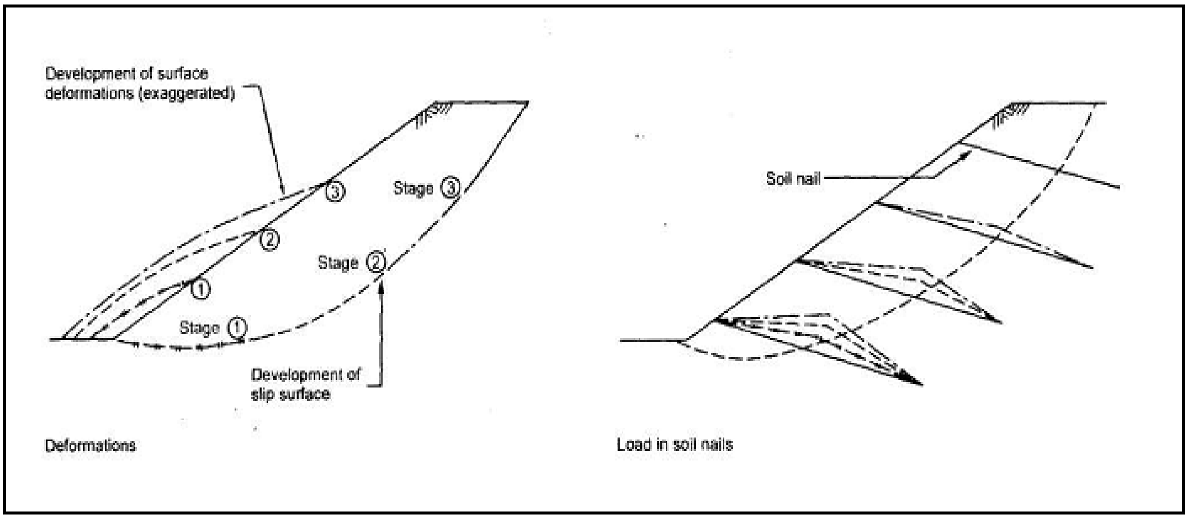


Figure 4.14. Development of deformations and loads in an existing slope stabilised by soil nails (with a flexible facing) (Phear *et al.*, 2005)

4.7.3. Role of Bending and Shear Resistance

Soil nails have bending and shear resistance, as well as tensile resistance. For most soils, the ability to mobilize shear resistance is a function of the section properties and the relative soil/nail stiffness and requires large deformations. It will therefore not occur under serviceability conditions (Clouterre, 1991; FHWA 1998). For typical soil-nailed walls and slopes, where the nails are inclined at 5-20° below horizontal, the contribution of nail bending and shear resistance can be neglected for practical purposes (Pedley *et al*., 1990).

When nails are installed at steeper angles than 20°, it is likely that they will intersect the potential failure plane almost perpendicularly and this will result in some mobilization of bending resistances (Jewell and Pedley, 1992). However, even at ultimate limit state, the tensile resistance of the nails will be dominant and bending/shear resistance will be only of secondary importance.

4.7.4. Role of the Facing

Where soil nails are used to stabilize an existing slope, or to construct a new slope or wall, they do not stabilize the surface soil. This is done by means of head plates and/or a facing. Separate measures to retain the surface soil need to be adopted and integrated with the soil-nailing system. The facing system, which could be hard, flexible or soft, can modify the internal failure mechanism dependent on facing strength, slope angle, and/or pre-existing failure mechanisms (Phear *et al*., 2005). As the bond stresses are developed throughout the nail length, the proportion of these stresses transferred to the facing varies depending on the type and stiffness of facing.

4.7.5. Deformation

The amount that a soil-nailed slope deforms not only depends on the nail lengths and spacings but also on the construction process. For slopes or walls with a hard facing constructed by excavation, it is generally expected that the maximum deflection would occur at the top of the wall. With existing slopes stabilised by soil nailing or for slopes or walls with a soft or flexible facing constructed by excavation, the maximum deflections may occur near the toe of the slope. Factors that directly affect the deformation characteristics of soil-nailed slopes and walls include Geological structure and profile, soil type and stiffness, and the geometry of wall as slope angle, inclination of nails from the horizontal, nail lengths and spacing, are the factors that directly affect the deformation characteristics of soil-nailed slopes and walls.

4.7.6. Redundancy in Soil Nailing Systems

Because of the large number of nails that comprise most soil-nailing systems, soil nailing has a higher system redundancy than other earth retaining techniques. The main benefit of this is that the failure mode is less brittle. Simply raising the amount of redundancy will result in a lower risk of failure. It is unlikely that the failure of any individual nail will result in excessive deformation, and/or total slope failure.

4.7.7. Limit State Design

When a soil-nailed wall or slope, or part of it, fails to satisfy any of its performance criteria, the wall or slope is deemed to have reached a limit state. Various ultimate and serviceability limit states are considered separately in the design and their occurrence is either eliminated or shown to be sufficiently unlikely. Predicting deformations of soil-nailed walls and slopes can be difficult. However, deformations will generally be acceptable if adequate partial factors are adopted and soil nailing is appropriately applied to the situation with good construction control (Phear *et al*., 2005).

4.7.7.1. Ultimate Limit State. An ultimate limit state is defined as a state beyond which a failure mechanism can form in the ground around or through a soil-nailed system, or failure of the principal structural elements occurs. Additionally, the ultimate limit state of other structures supported by the surrounding ground needs to be considered.

The design should guard against the occurrence of brittle failure, such as sudden collapse without obvious preliminary deformation. The retaining wall system should be designed to exhibit sufficient ductility in approaching geotechnical limit states to give visible warning of failure. For analysis purposes, the hazards are related to particular postulated failure mechanisms.

For soil-nailed slopes and walls, Figure 4.15 illustrates ultimate limit states for soil-nailed slopes and walls. External stability hazards are loss of overall stability at any stage, rotation, sliding and foundation failure. Whereas the internal stability hazards are pullout of nails through failure at the soil/nail interface, rupture of the soil nails, toppling of the facing, bending or punching failure of structural facings, punching or bearing failure of head plates, failure of soil between the nails and bearing capacity or structural failure of head bearing pads.

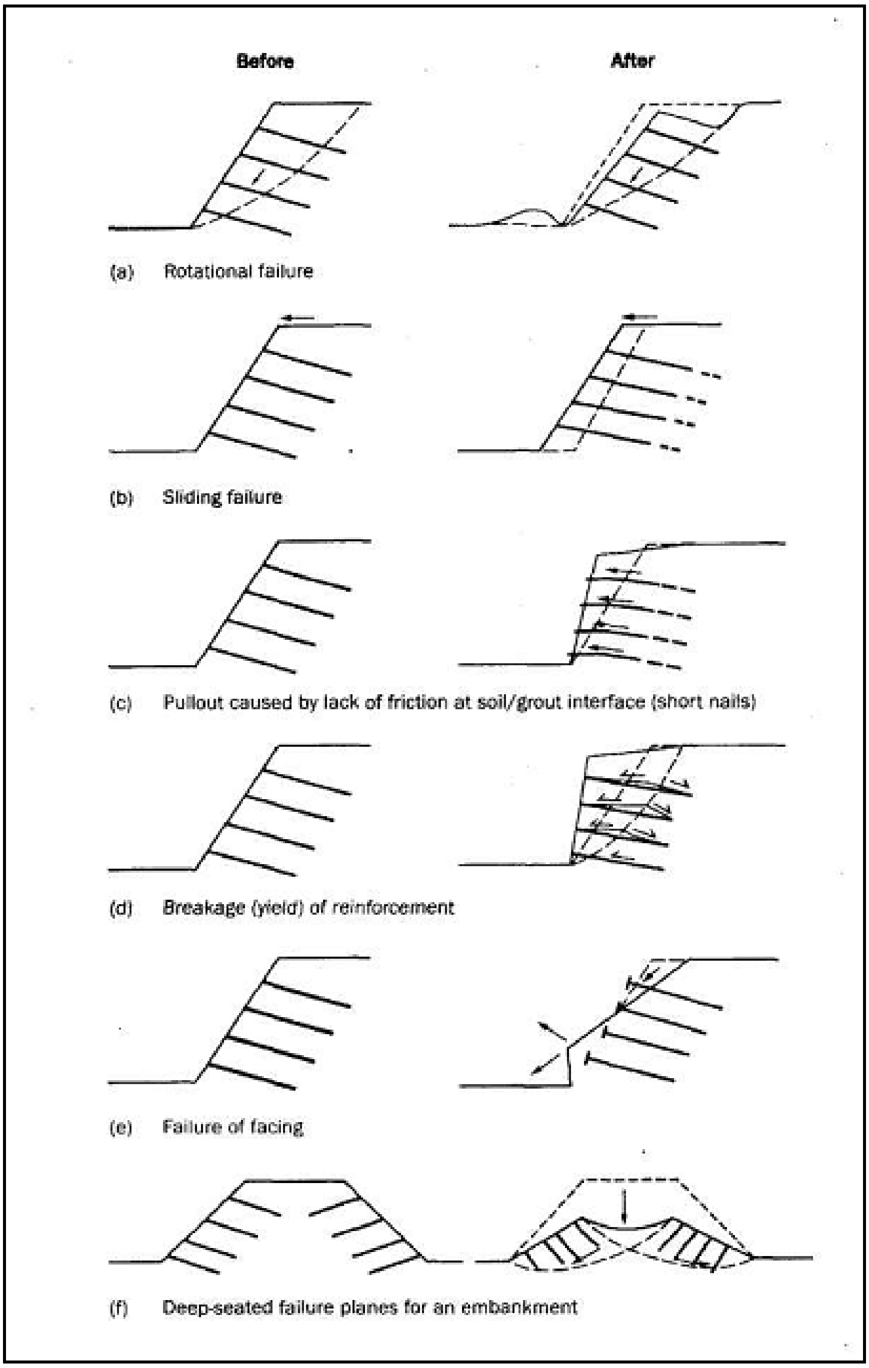


Figure 4.15. Illustrations of ultimate limit states for soil-nailed slopes and walls (Phear *et al.*, 2005)

<u>4.7.7.2. Serviceability Limit State</u>. A serviceability limit state is defined as a state at which excessive deformation of the ground or structure occurs, or seepages or blockage of the drainage system occur. Additionally, the serviceability limit state of other structures supported by the surrounding ground may have to be considered. The design life is the period for which all serviceability criteria need to be met. Common serviceability limit states are shown in Figure 4.16. These are the strains or movements of the facing that could affect the visual appearance of the facing or result in unforeseen maintenance, deformations in the facing that could affect the serviceability of any adjacent structures, services or infrastructure and excessive total, or differential, settlements at the top or the base of the wall or slope.

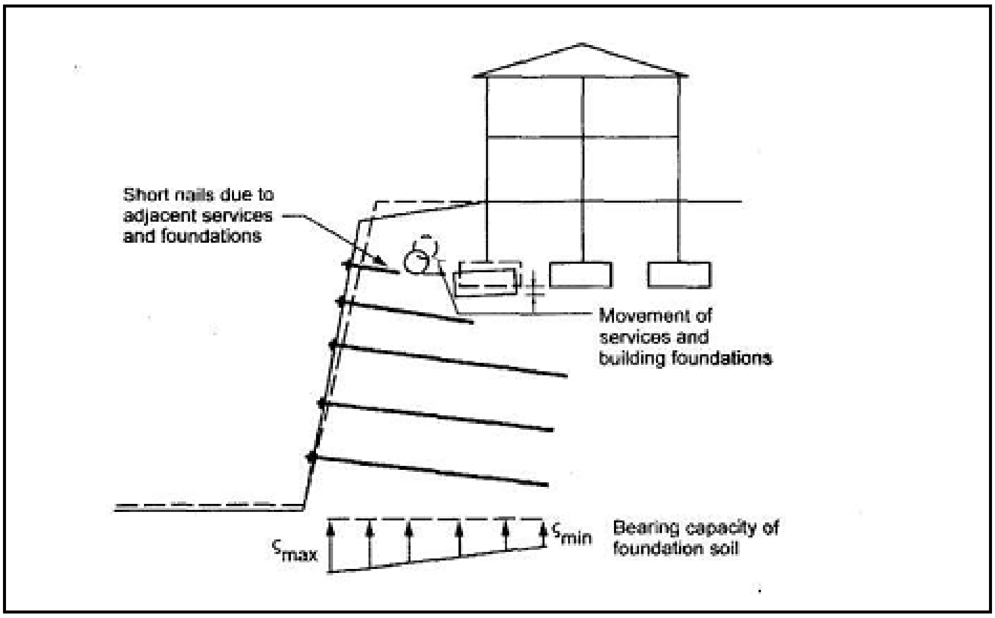


Figure 4.16. Illustrations of serviceability limit states for soil-nailed slopes and walls (Phear *et al.*, 2005)

4.7.8. General Requirements and Factors of Safety

Previous slope stability and retaining wall methods relied on the provision of adequate restraining forces to resist a set of disturbing forces. For soil-nailed systems, the restraining forces are a combination of resistive forces attributable to soil strength and mobilized nail pullout forces in the resistant zone. The movements needed to generate these resistive forces were usually ignored and generally little guidance was given on the soil parameters that should be used in the calculations. The approach was based on a simple definition of a factor of safety, defined as the ratio of resistive forces or moments to disturbing forces or moments. Values of 1.3-1.5 were usually sought for overall stability, 2.0 for overturning and sliding and 3.0 for bearing capacity. These were usually in combination with a factored pullout resistance of 2.0-2.5 (Phear *et al.*, 2005). This approach is expressed below and shown in Figure 4.17.

$$\frac{\Sigma R_N}{\gamma_N} + \frac{R_S}{\gamma_S} \geq R_D \gamma_D \tag{4.1}$$

where:

R_N and R_S	are characteristic values of resistive forces against failure for the nails and soil respectively.
R_D	is the characteristic value of the disturbing forces.
γ_N, γ_S and γ_D	are partial factors to obtain design values (> 1.0).

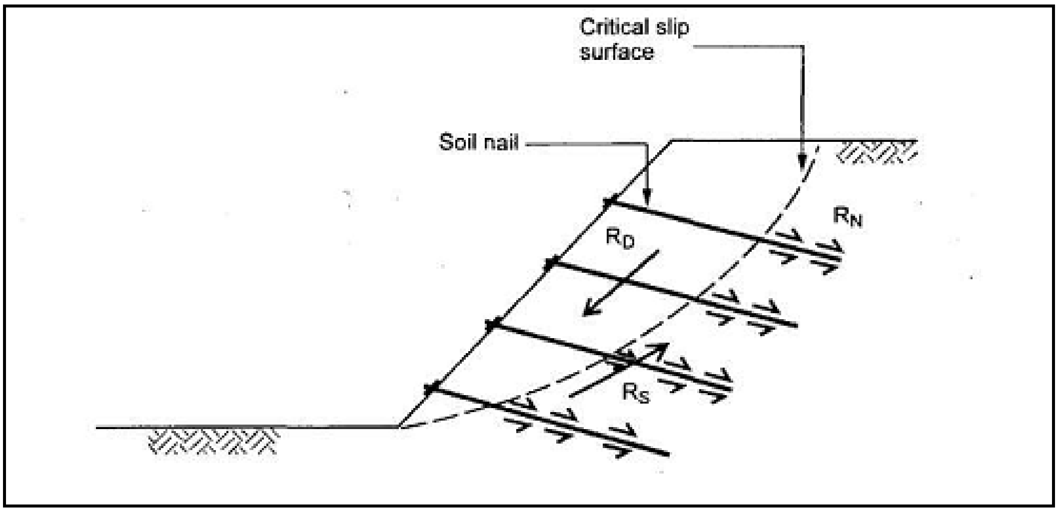


Figure 4.17. Resisting and disturbing forces for slope stability (Phear *et al.*, 2005)

For any soil-nailed slope or wall, the deformation of soil nails, and therefore also the mobilization of resisting forces, varies with depth. Consequently, although the global factor of safety of the system might be higher than the target value, individual factors of safety against pullout at each nail level may not be. This is why a separate factor of safety should be applied to the nail pullout resistance even though some designers might consider it over-conservative.

4.8. Site Investigation for Soil Nailing

The key to minimizing geotechnical risk in the design of a soil-nailing scheme is to develop a thorough understanding of the ground conditions through appropriate, timely site investigation (Phear *et al*., 2005). While standard site investigation techniques are essential to develop an understanding of the ground conditions and to derive parameters for design, field trials provide a more direct means of confirming the suitability of the ground for soil nailing and measuring the potential bond capacity. Trial excavations are essential to establish safe stand-up times for excavation of soil-nailed benches. Particularly for larger structures, installation and testing of preliminary sacrificial nails can be a cost-effective way to reduce risk in the design and construction of the main works.

4.8.1. Planning a Site Investigation

It is important to establish the purpose and objectives of the investigation before starting the work. A properly conducted investigation can save money by minimizing over-conservative design, redesign during construction, delays and costs arising from unforeseen ground conditions and risk of subsequent ground problems. A site investigation carried out for a proposed soil-nailed slope or wall has four main purposes (Phear *et al*., 2005):

(i) To establish the geological and hydrogeological conditions at the site, likely variations in the stratigraphy and any adverse geological features or materials.
(ii) To confirm whether or not the ground conditions are suitable for soil nailing.
(iii) To determine geotechnical parameters for design of the soil-nailed structure including values for the bulk properties of the ground (compressibility and permeability of the strata, bond strength) and the specific behaviour of individual materials (tests to establish the range of plasticity, density, grading, strength and aggressivity of each material).

(iv) To provide information to assess construction aspects such as safe stand-up time for temporary excavations and drilling or driving rates through different strata.

4.8.2. Preliminary Site Appraisal

Before undertaking a site investigation for a soil-nailing scheme, or indeed any construction works that affect the ground, a preliminary appraisal of the site should be made. This should include a desk study and a walk-over survey of the site. A thorough preliminary appraisal provides valuable information about the site at a very low cost. Aerial photographs are of particular importance for soil nailing in existing slopes, as they can provide evidence of previous instability and show sub-surface features that could affect the overall stability of the proposed works.

Once the desk study has been completed, a walk-over survey should be carried out to verify the information obtained in the desk study. Surface seepage or runoff may be evident, particularly during wet weather, which can inform the need for temporary and permanent drainage within the soil-nailing scheme. Cuts or exposures at the site can be very useful for soil-nailing schemes. The condition of existing structures in the ground can be inspected for signs of corrosion, which may indicate aggressivity in the ground that could affect the choice and subsequent durability of the soil nails. The walk-over also provides the opportunity to identify other structures and services that could be affected by the proposed works.

4.8.3. Subsurface Investigation

Subsurface investigations typically consist of in situ testing of soil/rock properties, retrieval of representative samples of soil/rock for visual classification and/or laboratory testing, characterization of the stratigraphy, and identification and observation of groundwater location. The soil/rock properties of interest to be determined from in situ and laboratory testing include classification, index parameters, strength, compressibility, and corrosion potential. In addition, determining the location and nature of the groundwater is important in soil nail wall projects because these systems are difficult to construct and more costly when the groundwater is high.

A suitably qualified geotechnical specialist should determine the type, depth and spacing of exploratory holes to suit the site. In Clouterre (1991), subsurface stratigraphy should be developed from site investigation for every 200-600 m^2 of soil-nailed wall and no recommendations on depth. In Eurocode 7 Part 1, for Category 2 structures covering a large area, exploratory holes should be spaced at 20-40 m. In FHWA (1998), in flat or gently sloping ground, boreholes should be provided at 30 m centers along the line of a soil-nailed wall. To investigate the soil zone to be nailed, additional holes should be bored, as shown on Figure 4.18. For sloping ground conditions, this offset should be increased to 1.5-2.0 times the proposed wall height. At critical sections, additional boreholes should be added in front of the proposed structure to define better the soil/rock stratigraphy.

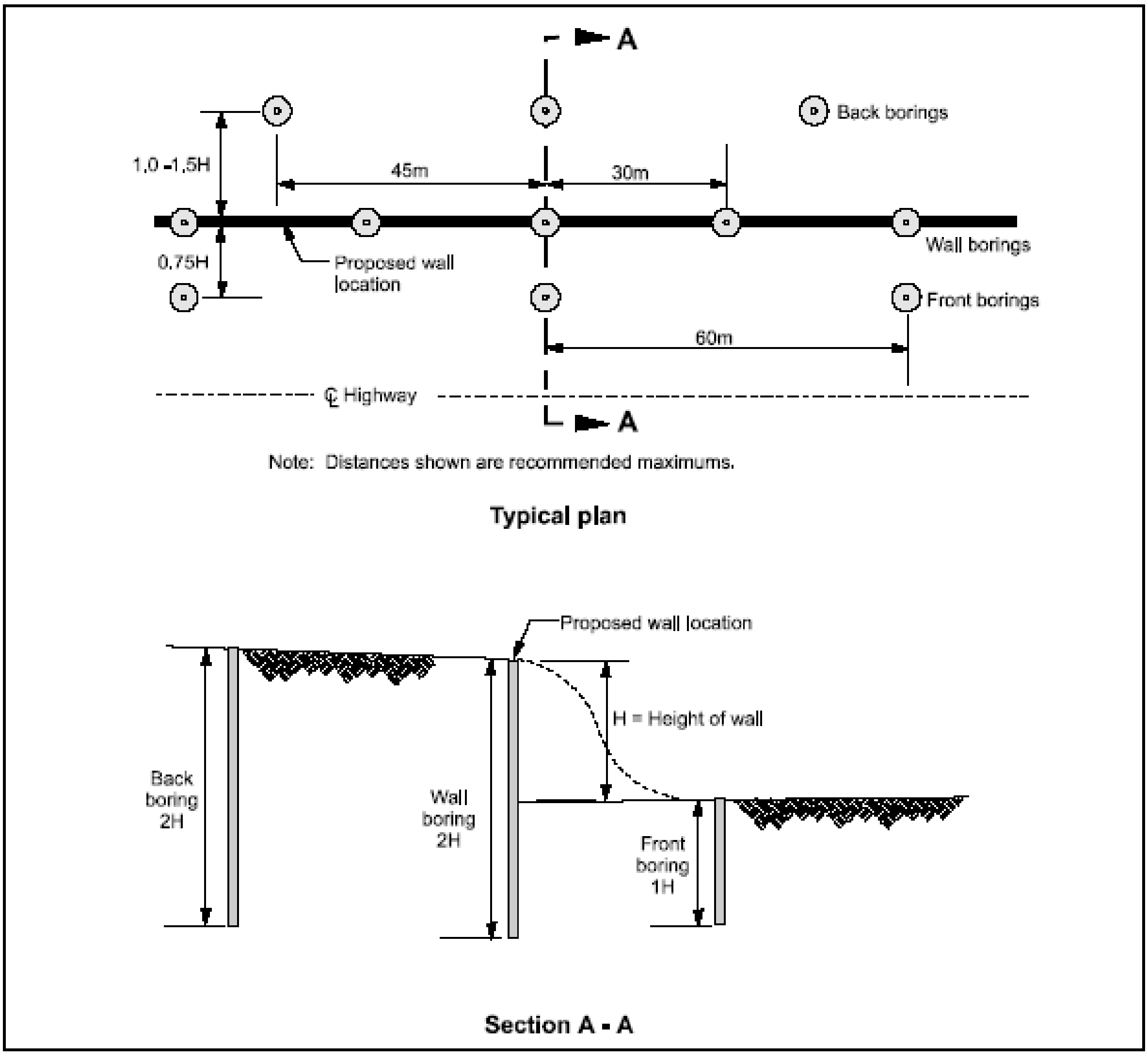


Figure 4.18. Site exploration guidelines for soil-nailed walls (FHWA, 1998)

In-situ tests are particularly useful in granular soils and soft cohesive soils where undisturbed samples are difficult to obtain. Well-established correlations with soil properties exist for standardized techniques, such as standard penetration test (SPT), cone penetration test (CPT) and pressuremeter testing. Plate load testing may provide useful data on load-settlement performance near the ground surface for use in designing nail head restraint details (plates, anchor blocks or facings) for soil-nailed slopes. Permeability testing may be carried out to determine the rate of dissipation of pore pressures in the ground, to provide information on the rate of groundwater flow for design of drainage and to assess the mobility of groundwater in transporting aggressive chemicals. Common field tests that can be used in site investigations of soil nail wall projects are summarized in Table 4.1.

Table 4.1. Common geotechnical field procedures and tests (FHWA, 2003)

	Activity	**Most suitable for**	**Not suitable for**	**Obtained from field activity**
Field Procedure	Preservation and Transportation of Soil Samples	All	NA	Representative samples
	Thin-Walled Tube Sampling	Clays, Silts	Sands, Gravel	Undisturbed samples
	Subsurface Explorations (Soil and Rock)	All	NA	Various
Field Test	Standard Penetration Test (SPT)	Sand, Silt		Stratigraphy, SPT N-values relative density, groundwater, samples
	Cone Penetration Test (CPT)	Sand, Silt, and Clay	Gravel, bouldery soil	Continuous stratigraphy, soil type, strength, relative density, K_0, pore pressures, no sample
	Field Vane Shear Test (VST)	Soft to Medium Clay	Sand and Gravel	Undrained shear strength
	Pressuremeter Test (PMT)	Soft Rock, Dense Sand, Non-Sensitive Clay, Gravel, Till	Soft Clays, Loose Silts and Sands	Soil type, strength, K_0, OCR, compressibility, soil modulus, no sample
	Flat Plate Dilatometer Test (DMT)	Sand and Clay	Gravel	Soil type, K_0, OCR, undrained shear strength, soil modulus, no sample

Samples obtained with the SPT sampler are disturbed and only adequate for soil classification and some laboratory tests, including particle gradation (sieve analysis), fines content, natural moisture content, Atterberg limits, specific gravity of solids, organic contents, and unconfined compressive strength tests. Grab samples obtained from cuttings in borings, test pits, and test cuts can also be used for soil classification and laboratory determination of index parameters, as long as they are sufficiently representative and the in situ moisture content was preserved during sampling and transportation (FHWA, 1998). Undisturbed thin-walled samplers, including the Shelby tube sampler with an outer diameter (OD) of 76 mm, should be used instead to obtain samples of fine-grained soil for laboratory testing of shear strength and consolidation.

In soil nailing applications, the use of test pits in flat areas or test cuts in sloping ground can be particularly beneficial. Test pits are relatively inexpensive and can help assess whether an excavation face will stand unsupported and define the feasibility of soil nailing at a site. Test pits should be approximately 6 to 8 m long and 2 to 2.5 m deep and should be excavated parallel to and in front of the proposed wall face. To evaluate the stand-up time for the excavation, the test pit should be left open for 3 to 4 days (FHWA, 2003).

Groundwater monitoring is necessary to determine the groundwater regime at the site. Both the groundwater table and any perched water in the overlying strata should be identified. Observation of seepages and standing water should be recorded during drilling or excavation of exploratory holes. These observations only provide information in relatively permeable ground for as long as the exploratory hole is open. To measure pore water pressures in relatively impermeable ground and to understand seasonal fluctuations in groundwater level, standpipes or piezometers should be installed in boreholes and monitored over a suitable period. Ideally, monitoring data should be collected for at least 12 months to determine the full range of seasonal variations.

4.8.4. Laboratory Testing

Laboratory testing of soil samples recovered during the site investigation is performed to produce soil classification, index properties, unit weight, strength, and compressibility. Table 4.2 presents laboratory tests commonly used to develop index parameters and other engineering properties of soils that may be necessary for the design of a soil nail wall.

Table 4.2. Common procedures and laboratory tests for soils (FHWA, 2003)

PROCEDURE	TEST NAME	APPLICABILITY
Classification	Visual and Manual Description and Identification of Soils	All soils
	Classification of Soils according to USCS	All soils
Index Parameters	Particle-Size Analysis (with sieves)	Granular soils
	Soil Fraction finer than No. 200 (75-μm) Sieve	Fine-grained and granular materials boundary
	Moisture Content	All soils
	Atterberg Limits	Fine-grained soil
	Organic Contents	Fine-grained soil fraction
	Specific Gravity of Soil Solids	All soils
Strength	Unconfined Compressive Strength (UC)	Fine-grained soil
	Unconsolidated Undrained Triaxial Comp. (UU)	Fine-grained soil
	Consolidated Undrained Triaxial Comp. (CU)	Fine-grained soil
	Direct Shear (Consolidated)	Sands and Fine-grained soils
Hydraulic Conductivity	Permeability (Constant Head)	Granular soils
Compressibility	One-Dimensional Consolidation	Fine-grained soil
	One-Dimensional Consolidation (Controlled-Strain Loading)	Fine-grained soil
Other	Frost Heave and Thaw Weakening Susceptibility	Silts
	Collapse Potential	Loess, silt
	Swelling Potential	Fine-grained soil

4.8.5. Characteristic Soil Parameters for Design

For granular soils, the in-situ shear strength is highly dependent on in-situ density. Sampling of granular soils usually causes disturbance and alters the density of the soil. Density and shear strength are more usually determined from correlations with in-situ tests such as SPTs or CPTs.

For cohesive soils, density and shear strength are determined in the laboratory. Both undrained (total) shear strength and drained (effective) shear strength parameters should be determined to understand both short-term and long-term behaviour, respectively.

Direct shear tests may be carried out on most soil and rock types to determine drained shear strength parameters at large strains (post-peak and residual shear strength). This is particularly important to assess shear strength along planes of weakness such as existing shear surfaces and joints.

For soft or loose soils and fills where consolidation is likely to result in significant settlement of the proposed soil-nailed structure, consolidation tests may be carried out using odeometers, triaxial apparatus or hydraulic cell apparatus. Swelling tests may also be useful to quantify swelling pressures that might arise due to unloading causing degradation of slopes in plastic clays.

4.8.6. Field Trials

Probably the most valuable means of ground investigation to predict the likely behaviour of soil nails at a site is the use of a field trial. Preliminary sacrificial soil nails tested to failure provide the most reliable method of predicting the bond capacity of the proposed nails. Test nails can be de-bonded through different strata to determine the bond capacity in different materials. Field trials also highlight any construction problems in advance of the main contract works, such as problems with stability of temporary cuttings, capability of equipment to penetrate different strata, problems with flush fluids.

4.9. Conceptual Design of Soil Nailing

As with any other civil engineering activity, the design of a soil-nailed wall or slope comprises two main stages; conceptual (preliminary) design, which is followed by detailed design. Design is a process that begins with feasibility studies. Verification and improvement of design should continue during the construction stage by means of close observation and monitoring techniques to identify and feed back potential problems and facilitate potential cost savings. Observation and design modification during construction are particularly important for soil-nailed slopes or walls.

Before beginning design, the designer needs to understand the fundamental mechanisms of behaviour of soil nailing. The decision about the appropriateness of soil nailing is likely to be resolved by a risk assessment process in which other options are considered.

A conceptual design is useful at the pilot planning stage, or when alternatives are being considered, it also enables the feasibility of a soil nailed wall to be assessed and calculated sufficiently accurately so the cost involved can be estimated.

4.9.1. Risk Based Approach

The fundamental philosophy of the risk-based approach is that the scale of the works and the consequences of failure need to be assessed in order to confirm the suitability of soil nailing and to assess the level of analysis required in the design, site investigation requirements, type of specification, and the testing and monitoring regimes.

4.9.2. Characterization of the Ground

It is crucial to know ground and groundwater profile, soil strength for design and stand-up time, potential variability of the ground, including likely obstructions and permeability, proposed geometry and knowledge of adjacent structures at conceptual design stage.

4.9.3. Groundwater

Assessment and control of groundwater are fundamental aspects of the design of all soil-nailed slopes and walls. Soil nailing is not well suited to permanent excavations below the water table, particularly in predominantly granular soils. However, it can be used for temporary excavations below the water table if temporary dewatering is also undertaken. Risk assessments should be carried out before starting any temporary dewatering to estimate the magnitude and rate of settlement it may cause to adjacent structures and utilities and to consider the consequences of failure of the dewatering system, and hence the amount of redundancy required (Phear *et al*., 2005).

4.9.4. Construction Sequence and Buildability

Soil nailing for new slopes and walls is constructed using a top-down technique, which is undertaken in discrete excavation stages. The buildability issues should be considered at the conceptual design stage are temporary excavation stability, groundwater, soil nail installation method, potential obstructions, working procedures on slopes, materials handling, working space and facing type and installation method.

4.9.5. Site Constraints

One of the principal advantages of soil nailing is its suitability for sites with difficult access. This is because of the relatively small size and mobility of the plant and equipment used compared with other techniques. Nevertheless, for new slopes and walls, the site geometry has to afford sufficient space for plant safely to access the slope and form benches from which to install the nails as shown in Figure 4.19.

If the soil nailing is going to extend beyond the site boundary, it will be necessary to obtain the agreement of the adjacent landowner. Soil nails cannot be installed unless easement can be obtained, nor if an adjacent development is likely to obstruct the nails as in Figure 4.19.

Existing services may need to be diverted to allow the soil nails to be installed without disrupting nearby infrastructure. If services are to be retained or installed within a soil-nailed slope, some modification may be needed to make sure that the services are not affected by slope deformations or nail installation.

Vegetation will generally need to be removed to allow for excavation and soil nail installation. Where trees are required to be retained, localized over-steepening and changes in the nail spacing and length may be required to support the ground.

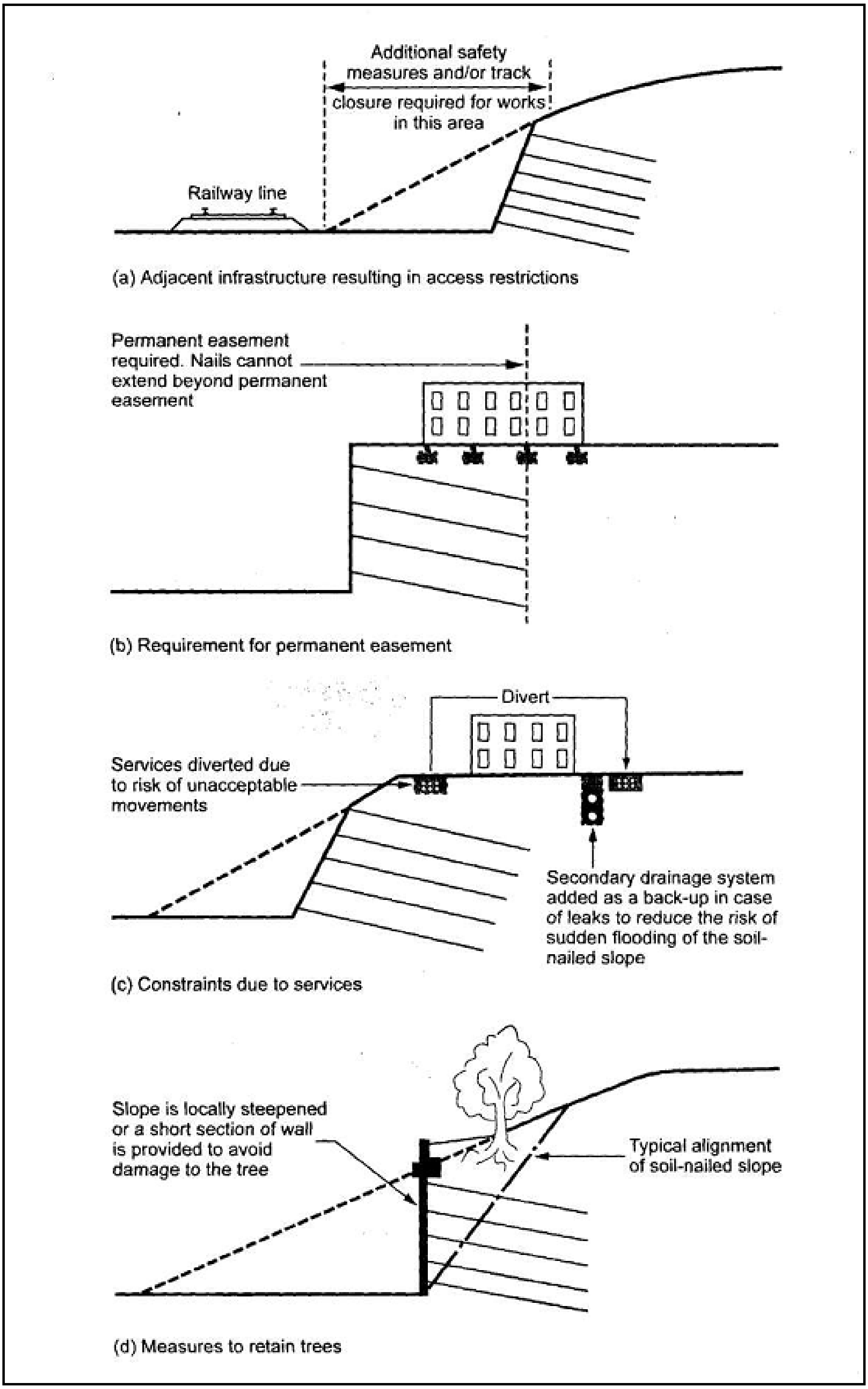


Figure 4.19. Examples of site constraints (Phear *et al*., 2005)

4.9.6. Deformation

An essential and inevitable aspect of the performance of a soil-nailed slope or wall is the amount that it deforms during its design life and whether this can affect adjacent features. Limiting the movements of soil-nailed walls or slopes may be vital in some situations, especially where there are buildings or infrastructure immediately adjacent.

Deformations of soil-nailed walls and steep slopes are usually predicted using empirical correlations such as Clouterre (1991). These correlations, which are based on monitoring results in different soil conditions, typically predict lateral deformations between 0.1 and 0.3 per cent of wall height.

Lateral earth pressure and surcharge loading are not the sole reason for wall movements. Base heave, excessive pore pressures, swelling of plastic clays and seismic loads may cause movements in addition to those caused by static loads and may dictate the final design scheme.

Few published data are available on the deformations that occur as a result of the stabilisation of existing slopes by soil nailing. Vegetation plays a profound role in the deformation of old embankments, especially if composed of high-plasticity clays (O'Brien *et al*., 2004). If post-construction deformations of less than 30 mm are required, soil nailing may not be suitable on its own.

4.9.7. Design Life

The design life should be established based on the client's requirements and whole-life versus capital cost considerations, such as durability requirements, maintenance requirements, consequences of failure, such as proximity of neighboring structures and/or critical infrastructure, ground aggressivity conditions, site access and mobilisation costs, client objectives. Excavations for basement construction in densely built-up urban areas or excavations adjacent to railways, highways and major services are examples of applications that may have a short design life but with demanding design requirements. These requirements may be as robust as many longer design life applications.

4.9.8. Geometry

The geometry of the slope or wall to be soil-nailed is fundamental to the development of the conceptual design and needs to be established at an early stage. The available space at the top and bottom of a slope and the slope angle are important considerations for assessing buildability issues. The geometric parameters are needed to develop all soil nailing designs are the levels at the top and bottom of the slope or wall, and thus the retained height, the slope of the ground above or below the soil-nailed structure, the angle of the face of the soil-nailed structure, the geometry in relation to the geological model, the distance between the facing and the site boundary, the space at the toe of the slope, for plant access, the location of surcharges and the other topographic features that will affect geometry, such as services, significant vegetation and adjacent structures.

4.9.9. Layout and Spacing of Nails

The conceptual design concludes with the preliminary layout, angle of installation, and length of the nails. The most important factors that influence these are ground strength, height of face, angle of face, type of nail (drilled and grouted, or driven), unit pullout resistance, environmental constraints, facing type (rigid or flexible).

Where new slopes are often of considerable height a combination of nails and intermediate rows of pre-stressed anchors is sometimes used. This allows the nails to be of lengths far less than the slope height, while the longer anchors ensure overall stability and, by pre-stressing, a reduction in the overall movement.

The spacing of the soil nails should reflect the choice of facing type as well as overall stability requirements. When nail spacings exceed one nail per 6 m^2 of hard facing or one nail per 2-4 m^2 of flexible facing (depending on soil type), the soil-nailed ground will no longer behave as a coherent reinforced soil block, and the nails will start to behave as individual elements. Therefore maximum horizontal and vertical nail spacings are typically in the range of 1.0-2.0 m (Phear *et al*., 2005).

4.9.10. Nail Orientation

Generally, nails are installed in rows at a slight inclination below the horizontal of between 5° and 20°. For grouted nails, this is to permit gravity installation of the grout. The theoretical tensile efficiency of the nails decreases significantly with increased downward inclination. Johnson *et al*. (2002) showed that for a nail intersecting a failure plane inclined at 60° to the horizontal in a soil with an internal friction angle, ø' of 25°, the most effective (100 per cent) nail inclination was -35° above the horizontal. Effectiveness continued to decrease until at a nail declination of 55° from the horizontal the nail had no effect. These situations are shown in Figure 4.20.

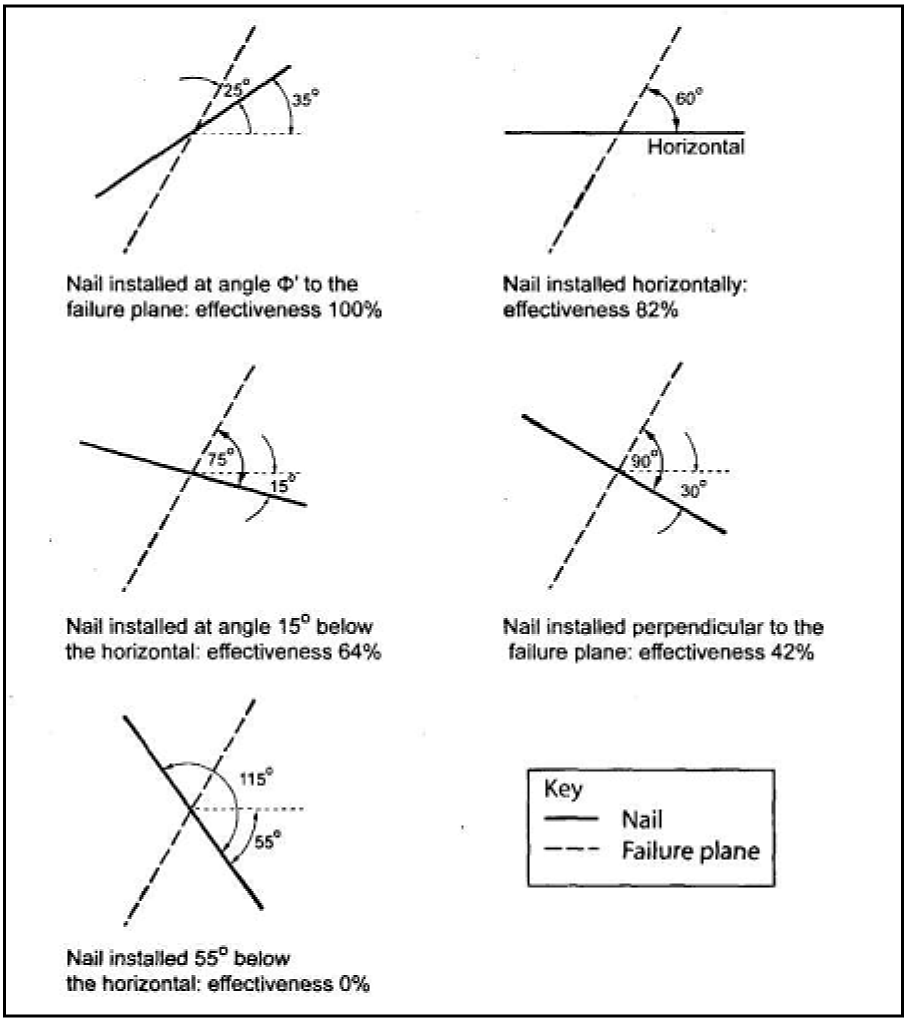


Figure 4.20. Theoretical tensile efficiency of nails installed at various inclinations (Johnson *et al*., 2002)

Figure 4.21 shows the above analysis applied to a 6 m-high slope reinforced with a single 6 m-long nail. The nail installed at the optimum angle has a short (2.3 m) length in the resistant zone and little (1.2 m) depth of overburden. Nails installed horizontally or inclined downwards have a length in the resistant zone of 4.2-4.4 m, and an average overburden of 4-5.9 m. Although the nail installed at 15° below the horizontal has an efficiency of 64 per cent of the nail installed at the optimum angle, it has nearly twice the length in the resistant zone and more than four times the average overburden. Based on the length in the resistant zone alone, the nail inclined slightly downwards is actually more effective. If pullout is considered to be a function of overburden it is four times more effective. However, as the nail angle becomes steeper than 15° the efficiency rapidly decreases without any increase in pullout length or significant increase in overburden.

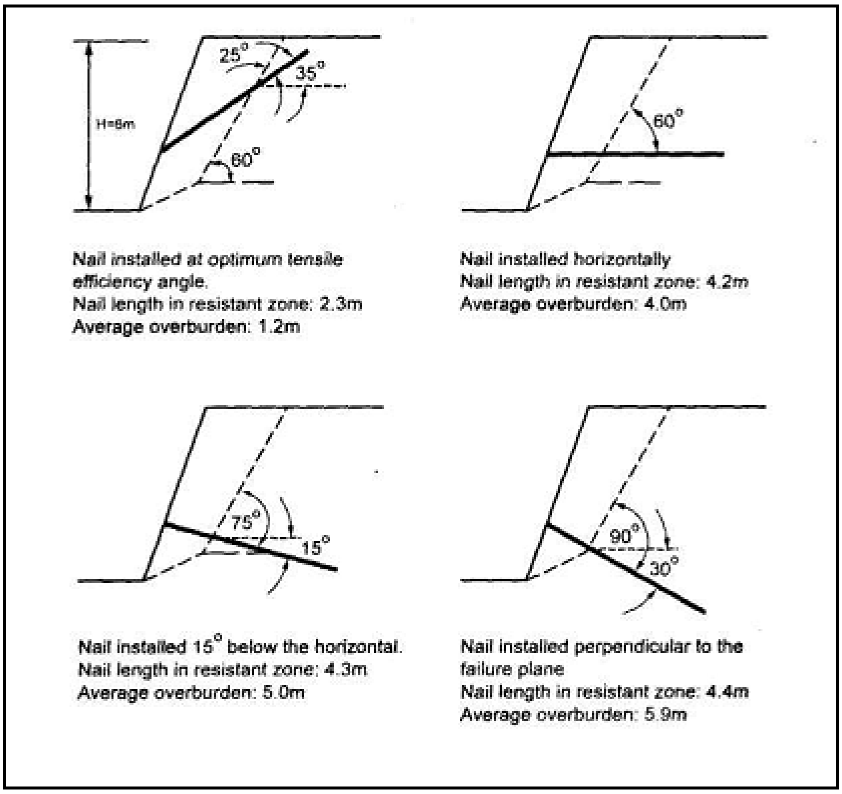


Figure 4.21. Nails installed at different efficiency levels, showing corresponding overburden and length in the resistant zone (Johnson *et al*., 2002)

Based on the above theoretical analysis, it would appear that the optimum design angle is between about 10° and 15° below the horizontal.

4.9.11. Preliminary Assessment of Soil Nail Length

Three approaches are given here to enable designers to make preliminary predictions for the sizing of soil nails at conceptual design stage. These empirical methods and design charts should be used with caution because they do not take account of potential underlying geological features that might affect stability. Also, real design situations usually have heterogeneous ground conditions, variable nail lengths and loading conditions and different ultimate pullout resistances at each nail level because of varying soil stratigraphy.

Bruce and Jewell (1987) derived four parameters to allow comparison between the designs of different projects. These are length ratio (L_r), bond ratio (B_r), strength ratio (S_r) and performance ratio (P_r).

$$L_r = \frac{Maximum\ Nail\ Length\ (L)}{Excavation\ Height\ (H)} \tag{4.2}$$

$$B_r = \frac{Hole\ Diameter\ (D) \text{ x } Nail\ Length\ (L)}{Nail\ Spacing\ (S)} \tag{4.3}$$

$$S_r = \frac{Nail\ Diameter^2\ (D^2)}{Nail\ Spacing\ (S)} \tag{4.4}$$

$$P_r = \frac{Lateral\ Displacement\ (\delta)}{Excavation\ Height\ (H)} \tag{4.5}$$

Gassler and Gudehus (1981) investigated a number of potential failure mechanisms, including single wedges, two-part wedges and slip circles for a variety of slope angles and soil friction angles. FHWA (2003) also contains some design charts for soil-nailed walls.

Soil-nailed walls and steep slopes can be analyzed simplistically using simple or two-part wedge approaches. Sheahan and Ho (2003) presented a simplified simple wedge approach, which allows soil-nailed walls and steep slopes to be analyzed using a spreadsheet, as shown in Figure 4.22. There are several other available methods and commercially available computer programs that use simple or two-part wedge methods and that are suitable for conceptual design.

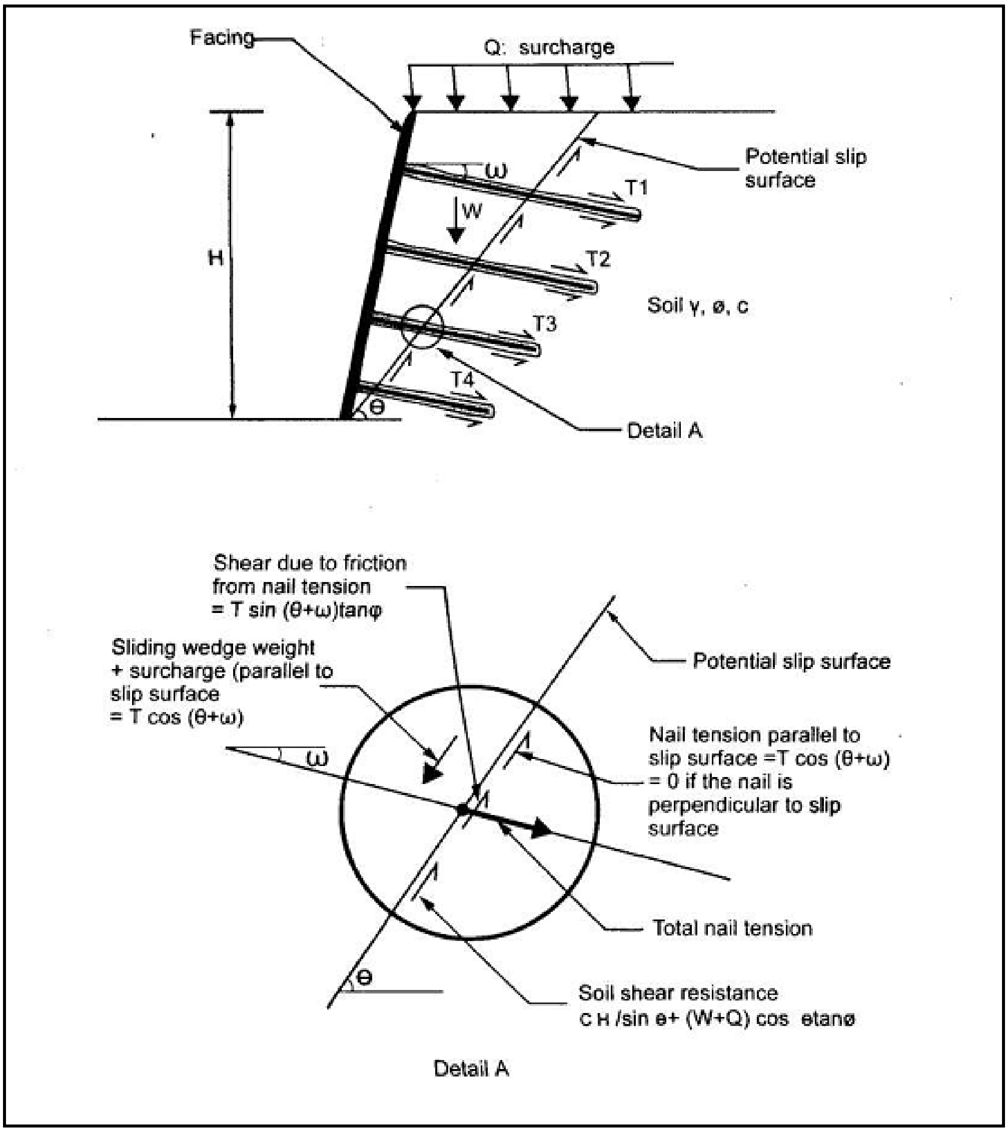


Figure 4.22. Simplified wedge analysis (Sheahan and Ho, 2003)

4.9.12. Head Plates

The correct sizing of head plates is important so that they do not fail through insufficient bearing capacity. For slopes with flexible facings, head plates play an important role in promoting arching between the nails.

4.9.13. Facing

The selection and detailing of an appropriate facing for soil-nailed slopes and walls is just as important as the design of the soil nails themselves and is fundamental to the performance of the soil-nailed slope. Usually a facing is required and its selection needs to consider the site constraints, and environmental and aesthetic requirements.

The major role of the facing is to stabilize the surface (and near-surface depth) of the ground between the nails. It provides lateral confinement for the retained soil between the nail head locations. There are three commonly used facing types as soft facings, flexible structural facings and hard structural facings.

Soft facings perform no long-term role but provide stability while vegetation becomes established. Their primary purpose is to retain the vegetation layer and topsoil and to prevent surface erosion. Typically they may be used on structures with a relatively shallow slope face. Materials commonly used for this purpose are geogrids, cellular geofabrics, geosynthetic sheet, light metallic mesh/fabric, or degradable coir mats.

Flexible structural facings provide long-term stability of the face of the soil-nailed structure by transfer of the soil load from the soil nails to the nail heads. The facing materials allow greater soil movement and minor bulging between the head plates should be expected, although this reduces with closer nail spacings. Materials used commonly comprise coated metallic meshes appropriately designed, in conjunction with the head plates, for the structural loads and durability requirements.

Hard structural facings perform the same function as the flexible structural facings but with less deformation. They generally comprise sprayed concrete reinforced with steel mesh. Most early soil-nailed structures were faced in this way. Other structural facing materials include conventional cast-in-situ concrete or pre-cast concrete panels. Hard structural facings are often used where steep, or vertical, soil-nailed slopes are required because of the face loading to be resisted. Unlike flexible structural facings, which usually are permeable, water pressures can readily build up behind the hard structural facing, so weep holes need to be included within the facing and/or a drainage system installed behind the facing.

4.9.14. Drainage

For most earth retaining structures, groundwater represents a major hazard for soil-nailed slopes and walls. It can take the form of surface water flows, groundwater inflows or the build-up of water pressures beyond those anticipated. Appropriate drainage should be installed to manage and control water flows and suitable whole-life monitoring and maintenance will need to be carried out. Typical drainage types are shown in Figure 4.23 and may be categorized as surface water interceptors, slope face drainage, sub-surface drainage and weep holes through facings.

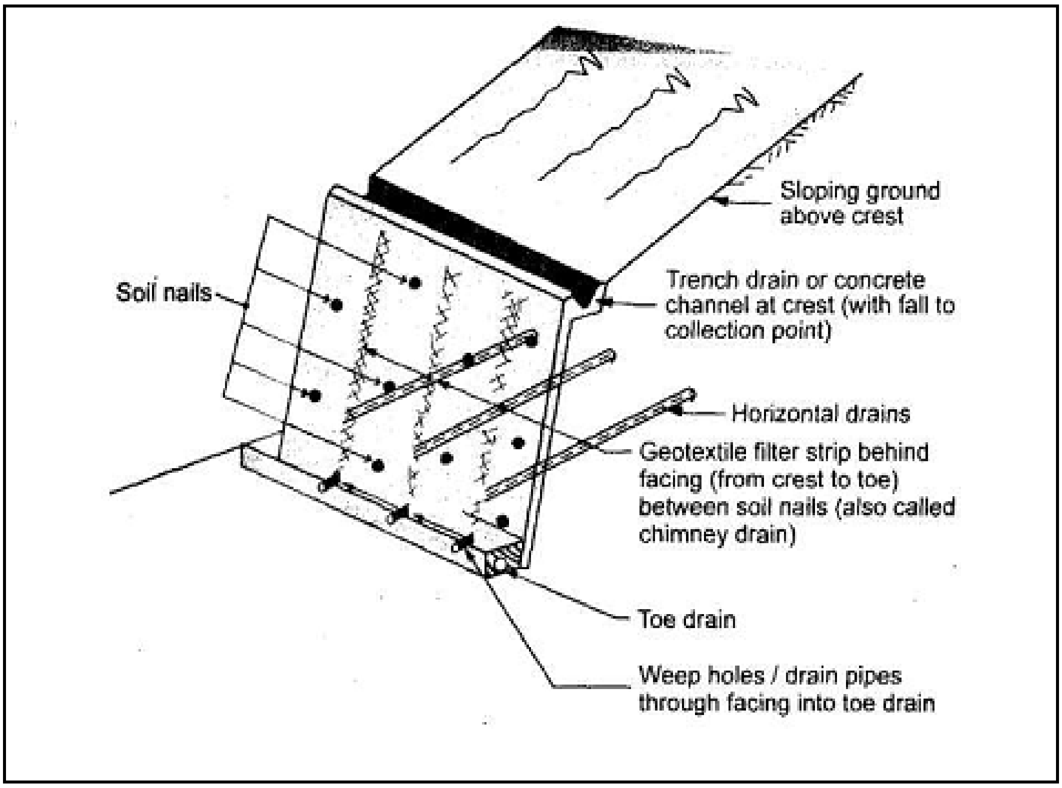


Figure 4.23. Typical types of drainage for soil nailing (Phear *et al*., 2005)

Surface water interceptors collect and manage surface water runoff. Uncontrolled surface water runoff can cause face erosion and instability during and after rainfall depending upon the catchment area and intensity of rainfall. During the construction of a soil-nailed structure an excavated soil face will be significantly affected, particularly in non-cohesive soils or those of marginal stability, as washout and instability are unrestrained by facing or vegetation. Types of surface water drainage include lined drainage channels, French drains, bunds, sumps and appropriate pavement drainage systems.

Slope face drainage manages flows out of the soil face, preventing localized erosion and instability of the face and localized water pressure build-up behind the facing. Most types of slope face drainage are connected to a collector drain running along the excavation below the formation level. Where a permeable facing material is adopted, French drain-type systems can be used within the slope face. French drains are not generally used where the slope face is steeper than about 35° to the horizontal because the slope angle is too steep for their practical installation. The alignment of the drains with the nails needs to be considered.

Subsurface drainage, also known as horizontal drains, is designed to reduce groundwater pressures and target perched groundwater within and behind the soil-nailed structure. The drainage boreholes are normally drilled upwards at 5-10° to the horizontal to facilitate gravitational flow. The drainage boreholes can often be formed using the same plant used for soil nail installation. A slotted pipe of 40 mm minimum internal diameter and a geofabric, to prevent loss of fines from the soil, is commonly used. Any joints should be secure and watertight. An example of a horizontal drain for a steep soil-nailed slope is shown in Figure 4.24. If high flows from the sub-surface drainage are predicted, then soil nailing is unlikely to be an appropriate solution and other earth retaining techniques may be better. Flows should be recorded and compared with those predicted.

Weep holes are generally 300-400 mm-long pipes, of 35 mm diameter or greater, that are cast into impermeable facing materials such as sprayed concrete to prevent the build-up of water pressures behind the facing. They are inclined downwards towards the front of the wall to allow the free flow of water. Where the weep holes are in direct contact with the soil they should be lined or wrapped with a geotextile filter fabric. As flows are normally small, weep holes are usually not directly connected to a collector system and flows arc collected by the surface water interceptor drainage at the toe of the soil-nailed structure.

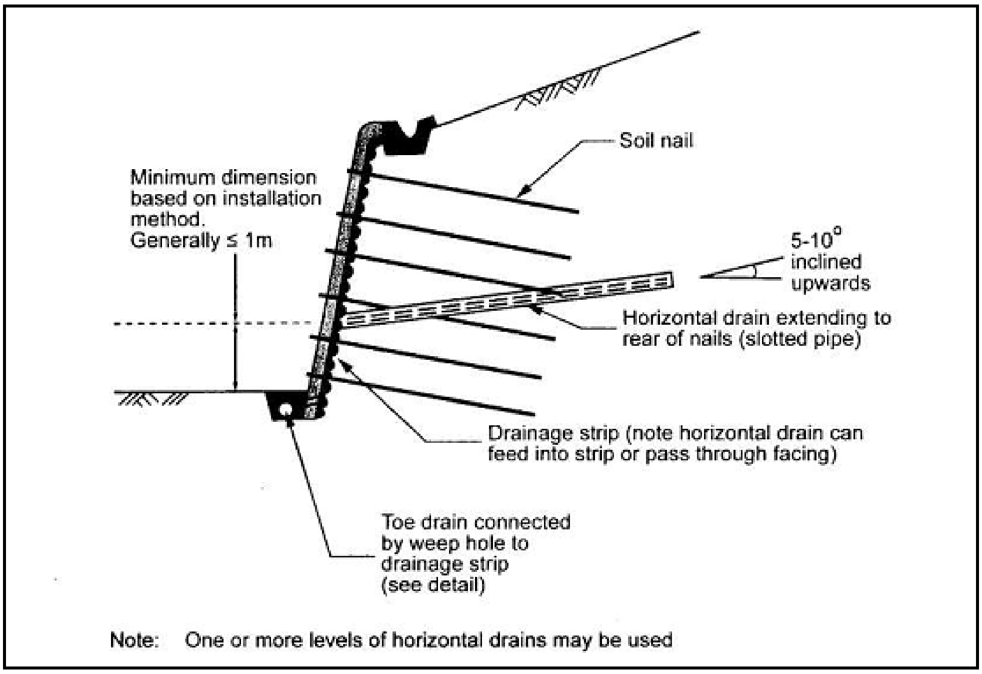


Figure 4.24. Example of a horizontal drain and a weep hole in a steep soil-nailed slope (Phear *et al*., 2005)

4.10. Detailed Design of Soil Nailing

Conceptual design is only the first stage in any project involving soil-nailed walls or slopes. Whatever the method used for conceptual design, a detailed design should always be prepared before construction. The detailed design of soil-nailed slopes or walls is based on information about the soil, groundwater conditions, loads, geometry and type of soil nail to be used. Detailed design is undertaken to satisfy equilibrium of forces and moments (strength and stability), to limit displacements (serviceability) and to maintain these performance criteria throughout the specified design life (durability). The preliminary design procedure should not replace the findings and results obtained with the final design presented herein. The steps of the final design (Phear *et al*., 2005) are described as follows.

4.10.1. Step 1: Geometry and Design Cross-sections

A detailed topographic survey and services search should be used to confirm or amend the conceptual design geometry. The designer has to decide how many cross-sections should be analyzed. Typically, cross-sections are selected at each significant change of slope height, slope angle and/or ground conditions. The geometrical parameters used to define slopes are shown in Figure 4.25.

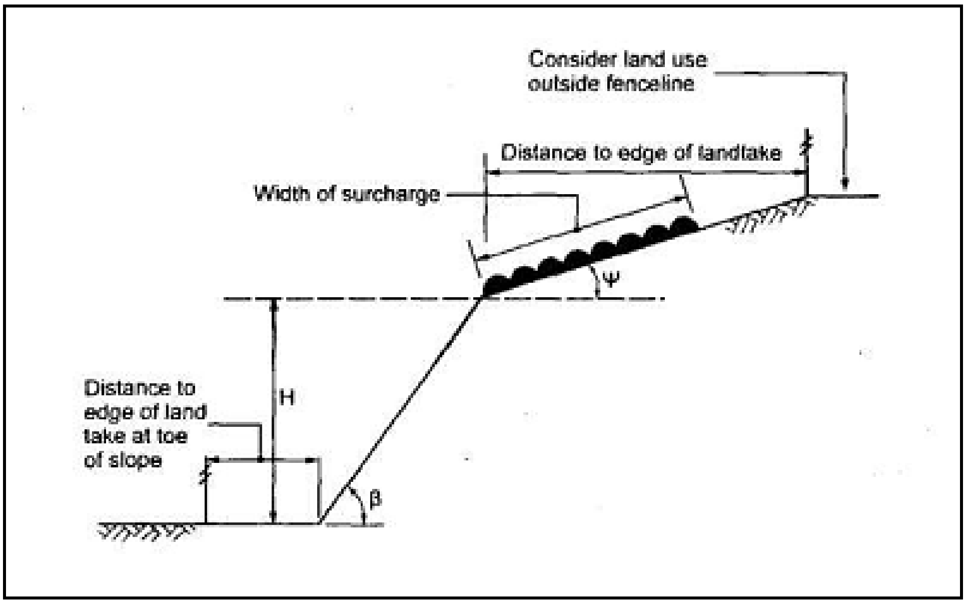


Figure 4.25. Geometrical parameters used to define slopes (Phear *et al*., 2005)

4.10.2. Step 2: Surcharges and Loads

The design of all slopes and walls should allow for a surcharge on the surface of the retained ground or top of the slope. This allowance includes variation in ground levels, construction surcharges such as plant or stacked construction materials, permanent surcharges such as buildings, roads, railway tracks or large trees, transient surcharges such as vehicle loading (live load) or stored goods (dead load).

4.10.3. Step 3: Ground Model

When producing the detailed design it is essential to have information about the ground stratigraphy and the properties of each soil type. It is particularly important to identify strata, lenses or areas of the slope exhibiting different soil characteristics. Where strata thicknesses vary in close succession, a sensitivity analysis may be required.

4.10.4. Step 4: Groundwater Profile

The assessment and control of groundwater are fundamental aspects of the design of all soil-nailed slopes and walls. Pore water pressures affect the stability of a soil-nailed slope or wall. Higher pore water pressures require a higher restoring force to maintain the stability of a potential failure zone, and they also reduce the effective stress acting on the nail/ground interface along which pullout resistance is generated. Both effects require increased nail length to maintain stability for a given nail diameter or nail spacing.

Nearly all designs for soil-nailed slopes and walls use effective stress parameters. Such designs require knowledge of the likely pore water pressure regime in the ground both at the construction stage and in the longer term. The construction case is often less onerous than that for the long-term situation because of short-term beneficial pore suctions (apparent cohesion). It is important to check the stability of the slope for the worst (ie long-term) case, because the pore suctions dissipate over time.

The methods for including pore water pressures in a stability analysis are often imprecise. An appropriate value for the pore water pressure parameter (r_u) may be estimated and included in an analysis, but this cannot easily account for high pore water pressure from surface infiltration. Modern slope stability software usually allows a grid of pore pressure points to be specified, and this enables complex groundwater conditions to be modeled.

4.10.5. Step 5: Design of Drainage System

Drainage will generally be cost-effective in helping to stabilize all slopes including nailed slopes. After evaluating the surface water and groundwater conditions at the site, the installation of surface water control and groundwater control systems such as geocomposite drains, weep holes and pipe drains should be considered to avoid construction difficulties and/or long-term performance deterioration.

4.10.6. Step 6: Design Codes and Design Methods

In soil nailing design the most commonly used manuals are Eurocodes - Eurocode 7 (BS EN 1997-1:2004) Geotechnical design. Part 1: General rules and prEN 14490:2002 Execution of special geotechnical works - soil nailing, US Department of Transportation Federal Highway Administration (FHWA,1998) and French National Research Project Clouterre (Clouterre, 1991). The factoring approaches used by this codes are summarized in Table 4.3. In Table 4.3, FHWA Service Load Design uses lumped factors while the other codes use partial factors.

Table 4.3. Factors recommended by soil nailing design codes

	FHWA	**Clouterre**	**Eurocode 7 (Design Approach 1-1)**	**Eurocode 7 (Design Approach 1-2)**
Load factors				
Soil weight	1.00	1.05/0.95	1.00 (M1)	1.00 (M2)
Dead surcharge	1.00	1.20	1.35 (A1)	1.00 (M2)
Live surcharge	1.00	1.33	1.50 (A1)	1.30 (M2)
Soil parameters				
$\tan \phi'$	1.00	0.77-0.83	1.00 (M1)	0.80 (M2)
c'	1.00	0.61-0.67	1.00 (M1)	0.80 (M2)
c_u	1.00	0.71-0.77	1.00 (M1)	0.71 (M2)
Pore water pressures	1.00	1.00	Most unfavourable	Most unfavourable
Soil nail pullout	2.00	1.40-1.50 (tests) 1.80-1.90 (charts)	1.5 on characteristic multiplied by extra factor of 1.1 to 1.5 (or more for high-plasticity clays) for testing variability, long-term strength reduction, etc	1.5 on characteristic multiplied by extra factor of 1.1 to 1.5 (or more for high-plasticity clays) for testing variability, long-term strength reduction, etc
Testing		Required	Required	Required
Overall stability	1.35	1.125	1.00	1.00

4.10.7. Step 7: Characteristic Soil Strengths

Eurocode 7 uses characteristic values of soil and rock parameters. The characteristic values for ultimate limit state calculations have to be estimated such that the probability of failure will be acceptably low. The characteristic value is a cautious estimate of the mean of the parameter values from tests on material from the zone of ground governing the behaviour of the slope at a limit state. If sufficient information is available, the characteristic value may be derived using statistical methods. The information obtained in the site investigation should be interpreted and used to obtain the characteristic soil strengths.

4.10.8. Step 8: Determination of Design Soil Parameters and Design Loads

The design approaches defined in Eurocode 7 Geotechnical Design are summarized in Table 4.4. For both serviceability limit state and ultimate limit state cases, design soil strengths are obtained by dividing the characteristic strengths by a partial factor as follows:

$$\textit{Design Strength} = \frac{\textit{Characteristic Strength}}{\textit{Partial Factor}} \tag{4.6}$$

For serviceability limit states, all partial factors for soil strength are 1.0. For ultimate limit states, the relevant partial factors are summarized in Table 4.4. Eurocode 7 (BS EN 1997-1:2004) defines the following actions in a similar manner to load cases as direct action, where the load is directly applied to the structure and indirect action, where the load indirectly acts on the structure (such as deformations, temperature changes and seismic loads). Actions are subdivided into different categories such as permanent, variable, accidental and construction. For serviceability limit states, all partial factors applied to actions are 1.0. Limit equilibrium models do not allow serviceability deformations to be assessed. For Design Approach 1 and ultimate limit states, actions are multiplied, and so increased, by the partial factors in Table 4.4 to obtain design actions. Only the actions that can occur simultaneously should be combined. For accidental situations, all partial factors for actions are equal to 1.0. The basic principle is to combine permanent actions, a dominant variable action and combinations of values for other variable actions.

Table 4.4. Design approaches and partial factors in Eurocode 7

	Design Approach 1-1 (A1+M1+R1)	**Design Approach 1-2 (A2+M2+R2)**
Actions (A) Multiply action by partial factor		
Permanent unfavourable	1.35	1.00
Permanent favourable	1.00	1.00
Variable unfavourable	1.50	1.30
Variable favourable	-	-
Materials (M) Divide material strength by partial factor		
$\tan \phi'$	1.00	1.25
c'	1.00	1.25
c_u	1.00	1.40
q_u	1.00	1.40
Bulk density	1.00	1.00
Resistances (R)		
Overall stability	1.00	1.00
Bearing capacity	1.00	1.00
Sliding	1.00	1.00

4.10.9. Step 9: Internal Stability and Pullout Resistance

Internal failure modes refer to failure in load transfer mechanisms between the soil, the grout, and the nail. The design nail resistance is the lowest of the pullout resistance between the soil nail and the ground or the pullout resistance between the nail tendon and the grout or the rupture strength of the soil nail tendon itself. The governing criterion is usually either the first or the last, provided that consistent good-quality grout is used and it is allowed to gain sufficient strength before loading. If the aim is to achieve a ductile slope failure mechanism, first criterion is preferred.

The pullout resistance between the nail tendon and the grout for grouted nails is derived mainly from mechanical interlocking of grout with the protrusions and depressions in the surface of the nail tendon. Mechanical interlocking provides significant resistance when threaded tendons are used and is negligible with smooth tendons. If threaded tendons are used then this criterion is rarely critical.

The ability of a soil nail to generate sufficient pullout resistance (soil/nail) is of fundamental importance to the stability of a soil-nailed slope or wall. The ultimate pullout resistance of a soil nail is a function of the soil type, surface roughness, drilling or installation technique, time that the drillhole is left open and ungrouted, grout pressure, nail diameter, nail length in the active zone, nail length in the resistant zone, elasticity of the tendon time for soils susceptible to creep action, rate at which test load is applied and confining stress and presence of groundwater and its effect on effective stress and radial stress.

Confirmation of ultimate pullout resistance is particularly important since a large proportion of failures of soil-nailed slopes and walls results from overestimation of the pullout resistance of the nails which results from either to overestimation of bond stress or to inadequate nail length. Site pullout tests should be considered to be an extension of the design process. There are five methods of determining the pullout resistance, as discussed below.

The first method to determine the pullout resistance is using empirical correlations and charts. Data for ground conditions by Elias and Juran (1991) and FHWA (1998) are presented in Table 4.5. All these data are for drilled and grouted soil nails using various construction methods in various soil types.

The second method is carrying out pullout tests. This may not always be practicable for small soil nailing sites, where, if the design is thorough, tests at an early stage on working nails to 1.5 times of design load should be sufficient. The main purpose of such tests is to measure the ultimate pullout resistance, P_{ult}. It is recommended that a set of correction factors, ξ, to be applied to the ultimate pullout resistance values obtained from site tests to reduce them to characteristic values and these are summarized in Table 4.6.

Table 4.5. Ultimate bond stress for various construction methods and soil types (Elias and Juran, 1991; FHWA, 1998)

Construction Method	**Soil Type**	**Ultimate Bond Stress (kN/m2)**
Augered	Loess	25-75
	Soft clay	20-30
	Stiff to hard clay	40-60
	Clayey silt	40-100
	Calcareous sandy clay	90-140
	Silty sand fill	15-20
Open hole	Non-plastic silt	20-30
	Medium-dense sand /silty sand/sandy silt	50-75
	Dense silty sand and gravel	80-100
	Very dense silty sand and gravel	120-240
	Stiff clay	40-60
	Stiff clayey silt	40-100
	Stiff sandy clay	50-100
Rotary-drilled	Marl/limestone	300-400
	Soft dolomite	400-600
	Weathered sandstone	200-300
	Weathered shale	100-150
	Weathered schist	100-175
	Basalt	500-600
	Silty sand	100-150
	Silt	60-75
Driven casing	Dense sand/gravel	180-210
	Sandy colluvium	70-150
	Clayey colluvium	40-75
Jet-grouted	Sand	380
	Sand/gravel	700

Table 4.6. Correction factors for pullout test results to obtain characteristic values (Phear *et al.*, 2005)

Number of pullout tests	**1**	**2**	**3 or more**
ξ, based on mean of test results	N/A	1.35	1.30
ξ, based on lowest test results	N/A	1.25	1.10
ξ, based on test result	1.50	N/A	N/A

A partial factor (γ_p) to account for natural variation in the nail properties such as surface area, normal stress, surface roughness, and reduction in bond resistance over time should be applied to the characteristic pullout resistance to obtain the design pullout resistance. Appropriate values of these partial factors are 1.25 for temporary soil nails and 1.50 for permanent soil nails in soils other than high-plasticity clays. A partial factor of 2.0 should be used for permanent soil nails in high-plasticity clays. The design bond resistance (T_d) per meter of bond length of nail is:

$$T_d = \frac{P_{ult}}{\xi L_b \gamma_p} \tag{4.7}$$

or the design bond stress (τ_d) is:

$$\tau_d = \frac{P_{ult}}{\xi L_b \gamma_p \pi d} \tag{4.8}$$

The third method of determining the pullout resistance is estimation of ultimate pullout resistance using the undrained shear strength and an adhesion factor, α, in cohesive soils in a similar way to piles in clay:

$$P_{ult} = \gamma_p \pi d L_b \alpha c_u \tag{4.9}$$

A cautious value of undrained shear strength should be adopted, and the following issues should be borne in mind:

- undrained shear strength is not a unique soil property and is dependent on the method and rate of testing
- undrained shear strengths from triaxial tests on samples of 38 mm diameter overestimate the undrained shear strength of the soil mass and 102 mm diameter samples should be used
- •the value of α is also dependent on the rate of testing and the empirical database used for its derivation.

This method will tend to give an upper bound estimate of ultimate pullout resistance in cohesive soils. It is probably not appropriate where the nails need to be designed for long-term conditions, and particularly where excavation will reduce the strength in the long term.

The fourth method of calculating the pullout resistance is using an effective stress design method in granular soils. Such methods can also be used to calculate pullout resistances in cohesive soils. For reasonable confidence of long-term performance, it is probably most appropriate to calculate the pullout resistance of soil nails in cohesive soils using this approach.

Effective stress methods are thought to underestimate the pullout resistance in granular soils. One reason for this is a well-documented effect called constrained dilation, where the soil tries to dilate to accommodate the movement of the nail through the ground as it mobilizes tension. This dilation is prevented by the surrounding ground providing that it is not in a loose condition and the radial stress on the nail increases until some of the soil grains start to crush or passive failure occurs in the surrounding soil allowing the nail to move. Constrained dilation is present to a much lesser extent in cohesive soils.

The short term pullout resistance of a soil nail installed in clay may be higher than that attainable in the long-term because pore water pressures during construction and testing are generally lower than those encountered during the service life of the wall or slope. It is also possible that the movement and stresses generated during testing could produce temporary pore water suctions locally, leading to higher effective stresses on the soil nail and enhanced pullout resistance (Johnson el al, 2002).

The typical form of equation is:

$$P_{ult} = \gamma_p \pi d \left(K\sigma_v' \tan\phi_d' + c_d' \right) L_b \tag{4.10}$$

The fifth method is the pressuremeter testing. The ultimate bond stress, τ_{ult} can be derived from the results of pressuremeter tests carried out during the site investigation using the following correlation (FH WA, 2003):

$$\tau_{ult} = 14P_L(6 - P_L) \tag{4.10}$$

where P_L has units of MPa and τ_{ult} in kN/m^2.

Clouterre (1991) presents a series of design charts that can be used to estimate the ultimate unit skin friction of nails for different construction techniques and soil types. These charts correlate ultimate unit skin friction of nails against the limit pressure obtained in a pressuremeter test.

In soil nail design it is usually assumed that the pullout resistance is directly proportional to the length of the nail, this implies that it will be uniform along the length of the nail for a constant cover depth. Similarly in testing it is generally assumed that the distribution of load in the serviceability condition along the nail is uniform, yet opposite in direction in the active and resistant zones respectively. This is unlikely to be the case, since the development of pullout resistance is probably much more complex than this and is progressive (Barley *et al*., 1997b).

To summarize, soil nail pullout resistance should be assessed as follows:

(i) Estimate ultimate pullout resistance from tables and charts.
(ii) For cohesive soils calculate upper bound ultimate pullout resistance using undrained shear strength method and lower bound using effective stress methods. For granular soils calculate ultimate pullout resistance using effective stress methods.
(iii) Then validate the design with design investigation tests or suitability tests to failure on sacrificial nails.

On small sites, where design investigation tests may not be practicable, pullout tests can be carried out on working nails to 1.5 x design load, but these give much less information than design investigation tests especially if the nail tendon strength limits the test load.

The design tensile strength of the nail tendon (T_{nd}) is calculated as follows:

$$T_{nd} = \frac{f_y A_s}{\gamma_s} \tag{4.11}$$

γ_s, a partial factor for reinforcement material, is 1.05 for steel in tension and about 1.3 for geosynthetics. A_s and f_y should be the values applicable at the end of the design life.

4.10.10. Step 10: Internal and External Stability Checks

As with all earth retention systems and slopes, the stability of the proposed works needs to be assessed and confirmed. In general, a limit equilibrium analysis is used because this is relatively simple and gives satisfactory results. Numerical modelling methods such as the finite element method may also be used, but these are complex. They also require high-quality site investigation data coupled with careful calibration of the constitutive model to give meaningful and reliable results. The following external failure modes should be considered in the analysis of soil-nailed walls or slopes:

4.10.10.1. Overall Stability. Stability analyses should assess all potential failure surfaces both those that intersect the proposed soil nails (internal stability analyses) and those that pass behind and below the proposed soil nails (external stability analyses). If any of these analyses do not meet the design conditions with respect to pullout resistance, then the proposed nail lengths have to be increased and/or proposed spacings of the nails reduced until the required result is achieved. If the design conditions are not achieved with respect to rupture capacity, then the nail tendon diameter has to be increased and/or the proposed nail spacing reduced. A variety of potential failure mechanisms can be used in stability analyses which shown in Figure 4.26 and include single wedge, two-part wedge, log spiral slip and circular slip.

Gassler and Gudehus (1981) found that in intact clay the geometry of the critical slip surface tends to be circular and in frictional soils the critical slip surface tends to be a two-part wedge. If the slope angle is steep the critical slip surface tends to be a two-part wedge and a circular slip surface is more likely for more gentle slopes. The presence of weaker layers, and particularly pre-existing slip surfaces, should be carefully considered and modeled accordingly.

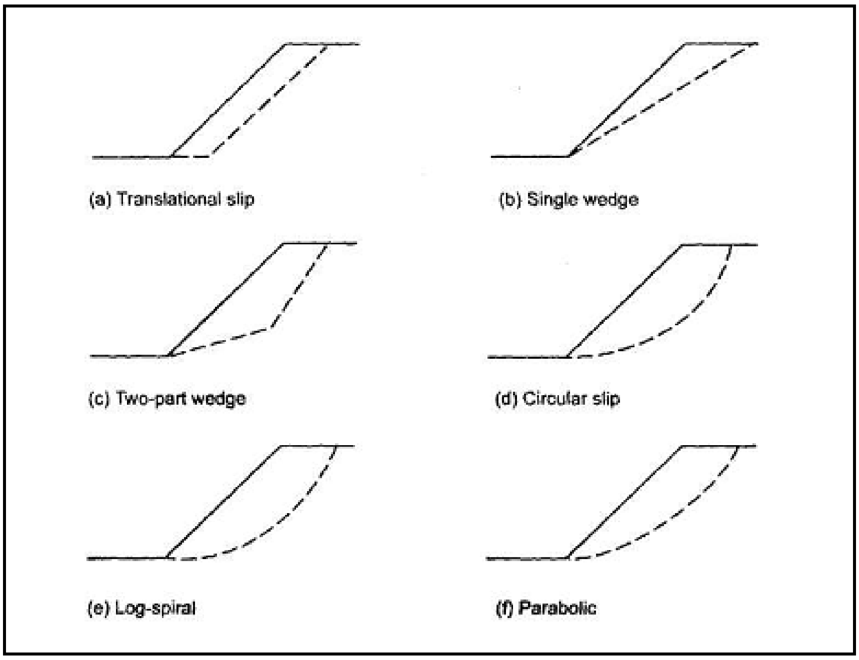


Figure 4.26. Failure surfaces used to assess stability of slopes (Johnson *et al.*, 2002)

Long-established slope stability analysis methods such as Bishop, Janbu, Sarma or others are often used to analyze soil-nailed systems. These methods use a number of slices to analyze the stability of the slope. They analyze the effect of the soil nails by incorporating the nail force at the intersection with the critical slip surface into the equilibrium equations. It is assumed that the nail forces do not modify the interslice forces. The typical approach using the method of slices for circular slip analysis is as follows. For equilibrium:

$$M_D \leq M_{RS} + M_{RR} \tag{4.12}$$

where:

M_D	is the disturbing moment
M_{RS}	is the restoring moment due to shear strength of soil
M_{RR}	is the restoring moment due to presence of soil nails

Using the method of slices and ignoring interslice forces, the disturbing moment for each slice, *i*, summed for *n* slices is given by:

$$M_D = \sum_{i=1}^{n}\left[\left(W_i\gamma_\gamma + q_{DLi}\gamma_{DL}B_i + q_{LLi}\gamma_{LL}B_i\right)\sin\theta_i\right]R_d \tag{4.13}$$

Similarly the restoring moment for *n* slices is given by:

$$M_{RS} = \sum_{i=1}^{n}\left[\frac{\left(c_d{}'b_i + (W_i\gamma_\gamma + q_{DLi}\gamma_{DL}B_i + q_{LLi}\gamma_{LL}B_i - u_iB_i)\tan\phi_d{}'\right)\sec\theta_i}{(1+\tan\phi_d{}'\tan\theta_i)}\right]R_d \tag{4.14}$$

The restoring moment for each nail, *j*, summed for m nails is given by:

$$M_{RR} = \sum_{j=1}^{n}\left(\underbrace{\frac{T_j}{\gamma_p}R_d\sin\left(\theta_j-\omega\right)}_{\text{restoring moment due to tension in nails}} + \underbrace{\frac{V_j}{\gamma_s}R_d\cos\left(\theta_j-\omega\right)}_{\text{restoring moment due to shear in nails}}\right) \tag{4.15}$$

An explanation of the application of partial factors in Equations 4.13, 4.14 and 4.15 is given in Table 4.7 and the notation is shown in Figure 4.27.

Table 4.7. Explanation of application of partial factors (Phear *et al.*, 2005)

Description	**Characteristic Parameter**	**Partial Factor**	**Design Parameter**	**Unit**
Unit weight of soil of slice *i*	γ_{ki}	γ_γ	γ_{ki} x γ_γ	KN/m^3
Tangent of effective stress friction angle	$\tan\phi'_k$	γ_ϕ	$\tan\phi'_k/\gamma_\phi$	-
Effective stress cohesion intercept	c'_k	γ_c	c'_k/γ_c	kPa
Pullout resistance	P_{ult}/ξ	γ_p	$P_{ult}/\xi\gamma_p$	KN
Dead surcharge load on slice *i*	q_{DLi}	γ_{DL}	q_{DLi} x γ_{DL}	kPa
Live surcharge load on slice *i*	q_{LLi}	γ_{LL}	q_{LLi} x γ_{LL}	kPa

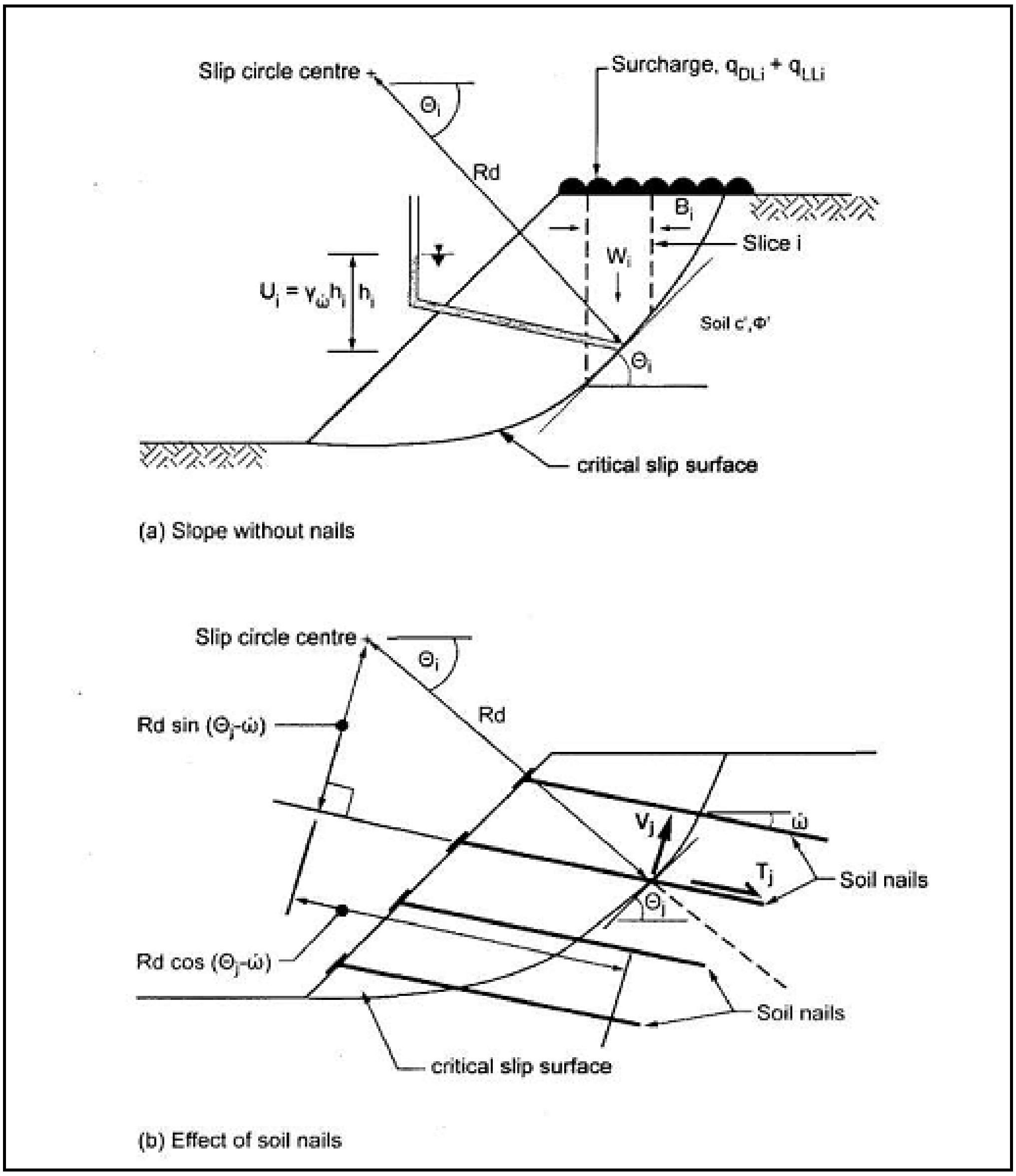


Figure 4.27. Method of slices for circular slip analysis (Phear *et al*., 2005)

4.10.10.2. Sliding Stability. Analysis of sliding stability considers the ability of soil-nailed walls or steep slopes to resist sliding along the base of the soil-nailed block in response to lateral earth pressures and water pressures behind it. Sliding failure may occur when the lateral earth pressures exceed the sliding resistance along the base. Such failures can occur if there is a weak horizontal, or nearly horizontal, seam or zone at or slightly below the toe of the wall or slope.

4.10.10.3. Bearing Capacity. Very rarely, bearing capacity may be a concern when a soil-nailed wall or steep slope is excavated in soft fine-grained soils. Since the soil-nailed block does not extend below the base of the excavation, the unbalanced load caused by the excavation may cause the base of the excavation to heave. This may result in a bearing capacity failure of the foundation of the wall or slope.

4.10.11. Step 11: Design of Facings and Head Plates

4.10.11.1. Design of Head Plates. The head plate should be correctly sized to prevent bearing failure and to promote soil arching and hence to reduce local surface instability between the soil nails. The head plate should also be in good contact with the soil behind it to prevent raveling. If the nails are too far apart and/or the head plates are too small then the soil may fail between the nails.

For shallow slopes, Terzaghi's basic bearing capacity equation for square footings gives a simple preliminary estimate of the plate size required. Note this gives ultimate load whereas the methods below are for design parameters and loads.

$$q_u = 1.3c'N_c + \gamma DN_q + 0.4a\gamma N_\gamma \qquad (4.16)$$

The adequacy of the head plate in bearing, to guard against front face pullout, should be checked by using a lower bound and an upper bound solution, as shown in Figures 4.28. Having determined the plan area of the head plate, the thickness then needs to be calculated to avoid overstressing the head plate in bending.

4.10.11.2. Design of Soft Facings. The primary function of a soft facing is erosion control and to support establishment of vegetation on the face of the slope. With such facings, the nail heads fix the facing to the ground. A soft facing will not be very effective against raveling, will not contribute significantly to slope stabilisation between the nails and will not guarantee the group action and integrity of soil nails. The nails will simply behave individually against any destabilising force. The use of soft facings should therefore be limited to shallow slope angles of up to about 30° to the horizontal.

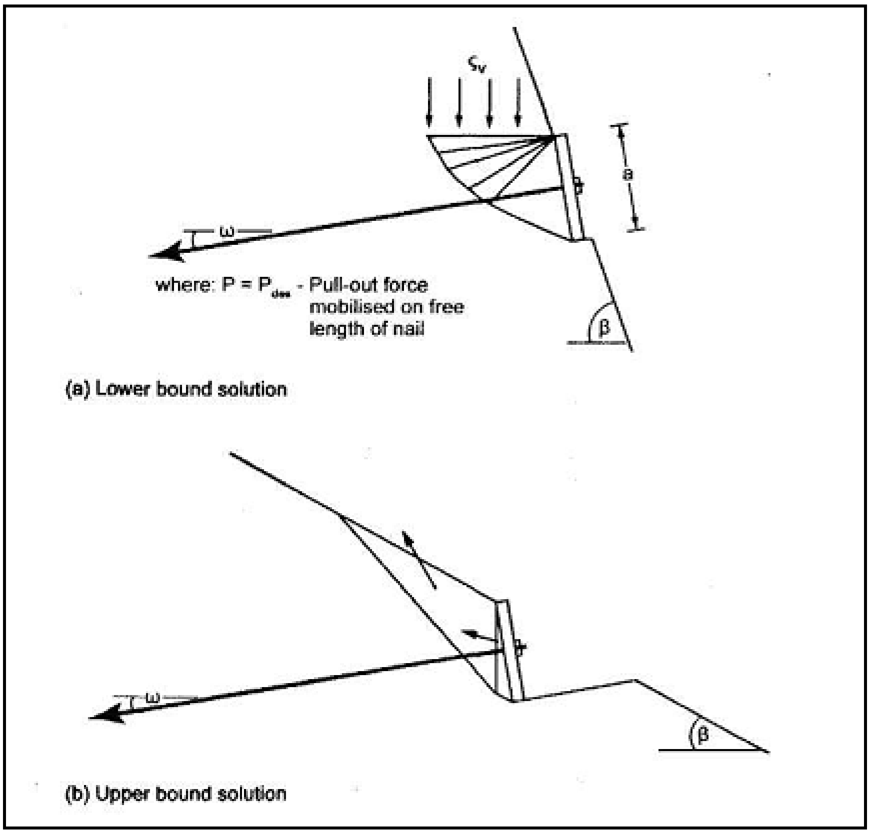


Figure 4.28. Calculation of head plate bearing capacity (UK Highways Agency, 1994)

4.10.11.3. Design of Flexible Facings. Flexible facings are not suitable for large spans between nails. The entire system of flexible facing, head plates and the soil nails themselves needs to be rigorously designed to provide restraint at the slope surface, to provide a restraining force at the nail head and to satisfy the agreed durability and maintenance requirements. Flexible facing design needs to consider nail spacing, soil type, potential deformation mechanisms, head plate dimensions, stiffness of the facing, nail and soil stiffness, groundwater and drainage conditions and steepness of the slope and future role of vegetation. The design considerations should include:

- check the ultimate limit states
- check the punching resistance of the facing
- calculate the head plate sizes based on the punching resistance of the facing
- check that the structural capacity of all connections is adequate
- check durability of the facing material itself and all connections for the design life.

The facing should be designed to resist potential out-of-balance forces, which can be transferred to the facing by failure of three-dimensional blocks between the nails. Such blocks can be conservatively modeled in two dimensions (ignoring side friction) using shallow two-part wedge mechanisms or slip circles, to achieve force equilibrium. Depending on the slope angle, the load on the facing may increase as the height below the top of the slope increases, and may be greatest at the toe of the slope. For medium- and high-plasticity clays, the influence of water-filled tension cracks should also be considered.

The pore water pressures that need to be adopted in these calculations should be carefully considered. For example, for embankment slopes in overconsolidated clay fills, higher pore water pressures can occur within the top 1-1.5 m. In such materials, an average r_u value of 0.25 is often appropriate for shallow slip surfaces in this zone, but may be higher if tension cracks develop. In granular soils, a lower r_u value may be appropriate.

The reinforcement strength adopted needs to be a characteristic strength reduced by partial factors appropriate to the required design life. If the load in the reinforcement is excessive, a grid of short nails should be added in between the other nails to carry some of this load.

The above design method is much simplified but conservative because the way in which a flexible facing performs involves complex interaction between the soil, soil nails and the facing, and is dependent on their relative stiffnesses. For slopes where the consequences of failure are high, it is recommended that the design of the flexible facing should be undertaken using greater rigour such as numerical modelling.

4.10.11.4. Design of Hard Facings. The design sequence for hard facings generally includes the following steps:

- decide on facing type based on performance and aesthetic requirements
- determine the nail head forces
- calculate head plate sizes and/or initial sprayed concrete thickness based on the punching resistance of facing
- check the flexural resistance of facing and reinforcement detailing

- check performance requirements under serviceability limit state conditions (crack width, deflection and durability) that are required for long-term applications.

Since the majority of resisting forces are generated along nails, only some of the maximum tension force in the nail is transferred to the facing. Therefore, soil nailing requires lighter facing elements than other conventional earth retention techniques. Investigation results reported by FHWA indicate that approximately 40-70 per cent of nail service loads can be transferred to the facing. FHWA recommends that 50 per cent of active earth pressure is used over the full height of the slope or wall for design purposes unless the designer has a site-specific value.

4.10.12. Step 12: Predicting Deformations

The main shortcoming of the limit equilibrium design methods is that they do not give a prediction of deformations. They also do not consider the deformation required, to mobilize the resisting forces in the soil and soil nails. These methods cannot therefore provide a thorough description of the contribution of each soil nail to overall stability based on the pattern of deformation behind the slope or wall (FHWA, 2003). Deformations can be predicted approximately using empirical correlations and while these are appropriate in many cases they have limitations.

In situations where more confidence is required, a higher level of analysis should be adopted by using numerical modelling such as finite element and/or finite difference methods. The accuracy of numerical modelling depends on the quality of data acquired, the estimation of in-situ stress and soil stiffness and the availability of good case histories to calibrate numerical models (FHWA, 2003). The stiffness parameters in these models should be adjusted to match the values obtained from actual site monitoring results at an early stage. Observations during construction are essential and cannot be replaced by numerical modelling. Even using numerical modelling, it is still relatively difficult to predict displacements for layered soil stratigraphies and weathered rocks with discontinuity surfaces. The accurate modelling of the grout/soil interface is also difficult, as mobilisation of tension forces is often not directly proportional to facing deflections and/or construction stages.

4.11. Construction Methods

For successful construction of soil-nailed slopes or walls, there should be an integrated design and construction team with efficient lines of communication. The design needs to be confirmed as excavation exposes the actual ground conditions. The design input is not finished until construction is complete, and there is a need for continuous assessment and review until then. The main subjects are safe systems of work, planning, programming and procurement issues, soil nail installation methods, construction of drainage systems and construction of facings.

4.11.1. Planning for Health and Safety

Like all construction and maintenance works, soil nailing must be carried out safely. Arriving at a safe system of work is a process that involves work and decisions by the client, the designer(s), the specifier(s) of the work, and the contractor(s), and is not an activity confined to the construction stage alone.

Before starting the site work, the "principal contractor" must prepare a construction stage health and safety plan, taking into account the information received from the pretender health and safety plan. The plan should be reviewed regularly and revised as appropriate, taking account of experience in addressing the actual health and safety risks encountered on site. It should define a clear chain of responsibility and channels of communication on engineering safety matters.

4.11.2. Programming and Procurement

The formation of a soil-nailed slope or wall comprises the installation of soil nails as well as the construction of drainage and facing components. Where excavation and platform construction are required in conjunction with the construction of the above components, the adoption of good planning and procurement strategies can significantly reduce health and safety, technical and commercial risks (Phear *et al*., 2005).

The relative flexibility of a soil-nailed solution can be used to great effect to overcome changes in ground conditions provided the changes are identified and action is taken. Conversely, the temporary stability requirement of an exposed soil face means that soil nailing works can also be more sensitive to changes in ground conditions.

The works should be programmed in a way that maximizes efficiency, produces a good-quality final product and minimizes delay so that the project is a commercial success. With a staged excavation sequence, the earthworks, nail installation, drainage installation, facing construction etc. all become discontinuous operations. On larger structures it is possible to maintain continuity by dividing the structure into zones with different operations taking place in each zone.

There are many procurement options that may be adopted. The two extremes are construction of the entire soil nailed slope or wall by a single soil nailing contractor on a design-and-build basis which includes excavation, drainage, nail installation and facing construction and build-only subcontracts for the individual work elements.

4.11.3. Soil Nail Installation Techniques

Various soil nail installation techniques are available. The installation method considered should be appropriate for the specific health and safety issues relating to the site and for the site constraints. Generally its selection will be a compromise to satisfy cost, access to site including the available working space and environment, ground conditions particularly temporary soil face and bore stability, durability, environmental issues and speed of construction.

The nail installation methods may be categorized into:

(i) bored and grouted nails
(ii) self-drilled nails
(iii) driven nails.

4.11.3.1. Bored and Grouted Nails. Bored and grouted soil-nailing systems use specific plant to construct a hole into which the tendon and grout are placed. The bore may be unsupported in soils exhibiting sufficient cohesion or supported by temporary casing. In soils of marginal stability, a hollow-stem auger may be used and grout placed as the auger is extracted to give hole stability. Because several bored open hole techniques are commonly available, this is a flexible option that can be used in all ground conditions.

Drilling can be carried out using augers, which mechanically remove the spoil from the hole; these are suitable in most soils. Alternatively, the use of rotary or rotary percussive techniques, using a flushing medium to lift spoil from the hole, can be adopted. These methods are suitable in more competent soils, weak rocks or where natural obstructions are anticipated in the ground.

After formation of the hole to the design depth, the reinforcing tendon, with centralizers at regular intervals to provide the required cover is installed and the grout placed using a tremie pipe to the base of the hole. Care should be taken when installing the tendon, or corrosion protection ducts, into an uncased hole to make sure that their surface does not become contaminated by soil from the sides of the hole. This could compromise the bond.

4.11.3.2. Self Drilled Nails. Self drilled soil nail systems use the reinforcing tendon as the drill rod to transfer applied energy from the installation plant to the sacrificial drill bit. Hollow bar reinforcement is used to facilitate the injection of the flushing medium at the drill bit location. Typically, grout is used as the flushing medium which has the benefit of maintaining bore stability in unstable soils and minimizing the effects of softening as the hole is grouted immediately. As the diameter of the hole is generally smaller than with open-bore techniques, hand-held tools can be used for short soil nails. This is an advantage where access is difficult (Phear *et al.*, 2005).

Several sacrificial drill bit options are available for different soil types to provide the necessary soil penetration. The combined control of the penetration rate, flushing pressure and flow rate is of particular importance to evacuate all the spoil from the hole, to provide adequate cover to the nail and to limit over-break and grout wastage.

<u>4.11.3.3. Driven or Rotated Nails.</u> Driven or rotated nails are installed directly into the ground and displace the soil surrounding them. Installation can be achieved using percussive, vibratory or ballistic or drilling/rotation methods. Typically the reinforcing tendon is in direct contact with the ground, resulting in limited bond capacity, although with some systems this can enhanced by post-grouting (Phear *et al*., 2005).

The reinforcing tendon needs to be of sufficient strength to withstand the installation forces. The bending stiffness should be such that buckling and lateral displacements are limited and do not compromise the bond. For these reasons the temporary loading condition during driving will normally dictate the size of the reinforcing tendon and the need for metallic reinforcement. The use of driven soil nails is suited only to relatively weak soils where obstructions are not foreseen.

4.11.4. Excavation

Where a soil-nailed slope or wall is being used as a retaining measure in an earthworks cutting, bulk excavation and, potentially, temporary dewatering will be required. Unless the slope height is low, the excavation typically progresses in stages. The height of the exposed slope face is dictated primarily by its temporary stability. The construction sequence on site needs to retain a level of flexibility to deal with any variations in ground conditions encountered that affect temporary stability .

Other criteria that should also be considered in developing the construction sequence include the vertical reach of soil nail installation plant, access to carry out soil nail pullout tests, access for drainage installation and access to construct facing and install head plates.

Where grouted soil nails are used, subsequent excavation stages should progress only when the grout has achieved a strength of at least 5 MPa and any pullout tests have been carried out to avoid the need for access at a later date.

During the excavation the slope face is exposed, providing an opportunity to review the geological structure relative to the design soil parameters. While this may be representative only of the soil at the face, and not necessarily of the subsoil, the sample size is several orders of magnitude greater than that available from boreholes. There is significant potential value for ground risk mitigation and design refinement in having a geotechnical engineer on site during the works to review the exposed face for strata types, groundwater seepages, discontinuities etc. and compare the observations made on site with the design parameters.

4.11.5. Construction of Drainage for Soil Nailing

The sequence of drainage construction needs to be considered. For example, temporary drainage is often required during soil nailing construction. The slope face and subsurface drains need to be installed after each excavation stage because there are usually access constraints, primarily with regard to the safety of personnel, after the slope or wall is fully excavated. Sometimes it may not be safe to install permanent crest drains on existing potentially unstable slopes before nailing because of the destabilising surcharge loading from the plant needed to install them.

4.11.6. Construction of Facings for Soil Nailing

The selection, design and detailing of an appropriate facing for a soil-nailed slope or wall are as important as the design of the soil nails themselves. There are three types of facing, soft, flexible structural and hard structural facings.

The long-term effectiveness of a slope with a soft facing depends on the growth, and subsequent management of the vegetation. For this reason, existing natural vegetation should be maintained where appropriate. The selection of suitable vegetation types or species is a specialist subject, but the primary factors include the local climatic conditions, orientation of the slope face, rainfall pattern, topsoil and subsoil type and soil chemistry.

Materials used for flexible facings include geogrids and coated metallic meshes, in conjunction with head plates. These facing materials need to have sufficient strength and durability and will require either jointing or lapping to provide structural continuity between the soil nails. Careful consideration needs to be given to the ability of the facing material to resist the loads imparted by the nail heads and head plates to avoid failure by puncturing or rupture and/or excessive bulging under working conditions.

Completion of the flexible structural facing includes the tying in of the facing material along vertical joints and at the base of the excavation. Up to this point temporary stability conditions apply and the potential for shallow translational movement of the soil under the flexible structural facing exists.

Hard structural facings perform the same function as flexible structural facings but generally comprise steel-mesh-reinforced sprayed concrete. Other structural facing materials could include conventional cast-in-situ concrete or precast concrete panels.

There is a natural tendency for the temporary stability to become more critical as the slope face angle increases. In some cases excavation, therefore, soil nail construction and the application of sprayed concrete are carried out in bays of limited width. Alternatively, the nails may be installed before the final cut slope is excavated, followed by rapid excavation and spraying the concrete in stages. Sprayed concrete may be applied either as a single layer or in two phases. The latter may be adopted where temporary excavation stability is marginal, as the thin initial layer protects the slope face and enhances the localized stability of the soil.

4.12. Construction Inspection And Performance Monitoring

Inspection is the primary mechanism to assure that the soil nail wall is constructed in accordance with the project plans and specifications. Short-term and long-term performance monitoring is conducted to assess the performance of the soil nail wall. The owner agency, the contractor, or a combination of both can carry out the construction inspection activities, depending on the contracting approach.

Inspection activities, if properly conducted, play a vital role in the production of a high-quality soil nail wall because conformance to project plans and specifications should result in a soil nail wall that will perform adequately for the intended service life. Inspection may involve evaluation of the conformance of system components to material specifications, conformance of construction methods to execution specifications, conformance to short-term performance specifications and long-term monitoring.

Monitoring activities may include short-term or long-term measurements of soil nail wall performance. Short-term monitoring is usually limited to monitoring measurements of soil nail wall performance during load testing. In some cases, short-term monitoring may include monitoring lateral wall movements and ground surface settlements. Oftentimes this monitoring is motivated by performance requirements. Long-term monitoring of the soil nail wall usually includes a continuation of measurements from short-term monitoring (FHWA, 2003).

4.12.1. Construction Inspection of Soil Nailing

For a soil nail wall contracted using the method approach, inspection activities are carried out by the owner agency based on comprehensive material and procedural requirements of owner-provided plans and specifications. The contractor's responsibility is to follow the project plans and specifications. The owner's inspection is conducted to assure strict compliance with each component of the plans and specification (FHWA, 2003).

The quality of all materials used is controlled on site by visual examination for defects due to poor workmanship, contamination, or damage from handling, certification by the manufacturer or supplier that the materials comply with the specification requirements; and/or laboratory testing of representative samples from materials delivered to the site or approved storage area.

Nails, cement, bars, and drainage materials must be kept dry and stored in a protected location. Note that bars should be placed on supports to prevent contact with the ground.

The common methods to protect nails from corrosion include encapsulation, epoxy coating, grout protection, or a combination of these measures. Encapsulated bars are usually delivered to the site completely assembled. The epoxy coating should be visually examined for damage. Corrosion protection is a critical component of most permanent soil nails. Soil nails with damaged corrosion protection should be either repaired or replaced.

Soil nail walls are constructed in staged lifts using "top-to-bottom" construction with each lift completed to closure prior to excavating subsequent lifts. It is the responsibility of the inspection staff to ensure that all required construction activities and testing for each lift has been completed in accordance with the contract specifications and plans.

For some projects, construction monitoring devices and installation of instrumentation, such as slope inclinometers, surface survey points, load cells, or strain gauges, may be required. The installation methods should be covered in the plans and specifications and should be the responsibility of the contractor to maintain these devices during construction. A discussion of inspection issues for each of the major construction steps is provided below (FHWA, 2003).

The two types of excavation that generally occur during construction of a soil nail wall are mass excavation, which is conducted to provide equipment access and general site grading and excavations required for construction of the soil nail wall. During mass excavation, the inspection staff must verify that the excavation does not encroach upon the partially completed soil nail wall because uncontrolled excavation near the wall location could affect the stability of the wall.

The soil nail drillholes should be located as shown on the plans and within the specification tolerances. Generally, the angle of the drill mast, as measured with a magnetic angle tool, is used to check the angle of the drillhole. Drillholes in soil should be kept open only for short periods of time. The longer the hole is left open, the greater the risk of caving or destressing of the soil. A mirror or a high intensity light should be used, prior to nail installation, to inspect the hole for cleanliness. Soil that may have sloughed into the hole should be removed either by redrilling or by cleaning with a tool, if feasible.

The inspection staff should check each nail to ensure that the length, diameter, steel grade, centralizers, and corrosion protection as required are in accordance with the plans and specifications. The nail must be inserted into the hole to the minimum specified length. The inability to do so indicates an unacceptable condition caused by caving/sloughing of the hole and/or insufficient drilled length. Nails must be handled carefully to avoid damage. The centralizers should be stiff and large enough to provide space for the minimum specified grout cover. Centralizers should be spaced closely enough to each other to keep the bar from sagging and touching the bottom of the hole, but should not impede the free flow of tremied grout into the hole.

The primary inspection activity associated with grouting involves verifying that the entire length of the nail is grouted without any voids or gaps in the grouted column. To minimize the potential for drillhole caving, open-hole tremie grouting should be performed as soon as possible after drilling and immediately following nail insertion. The grout should flow continuously as the tremie pipe is withdrawn. The withdrawal rate should be controlled to ensure that the end of the tremie pipe is always below the grout surface. A record of the volume of grout placed should be maintained.

Once the final wall line excavation and nail installation have been completed for each lift, the geocomposite drain strips are typically placed vertically, at specified intervals. Drain strips must be continuous from the top to the bottom of the wall. Maintenance of drainage continuity and capacity is critical to the overall stability of the system and must not be risked. At the base of the soil nail wall, drains are connected either to a footing drain below the finished grade, or to weep holes that penetrate the finished wall. Weep holes should be located and spaced as shown on the plans, coinciding with the drain locations. A filter fabric is usually placed against weep holes to prevent clogging. Footing drains are comprised of perforated pipe embedded in drainage gravel.

After the geocomposite drain strips are installed, the reinforcing steel is placed and shotcrete is applied to the lines and grades specified. The welded wire mesh or reinforcing steel must be installed with the proper dimensions, at the specified locations and with the prescribed overlap length. During shotcreting, construction equipment that causes excessive ground vibrations should not be operating in the vicinity of the shotcreting operations to reduce shotcrete rebound. The overlying cold joint must be cleaned prior to placement of the overlying lift of shotcrete. During shotcreting the nozzle should be held perpendicular to the exposed excavated surface, except when shooting around reinforcing bars. Optimum nozzle distance from the surface being shot against is: 0.6 to 1.5 m for wet-mix, 1 to 2 m for dry-mix and voids shall not be allowed to form behind bars, plates, or steel mesh. Temporary shotcrete facings typically consist of 100-mm thick welded wire mesh reinforced shotcrete, placed directly against the soil, as the excavation proceeds in staged lifts. The steel bearing plate is positioned while the shotcrete is wet. Deviations from perpendicularity are adjusted with tapered washers below the nut. Once the bottom of the excavation is reached, a permanent wall facing, if required, is built.

4.12.2. Load Testing

Soil nails are load tested in the field to verify that the nail design loads can be carried without excessive movements and with an adequate factor of safety. Testing is also used to verify the adequacy of drilling, installation, and grouting operations prior to and during construction of the soil nail wall. If test results indicate faulty construction practice or soil nail capacities are less than that required, the contractor should be required to alter nail installation/construction methods (FHWA, 2003).

A center-hole hydraulic jack and hydraulic pump are used to apply a test load to a nail bar. The axis of the jack and the axis of the nail must be aligned to ensure uniform loading. Once the jack is centered and aligned, an alignment load should be applied to the jack to secure the equipment and minimize the slack in the set-up. The alignment load should not be permitted to exceed 10 percent of the maximum test load. Figure 4.29 shows schematically layout of a soil nail testing system.

Movement of the nail head is measured with at least one, and preferably two, dial gauges mounted on a tripod or fixed to a rigid support that is independent of the jacking set-up and wall. The dial gauges should be aligned within 5 degrees of the axis of the nail, and should be zeroed after the alignment load has been applied.

A hydraulic jack is used to apply load to the nail bar while, a pressure gauge is used to measure the applied load. A center-hole load cell may be added in series with the jack for use during creep tests. For extended load hold periods, load cells are used as a means to monitor a constant applied load while the hydraulic jack pump is incrementally adjusted.

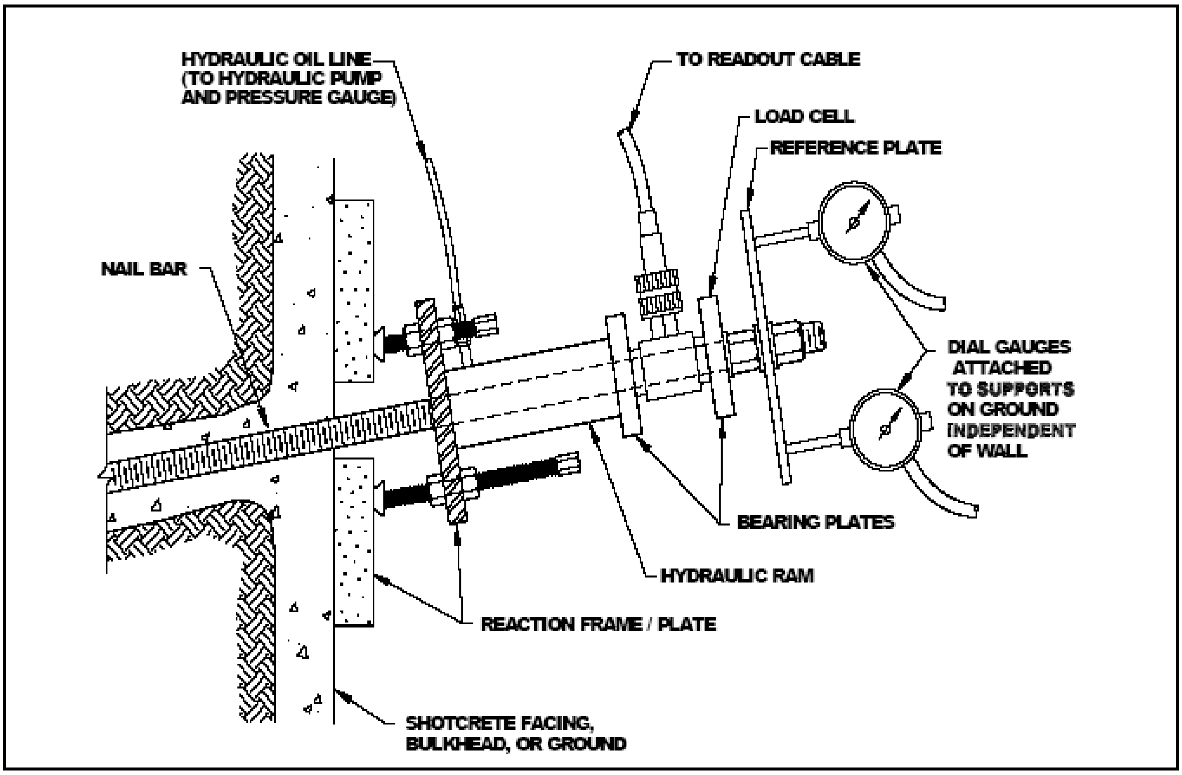


Figure 4.29. Schematic layout of a soil nailing testing system (FHWA, 1998)

Verification tests are completed on non-production, "sacrificial" nails prior to construction. In addition, verification testing may be required during production to verify capacities for different in situ conditions encountered during construction and/or different installation methods. Verification tests provide determination of the ultimate bond strength, verification of the design factor of safety; and determination of the soil nail load at which excessive creep occurs. As a minimum, verification test loading must be carried out to a load defined by the pullout factor of safety times the design allowable pullout capacity. The test acceptance criteria require that:

- no pullout failure occurs at 200 percent of the design load where pullout failure is defined as the load at which attempts to further increase the test load increments simply results in continued pullout movement of the tested nail; and
- the total measured movement (ΔL) at the test load of 200 percent of design load must exceed 80 percent of the theoretical elastic movement of the unbonded length (UL). This criterion is expressed as $\Delta L > \Delta L_{min}$, where ΔL_{min} is the minimum acceptable movement defined as:

$$\Delta L_{\min} = 0.8 \frac{PUL}{EA} \tag{4.17}$$

where:

P : maximum applied test load

UL : unbonded length (from the back of reference plate to top of the grouted length)

A : cross-sectional area of the nail bar

E : Young's modulus of steel [typically 200,000 MPa

A proof test is performed on a specified number, typically up to 5 percent, of the total number of production soil nails installed. This test is a single cycle test in which the load is applied in increments to a maximum test load, usually 150 percent of the design load capacity. Proof tests are used to ascertain that the contractor's construction methods and/or soil conditions have not changed and that the production soil nails can safely withstand design loads without excessive movement or long-term creep over the service life. The acceptance criteria require that no pullout failure occurs and that the total movement at the maximum test load of 150 percent of design load must exceed 80 percent of the theoretical elastic movement of the unbonded length. Again, the measured movement must be $\Delta L > \Delta L_{min}$, where ΔL_{min} has been defined in Equation 4.17.

Creep tests are typically performed as part of a verification or proof test. Creep testing is conducted at a specified, constant test load, with displacements recorded at specified time intervals. Acceptance criteria typically requires that creep movement between the 1 and 10 minute readings, at maximum test load, must be less than 1 mm, or that the creep movement between the 6 and 60 minute readings must be less than 2 mm at maximum test load. The creep criterion has been established to ensure that nail design loads can be safely carried throughout the structure service life.

4.12.3. Long Term Monitoring

Although several thousand soil nail structures have been constructed worldwide, only a limited number have been instrumented to provide performance data to support design procedures and ensure adequate performance (FHWA, 2003). Performance monitoring instrumentation includes inclinometers, top-of- wall survey points, load cells, and strain gauges. Inclinometers and survey points are used to measure wall movements during and after construction. Load cells are installed on selected production nails at the wall face to measure the magnitude of nail head forces. By installing strain gauges in individual nails, the development and distribution of the nail forces may be measured to provide information to improve future designs.

The most significant measurement of overall performance of the soil nail wall system is the amount of deformation of the wall or slope during and after construction. Inclinometers along the face and at various distances away from the face provide the most comprehensive data on ground deformations. Monitoring during wall construction should be performed to obtain data on the overall wall performance. As a minimum, a performance-monitoring plan should typically include requirements for the following features:

- face horizontal movements using surface markers on the facing and surveying methods, and inclinometer casings installed a short distance, typically 1 m, behind the facing,
- vertical and horizontal movements of the top of wall facing and the ground surface behind the shotcrete facing, using optical surveying methods
- ground cracks and other signs of disturbance in the ground surface behind the top of wall, through daily visual inspection during construction
- local movements and or deterioration of the facing using visual inspections and instruments such as crack gauges
- drainage behavior of the structure, especially if groundwater is observed during construction; drainage can be monitored visually by observing outflow points or through standpipe piezometers installed behind the facing

A typical instrumentation layout for a comprehensive monitoring plan is shown in Figure 4.30.

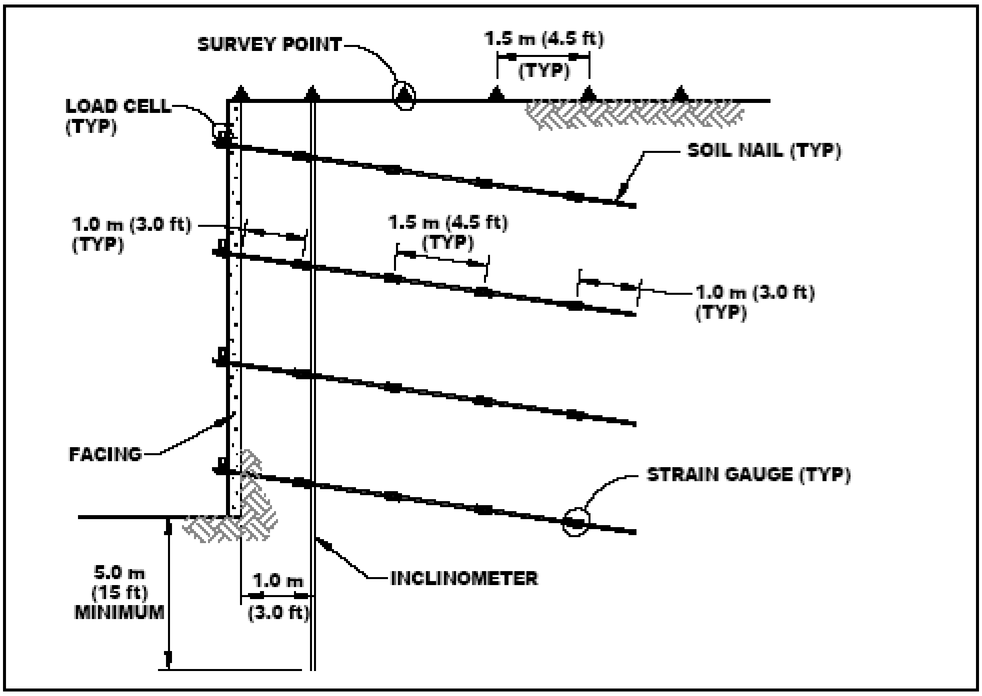


Figure 4.30. Typical instrumentation (FHWA, 1998)

A brief discussion of the various types of monitoring instruments typically employed for assessing soil nail wall performance is provided below.

Inclinometers are employed to measure horizontal movements of the structure, preferably installed about 1 m behind the soil nail wall face to provide the most comprehensive data on wall deformations. Inclinometers are a well-established technology and are commercially available from several manufacturers.

Wall face deformation can be measured directly by optical surveying methods or indirectly with electronic distance measuring (EDM) equipment. Also, ground movements behind the soil nail wall can be assessed by monitoring an array or pattern of ground surface points established behind the wall face and extending for a horizontal distance at least equal to the wall height. In addition, reflector prisms attached to selected nails allow for electronic deformation measurements of discreet points on the soil nail wall face. The survey system is typically capable of measuring horizontal and vertical displacements to accuracy of 3 mm or better.

Soil nails instrumented with strain gauges allow assessment of the soil nail load distribution as the excavation progresses and after the completion of the soil nail wall installation. Conventional strain gauges will measure loads carried by both the grout and nail and will thereby depend to some extent on the in-place deformational characteristics of the grout and the interaction between the grout and drillhole wall, both of which are difficult to evaluate. Ideally, strain gauges are attached to the nail bar in pairs, and are mounted top to bottom at a 1.5-m spacing, diametrically opposed to address bending effects. The end of the bar should be inscribed so that the final orientation of the strain gauge can be verified.

Load cells installed at the soil nail head are used to provide reliable information on the actual loads that are developed at the facing. High quality nail load data near the head of the nail can generally be obtained by load cells rather than by strain gauges attached to the nail.

5. PERFORMANCE OF TEMPORARY SOIL NAILED WALLS IN DEEP EXCAVATIONS

5.1. Introduction

The city of Istanbul due to its recent growth in economy caused a great attraction for the construction of high-rise residential and office buildings. In order to obtain parking space, deep excavations are employed to allow great number of basements below these tower structures. The depths of the excavations commonly reach to 25-40 meters below the ground surface. Most of these tower structures are constructed along the newly developed longitudinal axis of the city along Buyukdere Street having similar subsoil conditions and seismicity. Figure 5.1 represents the existing high-rise buildings on Buyukdere Avenue, red sections on the figure indicates the lands on which a number of tower structures are planned to built. Additionally, a picture from another location that the tower structures are concentrated in Maslak region, is given in Figure 5.2.

Figure 5.1. High-rise buildings on Buyukdere Avenue, Istanbul

Figure 5.2. A picture from Maslak, Istanbul

The encountered subsoil formation is generally soft rock greywacke locally known as Trakya formation, which is lithologically alternating sandstone, siltstone and claystones with various degrees of weathering and fracturing.

Since Istanbul is in very seismically active region and obviously at risk of being hit by major earthquake, flexible retaining walls should be preferred in deep excavations carried out in the city. Based on the previous positive records of flexible earth retaining structures during earthquakes in Turkey by Mitchell *et al*. (2000) and Durgunoglu *et al*. (2003a), soil nailed walls in such excavations performed within the city offer great advantage especially for the encountered subsoil and seismic conditions.

The results of the performance of walls with different heights in various sites having the similar greywacke subsoil formation are compiled herein. The performances of walls are monitored by inclinometer recordings taken at certain time intervals in parallel to the excavation at various locations.

The displacement and normalized displacement (i.e. performance ratio, P_r) data are presented together with some basic parameters of soil nailed walls such as, height of wall (H), area per nail (S), average nail length (L), nail density (η=L/S), length ratio (L_r), bond ratio (B_r) and strength ratio (S_r). As a result the values of performance ratio for soil nailed walls together with nail density in typical greywacke formation of the city of Istanbul are developed based on these extensive case studies as a guideline for future applications.

5.2. Case Studies of Soil Nailed Walls

During the last ten years soil nailed walls have been extensively constructed within the city of Istanbul as temporary retaining walls to support the basement excavations of various structures. According to recent compilation by Zetas (2006) about 160,000 m^2 of wall had been constructed in 60 different projects and the performances of some of these soil nailed wall structures have been reported previously by Ozsoy (1996), Durgunoglu *et al*. (1997) and Yilmaz (2000). In this study six major case of deep soil nailing walls constructed in Istanbul in locally well-known Trakya formation – greywacke are presented.

These six case studies of soil nailed walls having total surface area of 63,000 m^2 are BJK Fulya Complex, Istinye Park Complex, Kanyon Complex, Mashattan Residence, Tepe Shopping Mall and Besler Warehouse as presented in Table 5.1.

Table 5.1. Six major case studies of soil nailed walls in Istanbul

CASE STUDIES	**SURFACE AREA, m^2**
BJK Fulya Complex	6,500
Istinye Park Complex	20,000
Kanyon Complex	16,000
Mashattan Residence	6,500
Tepe Shopping Mall	6,000
Besler Warehouse	8,000
TOTAL	**63,000**

BJK Fulya Complex, planned to be erected in Fulya, Istanbul by Besiktas JK, consists of two residential towers, one office building, hospital and shopping mall. Due to the variations of soil formation of the entire project area, different retaining methods in accordance with the subsoil conditions are applied at different sections. In the 6,500 m^2 out of the total 15,000 m^2 of retaining area, soil nailed walls were constructed. Figure 5.3 demonstrates the three-dimensional architectural plan of the complex.

Figure 5.3. BJK Fulya Complex

Istinye Park Project, erected in Istinye, Istanbul, is a complex of residences, shopping mall and social facilities. Soil nailed walls were applied in the total 20,000 m^2 of retaining area. A picture of Istinye Park Project is given in Figure 5.4.

Figure 5.4. Istinye Park Project

Kanyon Complex, situated in Levent region of Istanbul, has shopping mall, residences and a multi-storey office building. The project has 16,000 m^2 retaining area, all soil nailed walls, with maximum height of 28.3 m. Figure 5.5 shows a picture from Kanyon Complex.

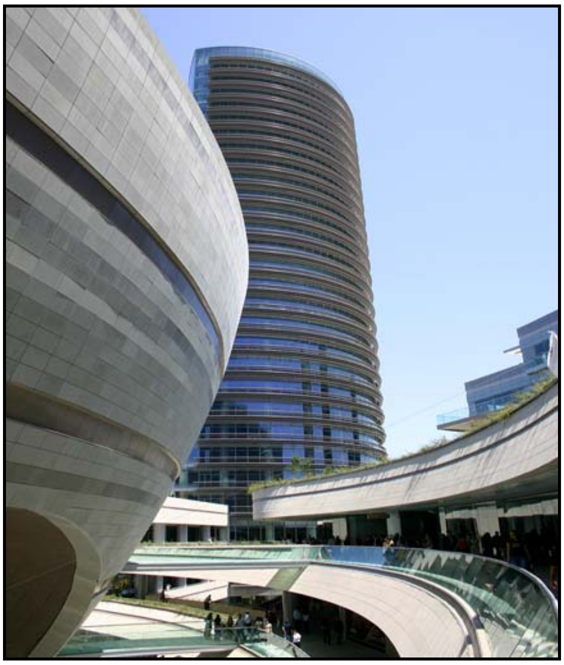


Figure 5.5. Kanyon Complex

Mashattan Residence in Maslak is a housing estate with eight apartment blocks each 30-storey with total retaining structure area of 10,000 m^2 which 6,500 m^2 of that was soil nailed walls. Figure 5.6 illustrates the three-dimensional architectural plan of Mashattan Residence.

Figure 5.6. Mashattan Residence

Tepe Shopping Mall, in Maltepe region of Istanbul, has 6,000 m^2 of retaining area with all soil nailed walls application. The last case, Besler Warehouse in Kurtköy, Istanbul with 8,000 m^2 soil nailed walls and the maximum height was 28.7 m.

5.3. Monitoring of Lateral Displacements in Soil Nailed Walls

Although several thousand soil nail structures have been constructed worldwide, only a limited number have been instrumented to provide performance data to support design procedures and ensure adequate performance (FHWA, 2003). Performance monitoring instrumentation for such walls should include inclinometers, top of wall survey points, load cells, and strain gauges. The most powerful method of monitoring a soil nailed structure is to observe its horizontal displacement because in literature most of the failures have reported because of the horizontal movement of nails by starting from the top. Therefore, a special concentration should be paid to the horizontal displacement and in order to observe these movements, inclinometers are the best solution.

5.3.1. Lateral Displacements

During construction and after its completion, a soil nail wall and the soil behind it tend to deform outwards. The outward movement is initiated by incremental rotation about the toe of the wall, similar to the movement of a cantilever retaining wall. Most of the movement occurs during or shortly after excavation of the soil in front of the wall. Post construction deformation is related to stress relaxation and creep movement, which are caused by post-construction moderate increases in tensile force in the soil nail (FHWA, 2003).

Maximum horizontal displacements occur at the top of the wall and decrease progressively toward the toe of the wall. Vertical displacements (i.e., settlements) of the wall at the facing are generally small and are on the same order of magnitude as the horizontal movements at the top of the wall. In general, horizontal and vertical displacements of the facing depend on the following factors:

- wall height, H, deformation increases approximately linearly with height,
- wall geometry, a vertical wall produces more deformation than a battered wall,
- the soil type surrounding the nails, softer soil will allow more deformation
- nail spacing and excavation lift heights, larger nail spacing and thicker incremental excavation lifts generate more deformation);

- global factor of safety, smaller FOS are associated with larger deformation,
- nail length to wall height ratio, shorter nail lengths in relation to the wall height generates larger horizontal deformation,
- nail inclination, steeper soil nails tend to produce larger horizontal deformation because of less efficient mobilization of tensile loads in the nails,
- magnitude of surcharge, permanent surcharge loading on the wall increases deformation.

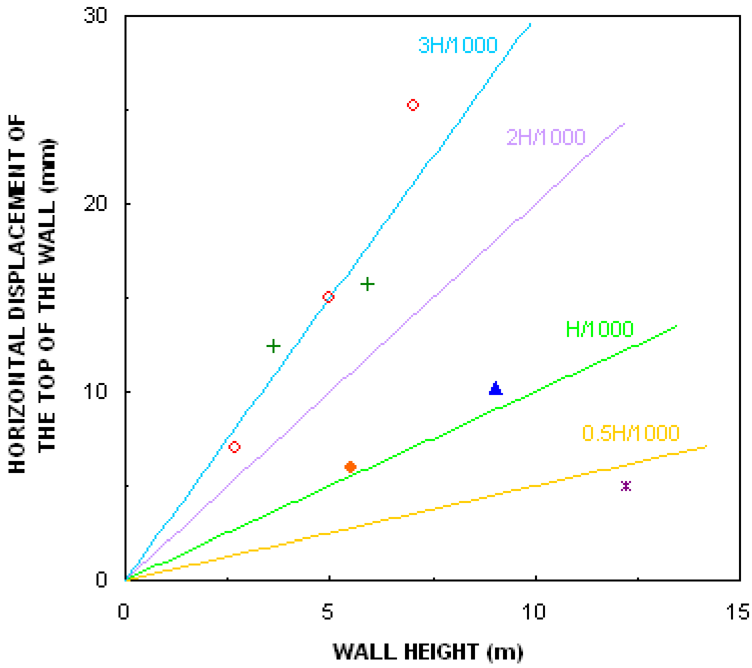
Evaluation of the measured lateral displacement data can be performed by means of the empirical approaches. These empirical relationships are determined from monitoring programs from different type of soils. A graph that is given by Juran (1991) can be used as a reference in Figure 5.7. According to this figure, horizontal displacement of the top of the wall is changed consistently with height of wall within the range of 0.5H/1000 to 3H/1000 by depending on the type of soil.

Typically, δ_h has been observed to be about equal to δ_v. The range of values for δ_h, defined as a percentage of the wall height H, observed in France is 0.1% to 0.3% (Schlosser and Unterreiner, 1991), while in USA it is 0.07% to 3.0% and in Germany it is 0.25% to 0.3% (Gassler and Gudehus, 1981).

Another approach is provided by Federal Highway Administration (2003). According to the report, on the ground surface at the top of a soil nailed wall, the lateral and vertical displacements which are maximum at the edge of the wall decrease to zero over a length as shown in Figure 5.8. For determining δ_h, an empirical formula is proposed by Federal Highway Administration (2003) as follows:

$$\delta_h = k(1 - \tan\alpha)H \tag{5.1}$$

where H is the wall height and α is the batter angle, which are demonstrated in Figure 5.7 and k is the coefficient, indicated in Table 5.2.



	SOIL	NAIL	REFERENCE
+	Medium sand	driven	Gassler *et al.* , 1981
▲	Silty sand (SM)	grouted	Shen *et al.* , 1981
◆	Fine sand (SP) to clayey sand (SC)	driven	Cartier and Gigan, 1983
✱	Residual clayey silt weathered shale sandstone	grouted	Juran and Elias, 1986
◇	Fontainbleau sand (SP)	grouted	Plumelle, 1986

Figure 5.7. Lateral displacement of soil nailed walls (Juran, 1991)

Table 5.2. Summary of data on displacements (FHWA, 2003)

Type of Soil	**Weathered Rocks Stiff Soils**	**Sandy Soils**	**Clayey Soils**
$\delta_h = \delta_v$	H/1000	2H/1000	3H/1000
Coefficient, k	0.8	1.25	1.5

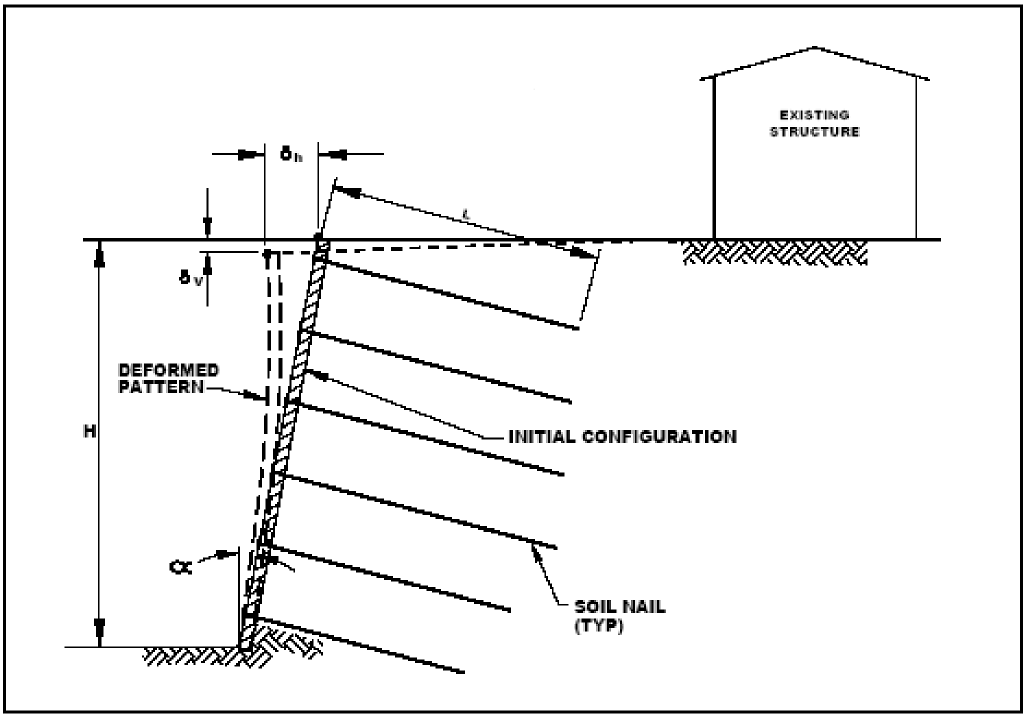


Figure 5.8. Deformation of soil nailed walls (FHWA, 2003)

5.3.2. Inclinometers

Inclinometers are defined as devices for monitoring deformation normal to axis of a pipe by means of a probe passing along the pipe. This pipe may be plastic, aluminum etc. The probe contains a gravity-sensing transducer designed to measure inclination with respect to the vertical. The pipe may be installed either in a borehole or in fill and in most applications is installed in a near-vertical alignment, so that the inclinometer provides data for defining subsurface horizontal deformation. Inclinometers are also referred in most references as slope inclinometers, probe inclinometers and slope indicators.

Most inclinometer systems have four major components:

(i) A permanently installed guide casing, made of plastic, aluminum alloy, fiberglass or steel. When horizontal deformation measurements are required, the casing is installed in a near vertical alignment. The guide casing usually has tracking grooves for controlling orientation of the probe.

(ii) A portable probe containing a gravity-sensing transducer.

(iii) A portable readout unit for power supply and indication of probe inclination.
(iv) A graduated electrical cable linking the probe to the readout unit.

After installation of the casing and surveying of its tip location, the probe is lowered to the bottom and an inclination reading is made. Additional readings are made as the probe is raised incrementally to the top of the casing providing data for determination of initial casing alignment (zero reading). The differences between these initial readings (generally 50 centimeter intervals) and a subsequent set defines any change in alignment. Provided that one end of the casing is fixed from translation or that translation is measured by separate means, these differences allow calculation of absolute horizontal displacement at any point along the casing. Figure 5.9 given by Dunnicliff (1988) shows the principle of inclinometer operation.

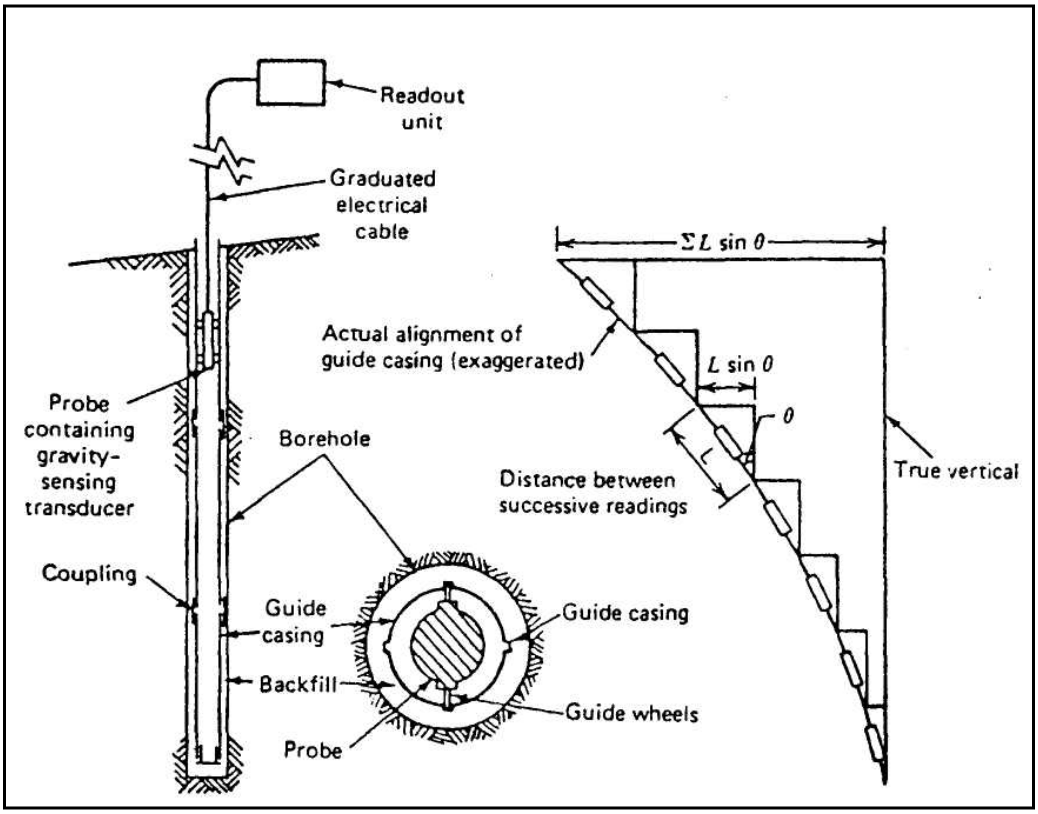


Figure 5.9. Principle of the inclinometer operation, (Dunnicliff, 1988)

Data processing and sign convention change to the manufactures' assumptions. The type of inclinometer and the procedure used in the case studies are described below.

5.3.3. Monitoring Performance and Data Processing

The inclinometer system used for all of the case studies is a force balance accelerometer type of inclinometer system 'SISGEO-S200SV', shown in Figure 5.10, which is the most extensively used type of inclinometer in Turkey (Ozsoy, 1996).

Figure 5.10. Sisgeo inclinometer system (Sisgeo, 1998)

System consists of inclinometer casing and probe, operating cable read, readout unit, software for data processing. The inclinometer probe consists of a stainless steel cylindrical element equipped, according to the required accuracy, with a biaxial servo-accelerometer or magneto-resistive sensor for inclination measurement in two mutually orthogonal directions. A submersible connector provide electrical and mechanical connection between probe and graduated cable. The inclinometer probe incorporates two rotating supports, equipped with two wheels each, located at a distance of 0.5 m, designed to properly fit the casing grooves during the measurement. The plane in which the four wheels lay defines the "A axis" while the orthogonal plane to it defines the "B axis". Knowing the reference wheel, the A and B axes in direction and sign are defined according to the Figure 5.11 (Sisgeo, 1998).

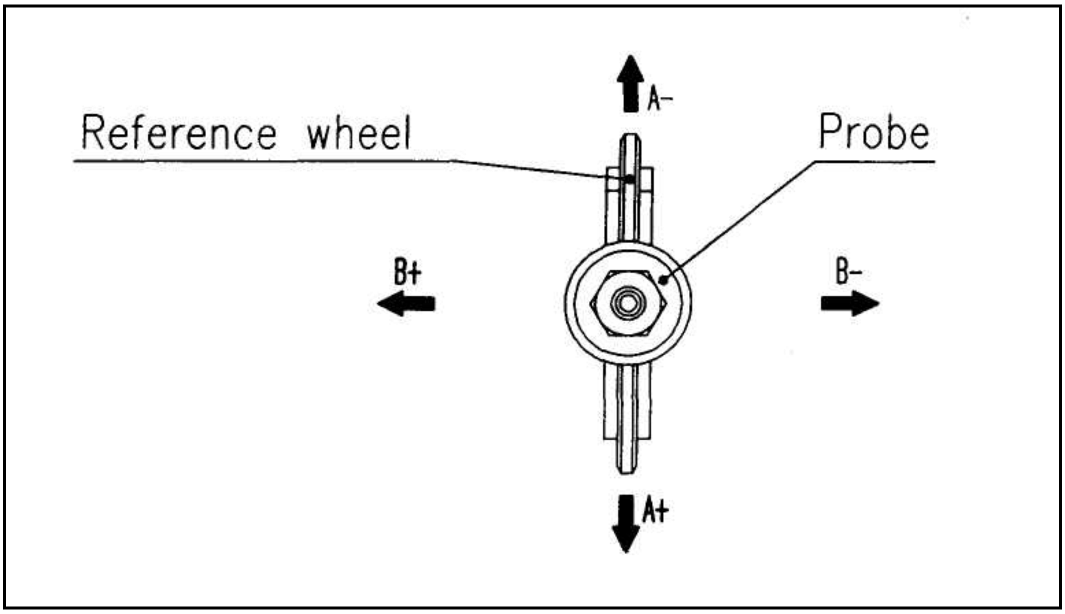


Figure 5.11. Inclination signs of Sisgeo probes (Sisgeo, 1998)

Measurement of A and B inclination component are performed every 50 cm along the inclinometric casing starting from the bottom. In each step two readings are taken as A and B from readout unit.

The readings may be performed on 2 grooves or 4 grooves. Remarking that the tube grooves are identified with number 1-2-3-4 clockwise. (Two grooves mean measurements have been taken in 1 -3 grooves and four grooves mean that measurements have been taken m 1-3 and 2-4 grooves. The routine readings are generally performed on grooves 1-3.

Theoretical 0° and 180° degrees readings should have the same value and opposite sign. Data set is usually influenced by electrical and mechanical offset introduced by the sensor. These differences can be eliminated averaging the A-, A+, and B-, B+ readings at each depth. This procedure is usually called "offset compensation" and improves the quality of the data interpretation.

Conversion for a servo-accelerometer probe independently from the readout unit adopted is presented herein. According to the calibration sheet, electrical output of the sensor is expressed in 20000 sin(α) unit. Giving two inclinometer probe electrical readings A_{mV} and B_{mV} expressed in mV:

$$\sin(\alpha_A) = A_{mV} / 20000 \qquad \text{A-axis} \tag{5.2}$$

$$\sin(\alpha_B) = B_{mV} / 20000 \qquad \text{B-axis} \tag{5.3}$$

Casing inclination is converted in lateral deviation and displacement for each axis at a certain depth according to the equations presented in the following formulas.

Casing lateral deviation:

$$A_d = \rho \times \sin(\alpha_A) \tag{5.4}$$

$$B_d = \rho \times \sin(\alpha_B) \tag{5.5}$$

Where ρ= is the measurement interval.

Casing total displacement:

$$D = \sqrt{(A_d{}^2 + B_d{}^2)} \tag{5.6}$$

Azimuth angle θ:

$$\theta = \arctan(A_d / B_d) \tag{5.7}$$

Casing lateral displacement is calculated every depth location. This calculated value is called "incremental lateral deviation". The sum of subsequent incremental lateral deviation versus depth is the "cumulative lateral deviation". Change in cumulative deviation defines the casing "displacement".

A dedicated software is provided for data reduction including data check, data processing and graphic output options (Sisgeo, 1998).

5.4. Evaluation of Lateral Displacements

In the case studies evaluated, the maximum height of soil nailed walls varied between 10.0 m to 32.5 m. Since it is known from the previous studies that a small inclination from the vertical has a great advantage in the performance of the walls (French National Research Project Clouterre, 1993; Elias and Juran, 1991), the slope angle β for the studied cases were 85° except for Case No. 2, Istinye Park having β=80°. Nail orientation with the horizontal were adopted as ω=10° in all the cases together with a common nail diameter of D=105 mm. The main lithological features of the greywacke formation are given together with other geometrical data; the maximum excavation height, H, slope angle, β, nail orientation, ω and the nail hole diameter, D in Table 5.3.

Table 5.3. Geometrical data and soil conditions of case studies of soil nailed walls

No.	Project Name	H_{max}(m)	β (°)	ω (°)	D (mm)	Subsoil Conditions
1	BJK Fulya Complex	32.5	85	10	105	fractured silicified **sandstone**
2	Istinye Park Complex	22.0	80	10	105	extensively fractured **siltstone, claystone**
3	Kanyon Complex	28.3	85	10	105	extensively fractured **sandstone, siltstone, claystone**
4	Mashattan Residence	24.9	85	10	105	extensively fractured **siltstone, claystone**
5	Tepe Shopping Mall	10.0	85	15	105	extensively fractured **sandstone, siltstone, claystone**
6	Besler Warehouse	28.7	85	10	105	extensively fractured **sandstone**

H_{max} (m) : Maximum soil nailed wall height
β (°) : Slope angle
ω (°) : Nail orienation
D (mm) : Nail hole diameter

Lateral displacement readings of the above mentioned six projects studied and evaluated in detail, are given in Figures 5.12 through 5.29 respectively. A picture view and the cross-section where the inclinometer readings made are also provided for reference.

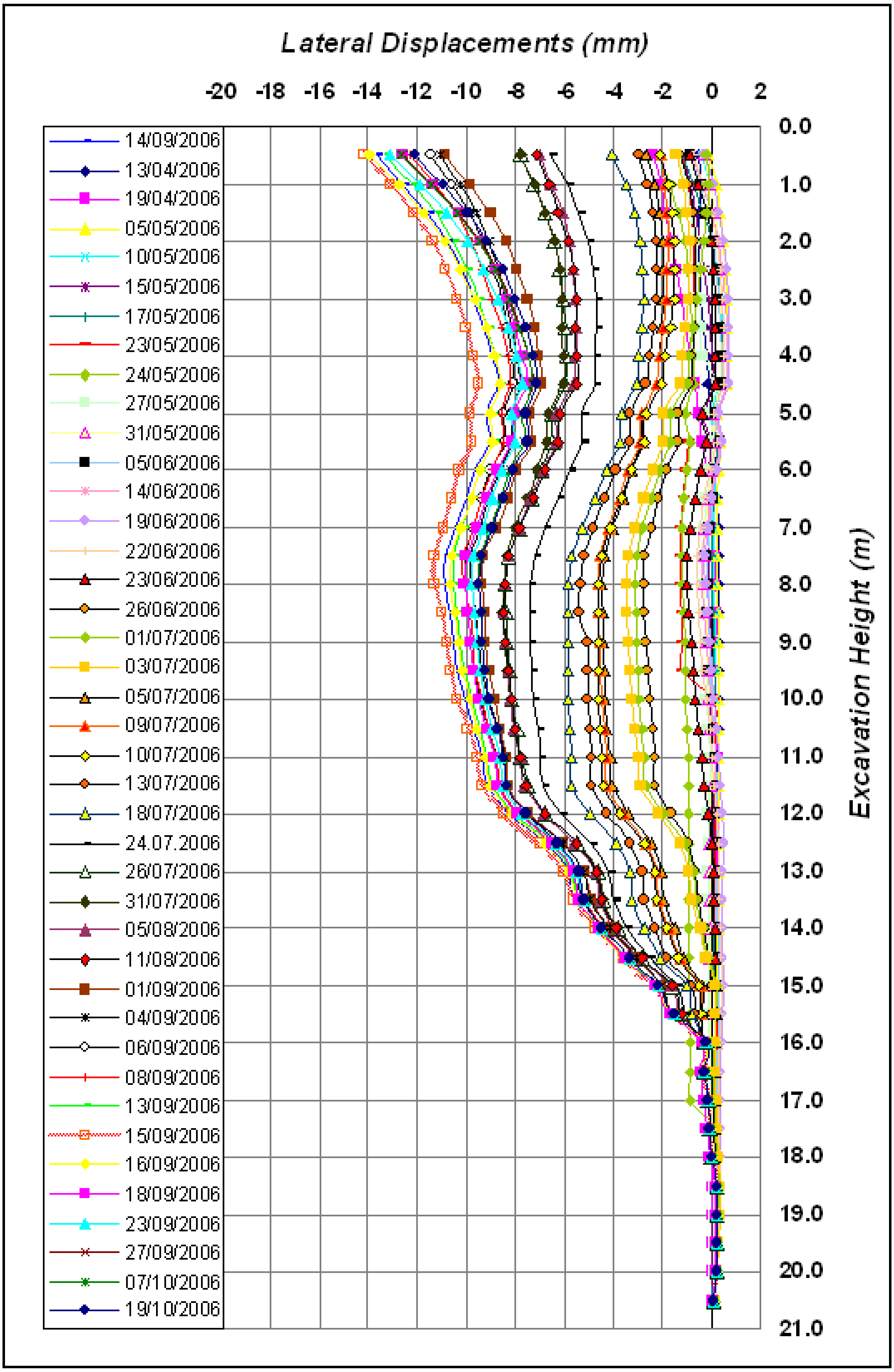


Figure 5.12. BJK Fulya Complex, inclinometer 1 readings

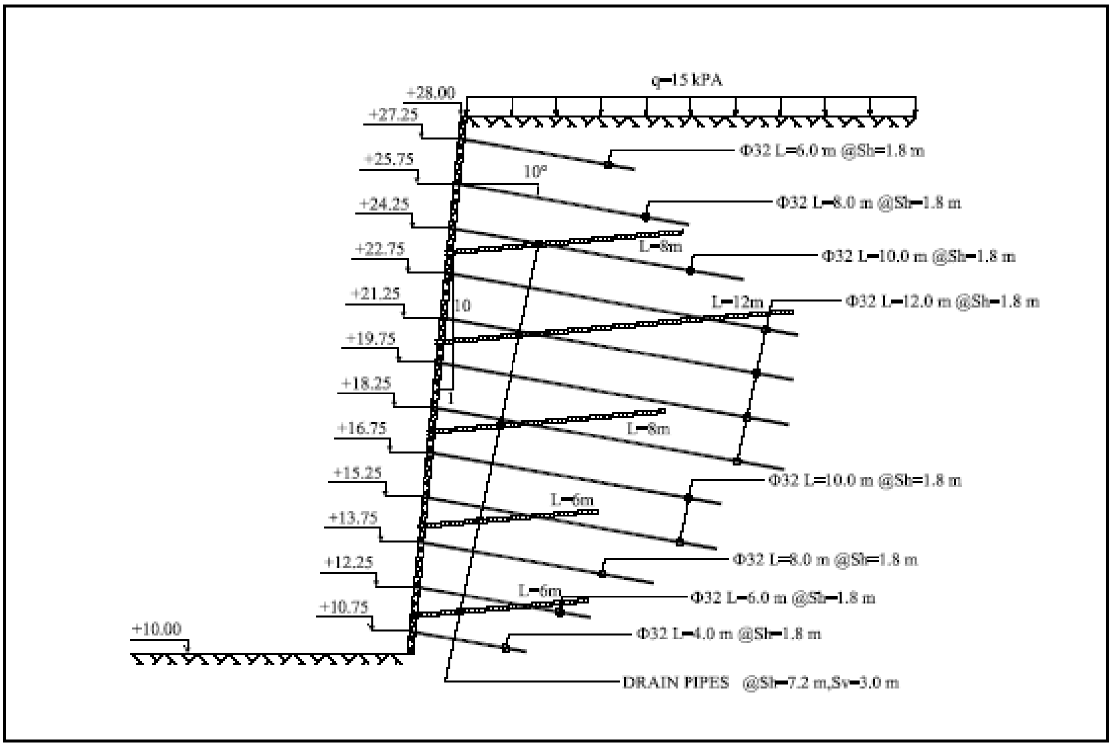


Figure 5.13. BJK Fulya Complex, detailed cross-section of inclinometer 1

Figure 5.14. A photograph from BJK Fulya Complex, inclinometer 1 section

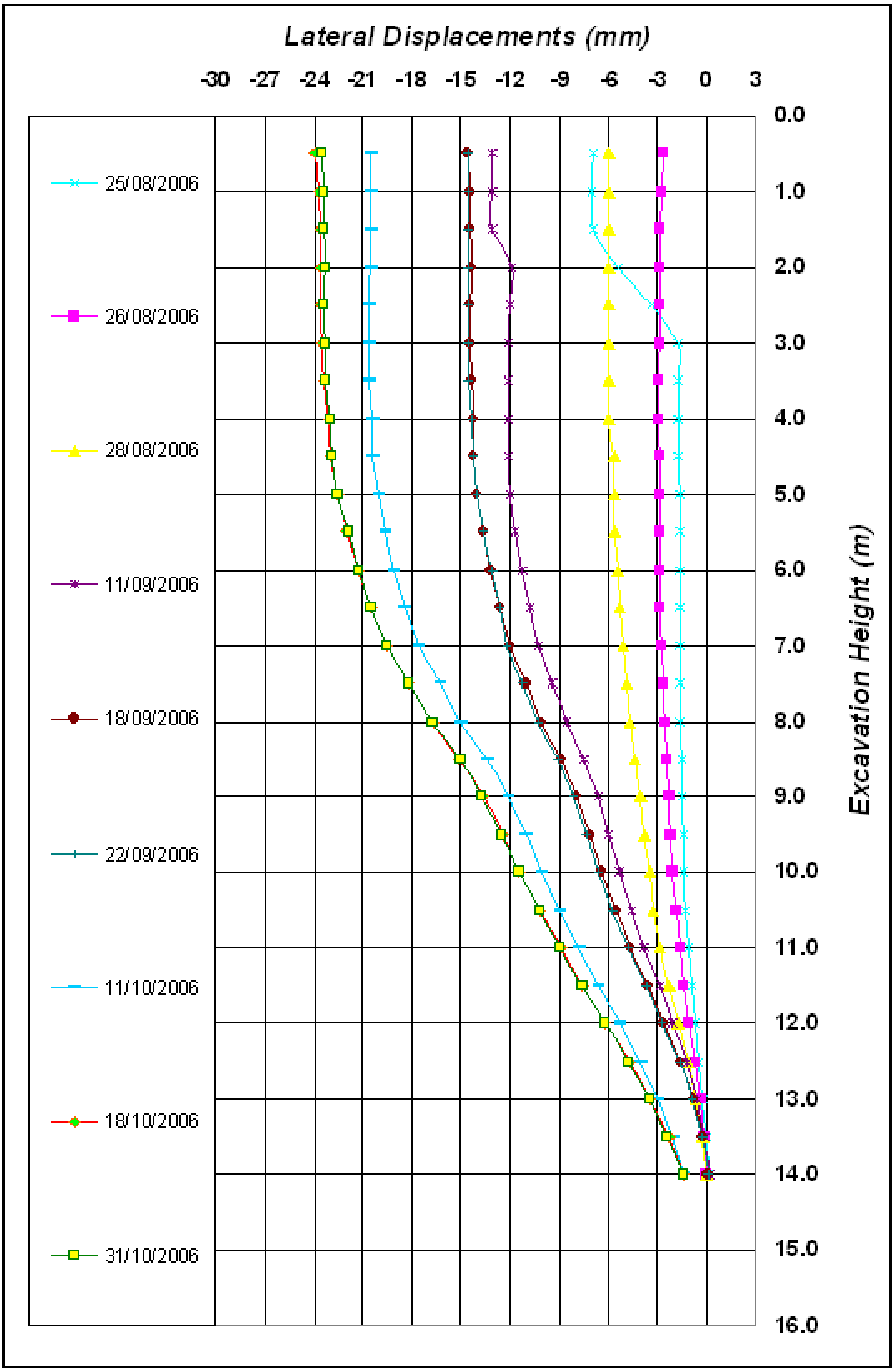


Figure 5.15. Istinye Park Complex, inclinometer 4 readings

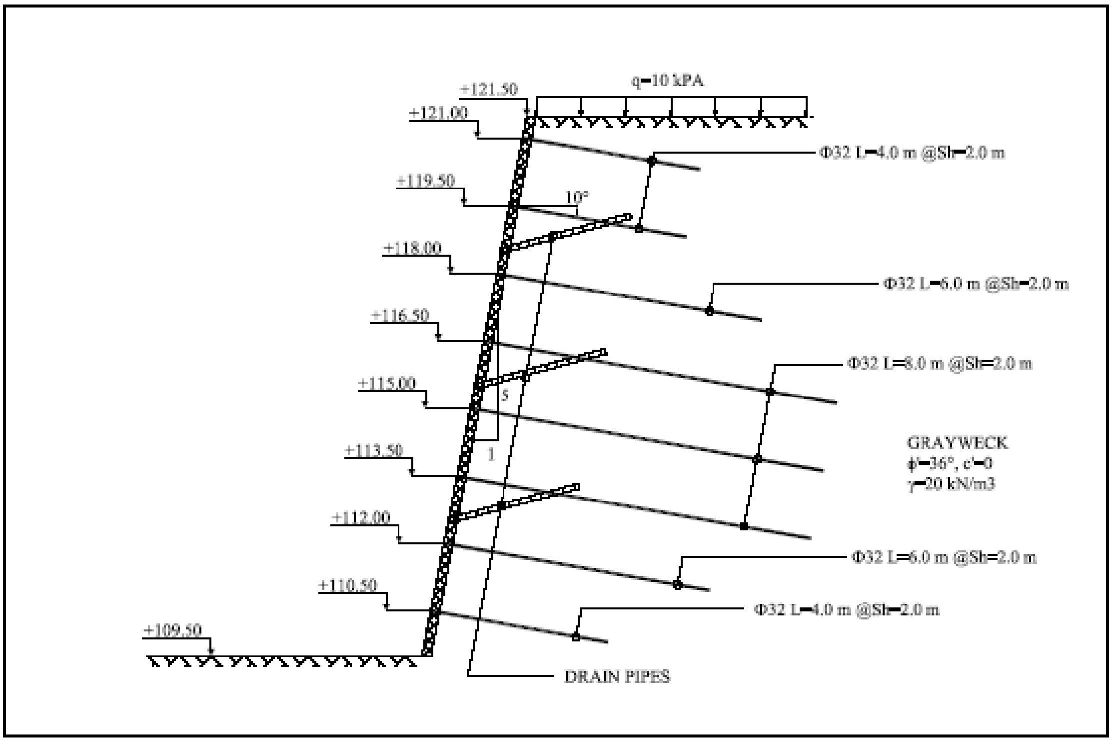


Figure 5.16. Istinye Park Complex, detailed cross-section of inclinometer 4

Figure 5.17. A photograph from Istinye Park Complex, inclinometer 4 section

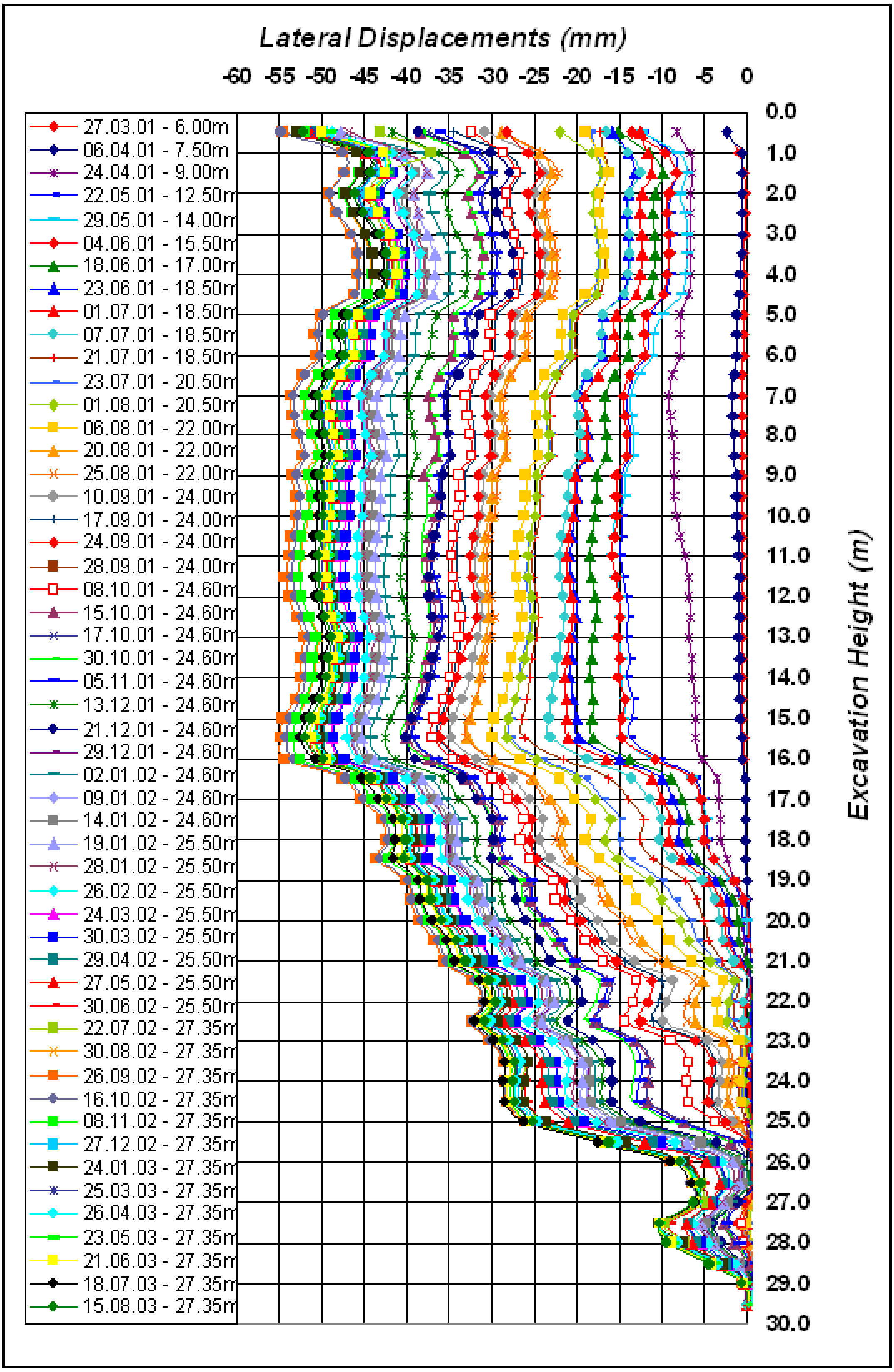


Figure 5.18. Kanyon Complex, inclinometer 7 readings

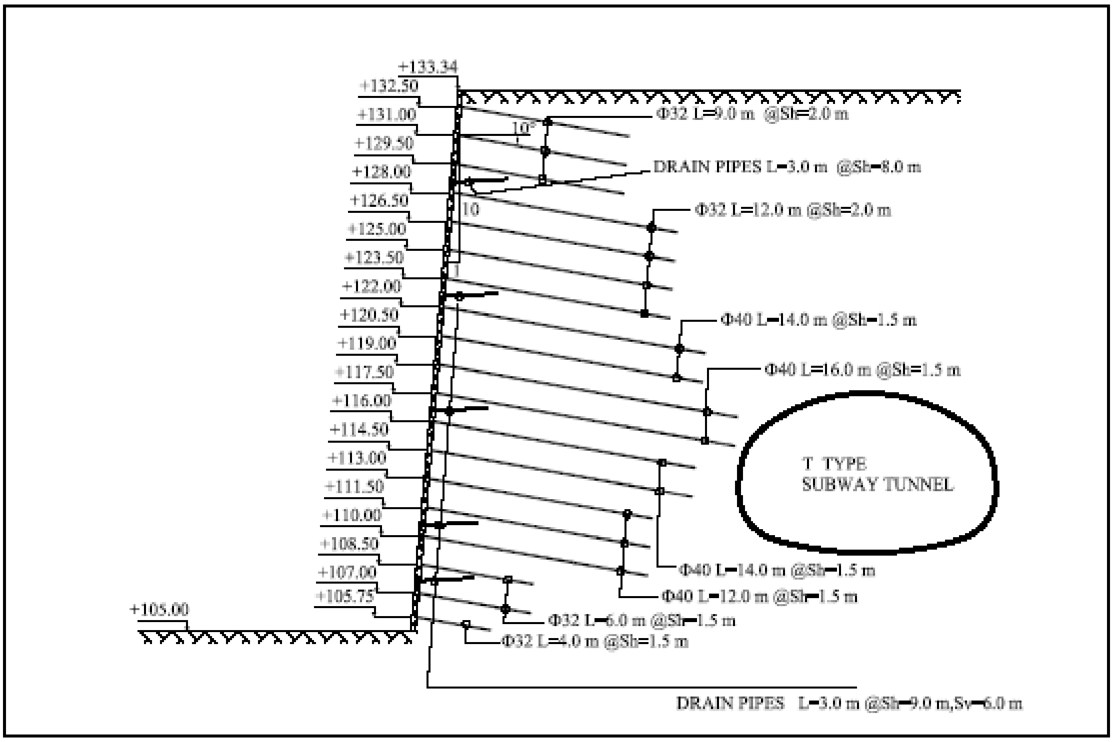


Figure 5.19. Kanyon Complex, detailed cross-section of inclinometer 7

Figure 5.20. A photograph from Kanyon Complex, inclinometer 7 section

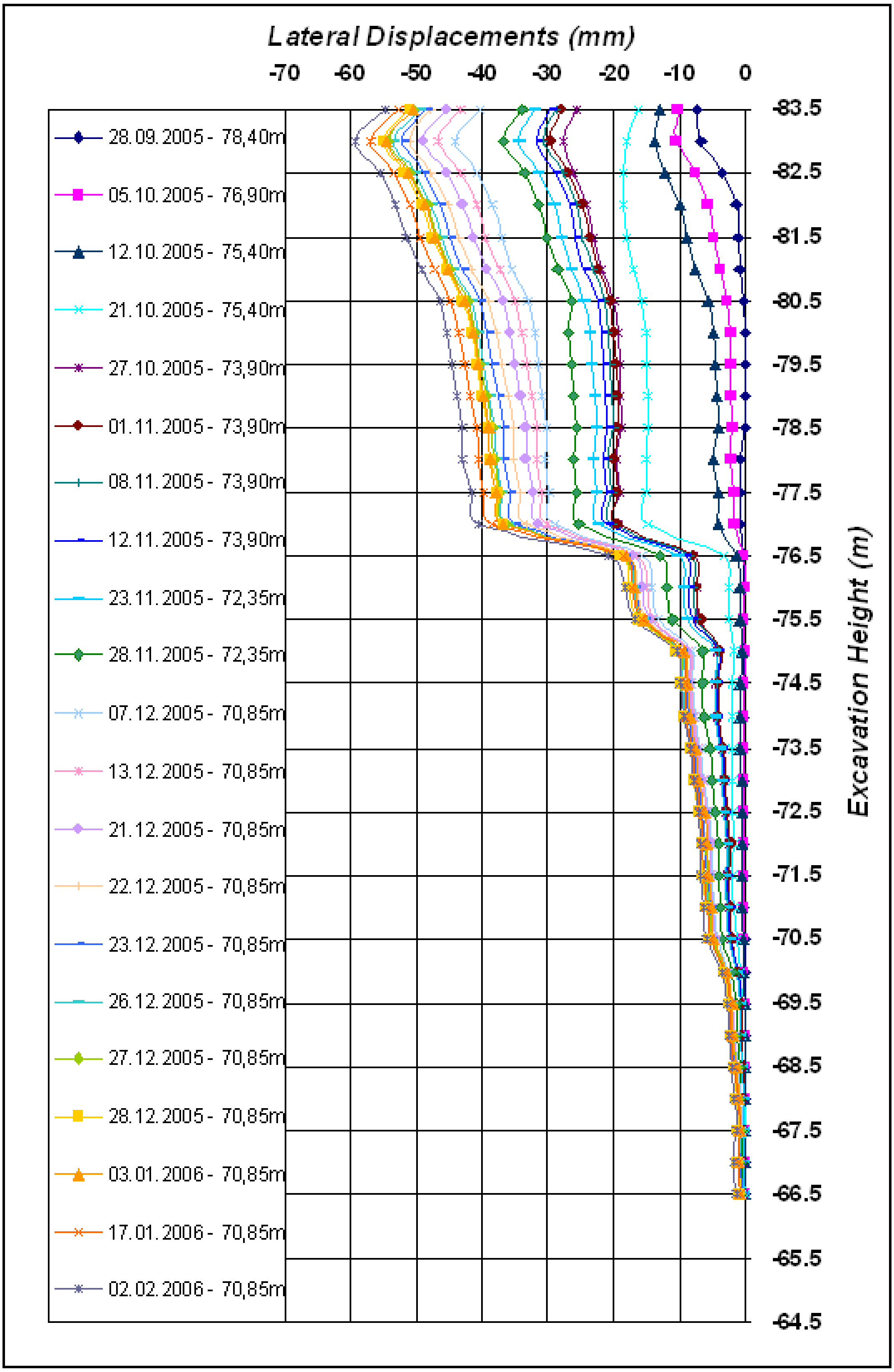


Figure 5.21. Mashattan Residence, inclinometer 1 readings

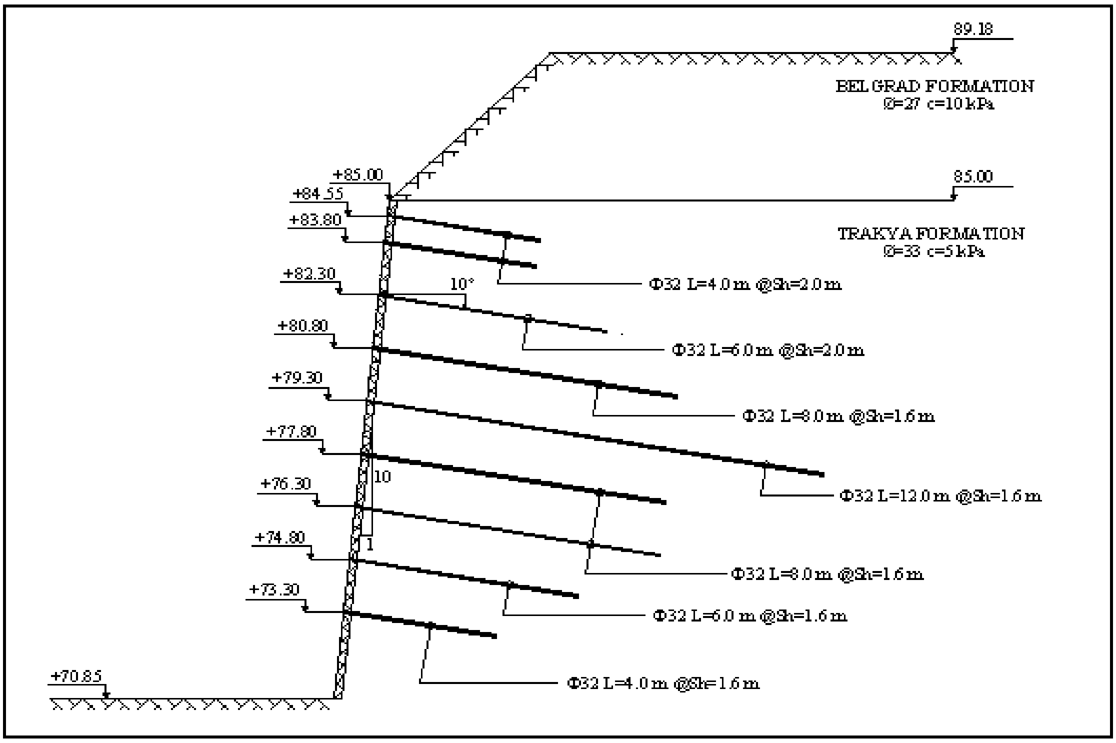


Figure 5.22. Mashattan Residence, detailed cross-section of inclinometer 1

Figure 5.23. A photograph from Mashattan Residence, inclinometer 1 section

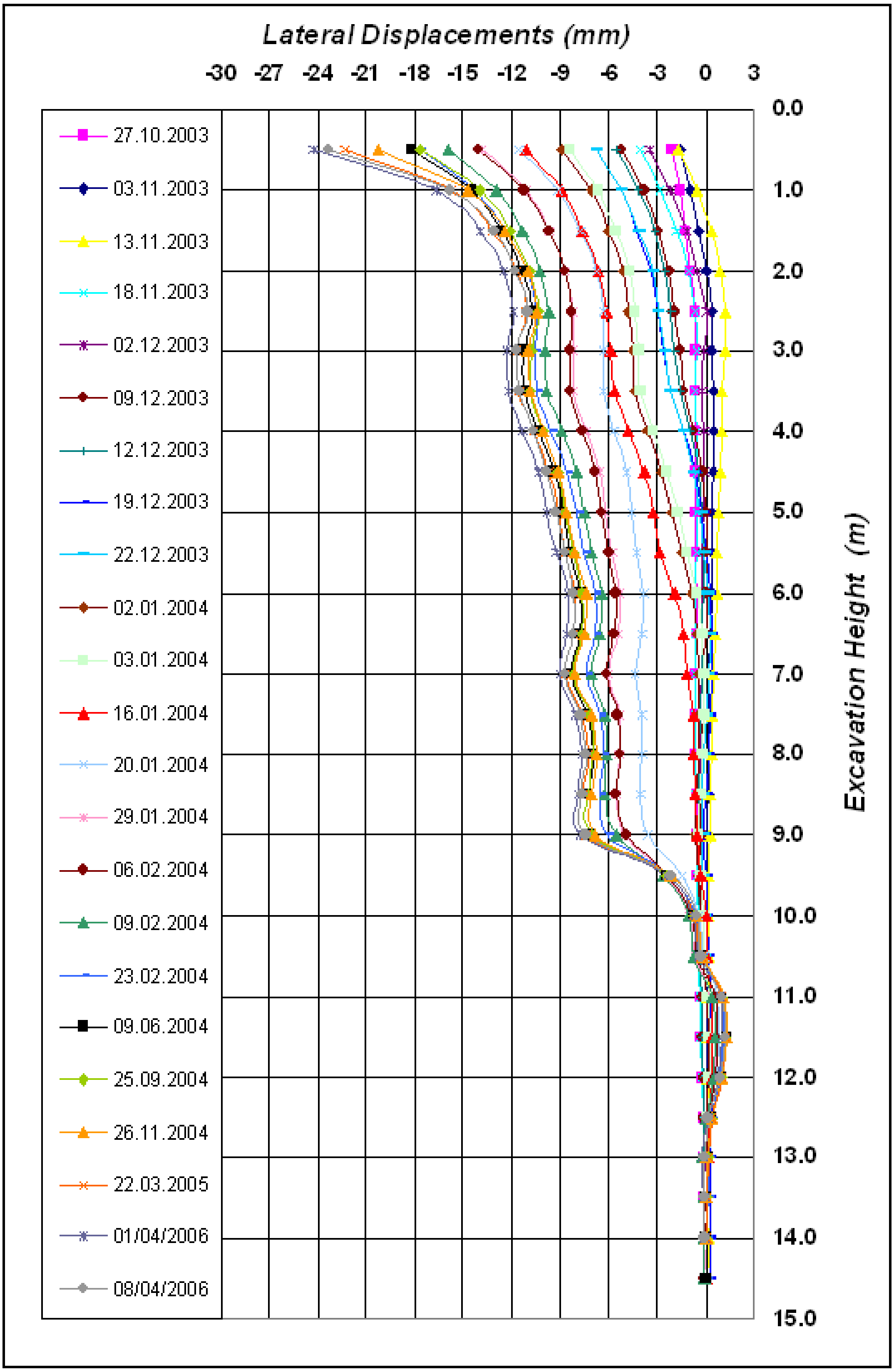


Figure 5.24. Tepe Shopping Mall, inclinometer 4 readings

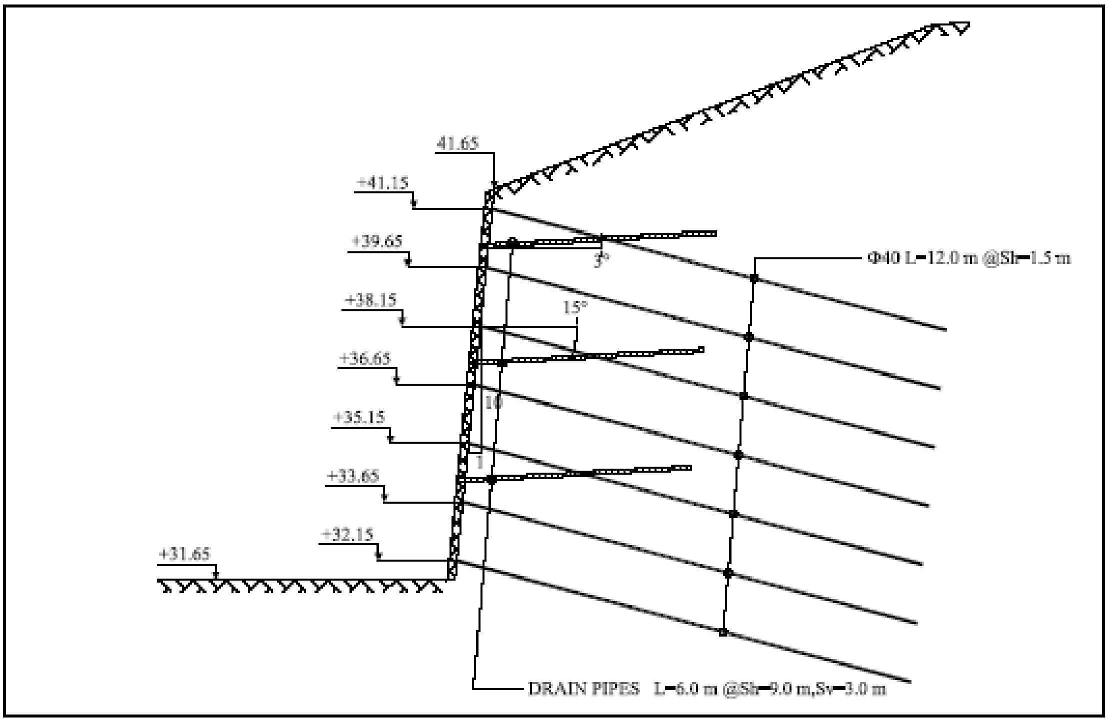


Figure 5.25. Tepe Shopping Mall, detailed cross-section of inclinometer 4

Figure 5.26. A photograph from Tepe Shopping Mall, inclinometer 4 section

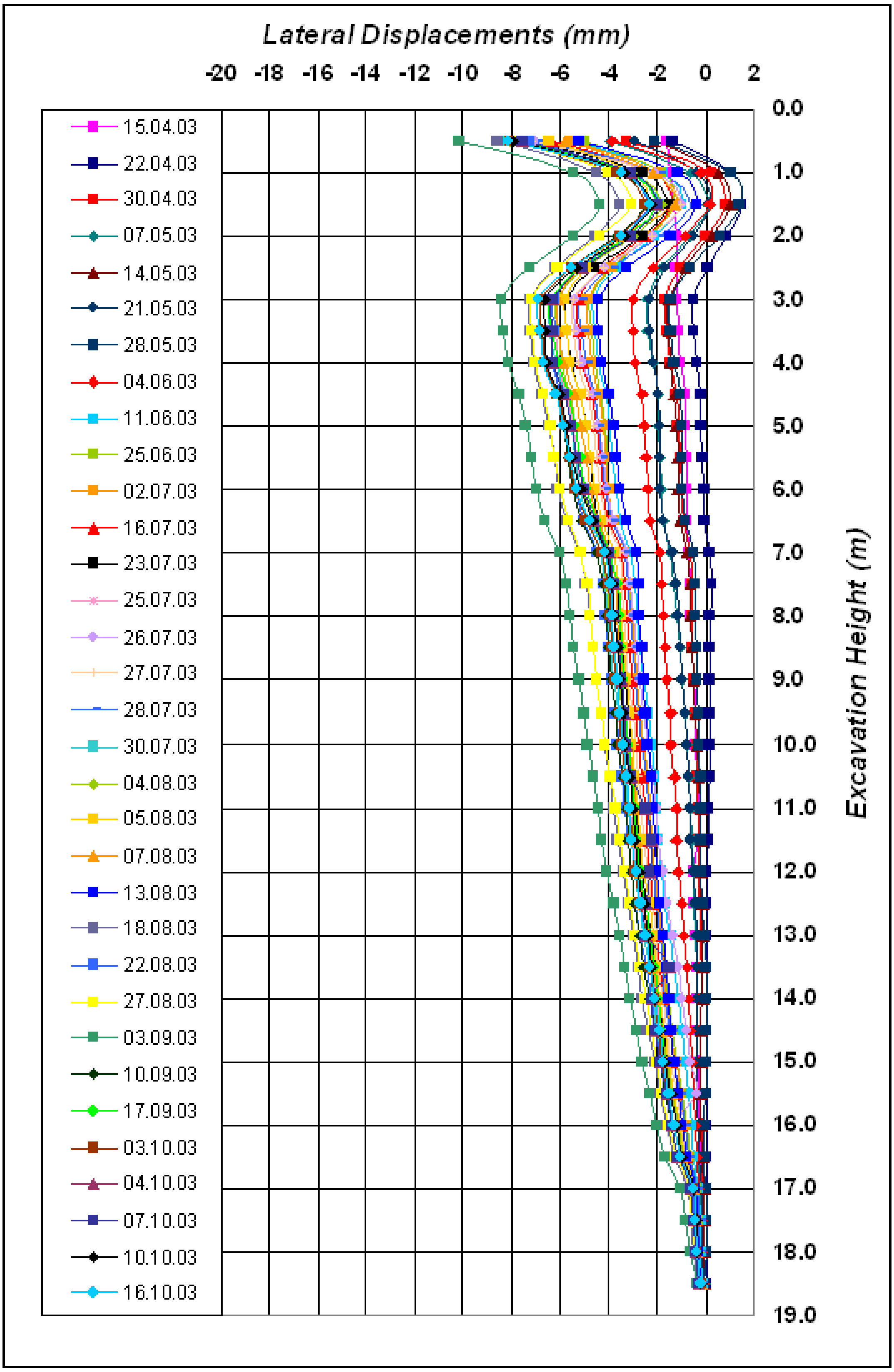


Figure 5.27. Besler Warehouse, inclinometer 1 readings

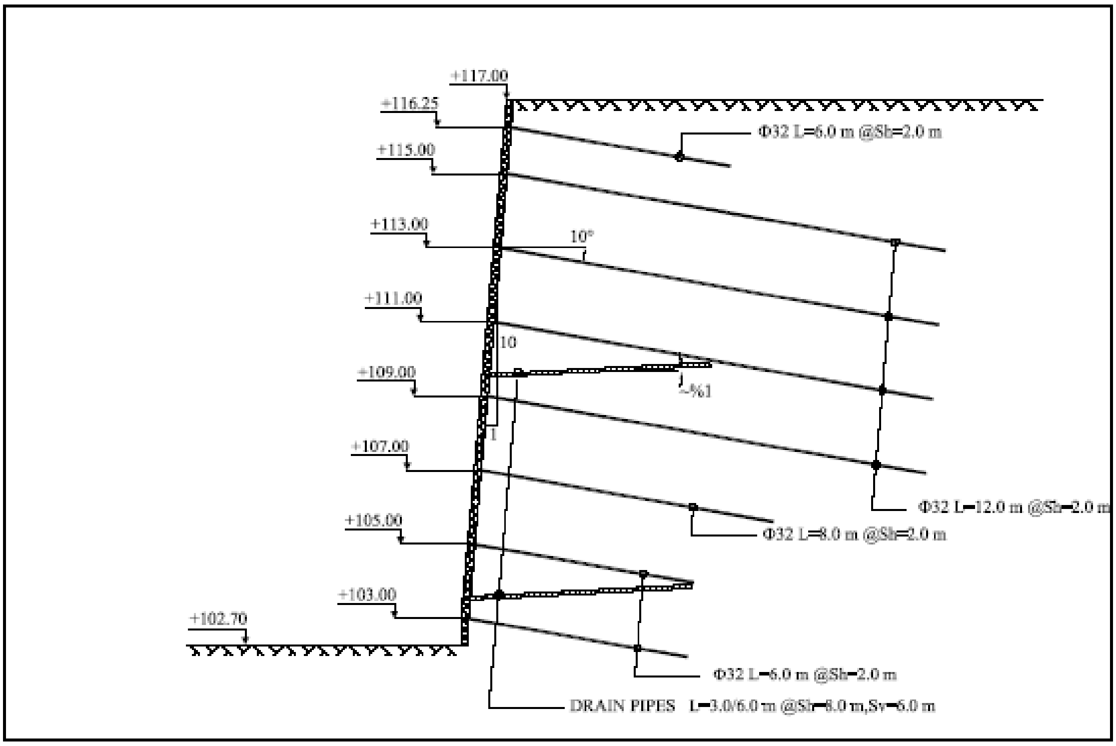


Figure 5.28. Besler Warehouse, detailed cross-section of inclinometer 1

Figure 5.29. A photograph from Besler Warehouse, inclinometer 1 section

5.5. Typical Lateral Displacement Data

A typical lateral displacement data for the Case No. 3 (Kanyon Complex, Istanbul), Inclinometer 7 is presented in Figure 5.30. At the top the excavation depth vs. date, in the middle lateral displacement vs. date and at the bottom lateral displacement vs. depth are provided. Inclinometer readings, detailed cross-section and a picture from this site is also given in Figure 5.18, Figure 5.19 and Figure 5.20.

Although the major height of the soil nailed wall was completed within six months, the excavation was kept open for almost another two years due to delay in final design of the upper structures to be constructed and obtaining related building permit from the municipality. It is interesting to note the followings:

- The lateral displacement has increased linearly with depth up to an excavation depth of approximately 18.0 m. For greater depths the rate of increase in the lateral displacement was increased considerably.

- Even though the temporary excavation with soil nailed retaining structure left open for more than two years, almost no additional lateral displacement was observed in spite of heavy rain and snow within that period indicating that the drainage system designed and implemented which is given in Figure 5.18 were performed satisfactorily. Subhorizontal drains, in length of l_d=3 m, were implemented at S_h = 8 to 9 m horizontal spacings with an inclination of 3˚ to the horizontal. The typical vertical spacings were S_v = 4 to 6 m.

5.6. Performance Analysis of Soil Nailed Walls

For each case study, some of the basic design parameters for the soil nailed walls, (Phear *et al*., 2005) are determined from the final design drawings and are given in Table 5.4. In Table 5.4 following parameters are calculated and summarized:

H = excavation height, m
$S = S_v \times S_h$, area per nail, m^2
L = average nail length, m
$\eta = L/S$, nail density, ave. nail length per area, m/m^2
$L_r = L/H$, length ratio
$B_r = DxL/S$, bond ratio
$S_r = D^2/S$, strength ratio

Design Parameters

δ_h = lateral displacement at top, mm
$P_r = \delta_h /H$, performance ratio

Performance Parameters

By the analysis of the data given in Table 5.4, lateral displacements, δ_h, performance ratio, P_r, average nail length, L, and nail density, η versus the height of the soil nailed wall, H were developed and presented in Figures 5.31 through 5.34. From these figures the following observations and evaluations are done for the soil nailed walls constructed in greywacke formation of the city Istanbul.

- It is seen that the linear increase in lateral displacement with the height of the wall is valid up to a certain height. The change in slope occurs at various heights and sooner for the weaker claystone than the stronger silicified sandstone case. Similar observation was made by Durgunoglu *et al*. (2003b) Using conventional methods of design, Federal Highway Administration (2003) and previously developed charts for estimating lateral displacements or performance ratio may be misleading in deep soil nailing applications.

- The performance ratio, Pr, for the greywacke formation is within the range of $1x10^{-3}$ to $3x10^{-3}$, depending on the nature of the lithological unit of the formation. For the strongest silicified sandstone with a typical value of E_m=250 MPa, $P_r \sim 1x10^{-3}$, on the other hand for the weakest claystone $P_r \sim 3x10^{-3}$ with E_m=50 MPa where Em is equal to $2(1+\nu)G_m$ and for ν =0.25, E_m=2.5 G_m. It is seen that these values tend to increase after 25 m for the case of sandstone and 15 m for the case of claystone.

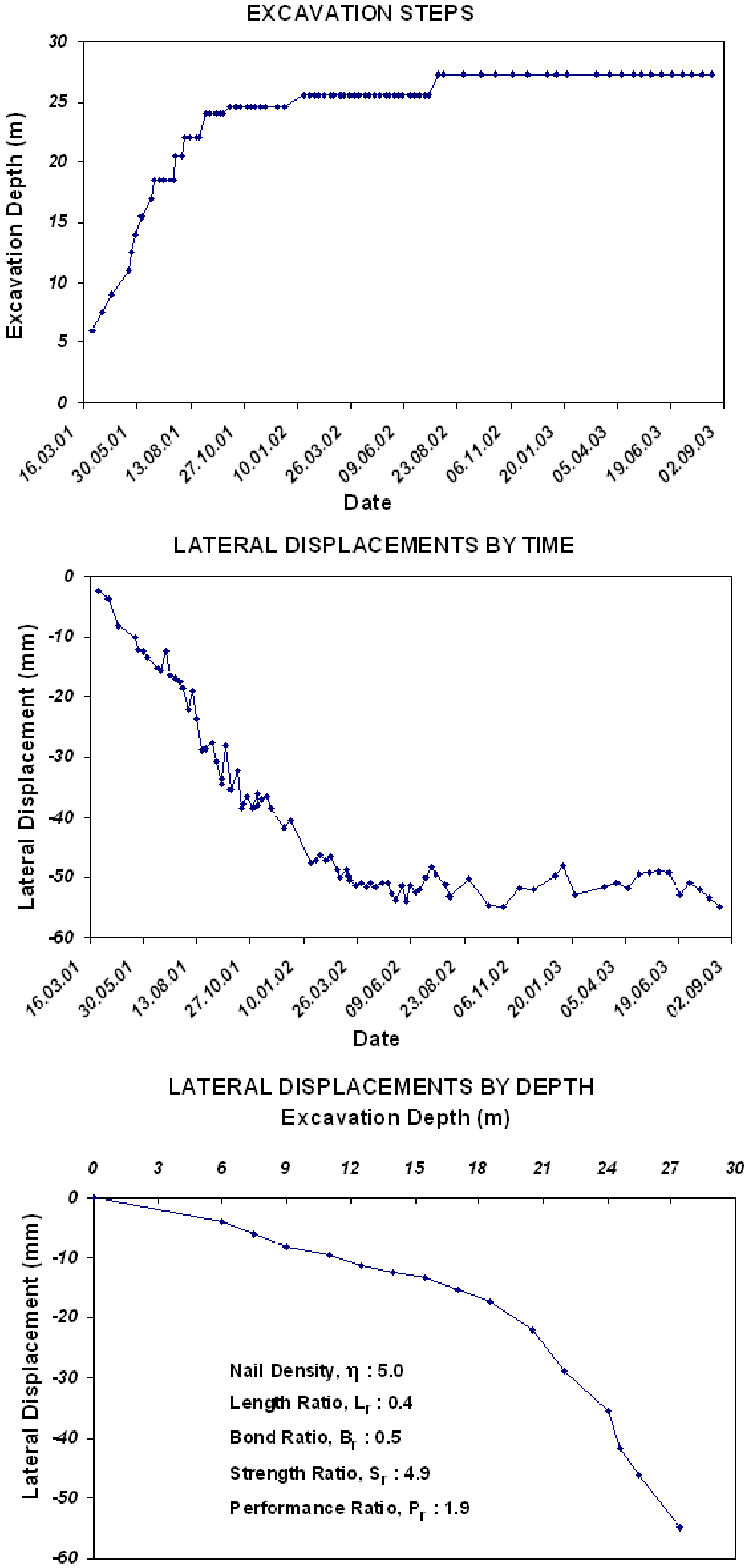


Figure 5.30. Lateral displacements for Case No.3, Kanyon Complex, Inclinometer 7

Table 5.4. Design and performance parameters for soil nailed wall case studies

Case 1: BJK Fulya Complex, Fulya,Istanbul

Inc. No.	H (m)	S (m^2)	L (m)	δ_h (mm)	η (m/m^2)	L_r	B_r	S_r (10^{-3})	P_r (10^{-3})
1	18,5	2,7	9,2	14,0	3,4	0,5	0,4	4,1	0,8
2	25,0	2,1	11,2	22,7	5,3	0,4	0,6	5,3	0,9
3	32,5	2,1	10,2	50,6	4,9	0,3	0,5	5,3	1,6

Case 2: Istinye Park Complex, Istinye, Istanbul

Inc. No.	H (m)	S (m^2)	L (m)	δ_h (mm)	η (m/m^2)	L_r	B_r	S_r (10^{-3})	P_r (10^{-3})
1	10,0	3,0	5,1	26,6	1,7	0,5	0,2	3,7	2,7
2	10,0	3,0	8,3	22,0	2,8	0,8	0,3	3,7	2,2
3	12,0	3,0	6,0	36,7	2,0	0,5	0,2	3,7	3,1
4	12,0	3,0	8,8	24,7	2,9	0,7	0,3	3,7	2,1
5	14,0	3,0	9,1	19,9	3,0	0,7	0,3	3,7	1,4
6	16,0	3,0	8,2	45,3	2,7	0,5	0,3	3,7	2,8
7	18,0	3,0	9,3	56,7	3,1	0,5	0,3	3,7	3,1
8	20,0	3,0	9,7	80,8	3,2	0,5	0,3	3,7	4,0
9	22,0	3,0	10,1	96,5	3,4	0,5	0,4	3,7	4,4

Case 3: Kanyon Complex, Levent, Istanbul

Inc. No.	H (m)	S (m^2)	L (m)	δ_h (mm)	η (m/m^2)	L_r	B_r	S_r (10^{-3})	P_r (10^{-3})
1	14,0	2,7	8,4	27,8	3,1	0,6	0,3	4,1	2,0
2	15,7	3,0	9,4	45,1	3,1	0,6	0,3	3,7	2,9
3	18,8	2,4	9,5	32,5	4,0	0,5	0,4	4,6	1,7
4	21,3	2,7	11,6	69,2	4,3	0,5	0,5	4,1	3,2
5	25,3	2,4	11,2	57,5	4,7	0,4	0,5	4,6	2,3
6	26,3	2,4	11,8	85,7	4,9	0,4	0,5	4,6	3,3
7	28,3	2,3	11,3	54,8	5,0	0,4	0,5	4,9	1,9
8	28,3	2,3	11,3	69,2	5,0	0,4	0,5	4,9	2,4
9	28,3	2,3	11,6	97,0	5,1	0,4	0,5	4,9	3,4

Case 4: Mashattan Residence, Maslak, Istanbul

Inc. No.	H (m)	S (m^2)	L (m)	δ_h (mm)	η (m/m^2)	L_r	B_r	S_r (10^{-3})	P_r (10^{-3})
1	18,3	2,4	6,7	59,3	2,8	0,4	0,3	4,6	3,2

Case 5: Tepe Shopping Mall, Maltepe, Istanbul

Inc. No.	H (m)	S (m^2)	L (m)	δ_h (mm)	η (m/m^2)	L_r	B_r	S_r (10^{-3})	P_r (10^{-3})
1	7,0	2,7	6,4	5,6	2,4	0,9	0,2	4,1	0,8
2	9,0	2,3	12,0	15,8	5,3	1,3	0,6	4,9	1,8
3	9,0	2,4	7,3	15,4	3,1	0,8	0,3	4,6	1,7
4	10,0	2,3	12,0	24,3	5,3	1,2	0,6	4,9	2,4

Case 6: Besler Warehouse, Pendik, Istanbul

Inc. No.	H (m)	S (m^2)	L (m)	δ_h (mm)	η (m/m^2)	L_r	B_r	S_r (10^{-3})	P_r (10^{-3})
1	14,7	4,0	9,3	10,2	2,3	0,6	0,2	2,8	0,7
2	16,2	4,0	9,6	18,3	2,4	0,6	0,3	2,8	1,1
3	18,4	3,6	9,6	13,4	2,7	0,5	0,3	3,1	0,7

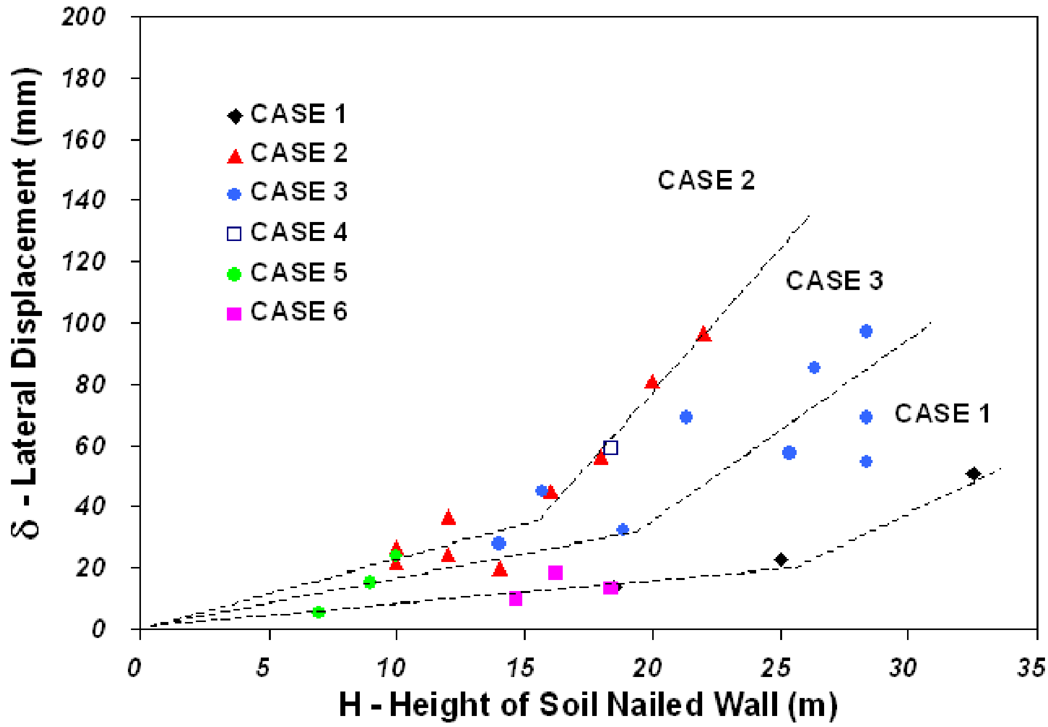


Figure 5.31. Lateral displacements, δ vs. height of soil nailed walls

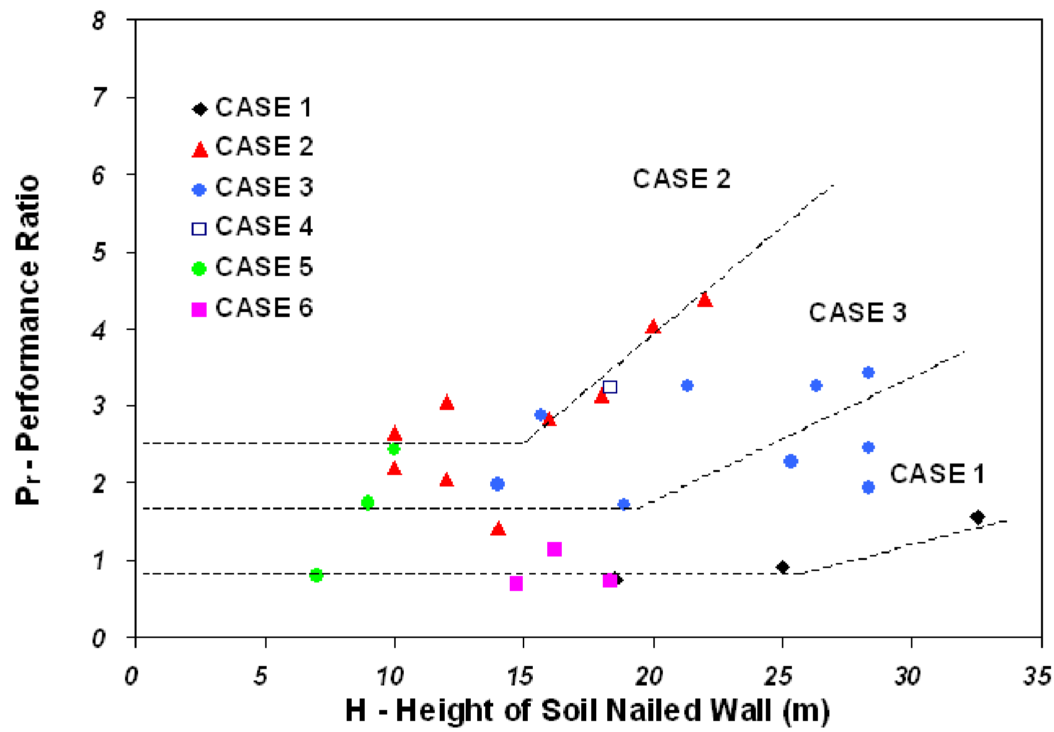


Figure 5.32. Performance ratios, P_r vs. height of soil nailed walls

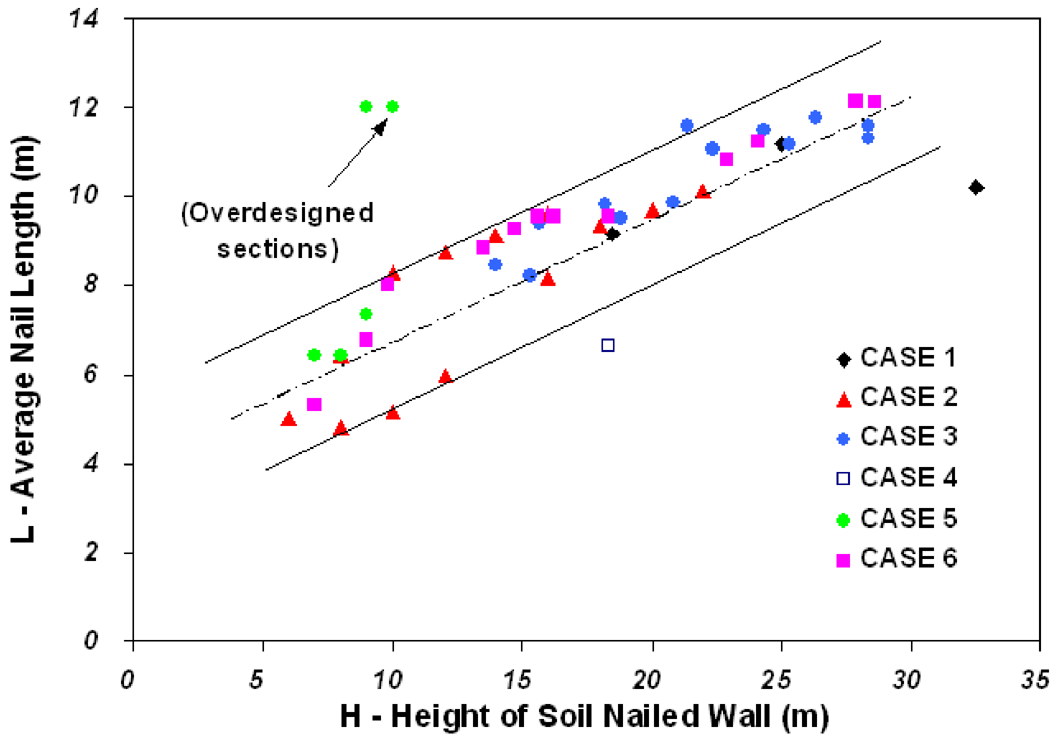


Figure 5.33. Average nail lengths, L vs. height of soil nailed walls

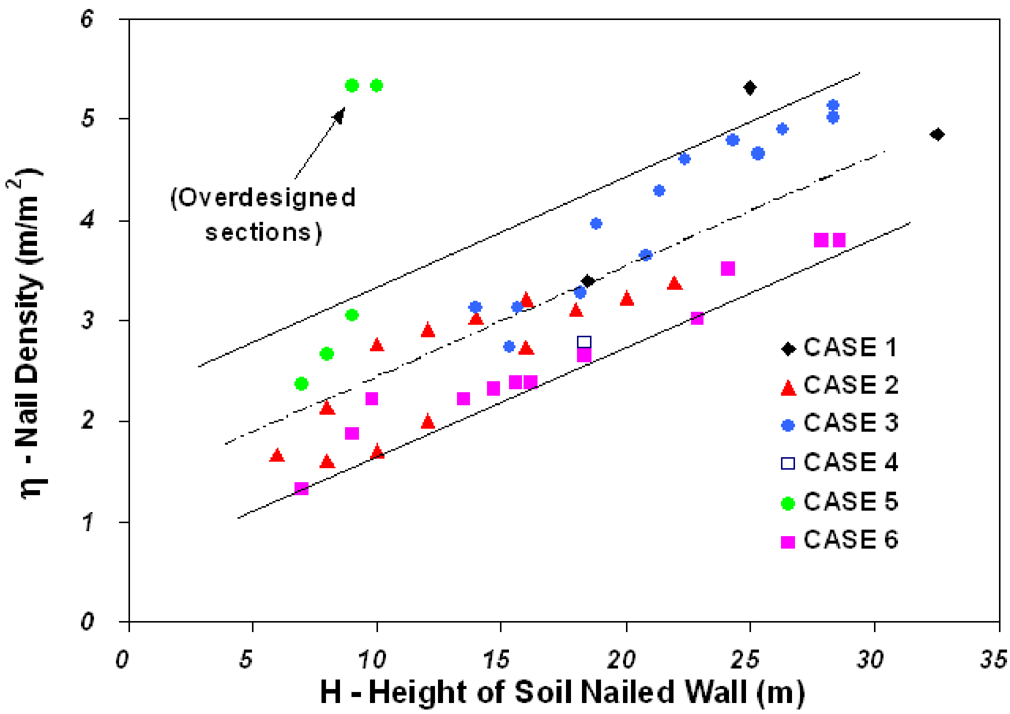


Figure 5.34. Nail density, η vs. height of soil nailed walls

- Average nail length, L, increases linearly with the height of soil nailed wall. The average nail length that could be utilized is about L=5 to 8 m for H=10 m and L=8 to 11 m for H=20 m.
- Nail density, L/S (m/m2) also increases linearly with the height of the soil nailed wall. It is about 1.6 to 3.2 m/m2 for H=10 m and 2.8 to 4.4 m/m2 for H=20 m.
- From Figure 5.33 and Figure 5.34 it is seen that two sections from Case No. 5, Tepe Shopping Mall, are overdesigned since they have noticeably long average nail lengths considering the small soil nailed wall heights. However, excessive nail lengths implemented on these sections have no or little effect on the lateral displacements or performance ratios as can be seen from the Figure 5.31 and Figure 5.32.

5.7. Concluding Remarks

Soil nailing is a very versatile excavation retaining system for deep excavations in urban areas surrounded by major structures and infrastructures provided that limiting lateral displacements are not exceeded.

Based on the previous positive records of flexible earth retaining structures during earthquakes in Turkey soil nailed walls in such excavations performed within the city of Istanbul offer great advantage especially for the encountered greywacke unit and described seismic conditions.

As a result the values of performance ratio for soil nailed walls together with nail density in typical greywacke formation of the city of Istanbul are developed based on these extensive case studies as a guideline for future applications.

For future applications performance of soil nailed walls are recommended to be monitored and to be compared with provided range of performance parameters based on the lithological description of the greywacke formation.

6. NUMERICAL ANALYSIS OF SOIL NAILED WALLS

6.1. Introduction

An essential and inevitable aspect of the performance of a soil nailed slope or wall is the amount that it deforms during its design life and whether this can affect adjacent features. Limiting the movements of soil nailed walls or slopes may be vital in some situations, especially where there are buildings, critical services or other infrastructure immediately adjacent. However, it should be recognized that in some cases, soil nailing may not be an appropriate solution because it does not guarantee sufficient security or sufficiently small deformations.

Although several thousand soil nail structures have been constructed worldwide, only a limited number have been instrumented to provide performance data to support design procedures and ensure adequate performance (FHWA, 2003).

The main shortcoming of the limit equilibrium design methods is that they do not give a prediction of deformations. They also do not consider the deformation required, to mobilize the resisting forces in the soil and soil nails. These methods cannot therefore provide a thorough description of the contribution of each soil nail to overall stability based on the pattern of deformation behind the slope or wall (FHWA, 2003). Deformations can be predicted approximately using empirical correlations and while these are appropriate in many cases they have limitations.

In situations where more confidence is required, a higher level of analysis should be adopted by using numerical modelling such as finite element and/or finite difference methods. The accuracy of numerical modelling depends on the quality of data acquired, the estimation of in-situ stress and soil stiffness and the availability of good case histories to calibrate numerical models. The stiffness parameters in these models should be adjusted to match the values obtained from actual site monitoring results at an early stage.

Observations during construction are essential and cannot be replaced by numerical modelling. Even using numerical modelling, it is still relatively difficult to predict displacements for layered soil stratigraphies and weathered rocks with discontinuity surfaces. The accurate modelling of the grout/soil interface is also difficult, as mobilisation of tension forces is often not directly proportional to facing deflections and/or construction stages (Phear *et al*., 2005).

In this study finite element computer program ‘PLAXIS’ was deployed to simulate the excavation sequence and installation of nails and to carry out back analysis of the six case studies in Istanbul of which performance analysis is achieved by means of inclinometer monitoring of lateral displacements. The finite element analyses were aimed to verify stiffness parameters of subsoil greywacke formation resulting the measured lateral displacements and to come across a relation between the actual lateral displacements and the stiffness parameters.

Detailed numerical analysis was performed on a specific section which thorough examination on the excavation steps and variation of the lateral displacement with depth and time was made in the previous section. Stiffness parameters were adjusted on every excavation stage in order to match the actual lateral displacements and thus actual subsoil profile in terms of stiffness were achieved. In this comprehensive finite element back analysis, the importance of verification and improvement of design during the construction stage through close observation and monitoring techniques is exposed. In this fashion it is possible to recognize and feed back potential problems and also make potential cost savings.

6.2. Numerical Modeling

Finite element method using the numerical modeling software ‘PLAXIS’ is used to perform back analysis of the typical cross-sections of the six soil nailing case studies; BJK Fulya Complex, Istinye Park Complex, Kanyon Complex, Mashattan Residence, Tepe Shopping Mall and Besler Warehouse.

The cross-sections where Inclinometer No. 1 readings made from BJK Fulya Complex, Inclinometer No. 4 from Istinye Park Complex, Inclinometer No. 7 from Kanyon Complex, Inclinometer No. 1 from Mashattan Residence, Inclinometer No. 4 from Tepe Shopping Mall and Inclinometer No. 1 from Besler Warehouse were analyzed as representing cross-sections of the mentioned case studies.

To achieve a fine relationship with the measured lateral displacements, the slope geometry, the material modeling, excavation and construction sequences have been simulated as close as possible to the actual conditions including the removal of elements to match the actual stages of excavation.

The 15-noded finite element mesh was adopted for the 2D analysis under plain strain conditions. The model boundary conditions were fixed by standard fixities, where side vertical boundaries were fixed in horizontal x-direction but free to move vertically, while the bottom boundary was restrained from any movement in all directions. The initial stresses of the model were calculated by gravity loading to reach its equilibrium (Liew and Khoo, 2006).

Fully drained Mohr-Coulomb Model was implemented as representing soil and interfaces behaviour. The elastic-plastic Mohr-Coulomb Model represents a “first-order” approximation soil or rock behaviour. It is recommended to use this model for a first analysis of the problem considered (Brinkgreeve, 2002). Mohr-Coulomb Model involves five input parameters; modulus of elasticity, E and Poisson’s ration, ν for soil elasticity, internal friction angle, Ø and cohesion, c for soil plasticity and ψ as an angle of dilatancy. Besides the five model parameters mentioned above, initial soil conditions play an essential role in most soil deformation problems (Brinkgreeve, 2002). Therefore proper K_0 values were selected to generate initial horizontal soil stresses.

As the nail was slender steel reinforcement and offered only little bending resistance (Liew, 2005), it was modeled using the one-dimensional geotextile element that was only capable of sustaining uniaxial tension (Brinkgreve, 2002). Extension stiffness, EA, of the soil nails, which are modeled as elastic geogrids members, were calculated using below formulation:

$$(EA)_{nail} = E_{st} \times A_{st} + E_c \times A_c \tag{6.1}$$

where;

E_{st} : Elasticity modulus of steel

A_{st} : Cross-sectional area of steel tendon

E_c : Elasticity modulus of concrete

A_c : Cross-sectional area of grout covering

For example soil nail with C30 grout diameter D = 110 mm having E = $3x10^7$ kPa and nail tendon Ø32 mm having E = $2x10^8$ kPa at 1.5 m horizontal spacing is calculated as:

$$(EA)_{nail} = 2\times10^8 \times \frac{\pi}{4}(32)^2 + 3\times10^7 \times \frac{\pi}{4}\left[(110)^2 - (32)^2\right] = 4.218\times10^5 kN/m \tag{6.2}$$

Properties of soil nail types that are utilized in case studies are presented in Table 6.1

Table 6.1. Properties of the soil nails of the soil nailed walls

Type	**Parameter**	**Name**	**Value**	**Unit**
Ø32 @ S_h=1.5m	*Normal stiffness*	*EA*	$2.8x10^5$	kN/m
Ø32 @ S_h=1.6m	*Normal stiffness*	*EA*	$2.6x10^5$	kN/m
Ø32 @ S_h=1.8m	*Normal stiffness*	*EA*	$2.3x10^5$	kN/m
Ø32 @ S_h=2.0m	*Normal stiffness*	*EA*	$2.1x10^5$	kN/m
Ø40 @ S_h=1.5m	*Normal stiffness*	*EA*	$3.3x10^5$	kN/m

Structural shotcrete facing member was modeled as an elastic plate member having thickness d = 0.25 m with flexural rigidity, EI, normal stiffness, EA which were calculated as below:

$$(EA)_{facing} = E_{concrete} \times A_{concrete} \tag{6.3}$$

$$(EI)_{facing} = E_{concrete} \times I_{concrete} = E_{concrete} \times \frac{1}{12} bh^3 \tag{6.4}$$

Using above equations:

$$(EA)_{facing} = 3\times10^{7}\times(1.0)\times0.25 = 7.5\times10^{6}\,kN/m \tag{6.5}$$

$$(EI)_{facing} = 3\times10^{7}\times\frac{1}{12}\times1\times(0.25)^{3} = 3.9\times10^{4}\,kNm^{2}/m \tag{6.6}$$

Note that while calculating normal and flexural stiffness of the facing element, effect of welded wire mesh was neglected. Properties of the shotcrete facing, used in all of the case studies, are presented in Table 6.2.

Table 6.2. Properties of the shotcrete facing of the soil nailed walls

Parameter	**Name**	**Value**	**Unit**
Type of behaviour	*Material Type*	*Elastic*	-
Normal stiffness	*EA*	7.5×10^{6}	kN/m
Flexural rigidity	*EI*	3.9×10^{4}	kNm^2/m
Equivalent thickness	*d*	0.25	m

6.3. Back Analysis of Six Case Studies

Finite element back analysis by using computer program 'PLAXIS' was utilized on the soil nailed walls of the six case studies in Istanbul. In all cases fully drained Mohr-Coulomb Model was implemented with the material data described above. The aim of this study is to confirm stiffness parameters of greywacke formation of the corresponding case that provide the measured lateral displacements and to come across a relation between the actual lateral displacements and the stiffness parameters.

A constant average stiffness was estimated for each layer in order to obtain lateral displacements. On every section analyzed, different elasticity modulus values were deployed ranging from 10 MPa to 450 MPa and graphs of the variation of the lateral displacement with modulus for each case are generated. From these graphs corresponding modulus values are obtained for all case studies.

6.3.1. BJK Fulya Complex

The finite element back analysis of BJK Fulya Complex was performed on the cross-section, in Figure 6.1, where Inclinometer No. 1 is placed. Subsoil parameters of the cross-section are tabulated in Table 6.3. The surcharge load of q=15 kPa employed in the model. The slope geometry and the variation of the lateral displacements with elasticity modulus are presented in Figure 6.2 and Figure 6.3, respectively

Table 6.3. Subsoil parameters of Case No. 1 - BJK Fulya Complex

Parameter	**Name**	**Greywacke**	**Unit**
Material Model	*Model*	*Mohr-Coulomb*	-
Type of material behaviour	*Type*	*Drained*	-
Soil unit weight above phreatic line	γ_{unsat}	20	kN/m^3
Poisson's ratio	ν	0.25	-
Cohesion	c_{ref}	5.0	kN/m^2
Friction angle	φ	33	^{o}
Dilatancy angle	ψ	0.0	^{o}
Interface reduction factor	R_{inter}	0.8	-

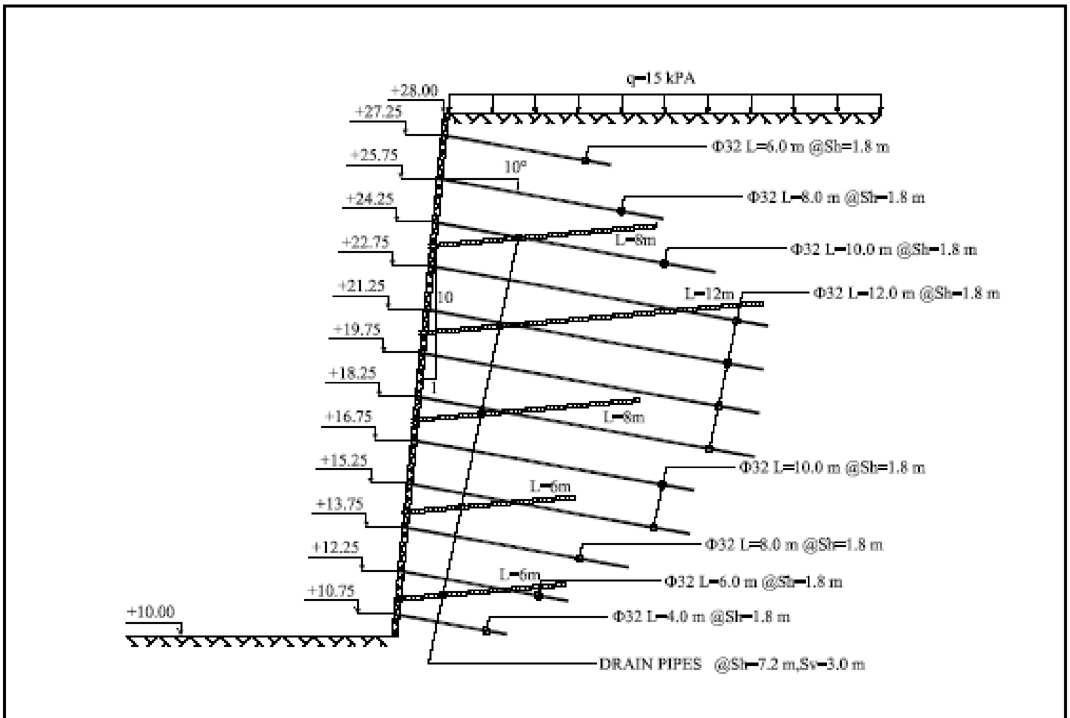


Figure 6.1. Case No. 1 - BJK Fulya Complex, detailed cross-section of inclinometer 1

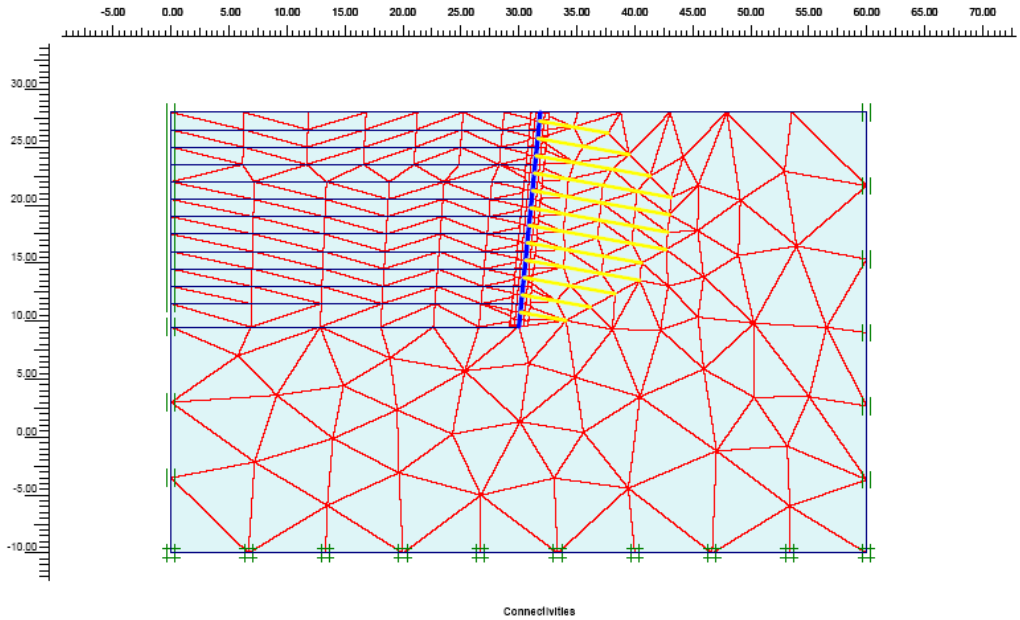


Figure 6.2. Slope geometry and mesh model of the cross-section for Case No. 1

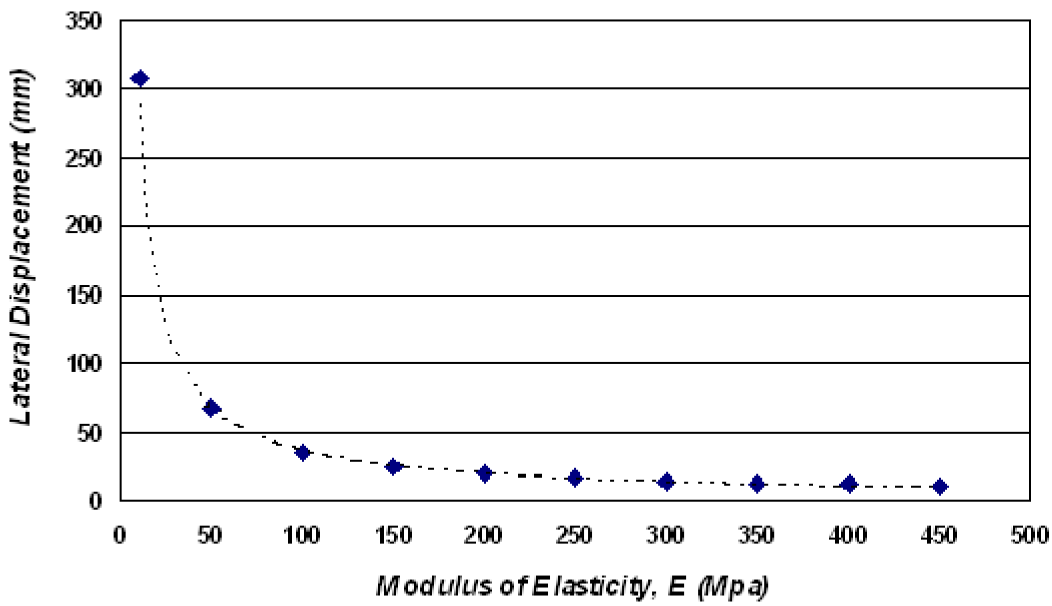


Figure 6.3. Variation of the lateral displacement with modulus for Case No. 1

Figure 6.3 emphasizes that lateral displacement and modulus elasticity are indirectly proportional. Measured lateral displacement of the cross-section is δ_h=14.0 mm. From Figure 6.3 representing modulus is about 300 MPa. Lateral displacement of the analyzed section with E_{ref} =300 MPa is presented in Figure 6.4.

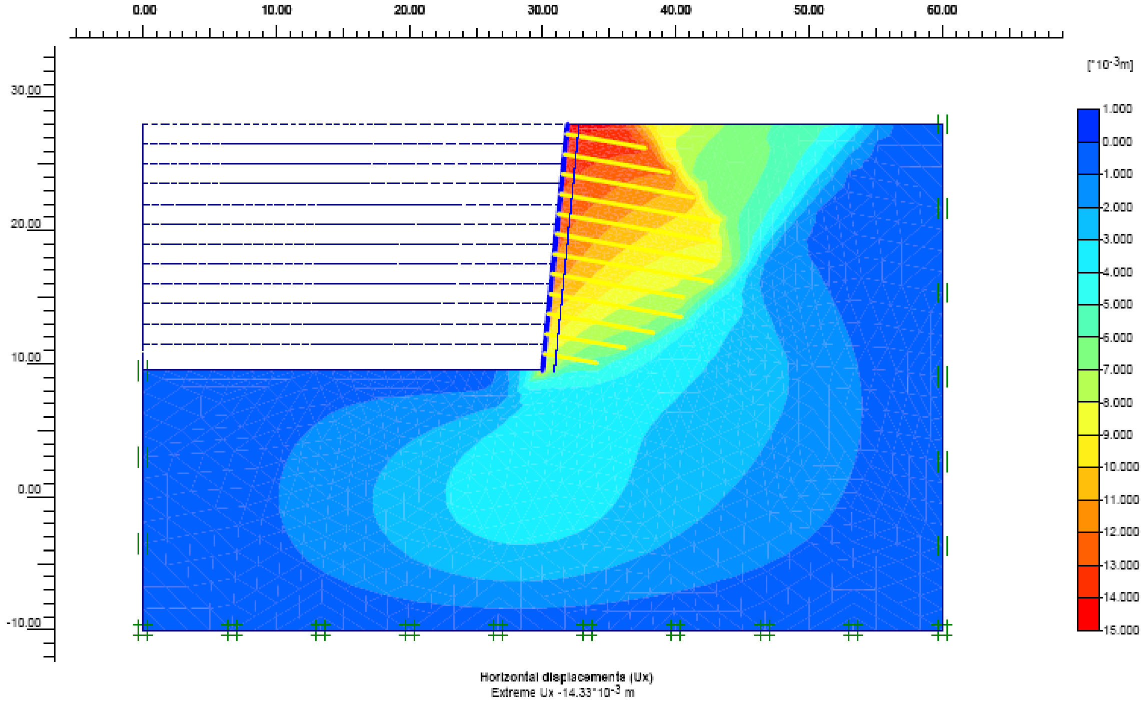


Figure 6.4. Lateral displacement for Case No. 1 with E_{ref} =300 MPa

6.3.2. Istinye Park Complex

The finite element back analysis of Istinye Park Complex was performed on the cross-section, in Figure 6.5, where Inclinometer No. 4 is placed. Subsoil parameters of the cross-section are tabulated in Table 6.4. The surcharge load of q=10 kPa employed in the model. The slope geometry and the variation of the lateral displacements with elasticity modulus are presented in Figure 6.6 and Figure 6.7, respectively

Table 6.4. Subsoil parameters of Case No. 2 - Istinye Park Complex

Parameter	**Name**	**Greywacke**	**Unit**
Material Model	*Model*	*Mohr-Coulomb*	-
Type of material behaviour	*Type*	*Drained*	-
Soil unit weight above phreatic line	γ_{unsat}	20	kN/m^3
Poisson's ratio	ν	0.35	-
Cohesion	c_{ref}	5.0	kN/m^2
Friction angle	φ	36	°
Dilatancy angle	ψ	0.0	°
Interface reduction factor	R_{inter}	0.8	-

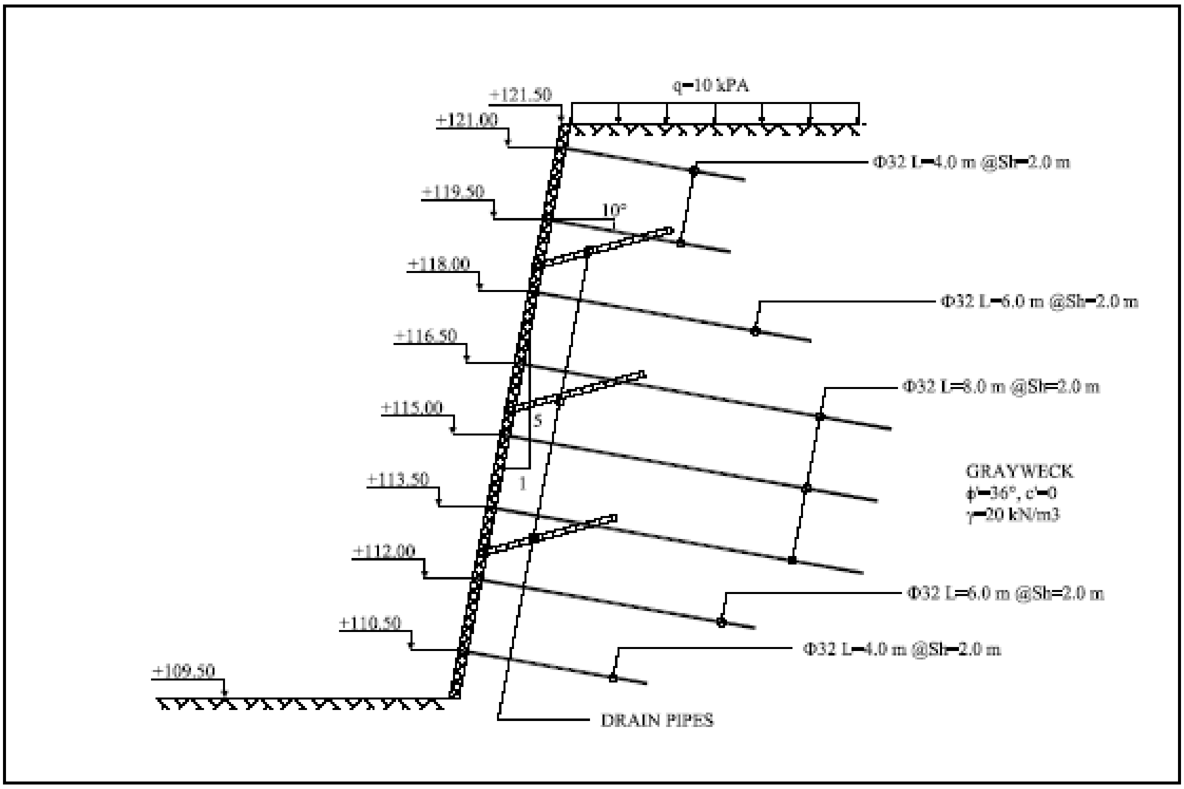


Figure 6.5. Case No. 2 – Istinye Park Complex, detailed cross-section of inclinometer 4

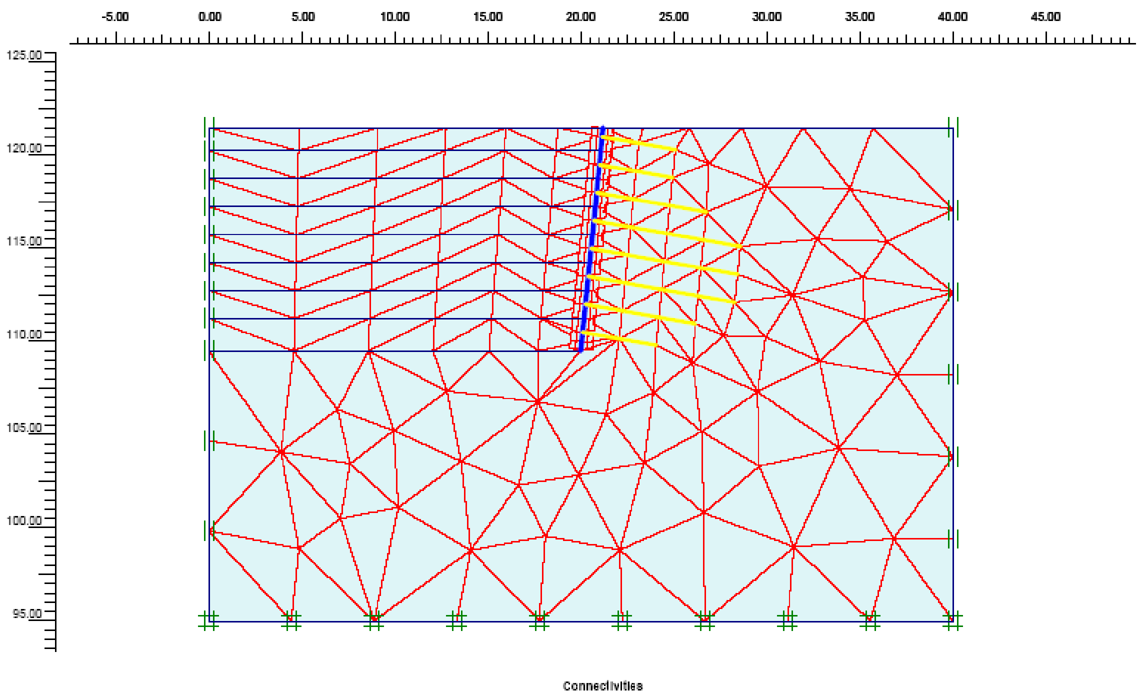


Figure 6.6. Slope geometry and mesh model of the cross-section for Case No. 2

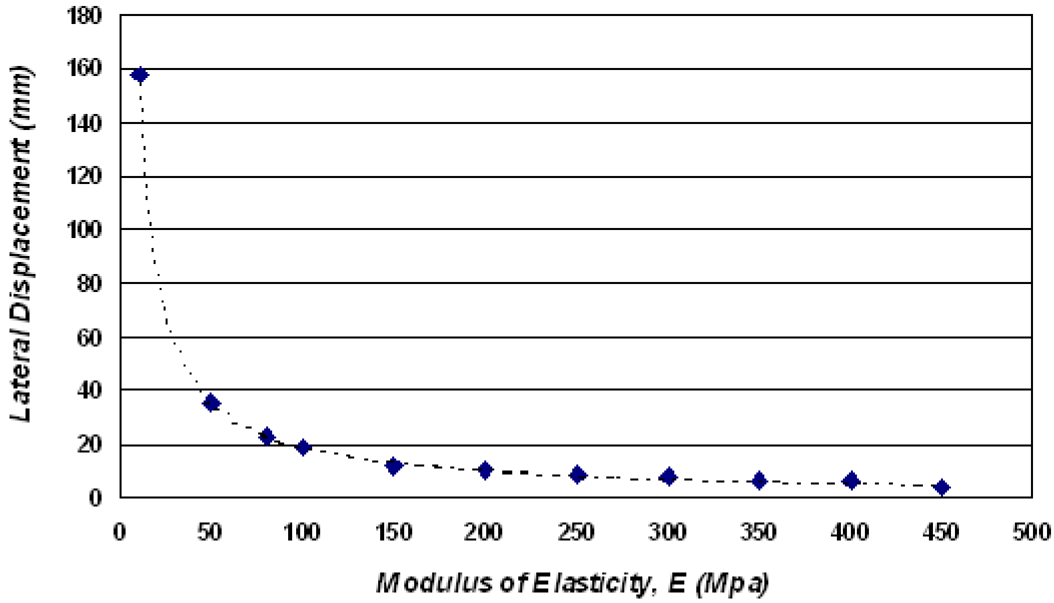


Figure 6.7. Variation of the lateral displacement with modulus for Case No. 2

Measured lateral displacement of the cross-section is δ_h=24.7 mm. From Figure 6.7 representing modulus is about 80 MPa. Therefore the section is analyzed for E_{ref} = 80 MPa and lateral displacement of the analyzed section is presented in Figure 6.8.

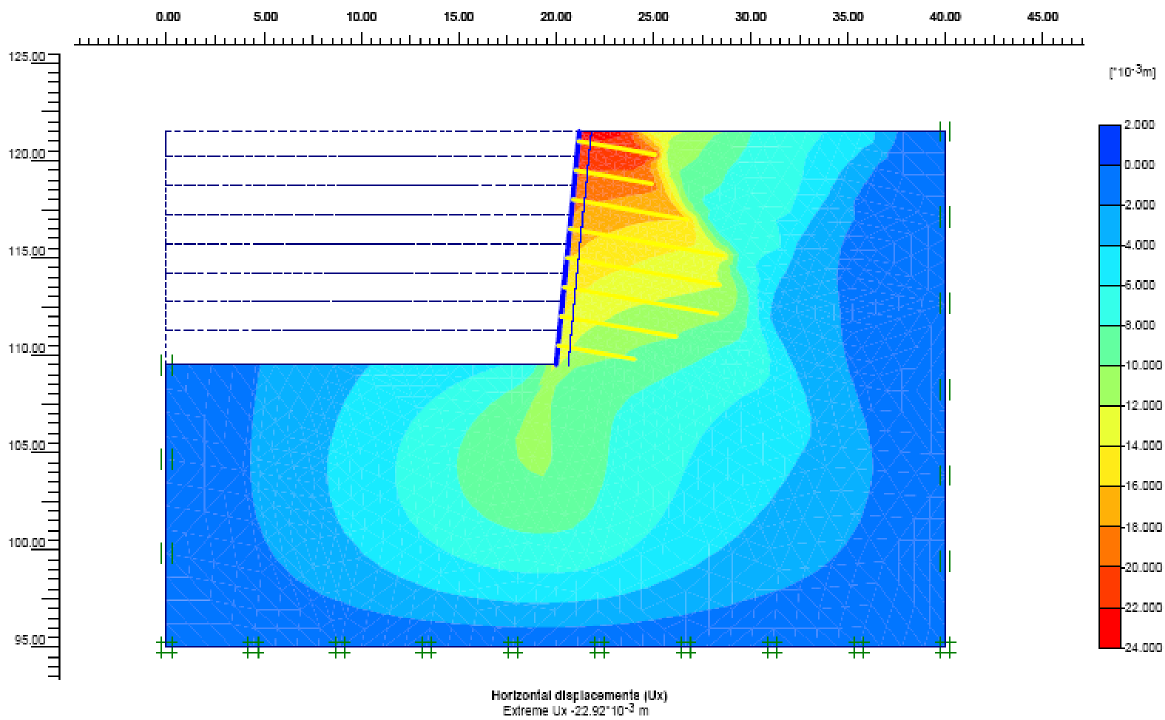


Figure 6.8. Lateral displacement for Case No. 2 with E_{ref} =80 MPa

6.3.3. Kanyon Complex

The finite element back analysis of Kanyon Complex was performed on the cross-section, in Figure 6.9, where Inclinometer No. 7 is placed. Subsoil parameters of the cross-section are tabulated in Table 6.5. There is no surcharge load. The slope geometry and the variation of the lateral displacements with elasticity modulus are presented in Figure 6.10 and Figure 6.11, respectively

Table 6.5. Subsoil parameters of Case No. 3 - Kanyon Complex

Parameter	**Name**	**Greywacke**	**Unit**
Material Model	*Model*	*Mohr-Coulomb*	-
Type of material behaviour	*Type*	*Drained*	-
Soil unit weight above phreatic line	γ_{unsat}	21	kN/m^3
Poisson's ratio	v	0.25	-
Cohesion	c_{ref}	5.0	kN/m^2
Friction angle	φ	37	°
Dilatancy angle	ψ	0.0	°
Interface reduction factor	R_{inter}	0.8	-

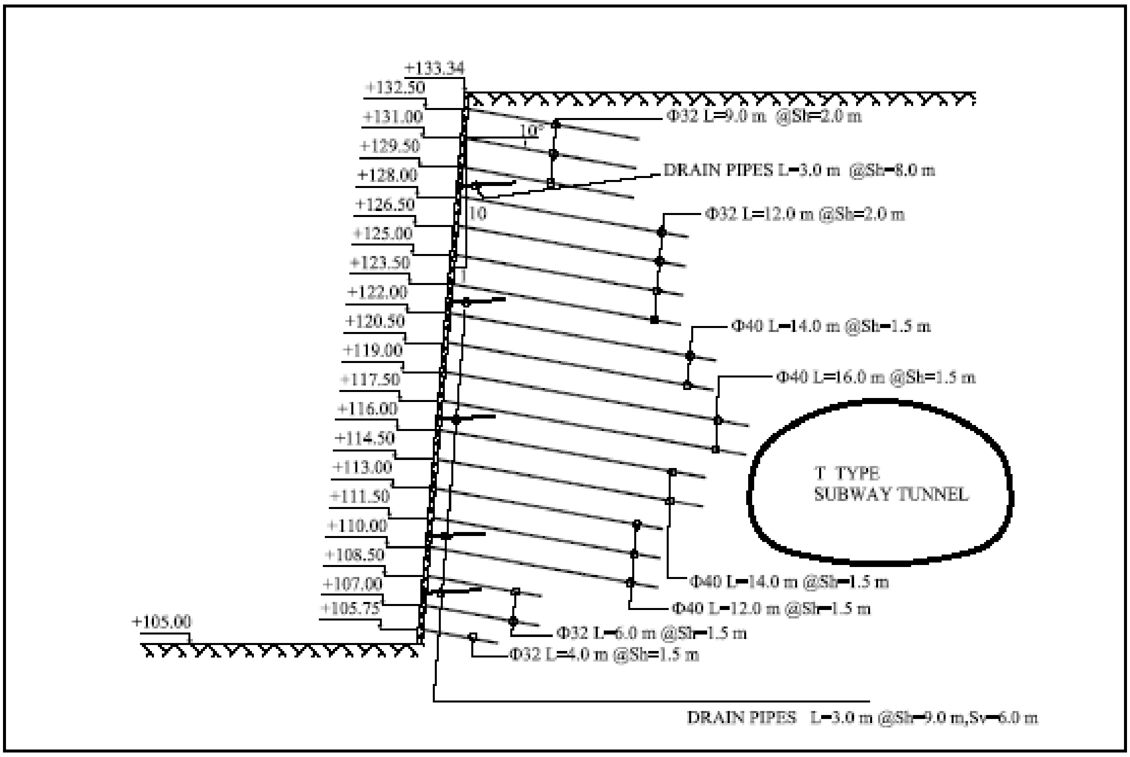


Figure 6.9. Case No. 3 – Kanyon Complex, detailed cross-section of inclinometer 7

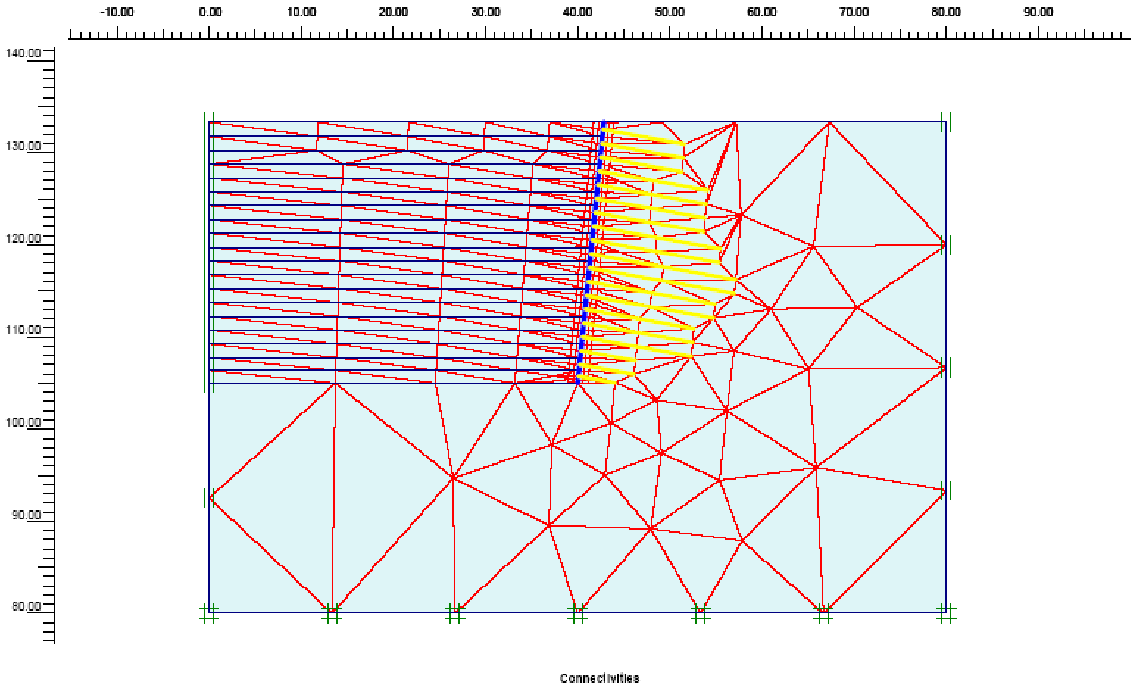


Figure 6.10. Slope geometry and mesh model of the cross-section for Case No. 3

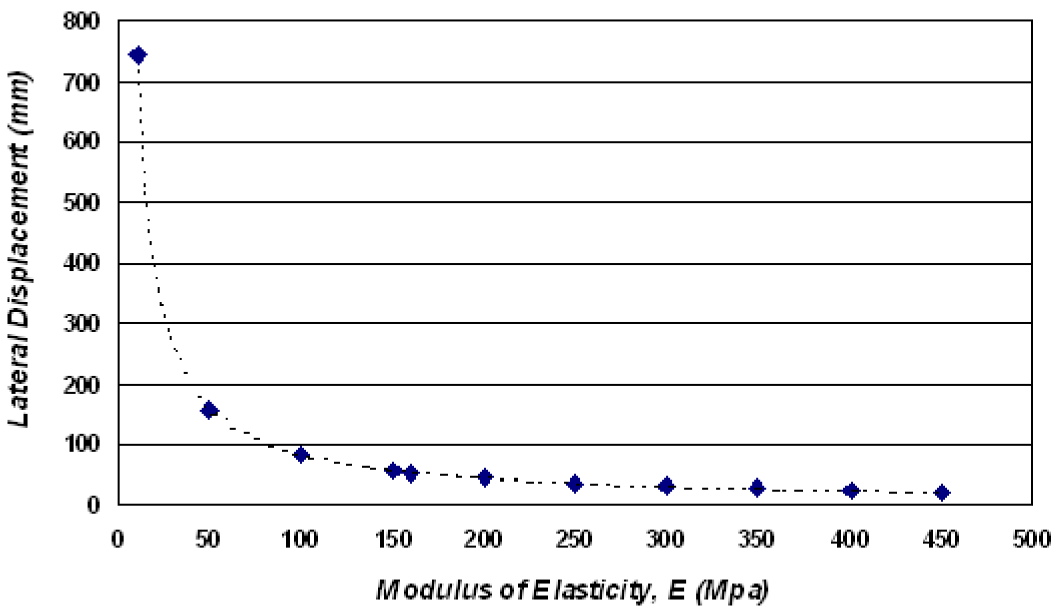


Figure 6.11. Variation of the lateral displacement with modulus for Case No. 3

Measured lateral displacement of the cross-section is δ_h=54.8 mm. From Figure 6.11 representing modulus is about 160 MPa. Therefore the section is analyzed for E_{ref} = 160 MPa and lateral displacement of the analyzed section is presented in Figure 6.12.

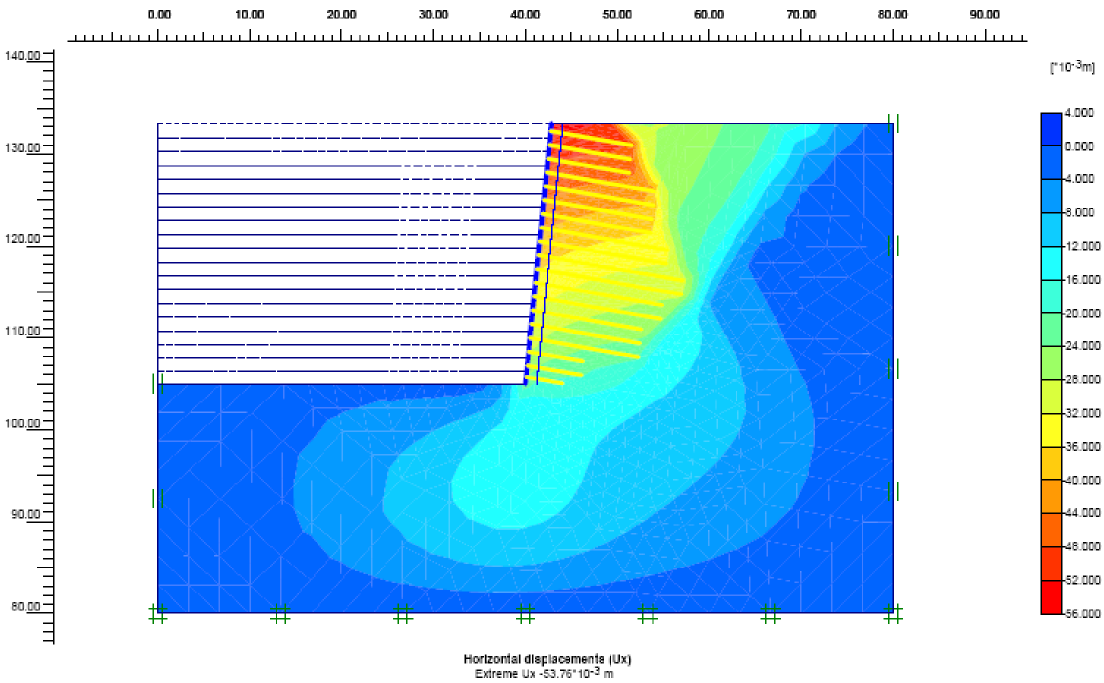


Figure 6.12. Lateral displacement for Case No. 3 with E_{ref} =160 MPa

6.3.4. Mashattan Residence

The finite element back analysis of Mashattan Residence was performed on the cross-section, in Figure 6.13, where Inclinometer No. 1 is placed. Subsoil parameters of the cross-section are tabulated in Table 6.6. There is a soil layer above the soil nailed wall sloped at an angle about 45°. The slope geometry and variation of lateral displacements with elasticity modulus are presented in Figure 6.14 and Figure 6.15, respectively.

Table 6.6. Subsoil parameters of Case No. 4 - Mashattan Residence

Parameter	**Name**	**Greywacke**	**Unit**
Material Model	*Model*	*Mohr-Coulomb*	-
Type of material behaviour	*Type*	*Drained*	-
Soil unit weight above phreatic line	γ_{unsat}	20	kN/m^3
Poisson's ratio	ν	0.30	-
Cohesion	c_{ref}	5.0	kN/m^2
Friction angle	φ	33	°
Dilatancy angle	ψ	0.0	°
Interface reduction factor	R_{inter}	0.8	-

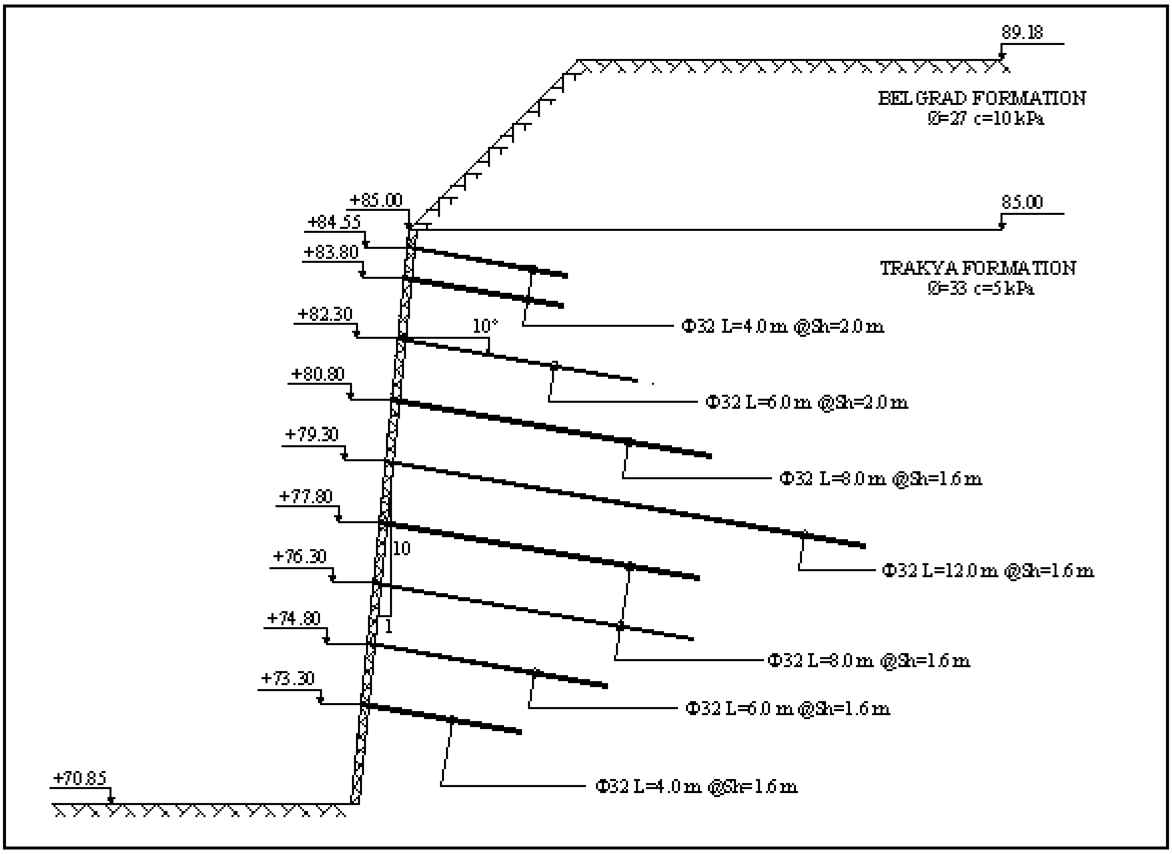


Figure 6.13. Case No. 4 – Mashattan Residence, detailed cross-section of inclinometer 1

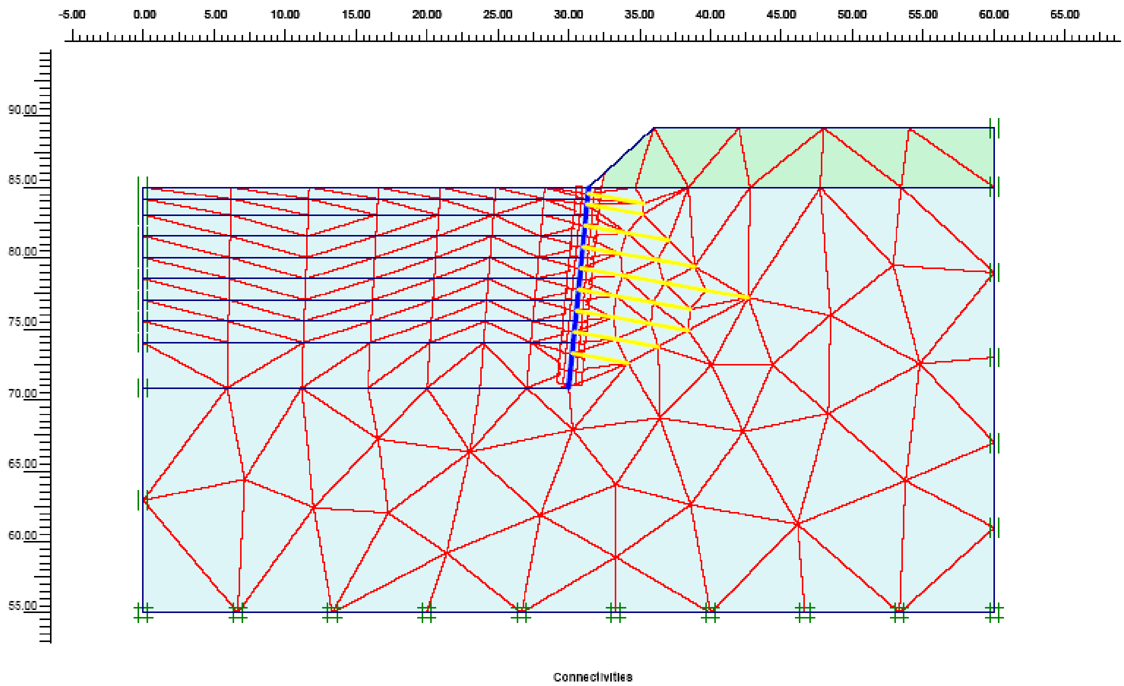


Figure 6.14. Slope geometry and mesh model of the cross-section for Case No. 4

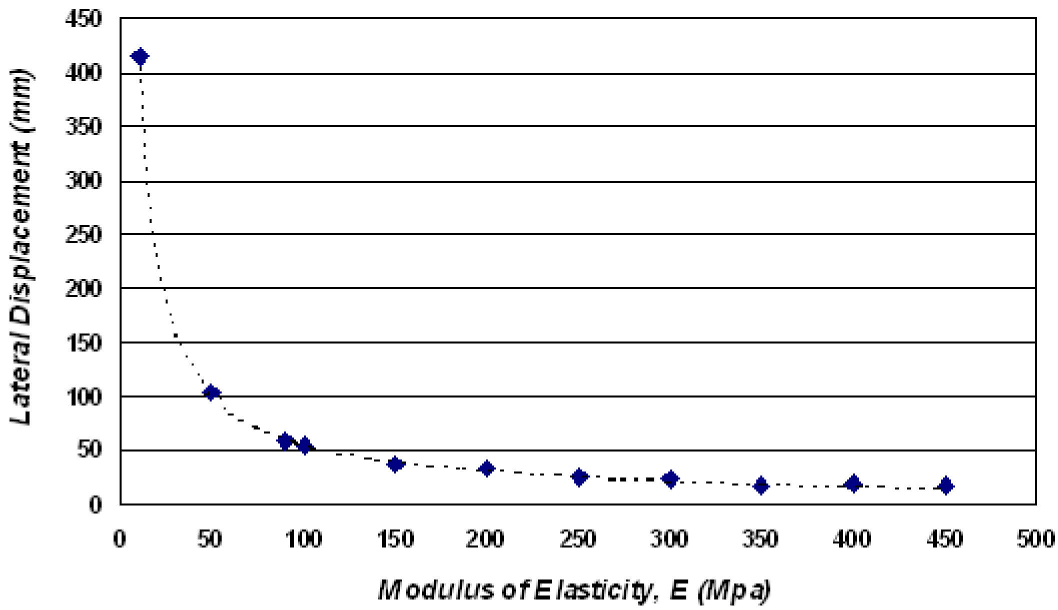


Figure 6.15. Variation of the lateral displacement with modulus for Case No. 4

Measured lateral displacement of the cross-section is δ_h=59.3 mm. From Figure 6.15 representing modulus is about 90 MPa. Therefore the section is analyzed for E_{ref} = 90 MPa and lateral displacement of the analyzed section is presented in Figure 6.16.

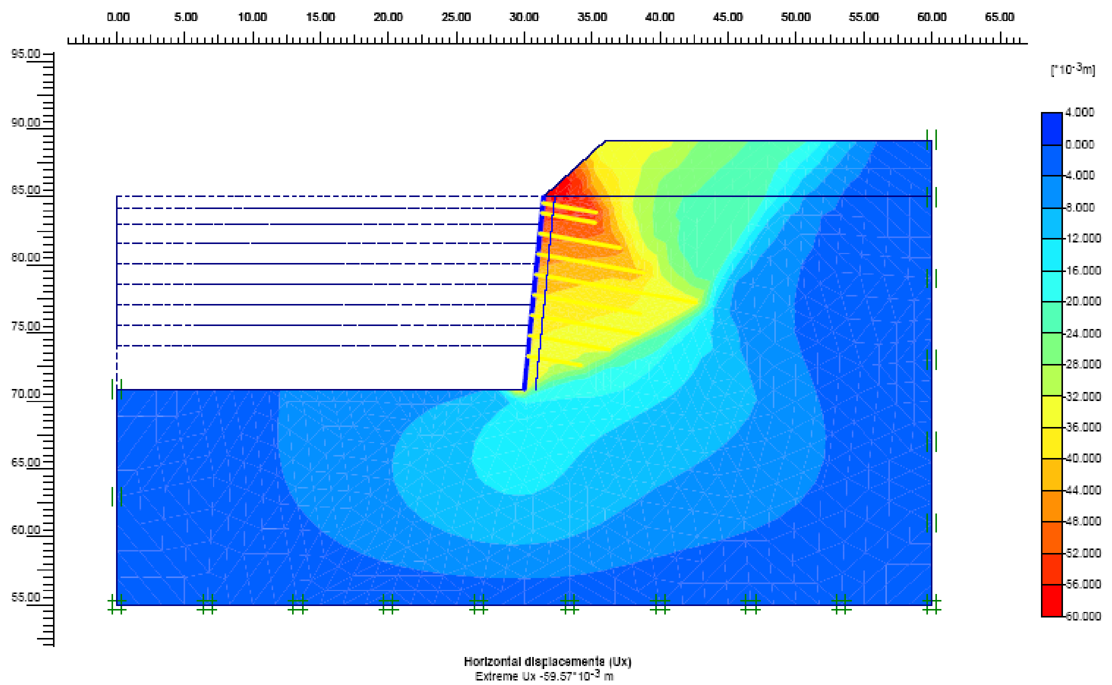


Figure 6.16. Lateral displacement for Case No. 4 with E_{ref} =90 MPa

6.3.5. Tepe Shopping Mall

The finite element back analysis of Tepe Shopping Mall was performed on the cross-section, in Figure 6.17, where Inclinometer No. 4 is placed. Subsoil parameters of the cross-section are tabulated in Table 6.7. The surcharge load of q=30 kPa employed in the model. The slope geometry and the variation of the lateral displacements with elasticity modulus are presented in Figure 6.18 and Figure 6.19, respectively

Table 6.7. Subsoil parameters of Case No. 5 - Tepe Shopping Mall

Parameter	**Name**	**Greywacke**	**Unit**
Material Model	*Model*	*Mohr-Coulomb*	-
Type of material behaviour	*Type*	*Drained*	-
Soil unit weight above phreatic line	γ_{unsat}	19	kN/m^3
Poisson's ratio	ν	0.35	-
Cohesion	c_{ref}	5.0	kN/m^2
Friction angle	φ	33	°
Dilatancy angle	ψ	0.0	°
Interface reduction factor	R_{inter}	0.8	-

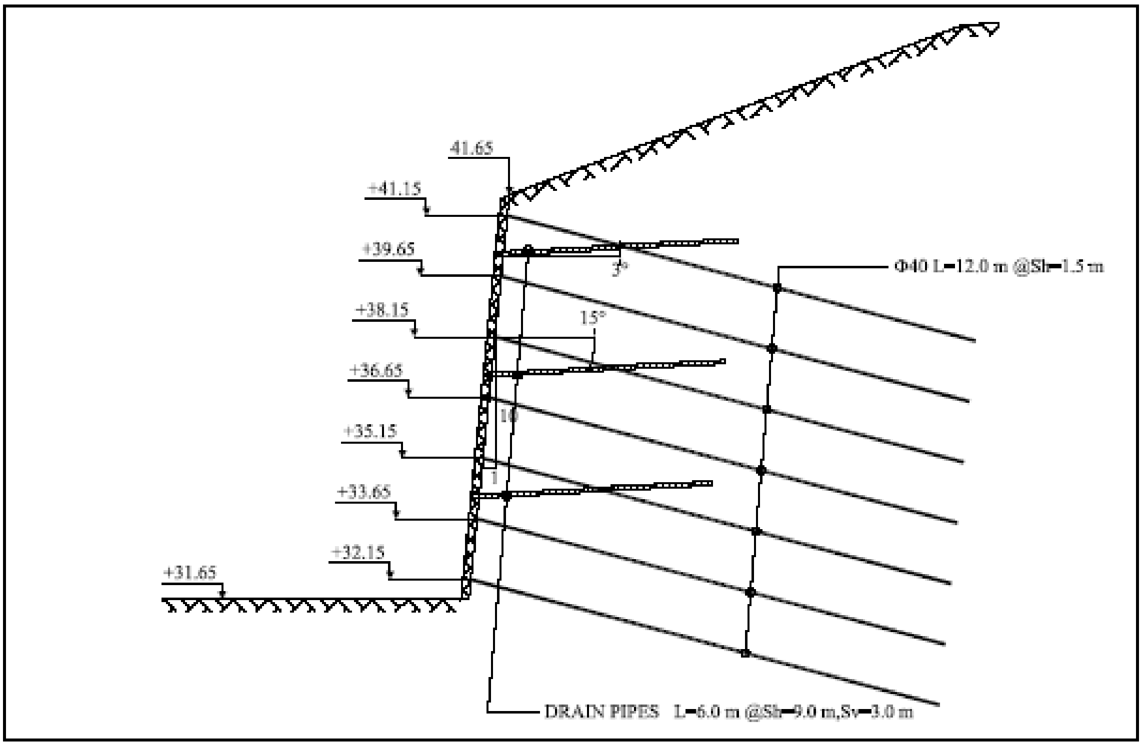


Figure 6.17. Case No. 5 – Tepe Shopping Mall, detailed cross-section of inclinometer 4

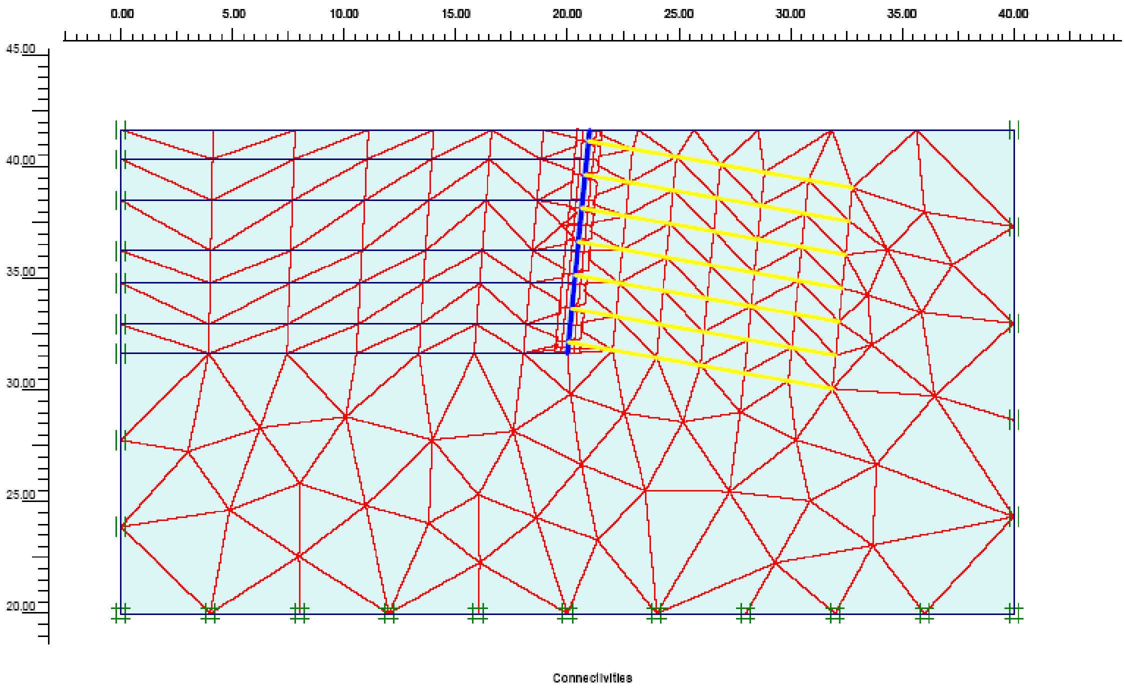


Figure 6.18. Slope geometry and mesh model of the cross-section for Case No. 5

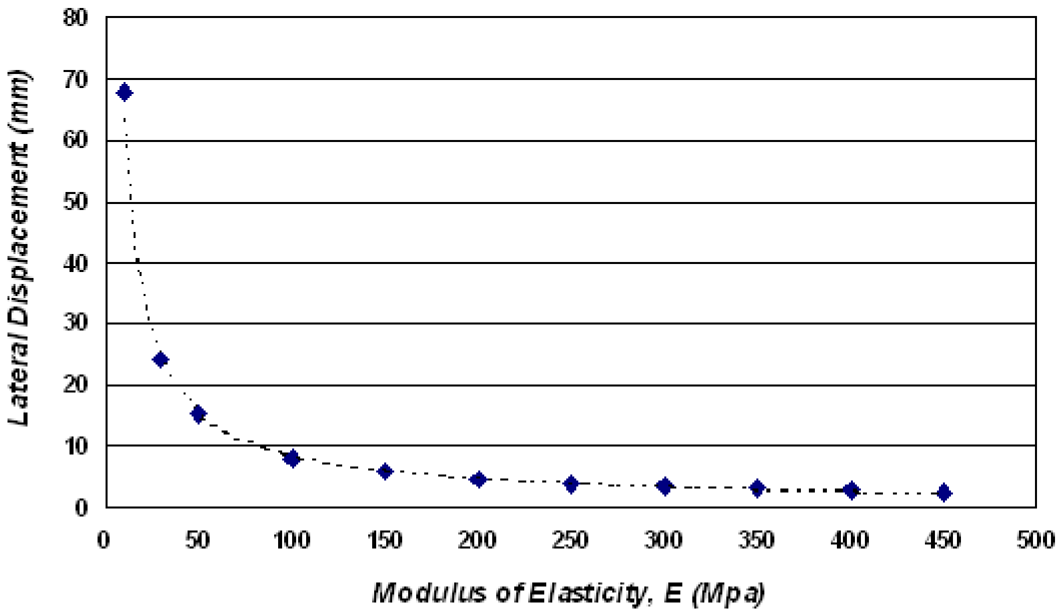


Figure 6.19. Variation of the lateral displacement with modulus for Case No. 5

Measured lateral displacement of the cross-section is δ_h=24.3 mm. From Figure 6.19 representing modulus is about 30 MPa. Therefore the section is analyzed for E_{ref} = 30 MPa and lateral displacement of the analyzed section is presented in Figure 6.20.

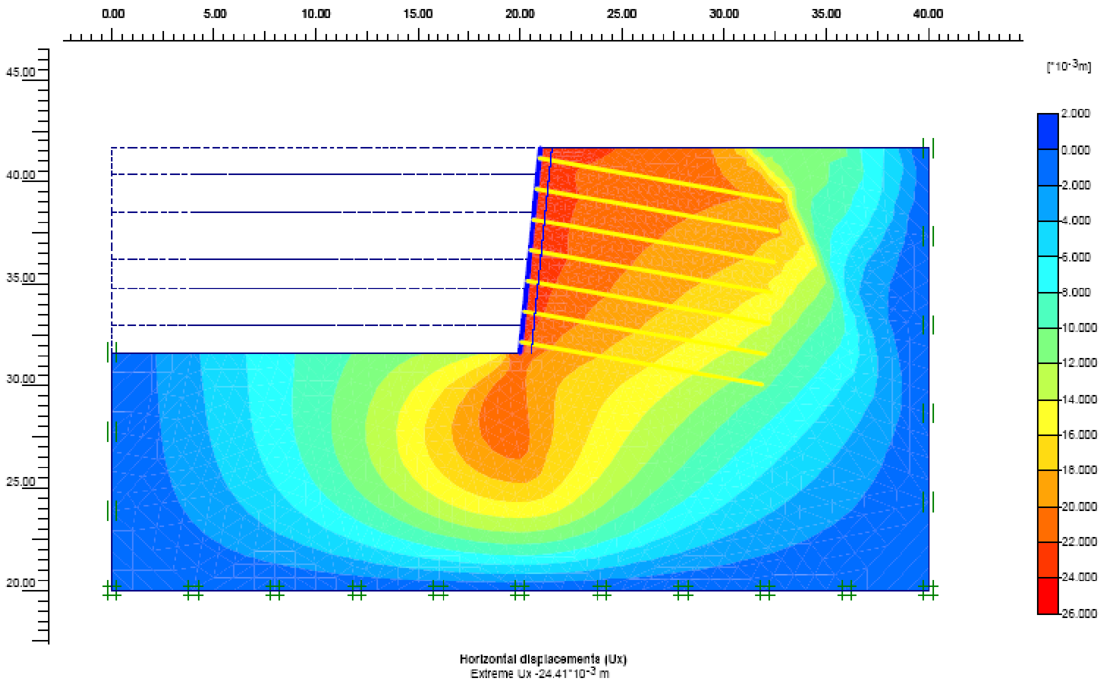


Figure 6.20. Lateral displacement for Case No. 5 with E_{ref} =30 MPa

6.3.6. Besler Warehouse

The finite element back analysis of Besler Warehouse was performed on the cross-section, in Figure 6.21, where Inclinometer No. 1 is placed. Subsoil parameters of the cross-section are tabulated in Table 6.8. The surcharge load of q=20 kPa employed in the model. The slope geometry and the variation of the lateral displacements with elasticity modulus are presented in Figure 6.22 and Figure 6.23, respectively.

Table 6.8. Subsoil parameters of Case No. 6 - Besler Warehouse

Parameter	**Name**	**Greywacke**	**Unit**
Material Model	*Model*	*Mohr-Coulomb*	-
Type of material behaviour	*Type*	*Drained*	-
Soil unit weight above phreatic line	γ_{unsat}	20	kN/m^3
Poisson's ratio	ν	0.35	-
Cohesion	c_{ref}	5.0	kN/m^2
Friction angle	φ	32	^{o}
Dilatancy angle	ψ	0.0	^{o}
Interface reduction factor	R_{inter}	0.8	-

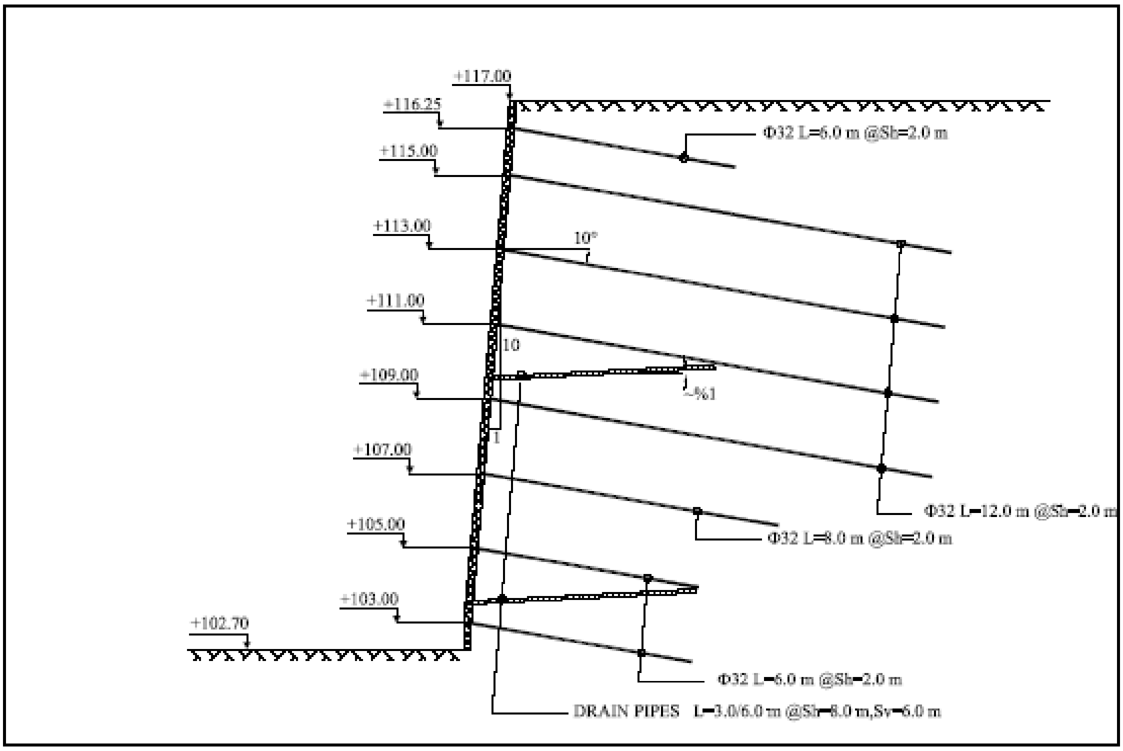


Figure 6.21. Case No. 6 – Besler Warehouse, detailed cross-section of inclinometer 1

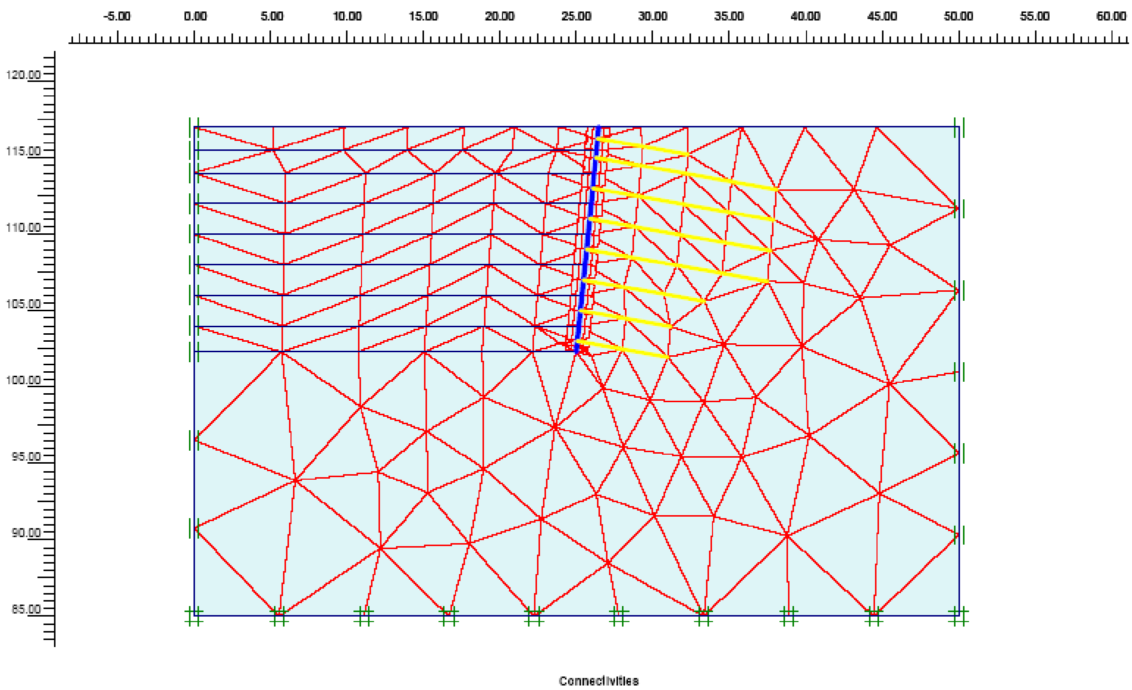


Figure 6.22. Slope geometry and mesh model of the cross-section for Case No. 6

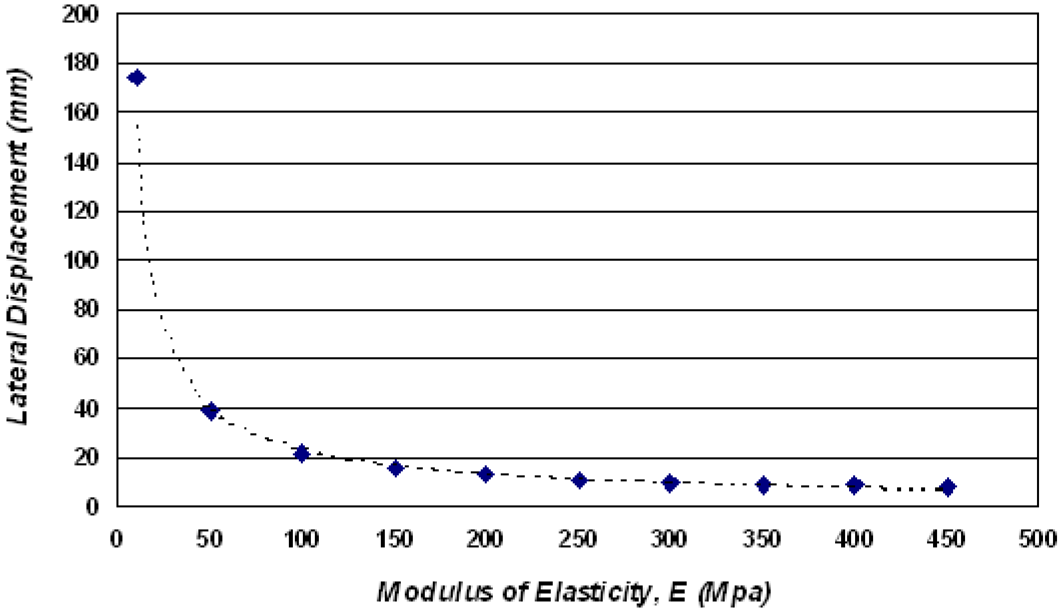


Figure 6.23. Variation of the lateral displacement with modulus for Case No. 6

Measured lateral displacement of the cross-section is δ_h=10.2 mm. From Figure 6.23 representing modulus is about 250 MPa. Lateral displacement of the analyzed section with E_{ref}=250 MPa is presented in Figure 6.24.

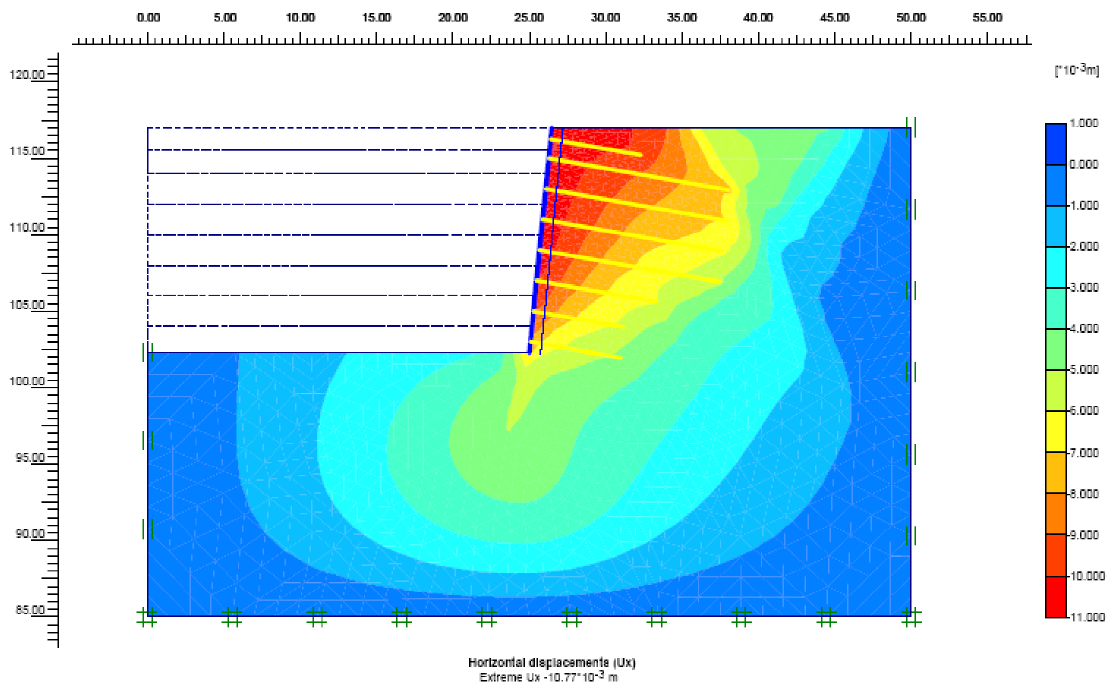


Figure 6.24. Lateral displacement for Case No. 6 with E_{ref}=250 MPa

6.4. In-depth Numerical Analysis for a Cross-section

Detailed numerical analysis was performed on the soil nailed cross-section where Inclinometer 7 of Case No. 3 – Kanyon Complex readings were made. In the previous numerical analysis it is established that the constant average stiffness of the cross-section is E_{ref} = 160 MPa. Due to this constant stiffness, computations tend to be relatively fast and first impressions on deformations can be obtained (Brinkgreeve, 2002).

In real soils, the stiffness depends on the stress level, therefore stiffness generally increases with depth. On the other hand, when using the Mohr-Coulomb model, the stiffness is a constant value. In order to account for the increase of the stiffness with depth while using 'PLAXIS', $E_{increment}$ value can be used which is the increase of the modulus of elasticity per unit of depth (Brinkgreeve, 2002).

Although this approach is convenient for the design, in reality, increase of the stiffness of soil or rock with depth is not perfectly linear. In this study subsoil behind the soil nailed wall was divided into several layers for defining different stiffness parameters in order to match the actual lateral displacements and hence actual subsoil profile in terms of stiffness were achieved. Subsoil was divided into 24 layers and by means of inclinometer readings which were recorded at a certain time interval and every excavation stage stiffness parameters were adjusted to match the measured lateral displacements on every calculation step using the method of "trial and error".

Since there are 19 rows of soil nails and therefore 19 stages of excavations made, same number of calculation step was deployed in the numerical analysis. For the relative simplicity of the calculations, subsoil behind the wall is divided into 19 layers, having a thickness of 1.5 m, except for the last layer which has the thickness of 1.34 m, same as the excavation stages. Subsoil below the wall is subdivided into 5 layers which have equal thickness of 5 m. Figure 6.25 illustrates the slope geometry and subsoil layering of the cross-section, Case No. 3 Inclinometer 7 while Figure 6.26 shows the to dimensional finite element mesh model of the cross-section.

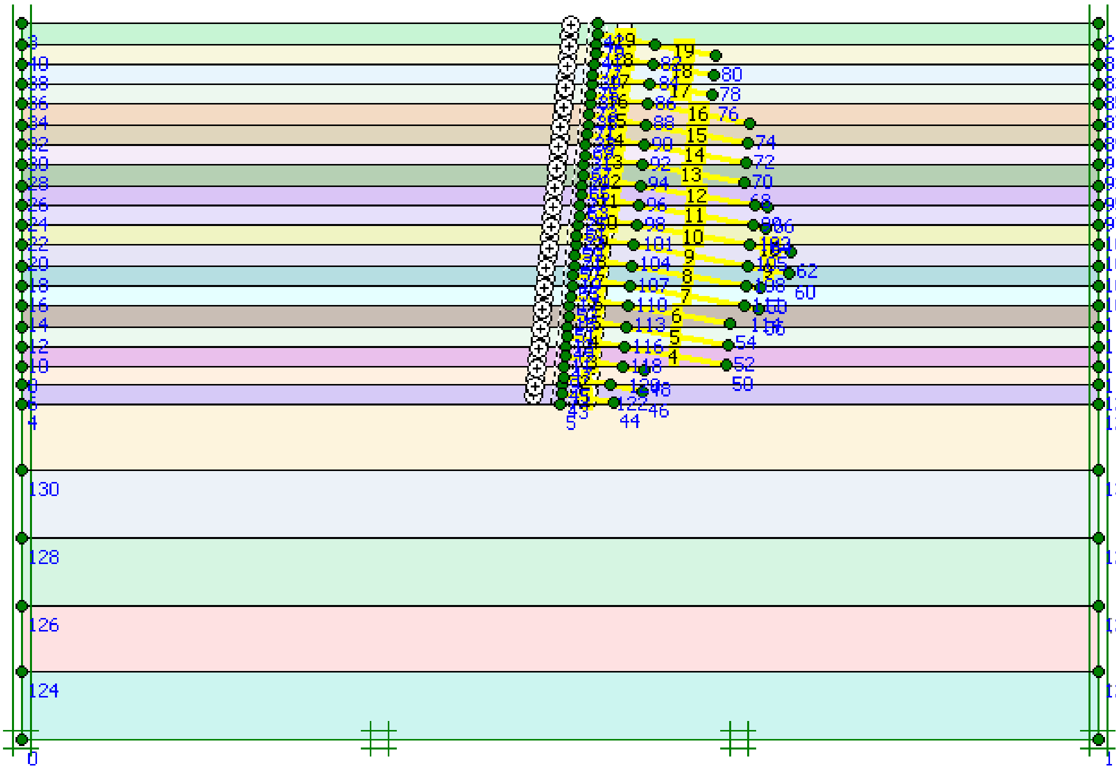


Figure 6.25. The slope geometry and subsoil layering of Case No. 3, Inclinometer 7

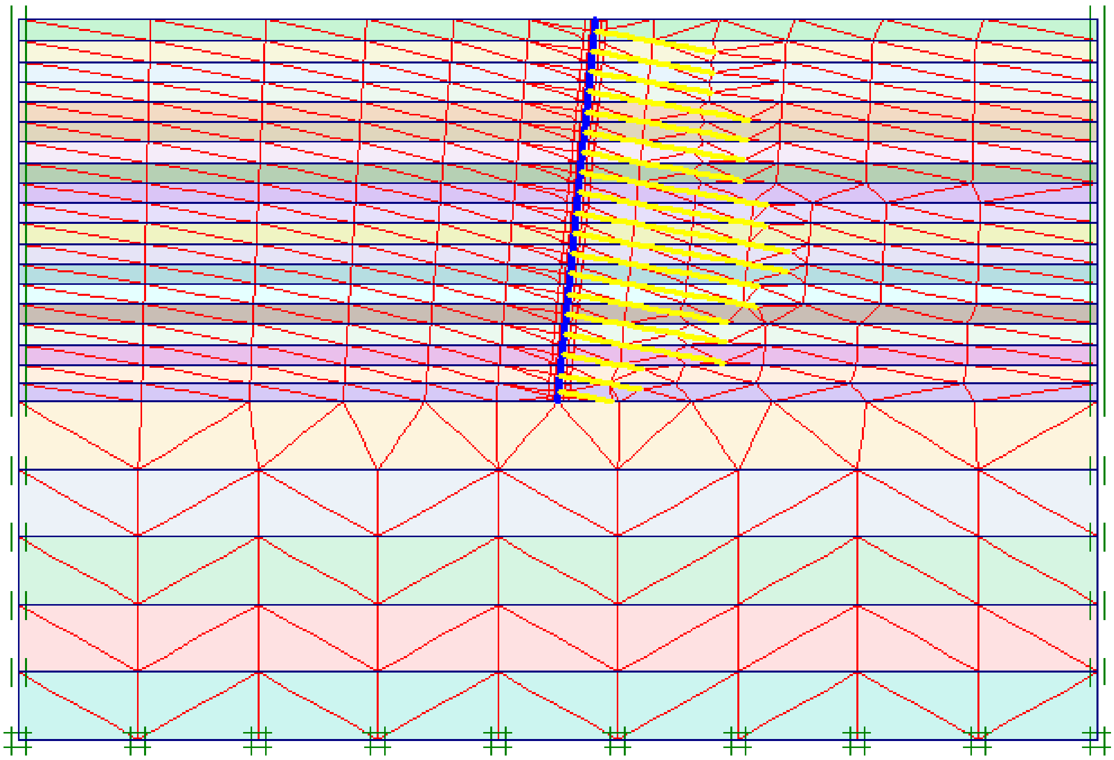


Figure 6.26. Mesh generation for Case No. 3, Inclinometer 7

In order to obtain close lateral displacement values from the model with the measured ones, an iterative procedure has to be implemented. Because the stiffness of the subsoil layers, even if it is not excavated, effects the lateral displacements of upper layers. That means for every calculation step needs adjustment of the all soil layers, therefore after changing stiffness parameter of any subsoil layer, calculation has to start from the beginning and deformations occurred on individual calculation steps must be checked and confirmed.

According to the above iterative procedure, lateral displacements of each individual calculation step, which are very close to the observed deformations, are obtained. Deformations of the soil nailed cross-section by means of horizontal displacements are presented for excavation depths of 7.5 m, 15 m, 22.5 m and final excavation depth of 28.3 m in Figures 6.27 through 6.30. Figure 6.31 gives the comparison of the modeled and actual lateral displacement with depth. Variation of elasticity modulus with depth in the numerical analysis is shown in Figure 6.32. As it is seen in the figure that stiffness of the greywacke formation increases with depth, but it is not a linear relationship since it depends on many factors and depth is just one of them.

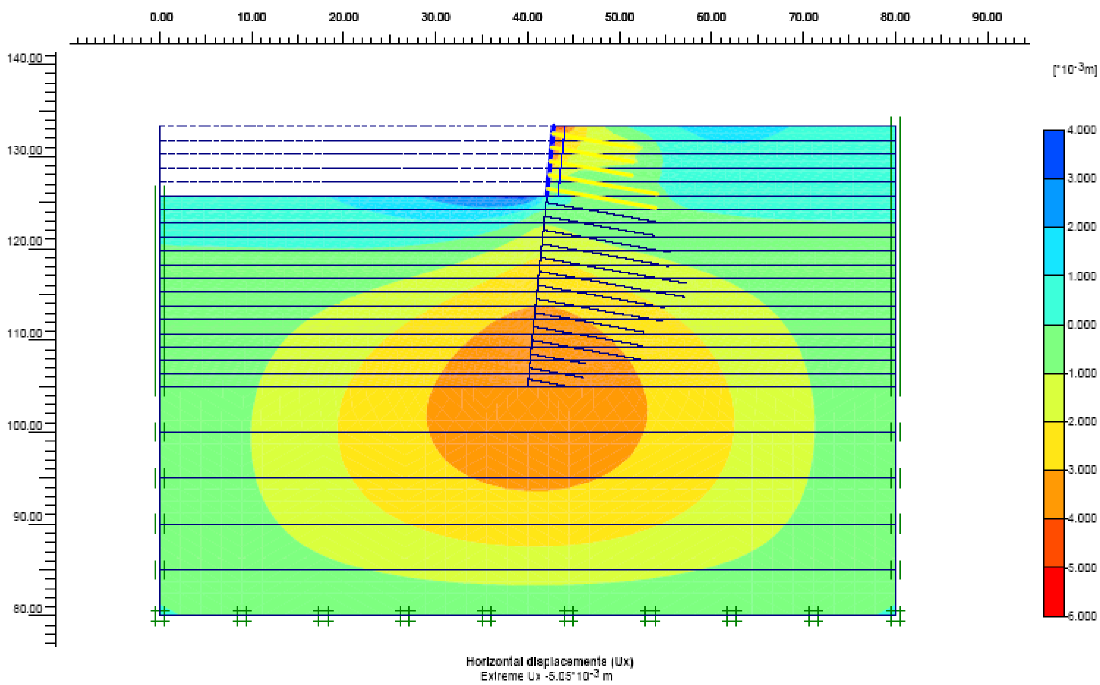


Figure 6.27. Lateral displacements for excavation depth of 7.5 m

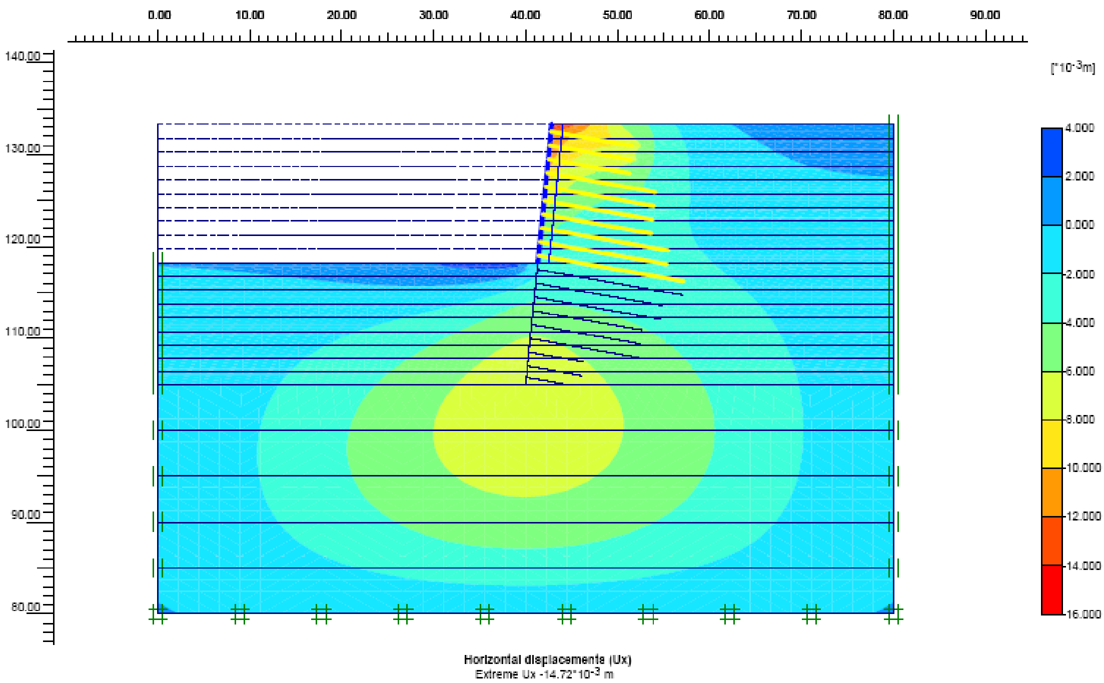


Figure 6.28. Lateral displacements for excavation depth of 15 m

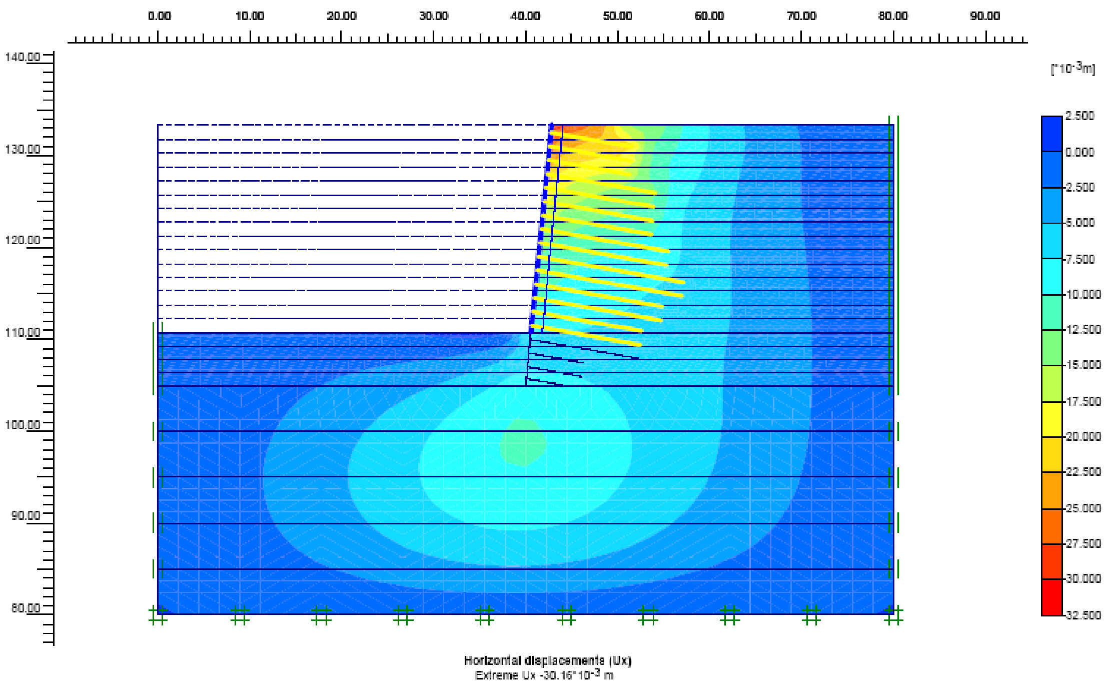


Figure 6.29. Lateral displacements for excavation depth of 22.5 m

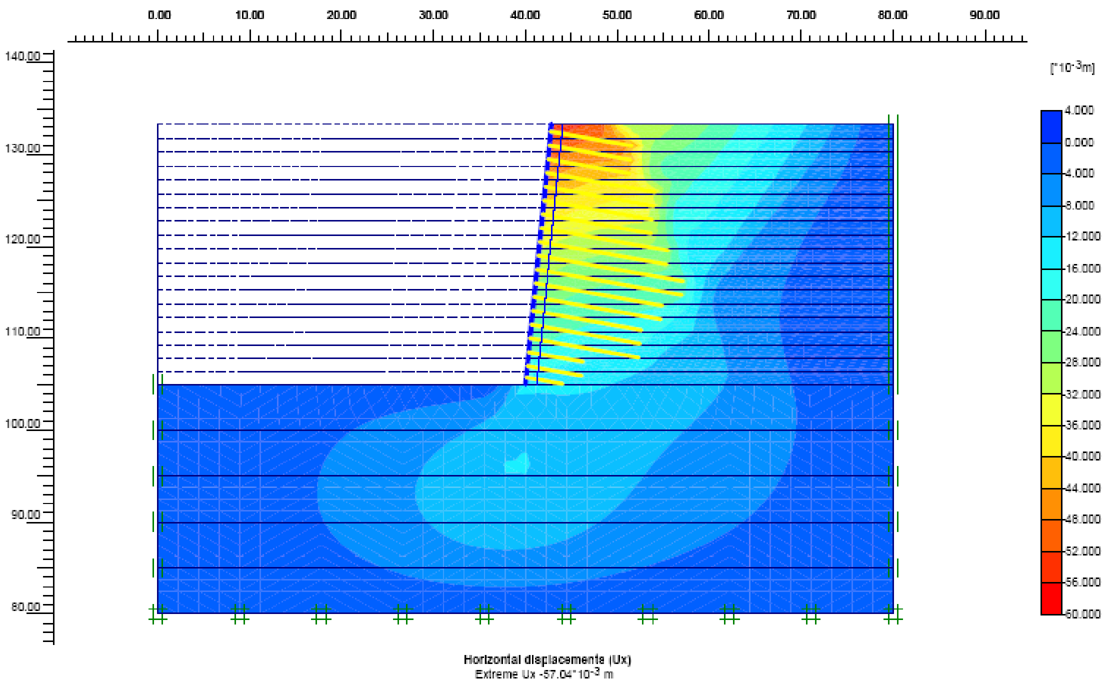


Figure 6.30. Lateral displacements for excavation depth of 28.3 m

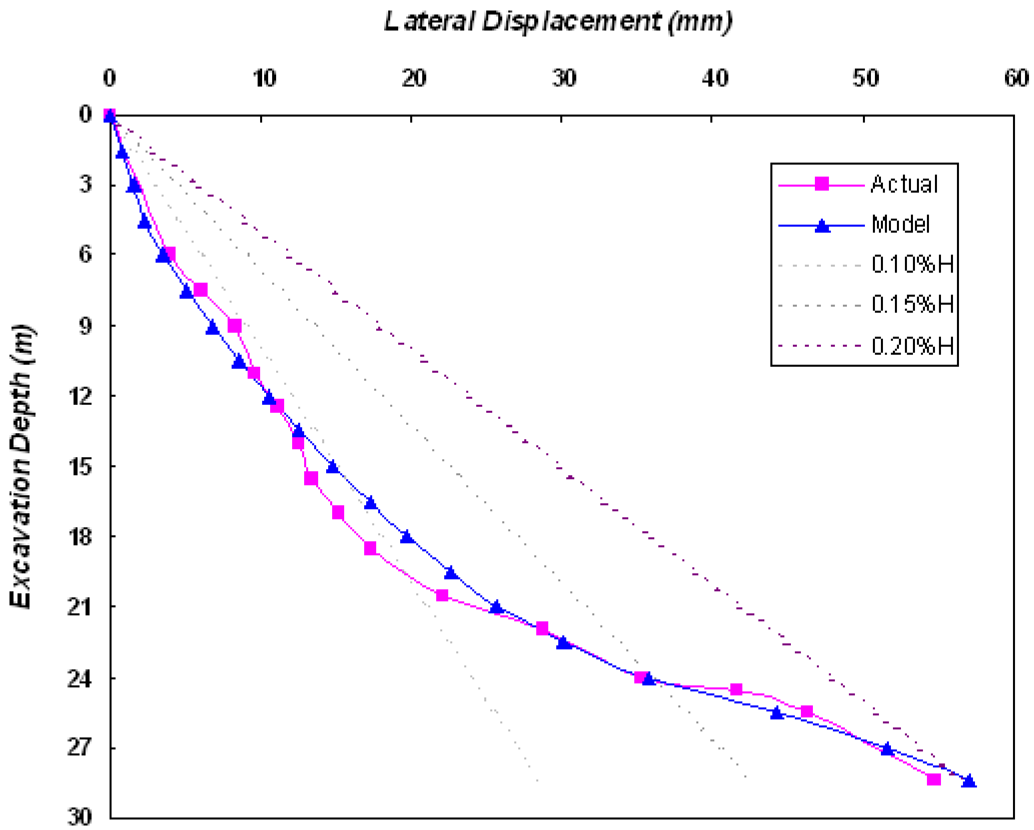


Figure 6.31. Comparison of the modeled and actual lateral displacement with depth

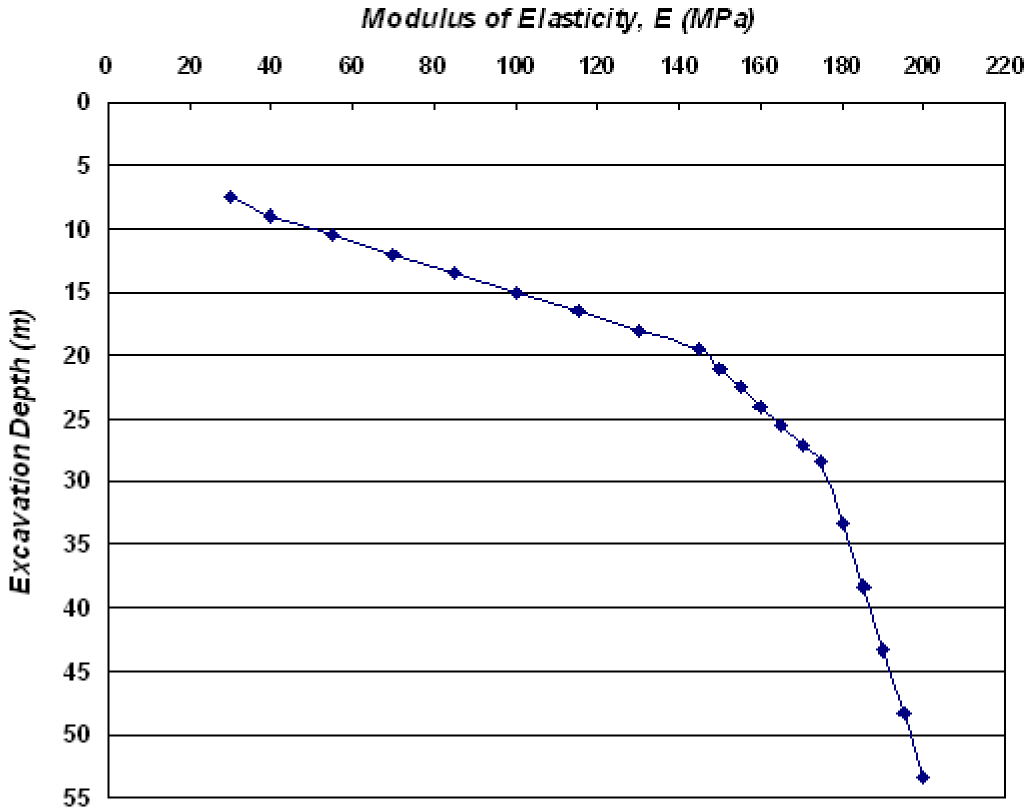


Figure 6.32. Variation of elasticity modulus with depth

6.5. Concluding Remarks

The most important deficiency of the limit equilibrium design methods is that they do not provide a prediction of deformations. Deformations can be predicted approximately using empirical correlations and while these are appropriate in many cases they have limitations. In situations where more confidence is required, a higher level of analysis should be adopted by using numerical modelling such as finite element or finite difference methods (Phear *et al*., 2005).

In this study finite element computer program ‘PLAXIS’ was deployed to simulate the excavation sequence and installation of nails and to carry out back analysis of the six case studies in Istanbul of which performance analysis were achieved by means of inclinometer monitoring of lateral displacements.

By means of back analysis average stiffness values of greywacke formation encountered in the case studies are obtained. Average stiffness values of the subsoil formation of the case studies are tabulated in Table 6.9. Average stiffness values obtained from finite element back analysis are in excellent relation with the previous study.

Table 6.9. Stiffness parameters of case studies of soil nailed walls

No.	Project Name	E_{ref} (MPa)	Subsoil Conditions
1	BJK Fulya Complex	300	fractured silicified **sandstone**
2	Istinye Park Complex	80	extensively fractured **siltstone, claystone**
3	Kanyon Complex	160	extensively fractured **sandstone, siltstone, claystone**
4	Mashattan Residence	90	extensively fractured **siltstone, claystone**
5	Tepe Shopping Mall	30	very extensively fractured **sandstone, siltstone, claystone**
6	Besler Warehouse	250	fractured **sandstone**

Finite element back analysis of the case studies show that lateral displacements are inversely proportional with stiffness and the alteration of the stiffness value changes the lateral displacements considerably. The degradation ratios for greywacke formation to obtain E_m to be utilized in prediction of lateral displacement of soil nailed walls are large as well. (Durgunoglu and Yilmaz, 2007). Therefore, the modulus value of greywackes of Istanbul to be utilized in numerical models such as finite element computer program, PLAXIS, to predict the displacements do vary in a very big range. Consequently, the displacements of soil nailed walls based on selected modulus values are only indicative.

In addition, although the lateral displacements of the soil nail walls can be predicted by estimating an average stiffness of the subsoil layers in numerical modelling, in reality, the stiffness depends on the stress level, therefore stiffness generally increases with depth. Even the computer programs put some incremental parameters with depth to raise the stiffness with depth, it is seen from the back analysis of a cross-section from Case No. 3 Inclinometer 7, the increase of the modulus with depth is not a linear function and the rate of increase of the stiffness decreases with depth, as expected, especially when it reaches the intact bedrock.

The accuracy of numerical modelling depends on the quality of data acquired, the estimation of in-situ stress and soil stiffness and the availability of good case histories to calibrate numerical models. The stiffness parameters in these models should be adjusted to match the values obtained from actual site monitoring results at an early stage. Verification and improvement of design should continue during the construction stage through close observation and monitoring techniques in order to recognize and feed back potential problems and also to make potential cost savings.

As a result, direct measurement of lateral displacements of deep soil nailed walls in Istanbul is compulsory in order to follow performance of such structures during and after construction.

7. HARMONY OF RETAINING SYSTEMS TO VARIOUS LOCAL SUBSOIL CONDITIONS

7.1. Introduction

A significant case study, in which the author involved throughout the soil investigation and modeling, design, construction and monitoring phases as part of this thesis assignment, is presented herein for the utilization of various retaining systems for different subsoil and groundwater conditions encountered within a given site, considering the output of optimization of the cost as well. The project is known as “BJK Fulya Complex” consisting of high-rise residential twin towers, a hospital building and a hotel building covering approximately 160,000 m^2 floor area including a hypermarket, a technomarket, a cultural center, entertainment facilities and underground parking area. The project is located at a very prestigious district of the city, therefore maximum underground space gain were desired. As a result nearly 20 m of excavation was performed partly under groundwater. Figure 7.1 shows the layout plan, where the twin towers are on the right, the hospital block is on the left and the hotel block is in between.

The project site has a very rugged topography having about 25 m difference in elevation in perpendicular direction to the covered old creek located at the bottom of the valley along the main street. Due to unique topography and geology, subsoil and groundwater conditions at various faces of the excavation differ considerably. Furthermore, again due to unique topography, at the hillside in addition to 18.5 m of temporary retaining structure, permanent retaining structure of about 15-20 m high had to be constructed over the temporary wall leading to a retaining structure as high as 36 meters.

Due to complicated geology and the high seismicity of this site, it was necessary to employ extensive soil investigations to identify the limits of various lithological units and the ground water conditions. As a result, various types of retaining structures were employed having both flexible and rigid retaining systems at various locations within the

Figure 7.1. BJK Fulya Complex

site of “BJK Fulya Complex”. Various forms of retaining structures that have been utilized at the site include temporary soil nailing, permanent soil nailing, temporary and permanent soil nailing along with the permanent tied-back cast in-situ reinforced concrete caisson wall and temporary tied-back diaphragm wall consist of soldier cast in-situ piles with jet grout columns in between.

The performances of various systems are closely monitored by means of inclinometer recordings taken at certain time intervals in parallel to the staged excavation. Readings from sixteen inclinometers at different locations were recorded throughout the construction.

Displacement data and experience obtained from this case study together with previous experience (Durgunoglu *et al.*, 2007a) serves an excellent source of data and example for future applications in similar conditions within the city.

7.2. Project Description

Besiktas JK, founded in 1903, is one of the famous football clubs in Europe and often participates for the UEFA European competitions and also professionally contributes in many sports branches including basketball, volleyball and handball. BJK is the legal owner of the real property, BJK Fulya Complex, which is located at a very prestigious district of Besiktas, Istanbul.

Total area of the land is more than 43,000 m^2, consisting of 29,000 m^2 construction area and the rest is spared for sporting facilities. Total construction area is 160,000 m^2 and more than half of it is constructed underground. The construction cost is more than 100 million US dollars excluding the land purchase cost which is about twice of the construction cost.

“BJK Fulya Complex” is a high standard, modern architecture, multi-functional complex that contains two residential towers, a five-star hotel building, a fully equipped hospital building, a cultural center, a hypermarket, a technomarket, dining and entertainment facilities, and parking lot. There are 240 high-tech residential units in twin towers, which are more than 150 m in height. 12,000 m^2 of hospital building is on the north side of the project, having a height of approximately 100 m. Between the hotel and twin towers there is a hotel building which is 125 m in height and has 15,000 m^2 floor area. The three-dimensional rendering of the final architectural design of “BJK Fulya Complex” is illustrated in Figure 7.2.

Figure 7.2. BJK Fulya Complex, architectural design

On the ground level dining and entertainment facilities, luxury restaurants and fancy boutiques are located, besides there is a cultural complex that has a number of ateliers, exhibition halls and an amphitheatre. There are also swimming pools, tennis courts and sports areas alongside of the underground facilities.

As in the other densely populated metropolitan cities, the value of the land in high-status areas is a major cost issue in construction. Therefore shopping and entertainment facilities as well as parking lots and the electro-mechanical technical spaces are positioned in underground levels. “BJK Fulya Complex” has also more floor area on underground than upper ground levels. There are 4 to 5 underground levels covering almost 90,000 m^2 floor area. There are 16,000 m^2 of large sized hypermarket, 8,000 m^2 of very large sized technomarket, 3-level parking lot which has the capacity of 2,000 car parking and right under the hospital building almost 8,000 m^2 of side facilities of hospital consisting of various surgery rooms, intensive care units, etc. Figure 7.3 shows the construction area of

"BJK Fulya Complex" just before the beginning of the excavation stage, on September, 2005.

Figure 7.3. BJK Fulya Complex, before the excavation

The completing date of the construction of "BJK Fulya Complex" is planned to be September, 2008. Figure 7.4 demonstrates the current construction stage, September 2007, where shell construction of the twin towers, at south, nearly completed and the sixth floor of hospital building, above the ground, at north, is under construction. In between, hotel building has a little of shell construction works left.

Figure 7.4. BJK Fulya Complex, during the construction, September 2007

7.3. Subsoil Modeling

The subject site has a very rugged topography having about 25 m difference in elevation in perpendicular direction to the covered old creek located at the bottom of the valley where the main street is positioned. The topographic elevations at site vary from +28.0 m to + 52.0 m LD, elevation above local Istanbul datum. The basement of the complex has the formation level of +9.5 m. Since the main construction axis is alongside the valley, the existing ground should be excavated starting from around +28.0 m to the bottom level of +9.5 m. Therefore the average height of the excavation would be nearly 20.0 m.

The scope of the site investigations at the initial stage originally consisted of six boreholes, covering total length of 94.0 m. However considering the planned structures and encountered subsoil conditions, additional second stage ten boreholes with total length of 235.0 m were realized within the scope of soil investigation programme. The locations of the investigation points are shown on the general layout plan of the site as given in Figure 7.5.

Standard Penetration Testing SPT with regular intervals and representative sampling were performed at the alluvial sand and gravel subsoils located above the lithological bedrock unit according to ASTM D-1586. Energy corrected SPT/N_{60} blow counts are determined. The drilling method was rotary drilling and bentonite slurry was used in all boreholes for circulation in order to minimize the bottom heave of alluvium during drilling and to get higher total core recovery from the main lithological unit of closely fractured greywackes.

TCR, SCR and RQD values of rock formations are also determined and their variations with elevation are presented on borehole log charts. Additionally, after the drilling of the boreholes groundwater levels in each borehole are recorded and monitored by means of piezometers. It is seen that the depth of the groundwater differ from h_w=2.5 to 7.3 m below the ground level.

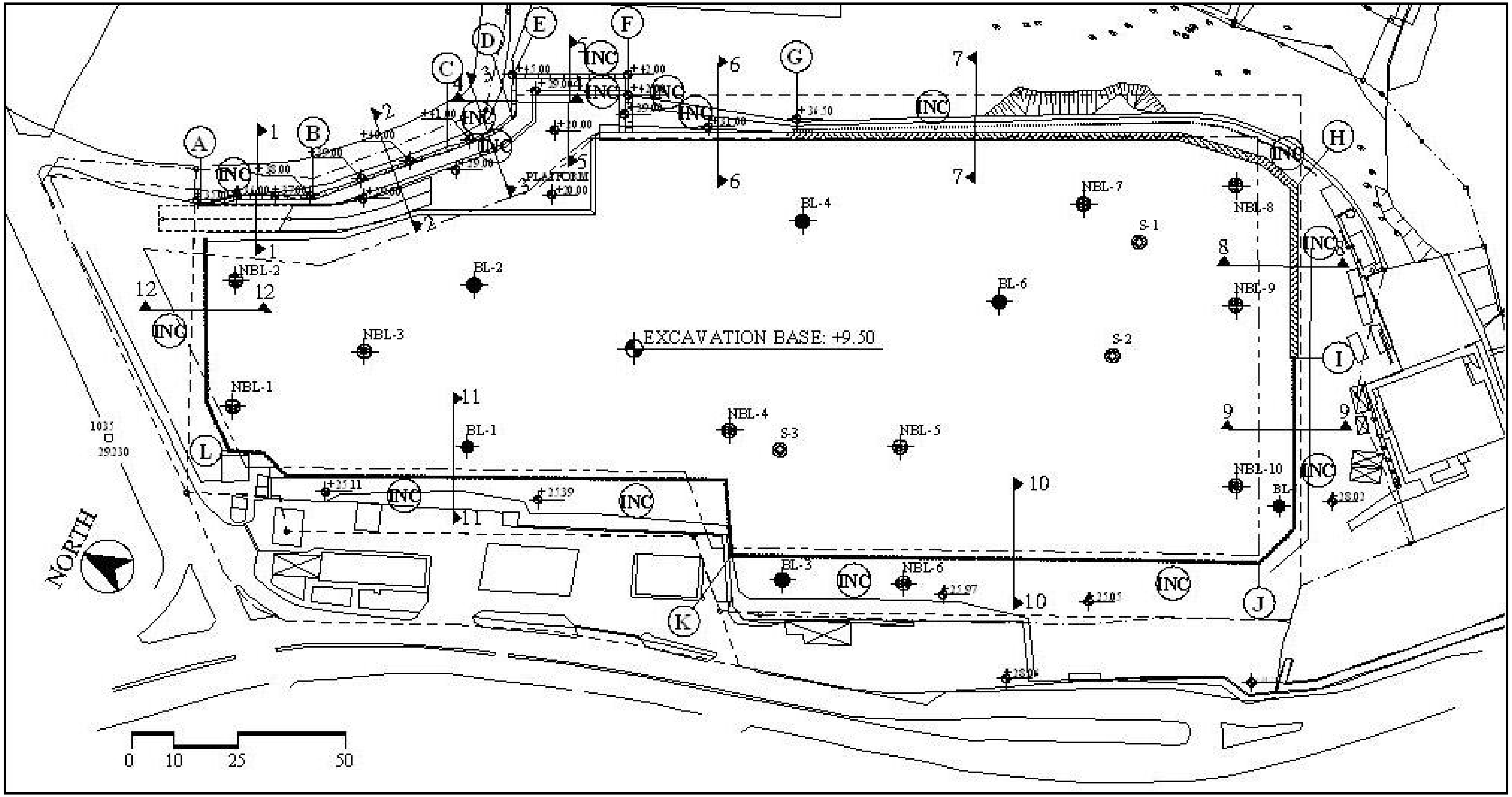


Figure 7.5. General layout plan of BJK Fulya Complex

Since the subject site is in perpendicular to the old creek, the upper levels of the subsoil is formed of alluvium and fill. The depth of the alluvium starts from zero ground level, at the east side, and reaches to eleven meters, at the west side of the subject site. Alluvium and fills lie on the bedrock greywacke, which is classified as Trakya Formation. The fill is uncontrolled manmade to correct the topography. Trakya formation is composed of alternating claystones, shales siltstones and sandstones with various degrees of weathering and fracturing. Figure 7.6 demonstrates the boundary between greywacke and alluvium zones where greywacke formation of the site has gray in color, while alluvium and the upper fill is brownish.

Figure 7.6. A picture from early stages of excavation, September 2005

Laboratory testing is performed on soil, rock and groundwater samples obtained from boreholes in order to determine geotechnical parameters and aggressiveness of the water for foundation engineering evaluations. Soil mechanics and rock mechanics test results of "BJK Fulya Complex" are given in Table 7.1 and Table 7.2, respectively. In addition, the change in the energy corrected SPT/N_{60} blow counts with depth is also presented in Figure 7.7.

Table 7.1. Soil mechanics test results for alluvium and fill

Borehole No.	Sample No.	Depth (m)	Water content w_n (%)	Atterberg limits			Sieve analysis		Soil Classification USCS
				LL	PL	PI	+No.4 (%)	-No.200 (%)	
BL-1	SPT2	3.00-3.45	21	36	21	15	18.32	40.58	SC
	SPT4	6.00-6.45	15	29	13	16	16.00	42.00	SC
	SPT6	9.00-9.45	14	31	17	14	26.55	41.46	SC
BL-2	SPT1	1.50-1.95	12	25	14	11	41.00	21.00	GC
	SPT3	4.50-4.95	20	30	15	15	0.00	61.68	CL
	SPT5	7.50-.795	18	34	16	18	0.91	66.11	CL
BL-3	SPT1	1.50-1.95	11	28	17	11	34.86	27.02	SC
	SPT3	4.50-4.95	24	-	-	-	8.31	40.40	SC/SM
	SPT4	6.00-6.45	16	27	17	10	14.04	52.22	CL
	SPT7	10.50-10.95	23	32	14	18	24.00	37.00	SC
BL-4	SPT2	3.00-3.45	7	-	-	-	34.71	29.87	SC/SM
BL-5	SPT2	3.00-3.45	8	27	14	13	47.00	20.00	GC
	SPT4	6.00-6.45	23	NP	NP	NP	53.56	30.51	GM
	SPT6	9.00-9.45	14	27	13	14	38.00	32.00	GC
	SPT8	12.00-12.45	16	26	13	13	41.28	21.50	GC
BL-6	SPT1	1.50-1.95	13	29	13	16	24.00	34.00	SC
NBL - 1	SPT1	1.50-1.95	13	NP	NP	NP	32.89	24.04	SM
	SPT3	4.50-4.95	11	NP	NP	NP	61.49	9.92	GM-GP
	SPT4	6.00-6.45	11	NP	NP	NP	43.80	13.91	GM
	SPT5	7.00-7.50	15	NP	NP	NP	25.70	35.20	SM
	SPT6	9.00-9.45	14	NP	NP	NP	46.91	13.59	GM
	SPT7	10.50-10.95	17	NP	NP	NP	33.11	15.51	SM
NBL - 2	SPT1	1.50-1.95	19	NP	NP	NP	33.79	24.01	SM
	SPT4	6.00-6.45	16	NP	NP	NP	31.97	17.57	SM
NBL - 3	SPT1	1.50-1.95	19	32	14	18	1.32	57.49	CL
	SPT4	6.00-6.45	11	NP	NP	NP	-	-	-
NBL - 4	SPT2	3.00-3.45	14	27	14	13	27.00	32.00	SC
	SPT3	4.50-4.95	17	31	16	15	9.27	49.12	SC
NBL - 5	SPT2	3.00-3.45	9	NP	NP	NP	76.49	6.49	GM-GP
	SPT5	7.50-7.95	19	NP	NP	NP	39.67	29.44	GM
NBL - 6	SPT2	3.00-3.45	10	NP	NP	NP	55.78	13.07	GM
	SPT4	6.00-6.45	16	22	15	7	10.00	24.00	SC
	SPT6	9.00-9.45	14	NP	NP	NP	11.24	25.46	SM
NBL - 9	SPT1	1.50-1.95	15	NP	NP	NP	19.66	37.73	SM
	SPT3	4.50-4.95	15	-	-	-	16.00	25.00	SC/SM
NBL - 10	SPT2	3.00-3.45	15	32	19	13	43.03	24.54	GC
	SPT5	7.50-7.95	17	25	13	12	29.00	23.00	SC
	SPT7	10.50-10.95	18	31	14	17	18.16	47.19	SC

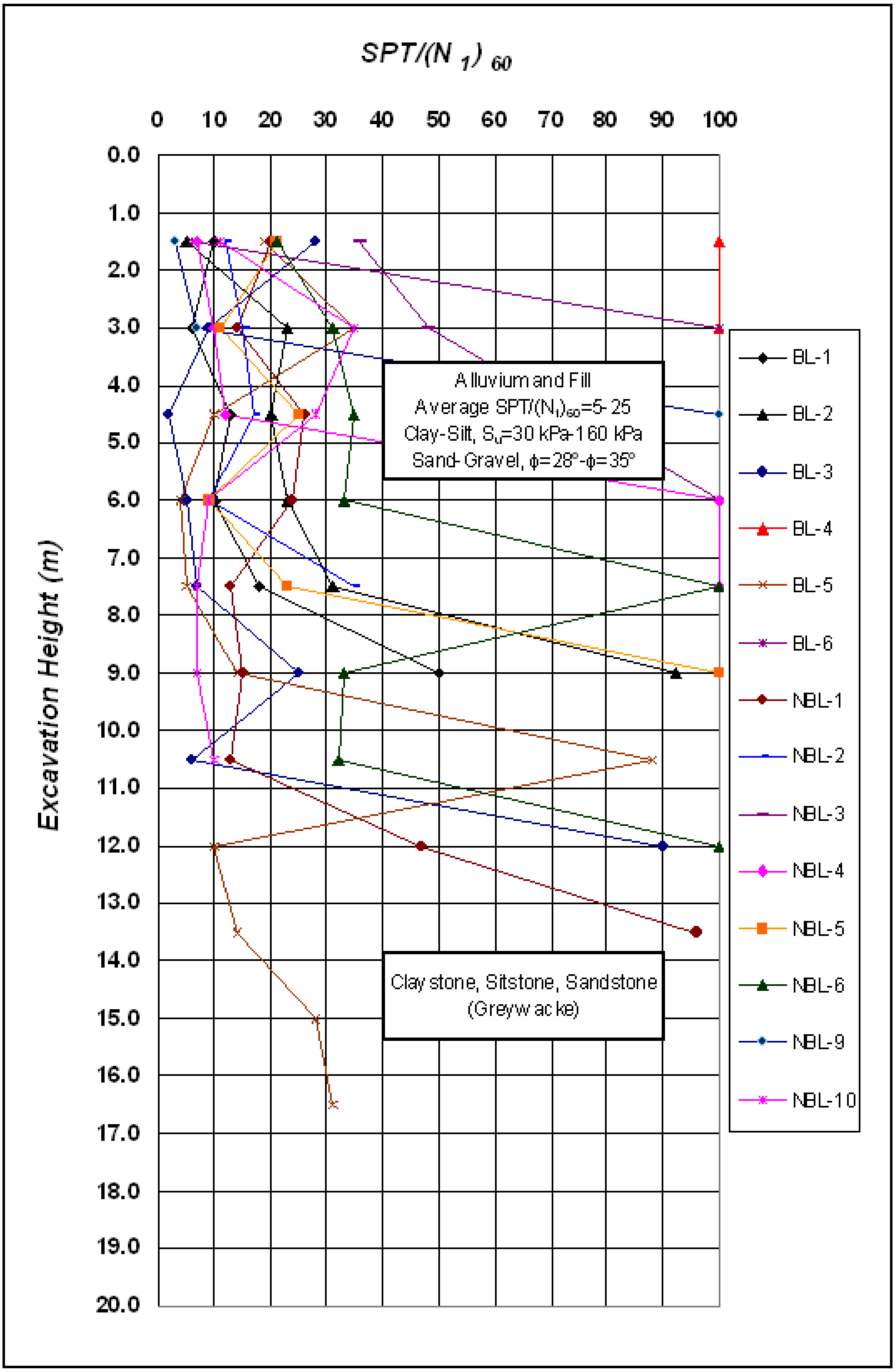


Figure 7.7. The change in the energy corrected SPT/N_{60} blow counts with depth

Table 7.2. Rock mechanics test results for greywacke

Borehole No.	Depth (m)	Sample No.	Uniaxial Compressive Strength, MPa
YSK-4	15.00-16.50	Core # 6	22.2
YSK-4	19.00-20.00	Core # 9	44.9
YSK-6	22.50-24.00	Core # 8	74.9
YSK-6	24.00-25.50	Core # 9	27.2
YSK-9	15.00-16.50	Core # 8	35.9
YSK-9	18.00-19.50	Core # 10	42.2
YSK-10	18.00-19.50	Core # 6	29.5
YSK-10	21.00-22.50	Core # 8	38.1
YSK-10	24.00-25.50	Core # 10	51.3
		average	40.7
		minimum	22.2
		maximum	74.9

The uniaxial compressive strength of the greywacke formation is determined between 22.2 MPa and 74.9 MPa, average of 40.7 MPa, according to laboratory test results on core samples. The range of index properties obtained from alluvium formations shown below:

Natural Water Content,	w_n (%)	= 7-24,
Liquid Limit,	LL (%)	= 22-36,
Plastic Limit,	PL (%)	= 13-21,
Plasticity Index,	PI (%)	= 7-18,
Unified Soil Classification;	CL, SC, SM, GC, GM.	

Over the site of investigation three shallow seismic survey was conducted to derive a 'geodynamical–seismic' model below the ground surface. The dynamic parameters such as the P-wave and S-wave velocities (v_p and v_s), shear modulus, and compressibility modulus of the subsoil were evaluated. Table 1 represents geodynamical properties of the subject site. There were three different seismic zones representing various geological units present. The first zone is composed of loose/soft soils, i.e. alluvium, the second is clay/sand, i.e. fill, and the third is fractured greywacke. Summary of geodynamical parameters are presented in Table 7.3.

Table 7.3. Geodynamical parameters of BJK Fulya Complex

Survey	Layer	d_i (m)	v_s (m/s)	v_p (m/s)	γ (kN/m^3)	ν	G_0 (MPa)	E_0 (MPa)
S 1	1	0 to 0.5-2	56	211	16.4	0.46	5	15
	2	0.5-2 to 2.5-5	288	737	17.4	0.41	147	415
	3	2.5-5 to -	667	1500	20.0	0.38	907	2500
S 2	1	0 to 4.5-5	225	499	16.9	0.37	87	239
	2	4.5-5 to 8-11	600	1500	20.0	0.40	734	2062
	3	8-11 to -	1091	2824	22.6	0.41	2742	7745
S 3	1	0 to 5-7.5	300	639	17.2	0.36	158	429
	2	5-7.5 to -	667	1625	20.2	0.40	916	2563

In Table 7.3., "d_i" is the depth of the subsurface layer in meters, "v_s" is the shear wave velocity in m/s, "v_p" is the pressure wave velocity in m/s, "γ" is total unit weight of the subsoil unit in kN/m^3, "ν" is the Poisson's ratio, "G_0" is the dynamic shear modulus in MPa and "E_0" is the dynamic elasticity modulus in MPa. To calculate dynamical parameters, below formulas are used;

$$\nu = 0.5\left[\frac{\left(v_p / v_s\right)^2 - 2}{\left(v_p / v_s\right)^2 - 1}\right] \tag{7.1}$$

$$G_0 = \rho {v_S}^2 \tag{7.2}$$

$$E_0 = 2G_0(1+\nu) \tag{7.3}$$

Note that, E_0 and G_0 are modulus values corresponding to very low strain level employed in seismic surveys. The representative values of shear wave velocities for each unit could be taken as 50 to 100 m/s for alluvium, 200 to 300 m/s for the fill and 600 to

1100 m/s for the greywacke depending on the extend of weathering and structural discontinuities and depth. The measured v_s values for greywacke are in good agreement with the results of previous seismic surveys (Durgunoglu and Yilmaz, 2007).

Based on the in-situ and laboratory tests carried out for the subject site, together with the previous performance experiences of such retaining structures in similar subsoil conditions following drained condition geotechnical parameters were used as stress based design geotechnical parameters for retaining systems of "BJK Fulya Complex" as given below in Table 7.4.

Table 7.4. Geotechnical parameters used for limit state design

Subsoil Layers	Parameters	Symbol	Value
Alluvium and Fill	Internal friction angle	ϕ'	27.0°
	Cohesion	c'	0 kPa
	Total Unit weight	γ	18 kN/m^3
	Active earth pressure	K_a	0.376
Sandstone, Siltstone, Claystone (Greywacke)	Internal friction angle	ϕ'	33.0°
	Cohesion	c'	0 kPa
	Total Unit weight	γ	20 kN/m^3
	Active earth pressure	K_a	0.295
	Earth pressure at rest	K_0	0.500

7.4. Various Types of Retaining Structures

Various locations along the perimeter basically five different types of retaining systems are utilized and shown in Figure 7.5. Retaining Structures - Type 1 is permanent pre-stressed anchored reinforced concrete caisson utilized on temporary soil nailing, from A to B in total length of 27 m. Retaining Structures - Type 2 is permanent pre-stressed anchored reinforced concrete caisson utilized on soil nailing that upper part is permanent and the lower part is temporary, from B to F in total length of 87 m. Retaining Structures - Type 3 is permanent soil nailing utilized on temporary soil nailing, from F to G in total length of 52 m. Retaining Structures - Type 4 is temporary soil nailing, from G to I in total length of 161 m. Retaining Structures - Type 5 is temporary pre-stressed anchored diaphragm wall, from point I to A in total length of 361 m.

7.4.1. Retaining Structures – Type 1

From A to B, permanent pre-stressed anchored reinforced concrete caisson walls are utilized on the upper side of the ramp, sloping down from elevation +28.0m to +20.0m and 7m in width. In lower elevations of the ramp beneath the caisson walls, temporary soil nailed walls are constructed. Typical detailed cross-section of Type 1 is presented in Figure 7.8.

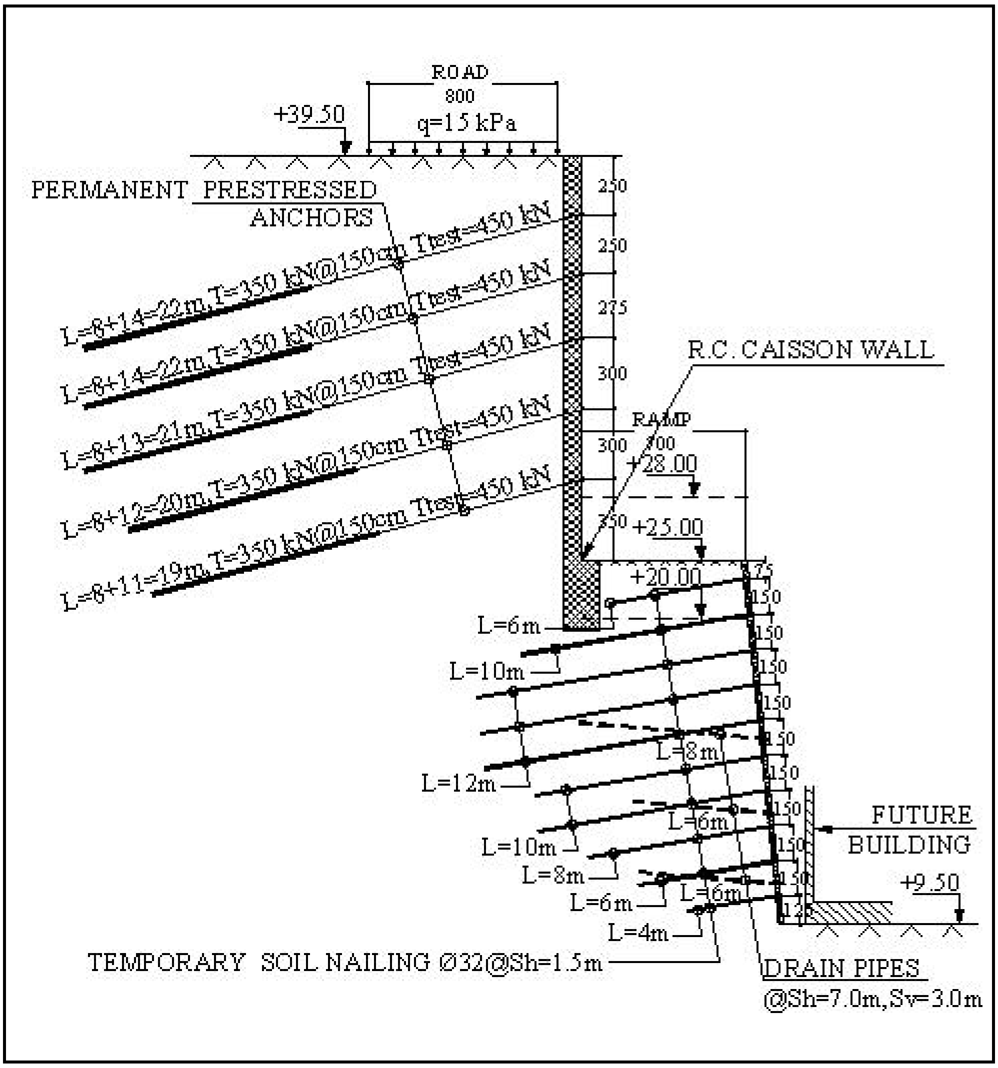


Figure 7.8. Detailed typical cross-section of Type 1

The height of the soil nailed walls is between 10.5 m and 18.5 m. Horizontal and vertical spacings of nails are $S_h = S_v = 1.5$ m and length of the nails are ranging from 4 m to 12 m.

Cast in-situ reinforced concrete caisson walls are 70 cm in thickness and 10.3 m to 20.5 m in height. The thickness of the footing of the caisson wall is 1.5 m. Caisson walls are manually constructed due to very limited space available and anchored by permanent pre-stressed anchors, with bond length of 8 m and total length ranging from 19 m to 22 m.

Horizontal and vertical spacings of anchors are $S_h = 1.5$ m and $S_v = 2.5$ m to 3.0 m, respectively. The lock-off load of tieback anchors is 350 kN while the test load is 450 kN. A picture from caisson wall is given in Figure 7.9.

Figure 7.9. A picture from caisson wall of Type 1

7.4.2. Retaining Structures – Type 2

After point B along the perimeter due to increase in elevation of existing topography, permanent soil nailing is utilized vertically between permanent pre-stressed anchored caisson wall and temporary soil nailing reaching to point F. The maximum height of the caisson wall is 12.5 m. Caisson walls are tied back permanently with anchors as in Type 1, 21 to 22 m in length. Permanent soil nailed walls are 15 m in height. One row of pre-stressed anchors are utilized on the soil nailed wall above the permanent nails to prevent excessive lateral displacement of the nailed walls, as shown in Figure 7.10 and Figure 7.11. At this section, the height of the permanent retaining system reaches to 25 m and the subsoil gets stronger 10 m below the road. Therefore, permanent soil nailing system between caisson wall and temporary soil nailing is preferred.

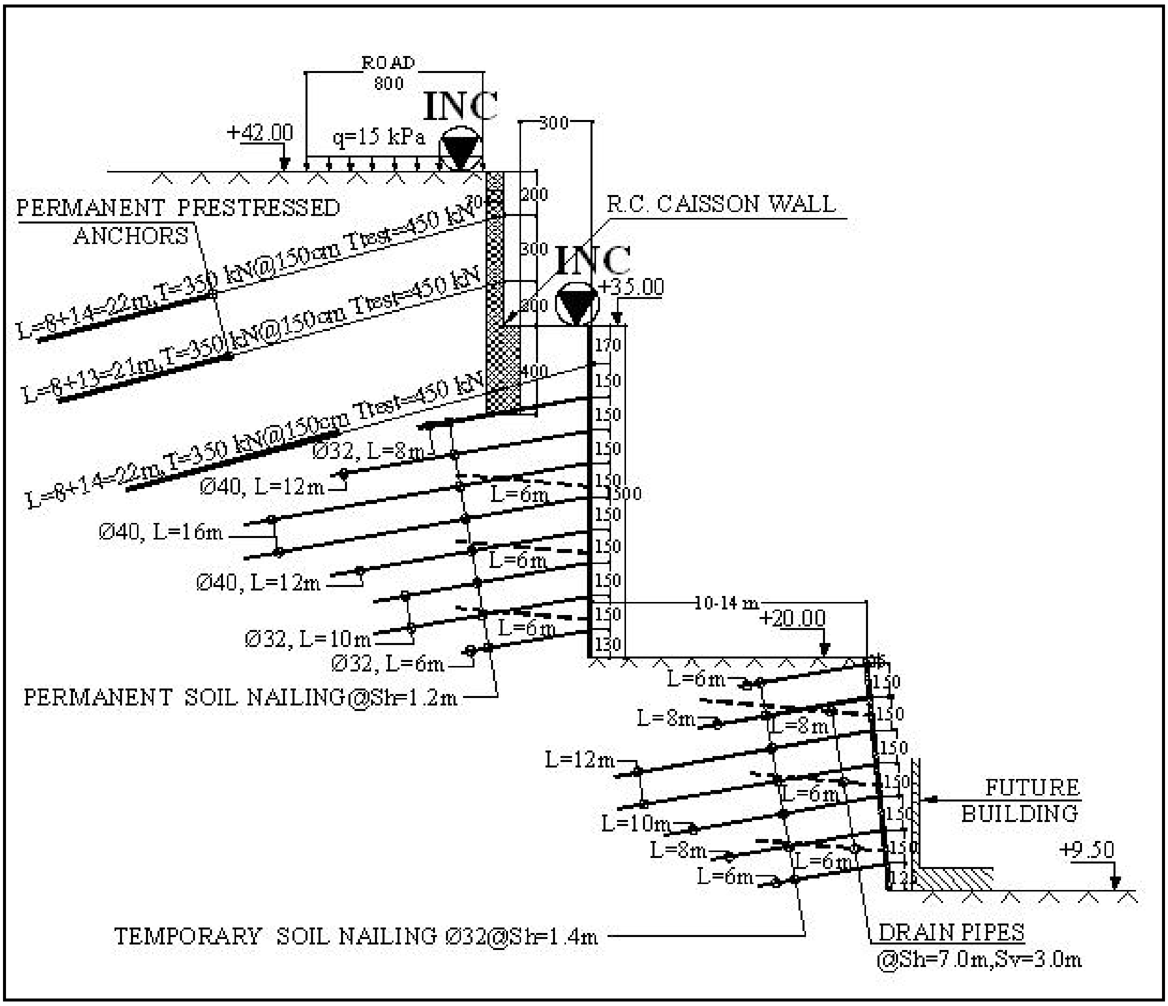


Figure 7.10. Detailed typical cross-section of Type 2

Figure 7.11. Pre-stressed anchors on permanent soil nailing to prevent excessive lateral displacement

Horizontal and vertical spacings of the permanent soil nails are S_h = 1.2 m and S_v = 1.5 m respectively, while those of temporary soil nailed walls are S_h = 1.4 m and S_v = 1.5 m. An overview from retaining structures Type 1 and Type 2 is given in Figure 7.12.

Figure 7.12. Overview from Type 1 and 2

7.4.3. Retaining Structures – Type 3

Between points F and G on the perimeter, permanent soil nailing is constructed for the permanent part of the retaining system. Below the permanent part, temporary soil nailing is again placed. The height of the permanent soil nailing is in the range of 7.5 m to 14 m. The slope of the permanent soil nailing wall is 1H/3V. The maximum nail length is 16 m. The height of the temporary soil nailed wall is constant and 18.5 m. Horizontal and vertical spacings of the permanent soil nails are S_h = 1.4 m and S_v = 1.5 m, and S_h = 1.5 m and S_v = 1.5 m for the temporary soil nailing, respectively. Figure 7.13 and Figure 7.14 present the typical detailed cross-section and a photo of retaining Type 3.

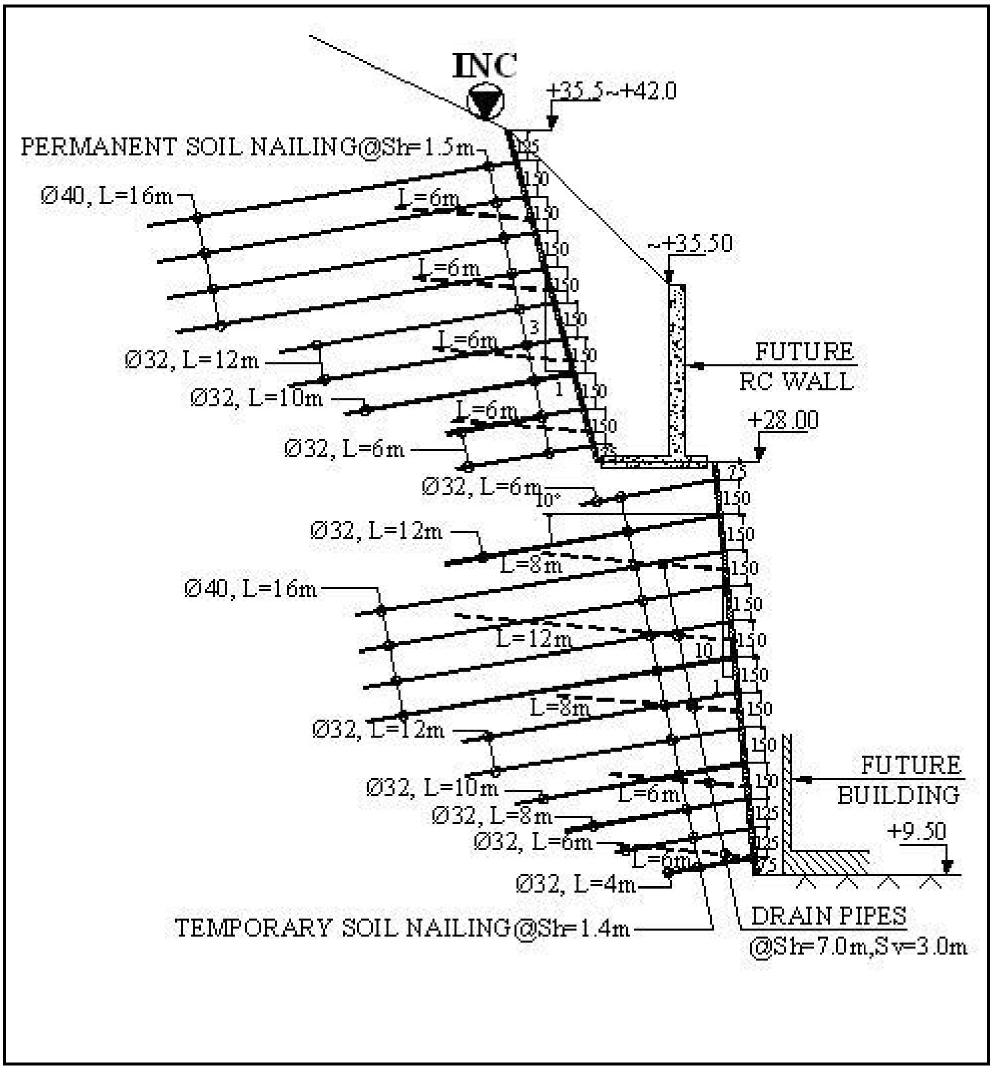


Figure 7.13. Detailed typical cross-section of Type 3

Figure 7.14. A picture from Type 3

Excessive lateral displacement is monitored by means of one inclinometer within this part of the retaining structure because of unforeseen potential slip plane due to adverse bedding of the greywacke formation during excavation. Additional long pre-stressed anchors are constructed at these locations reaching behind the instable wedge to provide further stability of excavated slope. Figure 7.15 shows two rows of additional pre-stressed anchors in temporary soil nailed part of the retaining system.

Figure 7.15. Additional rows of pre-stressed anchors on temporary soil nailing

7.4.4. Retaining Structures – Type 4

From point G to I, only temporary soil nailing retaining system is constructed. The typical nail diameter of D = 105 mm, nail orientation of $\omega = 10^\circ$ with the horizontal and slope angle of $\beta = 85^\circ$ (1H/10V) are utilized for all temporary soil nailed walls constructed within the site. Two different nail bars with Ø32 mm and Ø40 mm in diameter are used. The length of the nails is ranging from 4 m to 16 m horizontal, spacings of the nails are S_h = 1.4 m to 1.8 m while vertical spacings are S_v = 1.5 m. Typical cross-section and a photograph from temporary soil nailing are given in Figure 7.16 and Figure 7.17, respectively.

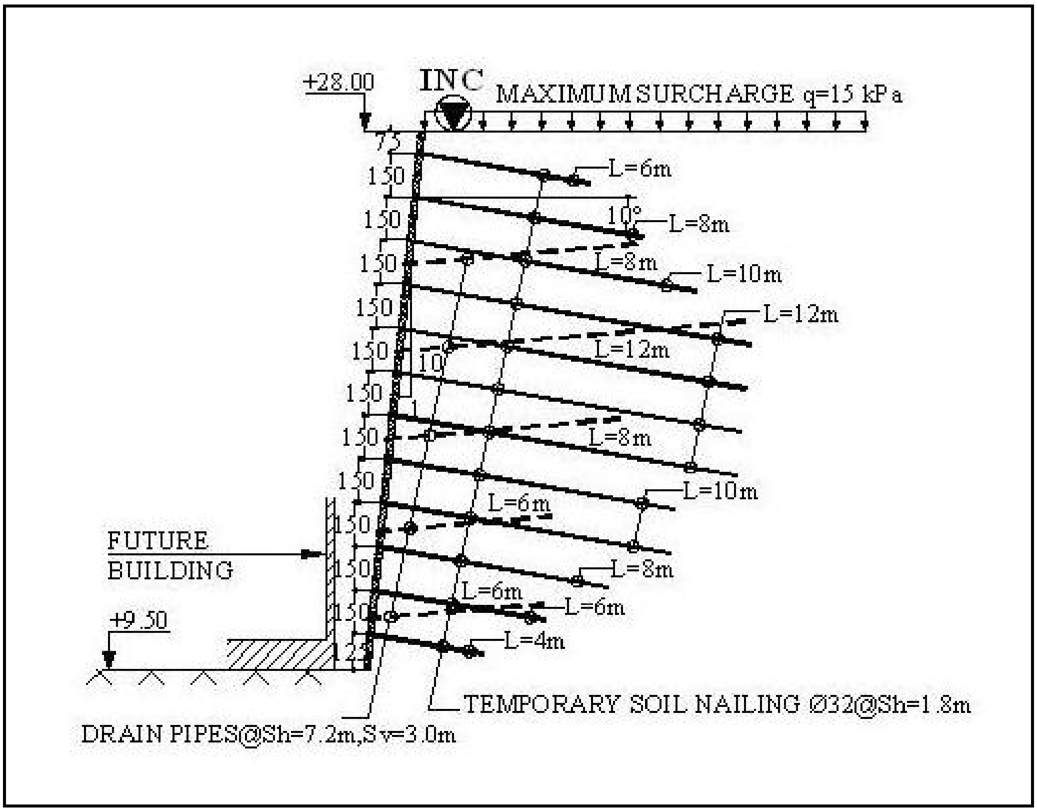


Figure 7.16. Detailed typical cross-section of Type 4

A minor variation in the temporary soil nailing section is described below. There exists an old 7 m high retaining wall between point G and H. The retaining wall is to be kept in place by improving its stability with mini piles of diameter Ø225 mm in front of the

Figure 7.17. A picture from Type 4

toe. After construction of mini piles, temporary soil nailed wall is utilized below the existing retaining wall. Figure 7.18, shows a view of the initial stage of the excavation where mini piles had been constructed in front of the toe of the old retaining wall and the first row of soil nailing has just been completed. Detailed typical cross-section of this section is presented in Figure 7.19.

Figure 7.18. Completion of the first row of soil nailing under the existing retaining wall

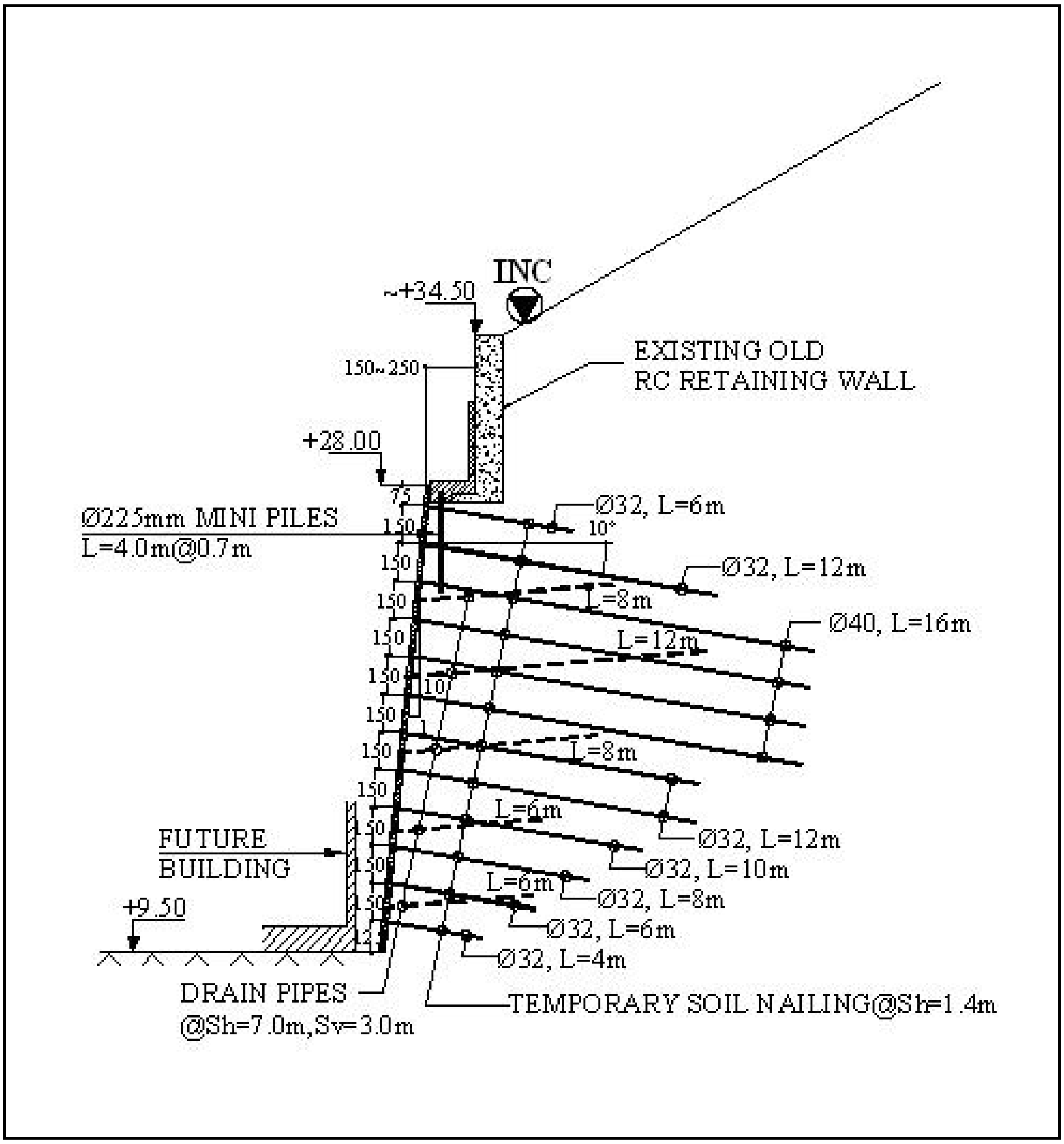


Figure 7.19. Detailed cross-section from temporary soil nailing under the existing old retaining wall

7.4.5. Retaining Structures – Type 5

Temporary pre-stressed anchored diaphragm wall from point I to A is constructed. The wall consists of bored piles of diameter Ø65 cm, spaced at 90 cm intervals from center to center. To prevent ground water intrusion, jet grout columns in 60 cm diameter are constructed between piles. The height of the diaphragm wall is 15.5 m to 18.0 m.

Five rows of pre-stressed anchors, 18 to 20 m in length, are constructed to overcome both earth and hydrostatic water pressures on the diaphragm wall. The horizontal spacings of the anchors are 0.90 m for upper rows and 1.25 to 2.70 m for bottom rows. The lock-off load for the anchors located in alluvium and fill is 300 kN and for the anchors drilled in greywacke is 350 to 450 kN.

The concrete pile cap is 60 x 70 cm in section and 100 x 35 cm reinforced concrete beams are placed continuously for each row of anchors. Figure 7.20 presents the typical detailed cross-section of Type 5. Two photographs taken after the completion of the work are presented in Figure 7.21 and Figure 7.22.

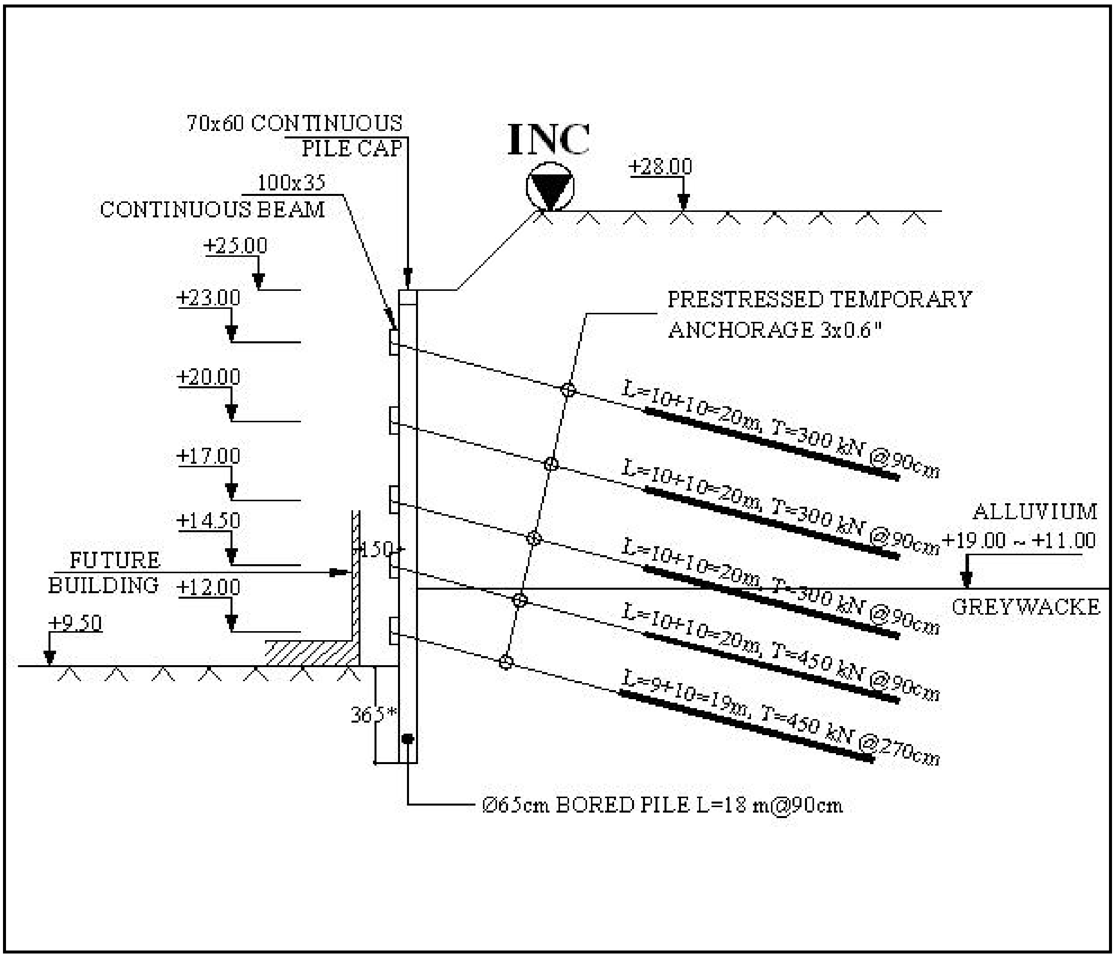


Figure 7.20. Detailed typical cross-section of Type 5

Figure 7.21. A picture from Type 5

Figure 7.22. Another picture from Type 5

A minor variation is unavoidable at certain length along the subject periphery. At some sections between point L and A, bored piles could not be drilled for the last 3 to 6 m due to very strong silicified sandstone formation. This formation is a different variation of main lithological unit of greywacke, having much greater strength, hardness and compressibility modulus.

At these sections, cast in-situ reinforced concrete walls in segments are constructed below the piles after partial excavation as shown in Figure 7.23. Lateral displacements were monitored carefully during partial excavations and it was seen that there were no appreciable displacement increase as a result of segmental construction procedure followed.

Figure 7.23. Cast in-situ reinforced concrete walls in segments under the bored piles

Two photographs showing completed retaining walls facing both north and south side of the subject site are presented in Figure 7.24 and Figure 7.25, respectively.

Figure 7.24. Retaining system facing north side

Figure 7.25. Retaining system facing south side

7.5. Inclinometer Readings And Lateral Displacements

The performances of deep excavations and retaining structures are monitored by means of observed lateral displacements. It is well known that, there are complicated numerical models and computer programs are available to predict the lateral displacements during design stage prior to construction. However, where complicated subsoil geology is prevailing, such as in this case study, the strict performance evaluations should be made based on the measured displacement data rather than the predicted values, especially where the design is based on both stability (certain factor of safety for each retaining member) as well as occurred displacements of the retaining structures at different phases of the excavations.

Although, the procedure followed in design guarantees the safety against lateral earth pressures and the hydrostatic water pressures, design requirements further implement that developed lateral displacements at various stages and various retaining systems should be kept below the acceptable limiting values. Further, even in simpler geological conditions, i.e. only in presence of greywacke formation, results of previous case studies in the city have shown that, the prediction of displacement even employing sophisticated computer programs such as PLAXIS and/or FLAC is mainly governed by the deformation modulus formulation of the subsoil unit.

It is also known that, the modulus of soils such as encountered in this case study alluvium, man made uncontrolled fills and even greywacke are dependent on many factors, including the non-homogeneity of the unit and even more important to excavation induced displacement, strain. Previous experience, Durgunoglu *et al*. (2007a) have demonstrated that, the correct predictions of displacements in such conditions as in this case study is almost impossible, therefore strict displacement monitoring during various stages of the construction is compulsory.

Total of sixteen inclinometers were installed prior to any earthworks at various locations along the periphery covering considering the presence of various types of retaining structures that are planned to be constructed, as shown in Figure 7.5. The inclinometer boreholes are located just outside the retaining wall, in order to guarantee that the measured displacements are not influenced by the relative rigidity of the various retaining wall systems that are constructed. Inclinometers are recorded daily throughout the construction, covering all phases of the excavation steps. Typical inclinometer recording for each retaining systems except from Type 1 are presented in Figures 7.26 through 7.29.

It is seen that lateral displacement vs. depth relations for retaining walls of Type 2, 3 and 4 are about the same form i.e. maximum displacements have occurred at the surface leading to spandrel type curve. On the other hand, the maximum lateral displacement has occurred at certain depth for retaining wall Type 5 leading to concave type curve. The observed shape of lateral displacement vs. depth curves are in agreement with the previous displacement curves obtained for that specific retaining wall system.

The maximum lateral displacement values, δ_{hm} with the corresponding height of excavation, H, together with performance ratios, $P_r = \delta_{hm}/H$ are summarized in Table 7.5. The measured δ_{hm} values for soil nailed systems described in retaining walls, Types 2, 3 and 4, are between δ_{hm}=0.1 to 0.2%H, depending on the nature of greywacke formation, which is in good agreement with the results reported for similar conditions by Durgunoglu *et al*. (2007a).

On the other hand δ_{hm} is about 0.12%H for the diaphragm wall, which is given as Type 5. Considering the subsoil alluvium layers were sand and gravel, this value is in good agreement with the value obtained by Ou (2006) for excavations performed in sandy soils at Taipei area.

Table 7.5. Typical lateral displacements of retaining types

Retaining Types	Excavation Height, H (m)	Max. Lateral Displacement, δ_{hm} (mm)	Performance Ratio, P_r (δ_{hm}/H, 10^{-3})
Type 2	17.0	21.4	1.26
Type 3	32.5	50.6	1.56
Type 4	18.5	14.0	0.76
Type 5	17.0	20.7	1.22

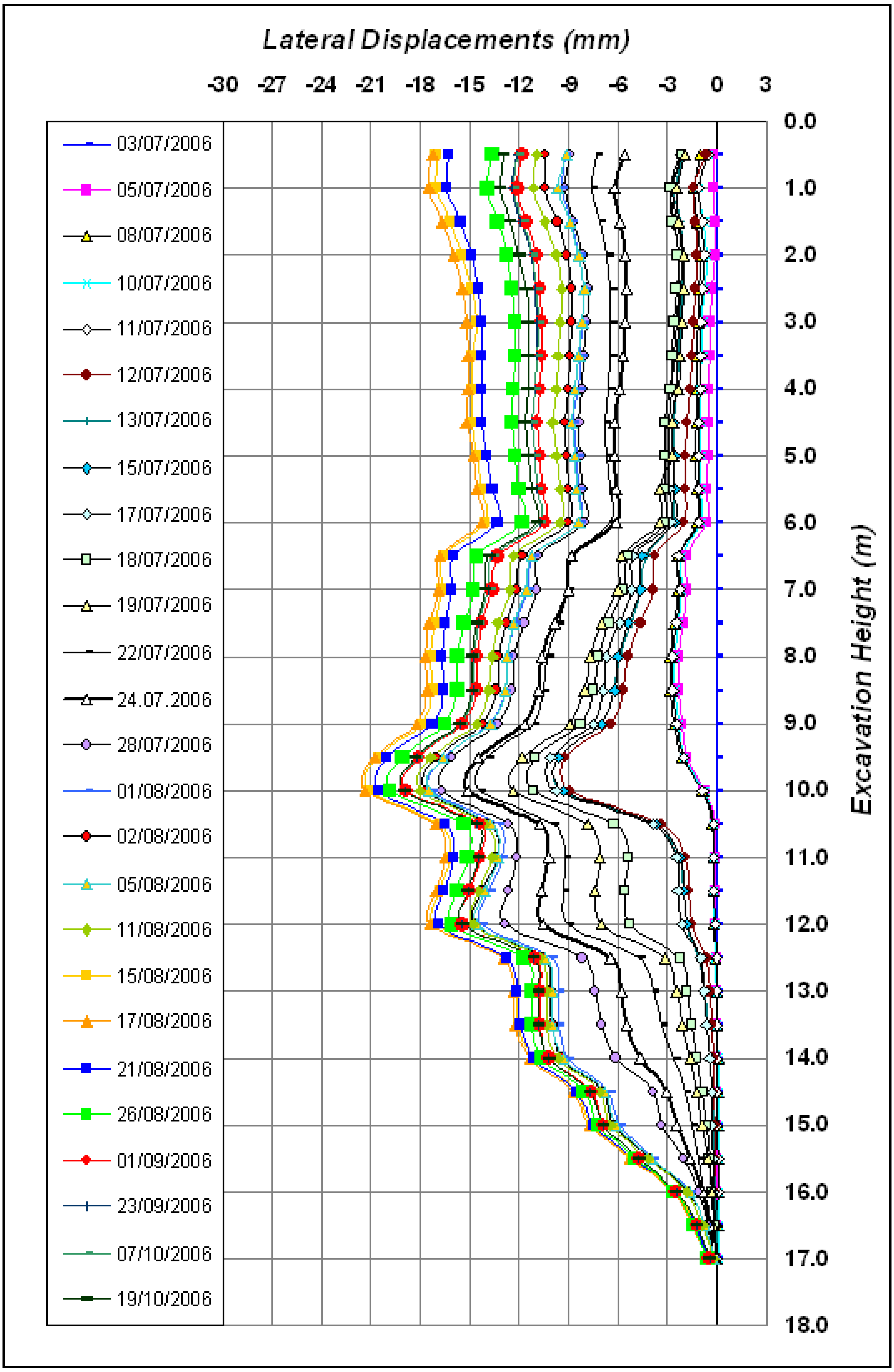


Figure 7.26. Typical inclinometer readings from Type 2

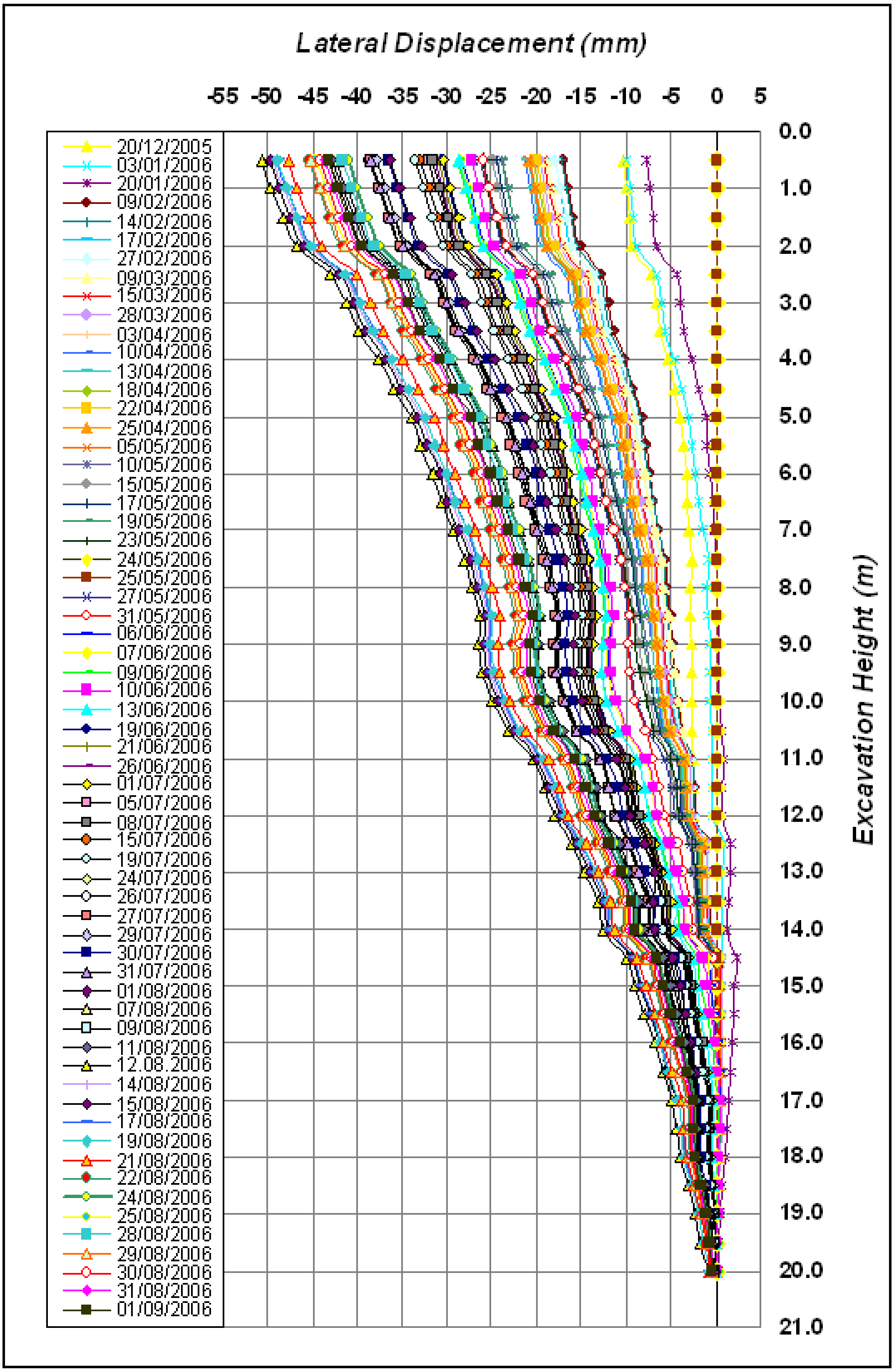


Figure 7.27. Typical inclinometer readings from Type 3

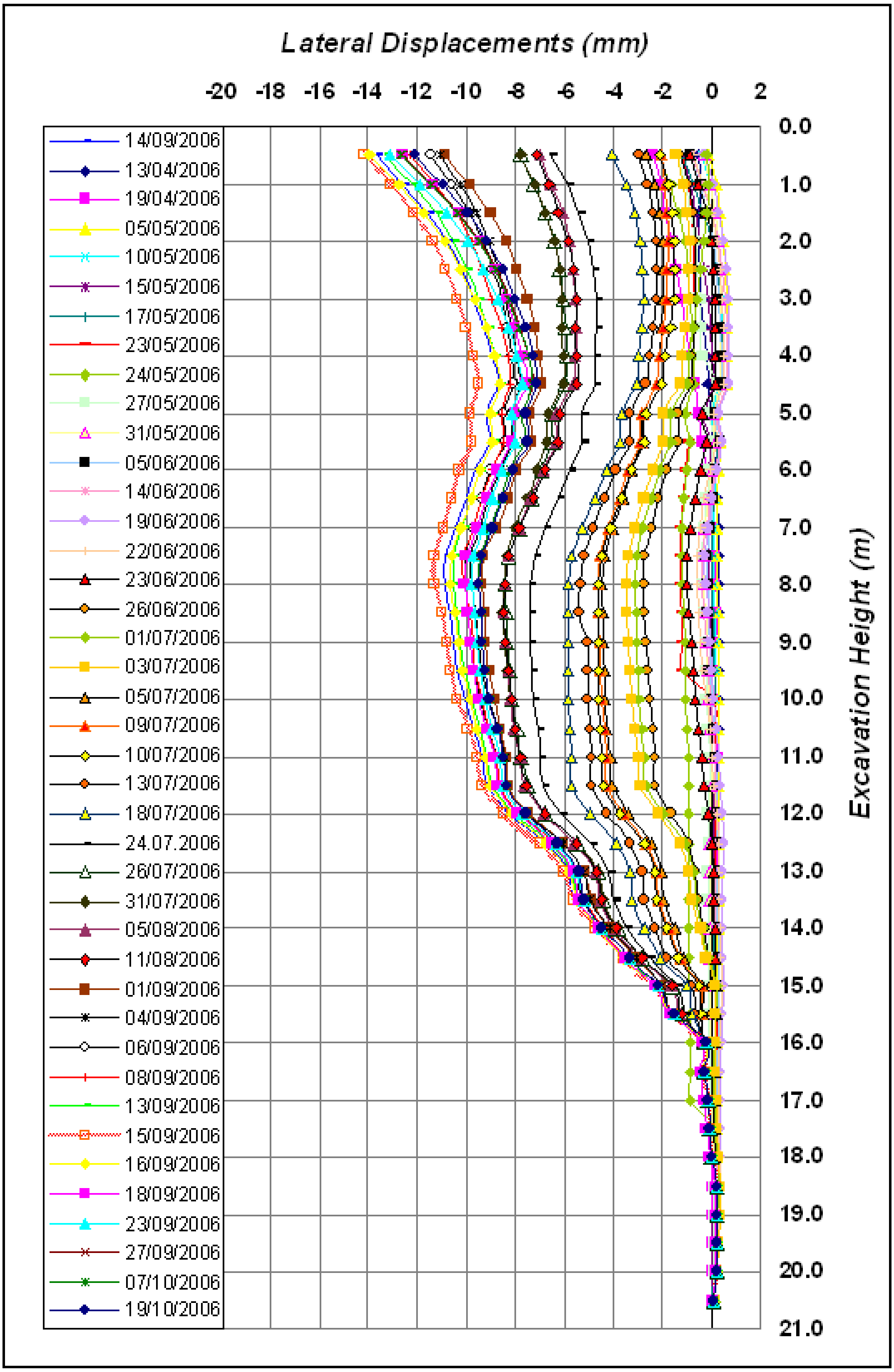


Figure 7.28. Typical inclinometer readings from Type 4

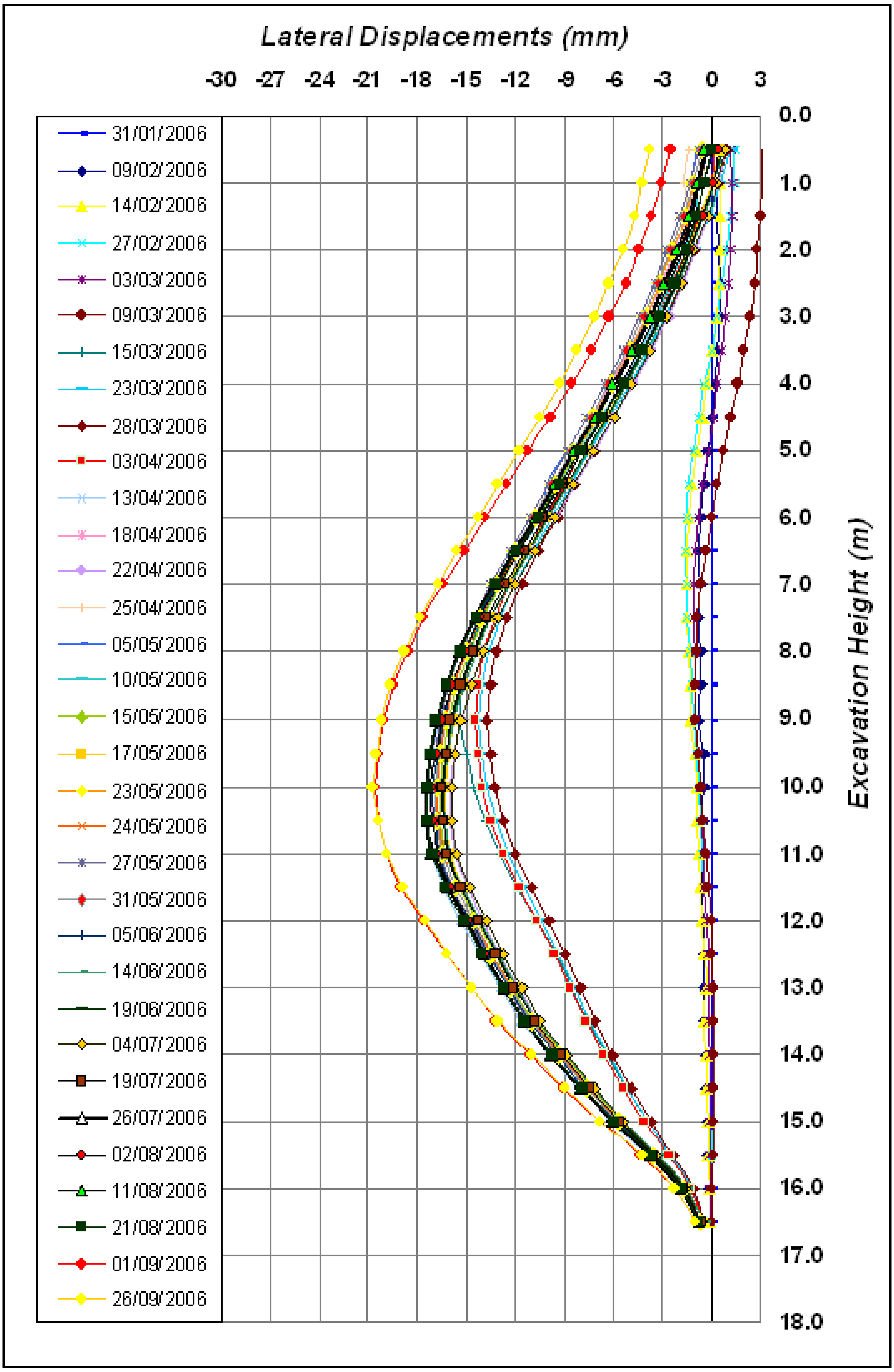


Figure 7.29. Typical inclinometer readings from Type 5

7.6. Concluding Remarks

An interesting case study is presented for the implications of various retaining wall systems at a specific project site based on the observed complex geological and groundwater conditions. It is shown that, site subsoil conditions, ground water regime, the topography and the architectural elevations and locations imposed by the project have dictated the tailor made retaining wall design and construction for this specific case.

Except some minor variations employed as described herein total of five different retaining wall system have been employed having temporary and permanent parts. Due to the high seismicity of the site the permanent walls are preferred to be flexible type, i.e. soil nailing, except from the top part of the Type 1 retaining system which had to employed manually constructed very rigid caissons due to limitations of the space for construction equipment and more strict lateral displacement limitations towards nearby infrastructures. Both flexible and rigid types of retaining wall systems, on the other hand, have been employed for the temporary structures.

The performance criteria for the walls were based on the observed lateral displacement during excavation. The careful monitoring of the various systems by means of inclinometers have provided the opportunity to implement further measures as in one instance described here. Further, the observed form of lateral displacement vs. excavation depth relationships are in good agreement with the previous findings. In addition, the performance ratios defined as the ratio of maximum lateral displacement to excavation height were within the range of 0.1 to 0.2 % which were below the critical value of 0.3 % imposed in the contract documents. It is further concluded that with the tailor made approach it was possible to complete the project within budget on time.

8. SUMMARY OF CONCLUSIONS

The city of Istanbul has performed significant growth in economy lately, consequently the need for high-rise residential and office buildings and shopping malls with multiple basement levels increased noticeably. In order to build multiple basement levels deep excavations and construction of retaining structures became compulsory. The depths of the excavations commonly reach to 25 to 40 meters below the ground surface.

The encountered subsoil formation mostly in the deep excavations of high-rise buildings of Istanbul is soft rock greywacke locally known as Trakya formation, which is lithologically alternating sandstone, siltstone and claystones with various degrees of weathering and fracturing, especially along European side of the city, the newly developed axis, Buyukdere Avenue. The extent of weathering and fracturing controls the physical and mechanical properties of greywacke formations. The weathering of the greywackes decreases gradually and progressively with increasing depth.

The greywackes, which constitute the basement rock formation of the city of Istanbul, are classified as critical as far as their stability upon excavation is concerned. They contain very complex discontinuity surfaces and sliding and rock falls along these discontinuities are very common at unsupported deep excavations made in this rock formation. The greywacke beds, which possess rather high shear strength and uniaxial compressive strength, are very poor from the slope stability viewpoint due to relatively low residual shear parameter they show when exposed to water and atmosphere. As a result, the deep excavations to be utilized in Istanbul greywackes should be supported by various means (Saglamer, 1986).

Considering the seismicity of the city of Istanbul, one of the principal concerns while selecting appropriate retaining system whether it is flexible or rigid. Other factors affecting the predetermination of the retaining system are depth of cut, location of neighboring structures and infrastructures, time schedule, budget constraints, access to the site and the available techniques that can be readily employed by the geotechnical contractor. The retaining systems widely used in the deep excavations in the city of

Istanbul are tied back cast in-situ piles, tied-back micro piles, tied back cast in-situ reinforced concrete walls (manual caisson, segmented or diaphragm walls), strutted or shored support walls, soil nailing or combinations of these.

Since Istanbul is in very seismically active region and obviously at risk of being hit by major earthquake, flexible retaining walls should be preferred in deep excavations carried out in the city. Based on the previous positive records of flexible earth retaining structures during earthquakes in Turkey by Mitchell *et al*. (2000) and Durgunoglu *et al*. (2003a), soil nailed walls in such excavations performed within the city offer great advantage especially for the encountered subsoil and seismic conditions.

Soil nailing is a very versatile excavation retaining system for deep excavations in urban areas surrounded by major structures and infrastructures provided that limiting lateral displacements are not exceeded. As a result the values of performance ratio for soil nailed walls together with nail density in typical greywacke formation of the city of Istanbul are developed based on these extensive case studies as a guideline for future applications.

By the analysis of the data, lateral displacements, δ_h, performance ratio, P_r, average nail length, L, and nail density, η versus the height of the soil nailed wall, H were developed and presented. It is seen that the linear increase in lateral displacement with the height of the wall is valid up to a certain height. The change in slope occurs at various heights and sooner for the weaker claystone than the stronger silicified sandstone case. Similar observation was made by Durgunoglu *et al*. (2003b). Using conventional methods of design, FHWA (2003) and previously developed charts for estimating lateral displacements or performance ratio may be misleading in deep soil nailing applications.

The performance ratio, P_r, for the greywacke formation is within the range of $1x10^{-3}$ to $3x10^{-3}$, depending on the nature of the lithological unit of the formation. For the strongest silicified sandstone with a typical value of E_m = 250 MPa, $P_r \sim 1x10^{-3}$, on the other hand for the weakest claystone $P_r \sim 3x10^{-3}$ with E_m = 50 MPa. It is seen that these values tend to increase after 25 m for the case of sandstone and 15 m for the case of claystone.

Average nail length, L, increases linearly with the height of soil nailed wall. The average nail length that could be utilized is about L = 5 to 8 m for H = 10 m and L = 8 to 11 m for H = 20 m. Nail density, L/S (m/m^2) also increases linearly with the height of the soil nailed wall. It is about 1.6 to 3.2 m/m^2 for H = 10 m and 2.8 to 4.4 m/m^2 for H=20 m.

The most important drawback of the limit equilibrium design methods is that they do not provide a prediction of displacements. Displacements can be predicted approximately using empirical correlations and while these are appropriate in many cases they have limitations. In situations where more confidence is required, a higher level of analysis should be adopted by using numerical modelling such as finite element or finite difference methods (Phear *et al*., 2005).

In this study finite element computer program 'PLAXIS' was deployed to simulate the excavation sequence and installation of nails and to carry out back analysis for the six case studies in Istanbul of which performance analysis were achieved by means of inclinometer monitoring of lateral displacements.

By means of back analysis average stiffness values of greywacke formation encountered in the case studies are obtained. Average stiffness of the subsoil formation of the case studies are; BJK Fulya Complex, which has the strongest silicified sandstone, E_{ref}=300 MPa, Istinye Park Complex, which is formed of extensively fractured siltstone and claystone, E_{ref} = 80 MPa, Kanyon Complex, which is composed of extensively fractured sandstone, siltstone and claystone, E_{ref} = 160 MPa, Mashattan Residence, same as Istinye Park Complex, which is formed of extensively fractured siltstone and claystone, E_{ref} = 90 MPa, Tepe Shopping Mall, which has the weakest very extensively fractured siltstone and claystone, E_{ref} = 30 MPa, Besler Warehouse, which is formed of strong sandstone with fractures, E_{ref} = 250 MPa. Average stiffness values obtained from finite element back analysis are in good agreement with the previous study.

Finite element back analysis of the case studies show that lateral displacements are inversely proportional with stiffness and the alteration of the stiffness value changes the lateral displacements considerably. The degradation ratios for greywacke formation to obtain E_m to be utilized in prediction of lateral displacements of soil nailed walls are large as well. (Durgunoglu and Yilmaz, 2007). Therefore, the modulus value of greywackes of Istanbul to be utilized in numerical models such as finite element computer program, PLAXIS, to predict the displacements do vary in a very big range. Consequently, the prediction of displacements of soil nailed walls based on selected modulus values are only indicative.

In addition, although the lateral displacements of the soil nail walls can be predicted by estimating an average stiffness of the subsoil layers in numerical modelling, in reality, the stiffness depends on the stress level, therefore stiffness generally increases with depth. Even the computer programs put some incremental parameters with depth to raise the stiffness with depth, it is seen from the back analysis of a cross-section from Case No. 3 Inclinometer 7, the increase of the modulus with depth is not a linear function and the rate of increase of the stiffness decreases with depth, as expected, especially when it reaches the intact bedrock.

The accuracy of numerical modelling depends on the quality of data acquired, the estimation of in-situ stress and soil stiffness and the availability of good case histories to calibrate numerical models. The stiffness parameters in these models should be adjusted to match the values obtained from actual site monitoring results at an early stage. Verification and improvement of design should continue during the construction stage through close observation and monitoring techniques in order to recognize and feed back potential problems and also to make potential cost savings. As a result, direct measurement of lateral displacements of deep soil nailed walls is compulsory in order to follow performance of such structures during and after construction.

Finally, an interesting case study is presented for the implications of various retaining wall systems at a specific project site based on the observed complex geological and groundwater conditions over greywacke bedrock. It is shown that, site subsoil conditions, ground water regime, the topography and the architectural elevations and locations imposed by the project have dictated the tailor made retaining wall design and construction for this specific case.

Except some minor variations employed as described herein total of five different retaining wall system have been employed having temporary and permanent parts. Due to the high seismicity of the site the permanent walls are preferred to be flexible type, i.e. soil nailing, except from the top part of the Type 1 retaining system which had to employed manually constructed very rigid caissons due to limitations of the space for construction equipment and more strict lateral displacement limitations at nearby infrastructures. Both flexible and rigid types of retaining wall systems, on the other hand, have been employed for the temporary structures.

The performance criteria for the walls were based on the observed lateral displacement during excavation. The careful monitoring of the various systems by means of inclinometers have provided the opportunity to implement further measures. The observed form of lateral displacement vs. excavation depth relationships are in good agreement with the previous findings for various retaining systems. In addition, the performance ratios defined as the ratio of maximum lateral displacement to excavation height were within the range of 0.1 to 0.2 % which were below the critical value of 0.3 % imposed in the contract documents indicating a satisfactory performance of various retaining systems employed within the same site.

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