DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

# A CASE STUDY: A DEEP RETAINING SYSTEM CONSTRUCTION WITH PRESTRESSED ANCHORS

by
**Mustafa SÜTCÜOĞLU**

April, 2010
İZMİR

# A CASE STUDY: A DEEP RETAINING SYSTEM CONSTRUCTION WITH PRESTRESSED ANCHORS

**A Thesis Submitted to the**
**Graduate School of Natural and Applied Sciences of Dokuz Eylül University**
**In Partial Fulfilment of the Requirements for the Degree of Master of Science in**
**Civil Engineering, Geotechnics Program**

**by**
**Mustafa SÜTCÜOĞLU**

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**İZMİR**

## M.Sc. THESIS EXAMINATION RESULT FORM

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## ABSTRACT

Pre-stressed anchored walls are one of the most common deep excavation support systems in Istanbul. The risk associated with deep excavation works is high as failures of retaining wall or anchors could be catastrophic and may affect surrounding areas. Therefore, the design of anchored walls for deep basement construction works requires careful consideration of soil/rock-structure interaction. This is usually accomplished using the finite element method. However, in order to produce safe and economical design, proper understanding of the soil/rock behaviour is very important.

In this study, applicability of pre-stressed anchors as stability increasing elements in deep excavations performed in highly weathered graywackes involving considerable discontinuous features is investigated with reference to field data obtained in a deep cut retaining system project. It was concluded that application of tension forces to already present discontinuous zones in jointed graywackes might have led to unexpected deformations in spite of the fact that the original project appeared to be over safe in many respects. It would be a better engineering approach to employ rock nails instead of pre-stressed anchors to increase overall shear strength of the rock mass and to avoid opening of the cracks and triggering of slides on residual sliding surfaces. One should be cautious to determine the weathering state of the rock mass and discontinuities during the site investigation stage.

**Keywords:** Anchor, deep excavation, finite element, greywacke, jointed rock, segmented reinforced concrete wall

# ÖNGERMELİ ANKRAJLI BİR DERİN İKSA SİSTEMİ İNŞAATI

## ÖZ

İstanbul'da yapılan derin kazılarda uygulanan en yaygın derin kazı destek sistemlerinden biri öngermeli ankrajlı dayanma sistemleridir. Derin kazı çalışmaları yüksek riskler içerir ve ankraj veya dayanma yapısının göçmesi gibi çevresini etkileyebilecek felaketlere neden olabilir. Bu nedenle, derin kazılar için uygulanacak dayanma yapısı ve ankraj sistemlerinin tasarımında zemin/kaya-yapı etkileşiminin dikkatle değerlendirilmesi gereklidir. Bu konuda genellikle sonlu elemanlar yöntemi kullanılır. Ancak, ekonomik ve güvenli bir tasarım yapabilmek için bu analizlerde kullanılacak zemin/kaya malzemesi bünye modellerinin iyi anlaşılması ve zemin/kaya davranışına uygun modelin kullanılması gereklidir.

Bu çalışmada, öngermeli ankrajların çok ayrışmış ve önemli derecede süreksizlikler içeren grovaklarda yapılan bir derin kazı destek projesindeki performansı saha ölçümleri ışında incelenmiştir. Sonuç olarak, uygulama projesinin birçok bakımdan aşırı güvenli olmasına rağmen, öngermeli ankrajlarda uygulanan çekme kuvvetinin çatlaklı grovaklarda zaten var olan süreksizliklerin daha ilerlemesine ve beklenmeyen hareketlerin ortaya çıkmasına neden olabileceği anlaşılmıştır. Öngermeli ankrajlar yerine, kaya çivilerinin kullanılması kayanın genel kayma dayanımı arttırması ve çatlakların daha ileri seviyede açılmasını tetiklememesi nedeni ile mühendislik yaklaşımı bakımdan daha uygundur. Bu tür kayalardaki çatlak ve ayrışma düzeyinin belirlenmesine ayrıca özen gösterilmelidir.

**Anahtar sözcükler:** Ankraj, çatlaklı kaya, derin kazı, grovak, sonlu elemanlar, yatay hareket, yerinde dökme betonarme duvar

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# CHAPTER ONE
# INTRODUCTION

The city of Istanbul has performed significant growth in economy and population especially during the last decade. While becoming the biggest metropolitan city of the region, the need for high-rise 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 entertainment facilities, deep excavations and construction of retaining structures became compulsory. The depths of the excavation commonly reach to 25 to 40 meters below the ground surface.

Selection of an appropriate excavation method and the retaining system necessarily considers many factors, such as depth of cut, area of construction site, subsoil profile and engineering characteristics of soil and/or rock formations, groundwater profile, construction budget, allowable construction period, existence of adjacent excavations, availability of construction equipment, conditions of adjacent buildings, foundation types of adjacent buildings, and so on.

Retaining systems with prestressed anchors are very common type of the supporting deep excavation in the city of Istanbul. Bored piles, micro piles and cast in-situ reinforced concrete walls with pre-stressed anchors are used to support deep excavations in all subsoil/rock and ground water conditions.

In order to facilitate deep excavations, many projects around the world have utilized cast in-situ reinforced concrete walls (usually diaphragm walls) for both temporary lateral earth support and permanent basement walls. In this type of retaining system reinforced concrete walls are used instead of bored piles, micro piles and other retaining systems. The types of cast in-situ reinforced concrete walls are manual caisson walls, segmented reinforced concrete walls, where there is no groundwater table and the diaphragm walls under groundwater table.

In the second and third subchapter of the chapter two, general overview of pre-stressed grouted ground anchors and cast in-situ reinforced concrete walls including manual caisson walls, segmented reinforced concrete walls and diaphragm walls are presented severally. In the forth subchapter introduces the general design procedure of the cast in-situ reinforced concrete walls with pre-stressed ground anchors.

The process of an excavation may encounter different kinds of soils and/or rocks underneath the same excavation site - from soft clay to hard rocks. The closer the construction site to a hillside, the more complicated the geological condition. The geological condition determines the type and construction of retaining system and greatly influences the excavation behaviour as well. In addition to the geological condition, the distribution of groundwater also contributes to the excavation behaviour.

One of the most specific common traits of the deep excavation projects in the city of Istanbul is that they are generally constructed in greywackes, a rock type frequently encountered in Istanbul construction sites. The greywackes identified as a jointed rock with various degrees of weathering by many investigators. In chapter three, general information on Istanbul greywackes are presented.

Rock differs from most other engineering materials in that it contains discontinuous features. The mechanical behaviour of a rock mass depends on both the properties of the intact rock and the joints (discontinuities). For this reason, general information on jointed rocks and effects of the discontinuities on the deformability end strength characteristics are presented in the fourth chapter. Additionally, a general overview on the modelling and design of excavations in the jointed rock mass with finite element method are presented in this chapter.

In chapter five, the effects of discontinuities on rock mass behaviour of greywacke during excavation are investigated by means of a parametric study. For that purpose four cases are distinguished using a “jointed rock model” that is available in a commercially available finite element (FE) software, according to joint set number and to whether excavation is supported or not.

In chapter six, a deep excavation case study in locally well-known greywacke formations supported by segmented reinforced concrete walls with prestressed anchors with a total surface area of approximately 4,100 $m^2$ (Point Hotel Barbaros, Istanbul) is examined. The performance of the retaining system is monitored by inclinometer recordings taken at certain time intervals in parallel to the excavation at various locations. As a result the performance assessment for cast in-situ segmented reinforced concrete walls with prestressed anchors in typical greywacke formation of the city of Istanbul is made based on this case study as a guideline for future applications.

In the last chapter the results and the general evaluations on the performance of the deep excavations supported by segmented reinforced concrete walls with prestressed anchors in jointed rock greywacke are given. The borehole log charts of case study area are given in the appendices.

# CHAPTER TWO
# GENERAL OVERVIEW OF CAST IN-SITU REINFORCED CONCRETE WALLS WITH PRE-STRESSED GROUTED GROUND ANCHORS

## 2.1 Introduction

During the last ten years cast in-situ reinforced concrete walls with pre-stressed anchors have been extensively constructed within the city of Istanbul as temporary or/and permanent retaining walls to support the basement excavations of various structures. According to recent compilation by Zetas (2009) about 20,000 $m^2$ of wall structures have been constructed in different projects in last three years. Figure 2.1 is a picture of an anchored cast in-situ retaining wall system.

Figure 2.1 Anchored cast in-situ reinforced concrete retaining wall construction (Zetas Zemin Tenolojisi A.S., 2008).

In this chapter general overview of pre-stressed grouted ground anchors, cast in-situ reinforced concrete walls and systems of their combination are presented.

## 2.2 A General Overview of Pre-stressed Grouted Ground Anchors

### *2.2.1 General*

Ground anchors and anchored systems have become increasingly more cost-effective through improvements in design methods, construction techniques, anchor component materials, and on-site acceptance testing. This has resulted in an increase in the use of both temporary and permanent anchors.

Ground anchors can be classified in several different configurations. However, anchors are divided into two main categories - strand and bar anchors. The type of anchors used depends on whether it is for rock or soil, for temporary or permanent use, whether or not it is to be tensioned, and whether or not permanent corrosion protection is required, etc.

Both bars and multi strand tendons are commonly used for soil and rock anchors for excavation support around the World. In Turkey, multi-strand tendons are usually used for excavation support systems. Generally, multi-strand tendon type anchors are used for deep excavation projects as temporary lateral support members. Therefore, only temporary ground anchors with multi-strand tendons are mentioned in the scope of this document.

Multi-strand tendon anchors are always pre-stressed (post-tensioned). A pre-stressed ground anchor is a structural element installed in soil or rock that is used to transmit an applied tensile load into the ground. Multi-strand tendon ground anchors, referenced simply as ground anchors, are installed in grout filled drill holes. Pre-stressed ground anchors are also referred to as "tiebacks".

In general, a pre-stressed ground anchor consist of multi-strand tendons embedded in a drill hole and secured at its end (root) region via grout injection. Fixed end section of the anchor is located at a stable region outside the failure wedge or spiral. As the support system is tied to this anchor, a reaction force is obtained to stabilize the wall against the earth pressure acting on it.

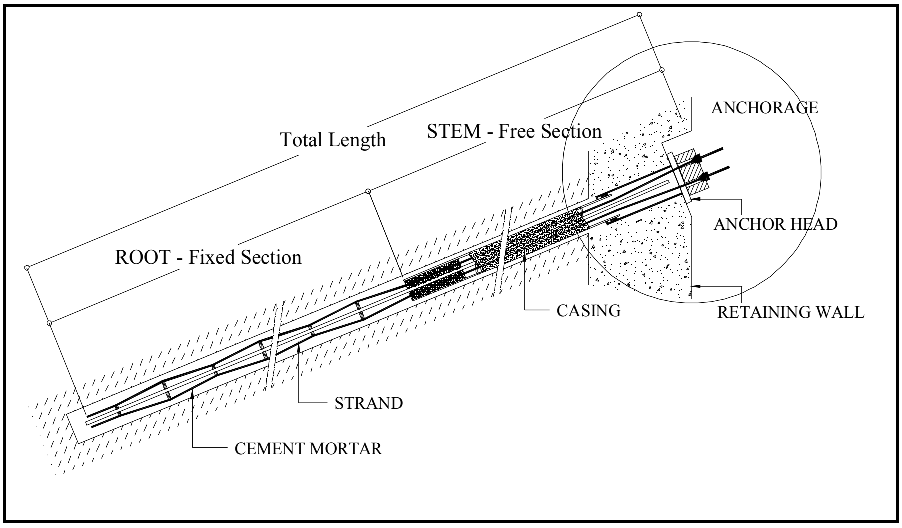

Figure 2.2 Basic configuration of a multi strand ground anchor.

The configuration of a temporary pre-stressed ground anchor can be divided into (1) Root; the fixed section—which offers anchoring force, (2) Stem; the free section—which transfers the anchoring force to the anchor head, and (3) Anchorage; the anchor head—which locks the tendons and transfers the anchoring force to the structure (retaining wall) (see Figure 2.2).

Root is the grouted tip of the anchor consisting of tendons. The tendons are equipped with rust covers to keep them at the centre of the hole. The tendons are separated by spacers, and connected by wires or tapes to enhance their adhesion to the grouted anchor root.

Stem is the stressed (post-tensioned) central portion of the anchor. The stressing jack pulls, elongates and pre-stresses the stem which is normally embedded into a plastic hose or a flexible PVC pipe (sheath) to protect it from the grout and moisture.

Anchorage is the fastened part that sticks out of the anchor hole and the special mechanism that lock the anchor to stable head block. For multi tendon anchors, main locking elements are conical wedges that fit into grooves of the steel anchor head which then rests on a bearing plate supported by a reinforced concrete or steel element on the support wall.

### 2.2.2 Ground Anchor Production

Ground anchors are composed of anchor body (stem and root) and anchorage (bearing plate, anchor head and capping). Manufacturing of ground anchors comprises of preparing the metal, plastic and other mechanical components of the anchor body and preassembling them so as to be ready for installation. Success and durability of ground anchors are directly related to their production quality.

The temporary anchors only fulfil their function for a limited period, generally for a maximum of two years. All materials used must be mutually compatible and material properties must not change during the design life of the ground anchor in such a way that the anchor loses its serviceability.

#### 2.2.2.1 Materials

Contract specifications for anchored systems include a description of acceptable materials and prefabricated elements for use as ground anchor components. General anchor components include prefabricated tendons (or materials for on-site fabrication), anchorage components, grout, spacers, centralizers, and various corrosion inhibiting materials such as greases and concrete.

Conformance to a specification requirement is commonly assessed in one of the following ways: (1) reviewing manufacturer or supplier certification submitted by the contractor; (2) reviewing of product literature and visual inspection; or (3) conformance testing.

*2.2.2.1.1Tendon Materials – Strands.* The multi-strand tendons have been first developed for pre-stressed concrete construction. Strand tendons comprise multiple seven-wire strands. High strength steel wires are woven into 0.5 or 0.6 inch strands. Then, multi-strands, from 3 to 20 are used to form the anchor. Multi-strand tendons have been applied to ground anchors first by Vorspann System Losinger (VSL) from Switzerland and such anchors are often referred to as VSL type anchors.

The common strand in Turkey practice is 15 mm (0.6 inch) in diameter. Tendons using multiple strands have no practical load or anchor length limitations. Steel strands have sufficiently low relaxation properties to minimize long-term anchor load losses. Specific information on relaxation losses and other technical properties should be obtained from the steel strand supplier. Some of the sectional and technical properties of 0.6 inch steel strand produced by VSL (one of the main manufacturers of steel strand all around the world) are presented in Table 2.1 and Figure 2.3 below.

Table 2.1 Specifications for 0.6” steel strand tendon, (VSL, 1995).

| | | |
|---|---|---|
| Characteristic tensile strength of a strand, $f_{tk}$ | N/mm² | 1770 |
| Nominal diameter (0.6”) | mm | 15.7 |
| Cross sectional area of a strand, $A_p$ | mm² | 150 |
| Characteristic load capacity of a strand, $P_{tk}$ | kN | 265.5 |
| Characteristic yield strength of a strand, $f_y$ | N/mm$^2$ | 1590 |
| Specific elongation under maximum load | % | 3.5 |
| Contraction, y | % | 30 |
| Elasticity modulus (average value), $E_p$ | kN/mm$^2$ | 195 |
| Fatigue resistance | Cycles | 2 x $10^6$ |
| - maximum stress, $\sigma_0$ | % of $f_{tk}$ | 70 |
| - stress variation, $\Delta\sigma$ | N/mm$^2$ | 200 |
| Relaxation after 1000 h, 20° C, 0.70 $f_{tk}$ | % | max. 2.5 |

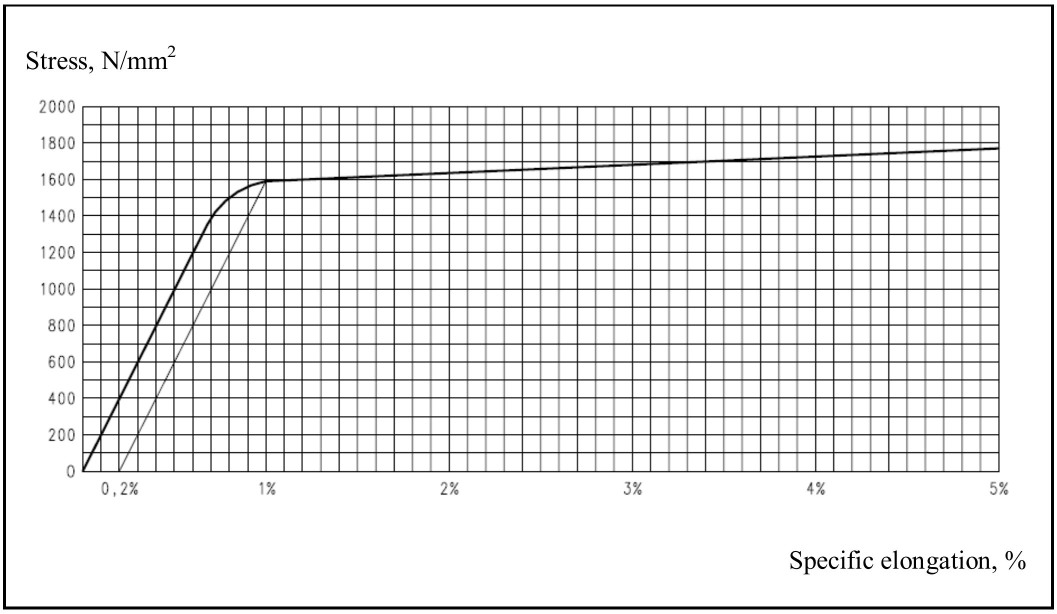

Figure 2.3 Stress-elongation diagram (VSL, 1995).

To account for these load losses, the load that is transferred to the anchorage may be increased above the desired load based on results of a lift-off test. For strand tendons, the lift-off test is performed by gradually reapplying load to the tendon until, for restressable anchor heads, the wedge plate lifts off the bearing plate (without unseating the wedges) or, for cases where the hydraulic jack rests on the anchor head, the wedges are lifted out of the wedge plate. After the losses, the transferred load will reduce presumably to the desired long-term load.

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*2.2.2.1.2 Spacers and Centralizers.* Spacer and centralizer units are placed at regular intervals (e.g., typically 3 m) along the anchor bond zone. For strand tendons, spacers usually provide a minimum interstrand spacing of 6 to 13 mm to prevent the intertwining (tie or link together) of adjacent strands and a minimum outer grout cover of 13 mm. According to United States Department of Transportation Federal Highway Administration [US-FHWA], (1999), the centralizer shall be able to support the tendon in the drill hole and position the tendon so a minimum of 13 mm of grout cover is provided and shall permit grout to freely flow around the tendon and up the drill hole.

Both spacers and centralizers should be made of noncorrosive materials and be designed to permit free flow of grout. Figure 2.4 shows typical spacers and centralizers. In many instances, it is feasible and practical to combine the characteristics of spacers and centralizers into one unit.

Figure 2.4 Typical spacers and centralizers.

*2.2.2.1.3 Sleeves – Sheaths.* The elongation of the strands in the free length is provided by the sleeve during stressing. This sleeve is also referred to as “bondbreaker”. A bondbreaker is a smooth sheath used in the unbonded length that allows the pre-stressing steel to freely elongate during testing and stressing, and to remain unbonded to the surrounding grout after lock-off.

The bondbreaker shall be fabricated from a smooth plastic tube or pipe having the following properties: (1) resistant to chemical attack from aggressive environments, grout, or corrosion inhibiting compound; (2) resistant to aging by ultra-violet light; (3) fabricated from material nondetrimental to the tendon; (4) capable of withstanding abrasion, impact, and bending during handling and installation; (5) enable the tendon to elongate during testing and stressing; and (6) allow the tendon to remain unbonded after lock-off.

*2.2.2.1.4 Grout Tubes*. Grout tubes must have an adequate inside diameter to enable the grout to be pumped to the bottom of the drill hole (see Figure 2.5). Grout tubes shall be strong enough to withstand a minimum grouting pressure of 1 MPa (10 bar). Postgrout tubes shall be strong enough to withstand post grouting pressures.

Figure 2.5 Heavy duty plastic grout tube.

### *2.2.2.2 Anchor Fabrication*

Anchors shall be either shop or field fabricated according to the approved working drawings and schedules by personnel trained and qualified for this work.

All materials and prefabricated elements delivered to the site must be visually examined prior to installation to verify required geometry and dimensions and to identify any defects in workmanship, contamination, or damage by handling. All nonconforming materials are unacceptable, unless appropriate corrections are made in accordance with specifications or by written approval of the project engineer.

Pre-stressing steel shall be cut with an angle grinder or when approved by the pre-stressing steel supplier, an oxyacetylene torch may be used. All of the bond length shall be free of dirt, manufacturers' lubricants, corrosion-inhibiting coatings, or other deleterious substances that may significantly affect the grout-to-tendon bond or the service life of the anchor.

Centralizers and spacers help to maintain anchor components parallel and in their correct alignment, and thus prevent contact friction from generating between them. Figure 2.6 shows a cut away section of a multi-strand tendon anchor. This is particularly important in the free length of long anchors where tangling or rubbing of individual wires or strands resulting from a distorted design geometry and initial alignment can cause the loads to dissipate during stressing. Furthermore, extremely high stress concentrations may be generated, especially under the top anchor head, and rupture of individual elements can thus occur.

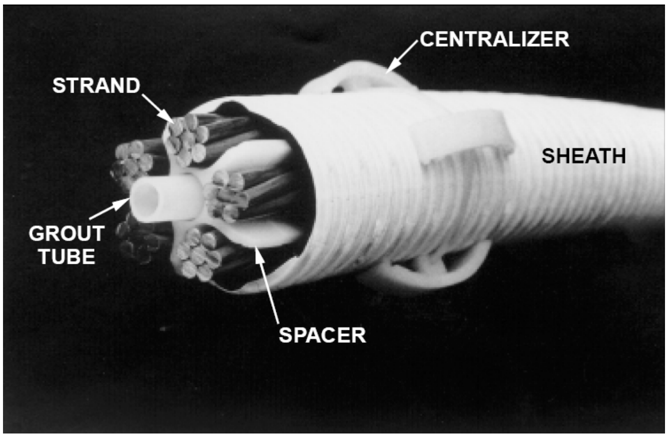


Figure 2.6 Cut away view of multi strand tendon anchor.

Corrosion inhibiting compounds are placed in the free stressing areas. They are of an organic compound with either a grease or wax base. They provide the appropriate polar moisture displacement and have corrosion inhibiting additives with self-healing properties. They can be pumped or applied manually. Corrosion inhibiting compounds stay permanently viscous, chemically stable and non-reactive with the pre-stressing steel, duct materials or grout.

In the fixed anchor zone spacers serve three primary purposes: (1) to centralize the tendon system in the borehole for an adequate and uniform grout cover, which enhances corrosion protection and provides good grout bond at the borehole interface; (2) to provide a positive grip for the tendon and grout without restricting the flow of the latter in the hole in order to completely penetrate the space between tendon units for full cover, a condition ensuring efficient transmission of bond stress; and (3) to help prevent contamination of the tendon parts such as clay smear. Spacers in this zone can also be used in conjunction with intermediate fastenings to form nodes and waves, intended to provide a more positive mechanical interlock between tendon and surrounding grout.

### *2.2.3 Anchor Installation*

The most difficult task of producing a ground anchor is its actual installation at the site. This installation process comprises of three equally important stages:

1. Anchor hole drilling,
2. Placement of the anchor, and
3. Grouting of the anchor.

Only highly specialized geotechnical contractors with experienced project engineers, superintendents and crew can successfully install ground anchors.

#### *2.2.3.1 Anchor Hole Drilling*

This section only dwells on the practical aspects of the drilling for anchors and describes a few common drilling methods for various typical ground conditions.

Unless specified in project documents, the selection of drilling methods and equipment for construction of the ground anchor should be left to the discretion of the contractor. The choice of a particular drilling method must also consider the overall site conditions and it is for this reason that the consultant may place limitations on the drilling method.

Most of the drilling methods selected by the specialty contractor are likely to be acceptable on a particular project, provided they can form a stable hole of the required dimensions and within the stated tolerances, and without detriment to their surroundings. It is important not to exclude a particular drilling method because it does not suit a predetermined concept of how the project should be executed. It is equally important that the drilling contractor be knowledgeable of the project ground conditions, and the effects of the drilling method chosen.

Anchor holes should be drilled at specified locations and tolerances as shown on the approved drawings. Drilling tolerances include length, orientation, and diameter. Common practice is to drill beyond the design length to permit better drill hole cleaning. The ground anchor must not be drilled at a location that requires the tendon to be bent to enable the anchorage to be connected to the anchored system. Orientation of the anchor hole both vertically and horizontally should be checked at the onset and during drilling.

*2.2.3.1.1 Drilling Methods.* The opportunity to use anchors will depend on the ownership of the land at the periphery of the excavations and on permission to found anchors in neighbouring land. The presence of existing substructures or basements may obstruct anchor installation. The drilling method must not adversely affect the integrity of structures near the ground anchor locations or on the ground surface. With respect to drilling, excessive ground loss into the drill hole and ground surface heave are the primary causes of damage to these structures. For example, the use of large diameter augers should be discouraged in sands and gravels since the auger will tend to remove larger quantities of soil from the drill hole as compared to the net volume of the auger. This may result in loss of support of the drill hole.

In unstable soil or rock, drill casing is used. Water or air is used to flush the drill cuttings out of the cased hole. Caution should be exercised when using air flushing to clean the hole. Excess air pressures may result in unwanted removal of groundwater and fines from the drill hole leading to potential hole collapse or these excess pressures may result in ground heave.

Casing the drill hole can increase the cost of anchored walls significantly, to the point where alternative wall construction methods may be more economical. Cased methods of drilling include the use of single tube and the duplex rotary methods. The single tube method involves drilling with one tube (drill string) and flushing the cuttings outside the tube by air, water, or a combination of water and air. The duplex rotary method has an inner element (drill rods) and an outer tube (casing). The assembly allows drill cuttings to be removed through the annular space between the drill rods and outer casing.

Soil and rock types and ground conditions should be recorded during drilling. Unexpected conditions should be carefully documented, and where appropriate, samples should be taken. Drill cuttings and soil exposed in the excavation should be visually classified to identify ground which may be susceptible to caving. Ground which may be susceptible to caving includes: (1) cohesionless soils below the groundwater table; (2) highly fractured or weathered rock; and (3) ground where artesian water pressures exist. Signs of caving include: (1) an inability to withdraw drilling tools; (2) a large quantity of soil removed with little or no advancement of the hole; (3) abnormally large drill spoil pile in comparison to other holes; (4) settlement of ground above the drilling location; and (5) an inability to easily insert the anchor tendon the full length of the drill hole. Where excessive caving occurs drilling should be halted, and alternative drilling methods should be used, such as using drilling fluids or casing to stabilize the drill hole.

The selection of drilling method should account for special concerns identified in project specifications such as noise, dust exposure, vibrations, hole alignment, and damage to existing structures. The inability of the contractor to establish a stable hole of adequate dimensions and within specified tolerances may be cause for modification of drilling methods. The main drilling methods for each of the three main soil and rock ground anchors include rotary, rotary-percussive, or auger drilling.

*2.2.3.1.1.1 Rotary Drilling Metho*. However, air or any other liquid or mixture can be used this method often called "mud rotary drilling". Rock bit (Figure 2.7-a and Figure 2.7-b), soil bit (Figure 2.7-c) or drag bit (Figure 2.7-d) are used on the end of the rod (Figure 2.7-e) to drill the holes. Cuttings are carried away by fluid circulated down through the centre of the shaft and up through the annulus of the hole.

a - Rock bit (tri-cone) b - Rock bit c - Soil bit d - Drag bit (Clay)

e – Drilling rod

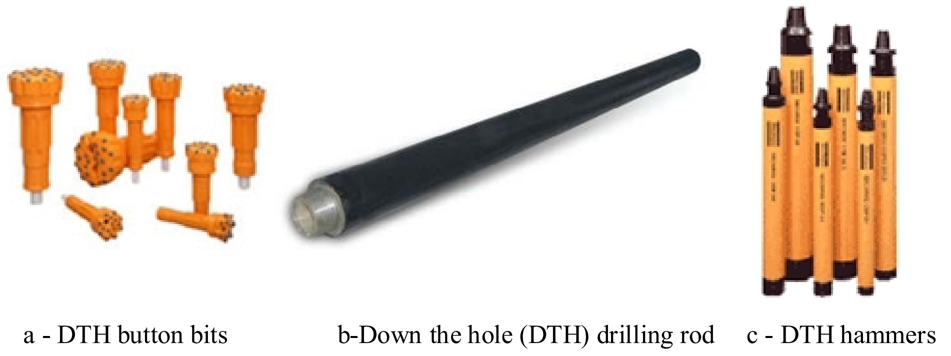
Figure 2.7 Some of the basic components of the rotary type drilling tools.

Rotary drilling method is usually used to drill fine-grained (or cohesive) soils may include stiff to hard clays, clayey silts, silty clays, sandy clays, sandy silts, and combinations thereof. In general, rotary-percussive method is used to drill rocks. But, in some projects restrictions may be imposed on certain drilling method if it is deemed that they might have an effect on the integrity of adjacent structures or underground utilities. In that case, rotary drilling method is used instead of rotary-percussive method.

Drilling muds and/or foams used for drilling anchor holes must be approved in project specifications or by the design engineer. Bentonite mud should not be used in uncased holes because the bentonite mud will tend to weaken the grout/ground bond. Control and disposal of drilling fluids (i.e., water, muds, and foams) is the responsibility of the contractor.

*2.2.3.1.1.2 Rotary-Percussive Drilling Method.* Particularly for rocks of high compressive strength, rotary percussive techniques using either top-hammer or down-the-hole hammers are utilized. For the small hole diameters used for ground anchors down-the-hole techniques are the most economical and common.

The drill bit (generally button bit, Figure 2.8-a) is both percussed and rotated. In general the percussive energy determines the penetration rate. With a top hammer, the drill rods are rotated and percussed by the drill head on the rig. With a down-the-hole hammer, the (larger diameter) drill rods (Figure 2.8-b) are only rotated by the drill head, and compressed air fed down the rods activates the percussive hammer (Figure 2.8-c) mounted directly above the drill bit. The drill cuttings are carried out through the space between the rod and the wall of the hole.



a - DTH button bits    b-Down the hole (DTH) drilling rod    c - DTH hammers

Figure 2.8 Some of the basic components of the DTH rotary-percussive type drilling tools.

*2.2.3.1.1.3 Auger Drilling.* Auger drilling method one of the most common drilling methods used for ground anchors because of high installation rates and low costs. It is anticipated that either auger (first option) or driven casing methods will be used for the drilling considering subsoil and groundwater conditions at the site.

The method consists of rotating an auger while simultaneously advancing it into the ground either hydraulically or mechanically. The auger (Figure 2.9-a) is advanced to the desired depth and then withdrawn. The cuttings can be removed from the auger. However, it cannot be used effectively in soft or loose soils below the water table without casing (Figure 2.9-b) or drilling mud to hold the hole open.


a-Auger b-Casings

Figure 2.9 Some of the basic components of the auger type drilling tools.

Various forms of cutting shoes or drill bits can be attached to the lead auger, but heavy obstructions, such as old foundations and cobble and boulder soil conditions, are difficult to penetrate economically with this system. In addition, great care must be exercised when using augers. Uncontrolled penetration rates or excessive hole cleaning may lead to excessive spoil removal, thereby risking soil loosening or cavitation in certain circumstances.

In stiff to hard clays without boulders and in some weak rocks, drilling may be conducted with a continuous flight auger. Such drilling techniques are rapid, quiet, and do not require the introduction of a flushing medium to remove the spoil. There may be the risk of lateral decompression or wall remoulding/interface smear, either of which may adversely affect grout/soil bond. Such augers may be used in conditions where the careful collection and disposal of drill spoils are particularly important environmentally.

*2.2.3.1.2 Unfavourable or Difficult Soil Conditions for Anchor Hole Drilling.* Large amounts of groundwater can cause drill holes (particularly in loose granular soils) to collapse. Additionally this type of soil may overflow from the hole easily even temporary casing will used. In this case, pre-grouting or jet-grouting is performed to stabilize the ground and allow the hole for the tendon installation to be advanced to the final location.

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. When only a few boulders and cobbles are present, modifying the drilling orientation from place to place may minimize or eliminate most of the difficult drilling. However, this approach has practical limitations when too many boulders are present.

Weathered rock may provide a suitable supporting material for anchors as long as weakness planes occurring in unfavourable orientations are not prevalent (e.g., weakness planes 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. Conversely, a highly variable degree of rock weathering at a site may require changes in drilling equipment and/or installation techniques and thereby cause a costly and prolonged ground anchor installation.

*2.2.3.1.3 Other Drilling Difficulties.* The depth is always an important factor of difficulty in foundation engineering. The drilling rods' torsional and flexural rigidities need to be increased with depth which means that their weights also increase. The manoeuvring times also increase as additional segments of rods would need to be added or extracted. All these factors result in the usage of heavier and more expensive drilling rigs which have higher extracting and penetration capacities. Also the flushing of the excavated debris more and more difficult as the depth increases. Thus, capacities of compressors and water pumps must be increased. All these increases in capacity mean more expensive and heavier accessory equipment. Heavier drilling and accessory rigs are difficult to mobilize and costs add up to make deep drilling, a very expensive operation.

The diameter is also an important factor of difficulty in drilling. Needed torques, and therefore, torsional rigidities, and thus, weights of the rods increase. This would mean more powerful, heavier and more expensive drilling rigs. Flushing volumes and pressures must be increased which requires more powerful, heavier and more expensive accessory equipment. Mobilizations would be more difficult and costly for the heavier rigs.

*2.2.3.1.4 Drilling Rigs.* The drill rigs typically used are hydraulic rotary (electric or diesel) power units. They can be track mounted; this allows manoeuvrability on difficult and sloped terrain. Figure 2.10 shows a modern drilling rig used for construction of the ground anchor.

Figure 2.10 Track mounted drilling rig, EGT MD 1500.

The size of the track-mounted drills can vary greatly. With the larger rigs allowing use of long sections of drill rods and casing in high, overhead conditions, and the smaller rigs allowing work in lower overhead and harder-to-reach locations.

These rigs are mostly of the rotary-percussive type that use sectional augers or drill rods. For deeper ground anchor excavations requiring longer anchor lengths, larger hydraulic-powered track-mounted rigs with continuous-flight augers may be used.

The rotary head that turns the drill string (casing, augers, or rods) can be extremely powerful on even the smallest of rigs, allowing successful installation in the most difficult ground conditions.

*2.2.3.1.5 Cost Parameters of Drilling.* The main purpose of using a particular drilling technique is the keep the drilling unit and total costs at a minimum while getting the work done. Drilling unit costs depend on various parameters:

Speed of drilling directly reduces the labour and machinery hours. Labour and machinery hour costs are main cost parameters in construction and their reduction directly reduces unit drilling costs.

Cost of labour is a function of the staff numbers and their skill levels. A mainly manual operation requires a higher number of skilled labor and supervisor, increasing overall labour costs. An automated drilling system, on the other hand, would need less personnel and lower labour costs.

Cost of the drilling and accessory equipment is another item which affects the unit cost of drilling. If the equipment is heavy and expensive, the rental (or finance and depreciation), transportation and working platform costs are higher. Therefore, the unit drilling cost increase as a consequence.

Cost of in-the-hole equipment (drilling consumables) may be a decisive factor in determining the minimum unit rate of drilling. Drilling rods, bits, and other consumables add up to significant costs.

Energy costs, drilling water or mud costs, maintenance costs, and other costs are additional cost factors which need to be carefully calculated in determining and minimizing the unit cost of drilling.

#### *2.2.3.2 Placement of the Anchor*

Placement of ground anchors is normally the next stage after the drilling. However, it is not uncommon to place the assembled anchors into hole already gravity (or less often pressure) filled with grout so it can also be the final stage. Also, often anchors are placed within the protective casing of the duplex or overburden drilling. Then, anchor placement can also be a phase of drilling process as the drilling operation is considered as completed only after the casing extraction. Thus, drilling the anchor hole, placing in the anchor and grouting the anchor are three interrelated operations which are conducted by the same site crew in a series of steps that complete the production of an anchor with the exception of stressing and mounting the head.

*2.2.3.2.1 Drill Hole Inspection.* Drill holes 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. After drilling, uncased and drilled casing holes should be thoroughly cleaned to remove loose material within the design length. For uncased holes in cohesionless soils, excessive cleaning should be avoided such as would cause significant ground loss. After cleaning is complete, uncased holes should be inspected with a mirror, high intensity light, or by probing. If the hole is to be grouted prior to insertion of the anchor, the hole depth should be measured to ensure that the anchor can be installed to the full depth. Drill holes may be considered clean if the full length of the anchor can be easily inserted to the desired depth.

*2.2.3.2.2 Tendon Inspection.* Just prior to insertion of the anchor, exposed steel surfaces should be inspected for unacceptable amounts of corrosion. Loose flaky rust must be removed and the tendon surface inspected for corrosion appearing to be deeper than the steel surface (i.e., pitting). Where corrosion penetrates the steel surface, the steel is unsuitable for use. The presence of light non-flaky rust is not necessarily harmful and is not cause for rejection of the tendon. The tendon bond length must be clean and free from any foreign substances.

The presence of rust on pre-stressing steel strand has been a source of controversy between the contractor and inspector/client. This is due, at least in part, to a lack of a clear understanding of how much rust can exist on the strand surface without any detriment to the performance of the strand.

Bright strand refers to the surface quality of uncoated strand with no signs of rusting (Figure 2.8-a1).

When pre-stressing strand is exposed to a humid atmosphere, the original bright surface condition of the strand will not last very long. Weathering, which is the initial stage of oxidation, starts to take place. It is difficult to determine the degree of weathering until visible rust begins to appear on the strand surface. Rusting will inevitably take place when the weathered surface is continuously exposed to dampness or a humid atmosphere.

Light rust does not harm any of the properties of the strand and it actually enhances bond. Rust alone is not a cause for rejection.

A pit visible to the unaided eye, when examined as described herein, is cause for rejection. A pit of this magnitude is a stress raiser and greatly reduce the capacity of the strand to withstand repeated or fatigue loading. In many cases, a heavily rusted strand with relatively large pits will still test to an ultimate strength greater than specification requirements. However, it will not meet the fatigue test requirements.

In order to evaluate the extent of pitting, the superficial rust has to be removed. Care shall be taken to not abrade the strand surface below the iron oxide or rust layer. This may be accomplished by cleaning the surface with Scotch Brite™ cleaning pads in order to expose the pits. Scotch Brite Cleaning Pad or its equivalent is a synthetic material which is non-metallic. This material is available from cleaning supply retailers or supermarkets for general purpose cleaning.

After the strand is cleaned, it can be checked again. A procedure has been developed by Sason (1992), for classifying the degree of rust on the stands. For this purpose, a number of sample pictures have been used by Sason (1992), (see Figure 2.11). These pictures can be used as visual standards from which the contractor and inspector may agree on the surface quality that is acceptable. Following the above-mentioned procedure, the strand in question may be accepted or rejected by comparing the cleaned surface with the picture that is previously agreed upon as the standard.

According to Sason's (1992) Figure Sets a through c are acceptable. Figure Set d is borderline and is subject to discussion, agreement or compromise. Some engineers may find this level of rusting objectionable for critical applications. Figure Sets e and f are pitied and unacceptable.

*2.2.3.2.3 Insertion of the Anchor.* The dimensions of each tendon should be checked to ensure that the minimum bond and unbonded lengths are equal to or exceed the minimum values specified for that anchor. Anchors may have specified maximum values if right-of-way restrictions exist. Coatings, sheaths, and encapsulations must be undamaged. Damage to protective layers should be repaired; otherwise the anchor should not be used.

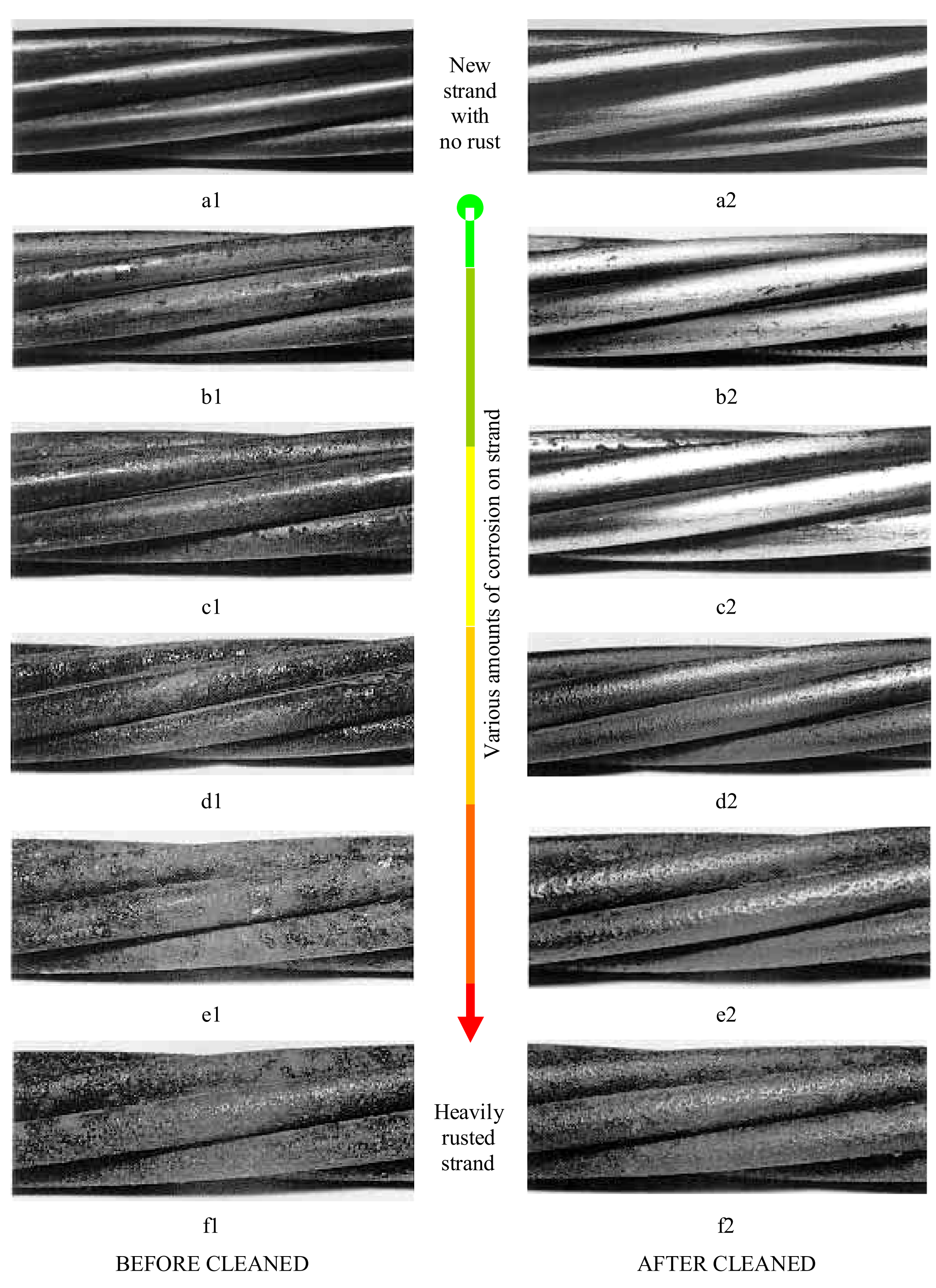


Figure 2.11 Various degrees of corrosion on strands (Sason, 1992)

The main gimmick of anchor placement is the placement of the anchor without causing any damage to the manufactured anchor and its components. Equally important is the avoidance of damage to the excavated hole and the ground surrounding the hole. The target is the getting a groutable and sound anchor which is able to safely carry its design loads and maintain its protection as design.

If caving occurs during installation, the anchor should be withdrawn and the hole redrilled. The tendon must not be driven, or the unbonded length cut off. In some instances, the contractor may desire to remove the tendon and cut off enough of the bond length such that the full unbonded length and shortened bond lengths can be inserted. If this is done, the bond length must still exceed the minimum specified bond length. Shortening the bond length may result in the anchor not meeting load testing acceptance criteria.

#### *2.2.3.3 Grouting of the Anchor*

The grout is a cement based mixture that provides load transfer from the tendon to the ground and provides corrosion protection for the tendon. Hence, the success of the grouting process is the decisive factors of an anchor's overall performance and load bearing capacity.

Anchor grout for soil and rock anchors is typically a neat cement grout (i.e., grout containing no aggregate) although sand-cement grout may also be used for large diameter drill holes.

High speed cement grout mixers are commonly used which can reasonably ensure uniform mixing between grout and water. A water/cement (w/c) ratio of 0.4 to 0.55 by weight and Type I cement will normally provide a minimum compressive strength of 21 MPa at the time of anchor stressing (FHWA, 1999). Type I Portland cement (CEM-I in Turkish Standards) is normal, general-purpose cement suitable for all uses.

##### *2.2.3.3.1 Materials.*

*2.2.3.3.1.1 Cement.* CEM-I (ordinary Portland) type cement is normally used for grout in normal environments. But sometimes, the deterioration of the grout leaves prestressing steel vulnerable to corrosion. The primary mechanism for degradation of cement-based grout is chemical attack in high sulphate environments, such as in marshy areas and in sulphate bearing clays.

The common approach to minimizing the potential deterioration of grout in high sulphate environments is to select a cement type based on the soluble sulphate ion (SO4) content of the ground. The aggressivity of the environment shall be defined in accordance with TS-EN 206.

*2.2.3.3.1.2 Water.* Water for mixing grout shall be potable, clean, and free of injurious quantities of substances known to be harmful to Portland cement or prestressing steel.

*2.2.3.3.1.3 Admixtures.* Admixture, if used, shall be provided at the Contractor's own expense. Admixtures shall impart to the grout the properties of low water content, good flow ability, minimum bleeding and controlled expansion. Its formulation shall contain no chlorides or other chemicals in quantities that may have harmful effects on the cement or prestressing steel. The Contractor shall submit to the Consultant the manufacturer's literature indicating the type of admixture and the manufacturer's recommendations for mixing the admixture with the grout. All admixtures shall be used in accordance with the instructions of the manufacturer.

Sometimes excavation projects are constrained by short completion time. In this case, admixtures may be used for increasing rate of strength development. By this way, the admixture increases early and ultimate flexural and tensile strength of the grout. After all, the anchors could be stressed earlier. By this way, especially in staged deep excavations with pre-stressed anchors could be completed earlier.

*2.2.3.3.2 Grouting Methods.* Possibly the most crucial and critical part of the anchor installation operation is the grouting phase. After the drilled hole is cleaned, gravity (no or low pressure) grout is applied by grouting pipe or hose extended to the bottom of the anchor hole until the grout comes out of the anchor hole. To achieve higher anchor capacities pressurized post grouting from a second and even third pipe is common. Also, simple fabric or more elaborate inflatable packers can be used between the root and the stem to enhance grouting pressures and the success of grouting in the first section of the root is most important.

Grouting an anchor rooted in rock is fairly simple, and a carefully conducted gravity grouting from the bottom of the hole is often sufficient. For fissured, porous or karstic rocks a secondary contact grouting may be necessary. However, grouting of anchors in soils is greatly enhanced by post grouting techniques which ensure a sound anchor root bonding in a properly grouted soil zone. To achieve a collinear drilling hole, a temporary support casing is used in most grounds.

The anchor can be placed before or after the casing is extracted. The filling grout can be placed before or after the casing is extracted. In any case, the soil may partially cave in on the root section of the anchor and the root’s adhesion and grout qualities may be insufficient. Post grouting through a secondary hose often remedies these problems. Granular soils, however, usually have rather high permeability, while clayey soils do not. Thus, an anchor installed in a granular soil with high groundwater level often encounters difficulty in sealing bores and grouting because of the higher water pressure outside the excavation area as shown in Figure 2.12. Loose fabrics or plastics placed around the open tendons of the root and grouting into the fabric is useful if there is water movement washing the fresh grout.

Figure 2.12 Problem of the anchored excavation method when applied in the chessionless soil with high groundwater level (Zetas, 2007)

When soils are loose or even if a good anchor root is formed, the soil root friction is insufficient. In such cases, post grouting under pressure from various points along the root can be done to enhance the bonding capacity by better permeation of the grout in granular soils or forming bulges of higher diameter grouted zone in soft cohesive soils.

### *2.2.4 Installation of Anchorage*

Once grouting is complete, the anchorage should be installed. The anchorage and tendon must be properly aligned. The anchor bearing plate and the anchor head are installed perpendicular to the tendon, within plus/minus three (3) degrees and centred on the bearing plate, without bending or kinking of the prestressing steel elements (see Figure 2.13). Wedge holes and wedges shall be free of rust, grout, and dirt.

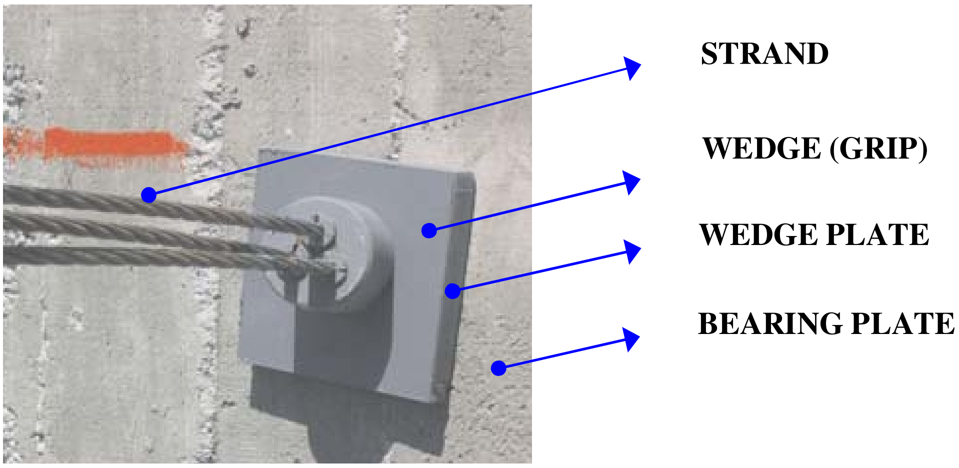


Figure 2.13 Anchorage components for a strand tendon.

The stressing tail shall be cleaned and protected from damage until lock-off. After the anchor has been accepted by the Consultant, the stress tail is cut to its final length according to the tendon manufacturer's recommendations.

The sleeve surrounding the unbonded length of the tendon shall not contact the bearing plate or the anchor head during testing and stressing. If it is too long, the Contractor shall trim the sheath to prevent contact.

### *2.2.5 Stressing and Locking*

A stressing anchorage is used in front of the jack head to grip the prestressing strand tendons during loading. Stressing anchorages are consisting of a combination of a steel bearing plate with wedge plate and wedges. Figure 2.13 shows a set of stressing components. Stressing is performed by load to the tendon until, for restressable anchor heads, the wedge plate lifts off the bearing plate (without unseating the wedges) or, for cases where the hydraulic jack rests on the anchor head, the wedges are lifted out of the wedge plate.

### *2.2.6 Load Testing and Stressing*

The final stage of anchor production is the stressing and locking the stressed anchor with an anchor head. The service life of temporary earth support systems is based on the time required to support the ground while the permanent systems are installed. In this respect, for temporary application, testing scope and program is determined by consultant/client for anchors. Generally a percentage of anchor number of each excavation and/or load stage is decided. Sometimes testing may not be necessary if the anchors have very short service life.

Short-term monitoring is usually limited to monitoring measurements of anchored system performance during load testing. There are three main categories of tests:

1. Design: pull-out test to failure to determine anchorage capacity of ground
2. Fit-for-purpose: pull-out test to failure to check anchorage capacity
3. Acceptance test (+ inspection): all anchors are tested to 1.15 times their service tension load.

Typical load test equipment includes: (1) hydraulic jack and pump; (2) stressing anchorage; (3) pressure gauges and load cells; (4) dial gauge to measure movement; and (5) jack chair. A typical load test setup is shown in Figure 2.14.

After load testing is complete and the anchor has been accepted, the load in the anchor will be reduced to a specified load termed the “lock-off” load. When the lock-off load is reached, the load is transferred from the jack used in the load test to the anchorage. The anchorage transmits this load to the wall or supporting structure.

The lock-off load is selected by the designer and generally ranges between 75 and 100 percent of the anchor design load, where the anchor design load is evaluated based on apparent earth pressure envelopes. Lock-off loads of approximately 75 percent of the design load may be used for temporary support of excavation systems where relatively

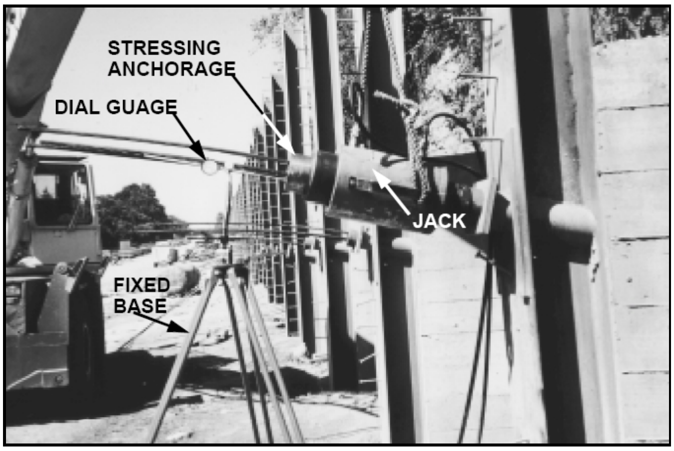


Figure 2.14 Typical equipment for load testing of strand ground anchor.

large lateral wall movements are permitted. Since apparent earth pressure diagrams result in total loads greater than actual soil loads, lock-off at 100 percent of the design load typically results in some net inward movement of the wall. Lock-off loads greater than 100 percent of the design load may be required to stabilize a landslide. For this case, structural elements must be sized to transmit potentially large landslide forces into the ground. Loads consistent with the required landslide restraint force to obtain a target slope stability factor of safety are selected for the lock-off load.

## 2.3 A General Overview of Cast In-situ Reinforced Concrete (RC) Walls

### *2.3.1 General*

In order to facilitate deep excavations, many projects around the world have utilized cast in-situ reinforced concrete walls (usually diaphragm walls) for temporary lateral earth support and/or permanent perimetral basement walls. In this type of retaining system reinforced concrete walls are used instead of bored piles, micro piles and other retaining systems. The types of cast in-situ reinforced concrete walls are manual caisson walls, segmented reinforced concrete walls, where there is no groundwater table and the diaphragm walls under groundwater table.

High costs of construction and especially for diaphragm wall large pit layout occupation are main disadvantages of the construction technologies. These disadvantages can be substantially reduced if the walls are proposed as permanent perimetral basement walls. But, in this type of (multi-purpose) project, wide knowledge and experience required for both design and construction phase. Furthermore, retaining structure design should be verified by structural designer that it is capable for use as a structural member of the building.

### *2.3.2 Forms of Cast In-situ RC Walls*

#### *2.3.2.1 Manual Caisson Walls*

Manual caisson walls are preferred when there is lack of space for construction equipment. Main advantages of the manual caisson walls are; manual installation can be made by excavating with hand tools when the access of the machinery to site is restricted and all of the excavation can be done without the risk of causing damage to existing infrastructure especially when there is lack of information of the location of the existing infrastructures such as excavations at the old residential area. Also some special executions can be done by manual excavation method e.g. the anchors of the existing adjacent excavation can be use under permission of the adjacent owners as a support members of the new excavation (see Figure 2.15).

On the other hand, manual excavations are severely hampered by limitations of depth and by the fact they cannot be used in hard rock. Furthermore, this method is extremely affected by groundwater and surface water. Prior to any excavation, surface water controls should be constructed to prevent surface water from flowing into the excavation.

Reinforced concrete wall construction by hand excavation is the former method of cast in-situ wall construction method and most of the applications are done by manually.

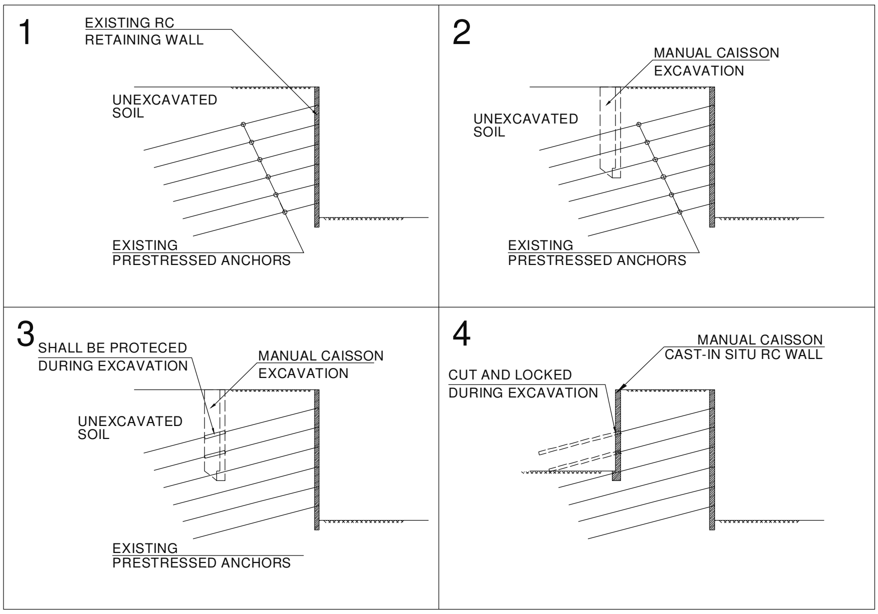


Figure 2.15 Using of the existing anchor strands in a new excavation.

Therefore, the use of manual excavation method has high accident rate. In that respect, to take proper health and safety precautions is very important. All reasonable and appropriate precautionary measures must be taken to ensure adequate protection of workers.

*2.3.2.1.1 Method of Excavation.* The manual caisson retaining wall is constructed as a series of primary and secondary panels (Figure 2.16). Caisson pits are excavated rectangular in plan view, with typical dimensions of 2 m by 2 m, 1.5 m by 3 m or 2 m by 3 m. The excavation width should be at least 1 (one) meter larger than the wall thickness. This area is used as a service hole during form work, concreting after reinforcement installation and form striking after concreting. Personnel and material access within a wall excavation may be provided by an elevator platform rigging using power from a crane hoist. Work may be performed at any depth in the caisson using steel or timber platform decking attached to the timber wall support, from steel scaffolding, or from an elevator platform. This service hole is back filled after concrete will harden.

Figure 2.16 Construction of the primary panels.

The Wall support must be provided for the total depth during excavation (see Figure 2.17). Typical wall support may consist of timber plate segments at the edge of the excavation. Horizontal timber stumps are used as temporary support elements between the wall support plates at different intervals. The thickness of the plates and the distance between the stumps may change with depth. Most of these stumps are lost during excavation. Therefore, the unit drilling cost increase as a consequence.

All obstructions, whether naturally occurring or otherwise, met during the course of excavation shall be removed. This shall include all boulders, old concrete, brick, steel or timber foundations, and roots and buried tree trunks, non-serviceable drains, manholes and gulleys or any other obstructions. The open ends of any such drains etc. shall be sealed with concrete and all voids filled in with granular materials.


(a) (b)

Figure 2.17 A typical manual caisson support configuration. (a) Excavation, (b) Placing of reinforcement.

Water pumped from caissons or the ground shall not be discharged directly onto the ground surface without suitable provisions for drainage being made. The contractor shall be responsible for obtaining all necessary permissions from relevant authorities for discharging water into the public drainage system.

If the excavation and pumping from pit results in settlement adjacent structure of more than as specified in the calculation report, caisson wall construction and dewatering at the appropriate locations shall immediately cease. Excavation or dewatering shall not be recommenced until the construction sequence has been reviewed and measures have been taken to prevent further settlement from occurring. In all cases, work shall not be resumed without the approval of the Consultant. In such cases, caisson wall excavation can be done under the foundation of the existing structure. By the way, the wall can be used as an underpinning system after concreting.

The underpinning guaranties zero settlement at adjacent structure. The thickness of the wall can be thus be partially built under the adjacent building and this means that if a 50 cm thick wall is built and 25 cm is placed under the adjacent building, the total space loss can be as low as 25 cm. For urban structures in congested areas, wall width losses are very important as basement areas can be quite valuable.

The walls are always excavated and constructed deeper than the final excavation level. The embedded length of the wall varies depending on the supporting system of the wall and this socket is generally constructed thicker than the rest.

*2.3.2.1.2 Placing Reinforcement.* Reinforcement shall not be placed until it is inspected and accepted. The minimum cover to all reinforcement shall not be less than specified in the drawings.

Lengths of reinforcing bars are generally joined by lapping. Lateral continuity of the wall is provided by horizontal reinforcement lapping during secondary panel construction. For this purpose, horizontal rebar of the primary panel are bent into L shape at the edge of the adjacent (secondary) panels then bent into straight shape and lapped during secondary panel construction (see Figure 2.18).

By the manual caisson method walls can be constructed as the permanent perimeter structural walls of building basements. Hence, the panels can be properly reinforced to act as the shear walls or columns of the structure. The columns can be integrated within the panels. Also bars for any application can be attached to the caisson reinforcing cage prior to installation and in situ casting.

Connection of the floor slabs into the RC walls is enabled by the use of a box out detail at the slab location. This will allow the connection of the basement and lower ground floor reinforcement into the panel.

*2.3.2.1.3 Placing of Concrete.* All subsoils and debris shall be removed from the caisson before concreting. No concrete shall be placed without the approval of the Consultant. Each caisson shall be concreted in one continuous operation immediately after the excavation has been completed, inspected and approved. If the continuity of placing concrete is interrupted, no further concrete shall be placed until agreed by the Consultant.

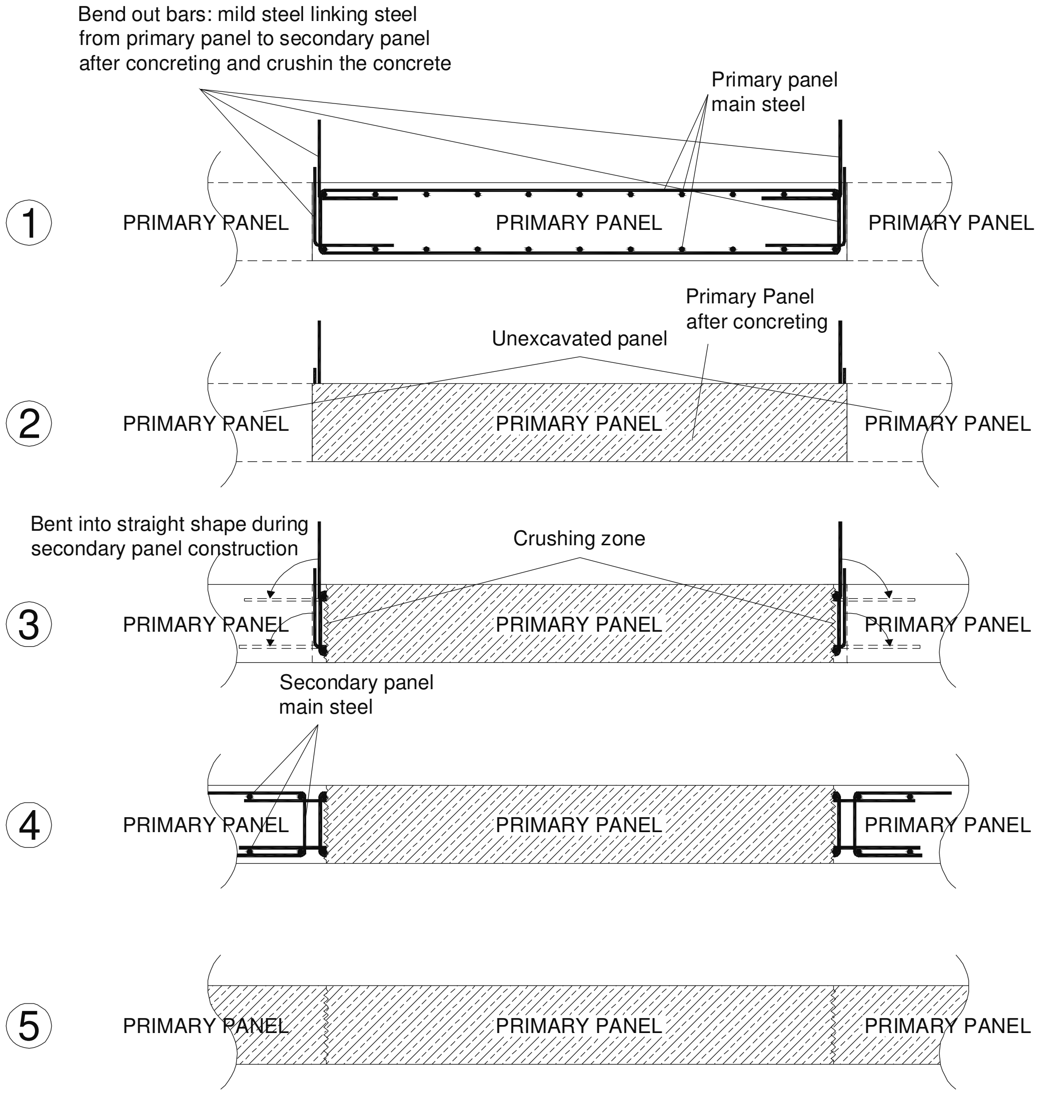


Figure 2.18 Horizontal joint process between primary and secondary panel.

Concrete shall be placed by a method which ensures that the concrete is discharged at a level not higher than 1 m above the surface of the concrete in the caisson. Concrete is generally placed by the concrete pump. Formwork should be continuously watched during and immediately after concreting.

### *2.3.2.2 Diaphragm Walls*

Another type of cast in-situ retaining wall systems widely used in deep excavations in the city centres around the world is diaphragm wall. Diaphragm walls are widely used for the construction of deep basements, underpasses, dock walls, water cut-off walls and similar structures.

The walls are built with a low noise and vibration method; its thickness depends on the structural usage and the equipment used. Diaphragm walls have small deformations, and are therefore mainly used in inner-city foundations as retaining wall (Figure 2.19).

Figure 2.19 Deep excavation with diaphragm wall in the city center of Dubai (Zetas, 2007)

A diaphragm wall (also referred to as slurry wall in the US) consists of RC components constructed in a mechanically excavated trench. The trench is filled with bentonite mud, a suspension of the clay mineral bentonite in water, to support it against collapse during excavation. The term "diaphragm walls" refers to the final condition when the slurry is replaced by tremied concrete that acts as a structural system either for temporary excavation support or as part of the permanent structure.

The diaphragm wall is generally efficient in cost and construction time where it is used for both permanent and temporary subsoil retention for walls of medium and greater depth. In general terms, where they are used for temporary retaining walls, diaphragm walls are more costly than other two types of cast in-situ RC walls because of the high operating cost of machinery. But on the other hand, diaphragm walls can be constructed in practically all types of soil and groundwater lowering is normally not necessary.

The development of diaphragm walling was encouraged by competition among specialist international firms. These firms developed excavation systems of their own. Two preferences quickly became evident: rope grabs suspended by rope either from tripod rigs or cranes; or grabs, generally hydraulic powered, mounted on kelly bars mounted on cranes. The major problem which these methods failed to solve was the excavation of rock.

In the early 1970s the reverse circulation system was used in Japan for a new generation of rail-mounted rigs. Although these rigs were adequate for excavation in granular soils (but less so in cohesive soils), they were the forerunners of modern reverse circulation rigs and used shaker screens and cyclones to remove cuttings from the slurry in a similar way to the latest Hydrofraise and Trenchcutter rigs (Puller, 2003).

Incentives to develop excavation equipment (particularly in rock) have come mainly from the need to improve output at the lowest cost: without exception, panel excavation still remains the critical operation in diaphragm wall contracts despite the complexity of all other operations. Minimization of machine downtime is a vital factor in overall excavation production.

In addition to excavation output, increasing wall depths and the requirements of wall vertically tolerance have placed demands on the development of excavation equipment. To a large extent the requirements of depth and tolerance, together with the particular soil or rock type, define the optimum excavation equipment: rope grab or cutter. Modern cutter machines are capable of cutting through rock with a compressive strength of the order of 200 MPa, and of excavation in soils to depths in excess of 100 m. Cutter machines with small roller bits on the outer edge of the cutters were developed for similar tasks. Vertically tolerances of the order of 0.3% can be made to depths of 50 m. The manufacturers of these rigs currently include Soletanche, Bauer, Casagrande, Soilmec and Tone.

*2.3.2.2.1 Method of Excavation.* Before the main diaphragm wall excavation starts, guide walls are constructed (prefabricated guide walls can be used) (see Figure 2.20). Guide walls not only serve as guides for panel excavation and flow trenches for the excess bentonite suspension, but are also used as reference elevation and for hanging the rebar cages.

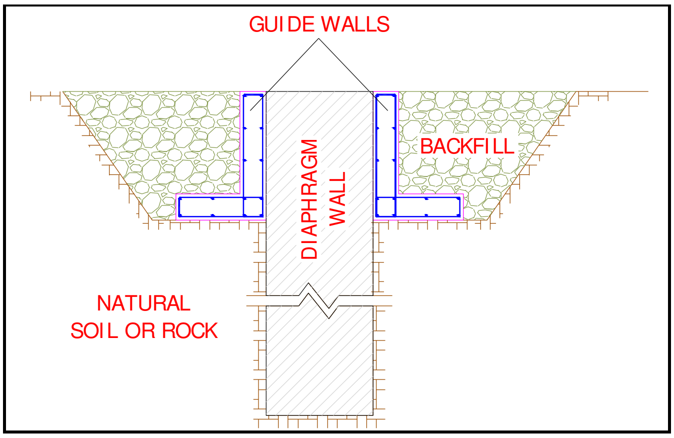


Figure 2.20 Reinforced concrete guide wall.

*2.3.2.2.1.1 Excavation by Grabs*. The diaphragm walls can be excavated by special grabs, which can be mechanical or hydraulic. The heavier and bulkier mechanical grabs are often carried by the cable system of heavy duty crawler cranes. Hydraulic grabs are often lighter, and attached to the end of a hydraulic kelly where the whole system supported by a carrier rig. Each grab has its advantages and disadvantages.

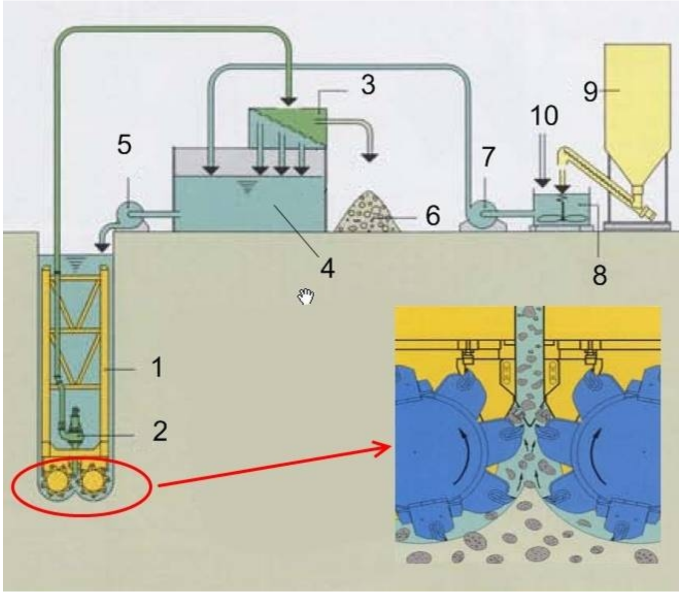
In difficult excavation conditions and for thick or deep panels, heavier and bulkier mechanical grabs are used (see Figure 2.21a). Their weight has the pendulum effect and helps keep the vertically of the wall. In addition, their longer length is a verticality stabilizer. Their weights which can exceed 12 tons help the excavation of hard ground formations, including softer and fissured rocks. However, they would require large carrier rigs. In addition, the manoeuvring of the steel cable mechanized mechanical grabs is slower.

(a) (b)

Figure 2.21 (a) Crane mounted mechanic grab, (b) kelly mounted hydraulic grab.

The hydraulic grabs are lighter and more agile (Figure 2.21b). They are attached to hydraulic kellies which move up and down much faster. The closure of the grab is faster as it is hydraulically powered. However, maintaining the verticality of walls deeper than 30 meters starts to get difficult. Also reaching depths greater than 50 meters requires expensive hydraulic grabs and kellies. Plus, the hydraulic power of the grab action may be not be as strong as the weight enhanced grab action of the mechanical grab.

2.1.1.1.1.1 *Excavation by Reverse Circulation Rotary Cutters.* In the reverse circulation rotary cutter technique, a complex hydraulic cutter is used the excavate rocks or hard grounds by laterally or vertically rotating cutter heads while the reverse circulation of the bentonite mix is regulated by large sludge pumps so as to remove the debris from the bottom of the excavation. Then desanders and desilters are used to clean the bentonite mix which is reused while the filtered gravel, sand and silt are damped away (Figure 2.22).



1. ***Trench Cutter***
2. ***Cutter mud pump***
3. ***Desander***
4. ***Slurry tank***
5. ***Centrifugal pump***
6. ***Excavated soil***
7. ***Centrifugal pump***
8. ***Bentonite mixer***
9. ***Bentonite silo***
10. ***Water***

Figure 2.22 Hydraulic cutter and reverse circulation technique of diaphragm wall production.

The advantages offered by the cutter over the other available systems include a consistently high output, an extremely high degree of verticality control as it is equipped with electronic inclinometers and its position adjusted with the help of hydraulically operated steering plates, watertight joints and the ability to cut through even the hardest boulders and also key into bedrock if required. However, they are very expensive and are generally not always viable as their stocking, back up and maintenance costs are very high. In addition, their mobilization is difficult and they would require very heavy carrier rigs.

Currently, the rotary cutters come in two sizes and the smaller size is known as the city cutter (see Figure 2.23). There is still debate on whether these cutters are viable in most of the projects as their break down and replacement issues are hard to solve. Some experienced specialty contractors still prefer to use chiselling and heavy mechanical grabs in lieu of the cutters.

Figure 2.23 Compact hydraulic cutter - city cutter (Zetas, 2009).

*2.3.2.2.2 Stopend Placement (only in Grab Excavation).* First, the predesigned panel lengths are excavated in alternating sequences. Then, specially designed or simple casing stopend pipes or profiles are placed at the ends of the completed panel (Figure 2.24). Stopends are used to control the concrete placement so that adjacent secondary panels are not excavating monolithic concrete. Stopends may be permanent or removed after concrete placement. Permanent stopends are typically wide flange shapes. Removable stopends can be pipe or special keyway end stops.

Figure 2.24 Stopends (Zmakina, 2009).

*2.3.2.2.3 Panel Desanding*. The panel must be de-sanded to remove excess sand in the slurry and bottom of panel (Figure 2.25). The removal of sand from the slurry decreases the density of the slurry so that tremie concrete does not mix with the slurry or trap pockets of sand.

If geotechnical data indicate that sandy soils (or bedrock) will be encountered in the trench excavation, then strong emphasis must be placed in the specifications regarding desanding requirements, prior to concrete placement, e.g., maximum sand content of slurry and/or maximum density of the slurry. In some cases it may be necessary to replace contaminated slurry with fresh slurry prior to concreting.

Figure 2.25 Panel desanding.

*2.3.2.2.4 Placing Reinforcement.* Carefully fabricated three-dimensional reinforcing cage are then inserted into the panel excavation (Figure 2.26). The length of the reinforcement cages can vary project to project. Generally, more than two cage sections are joined to form one continuous cage for individual panels. Cage sections are connected prior to and during lowering into the trench.

The reinforcing cage may also support future structural or utility connections using "box outs" that are pre-set in the wall (Figure 2.27). In order to place the box-out in position and to achieve an exact match of the cage sections, markings are made on the main bars of each cage.

Figure 2.26 Reinforcement placing.

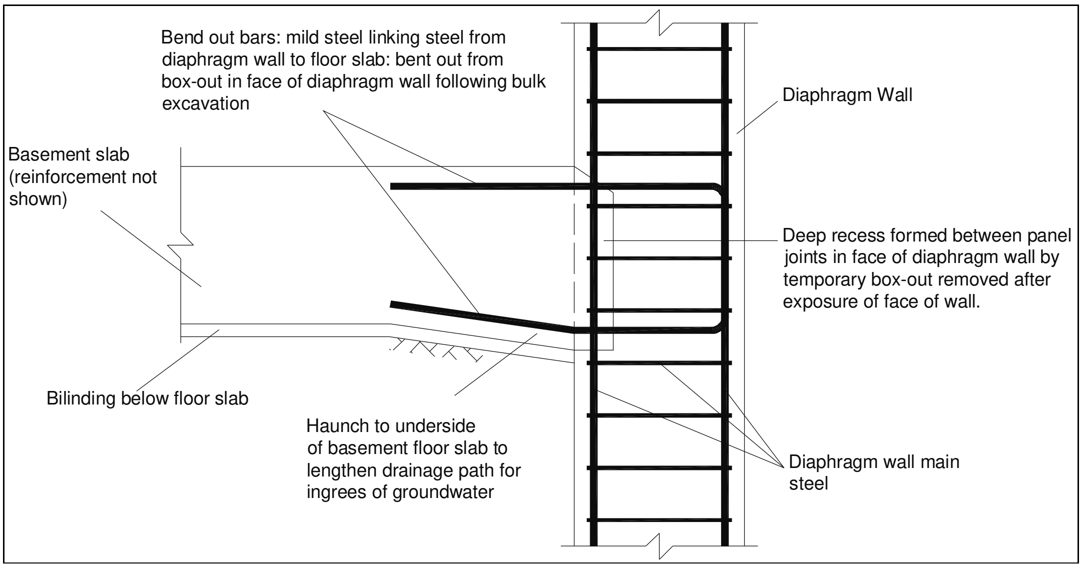


Figure 2.27 Horizontal joint between diaphragm wall and slab using box out method.

*2.3.2.2.5 Placing of Concrete.* The standard tremie concreting processes are applicable in the placement of concrete in bentonite slurry diaphragm wall construction. The tremie pipe(s) should remain embedded in the fresh concrete a minimum of 1.5 m at all times. When concreting wide panels more than one tremie pipe can be used (Figure 2.28). The tremie tubes shall be clean and watertight throughout. While the concrete is being placed, the bentonite slurry is pumped back to desanding plant for treatment and reuse.

It is important that in the tremie process, sufficient concrete be on hand to adequately backfill a panel section without interruption so as not to cause a formation of horizontal cold joints and, thus, slurry seams during the concreting process.

The design of the concrete mix must ensure that the maximum size of aggregate is appropriate to the tremie process as regards the spacing of steel reinforcement. The concrete mix is special to provide specified strength with high slump and contains fairly high cement content.

Figure 2.28. Concreting in progress.

It is normally considered good practice to specify concrete fluidity (as measured by the standard concrete slump test) in the range of 175 mm to 225 mm. This will ensure adequate flow of the concrete in the tremie system and the displacement of the slurry in the wall panels. Stiffer mixes leads to voids and open honeycombs in the panels, particularly when the reinforcing cage is dense.

*2.3.2.2.6 Panel Connections.* There are various positive panel construction connection techniques that can be specified to ensure that there is a structural tie between adjacent panel sections and that these sections have adequate strength and integrity to support the design loads imposed upon them. Some of these connections are proprietary, others have become standard practice. In many cases, this is a good place to require an end-result specification, i.e., require that the strength, integrity, and continuity of the joints are maintained and allow the contractor to use his ingenuity to develop an appropriate scheme.

The circular stop end joint is the most commonly used panel connection. By removing the stop end tubes from the primary panels, a semi-circular guide is provided for the adjacent secondary panels. After the concrete's preliminary setting, which may be 4 to 8 hours, the stop end pipes are pulled out by specially designed hydraulic extractors. Cleaning of the joint with a scratcher or other device is important to remove any contaminated material from the panel connection.

To achieve water proofing, several techniques can be used to make the joints impermeable. One common technique is using resilient but sturdy plastic water stops which are placed in between the panels by utilizing specially stop-end elements. Another technique is placing key tube pipes at the joints and forming single or double key joints. These keys can be tube-a machete grouted directly, or, be reamed with a down-the-hole hammer and the holes can be filled with a mastic grout. Sometimes, the joints are formed by grooved stop-end elements specially designed so that the teeth of the grabs would fit. Then, the limited amount of water seeping through is controlled by water-proofing and architectural detailing inside the diaphragm wall.

The trench cutters allow all types of joints to be constructed. Besides the conventional method of using stop-end tubes cutters can form overlap cutter joints. A secondary panel is formed between two previously completed primary panels, usually as single or three cuts (see Figure 2.29). As the cutter descends it encroaches on the adjacent primary panels and thereby cuts back a fillet from each end of the previously concreted primary panels. Joints produced in this manner are watertight, because they consist of a serrated surface resulting from the formation of grooves cut into the concrete of the primary panels by cutter wheels.

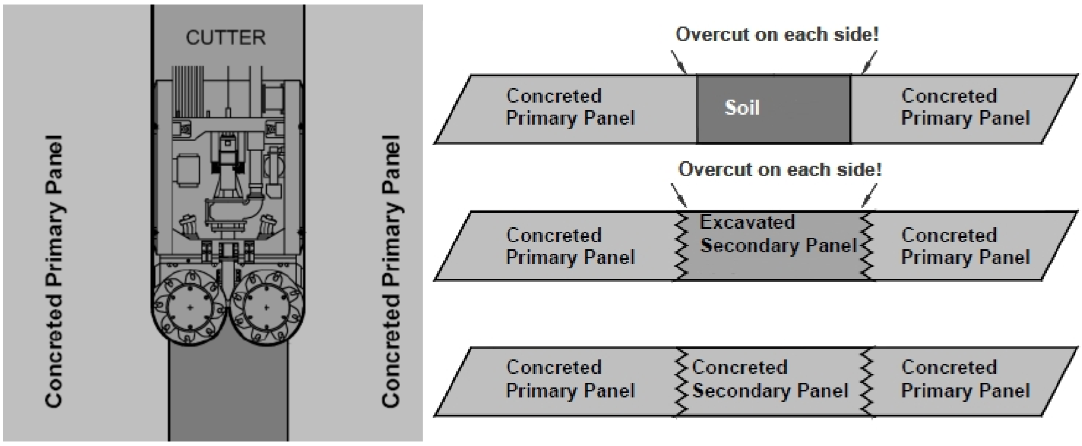


Figure 2.29. Overlap Cutter Joint.

#### *2.3.2.3 Segmented RC Walls*

In comparison with the other two methods, segmented RC wall construction is the most optimized method in many ways. When compared to manual caisson method, deeper excavations can be done in the rock by this method. Also, this method is highly preferred when there is lack of space for construction equipment (e.i. diaphragm wall machine).

On the other hand, during the construction stages including excavation, placing reinforcement and concreting, the cut face of the rock formations are directly exposed water and atmosphere because of the construction methodology. In such cases, some rock formations as well as graywacke, can be weathered and lost their integrity. As a result, soil losses can cause significant settlements even if the retaining walls do not move towards the excavation.

Segmented RC walls are constructed top to down step by step. Pre-stressed anchors are generally used as support system of the wall. But sometimes, braces are used instead of the anchors especially where there are adjacent underground utilities or when adjacent owners do not grant permission to drill them under their properties.

*2.3.2.3.1 Method of Excavation.* In this method, mass excavation of the construction area is performed concurrently with support wall excavation. During the wall construction, excavation is conducted in segments at each vertical stage. Each segment of stages shall be identified including maximum allowable height of excavation lifts and limitations for unsupported horizontal length including maximum time duration of cut face exposure between excavation and support installation during RC wall construction. 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 mechanical properties of the bedrock formation shall be understood prior to excavation. Bedrock can often comprise of an upper weathered/fractured zone and a lower more intact deposit. Depending on the strength of the rock, the degree of weathering/fracturing, the depth of the excavation and the scale of the project various excavation machines will have different capabilities to excavate rock can be used. Wall excavation is commonly made by backhoe to provide accurate vertical cuts (see Figure 2.30). Sometimes, widely spaced small diameter mini piles are installed before the excavation. Then, excavation borders are defined and the cut ground stays in place until the RC wall is cast Plus, mini piles provide additional resistance to the drag down forces acting on the wall.

*2.3.2.3.2 Placing Reinforcement.* Soon after the excavation, reinforcement is placed to the open-cut rock. The most typical reinforcement consists of welded wire mesh, which is placed at the any position of the wall thickness specified on the design drawings. The lateral and vertical continuity of the wall is provided by reinforcement lapping during next panel construction as well as manual caisson wall construction

Figure 2.30. Excavation of the RC wall segment (with pre-stressed ground anchors) at the later stage of the construction (Zetas, 2007).

The panels can be properly reinforced to act as the shear walls or columns of the structure. The columns can be integrated within the panels. Also bars for any further application can be attached to the wall reinforcement prior to casting. Box-outs and gaps can be easily made by placing styrofoam blocks for various purposes before form works (see Figure 2.31).

*2.3.2.3.3Placing of Concrete.* Where walling is being placed within a cast in-situ RC wall excavation single sided work is involved. Normally the pressures from the concrete are collected and contained by ties passing between the two form faces to a wall. In the case of single sided work, these forces must be transmitted into a previously cast concrete element or backfill, either using a propping system or utilizing anchors cast into upper lifts (see Figure 2.32).

Figure 2.31. Placing of wire mesh reinforcement of the panel and placing of the styrofoam blocks for the anchor head installation (Zetas, 2007).

Formworks can be designed in various ways which are used in cast in-situ RC wall. There are two main materials for creating formwork - traditional timber formwork and engineered formwork (usually a metal frame). Using these types of formwork, there are a variety of different ways to create walls from formwork.

Figure 2.32 Placing of single sided metal frame formwork panel with back fill method (Zetas, 2007).

Concrete is generally placed by the concrete pump. Formwork should be continuously watched during and immediately after concreting.

Wall formwork can be removed between 15 and 24 hours after casting, depending upon the ambient temperature, the concrete temperature and the degree, if any, of acceleration applied to the curing process. This operation commences with careful removal of the majority of ties, bolts or wedges. When metal frame panel is used, a crane (or generally backhoe) is attached via a suitable spreader bar and lifting arrangements so that the full weight of the panel is taken by the crane. The remaining ties are removed and the form is peeled away from the concrete face.

## 2.4 A General Overview of Pre-stressed Anchored Retaining Wall Design

### *2.4.1 General*

Deep excavation or the construction of basements includes the construction of retaining walls, excavation, the installation of anchors (or struts), the constructions of foundations and floor slabs, etc. With the great variety of excavation methods and lateral supporting systems, to come the most appropriate design, we have to consider, in combination, the local geological conditions, the environmental conditions, the allowable construction period, the budget, and the available construction equipments and make an overall plan accordingly.

Retaining walls with pre-stressed anchors have become popular in braced excavations because of the substantial progress in the technology and availability of high-capacity anchor systems, and the absence of interior obstructions that permit uninterrupted earth moving and thus improve the construction conditions of the underground portion of a building.

The risk associated with deep excavation works is high as failures of retaining wall or anchors could be catastrophic and may affect surrounding areas. Therefore, the design of anchored walls for deep basement construction works requires careful consideration of soil/rock-structure interaction. This is usually accomplished using the finite element method. However, in order to produce safe and economical design, proper understanding of the constitutive material models and using the proper model is very important.

This subchapter presents general information about design procedure of a retaining wall with pre-stressed anchors.

### *2.4.2 Establishing Project Requirements*

The first step of the anchored wall design involves establishing overall geometric requirements for the anchored system and identifying project requirements and constraints. This step involves developing the wall profile, locating wall appurtenances such as safety barriers, utilities, and drainage systems, establishing right of- way limitations, and construction sequencing requirements. Project requirements and constraints may significantly affect design, construction, and cost of the wall system and should therefore be identified during the early stages of the project. Since the information in first step is required prior to actual wall selection and design.

### *2.4.3 Pre-Design Investigations*

A number of investigations must be undertaken prior to the design and construction of an excavation and retaining system. Ground and site investigations usually are carried out to confirm the feasibility of a project as a whole, but adequacy in this respect does not necessarily ensure the feasibility of a proposed anchorage installation. This thesis, therefore, deals with the special topics that are relevant to the design, construction, testing, protection, and monitoring of anchor systems. Whereas adequate data may be available to indicate the feasibility and advantages of anchors for a given project, these data nonetheless may be insufficient to permit their economic design and construction, and this demonstrates the importance of a detailed knowledge of the ground.

In general, an adequate pre-design investigations include the drilling of soil borings, performing of soil or/and rock tests (in situ and lab) and developing of geotechnical reports, the performing of pre-construction surveys of adjacent properties and utilities, and the analyzing of the location of those utilities to verify potential interferences.

#### *2.4.3.1 Project-Specific Subsurface Investigation*

Prior to the design and installation of any underground improvement, an analysis of subgrade materials and conditions must be performed. This is usually done by drilling test borings and performing lab and field tests to characterize the subsurface conditions and ascertain various parameters of the soils or/and rocks.

Using Borings, soil and rock stratigraphy at the project site, including the thickness, elevation, and lateral extent of various layers, should be evaluated through implementation of a project-specific subsurface investigation. The following potentially problematic soils and rock should also be identified during the subsurface investigation which may significantly affect the design and construction of the anchored system (FHWA, 1999):

- Cohesionless sands and silts which tend to ravel (i.e., cave-in) when exposed, particularly when water is encountered, and which may be susceptible to liquefaction or vibration induced densification.
- Weak soil or rock layers which are susceptible to sliding instability.
- Highly compressible materials such as high plasticity clays and organic soils which are susceptible to long-term (i.e., creep) deformations.
- Obstructions, boulders, and cemented layers which adversely affect anchor hole drilling, grouting, and wall element construction.

Subsurface borings should be advanced at regular intervals along, behind, and in front of the wall alignment. Borings should be located at the site extremities along the wall alignment so that stratigraphy information can be interpolated from the boring information. Typical boring spacing is 15 to 30 m for soil anchors and 30 to 60 m for rock anchors (FHWA, 1999). The back borings are located such that the borings are advanced within the anchor bond zone so that potentially weak or unsuitable soil or rock layers can be identified. Back and wall boring depths should be controlled by the general subsurface conditions, but should penetrate to a depth below the ground surface of at least twice the wall or slope height. Front borings may be terminated at a depth below the proposed wall base equal to the wall height. Borings should be advanced deeper if there is a potential for soft, weak, collapsible, or liquefiable soils at depth. For very steeply inclined ground anchors borings should also, at a minimum, penetrate through to the depth of the anchor bond zone. Additional borings may be required to characterize the geometry of a landslide slip surface.

The groundwater table and any perched groundwater zones must be evaluated as part of a subsurface investigation program. The presence of ground water affects overall stability of the system, lateral pressures applied to the wall, vertical uplift forces on structures (structural design), and drainage system design, watertight requirements at anchor connections, corrosion protection requirements, and construction procedures. At a minimum, the following items need to be considered for anchored systems that will be constructed within or near the groundwater table (FHWA, 1999):

- Average high and low groundwater levels.
- Corrosion potential of ground anchors based on the aggressivity of the ground water.
- Soil and/or rock slope instability resulting from seepage forces.
- Necessity for excavation dewatering and specialized drilling and grouting procedures.
- Liquefaction potential of cohesionless soils (long term analysis).

#### *2.4.3.2 Geotechnical Reports*

Soil borings are taken to develop an understanding of strength parameters of the soils or/and rocks and ground water conditions and document any incidence of contamination which may be evident. Borings can be undertaken in a number of ways to produce meaningful information.

A geotechnical report which is helpful to both the designer and constructor of a deep excavation support system will include the following materials (Macnab, 2002):

- A listing of all soils encountered, described in accordance with a well recognized soil classification system such as the Uniform Soil Classification System (USCS). All soils encountered should be carefully logged so as to present an accurate cross section of the soils encountered at the boring location.
- A description of any water levels encountered in the boring, both at the time of drilling and some time later when static levels can be determined.
- A discussion of the applicability of the boring results to known geologic mapping of the area.
- A description of any evidence of hazardous or contaminated materials noted in the borings.
- A full disclosure of any in-hole testing which was undertaken together with the test results such as standard penetration tests (SPT), cone penetration tests (CPT), shear vanes, and slugging tests. If samples were retrieved, the location where they can be viewed should be indicated.
- An accurate plan indicating boring locations referenced to property lines, building lines, or easily identified monuments. Elevation of the top of all borings should be referenced to an easily identified datum, preferably geodetic.
- A discussion of the relevant parameters of the soils investigated. This discussion should include parameters such as unit weight in natural conditions, in-situ density, angle of internal friction, cohesion, moisture content, liquid and plastic limits, grain size analysis, permeability, and unconfined compressive strength. In rock, such parameters as Rock Quality Designation (RQD), compressive strength, and the incidence of fissuring should be provided. If available, strike and dip data is also helpful.
- A discussion of the water levels. Is the water indicated perched, or is it representative of the true water table? If dewatering is indicated, what is the anticipated hydraulic conductivity?

- Although it is preferred that the Geotechnical Report give angle of internal friction and cohesion values, if they are not available, a discussion of suggested apparent earth pressures or earth pressure coefficients is necessary.

#### *2.4.3.3 Pre-Construction Surveys*

Pre-construction surveys are carried out prior to construction. Some of the information gleaned from these surveys is necessary for design of the retaining system, while the remainder is an important log of the pre-existing condition of the adjacent property. These surveys are done for the dual purpose of protecting the owner, designer, consultant and contractor on a project against liability for pre-existing conditions, as well as determining the type of retaining system required to ensure that no damage occurs to the adjacent property from the planned excavation procedures.

Required pre-design information regarding adjacent properties includes (Macnab, 2002):

- Description of the size, shape and type of construction of any adjacent facilities such as buildings, roads, retaining walls and utilities. Report on any asbuilt documentation of these adjacent facilities
- Accurate location and depth of adjacent building basements and footings, together with an estimate of the location and depth of buried utilities.
- An estimate of the loads on adjacent building footings together with a disclosure of the competence of the structure.
- A clear definition of any easements, fire lanes, or building exits adjacent to the proposed excavation.

Pre-Construction information which should be collected prior to starting excavation and retaining system construction includes:

- Crack surveys including pictures, or videos to record pre-existing conditions.
- Detailed water level and water quality sampling of wells in the vicinity.
- Traffic surveys if excavation is likely to affect neighbouring businesses.

#### *2.4.3.4 Utility Locates*

Although normally considered to be part of the pre-construction survey, this issue has been given its own segment due to the importance of due diligence in this matter. While overhead utilities can be seen and are often moved prior to construction, buried utilities, which are far greater in number, must be located prior to construction. Not only does the striking and subsequent disruption of service represent a significant potential liability to the project participants, it can be also be a considerable safety hazard to the personnel directly involved.

While utility locates are attempted by the design engineer prior to the design of the retaining system, this locate information should never be relied upon. The construction team (owner, general contractor, and specialty contractor) should undertake a utility location survey prior to construction. This survey should include:

- A review of all available as-built drawings.
- Utility marking by relevant institutions.
- Utility locates by each individual utility.
- Opening of manholes and vaults to measure the depth and location of inverts of adjacent utilities.
- Pot holing as necessary to locate utilities.

Each of the steps becomes increasing more difficult and invasive for the project team. However, the team should continue to fulfil each of these steps until such time as the utilities are located with a high degree of certainty.

Aside from the disruption to the utility caused by their breakage, utility damage can have a very detrimental effect to the project. Drilling into gas or electric lines can risk fatal injuries for any nearby personnel. Drilling and grouting operations that damage and subsequently fill sewers or waterlines can be very expensive to fix. Damage to any adjacent utility can cause severe disruption to the project schedule while the utility is repaired.

Successful excavation can only be performed when a complete understanding of the soils and adjacent facilities exists. Only by performing the above-mentioned tests in a diligent and thorough manner can the project team assure themselves that they have achieved this understanding.

### *2.4.4 Evaluation of Design Criteria*

Whether an excavation is successful is significant to the lives and properties of many people. Thus an appropriate design criterion must be selected before design.

The analyses necessary for an excavation design include stability analysis, and stress and deformation analyses. The aim of the stability analysis is to avoid collapse of an excavation due to insufficiency of the strength of the soils. Stress analysis is necessary for the design of structural components and deformation analysis aims at diagnosing the wall deflections and soil/rock movements caused by excavation to protect adjacent properties.

An anchored wall design criterion should include, at least, establishing of design factors of safety for stability analysis, the simplified and/or advanced deformation and stress analyses, the design of structural components, property protection, a dewatering scheme, selection of the level of corrosion protection, etc.

#### *2.4.4.1 Stability Analysis, Factors of Safety and Deformation Relationship*

Stability and deformation are related. If the factor of safety against collapse is large, strain in the soil around the excavation will be small, and the ground movements will be small. On the other hand, if the factor of safety against collapse is close to one, strains in the soil/rock around the excavation will be large, and ground movements can be large.

Thus, prediction of deep excavation performance involves analysis of both stability and deformation. Experience has shown that stability can e evaluated with sufficient accuracy using simple limit equilibrium calculations. Deformations, however, are significantly more difficult to predict, and finite element analyses are often used for this purpose when ground movements are particularly important.

Generally, the anchored wall should be stable for all potential failure planes with a factor of safety of 1.3 or more. The factor of safety may be increased to account for site-specific conditions at the discretion of the designer.

#### *2.4.4.2 Stress Analysis and Design of the Structural Components*

Under normal excavation conditions, the stress on and deformation of walls, or ground surface relate to the unbalanced forces acting on the walls, the stiffness of the wall-anchor system, the stability of the excavation and so on. The unbalanced forces are in turn related to the soil/rock conditions, ground water levels, water pressures, the excavation width, the excavation depth, the excavation area, etc. To analyze excavation-induced deformation using the numerical method, the analysis method has to simulate all these factors rationally. Theoretically, the FE method is capable of simulating these factors and therefore the results derived from the method would be more accurate than those derived from simplified methods or the beam on elastic foundation method.

Structural design parameters are derived from the dimensional and material properties of structural components used in the excavation support wall system. Example of wall components are cast in-situ segmented RC walls, diaphragm walls, etc. Examples of reaction elements are anchors, struts, etc. The most important stress analysis data of structural wall design is the moment diagram derived by imposing the ground and surcharge loads and reactions mechanism of the excavation support walls.

An initial design analysis is conducted with the structural parameters of assumed structural geometries and materials plus the acting earth pressures including surcharge. Then, certain redesign considerations and structural parameter adjustments based on the moment bearing capacities of a feasible and viable wall are configured. Then, the practically possible reaction element (anchor) axial loads are controlled and if necessary the wall dimensions are re-adjusted. Thus,

- Moment capacity of the wall in vertical direction,
- Moment capacity of the wall in horizontal direction, and
- Load capacity of the anchors

are carefully configured to construct a sound and safe excavation support wall.

#### *2.4.4.3 Estimation of the Excavation-induced Allowable Settlement of Adjacent Properties*

The protection of adjacent building during excavation can be divided into the following three procedures: (a) before-excavation plan (b) monitoring and prevention during the construction, and (c) compensation after damages have been done. The most important job in a before-excavation plan is a comprehensive geological investigation, on the basis of which is carried out the analysis and design.

Settlements of buildings will occur as a result of excavation. However, buildings may have settled due to their own weight before excavation. How large the allowable settlement of a building should be, with the start of the excavation, becomes a complicated problem. The determination of the settlement value and influence range can be solved by way of finite element analysis.

Theoretically, the allowable settlement of a building is constant. With the existing settlement caused by the weight of the building itself, the allowable settlement after excavations started will not be large. The amounts should be smaller than those allowable settlement values.

Increasing the thickness of the retaining wall, enhancing the stiffness of the anchors by means of reducing the horizontal and vertical span of anchors and installing the struts as close to the excavation surface are all helpful to reduce the deformation of the retaining wall and the ground settlement induced by excavation. However, the measures are limited in their effects. If the wall deformation or settlement is still too large, auxiliary methods such as ground improvement, the construction of counterforts or cross walls will be needed to further reduce wall deformation or settlement.

#### *2.4.4.4 Dewatering of Excavation and Ground Water Effect on the Selection of the Analysis Method*

According to investigations, most problems encountered in deep excavation have direct or indirect relations with groundwater. Therefore, whether groundwater has been properly dealt with is a crucial point for the success of an excavation. Groundwater-induced problems in an excavation may arise from insufficient investigation of groundwater or geological conditions that lead to inability to fully control the groundwater. It may also arise from misunderstanding of the influence of groundwater, so that a wrong excavation method is adopted.

Drained analysis is to assume that, in analysis, the excess porewater pressure has been all dissipated (i.e. $u_e = 0$) and the volume of soils thereby changes. Rained analysis is mainly applied to granular soils such as sandy soils or gravel soils. It is also applied to the analysis of long-term behaviours of clayey soils. Hereby, the analysis has to adopt the effective stress analysis and the required parameters in analysis are effective parameters ($c'$ and $\phi'$).

The undrained analysis, assumes that the excess porewater is not dissipated (i.e. $u_e \neq 0$) and the total stress analysis is adopted accordingly. Suppose the soils are in the saturated state, no volume change can be observed under the undrained condition. The parameter to be used in analysis would be $s_u$ and $\phi = 0$.

In some cases, the clay is in between the two conditions, called the partially drained behaviour. The analysis of this behaviour of clay can be carried out through undrained analysis, which takes advantages of the known excess porewater at each stage.

#### *2.4.4.5 Selection of the Corrosion Protection Level*

The design of corrosion protection systems is performed to protect the steel components of the ground anchor. The corrosion protection system consists of components that combine to provide an unbroken barrier for each part of the tendon and the transitions between them.

The minimum level of corrosion protection for ground anchors should be selected considering the service life of the anchored system, the aggressivity of the ground environment, the consequences of failure of the anchored system, and the cost of providing a higher level of corrosion protection.

Service life is used to distinguish between a temporary support of excavation and a permanent anchor. In the content of the thesis the corrosion protection issue will not be further detailed. However, it is acknowledged that, if the service life of a temporary support of excavation anchor is likely to be extended due to construction delays, an evaluation should be made to determine whether or not to provide additional corrosion protection for the tendon, particularly in aggressive ground conditions.

## 2.5 A few Remarks on Anchored Retaining Wall Design by FEM

For analysis of an excavation and retaining system it is important to create a geometry model first. A geometry model is a 2D representation of a real three-dimensional problem and consists of plane strain elements, bar elements, beam elements and interface elements. A geometry model should include a representative division of the subsoil into distinct soil layers, structural objects, construction stages and loadings. The model must be sufficiently large so that the boundaries do not influence the results of the problem to be studied.

In FEM analysis, proper understanding of the constitutive soil/rock models is important in order to produce safe design. Incorrect use of material models leads to incorrect predictions of deflection, bending moment and forces in the design. Various soil models have been incorporated in commercial software packages ranging from elastic-perfectly plastic Mohr-Coulomb model to the Cam clay model. For example, in the FEM computer program Plaxis, there are various soil models for different application, i.e. Mohr-Coulomb, Soft Soil Model, Hardening Soil Model and Jointed Rock Model.

The use of finite element analysis is common and various commercial software packages are available to the designer. Due to increasing user-friendliness of the commercial software packages, the use of finite element has become increasingly common and FEM is being routinely used by engineers of differing levels of experience and expertise. On the other hand, the users should properly understand of finite element analysis and also the coding and constitutive soil/rock models used in the software.

Some of the recommendations by Tan & Chow (2008) for the factors which may affect the results of FEM analysis are listed below:

a. Locations of the boundaries of the problem. The problem boundary should be located far enough away such that there is no stress rotation near the boundary. For undrained analysis, the extent of the model required will be greater.

b. Details of mesh. Higher order elements (such as Q8 elements) are to be preferred to simple elements (such as T3 elements), especially if high strain gradients are anticipated, or for failure analysis. More, smaller, elements need to be placed where gradients are expected to be highest, and at regions of stress concentration (Wood, 2004).

c. Long, thin elements will lead to calculation instability. As such, the layout of the model and mesh should avoid long, thin elements.

d. Stages of construction. As soils/rocks are non-linear, history dependent materials, proper modelling of the soil/rock at various stages from the past to its construction stages needs to be carried out.

e. Modelling of interfaces. Improper modelling or use of unconservative interface reduction factors may lead to dangerously unsafe design.

f. Use of suitable constitutive soil/rock models to model different geotechnical problems.

g. Sensitivity of various soil/rock parameters. For different constitutive material models adopted in different FEM software packages, different soil/rock parameters may have different effects on the analysis results.

# CHAPTER THREE
# A GENERAL OVERVIEW OF ISTANBUL GREYWACKES

## 3.1 Introduction

One of the most specific common traits of the deep excavation projects in the city of Istanbul is that they are generally constructed in greywackes, a rock type frequently encountered in Istanbul construction sites. For this reason, the author believes that, giving general information on Istanbul greywackes prior to the case study would be beneficial to understand the application.

## 3.2 Definition of the Greywacke

The term of “Greywacke” is written in different ways in Turkish and European Languages. In Turkish geological literature it is seen as: Grovvak, Gravak, Grovak, Grovake, Grivake etc. In Europe, it is usually known as: Greywacke, Graywacke, Grauvacke, Grauewacke (Erguvanli, 1985).

According to Erksoy (1985) greywacke defined as: a dark grey, firmly indurated, coarse grained sandstone that consist of poorly sorted, angular to sub-angular grains of quartz and feldspar, with a variety of dark rock and mineral fragments embedded in a compact clayey matrix having the general composition of slate and containing an abundance of very fine grained illite, sericite and chloritic minerals. Few examples of greywackes that are met in the deep excavations in the city of Istanbul are given in Figure 3.1.

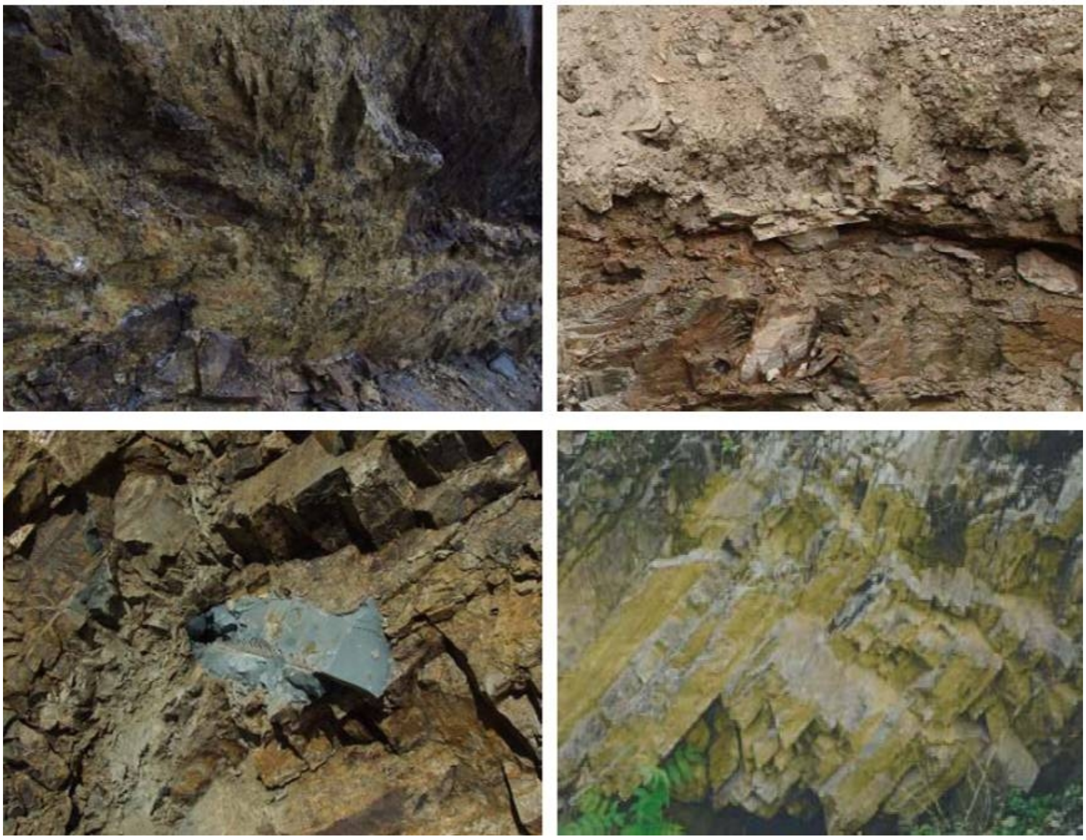


Figure 3.1 Features of typical greywacke formation in Istanbul (Keskin, 2008)

## 3.3 Geological Setting of the Greywackes

Paleozoic, Mesozoic and Cenozoic Period formations are underlying Istanbul. Paleozoic rocks are the most common rocks in Istanbul. From the Ordovician to the Carboniferous Periods, comprising several thousand metres of rock, a concordant rock sequence is observed. This stratigraphic sequence generally consists of clastic rocks and limestones in different facies (Onalan, 1981; Eroskay, 1985).

One of the most common formations encountered in Istanbul is Trakya Formation which substantially consists of greywacke. This study covers only the Carboniferous Trakya Formation.

The Trakya Formation consists of intercalated sequences of sandstone (greywackes), shale, siltstone and mudstone. 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). They are generally of marine origin and are believed to have been deposited by submarine turbidity currents (Pettijohn, 1972).

The Trakya formation is commonly exposed in the European side of Istanbul, especially in Ikitelli, Cebecikoy, Sisli, Besiktas, Levent and Gaziosmanpasa (Tugrul & Undul, 2006). General locations of greywacke formation in Istanbul are presented below (Figure 3.2).

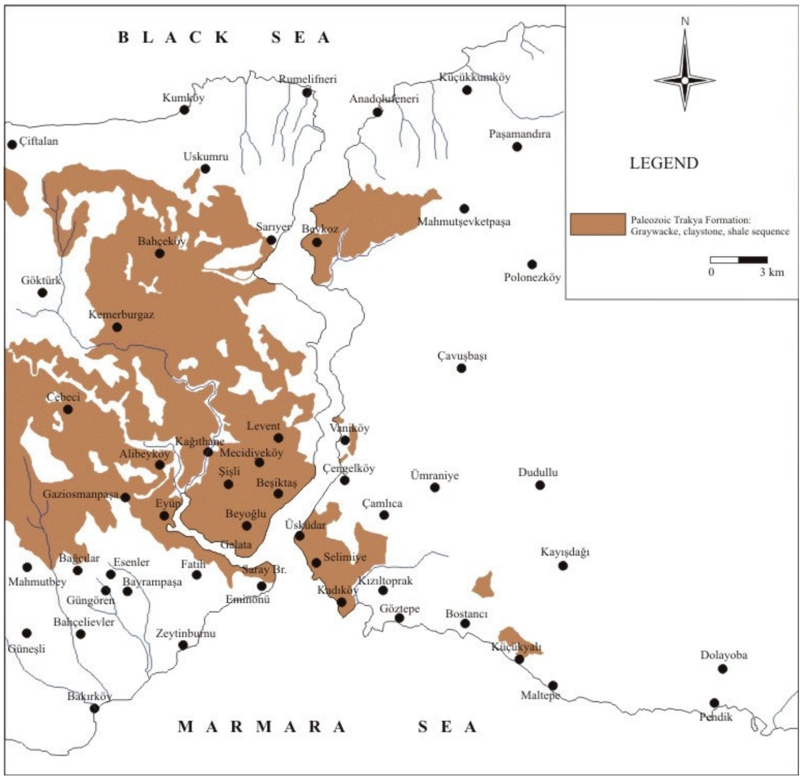


Figure 3.2 The location of greywackes in Istanbul (Ketin, 1991; Tugrul & Undul, 2006)

The greywacke is typically dark grey and green or brown sandstone, owing to secondary hematite cement. Thickness of layers changes between 10 cm and 2.5 metres (Tugrul & 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. Amount of quartz grains ranges 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 & 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.

## 3.4 Engineering Properties of the Istanbul Greywackes

Understanding of the term “Greywacke” is very vague for engineers. There are different types of greywackes with different lithological and physico-mechanical properties in Istanbul and surrounding areas. Therefore, all engineers who are working on the surface and/or subsurface excavations in Istanbul should have knowledge about the physico-mechanical properties of the greywackes (Erguvanli, 1985).

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 degree 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 modelling of formation, weathered zones, extend of fracturing and compressibility modulus of formation are usually obtained by means of integrated seismic survey and Menard pressuremeter testing performed within the boreholes at various locations and depths (Durgunoglu & Yilmaz, 2007).

The weathering of the greywackes decreases gradually and progressively with increasing depth (Tugrul & Undul, 2006).

The physical and mechanical properties of the fresh and weathered greywacke sandstones could be determined by a variety of laboratory tests. A number of test were made by many investigators on the cores including, dry unit weight, water absorption, porosity and uniaxial compression test, in accordance with the ISRM (1981) suggested methods. The results of the physical and mechanical tests are presented in Table 3.1 and Table 3.2 respectively. As seen in these tables, both porosity and water absorption increase and dry unit weight decreases with increasing weathering grade and the mechanical properties of the greywackes, i.e. strength and modulus, decrease considerably with weathering. The mechanical properties of the greywackes, i.e. strength and modulus, decrease considerably with weathering (Tugrul and Undul, 2006).

Pore size distribution could be determined using the mercury intrusion technique as suggested by ISRM (1981). 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 shown in Figure 3.3. 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).

Table 3.1 Physical properties of greywackes in different locations in Istanbul (Tugrul & Undul, 2006).

| Weathering | Location | Dry Unit Weight, $\gamma_d$ (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 | 2.8 | 8 | Erguvanli (1985) |
| | Halic Tunnel | 25.9 | 1.9 | - | Erguvanli (1985) |
| **Average** | | **26** | **3** | **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** | |

Table 3.2 Mechanical properties of greywackes in different locations in Istanbul (Tugrul & Undul, 2006).

| Weathering | Location | Uniaxial Comp. S., UCS (MPa) | Compression Modulus, E (GPa) | Poisson's 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 - Baltalimani | 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 - Baltalimani | 118 – 121 | 22 – 31 | - | Erguvanli et al. (1987) |
| **Average** | | **76.1** | **37.2** | **0.21** | |

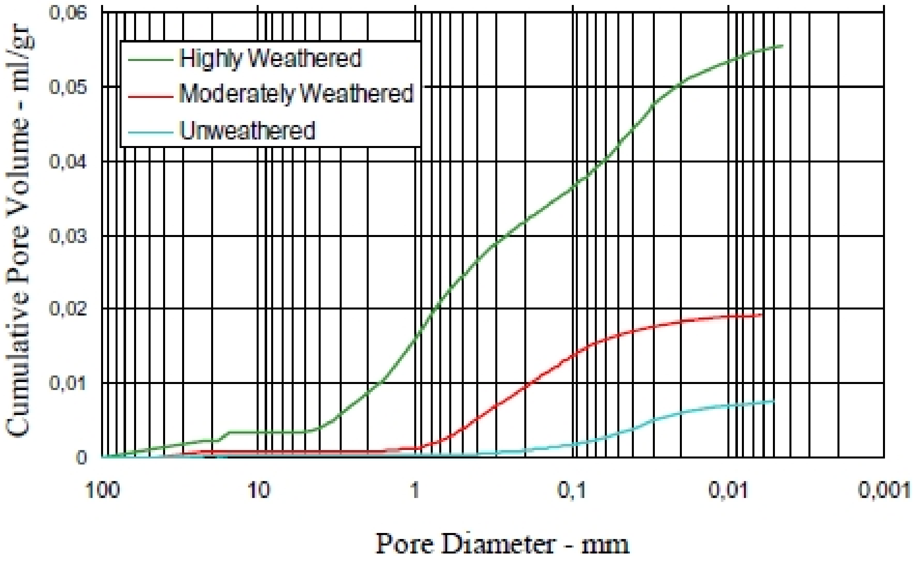


Figure 3.3 Porosity change with weathering of greywackes (Tugrul and Undul, 2006)

The strain levels that develop as a result of deep excavations are much higher than the levels resulted in geophysical measurements. The modulus value is also 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 (Keskin, 2008).

The modulus degradation values have been estimated for the soils in the past by various authors. However, almost no data exist for the rock conditions. The shear modulus ratios for the greywacke formation have been extensively studied by Durgunoglu and Yilmaz (2007) using shallow seismic survey data. The variation reported for three construction sites in Istanbul are presented in Figure 3.4. 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 (30+m), pertinent firm rock layer conditions the ratio is about 50 to 200 (Keskin, 2008). Where, $G_0$ is the dynamic shear modulus value for very low strains and $G_m$ is the measured shear modulus.

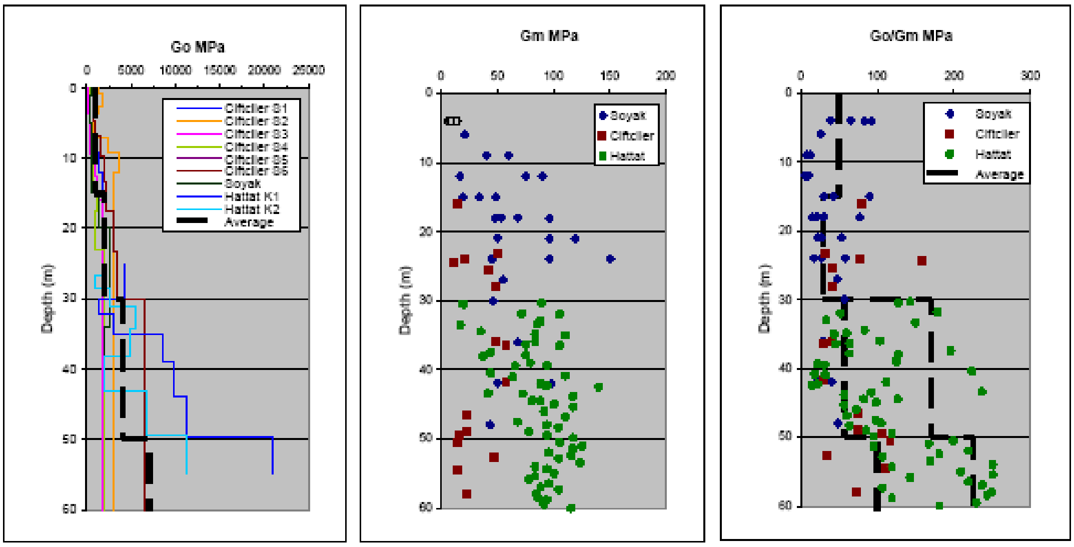


Figure 3.4 The variation of shear modulus with depth in greywackes (Durgunoglu ve Yilmaz, 2007).

## 3.5 Instability Problems Related to Greywackes Encountered During the Excavations

In practice clay, silt, sand and their compacted, cemented types namely claystone, siltstone and sandstone are well-known. However, the greywackes and their more than 20 subtypes are not distinguished precisely and easily in most respects of geotechnical engineering and engineering geology (Erguvanli, 1985).

The bearing capacity of the unweathered greywackes is generally high. However, deep excavations (i.e. deeper than 3 metres) should be supported, otherwise it can result in stability problems (Saglamer, 1985). For example, during the construction stages of the excavation the cut face of the rock formations could be directly exposed to water and atmosphere in accordance with the construction methodology. In such cases, greywacke may get weathered and loose its integrity during stand-up time and material losses can cause sudden movements. As a consequence, densely fractured and weathered greywackes should be investigated in detail.

# CHAPTER FOUR

# A GENERAL OVERVIEW OF THE MODELLING AND ANALYSIS OF EXCAVATIONS IN THE JOINTED ROCK MASS WITH FINITE ELEMENT METHOD

## 4.1 Introduction

Contemporary numerical methods should be employed in deep cuts made in greywackes considering the fact that material behaviour is highly complex and does not fit into traditional analysis methods. With the today's computer technology, numerical methods provide extremely powerful tools for analysis and design of engineering systems with complex factors that was not possible or very difficult with the use of the conventional methods that are often based on closed form analytical solutions. This has greatly enhanced the development of modern rock mechanics from the traditional empirical art of rock deformability and strength estimation and support design according to the rationalism of modern mechanics (Jing & Hudson, 2002).

In excavation and retaining system analysis with Finite Element Method (FEM), proper understanding of the constitutive material models (for soils and rocks) is important in order to produce safe and economical design. Incorrect use of material models leads to incorrect predictions of deflection, bending moment and forces in the design. As a result, project may be costly due to over design or could result in a catastrophic failure due to under design.

According to Hoek (2006), one of the most important contributors to the rock mechanics, the most crucial point of any practical rock mechanics analysis is the geological model and the geological data including the definition of rock types, structural discontinuities and material properties. If the geological model is inadequate or inaccurate the most sophisticated analysis can become a meaningless exercise.

Rock masses encountered during excavations in the City of the Istanbul such as greywackes are generally jointed. For this reason, the author believes that, giving general information on jointed rocks and effects of the discontinuities on the deformability end strength characteristics would be beneficial to understand the application. Additionally, a general overview on the modelling and design of excavations in the jointed rock mass with finite element method are presented in this chapter.

## 4.2 Rock Mass Structure

A clear distinction must be made between the rock element or rock material on the one hand and the rock mass on the other. Rock material is the term used to describe the intact rock between discontinuities. The rock mass is the total in situ medium containing bedding planes, faults, joints, folds and other structural features. Rock masses are discontinuous and often have heterogeneous and anisotropic engineering properties (Brady & Brown, 2004).

The collective term for the whole range of mechanical defects such as joints, bedding planes, faults, fissures, fractures and joints are discontinuity (Priest, 1993). The term discontinuity does not consider the mode of origin of the feature and thereby avoids any inferences concerning their geological origin. The mechanical behaviour of the rock masses depends on the material properties of the intact rock itself, the joint geometry (roughness), the joint genesis (tension or shear joints) and the joint fill material (Natau, 1990). The Rock Mass definition is illustrated in Figure 4.1.

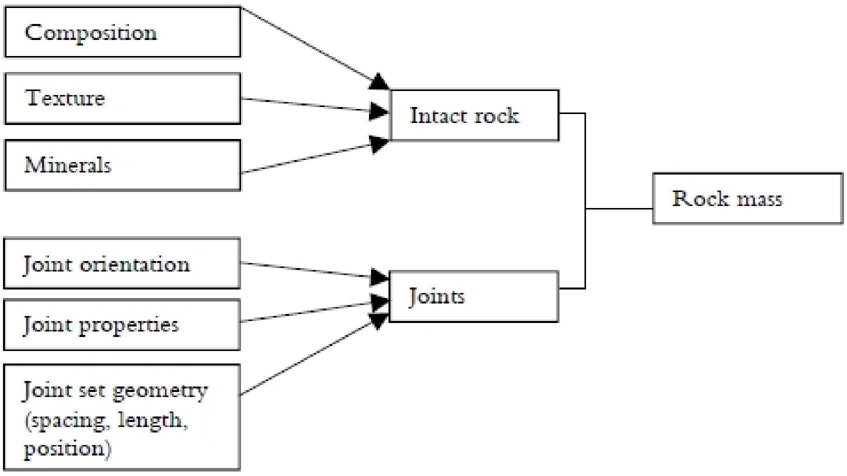


Figure 4.1 Illustration of the rock mass (Edelbro, 2003).

## 4.3 Rock Mass Classification

The processes and interrelations involved in determining the behaviour of the rock surrounding an excavation is sometimes so complex that it is not amenable to engineering analysis using existing techniques. In these cases, design decisions may have to take account of previous experiences. In an attempt to quantify these experiences so that it may be extrapolated from one site to another, a number of classification schemes for rock masses have been developed. These classification schemes seek to assign numerical values to those properties or features of the rock mass considered likely to influence its behaviour, and to combine these individual values into one overall classification rating for the rock mass. Rating values for the rock masses associated with a number of mining or civil engineering projects are then determined and correlated with observed rock mass behaviour (Brady & Brown, 2004).

Hoek and Brown (1980), Goodman (1993) and Brown (2003), among others, have reviewed the considerable number of rock mass classification schemes that have been developed for a variety of purposes. The schemes are currently widely used in civil engineering and in mining practice are The Norwegian Geotechnical Institute (NGI) tunnelling quality index (Q) developed by Barton et al. (1974), the South African

Council for Scientific and Industrial Research (CSIR) goemechanics or Rock Mass Rating (RMR) scheme developed by Bieniawski (1973, 1976) and GSI system introduced by Hoek (1994) and developed further by Marinos and Hoek (2000). These rock mass classification systems can be very useful practical engineering tools, not only because they provide a starting point for the design but also because they force users to examine the properties of the rock mass in a very systematic manner. The most recent GSI system will be outlined here.

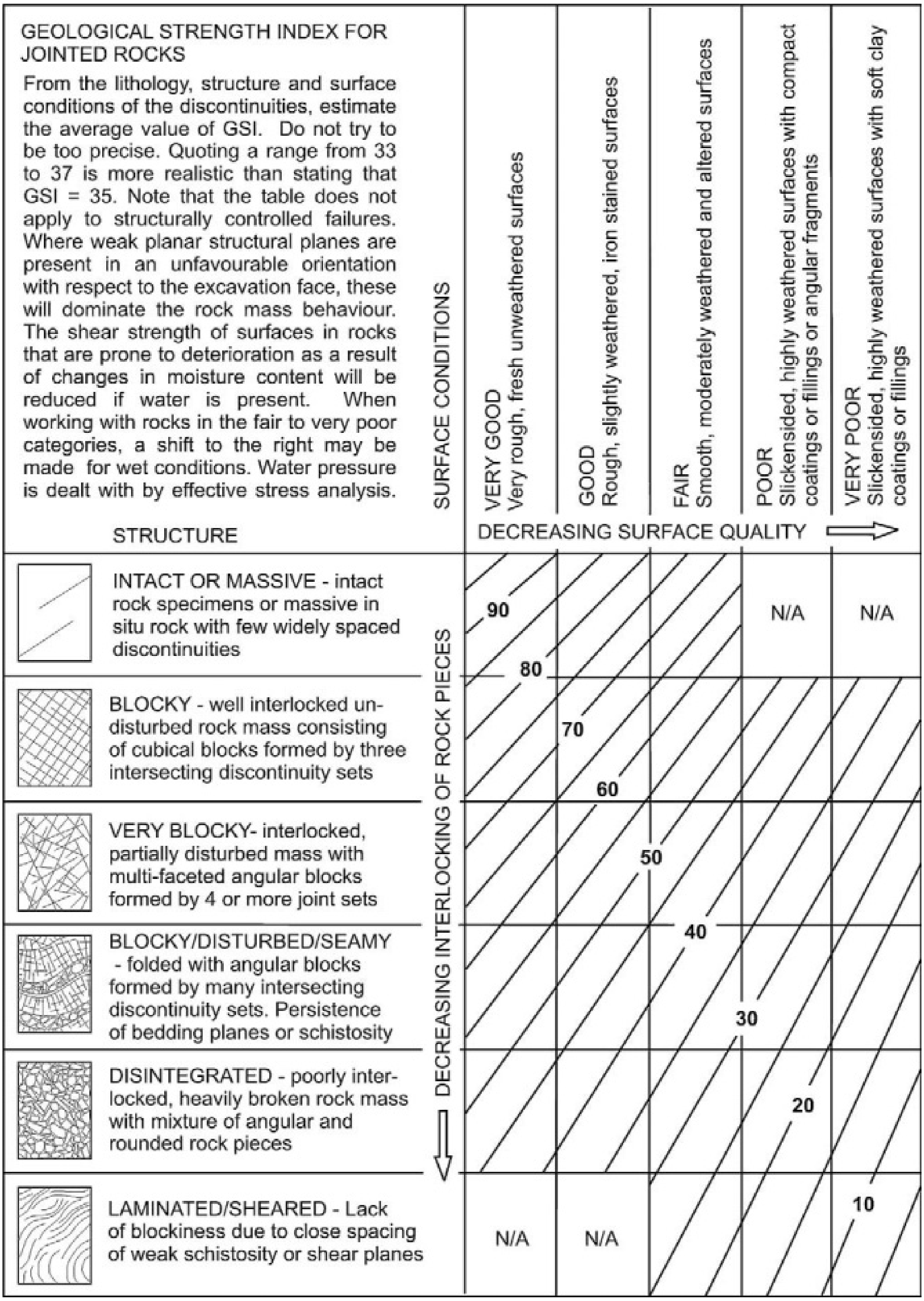
### *4.3.1 Geological Strength Index (GSI)*

The Geological Strength Index (GSI) was developed specifically as a method of accounting for those properties of a discontinuous or jointed rock mass which influence its strength and deformability. The strength of a jointed rock mass depends on the properties of the intact pieces of rock and upon the freedom of those pieces to slide and rotate under a range of imposed stress conditions. This freedom is controlled by the shapes of the intact rock pieces as well as by the condition of the surfaces separating them. The GSI seeks to account for these two features of the rock mass, its structure as represented by its blockiness and degree or interlocking, and the condition of the discontinuity surfaces. Using Figure 4.2 and with some experience, the GSI may be estimated from visual exposures of the rock mass or borehole core.

## 4.4 Mechanical Properties of Discontinuous Rock Mass

### *4.4.1 Strength*

Because of the difficulty of determining the overall strength of a rock mass by measurement, empirical approaches are generally used. The most completely developed of the empirical approaches is that introduced by Hoek and Brown (1980). Because of a lack of suitable alternatives, the Hoek- Brown empirical rock mass strength criterion was soon adopted by rock mechanics practitioners, and sometimes used in cases for



GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS

From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis.

Figure 4.2 Geological Strength Index (GSI) for jointed rock masses.

which it was not originally intended and which lay outside the limits of the data and methods used in its derivation. Because of this, and as experience was acquired in its practical application, a series of changes were made and new elements introduced into the criterion. Hoek and Brown (1997) consolidated the changes made to that time and gave a number of worked examples to illustrate the criterion's application in practice. A further update was given by Hoek et al. (2002). The summary of the criterion given here is based on these accounts and those of Marinos and Hoek (2000) and Brown (2003).

In effective stress terms, the generalized Hoek-Brown peak strength criterion for jointed rock masses is given by:

$$\sigma'_1 = \sigma'_3 + ( m_b \sigma_{ci} \sigma'_3 + s\sigma_{ci}^2 )^a \tag{4.1}$$

where $\sigma'_1$ and $\sigma'_3$ are the maximum and minimum effective principal stresses at failure, $m_b$ is the reduced value of the material constant $m_i$ for the rock mass, and $s$ and $a$ are parameters which depend on the characteristics or quality of the rock mass.

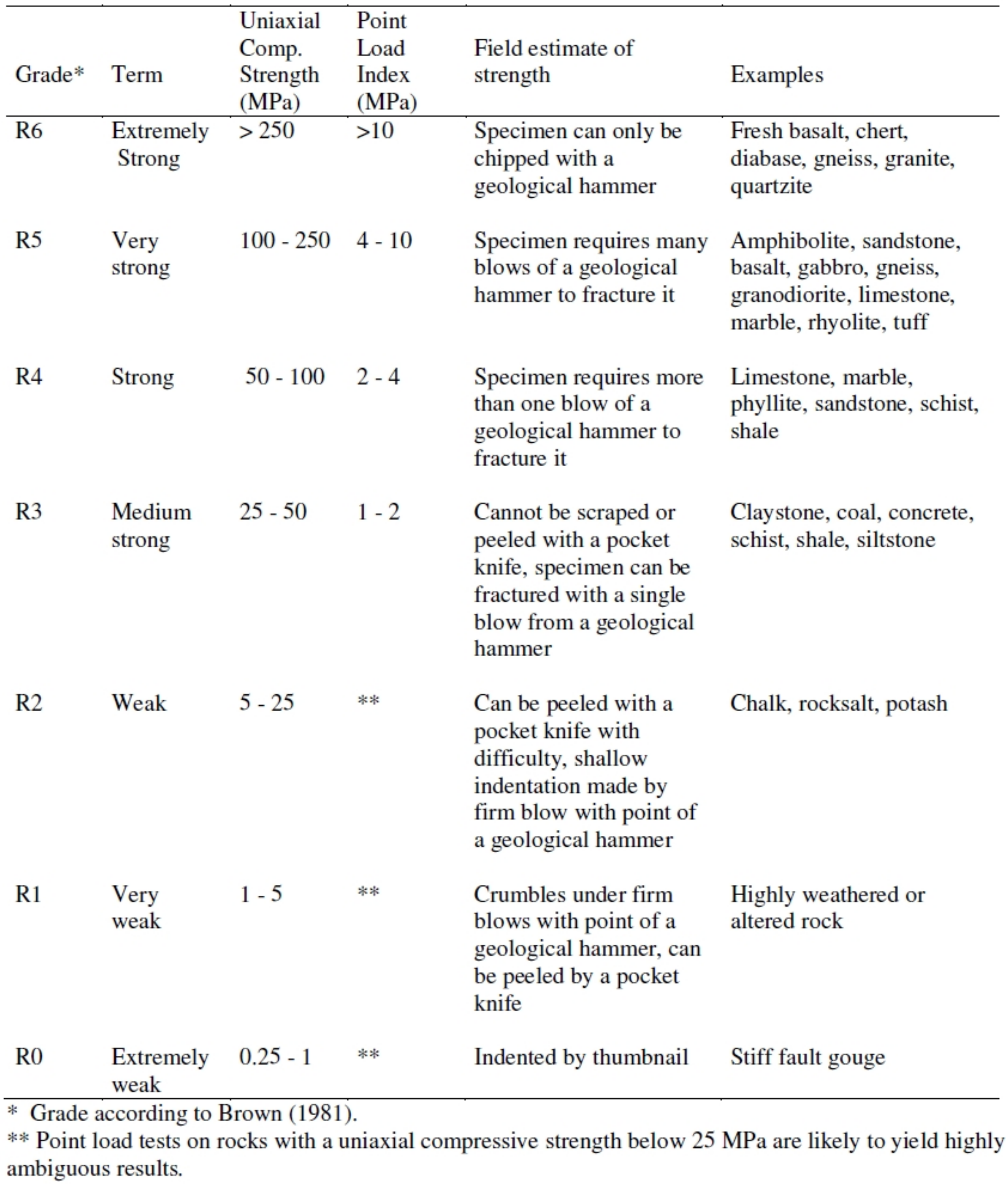
In order to use the Hoek-Brown criterion for estimating the strength and deformability of jointed rock masses, three properties of the rock mass have to be estimated. These are:

- uniaxial compressive strength $\sigma_{ci}$ of the intact rock pieces,
- value of the Hoek-Brown constant $m_i$ for these intact rock pieces, and
- value of the Geological Strength Index *GSI* for the rock mass.

The values of $\sigma_{ci}$ and $m_i$ constants should be determined by statistical analysis of the results of a set of triaxial tests on carefully prepared core samples. Once the five or more triaxial test results have been obtained, they can be analyzed to determine the uniaxial compressive strength $\sigma_{ci}$ and the Hoek-Brown constant $m_i$ as described by Hoek and

Brown (1980). When laboratory tests are not possible, Table 4.1 and Table 4.2 can be used to obtain estimates of $\sigma_{ci}$ and $m_i$.

Table 4.1. Field estimates of uniaxial compressive strength of intact rock (Marinos & Hoek, 2000).

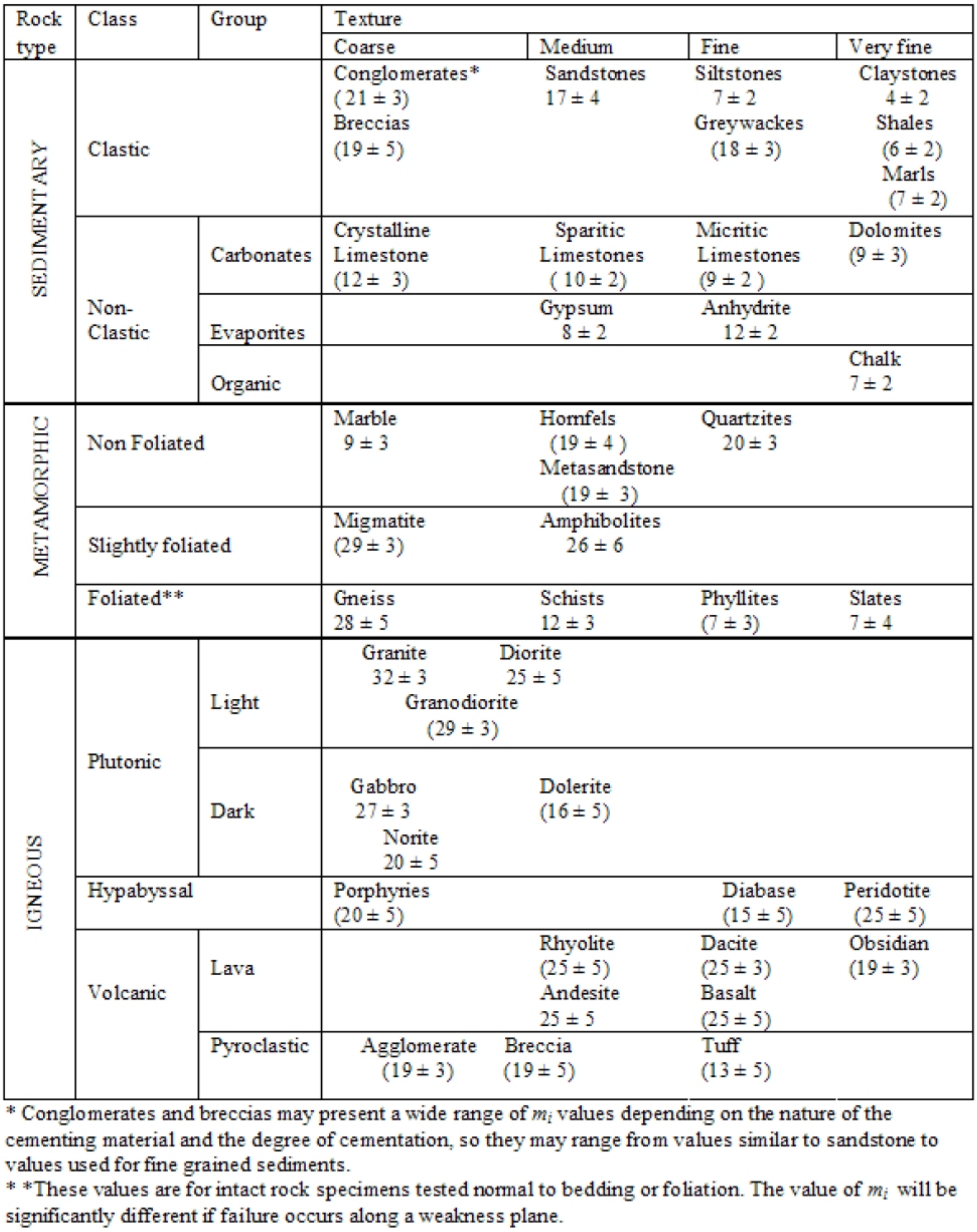


| Grade* | Term | Uniaxial Comp. Strength (MPa) | Point Load Index (MPa) | Field estimate of strength | Examples |
|---|---|---|---|---|---|
| R6 | Extremely Strong | > 250 | >10 | Specimen can only be chipped with a geological hammer | Fresh basalt, chert, diabase, gneiss, granite, quartzite |
| R5 | Very strong | 100 - 250 | 4 - 10 | Specimen requires many blows of a geological hammer to fracture it | Amphibolite, sandstone, basalt, gabbro, gneiss, granodiorite, limestone, marble, rhyolite, tuff |
| R4 | Strong | 50 - 100 | 2 - 4 | Specimen requires more than one blow of a geological hammer to fracture it | Limestone, marble, phyllite, sandstone, schist, shale |
| R3 | Medium strong | 25 - 50 | 1 - 2 | Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow from a geological hammer | Claystone, coal, concrete, schist, shale, siltstone |
| R2 | Weak | 5 - 25 | ** | Can be peeled with a pocket knife with difficulty, shallow indentation made by firm blow with point of a geological hammer | Chalk, rocksalt, potash |
| R1 | Very weak | 1 - 5 | ** | Crumbles under firm blows with point of a geological hammer, can be peeled by a pocket knife | Highly weathered or altered rock |
| R0 | Extremely weak | 0.25 - 1 | ** | Indented by thumbnail | Stiff fault gouge |

* Grade according to Brown (1981).

** Point load tests on rocks with a uniaxial compressive strength below 25 MPa are likely to yield highly ambiguous results.

Table 4.2. Values of the constant $m_i$ for intact rock, by rock group. Note that values in parenthesis are estimates. The range of values quoted for each material depends upon the granularity and interlocking of the crystal structure – the higher values being associated with tightly interlocked and more frictional characteristics (Marinos & Hoek, 2000).



| Rock type | Class | Group | Texture | | | |
|---|---|---|---|---|---|---|
| | | | Coarse | Medium | Fine | Very fine |
| SEDIMENTARY | Clastic | | Conglomerates* (21 ± 3)<br>Breccias (19 ± 5) | Sandstones 17 ± 4 | Siltstones 7 ± 2<br>Greywackes (18 ± 3) | Claystones 4 ± 2<br>Shales (6 ± 2)<br>Marls (7 ± 2) |
| | Non-Clastic | Carbonates | Crystalline Limestone (12 ± 3) | Sparitic Limestones (10 ± 2) | Micritic Limestones (9 ± 2) | Dolomites (9 ± 3) |
| | | Evaporites | | Gypsum 8 ± 2 | Anhydrite 12 ± 2 | |
| | | Organic | | | | Chalk 7 ± 2 |
| METAMORPHIC | Non Foliated | | Marble 9 ± 3 | Hornfels (19 ± 4)<br>Metasandstone (19 ± 3) | Quartzites 20 ± 3 | |
| | Slightly foliated | | Migmatite (29 ± 3) | Amphibolites 26 ± 6 | | |
| | Foliated** | | Gneiss 28 ± 5 | Schists 12 ± 3 | Phyllites (7 ± 3) | Slates 7 ± 4 |
| IGNEOUS | Plutonic | Light | Granite 32 ± 3<br>Diorite 25 ± 5<br>Granodiorite (29 ± 3) | | | |
| | | Dark | Gabbro 27 ± 3<br>Norite 20 ± 5 | Dolerite (16 ± 5) | | |
| | Hypabyssal | | Porphyries (20 ± 5) | | Diabase (15 ± 5) | Peridotite (25 ± 5) |
| | Volcanic | Lava | | Rhyolite (25 ± 5)<br>Andesite 25 ± 5 | Dacite (25 ± 3)<br>Basalt (25 ± 5) | Obsidian (19 ± 3) |
| | | Pyroclastic | Agglomerate (19 ± 3) | Breccia (19 ± 5) | Tuff (13 ± 5) | |

* Conglomerates and breccias may present a wide range of $m_i$ values depending on the nature of the cementing material and the degree of cementation, so they may range from values similar to sandstone to values used for fine grained sediments.

* *These values are for intact rock specimens tested normal to bedding or foliation. The value of $m_i$ will be significantly different if failure occurs along a weakness plane.

The most important component of the Hoek – Brown system for rock masses is the process of reducing the material constants $\sigma_{ci}$ and $m_i$ from their laboratory values to appropriate in situ values. This is accomplished through the Geological Strength Index *GSI* that is defined in Figure 4.2. The values of $m_b$ and *s* are related to the *GSI* for the rock mass.

$$m_b = m_i \, exp\left(\frac{GSI - 100}{28 - 14D}\right) \tag{4.2}$$

and

$$s = exp\left(\frac{GSI - 100}{9 - 3D}\right) \tag{4.3}$$

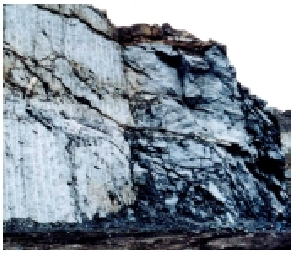
where *D* is a factor which depends on the degree to which the rock mass has been disturbed by blasting or stress relaxation. *D* varies from 0 for undisturbed in situ rock masses to 1.0 for very disturbed rock masses. For good quality blasting, it might be expected that $D \approx 0.7$. Guidelines for the selection of *D* are presented in Table 4.3.

In the initial version of the Hoek-Brown criterion, the index *a* took the value 0.5. After a number of other changes, Hoek et al. (2002) expressed the value of *a* which applies over the full range of *GSI* values as the function:

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \tag{4.4}$$

Note that for $GSI > 50$, $a \approx 0.5$, the original value. For very low values of *GSI*, $a \rightarrow 0.65$.

Table 4.3. Guidelines for estimating disturbance factor D (Marinos & Hoek, 2000).

| Appearance of rock mass | Description of rock mass | Suggested value of D |
|---|---|---|
|  | Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel. | D = 0 |
|  | Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass.<br>Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed. | D = 0<br>D = 0.5<br>No invert |
|  | Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass. | D = 0.8 |
|  | Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance. | D = 0.7<br>Good blasting<br>D = 1.0<br>Poor blasting |
|  | Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal.<br>In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less. | D = 1.0<br>Production blasting<br>D = 0.7<br>Mechanical excavation |

The uniaxial compressive strength of the rock mass, $\sigma_{cm}$ is obtained by setting $\sigma'_3$ to zero in equation 4.1 giving.

$$\sigma_{cm} = \sigma_{ci} s^a \tag{4.5}$$

Assuming that the uniaxial and biaxial tensile strengths of brittle rocks are approximately equal, the tensile strength of the rock mass, $\sigma_{tm}$ may be estimated by substituting $\sigma'_1 = \sigma'_3 = \sigma_{tm}$ in equation 4.1 to obtain

$$\sigma_{tm} = -\frac{s\sigma_{ci}}{m_b} \tag{4.6}$$

### *4.4.2 Mohr-Coulomb Criterion*

Since most geotechnical software is still written in terms of the Mohr-Coulomb failure criterion, it is necessary to determine equivalent angles of friction and cohesive strengths for each rock mass and stress range. This is done by fitting an average linear relationship to the curve generated by solving Equation 4.1 for a range of minor principal stress values defined by $\sigma_t < \sigma_3 < \sigma'_{3\max}$, as illustrated in Figure 4.3. The fitting process involves balancing the areas above and below the Mohr-Coulomb plot. This results in the following equations for the angle of friction and cohesive strength:

$$\phi' = sin{-}1\left[\frac{6am_b(s+m_b\sigma'_{3n})^{a-1}}{2(1+a)(2+a)+6am_b(s+m_b\sigma'_{3n})^{a-1}}\right] \tag{4.7}$$

$$c' = \frac{\sigma_{ci}\left[(1+2a)s+(1+a)m_b\sigma'_{3n}\right](s+m_b\sigma'_{3n})^{a-1}}{(1+a)(2+a)\sqrt{1+\left(6am_b(s+m_b\sigma'_{3n})^{a-1}\right)\Big/\left((1+a)(2+a)\right)}} \tag{4.8}$$

where, $\sigma_{3n} = \sigma'_{3max}/\sigma_{ci}$

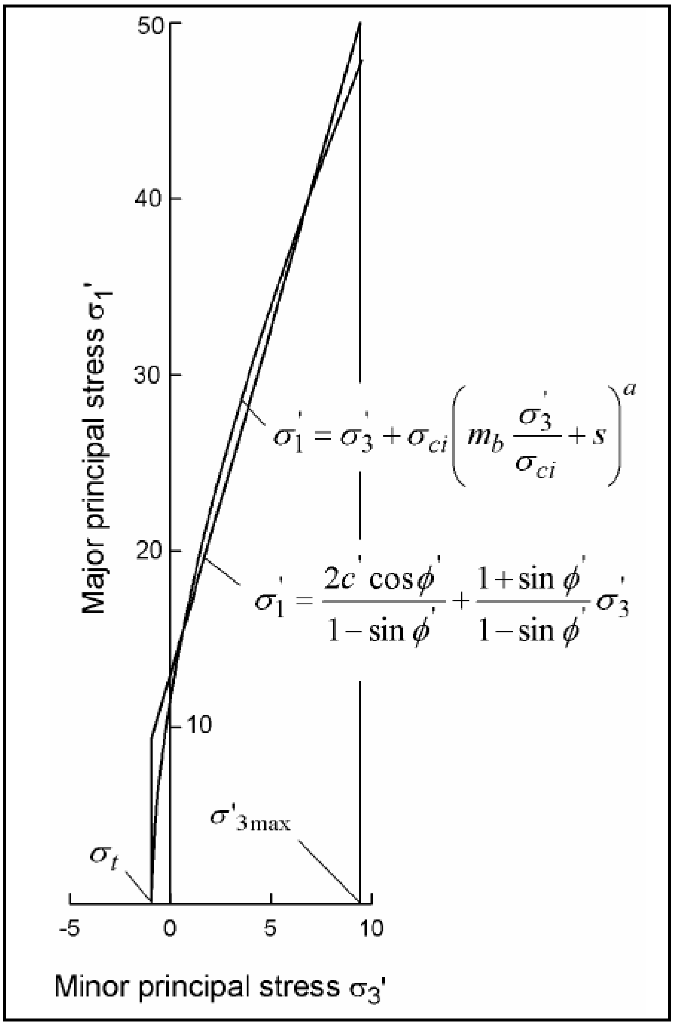


Figure 4.3 Relationships between major and minor principal stresses for Hoek-Brown and equivalent Mohr-Coulomb criteria (Hoek, Torres & Corkum 2002).

The value of $\sigma'_{3\max}$ the upper limit of confining stress over which the relationship between the Hoek-Brown and the Mohr-Coulomb criteria is considered, has to be determined for each individual case. Studies for slopes, using Bishop's circular failure analysis for a wide range of slope geometries and rock mass properties, yielded:

$$\frac{\sigma'_{3max}}{\sigma'_{cm}} = 0.72\left(\frac{\sigma'_{cm}}{\gamma H}\right)^{-0.91} \qquad (4.9)$$

where *H* is the height of the slope.

The equivalent plot, in terms of the major and minor principal stresses, is defined by:

$$\sigma'_1 = \frac{2c'cos\phi'}{1-sin\phi'} + \frac{1+sin\phi'}{1-sin\phi'}\sigma'_3 \tag{4.10}$$

These calculations, together with the Hoek-Brown criterion can also be performed by the program RocLab which is provided by Rocscience free of charge. In the next chapter the rock mass properties of greywacke determined by RocLab have been used as input for numerical analysis software Plaxis.

The Hoek-Brown failure criterion, which assumes isotropic rock and rock mass behaviour, should only be applied to those rock masses in which there is sufficient number of closely spaced discontinuities, with similar surface characteristics, where isotropic behaviour involving failure on discontinuities can be assumed. When the structure being analyzed is large and the block size is small in comparison, the rock mass can be treated as a Hoek-Brown material.

Where the block size is of the same order as that of the structure being analyzed or when one of the discontinuity sets is significantly weaker than the others, the Hoek-Brown criterion should not be used. In these cases, the stability of the structure should be analyzed by considering failure mechanisms involving the sliding or rotation of blocks and wedges defined by intersecting structural features.

It is reasonable to extend this argument further and to suggest that, when dealing with large scale rock masses, the strength will reach a constant value when the size of individual rock pieces is sufficiently small in relation to the overall size of the structure being considered. This suggestion is embodied in Figure 4.4 which shows the transition from an isotropic intact rock specimen, through a highly anisotropic rock mass in which failure is controlled by one or two discontinuities, to an isotropic heavily jointed rock mass.

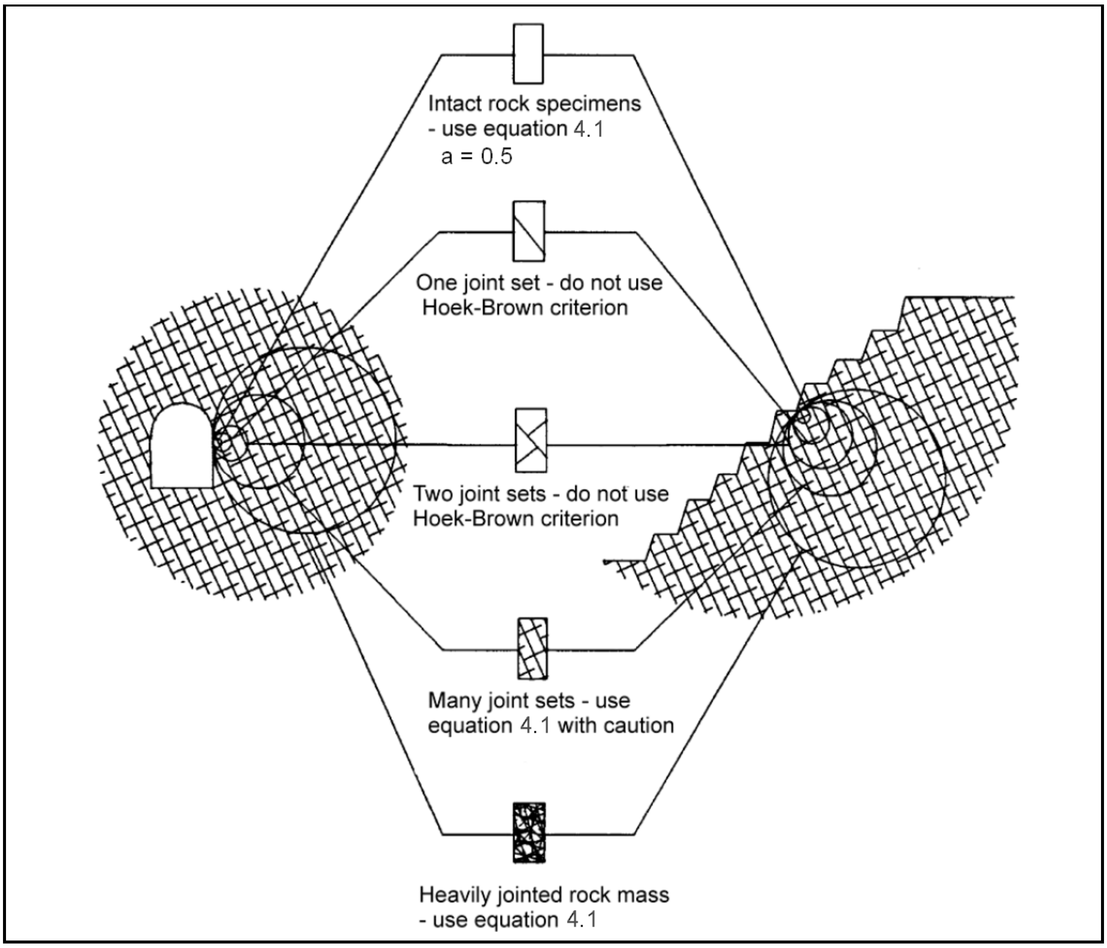


Figure 4.4 Idealized diagram showing the transition from intact to a heavily jointed rock mass with increasing sample size (Hoek, 1983).

### *4.4.3 Deformability*

The study of the complete stress–strain behaviour of jointed rock masses involving post-yield plastic deformation is beyond the scope of this study. However, it is of interest to consider the pre-peak behaviour with a view to determining equivalent overall elastic constants for use in design analyses.

In the simplest case of a rock mass containing a single set of parallel discontinuities, a set of elastic constants for an equivalent transversely isotropic (elastic anisotropy) continuum may be determined. For a case analogous to that shown in Figure 4.5, let the rock material be isotropic with elastic constants $E$ and, let the discontinuities have normal stiffness $K_n$ and shear stiffness $K_s$, and let the mean discontinuity spacing be $S$. By considering the deformations resulting from the application of unit shear and normal stresses with respect to the $x$, $y$ plane in Figure 4.5, it is found that the equivalent elastic constants required for use in the elastic constitutive equations (which are not given in detail in the thesis) for the material are given by:

$$E_1 = E \tag{4.11}$$

$$\frac{1}{E_2} = \frac{1}{E} + \frac{1}{K_n S} \tag{4.12}$$

$$v_1 = v \tag{4.13}$$

$$v_2 = \frac{E_2}{E} v \tag{4.14}$$

$$\frac{1}{G_2} = \frac{1}{G} + \frac{1}{K_s S} \tag{4.15}$$

Similar solutions for cases involving more than one set of discontinuities are presented by Amadei and Goodman (1981) and by Gerrard (1982). It is common to determine $E$ as the modulus of deformation or slope of the force–displacement curve obtained in a field compression test such as uniaxial compression, plate bearing, flatjack, pressure chamber, borehole jacking and dilatometer tests.

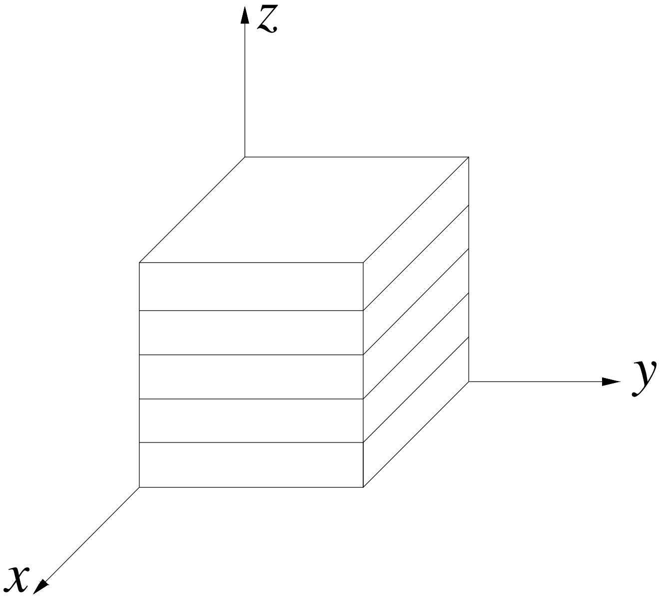


Figure 4.5 A transversely isotropic body for which the x, y plane is the plane of isotropy.

The results of such tests must be interpreted with care particularly when tests are conducted under deviatoric stress conditions on samples containing discontinuities that are favourably oriented for slip. Under these conditions, initial loading may produce slip as well as reflecting the elastic properties of the rock material and the elastic deformabilities of the joints. Using a simple analytical model, Brady et al. (1985) have demonstrated that, in this case:

(a) the loading–unloading cycle must be accompanied by hysteresis; and

(b) it is only in the initial stage of unloading (see Figure 4.6) that inelastic response is suppressed and the true elastic response of the rock mass is observed.

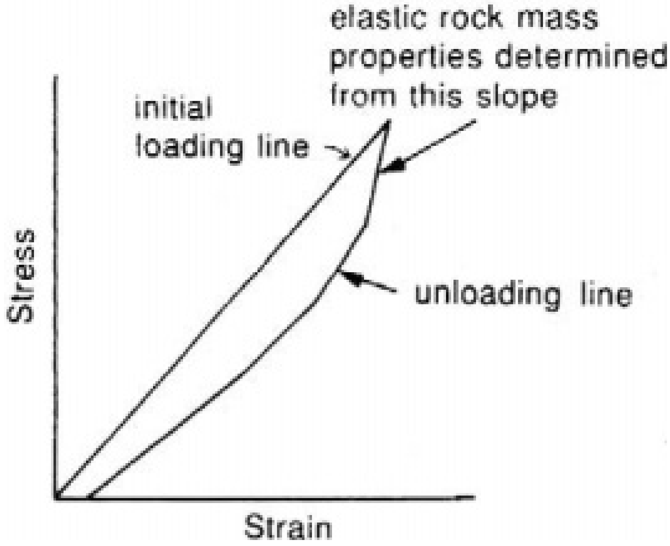


Figure 4.6 Determination of the Young's modulus of a rock mass from the response on initial unloading in a cyclic loading test (after Brady et al., 1985).

Bieniawski (1978) compiled values of in situ modulus of deformation determined using a range of test methods at 15 different locations throughout the world. He found that for values of Rock Mass Rating, *RMR*, greater than about 55, the mean deformation modulus, $E_M$, measured in GPa, could be approximated by the empirical equation

$$E_M = 2(RMR) - 100 \tag{4.16}$$

Serafim and Pereira (1983) found that an improved fit to their own and to Bieniawski's data, particularly in the range of $E_M$ between 1 and 10 GPa, is given by the Relation

$$E_M = 10^{\frac{RMR-10}{40}} \tag{4.17}$$

Figure 4.7 shows Equations 4.16 and 4.17 fitted to Bieniawski's (1978) and Serafim and Periera's (1983) data, respectively. It also shows further data provided by Barton (2002) fitted to the equation

$$E_M = 10Q_c^{1/3} \tag{4.18}$$

where $Q_c = Q_{\sigma c}/100$. (4.19)

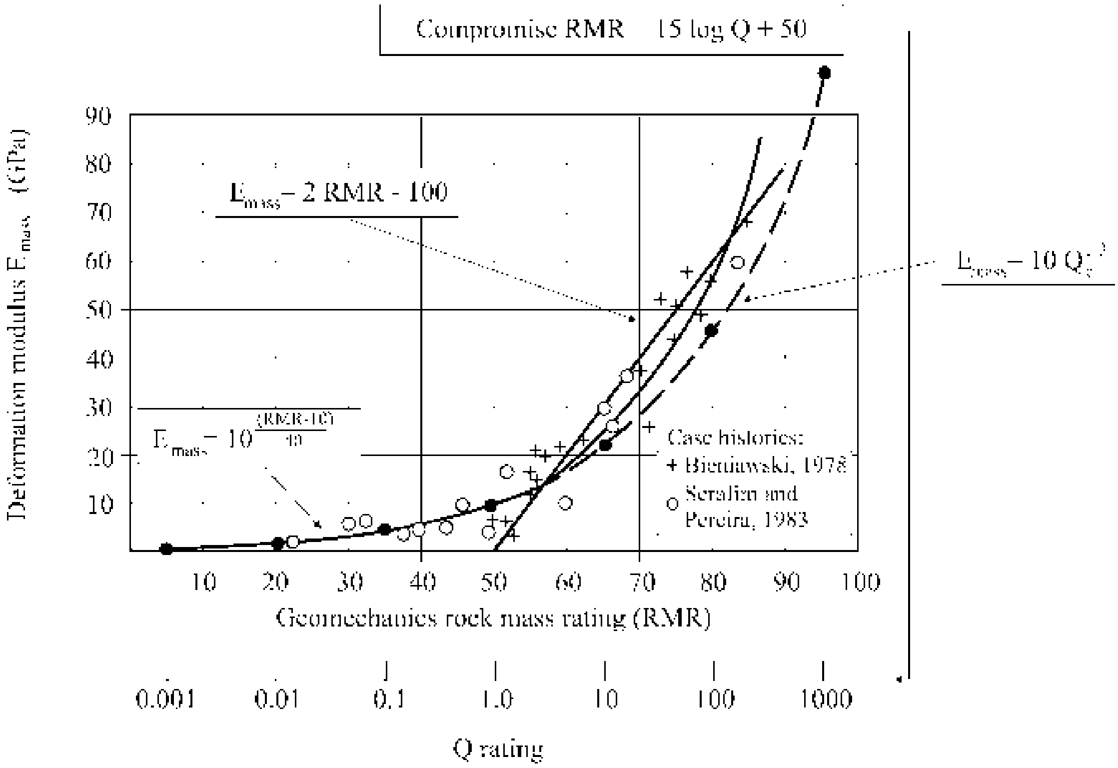


Figure 4.7 Measured values of static rock mass modulus, $E_M$, and some empirical relations (after Barton, 2002).

Following Hoek and Brown (1997), Hoek et al. (2002) proposed the more complex empirical relation

$$E_M = (1 - D/2)\sqrt{(\sigma_c/100)}10^{((GSI-10)/40)} \tag{4.20}$$

which is derived from equation 4.17 but gives an improved fit to the data at lower values of *RMR* (≈ *GSI* for *RMR* > 25), and includes the factor *D* to allow for the effects of blast damage and stress relaxation.

It must be recognized that equations 4.16 to 4.20 relate rock mass classification indices to measured static deformability values that show considerable scatter. Accordingly, it cannot be expected that they will always provide accurate estimates of $E_M$. It must also be remembered that, rock mass modulus may be highly anisotropic. They also vary non-linearly with the level of applied stress. Therefore, it can be expected to vary with depth.

## 4.5 Rock Mass Behaviour during Excavation

A rock mass can be said to be continuous if it consists of either purely intact rock, or of individual rock pieces that are small in relation to the overall size of the construction element studied, see Figure 4.8. The excavation dimension is kept constant, while the joint spacing is decreased.

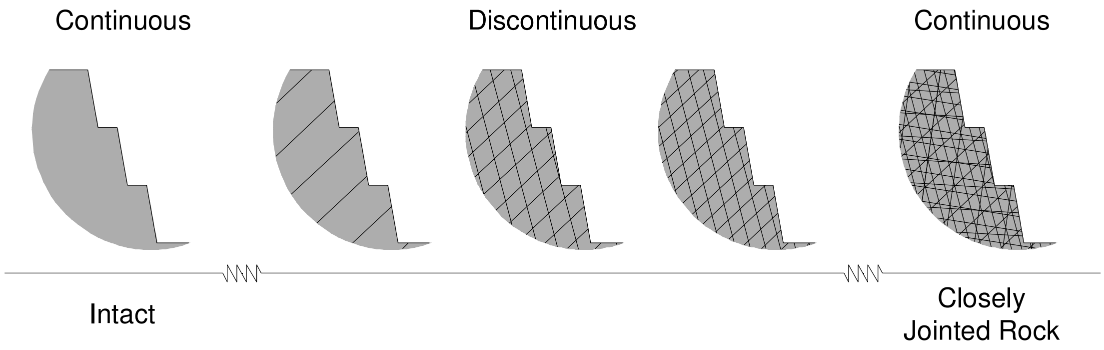


Figure 4.8 Example of continuous and discontinuous rock masses. The excavation size is constant.

A special case is when one discontinuity is significantly weaker than any of the others within a volume, as when dealing with a fault passing through a jointed rock mass. In such a case the continuous rock masses and the fault have to be treated separately.

For jointed rock masses, the issue of whether the rock mass can be considered continuous or discontinuous is also related to the construction scale in relation to the joint geometry. An increased excavation size in the same kind of rock mass can give different behaviour, as can be seen in Figure 4.9.

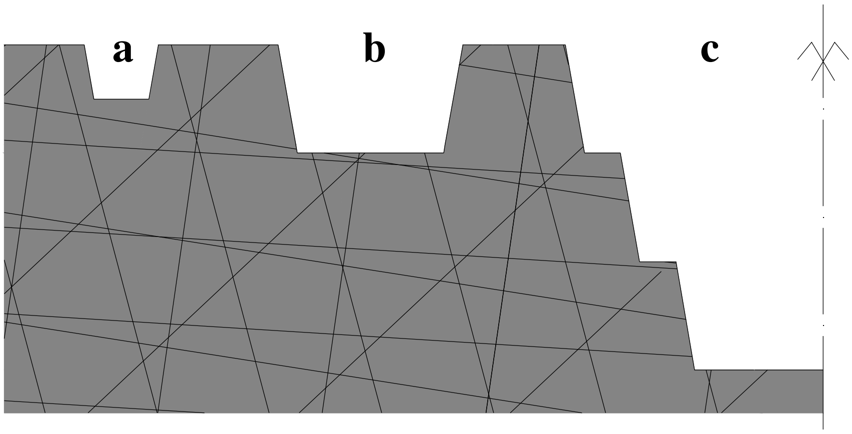


Figure 4.9 Example of continuous and discontinuous rock masses behaviour related to the excavation scale.

The smallest excavation in Figure 4.9 (a) is located inside a single block, which means that intact rock conditions holds. For the largest excavation (c), the rock mass can be assumed to be closely jointed. Thus both of these cases can be treated as continuum problems. The middle size excavation (b) is more likely to be a discontinuum problem. It is also important to notice that different construction sizes in a continuum material result in different strengths of the construction elements.

The strength of the rock mass is, in theory, determined by the combined strength of the intact rock and the various discontinuities in the rock mass. Instability of rock masses is often characterized by:

- Block failure : Structurally controlled failures (loosening, block fall). Normally treated as a discontinuum problem.
- Failures induced from overstressing :

  - Overstressing of massive rock (spalling, popping, strain burst) normally treated as a continuum problem.
  - Overstressing of jointed rock (shear failure, buckling) can be treated both as a continuum and discontinuum problem.
  - Overstressing of granular materials (soils, heavily jointed rocks) normally treated as a continuum problem.
- Instability in faults and weakness zones : Can be treated as a continuum or discontinuum problem, depending on the size of the weakness zone in relation to the construction size. For large scales, a fault or weakness zone can be treated as a joint and must therefore be analyzed as a discontinuum.

## 4.6 Numerical Methods for Mechanics of Discontinuous Rock

Rock mass can be generally classified into three groups, i.e., (a) continuous - refers to intact rock mass, (b) discontinuous – refers to jointed rock mass and (c) pseudo continuous – refers to highly fractured or weathered rock mass. The behaviour of intact rock mass can be analyzed by means of model based on continuum mechanics. The discontinuous model may be used for analyzing the jointed rock mass. In order to model the behaviour of discontinuous rock masses completely, special numerical techniques such as the Discrete Element Method (DEM) and Discontinuous Deformation Analysis (DDA) were developed. These methods explicitly model a rock mass as an aggregate of discontinuities and intact material. Discontinuous model similar to that of jointed rock mass can be used for type (c) rock mass. However, it is almost impossible to explore all the joint systems and it seems that this type of rock mass behaves just like a continuous body in a global sense. Therefore, a continuum mechanics model can be used with the effect of discontinuities adequately considered in the model. This is achieved by homogenization technique where equivalent continuum properties of the rock mass are derived based on the geometry of the contained fracture systems and physical properties of the intact rock matrix and the fractures.

Through the development of special elements – joint elements (sometimes also known as interface elements) – the continuum-based FEM can also be applied to the modelling of discontinuous rock masses. These elements can have either zero thickness or thin, finite thickness. They can assume linear elastic behaviour or plastic response when stresses exceed the strengths of discontinuities.

In the scope of the thesis continuum-based FEM computer program Plaxis 2D V8.2 and Jointed Rock material model is used to simulate jointed rock behaviour of greywacke. Analyses performed in this manner are presented in the following chapter.

# CHAPTER FIVE

# A PARAMETRIC STUDY ON EXCAVATIONS IN THE JOINTED ROCK-GREYWACKE WITH FINITE ELEMENT METHOD

## 5.1 Introduction

The skill in modelling is to spot the appropriate level of simplification to recognize those features which are important and those which are unimportant. The engineers may unaware of the simplification that they have made and problems may arise precisely because the assumptions that have been made are inappropriate in a particular application. Much more serious often are the idealizations of material behaviour that are necessary in order to develop a simple model.

The greywackes were identified as a jointed rock with various degrees of weathering by many investigators. However, it is commonly accepted in civil-geotechnical community of Istanbul that greywackes are modelled as a continuum mass without discontinuities.

One of the most widespread approaches of incorporating the influence of discontinuities on rock mass strength in numerical analysis is through modelling of rock masses as continua with reduced deformation and strength properties (Hammah, Yacoub, Corkum & Curran, 2008). However, method based on this approach is not able to model all deformation mechanisms of jointed rocks. In this chapter the effects of discontinuities on rock mass behaviour during excavation are investigated by means of parametric study. For that purpose four cases are distinguished with a “jointed rock model” that is available in the commercial Finite Element (FE) program Plaxis, according to joint set number and to whether excavation is supported or not.

## 5.2 Jointed Rock (JR) Model

The Finite Element Method (FEM) is the most widespread numerical analysis method. Its popularity can be primarily attributed to its ability to:

1. handle multiple materials in a single model (material heterogeneity)
2. readily accommodate non-linear material responses, and
3. model complex boundary conditions.

Although the FEM is a continuum method, special elements – joint (interface) elements – have been developed to directly represent the discontinuous behaviour characteristic of joints and interfaces between adjacent blocks of material (Hammah, Yacoub, Corkum, Wibowo & Curran, 2007).

The Jointed Rock model of Plaxis is an anisotropic elastic perfectly-plastic (transversely isotropic) model, especially meant to simulate the behaviour of jointed rocks. In this model it is assumed that there is intact rock with an eventual stratification direction and major joint directions. The intact rock is considered to behave as a transversely isotropic elastic material, quantified by five parameters and a direction. The anisotropy may result from stratification or from other phenomena (see Figure 5.1). In the major joint directions it is assumed that shear stresses are limited according to Coulomb's criterion. Upon reaching the maximum shear stress in such a direction, plastic sliding will occur. A maximum of three sliding directions can be defined as planes. The first plane is assumed to coincide with the direction of elastic anisotropy. Each plane may have different shear strength properties. In addition to plastic shearing, the tensile stresses perpendicular to the three planes are limited according to a predefined tensile strength (tension cut-off) (Plaxis Bv., 2006).

The application of the JR model is justified when joint sets are present. These joint sets have to be parallel, not filled with fault gouge, and their spacing has to be small compared to the characteristic dimension of the structure.

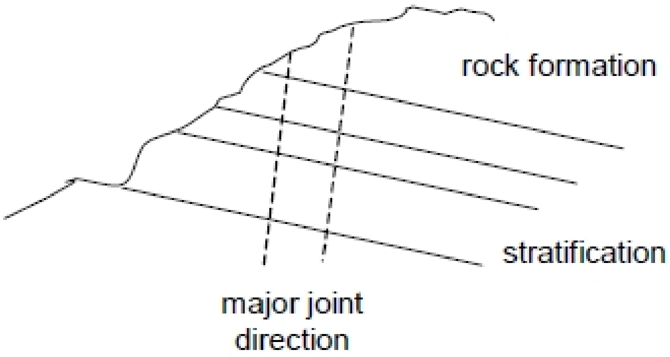


Figure 5.1 Visualization of concept behind the Jointed Rock model

### *5.2.1 Parameters of the Jointed Rock Model*

Most parameters of the jointed rock model coincide with those of the isotropic Mohr-Coulomb model. These are the basic elastic parameters and the basic strength parameters.

*Elastic parameters as in Mohr-Coulomb model:*

| | | |
|---|---|---|
| $E_1$ | : Young's modulus for rock as a continuum | [kN/m2] |
| $\nu_1$ | : Poisson's ratio for rock as a continuum | [−] |

*Anisotropic elastic parameters 'Plane 1' direction (e.g. stratification direction):*

| | | |
|---|---|---|
| $E_2$ | : Young's modulus in 'Plane 1' direction | [kN/m2] |
| $G_2$ | : Shear modulus in 'Plane 1' direction | [kN/m2] |
| $\nu_2$ | : Poisson's ratio in 'Plane 1' direction | [−] |

*Strength parameters in joint directions (Plane i=1, 2, 3):*

$c_i$ : Cohesion [kN/m2]

$\phi_i$ : Friction angle [°]

$\psi$ : Dilatancy angle [°]

$\sigma_{t,i}$ : Tensile strength [kN/m2]

*Definition of joint directions (Plane i=1, 2, 3):*

$n$ : Number of joint directions $(1 \leq n \leq 3)$

$\alpha_{1,i}$ : Dip angle [°]

$\alpha_{2,i}$ : Dip direction [°]

### *5.2.2 Definition of joint directions*

It is assumed that the direction of elastic anisotropy corresponds with the first direction where plastic shearing may occur (Plane 1). This direction must always be specified. In the case the rock formation is stratified without major joints, the number of sliding planes (= sliding directions) is still 1, and strength parameters must be specified for this direction anyway. A maximum of three sliding directions can be defined. These directions may correspond to the most critical directions of joints in the rock formation.

The sliding directions are defined by means of two parameters: The Dip angle (1) (or shortly Dip) and the Dip direction (2). Instead of the latter parameter, it is also common in geology to use the Strike. However, care should be taken with the definition of Strike, and therefore the unambiguous Dip direction as mostly used by rock engineers is used in Plaxis. The definition of both parameters is visualized in Figure 5.2.

Consider a sliding plane, as indicated in Figure 5.2. The sliding plane can be defined by the vectors $(s,t)$, which are both normal to the vector $n$. The vector $n$ is the 'normal' to the sliding plane, whereas the vector $s$ is the 'fall line' of the sliding plane and the vector $t$ is the 'horizontal line' of the sliding plane. The sliding plane makes an angle $\alpha_1$ with respect to the horizontal plane, where the horizontal plane can be defined by the vectors (*s**,*t*), which are both normal to the vertical *y*-axis. The angle $\alpha_1$ is the dip angle, which is defined as the positive 'downward' inclination angle between the horizontal plane and the sliding plane. Hence, $\alpha_1$ is the angle between the vectors *s** and *s*, measured clockwise from *s** to *s* when looking in the positive *t*-direction. The dip angle must be entered in the range [0, 90].

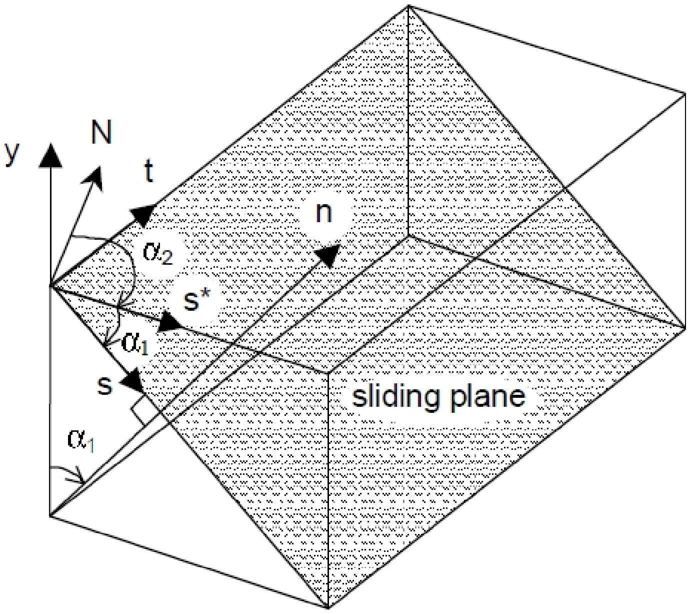


Figure 5.2 Definition of dip angle and dip direction

The orientation of the sliding plane is further defined by the dip direction, $\alpha_2$, which is the orientation of the vector *s** with respect to the North direction (*N*). The dip direction is defined as the positive angle from the North direction, measured clockwise to the horizontal projection of the fall line (=*s**-direction) when looking downwards. The dip direction is entered in the range [0, 360].

For plane strain conditions (the cases considered in Plaxis 2D V8) only $\alpha_1$ is required. By default, $\alpha_2$ is fixed at 90°.

## 5.3 Finite Element Analysis of Example Excavation Models in Greywackes

In this chapter a series of numerical analyses have been carried out to assess the effects of the discontinuities on rock mass during excavation. For this purpose two conditions have been numerically simulated for two different jointing degrees for excavation height, H=25.0m. Totally four configurations have been analyzed and result have been compared. These conditions are:

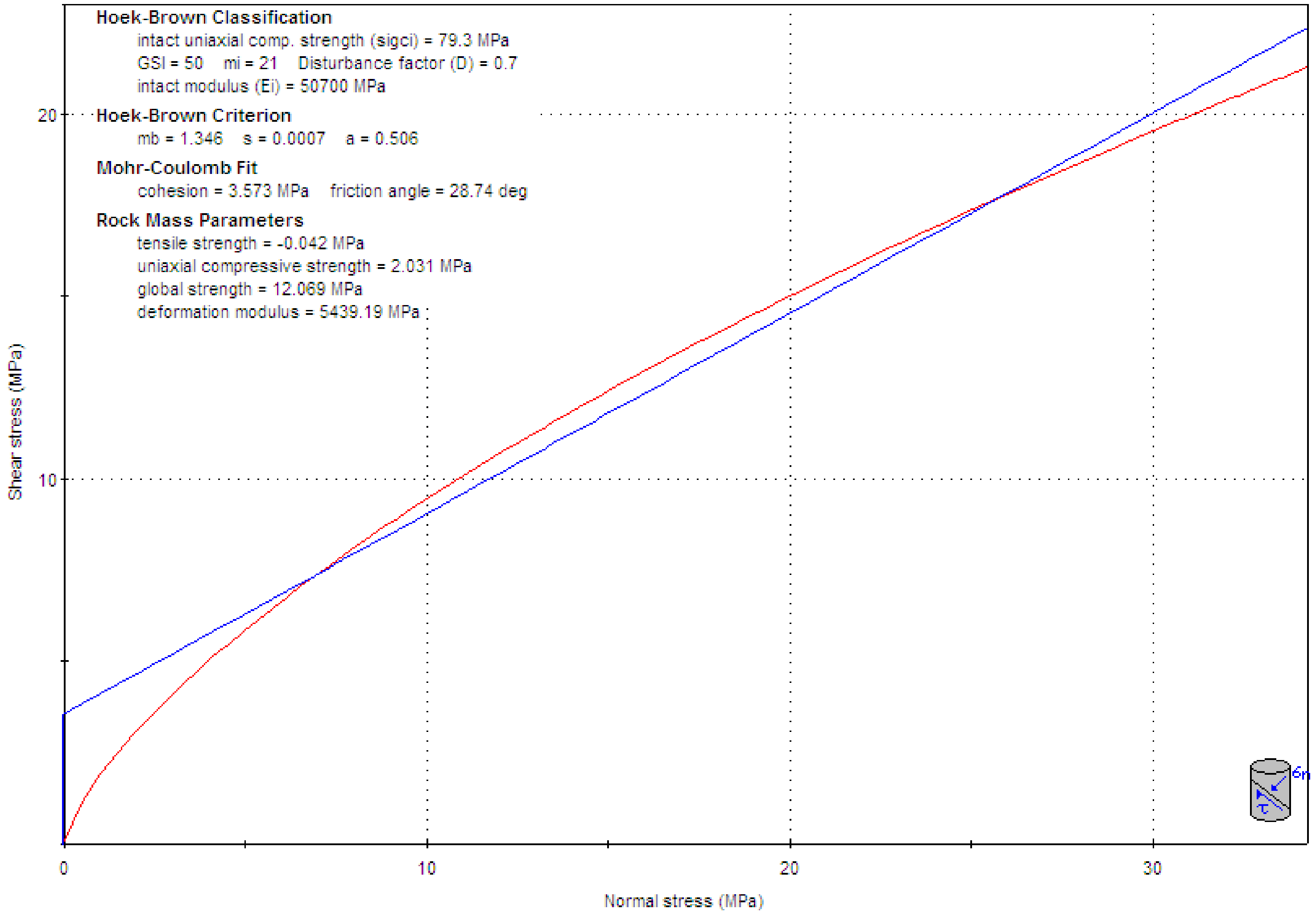
1. un-supported excavation,
2. excavation with pre-stressed anchor.

Each of these two conditions are analyzed for excavation depth 25m with slope angle 90° and two different jointing degrees (intact rock and rock with three joint sets) are used to assess the effect of the joints.

### *5.3.1 Geological Data and Model Parameters*

The intact rock mechanical and elastic properties of the greywackes were determined from the Table 3.1 and Table 3.2 and the rock mass properties of the greywacke determined using RocLab V1.031 (Rockscience, Inc., 2007) which is a free software based generalized Hoek-Brown failure criterion. Figure 5.3 and Figure 5.4 shows the RocLab analysis results for $\alpha_l$=0° and 90° and $\alpha_l$=45° respectively.

Table 5.1 shows the basic information about the properties of greywacke model parameters for Plaxis analysis.



**Hoek-Brown Classification**

intact uniaxial comp. strength (sigci) = 79.3 MPa

GSI = 50 mi = 21 Disturbance factor (D) = 0.7

intact modulus (Ei) = 50700 MPa

**Hoek-Brown Criterion**

mb = 1.346 s = 0.0007 a = 0.506

**Mohr-Coulomb Fit**

cohesion = 3.573 MPa friction angle = 28.74 deg

**Rock Mass Parameters**

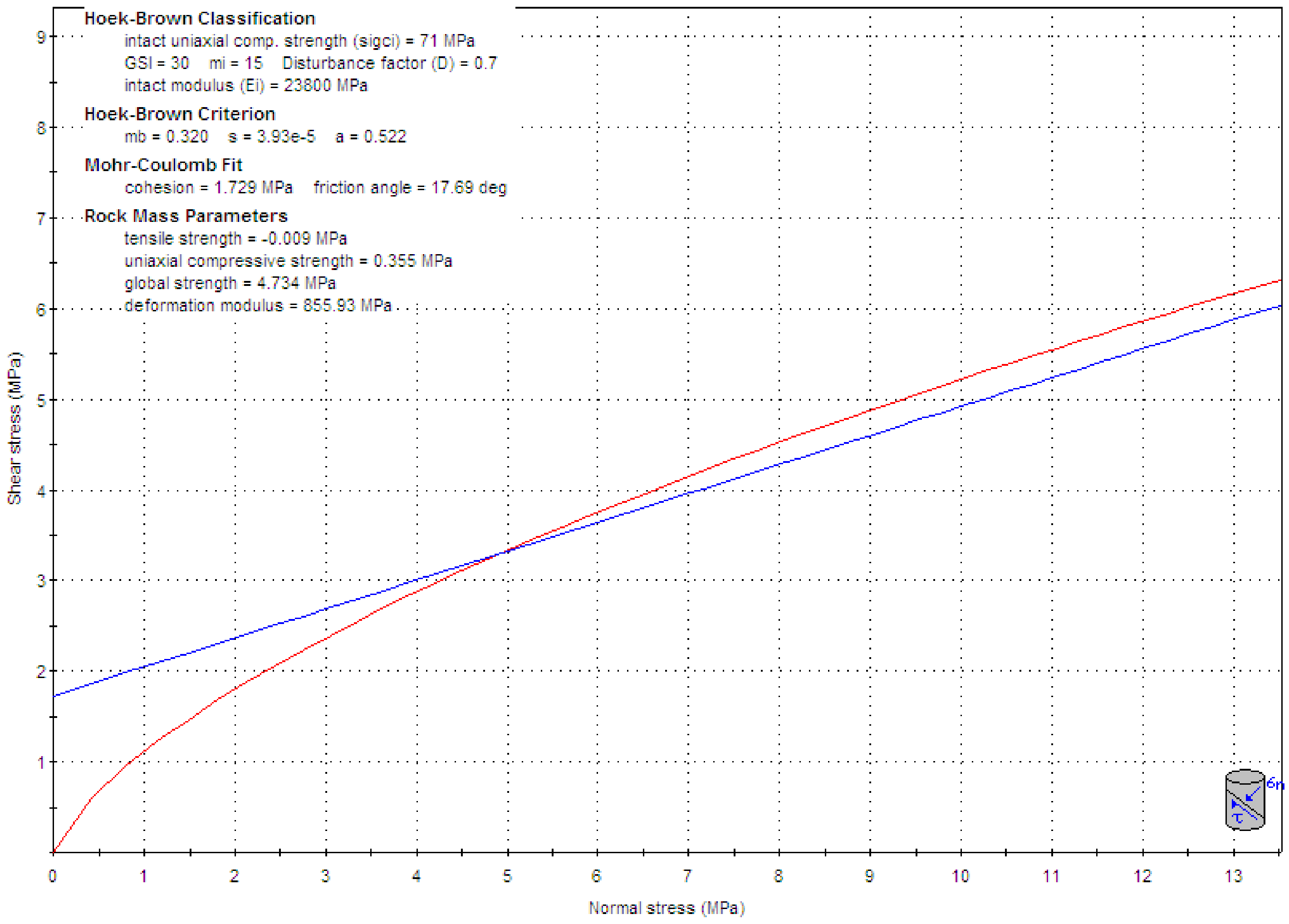
tensile strength = -0.042 MPa

uniaxial compressive strength = 2.031 MPa

global strength = 12.069 MPa

deformation modulus = 5439.19 MPa

Figure 5.3 RocLab results for dip angle $\alpha_l$=0° and 90°



**Hoek-Brown Classification**
intact uniaxial comp. strength (sigci) = 71 MPa
GSI = 30    mi = 15    Disturbance factor (D) = 0.7
intact modulus (Ei) = 23800 MPa

**Hoek-Brown Criterion**
mb = 0.320    s = 3.93e-5    a = 0.522

**Mohr-Coulomb Fit**
cohesion = 1.729 MPa    friction angle = 17.69 deg

**Rock Mass Parameters**
tensile strength = -0.009 MPa
uniaxial compressive strength = 0.355 MPa
global strength = 4.734 MPa
deformation modulus = 855.93 MPa

Figure 5.4 RocLab results for dip angle $\alpha_l$=45°

Table 5.1 Summaries of geomechanical properties of greywacke for Plaxis model.

| **Geomechanical Properties** | **Sliding Planes** | | |
|---|---|---|---|
| | **Plane-1 (Stratification Direction)** | **Plane-2** | **Plane-3** |
| Young's modulus, $E$ (GPa) | 5.4 | 0.9 | 5.4 |
| Shear modulus, $G$ (GPa) | 2.3 | 0.2 | 2.3 |
| Poisson's ratio, $\nu$ | 0.19 | 0.23 | 0.19 |
| Cohesion, $c$ (MPa) | 3.6 | 1.7 | 3.6 |
| Friction angle, $\phi$ (°) | 29 | 18 | 29 |
| Dilatancy angle, $\psi$ (°) | 0 | 0 | 0 |
| Tensile strength, $\sigma_t$ (MPa) | 0.042 | 0.009 | 0.042 |
| Dip Angle, $\alpha_1$ (°) | 0 | 45 | 90 |

### *5.3.2 Analysis Results*

The analysis results are presented in Table 5.2 below and graphics of stress distribution at failure for each case are presented Figure 5.5 to Figure 5.8. Analysis results indicate the need to perform FEM analysis with the effects of the discontinuities on excavations in jointed rock masses. Such analysis promotes greater understanding of problems. It increases the chances of success through more robust design of excavations, stabilization measures and improved monitoring decisions.

Table 5.2 Summaries of FEM (Plaxis) analysis results.

| **Cases** | **Rock Mass** | **Excavation Depth (m)** | **Support System** | **Factor of Safety** | **Failure Mode** |
|---|---|---|---|---|---|
| 1st Case | Intact Rock | 25 | Un-supported | 3.33 e13 | Base Failure |
| 2nd Case | Jointed Rock | 25 | Un-supported | 75 | Tension Failure |
| 3th Case | Intact Rock | 25 | 9 Level Pre-stressed Anchors | 6.13 e8 | Base Failure |
| 4th Case | Jointed Rock | 25 | 9 Level Pre-stressed Anchors | 190 | Base Failure / Tension Failure |

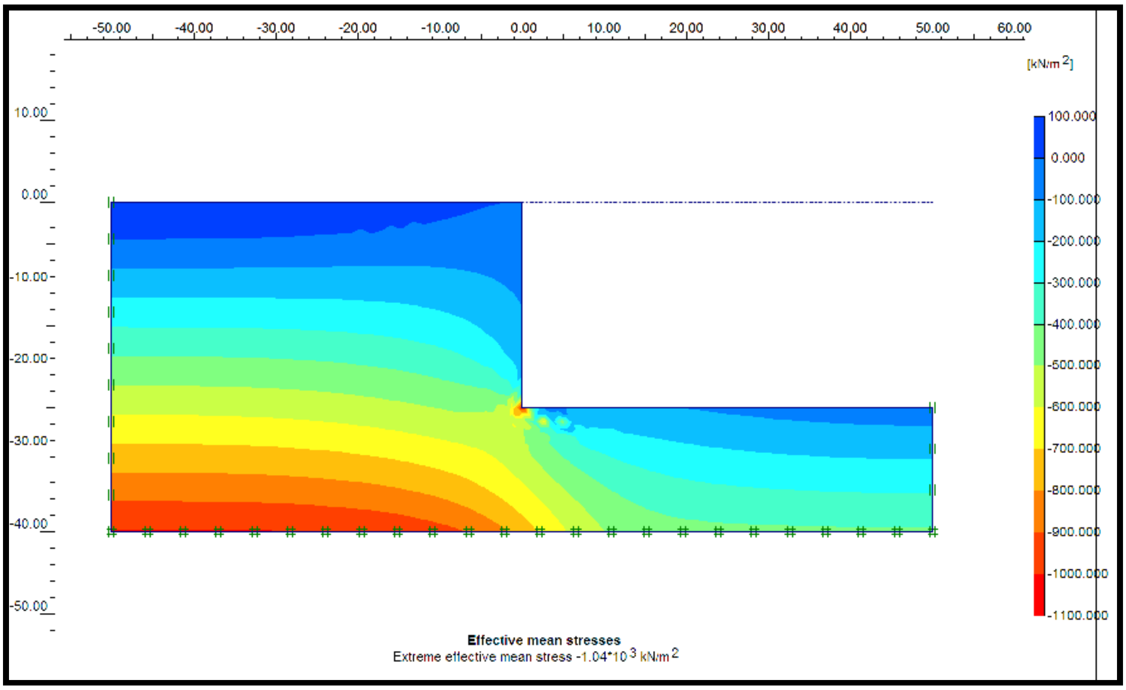


Figure 5.5 Effective mean stresses at failure for Case-1.

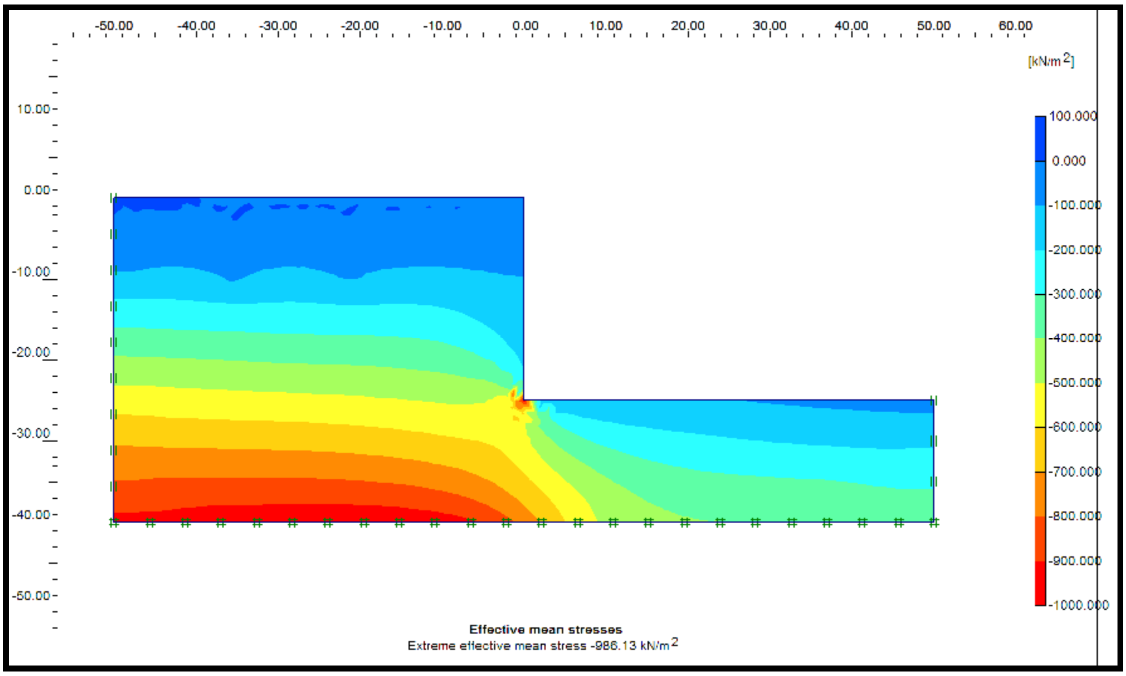


Figure 5.6 Effective mean stresses at failure for Case-2.

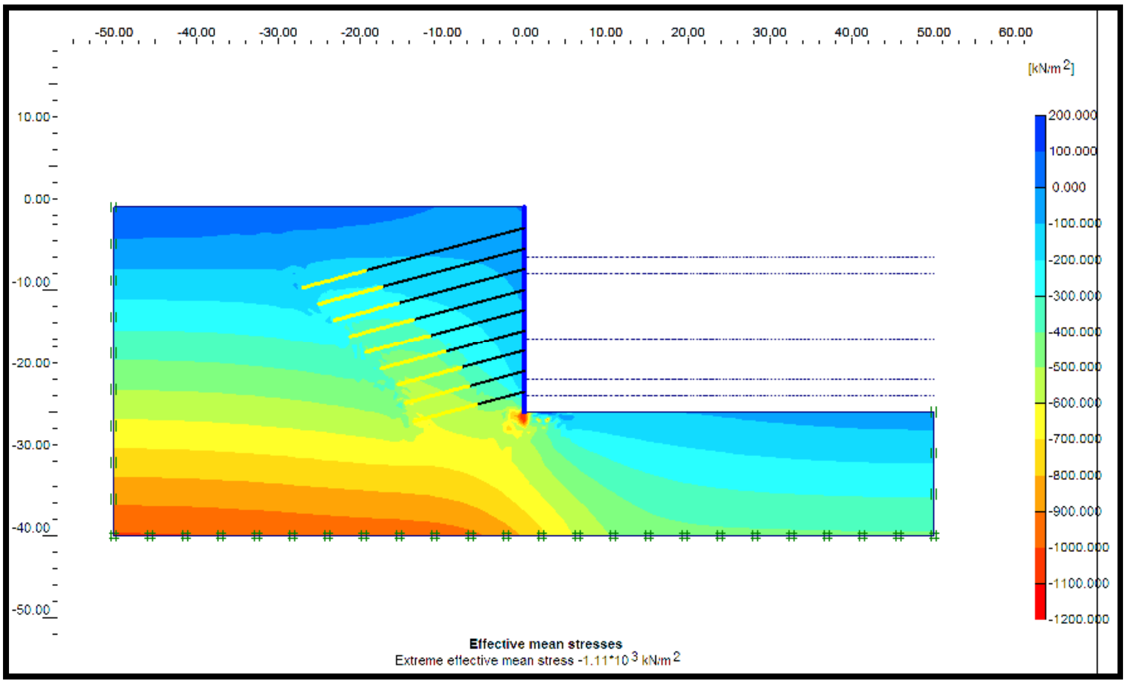


Figure 5.7 Effective mean stresses at failure for Case-3.

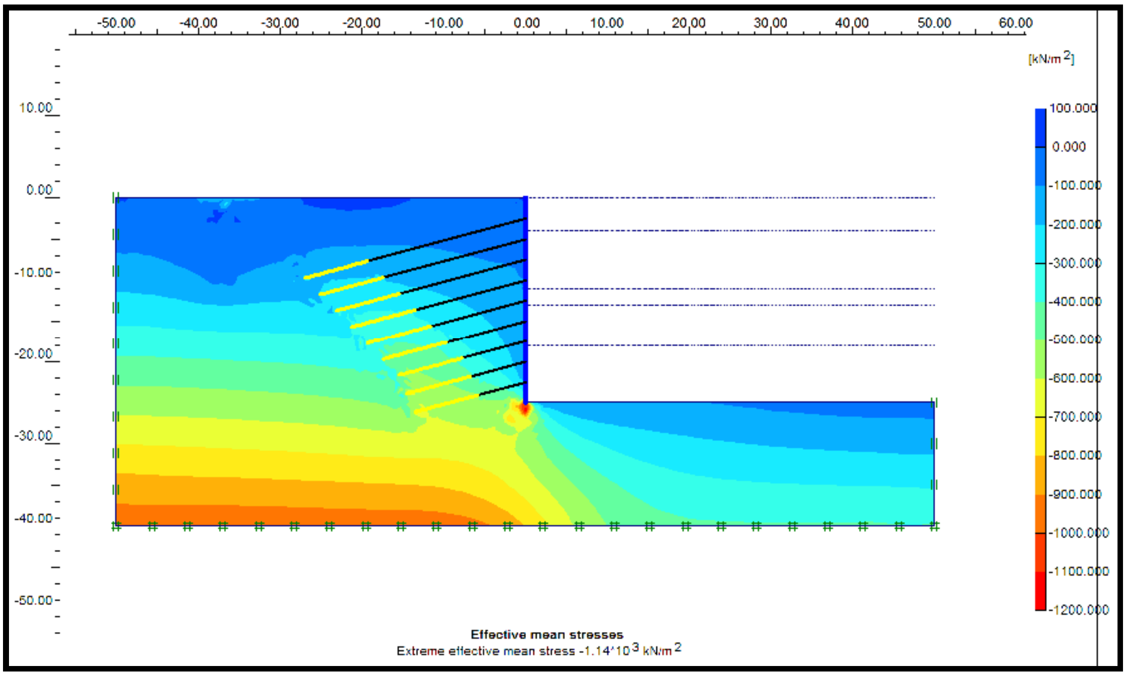


Figure 5.8 Effective mean stresses at failure for Case-4.

# CHAPTER SIX
# A CASE STUDY

## 6.1 Introduction

In this chapter, a unique case study, in which the author involved throughout design, construction and monitoring phases as site manager, is presented. The author thinks that the project is a special and challenging project because of the soil and rock stratigraphy at the project site, construction methodology of the excavation support system and the location of the adjacent properties, etc. Hereby, the performance of segmented cast in-situ reinforced concrete retaining wall system with pre-stressed ground anchors has been examined under difficult soil and jointed rock conditions and limitations. Additionally, some practical information about construction and monitoring of the subject system has been presented which will be a useful guide for future studies.

## 6.2 A Case Study: Point Hotel Barbaros Deep Excavation Project

The city of Istanbul has experienced significant growth in economy and population especially during the last decade. While becoming the biggest metropolitan city of the region, the need for high-rise 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 several number of basement levels, especially to obtain parking space and entertainment facilities, deep excavations and construction of retaining structures became compulsory. The case study of this thesis is called as “The Point Hotel”. The construction site is located at a very prestigious district of the city of Istanbul. Therefore, maximum underground space gain was desired.

According to the initial design, the structure was planned to have seven basements requiring soil and rock excavation to a maximum depth of -31.5 m below ground surface. However, gas pipeline displacement activities at the east and south sides of the construction site caused delays that are more than two months. As a result, the project was revised because of completion time limitation and maximum 24.5 m of excavation was performed partly under groundwater according to the revised project.

Anchored (tied back) retaining wall was utilized as temporary support system for Point Hotel excavation. In order to build foundation and basements of the hotel building, segmented reinforced concrete walls with a surface area of approximately 4,100 $m^2$ were cast in-situ and approximately 17,000 m pre-stressed anchor was installed.

The performances of the walls were monitored by inclinometer recordings taken at certain time intervals in parallel to the excavation at various locations. During excavation, excessive lateral displacements were monitored at one inclinometer (inc-3) on the west side of the site because of unforeseen potential slip planes at the contact zone of the greywacke and the siltstone formations. Additional lateral supports were constructed at this part of the system.

### *6.2.1 Project Description and Adjacent Properties*

Point hotel is located opposite the Dedeman Hotel at Yildiz Posta Street in Gayrettepe, Istanbul. The location of the hotel is shown in Figure 3.1. Point Hotel is completed and put in service in May 2009 by OZBEK TURIZM SANAYI VE TICARET A.S. (hereinafter referred to as the Owner) as the first art-tech hotel of the world with high standard and modern architecture. The Hotel consists of a high-rise tower with more than eleven meeting and conference rooms, a ballroom, executive floors, a helicopter landing area, swimming pools, a gaming floor, spa and underground parking area, etc. With these features the hotel construction cost is more than 60 million US dollars. Figure 3.2 demonstrates the construction of the building before its completion.

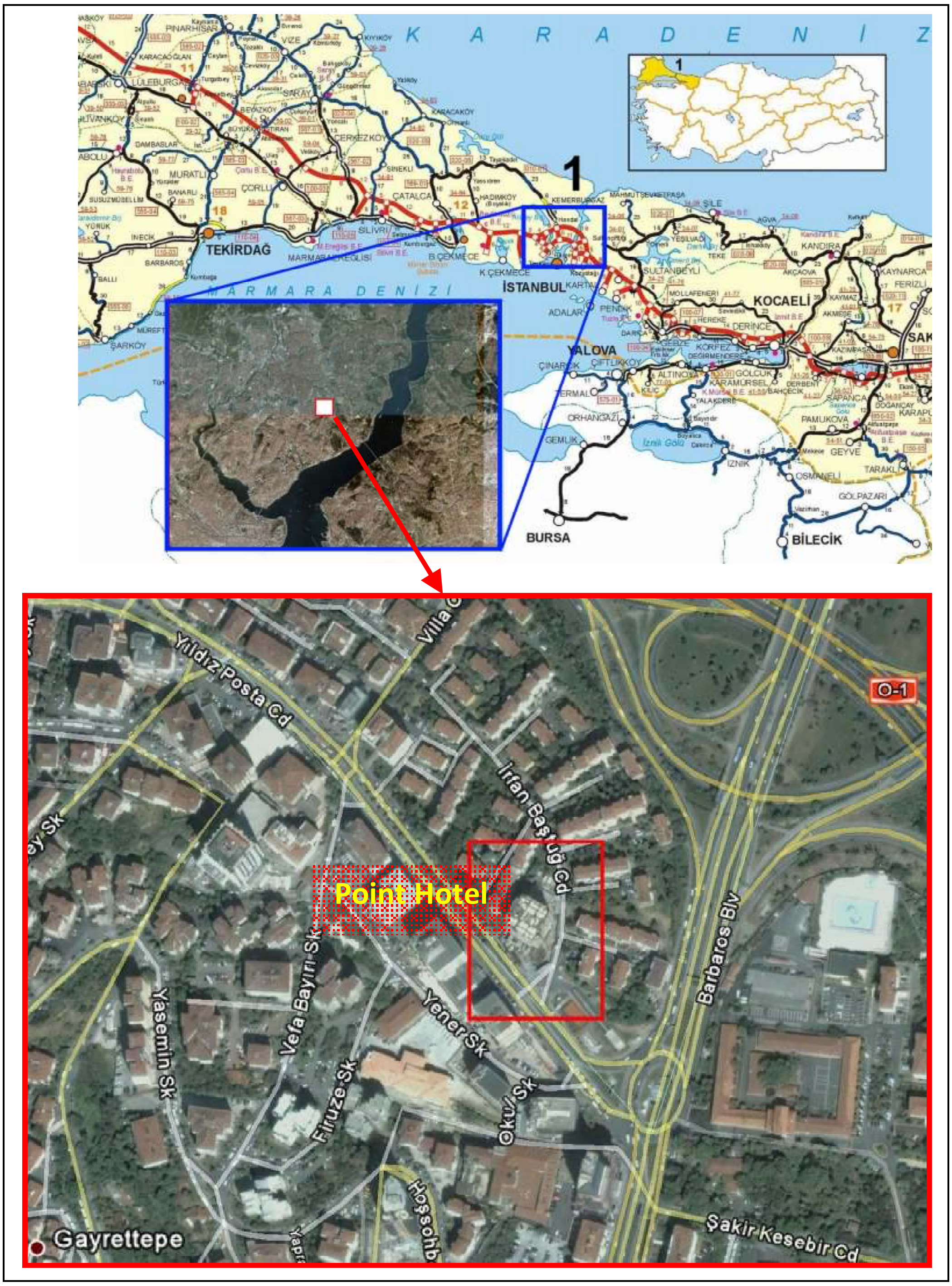


Figure 3.1 Construction site location.

Figure 3.2 Point Hotel Barbaros, during the construction, July 2008.

The project site is located in an urban area with highly dense population and currently occupied by a number of residential buildings, hotel buildings, and office buildings. Especially, ten–storey residential building is located too close at the west part of the site. The excavation boundary is about 8 m to 10 m away from this adjacent property boundary. Figure 3.3 shows the location of the construction site and adjacent buildings.

As in the other densely populated metropolitan cities, the value of the land in high-status area is a major cost issue in construction. Therefore, entertainment facilities as well as parking lots and the electro-mechanical technical spaces are positioned in underground levels. On the other hand, the issue of excavation support becomes further complicated in the event the excavation is feared to undermine the structural integrity of nearby foundations. In such cases, retaining systems must be designed to meet the specific requirements of the site and soil/rock conditions. Due to unique position of adjacent buildings and the roads, it was necessary to spend extensive time to both design and application stages of the subject project.

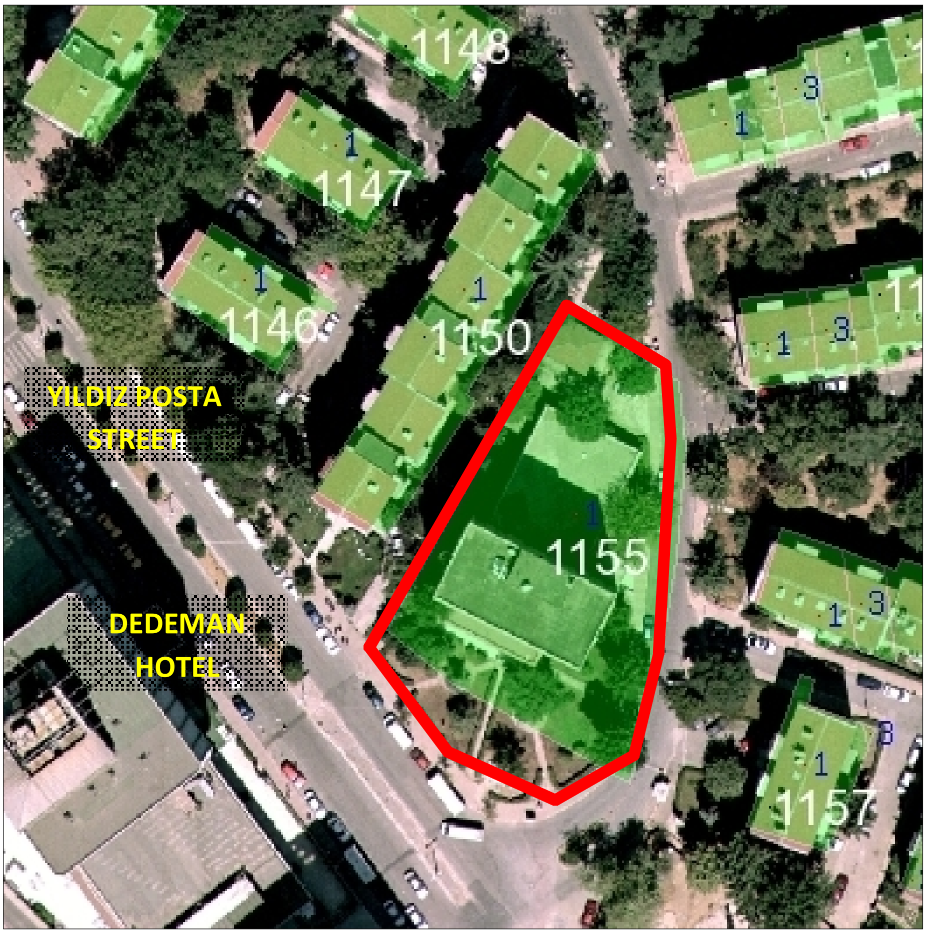


Figure 3.3 Aerial view of the construction site and adjacent buildings before excavation.

### 6.2.2 *Topography, Subsoil Conditions and Public Facilities*

The area of the construction site is approximately 2,060 m² and the perimeter of the site is approximately 185 m. The site has rugged topography with about 4 m difference in elevation orthogonal to the Yildiz Posta Street. The topographic elevations at the site vary between +125.0 m and +129.0 m according to the local Istanbul datum (i.e. sea level). According to the final project, basement elevation is +104.5 m. Therefore the average height of the excavation is nearly 22.5 m.

In order to establish project requirements and constraints, which may significantly affect design, construction, and cost of the wall system, a number of investigations must be undertaken prior to the design and construction.

In general, an adequate pre-design investigations include the drilling of soil borings, performing of soil and rock tests (in situ and lab.) and developing of geotechnical reports, the performing of pre-construction surveys of adjacent properties and utilities, and the determination of the location of those utilities to verify potential interferences.

An adequate subsurface borings should be advanced at regular intervals along, behind, and in front of the wall alignment and borings should be located at the site extremities along the wall alignment so that stratigraphy information can be interpolated from the boring information. However, only 3 boreholes, covering total length of 103.5 m, were performed in the scope of the sub-surface investigation because of the adjacent building location, existing old buildings and play ground at the construction site. Figure 3.4 shows these buildings and play ground before they are demolished. The locations of the investigation points are shown on the general layout plan of the site in Figure 3.4.

Standard Penetration Test (SPT) was performed at regular intervals in the subsoil located above the lithological bedrock unit. The drilling method was rotary drilling and water was used in all boreholes for circulation. TCR and RQD values of rock formations are also determined and their variations with elevation are presented on borehole log charts (see Appendix I).

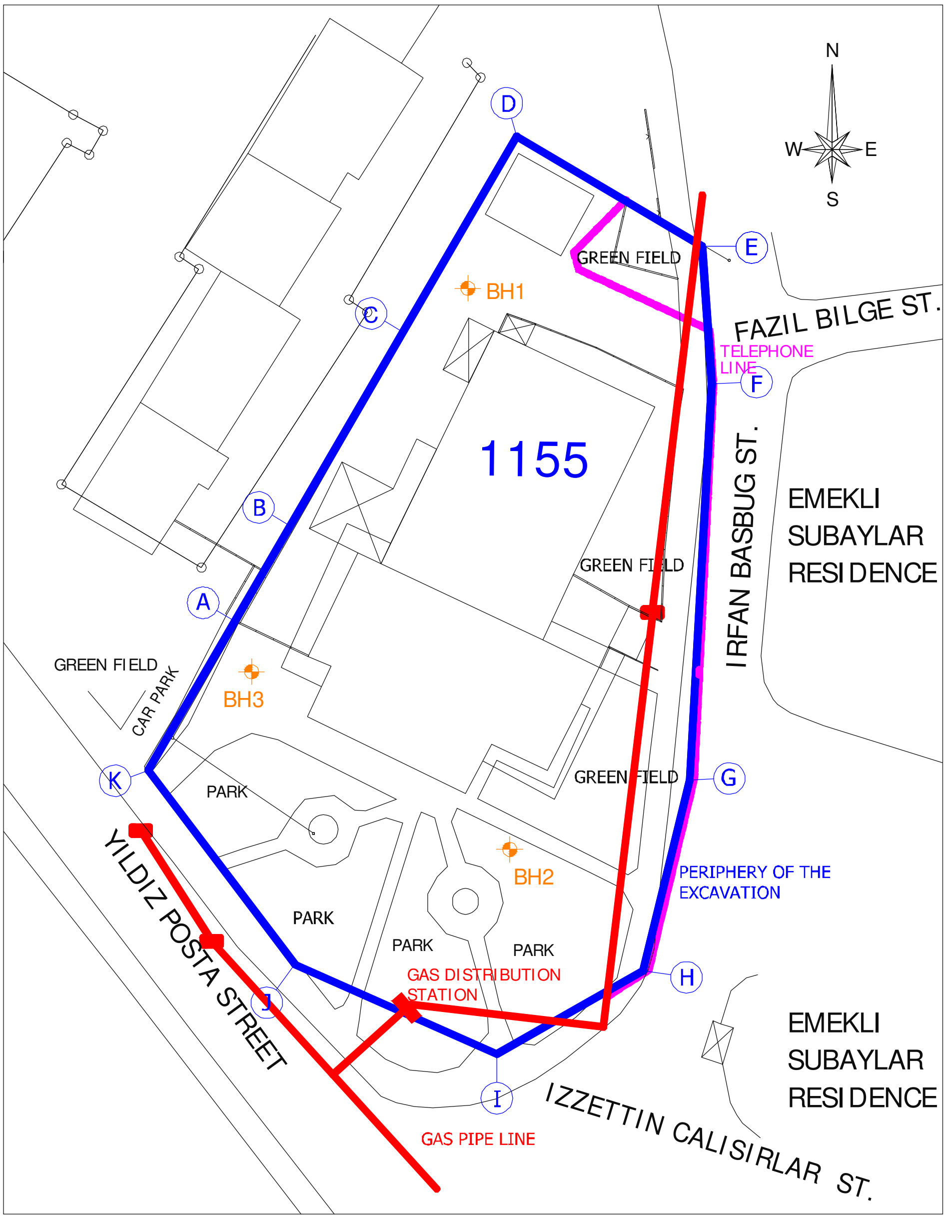


Figure 3.4 The locations of the investigation points and the public facilities on the general layout plan of the site.

According to the “Ground Investigation and Retaining System Interim Report” prepared by ENAR (2006), subsoil of the subject site consists of basically three different formation. The upper level of the subsoil is formed of fill and fully weathered rock of sandstone. The depth of the fill and weathered rock starts from zero ground level and reach to approximately 3 meters. Fills and weathered rock overlie the bedrock greywacke, which is classified as Trakya Formation.

Greywacke is jointed rock with various degrees of weathering and fracturing. Physical and mechanical properties of the greywacke, encountered at the site, are given in Table 3.1.

Table 3.1 Physical and mechanical properties of the greywacke at the project site.

| | |
|---|---|
| Unit Weight | $\gamma_n = 22.0$ kN/m$^3$ |
| RQD% | RQD% < 25% |
| Deformation Modulus | $E_S = 0.08$ GPa |
| Internal Friction Angle | $\phi = 40°$ |
| Horizontal Subgrade Modulus | $k_h = 50{,}000$ kN/m$^3$ |
| Vertical Subgrade Modulus | $k_v = 90{,}000$ kN/m$^3$ |

At borehole BH2, below 19.0 m depth, diabase underlies the greywacke at southeast part of the site. This rock unit consists of grey, slightly weathered, strong diabase. Diabase is jointed and medium grained to coarse grained. Physical and mechanical properties of the diabase, encountered at the site, are given in Table 3.2.

Table 3.2 Physical and mechanical properties of the diabase.

| | |
|---|---|
| Unit Weight | $\gamma_n = 22.0$ kN/m$^3$ |
| RQD% | RQD% = 25% - 50% |
| Deformation Modulus | $E_S = 0.1$GPa |
| Internal Friction Angle | $\phi = 45°$ |
| Horizontal Subgrade Modulus | $k_h = 70{,}000$ kN/m$^3$ |
| Vertical Subgrade Modulus | $k_v = 120{,}000$ kN/m$^3$ |

The groundwater table and any perched groundwater zones must be evaluated as part of a subsurface investigation program. The presence of ground water affects overall stability of the system, lateral pressures applied to the wall, vertical uplift forces on structures (structural design), and drainage system design, watertight requirements at anchor connections, corrosion protection requirements, and construction procedures.

Ground water elevations change -6.80 m (BH3) to -11.70 m (BH2) from west to southeast of the site. The aggressivity of the ground water was not a critical issue for the subject project due to relatively short construction period and the required service life of the anchors. Hereby, no further investigation was performed for ground water characteristics.

In this type of studies, a detailed soil/rock investigation should be made mandatory. However, the studies do not always provide satisfactory information about subsoil conditions which exist at the area behind the retaining wall.

Where the initial investigation suggests that ground conditions may be prone to random variations, data obtained during staged excavation and anchor drilling should be recorded on a daily basis and, where appropriate, samples should be taken. Drill cuttings and soil/rock exposed in the excavation should be visually classified to identify ground which may be susceptible to caving or sliding. All unexpected conditions should be carefully documented. This record should include variations in strata levels, ground types, and conditions that may require design changes and different installation procedures. The performance of test anchors should also be within the scope of the ground investigation, and analyzed with regard to field and laboratory data.

One of the most important issues of the pre-construction phase is the estimation of the underground utility locations. While overhead utilities can be seen and are often moved prior to construction, buried utilities, which are far greater in number, must also be located. Damages and subsequent disruption caused on such utility lines not only represent a significant potential liability to the project participants, it can also be a considerable safety hazard to the personnel directly involved in the construction.

Aside from the disruption to the utility caused by their breakage, utility damage can have a very detrimental effect to the project. Drilling into gas or electric lines can risk fatal injuries for any nearby personnel. Drilling and grouting operations that damage and subsequently fill sewers or waterlines can be very expensive to fix. Damage to any adjacent utility can cause severe disruption to the project schedule while the utility is repaired.

The construction site is located at an old residential area and many of the underground utilities were more than forty years old. Precise location and depth of the utilities and description of their size, shape and type were provided by the Owner to the ZETAS (hereinafter referred to as the Contractor) prior to the design of the retaining system. But some of the data could not be provided by the local authorities to the Owner. Therefore, construction team of the Contractor undertook a utility location survey including opening of manholes to measure the depth and location of inverts of adjacent utilities prior to construction. Also, crack surveys including pictures to record pre-existing conditions of neighbouring structures were conducted.

### *6.2.3 Pre-construction Phase*

#### *6.2.3.1 Retaining System Design*

Success of an excavation is significant to the lives and properties of many people. Thus, appropriate criteria must be selected before the design. A deep retaining system design criteria should include, at least, establishing safety design factors for stability analysis, the simplified and/or advanced deformation and stress analyses, the design of structural components, property protection, a dewatering scheme and selection of the level of corrosion protection.

*6.2.3.1.1 Initial Design.* The first job of excavation support system design is to decide the type(s) of retaining wall. Regarding the close proximity of the adjacent buildings and streets, it was reasonable to design a rigid retaining system to protect these properties during the Point Hotel excavation. One of the most common method of increasing the rigidity of the system is increasing the stiffness or thickness of the retaining wall. Thus, the retaining wall deformation and consequent ground surface settlement can be restricted to acceptable limits.

In the subject project, an initial retaining wall design is made by the Consultant in the scope of "Ground Investigation and Retaining System Interim Report". According to the initial project, excavation support system planned to be constructed consisted of 50 cm thick manual caisson walls with pre-stressed anchors.

Diaphragm walls or cast in-situ bored piles generally pose small deformations, and are therefore mainly used in inner-city excavations. On the other hand, the maximum underground space gain desired by the Owner of this project resulted in permanent basement walls to be very close to the retaining walls. Also, in these two construction methods, machine working tolerances can cause space loss at the adjacent property boundaries. Furthermore, there was lack of space for heavy and bulkier construction machines such as hydrofraise or pile boring machines which are capable of excavating greywacke and diabase. Under these circumstances, manual caisson method seems to be the most appropriate method for construction of thick retaining wall. However, this initial design was not applicable for existing ground condition because of limitations of the excavation method of the manual caisson. This method severely hampered by limitations of depth and by the fact that they cannot be used in hard rock. Especially, caissons cannot be excavated by hand tools in diabase layer.

*6.2.3.1.2 Application Project (the Contractor's Project).*Considering construction limits of the manual caisson method, the Contractor prepared an alternative anchored wall project, under the same design requirements and limits. Cast in-situ segmented reinforced concrete wall with pre-stressed anchors was chosen as the retaining system of the basement excavation of the hotel building. According to the contractor project, excavation support system planned to be constructed consisted of 30 cm thick cast in-situ segmented reinforced concrete (RC) walls with pre-stressed anchors. System was chosen based on the in-situ and laboratory tests carried out for the subject site, together with the previous experiences in similar subsoil conditions and practice.

In comparison with the other methods, segmented RC wall was the most optimized method for the subject project in many ways. When compared with manual caisson method, deeper excavations can be conducted in the rock by this method. Also, this method is highly preferred when there is lack of space for construction equipment (i.e. diaphragm wall machine or pile boring machine).

However, segmented RC walls have some disadvantages. Segmented RC walls are constructed top to down in continuous stages. During the construction stages the cut face of the rock formations are directly exposed to water and atmosphere in accordance with the construction methodology. In such cases, some rock formations as well as greywacke, can be further weathered and loose their integrity during stand-up time. As a result, material losses can cause sudden movements. Additionally, excessive concrete use is inevitable to fill these cavities. For these reasons, 25 cm diameter micro piles were planned to construct through the wall prior to the excavation stages. The micro piles planned to be constructed were located just behind the wall at 1 m intervals.

On the other hand, micro pile construction technique has some limitations. Vertical deviations increase very fast and disproportionably after 12 m. Furthermore, casting becomes a serious problem with depth especially in small diameter micro piles. Because of these limitations excavation system was designed in two-steps. The first step was planned to be constructed from ground level to +117.0 m. Once this level was reached, second stage micro piles were planned to be constructed 1 m away from the first stage piles. Finally, cast in-situ reinforced walls continued down to the final level (i.e. bottom of the excavation).

In order to design a rigid retaining system, stiff support members must be designed. Regarding the geometry of the construction area, depth of the excavation, transportation of the excavated material and the excavation technique, pre-stressed anchors were preferred over braces.

### *6.2.4 Construction Phase*

#### *6.2.4.1 Demolition and Site Preparation*

The site was handed over to the contractor on 14th day of February. However, construction site was not proper to start micro pile construction. Demolition of the existing building and boundary walls was still going on. Figure 3.5 shows site preparation works. Finally, the walls were demolished along the ABCD axis and the DE axis and micro pile construction started on 1st of March, 2007. However, demolition works were carried out until 20th of March while micro pile construction was going on (see Figure 3.6).

Figure 3.5 General site condition and site preparation works.

Figure 3.6 Demolition and site preparation works going on concurrently with micro pile construction.

#### 6.2.4.2 Site Establishment

Site establishment on an excavation project includes project preliminary items and preparations. The careful consideration of site establishment can have a considerable influence on the efficient and effective undertaking of site works.

Construction planning and management also includes organization of labour transportation and/or accommodation plan and establishment of the construction site facilities. Generally, working times are not regular in excavation works. Some of the activities frequently extend beyond the normal working time. For example, grouting activities sometimes start after entire daily anchor drilling and anchor installation is completed. For these reasons, site accommodation is generally preferred over labour transportation.

However, construction site accommodation is one of the most challenging tasks that the site manager should overcome. Especially, in the inner city excavation works, the area used for site camp and other social and construction facilities are generally very limited. Under these circumstances, the physical conditions in construction site can sometimes be quite poor, with inadequate facilities being provided on short term works.

The Point Hotel excavation boundaries extended very close to the fencing. Therefore, there was not too much area for site facility establishment even for site office. Only at the south side of the site, very limited municipally owned land was leased from local authority by the owner.

This area was very tight to place all facilities. As result, container type facilities including site office container, labour accommodation containers and storage container were replaced on top of the other. This setting was very poor by means of comfort and safety. Figure 3.7 shows the position of the site facilities and the site entrance.

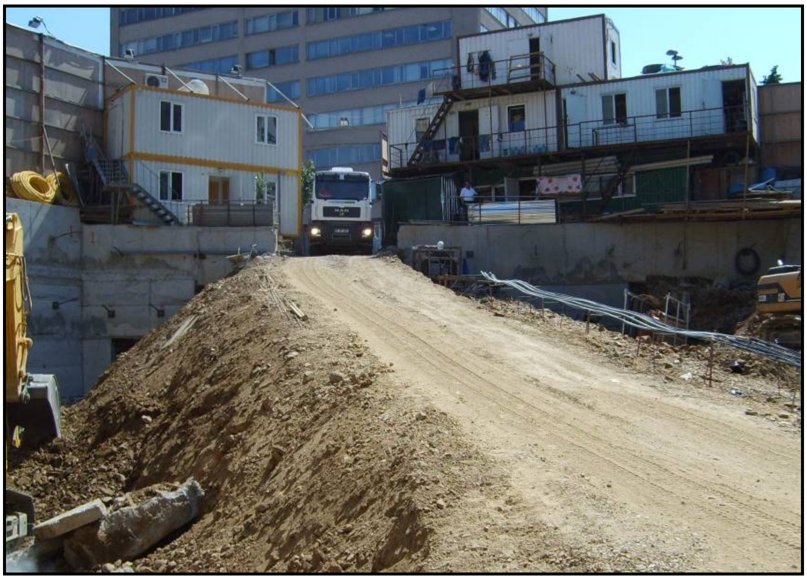


Figure 3.7 The position of the site facilities and the site entrance.

#### *6.2.4.3 Construction and Excavation Works*

*6.2.4.3.1First Construction Period (March 1 - April 12).* Excavation works started with micro pile construction on the 1st day of March as mentioned before. However, there was a gas pipeline along the Yildiz Posta Street and (EFGHI axis) and the Irfan Basbug Street (IJK axis). This pipeline was located inside the construction site. Additionally, a gas distribution station was located at the south of the site (nearly at the middle of the IJ axis). Because of these facilities, retaining system construction started only at two sides (KABCD axis and DE axis) of the site. The total length of these two sides was about 60 m. This means that the works could start in only one third of the site. Figure 3.5 shows the location of the utility lines on the general layout plan of the site.

Construction of the micro piles, RC walls and anchors at these sides continued about forty days. In this period, two stages were completed at these two sides. Figure 3.8, 3.9, 3.10 and 3.11 show construction stages of the retaining system. However, no more construction was conducted because of the site conditions. Forty two days later, construction stopped because of the gas pipe line transfer works.

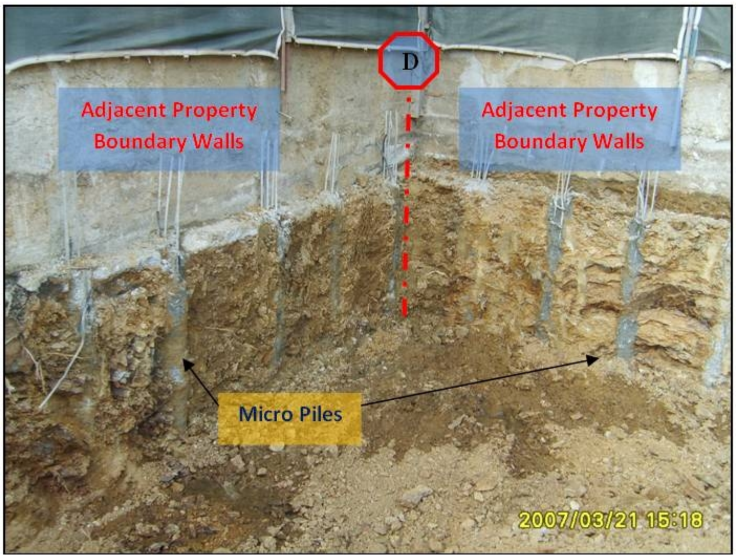


Figure 3.8 Completed micro piles.

Figure 3.9 Placing of plywood formwork panels after reinforcement installation.

Figure 3.10 Formwork support installation.

Figure 3.11 First stage reinforced concrete wall.

*6.2.4.3.2Stand-by Period (April 13 - May 1).* The owner organized a meeting to situation assessment just after the works stopped. According to progress, only (244 m$^2$ / 5100 m$^2$≅) 5% of the construction works were completed (see Figure 3.12). Nonetheless, 35% of the construction time elapsed. The local authority could not give a certain date of transfer works completion making things even worse.

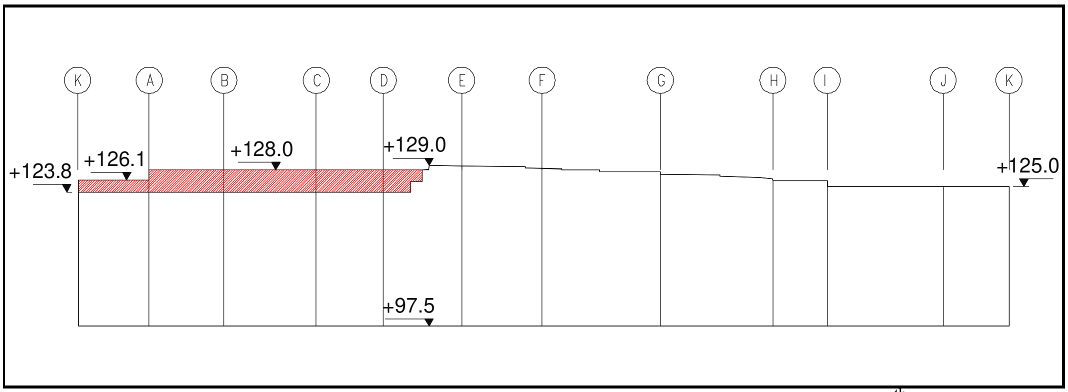


Figure 3.12 Completed retaining wall constructions (red hatched area) on the 12th day of April.

During the stand-by, the owner organized a number of meetings with the contractor, the architect and the consultant to discuss a new design of the basement facilities of the hotel and related retaining system design. As a consequence, the project was revised due to that unforeseen loss of time.

*6.2.4.3.3Revised Project.* According to the contract and the application project, the construction time of the retaining system was six months for nearly 5,100 $m^2$ RC walls with more than 20,000 m pre-stressed anchors. However, excavation works were obstructed by underground utility transfer works at the beginning of the construction time. Transfer works took longer time than expected and caused loss of valuable project time. These unforeseen delays caused to make provisions for construction schedule. As a result, the project was revised to meet the project deadline.

Two basement floors were cancelled and excavation depth was reduced by about 7 m to recover the time lost. These cancellations caused too much underground space loss. As a consequence, it was planned to cancel the second step micro piles and construct the RC walls in the same vertical position with the first step walls to recover some underground spaces. On the other hand, it was possible to increase concrete losses caused by excavation without pre-support. Finally, advantages and disadvantages of the revised design were evaluated and accepted. The typical cross-section of the revised project is presented in Figure 3.13.

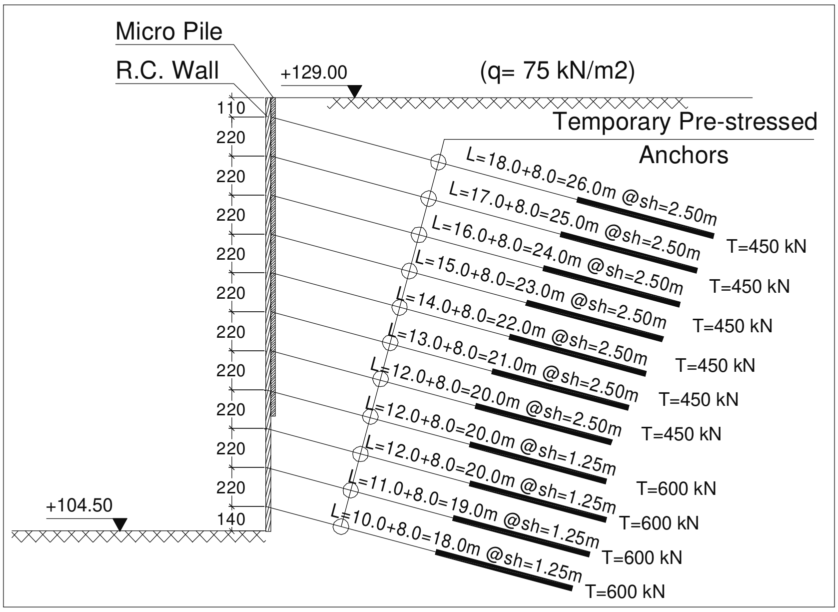


Figure 3.13 Detailed typical cross-section of Section-1 according to the revised design.

*6.2.4.3.4Second Construction Period (May 2 - October 10).* Excavation works started again on 2nd day of May, after the gas pipeline transfer works were completed. Micro pile constructions at the remaining boundaries of the retaining wall were completed in 25 days. RC wall construction generally continued in this part until the excavation levels become almost same all over the site. Any additional construction was conducted along the KABCD axis and DE axis during this period. Finally, at the end of the May, construction site was ready for efficient and effective work flow.

Because of the partial hand over of the construction site by the owner, almost two months were lost until the end of the May. However, construction works were conducted effectively in June and July. Almost 50% of the retaining wall construction was completed in this period. A panoramic view of the construction site is presented in Figure 3.14 below.

Figure 3.14 Retaining wall construction progress on the 17th day of July.

Though we can choose the most reasonable methods based on geotechnical theories, geological conditions, and neighbouring property conditions, an excavation analysis does not always predict the excavation behaviour exactly because (especially anchored wall project) geological investigation may not cover all kinds of soils/rocks and discontinuities to be encountered during excavation and because the simulation of the excavation process may not be thorough.

For these reasons, the performance of the retaining system is closely monitored by means of inclinometer recordings taken at certain time intervals in parallel to the staged excavation. Readings from seven inclinometers at different locations were recorded through out the Point Hotel construction. Figure 3.15 shows the location of the inclinometer boreholes on the general layout plan of the site.

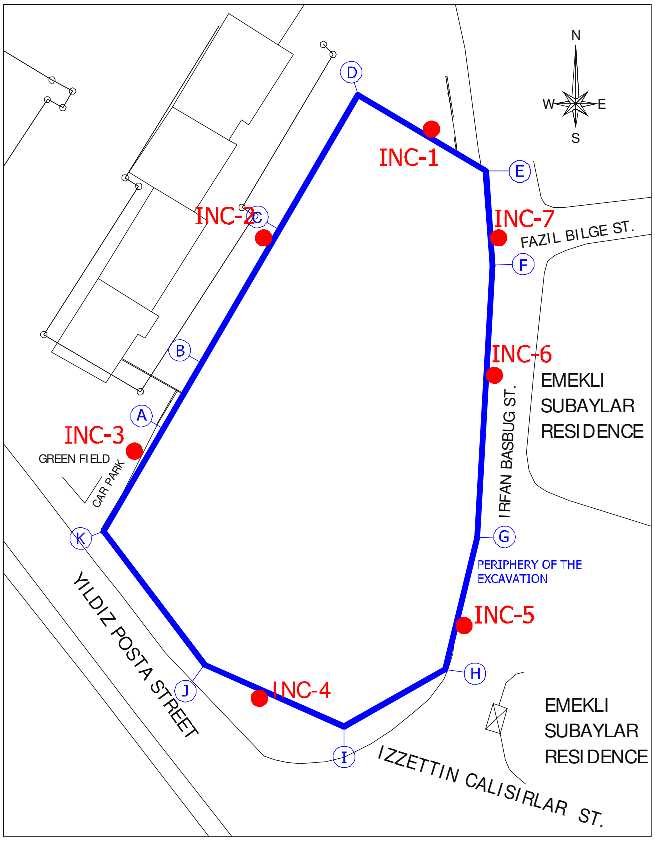


Figure 3.15 The locations of inclinometer boreholes on the general layout plan of the site.

Inclinometer readings must always be compared to pre-defined average and maximum values. When the measured value exceeds the maximum tolerable value, it follows that the excavation is under a certain abnormality. Under this circumstance, necessary measures must be taken to restore the stability.

There is no generally accepted maximum lateral wall movements for anchored walls constructed in rock in the literature. Hereby, the values specified for sands and stiff clays can be used. According to FHWA (1999), the lateral average and maximum wall movements for anchored walls constructed in sands and stiff clays are; 0.2%H and 0.5%H respectively, where H is the height of the wall.

All inclinometer readings taken from the site was within the acceptable limits until the end of July. Figure 3.16 and Table 3.3 present the excavation depths and inclinometer readings taken until the end of the July respectively.

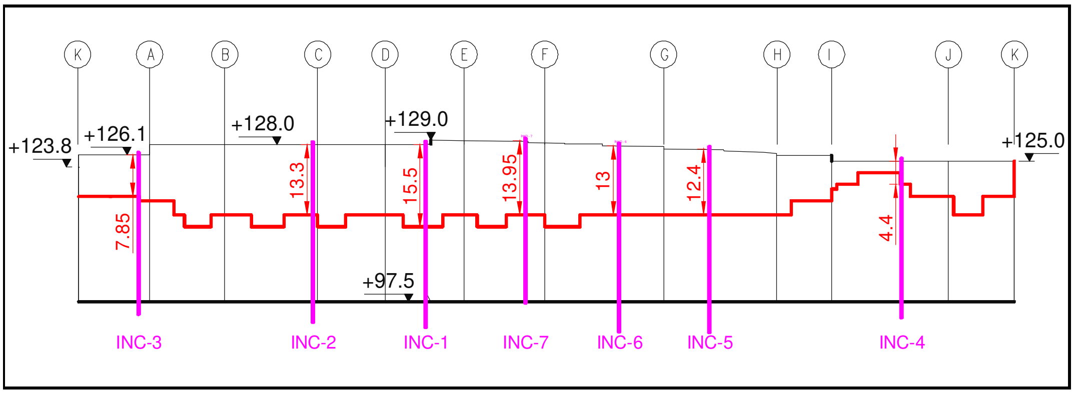


Figure 3.16 Excavation depths (red line) at the end of the July.

Table 3.3 Observed lateral movements at the end of July.

| | **INKLINOMETERS** | | | | | | |
|---|---|---|---|---|---|---|---|
| | **INC-1** | **INC-2** | **INC-3** | **INC-4** | **INC-5** | **INC-6** | **INC-7** |
| **Excavation Depth. H (m)** | 15.5 | 13.3 | 7.9 | 4.4 | 12.4 | 13.0 | 14.0 |
| **Observed Lateral Movement (mm)** | 10.5 | 7.8 | 6.3 | 5.4 | 14.1 | 13.8 | 13.3 |
| **Average Allowable Lateral Movement (0.2%H) (mm)** | 31.0 | 26.6 | 15.8 | 8.8 | 24.8 | 26.0 | 27.9 |
| **Maximum Allowable Lateral Movement (0.5%H) (mm)** | 77.5 | 66.5 | 39.5 | 22.0 | 62.0 | 65.0 | 69.8 |

In the segmented cast in-situ RC wall method, all activities are conducted parallel to excavation. As a consequence, working platforms and access roads change frequently. Furthermore, several activities included, excavation, drilling, reinforcement installation, formwork and concreting, are generally conducted at various locations and elevations of the site at the same time. Hereby, construction teams always move from some place to another. Additionally, storage area and anchor fabrication area must always be maintained. On the other hand, the access road becomes steeper as the excavation gets deeper.

At the beginning of August, the construction speed increased along JK and KAB axis. By this means, the access road of the site was intended to be maintained. Figure 3.17 shows the planned access road on the layout. Thereby, trucks, concrete mixer trucks, mobile concrete pumps could be used effectively inside the construction site.

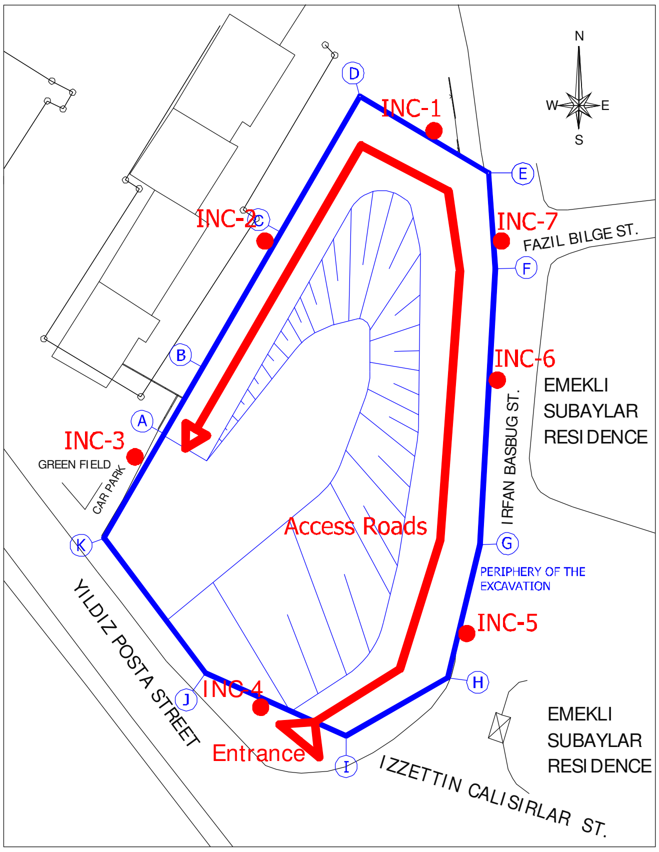


Figure 3.17 The Access roads and entrance of the site.

However, some sudden lateral movements were observed at the third inclinometer (INC-3). The readings taken from INC-3 are presented in Figure 3.18.

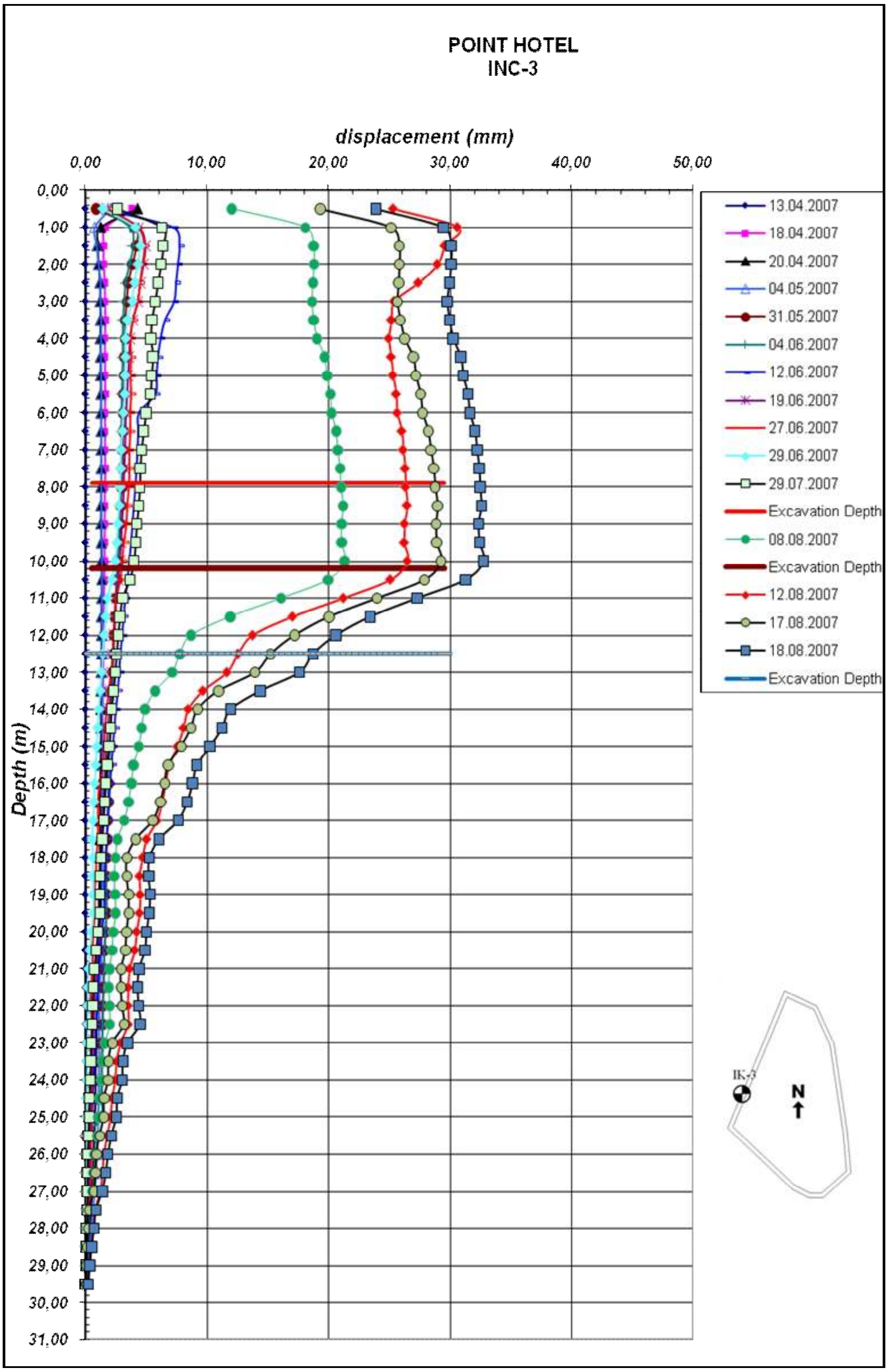


Figure 3.18 Inklinometer readings taken at INC-3.

In general terms, pre-stressed anchors are designed to limit lateral movements of retaining structure. However, some sudden lateral movements were observed at the third inclinometer (INC-3). In fact, recorded movements were within the acceptable limits. However, during the anchor drillings some weathered and water absorbed zones (discontinuities) were observed between sandstone and siltstone zones of greywacke. Furthermore, some thin cracks were observed on the road behind the wall.

Considering these observations, the Contractor decided to install additional long anchors at this part of the wall. Therefore, an additional support project was prepared. For this purpose, suspected slip surface was determined based on drilling records, inclinometer readings and the location of the cracks. Additional three anchor rows were planned. Figure 3.19 shows suspected slip line and the planned anchors in cross-sectional view.

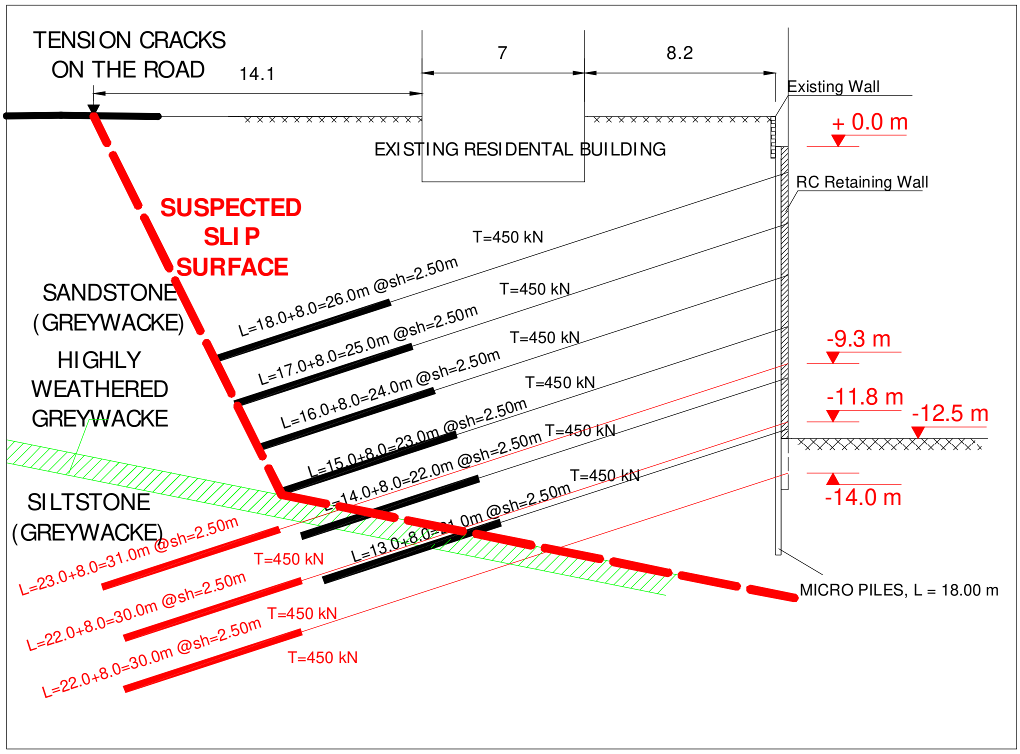


Figure 3.19 Suspected slip surface and additional anchors.

Twelve additional anchors were installed for each row. Totally 36 anchors were installed along the KA and BC axis. Figure 3.20 shows anchor locations. These sides were backfilled to the level of -10 m for anchor installation.

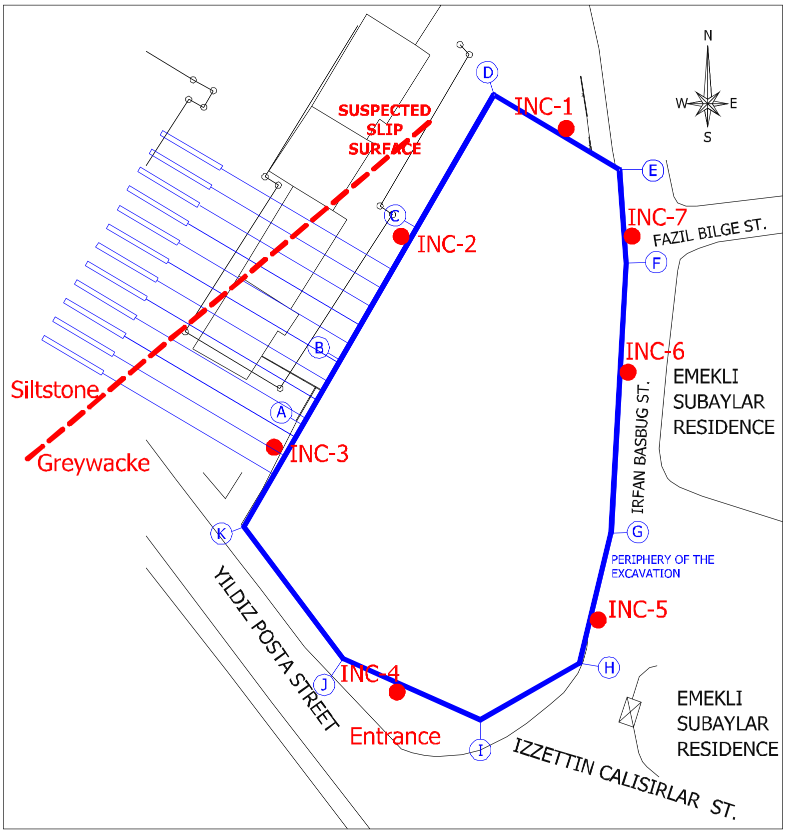


Figure 3.20 Location of the additional anchors.

The backfilling and additional anchor installations took nearly 20 days. During the additional anchor works excavation continued only along the CD, DE and EFG axes. As a result, completion and handover programs were revised. According to revised program, excavation and retaining systems were planned to be completed in two parts.

However, this revised program had some disadvantages. First of all, the access road had to be cut. As a result, all activities especially concreting would be more difficult. These parts are illustrated in Figure 3.21.

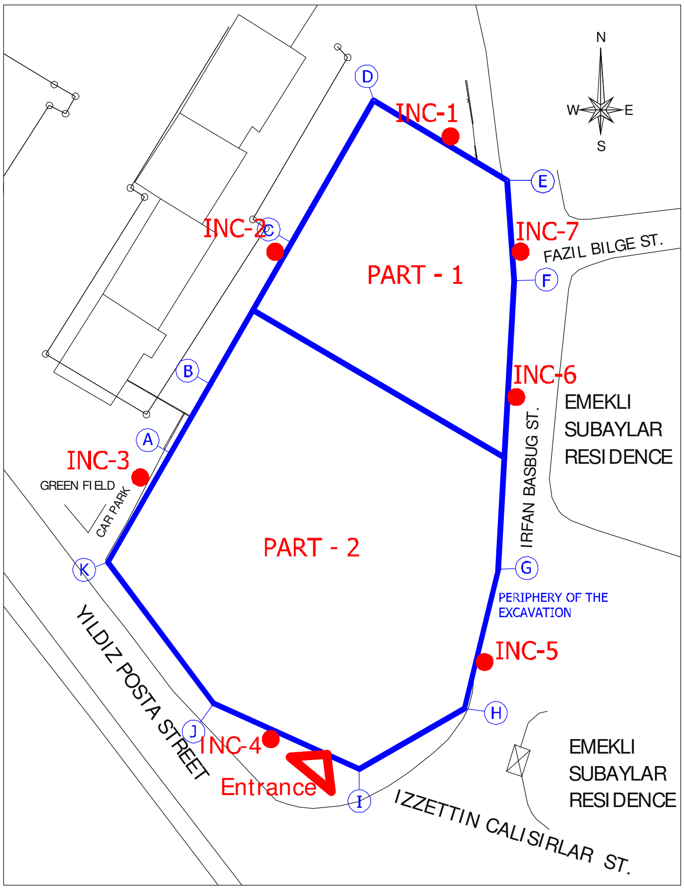


Figure 3.21 The partial completion program.

Additional anchor installation works were completed 5th of September. Inclinometers were recorded on daily basis during the construction. The readings taken between 8th of August and 6th of September are presented in Figure 3.22. Contrary to expectations, the records were evaluated as the additional supports (i.e. three levels of additional anchors) were not effective and did not contribute to the system.

In fact, safety factor of the system calculated using finite element analysis was more than two. Moreover, when compared to the data given in the Table 3.2, the mechanical parametric values of the greywacke that are used in FEM analysis are very conservative. As a result, the application project was almost over-designed.

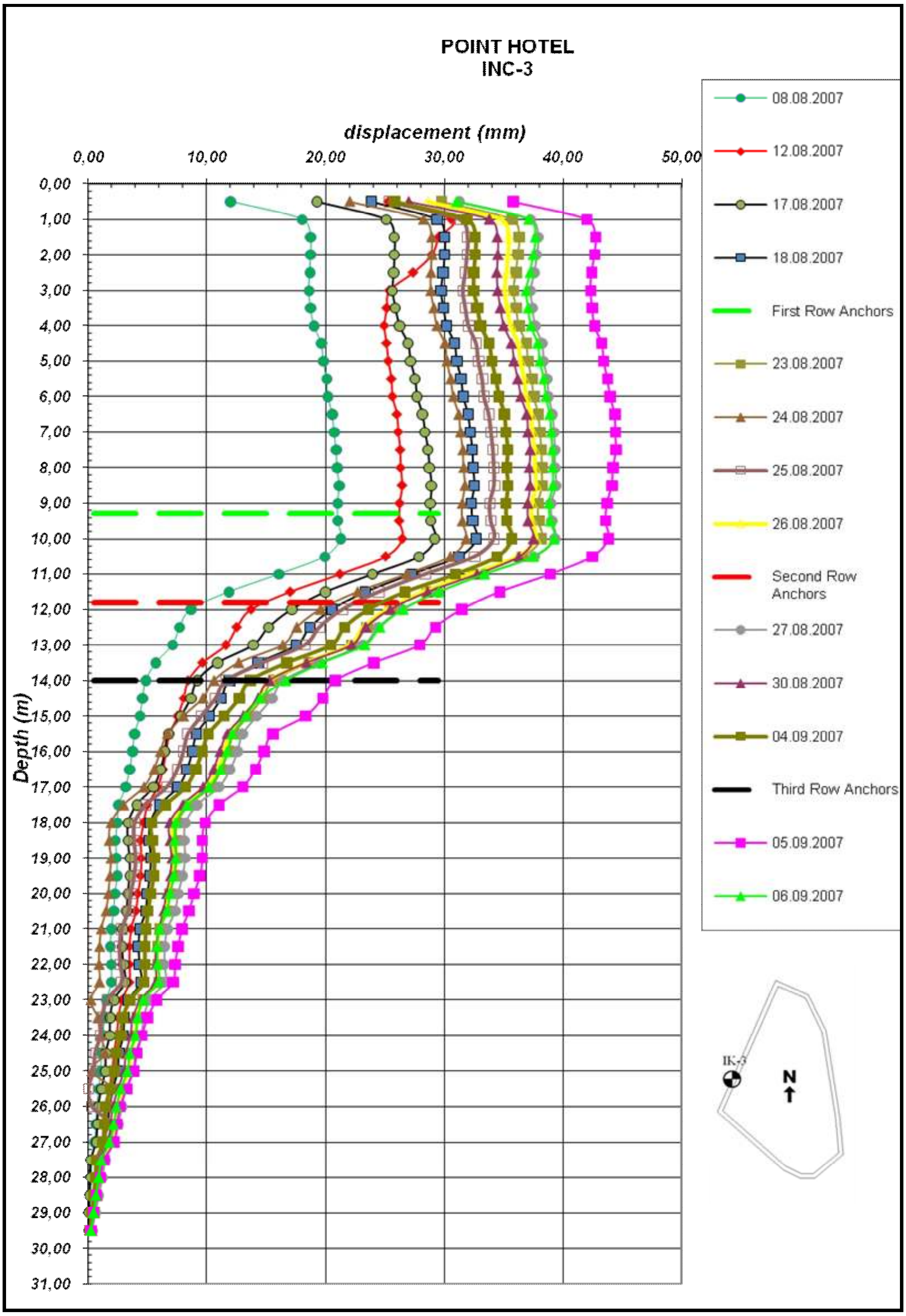


Figure 3.22 Inclinometer readings taken at inc-3 between 8th of August and 5th of September.

Considering the schedule, the Contractor had to take an urgent action. Steel pipe braces were planned to be used. A support project was prepared urgently. The braced support project is presented in Figure 3.23.

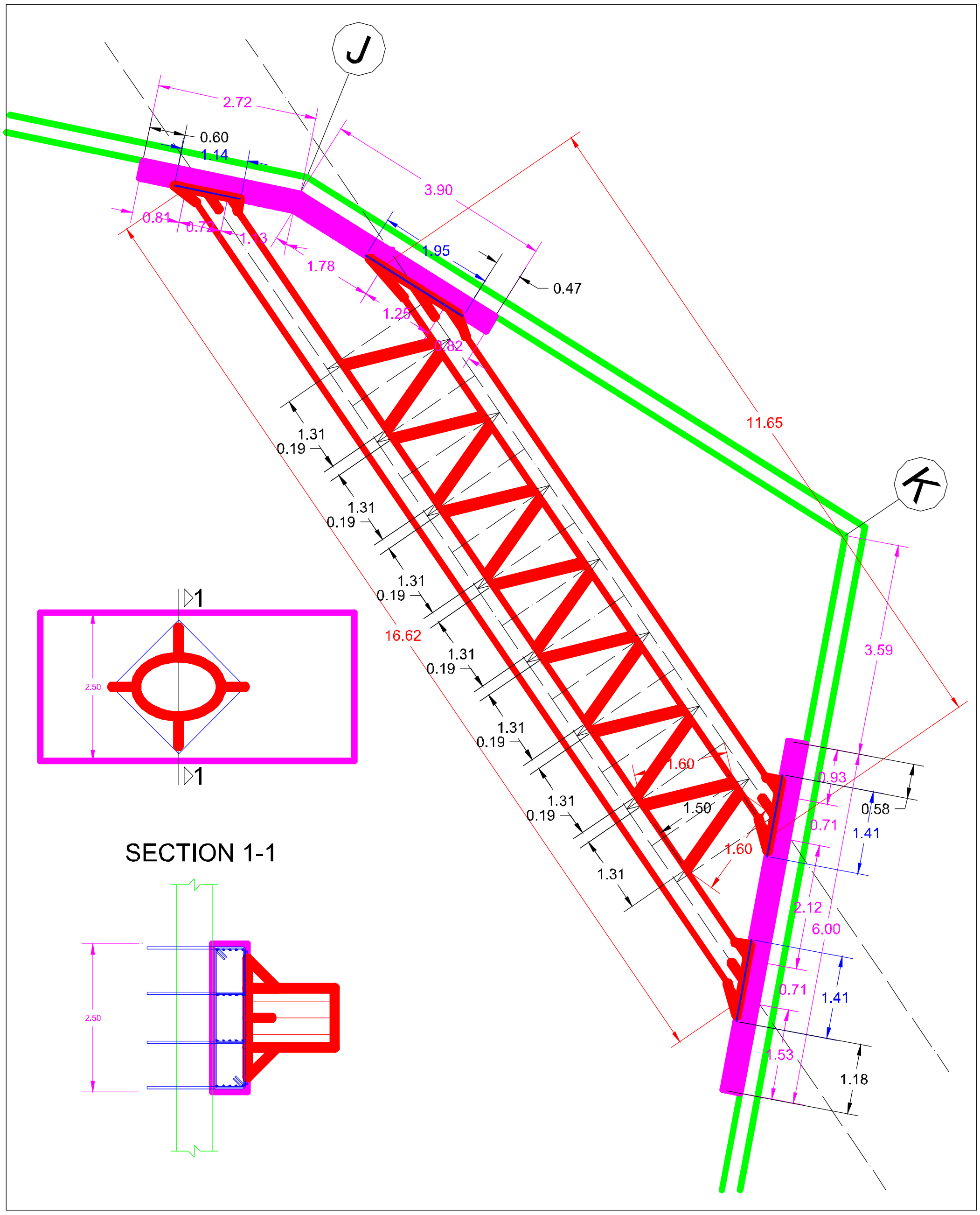


Figure 3.23 Stell pipe braced support project.

Steel pipe welding works were completed in 10 days. However, no RC wall construction was performed except Part-1. That is because; a working platform was prepared for mobile concrete pump and concrete mixer trucks. Remaining area except Part-1 was under the slopes of this platform.

Finally, on the 10th of October, excavation works on Part-1 was completed and area was handed back to the Owner. The contractor left the site obeying the Owner's request until the 1st of November. In this period the superstructure contractor constructed the foundation of the Part-1. Figure 3.24 and Figure 3.25 show the condition of the Part-1 just before and just after the construction was completed.

Figure 3.24 Construction of the last stage walls at the Part-1.

Figure 3.25 Part-1 just after the retaining walls were completed.

*6.2.4.3.5Third Construction Period (November 1 - December 17).*Excavation works started for the third time on the 1st of November. Remaining works were completed in nearly one and a half month. During construction, inclinometer readings were taken rarely parallel to the excavation. However, during the Part-2 constructions all movements were inside the acceptable movement limits. Figure 3.26 shows a panoramic view of the Part-2 one month later the handover.

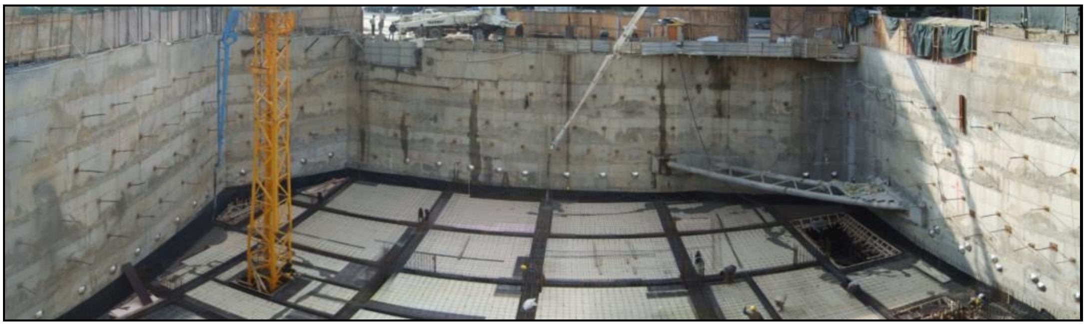


Figure 3.26 Point Hotel construction works after excavation.

## 6.3 The Views of the Author on the Causes of Lateral Movements Observed During Excavation

The initial design of the subject project may be regarded as somehow very safe. However, excessive lateral movements were observed during early stages of excavation. The Author believes that the main causes of the sudden movements observed during excavation are unforeseen discontinuities that were not noticed during site investigation. The retaining system did not comply with the nature of the weathered rock in the site.

As mentioned in the third, fourth and fifth chapters, the behaviour of jointed rock masses such as greywacke is highly complex and does not fit into traditional analysis methods. The effects of discontinuities including joints, weathered zones, etc. on rock mass behaviour during excavation should be taken into consideration during analysis, design and construction phases. However, it is commonly accepted in civil-geotechnical community that greywackes shall be modelled as a continuum mass without discontinuities. As a result, during the construction of the retaining systems some unexpected problems are encountered.

Generally, fully grouted rock nails are used with shotcrete facing (passive - non-tensioned anchors) in such cuts in greywackes. These nails are used to prevent surface rock from loosening, to prevent sliding on interconnected discontinuities and to prevent sliding of relatively intact rock on a thin shear zone. As a result these nails reinforce rock by increasing mechanical properties from cut face of the excavation to the end of nails.

However, pre-stressed anchors have some important distinctions compared to the passive anchors. The most crucial distinction is that pre-stressed anchors are not used to reinforce rocks, i.e. strand tendons are not used to resist shear forces. Pre-stressed anchors are only intended to take tension forces and generally used for supporting excavations in soils. Contrarily, tension forces can make the discontinuities (joints, faults, etc.) larger, deeper and thicker. As a result, mechanical properties of discontinuities and rock mass can decrease. Additionally, because of the extension, water can penetrate into discontinuities and water pressure can develop.

The Author thinks that one of these issues has lead to movements in the investigated case history. As a consequence, pre-stressed anchors may not be convenient for jointed rocks. This subject should be investigated more comprehensively.

# CHAPTER SEVEN

# CONCLUSIONS and RECOMMENDATIONS

Deep excavations and construction of retaining structures became compulsory in order to build various number of basement levels. Many factors should carefully be considered to select the most appropriate excavation method. These factors includes, depth of cut, area of construction site, subsoil profile and engineering characteristics of soil and/or rock formations, groundwater profile, construction budget, allowable construction period, existence of adjacent buildings, availability of construction equipment, conditions of adjacent buildings, foundation types of adjacent buildings.

While becoming the biggest metropolitan city of the region, the value of the land became a major part of the cost in construction of buildings in the city of Istanbul. Consequently, the need for construction of high-rise buildings and shopping malls with multiple basement levels increased noticeably. Therefore, deep excavation and retaining structures which commonly reach from 25 to 40 meters below the ground surface have been constructed especially in the last decade.

One of the most specific common traits of the deep excavation projects in the city of Istanbul is that they are generally constructed in greywackes, a rock type frequently encountered in Istanbul construction sites. There are different types of greywackes with different lithological and physico-mechanical properties in Istanbul and surrounding areas. For this reason, all engineers who are working on the surface and/or subsurface excavations in Istanbul should have knowledge about the physico-mechanical properties of the greywackes.

Both porosity and water absorption increase and dry unit weight decreases with increasing weathering grade and the mechanical properties of the greywackes, i.e. strength and modulus, decrease considerably with weathering. The mechanical properties of the greywackes, i.e. strength and modulus, decrease considerably with weathering (Tugrul and Undul, 2006).

The bearing capacity of the unweathered greywackes is generally high. However, deep excavations (i.e. deeper than 3 metres) should be supported; otherwise it can result in stability problems (Saglamer, 1985). For example, during the construction stages of the excavation the cut face of the rock formations could be directly exposed to water and atmosphere in accordance with the construction methodology. In such cases, greywacke may get weathered and loose its integrity during stand-up time and material losses can cause sudden movements. As a consequence, densely fractured and weathered greywackes should be investigated in detail.

Contemporary numerical methods should be employed in deep cuts made in greywackes considering the fact that material behaviour is highly complex and does not fit into traditional analysis methods.

In excavation and retaining system analysis with Finite Element Method (FEM), proper understanding of the constitutive material models (for soils and rocks) is important in order to produce safe and economical design. Incorrect use of material models leads to incorrect predictions of deflection, bending moment and forces in the design. As a result, project may be costly due to over design or could result in a catastrophic failure due to under design.

Rock masses encountered during excavations in the City of Istanbul such as greywackes are generally jointed. The collective term for the whole range of mechanical defects such as joints, bedding planes, faults, fissures, fractures and joints is discontinuity (Priest, 1993).

A clear distinction must be made between the rock element or rock material on the one hand and the rock mass on the other. Rock material is the term used to describe the intact rock between discontinuities. The rock mass is the total in situ medium containing bedding planes, faults, joints, folds and other structural features. Rock masses are discontinuous and often have heterogeneous and anisotropic engineering properties (Brady & Brown, 2004).

The mechanical behaviour of the rock masses depends on the material properties of the intact rock itself, the joint geometry (roughness), the joint genesis (tension or shear joints) and the joint fill material (Natau, 1990).

The processes and interrelations involved in determining the behaviour of the rock surrounding an excavation is sometimes so complex that it is not amenable to engineering analysis using existing techniques. In these cases, design decisions may have to take account of previous experiences. In an attempt to quantify these experiences so that it may be extrapolated from one site to another, a number of classification schemes for rock masses have been developed.

The most recent scheme is currently widely used in civil engineering and in mining practice is GSI system introduced by Hoek (1994) and developed further by Marinos and Hoek (2000). The Geological Strength Index (GSI) was developed specifically as a method of accounting for those properties of a discontinuous or jointed rock mass which influence its strength and deformability. Using schemes and with some experience, the GSI may be estimated from visual exposures of the rock mass or borehole core.

The Hoek-Brown failure criterion, which assumes isotropic rock and rock mass behaviour, should only be applied to those rock masses in which there is sufficient number of closely spaced discontinuities, with similar surface characteristics, where isotropic behaviour involving failure on discontinuities can be assumed. When the structure being analyzed is large and the block size is small in comparison, the rock mass can be treated as a Hoek-Brown material.

Where the block size is of the same order as that of the structure being analyzed or when one of the discontinuity sets is significantly weaker than the others, the Hoek-Brown criterion should not be used. In these cases, the stability of the structure should be analyzed by considering failure mechanisms involving the sliding or rotation of blocks and wedges defined by intersecting structural features.

A rock mass can be said to be continuous if it consists of either purely intact rock, or of individual rock pieces that are small in relation to the overall size of the construction element studied. A special case is when one discontinuity is significantly weaker than any of the others within a volume, as when dealing with a fault passing through a jointed rock mass. In such a case the continuous rock masses and the fault have to be treated separately.

Rock mass can be generally classified into three groups, i.e., (a) continuous - refers to intact rock mass, (b) discontinuous – refers to jointed rock mass and (c) pseudo continuous – refers to highly fractured or weathered rock mass. The behaviour of intact rock mass can be analyzed by means of model based on continuum mechanics. The discontinuous model may be used for analyzing the jointed rock mass. In order to model the behaviour of discontinuous rock masses completely, special numerical techniques such as the Discrete Element Method (DEM) and Discontinuous Deformation Analysis (DDA) were developed. These methods explicitly model a rock mass as an aggregate of discontinuities and intact material. Discontinuous model similar to that of jointed rock mass can be used for type (c) rock mass. However, it is almost impossible to explore all the joint systems and it seems that this type of rock mass behaves just like a continuous body in a global sense. Therefore, a continuum mechanics model can be used with the effect of discontinuities adequately considered in the model. This is achieved by homogenization technique where equivalent continuum properties of the rock mass are derived based on the geometry of the contained fracture systems and physical properties of the intact rock matrix and the fractures.

Through the development of special elements – joint elements (sometimes also known as interface elements) – the continuum-based FEM can also be applied to the modelling of discontinuous rock masses. These elements can have either zero thickness or thin, finite thickness. They can assume linear elastic behaviour or plastic response when stresses exceed the strengths of discontinuities.

In the scope of the thesis continuum-based FEM computer program Plaxis 2D V8.2 and Jointed Rock material model is used to simulate jointed rock behaviour of greywacke. Analyses performed in this manner are presented in the following chapter.

The greywackes were identified as a jointed rock with various degrees of weathering by many investigators. However, it is commonly accepted in civil-geotechnical community of Istanbul that greywackes are modelled as a continuum mass without discontinuities.

One of the most widespread approaches of incorporating the influence of discontinuities on rock mass strength in numerical analysis is through modelling of rock masses as continua with reduced deformation and strength properties (Hammah, Yacoub, Corkum & Curran, 2008). However, method based on this approach is not able to model all deformation mechanisms of jointed rocks.

In the content of the thesis, in order to evaluate effects of discontinuities on rock mass behaviour during excavation are investigated by means of parametric study. For that purpose four cases are distinguished with a “jointed rock model” that is available in the commercial Finite Element (FE) program Plaxis, according to joint set number and to whether excavation is supported or not.

The Jointed Rock model of Plaxis is an anisotropic elastic perfectly-plastic (transversely isotropic) model, especially meant to simulate the behaviour of jointed rocks.

In the fifth chapter, a series of numerical analyses have been carried out to assess the effects of the discontinuities on rock mass during excavation. For this purpose two conditions have been numerically simulated for two different jointing degrees for excavation height, H=25.0m. Totally four configurations have been analyzed and result have been compared. These conditions are un-supported excavation and excavation with pre-stressed anchor. Each of these two conditions are analyzed for excavation depth 25m with slope angle 90° and two different jointing degrees (intact rock and rock with three joint sets) are used to assess the effect of the joints.

Rock mass properties of the greywacke determined using RocLab V1.031 (Rockscience, Inc., 2007) which is a free software based generalized Hoek-Brown failure criterion. The intact rock mechanical and elastic properties of the greywackes were determined from the tables presented in the chapter three.

Analysis results indicate the need to perform FEM analysis with the effects of the discontinuities on excavations in jointed rock masses. Such analysis promotes greater understanding of problems. It increases the chances of success through more robust design of excavations, stabilization measures and improved monitoring decisions.

In the sixth chapter, a unique case study, in which the author involved throughout design, construction and monitoring phases as site manager, has been presented. The project is a special and challenging project because of the soil and rock stratigraphy at the project site, construction methodology of the excavation support system and the location of the adjacent properties, etc. Hereby, the performance of segmented cast in-situ reinforced concrete retaining wall system with pre-stressed ground anchors has been examined under difficult soil and jointed rock conditions and limitations.

The performance of retaining systems should always be monitored. The performance of the retaining walls was monitored by inclinometer recordings taken at certain time intervals in parallel to the excavation at various locations. These inclinometer readings have been compared to pre-defined average and maximum values. During excavation, rapid lateral displacements were monitored at one inclinometer (INC-3). In fact, recorded movements were within the acceptable limits. However, during anchor drillings some weathered and water absorbed zones were observed between siltstone and greywacke zones.

Considering the rapid lateral movements and the cracks on the road that was necessary to take measures on the time. Suspected slip surface is decided with observations including anchor drilling records. Totally 36 longer ground anchors were installed in three rows in very short time period. Anchor lengths were determined considering the suspected slip line.

Totally 36 additional anchors were installed along the retaining system. Additional anchor installation works were completed in a short period of time. Inclinometers were recorded on daily basis during the works. Contrary to expectations, the records were evaluated as the additional supports were not effective and did not contribute to the system.

Considering the schedule, the Contractor take an urgent action and steel pipe braces were used. Remaining works were completed in nearly one and a half month. During construction, inclinometer readings were taken rarely parallel to the excavation. However, during the constructions all movements were inside the acceptable movement limits. It was observed that internal rigid bracing system was much more successful in limiting deformations than pre-stressed anchors.

The design of the subject project appeared to be very safe. However, excessive lateral movements were observed during early stages of excavation. The Author believes that the main cause of the sudden movements observed during excavation is the fact that pre-stressed anchors were insufficient to limit the deformations probably because they resulted in further opening of the already present discontinues by applying tension forces at the boundaries.

Generally, fully grouted rock nails are used with shotcrete facing (passive - non-tensioned anchors) in such cuts in greywackes. These nails are used to prevent surface rock from loosening, to prevent sliding on interconnected discontinuities and to prevent sliding of relatively intact rock on a thin shear zone. As a result these nails reinforce rock by increasing mechanical properties from cut face of the excavation to the end of nails.

However, pre-stressed anchors have some important distinctions compared to the passive anchors. The most crucial distinction is that pre-stressed anchors are not used to reinforce rocks, i.e. strand tendons are not used to resist shear forces. Pre-stressed anchors are only intended to take tension forces and generally used for supporting excavations in soils. Contrarily, tension forces can make the discontinuities (joints, faults, etc.) larger, deeper and thicker. As a result, mechanical properties of discontinuities and rock mass can decrease. Additionally, because of the extension, water can penetrate into discontinuities and water pressure can develop.

The Author thinks that any one or probably more than one of the above mentioned mechanisms have lead to movements in the investigated case history. As a consequence, it may be stated that pre-stressed anchors may not be convenient for jointed rocks. This subject should be investigated more comprehensively.

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# APPENDICES

# APPENDIX A

# BOREHOLE LOG CHARTS

**YER :** BALMUMCU-NİPPON OTEL
**KUYU NO :** SK1
**EKİPMAN :** GMS-300
**ZEMİN KOTU :** 0.00 m.
**SONDAJ YÖNTEMİ :** ROTARY : 0.00-33.50m arası.
**KOORD. : N :** **E :**
**KUYU ÇAPI :** 0.00-31.50m arası - 89mm.
**BAŞLANGIÇ :** 29.07.2006 **BİTİŞ :** 01.08.2006

| NUMUNE VE YERINDE DENEY. Drn. (M) | TİP | S.P.T. darbe sayısı 15 | 7.5 | 7.5 | 7.5 | 7.5 | Muh. Drn. (m) Tarih | Y.A.S. Drn. (m) | TCR % | RQD % | SCR % | Drn. (m) | ZEMİN CİNSİ | KOT (m) | LEJAND |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 29.07.20 | | | | | 0.15 | Beton zemin | -0.15 | |
| | | | | | | | | | | | | 1.00 | Dolgu zemin | -1.00 | |
| 1.50<br>1.60 | D1 | 50/10 | | - | | - | | | | | | | Çok sıkı, kahve renkli, killi, çakıllı KUM, yer yer çok zayıf çimentolu<br>(Tamamen ayrışmış Kumtaşı) | | |
| 3.00<br>3.05 | D2 | 50/5 | | - | | - | | | | | | 3.20 | | -3.20 | |
| 3.20 | | | | | | | | | 100 | 8 | | | | | |
| 4.50 | | | | | | | | | 100 | 7 | | | | | |
| 6.00 | | | | | | | | | 100 | 20 | | | | | |
| 7.50 | | | | | | | | YAS<br>▼<br>7.70 | 100 | 30 | | | Orta zayıf-orta sağlam, kahve-gri renkli, iri daneli KUMTASI ,çok-orta derecede ayrışmış<br>(8.80-9.30, 10.00-10.90, 12.10-12.80 m derinlikler arası ince daneli)<br>(8.00-8.50, 10.00-10.50, 11.30-11.60, 12.30-12.60, 14.50-15.00 m derinlikler arası yer yer tamamen ayrışmış-killeşmiş) | | |
| 9.00 | | | | | | | | | 100 | 7 | | | | | |
| 10.50 | | | | | | | | | 100 | 0 | | | | | |
| 12.00 | | | | | | | | | 100 | 23 | | | | | |
| 13.50 | | | | | | | | | 100 | 13 | | 15.30 | | -15.30 | |
| 15.00 | | | | | | | | | 100 | 37 | | | | | |
| 16.50 | | | | | | | | | 100 | 30 | | | Orta sağlam, mavimsi gri renkli, orta-iri daneli, çatlaklı-kırıklı KUMTASI, orta derecede ayrışmış<br>(15.30-15.50, 17.30-17.60, 22.30-22.50, 23.50-23.80 m derinlikler arası çok çatlaklı-çok kırıklı, çok ayrışmış)<br>(18.20-18.40, 21.30-21.95, 22.50-23.00, 25.00-25.50 m derinlikler arası çok çatlaklı-çok kırıklı) | | |
| 18.00 | | | | | | | | | 100 | 7 | | | | | |
| 19.50 | | | | | | | | | 100 | 33 | | | | | |

**NOTLAR :**

**SONDÖR :** İ.KOLUKISA
**LOGU HAZIRLAYAN :** A.ARSLAN
**KONTROL EDEN :** T.ÖZBEK

**YER :** BALMUMCU-NİPPON OTEL **KUYU NO :** SK1

**EKİPMAN :** GMS-300 **ZEMİN KOTU :** 0.00 m.

**SONDAJ YÖNTEMİ :** ROTARY : 0.00-33.50m arası. **KOORD. : N : E :**

**KUYU ÇAPI :** 0.00-31.50m arası - 89mm.
31.50-33.50m arası - 76mm.
**BAŞLANGIÇ :** 29.07.2006 **BİTİŞ :** 01.08.2006

| NUMUNE VE YERİNDE DENEY. | | S.P.T. darbe sayısı | | | | | Muh. Drn. (m) | Y.A.S. Drn. | TCR | RQD | SCR | Drn. | ZEMİN CİNSİ | KOT | LEJAND |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Drn. (M) | TİP | 15 | 7.5 | 7.5 | 7.5 | 7.5 | Tarih | (m) | % | % | % | (m) | | (m) | |
| | | | | | | | 29.07.20 | | 100 | 33 | | | | | ⋯⋯ |
| 21.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 0 | | | Orta sağlam, mavimsi gri renkli, orta-iri danelı, çatlaklı-kırıklı KUMTASI, orta derecede ayrışmış<br>(15.30-15.50, 17.30-17.60, 22.30-22.50, 23.50-23.80 m derinlikler arası çok çatlaklı-çok kırıklı, çok ayrışmış)<br>(18.20-18.40, 21.30-21.95, 22.50-23.00, 25.00-25.50 m derinlikler arası çok çatlaklı-çok kırıklı) | | |
| 22.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 23 | | | | | |
| 24.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 67 | 7 | | | | | |
| 25.50 | | | | | | | | | | | | 25.50 | | -25.50 | |
| | | | | | | | | | 47 | 0 | | | | | ×××××× |
| 27.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 40 | 0 | | | | | |
| 28.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 87 | 0 | | | Orta sağlam, koyu gri renkli, ince-orta tabakalı, çok çatlaklı-çok kırıklı SILTTASI, orta derecede ayrışmış | | |
| 30.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 87 | 13 | | | | | |
| 31.50 | | | | | | | | 31.50 | | | | | | | |
| | | | | | | | | | 93 | 0 | | | | | |
| 33.00 | | | | | | | | | | | | | | | |
| 33.50 | | | | | | | | | | 100 | 0 | | 33.50 | SONDAJ BİTİMİ | -33.50 |

**NOTLAR :**

**SONDÖR :** İ.KOLUKISA **LOGU HAZIRLAYAN :** A.ARSLAN **KONTROL EDEN :** T.ÖZBEK

**GEOS Geoteknik ve Sondajcılık Ltd.**

NİPPON OTEL
JEOTEKNİK ARAŞTIRMALAR

Sayfa 1 / 2

| | |
|---|---|
| **YER :** BALMUMCU-NİPPON OTEL | **KUYU NO :** SK2 |
| **EKİPMAN :** GMS-300 | **ZEMİN KOTU :** 0.00 m. |
| **SONDAJ YÖNTEMİ :** ROTARY : 0.00-35.00m arası. | **KOORD. : N :** **E :** |
| **KUYU ÇAPI :** 0.00-12.00m arası - 89mm.<br>12.00-35.00m arası - 76mm. | **BAŞLANGIÇ :** 02.08.2006 **BİTİŞ :** 04.08.2006 |

| NUMUNE VE YERINDE DENEY. | | S.P.T. darbe sayısı | | | | | Muh. Drn. (m) | Y.A.S. Drn. | TCR | RQD | SCR | Drn. | ZEMİN CİNSİ | KOT | LEJAND |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Drn. (M) | TİP | 15 | 7.5 | 7.5 | 7.5 | 7.5 | Tarih | (m) | % | % | % | (m) | | (m) | |
| | | | | | | | 02.08.20 | | | | | 0.20 | Dolgu zemin | -0.20 | |
| 1.50<br>1.70 | D1 | 42 | | 50/5 | | | | | | | | 3.00 | Çok sıkı, kahve renkli, killi, çakıllı KUM ,yer yer çok zayıf çimentolu<br>(Tamamen ayrışmış Kumtaşı) | -3.00 | |
| 3.00 | | | | | | | | | 33 | 0 | | | | | |
| 4.50 | | | | | | | | | 33 | 0 | | | | | |
| 6.00 | | | | | | | | | 17 | 0 | | | | | |
| 7.50 | | | | | | | | | 30 | 0 | | 9.10 | Orta zayıf, kahve-gri renkli, ince-orta taneli, çok çatlaklı-çok kırıklı, KUMTASI ,çok-yer yer tamamen ayrışmış, killeşmiş | -9.10 | |
| 9.00 | | | | | | | | | 20 | 7 | | | | | |
| 10.50 | | | | | | | 12.00 | YAS<br>11.70 | 73 | 57 | | | | | |
| 12.00 | | | | | | | | | 87 | 80 | | | | | |
| 13.50 | | | | | | | | | 100 | 77 | | | | | |
| 15.00 | | | | | | | | | 100 | 7 | | | | | |
| 16.50 | | | | | | | | | 100 | 63 | | | | | |
| 18.00 | | | | | | | | | 73 | 50 | | 19.00 | Orta sağlam, mavimsi gri renkli, orta-iri taneli, çatlaklı-kırıklı, KUMTASI ,orta derecede-az ayrışmış<br>(15.30-15.50, 16.00-16.50 m derinlikleri arası ince taneli)<br>(9.10-9.80, 11.60-11.70, 15.40-15.70, 16.00-16.50, 17.30-17.40 derinlikleri arası çok çatlaklı-çok kırıklı)<br>(17.30-17.45 m derinlikleri arası boyuna çatlaklı) | -19.00 | |
| 19.50 | | | | | | | | | 53 | 7 | | | See next sheet (2) | | |

**NOTLAR :**

**SONDÖR :** İ.KOLUKISA **LOGU HAZIRLAYAN :** A.ARSLAN **KONTROL EDEN :** T.ÖZBEK

**YER :** BALMUMCU-NİPPON OTEL
**KUYU NO :** SK2
**EKİPMAN :** GMS-300
**ZEMİN KOTU :** 0.00 m.
**SONDAJ YÖNTEMİ :** ROTARY : 0.00-35.00m arası.
**KOORD. :** **N :** **E :**
**KUYU ÇAPI :** 12.00-35.00m arası - 76mm.
**BAŞLANGIÇ :** 02.08.2006 **BİTİŞ :** 04.08.2006

| NUMUNE VE YERİNDE DENEY. | | S.P.T. darbe sayısı | | | | | Muh. Drn. (m) Tarih | Y.A.S. Drn. (m) | TCR | RQD | SCR | Drn. | ZEMİN CİNSİ | KOT | LEJAND |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Drn. (M) | TİP | 15 | 7.5 | 7.5 | 7.5 | 7.5 | | | % | % | % | (m) | | (m) | |
| | | | | | | | | | 53 | 7 | | | | | |
| 21.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 87 | 13 | | | | | |
| 22.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 17 | | | | | |
| 24.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 43 | | | | | |
| 25.50 | | | | | | | | | | | | | Sağlam, gri renkli, orta-iri taneli, çatlaklı-kırıklı, DIYABAZ , az-orta derecede ayrışmış<br>19.00-22.50 m derinlikler arasında makaslama zonu ve bu zonda metamorfizmaya bağlı olarak yüksek ısı nedeniyle hornfelsleşme görülmektedir.<br>(23.00-23.50, 24.00-24.10, 24.90-25.00, 25.40-25.60, 26.40-26.60, 27.00-27.10, 27.80-27.90, 29.60-30.00, 30.40-30.60, 31.35-31.50, 32.30-32.60, 33.10-33.50, 33.75-34.50 m derinlikleri arası çok çatlaklı-çok kırıklı)<br>(25.40-25.50 m derinlikleri arası tamamen ayrışmış killeşmiş)<br>(28.20-28.40, 28.50-29.00 m derinlikleri arası boyuna çatlaklı) | | |
| | | | | | | | | | 100 | 30 | | | | | |
| 27.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 20 | | | | | |
| 28.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 10 | | | | | |
| 30.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 17 | | | | | |
| 31.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 67 | 33 | | | | | |
| 33.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 20 | | | | | |
| 33.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 10 | | 35.00 | SONDAJ BİTİMİ | -35.00 | |
| 35.00 | | | | | | | | | | | | | | | |

**NOTLAR :**

**SONDÖR :** İ.KOLUKISA
**LOGU HAZIRLAYAN :** A.ARSLAN
**KONTROL EDEN :** T.ÖZBEK

**GEOS Geoteknik ve Sondajcılık Ltd.**

NİPPON OTEL
JEOTEKNİK ARAŞTIRMALAR

Sayfa 1 / 2

**YER :** BALMUMCU-NİPPON OTEL
**KUYU NO :** SK3
**EKİPMAN :** GMS-300
**ZEMİN KOTU :** 0.00 m.
**SONDAJ YÖNTEMİ :** ROTARY : 0.00-35.00m arası.
**KOORD. : N :** **E :**
**KUYU ÇAPI :** 0.00-12.00m arası - 89mm. 12.00-24.00m arası - 76mm.
**BAŞLANGIÇ :** 05.08.2006 **BİTİŞ :** 08.08.2006

| NUMUNE VE YERINDE DENEY. Drn. (M) | TİP | S.P.T. darbe sayısı 15 | 7.5 | 7.5 | 7.5 | 7.5 | Muh. Drn. (m) Tarih | Y.A.S. Drn. (m) | TCR % | RQD % | SCR % | Drn. (m) | ZEMİN CİNSİ | KOT (m) | LEJAND |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | 05.08.20 | | | | | 0.70 | Dolgu zemin | -0.70 | ▨ |
| 1.50<br>1.58 | D1 | 50/8 | | | | | | | | | | 3.00 | Çok sıkı, kahve renkli, killi, çakıllı KUM ,yer yer çok zayıf çimentolu<br>(Tamamen ayrışmış Kumtaşı) | -3.00 | ⁘ |
| 3.00 | | | | | | | | | 40 | 0 | | 14.50 | Orta zayıf-orta sağlam, kahve-gri renkli, ince-orta taneli, çok çatlaklı-çok kırıklı, KUMTASI ,çok-orta derecede ayrışmış<br>(4.00-4.50, 6.00-7.00, 12.50-12.90 m derinlikleri arası ince taneli)<br>(9.00-9.50, 10.50-10.70, 13.10-13.25 m derinlikleri arası boyuna çatlaklı) | -14.50 | ░ |
| 4.50 | | | | | | | | | 53 | 7 | | | | | |
| 6.00 | | | | | | | | YAS<br>6.80 | 47 | 7 | | | | | |
| 7.50 | | | | | | | | | 67 | 7 | | | | | |
| 9.00 | | | | | | | | | 100 | 0 | | | | | |
| 10.00 | | | | | | | | | 100 | 10 | | | | | |
| 11.00 | | | | | | | | | 50 | 0 | | | | | |
| 12.00 | | | | | | | 12.00 | | 100 | 13 | | | | | |
| 13.50 | | | | | | | | | 100 | 27 | | | | | |
| 15.00 | | | | | | | | | 100 | 30 | | | Orta sağlam, gri renkli, orta-iri taneli, çatlaklı-kırıklı, KUMTASI ,az-orta derecede ayrışmış<br>(14.50-15.00 m derinlikleri arası boyuna çatlaklı)<br>(16.50-17.90, 27.00-27.80, 33.30-34.70 m derinlikleri arası ince taneli)<br>(17.80-18.00, 21.80-22.50, 22.90-23.30, 27.50-31.50, 33.00-33.15 m derinlikleri arası çok çatlaklı-çok kırıklı) | | ░ |
| 16.50 | | | | | | | | | 100 | 10 | | | | | |
| 18.00 | | | | | | | | | 100 | 20 | | | | | |
| 19.50 | | | | | | | | | 20 | 7 | | | | | |

**NOTLAR :**

**SONDÖR :** İ.KOLUKISA **LOGU HAZIRLAYAN :** A.ARSLAN **KONTROL EDEN :** T.ÖZBEK

| | | | |
|---|---|---|---|
| **YER :** BALMUMCU-NİPPON OTEL | | **KUYU NO :** SK3 | |
| **EKİPMAN :** GMS-300 | | **ZEMİN KOTU :** 0.00 m. | |
| **SONDAJ YÖNTEMİ :** ROTARY : 0.00-35.00m arası. | **KOORD. :** **N :** | **E :** | |
| **KUYU ÇAPI :** 12.00-24.00m arası - 76mm.<br>24.00-35.00m arası - 66mm. | **BAŞLANGIÇ :** 05.08.2006 | **BİTİŞ :** 08.08.2006 | |

| NUMUNE VE YERİNDE DENEY. | | S.P.T. darbe sayısı | | | | | Muh. Drn. (m) Tarih | Y.A.S. Drn. (m) | TCR | RQD | SCR | Drn. | ZEMİN CİNSİ | KOT | LEJAND |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Drn. (M) | TİP | 15 | 7.5 | 7.5 | 7.5 | 7.5 | | | % | % | % | (m) | | (m) | |
| | | | | | | | | | 20 | 7 | | | | | |
| 21.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 0 | | | | | |
| 22.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 13 | | | | | |
| 24.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 33 | 0 | | | | | |
| 25.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 33 | 10 | | | Orta sağlam, gri renkli, orta-iri taneli, çatlaklı-kırıklı, KUMTAŞI ,az-orta derecede ayrışmış<br>(14.50-15.00 m derinlikleri arası boyuna çatlaklı)<br>(16.50-17.90, 27.00-27.80, 33.30-34.70 m derinlikleri arası ince taneli)<br>(17.80-18.00, 21.80-22.50, 22.90-23.30, 27.50-31.50, 33.00-33.15 m derinlikleri arası çok çatlaklı-çok kırıklı) | | |
| 27.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 67 | 0 | | | | | |
| 28.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 30 | 0 | | | | | |
| 30.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 40 | 0 | | | | | |
| 31.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 53 | 7 | | | | | |
| 33.00 | | | | | | | | | | | | | | | |
| | | | | | | | | | 67 | 20 | | | | | |
| 34.50 | | | | | | | | | | | | | | | |
| | | | | | | | | | 100 | 20 | | 35.00 | SONDAJ BİTİMİ | -35.00 | |
| 35.00 | | | | | | | | | | | | | | | |

**NOTLAR :**

**SONDÖR :** İ.KOLUKISA **LOGU HAZIRLAYAN :** A.ARSLAN **KONTROL EDEN :** T.ÖZBEK