# CATALOGING AND STATISTICAL EVALUATION OF COMMON MISTAKES IN GEOTECHNICAL INVESTIGATION REPORTS FOR BUILDINGS ON SHALLOW FOUNDATIONS

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# ABSTRACT

**CATALOGING AND STATISTICAL EVALUATION OF COMMON MISTAKES IN GEOTECHNICAL INVESTIGATION REPORTS FOR BUILDINGS ON SHALLOW FOUNDATIONS**

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Information presented in site investigation reports has a strong influence in design, project costs and safety. For this reason, both the quality and the reliability of site investigation reports are important. However in our country, geotechnical engineering is relegated to second place and site investigation studies, especially parcel-basis ground investigation works; do not receive the attention they deserve. In this study, site investigation reports, that are required for the license of design projects, are examined and the missing/incorrect site investigations, laboratory tests, geotechnical evaluations and geotechnical suggestions that occur in the reports are catalogued. Also, frequency of each mistake is statistically examined; for geotechnical engineers, recommendations and solutions are presented to help them avoid frequent problems.



**Key words:** Geotechnical report, site investigation, in-situ testing, laboratory testing, shallow foundations, geotechnical engineering.

# ÖZ

## SIĞ TEMELLİ BİNALAR İÇİN HAZIRLANAN GEOTEKNİK ETÜT RAPORLARINDAKİ YAYGIN HATALARIN BELİRLENMESİ VE İSTATİSTİKSEL DEĞERLENDİRİLMESİ

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Zemin etüt raporlarında sunulan bilgilerin, yapılacak olan yapının tasarımı, maliyeti ve dayanıklılığı üzerindeki etkisi büyüktür. Bu nedenle etüt raporlarının kalitesi ve güvenilirliği önem arz etmektedir. Ancak ülkemizde, geoteknik mühendisliği ikinci plana itilmekte ve özellikle parsel bazındaki zemin etüt incelemelerine gereken önem verilmemektedir. Bu çalışmada, yapıların ruhsatına esas üstyapı projelerinin hazırlanması için gerekli olan ada/parsel bazında zemin-temel etüdü raporlarının incelenmesi yapılmış olup, bu raporlardaki yanlış/eksik arazi araştırmaları, laboratuvar çalışmaları, geoteknik hesaplar ve geoteknik öneriler tespit edilmeye çalışılmıştır. Ayrıca, bulunan hata ve eksikliklerin istatistiksel değerlendirilmesi yapılmış, geoteknik mühendislerinin sıkça karşılaşılan hatalardan kaçınmasını kolaylaştıracak tavsiyeler ve çözüm yolları sunulmuştur.

**Anahtar kelimeler:** Geoteknik rapor, saha araştırmaları, arazi testleri, laboratuvar testleri, sığ temeller, geoteknik mühendisliği.

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**TABLE OF CONTENTS**

# LIST OF TABLES

TABLES

## LIST OF FIGURES

FIGURES

# LIST OF ABBREVIATIONS

| | |
|---|---|
| $C_c$ | compression index |
| $C_r$ | recompression index |
| $C_s$ | swelling index |
| CD | consolidated drained |
| CU | consolidated undrained |
| CPT | cone penetration test |
| c | cohesion of soil |
| $D_r$ | relative density |
| $E_s$ | stress-strain modulus or modulus of deformation (also modulus of elasticity) of soil; may include additional subscripts to indicate method of determination |
| $k_s$ | modulus of subgrade reaction |
| N | SPT blow count |
| $N_i$ | SPT blow count at i = efficiency of 55, 60, 70, etc., percent |
| OCR | overconsolidation ratio |
| $q_c$ | cone bearing pressure |
| $q_{ult}$ | ultimate computed bearing capacity |
| RQD | rock quality designation |
| $s_u$ | undrained shear strength |
| SPT | standard penetration test |
| UU | unconsolidated undrained |
| $\gamma$ | unit weight of material; subscript is used with $\gamma$ to identify type or state, as c = concrete, dry, wet, sat, etc. |
| $\delta$ | differential settlement between two points |
| $\mu$ | Poisson's ratio |

# CHAPTER 1

# INTRODUCTION

Every civil engineering structure is in a direct relationship with the ground due to being founded in, on or with ground. The structure we put in or on the ground is man-made; every item that goes into it, like concrete, steel and bricks can be controlled and designed as desired. However, our knowledge about the ground is limited and we have no way of exactly controlling the behavior of the ground. For this reason, detailed investigation and accurate determination of natural ground characteristics is necessary, because reliability and cost of the engineering structure is substantially affected by ground properties.

## 1.1 Research Motivation

Although our country is located in one of the world's prominent earthquake zones (Alpine-Himalayan earthquake belt), geotechnical engineering is relegated to second place and geotechnical studies, especially parcel-basis ground investigation works, do not receive the attention they deserve. The geological/geotechnical reports that are prepared for low-rise housing projects are undertreated and seen merely as a procedural requirement.

In recent years, geotechnical engineering has regularly been in the news not only due to the earthquake disasters that we have experienced, but also to a lesser extent, due to slope stability and urban excavation failures. And yet, geotechnical investigations and reports that are inaccurate, incorrect or inadequate are still an important problem of geotechnical engineering in Turkey. It is hoped that the results of this study will attract attention and create awareness on this important issue, which are the first necessary steps towards widespread engagement of the problem.

## 1.2 Purpose and Scope

The purpose of this study is to determine the deficiencies, mistakes and incorrect suggestions that are frequently performed in geological/geotechnical reports prepared by different institutions for low-rise housing projects. An addition aim is to provide information and solutions for avoiding common errors and for obtaining more reliable geotechnical reports. The scope of the work is to collect geotechnical reports of housing projects, to examine the geotechnical investigations, calculations and suggestions according to the criteria considered by the literature to be important, and to emphasize common mistakes. Not only statistical evaluations of these frequent problems are carried out, but also guidelines of true solutions for engineers to avoid these frequent problems are presented. In this study, a total number of 66 geological/geotechnical reports are pitted against 36 different technical criteria.

From municipalities of all central districts of Ankara (Altındağ, Çankaya, Etimesgut, Gölbaşı, Keçiören, Mamak, Sincan and Yenimahalle), 60 different geotechnical reports are randomly collected and each are evaluated according to different technical criteria. In order to add breadth to the data and to attribute the results of this study to whole country, 6 additional reports that are obtained from other city municipalities are also included into the assessment. The missing/incorrect site investigations, laboratory tests, geotechnical evaluations and geotechnical suggestions that occur in these reports are investigated and catalogued. Also, frequency of each deficiency and mistake is statistically examined.

Examination is only made on geotechnical point of view, parts related to geology and geophysics are not included in this study. In Turkey, both the field investigations and the geological/geotechnical reports are carried out by geological or geophysical engineers. It should clearly be noted that; the aim of this study is not to question this situation or not to create a feud between engineering branches; aim is only to help prevent common errors to obtain more reliable geological/geotechnical reports.

## 1.3 Outline of Thesis

In order to complete this study, several stages were considered. As a first stage, literature survey about site investigation methods, geotechnical calculation procedures and geotechnical reports are reviewed. The extensive background knowledge required for the scope of work is presented in Chapters 2, 3 and 4. In Chapter 2, necessary information, which will be used and cited in the later chapters, is provided for proper and correct procedures of site investigation and laboratory test experiments. In the same manner, Chapter 3 includes information about bearing capacity, foundation settlement and foundation design. Chapter 4 is the part that includes guidelines for geotechnical report writing. Information about not only geotechnical knowledge and instructions to be included in the report content, but also information about their format and sequential order is given in Chapter 4.

Second stage of the study comprises the evaluation of collected geotechnical reports that are required for the license of design projects. In Chapter 5, assessment criteria and obtained results are presented in tables. Also, discussions and opinions about the results given in tables are expressed. In Chapter 6, as a final stage, comments and conclusions are presented. Additionally, recommendations and solutions to various issues are presented to help geological and/or geotechnical engineers avoid frequent problems.

# CHAPTER 2

# SITE INVESTIGATION

Site investigation is the general process of collecting information, evaluating, interpreting and reporting of data. The purpose of site investigation is to gather and identify the geological, geotechnical, and other relevant information of the ground at a site in order to accomplish efficient, safe and economic designs.

The sub-soils, in/on which a structure will stand, are created by many geological processes out of a wide variety of materials. We usually know very little about them, therefore, an adequate and properly configured site investigation is essential to understand the distribution of the materials, their properties and behavior under various influences during the construction and lifetime of the structure.

Geological conditions can be extremely complex and may change over time. It is not possible to identify all the information of the ground exactly, regardless of the comprehensiveness of the investigation, which means no one can always be 100% right when site investigation is completed. However, a properly procured, supervised and well interpreted site investigation that is tailored both to the conditions existing on site and to the form of construction, represents reliable and representative information that can be used in design by the engineer with confidence.

Inadequate or improper site investigation may result in construction delays, extra costs, or even structural collapse. According to Institution of Civil Engineers (1991), Littlejohn et al. (1994), Whyte (1995), ground engineering risk is one of the largest elements of technical and financial risk in civil engineering and building projects. In order to reduce the risk of cost overrun and structural collapse, expenditure of site investigations, which is sometimes as low as 0.1% to 0.2% (Building Research

Establishment, 1987) of the total project cost, should be increased. In today's general practice, site investigation is often based on minimum cost and maximum speed. Without a doubt, this increases the risk of poor quality work. The National Research Council (1984) suggests that site investigation expenditure should be at least 3% of the cost of the project. However, it should not be forgotten that site investigations that are not planned and conducted with geotechnical expertise, but only just fill this financial percentage, are useless.

## 2.1 Phases of Site Investigation

Site investigation process can be divided into a number of phases based on their purpose, with various investigation stages in each phase. Table 2.1 provides information about the general phases of site investigation and their properties.

Table 2.1 Geotechnical requirements (Look, 2007)

| *Geotechnical Study* | *Key Model* | *Relative (100% total)* | | *Key data* | *Comments* |
|---|---|---|---|---|---|
| | | *Effort* | *Benefit* | | |
| Desktop study | Geological model | <5% | ~20% | Geological setting, existing data, site history, aerial photographs and terrain assessment. | Minor SI costs (site reconnaissance) with significant planning benefits. |
| Definition of needs | | <5% | ~20% | Justify investigation requirements and anticipated costs. | Safety plans and services checks. Physical, environmental and allowable site access. |
| Preliminary investigation | Geological and geotechnical model | 15% | ~20% | Depth, thickness and composition of soils and strata. | Planning/Preliminary Investigation of ~20% of planned detailed site investigation. |
| Detailed site investigation | Geotechnical model | 75% | ~20% | Quantitative, and characterisation of critical or founding strata. | Laboratory analysis of 20% of detailed soil profile. |
| Monitoring/ Inspection | | <10% | ~20% | Instrumentation as required. QA testing. | Confirms models adopted or requirements to adjust assumptions. Increased effort for observational design approach. |

### 2.1.1 The desk study and walk over survey

The desk study and walk-over survey are the two necessary constituents of the site investigation. Both the desk study and walk-over survey provide many benefits at negligible cost. They are by far the most cost-effective parts of the site investigation process (Clayton et al., 1995).

The early stage of site investigation usually involves a desk study to collect and collate information already available about a site, and the likely problems that they will produce for the proposed type of construction. This is likely to involve multiple sources of information including ordnance survey maps, geological and groundwater vulnerability maps, aerial photographs, records of previous site investigation reports, service records to locate subsurface utilities such as electricity cables, sewers and telephone wires.

Subsequent to the initial document search, the walk-over survey is performed to collect extra information about the current condition of the site and on likely construction problems. The walk-over survey should complement the desk study and typically provide valuable information on features such as; topography, geology, surface and groundwater, ecology, damage of existing structures (settlement, cracks), access and services.

When these surveys are completed, the outputs should be reported in a formal way. The routine check-list recommended by the Building Research Establishment (1987) for desk and walk-over studies associated with low-rise building is shown in Table 2.2.

Table 2.2 Desk study and walkover survey checklist for low-rise buildings (Building Research Establishment, 1987)

**Topography, vegetation and drainage**

Does the site lie on sloping ground, and if so what is the maximum slope angle ?
Are there springs, ponds or watercourses on or near the site ?
Are there or were there trees or hedges in the area of proposed construction ?
Is there evidence of changes in ground level, e.g. by placement of fill or by the demolition of old structures ?

**Ground conditions**

What geological strata lie below the site and how thick are they ?
What problems are known to be associated with this geological context ?
Is the site covered by alluvium, glacial till(boulder clay) or any possible soft deposits?
Is there available information on the strength and compressibility of the ground ?
Does experience suggest that groundwater in these soil conditions may attack concrete ?
Is there evidence of landslipping either on or adjacent to the site or on similar ground nearby ?
Is there, or has there ever been, mining or quarrying activity in this area ?

**The proposed structure**

What area will the buildings occupy ?
What foundation loading is specified
How sensitive is the structure likely to be to differential foundation movements ?
What soils information is required for the design of every likely type of foundation ?
Is specialist geotechnical skill required ?

### 2.1.2 Surface investigation (Geophysical exploration)

Geophysical techniques consist of making indirect measurements from ground surface or in subsurface explorations to obtain subsurface information. They are helpful in correlating geologic features such as stratigraphy, ground water, locating cavities and discontinuities. For example, subsurface distribution of the geologic materials and groundwater conditions between boreholes can be checked to see whether ground conditions at the boreholes are representative of that elsewhere. The cost of geophysical explorations is generally low and considerable savings may be obtained by using the right technique at the right place. The main geophysical exploration techniques are seismic, electrical, sonic, magnetic, radar and gravity. Geophysical exploration is not within the scope of this study, so detailed information about geophysical methods is not provided in this study. Some basic geophysical methods are summarized in Appendix Table A.1. Their abilities to obtain different types of subsurface data are summarized in Appendix Table A.2.

### 2.1.3 Sub-Surface investigation

Sub-surface investigation is carried out for the purpose of detailed site characterization to be used for design. This involves using direct methods of investigation, such as drilling, sampling, field tests, and it requires use of specialized equipment. According to Clayton et al. (1995) exploration survey are carried out for a number of reasons, such as:

1. to establish the general nature of the strata below a site;
2. to establish the vertical or lateral variability of soil conditions;
3. to verify the interpretation of geophysical surveys;
4. to obtain samples for laboratory testing;
5. to allow in situ tests to be carried out; and
6. to install instruments such as piezometers, inclinometers or extensometers.

When compared with the other stages of site investigation, exploration techniques are relatively expensive and therefore should be carefully planned and controlled to increase benefits. Irrelevant and incorrect conclusions can be acquired if the procedures are not followed carefully and data not interpreted properly. For instance, inaccurate soil profile and strength parameters can be obtained as a result of poor drilling and sampling techniques. According to U.S. Army Corps of Engineers (2001), only competent, senior geotechnical personnel should be charged with planning a subsurface investigation, and only qualified geotechnical professionals and technicians should do the drilling, data collecting, analyzing and interpreting.

#### 2.1.3.1 Subsurface exploration planning

After evaluation of available information from the previous stages, the next step is to plan the field exploration program. The field exploration methods, locations and frequency are mostly controlled by the geological conditions, project design requirements and the availability of equipment.

Boring and test pit locations and frequency depend largely on the proposed structure. The layout of the subsurface investigation should aim not only to characterize geotechnical conditions related to the proposed structures and their foundations but also to verify the collected information from previous investigation stages. At the site, all subsurface exploration locations and elevations should be determined and recorded using either conventional surveying methods or by global positioning systems (GPS). It is important to allow cross-sections to be drawn when needed and to interpret the ground conditions between boreholes properly. In general, boring layouts should not be random. For example, borings could be drilled at the four corners of a proposed building, with an additional (and deepest) one located at the center of the proposed building.  Table 2.3 provides guidelines on the typical boring layout versus type of project.

Table 2.3 Guidelines for boring layout (Day, 2006)

| Areas of investigation | Boring layout |
|---|---|
| New site of wide extent | Space preliminary borings 60 to 150 m (200 to 500 ft) apart so that area between any four borings includes approximately 10 percent of total area. In detailed exploration, add borings to establish geological sections at the most useful orientations. |
| Development of site on soft compressible soil | Space borings 30 to 60 m (100 to 200 ft) at possible building locations. Add intermediate borings when building site is determined. |
| Large structure with separate closely spaced footings | Space borings approximately 15 m (50 ft) in both directions, including borings at possible exterior foundation walls, at machinery or elevator pits, and to establish geologic sections at the most useful orientations. |
| Low-load warehouse building of large area | Minimum of four borings at corners plus intermediate borings at interior foundations sufficient to define subsoil profile. |
| Isolated rigid foundation | For foundation 230 to 930 $m^2$ (2500 to 10,000 $ft^2$) in area, minimum of three borings around perimeter. Add interior borings, depending on initial results. |
| Isolated rigid foundation | For foundation less than 230 $m^2$ (2500 $ft^2$) in area, minimum of two borings at opposite corners. Add more for erratic conditions. |

*Source:* From NAVFAC DM-7.1, 1982.

In Turkey, it is recommended to drill at least five boreholes, four at corners and one at center, for parcels greater than 1000 $m^2$ and for smaller ones at least one drilling for every 300 $m^2$ (GDDA, 2005). Özdemir (2005) suggests 2 boreholes for building area smaller than 500 $m^2$, 3 boreholes for area between 500 $m^2$ and 1000 $m^2$ and 5 boreholes for the ones greater than 1000 $m^2$.

As in the boring layout, experience plays an important role and there is no simple answer in determining the extent of the subsurface exploration. The extent of the subsurface exploration depends on the size, loading level, sensitivity of the proposed structure and properties of the strata that will underlie the foundation.

Hvorslev (1949) proposed some general rules which remain applicable:

> The borings should be extended to strata of adequate bearing capacity and should penetrate all deposits which are unsuitable for foundation purposes — such as unconsolidated fill, peat, organic silt and very soft and compressible clay. The soft strata should be penetrated even when they are covered with a surface layer of high bearing capacity.
>
> When structures are to be founded on clay and other materials with adequate strength to support the structure but subject to consolidation by an increase in the load, the borings should penetrate the compressible strata or be extended to such a depth that the stress increase for still deeper strata is reduced to values so small that the corresponding consolidation of these strata will not materially influence the settlement of the proposed structure.
>
> Except in the case of very heavy loads or when seepage or other considerations are governing, the borings may be stopped when rock is encountered or after a short penetration into strata of exceptional bearing capacity and stiffness, provided it is known from explorations in the vicinity or the general stratigraphy of the area that these strata have adequate thickness or are underlain by still stronger formations. When these conditions are not fulfilled, some of the borings must be extended until it has been established that the strong strata have adequate thickness irrespective of the character of the underlying material.
>
> When the structure is to be founded on rock, it must be verified that bedrock and not boulders have been encountered, and it is advisable to extend one or more borings from 3 to 6 m into solid rock in order to determine the extent and character of the weathered zone of the rock.
>
> In regions where rock or strata of exceptional bearing capacity are found at relatively shallow depths — say from 30 to 45 m — it is advisable to extend at least one of the borings to such strata, even when other considerations may indicate that a smaller depth would be sufficient.

Another commonly used rule is De Beer's (1976) proposal which suggests that the depth of exploration should reach such a depth where vertical stress increase due to weight of structure would approximately be equal to ten percent of the existing overburden pressure. Figure 2.1 presents additional guidelines for different types of foundation projects.

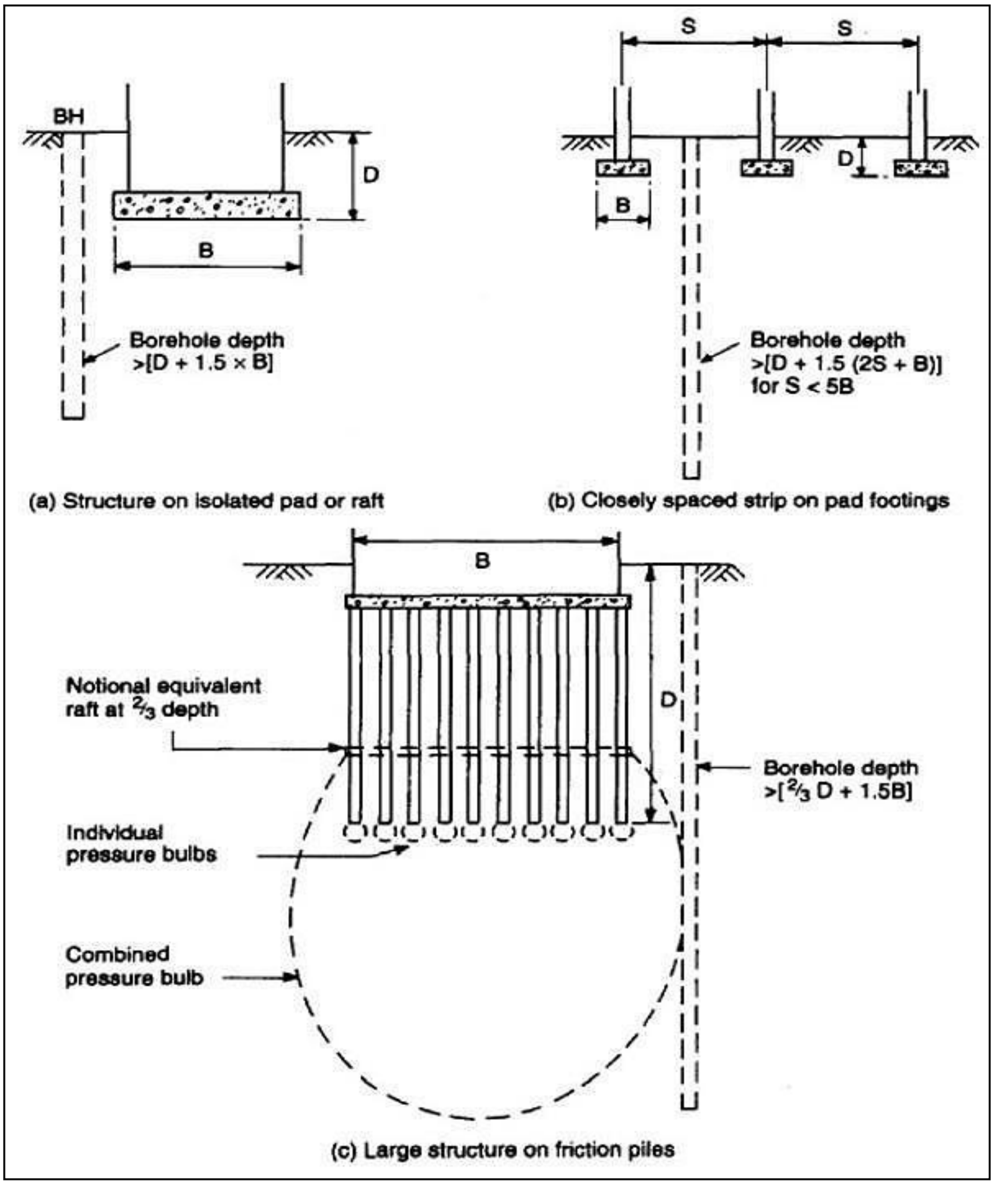


Figure 2.1 Necessary borehole depths for foundations (Clayton et al., 1995).

In our country, for ordinary buildings Özdemir (2005) suggests;

**1-** Drilling should continue until 3 consecutive SPT N values>50 are obtained.

**2-** If rock is encountered during drilling, a minimum of 3 m of rock core shall be obtained at each exploration location and if rock shows a fractured feature, drilling should extend to a minimum depth of 5 m.

**3-** If the above circumstances do not occur, the exploration depth would be up to 1.5 times the short side of the building plan below the foundation level.

**4-** Drilling depth should not be less than 12 m in any case, should be minimum 15 m for buildings with basement and 20 m for building taller than 10 floors.

### 2.1.3.2 Recovery of samples and cores

Sampling is carried out to allow detailed examination of soil and rock, and to supply specimens for laboratory testing to determine their physical and engineering properties. Samples obtained should represent all the characteristics of the ground from which they are taken. They should be large enough to contain representative particle sizes, fabric, and fissuring and fracturing (Clayton et al., 1995).

#### *2.1.3.2.1 Soil sampling*

There are lots of samplers and sampling methods. In order to provide that the sample disturbance is sufficiently small, a suitable technique of sampling and adequate sample size should be selected. In general, two types of samples are specified:

- Disturbed (but representative)
- Undisturbed

A disturbed sample is one in which the in-situ properties of the soil has been destroyed sufficiently during the collection process that only visual classification can be done and some laboratory tests can be carried out to determine properties of the soil grains accurately.

Undisturbed samples preserve in situ structural properties of soil, however, it should not be forgotten that no soil sample can be obtained in a perfectly undisturbed state. Considerable experience and specialized equipment is needed to minimize the disturbance of sample. According to Mayne et al. (2001), undisturbed samples are obtained in clay soil strata for use in laboratory testing to determine the engineering properties such as strength, permeability, compressibility and fracture patterns of those soils. They also state that undisturbed samples of granular soils can be obtained, but often specialized procedures are required such as freezing or resin impregnation and block or core type sampling. Common methods for obtaining disturbed and undisturbed samples are summarized in Table 2.4.

Table 2.4 Common soil sampling methods (Mayne et al., 2001)

| *Sampler* | *Disturbed / Undisturbed* | *Appropriate Soil Types* | *Method of Penetration* |
|---|---|---|---|
| **Split-Barrel (Split Spoon)** | Disturbed | Sands, silts, clays | Hammer driven |
| **Thin-Walled Shelby Tube** | Undisturbed | Clays, silts, fine-grained soils, clayey sands | Mechanically Pushed |
| **Continuous Push** | Partially Undisturbed | Sands, silts, & clays | Hydraulic push with plastic lining |
| **Piston** | Undisturbed | Silts and clays | Hydraulic Push |
| **Pitcher** | Undisturbed | Stiff to hard clay, silt, sand, partially weather rock, and frozen or resin impregnated granular soil | Rotation and hydraulic pressure |
| **Denison** | Undisturbed | Stiff to hard clay, silt, sand and partially weather rock | Rotation and hydraulic pressure |
| **Modified California** | Disturbed | Sands, silts, clays, and gravels | Hammer driven (large split spoon) |
| **Continuous Auger** | Disturbed | Cohesive soils | Drilling w/ Hollow Stem Augers |
| **Bulk** | Disturbed | Gravels, Sands, Silts, Clays | Hand tools, bucket augering |
| **Block** | Undisturbed | Cohesive soils and frozen or resin impregnated granular soil | Hand tools |

#### *2.1.3.2.2 Coring of rocks*

Where borings must extend into rock formations, rock coring is required. According to Mayne et al. (2001), defining the top of rock from drilling operations can be difficult, especially where large boulders exist, below irregular residual soil profiles, and in karst terrain. They also assert that a penetration of 25 mm (1 in) or less by a 51 mm (2 in) diameter split-barrel sampler following 50 blows using standard penetration energy indicates that soil sampling methods are not applicable and rock drilling or coring is required. Also geophysical methods, such as seismic refraction, can be used to specify the elevations of rock layers.

For coring of rocks, a core barrel, with a coring bit at the bottom, is attached to a drilling rod. There are three basic types of core barrels, namely, single-tube, double-tube, or triple-tube. Appendix Figure A.1 illustrates single and double tube core barrels. Rock cores obtained by single-tube core barrels can be highly disturbed and fractured because of torsion so they are often used as a starter barrel during the beginning of coring operations. Double-tube core barrel, which is the standard, consists of an inner and outer core barrel and offers a better recovery because it isolates the rock core from the drilling fluid stream. Triple-tube barrels are identical to double-tube barrels except that a liner tube is used inside the inner barrel which retains the cored sample and helps to reduce the frictional heat that may damage samples. Triple-tube barrels are useful in coring fractured and highly weathered rocks.

In core logging, relevant information such as the method of drilling and summary of rock coring parameters including total core recovery (TCR) and rock quality designation (RQD) shall be provided. TCR is the length of the total amount of core sample recovered, expressed as a percentage of the length of the core run, and RQD is a modified core recovery percentage in which the lengths of all recovered core pieces (NX size) equal to or longer than 100 mm are summed and divided by the length of the entire core run. The correct procedure for measuring RQD is illustrated in Appendix Figure A.2.

#### *2.1.3.2.3 Sampling interval and appropriate type of sampler*

The sampling interval varies according to project requirements. Generally, sampling intervals range between 0.75 and 1.5 m (Rowe, 2001). Seldom is the interval greater than 3 m and sometimes continuous sampling may be necessary through formations with highly variable strata. In general, disturbed samples are taken in both granular and cohesive soils for 1.5 m intervals as a result of standard penetration test. However, for granular soils under groundwater level, especially in the first 10 m depth, standard penetration test should be performed on every 1 m intervals (GDDA, 2005). In cohesive soils, with the help of information collected by standard penetration test samples, sufficient number of undisturbed samples should be obtained to determine the properties of each layer. For example if the first 4 m below the foundation base is a cohesive layer, at least two undisturbed samples should be taken (GDDA, 2005). In some instances, a greater sample interval, often 3 m, is allowed below depths of 30 m (Mayne et al, 2001).

An important aspect for minimizing the sample disturbance is selecting the most appropriate sampler type. According to Hunt (2005), a number of factors are considered in the selection of samplers, including:

- Sample use, which varies from general determination of material, to examination of material and fabric and in situ testing, to performing laboratory index tests, and to carrying out laboratory engineering-properties tests.
- Soil type, since some samplers are suited only for particular conditions, such as soft to firm soils vs. hard soils.
- Rock conditions, since various combinations of rock bits and core barrels are used, depending on rock type and quality and the amount of recovery required.
- Surface conditions, which vary from land or quiet water to shallow or deep water with moderate to heavy swells.

Applications of some common sampling tools to various subsurface conditions are illustrated in Figure 2.2. More detailed information about sampling methods and limitations are described in Appendix Table A.3.

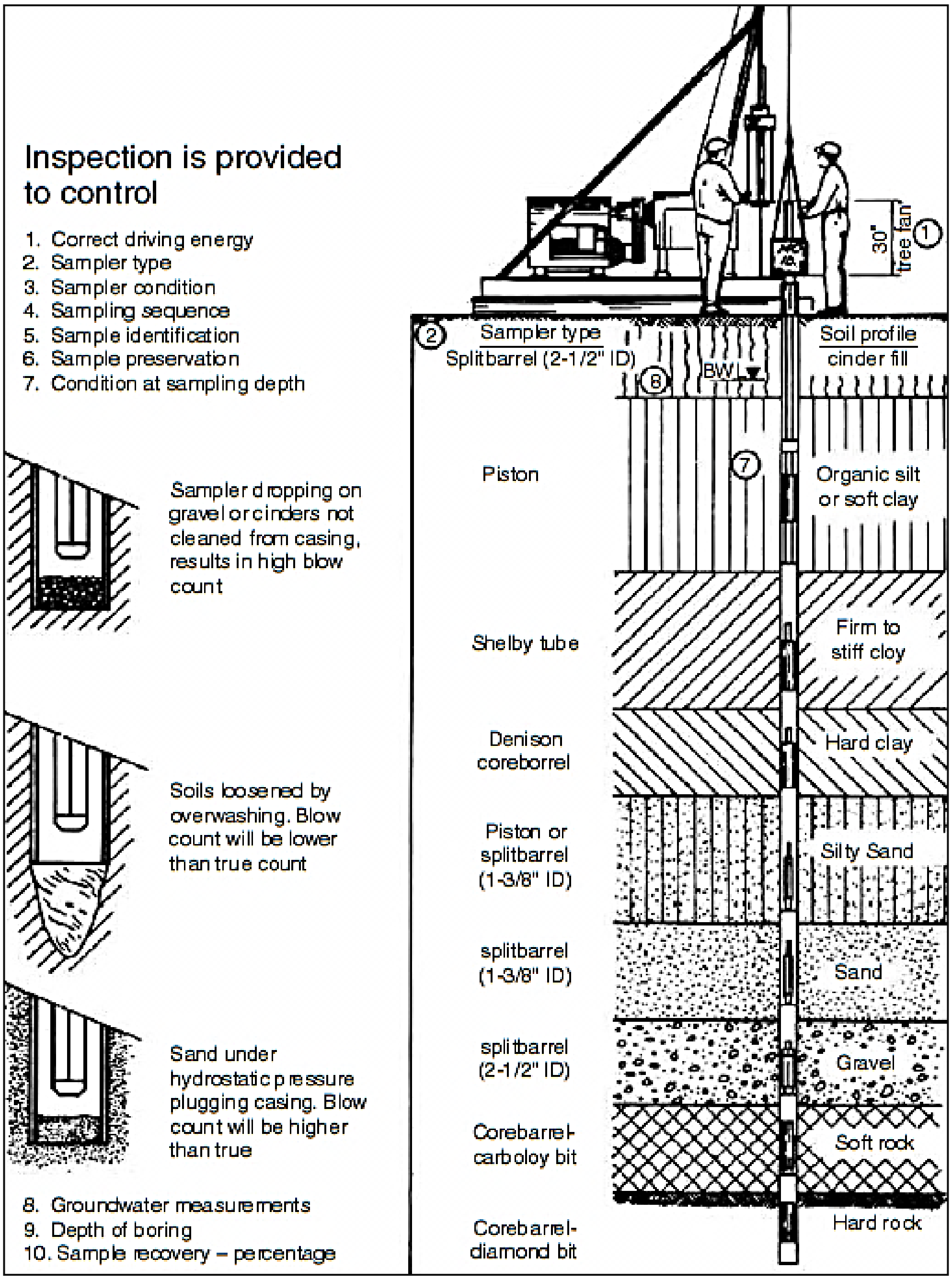


Figure 2.2 Common sampling tools for various soil and rock conditions (Hunt, 2005).

#### *2.1.3.2.4 Sample quality*

In evaluating consolidation and strength data it is useful to evaluate sample quality although this is not common in practice. Currently, the simplest and most effective method in determining sample quality is the measure of $\varepsilon_{vol}$ at $\sigma'_{v0}$. Andresen and Kolstad (1979) first developed this method with a ranking system that assigns a description of sample quality ranging from poor to excellent. Terzaghi et al. (1996) adopted this method and coined the term Specimen Quality Designation (SQD) with sample quality ranging from A (best) to E (worst) as listed in Appendix Table A.4. Terzaghi et al. suggest that reliable estimates of engineering parameters such as preconsolidation stress ($\sigma'_p$) and the undrained shear strength ($s_u$) require samples with SQD equal to B or better.

Thick sampler walls increases the sample disturbance while the use of very thin walled samplers may lead them to bend or buckle during driving the sampler into the soil. Bent and deformed sampler cutting edge increases the sample deformation. Sampler cutting edge should be smooth and maybe a thin film of oil can be applied at the cutting edge to reduce the friction between the soil and metal tube during sampling operations. Also inside clearance should be provided because rust and dirt causes distortions. Stress relief can result in base heave, caving, and piping in the borehole so the sampler should be lowered to the bottom immediately after the hole has been cleaned. Length of the sampler and rods should be measured carefully to make it certain that the sampler is resting at the bottom of the borehole and is seated precisely.

### 2.1.3.3 Groundwater observations

Groundwater conditions and the potential for groundwater seepage are fundamental factors in virtually all geotechnical analyses and design studies. Accordingly, the evaluation of groundwater conditions is a basic element of almost all geotechnical investigation programs (Mayne et al., 2001). Groundwater investigations include measurements of the elevation of the groundwater surface or water table and its variation in short term (couple of weeks) and in long term (season of the year); the location of aquifers; and the presence of artesian pressures. Piezometers are used where the measurement of the ground water pressures are specifically required.

### 2.1.4 In-situ geotechnical tests

In-situ tests are conducted to obtain direct measurements of geotechnical parameters and soil properties. In-situ tests are generally performed to investigate a much greater volume of material more quickly than possible for sampling and laboratory tests. They also facilitate testing at the in situ stress state. Therefore, they have the potential to realize high statistical reliability for foundation design (Failmezger, 2008). For designs involving coarse-grained foundation materials, where undisturbed sampling is usually impractical, in situ testing is the only feasible way to estimate the material properties.

This section presents an overview of the most common in-situ tests in Turkey and points out some important details that are often overlooked. Further information can be obtained from Sabatini et al. (2002) and Mayne et al. (2001), which are presented in the references, and from relevant testing standards (ASTM). Appendix Table A.5 and Table A.6 list in-situ test methods and their general application and purposes.

#### 1- Standard Penetration Test (SPT)

The standard penetration test (ASTM D-1586) is probably the most common in-situ soil test performed in the world. The SPT is performed during the advancement of a soil boring to obtain an approximate measure of the dynamic soil resistance. The SPT involves the driving of a hollow thick-walled tube into the ground and measuring the number of blows to advance the split-barrel sampler a vertical distance of 300 mm (Figure 2.3). A drop weight system is used for the pounding where a 63.5-kg hammer repeatedly falls from 0.76 m to achieve three successive increments of 150-mm each. The second and third increments, following the 150 mm seating drive, are summed to give the N-value ("blow count") or SPT-resistance. The SPT can be halted when 100 blows has been achieved or if the number of blows exceeds 50 in any given 150-mm increment (Mayne et al., 2001).

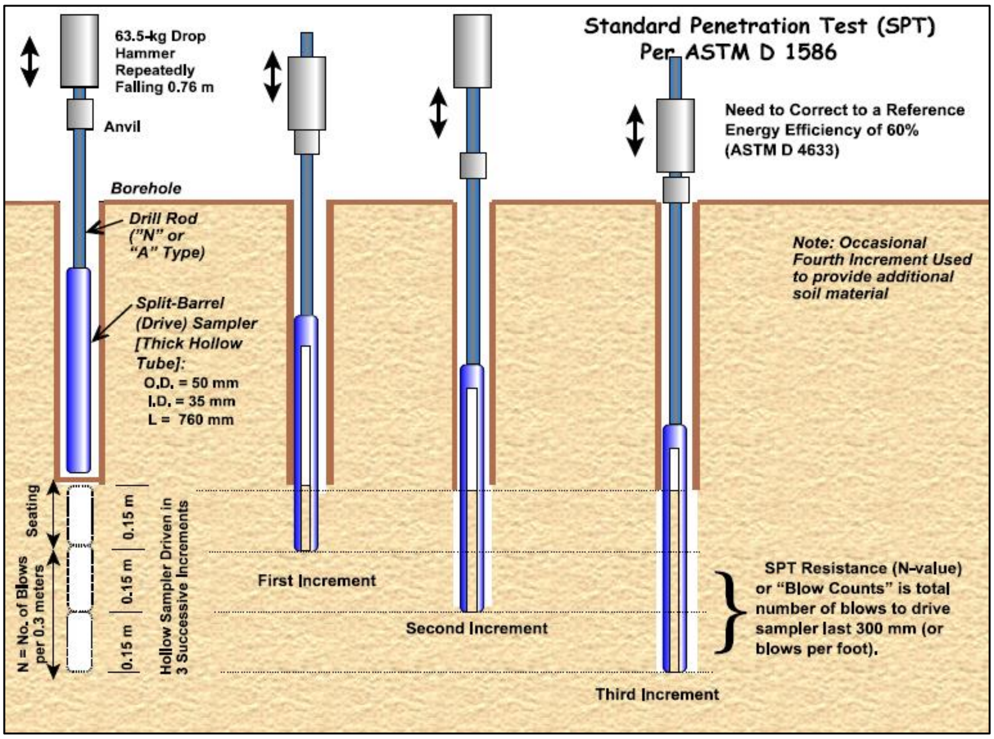


Figure 2.3 Sequence of driving split-barrel sampler during the SPT (Mayne et al., 2001).

The test can be performed in a wide variety of soil types, as well as weak rocks, but it is not particularly useful in the characterization of gravel deposits and soft clays (Mayne et al., 2001). SPT is recommended for essentially all subsurface investigations since it both provides a disturbed sample and a useful number, N-value. It is also a very fast and inexpensive test. Although the test is relatively simple to perform, it should be performed with only appropriate equipment and by only skilled drillers to achieve meaningful results. The main factors affecting the SPT results are summarized in Table 2.5.

Table 2.5 Factors affecting the SPT (after Kulhawy and Mayne, 1990)

| **Cause** | **Effects** | **Influence on SPT N Value** |
|---|---|---|
| Inadequate cleaning of hole | Soil may become trapped in sampler and may be compressed as sampler is driven, reducing recovery | Increases |
| Failure to maintain adequate head of water in borehole | Bottom of borehole may become quick | Decreases |
| Careless measure of drop | Hammer energy varies (generally variations cluster on low side) | Increases |
| Hammer weight inaccurate | Hammer energy varies (driller supplies weight; variations of 5 – 7 percent common) | Increases or decreases |
| Hammer strikes drill rod collar eccentrically | Hammer energy reduced | Increases |
| Lack of hammer free fall because of ungreased heaves | Hammer energy reduced | Increases |
| Sampler driven above bottom of casing | Sampler driven in disturbed, artificially densified soil | Increases greatly |
| Careless blow count | Inaccurate results | Increases or decreases |
| Use of non-standard sampler | Correlations with standard sampler invalid | Increases or decreases |
| Coarse gravel or cobbles in soil | Sampler becomes clogged or impeded | Increases |
| Use of bent drill rods | Inhibited transfer of energy of sampler | Increases |

For routine engineering practice, correlations for engineering properties are based on SPT N values measured based on a 60 percent efficient system (Sabatini et al., 2002). The N values corresponding to 60 percent efficiency are termed $N_{60}$. In Turkey, notation $N_{30}$ is common to indicate N value of the last 30 cm penetration of SPT hammer, which is misleading, because the N subscript indicates the energy efficiency, not the penetration depth.

Many different correction factors to the measured N-value are necessary because of energy inefficiencies and procedural variation in practice. By applying certain correction factors to the field recorded raw N-value, the corrected value is calculated as:

$$N_{60} = N_{field}\, C_E\, C_B\, C_S\, C_R \quad (2.1)$$

where; $C_E$ is energy correction factor, $C_B$ is borehole diameter correction, $C_S$ is sampler correction, $C_R$ is rod length correction. The correction factors are presented in Table 2.6.

Table 2.6 Corrections to the SPT (Skempton, 1986)

| Factor | Equipment Variable | Term | Correction |
|---|---|---|---|
| Energy Ratio (ER) | Donut Hammer<br>Safety Hammer<br>Automatic Hammer | $C_E$ = ER/60 | 0.5 to 1.0<br>0.7 to 1.2<br>0.8 to 1.5 |
| Borehole Diameter | 65 to 115 mm<br>150 mm | $C_B$ | 1.0<br>1.05 |
| Sampling method | Standard sampler<br>Non-standard sampler | $C_S$ | 1.0<br>1.1 to 1.3 |
| Rod Length | 3 to 4 m<br>4 to 6 m<br>6 to 10 m<br>10 to >30 m | $C_R$ | 0.75<br>0.85<br>0.95<br>1.0 |

The corrected SPT $N_{60}$ value may also be adjusted using an overburden correction that balances the effects of stress level. Since in a uniform soil deposit deep tests will have higher N values than shallow tests, the overburden correction factor is used to adjust the $N_{60}$ value to a reference point of vertical stress equal to 100 kPa. The overburden corrected $(N_1)_{60}$ values are expressed as:

$$(N_1)_{60} = C_N\, N_{60} \quad (2.2)$$

where $C_N$ is the correction factor for overburden stress. The expression for $C_N$ is given below with a restriction that $C_N \leq 2$ (Liao and Whitman, 1986):

$$C_N = (100 / \sigma'_v)^{0.5} \quad (2.3)$$

where $\sigma'_v$ the effective overburden pressure at the point of measurement. It should be noted that the overburden correction generally is applied only for granular soils (Das, 2011).

When the test carried out in very fine sand or silty sand below the water table, field measured N values greater than 15 should be corrected by using the following equation (Terzaghi & Peck, 1948):

$$N=15+1/2*(N-15) \quad (2.4)$$

**2- Cone Penetration Test (CPT)**

The cone penetration test involves insertion of an instrumented, cone-tipped cylindrical steel probe into the ground to determine the penetration resistance of the soil (Figure 2.4). The mechanical system (ASTM D-3441) and the electronic system (ASTM D-5778) are the two most common types of cone penetration testing. The mechanical cone measures cone tip resistance ($q_c$) and side resistance ($f_c$) at intervals of about 20 cm, whereas the electric cone is able to measure $q_c$ and $f_c$ continuously with depth. Also by using cones equipped with pore pressure transducers (piezecones), the excess pore pressures that develop during the advancement of the probe can be measured. This enhanced procedure is known as a CPTU test. Especially in saturated clays, it is very important and useful to monitor the pore water pressure.

The test is applicable to most soils, except gravelly soils, soil fills containing stones and brick bats, and soil with standard penetration resistance N greater than 50 (Das, 2011). In order to perform this test, boreholes are not necessary. It is also an inexpensive and a fast method but not recovering a soil sample and the necessity of a special rig to perform the test can be listed as the two disadvantages. Besides, raw cone penetration test measurements may require pore water correction and overburden stress normalization. These correction methods are available in the paper published by Mayne et al. (2001). Despite not recovering any soil samples, it is possible to obtain an approximate soil classification using the chart shown in Appendix Figure A.3. By using correlations, based on the soil type as determined by the CPT, the undrained strength can be estimated for clays (Jamiolkowski et al., 1982; Schmertmann, 1970), and the relative density (and friction angle) estimated for sands (Durgunoglu and Mitchell, 1975; Mitchell, Guzikewski and Villet, 1978; Schmertmann, 1978).

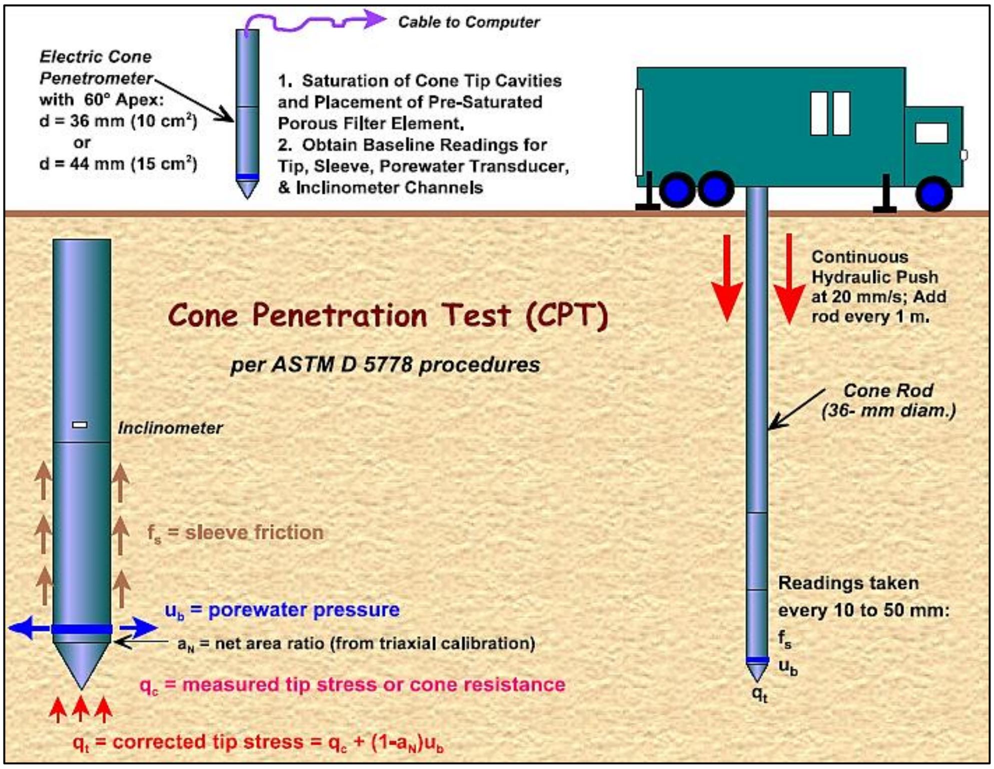


Figure 2.4 Procedures and components of the cone penetration test (Mayne et al., 2001).

### 3- Pressuremeter Test (PMT)

The pressuremeter test (ASTM D-4719) consists of a radially expanding cylindrical probe and a ground monitoring unit. The cylindrical probe consists of three cells

(Appendix Figure A.4). The top and bottom guard cells ensure that the central cell exerts a uniform pressure against the side walls of the boring. The central cell is connected to a pressure-volumeter that records the increasing inflation pressure and the volume change. The test is conducted in equal increments of time and pressure, and the resulting borehole expansions recorded at 15, 30, 60 and 120 seconds after each pressure enhancement. In general, pressuremeter test is conducted in predrilled boreholes. To obtain accurate results, the borehole disturbance should be minimized and the borehole diameter should lie within the range of standards. To offset this limitation, a self-boring pressuremeter has also been developed.

The PMT result data can be interpreted to give complete stress-strain-strength properties (Mayne et al., 2001). The test is applicable to a wide variety of soil types, weathered rock, and low to moderate strength intact rock (Das, 2011).

### 2.1.5 Laboratory testing

The purpose of geotechnical laboratory tests is to investigate the physical and hydrological properties of natural materials such as soil and rock, determine index values for identification and correlation by means of classification tests, and define the engineering properties in parameters usable for design of foundations (US Army Corps of Engineers, 2001). Laboratory tests present tangible results to the engineer to accomplish safe and economical designs for engineering structures. In order to assure quality in laboratory testing and get the most reliable results, extra attention should be paid to the procedure details and factors affecting the sample quality and hence the laboratory results.

The laboratory testing program should be prepared by an experienced geotechnical engineer in the light of information obtained from subsurface exploration. The laboratory testing program may be oriented towards the testing of critical soil layers or subsurface conditions that will have the most impact on the design. The number and scope of laboratory tests may be increased and expanded in critical layers to improve reliability. Conversely, the laboratory program may be limited on the samples of layers with least effect on design, to avoid extra cost and time loss.

In this study, procedures for performing laboratory tests are not described; references are provided for that purpose (Table 2.7). However, basic definitions are provided and some discussions are set up on the commonly used laboratory tests for low-rise building foundation designs in the following section.

Table 2.7 AASHTO, ASTM and Turkish Standards for frequently used laboratory testing of soils and rocks

| Test Category | Name of Test | AASHTO | ASTM | TS 1900 |
|---|---|---|---|---|
| SOIL TESTS | | | | |
| Index Properties | Test Method for Determination of Water (Moisture) Content of Soil by Direct Heating Method | T 265 | D 4959 | 1/5.1.1 |
| | Test Method for Specific Gravity of Soils | T 100 | D 854 | 1/5.1.5 |
| | Method for Particle-Size Analysis of Soils | T 88 | D 422 | 1/5.1.6 |
| | Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils | T 89<br>T 90 | D 4318 | 1/5.1.2-3 |
| Compression Properties | Method for One-Dimensional Consolidation Properties of Soils (Oedometer Test) | T 216 | D 2435 | 2/5.2 |
| Strength Properties | Unconfined Compressive Strength of Cohesive Soil | T 208 | D 2166 | 2/5.3 |
| | Unconsolidated, Undrained Compressive Strength of Cohesive Soils in Triaxial Compression | T 296 | D 2850 | 2/5.4 |
| | Consolidated-Undrained Triaxial Compression Test on Cohesive Soils | T 297 | D 4767 | 2/5.5 |
| | Direct Shear Test of Soils For Consolidated Drained Conditions | T 236 | D 3080 | 2/5.6 |
| ROCK TESTS | | | | |
| Point Load Strength | Method for determining point load index (Is) | - | D 5731 | - |
| Compressive Strength | Compressive strength ($q_u$) of core in unconfined compression (uniaxial compression test) | - | D 2938 | TS 2028 |
| Deformation and Stiffness | Elastic moduli of intact rock core in uniaxial compression | - | D 3148 | TS 2030 |

#### 2.1.5.1 Laboratory testing for soils

##### 1-Moisture content

Determination of moisture content is one of the most common and least expensive laboratory tests. This test can be performed on disturbed or undisturbed soil specimens. The aim is to determine the amount of water present in a quantity of soil in terms of its dry weight and to provide general correlations with strength, settlement, workability and other properties (Mayne et al., 2001). Moisture content is defined as the ratio of the mass of the water in a soil specimen to the mass of the dry soil solids.

Water content, when combined with data obtained from other tests can provide valuable information on possible foundation problems. For example, Day (2006) claims that if a clay layer located below a proposed shallow foundation has a water content of 100 percent, then it is likely that this clay will be highly compressible. Likewise if the same clay layer below the shallow foundation has a water content of 5 percent, then it is likely that the clay layer is dry and desiccated and could subject the shallow foundation to expansive soil uplift.

##### 2-Total unit weight

In the laboratory, the total density, which is also known as the wet density, is simply determined by dividing mass of soil sample to sample volume and can only be obtained from undisturbed soil specimens. The international system of units for density is $kg/m^3$. To convert the wet density ($\rho_t$) to total unit weight ($\gamma_t$), the wet density is multiplied by g (where g is acceleration of gravity = 9.81 $m/sec^2$), which has units of $N/m^3$. For example, the density of water ($\rho_w$) is 1000 $kg/m^3$, while the unit weight of water is 9810 $N/m^3$.

### 3- Specific gravity of solids

The specific gravity is a dimensionless parameter that relates the density of the soil particles to the density of water. Specific gravity ($G_s$) of solids is defined as

$$G_s = M_s/(V_s \times \rho_w) \tag{2.5}$$

where $M_s$ is the mass of soil particles used for the test, $V_s$ is the volume of the soil solids and $\rho_w$ is density of water.

### 4 Particle size analysis of soils (Grain Size Distribution)

This test is performed to determine the percentage (by mass) of various grain sizes contained within a soil. The distribution of particles coarser than 0.075 mm (No. 200 sieve) is determined by sieving, while a sedimentation process (hydrometer test) is used to determine the distribution of particle sizes smaller than 0.075 mm. The particle size distribution is obtained from records of the weight of soil particles and is usually shown on a graph of percentage passing by weight versus particle size (Appendix Figure A.5). If the size distribution of particles finer than 0.075 mm is an important parameter, hydrometer analyses need to be performed. The most common purpose of the hydrometer analysis is to obtain the clay fraction (percentage of particles finer than 0.002 mm).

The grain size distribution is used to determine the textural classification of soils (i.e., gravel, sand, silty clay, etc.) which in turn is useful in evaluating the engineering characteristics such as permeability, strength, swelling potential, and susceptibility to frost action (Mayne et al., 2001).

### 5- Atterberg limits

The objective of the Atterberg limits test is to illustrate the consistency and behavior of fine-grained soils with varying degrees of moisture. The tests for the Atterberg limits are referred to as index tests because they serve as an indication of several physical properties of the soil, including strength, permeability, compressibility, and shrink/swell potential (Sabatini et al., 2002).

In geotechnical engineering practice, the term Atterberg limits refers to three stages of water content known as the liquid limit (LL), plastic limit (PL), and shrinkage limit (SL). These stages are shortly defined below but for laboratory testing procedures and details, see ASTM D-4318 (2004).

- Liquid Limit (LL): The water content at which the behavior of soil change from plastic state to liquid.
- Plastic Limit (PL): The water content corresponding to the behavior change between the plastic and semi-solid state of a silt or clay.
- Shrinkage limit (SL): The water content at which any further loss of moisture will not result in a decrease in the volume of the soil.

By using these limits, other indices including the plasticity index (PI), liquidity index (LI) and the activity (A) of a soil can be obtained. Plasticity index, a measure of soil plasticity, is calculated as

$$PI = LL - PL \tag{2.6}$$

The liquidity index, an indicator of stress history, is defined as

$$LI = (w_n - PL) / PI \tag{2.7}$$

where $w_n$ is the natural moisture content of the soil. The activity (A) of a soil is the PI divided by the percentage of particles finer than 0.002 mm. Appendix Table A.7 gives the ranges of liquid limit, plastic limit, and activity of some clay minerals. The use of the liquidity index and activity can provide very useful information. For example, a LI value less than or equal to zero usually indicates a heavily consolidated soil that may have considerable expansion potential and a LI value of unity indicates that the soil likely is relatively weak and compressible.

If the soil is nonplastic, the Atterberg limits tests are not performed. According to ASTM (2004), the liquid and plastic limit tests must be conducted only on the portion of the soil finer than the No.40 (0.425 mm) sieve. By both using particle size and Atterberg limits data, the soil is classified using the pre-established group symbols. In Appendix Table A.8 shows the most widely used classification system, Unified Soil Classification System (ASTM D-2487 and D-2488). Atterberg limits tests results are not only used for classification of soils, but they also allow the use of a large number of rough empirical relationships for characterizing soils. It is important that these tests should be performed by skilled and careful technicians.

### 6- One-dimensional consolidation (Oedometer test)

One-dimensional consolidation test (or oedometer test) is the most common laboratory method to determine the consolidation and expansion properties of soils. Consolidation test is typically performed on undisturbed samples of fine-grained soils. It is relatively expensive and time consuming as compared to simpler index type tests but it provides one of the most useful and reliable laboratory measurements for soil behavior. The test determines the deformation parameters ($C_r$, $C_c$, $C_s$), stiffness in terms of constrained modulus ($D_r = 1/m_v$), preconsolidation stress ($\sigma'_p$), coefficient of consolidation ($c_v$), creep rate ($C_\alpha$), and approximate value of permeability (k) (Mayne et al., 2001). Results of one-dimensional consolidation tests are commonly presented on an e-log $\sigma'_v$ graph whereby the deformation indices ($C_r$, $C_c$, $C_s$) are determined as the slopes of $\Delta e$ vs. $\Delta \log \sigma'_v$ for the recompression, virgin compression, and swelling lines, respectively (Figure 2.5).

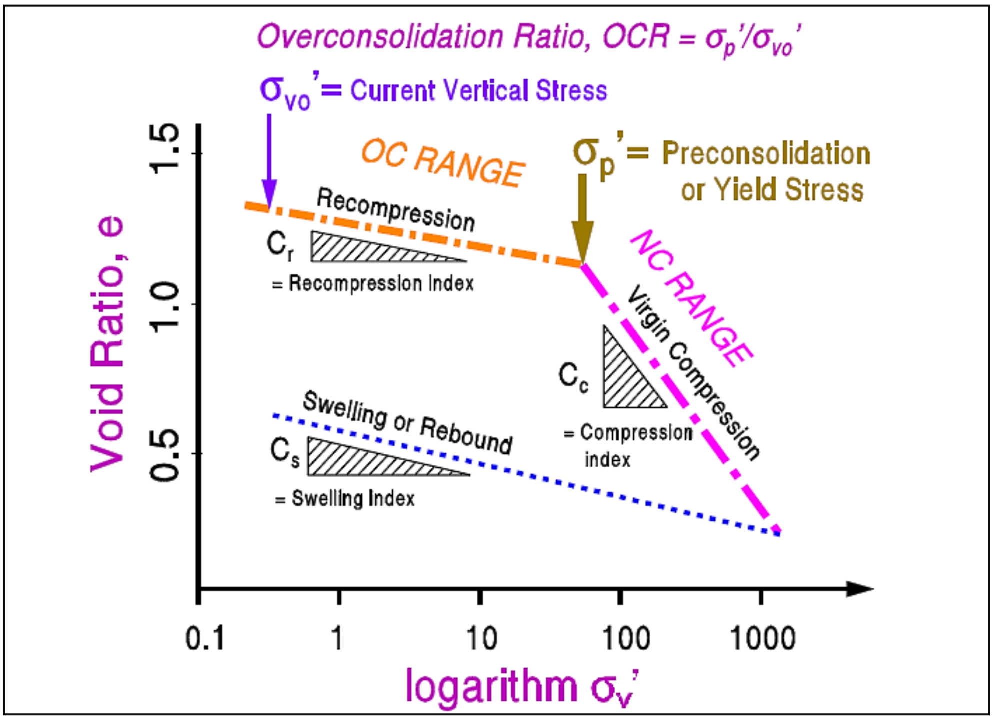



Figure 2.5 Idealized graph of e-log $\sigma'_v$ for obtaining parameters (Mayne et al., 2001).

A customary consolidation test is performed by increasing loading steps. The range of applied loads should cover the stress range from the initial in-situ state of the soil to the final stress level that is expected to take place due to the proposed construction. Also, Samtani (2006) advises that the anticipated preconsolidation stress should be exceeded by at least a factor of four during the laboratory test. The time period between the stress increments should be long enough to obtain reliable results. In order to understand elastic characteristics of soil layer, it is recommended that an unload-reload cycle be performed, especially for cases where accurate settlement predictions are required.

### 7- Unconfined compressive strength of cohesive soil

The unconfined compression test requires a short period of time to complete and is relatively inexpensive means to obtain approximate estimation undrained shear strength ($s_u$) of clay and silty clay soils. This test cannot be performed on granular soils, dry or crumbly soils, peat, or fissured materials. The unconfined compression test is a very simple type of test that consists of applying a vertical compressive pressure without any lateral confinement to a cylindrical cohesive soil sample, at a sufficiently high rate to prevent drainage. Despite some shortcomings and limitations due to the absence of lateral pressures and lack of control over pore pressures, in most cases test results from an unconfined compression test are consistent.

The shear stresses induced in the specimen by the axial load result in a shear failure. The maximum axial compressive stress applied to the specimen represents the unconfined compressive strength ($q_u$). The undrained shear strength ($s_u$) is calculated as half of the unconfined compressive strength ($q_u$) (Figure 2.6). The reliability of this test decreases with respect to increasing sampling depth because the sample tends to swell after sampling resulting in greater particle separation and reduced shear strength.

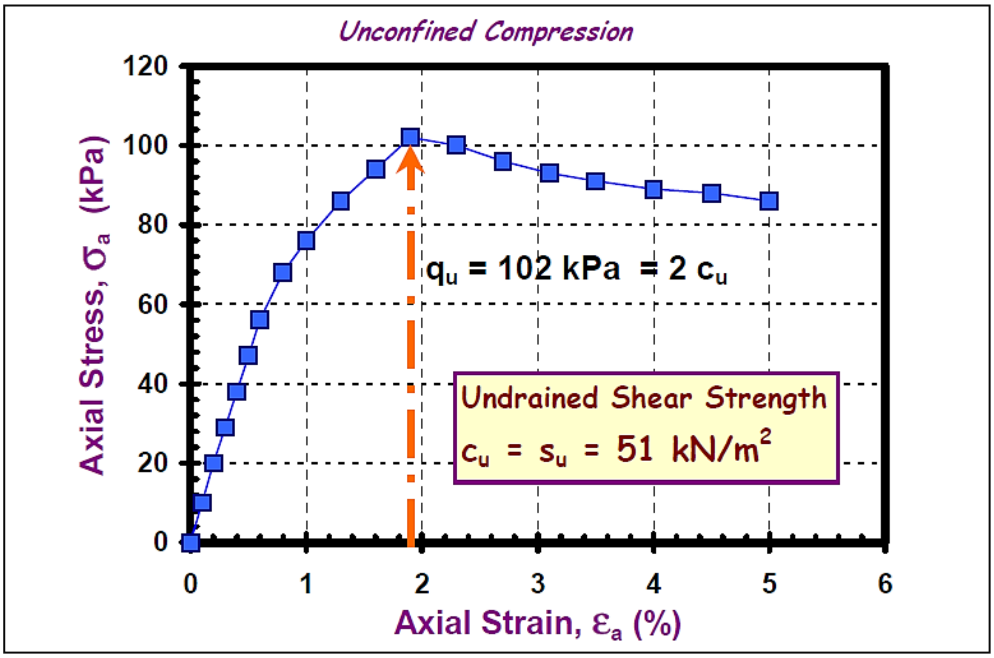


Figure 2.6 Representative stress-strain curve for unconfined compressive test (Mayne et al., 2001).

**8- Triaxial tests**

The triaxial test is probably the most important and extensively used laboratory test to determine strength characteristics of cohesive soils including detailed information on the effects of lateral confinement, porewater pressure, drainage. The triaxial test procedure is to place a cylindrical specimen of cohesive soil in triaxial apparatus, seal the soil with a rubber membrane, subject the specimen to an all-around confining fluid pressure and apply deviator stress through a vertical loading ram to cause shear failure in the sample. Traditionally, triaxial tests results are represented by graphical means using Mohr's circles and a failure envelope tangent to these circles (Figure 2.7). In theory only two circles would be sufficient to construct this tangent, but the recommended procedure involves doing at least three triaxial compression tests, each at a different lateral pressure, on the samples to define failure envelope consistently (Heck, 1970). Triaxial tests are classified according to whether the initial effective stress is controlled, and according to the soil specimen drainage conditions. There are three types of triaxial tests:

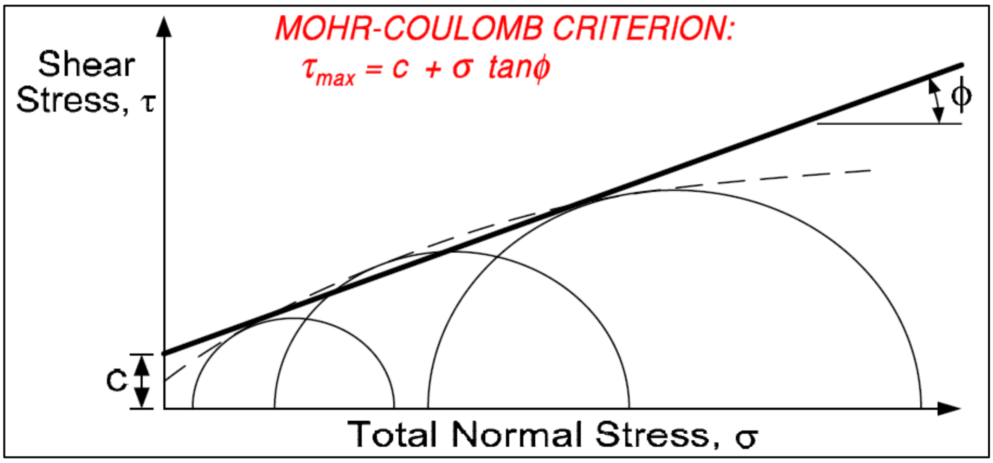



Figure 2.7 Equivalent linear representation of Mohr failure envelope for undrained shear strength of an unsaturated soil (Wright, 2005).

**Unconsolidated-Undrained test (UU):** In the UU test, no drainage or consolidation is allowed during either the application of the confining pressure or the application of the axial load that induces shear stress. This test models the response of a soil that has been subject to a rapid application of confining pressure and shearing load. The unconsolidated-undrained (UU) test provides a measure of the shear strength for short term stability which is in most circumstances the most critical case for buildings. Method generally does not cover measurement of pore water pressures and therefore parameters are determined in terms of total stresses. The failure envelope of a completely saturated cohesive soil is horizontal in an undrained test.

**Consolidated-Drained test (CD):** In this test, the specimen is allowed to completely consolidate under the confining pressure prior to performing the shearing portion of the test. During shearing, load is applied at a rate slow enough to allow drainage of pore water and no buildup of pore water pressures. The time required to conduct this test in low permeability soil may be as long as several months; therefore it is not common to conduct this test on low permeability soils. This test models the long-term (drained) condition in soil. Effective stress strength parameters (i.e., $\varphi'$ and $c'$) and volume deformations during shearing are evaluated in this test.

**Consolidated-Undrained test (CU):** The initial part of this test is similar to the CD test in that the specimen is allowed to consolidate under the confining pressure. The shearing for this test is undrained, and is more rapid than that for a CD test. Pore pressures are measured during axial load application so that both total stress and effective stress strength parameters can be obtained. Consolidated-undrained tests can be performed faster than CD tests, and results show that both tests (CD and CU) yield similar shear strength envelopes when plotted in terms of effective stress (Wright, 2005).

## 9- Direct shear test

The direct shear test is one of the oldest and simplest tests to determine the soil shear strength. In the direct shear test, the soil is first consolidated under a normal force.

The soil is then sheared with a constant rate tangential shear force along a predefined horizontal failure plane. Since there is no way to measure excess pore water pressures generated during shearing, the loading rates must be so slow to allow the specimen to fully drain. Hereby, the direct shear test is only appropriate to measure the shear strength under drained (long-term) conditions. Direct shear testing is commonly performed on compacted materials used for embankment fills and retaining structures. It is also applicable to natural materials where it is necessary to determine the angle of friction between the soil and the material of which the foundation is constructed (Mayne et al, 2001). In such cases, the upper box contains the foundation material and the lower box contains the soil sample. In addition to peak effective shear strength, the direct shear test can be used for the evaluation of effective stress residual strengths by repeating cycles of shearing along the slip surface.

#### 2.1.5.2 Laboratory testing of rocks

1- **Point load index test**

The point load strength test is an appropriate method used to estimate the unconfined compressive strength of rock in which both core samples and fractured rock samples can be tested (Sabatini et al., 2002). The test is conducted by compressing a piece of the rock sample between two conical hardened steel platens (Appendix Figure A.6) until the rock specimen fails in tension between these two points. In order to evaluate uniaxial compressive strength (UCS), index-to-strength conversion factor is applied to point load test index. Conversion factor is dependent upon rock type and generally varies between 16 and 24, with even lower values for some shales and mudstones (Rusnak, 2000). The relationship between UCS and the point load strength could be expressed as (Bieniawski, 1975; Broch and Franklin, 1972):

$$\mathrm{UCS} = \mathrm{k} * \mathrm{I}_{\mathrm{s}_{50}} \tag{2.8}$$

where k= conversion factor

$I_{s_{50}}$= size corrected point load index

Proposed conversion factors by various researchers are represented in Appendix Table A.9. Point Load Index Test is generally not appropriate for rock with uniaxial compressive strength less than 25 MPa (Sabatini et al., 2002).

**2- Unconfined compressive strength of intact rock core**

The unconfined compression test is conducted to identify the uniaxial compressive strength of a cored rock sample. The uniaxial test is probably the most important laboratory test in rock mechanics because it is the most direct way of determining the strength of rock. In this test, cylindrical rock specimens are tested in compression without any lateral confinement. The test sample should be cut with a length/diameter ratio of at least 2 and both the condition of the two ends of the rock core and the rate of loading should be within the tolerances. This test is more expensive than the point load strength test, but is also more accurate.

**3- Elastic moduli of intact rock core**

This test is performed similarly to the unconfined compressive test discussed above, as a plus deformation of the specimen is monitored as a function of load. This test is performed when it is necessary to estimate deformation characteristics of intact rock at intermediate strains. During the test both axial and lateral strain during compression are measured and axial stress versus axial strain curves are generated. The test results are reasonably reliable for engineering applications involving rock classification type, however because of localized variations in rock mass such as jointing, fissuring, and weathering the result are unique for each rock specimen.

# CHAPTER 3

# FOUNDATION DESIGN

A foundation is defined as the part of a structure that supports the weight of the structure and transmits the load to underlying soil or rock (Day, 2006). Foundations are generally divided into two categories: shallow foundations and deep foundations (Figure 3.1). Shallow foundations comprise footings and rafts, which convey the structural loads to shallow depths. However, if the soil stratum near the surface is not capable of supporting the structural loads adequately, deep foundations are used to transmit the loads to deeper and more stable soil (or rock). Deep foundations include pile foundations, stone columns, jet grout columns etc. The selection of foundation type is generally based on two main factors; bearing capacity and settlement.

## 3.1 Bearing Capacity of Foundations

Bearing capacity failure is defined as a foundation failure that occurs when the shear stresses in the soil exceed the shear strength of the soil (Day, 2006). There are three modes of shear failure: general, local and punching shear failures (Appendix Figure B.1). In the general shear failure mode, continuous failure surfaces are well defined and reach out from the edges of footing to ground surface. General shear failure is observed in dense or stiff soils. In the mode of local shear failure the failure surfaces do not reach to ground surface, only slight bulging occurs. Local shear failure occurs in medium dense soils with high compressibility. Punching shear failure occurs by shearing of the soil directly below footing in vertical direction while the area surrounding the footing remains relatively unaffected. In this mode, no bulging of ground surface and no tilting of footing is expected. Punching shear failure occurs in soils that are in a loose or soft state. Table 3.1 presents a summary of the type of bearing capacity failure that would most likely develop based on soil type and soil properties.

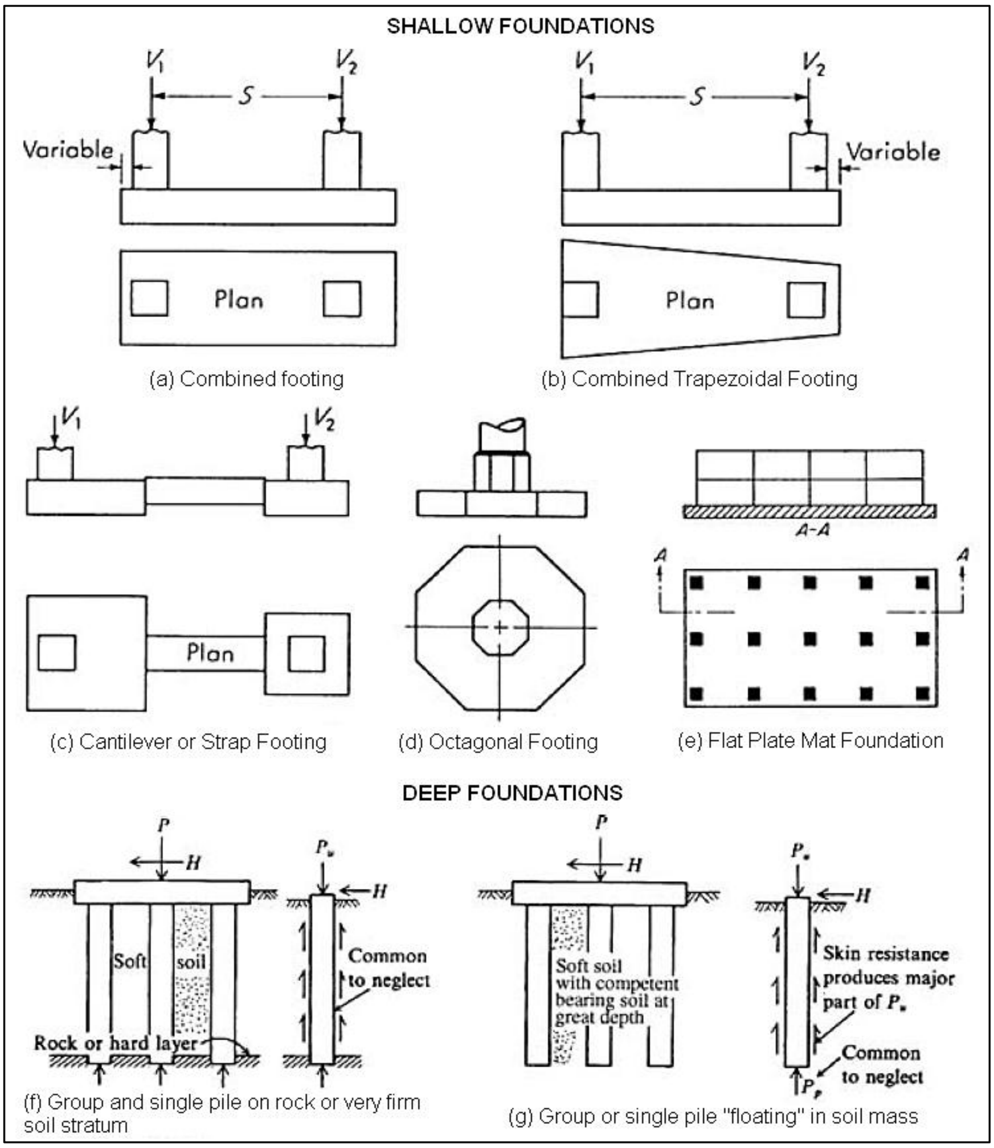



Figure 3.1 Various foundation types (Bowles, 1982).

Table 3.1 Summary of type of bearing capacity failure versus soil properties (Day, 2006)

| | Cohesionless soil (e.g., sands) | | | Cohesive soil (e.g., clays) | |
|---|---|---|---|---|---|
| Type of bearing capacity failure | Density condition | Relative density ($D_r$) | $(N_1)_{60}$ | Consistency | Undrained shear strength ($s_u$) |
| General shear failure | Dense to very dense | 65–100% | > 20 | Very stiff to hard | >100 kPa |
| Local shear failure | Medium | 35–65% | 5–20 | Medium to stiff | 25–100 kPa |
| Punching shear failure | Loose to very loose | 0–35% | < 5 | Soft to very soft | < 25 kPa |

### 3.1.1 Bearing capacity of shallow foundations

Most commonly used bearing capacity equation for shallow foundations is the equation developed by Terzaghi (1943) (Figure 3.2). For developing the equation, some assumptions (i.e. the soil is semi-infinite homogeneous and isotropic, the base of the footing is level and rough, the failure is by general shear mode, the load is vertical and without eccentricity) are made. The ultimate bearing capacity, $q_{ult}$, is expressed in Equation 3.1 (Terzaghi, 1943):

$$q_{ult} = \frac{Q_{ult}}{B.L} = cN_c s_c + \frac{1}{2}\gamma_t B N_\gamma s_\gamma + \gamma_t D_f\, N_q s_q \tag{3.1}$$

where $q_{ult}$ = ultimate bearing capacity for the footing, kPa

$Q_{ult}$ = vertical concentric load causing a general shear failure of the underlying soil, kN

B = width of the footing, m

L = length of the footing, m

$\gamma_t$ = total unit weight of the soil, kN/m$^3$

$D_f$ = vertical distance from ground surface to bottom of footing, m

c = cohesion of the soil underlying the footing, kPa

$N_c$, $N_\gamma$, and $N_q$ = Terzaghi dimensionless bearing capacity factors (Appendix Table B.1)

$s_c$, $s_\gamma$ and $s_q$ = Terzaghi dimensionless shape factors (Appendix Table B.2)

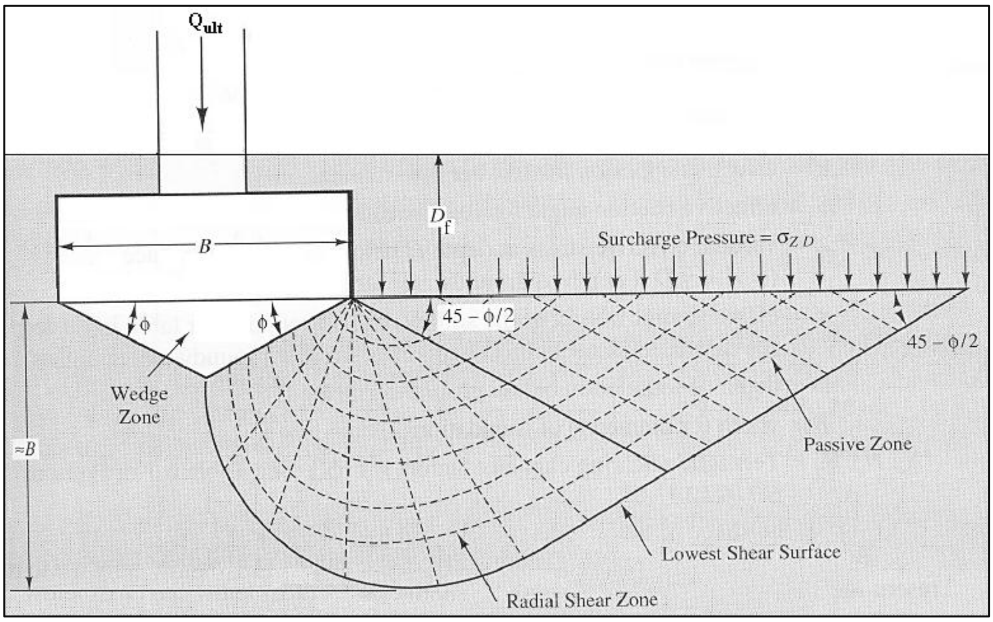


Figure 3.2 Failure surface in soil for a continuous rough rigid foundation as assumed by Terzaghi (Das, 1999).

Terzaghi bearing capacity equation is originally developed for a failure mode of general shearing. In case of loose layer as a foundation soil, where local or punching shear failure is expected, Terzaghi (1943) suggests to use reduced shear strength parameters (c*, ø*) (Eq. 3.2) and the modified dimensionless bearing capacity factors (Appendix Table B.3) for the calculations.

$$c^* = \frac{2}{3}c \ , \ ø^* = tan^{-1}(\frac{2}{3}\ tanø\,) \tag{3.2}$$

Meyerhof (1963) proposed adding depth factors to the terms of Terzaghi's equation (Eq. 3.1). The general form of Meyerhof bearing capacity equation for vertically loaded footing is:

$$q_{ult} = \mathrm{c}N_c s_c d_c + \frac{1}{2}\gamma_t B N_\gamma s_\gamma d_\gamma + \gamma_t D_f\ N_q s_q d_q \tag{3.3}$$

where $N_c$, $N_\gamma$, and $N_q$ = Meyerhof dimensionless bearing capacity factors (Appendix Table B.4)

$s_c$, $s_\gamma$ and $s_q$ = Meyerhof dimensionless shape factors (Appendix Table B.5)

$d_c$, $d_\gamma$ and $d_q$ = Meyerhof dimensionless depth factors (Appendix Table B.6)

(Other terms are previously defined)

Up to a depth of $D_f \approx B$ in Figure 3.2, the Meyerhof $q_{ult}$ is not greatly different from the Terzaghi value but the difference becomes more pronounced at larger $D_f/B$ ratios (Bowles, 1996). Bowles (1996) suggests that the Terzaghi bearing capacity method is useful for estimating of $q_{ult}$ of cohesive soils where $D_f/B \leq 1$. It is a good practice to use at least two methods and compare the computed values of $q_{ult.}$ If the two values do not compare well, it would be good to use a third calculation method (i.e. Hansen Method, Vesic Method).

The ultimate bearing capacity is the maximum foundation pressure that soil under the footing can support before failure in shear failure. In order to obtain the net ultimate bearing capacity ($q_{nu}$), which expresses the net maximum pressure that may be applied to the base of foundation, the overburden pressure at depth $D_f$ should be subtracted:

$$q_{nu} = q_{ult} - \gamma \, . D_f \qquad (3.4)$$

Dividing the net ultimate bearing capacity by a factor of safety, F, the net safe bearing capacity can be calculated. Net safe bearing capacity, $q_{n(safe)}$, is the maximum net intensity of loading that the soil can safely support without the risk of shear failure (Shroff & Shah, 2003) (Eq. 3.5). The vital point is not to confuse net safe bearing capacity with the allowable bearing capacity ($q_{all}$). Allowable bearing capacity embodies not only safety against shear but also acceptable settlement criteria and is used in foundation design.

$$q_{n(safe)} = \frac{q_{nu}}{F} \qquad (3.5)$$

where $q_{n(safe)}$ = net safe bearing capacity, kPa

$q_{nu}$ = net ultimate bearing capacity, kPa

F= factor of safety, (commonly 2.0~3.0 for apartments and office buildings) (Vesic, 1975)

#### 3.1.1.1 Effect of water table

The basic theory of bearing capacity is derived by assuming that the depth of water table from the ground surface is equal or greater than ($D_f$ + B). However, the presence of water table at any intermediate depth less than the depth ($D_f$ + B), the strength of the soil is affected due to the presence of the water table. In determining the effect of the water table on bearing capacity two cases may be considered.

*Case I;* $0 \leq d_w \leq D_f$ ($d_w$ is the depth of groundwater below ground surface)

For this case, the $\gamma_t D_f$ term should be changed to $\gamma_t d_w + (D_f - d_w)\gamma'$ and the term $\gamma_t$ associated with $N_\gamma$ should be replaced by $\gamma'$ ($\gamma'$ = effective unit weight of soil).

*Case II;* $D_f < d_w \leq D_f + B$

In this condition, ground water table is located below the bottom of the foundation. In such case, the term $\gamma_t D_f$ remains unchanged but the term $\gamma_t$ associated with $N_\gamma$ should be replaced by an average effective unit weight of soil, $\gamma_{av}$ (Das, 1999) ;

$$\gamma_{av} = \gamma' + \left(\frac{d_w - D_f}{B}\right)(\gamma_t - \gamma') \tag{3.6}$$

### 3.1.2 Bearing capacity of layered subsoil

The bearing capacity methods described in previous section presume that the soil underlying the foundation is uniform and extends to a great depth below the bottom of the foundation; however, this is not always the case. The underlying soil strata may be layered and may have different shear strength parameters. In the case of layered soil profile, the depth of failure surface and the bearing capacity of the footing are influenced.

For the case of foundation on layered Mohr-Coulomb soil, where stronger layer is underlain by weak soil, Meyerhof and Hanna (1978) developed a theory to estimate the ultimate bearing capacity of a shallow rough continuous foundation. According to their theory, at ultimate load per unit area, $q_{ult}$, the failure surface in soil will be as shown in Figure 3.3.

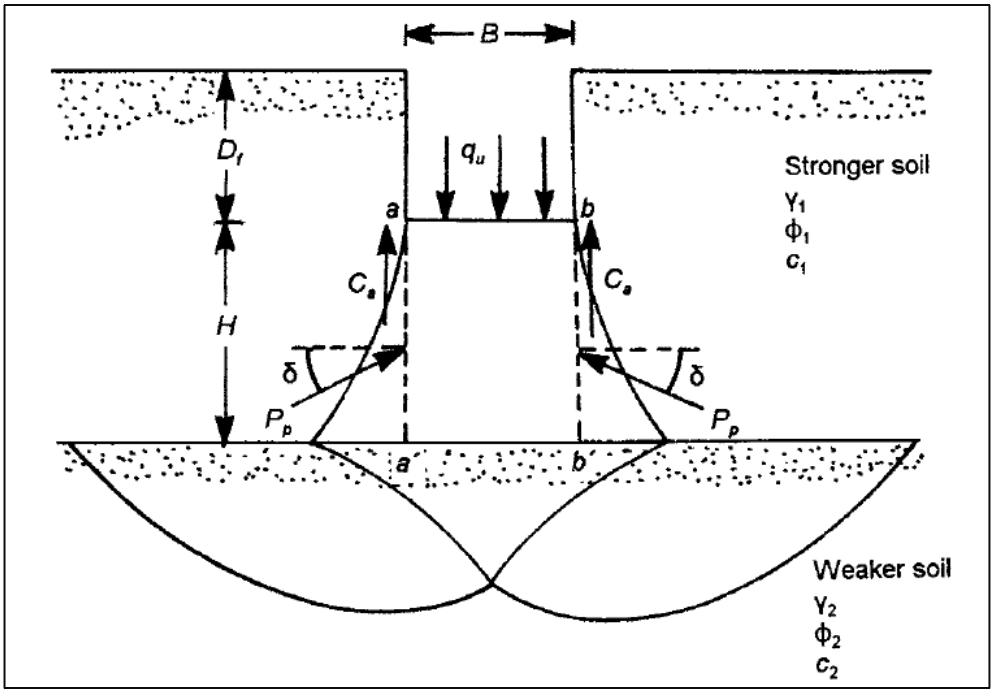



Figure 3.3 Rough continuous foundation on layered soil - stronger over weaker soil (Das, 1999).

If H, the thickness of the layer of soil below the footing, is relatively large then the entire failure surface will be within the top soil layer, and for this case the ultimate bearing capacity has been described previously. However, if the ratio H/B is relatively small, a punching shear failure will occur in the top (stronger) soil layer followed by a general shear failure in the bottom (weaker) layer (Das,1999). For this case, the ultimate bearing capacity of the shallow continuous foundation can be given as:

$$q_{ult} = q_b + \frac{2c_a H}{B} + \gamma_1 H^2 \left(1 + \frac{2D_f}{H}\right) \frac{K_s tan\text{ø}_1}{B} - \gamma_1 H \leq q_t \qquad (3.7)$$

where,

$$q_b = c_2 N_{c(2)} + \frac{1}{2} \gamma_2 B N_{\gamma(2)} + \gamma_1 (H + D_f)\, N_{q(2)} \qquad (3.8)$$

and

$$q_t = c_1 N_{c(1)} + \frac{1}{2} \gamma_1 B N_{\gamma(1)} + \gamma_1 D_f N_{q(1)} \qquad (3.9)$$

where H = height of the top layer

$D_f$ = vertical distance from ground surface to bottom of footing, m

B = width of the footing

$q_t$ = bearing capacity of the top soil layer

$q_b$ = bearing capacity of the bottom soil layer

$ø_1$ = angle of internal friction of top soil

$N_{c(1)}$, $N_{q(1)}$, $N_{\gamma(1)}$ = bearing capacity factors corresponding to soil friction angle $ø_1$ (Appendix Table B.4)

$N_{c(2)}$, $N_{q(2)}$, $N_{\gamma(2)}$ = bearing capacity factors for the bottom soil layer corresponding to soil friction angle $ø_2$ (Appendix Table B.4)

$c_{1,}$ $c_2$ = cohesion of the top and the bottom (weaker) layer of soil, respectively

$\gamma_{1,}$ $\gamma_2$ = unit weight of the top and the bottom soil layer, respectively

$c_a$ = unit adhesion (Appendix Figure B.2)

$K_s$ = punching shear coefficient (a function of $q_2/q_1$ ratio) (Appendix Figure B.3)

Note that $q_1$ and $q_2$ are the ultimate bearing capacities of a continuous surface foundation of width B under vertical load on homogenous beds of upper and lower soils, respectively, or (Das, 1999);

$$q_1 = c_1 N_{c(1)} + \frac{1}{2} \gamma_1 B N_{\gamma(1)} \qquad (3.10)$$

$$q_2 = c_2 N_{c(2)} + \frac{1}{2} \gamma_2 B N_{\gamma(2)} \qquad (3.11)$$

For the case of when a foundation is supported by a weaker soil layer underlain by a stronger soil at a shallow depth, as shown in the left-hand side of Figure 3.4, the failure surface at ultimate load will pass through both soil layers (Das, 1999).

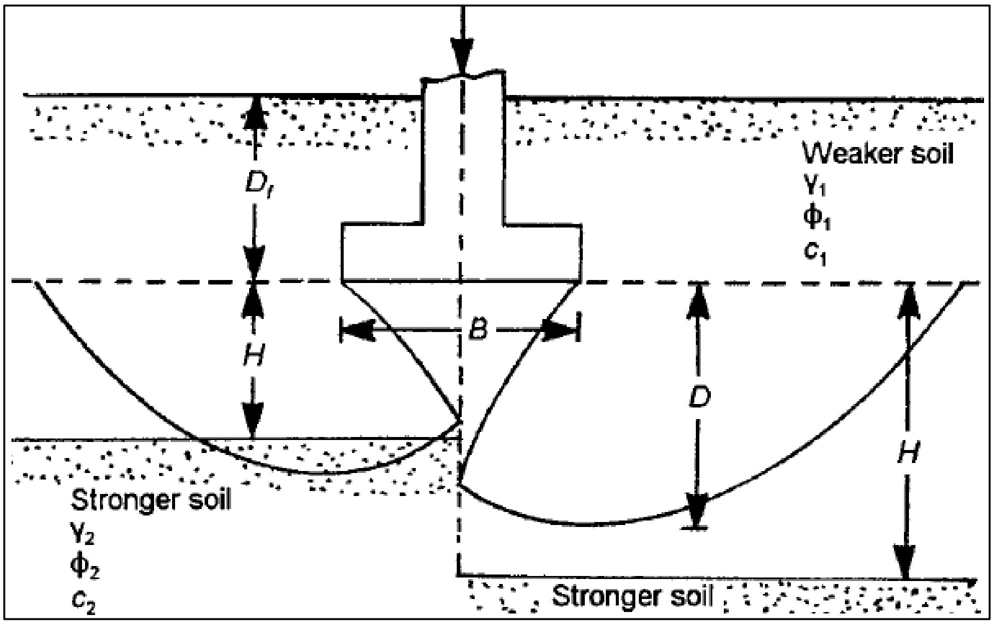


Figure 3.4 Foundation on weaker soil layer underlain by stronger layer (Das, 1999).

For estimating the ultimate bearing capacity of such foundations, Meyerhof (1974) and Meyerhof and Hanna (1978) proposed the following semi-empirical relationship.

$$q_{ult} = q_t + (q_b - q_t)\left(1 - \frac{H}{D}\right)^2 \geq q_t \qquad (3.12)$$

where D = depth of failure surface beneath the foundation in the thick bed of the upper weaker soil layer (The magnitude of *D*/*B* varies from 1 for loose sand and clay to about 2 for dense sands)

$q_t$ = ultimate bearing capacity in a thick bed of the upper soil layer

$q_b$ = ultimate bearing capacity in a thick bed of the lower soil layer

So

$$q_t = c_1 N_{c(1)} s_{c(1)} + \frac{1}{2}\gamma_1 B N_{\gamma(1)} s_{\gamma(1)} + \gamma_1 D_f N_{q(1)} s_{q(1)} \quad (3.13)$$

And

$$q_b = c_2 N_{c(2)} s_{c(2)} + \frac{1}{2}\gamma_2 B N_{\gamma(2)} s_{\gamma(2)} + \gamma_1 (H + D_f)\, N_{q(2)} s_{q(2)} \quad (3.14)$$

### 3.1.3 Bearing capacity of shallow foundations on rock

The bearing capacity of foundations founded on rock masses depends mostly on the ratio of joint spacing to foundation width, as well as intact and rock mass qualities, joint condition, rock type, and intact and mass rock strengths. Various empirical procedures for estimating allowable bearing capacity of shallow foundations on rock are available in the literature. Peck et al. (1974) suggest an empirical procedure based on the rock quality designation (RQD) index for estimating allowable bearing pressures of foundations on jointed rock. In this regard, the approach of Peck et al. (1974) uses the RQD directly to assess the allowable bearing stress ($q_{all}$), provided that the applied stress does not exceed the uniaxial compressive strength of the intact rock ($q_{all} < q_u$). The predicted bearing capacities by this method shall be used with the assumption that the foundation settlement does not exceed 12.7 mm (Peck et al., 1974). The RQD relationship is shown in Figure 3.5.

Another empirical approach is proposed by Carter and Kulhawy (1988) by which ultimate bearing capacity of fractured rock can be estimated. They suggest that the Hoek and Brown strength criterion for jointed rock masses can be used in the evaluation of bearing capacity. Their method is based on the unconfined compressive strength of the intact rock core sample and rock mass quality (For detailed information please see reference NCHPR, 2010). The ultimate bearing capacity of the strip footing may be evaluated from Equation 3.15 as:

$$q_{ult} = \left[\sqrt{s} + \left(m\sqrt{s} + s\right)^{0.5}\right] q_u \quad (3.15)$$

where $q_{ult}$ = ultimate bearing pressure, kPa

$q_u$ = uniaxial compressive strength of the intact rock, kPa

s and m = empirically determined strength parameters for the rock mass (Appendix Table B.7)

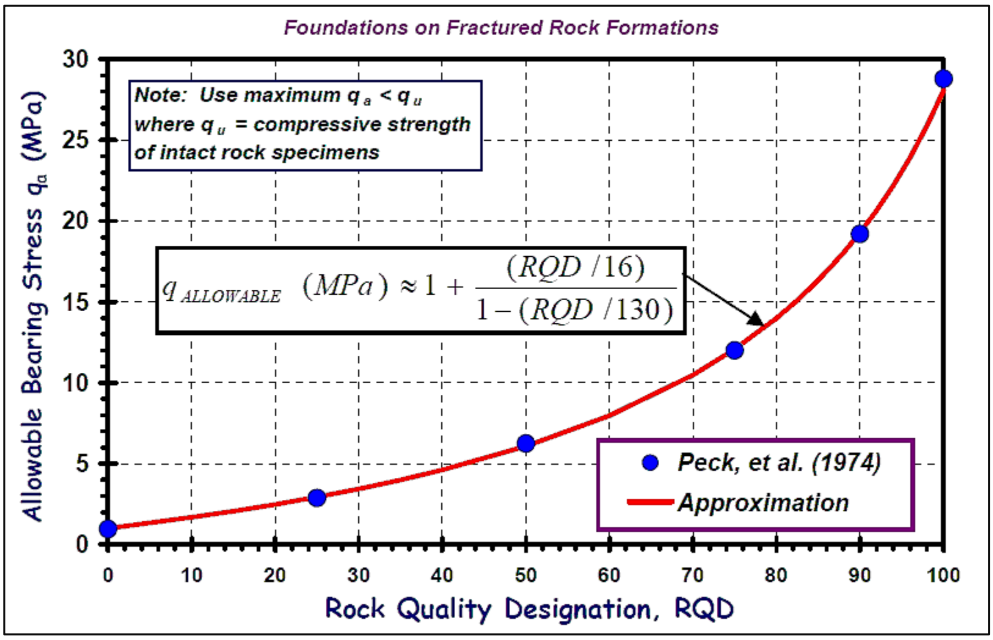



Figure 3.5 Allowable bearing stress on fractured rock from RQD (after Peck et al., 1974).

In order to obtain allowable bearing capacity, the ultimate capacity is divided by a safety factor which is generally dependent on RQD. It is common to use large safety factors in rock bearing capacity from 6 to 10 with the higher values for RQD less than about 0.75 (Bowles, 1996). When rock coring procedures result with no intact pieces (RQD=0), estimates of soil parameters (ø and c) from the Geological Strength Index (GSI) approach (Hoek et. al., 1995) may be used in traditional bearing capacity equations.

## 3.2 Bearing Capacity from Field Tests

In order to determine the bearing capacity of a foundation by using bearing capacity equations, experimental determination of shear strength parameters is necessary but it is better to remember that that field tests, if administered properly in the field, ensure accurate results and take precedence because they constitute to lowest level of disturbance (GDDA, 2005). In addition, for noncohesive foundation materials, where undisturbed sampling is usually impractical, field tests are the only way to estimate the material properties. So it is a common practice to estimate both the bearing capacity and other soil properties by using internationally agreed empirical correlations with field tests. The following in-situ tests may be used in determining bearing capacity:

- Standard Penetration Test (SPT)
- Cone Penetration Test (CPT)
- Pressuremeter Test (PMT)

### 3.2.1 Bearing capacity from SPT

The SPT is widely used to obtain the bearing capacity of soils directly. Meyerhof (1956, 1974) proposed equations for computing allowable bearing capacity. Since Meyerhof published equations, researchers have observed that its results are rather conservative. Bowles (1977) adjusted the Meyerhof equation for an approximate 50 percent increase in allowable bearing capacity to obtain the following:

$$q_{all} = 20NK_d\left(\frac{S}{25}\right) \qquad \text{(for B≤1.2 m)} \qquad (3.16)$$

$$q_{all} = 12.5NK_d\left(\frac{B+0.3}{B}\right)^2\left(\frac{S}{25}\right) \qquad \text{(for B>1.2 m)} \qquad (3.17)$$

where $q_{all}$ = allowable bearing pressure, kPa

$K_d$ = depth factor = $1 + 0.33(D_f/B) \leq 1.33$

S = tolerable settlement, mm

B = width of the footing, m

In these equations, corrections, including overburden stress effects, should be done on SPT "N" values and energy level adjustment should be 55 percent. N is the statistical average value for the footing influence zone of about 0.5B above footing base to at least 2B below (Bowles, 1996). Parry (1977) proposed computing the allowable bearing capacity of cohesionless soils as

$$q_{all} = 30N_{55} \qquad \text{(kPa)} \qquad (D_f \leq B) \qquad (3.18)$$

where $N_{55}$ is the average SPT value at a depth about 0.75B below the proposed base of the footing.

### 3.2.2 Bearing capacity from CPT

According to Meyerhof (1956) the allowable bearing capacity of foundations can be computed using Eqs. (3.19) and (3.20).

$$q_{all} = \frac{q_c}{15}\left(\frac{S}{25}\right) \qquad \text{(for } B \leq 1.2 \text{ m)} \qquad (3.19)$$

$$q_{all} = \frac{q_c}{25}\left(\frac{B+0.3}{B}\right)^2\left(\frac{S}{25}\right) \qquad \text{(for } B > 1.2 \text{ m)} \qquad (3.20)$$

where $q_{all}$ = allowable bearing pressure, kPa

$q_c$ = average value of cone penetration resistance measured at depths from footing base to 1.5B below the footing base, kPa

S = tolerable settlement, mm

B = width of the footing, m

Bowles (1988), by reference to Schmertmann's studies, suggests using the following relationships between ultimate bearing capacity and cone penetration resistance. For cohesionless soils one may use

Strip foundation; $q_{ult} = 28 - 0.0052(300 - q_c)^{1.5}$ (kg/cm$^2$) (3.21)

Square foundation; $q_{ult} = 48 - 0.009(300 - q_c)^{1.5}$ (kg/cm$^2$) (3.21a)

For clay one may use

Strip foundation; $q_{ult} = 2 + 0.28 q_c$ (kg/cm$^2$) (3.22)

Square foundation; $q_{ult} = 5 + 0.34 q_c$ (kg/cm$^2$) (3.22a)

where $q_c$ is averaged over depth interval from about B/2 above to 1.1B below the footing base. This approximation should be applicable for $D_f/B \leq 1.5$.

### 3.2.3 Bearing capacity from PMT

Menard (1965) proposed using the limit pressure measured in PMT to estimate ultimate bearing capacity:

$$q_{ult} = P_0 + k(P_{Le}{}^*) \quad (3.23)$$

where $q_{ult}$ = ultimate bearing pressure, kPa

$P_o$ = initial total vertical pressure at the foundation level, kPa

k = dimensionless bearing capacity coefficient (Appendix Figure B.4)

$P_L{}^*$ = net limit pressure = $P_L$-$P_{0h}$, kPa

$P_L$ = limit pressure (from test), kPa

$P_{0h}$= total horizontal stress at rest, kPa

$P_{Le}{}^*$ = equivalent net limit pressure near the foundation level, kPa

$$P_{Le}{}^* = \sqrt[n]{P_{L1}{}^* x\, P_{L2}{}^* x \ldots\, P_{Ln}{}^*} \quad (3.24)$$

where $P_{L1}$…, $P_{Ln}$ are the net limit pressures obtained from pressuremeter tests performed within the depth from 1.5B above to 1.5B below foundation level.

### 3.3 Foundation Settlements

The other primary consideration that affects the selection and design of foundations is foundation settlement. In the design of any foundation, both the safety against bearing capacity failure and the excessive settlement of the foundation must be taken into consideration. Settlement can be defined as the permanent downward displacement of the foundation (Das, 1999). The settlement of a foundation can have three components and results from one, or more likely, a combination of the following:

a) Immediate Settlement ($s_i$): It takes place during load application and is completed shortly after loading. This settlement may result from elastic deformation of the material supporting the foundation without any change in the moisture content. Immediate settlement analyses are used for all fine-grained soils including silts and clays with a degree of saturation $S<90$ percent and for all coarse-grained soils with a large coefficient of permeability [say, above $10^{-3}$ m/s] (Bowles, 1996).

b) Primary Consolidation Settlement ($s_c$): It is a time-dependent process and takes place as a result of expulsion of some pore water from soil as the loads are applied. Consolidation settlement analyses are used for all saturated, or nearly saturated, fine-grained soils (Bowles, 1996).

c) Secondary Consolidation Settlement ($s_{sec}$): It is due to structural reorientation of soil particles under constant loading. It is also referred as creep settlement and occurs after the completion of the primary consolidation settlement. It occurs in organic and sensitive soils.

### 3.3.1 Immediate settlement calculations

The immediate vertical displacement ($s_i$) of a semi-infinite, homogeneous and isotropic mass under a uniformly loaded area can be calculated from an equation from the Theory of Elasticity as follows:

$$s_i = q_0 B' \frac{1-\mu^2}{E_s} m I_s \tag{3.25}$$

where $s_i$ = immediate settlement of the footing, m.

$q_o$ = net vertical footing pressure, kPa

B' = least lateral dimension of contributing base area, m,

B' = B/2 for center; = B for corner.

$\mu$ = Poisson's ratio (Appendix Table B.8)

$E_s$ = Elasticity modulus (Appendix Table B.9)

m = number of corners contributing to settlement,

m = 4 for center; = 1 at a corner

$I_s$ = shape and rigidity factor (dimensionless) (Appendix Table B.10)

Bowles (1996) states that the stratum depth (H) actually causing settlement is not infinitely deep, but is either equal to 5B, where B is the least total lateral dimension of base, or is the depth to where a hard stratum is encountered. Bowles (1996) suggests taking "hard" as that where $E_s$ in the hard layer is about $10E_s$ of the adjacent upper layer. Also in most cases, the modulus in the formula is not constant throughout the depth of soil. In order to obtain quite good settlement estimates, the use of weighted average $E_s$ over the influence depth H would be correct.

An alternative immediate settlement calculation method for estimating the average elastic settlement of a uniformly loaded flexible footing on a saturated clay ($\mu$=0.5) is proposed by Janbu et al., (1956). The equation for computing the settlement may be expressed as

$$s_i = \mu_0 \mu_1 \frac{qB}{E_s} \tag{3.26}$$

where $s_i$ = immediate settlement of the footing

$\mu_0$ and $\mu_1$ = empirical factors dependent on the foundation geometry (Appendix Figure B.5)

q, B, $E_s$ are described previously

### 3.3.2 Primary consolidation settlement calculations

Consolidation settlement of saturated cohesive soils is calculated on the basis of parameters obtained in the oedometer test. Equation 3.27 is used for one-dimensional consolidation ($s_{oed}$) settlements of normally consolidated soils.

$$s_{oed} = \frac{C_c\, H}{1+e_0} log \frac{\sigma'_o + \Delta\sigma_{av}}{\sigma'_o} \tag{3.27}$$

where $s_{oed}$ = One-dimensional oedometer consolidation settlement, m

$C_c$ = compression index from e versus log σ' plot

$e_0$ = in situ void ratio in the stratum where $C_c$ was obtained

H = Stratum thickness, m. If the stratum is very thick (say $6^+$ m) it should be subdivided into several sublayers of $H_i$ = 2 to 3 m, with each having its own $e_0$ and $C_c$ (Bowles, 1996)

$\sigma'_0$ = Effective overburden pressure at midheight of H

$\Delta\sigma_{av}$ = average stress increase from the foundation loads at the middle of the clay layer

For soils in an overconsolidated state, the settlement calculation is similar to that of normally consolidated ones, except that now some compression will be along the recompression line (recompression index $C_r$ is used for calculations) and then it follows the normal consolidation line. In Appendix Table B.11 several correlation equations are listed that might be used to make compression index estimates.

The alternative form of one-dimensional consolidation settlement calculation is given in Equation 3.28:

$$s_{oed} = \Sigma \; (H \; . m_v \; . \Delta\sigma') \qquad (3.28)$$

where $s_{oed}$ = One-dimensional consolidation settlement, m

H = Stratum thickness, m (If H is very thick, it should be subdivided as stated before.)

$m_v$ = coefficient of volume compressibility obtained from oedometer test

$\Delta\sigma'$ = average effective stress increase at the middle of the clay layer

is also used. It should be taken into consideration that the $m_v$ value varies with the range of vertical stress, and in the calculations, the $m_v$ value that corresponds to stress increase caused by foundation should be used. The average vertical stress increase in the stratum of thickness H due to foundation load can be obtained by using numerical integration process or by approximate considerations such as trapezoidal rule.

Correction is necessary for these two methods because compressibility parameters obtained from oedometer test is one-dimensional. However at site this condition is not valid and deformations are three-dimensional. The correction is made by applying a 'geological factor' $\mu_g$ to the one-dimensional oedometer settlement by the following expression.

$$s_c = \mu_g \; x \; S_{oed} \qquad (3.29)$$

where $s_c$ = Three dimensional consolidation settlement, m

Published values of $\mu_g$ have been based on comparison of the settlement of actual structures with computations made from laboratory oedometer tests. Values established by Skempton and Bjerrum (1957) are shown in Appendix Table B.12.

### 3.3.3 Secondary consolidation settlement calculations

The secondary consolidation settlement, $s_{sec}$, can be calculated as

$$s_{sec} = H \,. C_{\alpha} . log \frac{t}{t_p} \tag{3.30}$$

where $s_{sec}$ = Secondary consolidation settlement, m

$C_{\alpha}$ = secondary compression index (Appendix Table B.11)

H = thickness of consolidating stratum at the end of primary consolidation

$t_p$ = time corresponding to the completion of the primary consolidation

t = time at which the secondary compression settlement is to be computed

## 3.4 Structural Tolerance to Settlement and Differential Settlements

Both the total downward settlement and the differential settlement of various parts of a structure should be within acceptable limits to not to cause structural or architectural distress. Hence, it is important to determine the total settlement and differential settlements of a structure. Differential settlement is relative settlement between different parts of a structure and occurs due to one or more of the following reasons (Birand et al., 2002);

a. Variations in soil strata
b. Variations in foundation loading
c. Large loaded areas on flexible foundations
d. Differences in time of construction of adjacent parts of a structure
e. Variations in site conditions (old and new parts)

Another parameter that may be useful in the design of the foundation is the maximum angular distortion (δ/L), defined as the differential settlement between two points divided by the distance between them (Day, 2006). In Table 3.2, limiting values for maximum settlement, maximum differential settlement, and maximum angular distortion to be used for building purposes, are summarized.

Table 3.2 Tolerable differential settlement of buildings (Skempton and MacDonald, 1956)

| | |
|---|---|
| Maximum settlement, | |
| Isolated foundations in sand | 51 mm |
| Isolated foundations in clay | 76 mm |
| Raft in sand | 51–76 mm |
| Raft in clay | 76–127 mm |
| Maximum differential settlement, | |
| In sand | 32 mm |
| In clay | 45 mm |
| Maximum angular distortion, | 1/300 |

## 3.5 Modulus of Subgrade Reaction

The modulus of subgrade reaction, $k_s$, is a conceptual relationship between soil pressure and deflection that is widely used in the structural analysis of foundation members (Bowles, 1996) (Appendix Figure B.6). It is most commonly determined from plate loading tests and is affected by factors such as size, shape and embedded depth of the plate. Terzaghi (1955) proposed that $k_s$ for footings of width B could be obtained from plate load test data using the following equations:

For footings on clay;

$$k_s = k_1 \frac{B_1}{B} \tag{3.31}$$

For footings on sand;

$$k_s = k_1 \frac{(B+B_1)^2}{2B} \tag{3.32}$$

where $k_s$ = desired value of modulus of subgrade reaction for full size foundation

$k_1$ = value obtained from a plate-load test

$B_1$ = side dimension of the square base used in the load test. In most cases $B_1$=0.3 m, but whatever $B_1$ dimension was used should be input. Also this equation disrupts when $B/B_1 \approx >3$

On large projects it may be feasible to construct a test section and perform plate load tests but because the plate load test is time consuming and expensive, is not commonly run in practice. Besides the plate load test, the subgrade modulus can also be obtained from empirical correlations. One useful correlation proposed for sandy soils by Scott (1981) between coefficient of subgrade reaction (k) and corrected SPT blow count $(N_1)_{45}$ is:

$$k_1 = 1.8\,(N_1)_{45} \qquad (\mathrm{MN/m^3}) \qquad (3.33)$$

Another correlation is suggested by Bowles (1996) in which ultimate bearing capacity $q_{ult}$ furnished by the geotechnical consultant is used for approximating $k_s$. The equation can be expressed as:

$$k_s = 40\,q_{ult} \qquad (\mathrm{kN/m^3}) \qquad (3.34)$$

where $q_{ult}$ is furnished in kPa. This equation is based on the ultimate soil pressure causing a settlement of $\Delta H$=0.0254 m and $k_s$ is $q_{ult}$ / $\Delta H$. For $\Delta H$=6, 12, 20 mm, the factor 40 can be adjusted to 160, 83, 50 respectively. Table 3.3 may be used as guide and for comparison when using approximate equations.

Table 3.3 Range of modulus of subgrade reaction (Bowles, 1996)

| Type of Soil | $k_s$ (kN/m$^3$) |
|---|---|
| Loose sand | 4,800–16,000 |
| Medium dense sand | 9,600–80,000 |
| Dense sand | 64,000–128,000 |
| Clayey medium dense sand | 32,000–80,000 |
| Silty medium dense sand | 24,000–48,000 |
| Clayey soil: | |
| $q_u \leq 200$ N/mm$^2$ | 12,000–24,000 |
| $200 < q_u \leq 400$ N/mm$^2$ | 24,000–48,000 |
| $q_u > 800$ N/mm$^2$ | > 48,000 |

# CHAPTER 4

# GEOTECHNICAL REPORT

After completion of the field and laboratory works of a geotechnical investigation, the collected data is evaluated, interpreted and presented in a report. The preparation of geotechnical investigation report requires special knowledge and skills therefore they must be prepared by an appropriately qualified professional, geotechnical engineer.

## 4.1 What is Geotechnical Report?

The geotechnical report is the tool used to communicate the site conditions, design and construction recommendations to the contractor, design and construction personnel. A geotechnical report typically provides an assessment of existing subsurface conditions at a project site, by presenting, describing and summarizing the procedures and findings of any geotechnical analyses performed. In addition, the report provides appropriate recommendations for design and construction of foundations, earth retaining structures, embankments, cuts, and other required facilities (Mayne et al., 2001). The report also includes background information about site conditions, geologic features, work scope and data presentation obtained from field and laboratory tests. The report serves as the permanent record of all geotechnical data known to be pertinent to the project and is referred to throughout the design, construction, and service life of the project (NDOT, 2005). Hence, it is very important that all the obtained information, calculations and recommendations should be presented in a logical and orderly format in the geotechnical report. Most companies have their own format for presenting their reports, which makes it difficult to understand and control the reports. In Turkey, the format of Ministry of Environment and Urbanism is used for geotechnical investigation reports of ordinary buildings (Table 4.1). In Table 4.1, the major elements of the report contents are presented in subtitles.

## 4.2 General Information

In the first part of the report, the scope of the work is introduced and general information about the investigation site is presented. The purpose of the investigation should be explained briefly and site location should clearly be described. Information about the size and shape of the site and its location relative to any roads and access routes should also be presented. The geomorphological and environmental information, project information including formal name of the project, dimensions and purpose of use of the proposed construction, information from previous reports, and general geology information of the site are given in the first part.

Table 4.1 General report format of geotechnical investigations for ordinary buildings (Ministry of Environment and Urbanism, 2005)

COVER PAGE
TABLE OF CONTENTS

1. GENERAL INFORMATION
    - 1.1 Objectives and Scope of Study
    - 1.2 Introduction of Study Area
        - 1.2.1 Geomorphological and Environmental Information
        - 1.2.2 Information about the Project
        - 1.2.3 Development Plan Status
        - 1.2.4 Previous Site Investigations
    - 1.3 Geology
        - 1.3.1 General Geology
        - 1.3.2 Engineering Geology of the Study Area
2. FIELD STUDIES AND TESTS
    - 2.1 Description of field and laboratory working methods and equipment used in
    - 2.2 Trial Pits
    - 2.3 Drilling Wells
    - 2.4 Groundwater and Surface Water
    - 2.5 Field Experiments
        - 2.5.1 Standard Penetration Test
        - 2.5.2 Cone Penetration Test
        - 2.5.3 Pressuremeter Test
        - 2.5.4 Vane Test
        - 2.5.5 Plate Loading Test
        - 2.5.6 Geophysical Surveys
3. LABORATORY TESTS and ANALYSES
    - 3.1 Determination of Soil Index/ Physical Characteristics
    - 3.2 Determination of Mechanical Properties of Soils
    - 3.3 Determination of Mechanical Properties of Rocks
4. ENGINEERING ANALYSES and EVALUATIONS
    - 4.1 Examination of Building-Soil Relationship
    - 4.2 Evaluation of Soil and Rock Types
        - 4.2.1 Classification of Weathered Rocks and Soils
        - 4.2.2 Classification of Rocks
        - 4.2.3 Soil Profile Interpretation
        - 4.2.4 Liquefaction Analysis
        - 4.2.5 Evaluation of Shrink/Swell Potential
        - 4.2.6 Assessment of Karst Cavities
        - 4.2.7 Evaluation of Foundation Soil Properties
        - 4.2.8 Slope Stability Analysis
        - 4.2.9 Interpretation of Security of Excavation and Necessary Measures
        - 4.2.10 Natural Disaster Risk Assessment
5. CONCLUSION and RECOMMENDATIONS
6. REFERENCES
7. APPENDICES

### 4.3 Field Studies and Tests

This section should describe the procedures followed in field studies and tests. It should contain an identification of each fieldwork technique employed, the locations and elevations at which each was used, the range of depths to which each was taken and the dates over which this work was done (AGS, 2005). Any supplementary references which define procedures for each investigation technique and references appropriate to interpretation should be given. Both the constraints on access and the difficulties that are encountered during each field test should be explained.

The sampling strategy should be stated, the types of samples taken should be identified and their transportation and storage conditions should be described. Groundwater conditions encountered during the investigation should also be presented. Monitoring records, with their location information, should be given in this section.

### 4.4 Laboratory Tests and Experiments

In this part of the report, the type and number of tests, as well as the relevant test reference numbers together with the location in the report should be presented. The conformity of laboratory tests to ground conditions and the reasons of which laboratory test is chosen should declared. Test results should be presented, together with tabulations of the results of all tests of the same type and if appropriate the results of different test types on the same material (AGS, 2005). The detailed results and graphs of the laboratory tests are to be presented in tabular form in the appendices part of the report.

### 4.5 Engineering Analyses and Evaluations

The purpose of this section is to provide enough supporting analyses, computations and discussions so that the basis for the geotechnical conclusions is clear to the reader. Selection of design parameters, whether values are determined by laboratory/field testing or through other approaches such as correlations, should be discussed and any assumptions should be clarified. The methods of analyses, such as Meyerhof’s bearing capacity analysis or Terzaghi's consolidation theory for settlement analysis, should be identified. When applicable, analyses for alternate foundations including spread footings, driven piles and drilled shafts should be provided (NDOT, 2005).

### 4.6 Conclusions and Recommendations

The report should contain a general conclusion or opinion as to the adequacy of the site for the intended use, conclusions as to the site’s overall stability, and ability of the onsite materials to support the proposed structures (Technical Guidelines for Geotechnical Reports, 1993). The reader of the report should be able to understand the geotechnical settings and possible engineering limitations after reading the report. The detailed recommendations and discussions, such as (i) whether the proposed development will adversely affect the current state of stability of adjoining land, (ii) whether the proposed development should allow cuts and fills and if so, to what depth, (iii) whether any special design features are required to stabilize or maintain the stability of the subject land, or portions of the subject land; should be submitted. Also, construction recommendations should be included at the end of the report for a clear description telling the contractor what to or not to do during construction.

### 4.7 References and Appendices

After a formal list of references, appendices are to be presented. In typical appendices, topographic site plan, test location plans (Appendix Figure C.1), geologic cross-sections and idealized soil profile (Appendix Figure C.2), subsurface exploration data in the form of borehole logs, and finally laboratory test and instrumentation results are to be presented in a complete manner.

# CHAPTER 5

# EVALUATION OF GEOTECHNICAL REPORTS

In this study, a total number of 60 geotechnical reports from all central municipalities of Ankara (Altındağ, Çankaya, Etimesgut, Gölbaşı, Keçiören, Mamak, Sincan and Yenimahalle) and 6 geotechnical reports from other city municipalities (Antalya, Çankırı, Çorum, İskenderun, Kırıkkale, and Konya) are randomly collected and evaluated. In order to clarify the word "randomly", it can be said that these collected reports are not specifically chosen for being deficient. They are casually collected from current archives of municipalities by taking the necessary permissions from the related people and institutions, which is the most challenging part of this study. All the evaluated reports are listed in Appendix Table D.1 and the locations of reports from Ankara municipalities are illustrated in Appendix Figure D.1. In the collected reports, main soil types of the study areas are specified as 37% rock, 29% high plasticity clay, 23% low plasticity clay, 5% sand, 4% silt and 2% gravel.

## 5.1 Methodology

The evaluation of the collected reports is performed according to 36 different criteria. These criteria are shortly explained item by item below:

- *Is there project information?*

By this criterion, whether project information about proposed structure such as, site plan, building dimensions, number of floors, is presented in reports is checked.

- *Are boreholes properly distributed over the study area?*

With this criterion, the distribution plan of subsurface exploration locations on the study and building area is checked.

- *Is borehole frequency adequate?*

In evaluation of this criterion, minimum 2 boreholes for building area smaller than 500 $m^2$, and one extra borehole for each additional 250 $m^2$ is adopted as necessary. In reports in which project information is not presented, the building area is assumed to be one-third of the study area.

- *Are borehole depths adequate?*

In this point of evaluation, suggestions of Özdemir (2005), which are stated in section 2.1.3.1, are taken into consideration.

- *Are coordinate and elevation data of sub-surface explorations recorded?*

This audit question is incorporated in this study because this information is important in interpreting cross-section between sub-surface locations in a good manner. It can be also regarded as a proof of the undertaken explorations.

- *Are undisturbed soil samples taken from each layer?*

By this criterion, whether undisturbed samples are obtained from each defined soil/rock layer is checked.

- *Are core samples obtained from rock masses?*

With this criterion, it is controlled that whether core samples are obtained from rock masses or not.

- *Is the number of undisturbed soil samples or cores sufficient?*

If there are at least two samples taken from each identified appropriate layer is checked by this query.

- *Are core recovery parameters determined?*

Whether and how often core recovery parameters (TCR and RQD) of rock samples, are determined in field studies and specified in the reports.

- *Are the field experiments suitable for the ground they were applied?*

In accordance with the borehole logs, the control of suitability of the in-situ tests with the ground they were applied is done by this criterion. The most appropriate in-situ tests for the related ground conditions are explained in section 2.1.4.

- *Is information about level of groundwater presented?*

By this criterion, it is controlled that whether groundwater level of the study area is measured or not.

- *Is the frequency of SPT sufficient in a borehole?*

By this criterion, it is checked that whether SPT is performed in boreholes at regular intervals of 1.5 m.

- *Are the SPT correction factors applied to raw data?*

This criterion is used to evaluate the implementation of necessary correction factors, which are discussed in section 2.1.4, on obtained field SPT raw data.

- *Are the laboratory tests performed for each layer? (UD. core)*

By this criterion, laboratory tests, which are only able to be performed on undisturbed soil samples and rock cores, are checked. A minimum number of two tests for each layer is the basis of this criterion.

- *Are the laboratory tests performed for each layer? (index)*

With this audit question, it is checked that whether “index tests”, which are specified in Table 2.7, is performed for each layer.

- *Is hydrometer test performed on soil samples containing high ratio of fine particles?*

The frequency of implementation of hydrometer test on soil samples containing fine particles more than 70 percent is assessed by this criterion.

- *Is consolidation test conducted?*

The interest point of this criterion is, if consolidation test is conducted on proper/necessary samples.

- *Is unload-reload cycle performed in consolidation test?*

The aim of this inquiry is to determine that how often unload-reload cycle is implemented within the scope of consolidation tests.

- *Are swelling pressure and percentage data obtained from consolidation test?*

By this criterion, the information obtained and presented about the swelling properties of the ground is investigated.

- *Is any other parameter except from the coefficient of volume of compressibility ($m_v$) obtained from consolidation test?*

Whether other parameters, such as compression/swelling/reloading index/ratio are presented in consolidation test results are examined by this criterion.

- *Is triaxial test conducted?*

Whether triaxial test is performed or not on necessary samples is controlled with this question.

- *Are there 3 Mohr's circles in determining failure envelope?*

Although two compression test results are theoretically sufficient for determining the failure envelope, use of at least three Mohr's circles is suggested, as stated in section 2.1.5.1. By this interrogation, the number of Mohr's circles used in determining the failure envelope is controlled.

- *Are Mohr's circles that are used in determining failure envelope far enough away from each other?*

For a proper failure envelope, it is necessary that Mohr's circles should be distant from each other. Otherwise, small errors in measured stresses may cause large errors in the strength parameters. For this reason, it is checked that whether center points of Mohr's circles are not located in the area of former circles.

- *Is failure envelope properly drawn?*

This examination is concentrated on whether the failure envelope is the best tangent of Mohr's circles and whether the failure envelope is horizontal for a completely saturated cohesive soil in an UU test.

- *Which laboratory tests are conducted on core samples?*

By this question, the frequency of laboratory tests on rock core specimens (point load test, uniaxial (unconfined) compression test, triaxial (confined) compression test, etc.) is investigated.

- *Is idealized soil profile created?*

This question is about, whether an idealized sub-surface profile(s) showing the differentiation of the various formations of the study area is constructed or not.

- *Are the strength parameters of soil obtained with more than one way and get averaged?*

By this criterion, it is tried to be determined that how the strength parameters of soils or rocks ($c_u$, c, ø, E) are obtained, using both the laboratory test and in-situ test results or adhering to only a single test or method.

- *What is the bearing capacity calculation method?*

It is investigated that which formula or method for calculating the bearing capacity of soils and rocks is used more intensively.

- *Are SPT results used in determining bearing capacity of soil?*

Whether the corrected standard penetration test results are used in design calculations is controlled by this query. The reason of why specifically "SPT results" is stated in query will be clarified in section 5.2.5.

- *Is bearing capacity calculated from allowed settlement consideration?*

Since allowable bearing capacity comprises not only safety against shear but also acceptable settlement criteria, technical calculations in collected reports are controlled in this respect.

- *Are bearing capacity and settlement calculations correct?*

The calculations done for the two major causes for foundation failure, bearing capacity and settlement, is tested by this audit question.

- *Are bearing capacity and settlement calculations calculated for the same foundation dimensions?*

In order to represent meaningful information to design engineer, it is necessary to calculate both the bearing capacity and settlement for the same foundation dimensions. The validity of this keynote is examined by this criterion.

- *Is the depth of foundation used in calculations reasonable?*

By this criterion, the foundation depth used in technical calculations is checked in accordance with the project information presented in reports. For example, for a building with a basement floor, a foundation depth of 1 m from surface level, or a depth of 7 m for a building without a basement, are accepted as incorrect.

- *Is stress dissipation calculated?*

It is checked that whether stress dissipation with depth is computed and used in calculations, by this criterion.

- *Are the foundation dimensions taken into consideration in determining the modulus of subgrade reaction?*

As it is clearly stated in question, it is checked that whether foundation dimensions are considered in determining the subgrade modulus.

- *Is the subgrade modulus found by using correlations or tables?*

The frequency of using correlations or tables in estimating subgrade modulus is explored by this inquiry.

The results of the assessments are collected in a table. The full state of the prepared table for this study is presented in Appendix Table D.2 on the back side of the thesis in CD. In order to show a proper representation of the results, the complete table is turned into small summary tables and the evaluation results are shown in the following tables in parts.

## 5.2 Results

### 5.2.1 Discussions on results of desk study and subsurface investigations

The compliance of subsurface investigations with the construction project, the distribution and frequency of the investigations, and the coordinate and the depth information of the explorations have been checked by the questions presented in Table 5.1, in accordance with the principles described sections 2.1.1 and 2.1.3.

Table 5.1 Criteria and inspection results on desk study and subsurface investigations

| | **YES** | **NO** | **OTHER** |
|---|---|---|---|
| Is there project information? (Site plan, building dimensions, number of floors) | 9% | 14% | 77% (only floor information) |
| Are boreholes properly distributed over the study area? | 14% | 7% | 79% (no information) |
| Is borehole frequency adequate? | 73% | 27% | - |
| Are borehole depths adequate? | 32% | 68% | - |
| Are coordinate and elevation data of sub-surface explorations recorded? | 2% | 98% | - |

When Table 5.1 is analyzed, it is seen that the desk studies and field survey plans are insufficient in current practice. Information, such as the sitting area of the proposed structure, dimensions, numbers of floors, basement-foundation depth are only presented in 9% of the reports and the reports that have no such information are 14%. In 77% of the reports, this section is glossed over by just giving number of floors only. The same negligence is continued while establishing the drilling location plans, frequency and the depth of drillings. It is striking that coordinate and elevation information for subsurface explorations is given in only one of the reports, and in 79% of the reports, where the subsurface explorations were conducted is not shown on a scaled plan. Also, in a significant proportion of the field studies the frequency of explorations is insufficient. Furthermore, more than two thirds of drilling studies do not extend to the required depth.

### 5.2.2 Discussions on results of sampling

The number and frequency of samples taken from the site for the laboratory experiments are checked by also considering the soil structure of the site. The criteria and the evaluation results are presented in Table 5.2.

Table 5.2 Criteria related to sampling and core recovery, and the percentage values of the results

| | **YES** | **NO** |
|---|---|---|
| Are undisturbed soil samples taken from each layer? | 7% | 93% |
| Are core samples obtained from rock masses? | 86% | 14% |
| Is the number of undisturbed soil samples or cores sufficient? | 17% | 83% |
| Are core recovery parameters determined? (TCR, RQD) | 100% RQD, 75% TCR | |

Table 5.2 shows us that the sampling studies do not comply with relevant rules. A frequently seen condition on the subject of taking undisturbed samples is to obtain samples at a depth of two to three meters and not to obtain any additional samples from deeper depths, even if this sole sample is above the foundation level. With a starting point below foundation level, undisturbed sampling at regular intervals, and if stratification is present, sampling from each proper layer is necessary. However, in nearly all reports it seems like there is no awareness of this. Although, rock core samples are mostly taken from rock formations, the number of core samples taken is insufficient. The situation is same with that of undisturbed sampling, only taking a core from a level close to surface and not taking any from greater depths. 83% of the reports are not sufficient in undisturbed and core sampling. Core recovery parameters, RQD and TCR, are indicated in most of the reports, however, the type of coring equipment, which affects these parameters generally goes unspecified.

### 5.2.3 Discussions on results of in-situ tests

The experimental data of in-situ tests, presented in geotechnical reports, are examined not only according to their properness to ground they were applied, but also their number, frequency and correction states of obtained raw data are controlled. Assessments are done by considering the soil-ground structure. Results are presented in Table 5.3.

Table 5.3 Criteria related to in-situ tests and the percentage values of the examination results

| | **YES** | **NO** |
|---|---|---|
| Are the field experiments suitable for the ground they were applied? | 87% | 13% |
| Is information about level of groundwater presented? | 70% | 30% |
| Is the frequency of SPT sufficient in a borehole? | 79% | 21% |
| Are the SPT correction factors applied to raw data? | 29% | 71% |

As a result of the examinations related to in-situ tests, it is seen that no test other than SPT is conducted. In-situ tests suitable for the study area are not selected, implemented and standard penetration test is conducted in all kinds of soil and rock. Whereas, as discussed in section 2.1.4, the most suitable in-situ test method for the soil structure should be selected and properly applied. A promising 87% of the reports answer “yes” to the question “Are the field experiments suitable for the ground they were applied?”. This is not necessarily because people are adept in selecting the suitable field test type, but more likely it is because the SPT is suitable for a great variety of ground conditions. However, SPT is not particularly useful in the characterization of gravel deposits and soft clays. The percentage of insufficient standard penetration tests in boreholes is 21, which shows us that SPT is not occasionally performed regularly on 1.5 m intervals. In the reports of 71%, no correction factor is applied to the raw field SPT N-value and even information about the energy level of the test system used is not given. In thirty percent of the reports information about groundwater level is not submitted. Additionally in none of the studies, short or long term changes of groundwater level is measured or discussed.

### 5.2.4 Discussions on results of laboratory experiments

At the part of the study related to laboratory experiments, the tests are controlled whether they meet the project requirements and also whether they are compatible with soil structure or not. In addition, number and frequency of laboratory tests, and the experiment results presented in reports are checked. The results of examination are shown in Table 5.4.

By examining the table containing the criteria and results related to laboratory experiments, it can safely be said that, as in the previous stages, an adequate level of awareness does not exist in current practice. As a result of not taking sufficient number of undisturbed samples, it is observed that laboratory tests for determining engineering properties of soil is not performed in necessary numbers. The hydrometer test is performed on only 3% of samples in which the fine particles are identified at a high ratio. Consolidation test is not conducted on almost one-third of the appropriate samples and none of the performed tests have an unload-reload cycle. Additionally, triaxial compression test is not conducted on the appropriate samples of 51%. None of the triaxial test results includes three Mohr's circles. Even though, two circles is sufficient for determining the failure envelope, use of at least three Mohr's circles, each obtained at different lateral pressure, would prevent variability in experimental results. Additionally, circles that are used in determining failure envelope are not away from each other in 33% of the test results.

Table 5.4 Criteria related to laboratory experiments and the results of inspection in percentage

| | **YES** | **NO** |
|---|---|---|
| Are the laboratory tests performed for each layer? (two tests for each layer)(UD, core) | 3% | 97% |
| Are the laboratory tests performed for each layer? (two tests for each layer)(Index) | 17% | 83% |
| Is hydrometer test performed on soil samples containing high ratio of fine particles? | 3% | 97% |
| Is consolidation test conducted? | 70% | 30% |
| Is unload-reload cycle performed in consolidation test? | 0% | 100% |
| Is any other parameter except from the coefficient of volume comp. ($m_v$) obtained from consolidation test? | 0% | 100% |
| Is Triaxial test conducted? | 49% | 51% |
| Are there 3 Mohr's circles in determining failure envelope? | 0% | 100% |
| Are Mohr's circles that used in determining failure envelope far enough away from each other? | 67% | 33% |
| According to triaxial test results, is the failure envelope properly drawn? | 83% | 17% |
| Are swelling pressure and percentage obtained from consolidation tests? | 62% only percentage, 38% both of them | |
| Which laboratory tests are conducted on core samples? (Point load test, Uniaxial compressive strength) | 96% Point load test, 4% Uniaxial comp. | |

Whether the failure envelope is properly drawn or not is also examined. As a result, 17% of the envelopes are not properly drawn. The uniaxial compressive strength of rock core samples is mostly obtained by the indirect method such as point load test. The usage frequency of uniaxial compression test, which is more expensive but more accurate, is remains at 4% of the reports that characterize rock formations.

### 5.2.5 Discussions on results of foundation calculations

Within the scope of the study, the bearing capacity and settlement calculations presented in geotechnical reports are examined, and questioning of whether the results obtained from laboratory and field experiments are properly used in technical calculations is made. The resulting percentage values are presented in Table 5.5.

Table 5.5 Criteria related to foundation design and the results of inspection in percentage

| | **YES** | **NO** |
|---|---|---|
| Is idealized soil profile created? | 4% | 96% |
| Are the strength parameters ($c_u$, c, ø, E) of soil obtained with more than one way and get averaged? | 9% | 91% |
| Are SPT results used in determining bearing capacity of soil? | 32% | 68% |
| Is bearing capacity calculated from allowed settlement consideration? | 22% | 78% |
| Are bearing capacity and settlement calculations correct? | 67% | 33% |
| Are bearing capacity and settlement calculations calculated for the same foundation dimensions? | 29% | 71% |
| Is the depth of foundation used in calculations reasonable? | 67% | 33% |
| Is stress dissipation calculated? | 2% | 98% |
| Are the foundation dimensions taken into consideration in determining the modulus of subgrade reaction? | 0% | 100% |
| Is the subgrade modulus found by using correlations or tables? | 5% correlation, 95% table | |

As shown in Table 5.5, an idealized soil profile for the study area is being prepared and submitted in only three of the reports. In 91% of the reports, geotechnical design parameters, which are used in calculations and directly affect the design principles, are obtained by adhering to only a single test or method. As a result of investigations, no any other in-situ test other than SPT is encountered. In addition, in 68% of the reports, which have SPT in their contents, SPT results are not used in design calculations. Disregarding field test results, which reliably reflect the in-situ properties of underlying soil, is really surprising. If this were the proper conduct, there would be no need to perform in-situ tests that would not be used. Another mistake observed in reports is related to allowable bearing capacity values. The allowable bearing capacity is directly found by dividing the ultimate strength capacity by a safety factor. However, allowable bearing capacity embodies not only safety against shear but also acceptable settlement criteria. In 78% of the reports, acceptable settlement criterion is not taken into consideration in determining allowable bearing capacity. Also, bearing capacity or settlement calculation are incorrect in one-third of the reports. In 71% of the reports, foundation dimensions used in bearing capacity and settlement calculations are different from each other, which do not make any physical sense. As emphasized in the explanation for the related question, unacceptable foundation depth values are suggested and used in bearing capacity or settlement calculations in a third of the reports. Only in one the reports, dissipation of vertical stress with depth is specified. In determination of modulus of subgrade reaction, foundation dimensions are not taken into account, values of subgrade modulus were simply selected from related tables. The percentage distribution of bearing capacity calculation methods used in collected reports are presented in Figure 5.1.

None of the examined reports can be regarded as perfect, however, report no. 66, which is rated best, is presented on the back side of the thesis in CD to set as an example. It should be noted that none of the evaluated geological/geotechnical reports, except report no. 66 which is for a high-rise project, are signed by geotechnical engineers.

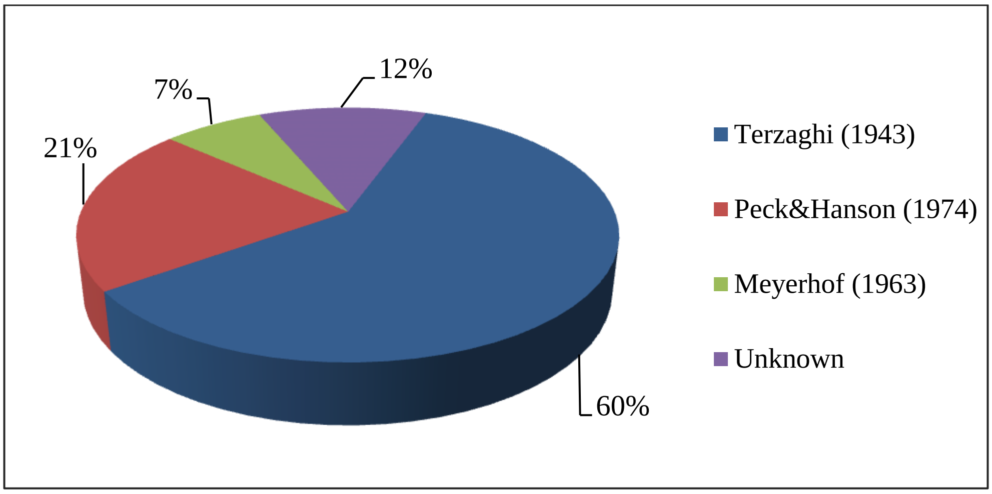


Figure 5.1 Bearing capacity calculation methods used in reports for shallow foundations on soil.

# CHAPTER 6

# CONCLUSION AND RECOMMENDATIONS

This thesis intends to identify missing/incorrect suggestions that are frequently performed in geotechnical reports prepared by different institutions for low-rise housing projects. Because of experiencing most of the loss of property and life due to earthquakes in low-rise buildings, it is clear that there are some problems in this field. For this purpose, 66 geotechnical reports are randomly collected from current archives of municipalities and evaluated. 60 pieces of reports out of these 66 are collected from municipalities of all central districts of Ankara (Altındağ, Çankaya, Etimesgut, Gölbaşı, Keçiören, Mamak, Sincan and Yenimahalle), and the remaining 6 reports are obtained from other city municipalities (Antalya, Çankırı, Çorum, İskenderun, Kırıkkale, and Konya). The evaluation is done according to 36 different technical criteria considered important. Statistical evaluation of deficiencies and mistakes, determined according to these assessment criteria, is made.

## 6.1 Conclusion

The results of this thesis study do find that there are significant deficiencies and mistakes in geotechnical reports of low-rise housing projects. Most of the reports are inadequate in terms of office work and walk-over studies. In addition, it is observed that in order to shorten the period of study and to reduce the work cost, the sampling studies, in-situ tests and laboratory tests are not conducted in adequate numbers and required context. In most of the reports, as there are insufficient numbers of tests, the results obtained or interpreted from these inadequate investigations are not used correctly in design calculations. Selections of suitable characteristic geotechnical design parameters for the requirements of the project, settlement and stability computations, which are perhaps the most important part of a geotechnical report, are incomplete, insufficient or incorrect in a significant part of the reports. Also, general recommendations concerning problems that may be encountered during excavations or construction of structures are mostly insufficient and consist of copy-paste sentences. As a result of this study, it is believed that most of the people who prepare and the people who control the reports either do not possess adequate knowledge and background about geotechnical subjects, or are simply unaware of the consequences of their substandard work.

## 6.2 Technical Recommendations

On the basis of the findings in this thesis, the following recommendations are made:

For an economic site investigation, also with an ability to meet project requirements, preliminary information related to project site should be collected by an office work. Geotechnical reconnaissance study should enclose review of geotechnical literature, maps and aerial photographs, and detailed description of geotechnical conditions of the site. A preliminary field investigation with a limited subsurface exploration may also be included. In determining the frequency and the depth of drillings, the geologic structure and seismicity of the region, load that will be applied by the planned structure on the ground and stress distribution should be taken into account.

In-situ tests should be conducted by trained technicians under the control of experienced engineers. In-situ testing should be done at frequent intervals and care should be taken for quality of both testing and sampling equipment. Expenditures for geotechnical site exploration should be increased. The numbers of undisturbed soil samples taken and the laboratory testing of subsurface materials generally need to be increased. In order to calculate primary foundation settlements, consolidation test should be absolutely carried out on samples taken from all necessary layers. In consolidation test loadings, the probable building load should be considered. Also, unconsolidated-undrained triaxial compression test should definitely be done on undisturbed samples taken from appropriate layers. It should not be forgotten that this type of triaxial test is done for determining short term (undrained) parameters of soil and failure envelope of saturated fine grained soils should be drawn as $ø=0°$. Consolidated triaxial tests must be employed if drained behavior is to be modeled. On core samples, not only point load index test, but also other laboratory tests such as unconfined compressive strength and elastic modulus should be performed.

In determining soil strength parameters, both the results obtained from laboratory tests and strength values obtained by empirical approaches using in-situ results should be considered together. In addition, an idealized soil profile must be created with a realistic approach. Geotechnical engineer should be in contact with both contractor and project engineer to be able to design all the investigations and calculations according to the proposed structure. Foundation and slab design recommendations based on site conditions should be provided. The report should contain a general conclusion or opinion as to the adequacy of the site for its intended use and ability of the onsite materials to support the proposed structures.

### 6.3 Policy Recommendations

All of the analyzed reports in this study are prepared and controlled without any contribution of civil engineers. It is seen that civil engineers are left outside of these studies. However, like many engineering issues, soil investigation and evaluation are multi-disciplinary engineering subjects. Especially in building foundation works, leaving civil engineers outside the issue and not allowing them to get involved in preparation of geotechnical reports create greater problems. For this reason, the correct approach for preparation of geotechnical reports is that different disciplines that study the ground come together to deal with the topic, with their respective education and experience.

In current practice, geotechnical investigations are performed with minimum cost and maximum speed. By adding ignorance and inattentiveness to these two erroneous approaches, the results that lack with engineering emerge. Also, there is not a functioning control mechanism that may prevent these malpractice issues. To sum up, it is clear that geotechnical investigations are just seen as a procedural detail. In order to construct safe and economic structures, greater attention to geotechnical investigations is needed, and both professional chambers and government agencies should seriously enhance their training and control mechanisms without losing more time.

## 6.4 Recommendations to Widen the Scope for Future Work

The scope of this study can be expanded by increasing the number of analyzed geological/geotechnical reports. Increasing the number of the reports results in obtaining more accurate results and as well as determining the common errors. Furthermore, the number of reports obtained from outside Ankara can be increased. Obtaining greater number of reports from other cities prevents study remaining limited to Ankara and attributes the results of study to whole country.

This thesis focused on reports that belong to small residential buildings. Other cases that may be investigated are the condition of geotechnical reports prepared for large projects (industrial buildings, skyscrapers, etc.) that have more capital to invest in the site investigation, or projects containing complex or difficult geotechnical problems.

# REFERENCES

Andresen A., Kolstad P., The NGI 54-mm Samplers for Undisturbed Sampling of Clays and Representative Sampling of Coarser Materials, 1979. Proc. of the Int. Conference on Soil Sampling, Singapore.

AGS (Association of Geotechnical and Geoenvironmental Specialists), 2005. Guidelines for the Preparation of the Ground Report, UK.

Bieniawski Z. T., 1975. The Point Load Test in Geotechnical Practice. Eng. Geol., Sept., pp. 1-11.

Birand, A., Ergun, U., Erol, O., 2002. CE 366 Foundation Engineering-1 Lecture Notes. METU, Ankara.

Broch, E. and Franklin, J.A., 1972. The Point Load Strength Test. International Journal Rock Mechanics and Mining Science, Vol. 9, pp. 669-697.

Bowles, J. E., 1996. Foundation Analysis and Design. McGrawhill Book Co, New York-USA.

Bowles, J. E., 1988. Foundatıon Analyses and Design, McGrawhill. Book Co, New York-USA.

Building Research Establishment, 1987. Site investigation for Low Rise Building: Procurement. Bldg Res. Dig. No: 322.

Carter, J. P., and F. H. Kulhawy, 1988. Analysis and Design of Foundations Socketed into Rock. Report No. EL-5918. Empire State Electric Engineering Research Corporation and Electric Power Research Institute, New York.

Clayton, C.R.I., Matthews, M.C., Simons, N.E., 1995. Site Investigation. Department of Civil Engineering, University of Survey.

Das, B. M., 2011. Geotechnical Engineering Handbook. J. Ross Publishing Inc. Company., USA.

Das, B. M., 1999. Shallow Foundations Bearing Capacity and Foundations. CRC Press, London.

Day, R. W., 2006. Foundation Engineering Handbook. The McGrawhill Companies.

Durgunoglu, H.T. and Mitchell, J.K., 1975. Static Penetration Resistance of Soils, Evaluation of Theory and Implication for Practice. Proceedings of the In-situ Measurement of Soil Properties, Raleigh, NC, vol. 1, pp. 172-189.

Failmezger, R.A., 2008. Which in-situ test should I use? - A designer’s guide. Ohio River Valley Soils Seminar 39, Cincinnati, Ohio.

GDDA (General Directorate of Disaster Affairs), 2005. Parsel Bazında Zemin-Temel Etütleri ve Zemin İyileştirme İşleri Hakkında Yönetmelik Taslağı Ön Raporu. Ankara.

Heck, W.J., 1970. Suggested Method of Test for Triaxial Compressive Strength of Undrained Rock Core Specimens with Induced Pore Pressure Measurements. Special Procedures for Testing Soil and Rock for Engineering Purposes, No: 479.

Hunt, R.E., 2005. Geotechnical Engineering Handbook. Second Edition, London.

Hvorslev, 1949. Subsurface Exploration and Sampling of Soils for Civil Engineering Purposes. Waterways Experimental Station, Vicksburg, USA.

Institution of Civil Engineers, 1991. Inadequate Site Investigation. London.

Janbu, N., Bjerrum, L. and Kjaernsli, B., 1956. Veiledring ved losning av fundermenteringsoppgaver. Norwegian Geotechnical Institute Publication No.16, Oslo.

Liao and R.V. Whitman, 1986. Overburden Correction Factors for SPT in Sand: Journal of Geotechnical Engineering. ASCE, v. 112:3, p. 373-377.

Littlejohn, G. S., Cole, K. W. and Mellors, T. W., 1994. Without Site Investigation Ground is a Hazard. Proceedings Inst. Civ. Eng., Vol. 102, May, pp. 72–78.

Look, B. G., 2007. Handbook of Geotechnical Investigation and Design Tables. London.

Mayne, P. W., Christopher, B. R., DeJong, J., 2001. Manual on Subsurface Investigations. Federal Highway Administration, Washington D.C.

Menard, L., 1965. The Determination of the Bearing Capacity and Settlement of Foundations from Pressuremeter Tests: Proc. 6th Int. Conf. SMFE, Montreal, V.2.

Meyerhof, G. G. and Hanna A. M., 1978. Ultimate Bearing Capacity of Foundations on Layered Soils under Inclined Load. Can. Geotech. J., vol. 15, pp. 565-572.

Meyerhof, G. G., 1974. Ultimate Bearing Capacity of Footings on Sand Layer Overlaying Clay. Can. Geotech. J., vol. 11, no. 2, pp. 223-229.

Ministry of Environment and Urbanism, 2005. Bina ve Bina türü Yapılar için Zemin ve Temel Etüdü Raporu Genel Formatı. http://www.csb.gov.tr/turkce/dosya/zemin.pdf [arrived on 27.01.2012].

National Research Council, 1984. Geotechnical Site Investigations for Underground Projects. US National Committee on Tunneling Technology, Vol. 1, National Academy Press, Washington.

NAVFAC DM (Naval Facilities Engineering Command), 1982. Soil Mechanics. Available from Naval Facilities Engineering Command, 200 Stovall Street, Alexandria

NCHPR (National Cooperative Highway Research Program), 2010. Design and Construction of Shallow Foundations for Highway Bridge Structures. Washington.

NDOT (Nevada Department of Transportation), 2005. Geotechnical Policies and Procedures Manual. Carson City, Nevada.

Özdemir, A., Ülgen, D., Özkan, Y., 2007. Ülkemizde Yapılan Geoteknik Etüt Sondajları İle İlgili Bazı Değerlendirmeler. İnşaat Mühendisleri Odası 2. Geoteknik Sempozyumu, Bildiriler Kitabı, pp. 473-479.

Özdemir, A., 2005. Türkiye Sondaj Yayınları (1934-2004). Sondaj Dünyası Dergisi.

Parry, R.H.G., 1977. Journal of Geotechnical Eng. Div. ASCE, 103, 1014.

Peck, R.B., Hanson, W.E. and Thornburn, T.H., 1974. Foundation Engineering. Second edition, John Wiley and Sons, New York.

Rowe, R.K., 2001. Geotechnical and Geoenvironmental Engineering Handbook. Academic Publishing, USA.

Rusnak, J., 2000. Using the Point Load Test to Determine the Uniaxial Compressive Strength of Coal Measure Rock. St. Louis, MO.

Sabatini, P.J., Bachus, R.C., Mayne, P.W., Schneider, T.E., 2002. Geotechnical Engineering Circular No: 5, Georgia.

Samtani, N. C. and Nowatzki, E. A., 2006. Soils and Foundations Reference Manual. National Highway Institute, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.

Schmertmann, J. H. 1978. Guidelines for Cone Penetrometer Test, Performance and Design. FHWA-TS-78-209, Federal Highway Administration, Washington, DC.

Schmertmann, J. H., 1970. Static Cone to Compute Static Settlement over Sand. Journal of the Soil Mech. and Foundation Division. Vol. 96, No. 3, pp. 1011 1043.

Scott, R.F. 1981. Foundation Analysis. Prentice-Hall. Englewood Cliffs, N.J.

Shaik, A.W., 2007. Towards More Reliable Site Investigation Information. Jurutera Perunding GEA (M) Sdn Bhd, Malaysia.

Shroff, A.V., Shah, D.L., 2003. Soil Mechanics and Geotechnical Engineering. India.

Skempton, A. W., 1986. Standard Penetration Test Procedures and the Effects in Sands of Overburden Pressure, Relative Density, Particle Size, Ageing and Over Consolidation. Geotechnique 36: 3, September.

Skempton, A. W. and Bjerrum, L., 1957. A Contribution to the Settlement Analysis of Foundation on Clay. Geotechnique, Vol. 7, pp. 168-178.

Skempton, A. W. and McDonald D. H., 1956. The Allowable Settlements of Buildings. Proc. Inst. Civ. Eng. Part III 5, 727-768, Discussion Closure: 783-784.

Skempton, A. W., 1953. The Colloidal Activity of Clays, Proceedings, 3rd International Conference on Soil Mechanics and Foundation Engineering, London.

Technical Guidelines for Geotechnical Reports, 1993. City of Carlsbad, California.

Terzaghi, K. and Peck, R. B., 1948. Soil Mechanics in Engineering Practice. John Wiley, New York.

Terzaghi K., Peck R.B., Mesri G., 1996. Soil Mechanics in Engineering Practice, John Wiley and Sons, New York.

Topal, T., 2000. Nokta Yükleme Deneyi İle İlgili Uygulamada Karşılaşılan Problemler. Department of Geological Engineering, METU, ANKARA.

U.S. Army Corps of Engineers, 2001. Geotechnical Investigations. Engineering and Design Manual, Washington.

U.S. National Research Council, 1984. Geotechnical Site Investigations for Underground Projects, Washington.

Whyte, I. L., 1995. The Financial Benefit from Site Investigation Strategy. Ground Engineering, Oct., pp. 33–36.

Wright, S.G., 2005. Evaluation of Soil Shear Strengths for Slope and Retaining Wall Stability Analyses with Emphasis on High Plasticity Clays, Austin.

Vesic, A. S., 1975. Bearing Capacity of Shallow Foundations. In Foundation Engineering Handbook, Ed. Winterkorn, H F and Fang, H Y. Pub: Van Nostrand Reinhold Co.

# APPENDIX A

## SITE INVESTIGATION

Table A.1 Application of selected geophysical methods for determination of engineering parameters (US Army Corps of Engineers, 2001)

| Method | Advantages | Limitations |
|---|---|---|
| Refraction seismic | Rapid, accurate, and relatively economical technique. Interpretation theory generally straightforward and equipment readily available | Incapable of detecting material of lower velocity underlying higher velocity. Thin stratum sometimes not detectable. Interpretation is not unique |
| Rayleigh wave dispersion | Rapid technique which uses conventional refraction seismographs | Requires long line (large site). Requires high-intensity seismic source rich in low-frequency energy. Interpretation complex |
| Vibratory (seismic) | Controlled vibratory source allows selection of frequency, hence wavelength and depth of penetration (up to 200 ft). Detects low-velocity zones underlying strata of higher velocity. Accepted method | Requires large vibratory source, specialized instrumentation, and interpretation |
| Reflection profiling (seismic-acoustic) | Surveys of large areas at minimal time and cost; continuity of recorded data allows direct correlation of lithologic and geologic changes; correlative drilling and coring can be kept to a minimum | Data resolution and penetration capability are frequency-dependent; sediment layer thickness and/or depth to reflection horizons must be considered approximate unless true velocities are known; some bottom conditions (e.g., organic sediments) prevent penetration; water depth should be at least 15 to 20 ft for proper system operation |
| Electrical resistivity | Economical nondestructive technique. Can detect large bodies of "soft" materials | Lateral changes in calculated resistance often interpreted incorrectly as depth related; hence, for this and other reasons, depth determinations can be grossly in error. Should be used in conjunction with other methods, e.g., seismic |
| Ground Penetrating Radar | Very rapid method for shallow site investigations. Online digital data processing can yield "onsite" look. Variable density display highly effective | Transmitted signal rapidly attenuated by water. Severely limits depth of penetration. Multiple reflections can complicate data interpretation |
| Gravity | Reasonably accurate results can be obtained, provided extreme care is exercised in establishing gravitational references | Equipment very costly. Requires specialized personnel. Anything having mass can influence data (buildings, automobiles, etc). Data reduction and interpretation are complex. Topography and strata density influence data |
| Magnetic | Minute quantities of magnetic materials are detectable | Only useful for locating magnetic materials. Interpretation highly specialized. Calibration on site extremely critical. Presence of any metallic objects influences data |

Table A.2 Numerical rating of geophysical methods to provide specific engineering parameters for engineering applications (US Army Corps of Engineers, 2001)

| Geophysical Method | Depth to Rock | P-Wave Velocity | S-Wave Velocity | Shear Modulus | Young's Modulus | Poisson's Ratio | Lithology | Material Boundaries Stratigraphy | Dip of Strata | Density | In Situ State of Stress | Temperature | Permeability | Percent Saturation | Ground water Table | Ground water Quality | Ground water Aquifers | Flow Rate and/or Direction | Borehole Diameter | Obstructions | Rippability | Fault Detection | Cavity Detection | Cavity Delineation | Location of Ore Bodies |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Surface | | | | | | | | | | | | | | | | | | | | | | | | | |
| Refraction (seismic) | 4 | 4 | 4 | 4 | 4 | 4 | 1 | 3 | 4 | 2 | 1 | 0 | 0 | 2 | 2 | 0 | 2 | 0 | 0 | 2 | 4 | 3 | 2 | 2 | 3 |
| Reflection (seismic) | 4 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 4 | 0 | 0 | 0 | 0 | 0 | 2 | 0 | 1 | 0 | 0 | 2 | 0 | 4 | 3 | 3 | 3 |
| Rayleigh wave dispersion | 1 | 0 | 2 | 2 | 0 | 0 | 1 | 3 | 0 | 2 | 1 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | 1 | 2 |
| Vibratory (seismic) | 2 | 0 | 4 | 4 | 4 | 0 | 1 | 3 | 0 | 2 | 1 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 2 | 1 | 2 | 2 | 3 |
| Reflection profiling (seismic-acoustic) | 4 | 0 | 0 | 0 | 0 | 0 | 1 | 4 | 4 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 0 | 4 | 3 | 3 | 4 |
| Electrical potential[2] | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | 0 | 1 | 1 | 2 | 3 | 3 | 3 | 0 | 0 | 0 | 3 | 3 | 3 | 4 |
| Electrical resistivity | 3 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 2 | 0 | 0 | 0 | 2 | 1 | 4 | 0 | 4 | 2 | 0 | 3 | 2 | 0 | 4 | 4 | 4 |
| Acoustic (resonance)[2] | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 3 | 0 | 0 | 0 | 0 | 0 | 4 | 0 |
| Radar[2,3] | 3 | 0 | 0 | 0 | 0 | 0 | 1 | 3 | 2 | 0 | 0 | 0 | 2 | 3 | 3 | 0 | 0 | 2 | 0 | 3 | 0 | 3 | 3 | 3 | 3 |
| Electromagnetic[2] | 4 | 0 | 0 | 0 | 0 | 0 | 3 | 4 | 1 | 0 | 0 | 0 | 1 | 2 | 3 | 1 | 2 | 0 | 0 | 0 | 0 | 3 | 0 | 0 | 4 |
| Gravity | 3 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 4 | 0 | 1 | 3 | 3 | 3 |
| Magnetic[2,3] | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 2 | 2 | 4 |

[1] Numerical rating refers to applicability of method in terms of current use and future potential:
0 = Not considered applicable
1 = Limited
2 = Used or could be used, but not best approach
3 = Excellent potential but not fully developed
4 = Generally considered as excellent approach; state of art well developed
A = In conjunction with other electrical and nuclear logs

[2] Methods not included in EM 1110-1-1802.

[3] Airborne or inhole survey capability not considered.

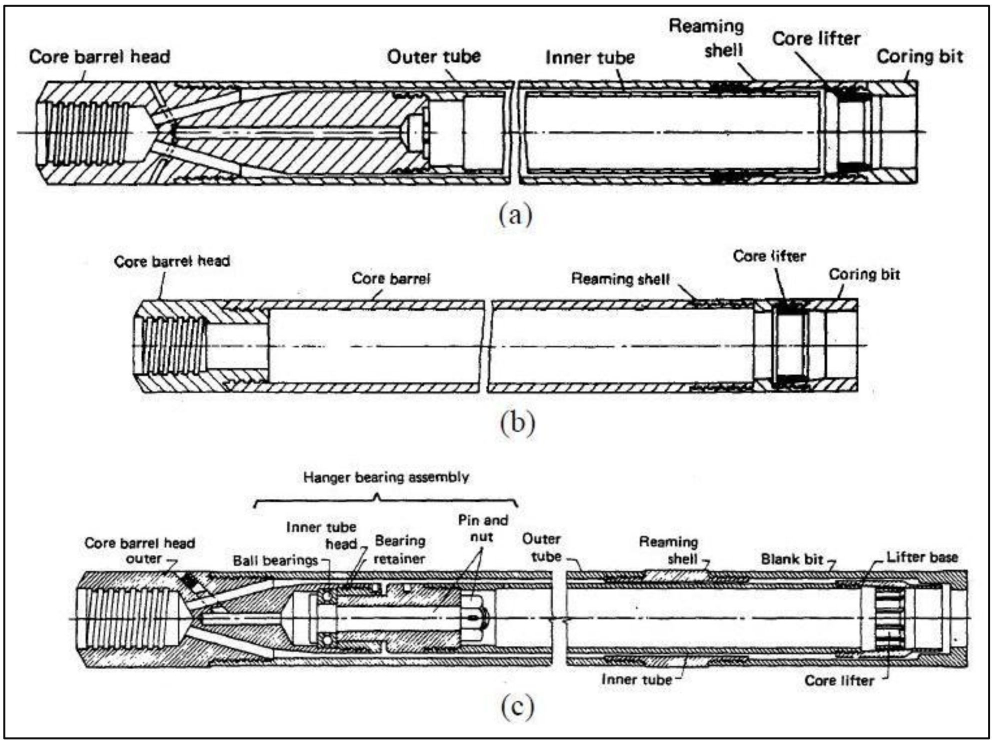



Figure A. 1 a) Single tube core barrel, b) rigid type double tube core barrel, c) swivel type double tube core barrel (Mayne et al., 2001).

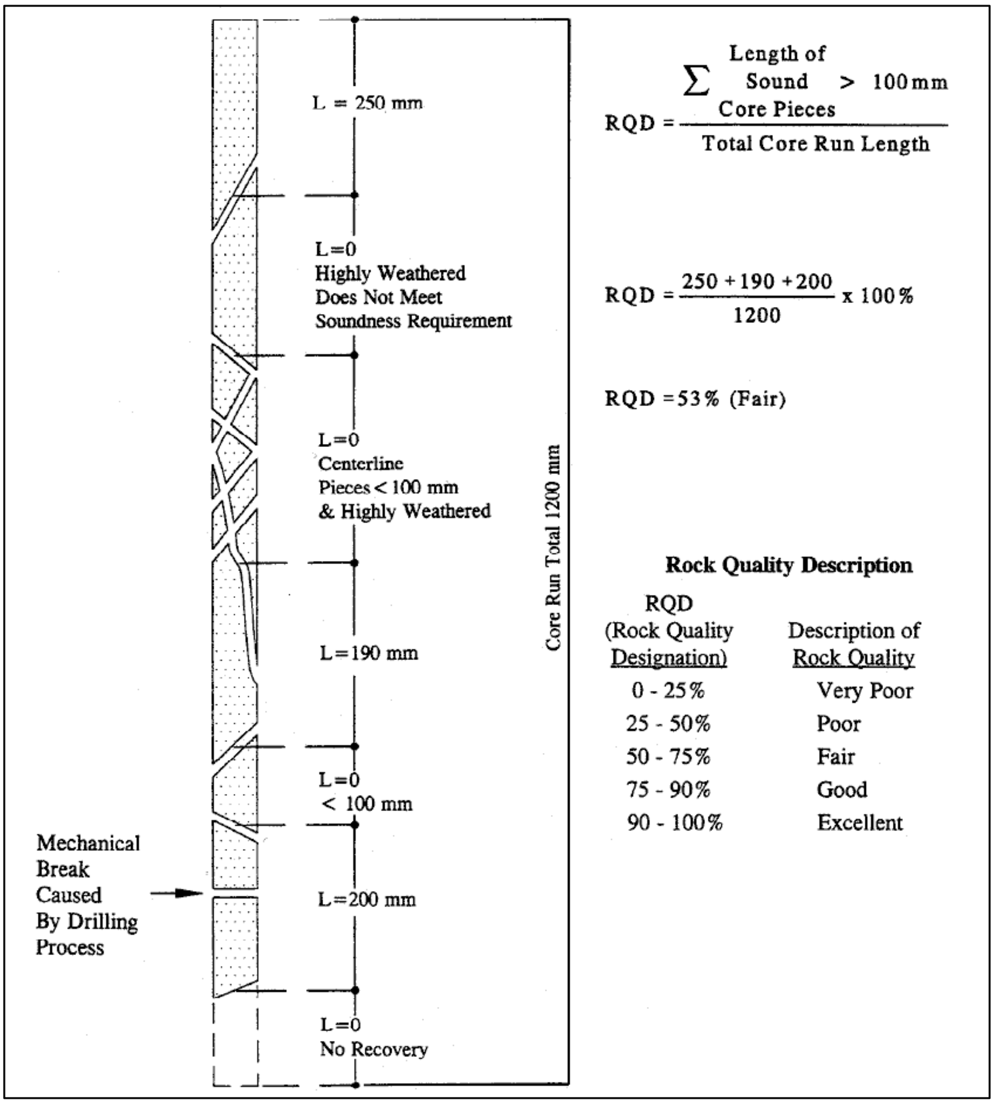



Figure A.2 Procedure for measurement and calculation of rock quality designation (Sabatini et al., 2002).

Table A.3 Sampling tools and methods (Hunt, 2005)

| Category-Method and Tool | Application | Limitations |
|---|---|---|
| | *Reconnaissance* | |
| Wash sample | Indication of material type only | Completely mixed, altered, segregated |
| Auger sample | Material identification | Completely disturbed |
| Retractable-plug sampler | Material identification | Slight disturbance, very small sample of soft soils |
| Black sample | Large undisturbed sample of cohesive materials | Taken from test pits, cohesive soils only |
| | *Test Boring Sampling (Soils)* | |
| Split barrel (spoon) | Undisturbed samples in soils suitable for identification and lab index tests | Samples not suitable for engineering properties testing<br>Sampling impossible in very coarse granular soils |
| Shelby tube | Undisturbed sample in firm to stiff cohesive soil. Can be driven into hard soils | Will not retrieve very or clean granular soils |
| Standard stationary piston | Undisturbed samples in soft to firm clays and silts | Will not penetrate compact sands, stiff clays, and other strong soils. Will not retrieve sands. Can be overpushed |
| Osterberg piston sampler | Undisturbed samples in all soils with cohesion except very strong. Less successful in clean sands | Usually cannot penetrate strong residual soil and glacial till. Some disturbance in sand and often loss of sample. User cannot observe amount of partial penetration |
| Shear-pin piston (Greer and McClelland) | Undisturbed samples in all soils with cohesion except very strong. Often recovers samples in sands and can be used to determine natural density | Usually cannot penetrate strong residual soil or glacial till. Disturbance in sands<br>Cannot observe amount of partial penetration |
| Swedish foil sampler | Continuous undisturbed samples in soft to firm cohesive soils | Gravel and shells will rupture foil. Cannot penetrate strong soils |
| Denison sampler | Undisturbed samples in strong cohesive soils such as residual soils, glacial till, soft rock alternating | Not suitable in clean granular soils, and soft to firm clays |
| Pitcher sampler | Similar to Denison above. Superior in soft to hard layers. Can be used in firm clays | Similar to Denison above |

Table A.3 Sampling tools and methods (Hunt, 2005) (continued)

| | | |
|---|---|---|
| | *Subaqueous Sampling Without Test Boring* | |
| Free-fall gravity coring tube | Samples firm to stiff clays, sand and fine gravel in water depths of 4000 m | Maximum length of penetration about 5 m in soft soils, 3 m in firm soils |
| Harpoon-type gravity sampler | Samples river bottom muds and silts to depths of about 3 m | Penetration limited to few meters in soft soils |
| Explosive coring tube (piggot tube) | Small-diameter samples of stiff to hard ocean bottom soils to water depths of 6000 m | Sample diameters only 1 7/8 in. Penetration only to 3 m below seafloor |
| Gas-operated free-fall piston (NG) | Good-quality samples up to 10 m depth from seafloor | Penetration limited to 10 m below seafloor |
| Vibracore | Undisturbed samples of soft to firm bottom sediment, 3 1/2 in. diameter to depths of 12 m | Limited to soft to firm soils and maximum penetration of 12 m. Water depth limited to 60 m |
| | *Subaqueous Sampling with Test Boring* | |
| Wireline drive sample | Disturbed sample in soils | Penetration length during driving not known |
| Wireling push samples | Relatively undisturbed samples may be obtained in cohesive materials | Often poor or no recovery in clean granular soils |
| | *Rock Coring* | |
| Single-tube core barrel | Coring hard homogeneous rock where high recovery is not necessary | Circulating water erodes soft, weathered, or fractured rock |
| Double-tube core barrel | Coring most rock types where high recovery is not necessary, and rock is not highly fractured or soft | Recovery often low in soft or fractured rocks |
| Double-tube swivel-type core barrel | Superior to double-tube swivel-core barrel, above Particularly useful to obtain high recovery in friable, highly fractured rock | Not needed in good-quality rock. Barrel is more costly and complicated than others mentioned above |
| Wireline core barrel | Deep hole drilling in rock or offshore because of substantial reduction of in–out times for tools | No more efficient than normal drilling to depths of about 30 m |
| Oriented core barrel | Determination of orientation of geologic structures | Procedure is slow and costly. Requires full recovery |
| Integral coring method | Recover cores and determine orientation in poor-quality rock with cavities, numerous fractures, and shear zones | Slow and costly procedure |

Table A.4 Specimen quality in terms of volumetric strain (Terzaghi et al., 1996)

| Volumetric strain (%) | SQD |
|---|---|
| < 1 | A |
| 1–2 | B |
| 2–4 | C |
| 4–8 | D |
| > 8 | E |

Table A.5 In-situ test methods and general application (Bowles, 1996)

| *Test* | *Area of ground interest* | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | *Soil identification* | *Establish vertical profile* | *Relative density $D_r$* | *Angle of friction $\phi$* | *Undrained shear strength $S_u$* | *Pore pressure u* | *Stress history OCR and Ko* | *Modulus: Es, G* | *Compressibility mv and Cc* | *Consolidation $c_h$ and $c_v$* | *Permeability k* | *Stress-strain curve* | *Liquefaction resistance* |
| Acoustic probe | C | B | B | C | C | | C | C | | | | | C |
| Borehole permeability | C | | | | | A | | | | B | A | | |
| Cone | | | | | | | | | | | | | |
| Dynamic | C | A | B | C | C | | C | | | | | | C |
| Electrical friction | B | A | B | C | B | | C | B | C | | | | B |
| Electrical piezocone | A | A | B | B | B | A | A | B | B | A | B | B | A |
| Mechanical | B | A | B | C | B | | C | B | C | | | | B |
| Dilatometer (DMT) | B | A | B | C | B | | B | B | C | | | C | B |
| Hydraulic fracture | | | | | | B | B | | | C | C | | |
| Nuclear density tests | | | A | B | | | | C | | | | | |
| Plate load tests | C | C | B | B | C | | B | A | B | C | C | B | B |
| Pressure meter menard | B | B | C | B | B | | C | B | B | | | C | C |
| Self-boring pressure | B | B | A | A | A | A | A | A | A | A | B | A | A |
| Screw plate | C | C | B | C | B | | B | A | B | C | C | B | B |
| Seismic down-hole | C | C | C | | | | | A | | | | B | B |
| Seismic refraction | C | C | | | | | | B | | | | | B |
| Shear vane | B | C | | | A | | B | | | | | | |
| Standard penetration test (SPT) | B | B | B | C | C | | | | C | | | | A |

Code: *A* = most applicable.
*B* = may be used.
*C* = least applicable.

Table A.6 In-situ tests for rock and soil (US Army Corps of Engineers, 2001)

| Purpose of Test | Type of Test | Applicability to Soil | Applicability to Rock |
|---|---|---|---|
| Shear strength | Standard penetration test (SPT) | X | |
| | Field vane shear | X | |
| | Cone penetrometer test (CPT) | X | |
| | Direct shear | X | |
| | Plate bearing or jacking | X | X[1] |
| | Borehole direct shear[2] | X | |
| | Pressuremeter[2] | | X |
| | Uniaxial compressive[2] | | X |
| | Borehole jacking[2] | | X |
| Bearing capacity | Plate bearing | X | X[1] |
| | Standard penetration | X | |
| Stress conditions | Hydraulic fracturing | X | X |
| | Pressuremeter | X | X[1] |
| | Overcoring | | X |
| | Flatjack | | X |
| | Uniaxial (tunnel) jacking | X | X |
| | Borehole jacking[2] | | X |
| | Chamber (gallery) pressure[2] | | X |
| Mass deformability | Geophysical (refraction) | X | X |
| | Pressuremeter or dilatometer | X | X[1] |
| | Plate bearing | X | X |
| | Standard penetration | X | |
| | Uniaxial (tunnel) jacking | X | X |
| | Borehole jacking[2] | | X |
| | Chamber (gallery) pressure[2] | | X |
| Relative density | Standard penetration | X | |
| | In situ sampling | X | |
| | Cone[2] penetrometer | X | |
| Liquefaction susceptibility | Standard penetration | X | |
| | Cone penetrometer test (CPT)[2] | X | |

[1] Primarily for clay shales, badly decomposed, or moderately soft rocks, and rock with soft seams.
[2] Less frequently used.

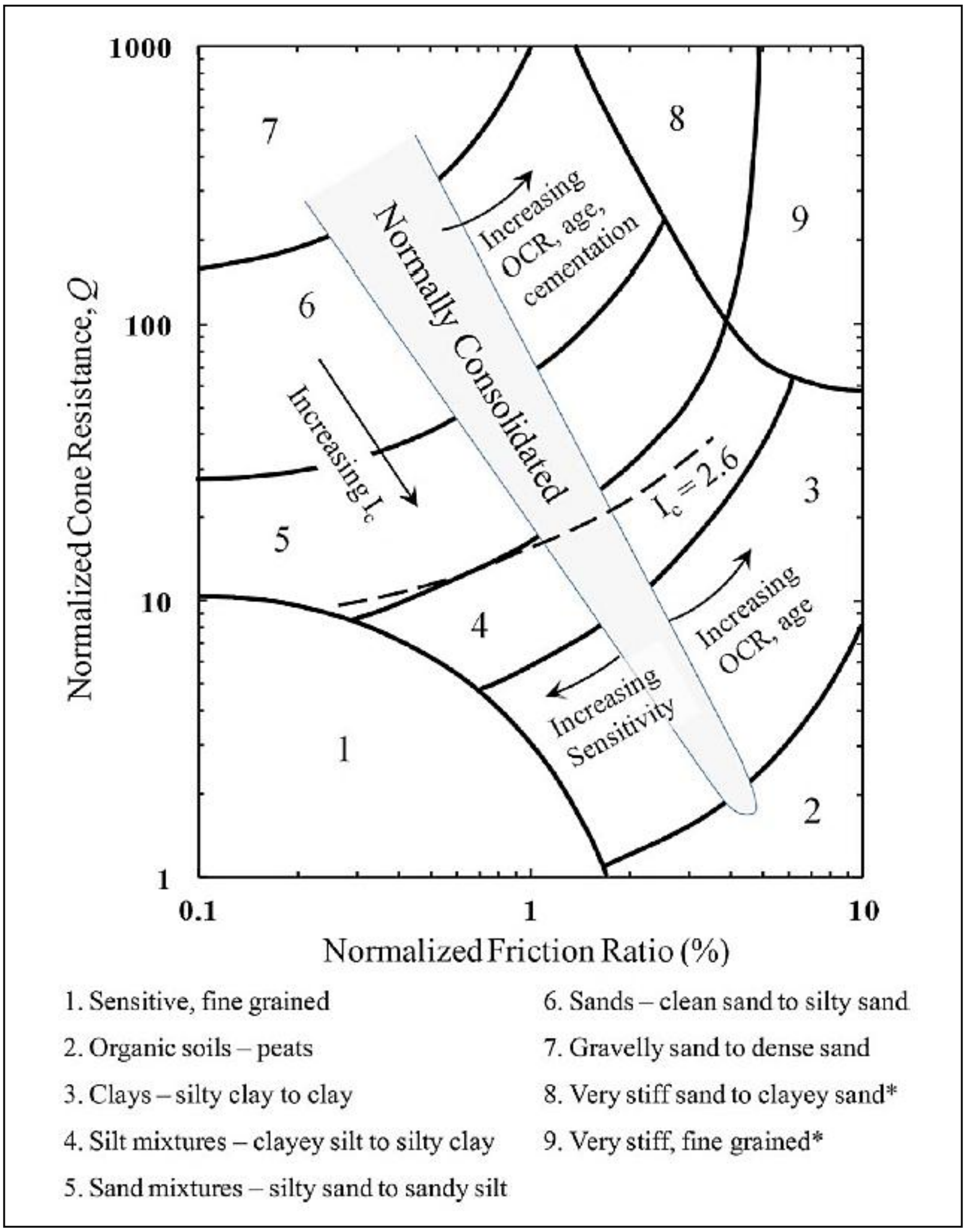



Figure A.3 Classification of soil based on CPT test results (Robertson and Wride, 1997).

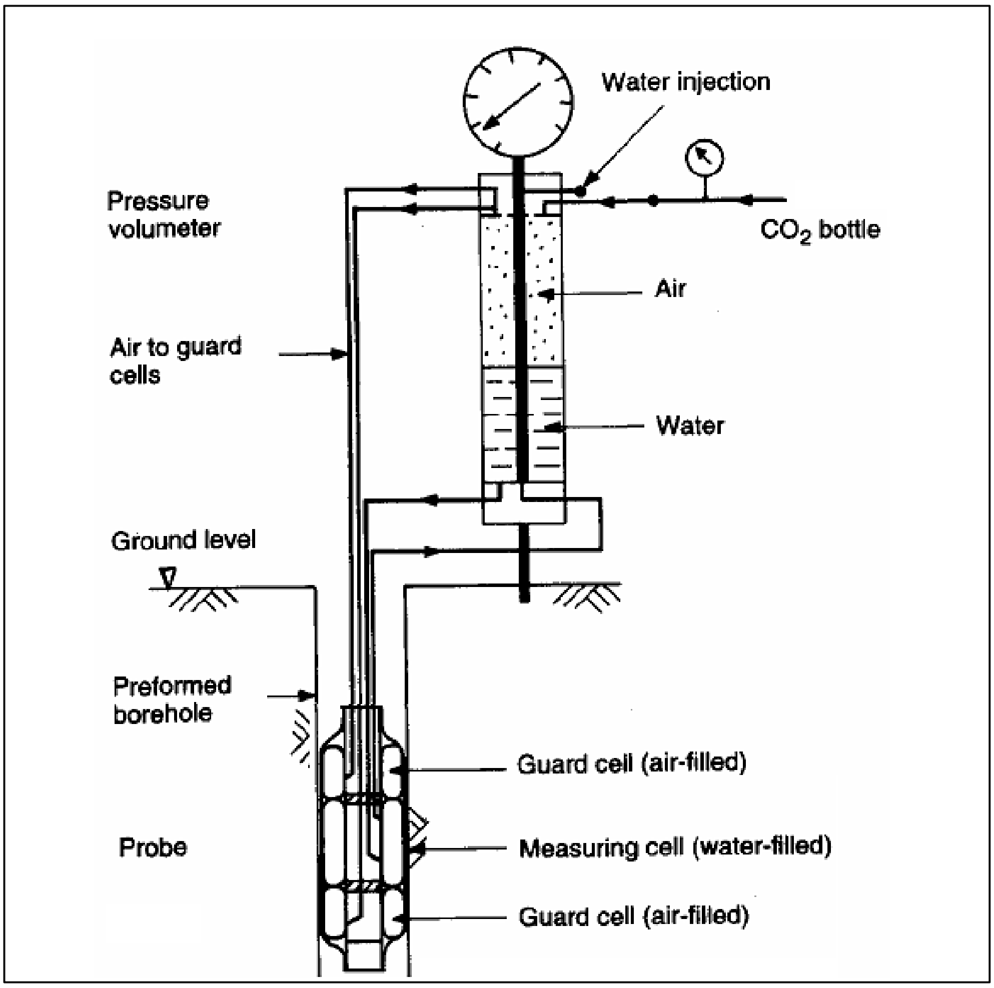


Figure A.4 Diagrammatic sketch of the Ménard pressuremeter (Gibson and Anderson 1961).

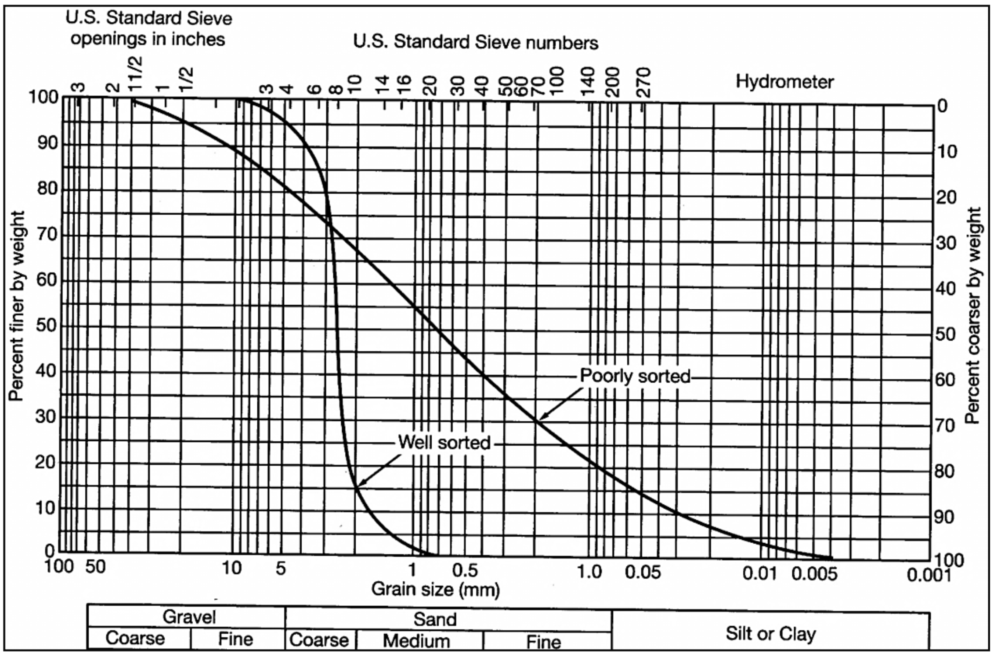



Figure A.5 Representative grain size distribution curve (San Diego State University Geo. 552 lec. notes, 2011).

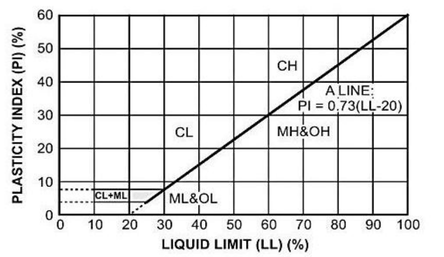
Table A.7 Typical values of liquid limit, plastic limit, and activity of some clay minerals (Mitchell, 1976; Skempton, 1953)

| Mineral | Liquid limit, *LL* | Plastic limit, *PL* | Activity, *A* |
|---|---|---|---|
| Kaolinite | 35–100 | 20–40 | 0.3–0.5 |
| Illite | 60–120 | 35–60 | 0.5–1.2 |
| Montmorillonite | 100–900 | 50–100 | 1.5–7.0 |
| Halloysite (hydrated) | 50–70 | 40–60 | 0.1–0.2 |
| Halloysite (dehydrated) | 40–55 | 30–45 | 0.4–0.6 |
| Attapulgite | 150–250 | 100–125 | 0.4–1.3 |
| Allophane | 200–250 | 120–150 | 0.4–1.3 |

Table A.8 Unified soil classification system (American Society for Testing and Materials, 1985)

| UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART | | | LABORATORY CLASSIFICATION CRITERIA |
|---|---|---|---|
| **COARSE-GRAINED SOILS** (more than 50% of material is larger than No. 200 sieve size.) | | | |
| **GRAVELS** More than 50% of coarse fraction larger than No. 4 sieve size | | Clean Gravels (Less than 5% fines) | |
| | GW | Well-graded gravels, gravel-sand mixtures, little or no fines | GW $C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3 |
| | GP | Poorly-graded gravels, gravel-sand mixtures, little or no fines | GP Not meeting all gradation requirements for GW |
| | | Gravels with fines (More than 12% fines) | |
| | GM | Silty gravels, gravel-sand-silt mixtures | GM Atterberg limits below "A" line or P.I. less than 4 — Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols |
| | GC | Clayey gravels, gravel-sand-clay mixtures | GC Atterberg limits above "A" line with P.I. greater than 7 |
| **SANDS** 50% or more of coarse fraction smaller than No. 4 sieve size | | Clean Sands (Less than 5% fines) | |
| | SW | Well-graded sands, gravelly sands, little or no fines | SW $C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3 |
| | SP | Poorly graded sands, gravelly sands, little or no fines | SP Not meeting all gradation requirements for GW |
| | | Sands with fines (More than 12% fines) | |
| | SM | Silty sands, sand-silt mixtures | SM Atterberg limits below "A" line or P.I. less than 4 — Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols. |
| | SC | Clayey sands, sand-clay mixtures | SC Atterberg limits above "A" line with P.I. greater than 7 |
| **FINE-GRAINED SOILS** (50% or more of material is smaller than No. 200 sieve size.) | | | Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent ........ GW, GP, SW, SP; More than 12 percent ........ GM, GC, SM, SC; 5 to 12 percent ........ Borderline cases requiring dual symbols |
| **SILTS AND CLAYS** Liquid limit less than 50% | ML | Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity | |
| | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays | |
| | OL | Organic silts and organic silty clays of low plasticity | |
| **SILTS AND CLAYS** Liquid limit 50% or greater | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts | |
| | CH | Inorganic clays of high plasticity, fat clays | |
| | OH | Organic clays of medium to high plasticity, organic silts | |
| **HIGHLY ORGANIC SOILS** | PT | Peat and other highly organic soils | |

**PLASTICITY CHART**



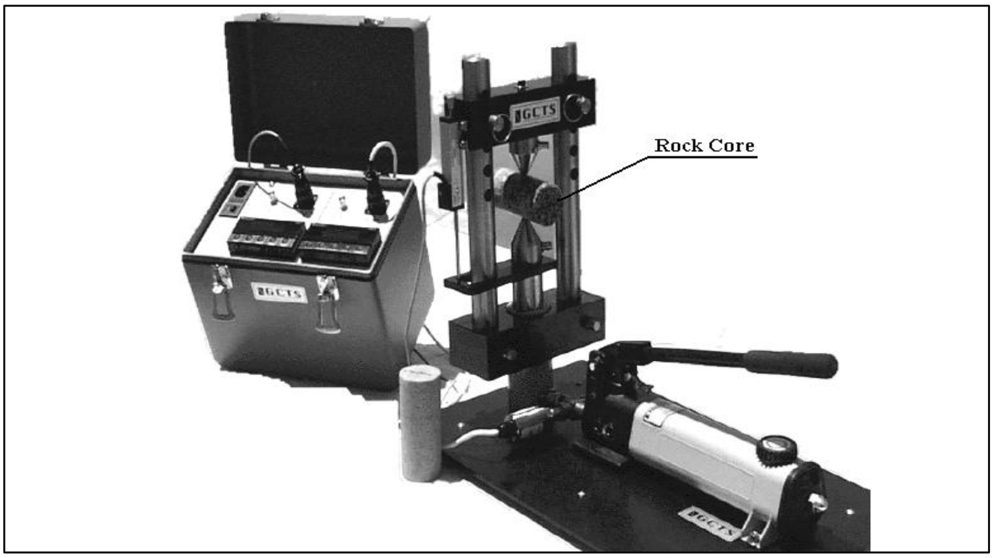




Figure A.6 The point load tester (Rusnak, 2000).

Table A.9 Conversion factors, k, by various researchers (Topal, 2000)

| **Researcher** | **Rock type** | **k value** |
|---|---|---|
| Bieniawski (1975) | Sandstone | 24 |
| Wilson (1976) | Very poor mudstone | 8 |
| Carter (1977 ) | Limestone | 26-28.5 |
| | Sandstone | 24.5 |
| Hassanı et al. (1980) | Sedimentary rocks | 29 |
| Beawisetal (1982) | Shale | 8 |
| Norbury (1986) | Sandstone | 8-30 (generally 20-25) |
| | Siltstone | 15-35 |
| | Mudstone | 18-35 (generally 20) |
| Hawkins & Olver (1986) | Limestone | 26.5 |
| Bell (1992) | Sandstone (dry) | 12-19 (dry) - 7-12 (wet) |
| | Limestone (dry) | 20-30 (dry) - 14-24 (wet) |
| Anıl et al. (1996) | Marble | 24 |
| Bowden et al. (1998) | Chalk | 11-21 (generally 14-17) |

# APPENDIX B

## FOUNDATION DESIGN

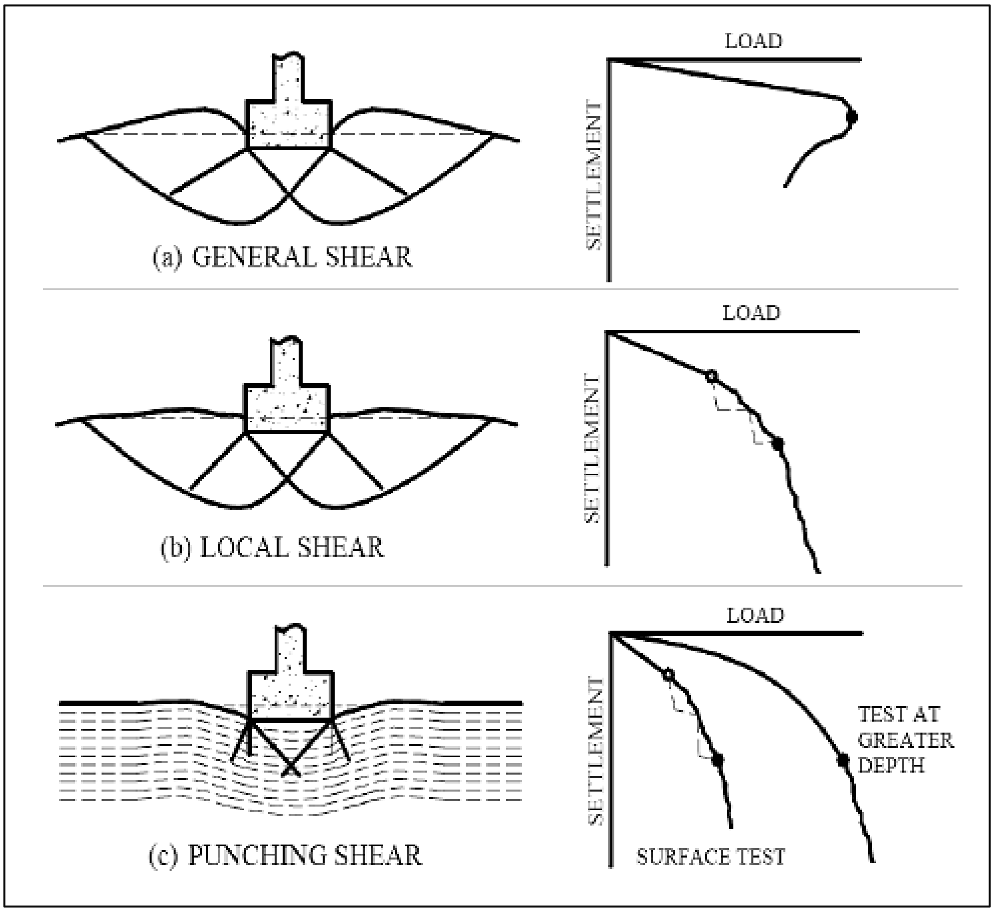



Figure B.1 Modes of bearing capacity failure (Vesic, 1973).

Table B.1 Terzaghi bearing capacity factors (Das, 1999)

| $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ | $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ | $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 5.70 | 1.00 | 0.00 | 17 | 14.60 | 5.45 | 2.18 | 34 | 52.64 | 36.50 | 38.04 |
| 1 | 6.00 | 1.1 | 0.01 | 18 | 15.12 | 6.04 | 2.59 | 35 | 57.75 | 41.44 | 45.41 |
| 2 | 6.30 | 1.22 | 0.04 | 19 | 16.57 | 6.70 | 3.07 | 36 | 63.53 | 47.16 | 54.36 |
| 3 | 6.62 | 1.35 | 0.06 | 20 | 17.69 | 7.44 | 3.64 | 37 | 70.01 | 53.80 | 65.27 |
| 4 | 6.97 | 1.49 | 0.10 | 21 | 18.92 | 8.26 | 4.31 | 38 | 77.50 | 61.55 | 78.61 |
| 5 | 7.34 | 1.64 | 0.14 | 22 | 20.27 | 9.19 | 5.09 | 39 | 85.97 | 70.61 | 95.03 |
| 6 | 7.73 | 1.81 | 0.20 | 23 | 21.75 | 10.23 | 6.00 | 40 | 95.66 | 81.27 | 115.31 |
| 7 | 8.15 | 2.00 | 0.27 | 24 | 23.36 | 11.40 | 7.08 | 41 | 106.81 | 93.85 | 140.51 |
| 8 | 8.60 | 2.21 | 0.35 | 25 | 25.13 | 12.72 | 8.34 | 42 | 119.67 | 108.75 | 171.99 |
| 9 | 9.09 | 2.44 | 0.44 | 26 | 27.09 | 14.21 | 9.84 | 43 | 134.58 | 126.50 | 211.56 |
| 10 | 9.61 | 2.69 | 0.56 | 27 | 29.24 | 15.90 | 11.60 | 44 | 151.95 | 147.74 | 261.60 |
| 11 | 10.16 | 2.98 | 0.69 | 28 | 31.61 | 17.81 | 13.70 | 45 | 172.28 | 173.28 | 325.34 |
| 12 | 10.76 | 3.29 | 0.85 | 29 | 34.24 | 19.98 | 16.18 | 46 | 196.22 | 204.19 | 407.11 |
| 13 | 11.41 | 3.63 | 1.04 | 30 | 37.16 | 22.46 | 19.13 | 47 | 224.55 | 241.80 | 512.84 |
| 14 | 12.11 | 4.02 | 1.26 | 31 | 40.41 | 25.28 | 22.65 | 48 | 258.28 | 287.85 | 650.87 |
| 15 | 12.86 | 4.45 | 1.52 | 32 | 44.04 | 28.52 | 26.87 | 49 | 298.71 | 344.63 | 831.99 |
| 16 | 13.68 | 4.92 | 1.82 | 33 | 48.09 | 32.23 | 31.94 | 50 | 347.50 | 415.14 | 1072.80 |

Table B.2 Commonly used shape factors for the Terzaghi equation (Day, 2006)

| **Shape of Foundation** | $s_c$ | $s_\gamma$ | $s_q$ |
|---|---|---|---|
| Strip | 1.0 | 1.0 | 1.0 |
| Rectangle | 1+0.3B/L | 0.8 | 1.0 |
| Square | 1.3 | 0.8 | 1.0 |
| Circle (dia. B) | 1.3 | 0.6 | 1.0 |

Table B.3 Terzaghi modified bearing capacity factors (Das, 1999)

| $\phi$ | $N_c'$ | $N_q'$ | $N_\gamma'$ | $\phi$ | $N_c'$ | $N_q'$ | $N_\gamma'$ | $\phi$ | $N_c'$ | $N_q'$ | $N_\gamma'$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 5.70 | 1.00 | 0.00 | 17 | 10.47 | 3.13 | 0.76 | 34 | 23.72 | 11.67 | 7.22 |
| 1 | 5.90 | 1.07 | 0.005 | 18 | 10.90 | 3.36 | 0.88 | 35 | 25.18 | 12.75 | 8.35 |
| 2 | 6.10 | 1.14 | 0.02 | 19 | 11.36 | 3.61 | 1.03 | 36 | 26.77 | 13.97 | 9.41 |
| 3 | 6.30 | 1.22 | 0.04 | 20 | 11.85 | 3.88 | 1.12 | 37 | 28.51 | 15.32 | 10.90 |
| 4 | 6.51 | 1.30 | 0.055 | 21 | 12.37 | 4.17 | 1.35 | 38 | 30.43 | 16.85 | 12.75 |
| 5 | 6.74 | 1.39 | 0.074 | 22 | 12.92 | 4.48 | 1.55 | 39 | 32.53 | 18.56 | 14.71 |
| 6 | 6.97 | 1.49 | 0.10 | 23 | 13.51 | 4.82 | 1.74 | 40 | 34.87 | 20.50 | 17.22 |
| 7 | 7.22 | 1.59 | 0.128 | 24 | 14.14 | 5.20 | 1.97 | 41 | 37.45 | 22.70 | 19.75 |
| 8 | 7.47 | 1.70 | 0.16 | 25 | 14.80 | 5.60 | 2.25 | 42 | 40.33 | 25.21 | 22.50 |
| 9 | 7.74 | 1.82 | 0.20 | 26 | 15.53 | 6.05 | 2.59 | 43 | 43.54 | 28.06 | 26.25 |
| 10 | 8.02 | 1.94 | 0.24 | 27 | 16.03 | 6.54 | 2.88 | 44 | 47.13 | 31.34 | 30.40 |
| 11 | 8.32 | 2.08 | 0.30 | 28 | 17.13 | 7.07 | 3.29 | 45 | 51.17 | 35.11 | 36.00 |
| 12 | 8.63 | 2.22 | 0.35 | 29 | 18.03 | 7.66 | 3.76 | 46 | 55.73 | 39.48 | 41.70 |
| 13 | 8.96 | 2.38 | 0.42 | 30 | 18.99 | 8.31 | 4.39 | 47 | 60.91 | 44.54 | 49.30 |
| 14 | 9.31 | 2.55 | 0.48 | 31 | 20.03 | 9.03 | 4.83 | 48 | 66.80 | 50.46 | 59.25 |
| 15 | 9.67 | 2.73 | 0.57 | 32 | 21.16 | 9.82 | 5.51 | 49 | 73.55 | 57.41 | 71.45 |
| 16 | 10.06 | 2.92 | 0.67 | 33 | 22.39 | 10.69 | 6.32 | 50 | 81.31 | 65.60 | 85.75 |

Table B.4 Meyerhof bearing capacity factors (Das, 1999)

| $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ | $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ | $\phi$ | $N_c$ | $N_q$ | $N_\gamma$ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 5.14 | 1 00 | 0.00 | 17 | 12.34 | 4.77 | 1.66 | 34 | 42.16 | 29.44 | 31.15 |
| 1 | 5.38 | 1.09 | 0.002 | 18 | 13.10 | 5.26 | 2.00 | 35 | 46.12 | 33.30 | 37.15 |
| 2 | 5.63 | 1.20 | 0.01 | 19 | 13.93 | 5.80 | 2.40 | 36 | 50.59 | 37.75 | 44.43 |
| 3 | 5.90 | 1.31 | 0.02 | 20 | 14.83 | 6.40 | 2.87 | 37 | 55.63 | 42.92 | 53.27 |
| 4 | 6.19 | 1.43 | 0.04 | 21 | 15.82 | 7.07 | 3.42 | 38 | 61.35 | 48.93 | 64.07 |
| 5 | 6.49 | 1.57 | 0.07 | 22 | 16.88 | 7.82 | 4.07 | 39 | 67.87 | 55.96 | 77.33 |
| 6 | 6.81 | 1.72 | 0.11 | 23 | 18.05 | 8.66 | 4.82 | 40 | 75.31 | 64.20 | 93.69 |
| 7 | 7.16 | 1.88 | 0.15 | 24 | 19.32 | 9.60 | 5.72 | 41 | 83.86 | 73.90 | 113.99 |
| 8 | 7.53 | 2.06 | 0.21 | 25 | 20.72 | 10.66 | 6.77 | 42 | 93.71 | 85.38 | 139.32 |
| 9 | 7.92 | 2.25 | 0.28 | 26 | 22.25 | 11.85 | 8.00 | 43 | 105.11 | 99.02 | 171.14 |
| 10 | 8.35 | 2.47 | 0.37 | 27 | 23.94 | 13.20 | 9.46 | 44 | 118.37 | 115.31 | 211.41 |
| 11 | 8.80 | 2.71 | 0.47 | 28 | 25.80 | 14.72 | 11.19 | 45 | 133.88 | 134.88 | 262.74 |
| 12 | 9.28 | 2.97 | 0.60 | 29 | 27.86 | 16.44 | 13.24 | 46 | 152.10 | 158.51 | 328.73 |
| 13 | 9.81 | 3.26 | 0.74 | 30 | 30.14 | 18.40 | 15.67 | 47 | 173.64 | 187.21 | 414.32 |
| 14 | 10.37 | 3.59 | 0.92 | 31 | 32.67 | 20.63 | 18.56 | 48 | 199.26 | 222.31 | 526.44 |
| 15 | 10.98 | 3.94 | 1.13 | 32 | 35.49 | 23.18 | 22.02 | 49 | 229.93 | 265.51 | 674.91 |
| 16 | 11.63 | 4.34 | 1.38 | 33 | 38.64 | 26.09 | 26.17 | 50 | 266.89 | 319.07 | 873.84 |

Table B.5 Meyerhof shape factors (Das, 1999)

For $\phi = 0°$:
$$s_c = 1 + 0.2\left(\frac{B}{L}\right)$$
$$s_\gamma = s_q = 1$$

For $\phi \geq 10°$:
$$s_c = 1 + 0.2\left(\frac{B}{L}\right)\tan^2\left(45 + \frac{\phi}{2}\right)$$
$$s_\gamma = s_q = 1 + 0.1\left(\frac{B}{L}\right)\tan^2\left(45 + \frac{\phi}{2}\right)$$

Table B.6 Meyerhof depth factors (Das, 1999)

For $\phi = 0°$:
$$d_c = 1 + 0.2\left(\frac{D_f}{B}\right)$$
$$d_q = d_\gamma = 1$$

For $\phi \geq 10°$:
$$d_c = 1 + 0.2\left(\frac{D_f}{B}\right)\tan\left(45 + \frac{\phi}{2}\right)$$
$$d_q = d_\gamma = 1 + 0.1\left(\frac{D_f}{B}\right)\tan\left(45 + \frac{\phi}{2}\right)$$

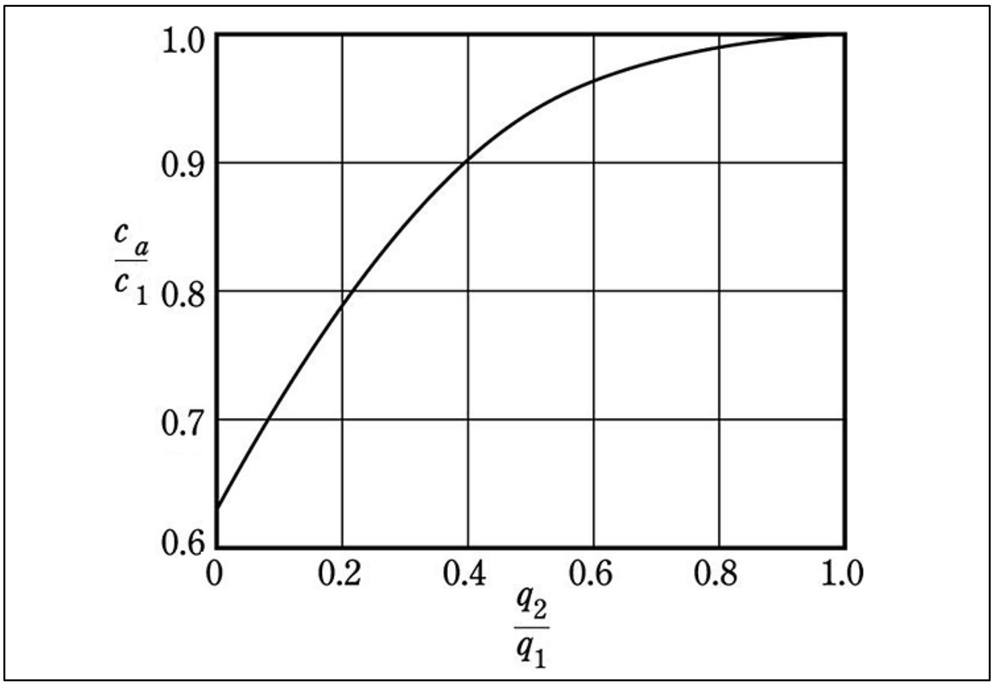



Figure B.2 Unit adhesion, $c_a$ (Das, 1999).

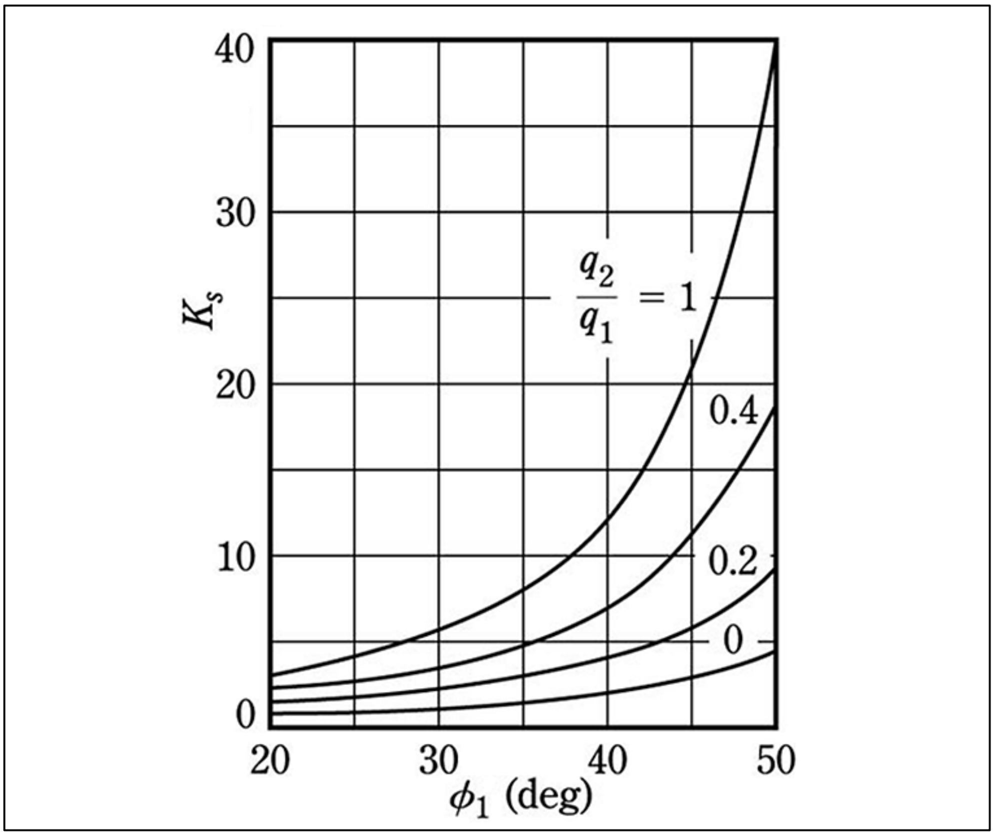
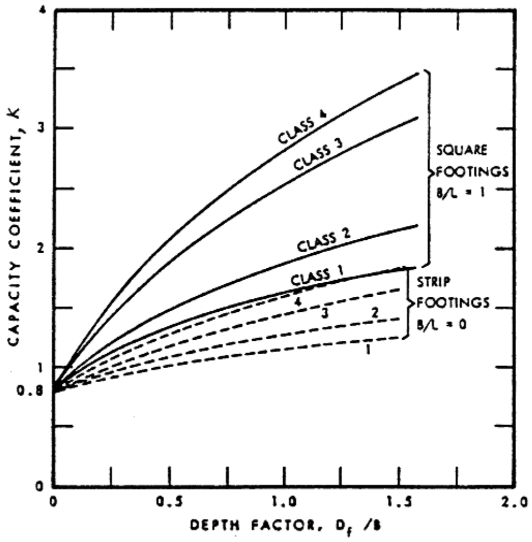



Figure B.3 Punching shear coefficient, $K_s$ (Das, 1999).

Table B.7 Approximate relationships between rock mass quality and material constants used in defining nonlinear strength (Hoek and Brown, 1988)

| **Rock quality** | **Constants** | **Rock type**<br><br>A = Carbonate rocks with well developed crystal cleavage— *dolomite, limestone, and marble*<br>B = Lithified argrillaceous rocks—*mudstone, siltstone, shale, and slate (normal to cleavage)*<br>C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite*<br>D = Fine grained polyminerallic igneous crystalline rocks— *andesite, dolerite, diabase, and rhyolite*<br>E = Coarse-grained polyminerallic igneous and metamorphic crystalline rocks—*amphibolite, gabbro, gneiss, granite, norite, quartz-diorite* | | | | |
|---|---|---|---|---|---|---|
| | | A | B | C | D | E |
| INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: *RMR* = 100 | m<br>s | 7.00<br>1.00 | 10.00<br>1.00 | 15.00<br>1.00 | 17.00<br>1.00 | 25.00<br>1.00 |
| VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft. CSIR rating: *RMR* = 85 | m<br>s | 2.40<br>0.082 | 3.43<br>0.082 | 5.14<br>0.082 | 5.82<br>0.082 | 8.567<br>0.082 |
| GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft. CSIR rating: *RMR* = 65 | m<br>s | 0.575<br>0.00293 | 0.821<br>0.00293 | 1.231<br>0.00293 | 1.395<br>0.00293 | 2.052<br>0.00293 |
| FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft. CSIR rating: *RMR* = 44 | m<br>s | 0.128<br>0.00009 | 0.183<br>0.00009 | 0.275<br>0.00009 | 0.311<br>0.00009 | 0.458<br>0.00009 |
| POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in; some gouge. Clean compacted waste rock. CSIR rating: *RMR* = 23 | m<br>s | 0.029<br>$3 \times 10^{-6}$ | 0.041<br>$3 \times 10^{-6}$ | 0.061<br>$3 \times 10^{-6}$ | 0.069<br>$3 \times 10^{-6}$ | 0.102<br>$3 \times 10^{-6}$ |
| VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced < 2 in with gouge. Waste rock with fines. CSIR rating: *RMR* = 3 | m<br>s | 0.007<br>$1 \times 10^{-7}$ | 0.010<br>$1 \times 10^{-7}$ | 0.015<br>$1 \times 10^{-7}$ | 0.017<br>$1 \times 10^{-7}$ | 0.025<br>$1 \times 10^{-7}$ |




| Soil Type | Consistency or Density | $(P_l\text{-}P_0)$ $kgf/m^2$ | Class |
|---|---|---|---|
| clay | soft to very firm | <12 | 1 |
| | stiff | 8 – 40 | 2 |
| sand and gravel | loose | 4-8 | 2 |
| | medium to dense | 10-20 | 3 |
| | very dense | 30-60 | 4 |
| silt | loose to medium | <7 | 1 |
| | dense | 12-30 | 2 |
| rock | very low strength | 10-30 | 2 |
| | low strength | 30-60 | 3 |
| | medium to high strength | $60\text{-}100^+$ | 4 |

Figure B.4 Values of empirical capacity coefficient, k (After Canadian Geotechnical Society, 1988).

Table B.8 Suggested values for Poisson's ratio, μ (Das, 1999)

| **Soil type** | **Poisson's ratio, μ** |
|---|---|
| Coarse sand | 0.15 - 0.20 |
| Medium loose sand | 0.20 - 0.25 |
| Fine sand | 0.25 – 0.30 |
| Sandy silt and silt | 0.30 – 0.35 |
| Saturated clay (undrained) | 0.50 |

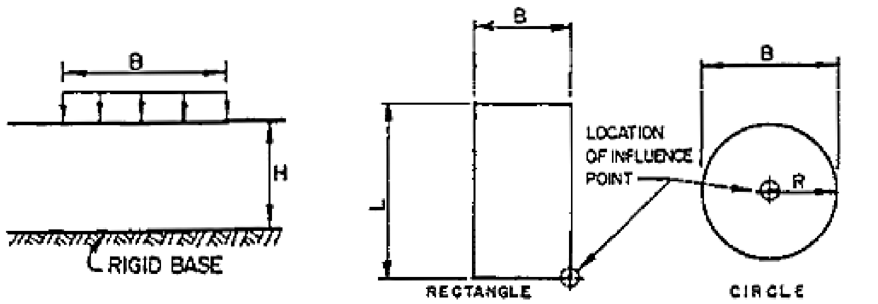
Table B.9 Elasticity modulus, $E_s$ (Erol, 2009)

| **Soil type** | **$E_s$ ($MN/m^2$)** |
|---|---|
| Soft clay | 2 – 5 |
| Hard clay | 7 – 20 |
| Sandy clay | 30 – 40 |
| Silty clay | 7 – 20 |
| Loose sand | 10 – 25 |
| Dense sand | 50 – 90 |
| Dense sand and gravel | 100 – 200 |

Table B.10 Shape and rigidity factors $I_s$ (NAVFAC DM, 1982)

Shape and Rigidity Factor I for Loaded Areas on an Elastic Half-Space of Limited Depth Over a Rigid Base

| H/B | Center of Rigid Circular Area Diameter = B | Corner of Flexible Rectangular Area | | | | |
|---|---|---|---|---|---|---|
| | | L/B = 1 | L/B = 2 | L/B = 5 | L/B = 10 | (strip) L/B = ∞ |
| for $\nu$ = 0.50 | | | | | | |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 0.5 | 0.14 | 0.05 | 0.04 | 0.04 | 0.04 | 0.04 |
| 1.0 | 0.35 | 0.15 | 0.12 | 0.10 | 0.10 | 0.10 |
| 1.5 | 0.48 | 0.23 | 0.22 | 0.18 | 0.18 | 0.18 |
| 2.0 | 0.54 | 0.29 | 0.29 | 0.27 | 0.26 | 0.26 |
| 3.0 | 0.62 | 0.36 | 0.40 | 0.39 | 0.38 | 0.37 |
| 5.0 | 0.69 | 0.44 | 0.52 | 0.55 | 0.54 | 0.52 |
| 10.0 | 0.74 | 0.48 | 0.64 | 0.76 | 0.77 | 0.73 |
| for $\nu$ = 0.33 | | | | | | |
| 0 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 0.5 | 0.20 | 0.09 | 0.08 | 0.08 | 0.08 | 0.08 |
| 1.0 | 0.40 | 0.19 | 0.18 | 0.16 | 0.16 | 0.16 |
| 1.5 | 0.51 | 0.27 | 0.28 | 0.25 | 0.25 | 0.25 |
| 2.0 | 0.57 | 0.32 | 0.34 | 0.34 | 0.34 | 0.34 |
| 3.0 | 0.64 | 0.38 | 0.44 | 0.46 | 0.45 | 0.45 |
| 5.0 | 0.70 | 0.46 | 0.56 | 0.60 | 0.61 | 0.61 |
| 10.0 | 0.74 | 0.49 | 0.66 | 0.80 | 0.82 | 0.81 |




NOTATION FOR LOADED AREAS, SHOWN IN PLAN VIEW

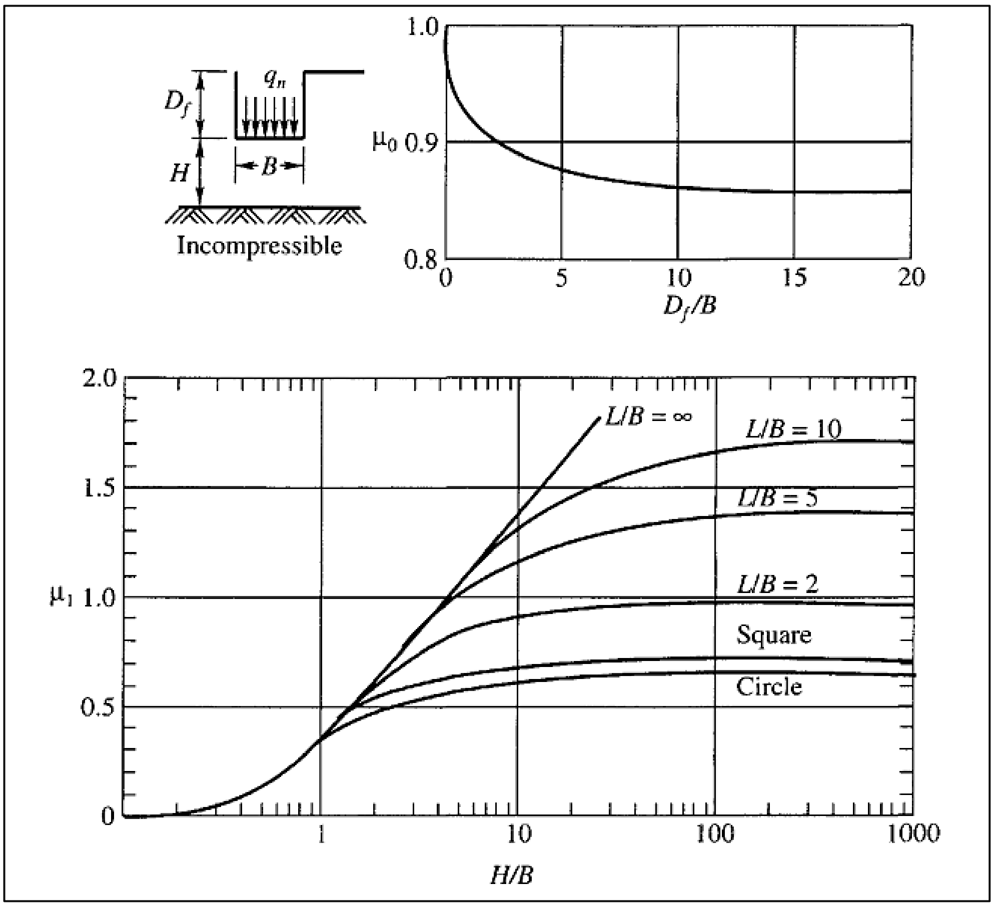



Figure B.5 Factors for calculating the average immediate settlement (after Christian and Carrier, 1978).

Table B.11 Correlation equations for soil compressibility/consolidation (Bowles, 1996)

| Compression index, $C_c$ | Comments | Source/Reference |
|---|---|---|
| $C_c = 0.009(w_L - 10)$ (±30% error) | Clays of moderate $S_t$ | Terzaghi and Peck (1967) |
| $C_c = 0.37(e_o + 0.003w_L + 0.0004w_N - 0.34)$ | 678 data points | Azzouz et al. (1976) |
| $C_c = 0.141G_s\left(\frac{\gamma_{sat}}{\gamma_{dry}}\right)^{2.4}$ | All clays | Rendon-Herrero (1983) |
| $C_c = 0.0093w_N$ | 109 data points | Koppula (1981) |
| $C_c = -0.0997 + 0.009w_L + 0.0014I_P + 0.0036w_N + 0.1165e_o + 0.0025C_P$ | 109 data points | Koppula (1981) |
| $C_c = 0.329[w_NG_s - 0.027w_P + 0.0133I_P(1.192 + C_P/I_P)$ | All inorganic clays | Carrier (1985) |
| $C_c = 0.046 + 0.0104I_P$ | Best for $I_P < 50\%$ | Nakase et al. (1988) |
| $C_c = 0.00234w_LG_s$ | All inorganic clays | Nagaraj and Srinivasa Murthy (1985, 1986) |
| $C_c = 1.15(e_o - 0.35)$ | All clays | Nishida (1956) |
| $C_c = 0.009w_N + 0.005w_L$ | All clays | Koppula (1986) |
| $C_c = -0.156 + 0.411e_o + 0.00058w_L$ | 72 data points | Al-Khafaji and Andersland (1992) |
| **Recompression index, $C_r$** | | |
| $C_r = 0.000463w_LG_s$ | | Nagaraj and Srinivasa Murthy (1985) |
| $C_r = 0.00194(I_P - 4.6)$ | Best for $I_P < 50\%$ | Nakase et al. (1988) |
| $= 0.05$ to $0.1C_c$ | In desperation | |
| **Secondary compression index, $C_\alpha$** | | |
| $C_\alpha = 0.00168 + 0.00033I_P$ | | Nakase et al. (1988) |
| $= 0.0001w_N$ | | NAFAC DM7.1 p. 7.1–237 |
| $C_\alpha = 0.032C_c$ | $0.025 < C_\alpha \le 0.1$ | Mesri and Godlewski (1977) |
| $= 0.06$ to $0.07C_c$ | Peats and organic soil | Mesri (1986) |
| $= 0.015$ to $0.03C_c$ | Sandy clays | Mesri et al. (1990) |

*Notes:* 1. Use $w_L$, $w_P$, $w_N$, $I_P$ as percent, not decimal.
2. One may compute the in situ void ratio as $e_o = w_NG_s$ if $S \to 100$ percent.
3. $C_p$ = percent clay (usually material finer than 0.002 mm).
4. Equations that use $e_o$, $w_N$, and $w_L$ are for both normally and overconsolidated soils.

Table B.12 Values of geological factor $\mu_g$ (Skempton-Bjerrum, 1957)

| Type of clay | $\mu_g$ |
|---|---|
| Sensitive clays, soft alluvial clays | 1.0 - 2.0 |
| Medium loose sand | 0.7 - 1.0 |
| Fine sand | 0.5 - 0.7 |
| Sandy silt and silt | 0.2 - 0.5 |

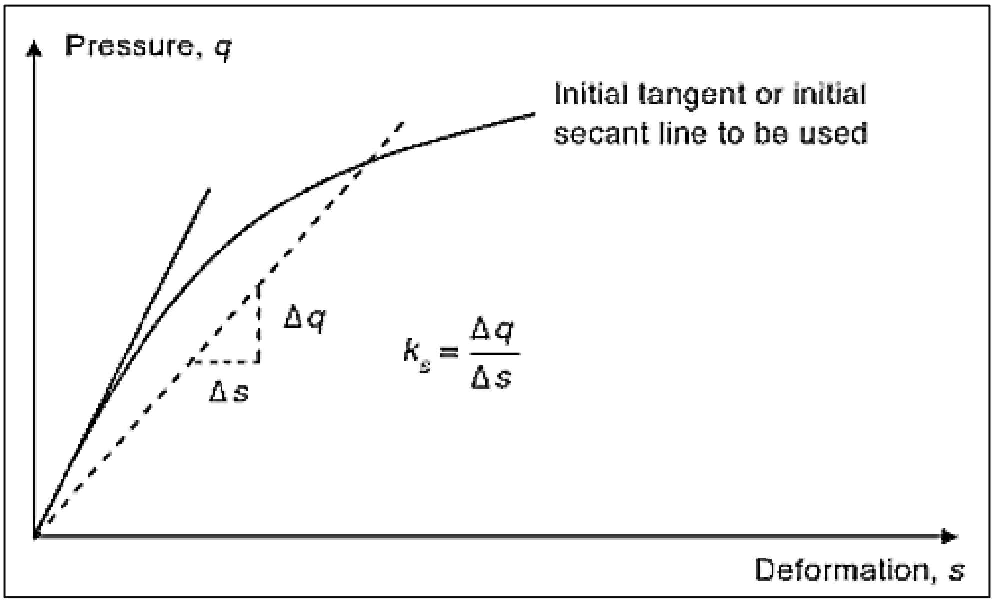


Figure B.6 Determination of modulus of subgrade reaction (Das, 2011)

# APPENDIX C

# GEOTECHNICAL REPORT

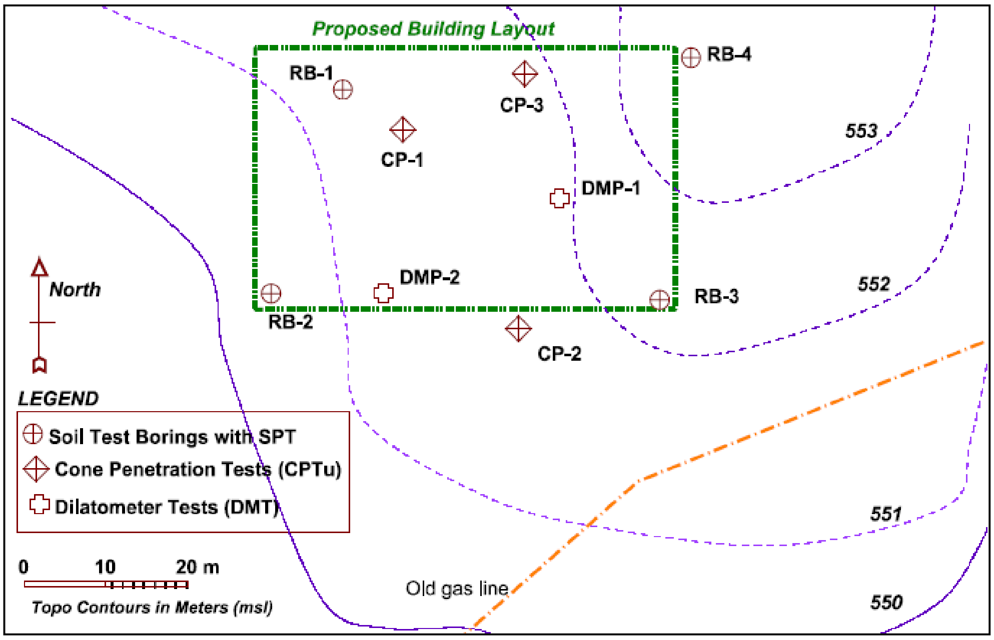



Figure C.1 Plan showing proposed boring and in-situ test locations (Mayne et al., 2001).

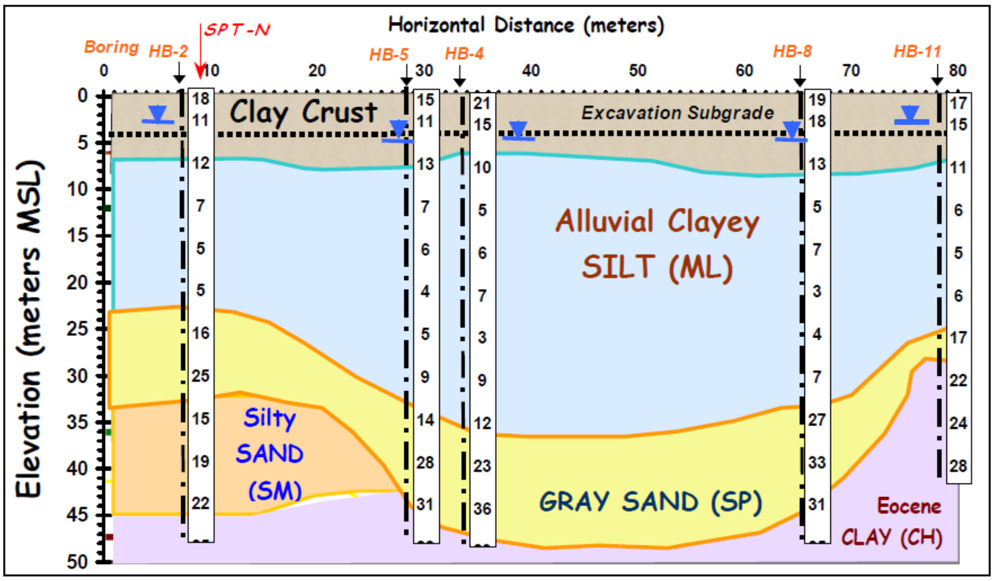



Figure C.2 Subsurface profile based on boring data showing cross-sectional view (Mayne et al., 2001).

## APPENDIX D

## EVALUATION OF GEOTECHNICAL REPORTS

Table D.1 List of reports

| |
|---|
| 1) Ankara Etimesgut, Block No: 45498, Parcel No: 16 |
| 2) Ankara Sincan-Yenikent, Block No: 472, Parcel No: 3 |
| 3) Ankara Sincan, Block No: 4353, Parcel No: 16 |
| 4) Ankara Yenimahalle-Yeşilevler, Block No: 60526, Parcel No: 1 |
| 5) Ankara Keçiören, Block No: 1834, Parcel No: 8 |
| 6) Ankara Keçiören, Block No: 30761, Parcel No: 9 |
| 7) Ankara Keçiören, Block No: 30359, Parcel No: 11 |
| 8) Ankara Keçiören, Block No: 31535, Parcel No: 7 |
| 9) Ankara Keçiören, Block No: 5483, Parcel No: 11 |
| 10) Ankara Yenimahalle, Block No: 42824, Parcel No: 1 |
| 11) Ankara Yenimahalle, Block No: 80, Parcel No: 6 |
| 12) Ankara Yenimahelle, Block No: 61140, Parcel No: 3 |
| 13) Ankara Yenimahalle, Block No: 9933, Parcel No: 7 |
| 14) Kırıkkale-Yahşiyan, Block No: 844, Parcel No: 3 |
| 15) Çorum, State Hydraulic Works 54. Branch Facilities |
| 16) Çankırı, State Hydraulic Works 52. Branch Facilities |
| 17) Ankara Sincan, Block No: 490, Parcel No: 1 |
| 18) Ankara Sincan, Block No: 4388, Parcel No: 6 |
| 19) Ankara Sincan, Block No: 2191, Parcel No: 3 |
| 20) Ankara Sincan, Block No: 262, Parcel No: 18 |
| 21) Ankara Sincan, Block No: 971, Parcel No: 11 |
| 22) Ankara Sincan, Block No: 739, Parcel No: 4 |
| 23) Ankara Sincan, Block No: 182, Parcel No: 9 |
| 24) Ankara Sincan, Block No: 877, Parcel No: 36 |
| 25) Ankara Etimesgut, Block No: 45415, Parcel No: 1 |
| 26) Ankara Etimesgut, Block No: 45755, Parcel No: 11 |
| 27) Ankara Etimesgut, Block No: 45072, Parcel No: 12 |
| 28) Ankara Etimesgut, Block No: 45962, Parcel No: 1 |
| 29) Ankara Etimesgut, Block No: 45476, Parcel No: 6 |

30) Ankara, Gülhane Military Medical Academy - Sports facility
31) Antalya Kumluca, New Courthouse, Block No: 115 Parcel No: 7
32) Konya, New Airport Terminal Building
33) İskenderun, Student Dormitory Block No: 2481, Parcel No: 1
34) Ankara Altındağ, Block No: 22102, Parcel No: 1
35) Ankara Çankaya, Block No: 27457, Parcel No: 5
36) Ankara Altındağ, Block No: 20760, Parcel No: 6
37) Ankara Çankaya, Block No: 27100, Parcel No: 5
38) Ankara Çankaya, Block No: 26454, Parcel No: 7
39) Ankara Çankaya, Block No: 26946, Parcel No: 11
40) Ankara Altındağ, Block No: 23301, Parcel No: 7
41) Ankara Çankaya, Block No: 28145, Parcel No: 1
42) Ankara Altındağ, Block No: 20927, Parcel No: 2
43) Ankara Çankaya, Block No: 13104, Parcel No: 1
44) Ankara Altındağ, Block No: 20969, Parcel No: 1
45) Ankara Çankaya, Block No: 26074, Parcel No: 4
46) Ankara Altındağ-Güneşevler, Block No: 22041, Parcel No: 1
47) Ankara Çankaya-Ahlatlıbel, Block No: 59, Parcel No: 9
48) Ankara Mamak, Block No: 6681, Parcel No: 12
49) Ankara Keçiören, Block No: 5994, Parcel No: 7
50) Ankara Mamak, Block No: 36577, Parcel No: 8
51) Ankara Keçiören, Block No: 33052, Parcel No: 15
52) Ankara Keçiören, Block No: 34369, Parcel No: 5
53) Ankara Keçiören, Block No: 32269, Parcel No: 11
54) Ankara Keçiören, Block No: 30362, Parcel No: 3
55) Ankara Keçiören, Block No: 31776, Parcel No: 11
56) Ankara Mamak, Block No: 36938, Parcel No: 37
57) Ankara Mamak, Block No: 36216, Parcel No: 7
58) Ankara Keçiören, Block No: 7902, Parcel No: 11
59) Ankara Mamak, Block No: 36507, Parcel No: 8
60) Ankara Mamak, Block No: 35859, Parcel No: 9
61) Ankara Gölbaşı, Block No: 112578, Parcel No: 5
62) Ankara Gölbaşı, Block No: 112584, Parcel No: 2
63) Ankara Gölbaşı, Block No: 295, Parcel No: 1
64) Ankara Gölbaşı, Block No: 118, Parcel No: 4
65) Ankara Keçiören, Block No: 6011, Parcel No: 30
66) Ankara Çankaya, Block No: 27427, Parcel No: 4

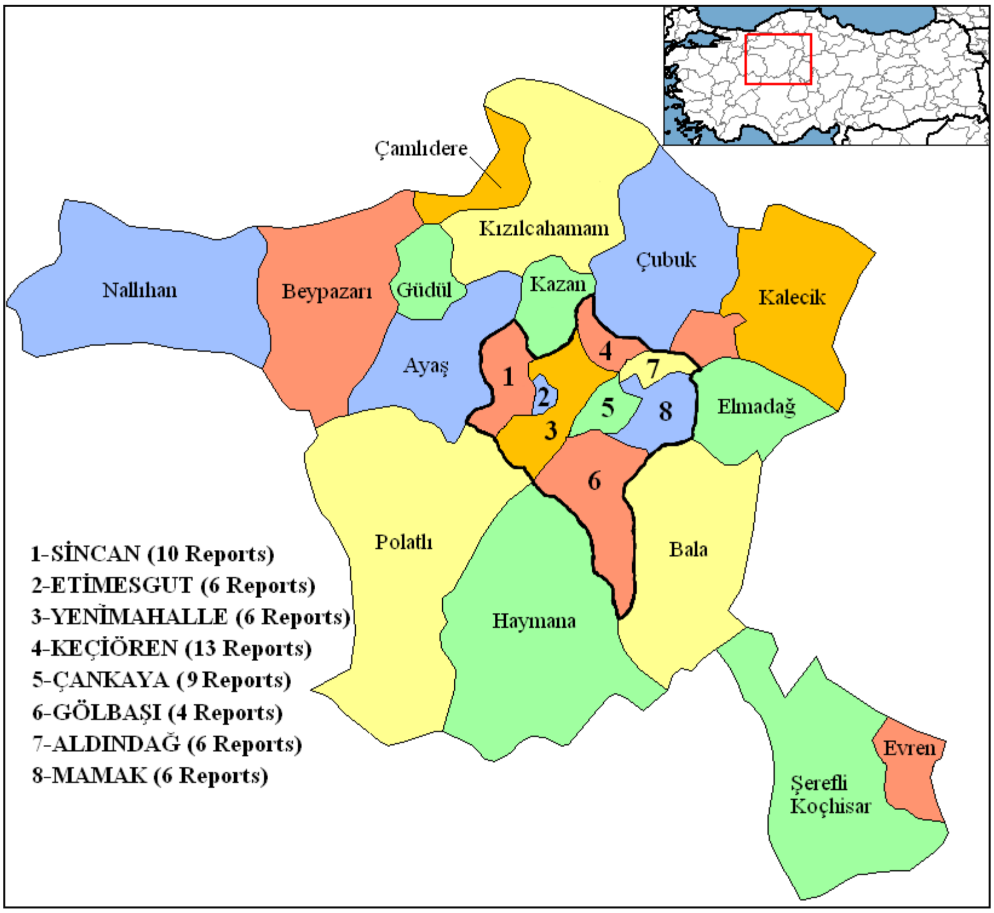


Figure D.1 Distribution of the geotechnical reports collected from central municipalities of Ankara.