Syllabus

BQS 312: CIVIL ENGINEERING TECHNOLOGY 45 Hours
This course introduces the student to civil engineering works.
Introduction to civil engineering works.

Introduction to soil mechanics and soil structures. Classification of soil into rocks, boulders, gravels, sands, silts, clays;

Properties of red coffee soils and black cotton soils;

Scope and conduct of site investigations; soil reports;

Earthworks in dams and road embankments; soil stabilization for improved bearing capacity;

Design and construction of foundations in difficult grounds - swelling clays, deep foundations, rafts and piling;

Design and construction of the following civil engineering structures: long span and short span bridges, retaining walls, tunnels and culverts.
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Chapter 1

Introduction to civil engineering works.

1.1 General

Civil engineering is one of the oldest of the engineering professions. Ancient feats such as the building of the Egyptian pyramids and Roman road systems are based on civil engineering principles.

Civil engineers can be found in all areas of society from small private contractors to municipal agencies, federal government organizations, and the military.

Branches of Civil Engineering

Some of the branches of civil engineering include:

1. Transportation – This branch of civil engineering is concerned with developing transportation systems, including highways, airports and runways, and rail systems.
2. Environmental – Environmental engineering involves wastewater treatment, air pollution management, and the handling and processing of hazardous wastes.
3. Geotechnical – Geotechnical engineering includes the design and construction of rock and soil based structures, including foundations and retaining walls, dams, and water retaining structures.
4. Structural – Structural engineering includes the design and construction of steel structures, including buildings, bridges, tunnels, and offshore structures such as oil rigs.
5. Water Resources

This branch includes construction of dams, canals, and water pipeline systems, as well as conservation and resource management.

Civil engineering works are mainly undertaken by government or public authorities. The employer needs to know the viability of the project and therefore requires to pass through a feasibility stage. The project then passes through a, preliminary design and detailed design before construction and eventual commissioning. Table 1 shows the various stages of a civil engineering project.
<table>
<thead>
<tr>
<th>Phase</th>
<th>Pre-construction</th>
<th>Construction</th>
<th>Post Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage</td>
<td>Inception/feasibility</td>
<td>Preliminary design</td>
<td>Detailed design</td>
</tr>
<tr>
<td>Activities</td>
<td>Planning Drawing or Writing of -Terms of Reference -Proposals</td>
<td>Design Alternatives Selection of the suitable alternative Site Investigation Engineering surveys Calculations Drawings Cost Estimates</td>
<td>Detailed Site Investigations -Boreholes -Trial pits etc Calculations Drawings Quantities Bidding documents Engineering estimates</td>
</tr>
<tr>
<td>Output</td>
<td>Terms of reference Proposals</td>
<td>Preliminary design report -Preliminary design -Choice of alternative -Requirements for detailed design -Cost estimate</td>
<td>Detailed design report -SI report -Calculations -Drawings -Bidding documents -Engineers cost estimate</td>
</tr>
<tr>
<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>
Chapter 2

Introduction to Geotechnical Engineering

2.1 Geotechnical engineering

Geotechnical engineering is the branch of engineering which deals with the design construction and maintenance of foundations. It also includes the investigation of sites for foundation purposes. It involves the application of laws of mechanics and hydraulics to engineering problems can be broadly be said to consist of:-

i) Assessment of the ability of soil deposits to provide support to various structures
ii) Evaluation of the magnitude and distribution of earth pressure against various structures
iii) Prediction of movement of water through soils
iv) Evaluation of stability of manmade and natural slopes
v) Analysis and design of dams
vi) Improvement of soil properties by chemical and/or mechanical methods

Definition of soil and rock

Soil is defined as a natural aggregate of mineral grains, loose or moderately cohesive, inorganic or organic in nature that has capacity of being separated by means of simple mechanical process e.g. by agitation in water. Soils are thus non-homogenous porous materials whose properties are greatly influenced by Rock on the other hand is hard a compact natural aggregate of mineral grains
cemented by strong and more or less permanent bonds. Geologists classify rocks into three basic groups, namely igneous, sedimentary a metamorphic

**Igneous rocks:** These rocks have formed from a melted liquid state. Extrusive igneous rocks are those that have arrived on the surface of the earth as molten lava and cooled. Intrusive igneous rocks are formed from magma (molten rock) that has forced itself through cracks into rock beds below the surface and solidified there. Examples of igneous rocks include granite, basalt and gabbro

**Sedimentary rocks:** Weathering result to fragmented particles which can be more easily transported by wind, water or ice. When dropped by agents of weathering they are termed as sediments. These sediments are deposited in layers or beds. When these sediments are compacted and cemented they form sedimentary rocks. The rocks in this classification include shale, sandstone and chalk

**Metamorphic rocks:** Metamorphic rocks result from action of high temperatures and pressure on sedimentary and igneous rocks. The original rock undergoes both chemical and physical alterations. These rocks include slate, quartzite and marble.

### 2.2 Soil formation

Mechanical and chemical weathering of rocks forms soils. Mechanical weathering of the rocks leaves the crystalline structure of the material unchanged. On the other hand the chemical weathering of the rocks results in crystalline change of the materials. Chemical and physical weathering often take place simultaneously and mutually accelerate each other

The soil formation in the tropics can be seen in Figure shown below

![Soil formation diagram](image)

- Completely weathered phase residual
- Finely spaced weathering joints ï gravel
- Largely spaced weathering joints
- Un-weathered joints rock
2.3 Particle Size Distribution and Classification

The sizes of soil particles and their distribution are important factors which influence the engineering properties and performance. It is common practice to classify soils on the basis of the particle size. The particle size in the soils varies from over 100mm to less than 0.001mm. In British standards the size ranges are detailed in Table 2.1. In the Table, the nomenclature of clay, silt, sand, gravel, cobbles and boulders are defined by the size ranges of the particles. However the same terms are used to describe particular types of soils. For example clay as will be explained later is a type of soil possessing cohesion and plasticity. The clay soils normally consist of soils in both clay silt ranges. Further all clay size particles are not necessary clay minerals. Thus the finest rock flour particles may be of clay size particles but are not referred to as clay. It is imperative that the process of weathering has formed the clay particles.

Table 2.1 Particle size and classification of soils

<table>
<thead>
<tr>
<th>Clay size (mm)</th>
<th>Silt</th>
<th>Sand</th>
<th>Gravel</th>
<th>Cobble</th>
<th>Boulders</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.002</td>
<td>0.006</td>
<td>0.02</td>
<td>0.06</td>
<td>0.2</td>
<td>0.6</td>
</tr>
<tr>
<td>0.6</td>
<td>2</td>
<td>6</td>
<td>20</td>
<td>60</td>
<td>200</td>
</tr>
</tbody>
</table>

In soils with clay mineral particles the clay particles influence the behavior of the soil in a proportion greater than the percentage of the clay content. Soils whose the properties are influenced mainly by clay and silt are referred to as fine grained soils while those whose properties are influenced by sand and gravel particles are referred to as coarse grained soils.

2.3 Plasticity of Fine Grained Soils

Plasticity describes the ability of a soil to undergo unrecoverable deformation at constant volume without cracking or crumbling. The plasticity is due to the presence of clay minerals or organic material. The physical state at particular water content is called consistency of the state.

Depending on the water content the soil may exist as liquid, plastic, semi solid or solid state. The water contents at which these transitions take place vary from soil to soil. The clay minerals have cohesive forces between them.
Any decrease in the water content increases the net attractive forces between the particles. In the plastic state the particles are free to slide relative to one another. A decrease in the water content also results in the decrease in the volume of the soil in the liquid, plastic, semi-solid and solid states.

In the liquid state the soil flows freely and has no shear strength. As the water content is reduced the soil gains some shear strength. The water content at which the soil gets some small shear strength is the liquid limit. At moisture content lower than the liquid limit the soil becomes plastic. In this state it can be molded into various shapes. It eventually loses its plasticity at plastic limit. After the plastic limit the soil exhibits the properties of a semi solid where it crumbles on being rolled into a thread instead of distorting plastically. It then passes over to the solid state. In the solid state a decrease in the water content does not result in the decrease of the volume of the soil. Figure 2. 1 shows the variation of the volume with water content and the various states of the soil.

The plasticity of soils is described by the limits described above and defined as below. The plasticity limits are commonly referred to as Atterberg limits after a Swedish scientist who developed the testing at the beginning of the 20th century and can be summarized as below: - the determination of these limits are presented in section 2.4.5.

*The liquid limit (LL)* is defined as the water content at which a moist soil in a special cup cut by a standard groove closes after 25 taps on a hard rubber plate

*The plastic limit (PL)* is the water content at which the soil starts to break apart when rolled into a thread which is approximately 3mm in diameter.
Figure 2.1 Various states of fine grained the soils

*The shrinkage limit (SL)* is the water content at which further loss of moisture content does not result in the decrease in volume.

*Plasticity index (PI)* is the range of water content over which the soil exhibits plasticity (Equation 2.22)

\[ PI = LL - PL \]  \(2.1\)

*Liquidity index (LI)* relates the natural water content of the soil to the plastic and limit and the plastic index (Equation 2.23)

\[ LI = \frac{w - PL}{PI} \]  \(2.2\)

*The activity* of the clay is the ratio of the PI to the percentage of clay size particles (Equation 2.24)

\[ Activity = \frac{PI}{\%Clay} \]  \(2.3\)
2.4 Soil Description

A detailed description of the method of describing soils is contained in BS 5930. The basic soils are boulders, cobbles, gravels, sand, silt and clay. The size ranges for these basic soils are given on Table 2.2. Often soils appear in mixtures and are referred to composite types.

In accordance to BS 5930 a soil is of basic type sand or gravel (coarse soil), if after removal of boulders and cobbles, over 65% of the material is in the sand and gravel range. A soil is of basic type silt or clay (fine grained soil) when over 35% of the soil is in the silt and clay range. Composite soils are named as described in Table 2.2. Mixtures containing over 50% boulders and cobbles are referred to as very coarse soils. The descriptions my be of the form COBBLES with finer material or gravelly SAND with occasional cobbly BOULDERS.

The firmness or strength of the soil in the field can be assessed by means of tests shown on Table 2.3.

Virgin soils
### Table 2.2 Soil description terminology

<table>
<thead>
<tr>
<th>Soil group</th>
<th>Soil description</th>
<th>Indicative particle size distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>COARSE SOILS</td>
<td>Slightly sandy GRAVEL</td>
<td>Up to 5% sand</td>
</tr>
<tr>
<td></td>
<td>Sandy GRAVEL</td>
<td>5-20% sand</td>
</tr>
<tr>
<td></td>
<td>Very sandy GRAVEL</td>
<td>Over 20% sand</td>
</tr>
<tr>
<td></td>
<td>GRAVEL/SAND</td>
<td>About equal proportions</td>
</tr>
<tr>
<td></td>
<td>Very gravelly SAND</td>
<td>Over 20% gravel</td>
</tr>
<tr>
<td></td>
<td>Gravelly SAND</td>
<td>5-20% gravel</td>
</tr>
<tr>
<td></td>
<td>Slightly gravelly SAND</td>
<td>Up to 5% gravel</td>
</tr>
<tr>
<td></td>
<td>Slightly silty SAND (or GRAVEL)</td>
<td>Up to 5% silt</td>
</tr>
<tr>
<td></td>
<td>Silty SAND (or GRAVEL)</td>
<td>5-15% silt</td>
</tr>
<tr>
<td></td>
<td>Very silty SAND (or GRAVEL)</td>
<td>15-35% silt</td>
</tr>
<tr>
<td></td>
<td>Slightly clayey SAND (or GRAVEL)</td>
<td>Up to 5% clay</td>
</tr>
<tr>
<td></td>
<td>Clayey SAND (or GRAVEL)</td>
<td>5-15% Clay</td>
</tr>
<tr>
<td></td>
<td>Very clayey SAND (or GRAVEL)</td>
<td>15-35% Clay</td>
</tr>
<tr>
<td>FINE SOILS</td>
<td>Sandy SILT (or CLAY)</td>
<td>35-65% Sand</td>
</tr>
<tr>
<td></td>
<td>Gravelly SILT (or CLAY)</td>
<td>35-65% Gravel</td>
</tr>
</tbody>
</table>

### Table 2.3 Field identification tests

<table>
<thead>
<tr>
<th>Soil</th>
<th>Term</th>
<th>Field test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sands and gravels</td>
<td>Loose</td>
<td>Can be excavated by spade. A 50 mm wooden peg can be easily driven</td>
</tr>
<tr>
<td></td>
<td>Dense</td>
<td>Requires a pick to excavate. 50 mm wooden peg difficult to drive</td>
</tr>
<tr>
<td></td>
<td>Slightly cemented</td>
<td>Visual examination. Pick removes lumps</td>
</tr>
<tr>
<td>Silts</td>
<td>Soft or loose</td>
<td>Easily molded or crushed in the fingers</td>
</tr>
<tr>
<td></td>
<td>Firm or dense</td>
<td>Can be molded or crushed by strong pressure in fingers</td>
</tr>
<tr>
<td>Clays</td>
<td>Very soft</td>
<td>Exudates between fingers when squeezed in the hand</td>
</tr>
<tr>
<td></td>
<td>Soft</td>
<td>Molded by light finger pressure</td>
</tr>
<tr>
<td></td>
<td>Firm</td>
<td>Can be molded by strong finger pressure</td>
</tr>
<tr>
<td></td>
<td>Stiff</td>
<td>Can not be molded by fingers</td>
</tr>
<tr>
<td></td>
<td>Very stiff</td>
<td>Can not be indented by thumb nails</td>
</tr>
<tr>
<td>Organic &amp; Peat</td>
<td>Firm</td>
<td>Fibers already compressed together</td>
</tr>
<tr>
<td></td>
<td>Spongy plastic</td>
<td>Very compressible and open structure. Can be molded by fingers and smears fingers</td>
</tr>
</tbody>
</table>
Mixed up soils

2.5 Properties of red coffee soils and black cotton soils;

The clay particles are formed by chemical process. In the chemical process the parent rock changes its structure in the presence of acid, alkali, water, oxygen or carbon dioxide. The chemical weathering results in the formation of crystalline particles of colloidal size less than 0.002mm. Most of the clay particles are plate like having a high surface area to mass ratio. They belong to a family of mineral particles called phylloclites and are principally from quartz and feldspar minerals.

Mixed with water the clays become plastic. At low water contents they are hard and susceptible to cracking. At high water contents they are fluid while at high temperatures they form hard industrial products of earthenware. When excess water is available the clays will tend to expand. As the water recedes from the soil there is an accompanying shrinkage. The common types of the clays are shown on Table 1.2. based on the susceptibility to change in volume.
Table 2. 4 Common types of clays

<table>
<thead>
<tr>
<th>Clay type</th>
<th>Susceptibility to volume change</th>
<th>Parent rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>Montimorillonite</td>
<td>High to very high</td>
<td>Shales</td>
</tr>
<tr>
<td>Illite</td>
<td>Moderate</td>
<td>Shales</td>
</tr>
<tr>
<td>Kaolinite</td>
<td>Low to very low</td>
<td>Felspar, granite and basalt</td>
</tr>
</tbody>
</table>

Structure and chemistry

The main structural units of clay minerals consist of silica tetrahedron an alumina octahedron (Figure 2.1). The basic unit in the silica tetrahedron mineral is chemically $(SiO_4)$ where one silicon cation combines with four oxygen cations. The repeating combined unit in the silica tetrahedron three dimension sheet is $(Si_4O_{10})^{-4}$ which has 4 negative charges. The repeating structure in the alumina octahedron is $(Al_3(OH)_3$. This is chemically stable.

Silicon and aluminum cations are generally replaced by other elements in what is known as isomorphous substitution to form various types of clays. The main groups of clays are kaolinite, illite and montimorillonite as shown on Figure 1.2. Their features are briefly outlined below:-

![Silica tetrahedron](image1)

Silica tetrahedron

![Alumina octahedron](image2)

Alumina octahedron

![Silica sheet](image3)

Silica sheet

![Alumina sheet](image4)

Alumina sheet

**Figure 2.1 Basic units of clay minerals**
**Kaolinite group of Clays**

The kaolinite group of clays consists of residual clay deposits. They are a combination of silica and alumina sheets. In basic combination the alumina octahedron sheets loose $4(OH)$ resulting in a positively charged chemical $[AL_4(OH)_8]^{4+}$. This chemical combines with the negatively charged $[Si_4O_{10}]^-$. The result is a stable structure. It has very few charges on the two layer system. The result is a clay of low swelling and shrinkage responses to water content variation. The basic structure is seen in Figure 2.2. The combined silica alumina sheets are tightly held in one direction and consist of hundreds of stacks. The kaolinite group of clays includes the red coffee soil found in large areas of the tropics.

![Figure 2.2 Kaolinite group of clays](image)

**Illite group**

The illite group has a basic structure of alumina octahedron looses 4 hydroxyl cations at the top and the bottom. The resulting positively charged alumina sheet combines with silica tetrahedron to form a chemically stable pyrophyllite as illustrated in Figure 2.3. There is partial substitution of aluminum with magnesium and iron which are lower valence minerals. Further substitution of silicon by aluminum also occurs. The result is a negative charge on the surface of the sheet. Potassium ions are then attracted between the sheets to neutralize the negative charges. But not all the negative charges are neutralized and the result is a charged clay soil. The illites tend to have higher swelling and shrinkage characteristics than the kaolinites.
"Remove 4(OH) ions at the top and bottom" $\left\{ \begin{array}{c} \frac{6(OH)}{4(Al)} \\ \frac{6(OH)}{} \end{array} \right\}^{+4} \quad \frac{2(OH)}{4(Al)} \quad \frac{2(OH)}{2}^{+4}$

"Combines with $[Si_4O_{10}]_4$ to form pyrophyllite" $\left\{ \begin{array}{c} \frac{2(OH)}{4(Al)} \\ \frac{2(OH)}{2} \end{array} \right\}^{+4}$

$\frac{Si_4O_{10}(OH)_2}{4(Al)} \quad \frac{Si_4O_{10}(OH)_2}{2(Al)}$

Substitution $\frac{Al^{3+} for Si^{4+}}{Mg^{2+} and Fe^{3+} for Al^{3+}}$

To give $\left\{ \begin{array}{c} AlSi_3O_{10}(OH)_2 \\ MgFe(Al)_2 \\ AlSi_3O_{10}(OH)_2 \end{array} \right\}^{+3}$

\( a) \quad \text{Isomorphous substitution} \)

\( b) \quad \text{General structure} \)

\textbf{Figure 2.3 Illite group of clays}

\textbf{Montmorillonite group}

These are characterized by a three-layered system. In the formation of montmorillonite silica octahedron initially combines with alumina octahedron to form the intermediate pyrophyllite as in the case of the illite group of clays. Subsequently there is partial substitution of aluminum (+3) with magnesium (+2) of a lower valency as illustrated in Figure 2.4. The result is a negatively charged three layer sheets. In the presence of ionized water the arrangement of the hydroxyl ions in the water result in a magnet type of arrangement where the hydroxyl end is negative and the hydrogen end is positive. There is attraction of the hydroxyl end of water between the three layered systems. The water is held by the clay structure and the result is swelling of the montmorillonite clay.
This is the black cotton soil which is abundant in the various parts of tropics. It is particularly present in the poorly drained parts of the country.

\[
\begin{align*}
\frac{Si_4O_{10}(OH)_2}{4(Al)} & \quad \text{Substitution} \\
& \quad \text{Mg}^{2+} \text{ for } Al^{3+} \\
& \quad \text{To give} \\
& \quad \frac{Si_4O_{10}(OH)_2}{Si_4O_{10}(OH)_2} \gamma^{-1/2}
\end{align*}
\]

\(a)\quad \text{Isomorphous substitution}

\[\text{Ionized water}\]

\(b)\quad \text{General structure}

Figure 2.4 Montmorillonite group of clay minerals

**General characteristics of clays**

The surfaces of clay minerals carry negative charges mainly as a result of the isomorphous substitution of aluminum and silicon by atoms of lower valency. The negative charges result in the positive cations present the water to be attracted to the particles. The effect is formation of a dispersed layer. The dispersed layer is known as a double layer as seen the montmorillonite structure in Figure 2.4. Forces of repulsion and attraction act between the particles. An increase in the cation valency resulting from positive cations from available metals reduces the repulsive forces and vice versa. The inter-particle forces are known as van de Waal forces.

The net inter-particle forces influences the structural form assumed by the clay minerals. When the forces are net repulsion particles are usually...
dispersed, while when the net forces are attraction the particles tend to flocculate. These arrangements can be seen in Figure 1.5a and b. The structural arrangement can be complex depending on the environment. Such arrangement include book-house, turbo-static and silt (Figure 1.5c-e)

a) Dispersed (repulsion)  
b) Flocculated (attraction)  
c) Book-house  
d) Turbo-Static  
e) Silt

Figure 1.5 Structural arrangement of clay soils
Chapter 3.

Scope and conduct of site investigations; soil reports;

3.1 Introduction

Site investigations are also referred to as soil exploration. It consists of investigating the condition on which construction is planned. From site investigation it should be possible to obtain information for the following geotechnical engineering activities

i. Design of new foundations
ii. Modification of existing foundations
iii. Location of materials of construction of roads, runways, etc
iv. Identification of materials needed for the construction of pavement structures for roads, runways etc
v. Identification of ground to be excavated in the construction of various facilities including water pipe lines, building foundations, earthworks in cut areas etc

The site investigation should form a part of a coordinated chain of design from inception of the project through preliminary to the final detailed design of a civil engineering project. It should indeed continue post construction monitoring of the completed schemes. Because of the diversity of civil engineering schemes a set of standard procedures is not possible for all site investigations. The varying civil engineering schemes require a variety of options in breadth and detail needed for the various schemes. The objectives for which a site
investigation is carried out also differ with various schemes. The main objectives of carrying out a site investigation are now presented

**Suitability of site for particular works:** In the case of option of site for particular works a detailed site investigation should be able to enable determination of the most suitable site. Thus it is possible to shift a bridge from one location which would call for expensive deep foundations to one where ordinary shallow foundations would be sufficient.

**Adequate and economic design:** A site investigation leads to safe structures during and after construction. Additionally sufficient information is obtained for quantifying the excavations needed in the preparation of the bills of quantities. This should minimizes the possibility of cost overruns due to unexpected ground conditions being met at construction time.

**Planning construction:** By identifying different materials along the construction paths and their locations a systematic procedure of carrying out the works is evolved. In the case of road works materials from the cut areas are analyzed for use in the fill areas. It is then possible to proceed with construction of the fills and cuts methodically with minimum haulages and waste of materials.

**Prediction in changes in structure:** Carefully and well executed site investigations should enable the prediction of the likely settlement of structures under construction. Equally important is the ability to predict the effect of excavations on the neighboring structures.

**Safe structural design of large structures:** Heavy modern structures require more detailed site investigations. Today we are seeing higher buildings, larger bridges and installations sensitive to settlement. Structures and civil engineering schemes are being put up very quickly. Immediate and consolidated settlement is taking place when the works are commissioned. Further settlement takes place during the useful life of the civil engineering installation. Accurate estimation of the settlement regime is particularly important considering that clients are becoming more and more sensitive to the performance of structures and the argument that cracks are minor and do not pose any danger to the structure is no longer good.
<table>
<thead>
<tr>
<th>Stage</th>
<th>Conceptual Initial design</th>
<th>Pre-construction</th>
<th>Detailed design</th>
<th>Supervision of construction</th>
<th>Post Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main activity</td>
<td>Conceptual design</td>
<td>Design Alternatives</td>
<td>Detailed Site</td>
<td>Construction control</td>
<td>Operation &amp; Maintenance</td>
</tr>
<tr>
<td>Site investigation activity</td>
<td>Define Scope of SI</td>
<td>Desk study of SI</td>
<td>Detailed investigations</td>
<td>Field observations</td>
<td>Monitoring and checking</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Review of existing data</td>
<td>- Boreholes</td>
<td>- field densities</td>
<td>performance</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Preliminary trial pits</td>
<td>- Trial pits etc</td>
<td>- field moisture</td>
<td>- pore water pressures</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Laboratory and field tests</td>
<td></td>
<td>contents</td>
<td>Settlement</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Inclinations</td>
</tr>
<tr>
<td>SI Reports</td>
<td>Terms of reference and bid documents</td>
<td>i) Preliminary SI investigation report</td>
<td>Detailed design report -SI report</td>
<td>As built SI report -</td>
<td>-Maintenance reports -Performance reports -Research reports</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ii) Cost estimate of SI</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
3.2 Site investigations details

i) A study of any existing site investigation reports for the area or in the neighborhood should form the basis of this stage of investigations.

ii) A study of geographical a geological maps of the site in the case of large sites. Topographical characteristics should lead to useful information such faulty areas. Heavily forested areas are an indication of deep rooted top soils.

iii) A site inspection of the existing buildings and any existing structures. Any signs of distress which can be related to the settlement of the foundations. Any information from archives, previous records held by the local authorities.

iv) Inspection of the soil profiles, in cut areas, old used quarries. Structured questions to local people with regard to the geotechnical information being sought yields considerable information. Such questions are:

   a) What is the depth of the pit latrines in the area?
   b) At what depth murram encountered?
   c) At what depth was water struck?

v) Aerial survey of the site could give useful information with regard to land formations and soil profiles.

vi) Seismic refractions could be carried out at this stage of investigations. Usually a specialist is needed to interpret the results.

vii) Preliminary trial pits

Geophysical methods
Geophysical methods involve sending of seismic or electrical waves through the ground. The determination of the soil strata is based on the fact that the velocity or the resistance seismic wave transmission or resistance to electrical flow differs with different rock types and soils. The method allows the boundaries of the soils to be determined seismic refraction is described below

Seismic refraction is conducted by having a source of seismic waves. The seismic waves are induced by detonating a small explosive or by striking a metal plate hard. Waves are subsequently emitted in all directions, through the
air, and through the soil in all directions. Seismic wave transducers called geophones are placed radially from the epicenter. A circuit connects the geophones and the detonator for accurate determination of time. A direct wave will reach the geophone first since it is the shortest distance covered. When there is a dense stratum at depth a refracted wave will travel along the top of the bed rock. As it travels it leaks energy to the surface which can be picked by the geophone.

Seismic refraction

For short distances the direct waves reach the geophones first. For longer distances the refracted wave reaches first though the distances is longer than the surface direct distance. This is so because the speed of the wave in the dense material is higher than that in the overburden material of less density. The geophone has a mechanism which records the first wave and ignores the others. This enables a plot of arrival time versus the distance.

![Seismic source](image)

Figure 3.2 Time versus distance for seismic waves

The first section of the graph represents the direct wave measurements while the second section represents the refracted wave measurements. The inverse of these curves are the velocities of the seismic waves. The general types of the rocks are determined by geophysics from the knowledge of velocity versus rock type. It is also used in the determination of depth to water table and thicknesses of multiple strata. The depth D to the bedrock can be estimated from the formula.
The trial pits
The pit and shaft technique supplies the most detailed and reliable data on the existing soil conditions. Once the trial pit has been dug stratification of the soil should be done usually in the field. In addition as much information should be recorded. This information includes
i. Depth to ground water table.
ii. Field assessment of the bearing capacity.
iii. Depth of the various strata encountered in the trial pit.
iv. The encountered soils should be classified by visual inspection

a. Coarse grained soils should be described with adjectives such as angular, rounded with traces of fines etc
b. Fine grained soils should be studied to indicate whether they are loamy, of low plasticity, whether they are sandy clays etc
c. All soils should be described indicating their color and odour if any. Decaying organic matter if encountered should be mentioned.

v. Obtain undisturbed samples when you can for the different layers of strata encountered. These samples can then be taken to the laboratory for tests

For large sites the pits should then be surveyed and located in a grid system for incorporation into the site investigation report.

Sounding tests
These are basically are penetration tests carried out to supplementing trial pits and borings. The penetration resistance is measured and related to the bearing capacity. They are widely used in site investigations. They consist of the cone penetrometer already presented in chapter 1. The other commonly used penetration equipment is the dynamic cone penetrometer used in the estimation of the California bearing ratio (CBR) of road pavement layers. This enables the design of the pavement layers to be carried out
**Boring methods**

When a deep stratum has to be investigated it will usually be necessary to perform boring operations to ascertain the strata below the ground to be used in the support of the proposed structures. Several boring methods are available and are summarized as follows

**Percussion drilling** consists of a derrick, a power unit and a winch carrying a light steel cable which passes thorough a pulley. The unit can be towed by a vehicle after the assembly is folded. The assembly drops a chisel on the ground and strata being drilled

![Figure 3.4 Schematic presentation of a drilling chisel](image)

The excavation is effected by the drilling chisel. The drilling rods provide the necessary weight for the penetration the strata. Further weight may be added when need arises.

**Power operated augers** are usually on vehicles. Downward pressure is applied by pressure or dead weight. The augurs are 75-300mm diameters. Augers are usually used in self supporting soils. Casing is usually not needed since the augers have to be removed before driving. In full flight augers the rod and the helix cover the entire length being investigated. The augur is then brought up. The soil is ejected by reverse rotation. The likely hood of soil from different strata being mixed up is very high. In the short flight augur the auger is advanced into the soil and then raised. The soil is also ejected by reverse rotation.
The continuous flight augurs are sometimes fitted with a hollow stem which is plugged during the drilling operations. When samples are needed the plug and the rods are removed and a sampler is introduced for the recovery of a sample. The sample may be undisturbed depending on the sampler utilized. The flight augurs are not suitable for use in loose soils which are likely to collapse as the augur is inserted and removed from the hole.

**Hand and portable augers** are usually operated by persons by turning the handle of the augur. The hand augers are typically of 75–300mm diameters. The soil is locked in the auger and frequent removal is needed to ensure that the augur does not get stuck in the soil. Undisturbed samples may be obtained by introduction of small diameter tubes which are hammered into the strata under investigation. This method is suitable for self supporting soils. It is not possible to penetrate coarse granular soils.
**Rotary drilling** is done by use of drilling bits that cuts and grinds the subsoil or rock at the bottom of the borehole. Water is usually pumped down hollow rods passing under pressure through to the drilling tools. This cools and lubricates the bits. The fluid also provides support for the borehole where there is no casing.

**Sampling**

Disturbed and undisturbed samples are recovered from trial pits and along drilling tools where there is no attempt to retain the soil constituents. Disturbed samples should however be collected carefully and placed in airtight tins or jars or in plastic sampling bags. The samples should be labeled to give the borehole or trial pit identification number, depth of recovery and field description should be done. The disturbed samples are used for identification tests namely Field moisture content, PI, grading, compaction and CBR.

Undisturbed samples are recovered from trial pits and along drilling tools where there is an attempt to retain the soil constituents. Such a sample is taken in an airtight container with wax at both ends to prevent moisture from escaping during transportation to the laboratory.

**Borehole logs**

Borehole logs summarizes all the laboratory an field tests carried out on samples representing the various strata encountered in the boring operations. All ground conditions encountered at the site are also included. The log enables a rapid accurate assessment of the soil profile on a vertical scale. The details of the various strata encountered including all their geological formation details which can be inferred are given. The details captured should include the depth to which ground water was encountered. The description is based on particle distribution and plasticity based visual inspection and feel. Soil color should also be recorded.
# Borehole Log

**Client:** NorKen Consulting Engineers (K) Ltd  
**Project:** Ndonyo Njeru - Kahunuko - Aberdares - Ihithe Road  

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Colour</th>
<th>Description</th>
<th>RQD (%)</th>
<th>Plastic Limits</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Reddish brown</td>
<td>CLAY</td>
<td></td>
<td></td>
<td>9</td>
</tr>
<tr>
<td>2.5</td>
<td>Brown</td>
<td>Multiple Fractured moderately weathered Basalt</td>
<td>80</td>
<td></td>
<td>25 m</td>
</tr>
<tr>
<td>4.5</td>
<td>Grey</td>
<td>Multiple Fractured slightly weathered Basalt</td>
<td>90</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.5</td>
<td>Grey</td>
<td>Multiple Fractured Basalt</td>
<td>100</td>
<td>43</td>
<td>II</td>
</tr>
<tr>
<td>7.5</td>
<td>Grey</td>
<td>Multiple Fractured Slightly Weathered Basalt</td>
<td>80</td>
<td></td>
<td>IV</td>
</tr>
<tr>
<td>8.5</td>
<td>Grey</td>
<td>End of BH 1 at 8.5 m</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Legend:**  
- SPT: where full bore penetration has not been achieved, the number of blows for the quoted penetration is given (soil samples taken at 10 blow intervals)  
- Depth: Meters (m) unless otherwise stated  
- Water: Water levels observed noted as given in meters  
- FI: Fracture Index  
- B: Bulk Sample  
- D: Disturbed Sample  
- RQD: Rock Quality Designation

---

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Scope of Site Investigation

**Spacing of the trial pits and or boreholes**
The scope of site investigation is dependent on the effect of the construction on the ground. The scope should be commensurate with the needed geotechnical parameters. Table *** shows the suggested minimum number of borings for the various structures.

<table>
<thead>
<tr>
<th>Project</th>
<th>Type of soil/Distance between borings</th>
<th>Minimum no</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Uniform</td>
<td>Average</td>
</tr>
<tr>
<td>Multistory</td>
<td>45</td>
<td>30</td>
</tr>
<tr>
<td>1 to 2 storeys</td>
<td>60</td>
<td>30</td>
</tr>
<tr>
<td>Bridge piers and abutments</td>
<td>30</td>
<td>30</td>
</tr>
</tbody>
</table>

For Highways and runways during preliminary design the subgrade soils along the proposed alignment should be sampled at 1000 metres and the samples should be tested to establish the in-situ CBR, grading and plasticity of the materials. At this stage the material site should be investigated at 60 meter intervals. In the detailed stage the subgrade is sampled at 500 metres while the material sites are sampled at 30 metres.

**Depth of investigation**
The depth should be such as to capture the geotechnical information needed for the design of the facility. Equally important is to capture the information needed in the quantification of the bill of quantities to ensure an accurate specification of the works is carried out. The recommended depths below the formation of investigation for the various civil engineering schemes is shown on table ** based on Figure *** below.
<table>
<thead>
<tr>
<th>Project</th>
<th>Depth</th>
<th>In rock</th>
<th>Parameters to be established</th>
</tr>
</thead>
<tbody>
<tr>
<td>Column foundations</td>
<td>1.5B-3B</td>
<td>1.5-3m</td>
<td>Bearing capacity</td>
</tr>
<tr>
<td>Raft foundations</td>
<td>1.5B</td>
<td>1.5-3m</td>
<td>Bearing capacity</td>
</tr>
<tr>
<td>Bridge piers and abutments</td>
<td>1.5B-3B</td>
<td>1.5-3m</td>
<td>Bearing capacity</td>
</tr>
<tr>
<td>Earthworks in fill for highways</td>
<td>0.5L</td>
<td>0.50m</td>
<td>Plasticity and fill material</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Strength of support</td>
</tr>
<tr>
<td>Earthworks in cut highways</td>
<td>0.5H</td>
<td>0.50m</td>
<td>Establish the type of excavated material</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>and strength of support</td>
</tr>
<tr>
<td>Pipe works</td>
<td>D</td>
<td>0</td>
<td>Investigate type of excavated material and</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>strength of support</td>
</tr>
</tbody>
</table>

![Diagram](image_url)

**a) Structural foundations**

**b) Highway earthworks**

**c) Pipe works**

Figure 3.6 Scope of foundation investigations
Site Investigation Reports

List of suitable headings

Title page
Gives the title of the project at a glance

Abstract
The abstract should be approximately 200 words. It is a very important element of the project and should be prepared with care. It must convey the essence of the site investigation and all the important findings without ambiguity.

List of contents
Guides the reader to the various chapters

Field work
A brief and complete description of what was done in the field. Boreholes, and trial pits performed, field testing etc. Actual procedures of standard tests need not be repeated. A mention of the tests performed is sufficient. New procedures and peculiar fieldwork should be explained.

Laboratory work
A brief and complete description of what was done in the laboratory work carried out. as in the case of field testing actual procedures of standard tests need not be repeated. A mention of the tests performed is sufficient. New procedures and peculiar laboratory equipment and procedures should however be explained

Site description and geology
An engineering summary of the nature of the site an its geology, including aspects such excavated areas and what was found, stability of natural slopes, drainage etc

Engineering properties of soils an rocks
A summary of the results of field and laboratory tests and other observations made at the site

Discussion
A reasoned discussion of what design and construction problems are likely to be encountered in relation to the site and its geological situations.

**Recommendations and conclusions**
A brief but clear statement of the recommended geotechnical parameters investigated. The treatment of the various aspects of design should come out clearly and without doubt. Values of use in design and construction should be summarized viz, allowable bearing capacity, estimated settlement, suitable types of foundations, construction requirements namely grouting, compaction etc.

**References**
A list of the books, papers, referred to in the work

**Appendices**
Appendix A should contain site plan, borehole logs, photographs, etc
Appendix B should contain tables of results of field and laboratory test those not included in Appendix A
Appendix C Any special or unusual test procedures adopted in the investigation
Chapter four

Earthworks in dams and road embankments

4.1 Introduction

Soil compaction is the process of increasing the density of soil by packing the particles together with reduction in volume of air. The process does not involve removal of water. The process primarily results in the increase of soil unit weight (density). The reduction of the air content results in the reduction of pores which act as conduits of water and consequently reduce the permeability of the soil. In addition compaction reduces the liquefaction and increases the erosion resistance of the soil. The result is increased shear strength and less compressibility of the soil.

The purpose of the compaction is to produce a soil having the physical properties appropriate to the particular project. A good measure of compaction is needed in the construction of road embankments, improvement of road subgrade, subbase and base layers. Compaction of materials in dams is needed to ensure stability and water tightness of the dam walls.

Variation in compaction effort

Increasing the compaction effort results in the increase in the maximum dry density and a decrease in the optimum moisture content as shown in Figure 3.2. Thus if light compaction is used more water would be needed to overcome the resistance of the soil grains to packing. With more compaction effort the soil grains need less water to occupy the available pore spaces.
**Variation in soil grading soil type**

The particle size distribution influences the arrangement of particles in the compaction of soils. A well-graded soil will compact to lesser voids than a poorly graded soil. The effect is that the well-graded soils will have an increased density and improved properties such as bearing capacity on compaction. These enhanced strength characteristics of compacted fills are important because of the increasing need of embankments capable of supporting higher loads (Mwea 2001).

The behaviour of soils containing clay particles is different from that of granular soils. Their behaviour is related to the arrangement of the clay minerals, the inter-particle forces and the nature of soil water (Lambe, 1958). Thus when two particles are pressed together the contact is initially between a few grains. These grains deform elastically to develop van der Waal forces. The van de Waal forces arise from electrical forces developed in atoms bonding the particles together (Singh and Prakash, 1985). They hold the particles together and when they are brought closer more forces have to be overcome during shearing. However, the overall effect of the cohesive forces and the plasticity is reduction in the maximum dry density achieved upon compaction.

Thus for the same compaction effort MDD and OMC increases and decreases respectively as the soil grain size increases and the plasticity decreases (Figure 3.3).

![Figure 3.1 Effect of compaction effort](image-url)
Figure 4.2 Effect of soil type on compaction

4.2 Road embankment materials

Embankment construction is generally made of locally available materials. This is generally economical and rarely would design engineers need to import embankment materials from long distances. The embankment geometry revolves around the choice of an appropriate slope. Only in very high embankments does the strength and settlement characteristics play a key role in the slope stability computations. In effect therefore, the height of embankments do not affect the choice of materials (Roads Department, 1987). The standard slopes have been determined by experience and monitoring of in-service slopes. The recommended slopes (vertical : horizontal) are as follows: -
Cohesionless materials
1:3 if $h \leq 1$ meter
1:2 if $h > 1$ meter

(ii) Other materials
1:3 if $h \leq 1$ meter
1:2 if $1 < h < 3$ meters
1:1.5 if $3 < h < 10$ meters

Where: - $h$ is the height of the embankment.

However, sufficient strength in the embankment material is required to prevent the settlement and deformation of the embankment layers which would result in the loss of support of the pavement structure. The engineering properties, which determine the design of embankment for support of pavements, include grain size distribution, atterberg limits, moisture content, dry density, shear strength and the California Bearing Ratio (CBR). These properties are described in many textbooks (Craig, 1987; Gichaga and Parker, 1988; Smith and Smith, 1998). Their determination is explained in BS 1377 (1990)

In brief however, the particle size distribution of soils and the plasticity as measured by the atterberg limits lead to the classification of the soil. The particle size distribution influences the arrangement of particles in the compaction of soils. A well-graded soil will compact to lesser voids than a poorly graded soil. The effect is that the well-graded soils will have an increased density and improved properties such as bearing capacity on compaction. Additionally the greater the soil density the less its water absorption tendencies and the better improved resistance to settlement. These enhanced strength characteristics of compacted fills are important because of the increasing need of road embankments capable of supporting pavement structures under heavy traffic loads.

The behaviour of soils containing clay particles is different from that of granular soils. Their behaviour is related to the arrangement of the clay minerals, the inter-particle forces and the nature of soil water (Lambe, 1958). The clay particles are generally less than two microns thin and flat. The surface area to the mass ratio is high and forces at the particle surfaces influences the behaviour of the atoms in the minerals. The net effect is electrical field at the particle
surfaces. These two fundamental properties determine to a great extent the engineering behaviour of a compacted clay.

In order to retain the strength during the usage of pavement structures the Roads Department (1987) has specified construction procedures of embankments, which include:

The lower embankment layers are filled in one hundred fifty millimetre compacted layers. The compaction is required to be at least ninety five per cent standard BS compaction.

The top three hundred-millimetre of the embankment forms the subgrade of the pavement. Its strength determines the pavement thickness and quality. It is considered together with traffic loading for pavement design. For this top layer the compaction is raised to one hundred per cent.

i. Excluding potentially weak soils which have CBR values less than two or plasticity index exceeding fifty per cent or containing more than five percent by weight of organic matter or material with moisture content greater than one hundred and five per cent of the optimum moisture content.

ii. Placing the expansive soils in the lower sections of fill where the weight counteracts the expansion of the soil.

iii. The above construction safety measures compare well with those of other road networks as can be seen from the requirements of California State in USA as outlined below (State of California, Department of Transportation, 1992).

iv. Treatment of expansive soils with lime for reduction of expansion tendencies.

v. Replacement of weak soils with non-expansive materials.

vi. Provision of surcharge over the expansive clays.

vii. Placing the surface courses after the underlying materials have been allowed to expand and stabilise.

viii. Limiting the entry into the pavement layers of water by geo-textile and asphalt membranes

ix. Relocation of project to areas with favourable conditions.

x. Compaction of earthworks to ninety five percent relative compaction
In conclusion the need of good compaction of earthworks and subgrade can not be underestimated. If the subgrade is not well compacted then it is not possible to effectively process and compact the overlying pavement layers. Besides, a poorly compacted subgrade will not offer effective support for the pavement layers. Material for subgrade should be selected from the stronger earthwork materials.

4.3 Soil stabilization for improved bearing capacity;

Cement and lime treatment can be regarded as the process where the stabilizing agent is mixed with soil and subsequently mixed with water to achieve an optimum. The mixture is subsequently compacted before curing process. This process ensures a strength which could otherwise not be achieved by the neat materials is realized. The use of stabilized materials is preferred when the cost of overcoming the deficiency of the in the material is less than importing a neat material which would perform the job satisfactory. The process of stabilizing results in an improved pavement section as can be seen from the properties of the pavement layer after stabilization

I. A substantial proportion of strength is retained when the treated layer is saturated with water.

II. Surface deflections are reduced

III. Resistance to erosion is increased

IV. Materials in the supporting layers cannot contaminate the stabilized layer

V. The effective elastic moduli are increased and so is the bearing capacity.

Lime stabilized material is particularly useful as a capping layer or working platform when insitu material is excessively wet and weak and cannot be removed. However, the following problems are associated with stabilized materials

I. Traffic, thermal and shrinkage stresses cause the stabilized layers to crack.

II. The cracks can reflect at the surface and let in water to lower pavement layers which enviably reduces the pavement carrying capacity.
III. Carbon dioxide reverses the process of stabilization and in the process reverses the gains made in the stabilization

**Lime stabilization**

When lime is added to a cohesive soil, calcium ions replace sodium ions in the clay fraction. The clay becomes saturated with calcium. The pH rises to a value in excess of 12 (highly alkaline). The quantity of lime needed to satisfy these reactions is determined by conducting the initial consumption of lime test (ICL) (British standard 1924, (1990))

The solubility of silica and alumina in the soil increase dramatically when pH rises to 12 and their reaction with lime proceeds producing cementitious calcium silicates and aluminates. The cementitious compounds form a skeleton which holds soil particles and aggregates together.

The primary hydration of cement forms calcium silicate and aluminates hydrates as described above. This releases lime which reacts with soil to produce additional cementitious material.

The type of stabilizer is based on the plasticity and the particle size distribution of the material. The grading should be such that the uniformity coefficient (D60/D10) is not less than 5. We are looking for a material with some spread of the particles. When the coefficient of uniformity is less than 5 the amount of stabilizer is high. Materials of low plasticity are best treated with cement. For soils with high plasticity lime is usually the better option. Cement is particularly difficult to mix with high plasticity soils. A solution which is often made is mixing the soil with a low percentage of lime (say 2%) and the adding cement

**Cement stabilization**

The cement content and curing time determines the strength of the mixture. In the first two to five days the gain in strength is rapid. Thereafter the rate of gain reduces. The choice of the percentage of cement depends on the target strength, the durability of the mixture and the soundness of the aggregate. The minimum amount of cement should exceed the quantity consumed in the initial consumption of cement (ICC). This is usually the same as ICL. To prepare stabilized samples, optimum moisture content for the mixtures are determined with the addition of 2, 4, 6 and 8 percent is determined. The mixtures are
compacted and then proceed to curing and soaking. In the field it is usual to have a delay of about 2 hours before the sample is compacted.

Cement and lime improved materials, namely, gravels, sands, and clayey sands can be used in base and subbase layers. In improvements small amounts of cement or lime are introduced in the material. This results in enhanced strength and reduction in the plasticity. Their enhanced strength enables them to be used for higher traffic levels than the natural materials. The material remains basically flexible. The specification for cement improved materials is shown on shows the major specifications for cement and lime improved materials.

On the other hand the usage of larger amounts of stabilizer results in the stabilization of the material. The result is usually increase in the cohesion of the material and increase in the material rigidity. Stabilization with cement is only justified when the gravel or coarse clayey sands are utilized in the base layers. Shows the major specifications for cement stabilized materials.
Chapter 5

Design and Construction of Foundations

5.1 Bearing capacity of soils and rocks

It is common to use presumed bearing capacity of soils and rocks. The values used have been derived after many years of testing and performance monitoring of existing structures. These values are usually conservative do not consider the overburden above the foundation level. They can be used as preliminary values for the very large structures where an accurate bearing capacity at the foundation level is needed. In the case of smaller structures these valued can be considered as final. Table 1.6 shows the presumed bearing capacity of soils as suggested by BS8004 (1986), while Table 1.7 shows the presumed bearing capacity values used in Kenya. It is to be noted that difficult soils such as expansive soils, loose sands and silts and made up ground should be investigated all the time.

Table 1.1 Presumed allowable bearing values in Kenya

<table>
<thead>
<tr>
<th>Soil and rock</th>
<th>Value (kN/m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Red coffee soil (Red silty clay)</td>
<td>80-120</td>
</tr>
<tr>
<td>Medium dense sand</td>
<td>100-300</td>
</tr>
<tr>
<td>Loose gravel (Murram)</td>
<td>100-150</td>
</tr>
<tr>
<td>Dense gravel</td>
<td>200-400</td>
</tr>
<tr>
<td>Compact gravel and weathered rock</td>
<td>350-600</td>
</tr>
<tr>
<td>Un-weathered rock</td>
<td>&gt;600</td>
</tr>
<tr>
<td>Expansive soils, loose sands and silts</td>
<td>Not Applicable</td>
</tr>
</tbody>
</table>
Table 1.2 Presumed allowable bearing values (BS 8004: 1986)

<table>
<thead>
<tr>
<th>Category</th>
<th>Types of soils and rocks</th>
<th>Value (kN/m²)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rocks</td>
<td>Strong igneous and gneissic rocks in sound</td>
<td>10000</td>
<td>The foundations should be taken to un-weathered rock</td>
</tr>
<tr>
<td></td>
<td>Strong limestone and strong sandstones</td>
<td>4000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Schists and slates</td>
<td>3000</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Strong shales, mudstones and siltstones</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Non cohesive soils</td>
<td>Dense gravel, or dense sand and gravel</td>
<td>&gt;600</td>
<td>The foundation width to be not less than 1m and water level not less than below the foundation level</td>
</tr>
<tr>
<td></td>
<td>Medium dense gravel or medium dense sand and gravel</td>
<td>&lt;200-600</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loose gravel or loose sand and gravel</td>
<td>&lt;200</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Compact sand</td>
<td>&gt;300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Medium dense sand</td>
<td>100-300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Loose sand</td>
<td>&lt;100</td>
<td></td>
</tr>
<tr>
<td>Cohesive soils</td>
<td>Very stiff boulder clay and hard clays</td>
<td>300-600</td>
<td>Soils susceptible to long-term consolidation and settlement</td>
</tr>
<tr>
<td></td>
<td>Stiff clays</td>
<td>150-300</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Firm clays</td>
<td>75-150</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Soft clays and silts</td>
<td>&lt;75</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Very soft clays and expansive clays and silts</td>
<td>Not applicable</td>
<td></td>
</tr>
</tbody>
</table>

5.2 Proportioning of shallow foundations

Proportioning the foundations

The proportioning of the foundations is usually the final step in the design of a structure. This principally dependent on the allowable bearing capacity of the soil. The type of foundation, sizes and the level of the foundation depend on the result of the site investigation. The general procedure for the design of the foundations follows the following steps

a) Evaluate the allowable bearing pressure in a site investigation exercise
b) Examine the existing and future levels around the structure and take into account the ground bearing strata and the ground water level to determine the final depth of the foundation
c) Calculate the loads and the moments if any on the individual footings.
d) Calculate the plan area of the foundation.

The plan area of the foundations is determined assuming that all the forces are transmitted to the soils without exceeding the allowable bearing pressure. The distribution of the pressure is assumed to be planar. In no case should the extreme pressure be less than zero. All parts of the foundation in contact with the soil should be included in the assessment of the contact pressure. Subsequently the designer carries out the structural design of the foundations. Typical foundations are now discussed.

*Strip and rectangular footings*

A strip footing is significantly greater in length than in width. This type of foundation is used to support walls and closely spaced columns. When an individual column is supported by a footing then this foundation is referred to as a pad footing. When two or more columns are supported by one footing, this is referred to as a combined footing.

*Axially loaded strip and rectangular foundations*

The contact pressure of these foundations is considered as uniform when loaded axially. The pressure under the foundations should not exceed the allowable bearing pressure of the supporting soil. Figure 1.17 shows the pressure distribution of such foundations.

![Diagram of foundations]

- a) Pad foundation
- b) Strip foundation
- c) Combined foundation
- d) Pressure distribution
**Rectangular combined footings**

It may not be possible to place columns at the centre of spread footings if they are near the property line, near mechanical equipment or irregularly spaced columns. Columns located off center will result in a non uniform soil pressure. In order to avoid the non uniform soil pressure, an alternative is to enlarge the footing and place one or more of the columns in the same footing to enable the center of gravity of the columns loads to coincide with the center of the footing (Figure 1. . The assumption here is that the footing is rigid. The column loads are taken as point loads and distributed into the footing. The footings are statically determinate for any number of columns. The column loads are known and the resulting pressure is shown in equation 1.37

\[ q = \frac{\Sigma P}{A} \]**

![Diagram](image)

**Figure 5.7 Combined rectangular footing**

**Raft foundations**

A raft foundation is a large concrete slab used as a foundation of a several columns in several lines. It may encompass the entire foundation area or only a portion. Raft foundations are generally used to support storage tanks, several pieces of industrial equipment or high rise buildings. Figure 1.24 shows some typical raft foundations
A raft foundation is used where the supporting soil has a low bearing capacity. Traditionally the raft is adopted when pad and structural wall foundations cover over half the area enclosed by the columns and the structural walls. However this should be evaluated on a case by case basis since the raft foundations end up with negative moments and top and bottom reinforcement. This arrangement could end up being more expensive than closely spaced pads which require only bottom reinforcement.

(a) Flat slab; (b) Thickened under columns or beam slab (c) Basement walls as part of the raft or cellular construction

Figure 5. 8 Common types of raft foundations

The advantages of the raft foundations over the other foundations include:

a) The effect of combining the column bases is increase in the bearing capacity of the foundation. This is because the bearing capacity increases with the breadth of the base.

b) The raft foundations bridge over the weak spots

c) They reduce settlement and are particularly suitable for structures sensitive to settlement.
5.3  **General consideration in the selection of the foundation depth**

Once the geometry of the foundation of the foundation has been found, it is necessary to determine an appropriate depth of the foundation. The following are general considerations which the designers should take into consideration.

a) Usually the foundation should be placed below the depth with minimum moisture variation over the years. This eliminates the shrinkage and collapse effects of the foundation soil. In this country a depth of between 1.0 and 1.5 metres is usually sufficient.

b) The foundation should be placed below top soil and below depths with roots of trees. The roots are potential water paths which weaken the foundations.

c) The foundations should be sited with due consideration to existing nearby structures. The exaction of the foundation in the vicinity of the existing structures could lead to loss of lateral support of the neighboring structures.

d) Special attention should be taken to foundations supported on expansive soils and those on loose sandy silts which are likely to be saturated during the lifetime of the structure.

e) For water structures viz: - river bridges it is necessary to take extra care to ensure that scouring of the foundation vicinity does not impair the safety of the foundation. It is usual to use gabions in areas where scouring is likely to erode the foundations such as downstream of box culverts and around abutments and pier foundations.

f) It is preferable to place foundations at one level throughout. None the less if it is not practical to have the foundations at one level, the change of level should be at one plane. Sloping foundation levels should be completely avoided even if they are on rock. There is a risk of the foundation sliding.

5.4  **Foundations for common buildings**

This section deals with foundations for ordinary common buildings. These are single and double storied buildings with structural walls as the main form of support. The spans should generally not be bigger than six metres. The buildings are generally on good bearing soils. The bearing soils include red
coffee soils, gravelly soils and firm sandy, gravelly clays. The footing for these common buildings is shown on Figure 1.27. The 600 mm width is a practical width which allows masons to maneuver into the trench.

**Figure 1.9 Typical strip footing for an ordinary building**

The following are the general considerations in the usage of the standard footing.

i. No reinforcement is needed for strips where the load can be distributed through 45°.

ii. The foundations should be excavated and the last 150 mm excavation be finalized when the concreting can be done without further delay. This minimizes the softening of the foundation.

iii. The mass concrete is in mass concrete usually by volume batching to achieve grade 15 concrete. A ratio of 1:3:6 for cement sand and ballast respectively is generally sufficient.

iv. Reinforced concrete foundations are done for areas with concentrated loads. These are usually column supports. Grade 25 concrete is the lowest class of concrete allowed in the new BS 8110, but grade 20 of concrete can be considered.
5.5 Pile foundations

Pile foundations are structural members used to transmit surface loads to lower levels in the soil mass. They are used when soil beneath the level at an appropriate raft or conventional footing is too weak or too compressible to provide adequate support to the structure load. The piles have small cross-section area compared to their lengths. The pile materials generally include timber, steel or concrete. The transfer is by vertical distribution of load along the pile surface and at the pile end point.

Piles may be used in the following circumstances
a) To transfer loads to a suitable bearing layer when weak strata is ignored and the load is transferred to an overlying strong bedrock or compact layer.

b) To transfer load through the shaft friction when compact layer is very deep and would be impractical to reach it.

c) To support structures over water where conventional exaction and construction of the foundation is not possible or very expensive to achieve.

d) To reduce settlement and in particular differential settlement.

e) Based on cost. It might prove economical to drive piles down the strata and then build on top of the piles instead of having to excavate deep layers and then construct ordinary foundations.

f) In structures which have considerable uplift, horizontal and/or inclined forces. This is especially true for marine and harbor works.

g) To increase the bearing capacity by vibration and compaction of granular layers of soil.

h) In soils where deep excavations would result in damage of existing buildings.

Piles can be distinguished by the function they are intended to perform or by the material and construction procedures used in their construction. The various types of piles by function are shown on Figure 2.1. The main function of the piles is to take the loads by end bearing or by friction or by combination of the two. Other functions exist and two which can be sited here include tension piles and fender piles. The tension piles take lateral forces in place of traditional retaining walls while fender piles also referred to as dolphin piles are marine structures principally for taking horizontal loads from vessels in the docking
areas. Section 2.2 is presentation of piles by their material and construction procedures.

![Diagram of pile types](image)

**Figure 2. 2 Types of piles by function**

**Classification of Piles by materials and construction**

Piles are constructed in a variety of properties of materials, construction methods and functions. This makes as simple classification difficult. Notwithstanding theses difficulties they are classified in accordance with the pile materials and method of construction (Figure 2.2). This classification also identifies the pile materials. The principal timber materials are timber, concrete and steel.
Types of piles

Driven piles

Large displacement

Preformed. Solid or hollow tubes closed at the end and left in position

Cast in place formed by driving closed tubular sections and then filling the void as the tube is withdrawn

Steel sections

H Piles

Open ended tubes unless a plug forms during driving

Small displacement

Replacement

A void is formed by excavation. the void is filled with concrete

Sides may be

Supported or unsupported

Solid

Hollow

Pre-cast concrete or Timber. Formed to required lengths as units with mechanical support

Steel or concrete tubes closed at the bottom. Filled or unfilled after driving

The supporting may be effected permanently by casing or

Temporarily by casing or drilling mud (Betonite) or

By soil on a continuous auger

Figure 5. 3 Principal Types of piles
Driven piles

To install prefabricated and some form of cast in place piles it is necessary to displace soil by driving the piles. The piling is commonly done by means of a hammer. The hammer operates between guides or leads by use of lifting cranes. The leads are carried by the cranes such that they can drive vertical or raking piles. The piling assembly may be mounted on base suitable for operation on land or on a floating pontoon in the case of piling in the sea.

The hammers may be free falling operated by a clutch release mechanism. Alternatively they are powered by diesel or steam. There are several forms of mechanical devices and equipment in the market used by piling contractors. In order to reduce the impact stresses on the hammer and the piles it is normal to strike the pile through a hammer cushion. The elements of cushion vary but are mainly wood packing in a steel cap or dolly. The various elements in the cushion not only protect the top of the pile but have a significant influence on the stress waves developed in the pile during the driving. The rating of a hammer is based on the gross energy per blow. For a drop hammer the rated energy is the product of the hammer and the height of fall. The efficiency of the hammer is the defined as the energy delivered at impact divided by the gross rated energy. Energy having been lost in the dropping of the hammer to pile. For driving piles to great length the hammers have energies of between of between 50kNm to over 180kNm.

Piles are installed by impact hammers and driven to a resistance measured by number of blows required in the final stages of piling. For wood piles the energy would be limited to about 3 to 4 blows per inch when energy of 15kNm is applied by the hammer. If the pile is to be driven through heaving strata then, it might be necessary to predrill the borehole where the pile is to be driven. This eliminates undesirable heaving. Additionally if the pile is to be driven through dense layers of sand and gravel it is possible to loosen the hard strata by sending a stream of water jet with specially adapted equipment. The various types of driven piles are now described.

Timber Piles

Timber piles are made of trunks of timber. The timber should be preserved to prevent decay. Untreated timber embedded below the ground water table has a long life. If the timber is exposed to alternating wetting and drying it is subject to decay. These types of piles are not very common.
Steel Piles

Steel piles (Figure 2.2a) are usually in form of H-Piles and pipe piles. H piles are preferred where high depth is required while the pipe piles are usually filled with concrete after driving.

In the case of H-Piles the flanges and the web are equal thickness in order to withstand large impact forces. Steel H piles penetrate the ground more readily than other pile types because of the relatively small cross-section area. They are subsequently used to reach stronger bearing stratum at great depth. Steel H piles have also relatively large bearing capacity of between 500 and 2,000 kN per pile depending on the size of the H section. The pile H sections are usually 250x250 to 350x350 with varying section thickness.

Pipe piles are of the range of 250mm to 750 mm diameter. The wall thickness is usually over 2.54mm. In the event that the wall thickness is less than 4.54mm the pile has to driven with a mandrel. When the thickness of wall is over 2.5mm the pipe acts with any concrete in carrying the load. Pipe piles are usually driven with the lower end closed with a plate. In some instances conical driving shoes have been attached. The advantage is not significant.

Steel piles are subjected to corrosion. The corrosion is minimal when the entire pile is embedded in natural soil. However, the corrosion can significantly increase in the event of entrapped oxygen. Zones of water table variation are particularly vulnerable. Severe attacks are encountered on sea structural sections exposed to high and low water tides where the salt sprays can significantly cause corrosion. The standard practice is to use piles which have a factory applied epoxy coating. The most vulnerable sections of the piles should be encased in concrete.

Hard driving and driving through obstructions causes the piles to twist and bend. They can easily go out of plumb without the piling team recognizing since the depth is at depth. Deviations from the vertical of below 10% are usually accepted. A penetration of 2 to 2.5mm per blow should be considered as refusal and further driving would generally cause deterioration.
Pre-cast Concrete Piles

Pre-cast Concrete Piles (Figure 2.2b) are usually cast in a casting yard and transported to the construction site. Where hard driving is expected the tip of the pile is fitted with a driving shoe. They are usually of square or octagonal section. The reinforcement is necessary within the pile to withstand both handling and driving stresses. It is necessary that the exact length to be installed be determined accurately. If the required length is underestimated, the extension can be done only with a lot of difficulties. If the length provided proves to be longer than needed at the site, the piles have to be cut again with a lot of difficulties.

Pre-stressed concrete piles are used and generally have less reinforcement. The pre-stressing reduces the incidence of tension cracking during handling and driving. The difficulties related to the pre-cast concrete piles also apply to the pre-stressed concrete piles.

Pre-cast concrete piles have relatively large bearing capacity of between 800 and 2,000 kN per pile. The presence of high concentrations of magnesium or sodium sulphate in the piled environments causes the piles to deteriorate.
The deterioration is in the form of rust in the reinforcement, cracking and spalling. The best practice is dense concrete of high quality or the use of pre-stressed piles which are not so much susceptible because tension cracks are minimized.

**Driven cast in place piles**

Driven cast in place fall in two categories namely case or uncased type. In the cased type also known as shell the shell type a corrugated steel or pipe which is driven into the ground. The driving is terminated when the desired length of the pile has been achieved. The concrete is poured in the shell and left place. In the shell is then left in place. Figure 2.4 shows the schematic installation of a shell type pile.

![Diagram of shell type pile](image)

**Figure 2.4 Shell type of pile**

In the uncased type a steel tube is driven into the ground and tube is withdrawn upon concreting. Figure 2.5 shows the schematic installation of a typical driven cast in situ pile where the casing is withdrawn. The pile illustrated is also known as a Franki pile.
Difficulties encountered in the installation of driven piles

The installation of driven piles has difficulties due to various factors incidental to the installation procedures and to the ground encountered at the sites. These difficulties are varied but the main ones include:

a) Handling of the preformed sections which could lead to damage of the piles before installation.

b) Noise arising from the hammer dropping on to the pile. This can be particularly undesirable in sites in the busy neighborhoods.

c) Spoiling of the pile in the driving operations include the spoiling of pile heads and or pile toes. This usually takes place due to overdriving piles when refusal has been reached. It is usually sufficient to achieve a penetration of 2-2.5 mm per blow in the last stages of piling.

d) Piles of small cross-section especially H piles driven in boulderly strata could easily alignment. Vertical piles could end up having bent up shapes and hence lose their carrying capacity.

Figure 2.5 Installation of a Franki pile
Bored piles

Bored piles are also known as cast in place concrete piles (Figures 2.2c-e). The borehole is effected by various methods using piling equipment. The bore is supported by casing or by drilling mud (bentonite suspension). At the required depth boring is stopped and the hole is filled with concrete. If required a cage of reinforcement is placed before concreting is done. With the use of bored piles larger diameter piles have been installed with corresponding high bearing capacities. They are constructed in diameters ranging from 300mm to as high as 2400mm. They have been performed to depths of 70 metres and below and can be constructed vertically or in rakes of up to 1:4. They are thus ideal for many site conditions. The construction sequence of bored piles depends on the method of construction adopted. The main construction methods include bored piles with casing support and bored piles with bentonite support.

Bored piles with casing support

In this type of pile the casing is advanced by a crane and a casing oscillator. The material below the casing area is excavated and brought up for examination and testing where necessary. After the depth needed has been achieved the reinforcement cage is inserted followed by concreting as shown on

Bored piles with bentonite support

In this type of pile a lead casing is advanced into the soil. The material below the casing area is excavated and brought up by use of drilling equipment with a bucket which can bail out the drilled soil. The excavated soil is examined and tested where possible. The drilled hole is supported by drilling mud. After the depth needed has been achieved the reinforcement cage is inserted followed by concreting as shown on Figure 2.6
This installation is particularly desirable in gravelly and boulderly conditions

a) **With casing**

<table>
<thead>
<tr>
<th>Install starter casing</th>
<th>Advance into the soil by drilling and supporting with bentonite</th>
<th>Insert reinforcement cage and recycle bentonite</th>
<th>Place concrete with a tremie pipe as casing is withdrawn</th>
<th>Complete pile</th>
</tr>
</thead>
</table>

This installation is suitable in all soils

b) **With bentonite support**

**Figure 2.6 Installation of a bored pile with drilling mud**
Difficulties encountered in the installation of bored piles

The difficulties associated with the installation of bored piles are also varied but the main ones include:

i. Poor base preparation after the bearing strata has been reached. Loose particles will have reached the bottom of the bore and will be difficult to detect or remove. The base the pile will consequently have a lower bearing capacity than would have been expected

ii. Poor concreting control where the pile is being cast under artesian conditions. This usually results from poor shaft control as the concreting continues. The result is necking of the concrete and/or washout of various sections of the pile. Under ideal conditions the concreter under tremie conditions should always be placed inside the wet concrete.

iii. Vibration and movement of the ground in the vicinity of the pile under construction.

It is to be noted that these difficulties are also present in the driven cast in place piles where the casing is withdrawn as concreting proceeds

5.6 Foundations on expansive clays

Introduction

The problems associated with expansive soils arise as a result of alternate heaving and shrinkage of the clays. These soils are typically black or grey and are referred to as black cotton soils in this country. The cycle of expansion and shrinkage is a result of ability of the clays to take in water and retain it in its clay structure. The water absorption leads to expansion of the clay and causes strains in the foundation and the structures supported thereupon. The strains eventually cause the cracks to appear on the walls. The result is structural safety and aesthetics of the buildings are compromised.

The clay minerals include montmorillonite, illite and kaolinite as discussed in FCE 311. The montmorillonite clay mineral is particularly prone to heaving and shrinkage. Soil having more than 20% of montmorillonite are particularly prone to swelling problems.
In addition to visual identification the expansive soils can be identified by assessing the swell potential of the soils. This is done by conducting an odometer test which measures the free swell and the swell pressure attained in an odometer when a sample held in an odometer ring is kept at the same volume as swelling is induced by allowing the sample to take in water. Some of the Nairobi black cotton soils have been found to have a swell pressure of up to 350 kN/m$^2$. Chen ( ) has related swell potential to plasticity index as shown on Table

The following methods can be applied to mitigate damage control

a) Moisture control
b) Soil stabilization
c) Structural measures

**Moisture control**
The main course of heave and shrinkage is the fluctuations of moisture under and around the structures in question. Depending on the topographical, geological and weather conditions the natural ground water fluctuates during the year. This seasonal fluctuation decreases with depth. In some areas the depth to the fluctuation zone is as low as 1.5 meters. In other areas it will be deeper going down to over three meters. In addition to the ground water fluctuation the surface water from rains or burst pipes seeps into the foundations and course moisture migration.

A satisfactory solution to the problem would to devise an economical way of stabilizing the soil moisture under and around the structure. It does not matter whether the moisture is maintained high or low in so far as it can be maintained throughout the year. An effective procedure of achieving this is to provide a water tight apron of approximately one metre round the building. A subsurface drain one metre round the building is provided with augur holes provided at every 2 meters. The holes are filled with sand and interconnected at the top. In effect the augur drain is and the impervious apron ensures that the moisture at the foundation area remains the same. Figure 1. 10 shows such an arrangement of the drains for ensures that the moisture content of the foundations remain the same.

The subsurface drain is used to intercept the gravity flow, or; perched water of free water to lower ground. It also arrests capillary moisture water movement. The subsurface drain should be lend to a positive outlet. In general
the ground surface around the building should be graded so that surface water will flow away from the building foundations all the time.

**a) Location of sand drain around a building**

**b) Sand drain and apron detail**

*Figure 1.10 Typical sand drain treatment of a building*

**Soil stabilization**

Soil stabilization consists of one of the following operations

(a) Pre-wetting or flooding the in-situ soil to achieve swelling prior to construction.

(b) Compaction control
Soil replacement

Chemical stabilization

Pre-wetting or flooding the in-situ soil to achieve swelling prior to construction involves the flooding of the site under consideration prior to construction. The soil would heave and the potential danger of cracking is eliminated. Pre-wetting has been used with success when the active zones are not large. It is very difficult to saturate high plasticity clays. There is danger that expansion of the clays could continue after the construction has taken place. This procedure should be considered for stabilizing pavement or canal linings. In only rare cases should the method be considered for use below ground floor slabs. Its application below building foundations is risky and questionable.

Compaction control has been used in pavement construction. Expansive clays expand very little when compacted at low densities and high moisture contents. But will expand considerably when compacted to high densities at low moisture contents. The approach is to compact swelling clays at moisture contents slightly above their natural moisture content for good result. In this method it is not necessary to introduce large amounts of water into the soil. Dry compaction of expansive soils was done along the Lodwar-Kakuma road.

Soil replacement is the simplest and easiest solution for slabs and footings founded on expansive soils. The expansive foundation soils are replaced with non-heaving materials. The method requires the selection of the replacement material and the depth to replacement. In Nairobi the depth of the expansive black cotton soils is in the region of 1.0 to 1.5 metres. In this case it has been found desirable to remove the entire expansive soil below buildings and replace with suitable granular material. When the expansive soil is deeper building slabs can be constructed above the compacted soil covering the expansive soil but the foundation of main structure needs further consideration.

This method is particularly useful for the construction of highway pavement in a site completely overlaid with expansive soils where the alternative to reroute the road is not viable. In this case it the lower expansive soils are overlaid with the compacted replaced material to a depth of 1.5 metres.

Chemical stabilization is the process of mixing additives like cement and lime to expansive soil to alter its chemical structure and in the process retard its
potential expansiveness. Lime reduces the plasticity of the soil and hence its swelling potential. The amounts used range from two to eight percent by weight. Cement on the other hand reduces the liquid limit, plasticity and potential volume change. Stabilization has been used mainly in highway and airport construction.

*Structural measures* include several methods have been reported in literature such methods include

(a) Floating foundation  
(b) Reinforcement of brick walls  
(c) Foundation on piles

*Floating foundation* concept is a providing a stiffened foundation. This is essentially a slab on ground foundation with the main supporting beams resting on non-cohesive non heaving material. The slabs are designed fixed on the beams that assuming a heave pressure of 20 kN/m². This magnitude is small considering that the swell pressure of the expansive soils commonly found in Kenya has been estimated at between 300 and 500 kN/m². Results of such an approach have been mixed where they have been tried. This method needs further research.

*Reinforcement of brick walls* have been tried in South Africa. In this method reinforcement is placed in brick walls. The reinforcement is placed where cracking usually takes place. This is typically above and below openings. The structure is made also semi flexible by providing joints in the brickwork so that when heave takes place the building will conform to the new ground shape and consequently reduce the bending moment induced in the walls. The joints are typically 1.5cm.

*Foundation on piles* is a very successful procedure which ignores the heave by placing the footing to a sufficient depth (Figure ). The depth of the pile should leave an expansion zone between the ground and the building to allow the soil to swell without causing detrimental effect to the building. One way of installing the piles is to provide a pile with bell at the bottom. The bell or under reamed section should be well below the active zone. The bell is installed with special equipment and anchors the pile into the ground. The pile can be
installed in an oversize shaft which is subsequently filled with straw saw dust as filler to eliminate uplifting of the pile by heaving soil. Alternatively the pile could be a straight and the effect of the uplift calculated using Equation 1.47. The friction below the active zone is utilized in the calculation of the bearing capacity of the pile.

![Diagram of pile systems for expansive soils]

**Figure 5.11 Pile systems for expansive soils**

**Foundations on loose sands**

Foundations on loose sands are particularly difficult due to the likelihood of collapse in the event of large storms. The storms result in the realignment of the sand particles and consequent settlement due to repacking of the sand support. This has resulted in large cracks in buildings which have been placed on this type of foundation soils. The foundation soils subsequently lose their bearing capacity and the result is settlement of the foundations. The superstructure has to absorb the settlement usually with resultant cracks of walls and structural elements.

A real case story is one of the Garissa teachers college whose buildings were placed on sand strata. The area is generally dry but when the rain comes, it usually very heavy and comes in large storms. The performance of the three building types of structures adopted at Garissa teachers college forms a case
study whose findings are used to suggest a construction procedure for foundations and masonry superstructures on loose sands.

The main teaching bungalow consisted of buildings constructed with a ground beam which was framed with columns and a concrete roof slab. The masonry was thus reinforced at the corners with columns and subsequently bound at the top by a ring beam and at the bottom with a ground beam. These types of buildings were found to have performed well several years after construction. This type of construction produced a satisfactory type of constructed and when the buildings were inspected ten years after construction the structural frames and the infill masonry walls were performing well.

The second type of buildings consisted of three and four and three storied flats. As in the case of the previous buildings these types of buildings were found to have performed well ten years after construction.

The third type of the buildings was the staff residential bungalows. These were constructed with a ground beam and masonry walls. The roof of the buildings was a concrete slab. However as the rains came and went in there stormy characteristics the residential houses developed cracks in the walls. The cracks were particularly severe in the external walls and after about 10 years of service and needed attention (Plate 1.1).

Based on the satisfactory behavior of the framed structures it was found prudent to introduce columns at the masonry wall corners in a repair scheme. Plate É It is therefore recommended for foundations on loose sands the masonry should be reinforced with columns at the corners. In addition the foundations should be kept as far as is possible free from percolating water. In this way the in the event of settlement the frame will be able to absorb the stressed attributable to additional settlement and reduce the severity of the cracks.
Plate 5.1 Cracks in the walls occasioned by settlement of the foundation

Plate 5.2 Introduction of columns to stiffen the walls
Chapter 6

Bridges, culverts retaining walls and tunnels

6.0 Bridge design and construction

A bridge is required in highway to enable traffic to cross rivers, seas, rail lines or other roads. There are also pedestrian bridges to enable pedestrians cross over the same facilities. The bridge design involves the correct choice of height, length and width of the bridge.

The height and width of the bridge should provide the right services required below the bridge. Consideration of height should be height of loaded trucks, while the width should consider the pedestrian traffic. The water and sea bridges should consider the peak floods and navigation requirements.

6.1 Bridge Superstructures

Reinforced concrete bridges

A Slab bridge is supported by beams over abutments and piers. The span of these kind of bridges is of the order of 6-10 metres.
A beam bridge is basically a rigid horizontal structure that is resting on two piers, one at each end. The weight of the bridge and any traffic on it is directly supported by the piers. The weight is traveling directly downward.

Many beam bridges that you find on highway overpasses use concrete. The size of the beam, and in particular the height of the beam, controls the distance that the beam can span. By increasing the height of the beam, the beam takes more load and the span of the bridge can be increased.

The span of these kinds of bridges is of the order of 10-25 meters when simply supported and 10-30 metres when continuous over the spans.
A beam bridge

Most of the concrete bridges are cast in situ. However it is possible to precast the beams and launch them with a crane and cast the deck in-situ as can be seen in the section seen below. The precast option is particularly attractive when it is not possible to keep the false work in place due to water passing below the deck or in the case of bridge over roads.
Pre-stressed concrete superstructures

Girder
Two or more longitudinal beams together with a deck incorporated so that they act compositely. They can be simply supported or can be continuous over the supports. The beams are usually precast on a yard and then launched by cranes. Many methods of launching have been devised by contractors. A truss launching arrangement is shown below. These type of bridges have been designed and constructed to span between 40 and 60metes

Box girder
The box girder bridges are cast insitu. They are cast over falsework and can span over two or more spans. In some cases they are continuous with piers and abutments to form rigid frames.
Segmental

These are constructed by the cantilevering technique. The sections of the bridge could be constant or are variable. The bridge sections may be formed by precast segments which are launched in place and then are pre-stressed. To achieve longer spans the segmental bridges are cast in-situ starting from the pier or from the piers or from the abutments. Construction is achieved by balancing over the piers and use of temporary supports. Our own Nyali Bridge has a span of 180 metres. The largest continuous span for segmental bridges are in the region of 200-250 meters.
Fig. 4.2 Typical cross section of a segmental bridge.

Fig. 4.3 Typical cross section of a segmental bridge.
Fig. 4.5 Superstructure-substructure temporarily fixed for unbalanced moment.

Fig. 4.6 Temporary struts for unbalanced moments.
Steel bridges

Steel girder are steel beams launched in place by crane. Composite action between the steel and the deck is achieved by incorporation of shear connectors in the form of angle or round small struts. An alternative to this is the use of steel trusses to span over larger spans. These kind of bridges span over 100 meters.
The suspension bridge is one where cables (or ropes or chains) are strung across the river (or whatever the obstacle happens to be) and the deck is suspended from these cables. Modern suspension bridges have two tall towers through which the cables are strung. Thus, the towers are supporting the majority of the roadway's weight.

The force of compression pushes down on the suspension bridge's deck, but because it is a suspended roadway, the cables transfer the compression to the towers, which dissipate the compression directly into the earth where they are firmly entrenched.

The supporting cables, running between the two anchorages, are the lucky recipients of the tension forces. The cables are literally stretched from the weight of the bridge and its traffic as they run from anchorage to anchorage. The anchorages are also under tension, but since they, like the towers, are held firmly to the earth, the tension they experience is dissipated.
Almost all suspension bridges have, in addition to the cables, a supporting truss system beneath the bridge deck (a deck truss). This helps to stiffen the deck and reduce the tendency of the roadway to sway and ripple.

Very large spans of up to 2000m have been achieved by this form of construction. The bridges must be tested to the most severe load in the case of earthquakes and wind.

The world's longest suspension bridges are listed according to the length of their main span (i.e., the length of suspended roadway between the bridge's towers). The most common method of comparing the sizes of suspension bridges, the length of main span often correlates with the height of the towers and the engineering complexity involved in designing and constructing the bridge.

Suspension bridges have the longest spans of any type of bridge. Cable-stayed bridges, the next longest design, are practical for spans up to around one kilometer. Thus, the 15 longest bridges on this list all are suspended-deck suspension bridges and currently are the 15 longest of all types of vehicular bridge.
<table>
<thead>
<tr>
<th>Rank</th>
<th>Name</th>
<th>Location</th>
<th>Main span metres</th>
<th>Year opened</th>
</tr>
</thead>
<tbody>
<tr>
<td>[1]</td>
<td>Akashi Kaikyō Bridge (The longest from 1998 to the present)</td>
<td>Kobe-Awaji Route, Japan</td>
<td>1,991</td>
<td>1998</td>
</tr>
<tr>
<td>[2]</td>
<td>Xihoumen Bridge</td>
<td>Zhoushan Archipelago, China</td>
<td>1,650</td>
<td>2009</td>
</tr>
<tr>
<td>[3]</td>
<td>Great Belt Bridge (also known as the Storebælt Bridge; Danish: Storebæltsbroen)</td>
<td>Halskov-Sprogø, Denmark</td>
<td>1,624</td>
<td>1998</td>
</tr>
<tr>
<td>[4]</td>
<td>Runyang Bridge</td>
<td>Yangtze River, China</td>
<td>1,490</td>
<td>2005</td>
</tr>
<tr>
<td>[6]</td>
<td>Jiangyin Suspension Bridge</td>
<td>Yangtze River, China</td>
<td>1,385</td>
<td>1999</td>
</tr>
<tr>
<td>[7]</td>
<td>Tsing Ma Bridge (the longest carrying road and rail traffic)</td>
<td>Tsing Yi-Ma Wan, Hong Kong</td>
<td>1,377</td>
<td>1997</td>
</tr>
<tr>
<td>[8]</td>
<td>Verrazano-Narrows Bridge</td>
<td>New York City (Brooklyn - Staten Island), USA</td>
<td>1,298</td>
<td>1964</td>
</tr>
</tbody>
</table>
An arch bridge is a semicircular structure with abutments on each end. The design of the arch, the semicircle, naturally diverts the weight from the bridge deck to the abutments.

Arch bridges are always under compression. The force of compression is pushed outward along the curve of the arch toward the abutments.

The tension in an arch is negligible. The natural curve of the arch and its ability to dissipate the force outward greatly reduces the effects of tension on the underside of the arch. The greater the degree of curvature (the larger the semicircle of the arch), however, the greater the effects of tension on the underside.

As we just mentioned, the shape of the arch itself is all that is needed to effectively dissipate the weight from the center of the deck to the abutments. As with the beam bridge, the limits of size will eventually overtake the natural strength of the arch.

### 6.2 Bridge Substructures

Bridge foundations generally fall into two categories:

**Strip footings**, one for each pier and abutment. However, it is sometimes convenient to split the deck into two halves longitudinally along the centre line, this is then continued to the footing.
Piled foundations.
It is possible to have a combination of both (i.e. piers being piled with abutments on strip footings).

Design Considerations
From the site investigation report decide upon which stratum to impose the structure load and its safe bearing pressure. Select the type of foundation, possibly comparing the suitability of several types. Design the foundation to transfer and distribute the loads from the structure to the ground. Ensure that the factor of safety against shear failure in the soil is not reached and settlement is within the allowable limits.

Strip Footings
The overall size of strip footings is determined by considering the effects of vertical and rotational loads. The combination of these two must neither exceed the safe bearing capacity of the stratum or produce uplift. The thickness of the footings is generally about 0.6 to 1.0 m but must be capable of withstanding moments and shears produced by piers or abutments.

Piled Foundations
The type of piles
Driven Piles: preformed piles of concrete or steel driven by blows of a power hammer or jacked into the ground.
Preformed Driven Cast In-Situ Piles: formed by driving a hollow steel tube with a closed end and filling the tube with concrete.
Driven Cast In-Situ Piles: formed by driving a hollow steel tube with a closed end and filling the tube with concrete, simultaneously withdrawing the tube.
Bored and Cast In-Situ Piles: formed by boring a hole and filling it with concrete.

The working load of an individual pile is based on providing an adequate factor of safety against the soil under the toe failing in shear and the adhesion between the shaft and the soil surrounding it passing its ultimate value and the whole pile sinking further into the ground. There are basically four methods for assessing this effect:
• Through soil parameters i.e. summing shaft friction and bearing capacity. The ultimate bearing capacity is usually modified to compensate for the driving effect of the pile.

• By means of test piles.

• By means of dynamic formulae i.e. Hiley formulae which equates the energy required to drive the pile with its ultimate bearing capacity.

• Piling contractors 'know how'.

6.2 Retaining walls,

A retaining wall is generally defined as a vertical wall that holds back earth. However a retaining wall has many uses such as:

• A retaining wall for roadside embankments
• A retaining wall can separate older roads from highways
• A house retaining wall for the garage
• A retaining wall helps keep river banks from eroding
• A retaining wall to keep earth from stairwells and driveways

Trial and error construction methods of the past and advancements in engineering knowledge have indicated that four basic types of retaining walls seem to perform quite well. Each design has its limitations however. The five basic types of retaining walls are: gravity wall; cantilever wall; counterfort wall; buttressed wall and the reinforced earth retaining wall

Gravity Walls
A gravity retaining wall is usually depends on its own weight or mass to hold back the earth behind it. This goal is achieved by constructing the wall with a volume of material so that when stacked together, the weight and friction of the interlocking material exceeds the forces of the earth behind it. The wall is thicker at the base than at the top. Also, note that as the front of the wall gets taller it slants backwards. This is often referred to as 'battering'.

This battering effect creates a visual message of strength, as over time, the wall will probably succumb to the forces of gravity and begin to tilt outward. By battering the wall backwards, you extend the visual life of the wall. Retaining walls that appear to be tipping over, tend to indicate faulty construction and impending failure.
Walls that are battered send a visual message that the wall is 'working' and that it is continuing to beat the force of gravity. Gravity walls become very cumbersome to construct as they get higher, because they require vast quantities of materials. The thickness of a gravity wall at its base should be one half to three fourths its height.
Cantilever Walls

A cantilever retaining wall is one that consists of a uniform thickness wall which is tied to a footing. A cantilever wall usually is asked to hold back a significant amount of earth, so it is a good practice to have these walls engineered. A simple example of a cantilever retaining wall is a typical basement wall of a house.

The width of the footing for a cantilever wall is very important. The footing is designed to resist tipping or sliding forces which the earth exerts upon the wall. Also, the wide area of the footer allows the weight of the earth to actually keep the wall from tipping in some instances. These walls require significant steel reinforcing in both the footer and the wall structures. The steel also has to extend from within the footer up into the wall so that the two pieces actually become one integral unit. As you can see, this is why these walls need to be designed by structural engineers. If you try to guess yourself at the amount, size and placement of structural steel in this type of wall, you are gambling. Also, the thickness of both the footer and the wall is extremely critical.

Counterfort Retaining Walls

A counterfort retaining wall is very similar to a cantilever wall, except that it has one additional feature. This wall has a triangular shaped wall which connects the top of the wall to the back of the footer. This added support wall is hidden within the earthen or gravel backfill of the wall. The footer, retaining wall and support wall must be tied to one another with reinforcing steel.
If the structure is poured concrete, often the retaining wall section and the support walls are poured as one unit at the same time. The support walls add a great deal of strength to the retaining wall. The supports make it virtually impossible for the wall to become detached from the footer. As with cantilever walls, a counterfort wall should be designed by a competent structural engineer. If you decide to attempt to construct this type of wall without approved plans, you are making a huge mistake. If the wall fails, the cost to remove the failed wall, construct the new one, etc. could be ten times or one hundred or more times the cost of engineering services. Remember, engineers have to eat just like you and me!

**Buttressed Retaining Walls**

A buttressed retaining wall is basically identical to a counterfort wall except for one thing. The support wall is on the outside of the retaining wall. They are visible. The buttresses add incredible strength to the wall system. For the retaining wall to fail or tip over, the buttresses would have to be crushed. The buttress concept was widely used in the construction of many cathedrals in Europe.

Because of the height of the cathedral walls, the buttresses helped to stabilize them. They do the exact same thing in a retaining wall. Once again, if you intend to build one of these walls, you must give serious consideration to hiring an engineer. Situations which demand this type of wall usually have tremendous loads which bear against the walls. The buttresses can often be designed to be decorative in nature and covered with stone or some other material. Depending upon the overall length of the wall, you may have several buttresses. They can be spaced to create rooms, parking spaces, handball courts or any other functional space. Use your imagination!
Fig. 7.1 Types of reinforced concrete retaining walls.

Fig. 7.3 Crib and gabion walls.

**DETAIL 2.**
STEPPED EMBANKMENT.

**DETAIL 3**
VERTICAL E
Reinforced Earth retaining walls are coherent gravity structures engineered to resist specific loading requirements. The primary components of a Reinforced Earth wall consist of alternating layers of granular backfill, and linear metallic, high-adherence soil reinforcing strips or ladders to which a modular precast concrete facing is attached. Its strength and stability are derived from the frictional interaction between the granular backfill and the reinforcements, resulting in a permanent and predictable bond that creates a unique composite construction material. A mechanical connection between the facing panels and the soil reinforcing strips is achieved by way of a special tie strip embed and high strength nut/bolt/washer assembly.

Reinforced Earth retaining walls are an economical way to meet every-day earth retention needs for highway and bridge grade separations, railroads and mass transit systems, waterfronts, airports, loading docks, industrial facilities and
commercial and residential developments. They are also used in response to difficult design conditions such as very high structures, restricted space, where obstructions within the MSE soil mass are present and poor foundation soils. The inherent strength and flexibility of the overall wall system gives designers a powerful way to economically solve difficult stability issues for structures subject to flooding or other hydrodynamic forces, or those in seismically active areas. Benefits include

- Considerable advantages over cast-in-place, both in construction time and quantity of materials
- Flexibility, making it possible to build directly upon compressible soils
- High load-carrying capabilities, both to static and dynamic loads
- Ease of installation since construction using prefabricated components is rapid and predictable
- Superior appearance since the facing is highly suited for architectural finishes
**Tunnels**

Artificial underground passage for roads, rails, pedestrian etc
Extensive geological investigations to determine the ground conditions.
Eventually needs ventilation, lighting systems and air-conditioning

Major works
Excavation
  Cut and fill procedure weight all necessary care to avoid collapses
  Working inside the tunnel without removal of the overburden
  Create a cavity inside the ground
  Take account of water pressures
  Collapsing ground into the cavity
In hard strata it is usually possible to excavate without support
In highly jointed strata it might be necessary to use some form of support
  Special tubes and temporary supports
Excavator done by drilling, explosives with machinery manufactured for the purpose. Newly exposed rock surfaces must be sealed to prevent further decomposition and further loosening. After excavation rock bolting is usually done to support unstable ring. Rock bolting is followed by application of rapid hardening concrete known as concrete. The shortcrete grouts the rock bolt, decreases the permeability and increase the strength of the supporting ring. Mesh reinforcement is usually incorporated in the concrete. The shortcrete could be applied in two layers. Finally an invert concrete is placed on the floor of the tunnel to complete the ring. This invert concrete may contain mesh reinforcement.

**Types of Tunnels**
There are three broad categories of tunnels: mining, public works and transportation. Let’s look briefly at each type.

**Mine tunnels are** used during ore extraction, enabling laborers or equipment to access mineral and metal deposits deep inside the earth. These tunnels are made using similar techniques as other types of tunnels, but they cost less to build. Mine tunnels are not as safe as tunnels designed for permanent occupation, however.
**Public works tunnels** carry water, sewage or gas lines across great distances. The earliest tunnels were used to transport water to, and sewage away from, heavily populated regions. Roman engineers used an extensive network of tunnels to help carry water from mountain springs to cities and villages. These tunnels were part of aqueduct systems, which also comprised underground chambers and sloping bridge-like structures supported by a series of arches. By A.D. 97, nine aqueducts carried approximately 85 million gallons of water a day from mountain springs to the city of Rome.

**Transportation tunnels** such as canals -- artificial waterways used for travel, shipping or irrigation. Just like railways and roadways today, canals usually ran above ground, but many required tunnels to pass efficiently through an obstacle, such as a mountain. Canal construction inspired some of the world's earliest tunnels.

The Underground Canal, located in Lancashire County and Manchester, England, was constructed from the mid- to late-1700s and includes miles of tunnels to house the underground canals. One of America's first tunnels was the Paw Paw Tunnel, built in West Virginia between 1836 and 1850 as part of the Chesapeake and Ohio Canal. Although the canal no longer runs through the Paw Paw, at 3,118 feet long it is still one of the longest canal tunnels in the United States.

By the 20th century, trains and cars had replaced canals as the primary form of transportation, leading to the construction of bigger, longer tunnels. The Holland Tunnel, completed in 1927, was one of the first roadway tunnels and is still one of the world's greatest engineering projects. Named for the engineer who oversaw construction, the tunnel ushers nearly 100,000 vehicles daily between New York City and New Jersey.
Tunnel construction takes a lot of planning. We'll explore why in the next section.

A transportation tunnel

Tunnel Planning

Almost every tunnel is a solution to a specific challenge or problem. In many cases, that challenge is an obstacle that a roadway or railway must bypass. They might be bodies of water, mountains or other transportation routes. Even cities, with little open space available for new construction, can be an obstacle that engineers must tunnel beneath to avoid.

In the case of the Holland Tunnel, the challenge was an obsolete ferry system that strained to transport more than 20,000 vehicles a day across the Hudson River. For New York City officials, the solution was clear: Build an automobile tunnel under the river and let commuters drive themselves from New Jersey into the city. The tunnel made an immediate impact. On the opening day alone, 51,694 vehicles made the crossing, with an average trip time of just 8 minutes. Sometimes, tunnels offer a safer solution than other structures. The Seikan Tunnel in Japan was built because ferries crossing the Tsugaru Strait
often encountered dangerous waters and weather conditions. After a typhoon sank five ferryboats in 1954, the Japanese government considered a variety of solutions. They decided that any bridge safe enough to withstand the severe conditions would be too difficult to build. Finally, they proposed a railway tunnel running almost 800 feet below the sea surface. Ten years later, construction began, and in 1988, the Seikan Tunnel officially opened.

How a tunnel is built depends heavily on the material through which it must pass. Tunneling through soft ground, for instance, requires very different techniques than tunneling through hard rock or soft rock, such as shale, chalk or sandstone. Tunneling underwater, the most challenging of all environments, demands a unique approach that would be impossible or impractical to implement above ground.

That's why planning is so important to a successful tunnel project. Engineers conduct a thorough geologic analysis to determine the type of material they will be tunneling through and assess the relative risks of different locations. They consider many factors, but some of the most important include:

**Soil and rock types**
- Weak beds and zones, including faults and shear zones
- Groundwater, including flow pattern and pressure
- Special hazards, such as heat, gas and fault lines

Often, a single tunnel will pass through more than one type of material or encounter multiple hazards. Good planning allows engineers to plan for these variations right from the beginning, decreasing the likelihood of an unexpected delay in the middle of the project.

Once engineers have analyzed the material that the tunnel will pass through and have developed an overall excavation plan, construction can begin. The tunnel engineers' term for building a tunnel is driving, and advancing the passageway can be a long, tedious process that requires blasting, boring and digging by hand.

In the next section, we'll look at how workers drive tunnels through soft ground and hard rock.
Photo courtesy Japan Railway Public Corporation

Construction of the Seikan Tunnel involved a 24-year struggle to overcome challenges posed by soft rock under the sea.

Figure – Typical tunnel in unstable rock
6.4 Culverts.
Drainage structures
Carry water across roads and estates
These are structures with a water opening less than 12m².
The size of the opening is dependent on the calculated flood calculated based on the maximum return flood usually of 10 to 25 years

The small culverts will be made up of pipe culverts while the larger culverts are made up of box culverts.

Because of their geometry the culverts can be laid on weak soils since they spread their load on large areas

Twin pipe culvert
Multi-cell box culvert