THE ANALYTICAL MODELS FOR GEOTECHNICAL PROBLEMS AROUND TUNNELS BAGHDAD METRO PROJECT

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IN THE NAME OF GOD

THE MOST COMPASSIONATE THE MOST MERCIFUL

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ABSTRACT

A new analytical techniques were developed based on mathematical relations to analyze the ground behavior modes, effective stress variations, limits of elastic behavior for tunnel wall points, and the ground surface settlement above tunnels in different soil types for free and compressed air excavation methods respectively.

Nine sites along proposed Baghdad Metro Project were selected to apply the study techniques. The results were compared with findings of numerical and monitoring studies in many parts of the world.

The results show that the analytical techniques which are developed in this study give a practical tool for analysis and predict of ground behavior during and after tunnel excavation.

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CHAPTER ONE

INTRODUCTION

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INTRODUCTION

1.1 GENERAL

Tunnels can be defined as , underground passages made without removing the overlying ground materials (ITA, 1986). In tunneling the ground actively participates in providing stability to the opening therefore , the design procedures for tunnels , as compared to above ground structures, depend mainly on geotechnical factors like the site situation, the ground characteristics and the method of excavation .

The selection of excavation method for any tunnel depends on the site situation and engineering properties of the ground materials (Duddeck and Erdman, 1985). In soft ground, soils, of a high water table, tunnels may be driven by two main methods;

i. In more cohesive soil, the ground may be strong enough to allow a certain open section at the tunnel face. In this case the tunnel is excavated sequentially with boring machines and the disturbed soil in the tunnel face will be supported firstly by shotcrete to prevent water inflow before final support installation. This excavation methodis named the free air method.

ii. In soft ground of high water table, immediate support must be provided by stiff lining. The tunnel is excavated with a cylindrical machines, shield, which have a radius slightly greater than the tunnel radius. The compressed air is used in this method to increase the internal pressure on the tunnel face and to prevent water seepage to the work space. This method is named the compressed air method.

The soil behavior around the opening will change widely depending on the excavation methods (Duddeck, 1995. Personal communication).

1.2 OBJECTIVE OF THE STUDY

The objective of this study is to build two dimensional models, based on an analytical approach in order to study the ground behavior around the tunnels which are excavated in soils.

The main objectives of the study are to assess the following problems:

1. *stress redistribution around tunnel opening.*
2. *vertical effective stress variations.*
3. *ground settlement above the tunnel.*
4. *limits of the soil elastic properties (The ground convergence curve GCC).*

These problems are studied for each excavation method (free and compressed methods respectively).

A Quick - BASIC computer program named BMP has been coded to calculate the variables required in each problem and to build the two dimensional models.

Nine locations along Baghdad Metro Project were selected to apply the study models. This project was chosen because it represents one of the important transport projects in Baghdad city as will be described in the next section. A great amount of geotechnical data, which are by experience societies, along metro alignment gave this study a strong tool to analyze the different geotechnical problems.

1.3 BAGHDAD METRO PROJECT

The Baghdad metro project was proposed by Baghdad Rapid Transit Authority (BRTA). The preliminary site investigations were completed in 1980 while the final investigations completed in 1983 by the Metro SI Joint Venture (MSIJV) which comprises:

1. *National Center for Construction Laboratories, (NCCL), Iraq.*

2. *Soil Mechanics Limited, (SML), U.K.*

3. *Fugro International BV., (FIBV), Netherlands.*

The project Fig.(1) comprises 32 km of Metro path which is made up of two lines meets at a central interchange, Khulfaa street; (Khallani SQ, S01) , thus forming four routes A to D. The proposed lines will have a 6.5 diameter tunnel with a minimum invert level at 20 m below existing ground level (BRTA, 1980).

This project is an important part of a massive program of public investment in a balance of transport system. It is designed to handle public transport in the higher density areas of the city. Other areas will be served by an integrated bus system providing feeder services to metro stations. There will also be convenient connections between the metro and Iraqi Railways on both sides of Tigris River, which flows through the center of Baghdad, and with the city's expanding highway network. The metro can be considered a catalyst to stimulate further development of the existing commercial and residential sectors (Edwards, 1982).

1.4 GEOLOGY OF BAGHDAD CITY

Iraq represents a part of the old Tethys sea which was affected by tectonic movements started 30 to 50 millions years ago, reached maximum intensity at the end of Tertiary Period. Structurally Iraq surface can be divided into three zones (Buday, 1980), as described below :

1. The Thrust Zone:

This zone occupies north and north east parts of Iraq. A complex structures of high mountains, thrust faults and igneous inclusions are found in this zone. This complexity resulted from subduction and collision of Arabian and Iranian Plates in Tertiary Period.

2. The Folded Zone :

This zone extends from north west to south east comprising narrow and parallel folds with reverse faults are located in the south west limbs of Some folds.

3. The Unfolded Zone :

This zone occupies the middle and southern parts of Iraq and extends to the West Desert in the west part of Iraq. This zone represents a flat area without surface outcrops for the subsurface Formations except some locations in the Desert. The south and south east parts of this zone are comprised mainly of alluvial deposits of cohesive and granular materials where Baghdad City is located.

Iraq surface was also divided topographically into five main units (Buringh,1960), as below :

1. *Zagros Mountains Area.*
2. *Foot Hills Area.*
3. *Jezira Area.*
4. *Lower Mesopotamian Plain Area.*
5. *Northern and Southern Desert Area.*

where Baghdad City lies in the Lower Mesopotamian Plain.

1.5 THE SOIL CONDITION IN BAGHDAD CITY

As discussed above, Baghdad City is located in the Lower Mesopotamian Plain. Which is an alluvial flood plain compris . Quaternary and Tertiary deposits derived largely from Zagros and Taurous mountains to the north east and north borders. These mountains are thought to be the result of the tectonic movements in the Tertiary Period. They consists mainly of limestones and dolomite with some calcareous mudstones and sandstones. Bedrock does not outcrop on the plain. Buday(1980) estimated the thickness of the Quaternary deposits about 150 to 200m discounted earlier estimates of 2000 m on the grounds that it probably included some of the underlying Tertiary sediments.

According to Buday; the Quaternary materials include deltaic, lacustraine and fluviatile sediments while the underlying Late Tertiary deposits are mostly of fluvio-lacustraine origin, this similarity contributes to the difficulty in identifying them.

Baghdad soil, Shows a generalized succession of essentially cohesive deposits overlying a major granular horizon; these have been termed the cohesive and granular strata respectively (BRTA, 1983a).(Figs, 1.2 and 1.3)

Significant changes in depositional environment must have occurred between granular and cohesive strata. Presumably a marked reduction in the hydraulic gradient of the river systems which might correspond to the general rise in the sea level at the start of Holocene some 8000 to 10 000 years ago. Since that time the climate has been stable and similar to that of the present day (BRTA, 1983b).

Wide changes in Baghdad soil strata, as will be discussed in Chap.3, may result from one or more of the factors (BRTA, 1983c):

1. *The presence of highly developed civilizations on the plain for at the last half of the Holocene.*

2. *The irrigation for agricultural purposes which has been carried out for the last 6000 years.*

3. *The annual floods of the Tigris River on its banks.*

4. *The change in the main river course some 4000 years ago by a dam constructed across it.*

The artifacts of earlier civilizations are more prevalent in the Recent Stratum, a term used by MSIJV reports to distinguish the upper part of the cohesive stratum.

1.6 SCOPE OF THE STUDY

In view of the objectives, this study was presented in seven chapters. Chapter one gives an introducing to the aims and problems of the study with a brief description to the geological and geotechnical conditions of the study area.

Chapter two shows a review of the early studies, and the more advanced studies on the subject where the important arithmetic relations and approaches are given.

Chapter three discusses the geotechnical assessment of Baghdad soil, and the geotechnical design factors were calculated.

In chapter four, the approaches used in this study are described, and all models are depicted. The multi purposes computer program BMP is also explained.

In chapter five, the geotechnical problems for every study site are represented in two dimensional models.

In chapter six a review of the findings of other studies

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In chapter six a review of the findings of other studies are given, and in chapter seven the conclusions and recommendations are recorded.

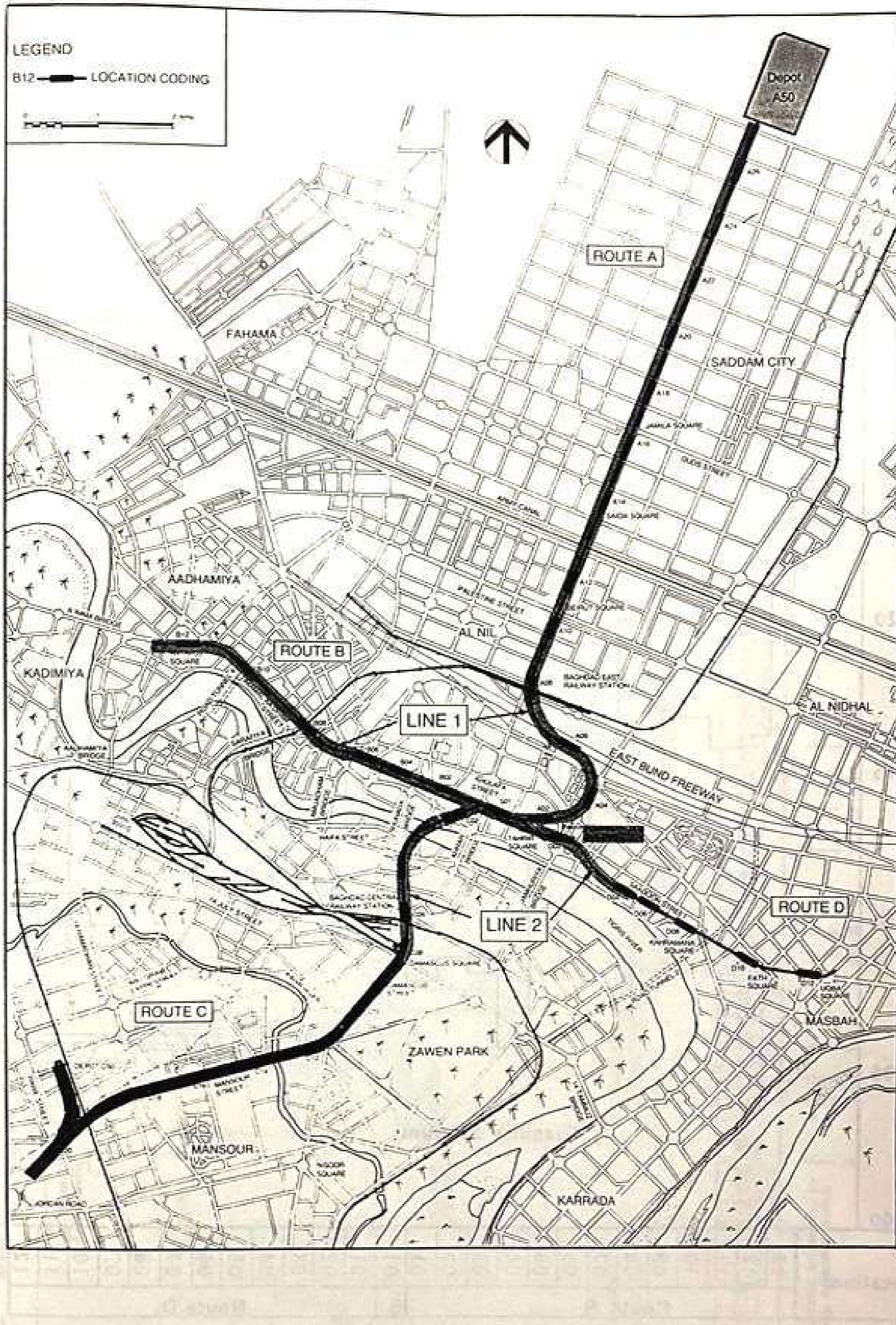


Fig.(1.1): Baghdad Metro Project (BRTA, 1980).

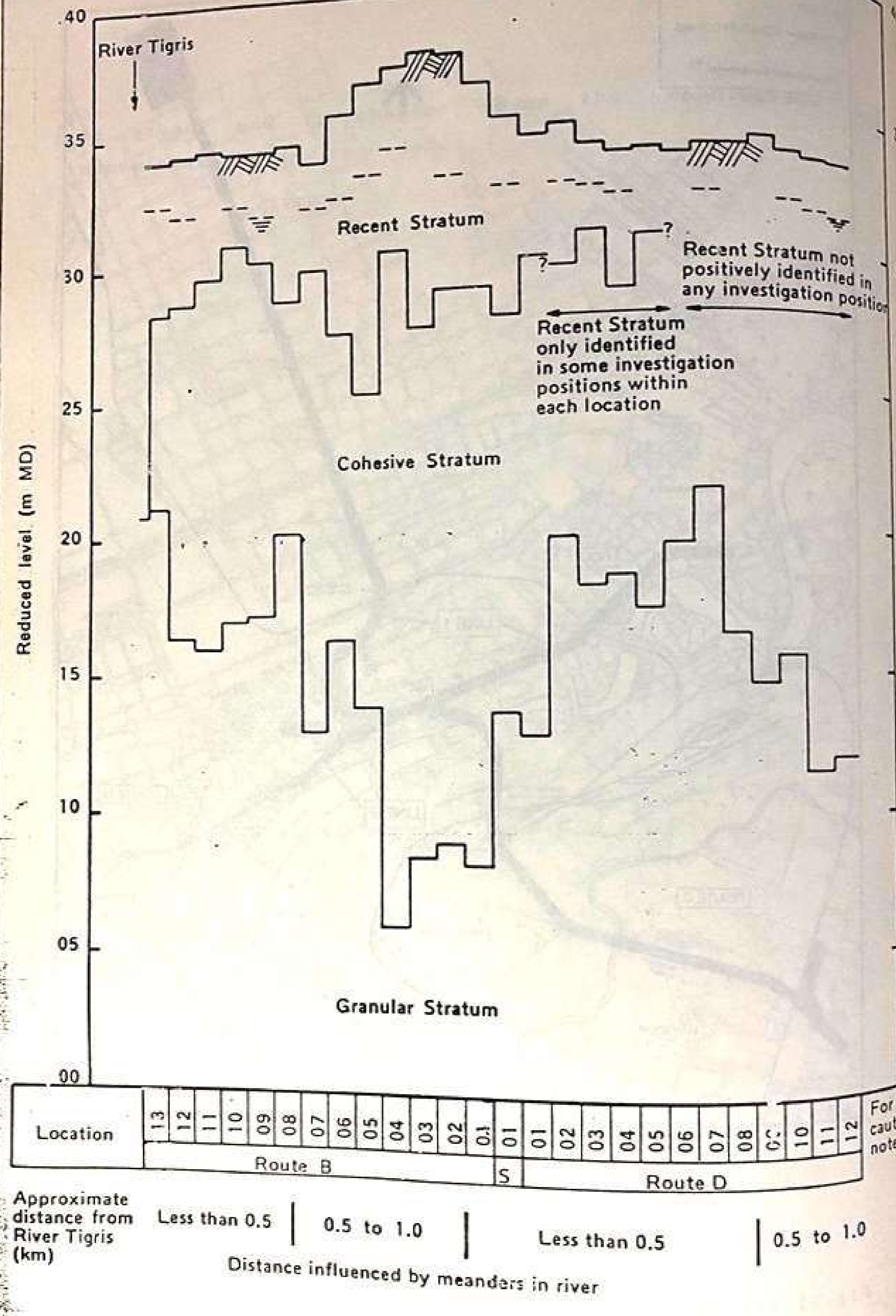


Fig.(1.3): SCHEMATIC SUCCESSION OF STRATA

(Routes B and D)

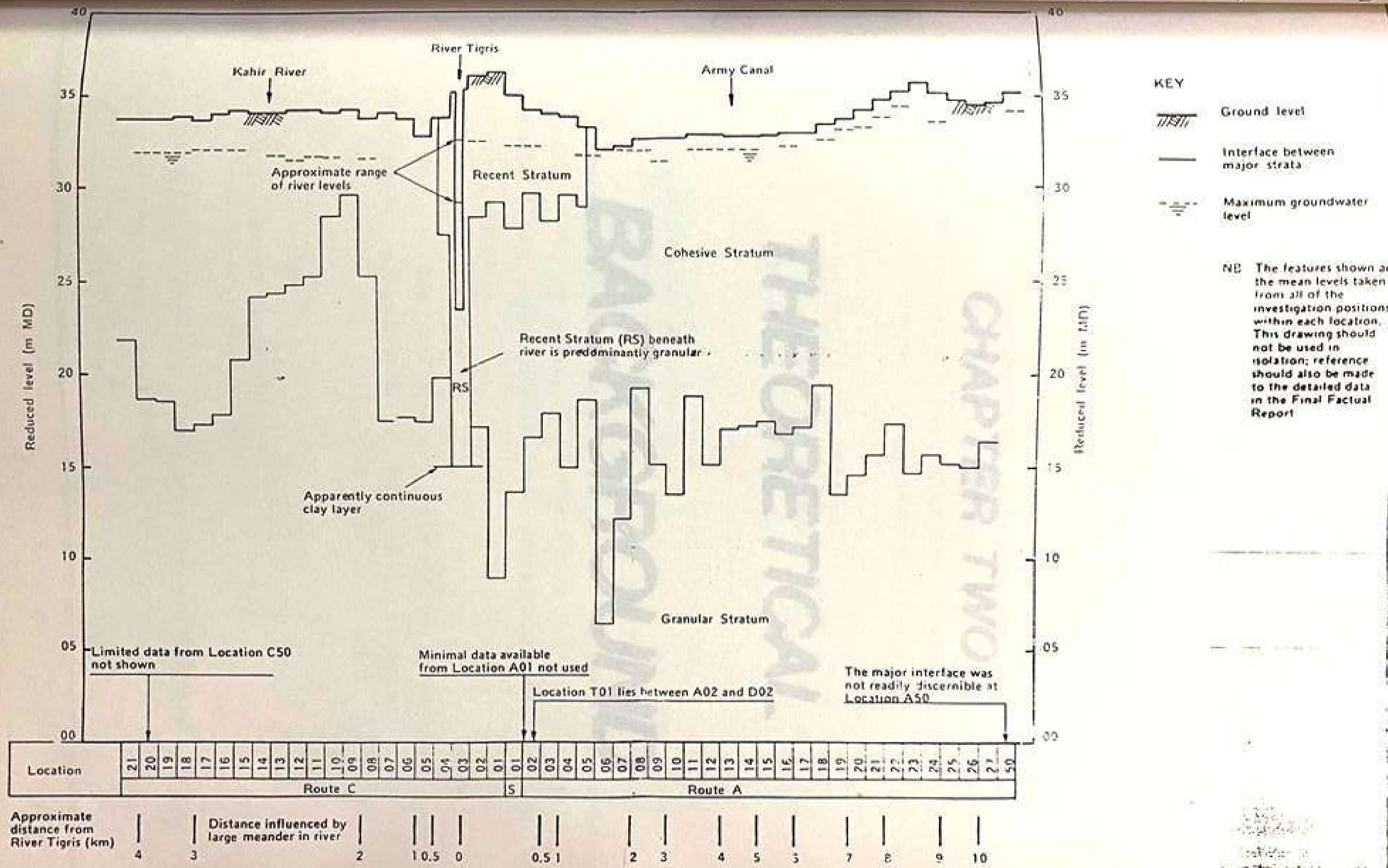


Fig. (1.2): SCHEMATIC SUCCESSION OF STRATA

CHAPTER TWO

THEORETICAL BACKGROUND

CHAPTER TWO
THEORETICAL BACKGROUND

2.1 GENERAL

This chapter gives the theoretical background and the previous researches and literatures dealing with the tunnel geotechnical design subject. In general there are three techniques which may solve the tunnel excavation and construction problems, these techniques are:

1. *Analytical Methods.*
2. *Experimental Methods.*
3. *Numerical Methods.*

2.1.1 ANALYTICAL METHODS

These techniques can be considered as the most economical method. One of their characteristics is the ability to reanalyze the problem with less complication for the same system geometry.

The serious deficiency with these techniques is the difficulty in representing the real material behavior in boundary conditions (Leco and Cloogh, 1992; Mair et al., 1982).

2.1.2 EXPERIMENTAL METHODS

The experimental techniques are widely used in solving the geotechnical engineering problems. The real ground conditions, and the engineering properties of structures ar simulated by a small model for purpose of testing study. On of the deficiencies this technique is the increment of it cost with increasing the tests and the model size in additio to the difficulty of simulating the real ground propertie (Glossop and Reilly, 1982).

2.1.3 NUMERICAL METHODS

The existence of computers helped in developing thi technique (Cundall, 1994). The finite element method is on of the most important methods because of its accuracy, generality, and ability of solving many and large problems. The main drawback is the difficulty to represent the rea internal properties of the model system and the hig expenses of the computers and the software needs in the bi problems (Cundall, 1994; Duddeck, 1991; Kielbassa an Duddeck, 1991; Konietzky,1994).

2.2 SOIL SUPPORT INTERACTIONS AROUND THE CIRCULAR TUNNEL

Excavation of tunnel involves removal of ground and installation of lining. Because of the relief of the initial in site stresses, the soil around tunnel will displace until it reaches a new state of equilibrium that is largely controlled by the manner of support activation (Bawden et al.,1996; Bulson, 1985).

Fig.(2.1) shows schematically the geomechanical system of a circular tunnel. A satisfactory model for ground movements and lining pressure prediction must include at less the below parameters(Fig. 2.1),(Wong and Kaiser, 1986):

1. *The depth ratio (H/a).*
2. *The horizontal to vertical stress ratio, Ko = (σ_h/σ_v).*
3. *The internal pressure, support pressure, Pi.*
4. *The ground properties as, cohesion C; friction angle ϕ; elastic modulus E; and Poisson's ratio ν.*

2.2.1 VERTICAL LOAD PREDICTION

The vertical load resulting from the overlying ground materials on the tunnel crown has long been investigated by many authors, (e.g., Bulson1985; Terzaghi, 1943).

Terzaghi (1943) studied the soil movements above tunnels and adopted the " *trapdoor* " approach to calculate the vertical pressure on the tunnel crown, roof. The problem of stress translation by mobilization of shear stresses in soil is defined by " *Arching* ",(Circular arrows in Fig 2.1). The studies showed that the shape and amount of arching depends on the relative flexibility of soil and lining (Wheby, 1982). When the tunnel lining is more flexible than the surrounding soil, or the wall is not supported, positive arching will result and the soil transfer most of the load side ways and only a small percentage of the load reaches the tunnel wall. Negative arching results when the lining is more rigid than surrounding soil. Where most of the load will be carried by lining, (Fig 2.2).

Duddeck and Erdman (1985); recommended based on review of common design approaches that the tunnel lining should designed against the following pressures:

1. *For H/a < 6:* Full overburden pressure at the crown.
2. *For H/a > 6:* appropriate reduction of ground pressure permitted (50% or more is debatable).

2.3 THE ASSESSMENT OF GROUND GEOTECHNICAL PROPERTIES

The ground responds during excavation greatly depen on the soil engineering properties. These properties mu be studied carefully to predict the initial ground stat elastic or plastic, because all computation methods of a theoretical approach to simulate the ground behavi will depend on this initial state. Three methods a used by researchers to indicate the initial ground stat these include the Wong and Kaiser method, the so consistency method, and the overload factor method (OFS).

Wong and Kaiser (1986) used the relation between stre ratio, Ko, and the Rankine's effective earth pressu coefficient, Ka, to estimate the initial state of soil a below:

When Ka > Ko , the soil is plastic. While when,
Ka < Ko , the soil is elastic.
where, Ka is calculated as:

$$Ka = \tan^2 (45 - \frac{\phi}{2}) \quad \text{..........} 2.3a$$

The soil consistency, the relation between soil water content and the Atterberg limits for the cohesive soil and the number of blows in standard penetration test for the granular soils (Bowels, 1984), gives an idea about the shear strength and initial state of soil as will be discussed in chapter three. For cohesive soils the consistency index, *Ic*, was also used. This index is defined as the ratio of the difference of liquid limit and natural water content to the plasticity index, as bellow:

$$Ic = \frac{LL - Wn}{PI} \quad \ldots\ldots\ldots \quad 2.3b$$

Where; LL: Liquid limit %, *Wn:* Natural water content%

PI: Plasticity index % .

The below table describes the relation between, *Ic*, and the soil consistency (Boniface et al., 1994).

State of Consistency	Ic
Very soft to soft	$Ic < 0.5$
Firm	$0.5 < Ic < 0.75$
Stiff	$0.75 < Ic < 1.0$
Very stiff or hard	$Ic > 1.0$

Table(2.1): The Cohesive soil Consistency (Boniface et al 1994).

The third method for determining the initial state of soil depends on the overload factor (OFS). This factor was introduced firstly by Peck (1969); then used by Deer et al., (1969) to describe the predicted soil behavior during tunnel excavation. The OFS is computed as :

$$OFS = \gamma . H / \tau \quad \ldots\ldots\ldots\ldots \quad (2.3c)$$

Where :

γ : Bulk unit weight of soil (kN/m3).

H : The depth to the tunnel axis (m).

τ : Shear strength of the soil (kN/m2).

By using the OFS factor , the soil reactions during excavation can be classified as below (Schmitter and Rendon, 1981):

1. *OFS < 2 to 3 :* The tunnel face movements are small and essentially elastic.

2. *3 < OFS < 6 :* The movements of the face are predominantly plastic, increasing generally as the OFS ratio increases.

3. *OFS > 6 :* The critical stability condition is reached and the face has a high risk of failure.

When the compressed air pressure *(Pi)* is used to support the tunnel face Eq.*(2.3c)* becomes :

$$OFS = (\gamma . H - Pi) / \tau \quad \ldots\ldots \quad (2.3d)$$

2. 3.1 THE GRAIN SIZE DISTRIBUTION

The grain size distribution of soil will affect the nature of the ground responses during excavation, This distribution will give an idea about shear strength components, permeability, and the stability of the ground against quick sand problems (Bowels, 1984; Lamb and Whitman, 1979). The quick sand problem results when the soil shear strength becomes equal to zero (this problem will be discussed in detail in chapter four).

Yokoyama,(1979); studied the problems resulting from tunnel excavation in a high water pressure ground. He recommended that the following soil characteristics play an important role in the soil stability against quick sand problems:

a. *Finer materials content, clay + silt, (f%)*: The percent of fine materials must be greater than 5%.

b. *Coefficient of uniformity, D_{60}/D_{10}, (Uc)*: This coefficient must be greater than 4.

2.4 THE STRESS DISTRIBUTION AROUND TUNNEL OPENING

When a circular tunnel is driven in any ground, the initial ground stresses (vertical and horizontal stresses) will be changed to new stress components named radial and tangential stresses. Obert and Duval (1967); Szechy (1967) and Peck (1969) gave many equations to calculate the radial and tangential stress components around the circular tunnel. Most of these equations depend on the theory of elasticity or on practical measurements. The following equations are widely used when the circular tunnel is excavated in rock or soil and when the ground is initially elastic, where no yielding occurs during unloading (Bulson, 1985; Obert and Duval, 1967):

$$\sigma_r = \left(\frac{\sigma_h + \sigma_v}{}\right) \cdot \left(1 - \frac{a^2}{r^2}\right) + \left(\frac{\sigma_h - \sigma_v}{2}\right) \cdot \left(1 - \frac{4a^2}{r^2} + \frac{3a^4}{r^4}\right) \cdot \cos 2\vartheta \qquad \text{.........2.4a}$$

$$\sigma_t = \left(\frac{\sigma_h + \sigma_v}{2}\right) \cdot \left(1 + \frac{a^2}{r^2}\right) - \left(\frac{\sigma_h - \sigma_v}{2}\right) \cdot \left(1 + \frac{3a^4}{r^4}\right) \cdot \cos 2\vartheta \qquad \text{.........2.4b}$$

Where:

σ_r : radial stress.

σ_t : tangential stress.

r : the radial distance from tunnel center.

a : the tunnel radius.

ϑ : polar coordinate, horizontal axis represent $\vartheta = 0$.

σ_v, σ_h : the vertical and horizontal stress respectively

From Eqs.(2.4a and b) it can be seen that the stresses are independent on the elastic constants of the ground, and the tunnel radius appears in these equations only in the dimensionless ratio (a/r) which specifies the distance from tunnel boundary (Obert and Duval, 1967).

Deer et al.(1969) developed the equations below to study the plastic initial ground behavior:

$$\sigma_r = P_i + 2.\tau.\ ln_{(a)} \qquad \text{................2.4c}$$

$$\sigma_t = P_i + 2.\tau.(1 + ln_{(a)}) \qquad \text{...........2.4d}$$

Where; P_i represents the internal pressure applied on the unlined tunnel for stabilization purposes.

The radial and tangential stress components, for each soil state, coincide with the principal total stresses or:

$\sigma_t = \sigma_v$ and $\sigma_r = \sigma_h$ at the spring line (tunnel axis level), and $\sigma_t = \sigma_h$ and $\sigma_r = \sigma_v$ at the invert and crown of tunnel (floor and roof).

Wong and Kaiser (1987a and b) used the finite element method to study the stress redistribution around the circular tunnels in two dimensional models. They found that the yielding initiation will be started when the stress difference ($\sigma_t - \sigma_r$) exceeds the shear strength of the soil. The yield location around the tunnel depends on the initial ground stresses (σ_v, σ_h) and the ground stress ratio (K_o).

2.5 THE GROUND CONVERGENCE CURVE GCC

The ground convergence curve, GCC, method was suggested by many authors (E.g, Deer et al., 1969; Duddeck, 1991; and Enistein and Schwatz, 1979) to study the soil behavior at the tunnel wall. Fig.(2.3) presents a schematic ground convergence curve for a point at the tunnel wall. Before excavation, the curve originates at point (1) or at the initial overburden pressure ($Po = Pi$), $Po = \gamma H$. Portion (1 - 2) represents the elastic unloading response of the ground. Point (2) indicates for the initiation of yielding. Portion (2 - 4) reflects the non-linear ground response due to yielding up to start of wall collapse mechanism.

Hoek et al.,(1995) used the following analytical approach to determine the different GCC portions for point at the tunnel wall:

Firstly the critical support pressure, P_{cr}, and the ground strength, σ_{cm}, are determined:

$$P_{cr} = (2.\sigma_v - \sigma_{cm})/(1 + K_o) \quad \ldots\ldots\ldots\ldots 2.5a$$

and

$$\sigma_{cm} = 2.c.\cos\phi \,/\, (1 - \sin\phi) \ldots\ldots\ldots\ldots\ldots 2.5b$$

where,

C and ϕ: The soil cohesion and friction angle respectively.

If the internal support pressure $P_i > P_{cr}$, no failure occurs and the ground behavior surrounding tunnel opening is elastic, portion 1-2 of Fig.(2.3). The inward radial elastic displacement U_{ie} of the wall is given by:

$$U_{ie} = (a.(1 + \upsilon)/E).(\sigma_v - P_i) \ldots\ldots\ldots\ldots\ldots 2.5c$$

Where,

E: Young modulus or deformation modulus.

υ: Poisson's ratio.

When $P_i < P_{cr}$, yielding occurs, portion (2-4), Fig.(2.3), and the total inward plastic displacement is given by:

$$U_{ip} = (a.(1 + \upsilon)/E).[2.(1 - \upsilon).(\sigma_v - P_{cr}).(rp/a)^2 - (1 - 2\upsilon).(\sigma_v - P_i)] \ldots\ldots\ldots\ldots 2.5d$$

Where,

$$rp = a.[(2.(\sigma v.(Ko - 1) + \sigma cm))/(1 + Ko).((Ko - 1).Pi + \sigma cm)]^{(1/(ko-1))} \quad ....2.5e$$

Hoek et al. also based on field measurements, concluded that the radial displacement starts about one half a tunnel diameter ahead of the advancing face, and reaches about one third of its final value of the tunnel face. This radial displacement gets to final value at about one and one half tunnel diameter behind the face (Fig 2.4)

This analytical approach have been used in this study to present the ground responses to the tunnel excavation, (Chap. 4)

Duddeck (1991) used finite element methods to draw the GCC for soil tunnels in order to analyze the predicted ground pressure on the tunnel lining.

2.6 THE GROUND SURFACE SETTLEMENT ABOVE THE TUNNEL

The ground surface settlement above tunnel axis takes more attention especially when the tunnels passed under urban areas, because this settlement may lead to destroy the buildings and streets.

The settlement that occurs during and after constructing the tunnel can be divided into two types:

1. *Type one is caused by uncontrolled reasons such as plastic deformation and property changes of soil due to the local over stress, unbalanced*

forces and soil movement inherent to the excavation itself and the movement of the boring machine through soil media.

2. Settlement type two occurs when the water lost from the soil through the boring, this type can be reduced to zero by using many methods as shotcrete or compressed air.

In order to determine the ground surface settlement the following questions must be answered (Rosza, 1979):

a. The value of the maximum surface settlement.

b. The extension of surface settlement.

c. The form of surface settlement profile.

The shape of the settlement profile can be approximated as given below (Wong and Kaiser, 1987b):

$$S = S_s \; EXP \; (-X^2 / 2i^2) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2.6a$$

Where;

S : The surface settlement at a transverse distance X from tunnel axis.

S_s : Maximum surface settlement.

i : Location of maximum settlement gradient or the point of inflexion.

The unknown parameters, i, and S_s, are calculated by many researchers based on model tests and field measurements. Wong and Kaiser (1987a), found from field observations that:

$$i = 0.43.H + 1.1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.6b$$

$$i = 0.28H + 0.1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.6c$$

Eqs.(2.6a and b) are used for cohesive and granular soil respectively.

These equations were used practically to determine the extension of ground settlement above tunnel axis, where the half width of settlement trough W can be defined by:

$$W = 3i \quad \ldots\ldots\ldots\ldots\ldots\ldots 2.6d.$$

and the maximum settlement S_s is given by:

$$S_s = VI\% \;/\; W \quad \ldots\ldots\ldots\ldots 2.6e.$$

Where;

VI% : is the volume of the settlement trough per unit length along the tunnel axis.

Deer et al.(1969) suggested that in sand, when there is n unusual problems, the *VI%* is taken as 1%. And when the tunnel is excavated below water table the conditions become difficult and the *VI%* is taken as 2.5 to 5%.

Peck (1969), depending the field measurements, suggested a relation between the *OFS* and *VI%* in cohesive and granular soils(Fig 2.5), this relation is very important and used widely in practical settlement determination (Schmitter and Rendon, 1981).

Rosza(1979) studied the settlement problem above the metro stations in Poland depending on analytical approaches.

while Wong and Kaiser (1987b) used finite element methods to determine the ground settlement above tunnels for different excavation methods, sequential and compressed shield.

2.7. THE STRESS VARIATIONS WHEN USING COMPRESSED AIR IN THE TUNNEL EXCAVATION

The compressed air used in the tunnel excavation process will cause a wide change in the initial ground stresses around the opening. Fernandez and Alvarez (1994) treated this problem by mathematical methods where the ground mass was taken as a homogeneous, isotropic, porous, elastic medium and the unlined tunnel is pressured with an internal pressure, P_i, and assumed the flow out of the tunnel to be radial and the Darcy's law was valid. Based on these assumptions, they estimated the excess pore pressure, induced by seepage within the surrounding ground medium as:

$$P_w = P_i \frac{\ln(b/r)}{\ln(b/a)} \quad \text{.......................} \quad 2.7a$$

Where P_i = internal pressure, and b = an arbitrary radial distance at which the seepage induced excess pore water pressure becomes nil.

Eq. (2.6a) was developed by the same authors to become more suitable to calculate the excess pore pressure around the tunnel. The excess water pressure P_w, induced by an internal pressure in the tunnel P_i, at any point around the opening can be estimated as:

$$Pw = \gamma w(hi-ho)\left[\frac{\ln\left[1+\frac{4ho}{r}\left(\frac{ho}{r}-\cos\theta\right)\right]}{\ln\left[1+\frac{4ho}{a}\left(\frac{ho}{a}-\cos\theta\right)\right]}\right] \quad \ldots\ldots 2.7b$$

Where;

Pw = excess pore water pressure.

γw = water unit weight.

ho = external hydrostatic head.

θ = angle measured clockwise from the crown of the tunnel.

Fernandez and Alvarez also used an analytical relationships to estimate the seepage induced stresses within the ground mass at angle equal to 90° from tunnel crown as below:

$$\frac{\sigma_r'}{\Delta Pw} = \frac{1}{2(1-\upsilon)}\left[(a^2/r^2-1)+\frac{2\ln r/a+[(1-2\upsilon)(1+4ho^2/r^2)-2(1-\upsilon)]\ln(\varepsilon)}{\ln(1+4ho^2/a^2)}\right]$$

$$\ldots\ldots\ldots\ldots\ldots\ldots 2.7c$$

$$\frac{\sigma_t'}{\Delta Pw} = \frac{-1}{2(1-\nu)} \left[(a^2/r^2 + 1) - \frac{2\ln\frac{r}{a} - [(1-2\nu)(1+4ho^2/r^2) + 2\nu]\ln(\varepsilon)}{\ln(1+4ho^2/a^2)} \right] \quad \text{.......... 2.7d}$$

Where;

$$\Delta Pw = \gamma W\,(hi - ho) \quad \text{.......... 2.7f}$$

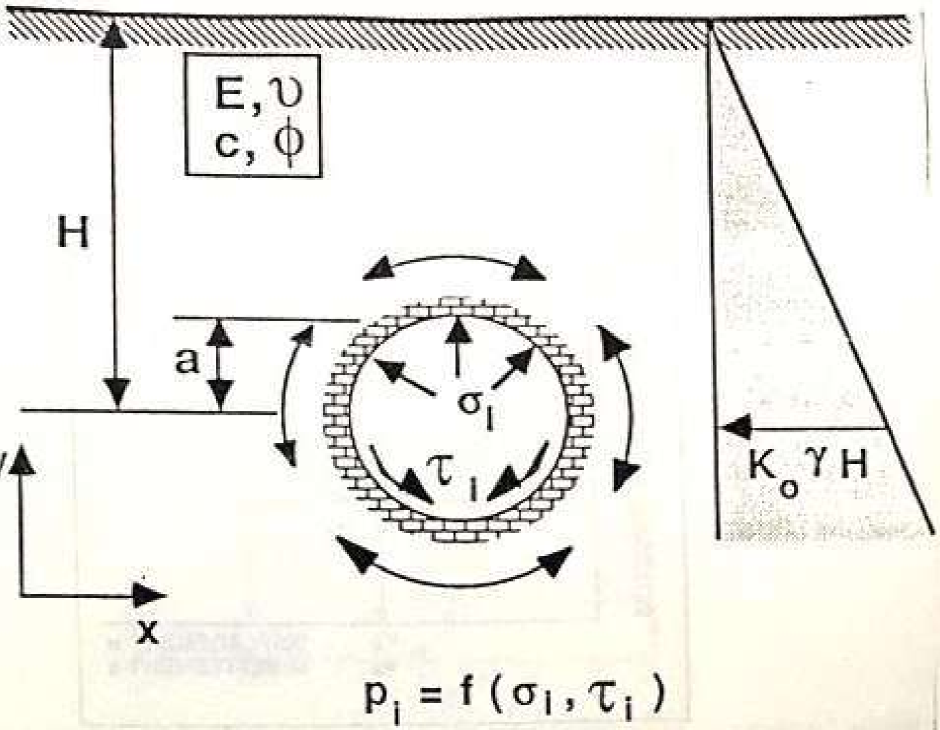
$$\varepsilon = \frac{(r^2/a^2 + 4ho^2/a^2)}{(1 + 4ho^2/a^2)} \quad \text{.......... 2.7g}$$

The air pressure required to support the tunnel during excavation can be calculated by the below relation (Snee and Javadi, 1996):

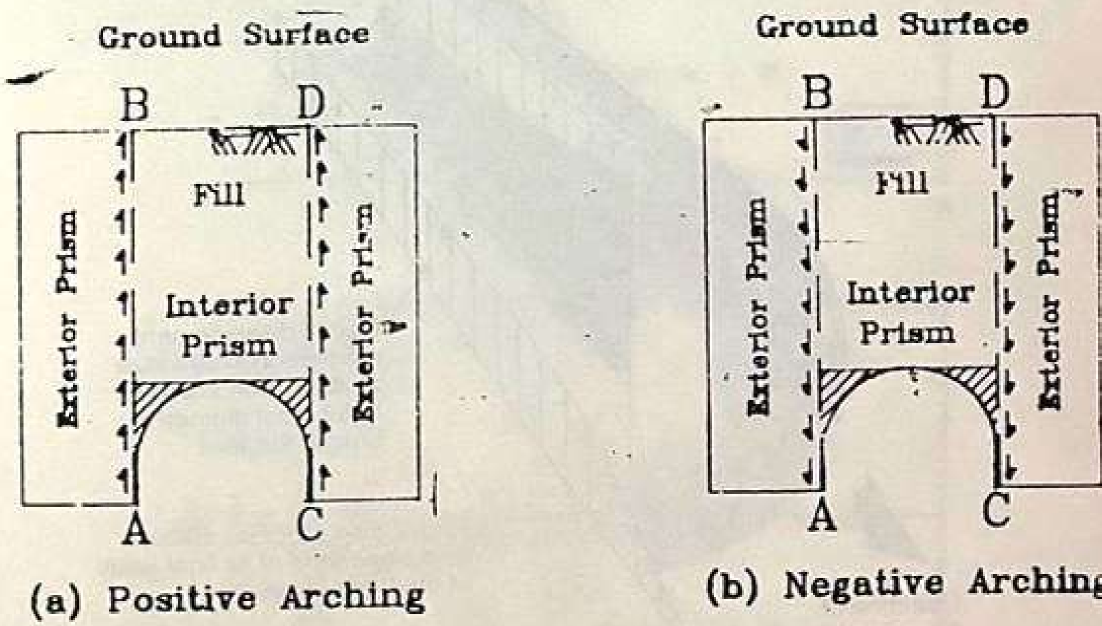
$$Pi = (H + 2/3\,D).\gamma W \quad \text{.......... 2.7h}$$

Where; D = the tunnel diameter .

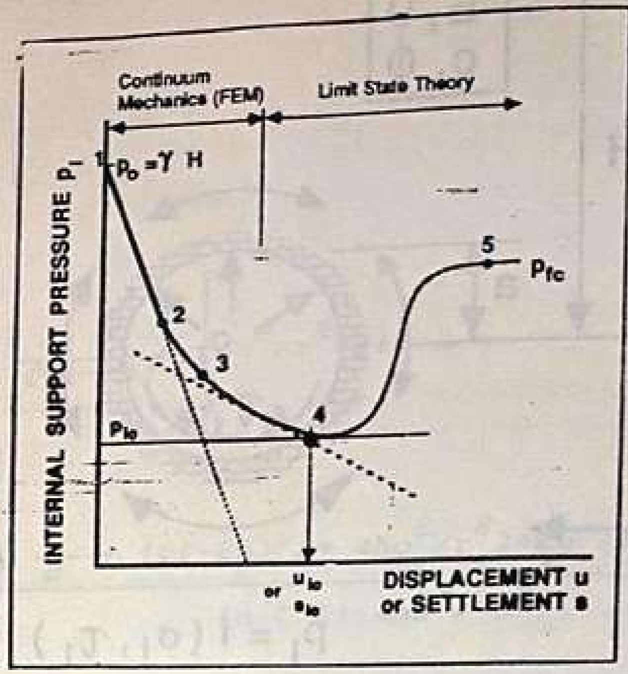
Fig (2.6) present the stress relations at angle 90° from tunnel crown as calculated by the above equations for different tunnel geometric relations.



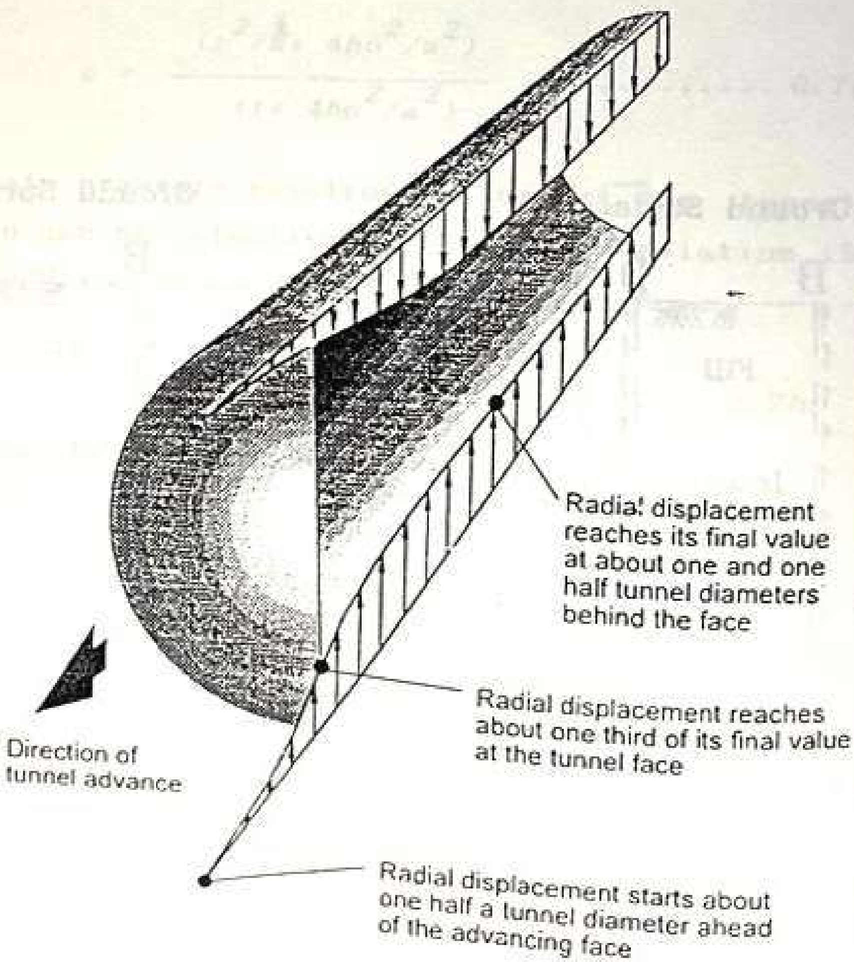
Fig(2.1): Geomechanical system of circular tunnel (Wong and Kaiser, 1986).



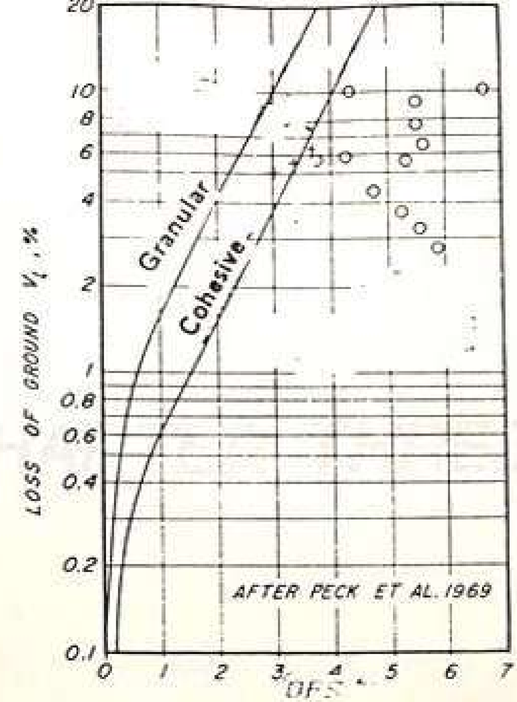
Fig(2.2): Passive and active arching (Bulson, 1985).



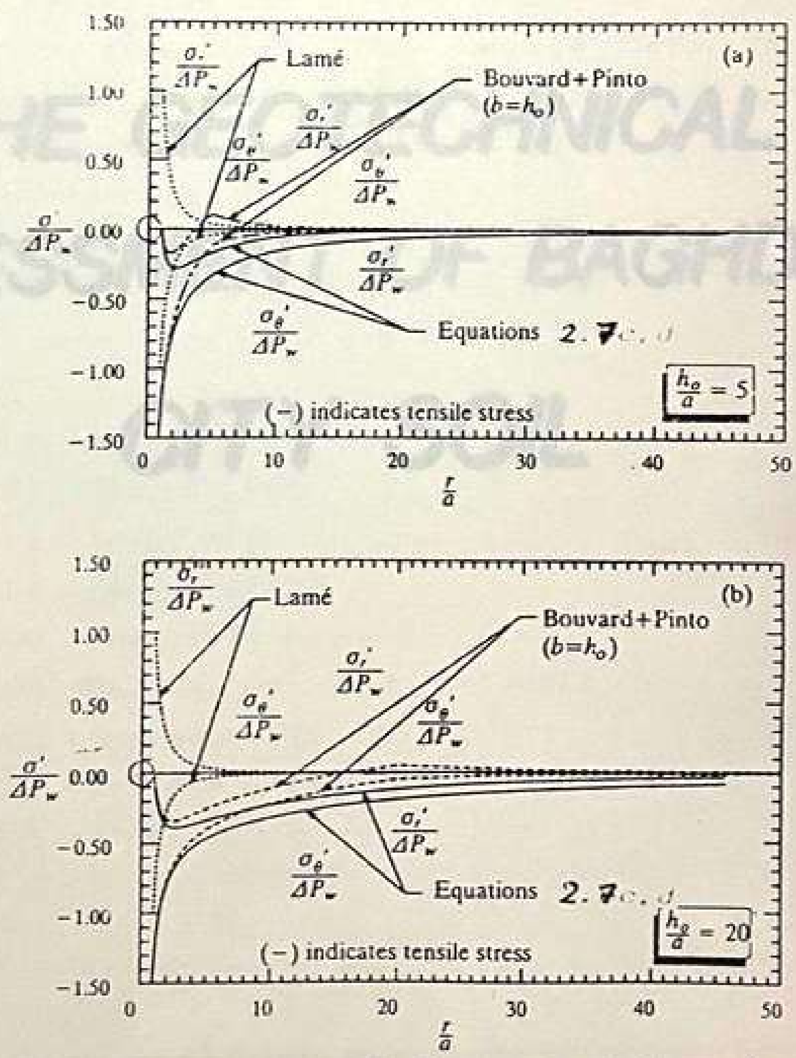
Fig(2.3): Ground convergence Curve For a point in the tunnel wall (Wong and Kaiser, 1986).



Fig(2.4): Ground Behaviour During Tunnel Excavation (Hoek et al.,1995).



Fig(2.5): Peck's chart for determine V/%, (Peck,1969).



Fig(2.6): Seepage induced stresses at tunnel springline, (Fernandez and Alvarez, 1994).

CHAPTER THREE

THE GEOTECHNICAL ASSESSMENT OF BAGHDAD CITY SOIL

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THE GEOTECHNICAL ASSESSMENT OF BAGHDAD CITY SOIL

3. 1 GENERAL

The engineering properties of Baghdad City soil were tested based on recommendations given in British standards (BS 5980) in 1980, along metro alignment, by many companies and the data have been recorded in several geotechnical reports, (BRTA, 1983a, b, and c). These data were used in this study for understanding the geotechnical situation, engineering properties and to estimate the initial soil behavior during excavation. The results obtained in this chapter were used in the next chapters to build the geotechnical models in order to simulate the various ground problems around tunnel opening.

Nine locations have been selected to use in this study. These locations are, see Fig (3.1):

1. S01 *(Khallani SQ)*
2. A02 *(Tahryer SQ)*
3. A14 *(Saida SQ)*
4. A24 *(Saddam City)*
5. B06 *(Jumhuryia SQ)*
6. C02 *(Rashied St., Near Rafidain Bank)*
7. C08 *(Demasqus SQ)*
8. C20 *(Mansoor City)*
9. D06 *(Al-Frdos SQ)*

These locations were selected according to the importance of its site . and to cover various soil properties and geometrical tunnel relations (H/a ratio).

3.2 THE MAIN GEOTECHNICAL CHARACTERISTICS OF BAGHDAD SOIL STRATA

The succession of Baghdad soil, as discussed in Chap.1, comprises essentially cohesive deposits overlying granular deposits . The upper part of the cohesive deposits near the Tigris River area comprises a low consistency with occurrence of brick, bone and pottery fragments, so, the cohesive materials are divided into Recent and Cohesive deposits (BRTA, 1983a).

According to the above discussion and references, Baghdad soils are divided into three main strata, these strata from the ground surface are:

1. *Recent Stratum.*
2. *Cohesive Stratum.*
3. *Granular Stratum.*

3.2.1 RECENT STRATUM

Apart from the river, the Recent Stratum is essentially cohesive in nature although the deposits contain variable proportion of up to gravel or even cobble size fragments of brick, pottery and bone, where the deposits have been identified as made ground which also include extraneous matter.

The Recent Stratum can be differentiated from the Cohesive Stratum by the presence of bone and pottery fragments, coarser gravel and cobble size brick fragments. It is usually, although not always, of a lower consistency and it could be postulated that it consists of cohesive materials eroded from the Cohesive Stratum and then re - deposited in a depositional environment which usually precluded desiccation.

3.2.2 THE COHESIVE STRATUM

This stratum is somewhat variable in nature; it is generally comprised of stiff to very stiff clay, although sandy clay and clay (very silty) also occur. Layers and/or lenses of silty sand and sandy silt are present throughout the Cohesive Stratum; these are generally relatively thin although layers up to several meters thick were encountered with a significant proportion of the investigated positions along metro alignment. There is some overall tendency for the layering and lensing to occur more frequently in the lower half of the stratum particularly in those locations remote from the river. However, it is not generally possible to correlate the variations within the Cohesive Stratum from one investigation position to the next (BRTA, 1983a).

3.2.3 GRANULAR STRATUM

The full thickness of the Granular Stratum was not proved at any of the investigations positions (BRTA, 1983c). The maximum depth achieved being 60.27m. The stratum