2015

Geotechnical Statistical Evaluation of Lahore Site Data and Deep Excavation Design

Aiza Malik
Portland State University

<u>CE 501</u>

<u>GEOTECHNICAL STATISTICAL EVALUATION OF LAHORE SITE DATA AND DEEP EXCAVATION DESIGN</u>

Submitted By:

Aiza Malik

A research project report submitted in partial fulfillment
of the requirement for the degree of

MASTER OF SCIENCE
IN
CIVIL AND ENVIRONMENTAL ENGINEERING

Project Advisor:

Dr. Trevor Smith

DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING

PORTLAND STATE UNIVERSITY

USA

ACKNOWLEDGEMENTS

I would like to thank Dr. Trevor Smith for his continuing support and help throughout the project. I would also like to thank the following companies and institutes for providing site data for Lahore, Pakistan.

1. Geo Associates and Consultants, Lahore
2. Building Standards, Lahore
3. National Engineering Services Pakistan (NESPAK), Lahore
4. University of Engineering and Technology Lahore (UET), Lahore

ABSTRACT

Geotechnical characterization for foundation design is critical during preliminary planning, designing and feasibility studies of various engineering projects. In this research, an effort has been made to develop a geotechnical database for the city of Lahore, Pakistan. This database would aid geologists and engineers involved in the geotechnical design and planning of engineering projects in Lahore. The project area has been divided into zones geographically. Soil profiles have been developed for all zones, which provide ranges of soil properties and SPT-N values at regular intervals. Furthermore, the research also focuses on deep excavations in urban areas of Lahore, Pakistan and the design of support systems. These systems have been designed using two different methods and a comparison has been drawn.

TABLE OF CONTENTS

CHAPTER#1

CHAPTER#2

CHAPTER#3

CHAPTER#6

CHAPTER#7

LIST OF TABLES

LIST OF FIGURES

NOTATIONS

A_s	Surface area of the pile in contact with the soil
A_T	Cross sectional area of the pile tip
c	Un-drained cohesion of soil
D	Diameter of pile
E	Modulus of Elasticity
f_s	Unit skin resistance
H	Depth of excavation
L	Length of pile
N_c, N_q	Bearing capacity factors
Q_s	Skin resistance
Q_t	End-bearing resistance
Q_{ult}	Ultimate Pile Capacity
Q_{all}	Allowable Pile Capacity
q	Unit end-bearing resistance
s_u	Un-drained shear strength of soil
α	Adhesion factor
γ	Unit weight of soil
φ	Angle of internal friction of the soil
ASD	Allowable Stress Design
FHWA	Federal Highway Administration
NAVFAC	Naval Facilities Command Engineering
NA	Not Available
NP	Non-Plastic
SPT	Standard Penetration Test

CONVERSIONS

1 tons per square foot (tsf) = 95.76 kilopascals (kPa)

1 pound per cubic foot (lb/ft^3) = 0.00013 kilonewton per cubic meter (kN/m^3)

CHAPTER -1

INTRODUCTION

1.1: General

In Pakistan, soil investigation has not been given its due importance. Most of the investors consider geotechnical investigations a mere waste of money and do not understand the importance of geotechnical engineering. However, after the devastating earthquake of 2005, geotechnical investigation is being given more importance. The earthquake resulted in the collapse of a number of structures. The investigation later on revealed that the structures had been built without conducting soil investigations and therefore could not take the seismic loads. After 2005, the government mandated that a geotechnical investigation report should be presented before construction of a structure. Therefore, abundant scattered geotechnical data is available which needs to be compiled and presented in a useful format.

During this research an effort has been made to develop a geotechnical data base for the aid of geologists and engineers involved in preliminary planning, designing and feasibility studies of engineering projects in Lahore city. The city of Lahore has been divided into zones geographically and the soil data has been presented in the form of soil profiles developed for each zone. The site locations have been visually displayed on a map of Lahore using GIS software.

Lahore commands a strategic political and administrative role as the capital of Punjab Province and the second largest city of Pakistan. It has been a centre of business, trade and politics since its inception. Therefore, the price of land is increasing and builders are looking to save money by developing multiple basements. Deep excavation support systems are required for such developments and these are a new concept in Lahore. Taking this into consideration, the research also focuses on deep excavations support systems and different methods of design being used in Lahore. For this project, these systems have been designed using two different methods and a comparison has been drawn.

1.2: Purpose and Objectives

The project is divided into two parts and the objectives are discussed below:

Part A

On the basis of the need to develop geotechnical characterization for foundation design for different zones of Lahore, the following have been identified as the basic objectives of the study:

- To divide Lahore into zones based on the geography of the region.
- To collect and analyze soil type and soil properties data at regular 2 m intervals for each zone.
- To present the soil data in a format that could be easily used by engineers and geologists in the design process.
- To visually display the site locations on a map of Lahore using GIS software.

Part B

The second part of the project deals with the deep excavations and following are the basic objectives of the study:

- To get a better understanding of the deep excavation support systems used in Lahore.
- To get an understanding of the deep excavation support system design methods used by contractors in Lahore.
- To perform deep excavation support system design according to two different methods being used by contractors in Lahore.
- To draw a comparison between both the design methods.

1.3: Work Plan and Methodology

The following methodology and work plan were prepared for the proposed study.

- Lahore was divided into five zones based on the geography of the region.
- Geotechnical investigation data for 60 sites, scattered throughout Lahore city, was collected from various specialized geotechnical consultants and contractors.
- The soil type was determined for each zone at a regular interval of 2 m and presented on the soil profile. The depth variation of different soil types were also presented in the profiles.
- The in-situ soil properties were also determined for each zone at regular intervals of 2 m and presented on soil profile.
- The site locations were shown on a map of Lahore using the GIS software. The zone boundaries were also shown on the map.
- Deep excavation support system types and design methods being used in Lahore were studied in detail.
- Deep excavation support systems were designed according to two different methods and a comparison was drawn.

CHAPTER -2

PROJECT AREA

2.1: Location

Lahore commands a strategic political and administrative role as the capital of Punjab Province and the second largest city of Pakistan. Lahore District lies between 31^{o}-15' and 31^{o} -42' north latitude, 74^{o} -01' and 74^{o} -39' east longitude [1]. It is situated in the north-eastern part of Pakistan with its centre lying within 25 km of the international border with India as shown in Figure 2.1 [2]. It occupies a focal position in the Upper Indus Plain and is located along the eastern bank (left bank) of River Ravi. Lahore is bounded on the north and west by the Sheikhupura District, on the east by Wagah, and on the south by Kasur District [1]. Lahore city covers a total land area of 404 square kilometres (156 sq mi) and is still growing [1].

Figure 2.1: Location Map of Lahore [2]

2.2: History of Lahore

Evolution of Lahore Metropolis dates back to the first millennium. During the regimes of this era, Hindu, Afghan, Turk and Mughal Rulers, made periodic changes in the physical form of Lahore, which were mostly confined within and around the Walled City. Development of Civil Lines and Cantonment by the British Empire in 1857, provided strong impetus towards urbanization [1, 2].

Partition of the Sub-continent in 1947 brought a major upheaval and everlasting changes in the socio-economic and physical set up of Lahore. Major roads connecting Lahore to other cities are G.T. Road, Multan Road, Raiwind Road, Ferozepur Road, Sheikhupura Road and Jaranwala Road. The main railway line connects Lahore to most of the settlements along northern and southern routes and also to the neighboring country-India, through Wagha in the east [1].

2.3: Topography

Lahore is generally flat and slopes towards south and south-west at an average gradient of 1:3000. It can be divided into two parts i.e. the low lying area along River Ravi and the comparatively upland area in the east away from Ravi [2]. The low lands are generally inundated by the river water during monsoon floods. River Ravi flows in the west of Lahore District forming a boundary with Sheikhupura District [2].

The original physiographic features like channels remnants and levees have been destroyed or changed by the construction of urban infrastructure. Flood plains have been confined by construction of embankments (bunds) and spurs. Sub-recent flood plain is 4 to 8 meters higher than the recent flood plain and can be identified at number of places i.e. Shalimar Garden, Moghalpura and Multan Road [2].

2.4: Climate

Lahore features a five season semi-arid climate and the seasons are winter, summer, spring, autumn and monsoon. The hottest month of the year is June when temperatures routinely exceed 40 ^{o}C. The wettest month is July, with heavy rain falls and evening thunderstorms with the possibility of cloudbursts. The coolest month is January with dense fog [3]. The mean maximum and minimum temperatures in summer are 48 ^{o}C and 38 ^{o}C and in winter 25 ^{o}C and -1 ^{o}C respectively [3].

2.5: Geology

Lahore city lies on the alluvial plain called Bari Doab. Doab is a local word for area between rivers as shown in Figure 2.2. Bari Doab is a part of the Indo-Gangatic alluvial plain formed by the Indus river and its tributaries. It is bounded by Ravi and Chanab rivers in the northwest and west and by Sutlej river in the southeast. Northeastern boundaries of Doab lies near the foothills of the Himalayan Ranges [4].

The Bari Doab is covered by Quarternary alluvium which overlies semi-consolidated Tertiary rocks or Metamorphic and igneous rocks of Precambrian age. Except for a small area in the northeastern part of Doab where basement rock was encountered no information is available at present regarding the distribution of Tertiary and precambrian rocks in the Doab [4].

2.5.1: Precambrian Basement Rock

The oldest rocks, the Kiranas, of Precambrian age are completely covered by Quarternary alluvial deposits. The same deposits also cover Bari Doab. The thickness of this alluvial plain extends beyond 610 meters. Out of several deep boreholes drilled in Bari Doab only one, drilled near Niazbeg in the vicinity of Lahore, encountered bedrock at 383 meters depth. This is possibly due to the underground ridge of Precambrian rocks extending from Shahpur to Dehli. From this it can be inferred that the thickness of alluvium under the city of Lahore is more than 380 meters [4].

2.5.2: Quaternary Alluvial Complex

The alluvium derived from the mountain/ranges to the north has been deposited by the present and ancestral tributaries of the Indus River. The alluvial complex of Pleistocene and recent age represent the latest phase of sedimentation in an environment that has its beginning in Mid-Tertiary times [4].

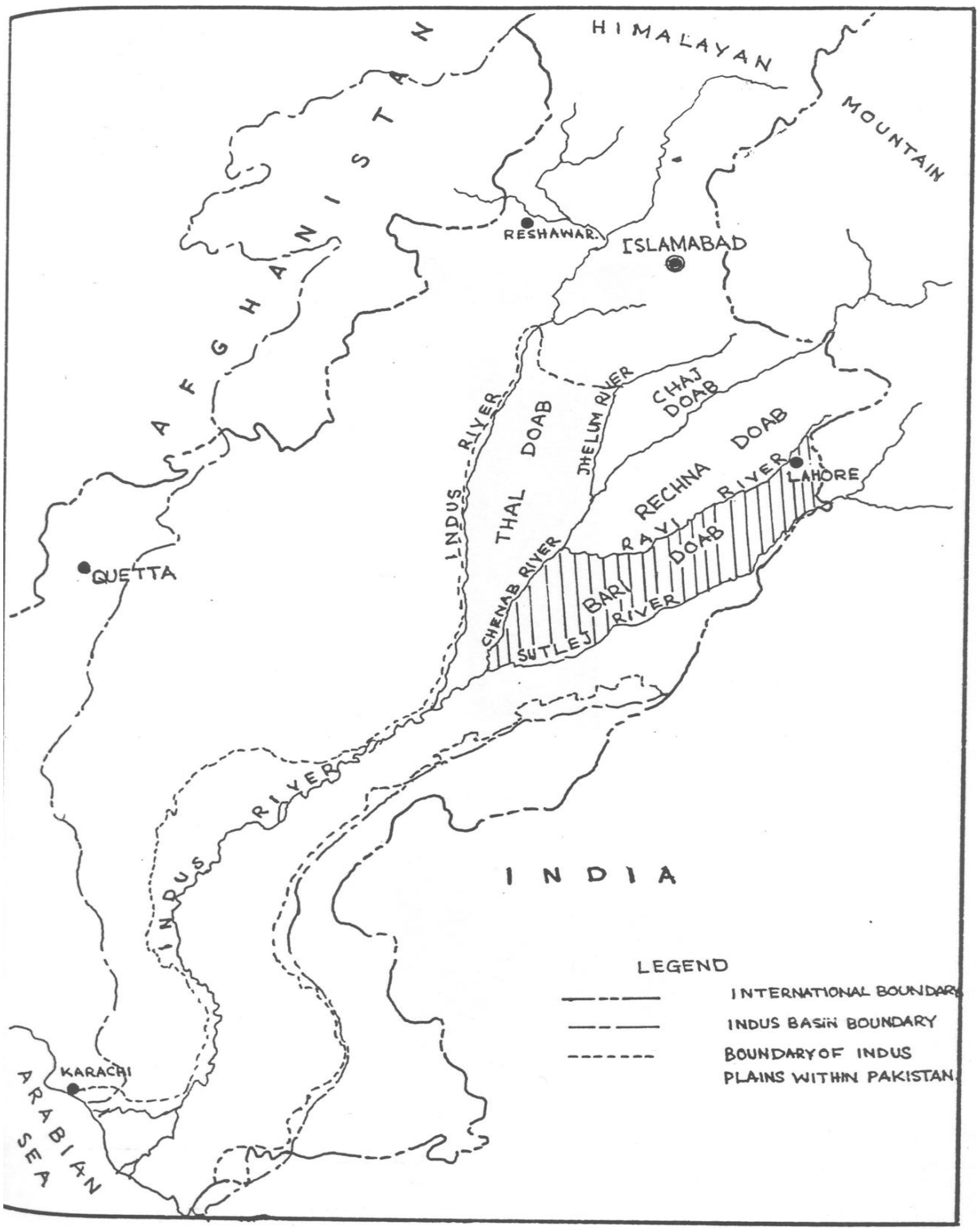


Figure 2.2: Map Showing Rivers and Doab's in Indus Plain [2]

The alluvial complex consists principally of fine to medium sand, silt and clay. Beds of gravel or coarse sand are uncommon. However pebbles of siltstone or mudstone may be found embedded in silty or clayey Sand in many places. Except for a few local lenses, few feet thick beds of hard compacted clay are rare in the area [4].

2.5.3: Surficial Geology

Lahore city is situated at an average elevation of 210 meters above mean sea level. The alluvial subsoil's are of late Pleistocene and were formed by the flood plains of river Ravi. These consist of clay, silt and sand. The thickness of clay increases with distance from the river bed [4].

2.6: Seismicity

The project site falls in the Punjab plain, which has low to moderate level of seismicity. The project region has been subjected to severe shaking in the past due to earthquakes in the Himalayas. The known main active fault of the Himalayas is the Main Boundary Thrust (MBT), which passes at a distance of about 180 km from Lahore towards northeast along the Himalayan front. Earthquakes of magnitude greater than 8 have been recorded along this fault during the past century [5].

The epicenters of low to moderate magnitude earthquakes, recorded in the Punjab plain are associated with the subsurface fractures in the basement rocks, which are concealed by thick alluvial deposits [5]. Probabilistic seismic hazard assessment recently carried out for Lahore area as part of the revision of Seismic Provisions of the Building Code of Pakistan shows that the Project area falls in Zone 2A as shown in Figure 2.3 [5]. It is, therefore, recommended that the design of the project structures should be based keeping in view the requirements of Zone 2A of Seismic Provisions of the Building Code of Pakistan (2007).

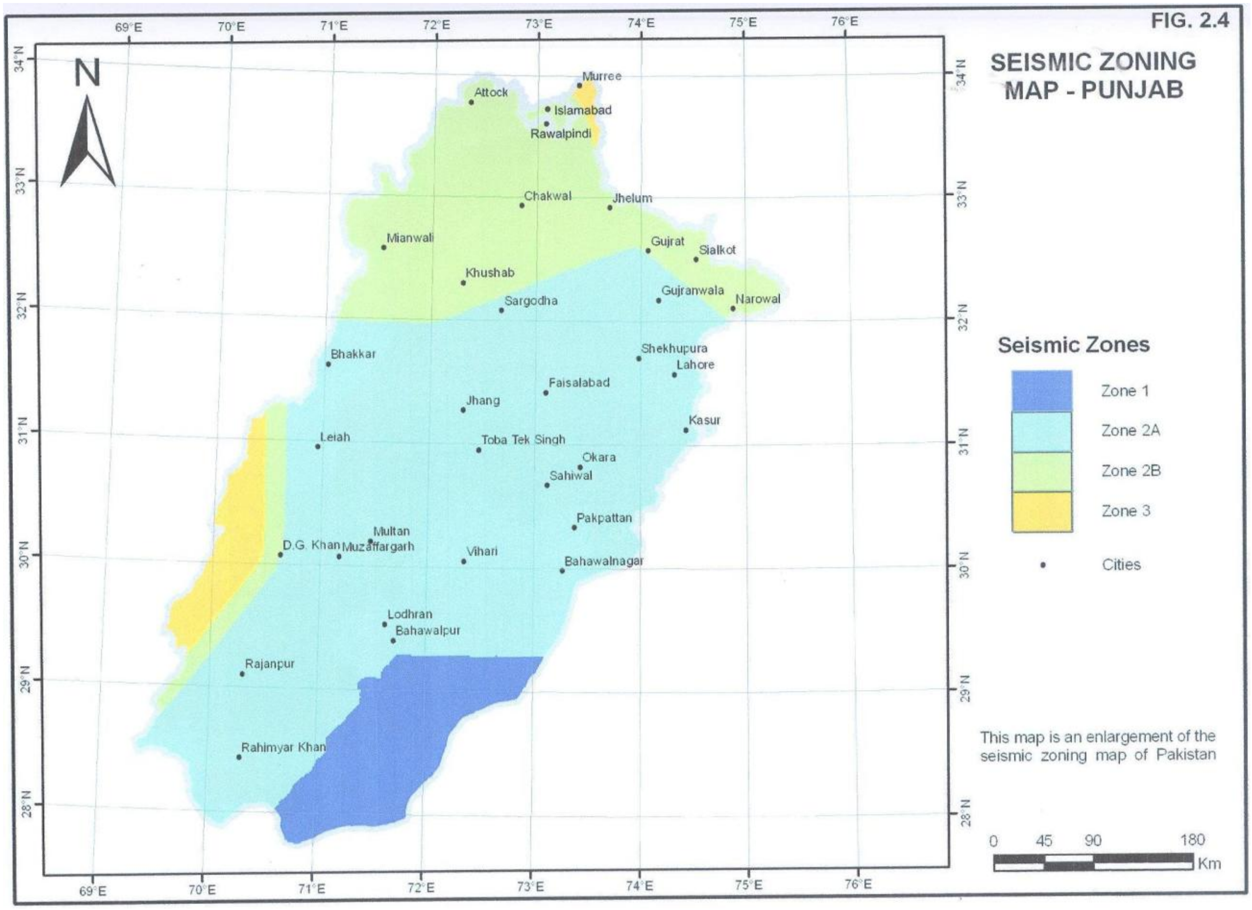


Figure 2.3: Seismic Zoning Map of Punjab [5]

CHAPTER -3

LITERATURE REVIEW

3.1: Geotechnical Investigations

Geotechnical investigations are the prerequisites to the economical design of a structure. It is performed by geotechnical engineers or engineering geologists to generally meet the following main objectives [6, 7, 8]:

- To determine the soil strata and establish a model of the soil profile.
- To determine the general geology of the site with particular reference to the main geological formations underlying the site.
- To learn more about the previous history and use of the site.
- To determine soil properties for the design of foundations for the structures.
- To determine the location of the ground water table.
- To identify possible environmental problems.
- To identify problematic soils i.e. swelling and shrinking soils.

The scope of geotechnical investigations vary from site to site depending on the nature of the project, substrata and available funds. The geotechnical investigation is carried out in two phases which are discussed below:

1) First phase: Field exploration including in situ testing
2) Second phase: Laboratory testing of disturbed and undisturbed samples retrieved during field investigations.

This chapter describes exploration techniques including field and in situ testing, laboratory testing and evaluation of sub-soil parameters / characteristics.

3.2: Field Exploration Methods

The extent of soil investigations and depth of borings should be approximately predetermined based on preliminary information and reconnaissance survey. Generally the field exploration carried out in Pakistan consists of all or some of the following tasks:

3.2.1: Test Pits and Exploratory Boreholes

The number and depth of exploration varies according to specific site conditions, type of project and cost allocated for geotechnical investigations. Boring should extend up to the depth where the stress increase due to the foundation load becomes insignificant. This value is often taken as 20% or less of the contact stress [6].

3.2.1.1: Test Pits

The most common and cheap method of shallow soil exploration in Pakistan is to excavate about 3.0 m deep open test pits. In the test pits usually field density tests are performed at varying depths. Disturbed and undisturbed (block) samples are recovered for detailed laboratory analysis and testing. Test pits are excavated using manual labour and hand digging tools. Open test pit is the best method of shallow exploration above ground water table GWT as it offers visual observation of the soil stratification, provides a direct assessment of foundation and soil conditions [6, 7].

3.2.1.2: Exploratory Boreholes

Exploratory boreholes into the soil may be made by hand tools, but more commonly mechanized tools are used. Generally the methods employed for advancing boreholes are as follows:

a) **Hand Driven and or Power Augers**

Auger Boring is the simplest method of making exploratory boreholes. Auger Boring above GWT is the best and probably the cheapest method of advancing boreholes. Hand augered holes can be drilled up to a depth of 35m but common depths are on the order of 2 to 5 m [6]. They are mostly employed for highways and small structures. Power driven augers are used for deeper boreholes. The soil samples obtained from such borings are highly disturbed. A casing is to be used in non-cohesive soils for advancing hole to prevent the soil from caving in [9].

b) Percussion Drilling

It is also known as cable tool drilling and is mostly used to advance hole through hard soil and rock. A heavy drilling bit is raised and lowered to chop the hard soil. The chopped particles are brought up by circulation of water. Percussion Drilling may require casing [10]

c) Wash boring

In case of Wash Boring a casing about 2-3 m long is driven into the ground. The soil inside the casing is removed by means of a chopping bit attached to a drilling rod. Water is forced through the drilling rod which exits at high velocity at the bottom of the chopping bit. The water and the chopped particles rise in the drill hole and overflow at the top of the casing through a T connection. The casing can be extended with additional pieces as the borehole progresses [6, 10].

d) Rotary Drilling

In case of Rotary Drilling a rapidly rotating drilling bit attached to drilling rods, cuts and grinds the soil. Rotary Drilling can be used in sand, clay and rocks. Water or drilling mud is forced down the drilling rods to the bits and returns cuttings to the surface. The drilling mud is slurry of water and bentonite. Several types of drilling bits are available for Rotary Drilling [9, 10].

3.2.2: Field Sampling

Field sampling is an important part of the exploration program. Two types of soil samples can be obtained during sub-surface exploration:

i. Disturbed and

ii. Undisturbed

i) Disturbed Samples

The disturbed soil samples are generally obtained through the split spoon sampler, used in carrying out the Standard Penetration Test (SPT). These samples are carefully examined to identify the soil types and their composition occurring at various depth horizons. Disturbed samples are also recovered using shovel and from auger cuttings. Some disturbed samples are tested in the laboratory to determine the physical properties of the subsoil. These samples cannot be used for consolidation, hydraulic conductivity or shear strength tests [6, 7 & 9].

ii) Undisturbed Samples

Extraction of undisturbed samples is a vital part of subsoil investigations. Undisturbed samples are those which are retrieved from the soil mass without disturbing the structure, density and natural moisture content. While the physical characteristics of the soil can be accessed through examination and testing of disturbed samples, the shear strength and compressibility characteristics of the soil must be determined through appropriate testing of undisturbed samples [7].

Undisturbed Samples (UDS) of cohesive soils are recovered by Denison or Shelby Tube, depending upon the consistency of the in-situ soils. Undisturbed samples of non-cohesive samples are very difficult to retrieve. They are generally obtained through thin walled Piston Samplers or Pitcher Sampler [6, 7]

iii) Ground Water Samples

The ground water level should be determined as soon as it is considered that the borehole has reached the stable water table level. The water sample is also taken for further quality tests [6].

3.3: In-Situ Testing Methods

Geotechnical investigations include in-situ testing and the results obtained from these tests are helpful in classifying the soil and determining the strength of soil. The most frequently used in-situ tests include the following:

- Standard Penetration Test
- Cone Penetration Test
- Pressure meter Test
- Dilatometer Test
- Pile Load Test

In Pakistan, in-situ testing is limited to a few tests. Generally, the Standard Penetration Test (SPT) is carried out on all kinds of strata. The results obtained from the SPT are used to determine various soil properties.

3.3.1: Standard Penetration Test

The standard penetration test developed around 1927 is currently the most popular and economical means to obtain subsurface information [6]. This test helps in assessing the in-place conditions of the sub-soil with regards to their relative density and or consistency (i.e. compactness or firmness) and at the same time provides high quality representative disturbed soil samples (DS) at testing depth. The test has been codified in ASTM D1586-92 for clayey soils and ASTM D6066-96 for sandy soils [8].

The test consists of the following activities [7]:

i. Driving the standard split - barrel sampler through a distance of 18 inches (460 mm) into the soil at the bottom of the bore using a standard force of 140 lbs (63.5 kg) free fall hammer from a height of 30 inches (762 mm).

ii. Counting the number of blows (N) to drive the sampler the last 12 inches (305 mm).This N-value is called SPT resistance of the soil.

iii. Using a 63.5 kg hammer driving mass falling from a free fall height of 30 inches (762 mm).

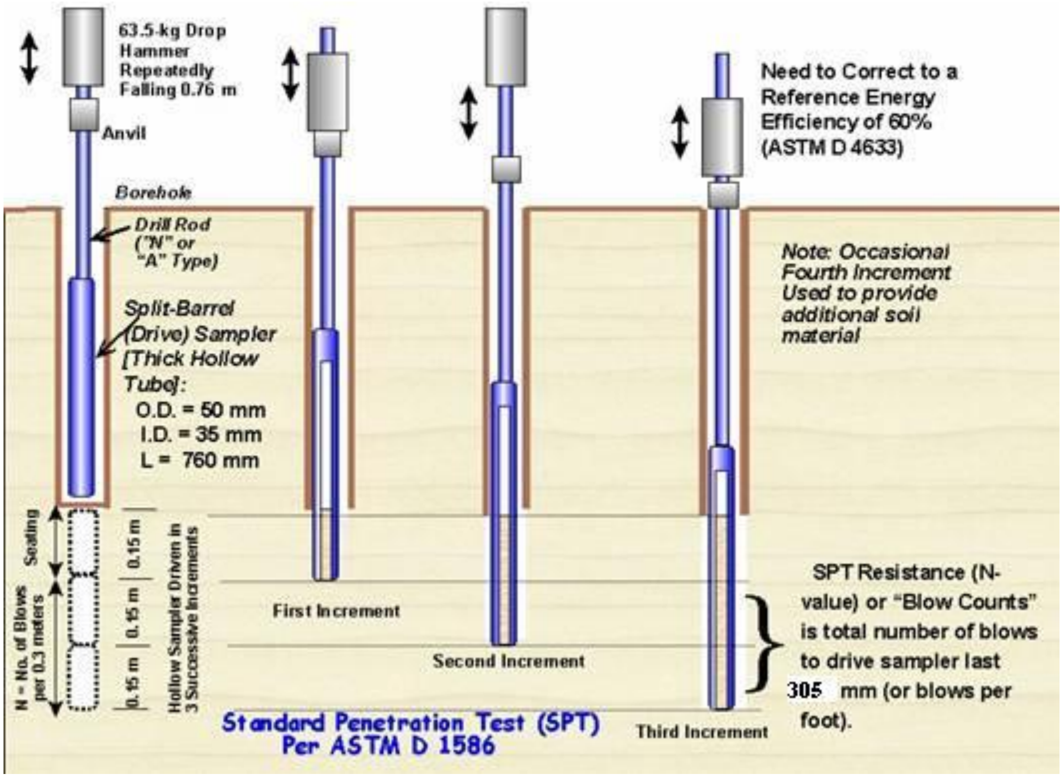


Figure 3.1: Schematic Diagram of the SPT Method [11]

The SPT apparatus has the following main components:

i) Split Spoon Sampler

A split spoon sampler, as the name implies, is designed in such a way that it could be longitudinally opened and the soil samples are collected and examined. It consists of a driving shoe to ensure a reasonable service life from driving into the soil and a barrel. The barrel consists of a tube split lengthwise with a coupling on the other end to connect a drill rod to the surface [6]. The sampler and its dimensions are shown in Figure 3.2 below:

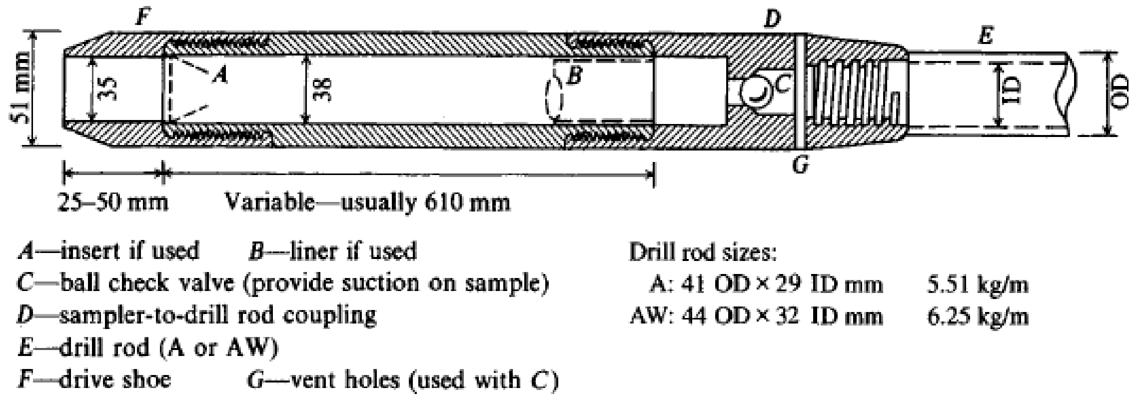


Figure 3.2: Standard Split Barrel Sampler [6]

ii) Sampler Rod

The rods used in pushing the penetration device (i.e. split spoon sampler) are stiff rods of varying length. The rods are increased to perform the test at greater depths. The rods should be straight and joints should be sufficiently tight to transmit the energy efficiently below [6].

iii) Drive Assembly

The drive assembly comprises of the following [6]:

a) A hammer weighing 63.5 kg (140 lbs).
b) A guiding assembly to ensure that the hammer has a free fall of 762 mm.
c) An anvil for transmitting the blows to the sampler rod.

3.3.1.1: Standard Penetration Test Procedure

The hole is cleaned of loose cuttings to the required depth. Whenever casing is used for advancing bore, it is not driven below the level at which the test is to be performed. A cleaned split spoon sampler is attached to the rod and lowered to the bottom of the hole. The drive assembly is connected to the rod. The sampler is seated by driving 150 mm (6 inch) into the soil with a 63.5 kg (140 lbs) hammer having a free fall of 762 mm and numbers of blows are recorded. The sampler is then driven 305 mm (12 inch), or until 50 blows were applied by 63.5 kg hammer falling 762 mm. The numbers of blows for each 150 mm (6 inches) of penetration are recorded but the number of blows for the first 6 inches of penetration is ignored. The total blows required for 305 mm (12 inches) penetration are called the penetration resistance and are denoted by N [6, 7].

The sampler is then withdrawn and opened. Samples are examined and some of them are properly labeled and placed in plastic jars or polythene sheets for laboratory testing. The field report for SPT performed generally consists of the following details:

a) The penetration resistance i.e. number of blows (N).
b) The depth at which penetration resistance is measured.
c) Number of blows for the first 150 mm (6 inches)

The test shows refusal and is halted if [6]:

a) 50 blows are required for any 150 mm penetration increment.
b) 100 blows are obtained (to drive the required 305 mm)
c) 10 successive blows produce no advance of sampler

SPT resistance is reliable for cohesion less soils but provides crude estimates for cohesive soils.

3.3.1.2: Overburden Pressure Correction

Corrections for overburden pressure are generally applied to the SPT-N values. All field SPT-N values after 1974 are corrected using the following equation [6].

$$C_N = \left(\frac{95.76}{p'_o} \right)^{1/2} \qquad \text{……. (3.1)}$$

$$N_C = (C_N)N \qquad \text{……. (3.2)}$$

Where,

p_o' = Overburden pressure in kN/m^2

C_N = Adjustment for effective overburden pressure p'_o

N = Uncorrected SPT-N values

N_c = Corrected SPT-N values

3.3.1.3: Determination of N'_{70}

The equation for determining N'_{70} is [6]:

$$N'_{70} = C_N \times N \times \eta_1 \times \eta_2 \times \eta_3 \times \eta_4 \qquad \text{……… (3.3)}$$

Where,

η_i = Adjustment Factors from Table 3.1

N'_{70}=Adjusted N value when E_{rb} is equal to 70

E_{rb} = Standard Energy Ratio

C_N = Adjustment for effective overburden pressure p'_o

Table 3.1: Factors η_i for N'_{70} [6]

Hammer for η_1					Remarks
	Average energy ratio E_r				
	Donut		Safety		
Country	R-P	Trip	R-P	Trip/Auto	R-P = Rope-pulley or cathead $\eta_1 = E_r/E_{rb} = E_r/70$ For U.S. trip/auto w/E_r = 80 η_1 = 80/70 = 1.14
United States/ North America	45	—	70–80	80–100	
Japan	67	78	—	—	
United Kingdom	—	—	50	60	
China	50	60	—	—	
Rod length correction η_2					
	Length	> 10 m	η_2 = 1.00		N is too high for L < 10 m
		6–10	= 0.95		
		4–6	= 0.85		
		0–4	= 0.75		
Sampler correction η_3					
		Without liner	η_3 = 1.00		Base value
	With liner:	Dense sand, clay	= 0.80		N is too high with liner
		Loose sand	= 0.90		
Borehole diameter correction η_4					
	Hole diameter:†	60–120 mm	η_4 = 1.00		Base value; N is too small when there is an oversize hole
		150 mm	= 1.05		
		200 mm	= 1.15		

3.3.1.4: Determination of N_{55}

N_{55} is the standard penetration value corresponding to an energy ratio (E_R) equal to 55. The energy ratio can be defined as:

$$E_R = \frac{Actual\ \ Hammer\ Energy\ to\ Sampler, E_a}{Input\ \ Energy, E_{in}} \times 100 \qquad \text{……… (3.4)}$$

$$E_{70} \times N'_{70} = E_{55} \times N_{55} \qquad \text{……… (3.5)}$$

Where,

$E_{70} = 70$

$E_{55} = 55$

$N'_{70} = C_N \times N \times \eta_1 \times \eta_2 \times \eta_3 \times \eta_4$

Therefore,

$$N_{55} = (70/55) \times N'_{70} \qquad \text{……… (3.6)}$$

3.4: SPT Correlations

Standard Penetration Test is the most commonly performed field test throughout the world. It provides an indirect method of determining the soil properties at various depths besides obtaining disturbed soil samples. It has been established that it has fairly reliable application to granular i.e cohesion less soil. However, the SPT results for cohesive soils are not reliable as they are influenced more by moisture content and clay mineral characteristics as compared to cohesionless soils.

3.4.1: Determination of Unit Weight and Shear Strength Parameters

The result of Standard Penetration Tests have been correlated with unit weight, relative density, angle of internal friction, and undrained compressive strength and are given in Table 3.2.

Table 3.2 (a): SPT Correlations for Granular Soils [6]

GRANULAR SOILS					
Description	Very Loose	Loose	Medium	Dense	Very Dense
Relative Density, D_r	0–0.15	0.15–0.35	0.35–0.65	0.65–0.85	0.85–1.00
Standard Penetration Test value, N	0–4	5–10	11–30	31–50	51–UP
Approximate angle of internal friction, ϕ (degree)	25–28	28–30	30–35	35–40	38–43
Approximate range of moist unit weight, γ (pcf*)	70–100	90–115	110–130	110–140	130–150
Submerged unit weight, γ_{sub} (pcf*)	60	55–65	60–70	65–85	75

Table 3.2 (b): SPT Correlations for Cohesive Soils [6]

COHESIVE SOILS						
Description	Very Soft	Soft	Firm	Stiff	Very Stiff	Hard
Unconfined compressive strength, q_u (tsf*)	0–0.25	0.25–0.5	0.5–1.0	1.0–2.0	2.0–4.0	4.0–UP
Standard Penetration Test value, N	0–2	3–4	5–8	9–16	17–32	33-UP
Approx. range of saturated unit weight, γ_{sat} (pcf*)	100–120		100–130	120–140		130^+

* 1 tsf = 95.76 kPa

*1 pcf = 0.00013 kN/m^3

3.4.2 Determination of Modulus of Elasticity

SPT-N values can be used to determine the modulus of elasticity (Es) for various soil types. The correlations are given in the Table 3.3. E_s values obtained from these correlations are in kilopascals (kPa).

Table 3.3: Equations for E_s by SPT and CPT Methods [6]

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $= 7000\sqrt{N}$ $= 6000N$ — — — ‡$E_s = (15\,000 \text{ to } 22\,000) \cdot \ln N$	$E_s = (2 \text{ to } 4)q_u$ $= 8000\sqrt{q_c}$ — — — $E_s = 1.2(3D_r^2 + 2)q_c$ *$E_s = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	$E_s = Fq_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$
Sands, all (norm. consol.)	¶$E_s = (2600 \text{ to } 2900)N$	
Sand (overconsolidated)	†$E_s = 40\,000 + 1050N$ $E_{s(\mathrm{OCR})} \approx E_{s,\mathrm{nc}}\sqrt{\mathrm{OCR}}$	$E_s = (6 \text{ to } 30)q_c$
Gravelly sand	$E_s = 1200(N + 6)$ $= 600(N + 6) \quad N \le 15$ $= 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = (3 \text{ to } 6)q_c$
Silts, sandy silt, or clayey silt	$E_s = 300(N + 6)$ If $q_c < 2500$ kPa use $\quad$ $^{\$}E'_s = 2.5q_c$ $2500 < q_c < 5000$ use $\quad E'_s = 4q_c + 5000$ where $E'_s = \text{constrained modulus} = \dfrac{E_s(1-\mu)}{(1+\mu)(1-2\mu)} = \dfrac{1}{m_v}$	$E_s = (1 \text{ to } 2)q_c$
Soft clay or clayey silt		$E_s = (3 \text{ to } 8)q_c$

3.5: Laboratory Testing

Laboratory tests are performed on carefully selected representative sub-soil and ground water samples recovered from the site during the site exploration process. Laboratory testing is an essential component of an exploration program to evaluate physical, engineering and chemical characteristics of the strata and GWT encountered at the project site. This section describes briefly the various laboratory tests, their utility and importance of various sub-soil parameters towards the design of foundation. Usually the following laboratory tests are carried out:

i) Tests for evaluation of physical characteristics of soils:

a) Grain size analysis, ASTM D 421, 422 & BS 1377 Part 2
b) Bulk & dry density, ASTM D 2216 & BS 1377 Part 2
c) Atterberg's limits (LL, PL and PI), ASTM D 4318 & BS 1377 Part 2
d) Specific gravity (Gs) ASTM D 854

ii) Tests for evaluation of engineering characteristics of soils:

a) Shear strength characteristics (c, φ, q_u)
   - Direct shear ASTM D 3080
   - Triaxial Compression ASTM D 2850.
   - Unconfined compression ASTM D 2166 & BS 1377 Part 7.

b) Tests for evaluation of compaction characteristics.
   - Standard Proctor test ASTM D 698
   - Modified Proctor test, ASTM D 1557
   - CBR test, ASTM D 1833

c) Compression characteristics tests
   - Consolidation test, ASTM D 2435

d) Tests for permeability
   - Permeability test, ASTM D 2434

Detailed procedures of performing these tests can be found from ASTM standards or testing manuals.

3.6: Statistical Evaluation

Statistical evaluation is critical for realistic estimates of the variability of design soil properties. Soil properties vary every few feet and geotechnical variability can be due to [12]:

- Soil variation
- Measurement errors
- Field or laboratory measurements that are transformed into design soil properties using empirical or other correlation models.

The variation in data can be determined by calculating the coefficient of variation (COV) for various soil properties. Advantages of determining COV and performing statistical evaluation are discussed as follows [12]:

1) Help engineers develop a physical feel for the probable range of variability inherent in the estimation of common design soil properties.

2) Atypical geotechnical variability's can be identified which in turn might lead to additional site investigation or improvement in the quality of the measurements.

3.6.1: Determination of COV

Coefficient of variation (COV) is a standardized measure of dispersion of a probability distribution or frequency distribution [13]. It is defined as the ratio of the standard deviation σ to the mean μ. Coefficient of variation (COV) can be determined using the following formulas [13]:

Standard Deviation, $\sigma = \sqrt{\frac{(x-\bar{x})^2}{n}}$ ……… (3.7)

Where,

x = observed values of the sample items

$\bar{x}$ = mean value of the sample items

n = total number of sample items

COV, $\mu = \sigma / \bar{x}$ ……… (3.8)

3.7: Deep Excavations

An excavation which is more than 15 ft or 4.5 m in soil or rock is generally termed as deep excavations. Careful design and proper planning is required to carry out deep excavation in urban areas. The decision of the type of retaining and support system required is an important part of deep excavation design. The important factors in the design and selection of appropriate retaining or support systems are time, cost and importance of structure. Excavations are shored or supported for a number of reasons which are discussed below [14]:

1) To limit the amount of over excavation required when sloping sides of the cut.

2) To protect the personnel who enter and work within the excavation.

3) To protect adjacent property such as buildings, utilities or property.

4) To minimize the excavation and therefore maximize the usable property around the excavation.

3.8: Deep Excavation Retaining Systems and Their Types

In Pakistan, especially in Lahore, Soldier Piles and lagging walls are being used as deep excavation retaining systems.

3.8.1: Soldier Piles and Lagging

Soldier Pile and lagging is the most common and the oldest shoring solution for urban construction. These walls have been successfully used in metropolitan cities like New York, Berlin and London. Soldier piles are vertical steel or concrete elements which define the perimeter of the excavation. They are spaced at 5-10 feet on center and stand at attention like soldiers, hence their name as shown in Figure [14, 15].

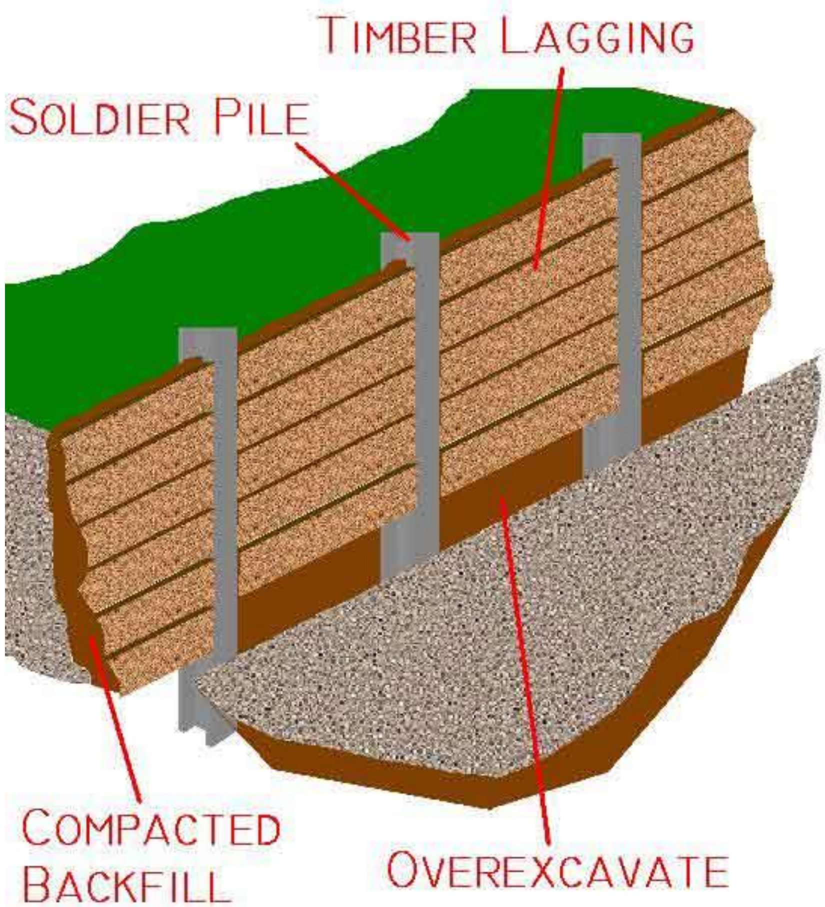


Figure 3.3: Soldier Piles and Lagging [16]

3.8.1.1: Types of Soldier Piles

Soldier piles can be drilled and concreted, driven, churn drilled or wet set in soil cement. Most commonly soldier piles are drilled or driven.

1) **Driven Soldier Piling**

These are usually H sections although when driving stresses are light then some wide flanged sections can be used [14, 15]. Driven soldier piles often reach a position differing from their intended location. The support system and wall design must therefore be able to accommodate this practical misalignment from design location [17].

2) **Drilled and Concreted Soldier Piling**

These are installed by drilling a hole of sufficient diameter to permit the introduction of a steel wide flange section. There should be enough space in the hole to overcome any variations from vertical. Once the hole is drilled, a steel wide flange section is introduced into the hole and hung to achieve verticality [14].

The toe of the soldier pile is always below the base of the excavation. It is backfilled with either structural concrete or with a lean sand grout such as CDF (Controlled Density Fill). The part of the drilled shaft above the toe is backfilled with lean sand grout. Typical soldier piles used in this application are 8 to 24 inch wide flange sections [14].

3.8.1.2: Advantages and Disadvantages

Following are the advantages of using Soldier Piles:

- They are the cheapest support system as compared to other retaining walls [15].
- They are easy and fast to construct [15].
- They can be used in relatively stiff soils that have underlying slip failure planes. They can also be designed to penetrate sufficient depth to intersect and strengthen slip planes [14].

Following are the disadvantages of using Soldier Piles [14]:

- They are primarily limited to temporary construction.
- They cannot be used in high water table conditions without extensive dewatering.

- They are not suitable in soils which exhibit basal instability as the lagging only extends to the base of the excavation.

3.9: Deep Excavation Support Systems

These systems are used to support lateral loads. In Lahore, tie back anchors are used for lateral support.

3.9.1: Tie Back Anchors

Tieback anchors are commonly used for temporary wall support on major excavation projects [16]. Tieback anchors secure the wall to a soil or rock mass which is behind that portion of the soil adjacent to the wall which is at risk of moving. A well designed tieback should be technically feasible, economical and safe. Tieback anchors should be installed in areas with reasonable soil strength and resistance [14].

3.9.1.1: Types of Anchors

Many methods of anchoring are available and the most common ones are discussed below:

1) **Mechanical Anchors**

   Two commonly used commercial anchors are helical and manta ray anchors. Helical anchors are a series of steel helical plates welded at intervals to a steel rod. The anchor is rotated into the soil with the helices screwing themselves into the ground. Manta ray anchors are steel plates which are attached to a rod. The plate is advanced into the ground by impact driving [14].

2) **Drilled and Grouted Anchors**

   There are two types of drilled and grouted anchors.

   Single Stage Anchors

   These anchors mobilize the shear strength of the soil by friction along their length. The anchors consist of a barrel anchorage located in a bearing layer after construction which is tensioned at the front face of the wall. The part of the anchor that transfers the force to the surrounding soil is called the “fixed length” or “bond zone”. The “free length” or no-load zone” transmits forces from the fixed length through the anchor head to the pile wall [14, 17, 18]. The anchor develops its capacity in the bond zone also called the anchor zone. The top of the bond zone for all strands is the bottom of the no-load zone so that all strands begin developing their capacity at the same depth in the drilled hole [14].

Multi Stage Anchors

These anchors also develop their capacity by mobilizing the soil shear strength. Some movement is necessary in order to mobilize their shear capacity. The entire load on the anchor is first brought to bear at the top of the bond zone because the bar or strand used for anchors elongates as it is stresses. As the anchor elongates, the stresses are distributed uniformly over the bond length of the anchor [14].

If the soil where the anchor is engaging is soft or the load is extremely high then the calculated anchor lengths can be very long. As mentioned above, the entire load is first brought to bear at the top of the bond zone. In some cases, the movement required to distribute the load over the entire bond length may be so great that the soils at the top of the bond zone will fail. This can lead to a progressive failure of the anchor. In order to overcome this problem, multi-stage anchors are used. In case of multi stage anchors, the top of the bond zone of each strand is in a different place and therefore, the onsets of bond stresses are more evenly distributed [14].

3.9.1.2: Installation of Anchors

Tie back installation follow the sequence shown in Figure 3.4 to minimize the soil movements and speed up the excavation construction.

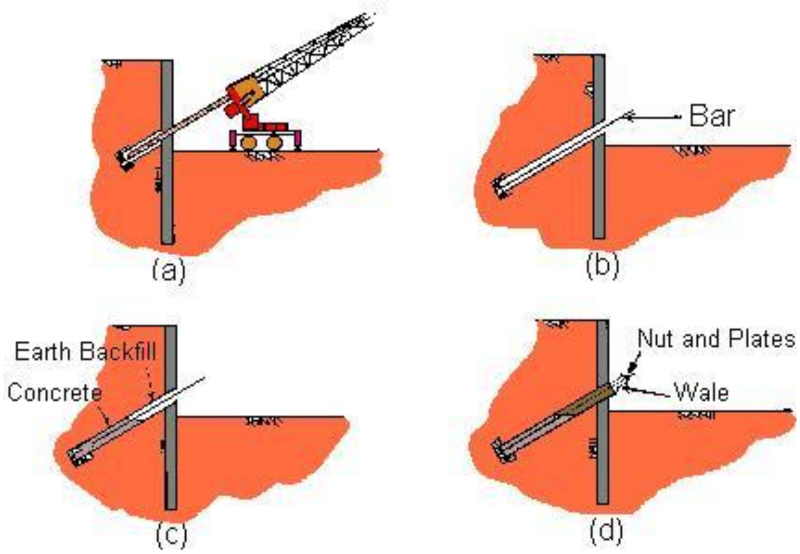


Figure 3.4: Installation steps for a tieback: (A) Drilling of Hole, (B) Bar Placed in Hole, (C) Grout Poured for Anchor Connection, (D) Installation of Nuts and Plates to Connect Anchor to Wall [16].

Anchor hole drilling should be performed using a method which permits reasonably accurate location control and provides the required holding capacity [17]. Holes can be drilled using auger rigs or continuous flight augers. Anchors can be installed by the following two techniques:

1) Anchors can be installed by hollow stemmed continuous flight augers in a method called auger casting. An anchor tendon is placed inside the auger and the auger drilled into the ground. Once the auger reaches the design depth, grout is forced down the hollow stem of the auger and the auger is withdrawn leaving the grout and tendon in place. The hole size ranges from 8 to 30 inch in diameter [14]. Excessive ground disturbance can be caused by using auger equipment to drill at shallow angles in cohesionless soils [17].

2) Anchors can also be installed by rotary techniques utilizing the air or water as a flushing medium. The hole is drilled using drag bits or rotary bits, the drill string is withdrawn and a tendon set and grouted in place. The hole size ranges from 4-10 inch in diameter [14]. Rotary percussion drilling methods can produce excessive ground disturbance in sands [17].

3.9.1.3: Grouting of Anchors

Grouting is usually performed with neat cement grouts. Bagged or bulk cement is mixed with water on site at a rate of 5-6 gallons per sack of cement. The grout is then pumped down the drill hole through 1 inch diameter lines. Grout is poured into dry holes or tremied into wet holes. The anchor tendon is usually placed before grouting but in some cases to achieve higher bond capacity, tendon is installed after grouting (this is called a wet setting) [14].

3.9.1.4: Stressing of Anchors/ Proof Testing

Proof testing is performed by staged application of load to tieback anchor with hydraulic jack and pump until reaching test load [17, 18]. The test load is generally taken as 1.33 times the design load [19]. The load is then reduced to a lock-off load which is usually 75-100% of the design load.

Anchors are stress or proof tested for the following reasons:

- To verify, ensuring that the design assumptions and techniques are correct [14].
- To ensure that the tieback has adequate capacity to bear the loads [17, 18].
- To pre-stress the tendon and support system [16].

3.10: Estimation of Pile Capacity using ASD Method

In Pakistan the design of foundations has traditionally been based on the Allowable Stress Design (ASD) method. The results of static analyses yield an ultimate pile capacity based on geotechnical considerations. The allowable geotechnical pile capacity (geotechnical pile design load) can be determined by dividing the geotechnical ultimate pile capacity by an appropriate factor of safety as follows:

$$Q_{all} = (Q_p/FOS_1) + (Q_s/FOS_2) \quad \text{……… (3.9)}$$

Where:

Q_{all} = Allowable geotechnical pile capacity

Q_p = Pile tip capacity

Q_s = Shaft Capacity

FOS_1 = Factor of Safety for Base Resistance

FOS_2 = Factor of Safety for Shaft Resistance

3.10.1: Estimation of Pile Capacity in Sand using ASD Method

Meyerhoff in 1976 recommended the following correlation for the axial capacity of a single pile in granular soil [6].

$$R = mNA_t + n\overline{N}DA_s \qquad \text{……… (3.10)}$$

Where,

R = Pile capacity (N)

$\overline{N}$ = Average SPT-N value along the pile

D = Pile embedment length (m)

N = SPT-N at the pile tip obtained by averaging the blows over a length of 6-10B above the pile tip and 2-4B below the pile tip

A_s = area of pile shaft (m^2)

A_t = area of pile tip (m^2)

m = 400 x 10^3 for driven piles

120 x10^3 for bored piles

n = 2x10^3 for driven piles

1x10^3 for bored piles

3.10.2: Estimation of Pile Capacity in Clay using ASD Method

Pile capacity in clays can be determined using the following equation [6]:

$$Q_{total} = A_p c_u N_c^{\ *} + \sum_{L=0}^{L=L_1} \alpha^* c_u \, p \Delta L \qquad \text{……… (3.11)}$$

Where,

Q_{total} = Total pile capacity (N)

α^* = 0.4

$p = \pi D_s$

ΔL = Length of each layer (m)

A_p = area of pile tip (m^2)

D_s = diameter of shaft (m)

L = length of shaft (m)

c_u = undrained cohesion (kPa)

P_a = atmospheric pressure (Pa)

N_c^* = 9 (if $c_u/p_a > 1$)

3.10.3: Factor of Safety

The factor of safety to be used in the static formulas depends on many factors such as the following:

- Reliability of soil parameters used for calculations
- The manner in which load is transferred to the soil
- The importance of the structure
- Allowable total and differential settlement tolerated by the structure

Table 3.4 gives the values of Factor of Safety generally used in the field.

Table 3.4: Factors of Safety for Static Formula for Piles [6]

CASE	FACTOR OF SAFETY
For Total Capacity	2.5
For Shaft Resistance	1.5
For Base Resistance	3.0

In Pakistan, quality control is an important issue during the construction of the piles. To hedge against substandard construction quality, a higher factor of safety is used (generally FOS = 3 to calculate allowable capacity)

CHAPTER -4

DATA ANALYSIS AND DEVELOPMENT OF ZONES

4.1: General

Geotechnical characterization for foundation design is critical during preliminary planning, designing and feasibility studies of various engineering projects. In several developed countries of the world, proper guidelines are readily available to practicing engineers and geologists in the form of maps and local building codes for geotechnical design purposes. Preparation of such guidelines would be helpful for the practicing engineers with considerable savings in time and expense in developing countries.

This chapter describes the data analysis procedure and its resulting outcome. Geotechnical data was derived from sixty sites, details of which are provided in Appendix-A. It was ensured that reliable geotechnical data was collected from specialized geotechnical consultants and contractors. The location of sites has been marked on the map of Lahore using GIS software as shown in Figure 4.1. The latitude and longitude coordinates for the sites were added to ArcMap software to plot the site locations on a map of Lahore.

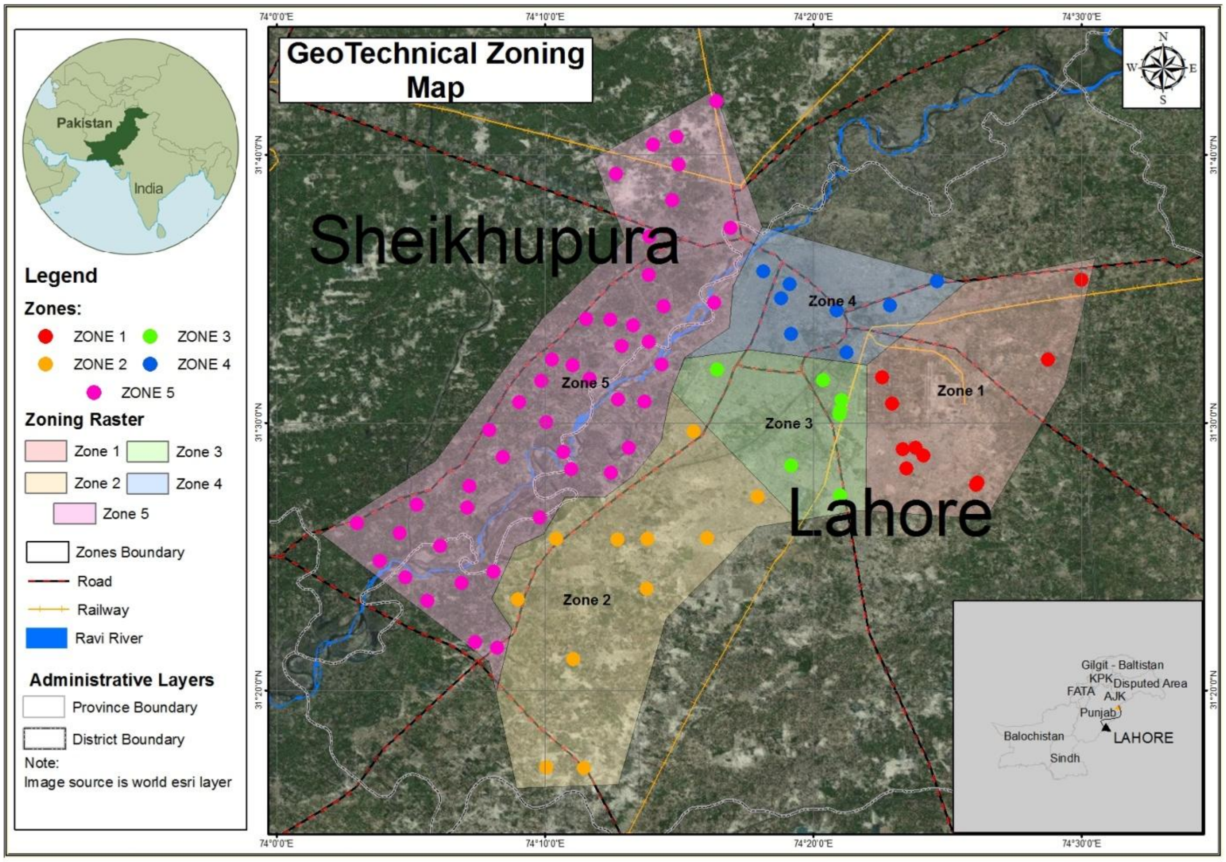


Figure 4.1: Visual Representation of Site Locations on a Map of Lahore

Geotechnical data collected from site investigation reports mainly includes information regarding soil stratigraphy, sub-soil characteristics of each stratum, ground water table position, SPT-N values and laboratory test data.

4.2: Data Collection and Analysis

The interpretation and analysis of data has resulted in the development of the following maps and profiles:

i. Map of Lahore visually representing the site locations.

ii. Preparation of Soil Log profile for all zones. This profile presents soil type and range of N values at a regular interval of 2 m.

iii. Preparation of Generalized Soil Properties profile for all zones. This profile presents range of shear strength parameters, Liquid Limit (LL), Plasticity Index (PI) and Elastic Modulus (Es) values at a regular interval of 2 m.

4.3: Division of Lahore into Zones

The first step in the geotechnical characterization process is to divide an area into zones. The division of an area into homogeneous sectors is done with respect to a certain criteria. For this study Lahore city has been divided into zones based on the geography of the region. The top thirty meter soil stratum has been considered for the study. Five zones have been developed on the basis of the geography of the region as shown in Figure. 4.1.

Zone-1

Zone 1 mainly encompasses a modern, newly developed housing society in Lahore called Defence Housing Authority (DHA) and its surrounding area. The main areas enclosed in this zone are Defence Housing Society (DHA), Walton Cantt, New and old airport terminals, Bedian Road, Barki Road, Paragon City, Sarwar Road, Defence Road and Attari Saroba. Data from a total of 15 sites were considered for this zone.

Zone-2

Lahore is now developing towards the south and Zone 2 includes the newly developed housing societies in Lahore. The main areas enclosed in this zone are WAPDA Town, EME-DHA, Valencia, Bahria Town, Izmir Town, Sundar, Chung, Bund Road, Raiwind, Kot Lakhpat, Multan Road and Lake City. A total of 22 sites were considered for this zone.

Zone-3

Zone 3 mainly encompasses a wide spread housing society in Lahore called Gulberg and its surrounding area. The main areas enclosed in this zone are Gulberg 2, Gulberg 3, M.M.Alam Road, Model Town, Township, Allama Iqbal Town, Johar Town, Bund Road, Hussain Chowk, Maratab Ali Road, Ferozepur Road and Upper Mall Road. Geotechnical data for this zone was gathered from 12 sites.

Zone-4

This zone includes the area within the interior city of Lahore. The main areas enclosed in this zone are Shad Bagh, Walled City, Baghbanpura, Saddar, Gulshan-e-Ravi, Lohari Gate, Ravi Road, Shalimar Link Road, Mall Road and Shalimar Town. For analysis, 7 sites were considered for this zone.

Zone-5

This zone includes areas around river Ravi. The main areas enclosed in this zone are Faizpur, Shahdara, G.T Road, Sharqpur, Karol, Babu Sabu and Mohlanwal. For this zone, data was obtained 6 sites.

4.4: Data Compilation

The data for all the sites was complied to prepare soil profiles. The main objective of preparing the profiles was to present the soil data in a form that could be easily used by engineers and geologists in the design process. Two soil profiles were prepared for each zone i.e. the Soil Log Profile and the Generalized Soil Properties Profile.

4.4.1: Soil Log Profile

The soil log profile provides information regarding the SPT-N values and soil types at regular 2 m intervals. The data compilation process is summarized below:

1) N values were taken at 2 m intervals for all the boreholes drilled at a site.

2) The N values obtained from all the boreholes drilled at a site were averaged to get one representative value for a site. This process was performed for all the sites in a zone.

3) These N values were then used to establish ranges.

4) Soil type varies every few feet and there was more than one soil type throughout the zone for a particular depth. To overcome this problem, borehole logs were considered for all sites in a zone.

5) All possible soil types were considered at regular 2 m intervals and presented on the profiles.

Groundwater table variation throughout a zone was also presented on the soil profiles. The soil log profiles for all five zones are attached in Appendix A.

4.4.2: Generalized Soil Properties Profile

The generalized soil properties profile provides information regarding various soil properties and their ranges. The data compilation process is summarized below:

1) The soil properties i.e. shear strength parameters (cohesion, c and friction angle, φ), liquid limit (LL), plasticity index (PI) and moduli of elasticity (Es) of soil were taken at 2 m intervals for all the boreholes drilled at a site.

2) The soil properties were averaged to get one representative value for a site. This process was performed for all the sites in a zone.

3) The soil properties obtained from all the sites in a zone were then used to establish ranges. This process was carried out for all five zones.

4) The ranges of soil properties are shown on soil profiles.

The generalized soil properties profiles for all five zones are attached in Appendix A.

4.4.3: Soil Properties in Generalized Soil Properties Profile

The soil properties considered for the generalized soil properties profile includes the shear strength parameters i.e c and φ, liquid limit (LL), plasticity index (PI) and modulus of elasticity (E_s) of soil. These soil properties were obtained from various test results and correlation and the details of which are discussed below:

- **Shear Strength Parameters**: The shear strength parameters were obtained from the results of Direct Shear Test performed on samples obtained from sites. In Pakistan, Direct Shear Tests are performed on samples which are at in-situ moisture content. Therefore, it is not possible to say whether or not the tests were performed in drained or undrained conditions. For this project it has been assumed that the shear strength parameters are undrained.

- **Liquid Limit and Plasticity Index**: Atterberg Limit Tests were performed on the soil samples obtained from sites to determine the LL and PI values.

- **Modulus of Elasticity (E_s)**: The E_s of soil was determined by using SPT-N correlations. The details of these correlations are discussed in Section 3.4.2 of this report.

- **Unit Weight (ɤ)**: The unit weight of soil was determined by using SPT-N correlations. The details of these correlations are discussed in Section 3.4.1 of this report.

4.5: Statistical Evaluation

Statistical evaluation was performed for all five zones. The objectives of performing statistical evaluation for the data are as follows:

- To determine the accuracy of the data i.e. soil properties and N values.
- To determine the variation of soil data throughout a zone.

COV were determined for friction angle and SPT-N values. The COV values were determined for soil layers with the same soil type. The calculated COV values were compared with the acceptable range of COV values given by EPRI. EPRI is the Electrical Power Research Institute and has established COV ranges by taking sites according to group type and test type and calculating soil properties [12].

The statistical evaluation tables for all five zones are attached in Appendix A. Statistical evaluation calculations for Zone 1 are also attached in Appendix A. From the tables it is clear that the COV values of friction angle and SPT-N for all five zones fall within the acceptable range of COV's given by EPRI. The SPT-N values from zone 1 have high COV values for the soil layer that extends up to a depth of 2 m below the GSL. The N values in the first 2 m vary throughout the zone as the soil strata is composed of gravel and stones. Therefore, the COV values are high and a conservative approach should be adopted while using these N values.

CHAPTER -5

DESIGN OF DEEP EXCAVATIONS SUPPORT SYSTEMS

5.1: General

Lahore is the second largest metropolitan area in Pakistan It has been a centre of business, trade and politics since its inception. Therefore, the price of land is increasing and builders are looking to save money by developing multiple basements. Deep excavation support systems are required for such developments and these are a new concept in Lahore. The second part of the project deals with deep excavation support systems and design methods used in Lahore.

In Lahore an anchor-pile system is generally used as deep excavation support system. Anchor-pile systems can be designed according to the methods given in the following codes i.e.

- Canadian Foundation Engineering Manual (2007)
- Federal Highway Administration (FHWA), US Department of Transportation Geotechnical Engineering Circular No.4, Ground Anchors and Anchored System (1999) and Soil Mechanics
- Naval Facilities Command Engineering (NAVFAC), US Army Corps of Engineers (1986 & 1997)

The above mentioned codes are generally used for design of deep excavation support systems in Pakistan, especially Lahore. This chapter will outline the main features of design of deep excavation support system according to FHWA and Canadian Method.

5.2: Federal Highway Administrations (FHWA), US Department of Transportation Approach

The FHWA approach focuses on procedures that should be addressed in designing specific components of an anchored wall. As part of the overall design, the relationship between type of ground, selection of ground anchors, type of soldier beam, connections (ground anchor/soldier beam, soldier beam/permanent facing), and type of facing must be considered. Detailed information on these considerations is not included in the FHWA method as decisions related to these considerations are typically made by the contractor. The engineer, however, should ensure that the specific components and combinations of components used for the anchored system are consistent with all performance requirements [18].

Design of deep excavation support systems according to FHWA Method has been discussed below in detail. The following excerpts are from the Federal Highway Administration (FHWA), US Department of Transportation Geotechnical Engineering Circular No.4, Ground Anchors and Anchored System (1999) and Soil Mechanics

5.2.1: Main Features

Typical design steps for an anchored wall are as follows [18]:

i. Establish project requirements including all geometry, external loading conditions (temporary and/or permanent, seismic, etc.), performance criteria and construction constraints.

ii. Evaluate site subsurface conditions and relevant properties of in situ soil and/or rock.

iii. Evaluate design properties, establish design factors of safety, and select level of corrosion protection.

iv. Select lateral earth pressure distribution acting on back of wall for final wall height. Add appropriate water, surcharge, and seismic pressures and evaluate total lateral pressure.

v. Calculate horizontal ground anchor loads and wall bending moments.

vi. Evaluate required anchor inclination based on right-of-way limitations, location of appropriate anchoring strata, and location of underground structures.

vii. Resolve each horizontal anchor load into a vertical force component and a force along the anchor.

viii. Evaluate horizontal spacing of anchors based on wall type and calculate individual anchor loads.

ix. Select type of ground anchor.

x. Evaluate vertical and lateral capacity of wall below excavation subgrade. Revise wall section if necessary.

xi. Evaluate internal and external stability of anchored system. Revise ground anchor geometry if necessary.

xii. Estimate maximum lateral wall movements and ground surface settlements. Revise design if necessary.

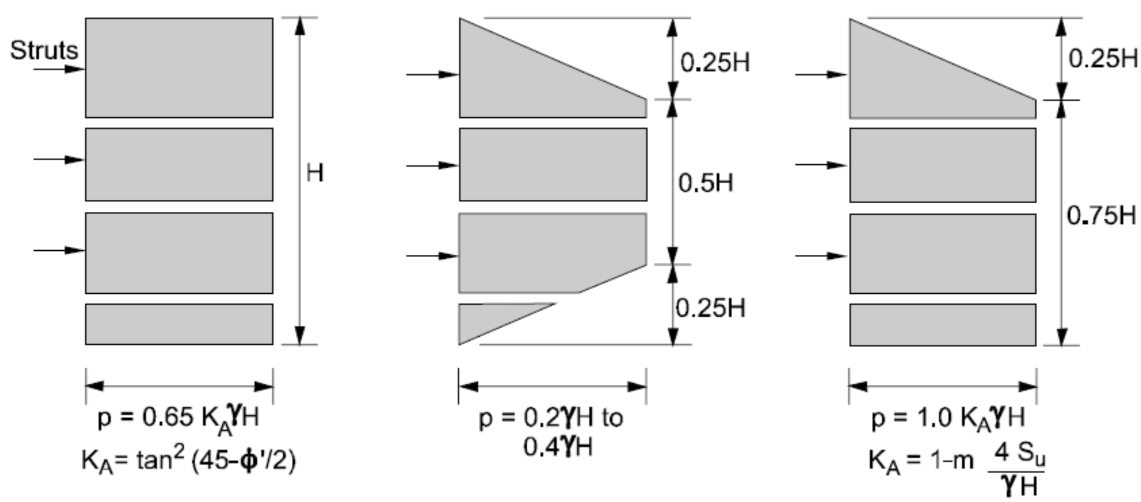
xiii. Select lagging, design wales, facing drainage systems, and connection devices.

5.3: Step-Wise Design Procedure for FHWA Method

5.3.1: Evaluation of Earth Pressures for Wall Design [18]

"The earth pressure distribution that develops on an anchored wall depends on the magnitude and distribution of lateral wall deformations. Some relatively flexible non-gravity cantilevered walls (e.g., sheet-pile or soldier beam and lagging walls which are not anchored) can be expected to undergo lateral deformations sufficiently large to induce active earth pressures for the entire wall height. For design of these systems, theoretical active earth pressure diagrams using either Rankine or Coulomb analysis methods can be used".

"The Terzaghi and Peck apparent earth pressure envelopes are rectangular or trapezoidal in shape. These diagrams are summarized in Figure 5.1. The maximum ordinate of the apparent earth pressure diagrams is denoted by p".



(a) Sand (b) Stiff-hard fissured clays (c) Soft to medium clays

Figure 5.1: Terzaghi and Peck Apparent Pressure Envelopes (After Terzaghi and Peck, 1967) [18]

5.3.1.1: Recommended Apparent Earth Pressure Diagram for Sands [18]

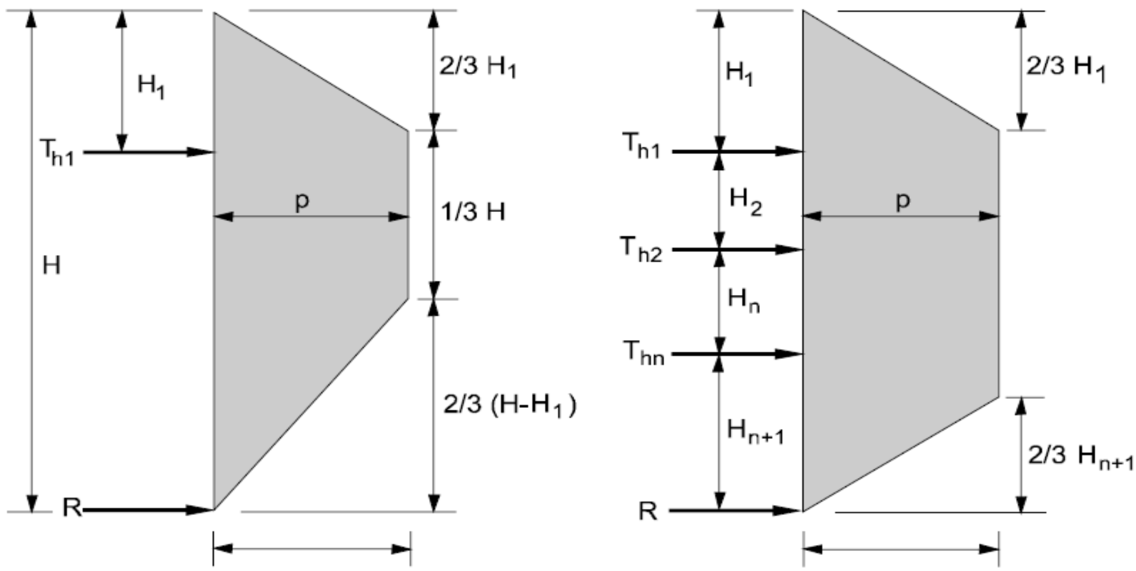
"For sands, the value for K_A in Figure 5.1 is given as [18]:

$$K_a = \tan^2\left(45 - \frac{\phi'}{2}\right) \qquad \text{……… (5.1)}$$

The maximum earth pressure ordinate is:

$$p = 0.65 K_a \gamma H \qquad \text{......... (5.2)}$$

Where Φ = Effective stress friction angle of the sand".



$$P = \frac{\text{Total load}}{2/3H} \approx K_a \gamma H$$

$$P = \frac{\text{Total load}}{H - 1/3H_1 - 1/3H_{n+1}}$$

H

1

a) Walls with one level of ground anchors

=

b) Walls with multiple levels of ground anchors

Distance from ground surface to uppermost ground anchor.

H_{n+1} = Distance from base of excavation to lowermost ground anchor.

T_{hi} = Horizontal load in ground anchor i.

R = Reaction force to be resisted by subgrade (i.e., below base of excavation).

P = Maximum ordinate of diagram.

Figure 5.2: Recommended Earth Pressure Diagram for Sands [18]

Total load = $P = 0.65 K_a \gamma H$

“Using the value of lateral earth pressure, total lateral earth load from the rectangular apparent earth pressure diagram (Figure 5.1a) for sands is 0.65 $K_a \gamma H^2$. The recommended apparent earth pressure envelope for single level anchored walls and walls with two or more levels of ground anchors is trapezoidal and is shown in Figure 5.2”.

“The trapezoidal diagram is more appropriate than the rectangular diagram for the following reasons:

- Earth pressures are concentrated at the anchor locations resulting from arching;
- Earth pressure of zero at the ground surface is appropriate for sands (provided no surcharge loading is present);
- Earth pressures increase from the ground surface to the upper ground anchor location; and
- Medium dense to very dense sands, earth pressures reduce below the location of the lowest anchor owing to the passive resistance that is developed below the base of the excavation.

This diagram is appropriate for both short-term (temporary) and long-term (permanent) loadings in sands. Water pressures and surcharge pressures should be added explicitly to the diagram to evaluate the total lateral load acting on the wall”.

5.3.2: Water Pressures [18]

“Permanent anchored soldier beam and lagging walls are typically not designed to resist large water loads. For these wall systems, drainage from the surface of the retained soil is collected in ditches at the top of the wall while subsurface water is collected using prefabricated drainage elements placed between the wall and the permanent facing. For temporary systems, it may be necessary to resist water forces associated with seepage behind and beneath the wall. A typical flow net is developed for this purpose”.

5.3.3: Earth Pressures due to Surface Loads [18]

Uniform surcharge loads are vertical loads applied at the ground surface which are assumed to result in uniform increase in lateral stress over the entire height of the wall. The increase in lateral stress for uniform surcharge loading can be written as [18]:

$$\Delta\sigma_h = Kq_s \quad \text{……… (5.3)}$$

Where:

$\Delta\sigma_h$ = the increase in lateral earth pressure due to the vertical surcharge load
q_s = the vertical surcharge stress applied at the ground surface
K = an appropriate earth pressure coefficient.
Standard SI units are: $\Delta\sigma_h$ (kPa), K (dimensionless), and q_s (kPa).

“Examples of surcharge loads for highway wall system applications include: (1) dead load surcharges such as that resulting from the weight of a bridge approach slab of concrete pavement; (2) live load surcharges such as that due to traffic loadings; and (3) surcharges due to equipment or material storage during construction of the wall system. When traffic is expected to come within a distance from the wall face equivalent to one half the wall height, the wall should be designed for a live load surcharge pressure of approximately of 12 kPa” [18].

Point loads, line loads, and strip loads are vertical surface loadings which are applied over limited areas as compared to surcharge loads. As a result, the increase in lateral earth pressure used for wall system design is not constant with depth as is the case for uniform surcharge loadings [15]. These loadings are typically calculated using equations based on elasticity theory for lateral stress distribution with depth [18].

5.3.4: Seismic Load Calculations [18]

Two modes of earthquake - induced failure for anchored walls are considered for design [18]:

- Internal failure and

- External failure.

"Internal failure is characterized by failure of an element of the wall system such as the tendons, ground anchors, or wall itself. External failure is characterized by a global failure of the wall similar to that which occurs in many slope stability problems, with the failure surface passing beyond the end of the anchors and below the toe of the wall" [18].

"The seismic loading on anchored walls is most commonly evaluated using pseudo-static analysis, as described subsequently. The most commonly used method for seismic design of retaining structures is the pseudo - static method developed by Okabe (1926) and Mononobe (1929). The Mononobe-Okabe method is based on Coulomb earth pressure theory" [18].

Using Mononobe - Okabe theory, the dynamic earth pressures in the active (P_{AE}) and passive (P_{PE}) state are given by the following [18]:

$$P_{AE} = ½\, \gamma H^2 (1- k_v)\, K_{AE} \qquad \text{……… (5.4)}$$

$$P_{PE} = ½\, \gamma H^2 (1- k_v)\, K_{PE} \qquad \text{……… (5.5)}$$

$$K_{AE} = \frac{\cos^2 (\phi-\theta-\beta)}{\cos(\theta).\cos(\beta)^2.\cos(\beta+\theta+\delta).D} \qquad \text{……… (5.6)}$$

$$D = \left(1 + \sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\theta-i)}{\cos(\delta+\beta+\theta).\cos(i-\beta)}}\right)^2 \qquad \text{……… (5.7)}$$

$$K_{PE} = \frac{\cos^2 (\phi-\theta+\beta)}{\cos(\theta).\cos(\beta)^2.\cos(\delta-\beta+\theta).D'} \qquad \text{……… (5.8)}$$

$$D' = \left(1 + \sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\theta+i)}{\cos(\delta-\beta+\theta).\cos(i-\beta)}}\right)^2 \qquad \text{……… (5.9)}$$

$$\text{Seismic inertia angle for soil} = \theta = \tan^{-1}\left(\frac{k_h}{1-k_v}\right) \qquad \text{……… (5.10)}$$

Where:

γ = Effective unit weight of the backfill (kN/m^3)

H = Height of the wall (m)

k_v = Vertical seismic coefficient expressed as fraction of g

k_h = Horizontal seismic coefficient expressed as fraction of g

δ = Angle of friction of the wall/backfill interface (degrees)

φ = Angle of internal friction of the backfill (degrees)

i = Slope of the surface of the backfill (degrees)

β = Slope of the backfill of the wall (degrees)

g = Acceleration due to gravity (m/s^2)

5.3.5: Ground Anchor Design [18]

"This section presents procedures that are commonly used to design a ground anchor and includes a brief discussion on analysis procedures to locate the critical potential failure surface, calculation of ground anchor loads from apparent earth pressure diagrams, design of the un-bonded and bonded lengths of the anchor, allowable load requirements for the prestressing steel element, and horizontal and vertical spacing and inclination of the anchor" [18].

5.3.5.1: Location of Critical Potential Failure Surface [18]

"The location of the critical potential failure surface must be evaluated since the anchor bond zone must be located sufficiently behind the critical potential failure surface so that load is not transferred from the anchor bond zone into the "no-load" zone. The "no-load" zone is defined as the zone between the critical potential failure surface and the wall, and is also referred to as the un-bonded length. The un-bonded length is typically extended either a minimum distance of H/5, where H is the height of the wall, or 1.5 m behind the critical potential failure surface. The critical potential failure surface can be assumed to extend up from the corner of the excavation at an angle of $45° + \varphi'/2$ from the horizontal (i.e., the active wedge)" [18].

5.3.5.2: Calculation of Ground Anchor Loads from Apparent Earth Pressure Diagrams [18]

"Ground anchor loads for flexible anchored wall applications can be estimated from apparent earth pressure envelopes. Methods commonly used include the tributary area method and the hinge method. Both methods, when used with appropriate apparent earth pressure diagrams, provide reasonable estimates of ground anchor loads and wall bending moments for anchored systems constructed in competent soils" [18].

"The calculations for horizontal ground anchor loads using the tributary area method and the hinge method are shown in Figure 5.3 for multi level anchored wall. Both methods assume that a hinge (i.e., zero bending moment) develops at the excavation subgrade and that the excavation subgrade acts as a strut support. This latter assumption is reasonable for walls that penetrate into competent materials. The maximum bending moment that controls the design of the wall typically occurs in the exposed portion of the wall, i.e. above the excavation subgrade" [18].

The values calculated using Figure 5.3 for the anchor loads are the horizontal component of the anchor load per unit width of wall, T_{hi}. The total horizontal anchor load, T_h, is calculated as [18]:

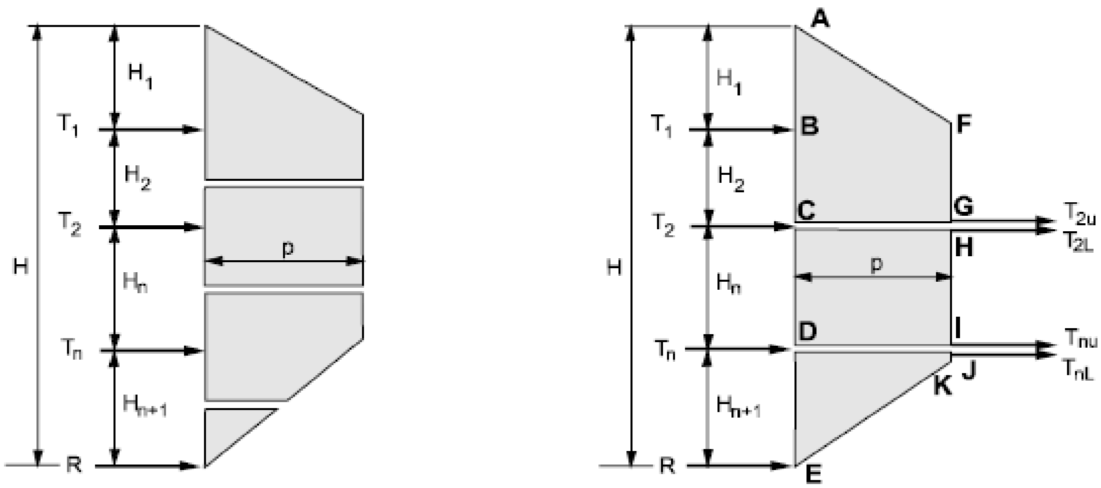
$$T_h = T_{hi} * s \qquad \text{……… (5.11)}$$

Where s is the horizontal spacing between adjacent anchors. The anchor load, T, to be used in designing the anchor bond zone (i.e., the design load) is calculated as [18]:

$$T = T_h/\cos\theta \qquad \text{……… (5.12)}$$

Where θ is the angle of inclination of the anchor below the horizontal. The vertical component of the total anchor load, T_v, is calculated as [18]:

$$T_v = T\sin\theta \qquad \text{……… (5.13)}$$



Tributary Area Method

T_1 = Load over length $H_1 + H_2/2$

T_2 = Load over length $H_2/2 + H_n/2$

T_n = Load over length $H_n/2 + H_{n+1}/2$

R = Load over length $H_{n+1}/2$

Hinge Method

T_1 = Calculated from $\Sigma M_C = 0$

T_{2u} = Total earth pressure (ABCGF) – T_1

T_{2L} = Calculated from $\Sigma M_D = 0$

T_{nu} = Total earth pressure (CDIH) – T_{2L}

T_{nL} = Calculated from $\Sigma M_D = 0$

R = Total earth pressure $T_1 - T_2 - T_n$

$T_2 = T_{2u} + T_{2L}$

$T_n = T_{nu} + T_{nL}$

Figure 5.3: Calculation of Anchor Loads for Multi - Level Wall [18]

5.3.5.3: Design of Un-bonded Length [18]

"The minimum un-bonded length for rock and soil ground anchors is 4.5 m for strand tendons and 3 m for bar tendons. These minimum values are intended to prevent significant reductions in load resulting from seating losses during transfer of load to the structure following anchor load testing" [18].

Longer un-bonded lengths may be required to [18]:

1. Locate the bond length a minimum distance behind the critical potential failure surface.

2. Locate the anchor bond zone in appropriate ground for anchoring
3. Ensure overall stability of the anchored system; and
4. Accommodate long term movements.

“In general, the un-bonded length is extended a minimum distance of H/5 or 1.5 m behind the critical potential failure surface to accommodate minor load transfer to the grout column above the top of the anchor bond zone” [18].

“As a general rule, the anchor bond zone and un-bonded zone should be grouted in one stage to maintain hole stability and to create a continuous grout cover for corrosion protection. However, for large diameter anchors in which the un-bonded length of the anchor extends just behind the critical potential failure surface, significant strains at the top of the anchor bond zone may cause load transfer into the grout column above the anchor bond zone. Large diameter anchors have been grouted in two stages (two stage grouting)” [18].

5.3.5.4: Design of Anchor Bond Length [18]

“For a specific project, the first step in estimating the minimum allowable capacity is to assume a maximum anchor bond length. In the case of a site with no restrictions on right-of-way, a 15^0 inclination of the anchor should be assumed with a bond length of 12 m in soil or 7.5 m in rock” [18].

“Anchors founded in soil and rock should be designed assuming the entire embedment is in soil, i.e. assume a bond length equal to 12 m. The bond lengths at sites with more restricted right-of-way may be evaluated assuming an anchor inclination of 30^0 and that the bond length is equal to the distance from the end of the un-bonded length to within 0.6 m of the right-of-way line” [18].

“For the purposes of preliminary design, the ultimate load transferred from the bond length to the soil may be estimated for a small diameter, straight shaft gravity-grouted anchor from the soil type and density (or SPT blow count value) (Table 5.1). The maximum allowable anchor design load in soil may be determined by multiplying the bond length by the ultimate transfer load and dividing by a factor of safety of 2.0” [18].

“Anchor bond lengths for gravity-grouted, pressure-grouted, and post-grouted soil anchors are typically 4.5 to 12 m since significant increases in capacity for bond lengths greater than approximately 12 m cannot be achieved unless specialized methods are used to transfer load from the top of the anchor bond zone towards the end of the anchor” [18].

Table 5.1: Presumptive ultimate values of load transfer for preliminary design of small diameter straight shaft gravity-grouted ground anchors in soil [18]

Soil type	Relative density/Consistency (SPT range)$^{(1)}$	Estimated ultimate transfer load (kN/m)
Sand and Gravel	Loose (4-10) Medium dense (11-30) Dense (31-50)	145 220 290
Sand	Loose (4-10) Medium dense (11-30) Dense (31-50)	100 145 190
Sand and Silt	Loose (4-10) Medium dense (11-30) Dense (31-50)	70 100 130
Silt-clay mixture with low plasticity or fine micaceous sand or silt mixtures	Stiff (10-20) Hard (21-40)	30 60

Note: (1) SPT values are corrected for overburden pressure.

“Pressure grouting in cohesionless soils significantly increases the normal stresses acting on the grout body (i.e., increases confinement). Small increases may also be observed in the effective diameter of the anchor bond zone, but capacity estimates should be based on the as-drilled hole diameter. Pressure grouting can be effective in increasing capacity in cohesive soils, however, post-grouting is a more effective means of increasing capacity in cohesive soils. Post grouting increases the radial stresses acting on the grout body and causes an irregular surface to be developed around the bond length that tends to interlock the grout and the ground” [18].

5.3.5.5: Spacing Requirements for Ground Anchors [18]

The horizontal and vertical spacing of the ground anchors will vary depending on project specific requirements and constraints, which may include [18]:

1. Necessity for a very stiff system (i.e. closely spaced anchors) to control lateral wall movements
2. Existing underground structures that may affect the positioning and inclination of the anchors
3. Type of vertical wall elements selected for the design.

"The vertical position of the uppermost ground anchor (i.e., the ground anchor closest to the ground surface) should be evaluated considering the allowable cantilever deformations of the wall. The vertical position of the uppermost anchor must also be selected to minimize the potential for exceeding the passive capacity of the retained soil during anchor proof and performance load testing" [18].

"For ground anchors installed in soil, a minimum overburden of 4.5 m over the center of the anchor bond zone is required (Figure 3.4). For gravity-grouted anchors, the minimum overburden criterion is required to provide the necessary soil overburden pressure to develop anchor capacity" [18].

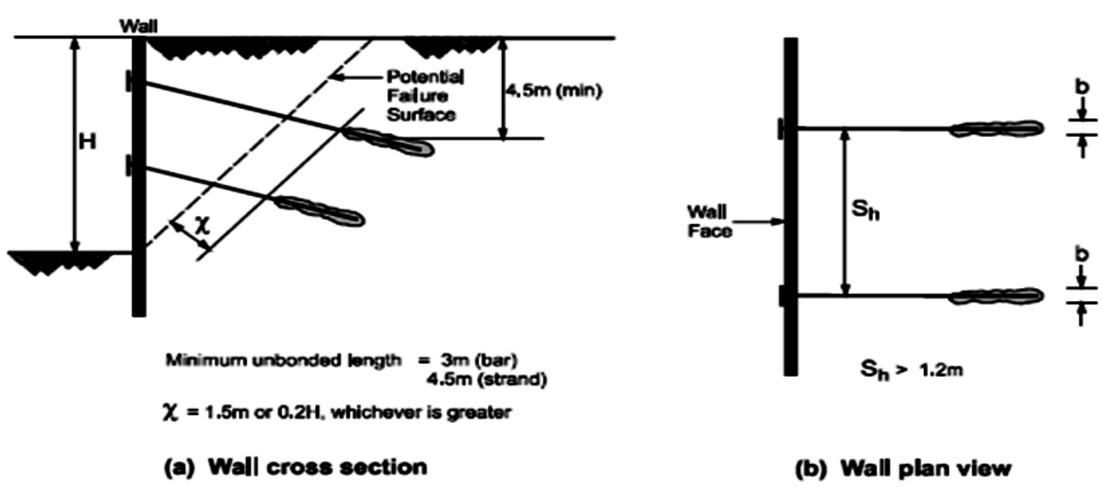


Figure.5.4 Vertical and Horizontal Spacing Requirements for Ground Anchors [15]

“Typical horizontal spacing for soldier beams is 1.5 to 3 m for driven soldier pile and up to 3 m for drilled-in soldier pile. The minimum horizontal spacing between anchors shown in Figure 5.4b ensures that group effects between adjacent ground anchors are minimized and that anchor intersection due to drilling deviations is avoided. Group effects reduce the load carrying capacity of individual ground anchors” [18].

5.4: Canadian Approach

According to the Canadian Foundation Engineering Manual (2007), the design of temporary supports of vertical faces of excavation is based on the combination of theoretical methods, empirical methods and experience based judgment [19].

There are two basic approaches to design the excavation support and flexible retaining structures [19]:

a. Design for the minimum requirements to satisfy load carrying capacity (those loads that the soil itself does not carry) and system stability; or

b. Design for control of deformations

In general, design of control of deformations will produce a support system stiffer than one designed based on an estimation of the loads imparted on the support system.

Design of deep excavation support systems according to the Canadian Method has been discussed below in detail. The following excerpts are taken from Canadian Foundation Engineering Manual (4th Edition, 2007).

5.5: Main Features of Canadian Method

According to the Canadian Foundation Engineering Manual (2007), design of supported excavation and flexible retaining structures require considerations of the following load and stability cases [19]:

5.5.1: Load Considerations [19]

- Earth pressure
- Water pressures
- Surcharge load from equipments, structures, adjacent roads
- Earthquake loading
- Loads from frost action
- Temperature-induced stresses in structural members
- Stresses from swelling ground
- Pre-stressing loads

5.5.2: Stability Considerations [19]

- Structural stability of the support system (loading)
- Stability of the excavation base related to shear failure in the soil
- Stability of the excavation base related to groundwater uplift forces
- Deep-seated failure encompassing wall and any ground anchors
- Stability of slopes above excavation

Flexible earth retaining structures can be walls, formed using soil mixing and/or jet grouting, small diameter drilled piles and soil nails.

5.5.3: Earth Pressures and Deformations [19]

"The earth pressure acting on an earth-supporting structure depends mainly on the lateral deformations of the soil as shown in Figure 5.5. The deformation conditions should be estimated with reasonable accuracy. For rigid walls, a fairly simple relationship exists between the wall movement and the earth pressure, if the displacement of the top of the wall is not smaller then the bottom of the wall" [19].

"For flexible walls, the deformations and the earth pressures are more complex. The yield of one part of the flexible wall redistributes pressure on to the more rigid parts due to internal shear strength of the soil, a process called "arching". That is why the pressures in the vicinity of supports are higher than in unsupported areas and the loads on or between individual supports vary depending largely on the stiffness characteristics of the various wall components themselves (e.g. piles, struts, anchors, lagging etc)" [19].

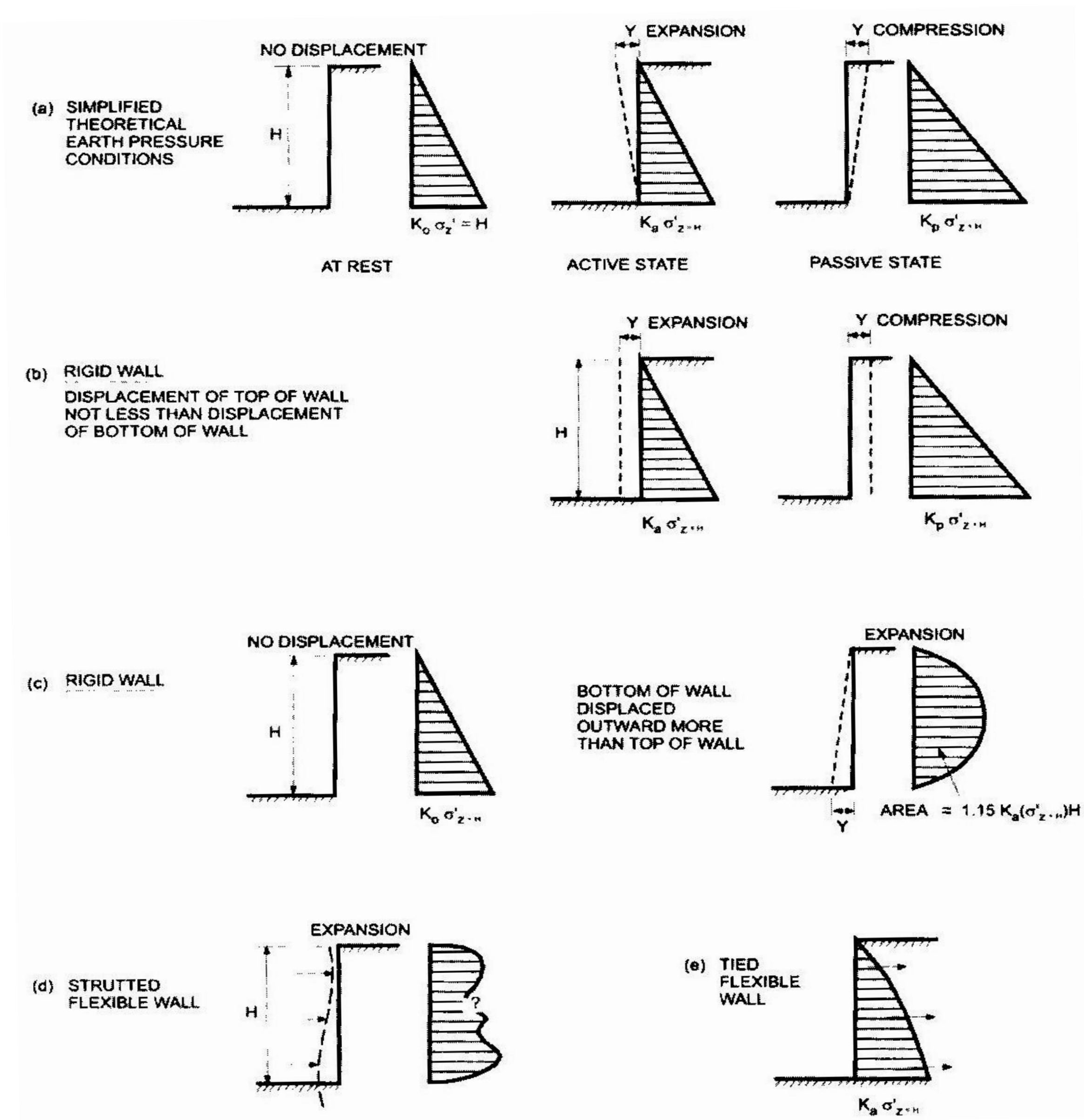


Figure 5.5: Effect on Earth Pressures in Cohesion Less Material [19]

"The deflection characteristics of anchors can provide nearly constant-load supports and anchored walls come nearer to having a triangular pressure distributions then strutted walls if the anchors are not heavily prestressed to a predetermined design load. In calculation for anchored walls, it may be desirable to assume a trapezoidal or rectangular distribution to ensure more positive support of adjacent footing or buried services" [19].

5.5.4: Surcharge Pressures [19]

"Theoretical surcharge pressures should be applied as per following guide lines [19]:

Uniform Area Loading: The surcharge behind the wall consists of a large uniformly loaded area, with intensity that is small compared to the total backfill forces (total force on wall from surcharge is less than 30% of the active force), the wall pressure may be calculated using [19]:

$$\sigma'_{hs} = q K \qquad \text{……… (5.14)}$$

σ'_{hs} = horizontal pressure due to surcharge (kPa)

q = uniform surcharge pressure (kPa)

K = applicable earth pressure coefficient (K_o or K_a)

5.5.5: Earthquake Induced Pressures [19]

"Earthquake will induce additional pressure on retaining structures. The magnitude and distribution of earthquake induced loads is determined using the Mononobe - Okabe (1926) equations according to the Canadian Method".

For active earth pressure loads [19]:

$$Pae = \frac{1}{2} \gamma H^2 (1- k_v) K_{ae} \qquad \text{……… (5.15)}$$

Where,

P_{ae} = resultant active lateral earth load including static and dynamic loads

ɤ = unit weight of the soil behind the wall

k_v = vertical component of the earthquake acceleration (as a decimal fraction of acceleration due to gravity)

k_h = horizontal component of the earthquake acceleration (as a decimal fraction of acceleration due to gravity)

K_{ae} = horizontal component of the active earth pressure coefficient including effects of earthquake loading.

$$K_{ae} = \frac{\cos^2(\varphi-\psi)}{\cos(\psi).\cos(\psi+\delta).\left(1+\sqrt{\frac{\sin(\varphi+\delta).\sin(\varphi-\psi)}{\cos(\delta+\psi)}}\right)^2} \qquad \text{……… (5.16)}$$

Seismic inertia angle for soil = $\psi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right)$ ……… (5.17)

For passive earth pressure loads [19]:

Ppe = ½ γH^2 (1- k_v)*K_{pe} ……… (5.18)

$$K_{pe} = \frac{\cos^2(\varphi-\psi)}{\cos(\psi).\cos(\psi+\delta).\left(1-\sqrt{\frac{\sin(\varphi+\delta).\sin(\varphi-\psi)}{\cos(\delta+\psi)}}\right)^2} \qquad \text{……… (5.19)}$$

Where,

P_{pe} = resultant passive lateral earth load including static and dynamic loads.

K_{pe} = passive earth pressure coefficient including effects of earthquake loading.

“In static earth pressure calculations, the effect of wall friction should be used with caution as unrealistically high values may result if values of δ greater then about φ/3 or φ/2 are used. The location of the resultant forces needs to be defined to calculate moments for completion of retaining structure design. The Mononobe-Okabe determination of the active and passive earth pressures does not provide any indication of the distribution of loads” [19].

“It is considered that the increases in active and passive earth pressures are greater near the top of the wall. Therefore, it is common to apply the resultant incremental earthquake loads at a height of 0.6H where H is the height above the bottom of the wall. If earthquake forces are to be considered in retaining wall design it is also reasonable to utilize a lower factor of safety of about 1.2”.

5.5.6: Design Approach for Canadian Method [19]

Two different methods can be used to design anchor - pile system. These methods are commonly referred to as “fixed earth” and “free earth” methods [19]:

1. The “free earth” approach assumes that the wall acts as beam spanning between two supports, these being the top anchorage and the passive pressures of the earth below the excavation line (wall is free to rotate or translate horizontally at its bottom end);

2. The “fixed-earth” approach assumes that the wall extends sufficiently in to the ground to develop fixity at some point below the excavation or dredge line and the wall cannot rotate or translate at this point”.

“The design of wall supported by multiple anchors can be carried out by using either triangular or apparent earth pressure diagram. For walls designed using multiple anchors and a triangular earth pressure distribution, the individual anchor loads can be solved through calculation of horizontal force equilibrium. Walls supported with multiple anchors typically experience large number of deformations at the top then their bottom. All the horizontal loads should be applied including those from active and passive earth pressures, surcharges, unbalanced water pressures, seepage pressures and seismic loads as appropriate” [19].

“Following are the design steps for anchor system design [19]:

1. Assume that the highest load in the nth level anchor occurs just before placing the next anchor, and draw the excavation cross section for that condition.

2. For the first anchor level, calculate the depth of penetration of soldier pile to result in moment equilibrium taken about the first anchor level, and the first anchor load will be equal to the load required for horizontal force equilibrium.

3. For all anchors, other than lowest, determine the depth of penetration of the wall required to establish a factor of safety of 1.0 against rotation about the wall top, using the pressure diagram previously established and taking into account the design forces in previously installed anchors.

4. Determine the required force in the nth anchor for stability of wall, based on equilibrium of all horizontal forces.

5. For the next to lowest anchor, check the intermediate depth of penetration as indicated by the analysis described is adequate to allow safe excavation to lowest anchor level.

6. For the lowest anchor, take the depth of penetration at the proposed design value and calculate the anchor force from horizontal force equilibrium.

7. If the lowest anchor is more than 1 m from the bottom of the wall, the wall should penetrate below the base of the cut at least to the depth at which the computed resultant force is zero. If this is not the case then, substantial bending moments may exist in the bottom section of the wall and the load on the lowest anchor increases as a result of stress redistribution as shown in Figure 5.6".

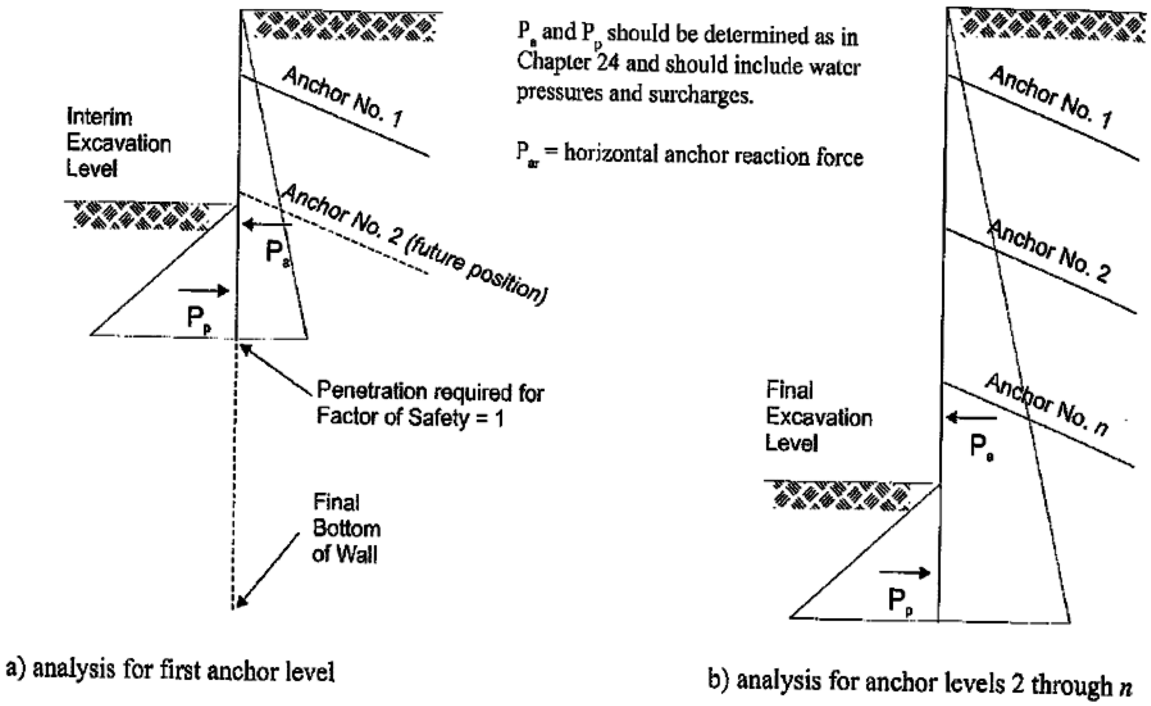


Figure 5.6: Calculation of Anchor Forces and Conditions for Multiple Anchors [19]

5.5.7: Effect of Anchor Inclination [19]

“Anchors are usually inclined downward transmitting the vertical component of the anchor force in to the anchored vertical member. This force should be considered in design, together with the weight of the vertical member itself. With soldier pile and lagging systems, the available shaft resistance is reduced during the excavation process; additional toe capacity may be required to limit the vertical deformations” [19].

“A conservative approach to retaining structure design is to ignore friction or adhesion along the back of the wall. Such vertical forces must be supported in bearing at the toe of the support system. The toe capacity of the wall must be checked otherwise unacceptable vertical or horizontal displacement may take place. Settlement of vertical members produces some reduction in anchor loads with the consequent tendency for outward movement of the supported face. It is advisable to monitor vertical and horizontal movements at the top and bottom of the excavation at regular intervals throughout the course of the work” [19].

5.5.8: Estimated Capacity of Soil Anchors [19]

The pull out resistance P_{ar}, for tremie grouting anchors in cohesionless soils can be estimated from the following equation [19]:

$$P_{ar} = \sigma'_z A_s L_s \alpha_g \qquad \text{........ (5.20)}$$

σ'_z = effective vertical stress at the midpoint of the load carrying length

A_s = effective unit surface area of the anchor bond zone

L_s = effective length of the anchor bond zone (limited to about 8 m)

α_g = anchorage coefficient dependent on the soil type and condition as given in Table 5.2.

Table 5.2: Anchorage Coefficient α_g [19]

Soil Type	Relative Density		
	Loose	Compact	Dense
Silt	0.1	0.4	1.0
Fine sand	0.2	0.6	1.5
Medium sand	0.5	1.2	2.0
Coarse sand, gravel	1.0	2.0	3.0

“The capacity of anchors estimated using the above method presumes a relatively linear increase of capacity with a corresponding increase in bond zone length. However, anchor capacities generally do not increase once the length of the bond zone increases beyond 8 m” [19].

“The allowable anchor load is determined by dividing the ultimate capacity of anchors by the factor of safety. Where no pull-out tests are carried out, the allowable anchor load is commonly obtained by dividing the computed capacity of the anchor by a factor of safety of 3 or more” [19].

5.5.9: Anchor Diameter and Spacing [19]

Preliminary capacities of pressure-grouted anchors may be calculated according to the values provided in Table 5.2. Following are the assumptions for anchor diameter and spacing [19]:

- The nominal diameter of the anchor is between 150 mm and 200 mm,
- Grout is injected using a pressure of about 1 MPa,
- The centre-to-centre spacing of the anchors in the bond zone should be more than 4 times the anchor diameter of the 20% of the bond zone length as shown in Figure 5.7.

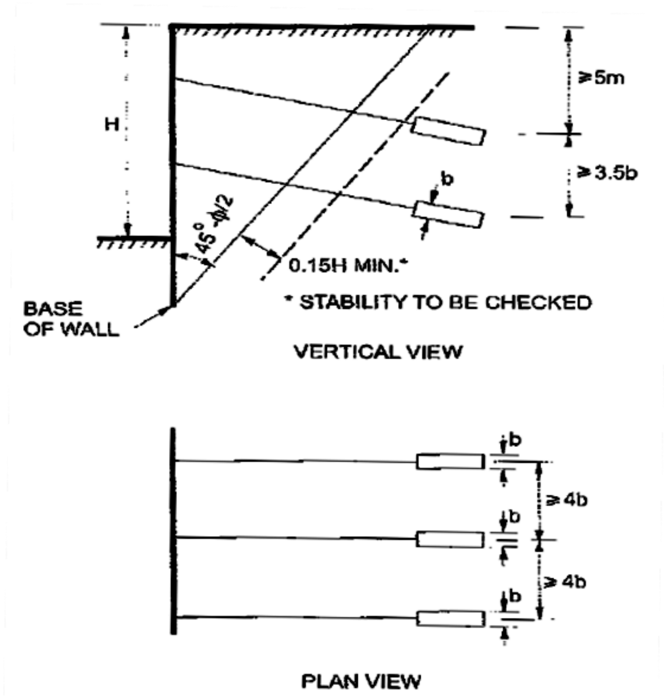


Figure 5.7: Minimum Spacing and Depth for Ground Anchors [19]

5.5.10: Stability of Flexible Retaining Systems

5.5.10.1: Excavation Base Stability

The base of a supported excavation can fail in three general modes including [19]:

1. Shear failure within ground from inadequate resistance of the loads imposed by the differences in grades inside and outside the excavation;

2. Piping or quick conditions from water seepage through granular soils at the excavation bottom.
3. Heave of layered soils due to water pressures confined by intervening low permeability soils".

5.5.10.2: Overall Stability of Anchored System [19]

"Even if the appropriate retaining system pressures and anchor design criteria are satisfied, an excavation support system or retaining structure supported by anchors can fail if the entire block encompassing all wall components is not stable. The overall stability of the anchor system is checked by analyzing the stability of the block of soil lying between the wall and the mid-point of the anchors. Overall stability of single level anchor system is shown in Figure 5.8".

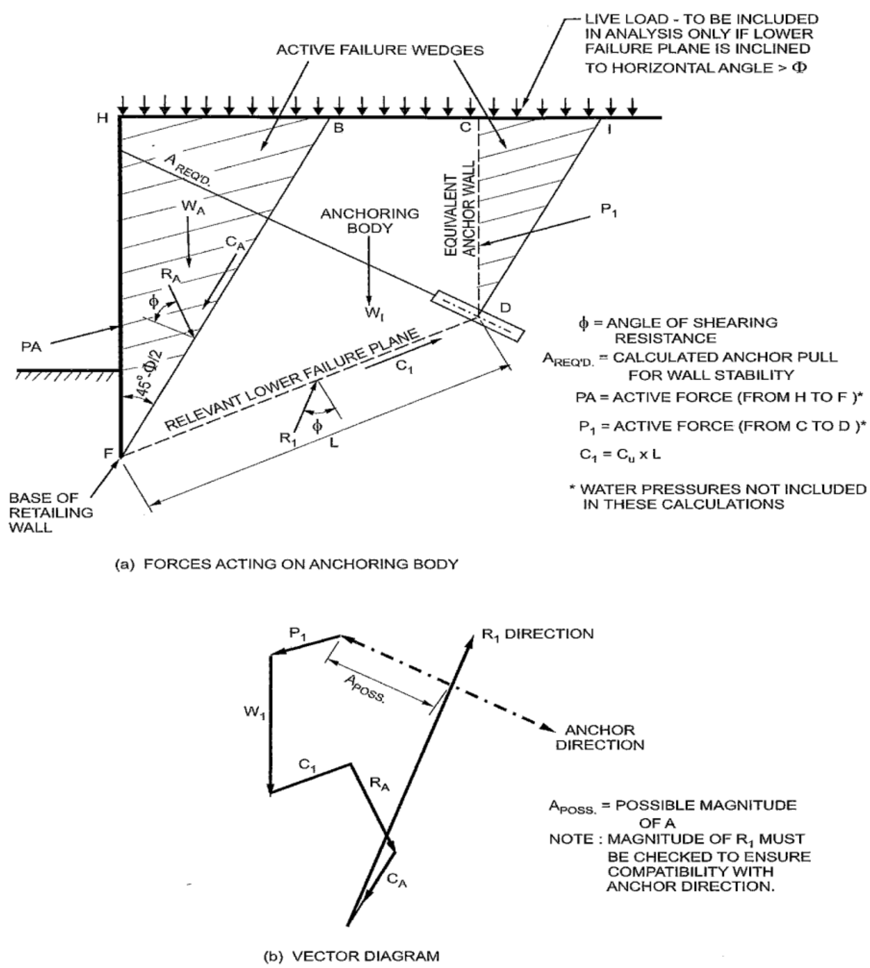


Figure 5.8: Graphic Analysis of Anchored Wall in Uniform Soil [19]

Multiple Level Anchor System: "The stability of each level of anchoring system should be checked, commencing at the top anchor. At each level, the required anchor force is the sum of all anchor forces above the relevant lower failure plane. Three typical possible cases for the location of anchors with respect to the base of the retaining wall are shown in Figure 5.9. The failure planes requiring stability analysis are indicated in each case. The method of analysis for each anchoring body is the same as that indicated for the single anchor system" [19].

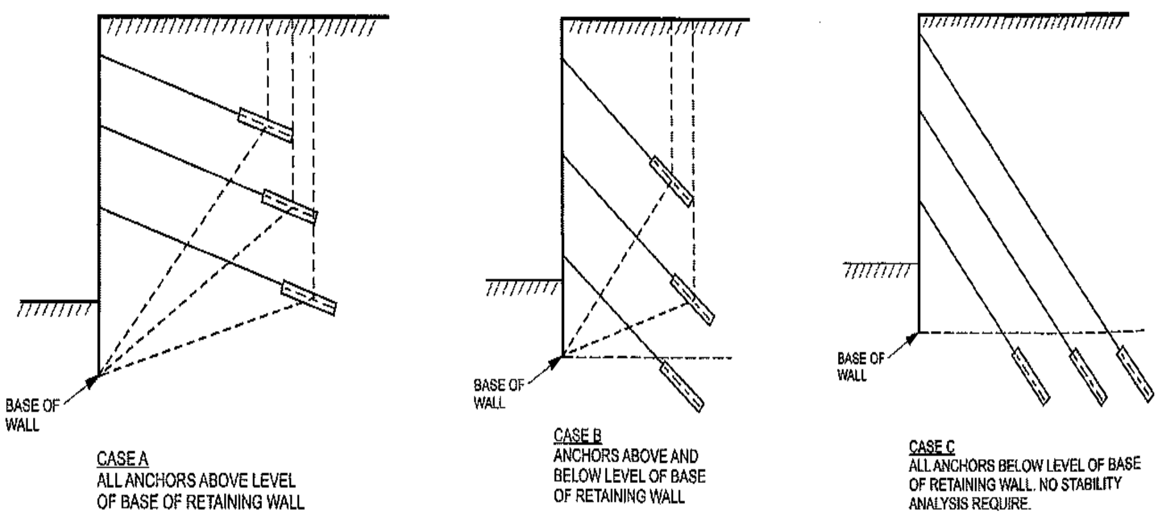


Figure 5.9: Typical Multiple-Level Anchor Systems Showing Potential for Failure Planes requiring Analysis [19]

CHAPTER -6

DEEP EXCACATION SUPPORT SYSTEM DESIGN

6.1: General

This chapter presents the final design for anchor - pile support system using the following two codes i.e.

- Canadian Foundation Engineering Manual (2007),

- Federal Highway Administration (FHWA), US Department of Transportation, Geotechnical Engineering Circular No. 4, Ground Anchors and Anchored System (1999) and Soil Mechanics,

6.2: Design of Anchor-Pile System

A 9 storey plaza was proposed to be constructed in Gulberg with 5 basements. The depth of the excavation was 15 m. The plaza is located at Plot No 92-B-2 Hussain Chowk Gulberg 3 Lahore. As discussed in the chapter 4, Lahore was divided into zones geographically. According to the division of Lahore, the site considered for deep excavation support system design lies in Zone 3.

The site is flanked by buildings on three sides and a road on one side as shown in Figure 6.1. Therefore, proper bracing of the deep excavation was considered necessary. Several options were considered and it was decided that an anchored tie-back system with soldier pile and wales would be optimum as it would clear space for construction operations.

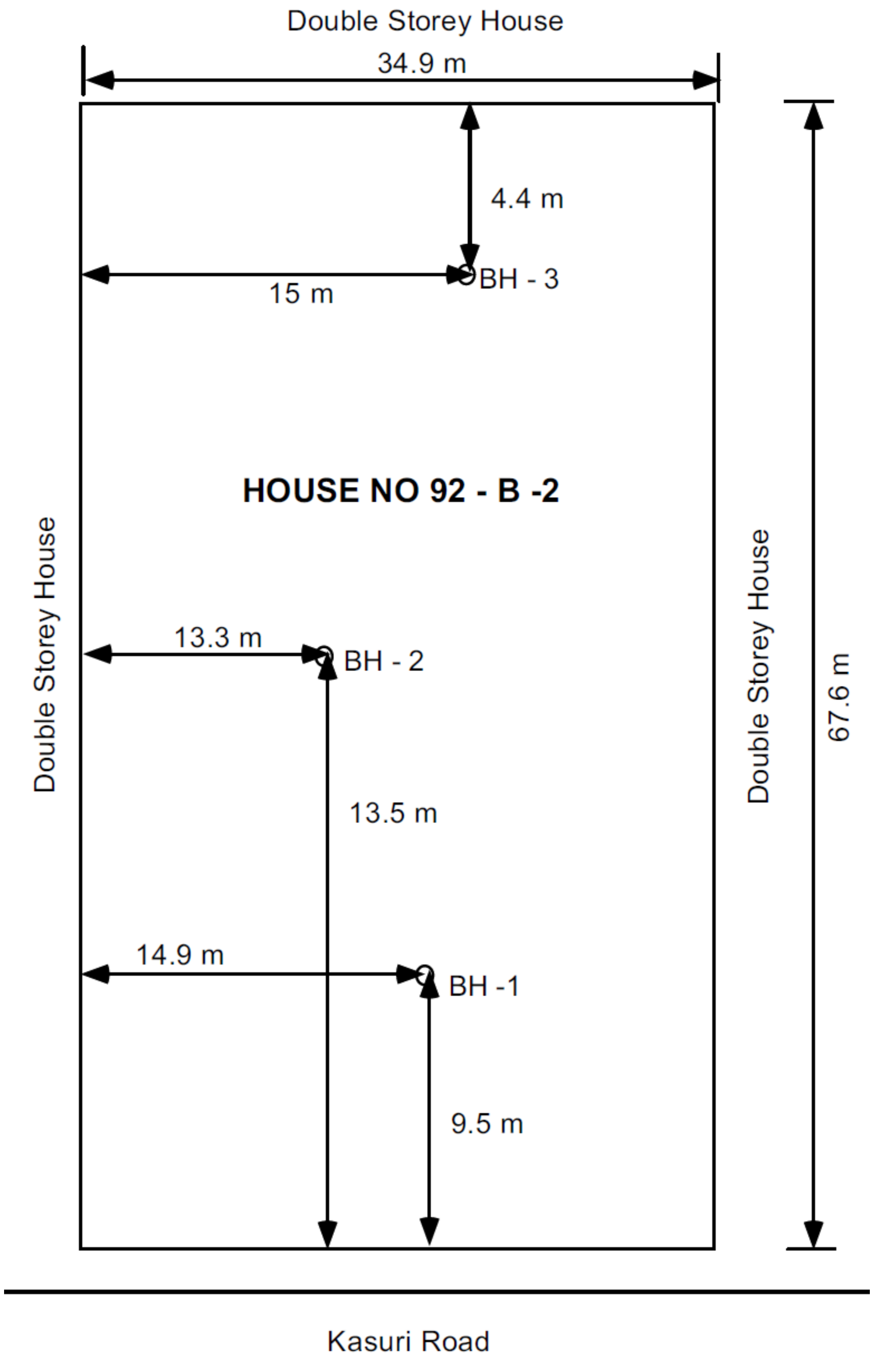


Figure 6.1: Location Plan for Site

6.3: Design Parameters Adopted for Research

The soil parameters chosen for design were selected based on:

1) Geotechnical investigation and laboratory test results and

2) The soil property ranges developed for Zone 3 as shown in the generalized soil properties profile for Zone 3 attached in the Appendix A. Table 6.1 shows the soil property ranges extracted from the generalized soil properties profile for Zone 3.

Table 6.1: Soil Property Ranges from the Generalized Soil Properties Profile for Zone 3

Depth	Friction Angle, φ	Unit Weight, ɤ
m	**Degrees**	**kN/m^3**
0 to 4	16 to 25	17 to 18
5 to 15	18 to 31	17 to 19
16 to 30	20 to 33	18 to 19

The subsurface profile is shown in Figure 6.2.

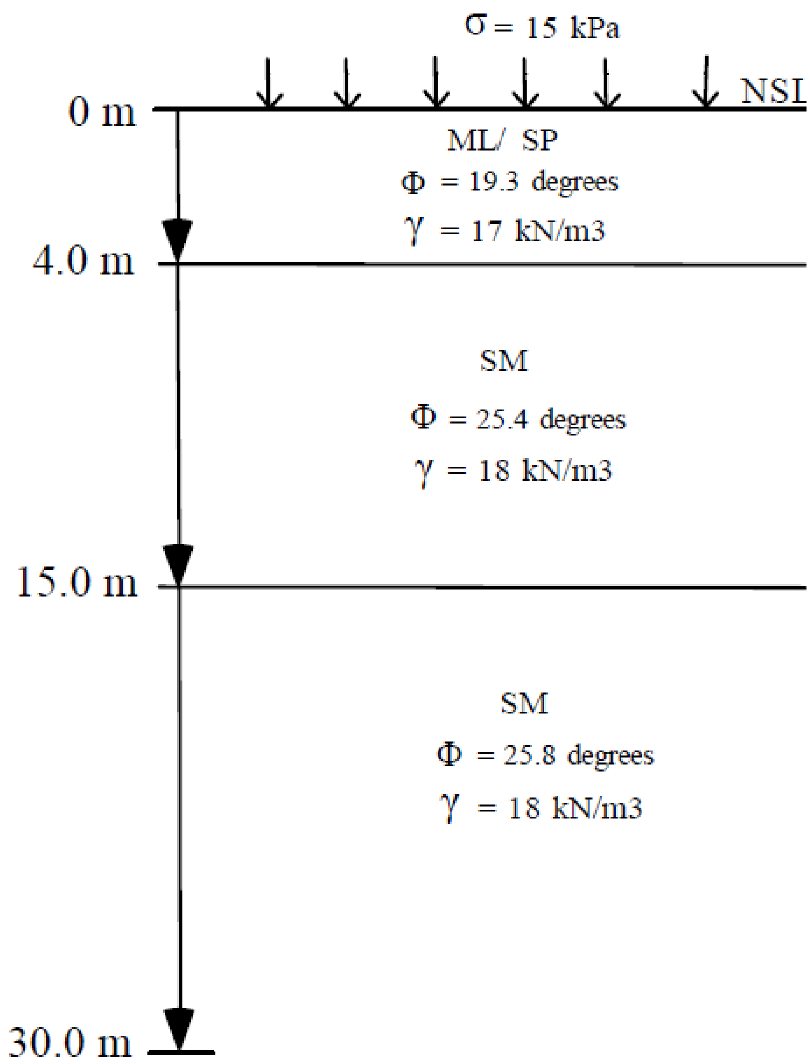


Figure 6.2: Soil Profile

6.4: Final Design According to FHWA Method

The final cross-section as per FHWA Method is as follows:

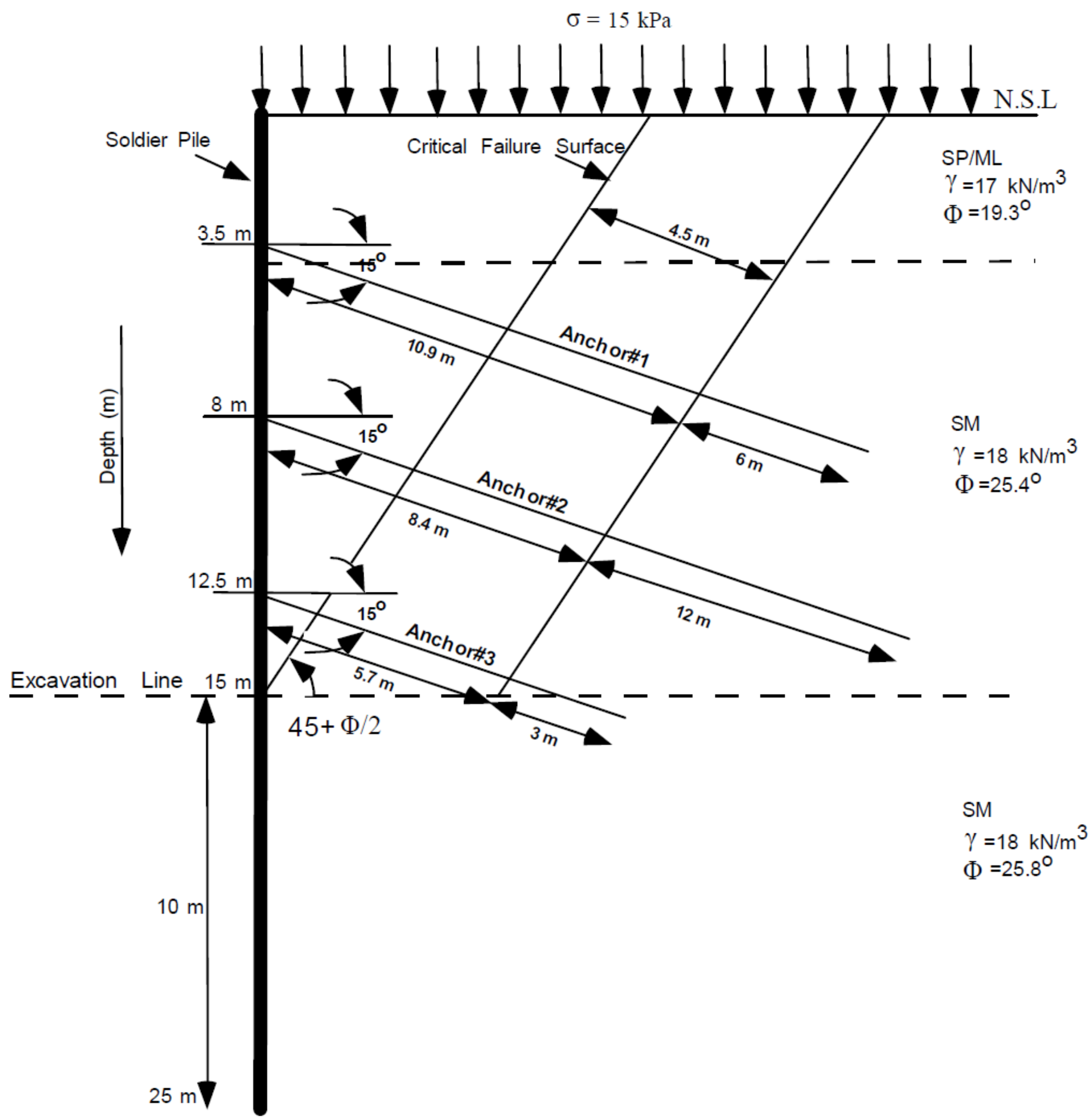


Figure 6.3: Final cross-section according to FHWA

The detailed design calculations are attached in Appendix B.

6.5: Final Design According to Canadian Method

The final cross-section as per Canadian Method is as follows:

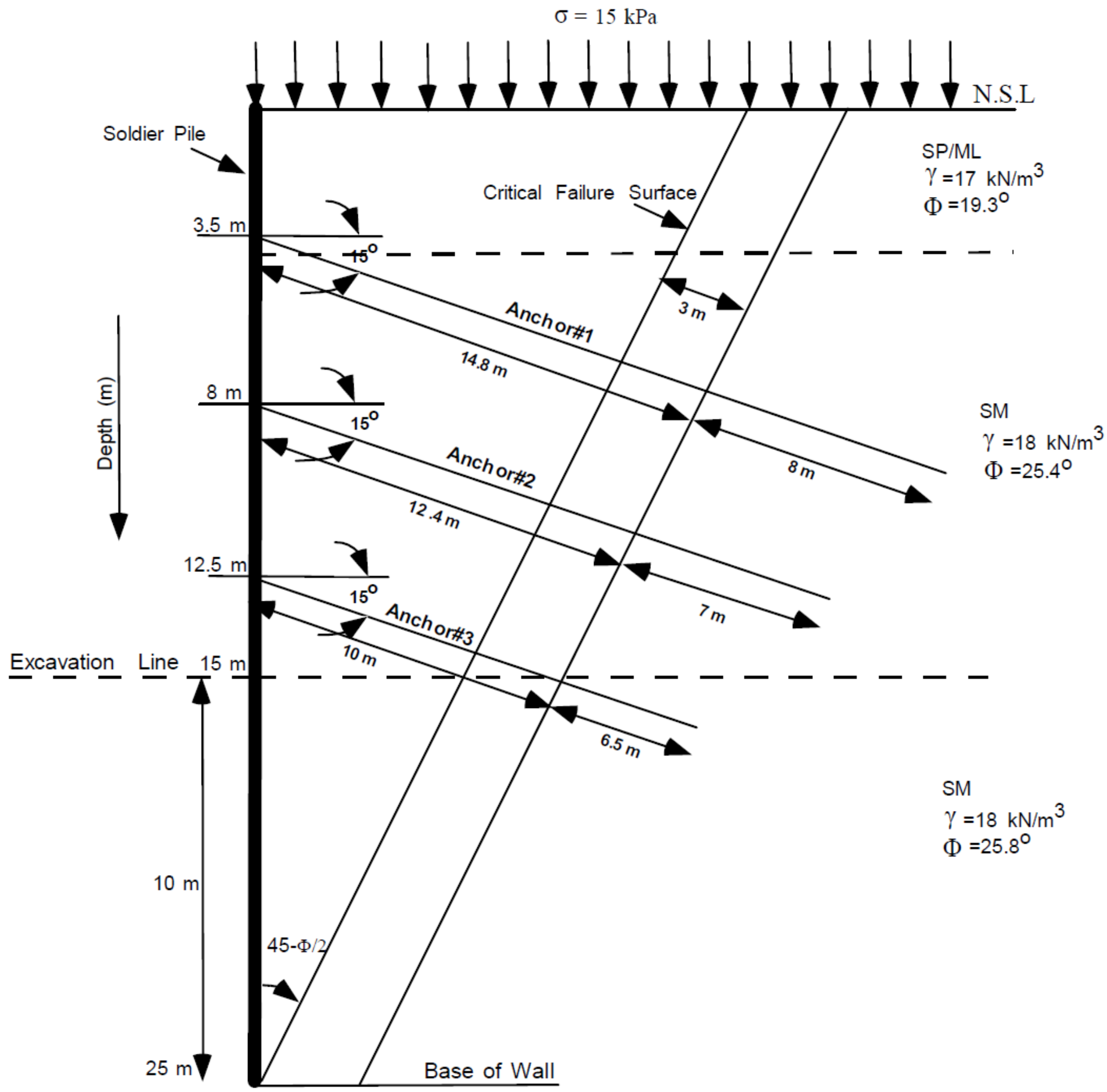


Figure 6.4: Schematic Arrangement for Anchor-pile system by Canadian Approach (2007)

The detailed design calculations are attached in Appendix B.

6.6: Comparison between FHWA and Canadian Method

The deep excavation support system was designed according to FHWA and Canadian Method. The final design obtained from both methods was different. The main differences in results are discussed below:

- **Differences in Earth Pressure Distribution**

  The FHWA and Canadian Method have different concepts for earth pressure distribution as discussed in chapter 5. FHWA method considers that the earth pressure distribution is trapezoidal. However, Canadian Method considers that earth pressure distribution is triangular. The difference in earth pressure distribution is the reason that the anchor loads calculated from both methods are different.

- **Differences in Critical Failure Surface Location**

  The critical failure surface defines the unbounded length of the anchor. According to FHWA method, the critical failure surface starts from the excavation line as shown in Figure 6.3. However, according to the Canadian Method, the critical failure surface starts from the base of the wall as shown in Figure 6.4. The difference in the critical failure surface location affects the length of the unbounded portion of the anchor.

- **Extension of Un-bonded Length**

  According to FHWA and Canadian Method, unbounded length is extended a minimum distance beyond the critical failure surface. In case of FHWA method the minimum unbounded length is selected to be greater of 0.2H or 1.5 m [18]. For Canadian Method, the unbounded length extends up to 0.15H beyond the critical failure surface [19].

- **Differences in Bonded length**

  The bonded length of the anchor starts after the critical failure surface location. The bonded lengths of the anchors are decided based on the anchor loads and the anchor loads are calculated using earth pressure diagrams. Different earth pressure distributions give different anchor loads and so the bonded lengths calculated from both the methods are different.

- **Calculation of Bonded and Un-bonded Lengths**

  Different approaches are used to calculate bonded and unbounded lengths according to FHWA and Canadian Method. According to FHWA Method the bonded lengths are determined by taking into consideration the load transfer rate as suggested in code for silts and sandy silt [18]. The Canadian Method however, determines bonded and un-bonded lengths using formulas and tables for anchorage coefficients given in the manual [19].

- **Stability Checks**

  According to the Canadian Method, additional checks are applied to determine the overall stability of the anchor. The FHWA method has no such stability requirements or checks.

CHAPTER -7

SUMMARY AND RECOMMENDATIONS

7.1: Summary

Summary of the report is discussed hereunder:

- Lahore was divided into five zones geographically and data was complied for each zone.
- Soil properties and SPT-N value ranges were established for each zone.
- The soil type and SPT-N value ranges were shown on “Soil Log Profile” developed for each zone.
- The soil property ranges were shown on the “Generalized Soil Properties Profiles” developed for each zone. Preparation of such profiles can be helpful and provide guidance to the practicing engineers and geologists with considerable savings in time and expense in Lahore.
- Deep excavation support system was designed according to FHWA and Canadian Method.
- The soil parameters for the site were determined by taking into consideration the profiles developed for Zone 3.
- The results obtained for bonded and unbounded lengths from both methods are different.
- Canadian Method is more reliable for design purposes as overall stability of anchors is checked.

7.2: Future Recommendations

Following are a few recommendations made for future studies:

- The accuracy of the study could be improved by increasing the database. Soil data could be collected from all over Lahore and used to develop profiles.

- Similar type of studies could be carried for other major cities in Pakistan and soil profiles could be developed. These profiles will be helpful for engineers and geologists in the design process.

- Deep excavation support system could also be designed according to NAVFAC method.

- A comparison could be drawn between all three methods of deep excavation support system design being used in Lahore i.e.

  1) Canadian Foundation Engineering Manual.
  2) FHWA Method
  3) NAVFAC

- A cost analysis/ comparison could be performed for all three methods of deep excavation support system design to determine which method is more economical.

REFERENCES

[1] http://en.wikipedia.org/wiki/Lahore

[2] Kazmi, A.H. & Jan, M.Q (1997), "*Geology and Tectonics of Pakistan*", Graphic Publishers, Karachi

[3] http://en.wikipedia.org/wiki/Climate_of_Lahore

[4] Kadwai, S.U. and Siraj, A. (1964), "*The Geology of Bari Doab, West Pakistan*", WAPDA Water and Soil Investigation Division. Bulletin No. 8.

[5] National Engineering Services Pakistan (NESPAK) (2007), "*Seisimic Building Code of Pakistan*", Ministry of Housing and Works.

[6] Bowles, J.E., (1996). "*Foundation Analysis and Design*", Fifth Edition, McGraw Hill, Singapore.

[7] Tomlinson, M.J, "*Foundation Design and Construction*", Seventh Edition, Prentice Hall.

[8] Salgado Rodrigo (2008), "The Engineering of Foundations", McGraw Hill.

[9]Das, B.M., (2011). "*Principles of Foundation Engineering*". Seventh Edition, Cengage Learning, USA.

[10] Das, B.M., "*Fundamentals of Geotechnical Engineering*". Fourth Edition, Cengage Learning, USA.

[11] http://teaching.ust.hk/~civl607d/Standard%20penetration%20test.pdf

[12] Phoon, Kok-Kwang (1995), "*Reliability-based design of foundations for transmission line structures*", Diss. Cornell University.

[13] http://en.wikipedia.org/wiki/Coefficient_of_variation

[14] Macnab Alan, (2002), "Earth Retaining Systems Handbook", McGraw Hill, New York.

[15] Naval Facilities Command Engineering (NAVFAC), US Army Corps of Engineers (1986 & 1997), "*Foundations and Earth Structures*".

[16] www.deepexcavation.com

[17] The Committee on Earth Retaining Structures of The Geo Institute (ASCE) (1997), "*Geotechnical Special Publication No 74: Guidelines of Engineering Practice For Braced and Tied Back Excavations*".

[18] Federal Highway Administration (FHWA), US Department of Transportation Geotechnical Engineering Circular No.4 (1999), "*Ground Anchors and Anchored System*".

[19] Canadian Geotechnical Society Foundations Committee (2007), "*Canadian foundation engineering manual*", 4th Edition, Canadian Geotechnical Society.

APPENDIX A

- **List of Sites in Lahore**
- **Soil Profiles for All Zones**
- **Statistical Evaluation for All Zones**
- **Statistical Evaluation Calculations for Zone 1**

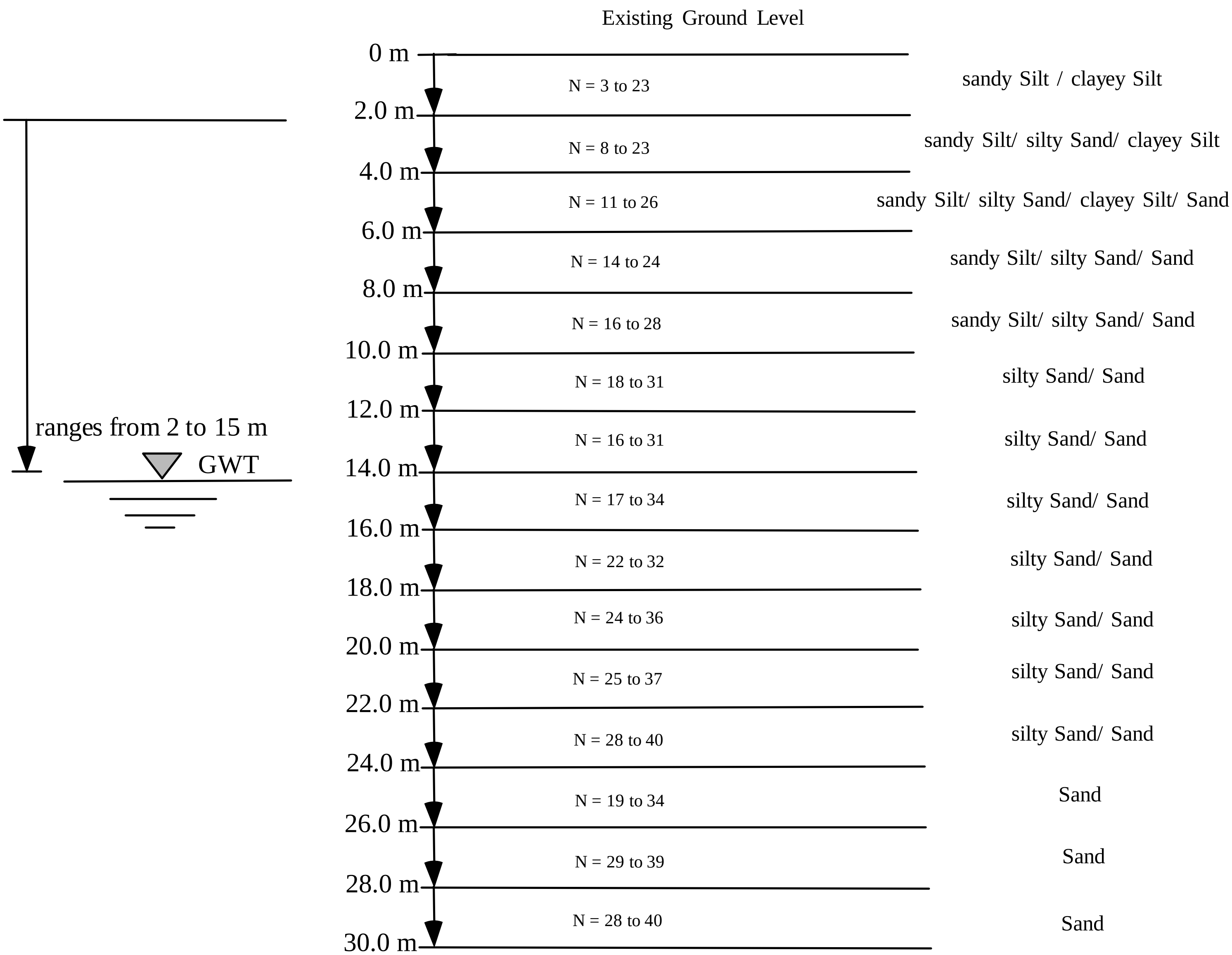
LIST OF SITES/PROJECTS			
Sr No.	**Zone**	**Project**	**Location**
1	1	Construction at National Hospital	Defence
2	1	Construction of House	Bedian Road
3	1	Construction of Megaplex Cinema	New Airport Road
4	1	Construction of Biogas Power Generation Plant	Burki Road
5	1	Construction of Engineering Block at Lahore University	Defence Road
6	1	Construction of Gym	Sarwar Road
7	1	Construction of Afaq Butt Residence	Bedian Road
8	1	Construction of House	Arif Jan Road, Cantt
9	1	Construction of Commercial Building	Bedian Road
10	1	Construction of Commercial Building	DHA Road
11	1	Construction of Apartment Buildings	Ghazi Road
12	1	ASKARI XI	Cantt
13	1	Construction of Ashiana Housing Scheme	Attari Saroba, Cantt
14	2	Construction of New Packing Hall in Aftab Qurshi Factory	Chung
15	2	Construction of Building	Quaid-e-Azam Town Scheme
16	2	Construction of Leach Field at Lahore Clear Project	Raiwind Road
17	2	Construction of Building at University of Education	Township
18	2	Construction of Golden Food Industries Sunder Industrial Estate	Township
19	2	Construction of Buildings at Academy of Arabic Sciences	Raiwind Road
20	2	Construction of Bridge	Gulshan-e-Ravi
21	2	Construction of Building/ Hospital	Kot Lakhpat
22	2	Construction of Steam Turbine Foundation at Japan Power Generation	Raiwind Road
23	2	Construction of Secondary Clarifier and Aeration Tank	Raiwind Road
24	2	Construction of Residence of Dr. Naseem Maqsood	Raiwind Road
25	2	Construction of Bridge at Style Textile LIM	Kot Lakhpat

26	2	Construction of Golden Food Industries Sunder Industrial Estate	Raiwind Road
27	2	Construction of Lahore Clear Project	Raiwind Road
28	2	Construction of Building at Venus Pakistan PVT LIM	Raiwind Road
29	2	Construction of Commercial Building and Mosque	WAPDA Town
30	2	Construction of Overhead Tank	EME
31	2	Construction of Wet Well	EME
32	2	Construction of Malik Flour and General Mills at Sunder Industrial Estate	Raiwind Road
33	2	Construction of Sheds at Sohail Engineering	Main Bund Road
34	2	Construction of Machine Foundation	Bhugtian, Defence Road
35	2	Construction of Mohammed Amin Vocational Training Institute	Saidpur, Multan Road
36	2	Installing Tower for WARID	Bund Road
37	2	Lahore Waste Management Company at Sunder Village	Multan Road
38	3	Construction of Plaza	M.M. Alam Road, Gulberg
39	3	Construction of Plaza	Hussain Chowk, Gulberg 3
40	3	Construction of Swimming Pool	Maratab Ali Road, Gulberg
41	3	Construction of Interhome Heights	Hali Road, Gulberg 3
42	3	Construction of Tivoli Tower	Upper Mall
43	3	Construction of Varioline Kitchens	Gulberg 3
44	3	Construction of Cafeteria at Arif Memorial Hospital	Ferozepur Road
45	3	Construction of Rahmat Nazir Cardiac Complex	Ferozepur Road
46	3	Construction of Talha’s Ploy Clinic	Ferozepur Road
47	3	LDA Housing Scheme	Ferozepur Road
48	3	Construction of JW Marriot Hotel	Gulberg 2
49	4	Geotechnical Study of Lohari Gate	Interior City
50	4	Construction of Multi-Storied Building at Tent Factory	Ravi Road
51	4	Construction of Askari Bank	Baghbapura, Shalimar Link Road
52	4	Construction of Building for Master Paint Industries PVT LIM	Shalimar Town
53	4	Construction at Fatima Jinnah Institute of Dental Sciences	Mall Road

54	4	National Bank Building	Upper Mall
55	5	Construction of Al-Jalil Garden Housing Scheme	Faizpur
56	5	Construction of Road over Bridge between Shadara and Kalashakako Stations	Shahdara to Kalashakako
57	5	Construction of Shahab Banquet Hall	G.T Road
58	5	Construction of Building at Chatha Colony	Sharqpur Road, Begum Kot, Shahdara
59	5	Construction at Ravi River Front	Karol to Babu Sabu
60	5	Construction at Ravi River Front	Babu Sabu to Mohlanwal

SOIL PROFILES FOR ALL ZONES

SOIL LOGS PROFILE FOR ZONE 1

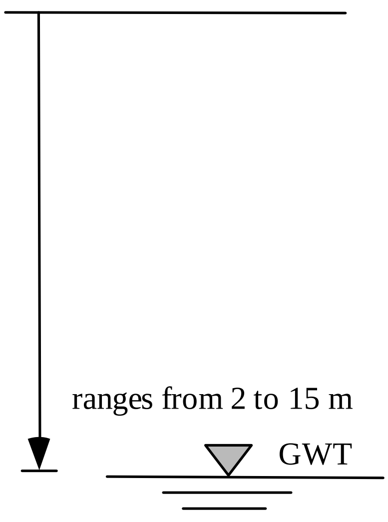


NOTE

1) The soil types may vary and the possible depth variations are presented below:
   sandy Silt varies from 0 to 10 m
   clayey Silt varies from 0 to 6 m
   silty Sand varies from 2 to 25 m
   Sand varies from 5 to 30 m

2) The SPT-N values are uncorrected

GENERALIZED SOIL PROPERTIES PROFILE FOR ZONE 1



Existing Ground Level

0 m – 2.0 m	LL = 21 to 36	PI = 0 to 13	c = 3 to 5 kPa	E =3 to 15 MPa	φ = 17 to 21 degrees	γ = 16 to 18 kN/m^3
2.0 m – 4.0 m	LL = 18 to 34	PI = 0 to 8	c = 1 to 5 kPa	E = 4 to 15 MPa	φ = 16 to 25degrees	γ = 16 to 18 kN/m^3
4.0 m – 6.0 m	LL = 24 to 33	PI = 0 to 7	c = 1 to 4 kPa	E = 4 to 16 MPa	φ = 19 to 27degrees	γ = 16 to 18 kN/m^3
6.0 m – 8.0 m	LL = 23 to 29	PI = 0 to 4	c = 0 to 3 kPa	E = 5 to 16 MPa	φ = 23 to 27 degrees	γ = 16 to 17 kN/m^3
8.0 m – 10.0 m	LL = 23 to 29	PI = 0 to 4	c = 1 to 2 kPa	E = 12 to 17 MPa	φ = 23 to 49 degrees	γ = 16 to 17 kN/m^3
10.0 m – 12.0 m	LL = -	PI = NP	c = 0 to 1 kPa	E = 13 to 17 MPa	φ = 26 to 29 degrees	γ = 16 to 17 kN/m^3
12.0 m – 14.0 m	LL = -	PI = NP	c = 0 to 1 kPa	E = 12 to 17 MPa	φ = 26to 31 degrees	γ = 16 to 17 kN/m^3
14.0 m – 16.0 m	LL = -	PI = NP	c = 1 to 2 kPa	E = 12to 17 MPa	φ = 23 to 27 degrees	γ = 16 to 17 kN/m^3
16.0 m – 18.0 m	LL = -	PI = NP	c = 1 to 2 kPa	E = 13 to 15 MPa	φ = 22 to 27 degrees	γ = 16 to 17 kN/m^3
18.0 m – 20.0 m	LL = -	PI = NP	c = 0 to 1 kPa	E = 13 to 15 MPa	φ = 23 to 26 degrees	γ = 16 to 17 kN/m^3
20.0 m – 22.0 m	LL = -	PI = NP	c = NA	E = 13 to 15 MPa	φ = NA	γ = 16 to 17 kN/m^3
22.0 m – 24.0 m	LL = -	PI = NP	c = NA	E = 14 to 15 MPa	φ = NA	γ = 16 to 17 kN/m^3
24.0 m – 26.0 m	LL = -	PI = NP	c = NA	E = 12 to 15 MPa	φ = NA	γ = 16 to 17 kN/m^3
26.0 m – 28.0 m	LL = -	PI = NP	c = NA	E = 15 to 16 MPa	φ = NA	γ = 16 to 17 kN/m^3
28.0 m – 30.0 m	LL = -	PI = NP	c = NA	E = 15 to 16 MPa	φ = NA	γ = 16 to 17 kN/m^3

NOTE

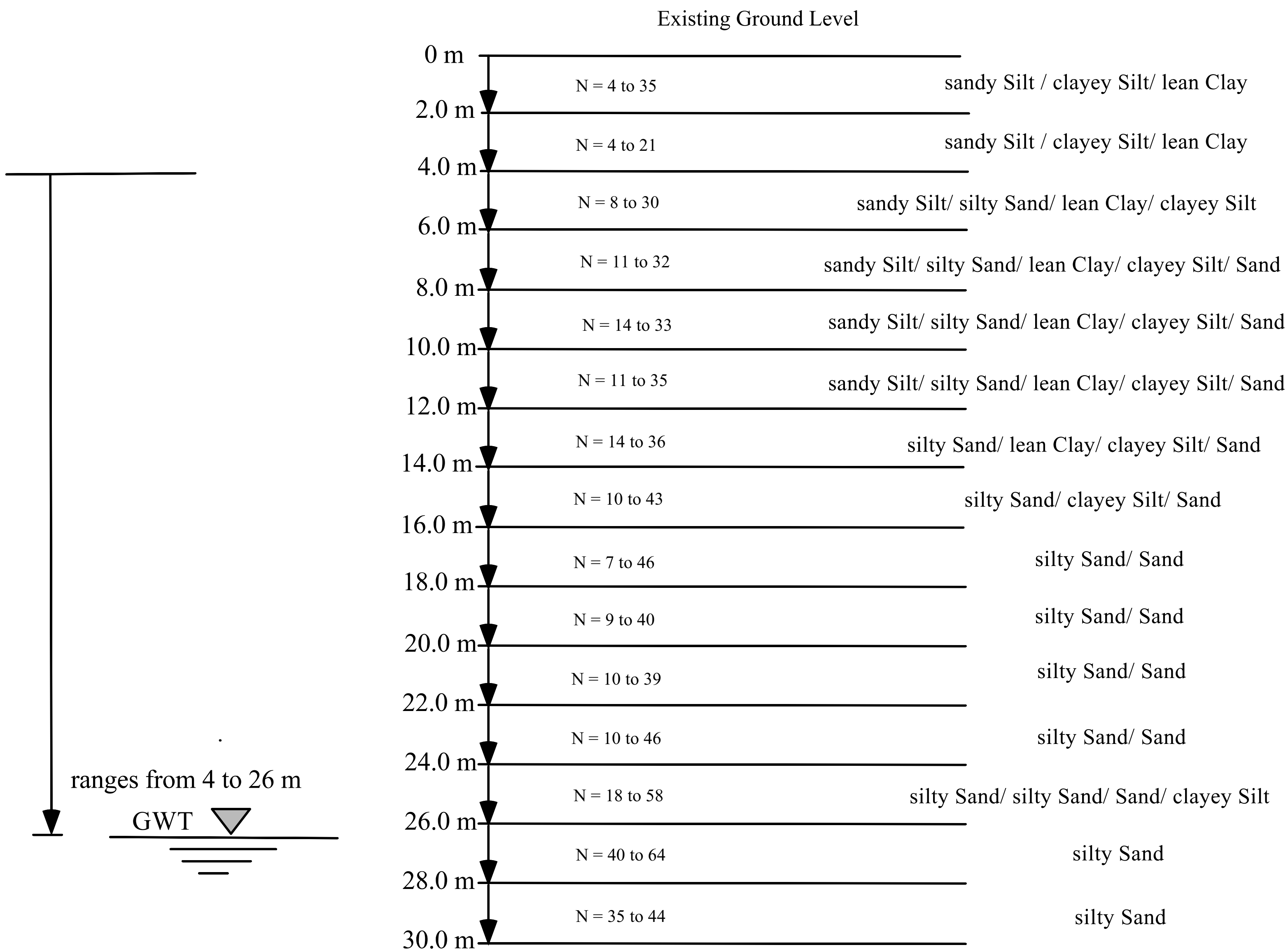
1) E = 300(N+6) for clayey Silt / sandy Silt and
E = 500(N +15) for silty Sand /Sand (Bowles, Joseph. E, "Foundation Analysis and Design", 5th Edition, The McGraw-Hill Companies, Inc.)

2) The values of c and φ were obtained from Direct Shear Tests.

3) The values of all the soil properties are precise upto two significant figures.

4) NA = Not Available, NP = Non-Plastic

SOIL LOGS PROFILE FOR ZONE 2

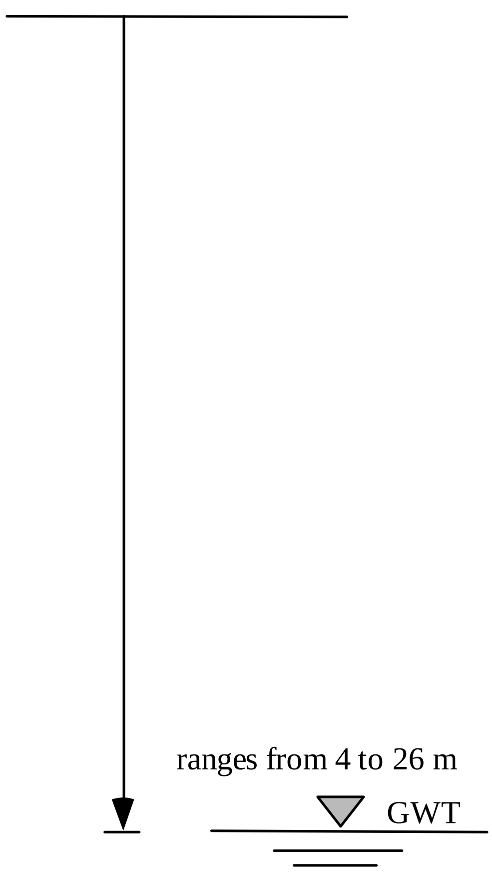


NOTE

1) The soil types may vary and the possible depth variations are presented below:
   - sandy Silt varies from 0 to 26 m
   - clayey Silt varies from 0 to 15 m
   - silty Sand varies from 3 to 30 m
   - Sand varies from 6 to 25 m
   - lean Clay varies from 0 to 15 m

2) The SPT-N values are uncorrected

GENERALIZED SOIL PROPERTIES PROFILE FOR ZONE 2



Existing Ground Level

0 m – 2.0 m	LL = 21 to 30	PI = 0 to 9	c = 2 to 4kPa	E =3 to 12 MPa	φ = 19 to 23 degrees	γ = 16 to 19 kN/m^3
2.0 m – 4.0 m	LL = 19 to 38	PI = 0 to 16	c = 0 to 4.kPa	E = 3 to 10 MPa	φ = 19 to 24 degrees	γ = 15 to 18 kN/m^3
4.0 m – 6.0 m	LL = 18 to 31	PI = 0 to 20	c = 1 to 3 kPa	E = 4 to 19 MPa	φ = 19 to 27 degrees	γ = 16 to 18 kN/m^3
6.0 m – 8.0 m	LL = 26 to 34	PI = 0 to 9	c = 1 to 3 kPa	E = 5 to 19 MPa	φ = 21 to 29 degrees	γ = 16 to 18 kN/m^3
8.0 m – 10.0 m	LL = 33 to 35	PI = 0 to 9	c = 0 to2 kPa	E = 5 to 18 MPa	φ = 24 to 28 degrees	γ = 15 to 17 kN/m^3
10.0 m – 12.0 m	LL = 26 to 36	PI = 0 to 10	c = 0 to 2 kPa	E = 5 to 19 MPa	φ = 24 to 27 degrees	γ = 15 to 17 kN/m^3
12.0 m – 14.0 m	LL = 30 to 33	PI = 0 to10	c = 0 to 2 kPa	E = 6 to 18 MPa	φ = 24 to 26 degrees	γ = 16 to 17 kN/m^3
14.0 m – 16.0 m	LL = 15 to 18	PI = NA	c = 0 to 1 kPa	E = 5 to 20 MPa	φ = 25 to 27 degrees	γ = 15 to 18 kN/m^3
16.0 m – 18.0 m	LL = NA	PI = NA	c = 0 to 1 kPa	E = 10 to 18 MPa	φ = 25 to 26degrees	γ = 15 to 17 kN/m^3
18.0 m – 20.0 m	LL = NA	PI = NA	c = 0 to 1 kPa	E = 10 to 18 MPa	φ = 25 to 31 degrees	γ = 15 to 17 kN/m^3
20.0 m – 22.0 m	LL = NA	PI = NA	c = 0 to 1 kPa	E = 11 to 17 MPa	φ = 27 to 30 degrees	γ = 15 to 17 kN/m^3
22.0 m – 24.0 m	LL = NA	PI = NA	c = 0 to 1 kPa	E = 11 to 19 MPa	φ = 27 to 30 degrees	γ = 15 to 17 kN/m^3
24.0 m – 26.0 m	LL = 18 to 26	PI = 0 to 5	c = 0 to 1 kPa	E = 6 to 19 MPa	φ = 30 to 32 degrees	γ = 16 to 18 kN/m^3
26.0 m – 28.0 m	LL = 23 to 25	PI = 0 to 5	c = 0 to 1 kPa	E = 6 to 19 MPa	φ = 26 to 28 degrees	γ = 16 to 18 kN/m^3
28.0 m – 30.0 m	LL = NA	PI = NA	c = 0 to 1 kPa	E = 14 to 15 MPa	φ = 24 to 26 degrees	γ = 16 to 17 kN/m^3

NOTE

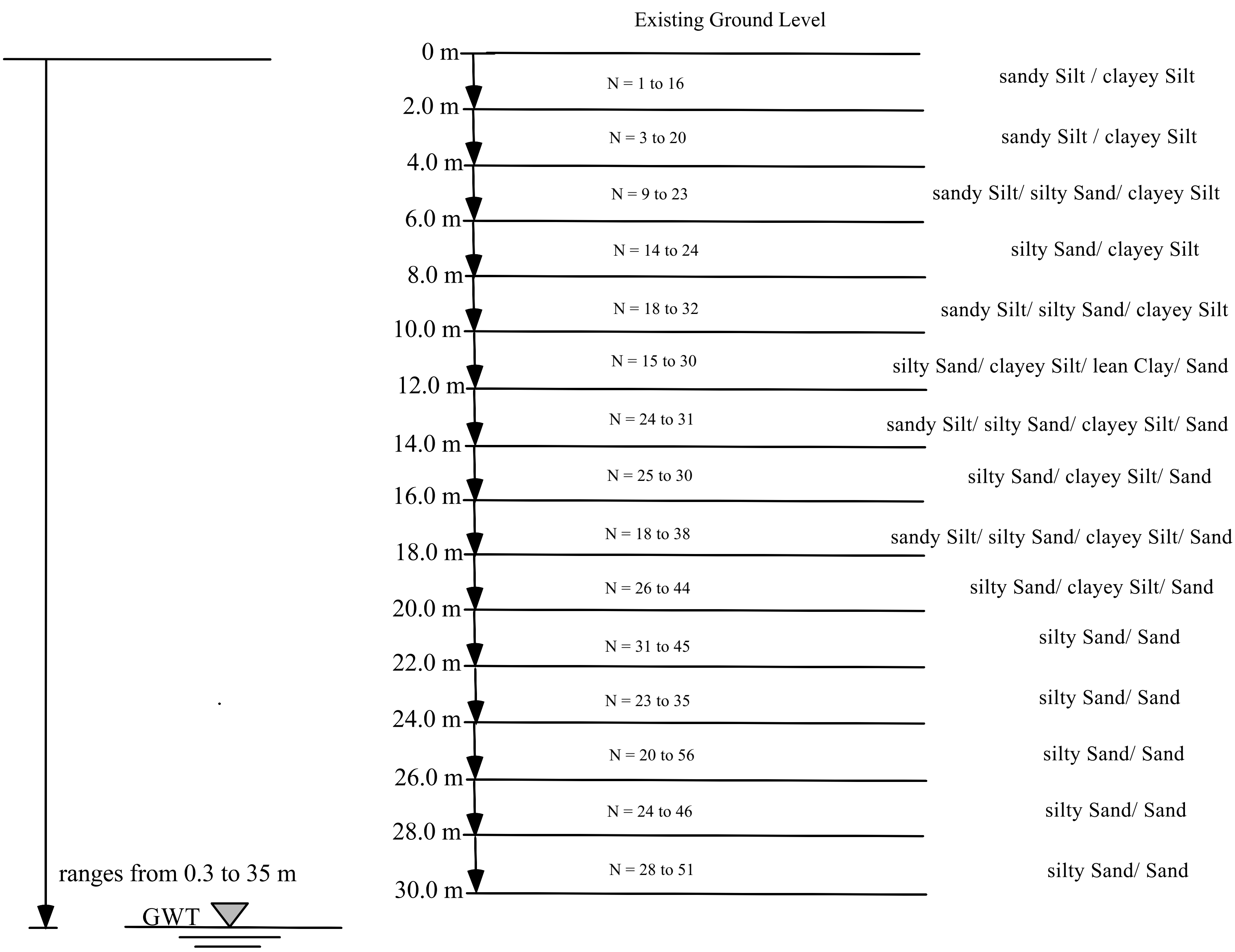
1) E = 300(N+6) for clayey Silt / sandy Silt and
E = 500(N +15) for silty Sand /Sand (Bowles, Joseph. E, "Foundation Analysis and Design", 5th Edition, The McGraw-Hill Companies, Inc.)

2) The values of c and φ were obtained from Direct Shear Tests.

3) The values of all the soil properties are precise upto two significant figures.

4) NA = Not Available

SOIL LOGS PROFILE FOR ZONE 3

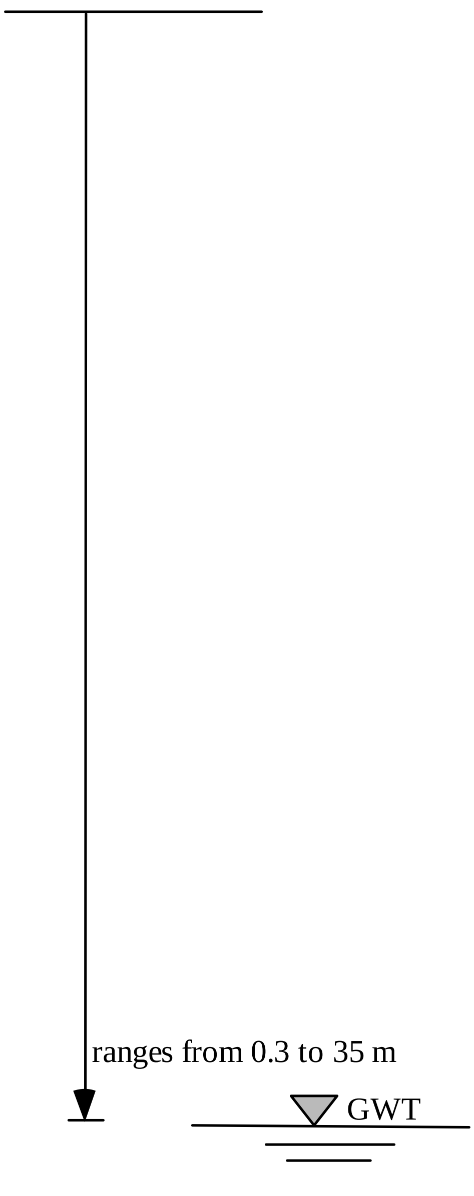


NOTE

1) The soil types may vary and the possible depth variations are presented below:
   - sandy Silt varies from 0 to 15 m
   - clayey Silt varies from 0 to 20 m
   - silty Sand varies from 5 to 30 m
   - Sand varies from 14 to 30 m
   - lean Clay varies from 10 to 17 m

2) The SPT-N values are uncorrected

GENERALIZED SOIL PROPERTIES PROFILE FOR ZONE 3



Existing Ground Level

0 m – 2.0 m	LL = 18 to 32	PI = 0 to 8	c = 2 to 5 kPa	E = 2 to 6 MPa	φ = 16 to 23 degrees	γ = 16 to 18 kN/m^3
2.0 m – 4.0 m	LL = 21 to 30	PI = 0 to 7	c = 2 to 4 kPa	E = 4 to 8 MPa	φ = 19 to 25 degrees	γ = 16 to 18 kN/m^3
4.0 m – 6.0 m	LL = 18 to 33	PI = 0 to 6	c = 0 to 4 kPa	E = 4 to 17 MPa	φ = 18 to 27 degrees	γ = 15 to 18 kN/m^3
6.0 m – 8.0 m	LL = 20 to 28	PI = 0 to 4	c = 0 to 2 kPa	E = 4 to 17 MPa	φ = 22 to 27 degrees	γ = 16 to 17 kN/m^3
8.0 m – 10.0 m	LL = 28 to 38	PI = 0 to 11	c = 0 to 2 kPa	E = 13 to 19 MPa	φ = 23 to 27 degrees	γ = 16 to 17 kN/m^3
10.0 m – 12.0 m	LL = 25 to 29	PI = 0 to 8	c = 0 to 2 kPa	E = 4 to 18 MPa	φ = 22 to 29 degrees	γ = 15 to 17 kN/m^3
12.0 m – 14.0 m	LL = 25 to 29	PI = 0 to 8	c = 0 to 1 kPa	E = 13 to 19 MPa	φ = 24 to 27 degrees	γ = 16 to 17 kN/m^3
14.0 m – 16.0 m	LL = 29 to 32	PI = 0 to 8	c = 0 to 1 kPa	E = 13 to 17 MPa	φ = 23 to 31 degrees	γ = 15 to 17 kN/m^3
16.0 m – 18.0 m	LL = 25 to 26	PI = 0 to 6	c = 0 to 1 kPa	E = 4 to 18 MPa	φ = 26 to 31 degrees	γ = 15 to 17 kN/m^3
18.0 m – 20.0 m	LL = -	PI = NP	c = 0 to 1 kPa	E = 5 to 17 MPa	φ = 25 to 29 degrees	γ = 16 to 17 kN/m^3
20.0 m – 22.0 m	LL = -	PI = NP	c = 0 to 1 kPa	E = 13 to 17 MPa	φ = 26 to 27 degrees	γ = 16 to 17 kN/m^3
22.0 m – 24.0 m	LL = 29 to 30	PI = 0 to 5	c = 0 to 1 kPa	E = 12 to 14 MPa	φ = 27 to 33 degrees	γ = 16 to 17 kN/m^3
24.0 m – 26.0 m	LL = 24 to 29	PI = 3 to 12	c = 0 to 4 kPa	E = 5 to 18 MPa	φ = 20 to 29 degrees	γ = 16 to 17 kN/m^3
26.0 m – 28.0 m	LL = 29 to 30	PI = 0 to 6	c = 0 to 1 kPa	E = 5 to 16 MPa	φ = 21 to 29	γ = 16 to 17 kN/m^3
28.0 m – 30.0 m	LL = 25 to 26	PI = 0 to 3	c = 0 to 4 kPa	E = 5 to 16 MPa	φ = 23 to 31 degrees	γ = 16 to 17 kN/m^3

NOTE

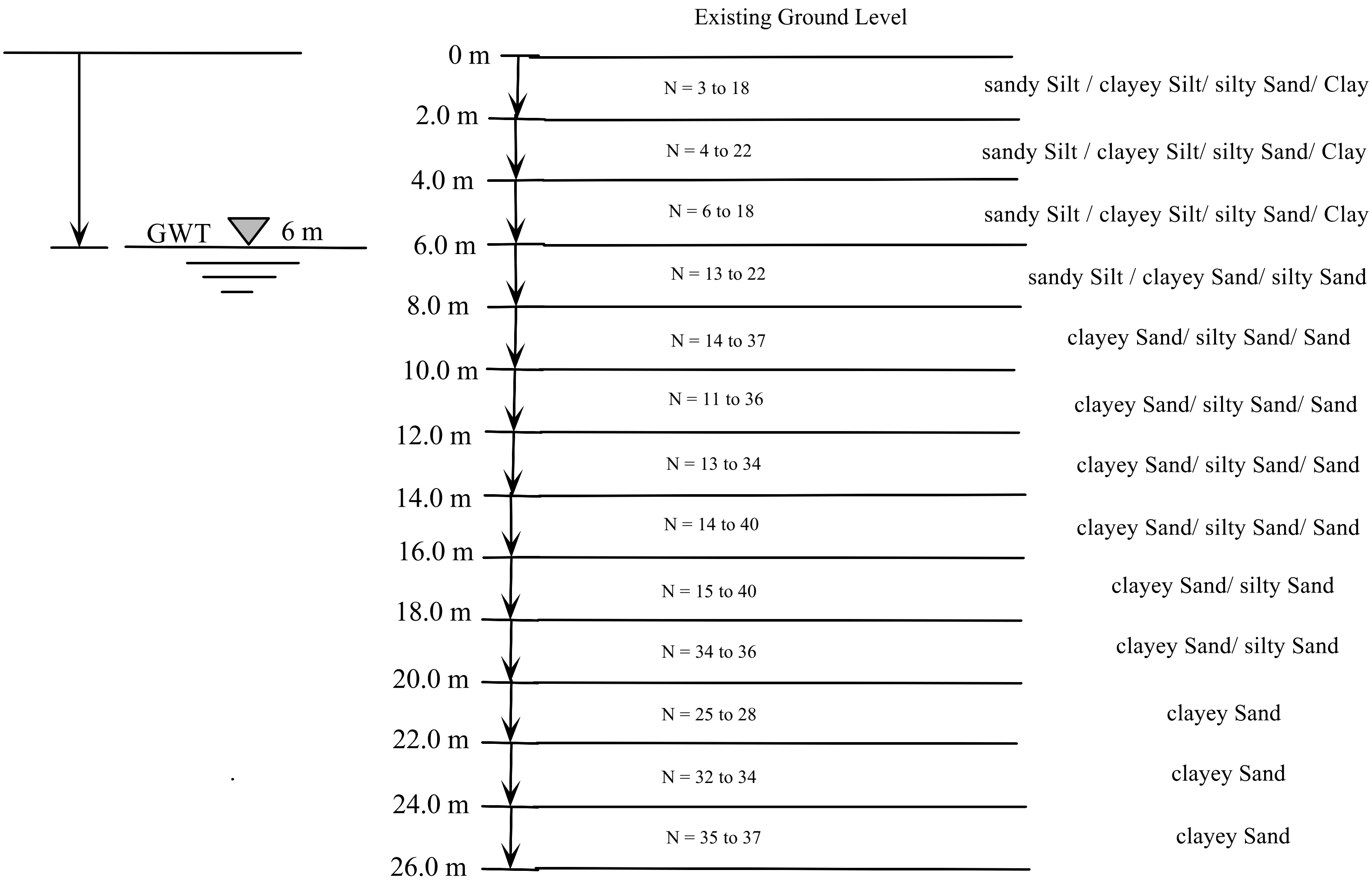
1) E = 300(N+6) for clayey Silt / sandy Silt and
 E = 500(N +15) for silty Sand /Sand (Bowles, Joseph. E, "Foundation Analysis and Design", 5th Edition, The McGraw-Hill Companies, Inc.)

2) The values of c and φ were obtained from Direct Shear Tests.

3) The values of all the soil properties are precise upto two significant figures.

4) NP = Non-Plastic

SOIL LOGS PROFILE FOR ZONE 4

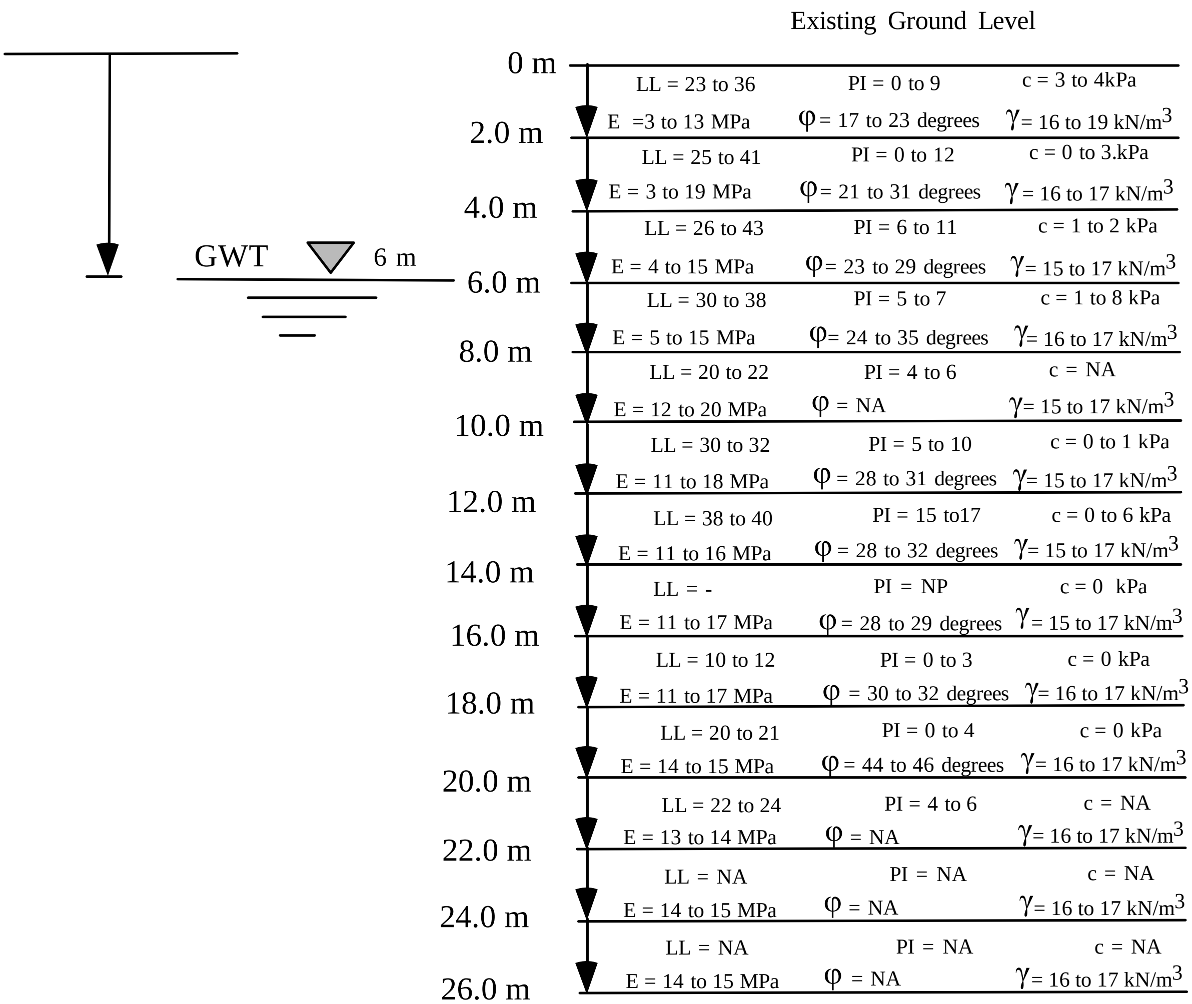


NOTE

1) The soil types may vary and the possible depth variations are presented below:
   - sandy Silt varies from 0 to 7.5 m
   - clayey Silt varies from 0 to 4.5 m
   - silty Sand varies from 0 to 20 m
   - Sand varies from 9 to 15 m
   - Clay varies from 0 to 5 m
   - clayey Sand varies from 6 to 25 m

2) The SPT-N values are uncorrected

GENERALIZED SOIL PROPERTIES PROFILE FOR ZONE 4



NOTE

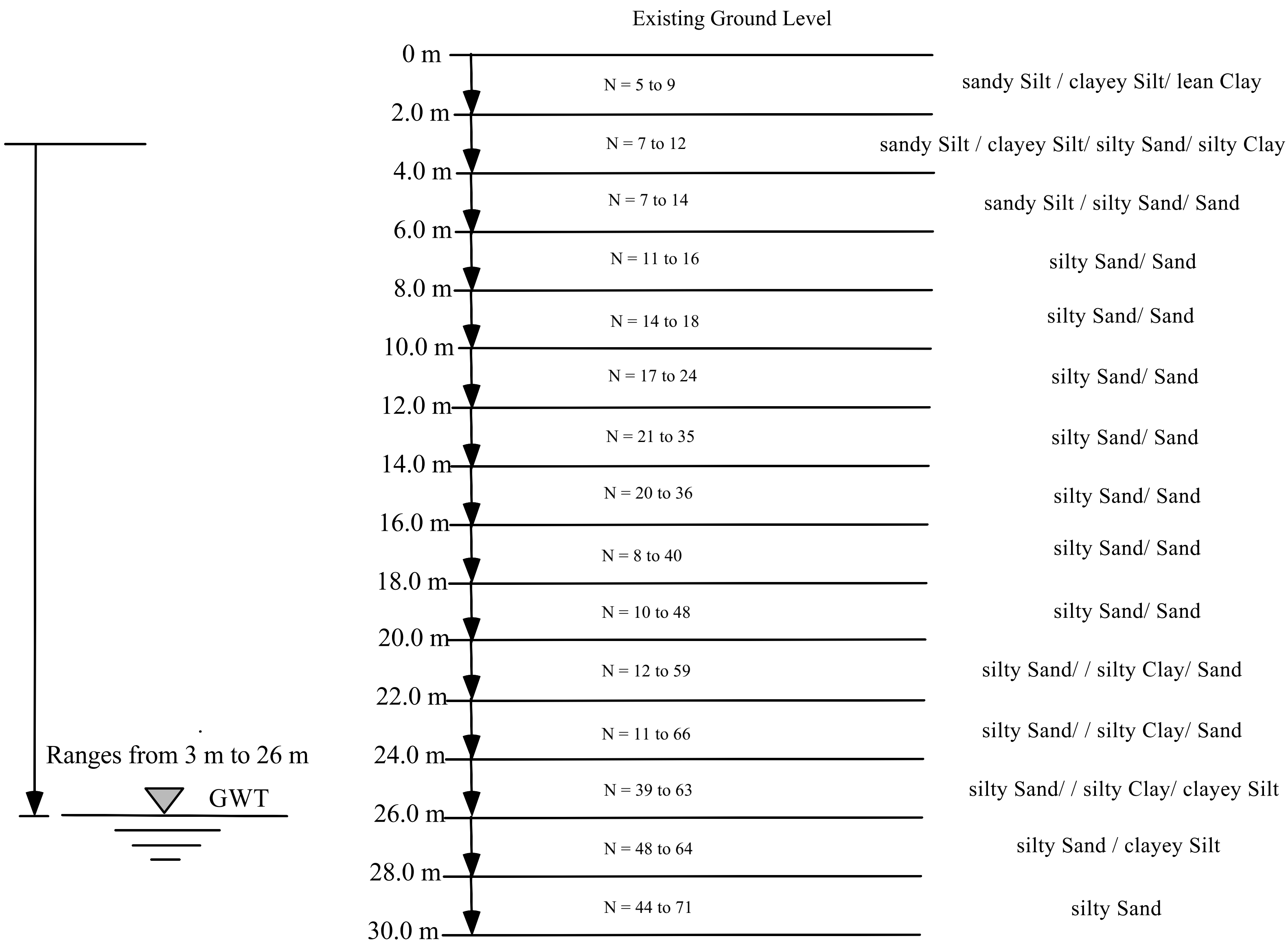
1) $E = 300(N+6)$ for clayey Silt / sandy Silt and
   $E = 500(N+15)$ for silty Sand /Sand (Bowles, Joseph. E, "Foundation Analysis and Design", 5th Edition, The McGraw-Hill Companies, Inc.)

2) The values of c and φ were obtained from Direct Shear Tests.

3) The values of all the soil properties are precise upto two significant figures.

4) NA = Not Avalilable, NP = Non-Plastic

SOIL LOGS PROFILE FOR ZONE 5

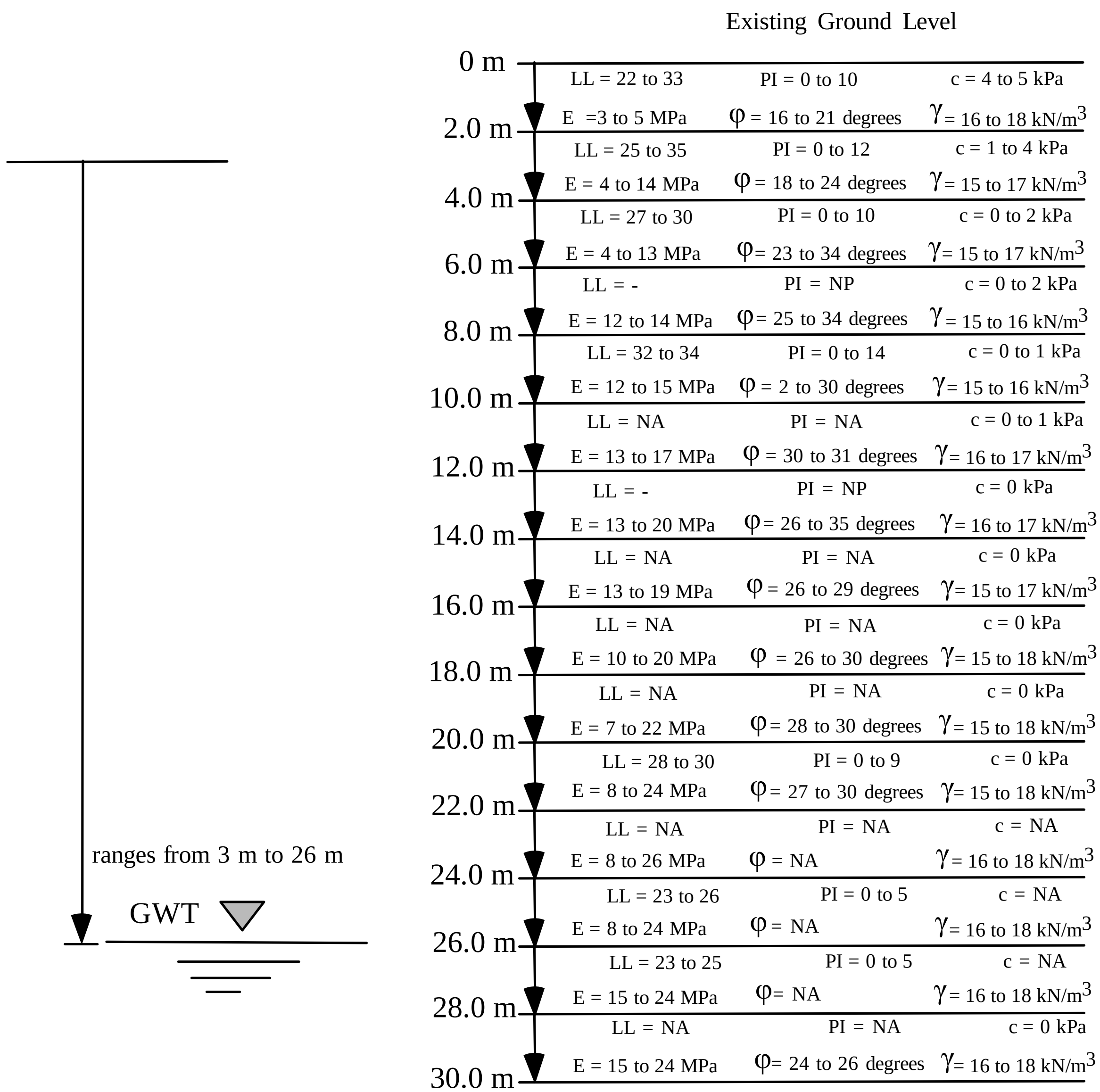


NOTE

1) The soil types may vary and the possible depth variations are presented below:
   - sandy Silt varies from 0 to 25 m
   - clayey Silt varies from 0 to 27 m
   - silty Sand varies from 3 to 30 m
   - Sand varies from 5 to 25 m
   - lean Clay varies from 0 to 1 m
   - silty Clay varies from 2 to 23 m

2) The SPT-N values are uncorrected

GENERALIZED SOIL PROPERTIES PROFILE FOR ZONE 5



NOTE

1) $E = 300(N+6)$ for clayey Silt / sandy Silty and
$E = 500(N+15)$ for silty Sand /Sand (Bowles, Joseph. E, "Foundation Analysis and Design", 5th Edition, The McGraw-Hill Companies, Inc.)

2) The values of c and φ were obtained from Direct Shear Tests.

3) The values of all the soil properties are precise upto two significant figures.

4) NA = Not Available, NP = Non Plastic

STATISTICAL EVALUATION TABLES

STATISTICAL EVALUATION FOR ZONE 1				
Property	**Test**	**Soil Type**	**Calculated COV (%)**	**Published COV* (%)**
φ	Direct Shear	clayey Silt/ sandy Silt	8	5 to 20
		silty Sand/ sandy Silt/ clayey Silt	12	
		silty Sand/ sandy Silt/ clayey Silt/ Sand	11	
		silty Sand/ sandy Silt/ Sand	22	
		silty Sand/ Sand	8	
		Sand	1	
N	SPT	clayey Silt/ sandy Silt	76	25 to 50
		silty Sand/ sandy Silt/ clayey Silt	27	
		silty Sand/ sandy Silt/ clayey Silt/ Sand	23	
		silty Sand/ sandy Silt/ Sand	21	
		silty Sand/ Sand	23	
		Sand	NA	

STATISTICAL EVALUATION FOR ZONE 2				
Property	**Test**	**Soil Type**	**Calculated COV (%)**	**Published COV* (%)**
φ	Direct Shear	sandy Silt/ clayey Silt/ lean Clay	6	5 to 20
		sandy Silt/ clayey Silt/ lean Clay/ silty Sand	8	
		sandy Silt/ clayey Silt/ lean Clay/ silty Sand/ Sand	8	
		clayey Silt/ lean Clay/ silty Sand/ Sand	8	
		clayey Silt/ silty Sand/ Sand	4	
		silty Sand/ Sand	6	
		silty Sand/ Sand/ sandy Silt/ clayey Silt	5	
		silty Sand	4	
N	SPT	sandy Silt/ clayey Silt/ lean Clay	51	25 to 50
		sandy Silt/ clayey Silt/ lean Clay/ silty Sand	30	
		sandy Silt/ clayey Silt/ lean Clay/ silty Sand/ Sand	27	
		clayey Silt/ lean Clay/ silty Sand/ Sand	26	
		clayey Silt/ silty Sand/ Sand	36	
		silty Sand/ Sand	33	
		silty Sand/ Sand/ sandy Silt/ clayey Silt	33	
		silty Sand	20	

STATISTICAL EVALUATION FOR ZONE 3				
Property	**Test**	**Soil Type**	**Calculated COV (%)**	**Published COV* (%)**
φ	Direct Shear	sandy Silt/ clayey Silt	10	5 to 20
		sandy Silt/ silty Sand/ clayey Silt	10	
		silty Sand/ clayey Silt	6	
		silty Sand/ clayey Silt/ lean Clay/ Sand	9	
		silty Sand/ clayey Silt/ sandy Silt/ Sand	11	
		silty Sand/ clayey Silt/ Sand	8	
		silty Sand/ Sand	13	
N	SPT	sandy Silt/ clayey Silt	53	25 to 50
		sandy Silt/ silty Sand/ clayey Silt	26	
		silty Sand/ clayey Silt	20	
		silty Sand/ clayey Silt/ lean Clay/ Sand	14	
		silty Sand/ clayey Silt/ sandy Silt/ Sand	6	
		silty Sand/ clayey Silt/ Sand	13	
		silty Sand/ Sand	23	

STATISTICAL EVALUATION FOR ZONE 4				
Property	**Test**	**Soil Type**	**Calculated COV (%)**	**Published COV* (%)**
φ	Direct Shear	sandy Silt/ clayey Silt/ silty Sand/ Clay	16	5 to 20
		sandy Silt/ silty Sand/ clayey Sand	18	
		silty Sand/ clayey Sand/ Sand	6	
		silty Sand/ clayey Sand	18	
		clayey Sand	NA	
N	SPT	sandy Silt/ clayey Silt/ silty Sand/ Clay	39	25 to 50
		sandy Silt/ silty Sand/ clayey Sand	16	
		silty Sand/ clayey Sand/ Sand	36	
		silty Sand/ clayey Sand	27	
		clayey Sand	11	

STATISTICAL EVALUATION FOR ZONE 5				
Property	**Test**	**Soil Type**	**Calculated COV (%)**	**Published COV* (%)**
φ	Direct Shear	clayey Silt/ sandy Silt/ lean Clay	11	5 to 20
		sandy Silt/ silty Sand/ clayey Silt/ silty Clay	11	
		sandy Silt/ silty Sand/ Sand	7	
		silty Sand/ Sand	11	
		silty Sand/ silty Clay/ Sand	NA	
		silty Sand/ silty Clay/ clayey Silt	NA	
		silty Sand/ clayey Silt	NA	
		silty Sand	NA	
N	SPT	clayey Silt/ sandy Silt/ lean Clay	22	25 to 50
		sandy Silt/ silty Sand/ clayey Silt/ silty Clay	18	
		sandy Silt/ silty Sand/ Sand	15	
		silty Sand/ Sand	40	
		silty Sand/ silty Clay/ Sand	42	
		silty Sand/ silty Clay/ clayey Silt	18	
		silty Sand/ clayey Silt	18	
		silty Sand	19	

NOTE

*The published values of COV were obtained from Table 4-11(Phoon, Kok-Kwang, "*Reliability-based design of foundations for transmission line structures"*, Diss. Cornell University, 1995.)

*The SPT-N values are uncorrected

*****NA = Not Available

STATISTICAL EVALUATION CALCULATIONS FOR ZONE 1

Statistical Evaluation for SPT-N Values

STATISTICAL EVALUATION FOR FIRST LAYER i.e. sandy Silt/ clayey Silt/ lean Clay

No	**SPT-N**	$X - \bar{X}$	$(X - \bar{X})^2$
1	3	-6	36
2	6	-3	9
3	23	14	196
4	4	-5	25
5	8	-1	1
6	6	-3	9
7	9	0	0
8	10	1	1
9	14	5	25
10	4	-5	25
11	8	-1	1
12	9	0	0
13	7	-2	4
SUM	111		554

Average = 9

$$s = \sqrt{\frac{\sum (X - \bar{X})^2}{n}} \quad = 6.53$$

To find the COV

COV = s/average = 0.76 = 76%

STATISTICAL EVALUATION FOR SECOND LAYER i.e. sandy Silt/silty Sand/ lean Clay

No	**SPT-N**	$X - \overline{X}$	$(X - \overline{X})^2$
1	10	-2.0	4
2	10	-2.0	4
3	19	7.0	49
4	8	-4.0	16
5	8	-4.0	16
6	9	-3.0	9
7	14	2.0	4
8	15	3.0	9
9	14	2.0	4
10	16	4.0	16
11	9	-3.0	9
12	13	1.0	1
13	12	0.0	0
14	11	-1.0	1
SUM	168		142

Average = 12

$$s = \sqrt{\frac{\sum (X - \overline{X})^2}{n}} = 3.18$$

To find the COV

COV = s/average = 0.27 = 27%

STATISTICAL EVALUATION FOR THIRD LAYER i.e. sandy Silt/silty Sand/ Sand/ lean Clay

No	**SPT-N**	$X - \bar{X}$	$(X - \bar{X})^2$
1	13	-3.0	9
2	13	-3.0	9
3	16	0.0	0
4	15	-1.0	1
5	11	-5.0	25
6	11	-5.0	25
7	14	-2.0	4
8	13	-3.0	9
9	11	-5.0	25
10	17	1.0	1
11	15	-1.0	1
12	20	4.0	16
13	26	10.0	100
14	23	7.0	49
15	16	0.0	0
16	15	-1.0	1
17	17	1.0	1
18	14	-2.0	4
19	17	1.0	1
20	16	0.0	0
21	19	3.0	9
22	18	2.0	4
23	14	-2.0	4
SUM	364		298

Average = 16

$$s = \sqrt{\frac{\sum (X - \bar{X})^2}{n}} = 3.60$$

To find the COV

COV = s/average = 0.23 = 23%

STATISTICAL EVALUATION FOR FOURTH LAYER i.e. silty Sand/ sandy Silt/ Sand

No	**SPT-N**	$X - \overline{X}$	$(X - \overline{X})^2$
1	22	-4.0	16
2	23	-3.0	9
3	17	-9.0	81
4	18	-8.0	64
5	20	-6.0	36
6	16	-10.0	100
7	23	-3.0	9
8	19	-7.0	49
9	22	-4.0	16
10	18	-8.0	64
11	26	0.0	0
12	25	-1.0	1
13	31	5.0	25
14	31	5.0	25
15	18	-8.0	64
16	19	-7.0	49
17	20	-6.0	36
18	22	-4.0	16
19	24	-2.0	4
20	25	-1.0	1
21	28	2.0	4
22	26	0.0	0
23	29	3.0	9
24	30	4.0	16
25	28	2.0	4
26	31	5.0	25
27	30	4.0	16
28	34	8.0	64
29	25	-1.0	1
30	26	0.0	0
31	29	3.0	9
32	31	5.0	25
33	33	7.0	49
34	32	6.0	36
35	36	10.0	100
36	37	11.0	121
37	40	14.0	196

38	28	2.0	4
39	26	0.0	0
40	24	-2.0	4
41	27	1.0	1
42	24	-2.0	4
43	31	5.0	25
44	30	4.0	16
45	21	-5.0	25
46	18	-8.0	64
47	24	-2.0	4
48	27	1.0	1
49	24	-2.0	4
50	26	0.0	0
51	31	5.0	25
SUM	1325		1517

Average = 26

$$s = \sqrt{\frac{\sum (X - \overline{X})^2}{n}} = 5.45$$

To find the COV

COV = s/average = 0.21 = 21%

STATISTICAL EVALUATION FOR FIFTH LAYER i.e.silty Sand/ Sand

No	**SPT-N**	$X - \overline{X}$	$(X - \overline{X})^2$
1	19	-14	196
2	34	1	1
3	39	6	36
4	40	7	49
5	34	1	1
SUM	166		283

Average = 33

$$s = \sqrt{\frac{\sum (X - \overline{X})^2}{n}} = 7.52$$

To find the COV

COV = s/average = 0.23 = 23%

Statistical Evaluation for Friction Angle Values

STATISTICAL EVALUATION FOR FIRST LAYER i.e. sandy Silt/ clayey Silt/ lean Clay

No	**ϕ**	$X - \overline{X}$	$(X - \overline{X})^2$
	degrees	**degrees**	**degrees**
1	17.5	-1.5	2.25
2	17.3	-1.7	2.89
3	20.5	1.5	2.25
4	17.1	-1.9	3.61
5	20.5	1.5	2.25
6	18.5	-0.5	0.25
7	19.8	0.8	0.64
8	21.0	2.0	4.00
SUM	152.2		18.14

Average = 19

$$s = \sqrt{\frac{\sum (X - \overline{X})^2}{n}} \quad = 1.51$$

To find the COV

COV = s/average = 0.08 8%

STATISTICAL EVALUATION FOR SECOND LAYER i.e. sandy Silt/ lean Clay / silty Sand

No	**ϕ**	$X - \overline{X}$	$(X - \overline{X})^2$
	degrees	**degrees**	**degrees**
1	17.9	-3.3	10.89
2	21.1	-0.1	0.01
3	20.3	-0.9	0.81
4	23.8	2.6	6.76
5	16.0	-5.2	27.04
6	18.8	-2.4	5.76
7	21.6	0.4	0.16
8	21.6	0.4	0.16
9	20.5	-0.7	0.49
10	20.7	-0.5	0.25
11	21.3	0.1	0.01
12	25.1	3.9	15.21
13	25.0	3.8	14.44
14	23.7	2.5	6.25
SUM	297.4		88.24

Average = 21.2

$$s = \sqrt{\frac{\sum (X - \overline{X})^2}{n}} \quad = 2.51$$

To find the COV

COV = s/average = 0/12 = 12%

STATISTICAL EVALUATION FOR THIRD LAYER i.e. sandy Silt/silty Sand/ Sand/ lean Clay

No	ϕ	$X - \overline{X}$	$(X - \overline{X})^2$
	degrees	**degrees**	**degrees**
1	19.0	-3.5	12.25
2	25.2	2.7	7.29
3	24.9	2.4	5.76
4	22.9	0.4	0.16
5	18.4	-4.1	16.81
6	18.1	-4.4	19.36
7	22.2	-0.3	0.09
8	22.2	-0.3	0.09
9	22.8	0.3	0.09
10	21.6	-0.9	0.81
11	22.1	-0.4	0.16
12	24.3	1.8	3.24
13	26.5	4.0	16.00
14	24.6	2.1	4.41
SUM	314.8		86.52

Average = 22.5

$$s = \sqrt{\frac{\sum (X - \overline{X})^2}{n}} \quad =2.49$$

To find the COV

COV = s/average = 0.11 = 11%

STATISTICAL EVALUATION FOR FOURTH LAYER i.e. silty Sand/sandy Silt / Sand

No	**ϕ**	$X - \overline{X}$	$(X - \overline{X})^2$
	degrees	**degrees**	**degrees**
1	23.4	-2.8	7.84
2	25.1	-1.1	1.21
3	26.4	0.2	0.04
4	26.7	0.5	0.25
5	26.5	0.3	0.09
6	24.2	-2.0	4.00
7	23.1	-3.1	9.61
8	49.4	23.2	538.24
9	26.4	0.2	0.04
10	25.7	-0.5	0.25
11	23.1	-3.1	9.61
12	24.2	-2.0	4.00
13	22.5	-3.7	13.69
14	23.2	-3.0	9.00
15	24.2	-2.0	4.00
16	25.2	-1.0	1.00
17	26.4	0.2	0.04
18	26.0	-0.2	0.04
19	25.3	-0.9	0.81
SUM	497.0		603.76

Average = 26.2

$$s = \sqrt{\frac{\sum (X - \overline{X})^2}{n}} \quad = 5.64$$

To find the COV

COV = s/average = 0.22 = 22%

STATISTICAL EVALUATION FOR FIFTH LAYER i.e. silty Sand /Sand

No	**ϕ**	$X - \overline{X}$	$(X - \overline{X})^2$
	degrees	**degrees**	**degrees**
1	26.9	-0.4	0.16
2	25.9	-1.4	1.96
3	27.9	0.6	0.36
4	22.7	-4.6	21.16
5	25.6	-1.7	2.89
6	31.0	3.7	13.69
7	25.8	-1.5	2.25
8	27.3	0.0	0.00
9	24.4	-2.9	8.41
10	26.0	-1.3	1.69
11	26.3	-1.0	1.00
12	26.2	-1.1	1.21
13	28.5	1.2	1.44
14	30.6	3.3	10.89
15	29.4	2.1	4.41
16	29.8	2.5	6.25
17	30.6	3.3	10.89
18	25.6	-1.7	2.89
SUM	490.5		91.55

Average = 27.3

$$s = \sqrt{\frac{\sum (X - \overline{X})^2}{n}} = 2.26$$

To find the COV

COV = s/average = 0.08 = 8%

STATISTICAL EVALUATION FOR SIXTH LAYER i.e. Sand

No	**ϕ**	$X - \overline{X}$	$(X - \overline{X})^2$
	degrees	**degrees**	**degrees**
1	27.9	0.2	0.04
2	27.5	-0.2	0.04
SUM	55.4		0.08

Average = 27.7

$$s = \sqrt{\frac{\sum (X - \overline{X})^2}{n}} \quad = 0.20$$

To find the COV

COV = s/average = 0.01 = 1%

APPENDIX B

- **Deep Excavation Support System Design**

    i. **Deep Excavation Design using FHWA Method**
    ii. **Deep Excavation Design using Canadian Method**

DESIGN OF DEEP EXCAVATION SUPPORT SYSTEM

Site Name: Geotechnical Investigation for the Construction of Plaza at Plot No 92-B-2 Hussain Chowk Gulberg 3 Lahore.

Depth of Excavation: 15 m (5 basements)

Type of Deep Excavation Support System: Anchored Tieback System with soldier piles.

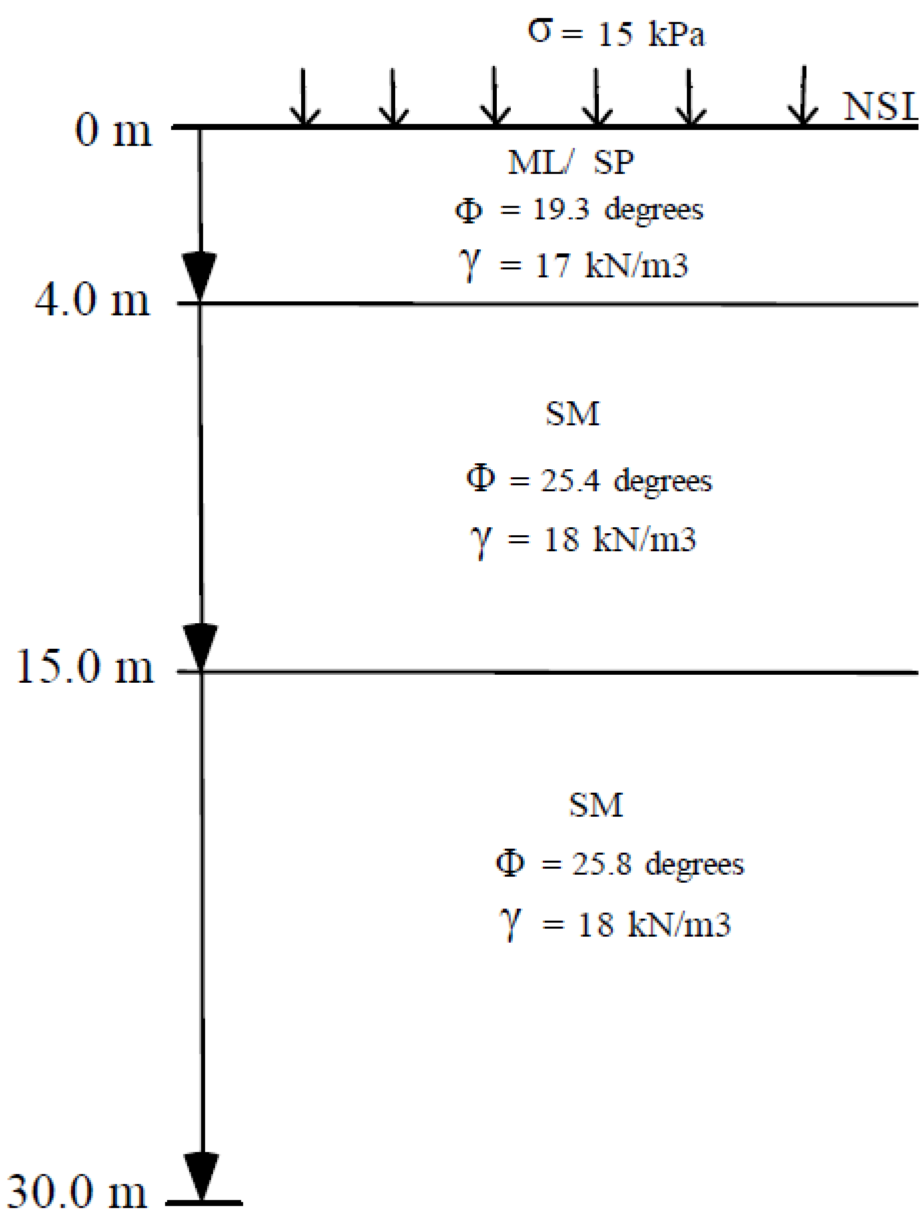


Figure 1.0: Soil Profile

DEEP EXCAVATION DESIGN USING FHWA METHOD, USA APPROACH

Loading Conditions

Loading conditions are [18]:

- Surcharge Load
- Earthquake Load
- Active Load

Surcharge Load

The minimum surcharge load should be equal to 14 kPa [15]. Considering surcharge load equal to 15 kPa.

Pressure due to Surcharge and Active Loading

Calculating the earth pressure coefficients [18]:

K_{a1} = Active Earth Pressure Coefficient

$$= \frac{1-\sin\varphi}{1+\sin\varphi}$$

$$= \frac{1-\sin(19.3)}{1+\sin(19.3)} = 0.50$$

K_{p1} = Passive Earth Pressure Coefficient

$$= \frac{1+\sin\varphi}{1-\sin\varphi}$$

$$= \frac{1+\sin(19.3)}{1-\sin(19.3)} = 1.99$$

K_{a2} = Active Earth Pressure Coefficient

$$= \frac{1-\sin\varphi}{1+\sin\varphi}$$

$$= \frac{1-\sin(25.4)}{1+\sin(25.4)} = 0.40$$

K_{p2} = Passive Earth Pressure Coefficient

$$= \frac{1+\sin\varphi}{1-\sin\varphi}$$

$$= \frac{1+\sin(25.4)}{1-\sin(25.4)} = 2.50$$

At h= 0 m

$p_1 = (\sigma + \gamma h)\ K_{a1}$

$= (15 + 17x0) \times 0.5 = 7.5\ kN/m^2$

At h= 4 m

$p_2 = (\sigma + \gamma h)\ K_{a1}$

$= (15 + 17x4) \times 0.5 = 41.5\ kN/m^2$

At h= 4 + dh

$p_3 = (\sigma + \gamma h)\ K_{a1}$

$= (15 + 17x4) \times 0.4 = 33.2\ kN/m^2$

At h= 15 m

$p_4 = (\sigma + \gamma h)\ K_{a1}$

$= (15 + 18x11) \times 0.4 = 85.2\ kN/m^2$

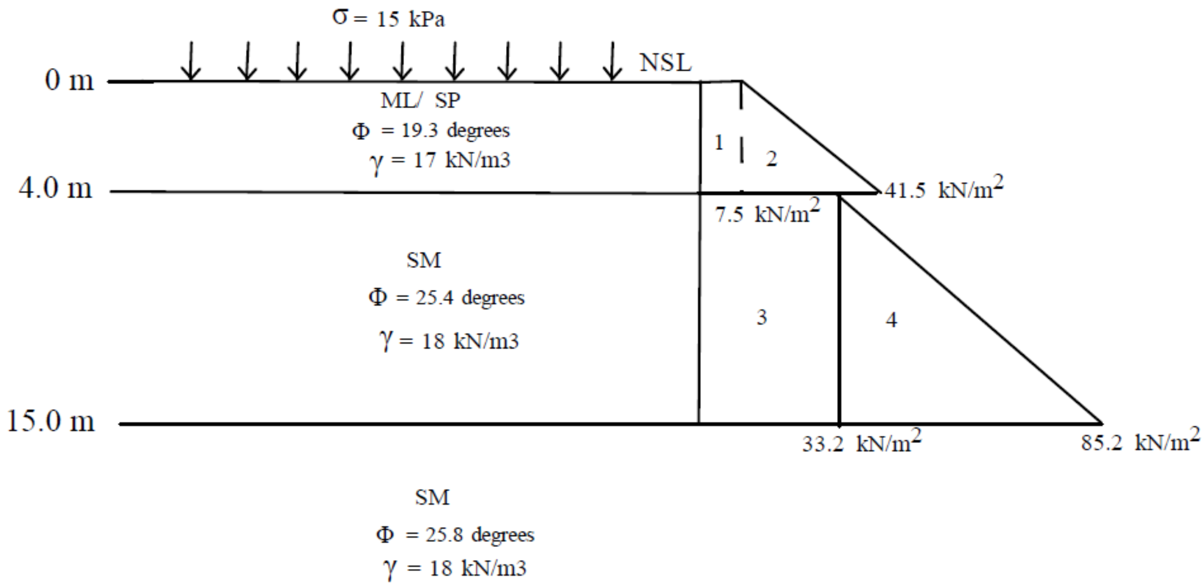


Figure 2.0: Active Earth Pressure Diagram

Calculating weight of each block from Figure 2.0

F_1 = 7.5 x 4 = 30 kN/m

F_2 = ½ x (41.5-7.5) x 4 = 68 kN/m

F_3 = 33.2 x 11 = 365.2 kN/m

F_4 = ½ x (85.2-33.2) x 11 = 286 kN/m

Total Load = $F_1+F_2+F_3+F_4$ = 749 kN/m

Increasing the total load by a factor of 1.3 for anchored soldier beam or sheet pile walls [18].

Total factored load = 1.3x749 = 974 kPa

Distributing the factored total force into an apparent pressure diagram using the trapezoidal distribution [18].

Lateral Earth Pressure, P = $\dfrac{Total\ Factored\ Load}{H-\frac{1}{3}H_1-\frac{1}{3}H_4} = \dfrac{974}{15-\frac{1}{3}(3.5)-\frac{1}{3}(2.5)} = 74.9kPa$

Pressure due to Seismic Loads

Using Mononobe-Okabe theory, the dynamic earth pressures in the active and passive state are calculated as follows [18]:

Dynamic Active Earth Pressure = $P_{AE} = 1/2\gamma H^2(1-k_v)K_{AE}$

$$K_{AE} = \frac{\cos^2(\phi-\psi-\beta)}{\cos(\psi).\cos(\beta)^2.\cos(\beta+\psi+\delta).D}$$

$$D = \left(1+\sqrt{\frac{\sin(\phi+\delta).\sin(\phi-\psi-i)}{\cos(\delta+\beta+\psi).\cos(i-\beta)}}\right)^2$$

Seismic inertia angle for soil = $\psi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right)$

H = Height of wall = 15 m

γ_{avg} = Average unit weight of backfill = 17 kN/m^3

φ_{avg} = Average angle of internal friction of the backfill = 23.5

δ = angle of friction of the wall/ backfill interface = $\varphi/2$ = 11.75

k_h = Horizontal seismic coefficient expressed as a fraction of g = 0.1

k_v = Horizontal seismic coefficient expressed as a fraction of g = $k_h/2$ = 0.05

ψ = 6

i = Slope of the surface of backfill = 0

β = Slope of the back of the wall = 0

By putting values in above equations:

D = 2.04

K_{AE} = 0.47

P_{AE} = 854 kN/m

Point of application of seismic load = 0.5H = 7.5 m from bottom of excavation.

Design of Tie-Back Anchors

Anchor Design Load

The inclination of all the anchors is assumed to be at 15^o. Anchor loads are the horizontal components of the anchor per unit width of wall [18].

$$T_h = T_{hi} \text{ x } s$$

Where, s is the horizontal spacing between adjacent anchors. The anchor load, T, to be used in the design of anchor bond zone [18],

$$T = T_h/\cos\theta,\ T_v = T \sin\theta$$

Horizontal anchor loads, maximum wall bending moment, and the reaction force to be resisted by the subgrade.

The horizontal anchor loads can be calculated using the tributary area method as shown in Figure 3.0:

T_1 = Load over length $H_1 + H_2/2$
 = P $[H_1+H_2/2]$
 = 74.9 [3.5 + 4.5/2] = 430.7 kN/m

T_2 = Load over length $H_2/2 + H_3/2$
 = P $[H_2/2 + H_3/2]$
 = 74.9 [4.5/2 + 4.5/2] = 337.1 kN/m

Adding earthquake load to T_2

T_2 = 337.1 + 854 = 1191 kN/m

T_3 = Load over length $H_3/2 + H_4/2$
 = P $[H_3/2 + H_4/2]$

$= 74.9\ [4.5/2 + 2.5/2] = 262.2$ kN/m

R = Load over length $H_4/2$

$= P\ [H_4/2]$

$= 74.9\ [2.5/2] = 93.6$ kN/m

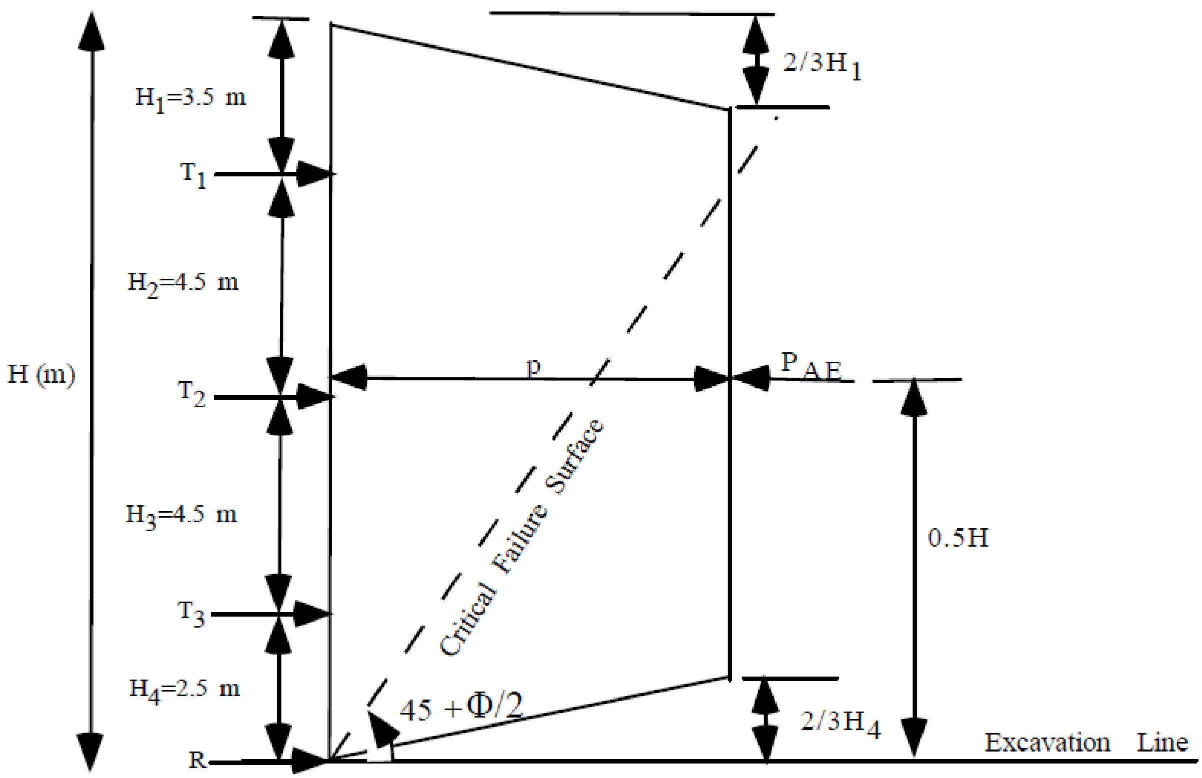


Figure 3.0: Apparent Earth Pressure Diagram [18]

Anchor Diameter and Spacing

Assuming anchor diameter = 6 in

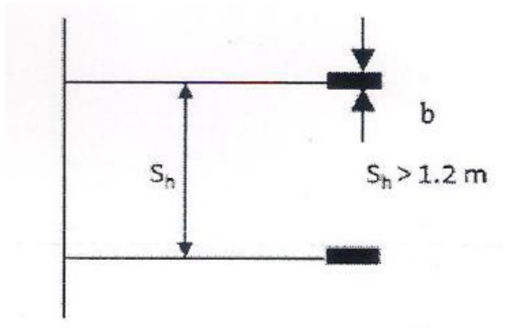


Figure 4.0: Horizontal Spacing of Ground Anchor [18]

Assume s_h = 1.1 m

Calculating the anchor loads by multiplying the horizontal spacing of ground anchors with above calculated anchor forces.

T_1 = (430.7 x 1.1)/ cos(15) = 491 kN for anchor # 1
T_2 = (1191 x 1.1)/ cos(15) = 1356 kN for anchor # 2
T_3 = (262.2 x 1.1)/ cos(15) = 299 kN for anchor # 3

Maximum anchor load over length is taken by anchor # 2 i.e. 1356 kN

Design of unbounded length

According to FHWA Method, for the design that includes strand anchors, the minimum unbounded length is selected to be greater of either 4.5 m or the distance from the wall to a location 2 m beyond critical failure surface [18].

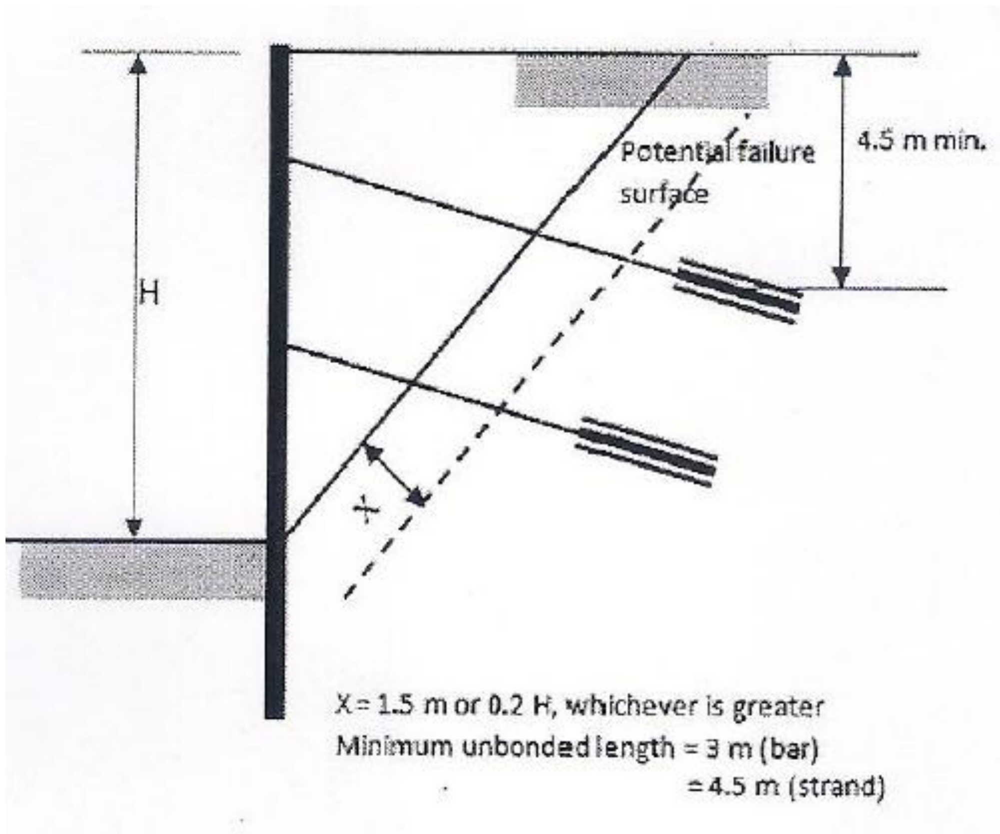


Figure 5.0: Vertical Spacing and Unbounded Length Criteria [18]

x = 1.5 m or 0.2 H whichever is greater

x = 0.2 x 15 = 3 m

Taking minimum unbounded length = 4.5 m

Estimated Capacity of Soil Anchors and Bonded Length

The anchor bond zones of the first anchor will be formed in medium dense Silt while anchor 2 and 3 will be placed in sandy Silt. Anchor bond lengths for gravity grouted, pressure grouted, post grouted soil anchors are typically 4.5-12 m since significant increase in capacity beyond 12 m is not much [18]. The design load with a factor of safety of 2 should be able to be achieved with typical with soil anchor bond length of 12 m. However, considering the earthquake loads, the factor of safety of 1.1 on wall elements is recommended for ductile failures [18].

Anchor # 1 (In Silt)

Assuming ultimate load transfer for Silt from Table 5.1 [15] = 100 kN/m

Assuming bond length = 12 m

Checking the allowable loads of Anchor # 1 = Ultimate Load Transfer x Bond Length/ FOS

= (100 x 12)/ 1.1 = 1091 kN > 491 kN, OK

Anchor # 2, # 3 (In sandy Silt)

Assuming ultimate load transfer for dense sandy Silt from Table 5.1 [15] = 130 kN/m

Assuming bond length = 12 m

Checking the allowable loads of Anchor # 2 = Ultimate Load Transfer x Bond Length/ FOS

= (130 x 12)/ 1.1 = 1418 kN > 1356 kN, OK

Calculation of bonded length:

Anchor #1 maximum bonded length = (491 x 1.1) / 100 = 5.4 m (taking bonded length equal to 6 m)

Anchor # 2 maximum bonded length = (1356 x 1.1) / 130 = 11.5 m (taking bonded length equal to 12 m)

Anchor # 3 maximum bonded length = (299 x 1.1) / 130 = 2.5 m (taking bonded length equal to 3 m)

Number of Anchor Strands

Anchor Load = 1356 kN

Diameter of Anchor Strand, A_s = 0.14 m (0.5 in)

Area of Anchor Strand = 0.00136 ft^2

Tensile Strength, f_u = 270 ksi

Allowable capacity of prestressing anchors = $0.6\ f_u A_s$

$= 0.6 \times 270 \times 0.00136 \times (12)^2$

= 31 kips = 138 kN

Required no of strands = 1356 / 138 = 9.8

Using 10 number of strands.

Soldier Pile Design

Total vertical weight caused due to anchors

Vertical component of anchor # 1 = 491 x sin(15) = 127 kN

Vertical component of anchor # 2 = 1356 x sin(15) = 351 kN

Vertical component of anchor # 3 = 299 x sin(15) = 77 kN

Total vertical load on soldier pile = 555 kN

Total self weight of pile:

γ_{conc} = 23.6 kN/m^3

Assuming pile diameter = 0.6 m

Embedded length of pile = 10 m

Total length of pile = 15+10 = 25 m

Self weight of pile = $\pi/4\,(0.6)^2$x 25 x 23.6 = 167 kN

Total vertical load, Q = 555+167 = 722 kN

Shaft Resistance [6]

$$Q_s = n\overline{N}DA_s$$

Where,

Q_s = Shaft capacity (N)

$\overline{N}$ = Average SPT index along the pile

n = $1x10^3$ for bored piles

Diameter of Pile = B= 0.6 m

D = Pile embedment length (m) = 10 m (Pile embedment starts 15m below NSL)

A_s = area of pile shaft (m^2)

Table 1.0: Calculations for Shaft Resistance of Pile

Depth	**SPT-N**	**Soil Strata**	SPT-N Values	Area (A_s)	Q_s
m			($\bar{N}$)	m^2	kN
16	29	Light Grey to greyish brown medium dense silty Sand	35	18.8	6525
17					
18	33				
19					
20	33				
21					
22	23				
23					
24	56				
25					
27	24				

Total shaft friction = 6525 kN

Tip Resistance [6]

$Q_p = mNA_t$

Where,

Q_p = Pile tip capacity

N = SPT-N at the pile tip obtained by averaging the blows over a length of 6-10B above the pile tip and 2-4B below the pile tip = 36

B = Diameter of pile = 0.6 m

m = 120 $x10^3$ for bored piles

D = Pile embedment length = 10 m

A_t = area of pile tip = $\Pi/4 \, D^2$ = 0.28 m^2

Q_p = 1210 kN

$Q_{total} = Q_p + Q_s$ = 1210 + 6525 = 7735 kN

Using a FOS = 2 for shaft and base resistance:

$Q_{safe} = Q_{total}$ / FOS = 7735/2 = 3868 kN > Q = 722 kN, OK

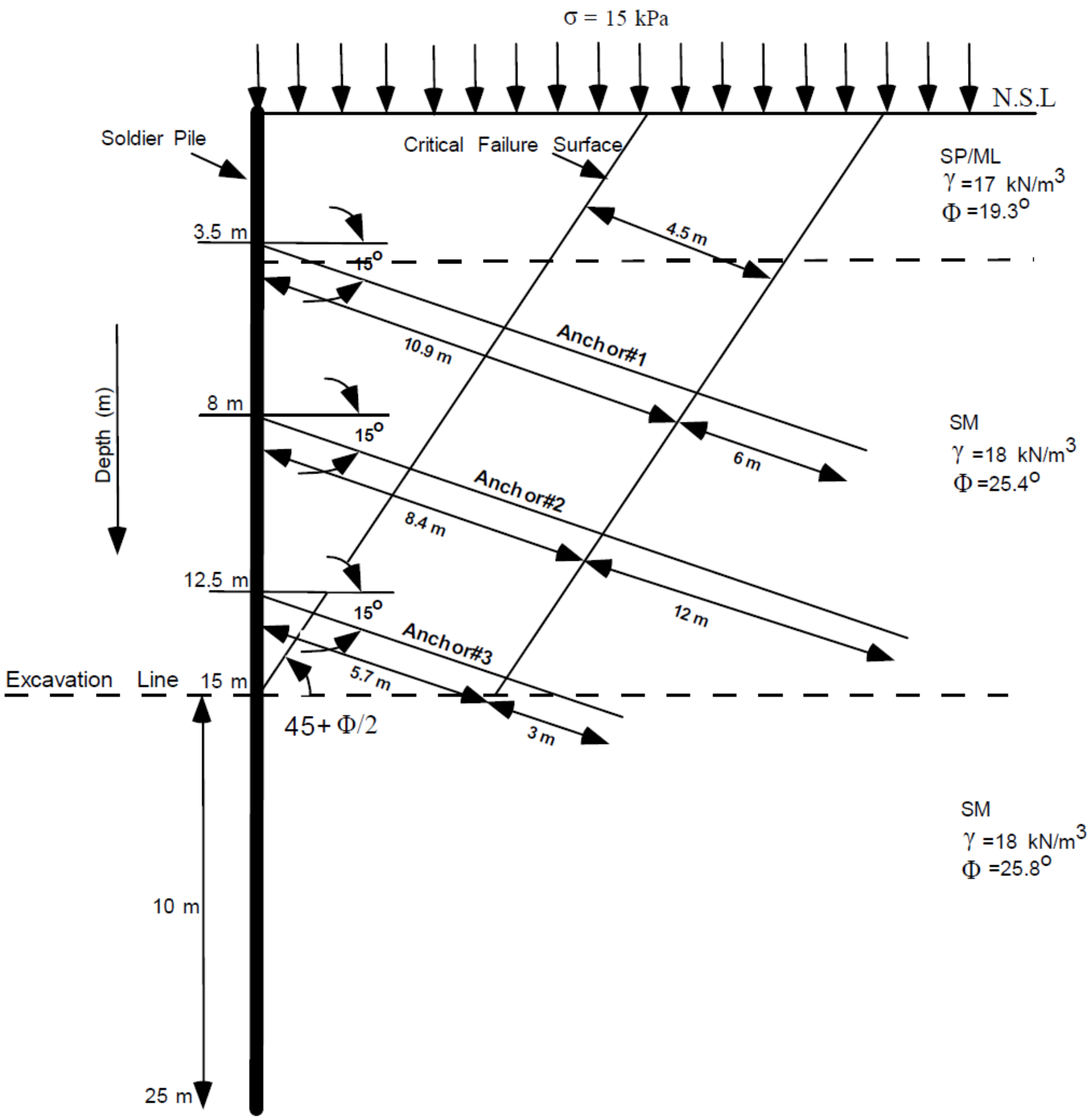


Figure 6.0: Schematic Arrangement for Anchor Pile System

DEEP EXCAVATION DESIGN USING CANADIAN FOUNDATION ENGINEERING MANUAL

The temporary support system i.e. soldier pile wall with tie-back anchors system is to be constructed to support the lateral earth pressures due to excavation of basements. Design procedures, as per Canadian Foundation Engineering Manual (4th edition, 2007), for multiple anchor retaining structure is given below:

Loading Conditions

As per Canadian approach, the following loading conditions will be applicable [19]:

- Lateral earth pressures i.e. active and passive due to vertical excavation,
- Surcharge loading from equipments, traffic loading, earth etc, as site is surrounded by a road and houses,
- Earthquake loading

Water pressures (due to GWT), loads from frost action, temperature induced stresses in structural member, stress from swelling ground, prestressing loading, loads on buried portion of wall, loads from sloping ground are not applicable in this case.

Pressure due to Surcharge Loading

The minimum surcharge load should be equal to 14 kPa according to NAVFAC [15]. Taking surcharge load equal to 15 kPa for design.

K_{a1} = Active Earth Pressure Coefficient

$$= \frac{1-\sin\varphi}{1+\sin\varphi}$$

$$= \frac{1-\sin(19.3)}{1+\sin(19.3)} = 0.50$$

K_{p1} = Passive Earth Pressure Coefficient

$$= \frac{1+\sin\varphi}{1-\sin\varphi}$$

$$= \frac{1+\sin(19.3)}{1-\sin(19.3)} = 1.99$$

K_{a2} = Active Earth Pressure Coefficient

$$= \frac{1-\sin\varphi}{1+\sin\varphi}$$

$$= \frac{1-\sin(25.4)}{1+\sin(25.4)} = 0.40$$

K_{p2} = Passive Earth Pressure Coefficient

$$= \frac{1+\sin\varphi}{1-\sin\varphi}$$

$$= \frac{1+\sin(25.4)}{1-\sin(25.4)} = 2.50$$

Calculation of Earthquake Induced Pressure

Determining horizontal and vertical seismic coefficients, considering peak ground acceleration coefficient $a_{(max)}$ = 0.1 g for Lahore (as per Seismic Code Provisions 2007) [5].

$k_h = 0.1$

$k_v = ½(k_h) = 0.05$

Determining dynamic earth pressure coefficients [19]:

Seismic inertia angle for soil = $\psi = \tan^{-1}\left(\frac{k_h}{1-k_v}\right) = 6^o$

$\Phi_1 = 19.3^o$, $\delta_1 = 19.3/2 = 9.7^o$

$$K_{ae} = \frac{\cos^2(\varphi - \psi)}{\cos(\psi).\cos(\psi + \delta).\left(1 + \sqrt{\frac{\sin(\varphi + \delta).\sin(\varphi - \psi)}{\cos(\delta + \psi)}}\right)^2}$$

$K_{ae} = 0.55$

$$K_{pe} = \frac{\cos^2\ (\varphi - \psi)}{\cos(\psi).\cos(\psi + \delta).\left(1 - \sqrt{\frac{\sin(\varphi + \delta).\sin(\varphi - \psi)}{\cos(\delta + \psi)}}\right)^2}$$

$K_{pe} = 2.27$

<u>Design of Anchors: Calculation of Forces in Anchors</u>

Anchor # 1, A_1

Assuming that the maximum load in the n^{th} level anchor occurs just before placing the next anchor and drawing the excavation cross-excavation for that condition as shown in Figure 7.0.

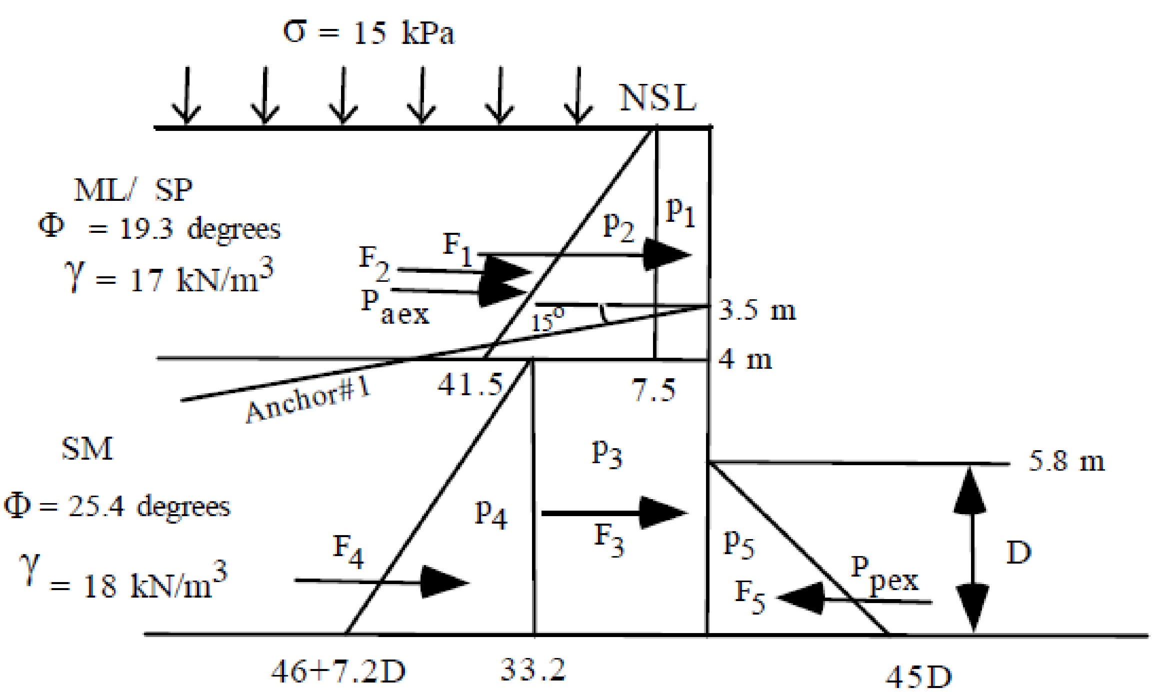


Figure 7.0: Earth Pressures for Calculation of Force in Anchor # 1

Anchor # 1 is installed at 3.5 m depth from NSL. Calculating active earth pressure forces:

At h = 0, $d_0 = 3.5$ m

$p_1 = (\sigma + \gamma*h)*K_{a1}$ $d_1 = 4$ m

$= (15 + 17*0)*0.50$ $d_2 = 1.8$ m

$=7.5$ kN/m^2 $d_3 = D$

At $h = d_1$

$p_2 = (15 + 17*4)*0.50 = 41.5$ kN/m^2

At $h = d_1 + dh$

$p_3 = (15 + 17*4)*0.40 = 33.2$ kN/m^2

At $h = d_1+d_2+d_3$

$p_4 = (\sigma + \gamma_1*h)*K_{a1} + (\gamma_2*h)*K_{a2}$

$p_4 = (15+17*4)*0.40 + (18*1.8)*0.40 + (18*D)*0.40$

$p_4 = 46 + 7.2D$ kN/m^2

Passive earth pressure forces at depth D:

$p_5 = 18D*2.5 = 45D$ kN/m^2

Calculating the weight of each block

$F_1 = p_1 * d_1$

$= 7.5 * 4 = 30$ kN/m

$F_2 = p_2 * ½*d_1$

$= ½ * (41.5-7.5) * 4 = 68$ kN/m

$F_3 = p_3 * d_2$

$= 33.2 (D+1.8) = 33.2D + 59.8$ kN/m

$F_4 = p_4 * ½*d_2$

$= ½ * (54.5+7.2D-33.2)*(D+1.8) = 3.6D^2 + 17.2D + 19.2$ kN/m

$F_5 = p_5 * ½*d_2$

$= ½ * 45D*D = 22.5D^2$ kN/m

For the first anchor level, the depth of penetration D was calculated by taking moment centre at anchor # 1 position. Required force in anchor 1 is calculated by horizontal force equilibrium, using the results of moment equilibrium.

Taking moments about Anchor # 1

$\Sigma M_1 = 0$

$-F_1*(d_0 – d_1/2) - F_2*(d_0 - 2/3d_1) + F_3*((d_2 + d3)/2 + d_1 - d_0) + F_4*(2/3(d_2+d_3) + d_1 - d_0)$
$– F_5*(2/3d_3 + d_2 + d_1 - d_0) =0$

$-45 – 56.7 + 16.6D^2 + 29.9D + 46.5D + 83.72 + 6.12D^2 + 21.9D + 19.6 + 2.4D^3 + 8.6D^2 + 7.7D$
$– 15D^3 – 51.8D^2 = 0$

$-12.6D^3 – 20.5D^2 + 84.1D + 23.5 = 0$

Solve for D

D = 2.1 m

So the embedded depth for Anchor # 1, D = 2.5 m

Calculating dynamic active and passive earth pressures for the second layer [19]:

Calculating K_{ae} and K_{pe} for second layer

By putting values, $\Phi_2 = 25.4^o$ $\delta_2 = 25.4/2 = 12.7^o$ $\psi = 6^o$

$$K_{ae} = \frac{\cos^2(\varphi - \psi)}{\cos(\psi).\cos(\psi + \delta).\left(1 + \sqrt{\frac{\sin(\varphi + \delta).\sin(\varphi - \psi)}{\cos(\delta + \psi)}}\right)^2}$$

$$K_{ae} = \frac{\cos^2\ (25.4 - 6)}{\cos(6).\cos(6 + 12.7).\left(1 + \sqrt{\frac{\sin(25.4 + 12.7).\sin(25.4 - 6)}{\cos(12.7 + 6)}}\right)^2}$$

$K_{ae} = 0.44$

$$K_{pe} = \frac{\cos^2\ (\varphi - \psi)}{\cos(\psi).\cos(\psi + \delta).\left(1 - \sqrt{\frac{\sin(\varphi + \delta).\sin(\varphi - \psi)}{\cos(\delta + \psi)}}\right)^2}$$

$$K_{pe} = \frac{\cos^2\ (25.4 - 6)}{\cos(6).\cos(6 + 12.7).\left(1 - \sqrt{\frac{\sin(25.4 + 12.7).\sin(25.4 - 6)}{\cos(12.7 + 6)}}\right)^2}$$

$K_{pe} = 3.3$

Dynamic active earth pressure = $P_{ae} = ½\ \gamma H^2\ (1 - k_v)K_{ae}$

$P_{ae} = ½(18 * (8.3)^2 * (1-0.05) * 0.44)$

$H = \left(\frac{8-3.5}{2} + 3.5\right) + D = 8.3$ m

= 259 kN/m

$P_{aex} = P_{ae} * \cos(\delta)$ = 253 kN/m

Point of application of P_{aex} = 0.6H = 5 m from the bottom of pile

$P_{pe} = ½\ \gamma H^2\ (1 - k_v) * K_{pe}$

$P_{pe} = ½ * 18 * (2.5)^2 * (1-0.05) * 3.3$

= 176 kN/m

$P_{pex} = P_{pe} * \cos(\delta)$ = 172 kN/m

Point of application of P_{pex} = 1/3D = 0.83 m from the bottom of pile

Calculating values of F_3, F_4 and F_5

Total static passive force P_p, F_5 = 141 kN/m

Total static active force P_a, $F_1 + F_2 + F_3 + F_4$ =322.5 kN/m

Force in anchor # 1, A_1 is calculated as:

$\Sigma F_x = 0$

$A_1 \cos(15) = P_a + P_{aex} - P_{pex}$

$A_1 = (322.5 + 253 – 172)/\cos(15)$

= 418 kN/m

Force in Anchor # 1, A_1 = 418 kN/m

Anchor # 2, A_2

According to the Canadian Foundation Engineering Manual, for all anchors, other than the lowest, determine the depth of penetration of the wall required to establish a factor of safety of 1.0 against rotation about the wall top, using the pressure diagram previously established, and taking into account the design forces in previously installed anchors [19].

Anchor # 2 is installed at 8 m depth from NSL as shown in Figure 8.0. Calculating active earth pressure forces:

At h = 0, d_0 = 3.5 m

$p_1 = (\sigma + \gamma_2 * h) * K_{a2}$ d_1 = 8 m

= (15+17*0)*0.50 d_2 = 10.3 m

=7.5 kN/m^2 d_3 = D

At h = d_2

p_2 = (15+ 18*10.3)*0.40

= 80.20 kN/m^2

p_3 = 80.20 kN/m^2

At h = $d_2 + d_3$

$p_4 = (\sigma + \gamma_2 * h) * K_{a2}$

$p_4 = (15+18*(10.3+D))*0.40$

$p_4 = 80.2 + 7.2D$ kN/m^2

Passive earth pressure forces at depth D:

$p_5 = 18D * 2.50$

= 45D kN/m^2

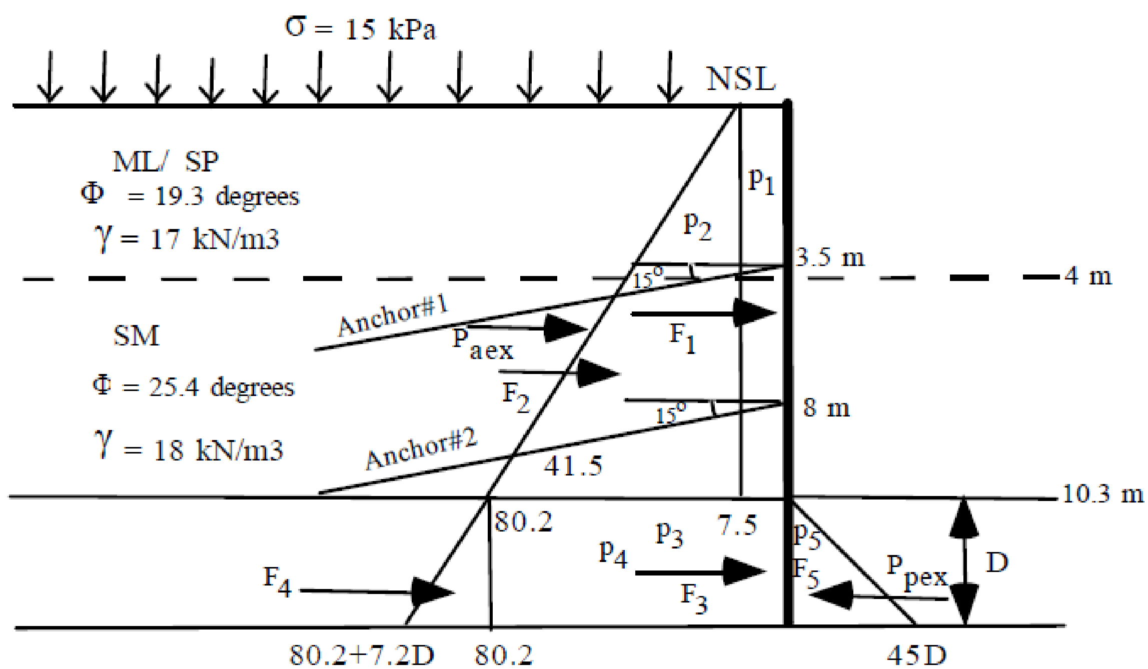


Fig. 8.0: Earth Pressures for Calculation of Force in Anchor # 2

Calculating the weight of each block

$F_1 = p_1 * d_2$

= 7.5 * 10.3 = 77.3 kN/m

$F_2 = p_2 * ½ * d_2$

= ½ * (80.2-7.5) * 10.3 = 374 kN/m

$F_3 = p_3 * d_3$

$= 80.2D$ kN/m

$F_4 = p_4 * ½*d_3$

$= ½ * (80.2 + 7.2D)*D = 40.1D + 3.6D^2$ kN/m

$F_5 = p_5 * ½*d_3$

$= ½ * 45D*D = 22.5D^2$ kN/m

Total static active force P_a, $F_1 + F_2 + F_3 + F_4 = 77.3 + 374 + 80.2D + 40.1D + 3.6D^2$

Calculating dynamic active and passive earth pressures by taking D = 3 m [19]:

Dynamic active earth pressure = $P_{ae} = ½\ \gamma H^2\ (1- k_v)K_{ae}$

$P_{ae} = ½\ (18 * (13.3)^2 * (1 - 0.05) * 0.44)$

$= 665.5$ kN/m $\qquad$ $H = d_2+D = 10.3+3 = 13.3$ m

$P_{aex} = P_{ae} * \cos(\delta_2) = 649$ kN/m

Point of application of $P_{aex} = 0.6H = 8$ m from bottom of pile

$P_{pe} = ½\ \gamma H^2\ (1- k_v)*K_{pe}$

$P_{pe} = ½(18*(3)^2 * (1 - 0.05) * 3.3)$

$= 254$ kN/m

$P_{pex}= P_{pe} * \cos(\delta) = 248$ kN/m

Point of application of $P_{pex} = 1/3D = 1$ m from bottom of pile

For equilibrium, sum of all horizontal forces, $\Sigma F_x = 0$

$- A_1 \cos(15) – A_2 \cos(15) + P_a + P_{aex} – P_{pex} = 0$

$- 418 \cos(15) – A_2 \cos(15) + 77.3 + 374 + 80.2D + 40.1D + 3.6D^2 + 649 - 248= 0$

$448.5– 0.97A_2 + 120D + 3.6D^2 = 0$

Take moment centre about wall top, $\Sigma M_{wall\ top} = 0$

FOS = 1

$\Sigma M_{active} / \Sigma M_{passive} = 1$

$\Sigma M_{active} = \Sigma M_{passive}$

$- (A_1 \cos(15) * d_0) – (A_2 \cos(15) * d_1) – (P_{pex} *(2/3\ d_3 + d_2)) + (P_{aex} * 0.4(d_2+d_3)) + (F_1*d_2/2) + (F_2 * 2/3\ d_2) + (F_3 * (d_3/2 + d_2)) + (F_4 * (2/3\ d_3 + d_2)) = 0$

$- (418 \cos(15) * 3.5) – (A_2 \cos(15) * 8) - (248 * (2/3\ D + 10.3)) + (649 * 0.4(10.3+D)) + (77.3 * 10.3/2) + (374* 2/3 * 10.3) + (80.2D * (D/2 + 10.3)) + (40.1D + 3.6D^2 * (2/3D + 10.3)) = 0$

$- 403.8 – 7.7A_2 – 165.3D – 2554.4 + 2674 + 259.6D + 398.1 + 2568 + 40.1D^2 + 826.1D + 26.7D^2 + 413D + 2.4D^3 + 37.1D^2 = 0$

$2682 + 1333.4D + 104D^2 + 2.4D^3 – 7.7A_2 = 0$

Solve both equations, D = 1.7 m

Force in Anchor # 2, A_2 = 683 kN/m

Anchor # 3, A_3

According to the Canadian Manual, for the lowest anchor, take the depth of penetration at the proposed design value and calculate the anchor force from horizontal force equilibrium [19].

K_{a3} = Active Earth Pressure Coefficient = $\frac{1-\sin\varphi}{1+\sin\varphi}$

$= \frac{1-\sin(25.8)}{1+\sin(25.8)} = 0.39$

K_{p3} = Passive Earth Pressure Coefficient = $\frac{1+\sin\varphi}{1-\sin\varphi}$

$= \frac{1+\sin(25.4)}{1-\sin(25.4)} = 2.54$

Anchor # 3 is installed at 12.5 m depth from NSL as shown in Figure 9.0. Calculating active earth pressure forces:

At h = 0,

$p_1 = (\sigma + \gamma*h)*K_{a1}$

$= (15+17*0)*0.50$

$= 7.5$ kN.m^2

$d_0 = 3.5$ m

$d_1 = 4$ m

$d_2 = 8$ m

$d_3 = 12.5$ m

$d_4 = 15$ m

$d_5 = D$

At h = d_1

$p_2 = (15 + 17*4)*0.50$

$= 41.5$ kN/m^2

At h = d_1 + dh

$p_3 = (\sigma + \gamma*h)*K_{a2}$

$p_3 = (15+17*4)*0.40$

$p_3 = 33.2$ kN/m^2

(for third layer, $\varphi = 25.8^0$)

$K_{a3} = 0.39$

$K_{p3} = 2.54$

At h = d_4

$p_4 = (\sigma + \gamma_1*h)*K_{a1} + (\gamma_2*h)*K_{a2}$

$p_4 = (15+17*4)*0.40 + (18*11)*0.40$

$p_4 = 112$ kN/m^2

At h = d_4 + dh

$p_5 = (\sigma + \gamma_1*h)*K_{a1} + (\gamma_2*h)*K_{a3}$

$p_5 = (15+17*4)*0.39 + (18*11)*0.39$

$p_5 = 110$ kN/m^2

At h = d_5

$p_6 = (\sigma + \gamma_1*h)*K_{a1} + (\gamma_2*h)*K_{a2} + (\gamma_3*h)*K_{a3}$

$p_6 = (15+17*4)*0.39 + (18*11)*0.39 + (18*D)*0.39$

$p_6 = 110 + 7D$ kN/m^2

Passive earth pressure forces at depth D:

$p_7 = 18D * 2.54$

$= 45.7D\ kN/m^2$

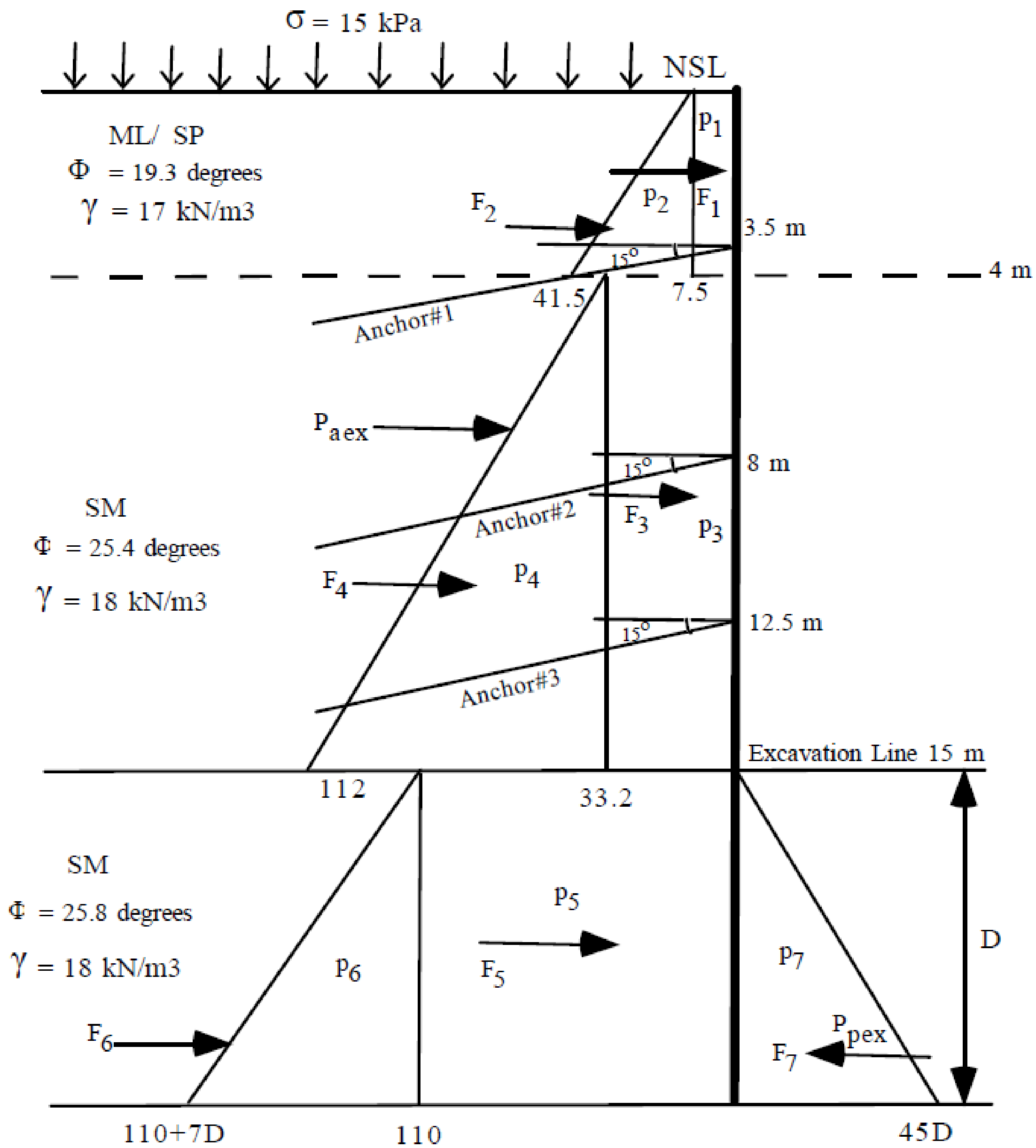


Fig. 9.0: Earth Pressures for Calculation of Force in Anchor # 3

Calculating the weight of each block

$F_1 = p_1 * d_1$

$= 7.5 * 4 = 30\ kN/m$

$F_2 = p_2 * ½*d_1$

= ½ * (41.5-7.5) * 4 = 68 kN/m

$F_3 = p_3 * (d_4 – d_1)$

= 33.2*11 = 365 kN/m

$F_4 = (p_4 – p_3) * ½* (d_4 – d_1)$

= ½ * (112 – 33.2)*11 = 433 kN/m

$F_5 = p_5 * d_5$

F_5 = 110D kN/m

$F_6 = (p_6 – p_5)* ½* d_5$

= ½ * (110+7D-110)*D = $3.5D^2$ kN/m

Total static active force P_a, $F_1 + F_2 + F_3 + F_4 + F_5 + F_6$= 30 +68 + 365 + 433 + 110D + $3.5D^2$ kN/m

Determining Dynamic Earth Pressures:

By putting values, $\Phi_3 = 25.8^o$ $\delta_3 = 25.8/2 = 12.9^o$ $\psi = 6^0$

$$K_{ae} = \frac{\cos^2(\varphi - \psi)}{\cos(\psi).\cos(\psi + \delta).\left(1 + \sqrt{\frac{\sin(\varphi + \delta).\sin(\varphi - \psi)}{\cos(\delta + \psi)}}\right)^2}$$

$$K_{ae} = \frac{\cos^2\ (25.8 - 6)}{\cos(6).\cos(6 + 12.9).\left(1 + \sqrt{\frac{\sin(25.8 + 12.9).\sin(25.8 - 6)}{\cos(12.9 + 6)}}\right)^2}$$

$K_{ae} = 0.43$

$$K_{pe} = \frac{\cos^2\ (\varphi - \psi)}{\cos(\psi).\cos(\psi + \delta).\left(1 - \sqrt{\frac{\sin(\varphi + \delta).\sin(\varphi - \psi)}{\cos(\delta + \psi)}}\right)^2}$$

$$K_{pe} = \frac{\cos^2\ (25.8 - 6)}{\cos(6).\cos(6+12.9).\left(1 - \sqrt{\frac{\sin(25.8+12.9)\,.\,\sin(25.8-6)}{\cos(12.9+6)}}\right)^2}$$

K_{pe} = 3.39

Calculating dynamic active and passive earth pressures by assuming D = 2 m [19]:

Dynamic active earth pressure = P_{ae} = ½ γH^2 (1- k_v)K_{ae}

P_{ae} = ½(18*(17)^2 * (1-0.05) * 0.43)

= 1063 kN/m

H = d_4+D = 15 + 2 = 17 m

P_{aex} = P_{ae} * cos (δ_2) = 1036 kN/m

Point of application of P_{aex} = 0.6H = 10 m from bottom of pile

P_{pe} = ½ γH^2 (1- k_v)*K_{pe}

P_{pe} = ½(18*(2)^2 * (1-0.05)) * 3.39

= 116 kN/m

P_{pex}= P_{pe} * cos (δ) = 113 kN/m

Point of application of P_{pex} = 1/3D = 0.7 m from bottom of pile

For equilibrium, sum of all horizontal forces, ΣF_x = 0

-A_1 cos (15) – A_2 cos (15) – A_3 cos (15) + P_a +P_{aex} - P_{pex} = 0

Put D = 2 m

- 418 cos (15) – 683 cos(15) –A_3 cos (15) + 30 +68 + 365 + 433 + (110*2) + 3.5(2^2) + 1036 - 113

Force in Anchor # 3, A_3 = 1024 kN/m

Anchor Diameter and Spacing

The nominal diameter of the anchor is between 0.15 and 0.20 m, so, let the diameter of the anchor hole, b = 0.15 m

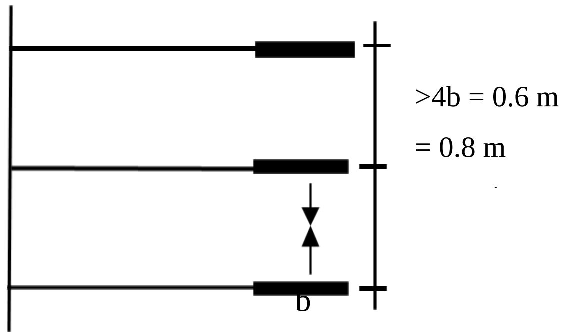


Figure 10: Minimum Anchor Spacing

Anchor loads are the horizontal components of the anchor per unit width of wall. Multiplying the above calculated anchor loads, A_1, A_2, A_3 and A_4 with the c/c spacing of anchors [19].

A_1 = 418 * 0.8 = 334 kN for anchor # 1

A_2 = 683 * 0.8 = 546 kN for anchor # 2

A_3 = 1024 * 0.8 = 819 kN for anchor # 3

Design of Unbonded Length

The unbonded length of anchor extends up to 0.15H minimum beyond the critical failure surface [19]. As shown in Figure 11.

x = 0.15H, where H = depth of excavation = 15 m

x = 0.15*(15) = 2.3 m, take x = 3 m

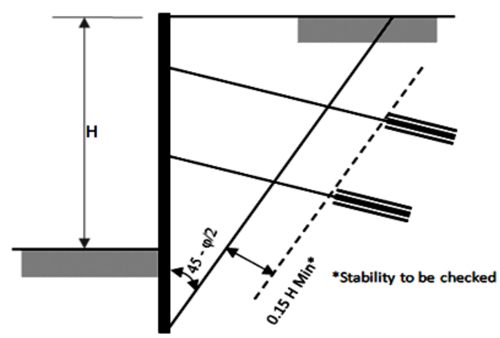


Figure 11: Minimum Unbonded Length

Estimated Capacity of Soil Anchors and Bonded Length

Computation of the pull out resistance P_{ar}, for tremie grouting anchors in cohesionless soils can be estimated by using the following equation [19]:

$$P_{ar} = \sigma'_z A_s L_s \alpha_g$$

Anchor # 1

σ'_z = (17*4) + (18*4.6) = 151 kN/m^2

L_s = 8 m

A_s = π*d*L = 3.14*0.15*1 = 0.47 m^2/m

α_g = 0.6 (from Table 5.2)

$P_{ar} = \sigma'_z A_s L_s \alpha_g$

P_{ar} = 151 * 0.47 * 8 * 0.6 = 341 kN > 334 kN ok (FOS = 1.0)

Anchor # 2

σ'_z = (17*4) + (18*8.3) = 217 kN/m^2

L_s = 7 m

A_s = π*d*L = 3.14*0.15*1 = 0.47 m^2/m

α_g = 0.8 (from Table 3.2)

$P_{ar} = \sigma'_z A_s L_s \alpha_g$

P_{ar} = 217 * 0.47 * 7 * 0.8 = 571 kN > 546 kN ok (FOS= 1.05)

Anchor # 3

σ'_z = (17*4) + (18*11.8) = 280 kN/m^2

L_s = 6.5 m

A_s = π*d*L = 3.14*0.15*1 = 0.47 m^2/m

α_g = 1.1 (from Table 3.2)

$P_{ar} = \sigma'_z A_s L_s \alpha_g$

P_{ar} = 280 * 0.47 * 6.5 * 1.1 = 941 kN > 819 ok (FOS= 1.15)

The final excavation cross section as per Canadian Code 2007 is shown as Figure 12:

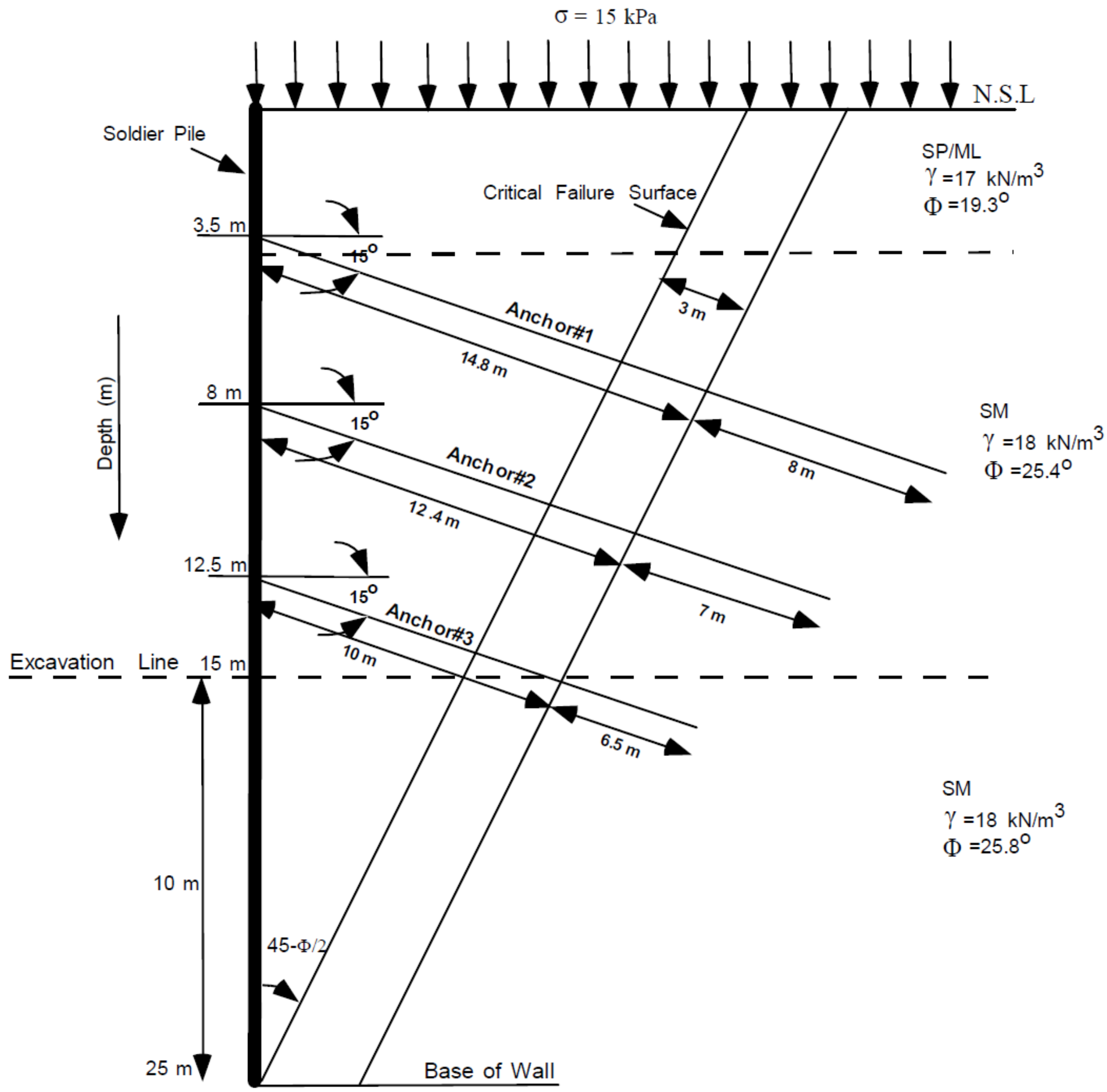
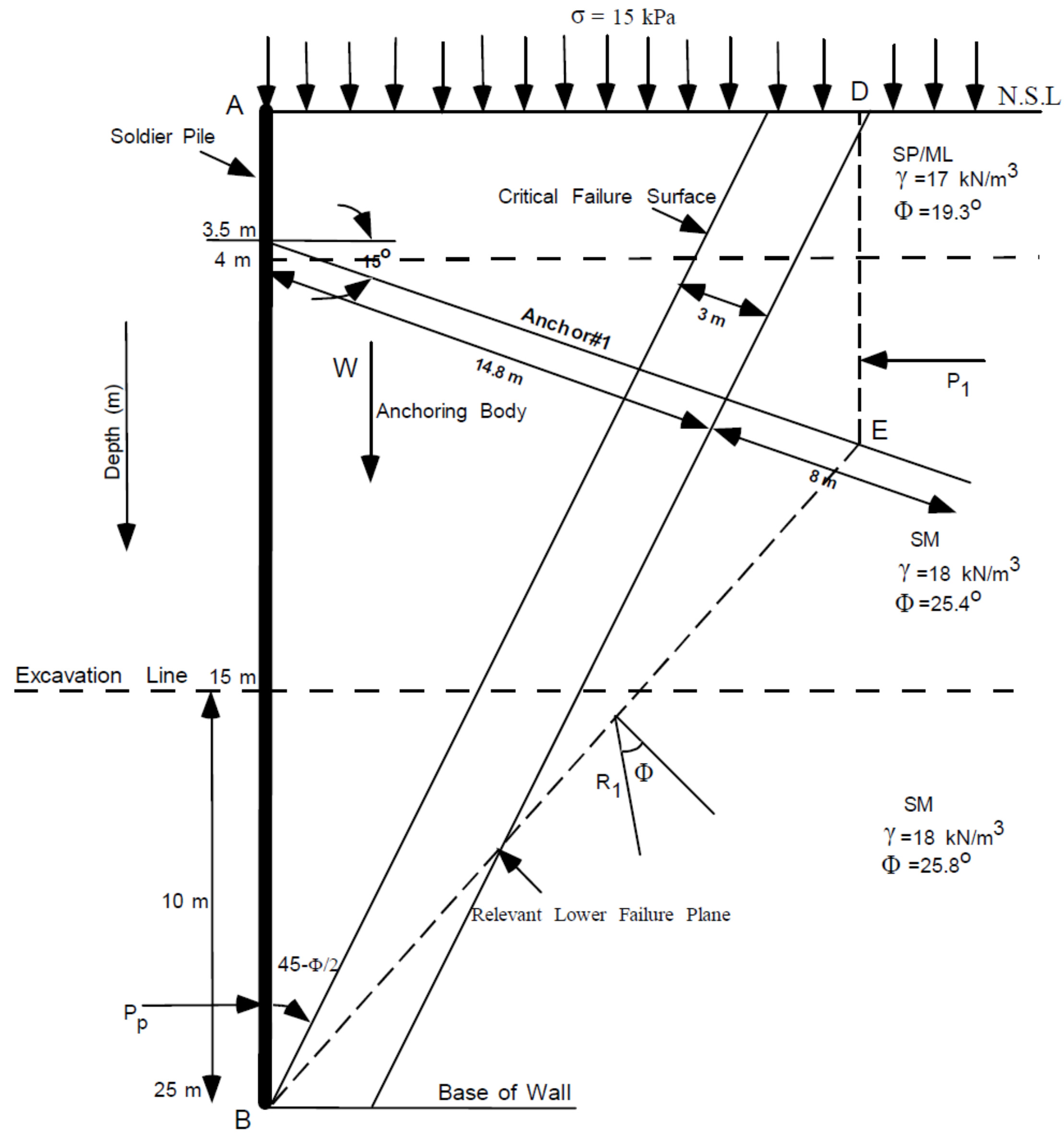


Figure 12: Schematic Arrangement for Anchor-pile system by Canadian Approach 2007

<u>Overall Stability of Anchored System</u>

According to the Canadian Manual, the overall stability of the anchor system is checked by analyzing the stability of the block of soil lying between the wall and the mid-point of the anchors. For multiple – level anchored systems, the stability of each level of the anchoring system should be checked, commencing at the top anchor. At each level, the required anchor force is the sum of all anchor forces above the relevant lower failure plane [19].

Anchor # 1 at 3.5 m depth: Considering Figure 13



a) Forces acting on Anchoring Body

W = Weight of anchoring body ABDE

P_1 = Active force from D to E

P_p = Passive pressure of embedded depth

Φ = Angle of shearing resistance

A_{reqd} = calculated anchor pull for wall stability

$W = \gamma_w \times (AB+DE)/2 \times AD$

$\gamma_w = 9.81\ kN/m^3$

$W = 9.81 \times (25+8.6)/2 \times 18.4$

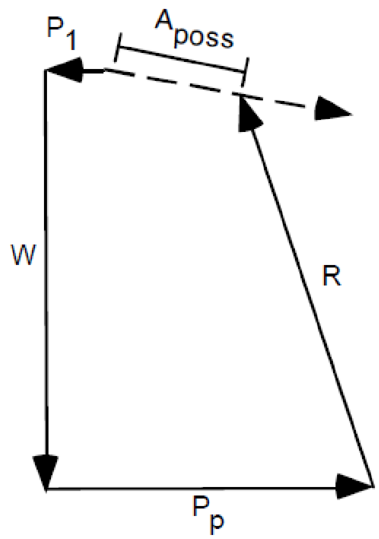
$= 3032\ kN/m$

$P_1 = ½ \times \gamma_1 \times K_{a2} \times (DE)^2$

$= 251\ kN/m$

$P_p = ½ \times \gamma_3 \times K_{p3} \times (10)^2$

$= 2286\ kN/m$



A_{poss} = Possible magnitude of Anchor#1

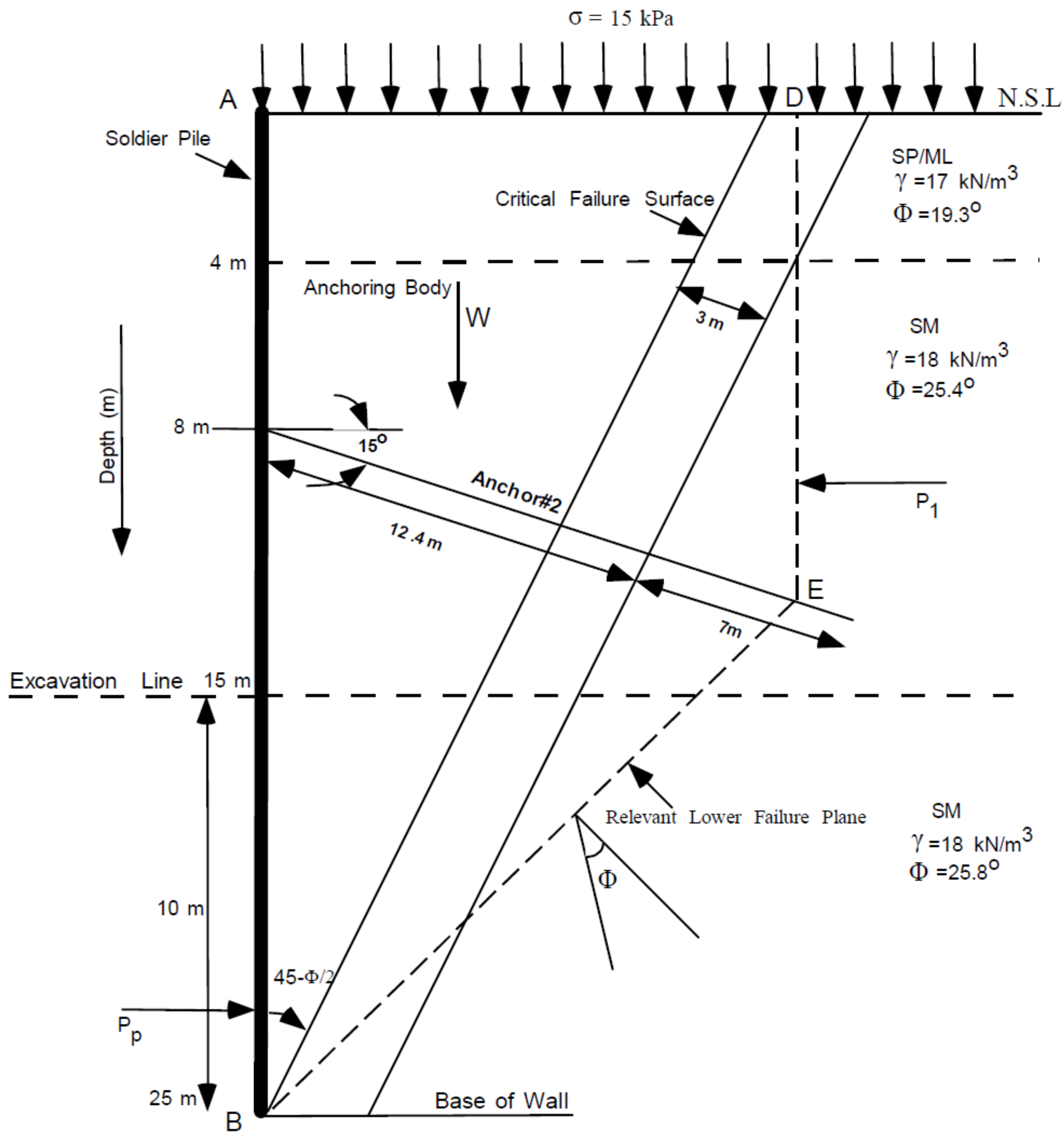
R = 2800 kN/m < W = 3032 kN/m OK

A_{poss} = 1250 kN > A_{reqd} = 418 kN OK

(b) Vector Diagram of Anchor # 1

Figure 13: Graphical Analysis of Anchored Wall at 3.5 m Level

Anchor # 2 at 8 m depth: Considering Figure 14



(a) Forces acting on Anchoring Body

W = Weight of anchoring body ABDE

P_1 = Active force from D to E

P_p = Passive pressure of embedded depth

Φ = Angle of shearing resistance

A_{reqd} = calculated anchor pull for wall stability

$W = \gamma_w \times (AB+DE)/2 \times AD$

$\gamma_w = 9.81$ kN/m^3

$W = 9.81 \times (25+12.3)/2 \times 15.6$

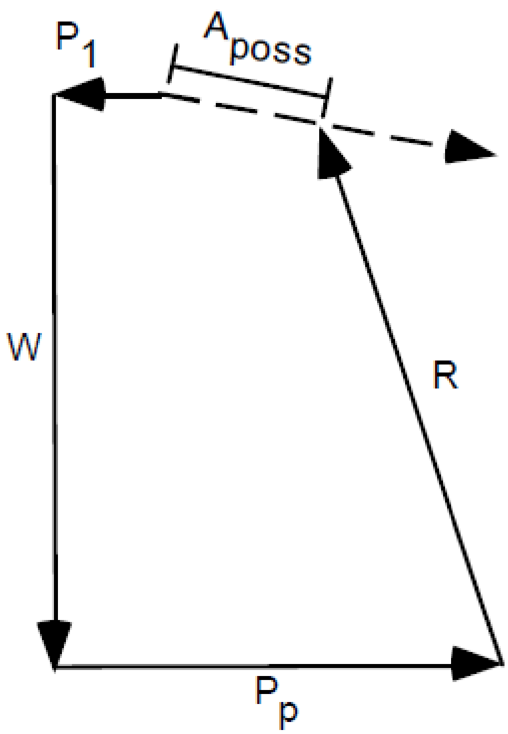
$= 2854$ kN/m

$P_1 = ½ \times \gamma_2 \times K_{a2} \times (DE)^2$

$= 545$ kN/m

$P_p = ½ \times \gamma_3 \times K_{p3} \times (10)^2$

$= 2286$ kN/m



A_{poss} = Possible magnitude of Anchor#1

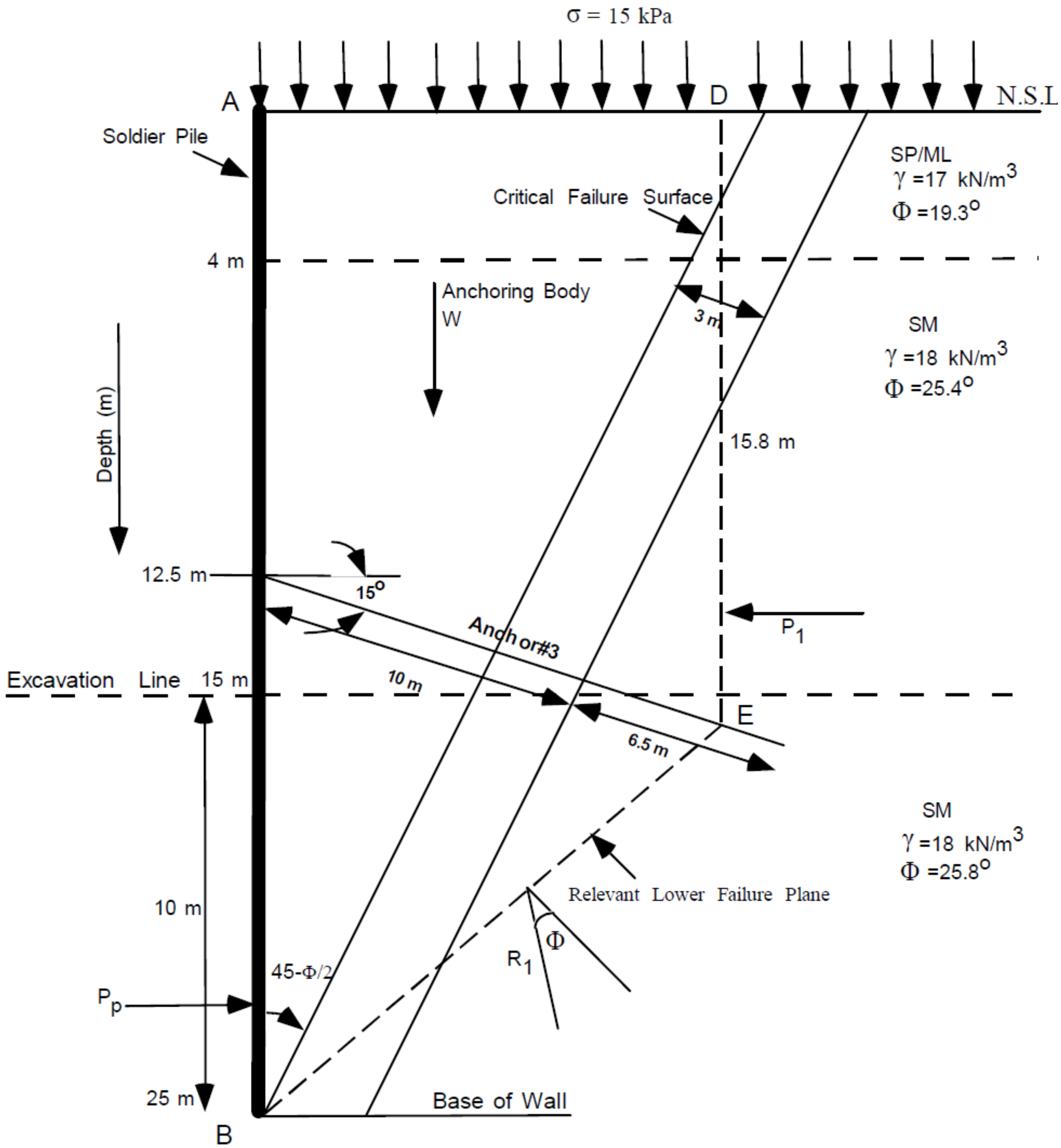
R = 2600 kN/m < W = 2854 kN/m OK

A_{poss} = 1150 kN > A_{reqd} = 683 kN OK

(b) Vector Diagram of Anchor # 2

Figure 14: Graphical Analysis of Anchored Wall at 8 m Level

Anchor # 3 at 12.5 m Depth: Considering Figure 15



(a) Forces acting on Anchoring Body

W = Weight of anchoring body ABDE

P_1 = Active force from D to E

P_p = Passive pressure of embedded depth

Φ = Angle of shearing resistance

A_{reqd} = calculated anchor pull for wall stability

$W = \gamma_w \times (AB+DE)/2 \times AD$

$\gamma_w = 9.81$ kN/m^3

$W = 9.81 \times (25+15.8)/2 \times 12.8$

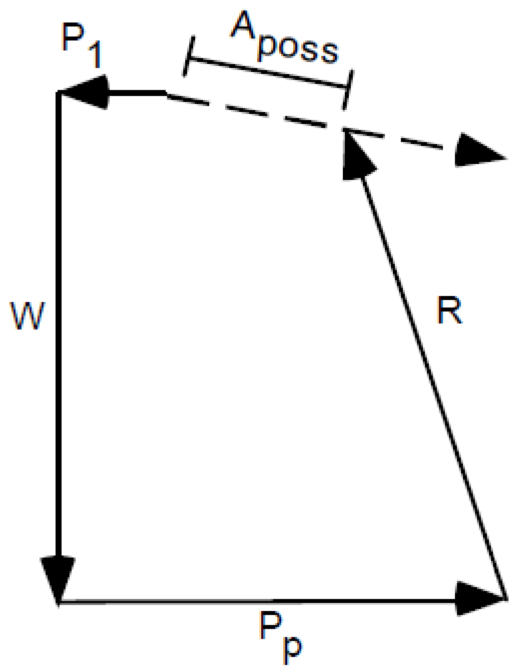
$= 2562$ kN/m

$P_1 = ½ \times \gamma_2 \times K_{a2} \times (DE)^2$

$=899$ kN/m

$P_p = ½ \times \gamma_3 \times K_{p3} \times (10)^2$

$= 2286$ kN/m



A_{poss} = Possible magnitude of Anchor#1

R = 2300 kN/m < W = 2562 kN/m OK

A_{poss} = 1250 kN > A_{reqd} = 1086 kN OK

(b) Vector Diagram of Anchor # 3

Figure 15: Graphical Analysis of Anchored Wall at 12.5 m Level

So, the stability of overall anchored system is verified through graphical analysis. The weight of anchoring body, W at every anchor level is greater than reaction R_1. The possible magnitude of anchors forces A_1, A_2, A_3 & A_4 are greater than required, indicating that the design is OK.

Number of Anchor Strands

Anchor load = 819 kN

Diameter of Anchor Strand, A_s = 0.013 m (0.5 in)

Area of Anchor Strand = 0.00136 ft^2

Use 270 grade steel

Tensile strength, f_u = 270 ksi

Allowable prestressing anchors = 0.6 f_u A_s = 0.6 * 270 * 0.00136 * $(12)^2$ = 32 kips = 142 kN

Required number of strands = 819/142 = 5.8

Use 8 numbers of strands.

Soldier Pile Design

Total vertical weight caused due to anchors

Vertical component of anchor # 1 = 334 x sin(15) = 86 kN

Vertical component of anchor # 2 = 546 x sin(15) = 141 kN

Vertical component of anchor # 3 = 819 x sin(15) = 212 kN

Total vertical load on soldier pile = 439 kN

Total self weight of pile:

γ_{conc} = 23.6 kN/m^3

Assuming pile diameter = 0.6 m

Embedded length of pile = 10 m

Total length of pile = 15+10 = 25 m

Self weight of pile = $\pi / 4\,(0.6)^2$x 23 x 23.6 = 167 kN

Total vertical load, Q = 439+167 = 606 kN

Shaft Resistance [6]

$$Q_s = n\overline{N}DA_s$$

Where,

Q_s = Shaft capacity (N)

$\overline{N}$ = Average SPT index along the pile

n = $1x10^3$ for bored piles

Diameter of Pile = B= 0.6 m

D = Pile embedment length (m) = 10 m (Pile embedment starts 15m below NSL)

A_s = area of pile shaft (m^2)

Table 2.0: Calculation of Shaft Resistance for Pile

Depth	**SPT-N**	**Soil Strata**	Avg SPT-N Values	Area (As)	Qs
m				m2	kN
16		Light Grey to greyish brown medium dense silty Sand	35	18.8	6580
17	29				
18					
19	33				
20					
21	33				
22					
23	23				
24					
25	56				
26		Greyish brown medium dense to very dense silty Sand			

Total shaft friction = 6580 kN

Tip Resistance [6]

$Q_p = mNA_t$

Where,

Q_p = Pile tip capacity

N = SPT-N at the pile tip obtained by averaging the blows over a length of 6-10B above the pile tip and 2-4B below the pile tip = 30

B = Diameter of pile = 0.6 m

m = 120 x10^3 for bored piles

D = Pile embedment length = 10 m

A_t = area of pile tip = $\Pi/4\, B^2$ = 0.28 m^2

Q_p = 1008 kN

$Q_{total} = Q_p + Q_s$ = 1008 + 6580 = 7588 kN

Using a FOS = 2 for shaft and base resistance:

$Q_{safe} = Q_{total}$ / FOS = 7588/2 = 3794 kN > Q = 606 kN, OK