ANALYSIS AND DESIGN OF SECANT PILES

Submitted in partial fulfillment of the requirements

for the degree of

Bachelor of Engineering

By

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2017-2018

CERTIFICATE

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2017-2018

This is to certify that, **Don Ahmed Sufiyan (15DCES64), Khot Shamikh A.Basit (15DCES74), Mhaskar M.Fuzail Ashraf (15DCES76)** and **Patait Ibad Asif (15DCES79)** has satisfactorily completed and delivered a Project report entitled, **"Analysis and Design of Secant Piles"** in partial fulfillment for the completion of the **B.E.** in **Civil Engineering** Course conducted by the University of Mumbai in Anjuman-I-Islam's Kalsekar Technical Campus, New Panvel, Navi Mumbai, during the academic year 2017-18.

Prof. Vedprakash Marlapalle

Guide

Dr. Rajendra B. Magar

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Director

Project Report Approval for B. E.

This B. E. Project entitled **"Analysis and Design of Secant Piles"** by **Mr. Don Ahmed, Mr. Khot Shamikh, Mr.Mhaskar M.Fuzail** and **Mr.Patait Ibad** is approved for the degree of ***"Bachelor of Engineering"*** in ***"Department of Civil Engineering".***

Examiners

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Supervisors

1 ___________________________

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Chairman (Director)

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Date:

DECLARATION

We declare that this written submission represents my ideas in our own words and where others ideas or words have been included, We have adequately cited and referenced the original sources. We also declare that I have adhered to all principles of academic honesty and integrity and have not misrepresented or fabricated or falsified any idea/data/fact/source in our submission. We understand that any violation of the above will be cause for disciplinary action by the Institute and can also evoke penal action from the sources which have thus not been properly cited or from whom proper permission has not been taken when needed.

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We take this opportunity to thank all our Professors and non-teaching staff who have directly or indirectly helped our project, we pay our respects and love to our parents and all other family members for their love and encouragement throughout our career. Last but not the least we express our thanks to our friends for their cooperation and support.

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ABSTRACT

In the recent years it has been observed that structures in urban areas has been facing problem due to unavailability of space. Therefore it is necessary of providing space for parking, public amenities etc in multi-storied building which needs to go deep excavation into ground for foundation of high rise buildings and infra structure projects. Deep excavations are supported by system like conventional retaining walls, sheet pile walls, braced walls, diaphragm walls and pile walls. The conditions of subsoil, the safety of neighboring structures, ground water regime,z imitation of vibration and noise caused by construction must all be considered for choice of an appropriate support system.

Keywords: deep excavation, secant pile, .

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Introduction

1.1 General:

Underground or basement walls are required to be constructed in case of underground water tank, basement parking, as a store room and many other purposes. These underground or basement walls are exposed to many types of loads and forces, moisture due to presence of ground water or due to rains etc.

Underground walls must support following functional requirements whether it is in a framed structure or load bearing structure:

1. Structural Stability
2. Durability
3. Moisture exclusion
4. Build ability

The presence of salts, high water table interfere with the construction process of the building, and it also affects the durability. These problems create restrictions on the nature of construction of walls below ground level and it is particularly important in case of basement to be used as internal building space.

Due to moisture conditions in case of high water table, materials with low porosity are to be used. Porous materials absorb moisture from the ground and expand on freezing, causing spalling and friability of the material. Non-porous materials also tend to perform better in terms of moisture exclusion, since they do not transfer moisture through capillarity.

The underground walls are subjected to high pressures, both axially and laterally. The lateral force exerted by the mass of earth which surrounds the walls can have a considerable effect, particularly in the case of walls to deep basements. These lateral loads must be adequately resisted if the stability of the wall is to be maintained. This is generally done either by bracing the walls or by constructing walls that are sufficiently robust to cope with the stresses involved.

To resist this loading, bracing walls below ground level with temporary supports or to utilize the floors of the buildings as permanent braces. Also, walls can be constructed to minimize the ground pressure by bracing them gradually as the work proceeds.

1.2 Retaining wall

Many types of excavation support are preferred. These may include:

1. Retaining walls.
2. Diaphragm walls.
3. Sheet pile walls.
4. Soldier piles with timber lagging walls.
5. Pile walls (contiguous, tangent or secant).

Underground construction has become a common practice worldwide. This is primarily because space for construction activities in urban areas is typically constrained by the proximity of adjacent infrastructure. Stiff excavation support systems (i.e., secant pile walls, diaphragm walls, tangent pile walls) have been employed successfully in protecting adjacent infrastructure from excavation-related damage. In particular, several literature are proposed for different excavation which shows different techniques of construction. However, for most underground construction projects in urban areas, excessive excavation-induced movements are major concerns. This is because these can lead to significant displacements and rotations in adjacent structures, which can cause damage or possible collapse of such structures. Therefore, accurate predictions of lateral wall deflections and surface settlements are important design criteria in the analysis and design of excavation support systems.

The choice of support type is dependent on many factors such as soil conditions, ground water and/ dewatering needs, excavation depth, space to nearest structures, dynamic effects of installation of driven piles, safety considerations as well as economic considerations. The availability of construction equipment, familiarity with their use, availability of materials and their cost are governing factors.

Bored concrete pile wall is the last option. Construction material i.e. concrete and steel reinforcement as well as drilling rigs are available in most places together with experienced personnel. For cases of high ground water secant piles are chosen while contiguous piles can be used in other cases.

In this report, analysis and design of secant piles is presented which deals with the moments of forces acting on pile at time of embedding them into the soil and after the construction how it transfers all the forces and moments and properly how it distributes to the strata. The design is done for a particular group of piles. The further comparison of manual calculations are compared in the PLAXIS 3D foundation. The idea of comparing both methods of design and analysis of the piles have been taken from the literature which concludes their results by designing and analyzing the different types of piles in different region of different countries. From the study of the literature it has been concluded that the secant piles are preferred for any type of the underground construction.

1.3 Aim:

This project has been taken up to **Analysis and Design of Secant Pile for Deep Excavation.** With the following objective.

1.4 Objectives:

1. To analysis of secant pile for basement excavation

2. To design of secant for basement excavation

3. To validated the manual calculation by FEM analysis

Chapter 2

Literature Review

2.1 General:

Underground construction has become a common practice worldwide. This is primarily because space for construction activities in urban areas is typically constrained by the proximity of adjacent infrastructure. Stiff excavation support systems (i.e., secant pile walls, diaphragm walls, tangent pile walls) have been employed successfully in protecting adjacent infrastructure from excavation-related damage. In particular, several literature are proposed for different excavation which shows different techniques of construction. However, for most underground construction projects in urban areas, excessive excavation-induced movements are major concerns.

2.2 Experimental and Theoretical Analysis:

Sastry and Meyerhof (1986) they compared the test results with theoretical pressure distributions and displacements of the pile for the working load range. They studied two different theories Terzaghi (1943) and Brinch Hansen (1961) for ultimate loads are extended to working loads. They showed a lateral pressure distribution for piles in sand and soft clay. From this an approximate theoretical lateral soil pressure distribution has been obtained for piles under vertical eccentric and central inclined loads having a given factor of safety.

Altuntas el at (2009) presented a state of practice of the design and construction of a secant pile wall for a major real estate development in Lower Manhattan. As an alternative, a secant pile wall was constructed to provide an excavation support system for the basement construction and to eliminate underpinning of adjacent buildings The secant pile wall also served as the permanent foundation wall of the new building. The secant pile was 58 meters which was constructed in 5 months. The design team concluded that the secant pile wall construction even drilled with casings can significantly impact adjacent structures. The use of bentonite slurry as a drilling agent to minimize base heave of the borehole will reduce impact to adjacent structures. However, use of bentonite slurry will result in smaller skin resistance warranting longer piles.

2.3 FEM Analysis:

A.M.A. Nasr (2013) has proposed an experimental and theoretical studies on lateral large load acting on pile foundation of bridge abutments, retaining walls and structure subjected to wind-earthquake forces. It shows that a pile with newly fins is capable of supporting the large lateral loads act on the structure. He performed a small scale experiment on piles with fins and without fins in a test tank which is made of mild steel with dimensions 1000 mm long × 500 mm wide × 1000 mm height. After that he performed experiment in three dimensional finite element analysis in non linear computer program PLAXIS 3D Foundation.

He concluded that Piles with fins provide considerably higher ultimate lateral loads and lateral resistance behavior compared with a regular reference pile. When a pile is finned with triangular and rectangular fins, the ultimate lateral load of the pile increases by about 64% and 86%, respectively, than that of a regular pile. At the same time, the lateral head deflection decreases by about 37% and 70%, respectively. Hence, using rectangular fins is more effective in improving the lateral behavior of piles

2.4 Field Analysis:

Georgiadis and Anagnostopoulos (1998) has tested a model sheet pile in sand to investigate the effect of surcharge strip loads on wall behavior. They used Coulomb analysis and the simple 45 load distribution while bending moments determined using elasticity theory. After they performed they tested in Finite-element analyses for lateral surcharge pressures and bending moment. From this they made the conclusion that the most accurate predictions (within 20%) of the model wall response were obtained using earth pressures determined with the Coulomb and the simple 45° distribution methods. The simplicity of the latter and its good performance make it very useful in preliminary design. The Beton Kalender uniform earth pressure solution overestimated maximum bending moments by 20 to 60%, while the triangular solution gave extremely high values. Finite-element analyses to demonstrated small lateral yielding of the wall drastically reduces lateral surcharge pressures and bending moments determined by elasticity theory.

Elfatih el at (2016) studied three cases and published paper in which shoring was used are presented. Two are for high rise building and one for infrastructure project. Continuous and secant bored concrete pile walls are presented as relevant solution. The main keywords are deep excavation, shoring, continuous pile, secant piles

Bilgin and Erten (2009) they resulted the behavior of anchored sheet pile walls constructed on slopes using finite element method. The study results show that for the cases studied the slope angle, varying between 5H:1V to 2H:1V, has a very minimal effect on wall behavior and concluded that the location of anchored sheet pile wall along the slope, on the other hand, has a significant effect on wall behavior. The main four parameters were also concluded, The top of wall starts moving backwards towards the soil due to the wall rotation about the anchor location. Maximum wall bending moments increase approximately 14 percent, when the wall moves from the top of the slope to the tip of the slope. Wall bending moments at the anchor level decrease almost 65 percent when the wall moves from the top of the slope to the middle of the slope and Anchor forces decrease significantly, approximately 30 percent when the wall moves from the top of the slope to the tip of the slope.

Altuntas el at (2009) presented a state of practice of the design and construction of a secant pile wall for a major real estate development in Lower Manhattan. As an alternative, a secant pile wall was constructed to provide an excavation support system for the basement construction and to eliminate underpinning of adjacent buildings The secant pile wall also served as the permanent foundation wall of the new building. The secant pile was 58 meters which was constructed in 5 months. The design team concluded that the secant pile wall construction even drilled with casings can significantly impact adjacent structures. The use of bentonite slurry as a drilling agent to minimize base heave of the borehole will reduce impact to adjacent structures. However, use of bentonite slurry will result in smaller skin resistance warranting longer piles.

Liao el at (2014) published a paper on mechanical behaviors of secant pile walls under lateral loading are complex with regard to their irregular cross sections and bonding quality of secant faces. They conducted a model test to investigate the stress states and failure modes of secant faces, the bearing capacities, and the flexural rigidity of secant pile walls under various lateral loading conditions which consist of three types of test (shear, tension, bending). Plain-reinforced-pile (PRP) and plain-concrete-pile (PCP) are used in model test. They concluded Under shear loading, the principal shear stress plane rotated an angle from the secant face. Under tension loading, the samples presented brittle fracture behavior and their failure surfaces occurred inside PCPs or and on secant faces. The shear capacities of secant pile walls were determined ac- cording to their respective failure modes of secant faces.

Underwood and Greenlee (2010) presented summarizes design and construction of steel sheet pile walls for permanent building foundations and earth retention on four projects in and around Minneapolis, Minnesota USA. From which it has resulted that Permanent sheet pile foundations can be cost effective where it is necessary or desirable to construct one or more below grade levels with building foundation walls close to the property lines. Construction was performed in stages which controls the sheet pile section, and often controls the required pile embedment depth below the bottom of excavation. For designing they consider both axial stresses from the superstructure and bending stresses from earth/surcharge pressures and from eccentric building loads.

2.5 Concluding remark:

Based on the published literature following concluding remarks are made.

1. Excavation support systems are conventionally designed based on anticipated earth pressures calculated from the apparent pressure diagrams developed by Peck (1969). These apparent earth pressure diagrams must only be used to calculate the strut loads and it is incorrect to use them for calculating the stress or bending moments in the retaining wall.
2. Results of experimental program on model cantilever sheet pile walls subjected to surcharge strip loads have been compared to predictions made using several different methods of computing lateral earth pressures.

3. Piles with fins provide considerably higher ultimate lateral loads and lateral resistance behavior compared with a regular reference pile.
4. Ultimate lateral load improvement depends greatly on length of the fins and increases significantly to the value of LF/LP = 0.4. Any further increase in the fin length does not show any note-worthy contribution to the pile capacity.
5. When a pile is finned with triangular and rectangular fins, the ultimate lateral load of the pile increases by about 64% and 86%, respectively, that of a regular pile. At the same time, the lateral head deflection decreases by about 37% and 70%, respectively. Hence, using rectangular fins is more effective in improving the lateral behavior of piles.

2.6 Research Gap:

Limited data has been reported in the literature presenting a fully three-dimensional finite element analysis of deep excavations. In addition, no one has presented design methodology for excavation support systems that relates system stiffness to excavation-related ground movements

2.7 problem and requirement of underground basements:

Chapter 3

Methodology

3.1 General:

Name of Structure: Shri Bhagubhai Mafatlal Polytechnic

Site Location: South Mumbai, Vile Parle, near airport.

Existing structure on the site is 54 years old and is of three storey structure which has to be demolished and new structure is to be constructed. Due to increase of the new courses in the polytechnic the structure is constructed to be high which is restricted by the government authority because of the airport nearer in the locality. To overcome this situation we have to construct underground storeys which requires deep excavation.

For deep excavation, safety of neighboring structure and side existing roadways a support system has to be provided which will act as a retaining structure to avoid failure of side soil. Secant pile has to be provided as a support system for which analysis and design will be done manually and in software. result between manually and in software will be compared. Analysis and Design will be performed with reference to the report submitted by Construction Industry Research and Information Association.

PLAXIS 3D Foundation software will be used for software analysis and design.

3.2 Analysis of Secant piles:

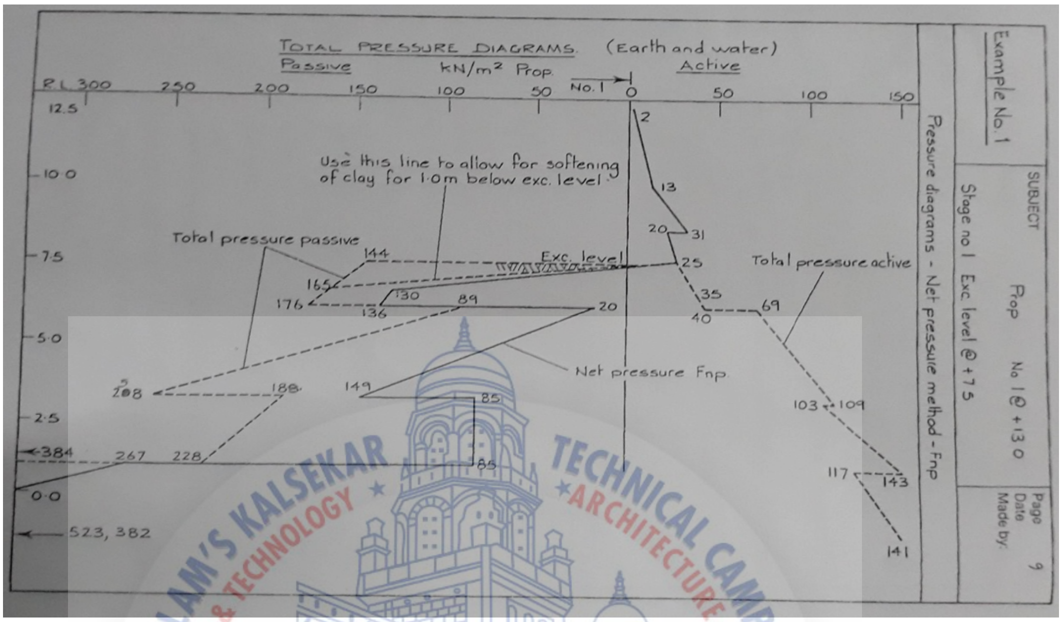

Fig.3.1 Total pressure diagram of an under driven pile

3.3 FEM Analysis:

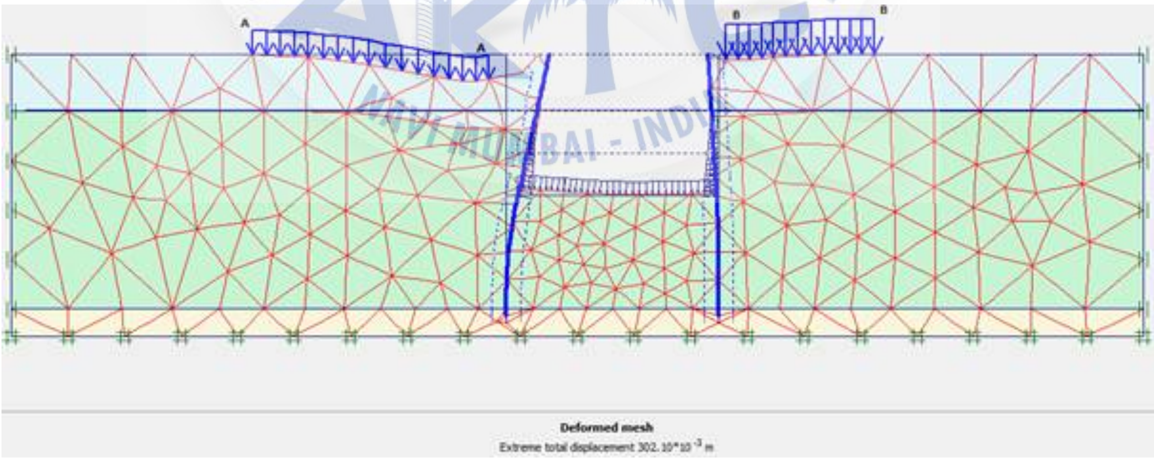

Fig.3.2 Deformation of side of the excavation due to loading

Chapter 4

Site Investigation Report

4.1 Introduction:

The present soil investigation report is for polytechnic at Chainage: 100+40 km.

The investigation comprises of sinking ten numbers of Boreholes of maximum depth of 21.50 m at bridge Location with collection of samples and conducting relevant field and laboratory tests. Based on the investigation the subsoil condition at the bridge location has been identified and an analysis has been done for the suitable foundation and Bearing capacities for the structure.

4.2 Field Investigation:

4.2.1 General:

For the finalization in the design of the foundation for the proposed structure to be constructed at the site, geotechnical investigation was done.

The total investigation programme has two phases.

a) Field Works.

b) Laboratory Testing

In field work, the type of sub-surface deposits and their characterization have been revealed. Laboratory testing actually helps in determining relevant geotechnical properties of the subsurface deposits leading to finalisation of foundation depths and type of the structures and the bearing capacity with particular reference to the sub-surface types and their strength parameters and settlement potentials at the site.

4.2.2 Boring/Sampling:

Of the different types of explorations, borings are the most practical and relatively correct method of obtaining sub-surface information. The most important aspect of the boring operations is to obtain information about the subsoil profiles, its nature and strength and to collect soil samples for strata identification and conducting laboratory tests.

The bore holes of 150 mm diameter (and NX size in rock) were sunk as per specifications and IS: 1892. Casings as required were used to retain the borehole. Bore Holes were taken at locations judiciously specified and were extended upto specified maximum depth around 10m. Boring was carried out by shell and augur/wash boring method in soil. Adequate care as per specification and Indian standard practice was taken to prevent any possible side collapse in bore holes.

The details of the bore hole including field tests of Standard Penetration tests and also collection of disturbed soil samples are given in Bore Log enclosed. All the representative samples of sub-surface deposits were collected from bore holes, labelled depth wise and placed in polythene bags. Reference Numbers and depth of these samples are shown in Bore Log Data Sheets.

4.2.3 Standard Penetration Test (SPT):

The standard penetration tests were conducted as per IS 2131-1981. The split spoon sampler, connected with a string of drill rods, was lowered into the bottom of the bore hole. The sampler was driven into the soil stratum upto a maximum depth of 450mm. by making use of 63.5 kg. weight falling freely from a height of 750mm on to an anvil fixed on the top of drill rod. The number of blows required to penetrate each of the successive 150mm depths was counted to produce a total penetration of 450mm. To avoid seating errors, the blows required for the first 150mm of penetration was not taken into account. Those required to increase the penetration from 150mm to with a detachable core bit, which is of diamond. All core bits were of 73mm size.

4.2.4 Details of Boreholes and Soil Samples Collected:

Table 4.1: Borelog Data BH-1

BORE LOG DATA SHEET

Bore hole no. BH-1	Bore hole diameter-150mm	field test	Nos.	samples
Location-Vile parle	Northing	SPT	3.0	Undisturbed(U)
Date- 02/02/2018	Easting	Cone	-	disturbed(D)
Water level-3.0 m	Termination depth-20m	Vane	-	samples

depth (m)	penetration			N value	strata description	symbol	thickness	SAMPLES			core recovery (%)	R.Q.D (%)	REMARK
	15cm	15cm	15cm					type	no.	depth			
0.00 0.50	4	6	9	14	TOP SOIL		0.5	D	0.5	0.5	0.0	0.0	
0.50 1.50	8	12	13	36	SANDY CLAY		1.0	D	2.0	0.5-1.0	0.0	0.0	
1.50 4.50	6	9	12	32	SILTY SAND		3.0	D S D S S	1 1 2 2 3	1.5-2.0 2.5-3.0 3.5-3.8 4.0 4.2-4.5	0.0 0.0 0.0 0.0 0.0	0.0 0.0 0.0 0.0 0.0	
4.50 14.50	9	13	16	42	SAND		10	D S D S S	1 1 2 2 3	1.5-2.0 2.5-3.0 3.5-3.8 4.0 4.2-4.5	0.0 0.0 0.0 0.0 0.0	0.0 0.0 0.0 0.0 0.0	
14.50 20.00	10	11	14	47	ROCK		5.5	C C C	1 2 3	3.5-3.8 4.0 4.2-4.5	18 20 26	13 20 40	

S=S.P.T	C=CORE	U=UNDISTURBED SAMPLE	D=DISTURBED SAMPLE	W=WATER SAMPLE	BGL=BELOW GROUND LEVEL

Table 4.2: Borelog Data BH-2

BORE LOG DATA SHEET

Bore hole no. BH-2	Bore hole diameter-150mm	field test	Nos.	samples
Location-Vile parle	Northing	SPT	3.0	Undisturbed(U)
Date- 02/02/2018	Easting	Cone	-	disturbed(D)
Water level-3.0 m	Termination depth-20m	Vane	-	samples

depth (m)	penetration			N value	strata description	symbol	thickness	SAMPLES			core recovery (%)	R.Q.D (%)	REMARK
	15cm	15cm	15cm					type	no.	depth			
0.00 0.50	5	7	10	15	TOP SOIL		0.5	D	0.5	0.5	0.0	0.0	
0.50 1.50	6	9	12	37	SANDY CLAY		1.0	D	2.0	0.5-1.0	0.0	0.0	
1.50 4.50	9	11	15	33	SILTY SAND		3.0	D S D S S	1 1 2 2 3	1.5-2.0 2.5-3.0 3.5-3.8 4.0 4.2-4.5	0.0 0.0 0.0 0.0 0.0	0.0 0.0 0.0 0.0 0.0	
4.50 15.50	11	13	15	43	SAND		11	D S D S S	1 1 2 2 3	1.5-2.0 2.5-3.0 3.5-3.8 4.0 4.2-4.5	0.0 0.0 0.0 0.0 0.0	0.0 0.0 0.0 0.0 0.0	
15.50 20.00	10	12	14	48	ROCK		4.5	C C	1 2	3.5-3.8 4.0	18 20	13 20	
S=S.P.T	C=CORE		U=UNDISTURBED SAMPLE			D=DISTURBED SAMPLE		W=WATER SAMPLE		BGL=BELOW GROUND LEVEL			

Table 4.3: Borelog Data BH-3

BORE LOG DATA SHEET

Bore hole no. BH-3	Bore hole diameter-150mm	field test	Nos.	samples	
Location-Vile parle	Northing	SPT	3.0	Undisturbed(U)	
Date- 02/02/2018	Easting	Cone	-	disturbed(D)	
Water level-3.0 m	Termination depth-20m	Vane	-	samples	

depth (m)	penetration			N value	strata description	symbol	thickness	SAMPLES			core recovery (%)	R.Q.D (%)	REMARK
	15cm	15cm	15cm					type	no.	depth			
0.00 0.50	6	8	11	16	TOP SOIL		0.5	D	0.5	0.5	0.0	0.0	
0.50 1.50	10	12	15	38	SANDY CLAY		1.0	D	2.0	0.5-1.0	0.0	0.0	
1.50 3.50	9	12	14	34	SILTY SAND		2.0	D	1	1.5-2.0	0.0	0.0	
								S	1	2.5-3.0	0.0	0.0	
								D	2	3.5-3.8	0.0	0.0	
								S	2	4.0	0.0	0.0	
3.50 15.50	12	13	15	45	SAND		12	S	3	4.2-4.5	0.0	0.0	
								D	1	1.5-2.0	0.0	0.0	
								S	1	2.5-3.0	0.0	0.0	
								D	2	3.5-3.8	0.0	0.0	
								S	2	4.0	0.0	0.0	
								S	3	4.2-4.5	0.0	0.0	
15.50 20.00	10	12	14	48	ROCK		4.5	C	1	3.5-3.8	20	15	
								C	2	4.0	25	23	

S=S.P.T	C=CORE	U=UNDISTURBED SAMPLE	D=DISTURBED SAMPLE	W=WATER SAMPLE	BGL=BELOW GROUND LEVEL

Table 4.4: Borelog Data BH-4

BORE LOG DATA SHEET

Bore hole no. BH-4	Bore hole diameter-150mm	field test	Nos.	samples	
Location-Vile parle	Northing	SPT	3.0	Undisturbed(U)	
Date- 02/02/2018	Easting	Cone	-	disturbed(D)	
Water level-3.0 m	Termination depth-20m	Vane	-	samples	

depth (m)	penetration 15cm	15cm	15cm	N value	strata description	symbol	thickness	SAMPLES type	no.	depth	core recovery (%)	R.Q.D (%)	REMARK
0.00 0.50	7	9	12	17	TOP SOIL		0.5	D	0.5	0.5	0.0	0.0	
0.50 1.50	9	12	14	39	SANDY CLAY		1.0	D	2.0	0.5-1.0	0.0	0.0	
1.50 3.50	9	11	13	32	SILTY SAND		2.0	D	1	1.5-2.0	0.0	0.0	
								S	1	2.5-3.0	0.0	0.0	
								D	2	3.5-3.8	0.0	0.0	
								S	2	4.0	0.0	0.0	
3.50 16.00	10	12	15	44	SAND		12.5	S	3	4.2-4.5	0.0	0.0	
								D	1	1.5-2.0	0.0	0.0	
								S	1	2.5-3.0	0.0	0.0	
								D	2	3.5-3.8	0.0	0.0	
								S	2	4.0	0.0	0.0	
								S	3	4.2-4.5	0.0	0.0	
16.00 20.00	11	12	14	49	ROCK		4.0	C	1	3.5-3.8	20	15	
								C	2	4.0	25	23	

S=S.P.T	C=CORE	U=UNDISTURBED SAMPLE	D=DISTURBED SAMPLE	W=WATER SAMPLE	BGL=BELOW GROUND LEVEL

4.3. Laboratory Testing:

For proper identification and classification of the sub-surface and for deriving adequate information regarding its relevant physical and geotechnical properties at the site under investigation, the following laboratory tests were conducted on the soil and rock samples collected from the bore holes.

For Soil Samples:

1. Grain size analysis (Sieve as well as Hydrometer).
2. Natural Moisture Content.
3. Bulk Density & Dry Density
4. Specific Gravity.
5. Liquid Limit.
6. Plastic Limit.
7. Triaxial Test (UU).
8. Consolidation Test.

For Rock Samples:

1.Bulk density, Specific Gravity & water absorption.

2.Unconfined Compression Strength Test.

The above mentioned laboratory tests were done following the testing procedure given in the relevant parts of IS: 2720 and other relevant codes. Results of all tests are furnished in Annexure of this report.

4.4 Subsoil Profile:

The subsoil is characterized by medium dense, silty sand layer at top followed by a layer of weathered rock and that continued up to the terminating depth of all boreholes. Around BH-1 a medium to stiff silty clay layer was observed followed by a weathered rock layer up to terminating depth.

4.5 Evaluation Of Strength And Deformation Parameters:

Stratum-I (Sandy Clay):

Design C = 0.40 kg/cm². Young's Modulus, E = 150 kg/cm².

(Refer to "Foundation Analysis and Design': 5th Edition. by J.E. Bowles. pp no. 125. Table 2-fill)

Now considering laboratory test result and pressure range between 0.25 kg/cm² to 2.00 kg/cm².

Average coefficient of volume change, mv = 0.018 cm²/kg.

From literature, coefficient of volume change, mv = 0.017 cm²/kg.

(Refer to "Standard Penetration Test, State-of-the-art-Report" by Ivan K. Nixon, Proceedings of the second European Symposium on Penetration Testing. Amsterdam. May 1982. pp-117)

Considering the above, use coefficient of volume change, mv = 0.018 cm²/kg. Stratum-Ill & IIIA

Weathered rock layer is treated as soil (Refer /RC: 78-2014. Clause 706.3.1.1.2). Es= Young's Modulus = 1500 kg/cm²

4.6 Determination of Bearing Capacity:

The Net Ultimate Bearing Capacity is given as (As per IS 6403):

$q_{nu} = (C\, N_e\, S_c\, D_e) + (q\, (N_q - 1)\, S_q\, D_q) + (0.5\gamma\, N_\gamma\, S_\gamma\, D_\gamma)$

Cohesion, C = 0,

The bearing capacity factors are (IS:6403 – 1981)

$N_c = 46.12$, $N_q = 33.30$, $N_\gamma = 48.03$

Depth of Foundation = Df = 3.00 m (below MSL) Width of Foundation = B = 3.00 m

Length of Foundation = L = 5.00 m

The Shape factors are (IS:6403 – 1981)

$S_c = 1.12$, $S_q = 1.12$, $S_\gamma = 0.76$

The Depth factors are (IS:6403 – 1981)

$D_e = 1.06$, $D_q = 1.05$, $D_\gamma = 1.05$

Computed Net Ultimate Bearing Capacity = **1357 kN/m²**

Using a factor of safety of 2.5, Net Safe Bearing Capacity = **542.88 kN/m²**

The above bearing capacity should be checked against settlement.

Settlement Calculation

For Stratum-I

As the founding material is sand, immediate is expected to occur.

Immediate settlement

$$Si= \frac{(P \times Z)}{Es}$$

Where,

P = Foundation Pressure at middle of layer = 2.23Kg/cm^2

Z = Thickness of compressible Layer = 1m

Es = young's Modulus = 250 Kg/cm^2

$$Si = \frac{(2.23 \times 1 \times 100)}{250} = 0.89 \text{ cm}$$

Considering permissible settlement as 50 mm, recommended allowable bearing capacity of Isolated foundation is **542.88 kN/m²**

Table 4.5: Recommended allowable bearing capacity values are as follows

Foundation location	Foundation Size (m x m)	Depth of foundation Below MSL (m)	Recommended allowable bearing capacity (kN/m²)
BH-1	(5 x 3)	3	542.88
BH-2	(5 x 3.1)	3	544.44
BH-3	(5 x 3.2)	3	547.45
BH-4	(5 x 3.3)	3	549.26

Note – 1. MSL means Maximum scour Level

2. Permissible Settlement 50 mm

4.7 Determination of Vertical Pile Capacity:

Sample calculation of safe vertical pile capacity around BH -1 (As per IS-2911(PART 1 / SEC 2:2010)

Embedded depth of Pile below ground level = 8.5 m

Diameter of pile = 800mm

Considering the Above Use, design C = 0 kg/cm² and $\phi = 35^\circ$

Also Use K = 1.25

The ultimate vertical pile capacity of Bored cast in situ RCC Pile in soil may be estimated using the formula as given below:

$Qu = (Ap \times P_D \times Nq) + (Ap \times Nc \times cp) + \sum(Ki \times P_{Di} \times \tan \delta i \times Asi) + \sum(\alpha i \times ci \times Asi)$

Where,

Qu = ultimate vertical load carrying capacity of RCC bored Pile

Ap = Cross sectional area of pile = $\frac{\pi}{4} \times D^2$

D = Diameter of pile

P_D= effective over burden pressure at pile tip.

Nq = Bearing capacity factor for bored pile depending on ϕ

Nc = Bearing capacity factor, may be taken as 9

Cp = Average cohesion at pile tip

Ki = coefficient of earth pressure in i^{th} layer

P_{Di} = Effective over burden pressure at the mid depth of i^{th} layer

δi = angle of wall friction between Pile and Soil for the i^{th} layer

Asi = surface area of Pile shaft In i^{th} layer = π x D x Li

Li = Length of pile in respective stratum

ϕ = Angle of internal friction of soil

$\alpha i =$ Adhesion factor for the i^{th} layer

Ci = Average cohesion for the i^{th} layer

Maximum depth upto which pressure will increase below MSL =18 m

(critical Depth = 15D)

Maximum effective OVP = 16.20 t/m²

4.8 Calculation of Safe End Bearing Resistance:

Ap = 3.14 x (0.8^2 / 4) = 0.502 m^2

Pᴅ = 100 kN/m²

Nq = 50 (AS per IS 2911 (PART 1 SEC -2): 2010)

Nc = 46.12

Pᴅi=0.4 x 15=6 kN/m²

Ultimate End Bearing Resistance = 0.502 x 100x 50= **2513.27 kN**

Safe End Bearing Resistance = 2513.27 / 5 (using FOS= 5) = **502.65 kN**

Safe vertical pile capacity

Recommended Vertical Pile capacity is **502.65 kN**

Similarly recommended vertical pile capacity of 700 mm Diameter pile is

Table 4.6: Recommended vertical pile capacity

Foundation location	**Pile diameter (mm)**	**Embedded Length of Pile below GL (m)**	**Recommended Vertical pile Capacity (kN)**
BH-1,BH-4	700	8.5	403.25
BH-2,BH-3	800	8.5	502.65

4.9 Determination of Lateral Pile Capacity:

The piles are often subjected to lateral forces under different conditions. In designing such piles, two criteria need to be satisfied: first, an adequate factor of safety against ultimate failure; and second, an acceptable deflection at working loads. The safe lateral load capacities as recommended may be moderated in design, keeping compatibility with structural design and acceptable horizontal deflection.

Safe lateral capacity of fixed head pile for permissible lateral deflection (As per IS 2911 (Part 1 / Sec 2) 2010) is given by

$$y = \frac{H\,(e+zf\,)^3}{12EI}$$

where,

y = Deflection of pile head. (1% of pile head as per IRC: 78-2014)

H = Safe lateral load capacity.

E = Young's Modulus of pile material.

I = Moment of inertia of pile cross section.

Zf = Depth of fixity.

e = Cantilever length below bottom of pile cap.

The sample calculation (as per appendix – C of IS 2911 (Part 1 / Sec 2): 2010)

Is provided for help of the foundation designer, who may use the actual data adopted in design, before ascertaining the pile capacity.

Sample Calculation of Lateral Pile Capacity (As per 2911 (Part 1 / Sec 2)2010):

Diameter of pile = 800 mm

Refer to IS: 2911 (Part1/Sec 2)-2010, Appendix – C

Constant Factor, k1 = 1.44 kg/cu.cm corresponding to cohesion = 0.40 kg/sq.cm

Now, K = (k_1 /1.5) x (30/D) which is coming as 0.24kg/cu.cm (D= Diameter of pile in cm)

Stiffness factor, $R = (EI/KD)^{1/4}$

Now, $I = 0.0201\ m^4$ (for 800mm diameter pile)

$E = 5000\sqrt{fck} = 5000 \times \sqrt{40} = 31622.47\ N/mm^2$

Hence, R = 3.392 m

R = 339.22 cm

Length of pile, L = 8.5m

Now, Moment = (H x L) = (24 x 8.5) = 204 kN.m per T of thrust

The reduction factor for computation of maximum moment in pile, m = 0.70

So, the corrected actual moment, M = 204 x 0.70 = 142.8 kN.m per T of thrust

Recommended Lateral Pile Capacity is 142.8 kN.m.

Similarly Recommended Lateral Pile Capacity of 700 mm diameter pile is 124.95

Table 4.7: Recommended lateral pile capacity and moment

Foundation Location	**Pile Diameter (mm)**	**Embedded Length of Pile Below G.L. (m)**	**Recommended Lateral Pile Capacity (kN)**	**Moment (kNm per 1T Of thrust)**
BH-1,BH-4	700	8.5	403.25	142.8
BH-2,BH-3	800	8.5	502.65	124.95

Chapter 5

Analysis

5.1 Lateral Earth Pressure:

It is well-known that an incorrect implementation in the design earth pressure may lead to uneconomical or even unsafe designs. Traditionally, apparent earth pressure diagrams are used for designing excavation support systems. These diagrams are semi-empirical approaches back-calculated from field measurements of strut loads which do not represent the actual earth pressure or its distribution with depth. Therefore, apparent earth pressure diagrams are only appropriate for sizing the struts. As previously mentioned, the use of these diagrams yield support systems that are adequate with regards to preventing structural failure, but may result in excessive wall deformations and ground movements.

5.2 Different Types of Lateral Earth Pressure:

1. **At-rest: The** lateral earth pressure is called at rest when the soil mass is not to any lateral yielding or movement. For example bridge abutment wall which is restrained at its top by

the bridge slab. The condition is also know as the elastic equilibrium, as no part of soil mass has failed and attained the equilibrium.

2. **Active pressure:** A state of active pressure occurs when the soil mass yields in such a way that it tends to stretch horizontally. It state of plastic equilibrium as the entire soil mass is on the verge of failure.
3. **Passive pressure:** A state of passive pressure exist when the movement of the wall is such that the soil tends to compress horizontally. It is another extreme of equilibrium condition.

5.3 Variation of pressure:

Fig. shows the variation of earth pressure with wall movement. Point B indicates the at-rest pressure. Point A indicates the active pressure. Point C indicates the passive pressure.

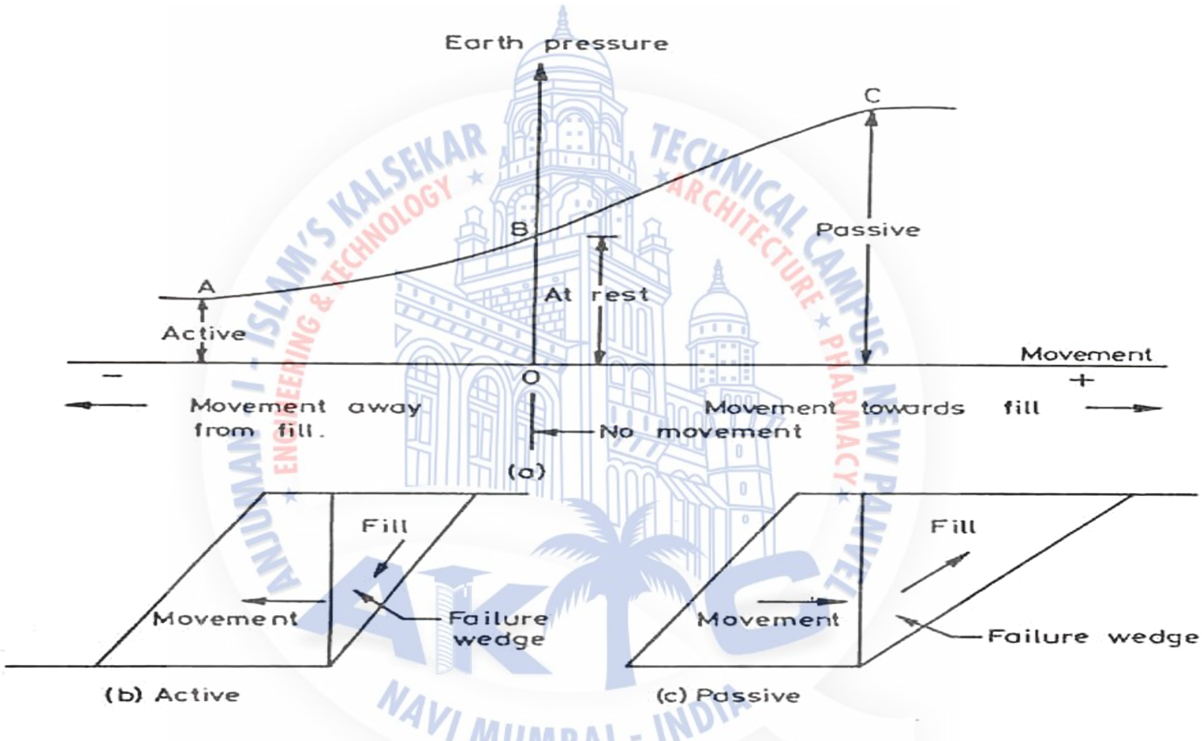

Fig. 5.1 Pressure variation at rest, active and passive with wall movement.

5.4 Rankine's Earth Pressure:

Rankine (1857) presented a solution for lateral earth pressures in retaining walls based on the theory of plastic equilibrium. He assumed that there is no friction between the retaining wall and the soil, the soil is isotropic and homogenous, the friction resistance is uniform along the failure surface, and both the failure surface and the backfilled surface are planar.

When the retaining wall in Figure a moves from AB to A'B' the horizontal stresses in back of and in front of the retaining wall will decrease and increase, respectively, while the vertical stresses remain constant. Rankine called the stresses in back of and in front of the retaining wall active earth pressure and passive earth pressure, respectively.

For a soil exhibiting both effective cohesion, 'c , and effective angle of internal friction, φ' , the Rankine earth pressures are given by:

Active case:

$\sigma'a = \sigma' Ka - 2c'\sqrt{Ka}$

where, Ka = $tan^2(45° - \emptyset'/2)$

Passive case:

$\sigma'p = \sigma'v\ Kp + 2c'\sqrt{Kp}$

where, Kp = $tan^2(45 + \emptyset'/2)$

the above expression are adequate for evaluating long-term lateral unloading conditions, which are the most critical in excavation.

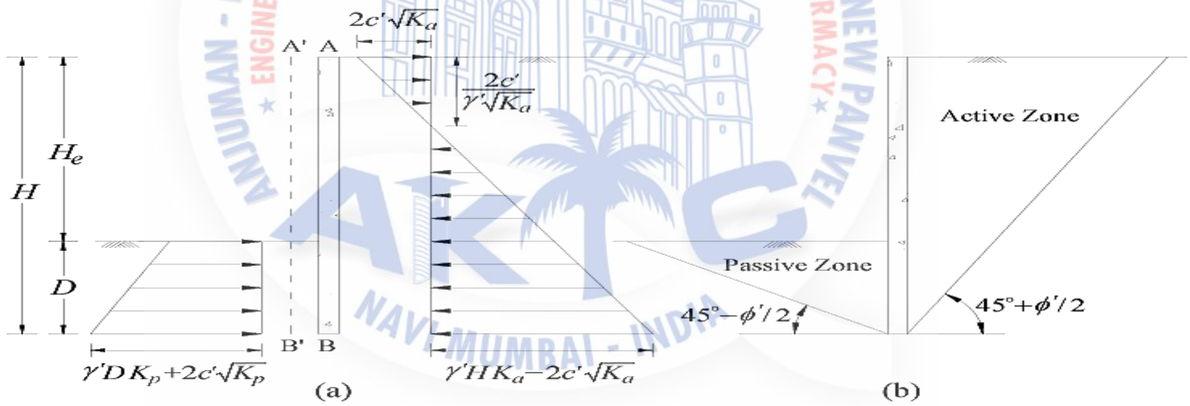

Fig.5.2 Rankine's pressure showing active and passive zone of retaining wall.

5.5 General Deflection Behavior of an Excavation Support System :

Lateral wall deformations and ground surface settlements represent the performance of excavation support systems. These are closely related to the stiffness of the supporting system, the soil and groundwater conditions, the earth and water pressures, and the construction procedures.

Excavation activities generally include three main stages:

(i) Installation of retaining wall,

(ii) Excavation of soil mass and installation of lateral support elements, and may or may not include,

(iii) Removal of the supports and backfill.

Fig. shows the general deflection behavior of the wall in response to the excavation presented by Clough and O'Rourke (1990). Figure 5.3.a shows that at early phases of the excavation, when the first level of lateral support has yet to be installed, the wall will deform as a cantilever. Settlements during this phase may be represented by a triangular distribution having the maximum value very near to the wall. As the excavation activities advance to deeper elevations, horizontal supports are installed restraining upper wall movements. At this phase, deep inward movements of the wall occur (Figure 5.3.b). The combination of cantilever and deep inward movements results in the cumulative wall and ground surface displacements shown in Fig.5.3.c below Clough and O'Rourke (1990) stated that if deep inward movements are the predominant form of wall deformation, the settlements tend to be bounded by a trapezoidal displacement profile as in the case with deep excavations in soft to medium clay; and if cantilever movements predominate, as can occur for excavations in sands and stiff to very hard clay, then settlements tend to follow a triangular pattern.

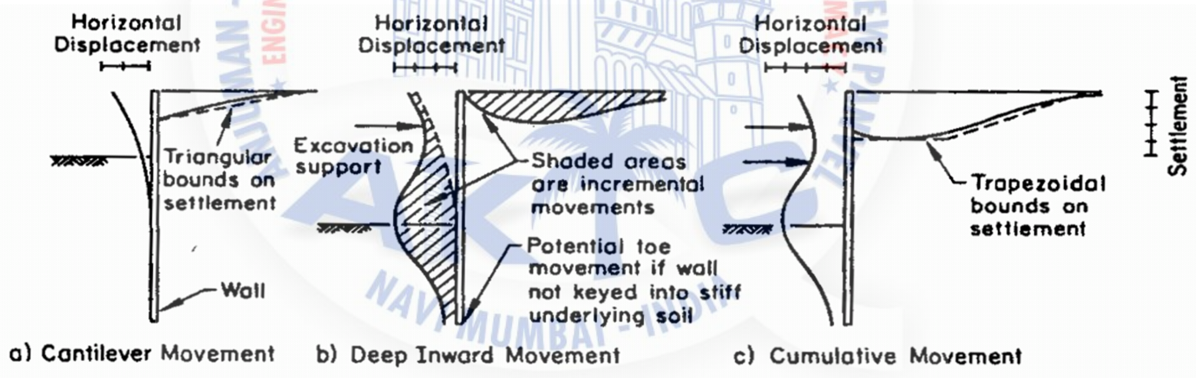

Fig. 5.3.a.b.c Deflection behavior of excavation support

5.6 Analysis of Pile for Underground Excavation for Support System:

An underground basement is to be constructed for a polytechnic for educational purpose. The length is 100 metres and width id 70 metres with and depth of 6 metres below ground level. Two boreholes have been sunk. Borehole A within the confines of the proposed area and borehole B is at distance of 50 metres distant. Borehole A and B extends to a depth of 8.5 metres and terminates in sand.

Table 5.1: Calculation Pressure of Active and Passive

RL	Ground	Parameters Surcharge= 10kN/m²	Depth Z	Total Vertical Stress (σ_v)	Pore water Pressure (γw=9.81kN/m²) u	Effective Vertical Stress σ_v'	Effective Horizontal Pressure Pa′	Total Horizontal Pressure Pa
6m	Top Soil	γ =18 kN/m² c = 20 kN/m² ø = 30 Ka = 0.33 Kp = 3	0m 0.5m	10 19	0 0	10 19	-19.67 -16.70	-19.67 -16.70
5.5m	Sandy Clay	γ =15 kN/m² c = 0 ø = 35 Ka = 0.27 Kp = 3	1m	19 34	0 0	19 34	5.13 9.18	5.13 9.18
4.5m	Silty Sand	γ =20.3 kN/m² c = 0 ø = 32 Ka = 0.31 Kp = 3.25	3m	34 94.9	0 29.43	34 65.47	10.54 20.29	10.54 49.72
1.5m	Sand	γ =15 kN/m² c = 0 ø = 38 Ka = 0.24 Kp = 4.2	1.5m	94.9 117.4	29.43 44.145	65.47 73.255	15.71 17.58	45.143 61.726
0.0m	Sand	γ =15 kN/m² c = 0 ø = 38 Ka = 0.24 Kp = 4.2	1.5m	117.4	44.145	73.255	17.58	61.726
-2.5m	Sand	γ =15 kN/m² c = 0 ø = 38 Ka = 0.24 Kp = 4.2	-2.5m	37.5	24.525	12.975	54.495	79.02

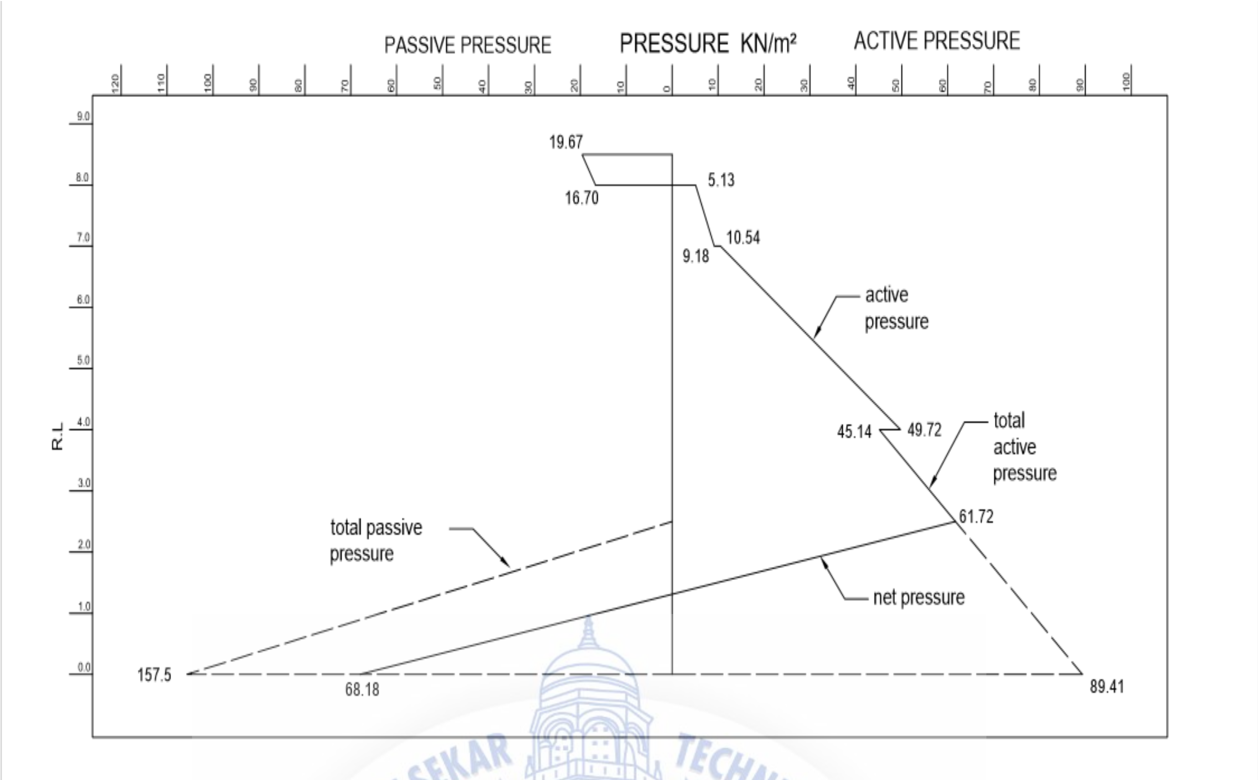

Fig.5.4 Total Pressure Diagram Of Active And Passive (pressure in kN/m²)

5.7 Analysis Using Net Pressure:

By inspection it is clear that there is enough passive pressure below 6m level to safely assume fixed earth support and that the eventual pile length will be adequate for this stage.

Assume that the point of contra flexure of the pile at this stage is at 6m

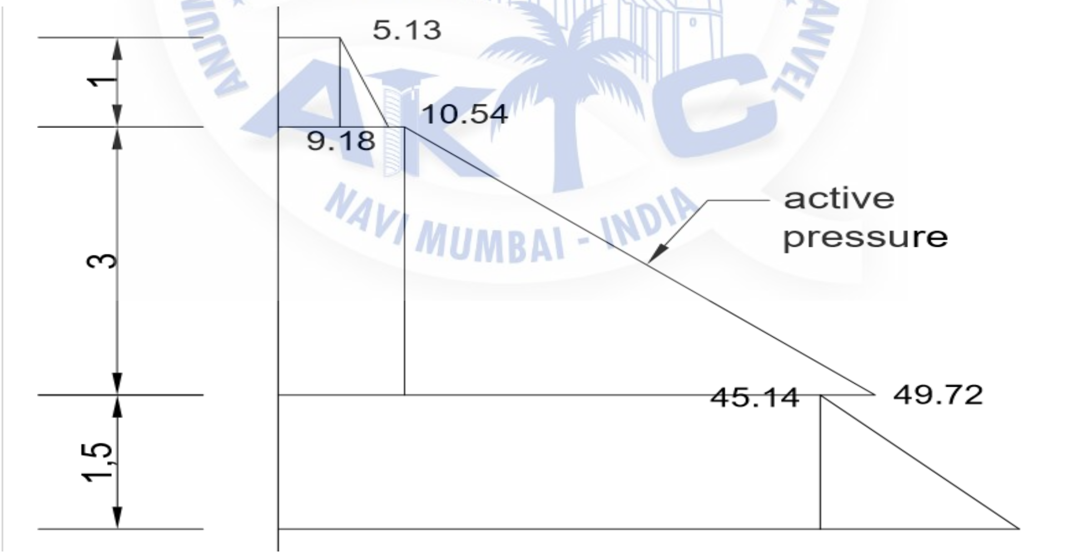

Fig.5.5 Active pressure (height in metres, pressure in kN/m²)

Table 5.2: calculation for load and moment of active pressure diagram

Force (kN)	Lever Arm (m)	Moment (kNm)
5.13 x 1 = 5.13	1.5+3+0.5 = 5	25.65
½ x 4.05 x 1 = 2.025	1.5+3+0.33 = 4.83	9.780
10.54 x 3 = 31.62	1.5+1.5 = 3	94.86
½ x 39.13 x 3 = 58.77	1.5+3/3 = 2.5	146.775
45.14 x 1.5 = 67.71	1.5/2 = 0.75	50.7825
½ x 16.58 x 1.5 = 12.435	1.5/3 = 0.5	6.217
Total = 177.69 kN	**16.58m**	**334.0645 kNm**

Load per metre run = $\frac{334.0645}{6}$ = 60.73 kN per m run.

5.8 Approximate analysis to find the embedded depth for pile:

The exact analysis of cantilever sheet pule as discussed above is quite involved. An approximate value of d can be obtain using pressure diagram as shown in figure. In this analysis, the resistance of the pile below the point is replaced by a concentrated force.

To find depth

$\frac{1}{2}$ x $(493.08\times31.6D)\times D\times\frac{D}{3}$ = $(5.13\times1)\times\{0.5\times(3+1.5+D)\}$ + $(0.5\times4.05\times1)\times\left\{\frac{1}{3}+3+1.5+D\right\}$+$(10.54\times3)\times\left\{\frac{3}{2}+1.5+D\right\}$+$(0.5\times39.18\times3)\times\left\{\frac{1}{3}\times3+1.5+D\right\}$+$(45.14\times1.5)\times(\frac{1.5}{2}+D)$+$(16.58\times0.5\times1.5)\times\left\{\frac{1}{3}\times1.5+D\right\}$+$(61.72\times D)\times\left\{\frac{D}{2}\right\}$+$0.5\times(33.55+11.06D)\times D\times(\frac{D}{3})$

$\frac{D^2}{6}(493.08+31.6D) = 5.13\times(5+D)+2.025\times(4.83+D)+31.62\times(3+D)+58.77\times(2.5+D)+67.71\times(0.75+D)+124.35\times(0.5+D)+33.86D^2+0.16D^2\times(33.55+11.06D)$

$82.18D^2+5.26D^3=25.65+5.13D+9.78+2.025D+94.86+31.62D+146.92+58.77D+50.78+67.71D+62.175+124.35D+33.86D^2+5.386D^2+1.7696D^3$

D = 1.85m

Depth factor = 1.2D to 1.5D

Therefore, D=1.2 x 1.85= 2.5m

Therefore, embedded depth of the pile in the soil is 2.5m below excavation level.

Chapter 6

Design of Piles

Using IS 2911 2010 for the design of secant pile.

6.1 Design Consideration:

6.1.1 General:

Pile foundation shall be designed in such a way that the load from the structure can be transmitted to the sub-surface with the adequate factor of safety against shear failure of sub-surface and without causing such settlement, which may result in structural damage and/or functional distress under permanent/transient loading. The pile shaft should be have adequate structural capacity to withstand all loads and moment which are to be transmitted to the subsoil and shall be designed to IS 456.

6.1.2 Pile Capacity:

The load carrying capacity of a pile depends on the properties of soil in which it is embedded. Axial load from a pile is normally transmitted to the soil through skin friction along the shaft and end-bearing as it tip. A horizontal load on vertical pile is transmitted to the subsoil primarily by horizontal subgrade reaction generated in the upper part of the shaft. Lateral load capacity depends upon the soil reaction developed and the structural capacity if the shaft under bearing. It would be essential to investigate the lateral load capacity of the pile using appropriate values of horizontal subgrade modulus of soil. Alternatively, pile may be installed in rake.

6.1.3 Spacing of Piles:

The minimum centre-to-centre spacing of pile is considered from the aspects, namely,

- Practical aspect of installing the piles,
- Diameter of the pile, and
- Nature of the load transfer to the soil and possible reduction in the load capacity of pile groups

6.1.4 Factor of Safety:

Factor of safety should be chosen after considering,

- The reliability of the calculated value of ultimate load capacity of pile,
- The type of superstructure and the type of loading,
- Allowable total/differential settlement of the structure.

6.2 Design Parameters:

6.2.1Design Material Parameters:

Table 6.1 : Material Parameter:

Item	Value
Density of Concrete	25 kN/m³
Density of Water	9.81 kN/m³
Concrete Grade of Pile	M40
Modulus of Elasticity of Concrete	$5000\sqrt{fck}$
Steel Grade	Fe500

Assumptions :

Following assumptions have been made during the design of secant pile.

- Factor of safety adopted for temporary slope is 1.35.
- Factor of safety adopted for the structural calculation is 1.5.
- Proper alignment of pile shall be ensured on site by the contractor.

6.2.2 Construction Levels :

Table 6.2 : Descriptions of size of the pile

Items	Value
Ground level	0 m
Diameter of pile	0.80m, 0.70m
Centre to centre distance of pile	1.30

6.3 Structural Design of Pile :

6.3.1 Designing for 0.80m diameter:

Axial Load (Pu) = 100KN

Bending Moment (M) = 334.064 KN.M

Factored Bending Moment (Mu) = 334.064 x 1.5

= 501.1 KN.M

Diameter of Pile = 800mm

Gross area of pile = $Ag = \pi r^2 = \pi \times 400^2 = 502654.825$ mm²

Assume cover = 80 mm

$$\frac{d'}{D} = \frac{80}{800} = 0.1$$

$$\frac{Pu}{fck \times D^2} = \frac{100\times10^3}{40\times800^2} = 0.0039$$

$$\frac{Mu}{fck \times D^3} = \frac{501.1\times10^{\wedge}6}{40\times800^3} = 0.0244$$

Referring IS:SP-16 to find the values for calculation

From IS:SP-16, Chart Number : 60,

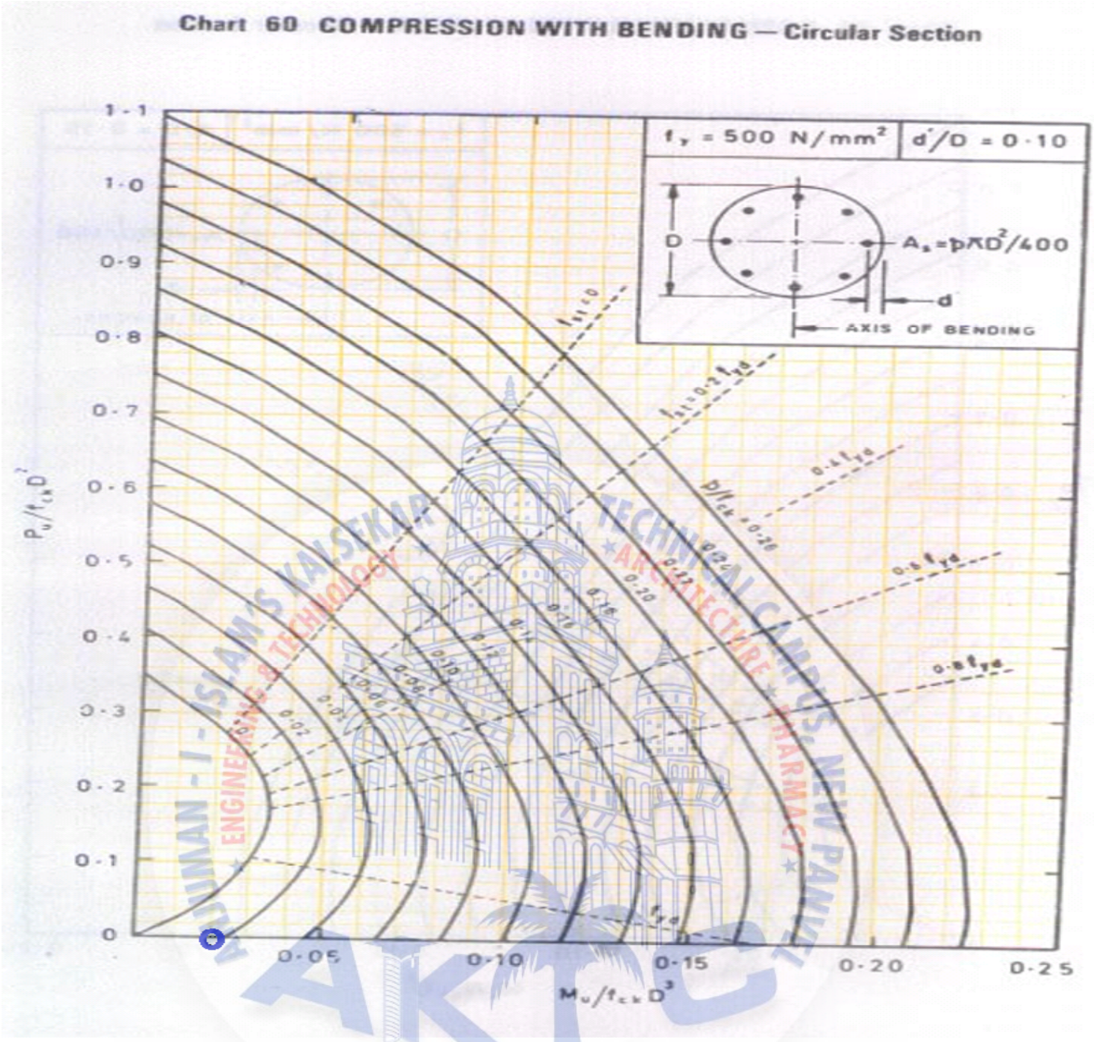

fig. 6.1 IS:SP 16, Chart:60, compression with bending

For fy = 500 N/mm² and $\frac{d'}{D}$ = 0.10

$\frac{p}{fck}$ = 0.02

p = 0.02 x 40 = 0.8 %

pt % = $\frac{Asc}{\pi \text{ x } r^2}$ x 100

$0.8 = \frac{Asc}{\pi \times 400^2} \times 100$

Area of steal in concrete (Asc) = 4021.24 mm²

Assuming 32 mm diameter of bars.

Number of Bars required = $\frac{Asc}{asc} = \frac{4021.24}{\frac{\pi}{4} \times d^2} = \frac{4021.24}{\frac{\pi}{4} \times 32^2}$ = 5 No.

But minimum number of bars required for circular section = 6 No.

Providing 6 bars of 32mm diameter.

Area of steal in concrete,

Asc provided = 6 x $\frac{\pi}{4}$ x 32² = 4825.486 mm²

Check for Asc:

Minimum Asc = 0.8% of Ag = $\frac{0.8}{100}$ x 502654.825 = 4021.239 mm²

Maximum Asc = 6% of Ag = $\frac{6}{100}$ x 502654.825 = 30159.29 mm²

Provided Asc within the limit.

Therefore, Design is Ok.

Puz = 0.45 x fck x Ac + 0.75 x fy x Asc

Puz = 0.45 x 40 x (Ag – Asc) + 0.75 x 500 x 4825.486

Puz = 0.45 x 40 x (502654.825 – 4825.486) + 0.75 x 500 x 4825.486

Puz = 10770.485 KN

$\frac{Pu}{Puz} = \frac{100}{10770.485}$ = 0

From the standard values given by IS code

αn = 1 for $\frac{Pu}{Puz} \leq 0.2$

αn = 2 for $\frac{Pu}{Puz} \geq 0.8$

Therefore, $\alpha n = 1$

From IS:SP-16, Chart number: 60. (Referring same figure 5.1)

For $\frac{Pu}{fck \times D^3}$ = 0.0244 and $\frac{p}{fck}$ = 0.02

Therefore, $\frac{Mux1}{fck \times D^3}$ = 0.025

Mux1 = 40 x 800^3 x 0.025 = 512 KN.M

$(\frac{Mux}{Mux1})^{\wedge}\alpha n = (\frac{501.1}{512})$ = 0.9787 < 1

Therefore, Ok.

Design of Lateral Ties :

Diameter of lateral ties = $\frac{1}{4}$ x diameter of main bar or 8mm whichever is more.

= $\frac{1}{4}$ x 32 or 8mm

= 8mm

Spacing For Lateral Ties is the minimum of the following;

- Least lateral dimension = 800mm
- 16 x diameter of main bar = 16 x 32 = 512mm
- 48 x diameter of lateral ties = 48 x 8 = 384mm
- 300mm

Therefore providing 8mm diameter bars at 300mm centre to centre.

6.3.2 Designing for 0.70m diameter:

Axial Load (Pu) = 100KN

Bending Moment (M) = 334.064 KN.M

Factored Bending Moment (Mu) = 334.064 x 1.5

= 501.1 KN.M

Diameter of Pile = 700mm

Gross area of pile = $Ag = \pi r^2 = \pi \times 350^2 = 384845.1$ mm²

Assume cover = 70 mm

$\frac{d'}{D} = \frac{70}{700} = 0.1$

$\frac{Pu}{fck \times D^2} = \frac{100 \times 10^3}{40 \times 700^2} = 0.005$

$\frac{Mu}{fck \times D^3} = \frac{501.1 \times 10^{6}}{40 \times 700^3} = 0.0365$

From IS:SP-16, Chart Number : 60,

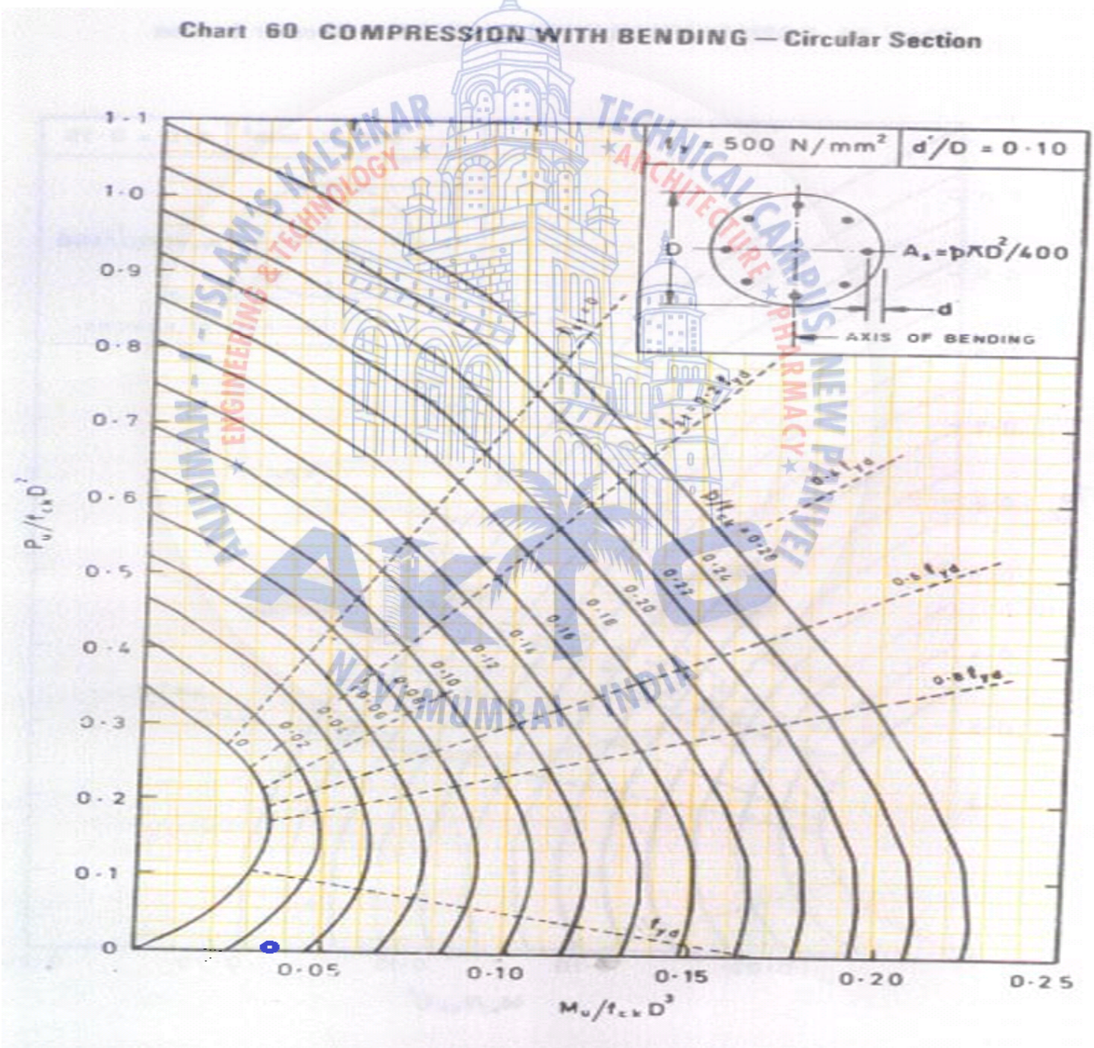

fig. 6.2 IS:SP 16, Chart:60, compression with bending

For fy = 500 N/mm² and $\frac{d'}{D}$ = 0.10

$\frac{p}{fck}$ = 0.03

p = 0.03 x 40 = 1.2 %

pt % = $\frac{Asc}{\pi \times r^2}$ x 100

1.2 = $\frac{Asc}{\pi \times 350^2}$ x 100

Area of steal in concrete (Asc) = 4618 mm²

Assuming 32 mm diameter of bars.

Number of Bars required = $\frac{Asc}{asc} = \frac{4618}{\frac{\pi}{4} \times d^2} = \frac{4618}{\frac{\pi}{4} \times 32^2}$ = 6 No.

Providing 6 bars of 32mm diameter.

Asc provided = 6 x $\frac{\pi}{4}$ x 32² = 4825.486 mm²

Check for Asc:

Minimum Asc = 0.8% of Ag = $\frac{0.8}{100}$ x 384845.1 = 3078 mm²

Maximum Asc = 6% of Ag = $\frac{6}{100}$ x 384845.1 = 23090.706 mm²

Provided Asc within the limit.

Therefore, Design is Ok.

Puz = 0.45 x fck x Ac + 0.75 x fy x Asc

Puz = 0.45 x 40 x (Ag – Asc) + 0.75 x 500 x 4825.486

Puz = 0.45 x 40 x (384845.1 – 4825.486) + 0.75 x 500 x 4825.486

Puz = 9505 KN

$\frac{Pu}{Puz} = \frac{100}{9505}$ = 0.01

From the standard values given by IS code

$\alpha n = 1$ for $\frac{Pu}{Puz} \leq 0.2$

$\alpha n = 2$ for $\frac{Pu}{Puz} \geq 0.8$

Therefore, $\alpha n = 1$

From IS: SP-16, Chart number: 60 (Referring same figure 5.2)

For $\frac{Pu}{fck \times D^3} = 0.005$ and $\frac{p}{fck} = 0.03$

Therefore, $\frac{Mux1}{fck \times D^3} = 0.04$

$Mux1 = 40 \times 700^3 \times 0.04 = 548.8$ KN.M

$(\frac{Mux}{Mux1})^\wedge\alpha n = (\frac{501.1}{548.8}) = 0.91 < 1$

Therefore, Ok.

Design of Lateral Ties :

Diameter of lateral ties = $\frac{1}{4}$ x diameter of main bar or 8mm whichever is more.

$= \frac{1}{4} \times 32$ or 8mm

= 8mm

Spacing For Lateral Ties is the minimum of the following :

- Least lateral dimension = 700mm
- 16 x diameter of main bar = 16 x 32 = 512mm
- 48 x diameter of lateral ties = 48 x 8 = 384mm
- 300mm

Therefore providing 8mm diameter bars at 300mm centre-to-centre.

Chapter 7

Plaxis Modelling

7.1 Introduction:

The Finite Element Method (FEM) has been used increasingly for the analysis of stress, deformation, structural forces, bearing capacity, stability and groundwater flow in geotechnical engineering applications. Besides developments related to the method itself (e.g. new constitutive models for soil and rock, new numerical procedures and calculation methods), the role of the FEM has evolved from a research tool into a daily engineering tool. The method has obtained a position next to conventional design methods, and offers significant advantages in complex situations.

Plaxis is a finite element package that has been developed specially for the analysis of deformation and stability in geotechnical engineering projects. The simple graphical input procedure enable a quick generation of complex finite element model, and the enhanced output facilities provide detailed presentation of computational results. The calculation itself is fully automated and based on robust numerical procedure. This concept enables new users to work with the package after only a few hours of training.

7.2Finite Element Simulation :

The problem was simulated using a complete two-dimensional model of the polytechnic in South Mumbai, Vile Parle, near airport and the secant pile wall is to cast. The finite element software PLAXIS 3D FOUNDATION was used to compute the response of the soil around the secant pile wall. Figure 3.14 shows a schematic of the PLAXIS input model. Details about the definition of the finite element problem, the calculation phases, and the model parameters used in the simulation described herein can be found in Appendix A. The soil stratigraphy was assumed to be uniform across the site (see Figure 3.12). Seven uniform soil layers were considered in the analysis: (1) a sand fill layer, (2) a clay crust, (3) a soft clay layer named Upper Blodgett, (4) a medium clay layer named Lower Blodgett, (5) a medium clay layer named Deerfield, (6) a stiff silty clay stratum known as Park Ridge, and (7) a hard clay stratum. The Hardening Soil Model (Schanz et al., 1999) was used to represent the elasto-plastic response of the clay soil layers while the sand fill and the clay crust layer were modelled using the classical Mohr-Coulomb soil model. A complete description of the Hardening Soil Model can be found in Appendix B.

7.3 Finite Element Models :

A total of 48 finite element simulations, performed in the three-dimensional software package PLAXIS 3D FOUNDATION, are the basis of the parametric study conducted to overcome the deficiencies of the actual methods used to predict maximum wall movements for deep excavations in cohesive soils. Figure 4.6 shows a PLAXIS 3D FOUNDATION schematic of one of the finite element models used in the analyses. Note that only half of the excavation was modeled because symmetry conditions applied to both the geometry and excavation sequence. In the simulations, soil elements were modeled with 15-node wedge elements that are generated from the projection of two-dimensional 6-node triangular elements between work planes. The 15-node wedge element is composed of 6-node triangles in the horizontal direction and 8-node quadrilaterals in the vertical direction. As expressed by Brinkgreve and Broere (2006), the accuracy of the 15-node wedge element and the compatible structural elements is comparable with the 6-node triangular element and compatible structural elements in a 2D PLAXIS analysis.

7.4 Secant Pile Wall Construction Simulation :

The wall is constructed on size of 100 x 70m plot. Half side is considered for calculation which shows all the displacement of the pile at certain stage. The first is to put all the material properties at for every soil layer. After the material properties mesh generation is to for local refinement of soil. Intial condition are given to current project for generation of water pressure deactivation of the structure and loads and the generation of initial stress. The required all data has been provided to the finite element and then it concluded the calculation and result if the pile.

There are 4 phases of calculation

- Phase1- External Load,
- Phase 2- excavation stage,
- Phase3- installation of strut,
- Phase4- second excavation stage (final stage)

Viewing of results:

In additional to the displacement the stresses in the soil, the output progam can be used to view the forces in structural objects. To examine the results of this project, follow the steps:

- Click on the final calculation phase in the calculation window,
- Click on the output button on toolbar. Output result is displayed,
- Then select total increments from deformation menu. The plot showed the displacement increment of all nodes as arrow and the length indicate the relative magnitude,
- Then select effective stress from stress menu. The plot shows the magnitude and direction of the principal effective stresses. The orientation of principal stress indicates large passive zone under bottom of excavation.

7.5 Results of calculation in the modelling software

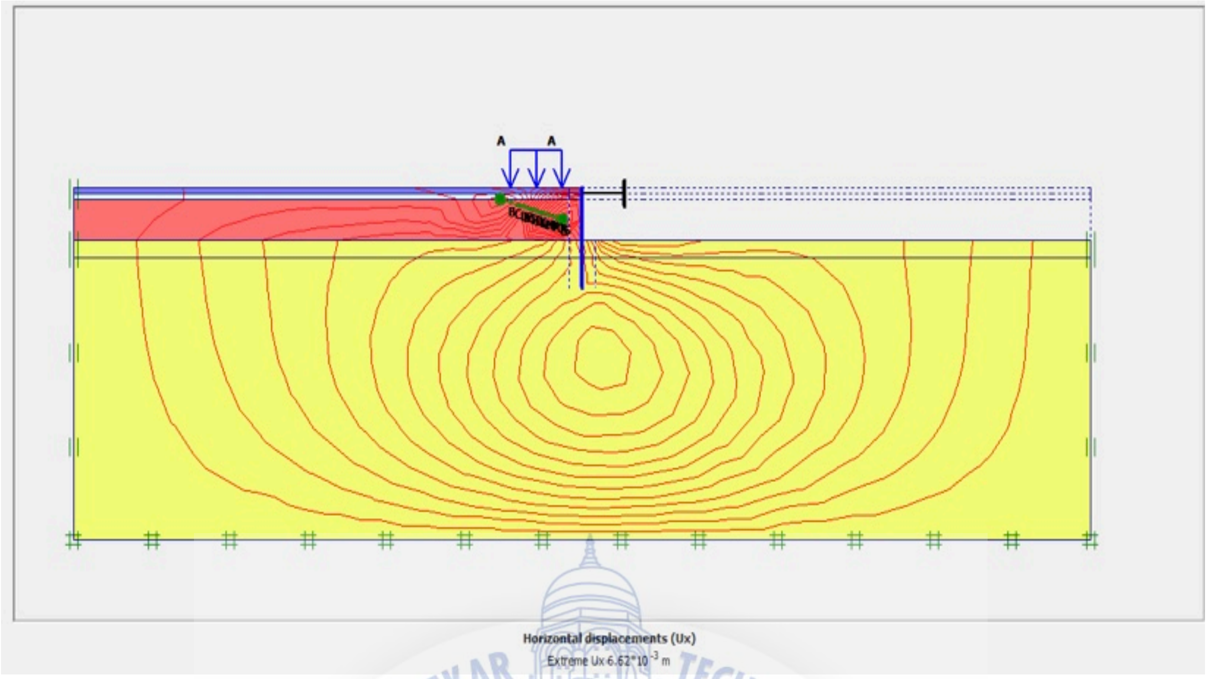

fig.7.1 horizontal displacement of pile due to surcharge loading

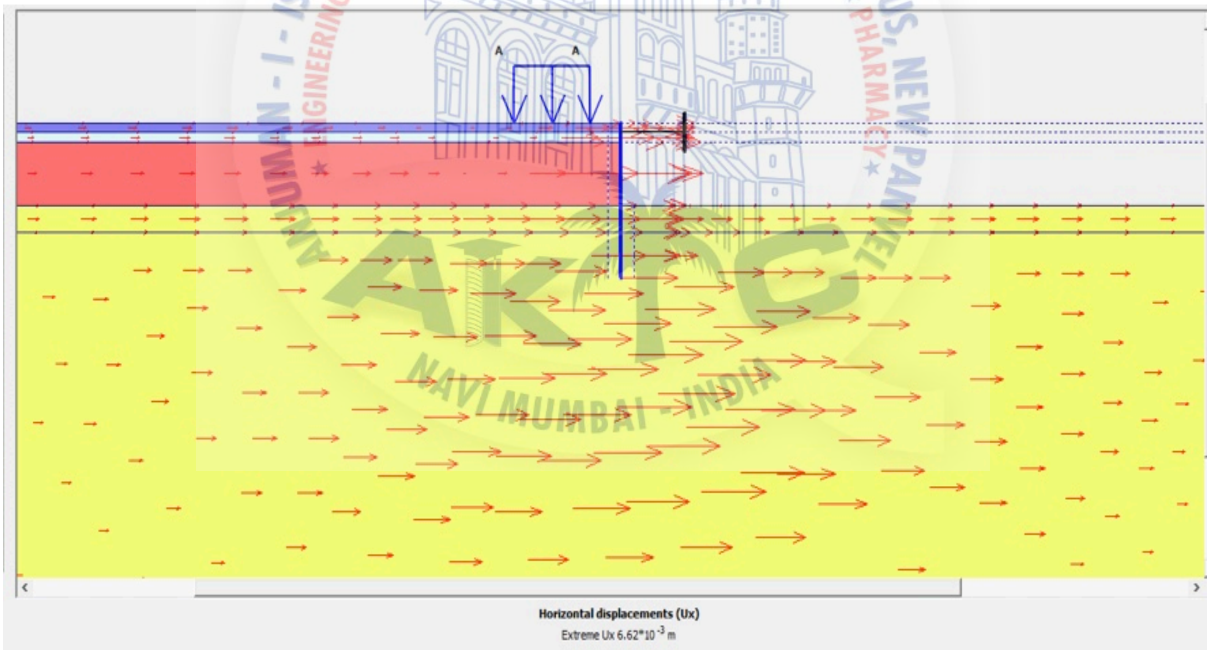

Fig. 7.2 Relative magnitude of the pile deformation

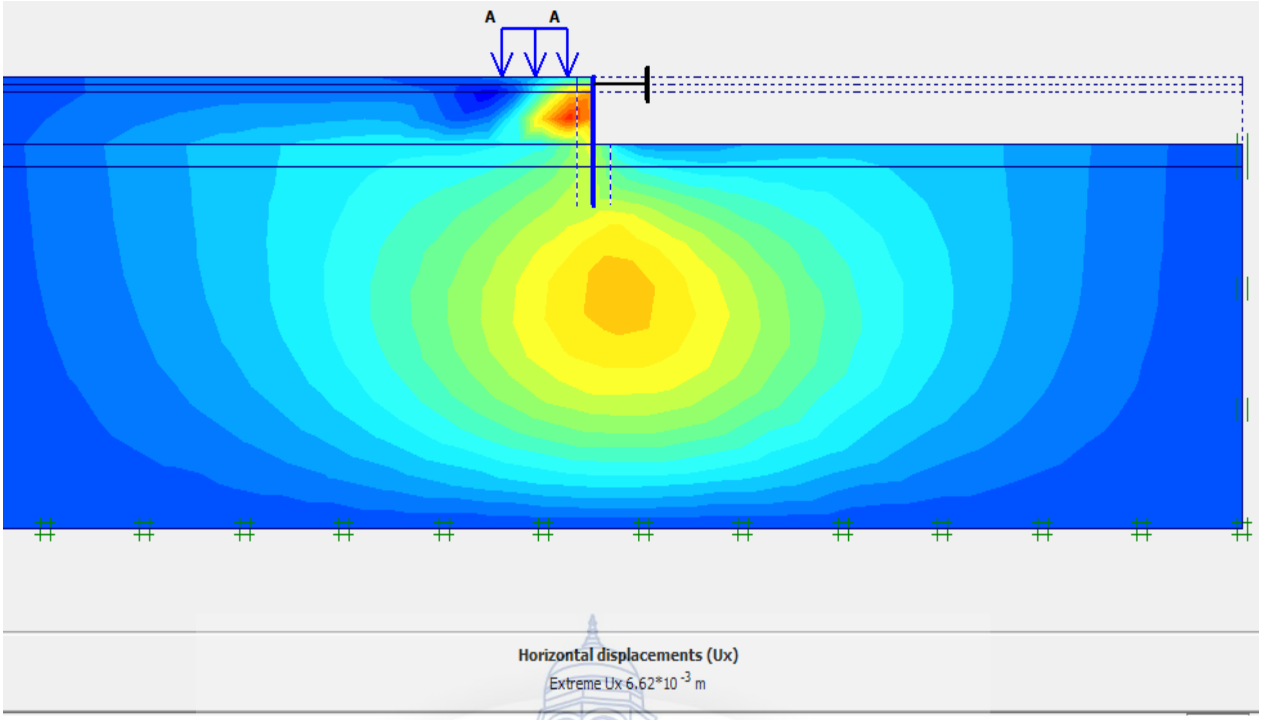

Fig.7.3 Horizontal displacement at final phase

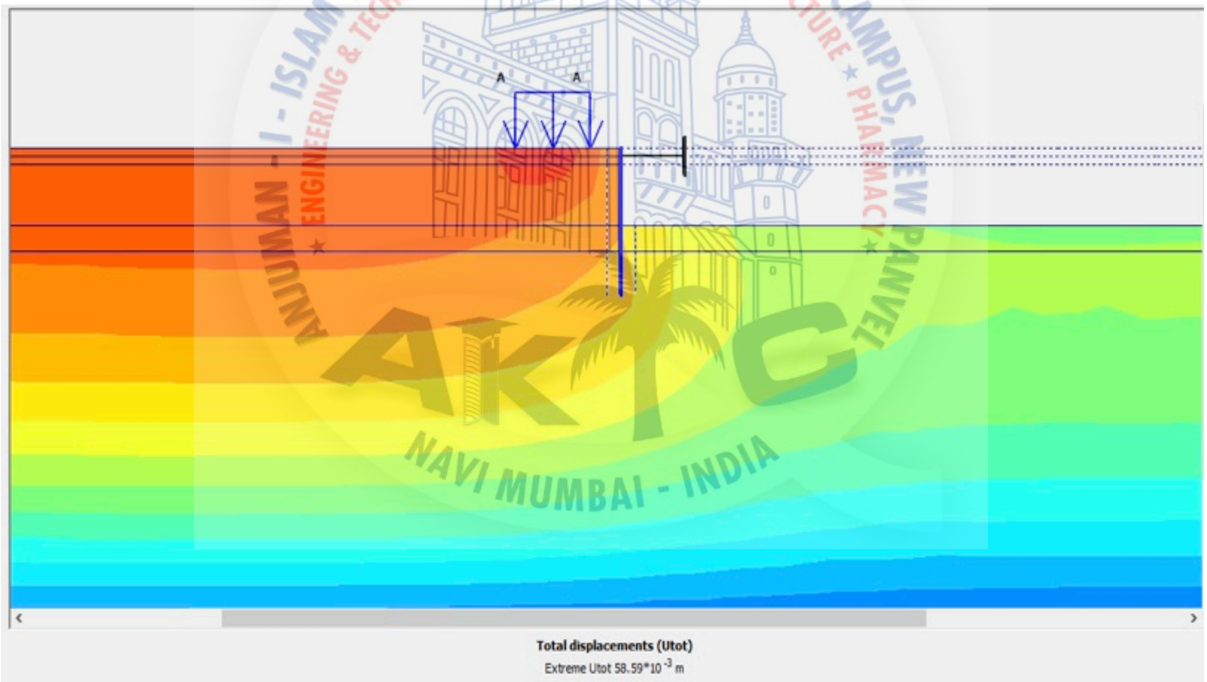

Fig.7.4 Total displacement after final phase with all loading

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