“Design of Deep Excavation with Different Methods”

A DISSERTATION

SUBMITTED IN PARTIAL FULFILLMENT

FOR REQUIREMENT OF THE DEGREE OF

MASTER OF TECHNOLOGY

IN

GEOTECHNICAL ENGINEERING

Submitted by

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CANDIDATE'S DECLARATION

I, **Akash ojha, 2K20/GTE/02**, student of M.Tech (Civil Engineering), hereby declare that the project dissertation titled **"Design of Deep Excavation with Different Methods"** is submitted to the Department of Civil Engineering, Delhi Technological University, Delhi, by me in partial fulfillment of requirement for the award of degree of **Master of Technology (Geotechnical Engineering)**. This thesis is original work done by me and not obtained from any source without proper citation. This project work has not previously formed the basis for award of any degree, diploma, fellowship or other similar title or recognition.

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CERTIFICATE

I hereby certify that project dissertation titled **"Design of Deep Excavation with Different Methods"** submitted by **Akash Ojha, 2K20/GTE/02,** Department of Civil Engineering, Delhi Technological University, Delhi, in partial fulfillment for the award of degree of Master of Technology, is a project work carried out by the student under my supervision. To the best of my knowledge, this work has not been submitted in part or full for any degree or diploma to this university or elsewhere.

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ABSTRACT

As of excavation supporting methods, sheet pile walls for supporting earth are commonly used. In this report, a validated 2-dimensional finite element (FE) model was used to test lateral earth pressure distribution on sheet pile walls, as well as an empirical approach derived from Peck and Terzaghi 1967, with the goal of comparing numerical modelling and empirical sheet pile wall design. Finite element analysis can be used to calculate the factor of safety for excavations. The results of the research were helpful in the development of safer and more cost-effective sheet pile walls. The empirical method has a limitation that they can be used for a the same type of problem for the problem with special or new design such as any ground stabilisation or cross wall or two approaches are mixed while design than the empirical method can lead to incorrect design.

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CHAPTER 1 – INTRODUCTION

1.1 BACKGROUND

Rapid urbanisation, industrialisation, and population increase in developed, developing, and underdeveloped countries of metropolitan and cosmopolitan cities has resulted in significant land space restrictions. As a result, civil engineers have begun to choose underground area for the building of various infrastructure facilities such as tunnels, basements, and underground utilities. Subsurface deep excavation presents a significant challenge to the engineer, not only in terms of providing a stable underground system, but also in terms of adjoining structure safety measures, both during the pre-construction and post-construction phases of the project.

Even though advanced underground construction technologies such as diaphragm walls, contiguous piles, and soil nailing have overcome most of the prevailing difficulties of traditional underground support systems such as braced excavations, an engineer still faces a difficult task in overcoming the problems encountered during execution for a safe design as well as safeguarding adjacent structures, utilities, and mankind's livelihood. The literature on real-time monitoring of subterranean excavation systems is quite sparse, and there is a tremendous vacuum in defining the many aspects responsible for the safe design and execution of subsurface support systems.

This engineering task is made more difficult by the deep excavation in built-up metropolitan regions surrounded by high-rise buildings with deep basements. The greater depth and densely built-up environment make it particularly tough. (Josifovski. J., 2011)

Every day, the world's population grows. Underground constructions are rapidly becoming more necessary as excavations deepen to supply the basic necessities of this growing population, including as transportation, shelter, and social activities. However, increasing the depth of the excavation poses some risks to the excavation as well as the neighbouring buildings and utilities.

The distributed prop load approach is quick and simple to apply, and it provides conservative prop load estimates for braced temporary excavations in a variety of soil conditions encountered in the UK. These loads will be appropriate for the design of the temporary propping structure in most cases. The method will provide early estimations of the loads for increasingly sophisticated retaining structures. Numerical methods of analysis are frequently used by designers of big or sophisticated retaining structures. They can compare the prop loads produced from these calculations to field experience by doing check calculations using the distributed prop load method or using the report's case histories.

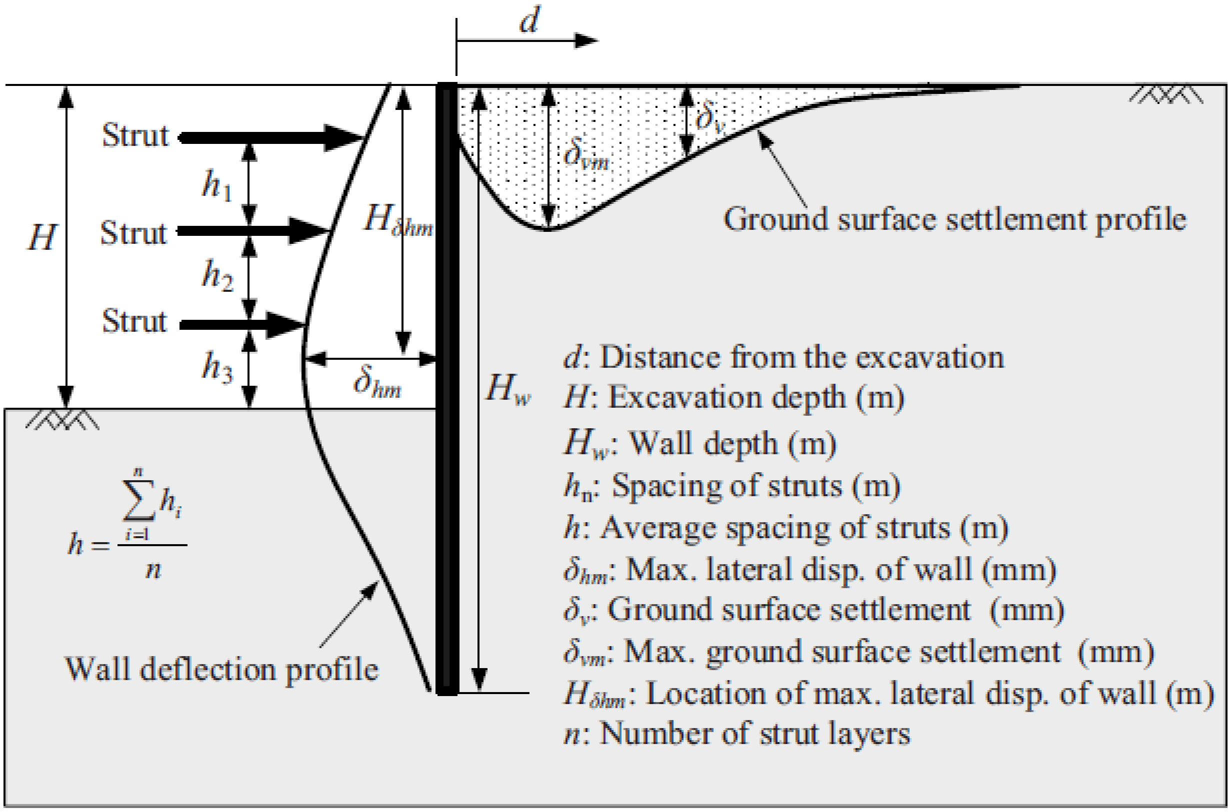
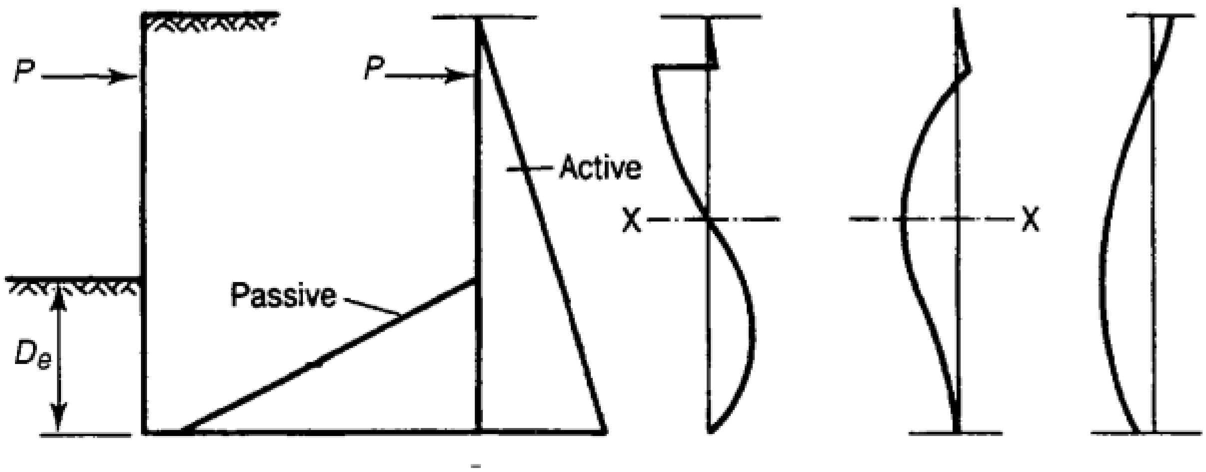


Figure 1.1 : Deflection profile of wall (Wang et al., 2010)

The accuracy of the finite element model is thought to be dependent on the formation of constitutive models and the selection of soil properties. The development of constitutive models is usually very hard since the stress strain behaviour of soil is highly nonlinear, anisotropic, porewater pressure dependent, and plastic. Furthermore, choosing soil parameters (such as strength and stiffness) is an important stage in the finite element analysis. Due to the difficulties of acquiring undisturbed in situ samples, a major concern in the study of deep excavations is that soil test data is often limited or of low quality.

More advanced methods of analysis have been developed and are generally available to practising engineers with the introduction of powerful desktop computers. The term "deformation methods" refers to all of these techniques.



(i) Cross-section (ii) pressure diagram (iii) Shear (iv) Bending moment (v) deflection

Figure 1.2: properties of wall

1.2 NEED OF THE PRESENT STUDY

The demand for buildings with wide basements and the construction of modern urban infrastructure is driving up the number of deep excavations for highway underpasses and rapid transit systems. Many of these constructions are in densely populated regions, near to existing structures. Negotiations with adjacent land owners and occupiers are often drawn out. A clear and authoritative statement on the realistic limit to which ground movements can be regulated is needed to inform and help these talks.

There are a variety of computation approaches that result in considerably varying prop loads. At several phases of the calculation, safety factors are used. It is a need to compare these methods and draw a difference in them to help the designer to evaluate the right design method for a particular case according to the soil conditions and other factors invole.

1.3 OBJECTIVE OF THE STUDY

- The objective of this study is to design a deep excavation with the Empirical Method which is based on Peck and Terzaghi given design method.
- The design values got from the empirical method is than analyzed with the Finite element method using Plaxis 2D.
- The results from the both the method are compared and difference are noted and the possible cause of difference are discussed.

CHAPTER 2 – REVIEW OF LITERATURE

2.1 DEEP EXCAVATION

Deep excavations, particularly in metropolitan locations, can result in ground displacements that are excessive, which can impact nearby building and soils condition. As a result, it is critical to anticipate these movements in order to reduce the impact of ground movements. Estimating these movements of soil is difficult because various factors influence deep excavation movements, including the type of retaining wall, soil conditions, dewatering or pore water pressue, building sequences, climatic conditions, creep effect, and so on. (Wang et al. 2010).

When analysing the performance of deep excavations, it is difficult to take all of these elements into account. As a result, various studies generated forecasts for stiff and soft clays based on site observations in order to determine the order of magnitude of the surrounding soil displacement.

Large lateral wall deformation and soil settlements are prevalent in excavations. According to (Ou et al. 1993), for soft clay excavations, the maximum lateral wall deflection can reach 0.005 times the depth of excavation, while the maximum soil settlement can reach upto 1.0 times. Deep excavations are frequently carried out near to existing structures or public facilities in cities because of the high density of buildings. Large ground settlements frequently cause damage to nearby structures, posing a serious public safety risk. As a result, supplemental measures like improvement of soil properties are frequently needed to protect surrounding structures during deep excavation. Two often used auxiliary methods in engineering practise include strengthening the bracing system and/or enhancing passive resistance, and case histories of these measures with varied degrees of success have been reported.

The placement of cross walls in excavations has been shown in several case studies to effectively reduce wall deflection and ground settlement. However, the impacts of numerous parameters on the wall deflection, including the number of cross walls, the distance to the cross wall, the cross-wall interval, the cross-wall height, and the cross wall embedment, were investigated (Chang O at el., 2012).
The influence of different cross wall installation patterns on the lateral wall deflection in excavations was investigated using a series of 3D numerical calculations, which included the cross-wall interval, height, and embedded depth as factors. The effect of nearby cross wall intervals on lateral wall deflection in the

center cross wall bay was investigated as well. The results can be used as a first approximation for designing a reasonable cross wall interval and a cost-effective cross wall depth to enable a reduction in lateral wall deflection equivalent to that offered by a complete cross wall depth. The following are some more inferences that can be drawn:

- The cross wall generated a corner effect similar to that of a diaphragm wall corner, with substantially less lateral wall deflections near the corner than at other parts of the wall. However, due to the elastic compression of the cross walls, a tiny degree of lateral wall deflection was recorded at the cross-wall section, and the removal of cross walls during excavation in close proximity to the diaphragm wall corner generated negligible lateral wall deflection. The PSR (Plane strain ratio) at the cross-wall section is approximately equal to the difference in plane strain ratio between the cross wall and diaphragm wall corners.
- The lateral wall deflection in a cross-wall bay produced by two cross walls was controlled only by the cross walls and cross wall interval in this bay, with the effect of nearby cross wall bays being low; consequently, the effect of various diaphragm walls or cross wall intervals would be negligible.
- The best reduction in lateral wall deflection can be achieved by installing a full cross wall depth, that is, from the ground level to the bottom of the wall. However, using the crucial cross wall height and embedment can produce a lateral wall deflection that is very near to that of the full cross wall depth while being far more cost effective.

With deep vertical excavation the problem is the settlement of nearby soil to the exaction can damage nearby building and other structure. In the soft soil like clay a the deep excavation can be done to a limited deapth because base failure can happen in soft soil like clay.

For the estimation of apparent soil pressure in the soil near to the retaining wall, Peck and Terzaghi have provided soil pressure graphs. For decades, the Peck approach has been used to assess apparent soil pressure and plan deep excavations.

Peck and Terzaghi (1967) suggested using apparent pressure diagrams to calculate strut loads in braced cuts.

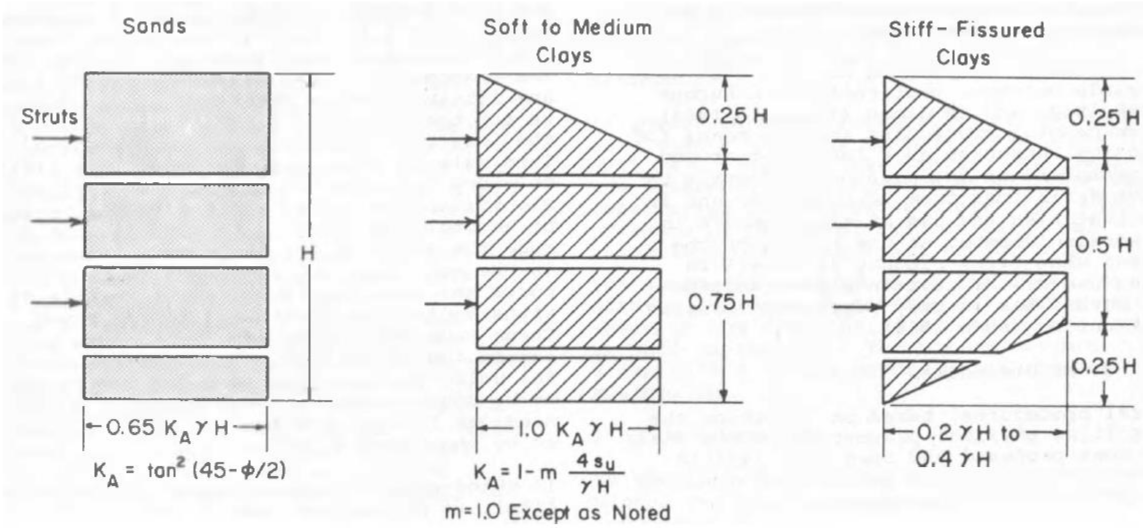


Figure 2.1: Pressure diagram given by Peck and Terzaghi (1967)

The measured and modeled lateral motions and strut forces in deep cement mixing walls under construction site of excavation utilising top-down construction methods in soft Bangkok clay are analysed. Fixed concrete slabs and short term struts support the walls laterally. A three-dimensional object, the numerical model is calibrated first using data out of a case study. (Jamsawang et al., 2017)

- For wall horizontal movement and strut forces, the numerical results generated from the three - dimensional model were compatible with the field measurements. For the computed lateral displacement and strut loads, comparisons of measured and computed data revealed 20 percent maximum and 12 percent average inaccuracies, respectively.
- The concrete slab utilised in the top-down construction had a profound influence on the lateral displacement of the wall, according to the numerical data.

Using the numerical tool FLAC, the best values of the design parameters (strut location, wall embedment depth, wall thickness, and strut stiffness) for strutted excavation in soft clay were identified. The design parameters are derived by examining their impacts on characteristics such as brace force, wall deformation, wall moment, and ground surface displacement near to the braced excavation, all of which are important in the braced excavation design.

It can be deduced that when ratio of excavation depth to embedment depth is between 0.8 and 1.0, the best values of strut force, wall moment, wall defection, and ground surface deformation are obtained. The experiment with varying wall thickness and strut stiffness demonstrated that when the ratio of wall thickness to depth of excavation is between 0.06 and 0.07, both wall deformation and ground displacement reach their lowest point, after which they increase again(Chowdhury at al. 2016).

Table number 2.1: Recommendation for strut depth

Strut no.	Depth recommended
First	2-3
Second	6-7
Third	10-12
Fourth	16-17

2.2 FINITE ELEMENT ANALYSIS OF DEEP EXCAVATION PROBLEMS

For the analysis of deep excavation problems, the finite element method is a strong tool. Because soil is a highly nonlinear, plastic, and porewater pressure dependent material, logical construction of constitutive models and flow rules is required to realistically forecast soil deformation. In the effective stress drained and undrained analyses, the generally used constitutive soil models suitable for the analysis of deep excavation problems, where the input parameters can be determined from the basic soil properties.

The efficiency of the finite element analysis is thought to be dependent on the construction of constitutive models and the selection of soil parameters. The development of constitutive models is usually very hard since the stress strain behaviour of soil is highly nonlinear, anisotropic, pore - water pressure dependent, and plastic. Furthermore, choosing soil parameters (such as strength and stiffness) is an important stage in the fem analysis. Due to the difficulties of acquiring undisturbed in situ samples, a major concern in the study of underground structures is that soil test data is often limited or of low quality.

Using a hybrid of the hyperbolic and Modified Cam-clay models, finite-element analysis was used to deep excavation in multilayer sandy and clayey soil layers. The hyperbolic and Modified Cam-clay models were used to represent the drained behaviour of dry sand and the un - drained behaviour of cohesive soil, respectively. In each of the models, a logical approach for obtaining soil properties was devised. It was designed to simulate the drainage process during excavation. An examination of 3 real excavations cases confirmed the analytical approach. Finally, during each construction phase, assessments of porewater pressure dispersion during the actual time elapsed were conducted.

For excavations with a prolonged construction duration, pore-water pressure dissipation analysis can provide more accurate forecasts than completely undrained analysis. However, in the case of a short construction period and no drained material in the soft clay layer, the excavation behaviour is similar to that of a completely undrained situation.

According to parametric analyses, the pore-water pressure on the passive face of the retaining wall drops dramatically following excavation and then progressively recovers over time. The water pressure on the active face of the retaining wall, on the other hand, does not change significantly during excavation. Following the conclusion of excavation, pore-water dissipation can reduce final wall distortion and ground-surface settlement. The creep effect of soft soil on excavation wasn't even taken into consideration in the investigation.(Chang at al., 1993)

A long-term observation and risk evaluation model for the excavation of a metro station has been designed and implemented. The database management system, the real time construction projects feedback mechanism, and the displacement forecasting system are all part of a software platform for analysing and processing surveillance data based on the notion of dynamic construction inversion analysis. The results of many kinds of field monitoring while deep excavation are provided. Following are the conclusions reached after reviewing the field measurement results:

- With increasing excavation depth, the diaphragm wall deformation and ground surface settlements increased.
- During excavation, the position of the largest horizontal displacement went downward to the excavation face.
- Axial forces passed in first layer to second layer and others during the excavation.
- The propped excavation remains overall stable during the building stage, according to the monitoring results.(Ran at al., 2011)

The Apparent Pressure Diagram (APD) approach created by Peck (1969) is often used to determine the forces operating on the struts in multi-propped retaining walls. Unfortunately, most earlier research focused on braced excavation performance in soft and stiff clays. where supported excavations in sand having high compactness and gravel with moderate stiff walls are limited publications In this case, The strut forces for these kind of excavations were investigated using 2D and 3D finite element analyses in soils with granular particles.

The conclusion is that the strut forces computed by 3D modeling are often higher than those calculated by 2D simulations for a given soil type. The usual pattern for strut loads is for them to decrease as soil strength increases and increase as wall system stiffness decreases. Empirical graphs for dense sand and gravel deposits have been proposed based on field data from a variety of recorded case histories and numerical results(Zhang at al., 2019).

CHAPTER 3 – MATERIALS AND METHODOLOGY

In this study, unbalanced forces exerted on the wall owing to the removal of soils within the excavation zone cause the stability and movements of the excavation. Many factors determine the amount of imbalanced forces, including soil layer characteristics, groundwater level and pressures, excavation depth, and excavation width. Because the finite element approach can simulate these factors, the findings obtained from it are more accurate than those obtained from basic stability formulas or empirical methods. As a result, the finite element method has been widely used in the analysis of deep excavation problems.

3.1 CORE CUTTER TEST FOR UNIT WEIGHT OF SOIL

For assessing the in-situ compression of cohesive or clay soils used as fill, cylindrical core cutters measuring 130mm long and 100mm in diameter are utilised. The bulk density of soil can indeed be readily measured using the core cutter method, and the dry unit weight of the fill can be computed by calculating the water content of the soil.

3.2 DIRECT SHEAR STRENGTH

The maximum resistance of soil to shearing determines its shear strength, as described below:

$$s = c' + \overline{\sigma} \tan \phi'$$

Where, C = Effective cohesion

Φ = Effective stress

$\overline{\sigma}$ = Angle of resistance

Q = kiA

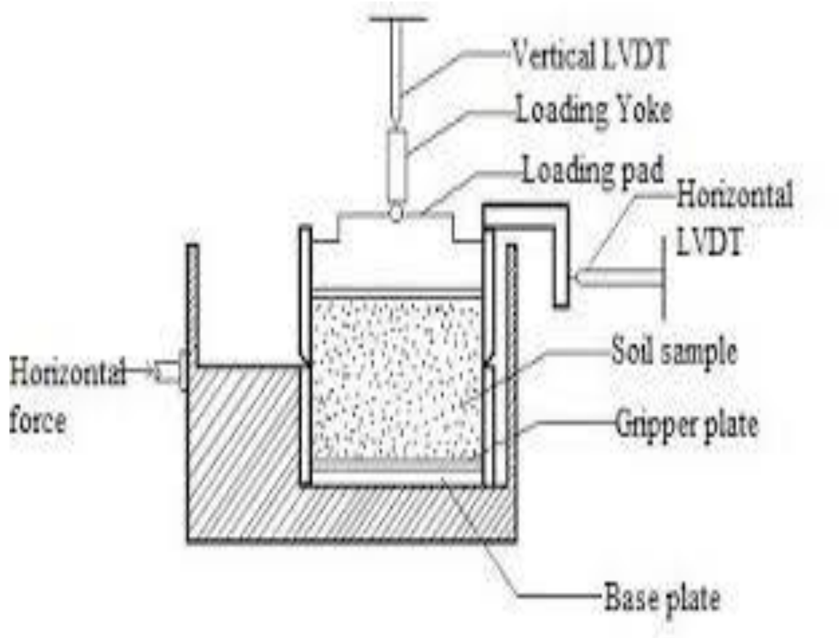


Figure 3.1: Direct shear stress test

3.3 PERMEABILITY OF SOIL

It is the ability of the medium to allow fluid to flow via its interconnecting voids. The permeability of coarse-grained soil is greater than that of fine-grained soil for the following reasons:

Table number 3.1:

Types of soil	**Permeability (K)**
Gravel	> 1
Sand	1 - 10^{-3}
Silt	10^{-3} - 10^{-7}
Clay	< 10^{-7}

According to Darcy's rule, laminar flow in saturated soil has a proportionality to hydraulic gradient, hence permeability can be defined as the rate of flow through the medium per unit area under a given hydraulic gradient.

Q = kiA

Where; Q = Discharge through soil voids
i = Hydraulic gradient
A = Cross sectional area of soil sample under test

3.3.1 Falling head method: The falling head method is used to determine permeability in fine-grained soil. In this procedure, a known-area stand pipe is put into the soil sample to be examined, and flow is permitted through it into the sample. Due to the flow of water via the stand pipe into the sample, the height of the water in the stand pipe decreases over time. The permeability of a given soil sample is determined by measuring the height of the water in the stand pipe at various times. Figure 1 shows the test setup and formula used:

$$K = \frac{2.303\ a\ L\ \log_{10}(\frac{h_1}{h_2})}{At}$$

Where; t = Time taken by the flow to reduce the height from h1 to h2.

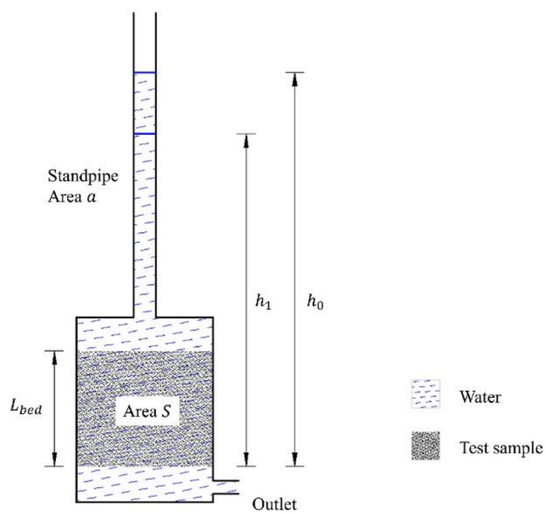


Figure 3.2: Falling head test

3.4 DESIGN PROCEDURE

- Assessing the soil properties above and below the depth of excavation, as well as the hydraulic regime and drainage characteristics of the soil profile. Establishing the right soil class for the ground conditions.

  The case histories have been categorised according to the type of ground that the excavation has taken place:

  Table number 3.2:

Class A	normally and slightly overconsolidated clay soils (soft and firm clays)
Class B	heavily overconsolidated clay soils (stiff and very stiff clays)
Class C	granular/cohesionless soils
Class D	walls retaining both cohesive and cohesionless soils (mixed soils).

  The case histories for firm and soft clays (Class A) and stiff clays (Class B) have been separated into two categories based on the stiffness of the walls. Timber sheet pile and soldier pile/king post walls are examples of flexible (F) walls. Contiguous, secant, and diaphragm concrete walls are stiff (S) walls.

  Flexible walls retaining soft clay soil (Class AF) have been subdivided into "stable" and "improved stability" examples based on base stability requirements.

  Granular soils (Class C) have two types of walls: "dry" and "submerged."

- Checking if the case under consideration falls inside the case history data collection that was utilised to build the distributed prop load approach. assessing if the excavation's base is stable for the final depth of excavation if the underlying conditions are soft clay.

  Is the data set covering the specific site stratigraphy? If the answer is no, do the site-specific soils act similarly to the soils in the data set, that is, do the specific soils behave differently than the generic Class A, B, or C soils? Is it practical to apply DPL guidelines to the site?

- Establishing the excavation's design depth to include a 10% vertical distance below the end prop of 0.5 m maximum provision for over dig.

- The design water pressures are established.

  The design water pressures for the ultimate limit state should be the most unfavourable values that could occur in extreme situations.The design water pressures for the serviceability limit condition should be the most unfavourable that might occur in normal circumstances.

- Whether or not any surcharge loads should be added. Nominal distributed surcharges of up to 10 kPa, such as general construction surcharges and average highway loadings, are included in the DPL diagrams, but other surcharges are not (from buildings, tower cranes, large stockpiles, etc).
- Within the remaining soil profile, determining the unit weight for each soil layer. The value chosen should be a conservative estimate of the value that controls the occurrence of the final limit state. Determining the remaining soil profile's average unit weight.
- Calculating the excavation's typical DPL diagram.
- Checking that the back of the wall beneath the segment of the wall allocated to the lowest prop level has enough embedment to support the earth and water pressures.
- Calculating the typical prop loads, DPL_k, for both excavation completion and prop removal from the characteristic DPL diagram.
- Calculating ultimate limit state(ULS) and serviceability limit state (SLS) prop load values during both excavation and prop removal.

  ULS prop load = (1.35 x DPL_k from permanent actions) +

  (1.5 x DPL_k from variable actions)

  SLS prop load = 1.0 x DPL_k

3.5 Analysis of design with FEM using Plaxis 2D

The steps involed in the ananlysis of the design excavation geometry are stated below

- Axisymmetric model is suitable for this analysis
- Soil material data sets creation and assignment (Mohr-Coulomb model)
- Using the Interface feature to model soil-structure interaction.
- Identifying required displacements
- Creating and allocating plate material data sets
- Developing loads
- Fixed-end-anchor definition and Material data sets for anchors are created and assigned.
- Producing the mesh
- Using the K_0 technique to generate initial stresses
- Excavation simulation
- Seeing the outcomes of the calculations
- Point selection for curves

CHAPTER 4 – RESULTS AND DISCUSSION

4.1 LABORATORY RESULTS

4.1.1 DIRECT SHEAR STRESS

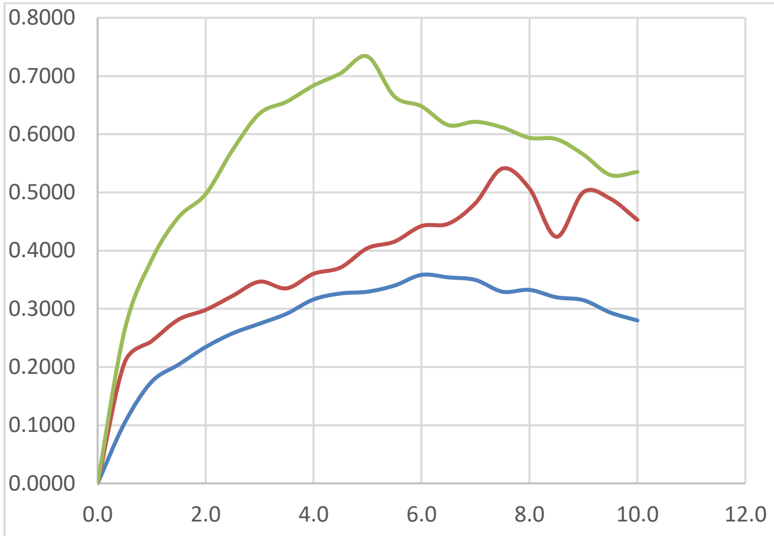
Table number 4.1

Horizontal Displacement		Corrected Area (cm^2) $=A_O(1-\delta/6)$	Normal Stress(kg/cm^2) =0.5		
			Shear Load from Proving Ring		Shear Stress kg/cm^2
Divisions	δ(mm)		Divisions	kg	
0	0.0	36.0	0	0	0.0000
50	0.5	35.7	15	3.705	0.1038
100	1.0	35.4	25	6.175	0.1744
150	1.5	35.1	29	7.163	0.2041
200	2.0	34.8	33	8.151	0.2342
250	2.5	34.5	36	8.892	0.2577
300	3.0	34.2	38	9.386	0.2744
350	3.5	33.9	40	9.88	0.2914
400	4.0	33.6	43	10.621	0.3161
450	4.5	33.3	44	10.868	0.3264
500	5.0	33.0	44	10.868	0.3293
550	5.5	32.7	45	11.115	0.3399
600	6.0	32.4	47	11.609	0.3583
650	6.5	32.1	46	11.362	0.3540
700	7.0	31.8	45	11.115	0.3495
750	7.5	31.5	42	10.374	0.3293
800	8.0	31.2	42	10.374	0.3325
850	8.5	30.9	40	9.88	0.3197
900	9.0	30.6	39	9.633	0.3148
950	9.5	30.3	36	8.892	0.2935
1000	10.0	30.0	34	8.398	0.2799

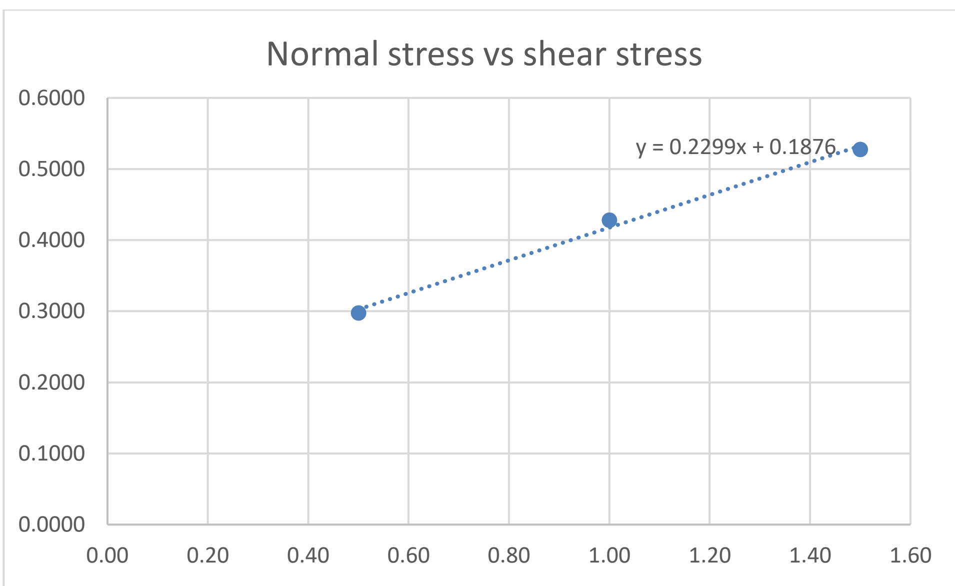
Normal Stress(kg/cm^2) =1.0			Normal Stress(kg/cm^2) =1.5		
Shear Load from Proving Ring		**Shear Stress kg/cm^2**	**Shear Load from Proving Ring**		**Shear Stress kg/cm^2**
Divisions	**kg**		**Divisions**	**kg**	
0	0	0.0000	0	0	0.0000
30	7.41	0.2076	38	9.386	0.2629
35	8.645	0.2442	55	13.585	0.3838
40	9.88	0.2815	65	16.055	0.4574
42	10.374	0.2981	70	17.29	0.4968
45	11.115	0.3222	80	19.76	0.5728
48	11.856	0.3467	88	21.736	0.6356
46	11.362	0.3352	90	22.23	0.6558
49	12.103	0.3602	93	22.971	0.6837
50	12.35	0.3709	95	23.465	0.7047
54	13.338	0.4042	98	24.206	0.7335
55	13.585	0.4154	88	21.736	0.6647
58	14.326	0.4422	85	20.995	0.6480
58	14.326	0.4463	80	19.76	0.6156
62	15.314	0.4816	80	19.76	0.6214
69	17.043	0.5410	78	19.266	0.6116
64	15.808	0.5067	75	18.525	0.5938
53	13.091	0.4237	74	18.278	0.5915
62	15.314	0.5005	70	17.29	0.5650
60	14.82	0.4891	65	16.055	0.5299
55	13.585	0.4528	65	16.055	0.5352

Table 4.2:

Max. Normal Stress (kg/cm^2)	Max. Shear Stress (Kg/cm^2)	
0.50	0.3583	
1.00	0.5410	
1.50	0.7335	
Angle of Internal Friction (˚)		20.5667
Cohession (c) kg/cm^2		0.1691



Graph 4.1: Shear vs horizontal displacement



Graph 4.2: Normal stress vs Shear stress

4.1.2 PERMEABILITY OF SOIL

Mould diameter= 10 cm

Mould diameter (L) = 11 cm

Specimen area (A) = 78.57 cm^2

Specimen volume (V) = AL = 864.28 cm^3

Area of stand-pipe (a) = 0.282 cm^2

Table number 4.4:

Reading in stand pipe	1	2	3
Initial (h_1)	80	65	40
Final (h_2)	70	55	30
Time (t)	59	103	150
Permeability (k)	8.83×10^{-5}	6.38×10^{-5}	4.66×10^{-5}

Average permeability = 6.623×10^{-5} cm/s

4.1.3 UNIT WEIGHT OF SOIL

Weight of empty mould of core cutter =968 gm

Weight of core cutter with soil = 2676 gm

Volume(V) of mould = $\pi/4 \times 10^2 \times 13^2$ = 1021 cc

Weight of soil = 2676 – 968 = 1708 gm

Υ_b = 1708/1021 = 1.672 gm/cc

Moisture content = 14.6 %

4.2 Excavation design

Excavation width = 15 m	Depth 18.5 m
supported by sheet pile walls	there is a 20 kN/m2 UDL on ground level
horizontal prop spacing = 4.2 m	Soil's unit weight is 17 kN/m3.

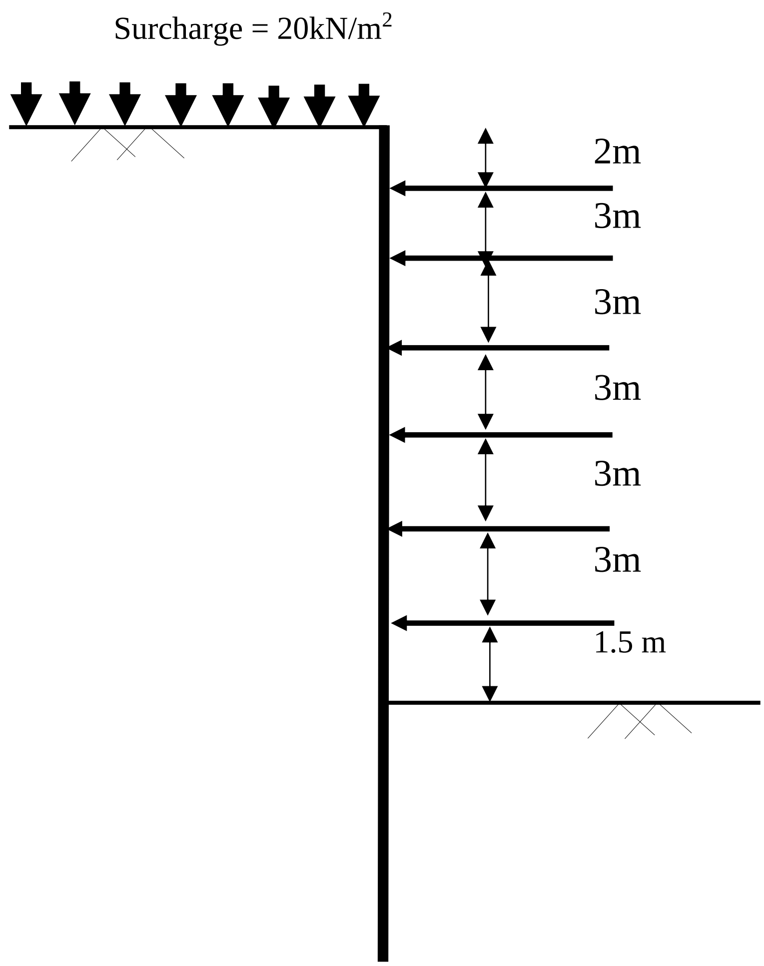


Figure 4.1: Proposed Strut levels in the excavation

Design depth = 18.5 + 1.5x0.1 = 18.7m

The characteristic distributed prop load diagram is being calculated according to the CIRIA recommendation

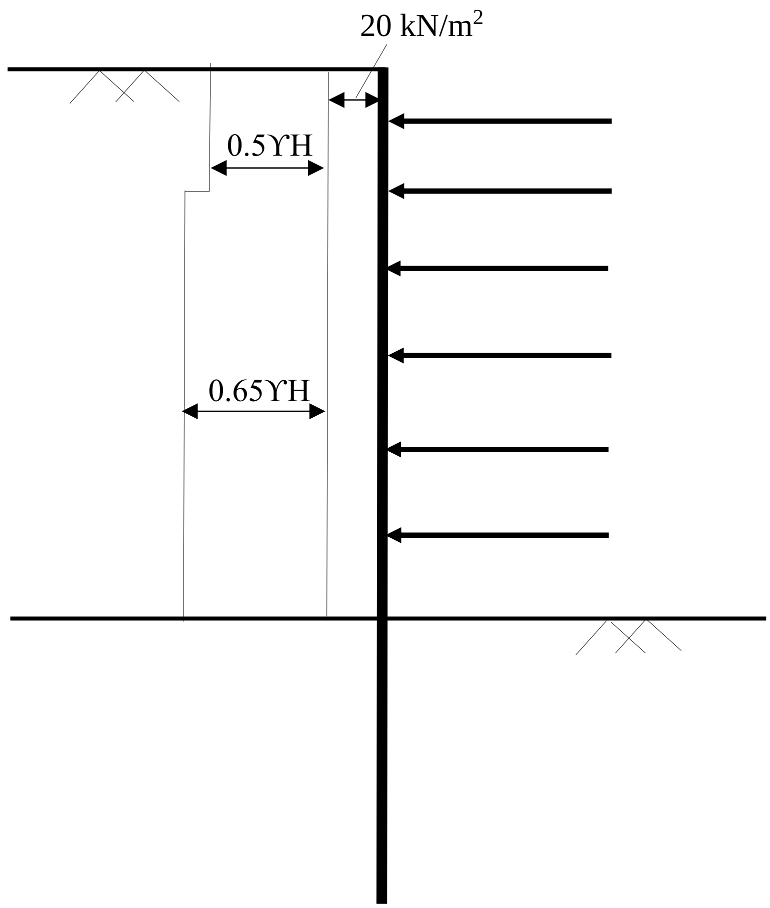


Figure 4.2: Pressure Diagram for the excavation

$0.5ϒH = 158.95\ kN/m^2$

$0.65ϒH = 206.63\ kN/m^2$

Maximum stress in the soil is $206.6 + 20 = 226.63\ kN/m^2$

Evaluating the Characteristic prop load.

Table 4.4:

Prop number	Characteristic prop load (in kN)		Total
	Due to permanent load (in kN)	Due to Variable load (in kN	
1	158.9 x 3.5 x 4.2 = 2336.5	20 x 3.5 x 4.2 = 294	2630
2	158.9 x 0.2 x 4.2 + 206.6 x 2.8 x 4.2 = 2563.1	20 x 3 x 4.2 = 252	2815
3-5	206.6 x 3 x 4.2 = 2603.1	20 x 3 x 4.2 = 252	2855
6	206.6 x x3.25 x 4.2 = 2820	20 x 2.3 x 4.2 = 197	3017

Table 4.5:

SLS prop load in kN	ULS prop load in kN
2630	2336.5 x 1.35 + 294 x 1.5 = 3532
2815	2563.1 x 1.35 + 252 x 1.5 = 3532
2855	2603.1 x 1.35 + 252 x 1.5 = 3532
3017	2820 x 1.35 + 197 x 1.5 = 3532

ANALYSIS OF DESIGN OF EXCAVATION WITH FINITE ELEMENT METHOD

For the analysis of design data of excavation Plaxis 2D is used, it is a well know software tool which can simulate variety of geotechnical model using finite element model.

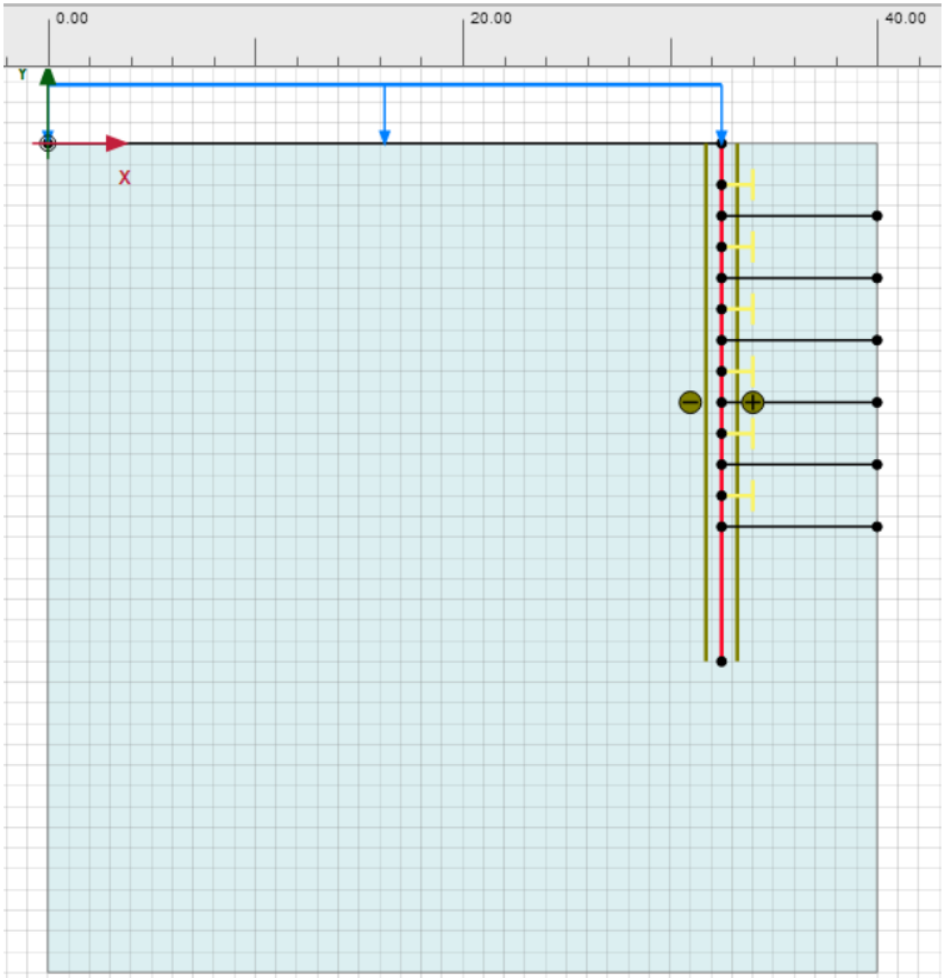


Figure 4.3: Model using FEM

The model with the help of Plaxis 2D is created above with following properties

- Six number of strut
- 20 kN/m^2 Surcharge
- Sheet pile with 26 m depth
- Interface Reduction factor is defined as 0.67

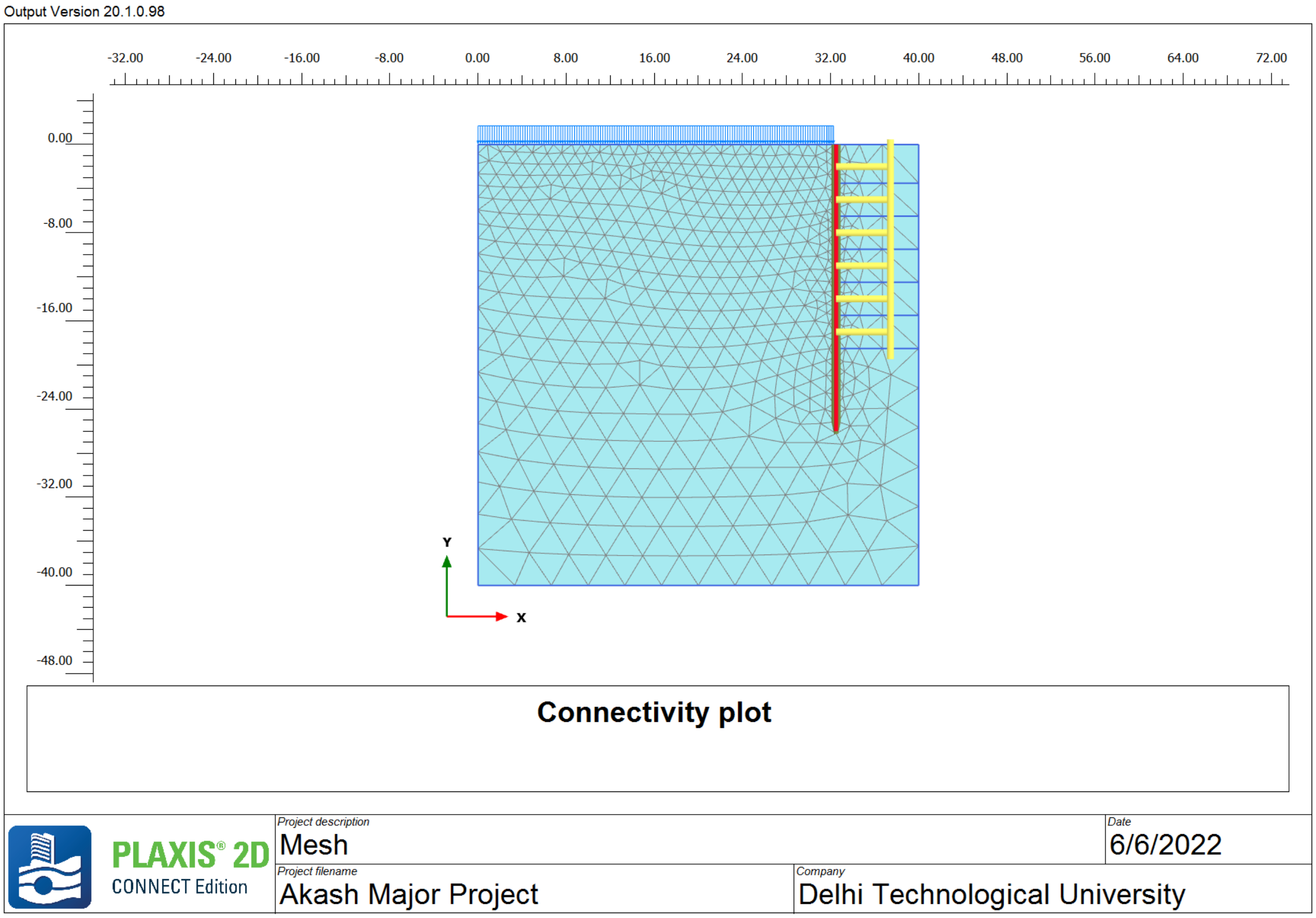


Figure 4.4: Mesh details for the model

- This figure describes the mesh which are formed before the calculation Phase.
- These are triangular elements which are having different size and 15 node element generated by the software.
- The element selection is important in Fem as this can produce more accurate data

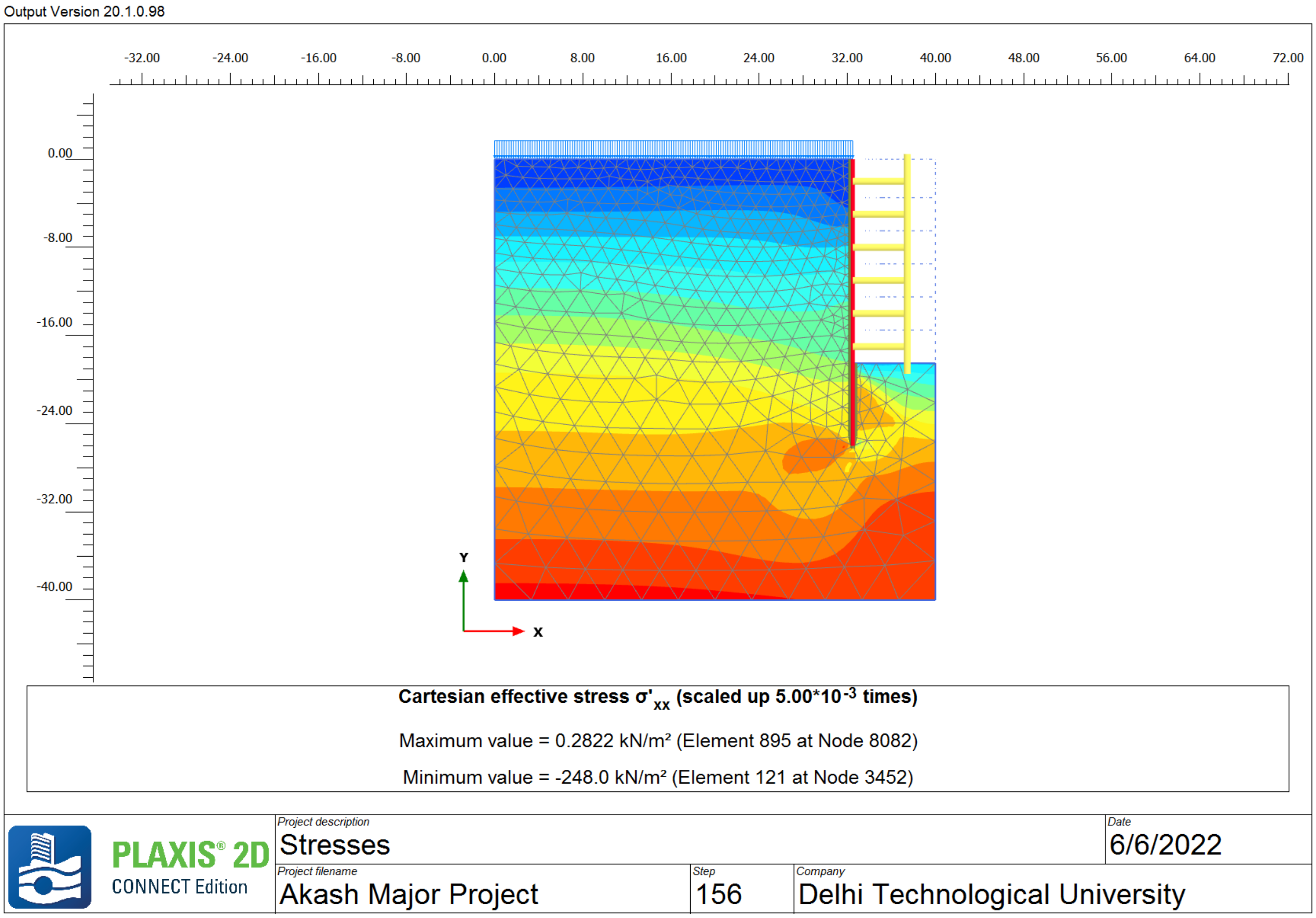


Figure no. 4.5: lateral stresses

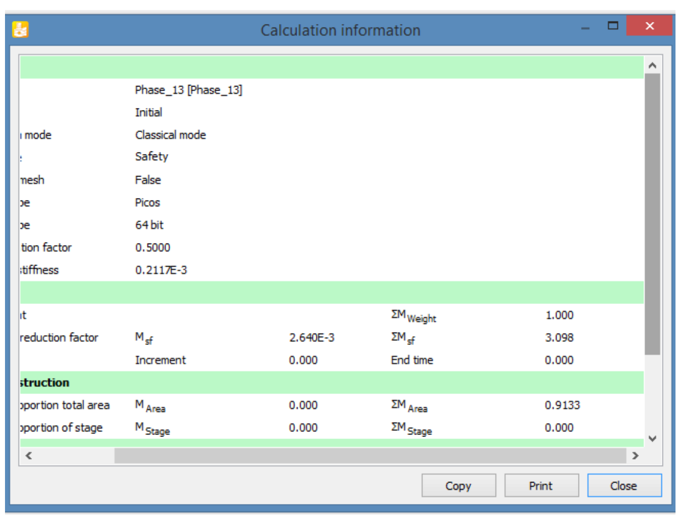


Figure no. 4.5 : FOS

- Factor of safety is coming out to 3.1

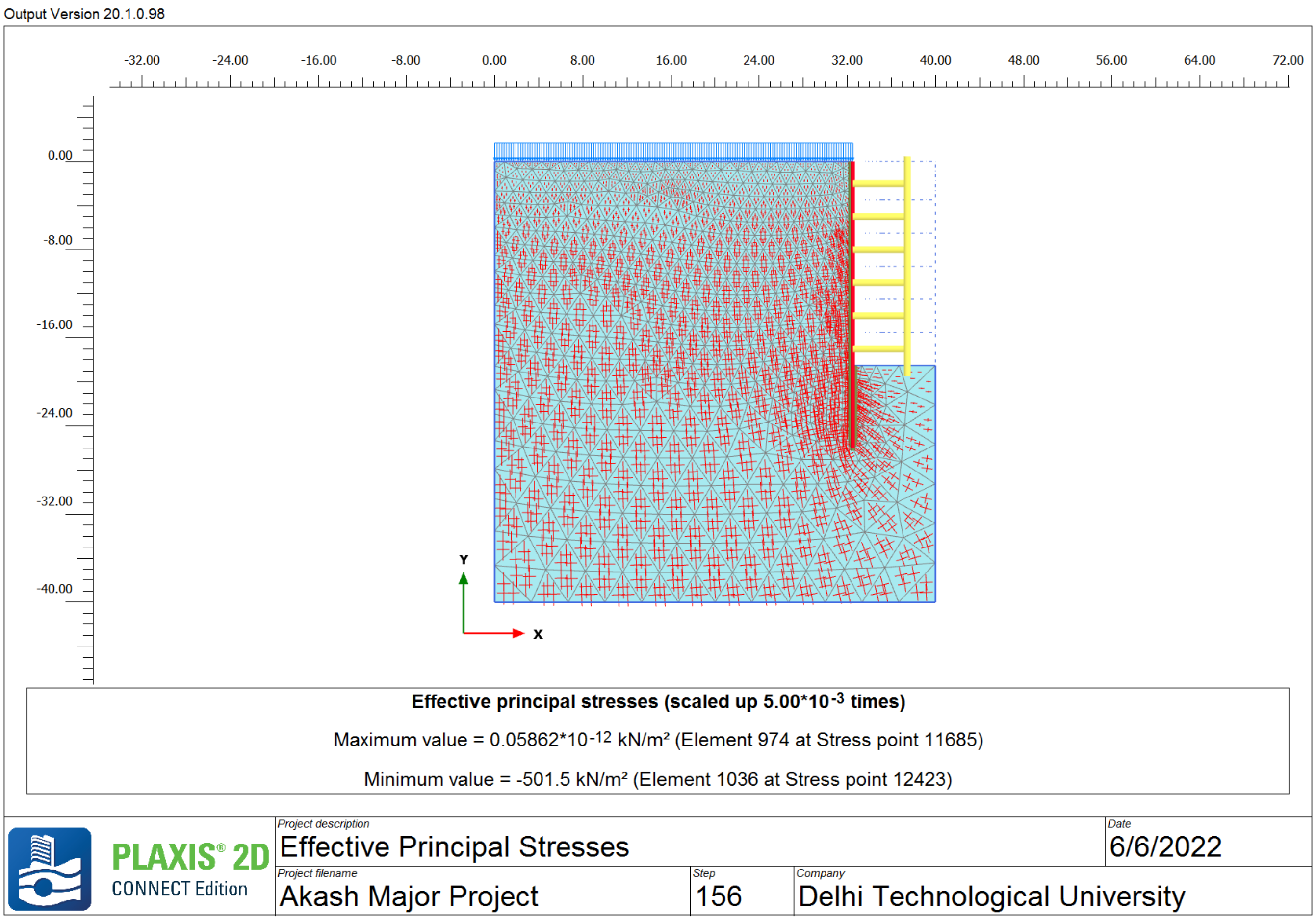


Figure 4.5: Effective stress diagram

- The figure gives the effective principal stresses in the model and maxium value of which is 501.6 kN/m^2.
- The arrow with each element in the diagram indicates the direction of effective principal stress for the element also the length of arrow indicates the magnitute of stress

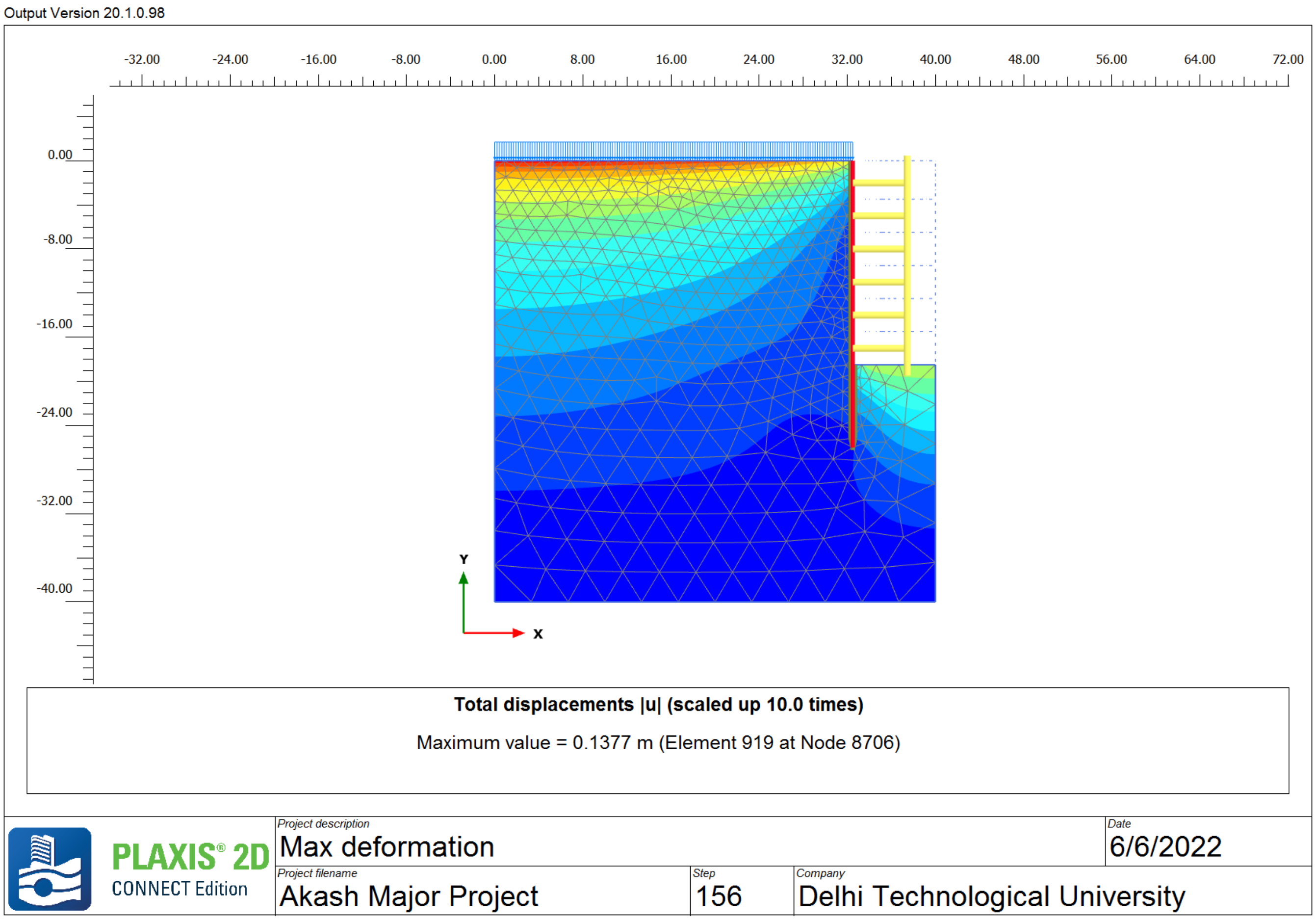


Figure 4.6: Deformation diagram

- Figure explain about the deformation in each element.
- The maximum displacement element is 13.77 cm in the model as calculated.
- The element with same color shades are the element having approximately same value of deformation.

Output Version 20.1.0.98

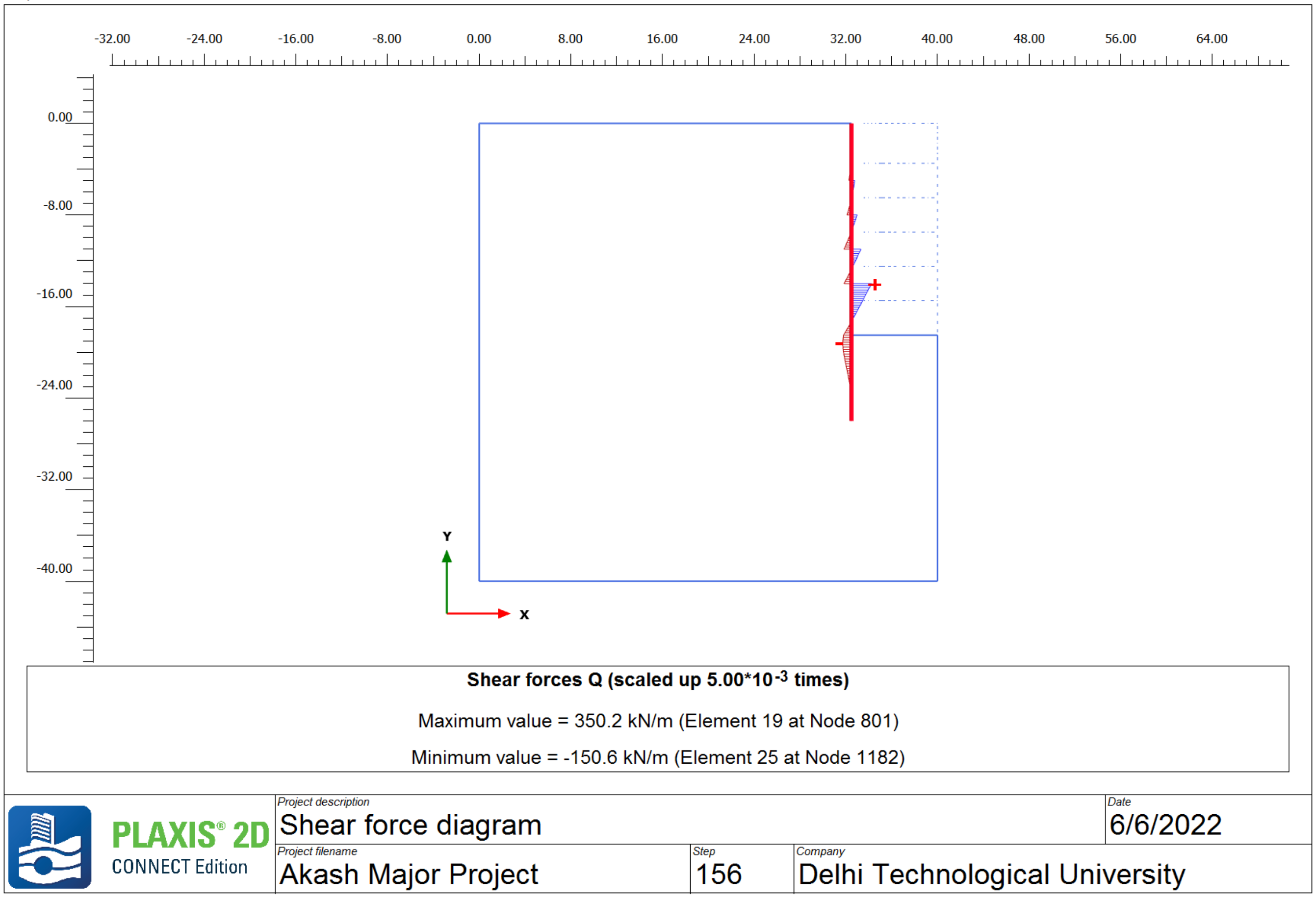


Figure 4.7 : Shear force diagram for the wall

- This diagram of the plate gives the shear force experienced by the plate due to the stresses due to soil and the forces due to the strut.
- In this diagram the location where end strut is located is giving very little load value on the plate which give us the result as this strut is taking a small load.
- The last strut is installed after the deflection in the plate had occurred hence the load is not transferred to the strut.
- The maximum value of shear force is 350.1 kN/m

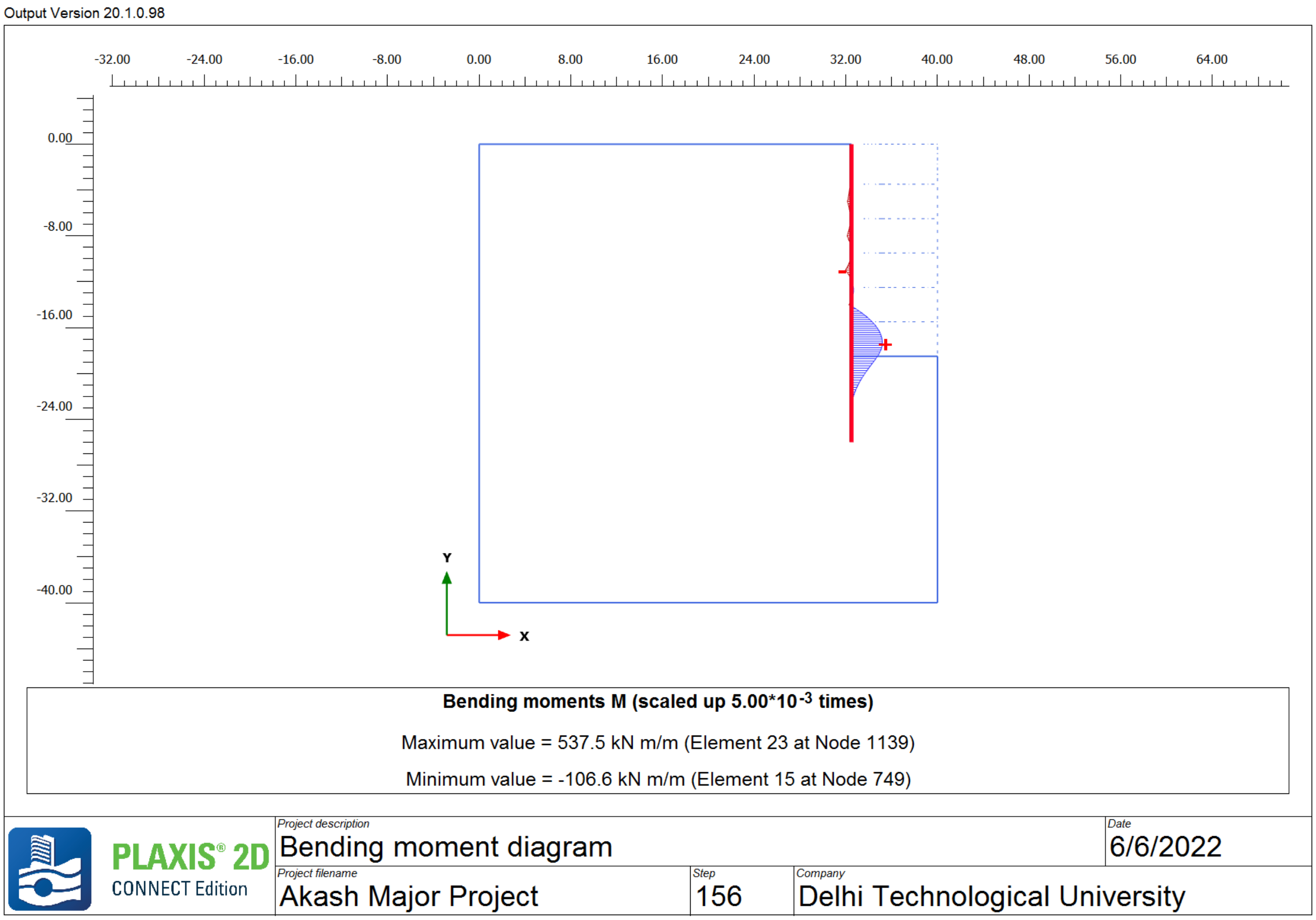
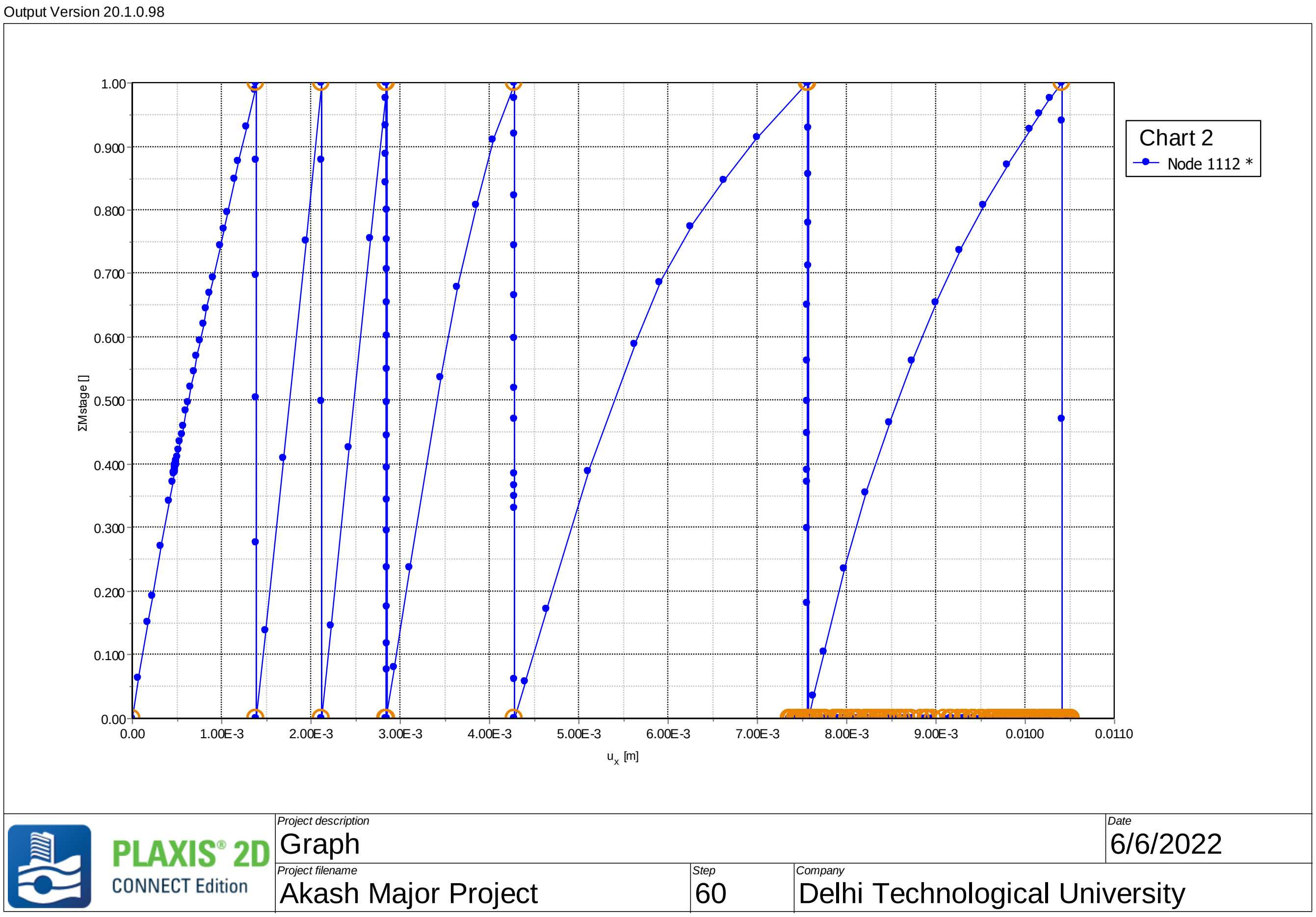


Figure 4.8: Bending moment diagram for the wall

- The bending moment in the plate is maximum in the bottom which gives the result as to decrease the distance between the strut as we move to the higher depth in the excavation considering all the factors.
- The maximum value of bending moment for the analysis is 537.3 kNm/m



Graph 4.3: Phase vs deflection graph

- The Graph is plotted between ΣStages vs deformation or load vs displacement curve for a point.
- The displacement of the point during each phase of construction is represented in the graph here.
- We can choose a point to plot this graph so that we can calculate the deflection for any point.

The forces exerted by plate on each strut is given below

Table 4.6: Strut calculated by Fem

Structural element	Node	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_1_1	633	1	32.500	-2.000	-21.918	-176.347	52.274	0.000	1.000

Structural element	Node	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_2_1	918	1	32.500	-5.000	-423.829	-626.165	0.000	0.000	1.000

Structural element	Node	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_3_1	849	1	32.500	-8.000	-738.526	-995.208	0.000	0.000	1.000

Structural element	Node	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_4_1	749	1	32.500	-11.000	-1230.021	-1426.762	0.000	0.000	1.000

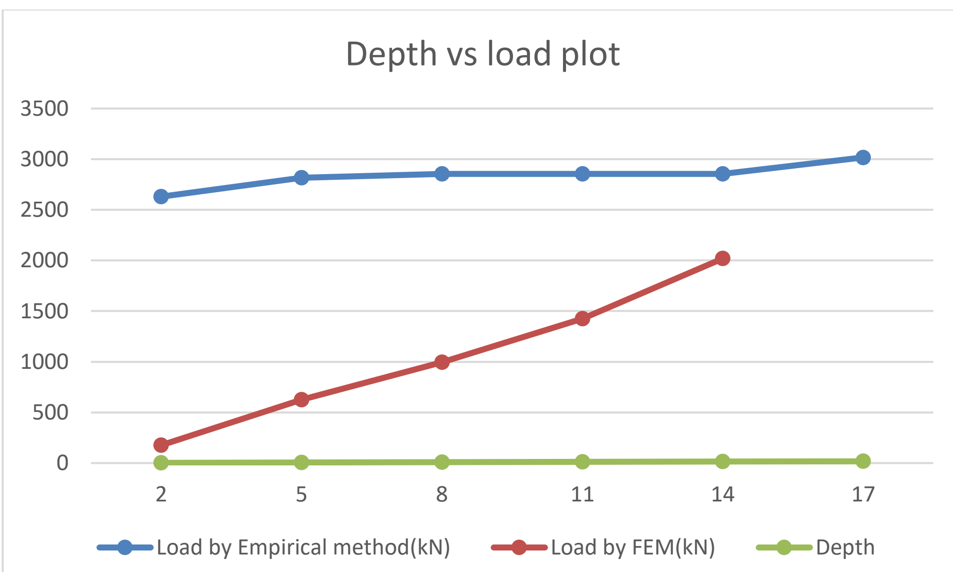
Structural element	Node	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_5_1	801	1	32.500	-14.000	-2019.637	-2019.904	0.000	0.000	1.000

Structural element	Node	Local number	X [m]	Y [m]	N [kN]	N_{min} [kN]	N_{max} [kN]	Φ_z [°]	Length [m]
FixedEndAnchor_6_1	1137	1	32.500	-17.000	-4.832	-4.832	0.000	0.000	1.000

- load in the end strut is 4.82 kN which is very less as compared to the other load gained by the strut is so because the end strut is installed after the deformation in the sheet wall has occurred in the strut and load is not transferred to the strut.

Table 4.7:

Strut no.	Depth (m)	Load by Empirical method(kN)	Load by FEM(kN)
1	2	2630	176
2	5	2815	626
3	8	2855	995
4	11	2855	1426
5	14	2855	2019
6	17	3017	4.8



Graph 4.4:

- Forces in the strut are increasing as we move in to the higher depth in the excavation as we got from the FEM method while from the Empirical method the change is not much have magnitude is also higher.
- The load in the last strut calculated by the Fem is negligible because the every strut is installed after excavation in each phase in the subsequent stage the load is transferred to the strut installed in the previous stage of excavation.

CHAPTER 6 – CONCLUSION

- The Empirical method was developed with test data from excavation in US and UK. The soil condition is different there so the excavation results are analysed by with another method with using FEM with Plaxis 2D is done.
- The stresses in soil calculated with Plaxis 2D is 248 kN/m^2 while soil stresses calculated by Empirical method is 226 kN/m^2 which is 11% less.
- Apparent soil pressure calculated by empirical method is conservative w.r.t soil pressure calculated by Plaxis 2D.
- The forces in strut which are installed near the ground surface are carrying less that is176 kN load w. r. t. strut in bottom strut which is 2019 kN as calculated by Fem while by Empirical method.
- The strut which is installed in the bottom is not carrying load in numerical modelling because the strut was installed after the deflection in plate has occurred and hence no force is transferred to the strut.
- More parameter of the excavation can be estimated from the Plaxis 2D like Shear stress in wall, Bending moment in wall , Stresses and deformations in soil etc.
- In the Shear force diagram, it is visible that the bottom strut is taking very less load.
- Forces in strut are overestimated by Empirical method than the numerical method and the forces in the struts are increasing as we move to the higher depth which is from 176 kN to 2019kN.
- The spacing between the strut is constant in the design but the spacing shall decrease as excavation is proceed further because strut load is increasing as we proceed to deeper depth.

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