SIMULATION OF DEEP EXCAVATIONS USING FINITE ELEMENT METHOD

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ABSTRACT

Finite Element calculations are frequently utilized for the design of deep excavations because a prediction of ground movements and wall deformation is not possible with classical limit equilibrium methods. To solve such Geotechnical boundary value problems, appropriate constitutive laws are necessary for the description of mechanical behaviours of soils.

The possible combinations of these behaviours are divided into two groups; those with a constitutive law based on plasticity and those without. The latter group contains linear elasticity and variable elasticity. A key distinction between the plasticity and the elasticity group is that in latter, strains are recoverable upon decreases in stresses, where as elastoplastic models strains are only partly recoverable.

A crucial point in determining the choice of a suitable soil constitutive model is the ease with, which values can be assigned to the constants defining it. With considering those pointed above, this paper summarizes the application of simplest linear elasticity analysis for convenience for modeling 15m deep base excavation supported with bored pile wall.

Finite element software GEO SLOPE/SIGMAW DEFINE is applied for 2-D plain strain analysis. The mechanical behaviour of the soil is modeled with linear elastic constitutive model using reasonable soil parameters.

Excavation was carried out in 4 steps. Pre-stressed anchors were used as additional supports, just before each excavation step. The FEM simulation was carried out to compare the computed and observed pattern.

INTRODUCTION

One of the main features of a developing country is to develop its infrastructure systems of its main cities. This consists mainly, the development of its transport system and gets the maximum utilization of land allocated for future construction. Especially due to the scarcity of land in highly commercialized areas like Colombo, it is the one of the urgent requirements that to get the maximum usage of the land available. Increasingly clients and architects are demanding larger and deeper basements, which can be used as car parks, shopping, malls etc in the substructure of the buildings. Alternatively for a remarkable improvement in the mass transport system, it is not over optimistic to think of having an underground transport network systems.

Recent studies reveal, the structures that have constructed in the city of Colombo consists of maximum of 2-3 basement levels which are mainly allocated for vehicle parking. Seldom does the location of a basement allow open battered excavation. Particularly in the urban sites, space is limited and insufficient to accommodate cut slopes of battered excavations, basement constructions inevitably occupy as much of the sites as possible.

The reconstruction process to the existing facilities of main cities and the new progressive urbanization strategies reveals that the application of the geotechnical aspects plays a leading role in the infrastructure development process specially in the area of deep excavation.

The selection of the most appropriate type of retaining structure depends on the prevailing site conditions and economic considerations. Restriction in space, necessitate to work with minimum disturbances in urban areas, difficult foundation conditions etc. impose restrictions on the use of gravity retaining walls. More over these could be used as to serve multipurpose functions, which will last several years mostly through the period of serviceability limit of the structure. Diaphragm walls &bored pile type walls have been used extensively in recent times to support underground transport network systems in the form of cut and cover tunnels, basements to high rise buildings etc. They are usually designed to be a part of the final structure.

When earth-retaining structures are used to retain soil close to existing developments, their displacement limiting potential is of critical importance. For example, in most cases of basement and underground transport network constructions, existing buildings have had to rely on retaining structure, to control their displacements. Unfortunately the classical methods of retaining structure analysis are not capable of predicting these deformations. Hence Finite Element calculations are frequently utilized for the design of deep excavations because a prediction of ground movement and wall deformation is not possible with classical limit equilibrium methods as stated above.

The Finite Element method provides a complete solution by satisfying simultaneously the compatibility conditions, equilibrium and constitutive relationships. The existing ground conditions and actual construction procedures can be numerically simulated while using a FEM package.

Finite Element methods have been successfully used to investigate various aspects of soil structure interaction; in the present study therefore Finite Element Software GEOSLOPE/SIGMADEFINE is applied for 2-D plain strain analysis. The mechanical behavior of the soil is modeled with linear elastic constitutive model using reasonable soil parameters. Excavation was carried out in three steps. Pre stressed anchors were used as additional supports, just before each excavation step. The FEM simulation was carried out to compare the computed and observed deformation pattern.

SCOPE OF THE PROJECT

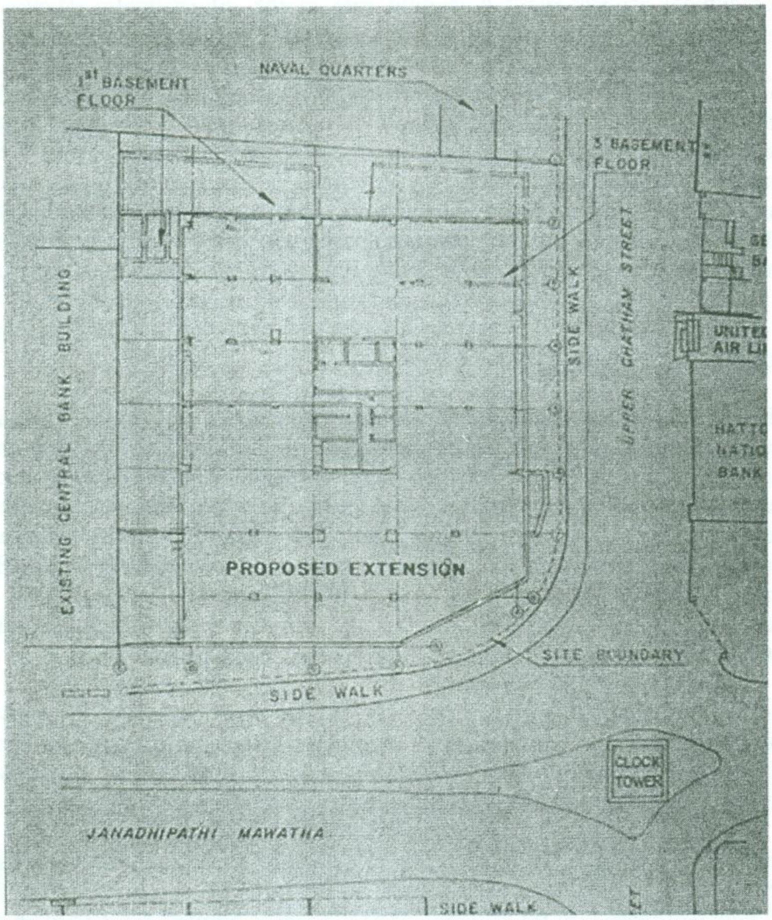

Figure 1 Site Location

The basement done at central bank extension project was selected to carry out the FEM analysis.

The site was occupied an area of approximately 60mx50m in the heart of Colombo city. The substructure of the building consists of a three-storey basement of cast in-situ concrete. The entire structure is supported partly on the bedrock and the balance on reinforced concrete piles bored into the bedrock.

Site investigation revealed that the sub-soil of the site mainly consists of lateritic soils of which the soil type varies widely, ranging from stiff clay to loose clayey sand overlying pre-Cambrian metamorphic bedrock. Bedrock level varies from 8.m to a depth in excess of 25m below the existing ground level.

The construction of basement required the excavation of the site down to a depth of 15m from the existing ground level. Since the excavation extended to the boundaries of the site a vertical cut was inevitable for the excavation pit. This vertical cut had to be retained not only against collapse but also against possible lateral movements, which would cause settlement and endanger the stability of the existing buildings and roads in vicinity of the site.

Further the lowering of ground water table outside the site had to be prevented during dewatering of the excavation pit to avoid settlements of the adjoining structures. For this in practice, anchorage systems are used or walls are braced internally at one or more levels by struts or more levels by struts or floor slabs of the final structure, to limit the wall and ground movement.

In order to resists high loads due to earth and water pressure, up to three to four layers of anchors were installed in the deep excavation area. The first layer of anchors was installed with in the over burden soil where the maximum permissible anchor force was limited to 400KN. For the second layer it was limited to 750KN.The third and fourth anchor layers were extended and anchored into the bedrock to achieve a maximum load carrying capacity of order of 1000KN. Each anchor consists of 3 to 9 strands depending on anchor loads, while the anchor lengths varied from 8m to 30m. Grouted bodies, which transferred the force from anchor to the ground, vary in lengths between 4 and 6m.

The traditional design methods do not provide any means of estimating displacements associated with these circumstances. They are effective only in estimating a depth of embedment necessary for stability by either free or fixed earth support method.

The finite element technique provides the most useful analytical tool in estimating the deformations associated with excavation in front of the embedded walls. Its ability to represents the soil by proper constitutive models and to simulate closely the construction sequence avoids any need for making over simplifying assumptions.

The actual deformation of the bored pile wall had been monitored at 6 locations by installing vertical digital inclinometers. The output had been directly fed into the computer and the results were obtained. In the present study FEM simulation was carried out to compare the observed results.

FINITE ELEMENT ANALYSIS

Finite element analysis consists of two steps. The first step is to model the problem, while the second step is to formulate and solve the associated finite element equations.

Modeling involves designing the mesh, defining the material properties choosing the appropriate constitutive soil model and defining the boundary conditions. It is good practice to first define a simplified version of the problem

and then add complexities in stages. Hence initially it was analyzed linear-elastic model, which include essentially the anchor simulation.
The mesh was prepared along the grid line where an inclinometer results could be obtained for comparison. While preparing the FE mesh it was stick to use quadrilateral elements. The mesh was discertized into fine elements where the critical deformations could take place, i.e. around the bored pile area and with in the excavation limits. Though the best performance could obtain by maintaing the interior angle 90° it was accepted for angle less than 180°. Hence the selected mesh was with in accepted limits (Fig. 2).

BOUNDARY CONDITIONS

Since the depth of the excavation was 15m the span of the FE mesh was selected as 15m away from the bored pile wall on one side and to left it spans 25m to facilitate the anchors in correct position. (Kulathilaka, 1990). On the vertical sides the X displacement was restricted while at the bottom it was done for Y displacement. (Fig. 2) The vertical dimension of the mesh was governed by the depth of the bored pile wall. In other words it was terminated at the bed rock level. The above said boundary conditions were used for initial stress analysis.

Prior to carry out the excavation analysis the material property of bored pile wall was specified and the boundary conditions at the base of the pile was defined as fixed in both X and Y directions (Fig. 3). This was reasonable as the bored pile wall was embedded in the bed rock.

SOFTWARE

The software used to formulate and solve the finite element equations is SIGMAW/DEFINE. It carries out 2-D plain strain analysis. SIGMA/W can be used to perform stress and deformation analyses of earth structures. Its comprehensive formulation makes it possible to analyze both simple and highly complex problems. For example, one can perform a simple linear elastic deformation analysis or a highly sophisticated nonlinear effective stress analysis. SIGMA/W has application in the analysis and design for geotechnical, civil, and mining engineering projects.

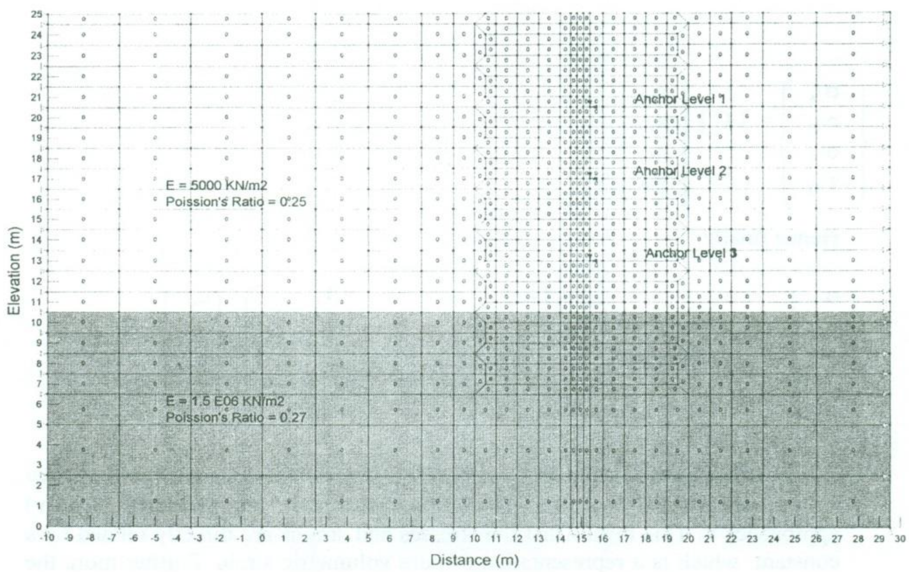

Figure 2 Finite element grid used for simulation

LINEAR ELASTIC MODEL

The analysis is performed based on Total Stress Conditions.

This is the simplest model for which stresses are directly proportional to the strains. The proportionality constants are Young's Modulus, E, and Poisson's ratio ν .The stresses and strains are related by the equations with in the theory of elasticity

$$\{\sigma\} = [C]\{\varepsilon\}$$

Where C is the constitutive matrix and is given by

$$[C] = \frac{E}{(1+\nu)(1-\nu)} \begin{pmatrix} 1-\nu & \nu & \nu & 0 \\ \nu & 1-\nu & \nu & 0 \\ \nu & \nu & 1-\nu & 0 \\ 0 & 0 & 0 & (1-2\nu)/2 \end{pmatrix}$$

SIGMA/W DEFINE compute the stresses and strains at each integration point with in each element once the nodal displacement have been obtained.
Stresses are computed at each Gauss point using the constitutive matrix C in the following manner

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \end{Bmatrix} = [C] \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \end{Bmatrix}$$

Hence finally

$$\begin{Bmatrix} \sigma_x \\ \sigma_y \\ \sigma_z \\ \tau_{xy} \end{Bmatrix} = \frac{E}{1+\nu)(1-\nu)} \begin{pmatrix} 1-\nu & \nu & \nu & 0 \\ \nu & 1-\nu & \nu & 0 \\ \nu & \nu & 1-\nu & 0 \\ 0 & 0 & 0 & (1-2\nu)/2 \end{pmatrix} \begin{Bmatrix} \varepsilon_x \\ \varepsilon_y \\ \varepsilon_z \\ \gamma_{xy} \end{Bmatrix}$$

For two dimensional plain strain analysis ε_z is zero.It is note worthy that when ν approaches 0.5 the term $(1-2\nu)/2$ approaches zero and the term $(1-\nu)$ approaches ν. This means that the stresses and strains are directly related by a constant, which is a representative of pure volumetric strain. Further more the term $E/[(1+\nu)(1-2\nu)]$ tends towards infinity as $(1-2\nu)$ approaches zero. Physically, this means that the volumetric strain tends towards infinity as $(1-2\nu)$ approaches zero. Physically, this means that the volumetric strain tends towards zero as Poission's ratio; ν approaches 0.5.For computation purposes hence ν can never be 0.5. Hence SIGMAW/DEFINE limits ν to 0.49.

The FEM analysis is performed in two stages. Initially in-situ stress analysis/layer construction is performed to determine the initial stresses and then followed by load deformation analysis. That is while performing the load deformation analysis the resultant stresses from in-situ stress analysis was coupled.

The soil strata were divided into two groups as normally consolidated and weathered rock. The modulus of elasticity was defined as 5000KN/m^2 and 1.5E06 KN/m^2 respectively while the Poisson's ratio as 0.25 and 0.27 .The bulk density used is 18 KN/m^3 & 21 KN/m^3. For the in-situ analysis the properties of concrete pile was not taken into account. Instead the relevant soil parameters were given.

The pre stressing effects on the anchors were simulated in the in-situ stress analysis. Here at the location of the anchor head the effective force per meter run were applied. The anchors were done at 1.5m apart.

Anchor Level	Max. Permissible Anchor Force (KN)	Force Applied in X Direction (per 1m run) (KN/m)	Force Applied in Y Direction (per 1m run) (KN/m)
Level 1	400	231	133
Level 2	400	188	188
Level 3	750	353	353

The excavation was done by merely inactivating the elements to be excavated in the actual situation and was performed under load/Deformation analysis. Before performing the excavation in the mesh the anchor properties were given in the mesh. Since the anchorage between the anchor and ground was transferred through a grouted body the simulation was done in those specified areas Fig (3) i.e. through the grouted body.

ANCHOR PROPERTIES

Anchor Level	No. of Strands	Cross Section Area m^2	E Modulus KN/m^2	Anchor Lengths (m)	Grouted Length (m)	Free Anchor Length (m)
Level 1	4	5.6 E-04	19500E03	10	4	6
Level 2	5	5.6E-04	19500E03	15	5	10
Level 3	6	5.6 E-04	19500E03	21	6	15

The anchor stiffness was applied in both x and y direction. The stiffness [K_s] was calculated taking the anchor properties into account.

The spring constant was defined as

$$Ks = \frac{EA}{L}$$

Where
A= Cross sectional area of member
E= Young's modulus of member
L= Free anchor Length

Anchor Level	Stiffness in X Direction (KN/m)	Stiffness in Y Direction (KN/m)
Level 1	6300	3640
Level 2	3800	3800
Level 3	3088	3088

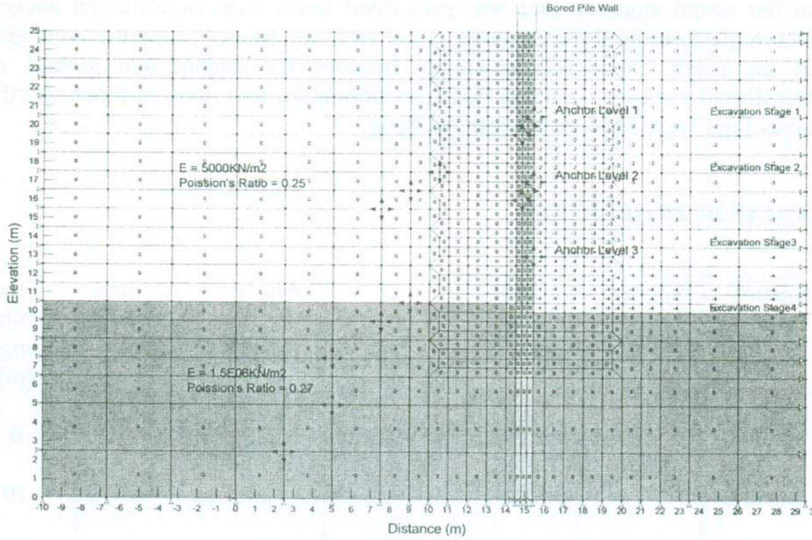

Figure 3 Placement of anchors and performance of stage excavation

RESULTS

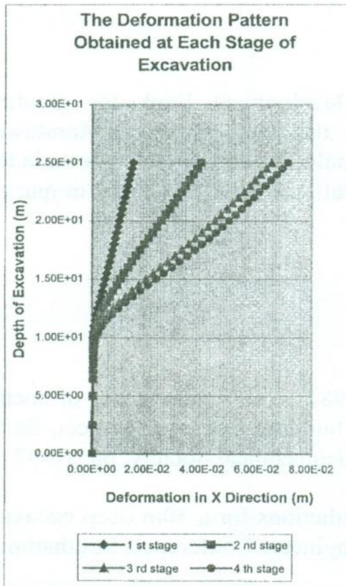

Figure 4 - The deformed pattern without having anchors

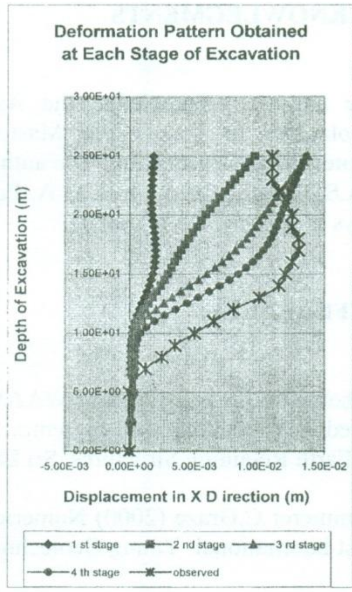

Figure 5 - The deformation pattern with anchor supports & observed pattern

Two types of analysis were performed. First, the excavation was simulated assuming no anchor supports. The inactivation of elements were performed in 4 stages so as to simulate the step excavation performed at the actual situation. The deformation pattern obtained was very much deviated from observed values. For the final stage of excavation the lateral deformation was amounted to around 80mm as shown in Fig. 4 where as it was 12mm in the observed data (in Fig. 5).

The analysis, which was carried out after introducing the anchors matches with the observed deformations qualitatively. The deformation obtained was somewhere round 13.8mm as shown in Fig. 5.

CONCLUSION

Finite Element Method can be used to simulate stage excavation. If a non-linear soil model is used and mesh re-generation is introduced, more realistic deformation pattern could have been obtained in stage excavation.

Once comparing the observed and simulated deformation pattern it can be concluded that they matches qualitatively within a reasonable limits for an acceptable extent

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