Modeling and Analysis of Risk Factors on Soil Anchoring

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Abstract

Given the limited scale of development and urbanization of land in the urban master plan and the necessity of using underground spaces, Excavation in urban areas is important. inevitably limits the engineers of the excavation with vertical or near vertical walls are built of land in urban areas. Accordingly, the stability of supply, particularly with regard to the applied load excavation from adjacent structures, civil engineering challenges facing the guard structure is built. One of the ways that today, especially for urban excavation is common in urban areas deep into the tailoring is back. In this paper the method of sewing the back by a excavation software PLAXIS 2D, It is modeled after the survey results of two-dimensional model, the effect excavation anchorage length, the mechanical parameters of soil (soil internal friction angle, modulus of elasticity of soil) has been investigated. The results of artificial neural network is seeking to determine the success of the guard wall is moving. For this purpose, the artificial neural network application software MATLAB was used. Two-dimensional numerical model of the wall (beam guard) wall with Germany, the German non-tangly inhibition anchor node to node, the tangly inhibition Germany GEOGRID and the soil is simulated by the model soil cured. Complete Excavation involves excavation to a depth of 2 to 3 meters, which is usually considered to be Find the anchorage in the sump pit dug in the structure and stress in the inhibition of force is also modeling.

Key words: Excavation, Soil Anchoring, Finite Element Analysis, PLAXIS 2D

1. Introduction

In recent years, growing land prices and lack of space have necessitated deeper and larger basement excavation in urban areas. Deep excavations are often used in urban areas, for example in underground transport systems, basement and water distribution networks, underground oil tanks, etc (Gwang,2005). A deep excavation in such soft soil can cause large settlements and horizontal movements of ground around the excavation (Wong,1989). Horizontal movements are also a major factor to consider during excavations (O'Rourke,1990). Terzaghi (1943) was the first person that define the excavations, he defined those whose excavation depths were smaller than their widths as shallow excavations while

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those with depths larger than their widths were deep excavations. Terzaghi et al (1943), revised that excavations whose depths were less than 6m could be defined as shallow excavations and those deeper than that as deep excavations, considering that use of sheet piles or soldier piles grows uneconomical once the excavation depth goes beyond 6m (Chana,2006). It is necessary to provide protection to existing buildings adjacent to the construction site, because excavation may damage them. There are different methods for stability of excavations urban areas that included, Anchorage system, Tie-Back system, Diaphragm wall, Reciprocal support, Piling, Truss Construction and combined. The tieback wall as the support system for urban excavations is a commonly used system all over the world. It has many advantages with respect to economy, construction speed, technical and partical considerations. The performance of the system can be monitored and additional precautions, if required, can be taken directly while construction proceeds. Analysis and design of tieback walls is a very wide subject and some assumptions have to made. Many researches have made investigations in that area as a result, different methods of analyses have been developed .In attention to this, the factor of safety determined is anchor uncertainty to be overcome by designers.

2. Methodology

In this article the methodology is based on two dimensional numerical modeling based on Finite Element method has been done and the results from numerical modeling was compared with in situ measurements.

3. Tieback Method

This method is one of the public method for stability in urban excavation. This method is similar to anchorage system. Excavation on it's by step by step and top to down construction .The excavation's steps as follows:

1. Set out the first stage excavation
2. Drilling the vertical or horizontal pits in the wall which's down by drilling special machine.
3. Putting the anchors in to the pits.
4. Inject grouts in the end of the pits; according to the direct contact of the end of anchors (in bonded) with grout and also the unbounded part of anchors for creating prestress force shouldn't be in direct contact with grout.
5. Preload anchors and lock them by special jacks
6. Proceed to the second stage of excavation

4. Structural Elements of a Tieback Wall

A tieback wall is a structural system that uses an anchor in the ground to secure a tendon that applies a force to a wall (Weatherby,1982). Tieback walls are designed to stabilize and support natural and engineered structures and to restrain their movements using tension-resisting elements (Juran,1991). As tension-resisting elements, ground anchors are used in tieback walls.

Ground anchor is a main element of a tieback that functions as load carrying element. An anchor transmits a tensile force from the main structure to the surrounding ground. The shear strength of the surrounding ground is used to resist this tensile force. The tensile force in anchor is that force which is necessary for equilibrium between the anchor, the structure and the ground. As a result of this equilibrium, the movement of the structure and the surrounding ground are kept to acceptable levels (Hanna, 1982).

Ground anchor consist essentially of a steel tendon inserted into ground formations in almost any direction. A ground anchor and its basic components include the head, the unbonded length, and the anchor bond length (Fig .1).

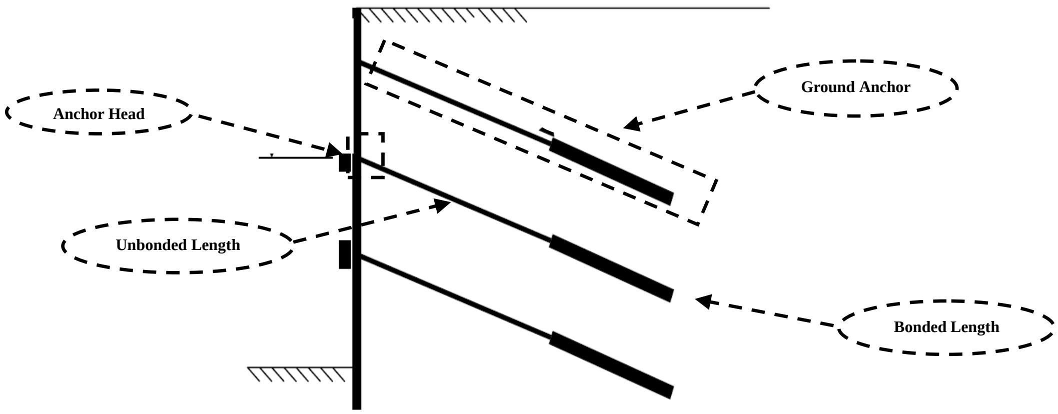


Fig 1: Profile of Tie-Back Method [10]

Ground anchors are classified as permanent and temporary ground anchors as to their service life. Temporary ground anchors are used for a specified construction period and their service life is generally less than two years. Permanent ground anchors, however, maintain the stability of a system on a permanent basis. To provide this long-term performance throughout the service life of the system, permanent ground anchors must be corrosion-protected.

A variety of anchors are developed using different anchor tendons, drilling methods, grout control procedures, and corrosion-protection system. By means of these different construction techniques, different anchorage pull-out capacities can be obtained. Figure (2) illustrates schematically the four types of commonly used ground anchors.

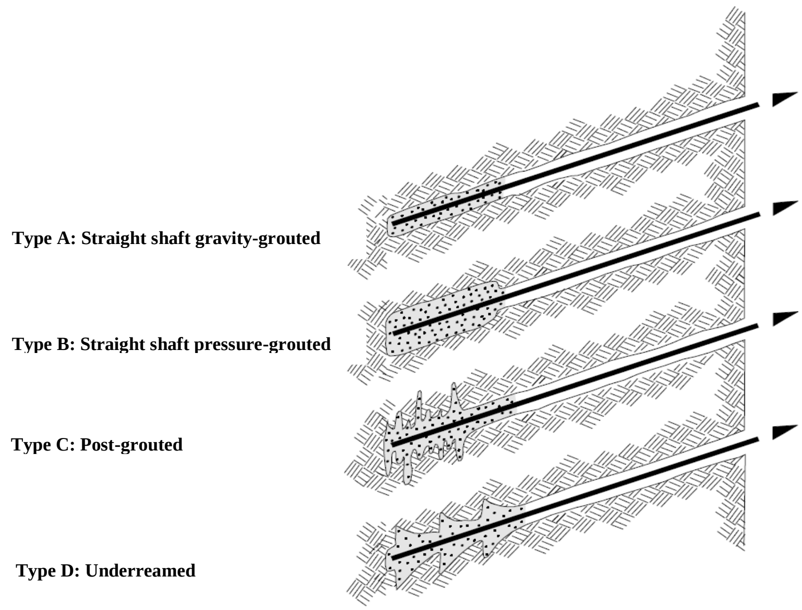


Fig 2: Main Types of Grouted Ground Anchors [10]

4.1. Straight Shaft Gravity-Grouted Ground Anchors

Straight shaft gravity-grouted ground anchors are typically installed in rock and very stiff to hard cohesive soil deposits using either rotary drilling or hollow-stem auger methods. Tremie methods are used to grout the anchor in a straight shaft borehole. Anchor resistance to pullout of the grouted anchor depends on the shear resistance that is mobilized at the ground/grout interface (Sabatini,1999) .

4.2. Straight Shaft Pressure-Grouted Ground Anchors

Straight shaft pressure-grouted ground anchors are most suitable for coarse granular soils and weak fissured rock. This anchor type is also used in fine grained cohesionless soils. With this type of anchor, grout is injected into the bond zone under pressures greater than 0.35MPa. The borehole is typically drilled using a hollow stem auger or using rotary techniques with drill casings. This grouting procedure increases resistance to pullout relative to tremine grouting methods by: (1) increasing the normal stress (i.e., confining pressure) on the grout bulb resulting from compaction of the surrounding material locally around the grout bulb; and (2) increasing the effective diameter of the grout bulb (Sabatini,1999).

4.3. Post-grouted Ground Anchors

Post-grouted anchors use delayed multiple grout injections to enlarge the grout body of straight shafted gravity grouted ground anchors. Each injection is separated by one or two days. Postgrouting is accomplished through a sealed grout tube installed with the tendon. The tube is equipped with check valves in the bond zone. The check valves allow additional grout to be injected under high pressure into the initial grout which has set. The high pressure grout factures the initial grout and wedges it outward into the soil enlarging the grout body (Sabatini, 1999).

4.4. Underreamed Anchors

Underreamed Anchors consist of tremie grouted boreholes that include a series of enlargment bells or underreams. This type of anchor may be used in firm to hard cohesive deposits. In addition to resistance through side shear, as is the principal load transfer mechanism for other anchors, resistance may also be mobilized through end bearing. Care must be taken to form and clean the underreams.

Anchor capacity depends upon the size and shape of the grouted anchor, the tendon type and size, the relative density of the soil, the in situ strength of the soil and rock, the drilling method, the method used to clean the drill hole, and the grouting method. These variables effect the load transfer mechanism between the grouted anchor and the soil or to date, there appears to be no theoretical relationship that can accurately predict tieback capacity. The relationships used to estimate tieback capacity assume that;

1- There is no local debonding at the grout/ground interface,

2- Failure takes place by shear at the grout/ground interface,

3- There is a uniformly distributed bond stress over the whole of the fixed anchor interface,

4- The anchor is formed is only one type of soil.

In reality, however, shear resistance between ground and grout column is more complex than the forgoing idealized model, and this complexity give rise to essentially nonuniform bond distribution. In addition to the anchor, the wall is the other structural element of tieback wall. The wall has to resist the tensile forces transferred by the anchors, the lateral pressure applied by the retained soil and the bending moments. The wall has to be stiff enough to restrain the ground displacement induced by the excavation process.

A tieback wall can be constructed with a wide variety of structural elements, using different installation techniques. Selection of the structural element for a specific application generally depends on the subsurface soil (or rock) type, groundwater conditions, local construction practice, availability of material and equipment, and performance requirements. The structural elements can be evaluated in terms of their stiffness, ease of handling and installation, durability, water-tightness or continuity, and ease of removal. The commonly used elements can be classified into four groups: sheet piles, soldier piles and lagging walls, cylinder walls, and concrete diaphragm or slurry walls.

Sheet-pile walls usually consist of interlocking steel sheets driven into the ground prior to excavation. They are fairly impervious and easy to handle and install in soft clays, cohesionless sites, or loose sands. However, it is not usable when the subsoil contains many boulders or is dense. As compared with other elements, they are relatively flexible and wall displacement is usually larger.

Soldier piles and lagging wall usually consist of steel H-beams that are either driven into the ground or placed in oredrilled boreholes prior the excavation. They can be readily adapted to different site conditions and irregular wall alignments. They are easy to install in most type of soils. There are two methods to tighten the sheeting timbers with the ground. In the first, a vertical face is carefully trimmed to expose the soil behind the front flange of steel pile. The timbers are then inseted and wedged in place. The second method, attaches timbers to the H-pile by special clips proposed by Hanna and Littlejohn, or by bolts and washers proposed by Wosser and Darragh (Wosser,1970). The anchor loads are transmitted to the H-pile through wale beams. The main disadvantage of this wall type is that the wall is rather pervious and subsurface water flow may cause local instabilities.

Cylinder walls consist of an array of cylindrical caissons that are usually constructed or reinforced concrete or mixed-in-place soil-cement and are closely spaced to form a continuous wall. They can be cast-in-place and installed using several techniques such as hallow-stem augers, rotary drilling equipment deep mixing methods, or jet-grouting. To achieve water-tightness and properly retain the soil, shotcrete or lagging in the space between the cylinders may be required. Alternatively, the cylinders can overlap to produce a continuous, impervious wall. In addition to their rigidity, cylinder walls offer the advantage of adaptability to irregular site alignments and can be used in a variety of ground conditions.
Slurry walls or concrete diaphragm walls are generally formed in a trench supported by viscous mud slurry. Concrete is tremied into the trench, displacing the mud slurry upward. Reinforcement of the wall is made by vertical steel sections, precast reinforced concrete members, or cages of reinforcing steel. With diaphragm walls, each panel is supported by anchors and no wale beam system is used. Their main disadvantage is the relatively high cost and the need for specialized construction equipments.

5. Finite Element Modeling

5.1. Site Description

A 7.5-m-high, instrumented, full-scale, tieback, H-beam and wood lagging wall supported by pressure-injected ground anchors was constructed in an alluvial sand deposit to study various aspects of anchored walls at the Texas A&M University National Science Foundation (NSF) designated site for geotechnical experimentation . The schematic plan view, instrument locations and elevation view of the Texas A&M wall are shown in Figure 3.

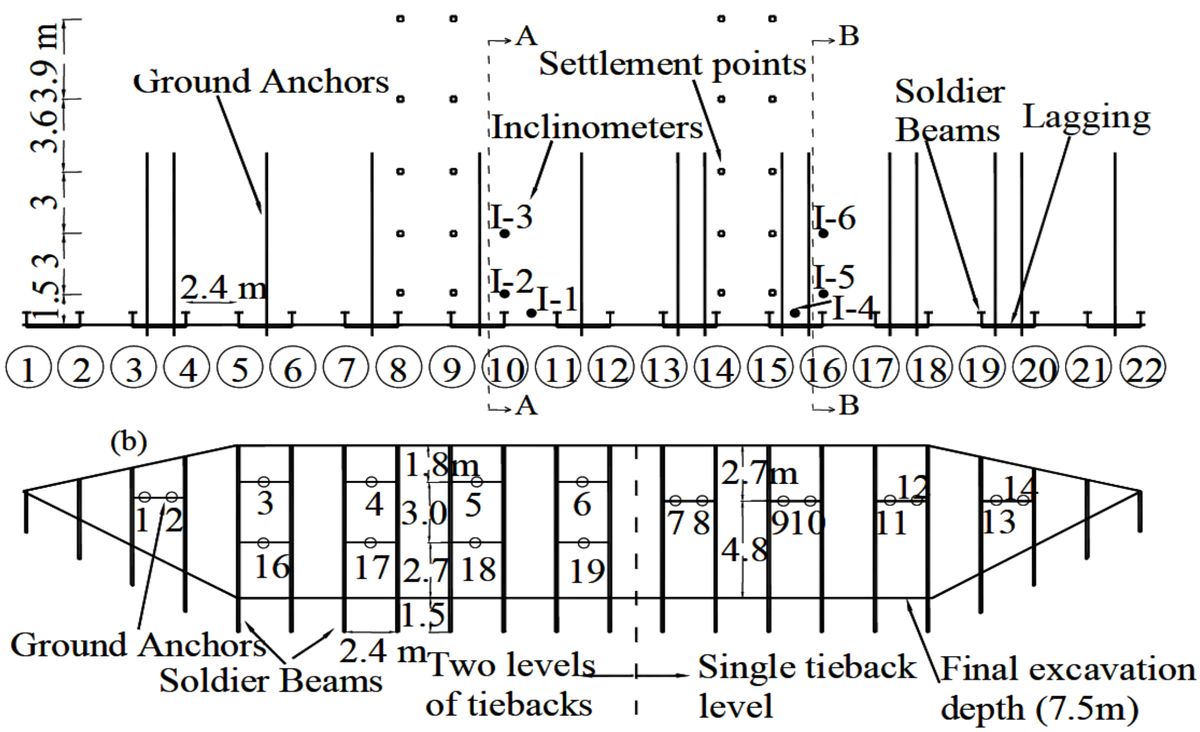


Fig 3: Schematic of Texas A&M University full scale model excavation site showing a) Plan view and instrument locations. b) Elevation view of the wall, inclinometer labels introduction in this study[15]

The excavation supported by Soldier beam and two levels of tiebacks and the friction angle is 30 degrees. The soil and grout body and anchors and soldier beam parameters are given in Table (1) and (2) and (3) and (4).

5.2. Soil Parameters

The wall was constructed in an alluvial sand deposit. A series of in situ and laboratory test were performed to characterize the soil at the site. The in situ tests included: Standard Penetration (SPT) borings, Cone Penetrometer (CPT) soundings, Preboring pressuremeter (PBPMT) boring, a dilatometer (DMT) boring, and a Borehole Shear Test (BHST) hole. Laboratory tests were used to determine moisture contents, the particle size distributions, the Atterberg limits, and the Unifies soil classification for disturbed samples obtained from the SPT borings. Soldier beams and the ground anchors for the two tier tieback wall were designed to support 25H (H is wall height) trapezoidal apparent earth pressure diagram. Material properties of soil in the numerical modeling are listed in Table (1).

Table1: Hardening Soil Parameters Used for Flexible Tieback Wall

Name	Model	γ (KN/m^3)	C_{ref} (KPa)	E_{50}^{ref} (KPa)	E_{oed}^{ref} (KPa)	E_{ur}^{ref} (KPa)	ν (-)	φ (°)	ψ (°)	R_{inter} (-)	m (-)
1st Layer(Silty Sand)	HS	18.065	0	3.5×10^4	3.5×10^4	1×10^5	0.2	32	2	0.9	0.8
2nd Layer(Med Dense Sand)	HS	18.065	0	1.5×10^4	1.5×10^4	4.5×10^4	0.2	32	2	0.9	0.8
3rd Layer(Clayey Sand)	HS	19.636	0.479	1.5×10^4	1.5×10^4	4.5×10^4	0.2	32	0	0.9	0.8
4rd Layer (Hard Clay)	HS	20.421	478.8	4.9×10^4	4.9×10^4	1.4×10^5	0.2	30	0	0.9	0.8

5.3. Soldier Beam Parameters

A two-tier segment of the test wall with driven soldiers beam sections was utilized in the assessment of a “flexible” tiebacks wall system. Soldier beams designated as 7 and 8 were instrumented in the driven test section and these beam sections were used in this article. Figure 3 shows a plan view and Figure 4 shows an elevation view of the wall. To protect the vibrating wire strain gauges from damage during installation of soldier beams, structural angles were welded over the gauges. Soldier beams 7 to 10 had 3× 3×5/16 angles welded to the WF 6×25 beams. The composite WF 6×25 beams and moments of inertia of 5531.71 cm^4. In two dimensional modeling soldier beam was modeled by Plate element. Material properties of soldier beam in the numerical modeling are listed in Table 2.

Table 2: Material Properties for the Soldier Beam

Identification	EA (KN/m)	EI ($KN.m^2/m$)	W (KN/m/m)	ν (-)
Soldier Beam	5.969×10^5	4691.107	1.19	0.2

5.4 Ground Anchors Parameters

Pressure-injected ground anchors were used to support the wall. The ground anchor angle was selected to be 30˚ from the horizontal so the ground anchors would apply a significant downward load on the soldier beams. The anchors were installed by driving a closed-end, 8.89 cm casing into the ground. After the casing reached the desired depth, then the ground anchor tendon was inserted in the casing and the closure point driven off. Cement grout was pumped down the casing as the casing was extracted. The top row of anchors had a 5.48m unbounded length and the bottom row of anchors had a 4.57m unbounded length. A plastic tube was used as a bond breaker over the unbounded length. In 2d modeling the grout body (second part of anchor) modeled by Geogrid element, the unbonded lengh (first part of anchor) modeled by node to node anchor. Material Properties for the Grouted Zone and anchors are listed in Tables 3 and 4.

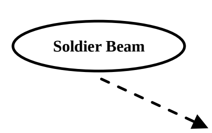
Table 3: Material Properties for the Grouted Zone

Identification	EA (KN/m)	ν (-)
Tieback Grout	3.415×10^4	0

Table 4: Material Properties for the Anchor

Identification	EA (KN)	F_{max},comp (KN)	F_{max},ten (KN)	L_s (m)
Anchor	8.34×10^4	1×10^{15}	1×10^{15}	2.438

Staged constructions was in 8 phases. The numerical modeling in this article has been done by PLAXIS 2D software and the selecting conditions is plane strain. Figure 4 show the two dimensional geometry model.



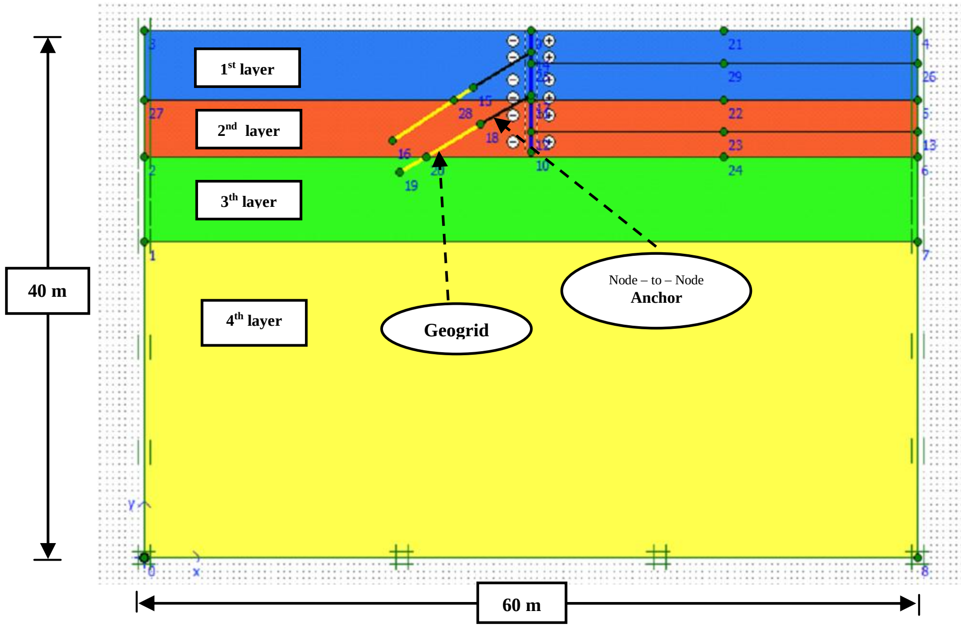
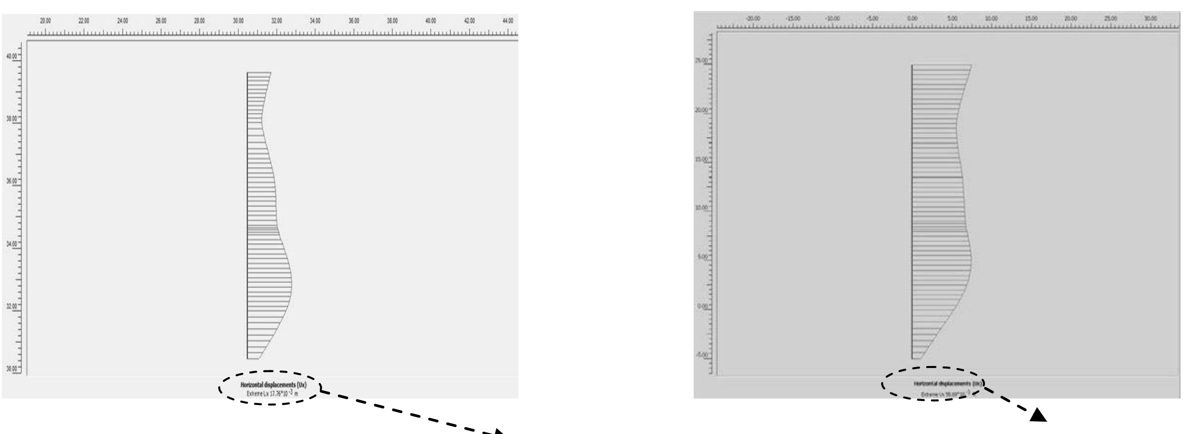


Fig 4: Geometry Two Dimensional Model

Figure (5) show the The maximum values of horizontal displacements from 2D numerical modeling in final stage excavation and comparison between 2d numerical modeling and from refrence (Fig 5).



a) From 2d Numerical Modeling 1.776 cm b) From Refrence [15] 1.778cm

Fig 5: The Maximum Values of Horizontal Displacement

6. Artificial neural networks

Learning algorithms can be classified into local and global algorithms. Global algorithms use knowledge of the current state of the entire network, such as the direction of the overall weight update vector. For instance, in the widely-used back-propagation learning algorithm the gradient descent algorithm is used. In contrast, local adaptation strategies are based on specific information of the weight values, such as the temporal behavior of the partial derivative of the weights. The local approach is better related to the natural neural networks concept of distributed processing, where the computations are performed independently. Moreover, it appears that for many applications local strategies achieve faster and more reliable predictions than global techniques [Riedmiller,1994].

ANN model with the data model 162 PLAXIS trained. Model input data for ANN (E_1, E_2, φ_1, φ_2, L_1, L_2), output guards are removable wall. ANN to predict the movement used is composed of three layers. Previous estimates of the training set and a full investigation to obtain more information and a better approximation of the time the forecasts were reduced. Network training error with respect to changes in input data, the number of hidden layers and number of neurons in the hidden layer is calculated. Network is trained to evaluate the data series of 18 shrimp were not used in the training phase. The following Table 5 Comparison between the output of network and application data is done.

Table5: Comparison Between the Output of Network and Application Data

Model	**Plaxis output** (m)	**Ann output** (m)	**Percent error** (%)
1	0.04129	0.04296	2.69
2	0.04052	0.04039	0.31
3	0.02279	0.02268	0.48
4	0.0308	0.03087	0.24
5	0.02739	0.02703	1.29
6	0.02055	0.02052	0.11
7	0.00762	0.00743	2.38
8	0.02462	0.02644	0.12
9	0.01591	0.01593	0.14
10	0.02843	0.02838	0.16
11	0.02335	0.02307	0.36

12	0.02063	0.02031	1.5
13	0.01158	0.01148	0.82
14	0.03238	0.03250	1.76
15	0.03318	0.0336	1.19
16	0.02692	0.0266	1.56
17	0.0311	0.03108	0.1
18	0.00699	0.00715	2.3

7. Conclusion

One of the most attention in recent years stabilization excavation civil Engineers for the city is located to the rear is a sewing technique. The advantages of this method can be implemented quickly and cost and also because of improved soil mechanical parameters of the injection and the absence of large structures deep inside the guard said.

This paper has explained that Excavation two-dimensional numerical modeling can be a method of sewing the back wall of the factors affecting the transformation the stitching on the back of a sustainable urban Excavation investigate.

This article about a study from Texas A&M University is used. This excavation has a depth of 7.5 m and has two rows of underground containment, July Guardian (Soldier Beam) and the wood lagging is used. Two-dimensional numerical modeling and comparison with results of Plaxis software is a good agreement with the results obtained from the reference.

This study showed that:

- With increasing soil mechanical parameters such as modulus of elasticity of soil and soil internal friction angle of the wall decreases the maximum amount of transformation.
- Reduction of soil mechanical parameters such as modulus of elasticity of soil and soil internal friction angle of the wall increases the maximum amount of transformation.
- With increasing amounts of inhibitory Excavation tangly and transformation up to the wall decreases.
- Reducing the inhibitory Excavation tangly and transformation values of maximum wall increases.
- During the transformation Excavation on the maximum wall length is more tangly.
- Stress reduction in force, with a maximum inhibitory amounts of deformation increases.

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