Journal of Civil & Environmental Engineering
ISSN: 2165-784X
Bakr, J Civil Environ Eng 2019, 9:1
DOI: 10.4172/2165-784X.1000327
Research Article
Open Access

The Impact of the Unsupported Excavation on the Boundary of the Active Zone in Medium, Stiff and Very Stiff Clay

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Abstract

Deep excavation adjacent to existing buildings with shallow foundations and/or old wall bearing buildings represents a big challenge to the geotechnical engineer. The restriction of the lateral movement of the soil underneath those building represents the primary objective for any excavation support system. The previous research resulted in many excavation support systems including; soldier pile walls, sheet pile walls, secant piles, tangent piles, diaphragm walls, etc. These techniques may be cost-effectively for large and important projects but they are not for small projects which represent the majority. The main objective of this research is to study the stability of excavation sides in medium, stiff and very stiff clay soils either with or without a minimum safe lateral horizontal distance to the adjacent building. A parametric study was carried out to determine the minimum horizontal distance, H, for selected excavation depths, de, ground water depth, dw, and surcharge stress (q) underneath the neighboring buildings. The research outcomes showed that a significant saving can be achieved by excavation in very stiff clay excavation to a relatively large depth up to 9 m without retaining system. While for stiff and medium clay soils, a horizontal distance should be left beside the neighbored buildings depending on the magnitude of the surcharge stress, q.

Keywords: Clay; Unsupported excavation; Lateral displacement; Supporting systems

Introduction

Providing space for parking, public amenities, etc., in multi-storey buildings at town centers has created increasing demand on deep excavations. Most civil engineering projects require this type of excavation. For example, basements of buildings in developed areas, underground transportation facilities at relatively shallow depths below ground surfaces using cut-and-cover type of construction, underground parking's, sewage pipelines, and water mains etc.

Deep excavations are supported by systems like conventional retaining walls, sheet pile walls, braced walls, diaphragm walls and pile walls. This article discusses various excavation supporting systems in terms of method of execution, cost, and the appropriate conditions for use. Deep excavation with vertical or near vertical faces can be considered one of the common and complex geotechnical problems.

Many building codes require that all trenches exceeding 4 to 5 feet in depth be shored. For example the Australian and New Zealandian code of practice for excavation requires shoring or stable side slopes for excavation depth more than 1.5 m while Hong Kong's building regulations restrict excavation for depth more than 1.2 m without support system. According to the British Health and Safety Executive, HSE; any unsupported excavation will be safe without support only if its sides are battered back sufficiently.

Support systems are temporary or permanent earth retaining structures that allow the sides of excavation to be cut vertical or near vertical. They are used to minimize the excavation area, to keep the sides of deep excavations stable, and to ensure that ground movements due to excavation will not cause damage to neighboring structures or to utilities in the surrounding ground due to settlement or bearing capacity failure.

Selection of the appropriate excavation method and lateral supporting system depends on, local geotechnical conditions, environmental conditions, the allowable construction period, the available budget, existence of adjacent excavations, area of construction site, conditions of adjacent buildings, foundation type of adjacent buildings, and the available construction equipment's.

Common Excavation Methods

The cantilever open cut method

It uses retaining walls to ensure the soil stability depending on the wall stiffness. This method doesn't require digging slopes and backfilling therefore the cost might not be necessarily higher than the slope open cut method.

The braced cut methods

It uses struts in front the wall to resist the earth pressure on the wall. This system consists of struts, wales end braces, corner braces, and center posts.

The anchored excavation methods

It uses anchors instead of struts in the method of braced cuts. The anchors offer lateral supports against lateral earth pressure. Anchors can be divided into fixed section, free section, and head. The fixed section provide the anchoring resistance. The free section transfers the anchoring force to the anchor head which locks the tendons and transfer the anchoring force to the structure.

The island excavation methods

In this method, the central part of the site is excavated first. The soil of outer part is kept to form slopes near the retaining walls. The central part of structure is built. Slope is then excavated and struts or rackers are installed between retaining walls and central structure. Struts are then dismantled and the outer part of structure is completed.

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Received October 21, 2018; **Accepted** January 18, 2019; **Published** January 25, 2019

Citation: Bakr RM (2019) The Impact of the Unsupported Excavation on the Boundary of the Active Zone in Medium, Stiff and Very Stiff Clay. J Civil Environ Eng 9: 327. doi: 10.4172/2165-784X.1000327

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The top-down construction methods

It erect molds and constructs permanent floor slabs right after excavation. These slabs replace struts in the braced excavation methods. The construction sequence is to construct retaining walls, pile foundations, install steel columns on piles, proceed first stage of excavation, construct slab for first basement, start construction of the superstructure, proceed the second stage of excavation for the second basement floor slab, repeat the same procedure till the designed depth.

The zoned excavation methods

It uses the principle of arching effect of soil which reduce the concrete wall deformation and ground settlement at or around corners. Therefore the deformation of shorter side of excavation is less than that on longer side. The excavation site is divided into smaller sized zones. The odd numbered zones are excavated while the adjacent interconnected even numbered zones one are left to support the wall by reducing deformations at corners. Struts are installed in the excavated zones. The even numbered zones are then excavated. This procedure is repeated till excavation is completed.

Types of Retaining Structures

Retaining structures are installed for all methods of excavations as a part of the supporting system. The common types of retaining structures are soldier piles, sheet piles, column piles, and diaphragm walls. A brief description for each type is explained in the following paragraph.

Soldier piles supporting system

It consists of steel rail pile or H-pile and laggings. The construction procedure of this type starts by installation of soldier piles by striking in non-urban areas or vibrating in urban areas. If hard soil layer exists prehole is excavated. Laggings are placed as excavation proceeds. Voids between soldier piles and laggings are then backfilled. Horizontal struts are installed in the proper places. After completion of excavation the Inner basement walls are constructed.

Sheet piles

These are driven into soil by striking or vibration. Common types of sheet piles are U-section and Z-section. Sheet pile can be efficient in water sealing if they are well interlocked. In granular soils with high permeability leakage soil will flow out causing settlement. The construction procedure of sheet piles starts with installation of sheet piles prior the first stage of excavation. Wales are placed in proper places and horizontal struts are installed. Next stage of excavation is processed. Installation of wales, struts, and excavation are repeated till the design depth. Foundations and basement walls are executed then dismantle the struts level by level and build the floor slab. After completion of the underground part of the structure, the sheet piles are dismantled.

Column piles method

It is to construct rows of concrete piles to serve as retaining walls. Column piles may be precast or cast *in-situ*. The later is divided into three types: packed in place piles, concrete piles, and mixed piles. Packed in place piles have diameter from 30-60 cm. This type is constructed by using helical auger to dig the hole to the designed depth. While lifting the chopping bit, mortar is casted till the hole is filled to the ground surface then steel cages or steel H-pile is placed into the hole. This type is not water tight therefore if it is used in soil with high permeability and high ground water level, sealing and grouting is always required. Concrete piles is constructed by drilling a hole to the design depth by drilling machine, placing steel cages in the hole, and filling the hole with concrete using tremie tube. Mixed piles is constructed by using special chopping bit to drill a hole with the concrete mortar sent out from the fron of the bit to be mixed with soil. When design depth is reached, the bit is lifted but keeping swirling and grouting simultaneously and mortar is mixed with soil thoroughly. Thereafter, steel cages or H-pile is placed into the hole.

Cost of the Supporting Systems

The cost of supporting systems ranges from low cost methods like soldier beam and lagging methods, moderate cost methods like column piles, and high cost methods like sheet piles and 1.3.4 Diaphram walls. Table 1 explains a comparison for the approximate cost of the most used systems of soil retaining structures.

The solder H pile wall with lagging is rarely used in our practice, although it is highly efficient and cost effective for situations where there is no ground water. Also a greater depth can be achieved when combined with adequate supporting system e.g., tieback. Although very formidable the systems with diaphragm wall are seldom used, partially because there is almost no experience nor there has been clear cost-benefit analysis. For a long period of time it has been thought that the costs are very height, which with the present study had proven not to be the case. Combined with the top-down method of construction where the wall is permanent structure according to our analyses remains very cost effective solution. The secant pile wall technique, in contrast, is very often used in our practice, sometimes in combination with anchors when greater depth is needed. It represents formidable solution but usually takes a lot of the available space and construction time, also brings high expenses since it is often a temporary structure.

Form the previous literature it can be noted that for each excavation method there are many possible supporting systems. It can be also noted that the cost of using such supporting systems may be uneconomic for many engineering projects. Therefore, this research focuses on utilizing the cohesive property of medium, stiff, and very stiff clay soils in dispensing with the use of these supporting systems.

Aims of Study

This research aims to study the maximum depth to which deep excavation can be performed without using lateral supporting systems in medium, stiff, and very stiff clayey soils. It aims also to study the effect of surface loads that represent the impact of neighboring facilities and existence of groundwater table on this depth.

Research Methodology

Problem characterization

Slope failure mechanisms can be classified in three categories: rotational slump in homogeneous clay, translational slice in cohesionless sand or gravel, and slip along plane of weakness. Driving forces are the component of soil weight downslope (forces causing instability), and resisting forces are the soil strength acting in the opposite direction (resisting forces). Slope failure occurs when driving forces exceed the resisting forces.

Number	Supporting system	Cost, $/m^2
1	Secnat concrete pile wall	1100
2	Soil nailing wall	1500
3	Soldier beam and lagging	250
4	Sheet pile wall	500
5	Diaphram wall	1150

Table 1: Average cost of some supporting systems.

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Many scenarios were investigated including the excavation directly adjacent to an existing building, providing safe horizontal distance, using stepped excavation, and side slopes instead of providing structural support systems. Figure 1 presents a model for an existing building represented by a uniformly distributed load (q) acts at the foundation level. Varying ground water level was also considered in the calculations. The ground water depth varies from 0.0 m at the ground surface to 9.0 m which represents excavation for approximately three basement floors.

The model also consider excavation depth de beside the existing building in a clayey soil with cohesion, c and safe horizontal distance, H away from the existing building.

Description of variables

The variables affecting the results of the study include the following independent variables:

- q, which represents the uniformly distributed vertical downward pressure due to the existing neighbor building.
- c, which represents the cohesion contribution of shear strength for clayey soil at the site of excavation. This study investigates the behavior of three types of clay based on the average value of c these are; firm, stiff, and very stiff clay.
- dw, represents the depth of ground water at the site of excavation. The effect of ground water depth was investigated in a range of 0.0 to 9.0 meter.
- de, represents the design excavation depth. The maximum depth of 9 m was selected to be investigated in this study which represents approximately the height of three basement floors. As we expected to use side supporting system for deeper excavation based on the stiffness of the soil.

Many scenarios of excavations representing practical cases in reality were investigated. The dependent variable was considered the minimum safe horizontal distance (H) which required for maintaining both the horizontal and vertical displacements within the allowable limits. This distance was measured from the existing building to the excavation limit. Table 2 presents the soil parameters used in the analysis of each model.

Numerical model

The analysis of the cases was performed using finite elements software (Plaxis). Many numerical models were designed to simulate the cases of the study. Figures 2 and 3 presents finite elements mesh for a sample case. The model size is 50 × 50 m, while the element size was selected as 1.0 × 1.0 m. The effective stress analysis was performed using effective model parameters. A bulk modulus for water is added to the bulk modulus of the soil and thereby transforms the effective stiffness parameters E and ν into undrained parameters Eu and νu. Any volumetric strain occurring in an undrained material during a Plastic calculation phase will now give rise to excess pore pressures. The Hardening Soil Model which built in Plaxis was used to simulate the clay soil. The Hardening Soil model is an advanced model for simulating the behavior of different types of soil, both soft soils and stiff soils, [1-8]. When subjected to primary deviatoric loading, soil shows a decreasing stiffness and simultaneously irreversible plastic strains develop. In the special case of a drained triaxial test, the observed relationship between the axial strain and the deviatoric stress can be well approximated by a hyperbola. Such a relationship was first formulated and later used in the well-known hyperbolic model [9-11]. The Hardening Soil model, however, supersedes the hyperbolic model by far: Firstly by using the theory of plasticity rather than the theory of elasticity, secondly by including soil dilatancy and thirdly by introducing a yield cap. Some basic characteristics of the model are: m is the power for stress-level dependency of stiffness, is the plastic straining due to primary deviatoric loading, is the plastic straining due to primary compression and are the elastic unloading/reloading. The failure parameters c, ϕ and ϕ are cohesion, angle of internal friction, and angle of dilatancy respectively are taken according to the Mohr-

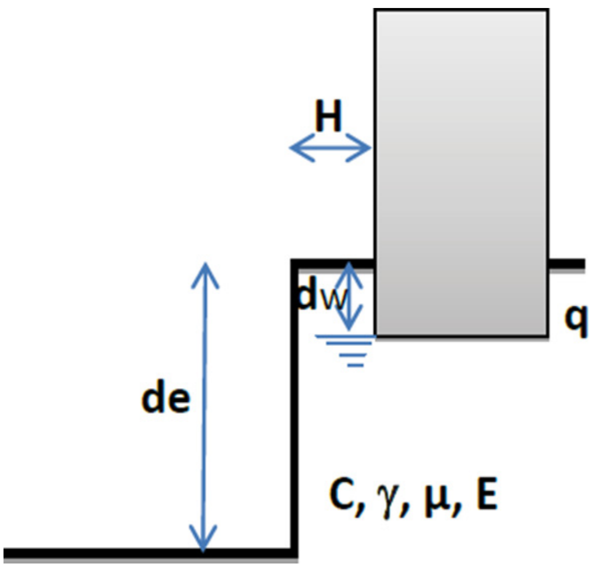

Figure 1: Model of study.

Clay type	Plasticity index	E_{ref}^{50}	E_{ref}^{oed}	E_{ref}^{ur}	C_u	I_P
Medium clay	C_L	45000	67500	3.00E+05	37.5	5.0-20
	C_H	18750	28125	1.00E+05	37.5	20-40
Stiff clay	C_L	90000	135000	7.00E+05	75	5.0-20
	C_H	37500	56250	3.00E+05	75	20-40
Very stiff clay	C_L	180000	270000	1.00E+06	150	5.0-20
	C_H	75000	112500	6.00E+05	150	20-40
Hard clay	C_L	240000	360000	2.00E+06	200	5.0-20
	C_H	100000	150000	8.00E+05	200	20-40

Table 2: Soil parameters.

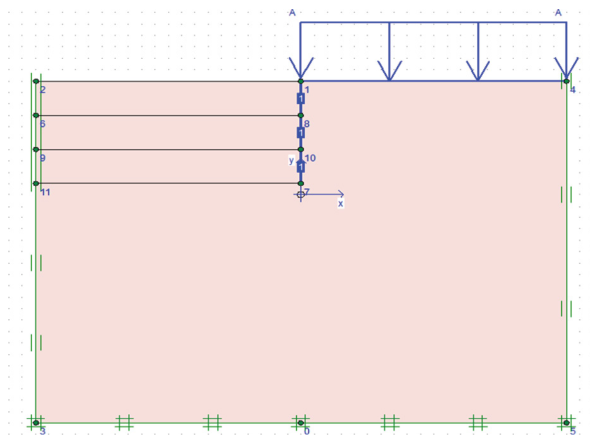

Figure 2: Geometric model.

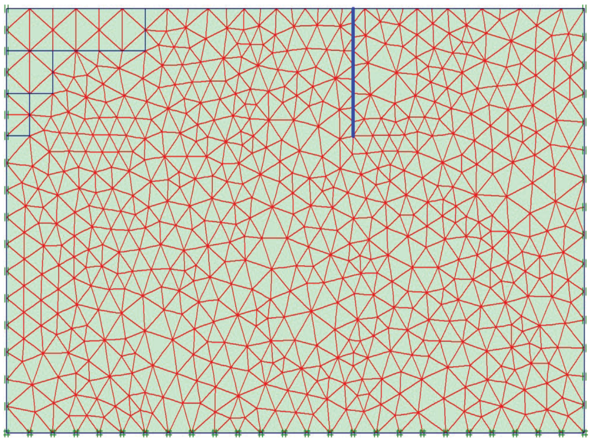

Figure 3: Finite elements mesh.

Coulomb model. As average values for various soil types, Eur ≈ 3E50 and Eoed ≈ E50 are suggested as default settings, but both very soft and very stiff soils tend to give other ratios of Eoed/E50. In contrast to the Mohr-Coulomb model, the Hardening Soil model also accounts for stress-dependency of stiffness moduli. This means that all stiffness's increase with pressure. Hence, all three input stiffness's relate to a reference stress, usually taken as 100 kPa (1 bar).

A diaphram wall with negligible stiffness was installed at the face of boundary of the excavation site adjacent to the neighbored building. This wall was installed to study the deformations along it. Interface elements are modelled by means of the bilinear Mohr-Coulomb model. The interface stiffness is set equal to the elastic soil stiffness. Hence, E =Eur where Eur is stress level dependent, following a power law with Eur proportional to σm.

Once the mesh has been generated, the finite element model is complete. The initial stress state was calculated and the initial water pressures for the clay layer been generated. These conditions are also taken into account to calculate the initial effective stress state in the initial calculation phase. In this phase gravity loading is used where Ko=1-sinϕ, σv=γH, σh=koγH. The second stage represents is the construction of the adjacent building before excavation of the study site. Third, fourth, and fifth stages of construction are the excavation of the site to depths 3, 6, and 9 meters respectively. For each stage of construction both depth of ground water and surcharge due to the adjacent building and the minimum horizontal distance was calculated. This distance was calculated for depths of ground water at 0, 3, 6, 9 meters. As for the surcharge due to the adjacent building, the following values were used: 0, 25, 50, 75, 100, 125, 150 kN/m^2.

Model Analysis

The geometric model and finite elements mesh are shown in Figures 2 and 3 respectively. Undrained analysis is used for a full development of excess pore pressures. Flow of pore water can sometimes be neglected due to a low permeability (clays) and/or a high rate of loading. All clusters that are specified as undrained will indeed behave undrained, even if the cluster or a part of the cluster is located above the phreatic level. The effective model parameters are entered, i.e., E', ν', c', ϕ'. For undrained behavior the total stiffness against isotropic compression of both soil and water is based on an implicit undrained bulk modulus:

The saturated unit weight γsat is used for soil under phreatic level while unsaturated unit weight γunseat is used above this level. Three cases were studied representing medium, stiff, and very stiff clay. For each case the effect of ground water was investigated by changing the water depth from ground surface to a depth of 1.0 m below the excavation depth in many steps. The impact of the surcharge load representing the stresses due to the existing building neighbor to the excavation site was also investigated. The surcharge ranges from 0.0 to 150 kN/m^2 with increment 25 kN/m^2. Each case is explained in the following paragraphs [12-20].

Excavation in medium clay

Table 3 presents the inputs (de, dw, H, q) and the corresponding outputs of the vertical and horizontal displacements sv and sh respectively for fourteen cases. These cases representing the excavation to depths 3, 6, and 9 m. In each case, several attempts were made to obtain the minimum horizontal safe distance that enables excavation to the design depth without soil collapse and maintain the safety of the neighboring building not affected. This was achieved by calculating the vertical and horizontal displacements at the most critical point which located on the boundary of the excavation directly under the neighboring building to ensure that neither of them exceeded the permissible limit of 10 mm. Figures 4-6 illustrate the relationship between the vertical surcharge due to an existing neighboring building at the boundary of the excavation site q and the horizontal safe distance H for different groundwater depths dw for excavation depths 3, 6, and 9 m respectively. As shown in Figure 4 the soil was excavated to depth de=3.0 m. Five water depths were investigated these are 0.0, 1.0, 2.0, 3.0, 4.0 m. The surcharge was increased stepwise from 0.0 to 50 kN/m^2 with step 25 kN/m^2. The safe horizontal distance ranges from 0.0 m at q=0.0, dw=0.0 m to 4.5 m at q=50 kN/m^2 and dw=4 m. At surcharge 25 kn/m^2 H was 0.5, 1.0, 1.25, 1.5 m for dw=1, 2, 3, 4 m respectively. The horizontal distance increased dramatically beyond this value of q. the soil collapsed at surcharge of 75 kN/m^2. For depth 6m the excavation was executed in two steps 3m each to reduce the space left for safety. The measurements of the safe horizontal distance versus the surcharge for different ground water depths are Figure 5. This time H was measured at both the upper and the lower points of each excavation step. In absence of surcharge loads H can be dispensed for any value of dw to depth 6 m where 0.5 m must be left as safe distance. For surcharge 25 kN/m^2, H=1.5, 2.5, 3.5, and 6.0 m for dw=3, 0, 1, 6 m respectively. At surcharge 50 kN/m^2 H=5, 6, 8, 10 m for de=0, 1, 3, 6 m respectively.

For excavation depth de=9 m, three steps are used each 3 m depth. Figure 6 presents the relationships between q and H for five cases representing dw=0, 1, 3, 6, 9 m. At q=0 kN/m^2, the safe horizontal distances H=1, 1.5, 1.5, 2, 3 m while H for q=25 kN/m^2 increased to 7, 8, 8, 8, 8 m for these case respectively. At q=50 kN/m^2, H=15, 16, 18, 24, and 26 m for dw=0, 1, 3, 6, and 9 m respectively. In medium clay the soil collapses beyond surcharge of value 50 kN/m^2 for all cases.

Excavation in stiff clay

The same cases of loading and ground water conditions were investigated but for stiff and very stiff clayey soil. Tables 4 and 5 present the analysis results for all the cases of stiff and very stiff clay respectively. Fourteen cases also were investigated representing three depths of excavation 3, 6, and 9 m, different ground water depths. In each case H was determined for different values of q to satisfy the soil stability conditions. Figures 7-9 present the results for de of 3, 6, 9 m for stiff clay. For very stiff clay curves can't be produced because most of trials results in zero horizontal distances.

It can be clearly observed that the excavation to 3 m depth in stiff clay can be easily executed without lateral supporting system if small horizontal distance (0-2.5) m left. Stiff clay also can accommodate

H			Stiffness	d_e	q, kN/m^2	d_w	s_v, mm	s_h, mm	No. of excav. steps
(d_e=3)	(d_e=6)	(d_e=9)							
0	N.A	N.A	Medium	3	0	0	0.78	1.3	1
0.5	N.A	N.A	Medium	3	25	0	5.09	8.8	1
3	N.A	N.A	Medium	3	50	0	10.68	6.81	1
0	N.A	N.A	Medium	3	0	1	1.74	1.76	1
1	N.A	N.A	Medium	3	25	1	6.75	8.89	1
3	N.A	N.A	Medium	3	50	1	6.86	10.25	1
0	N.A	N.A	Medium	3	0	2	4.11	3.81	1
1	N.A	N.A	Medium	3	25	2	7.79	10.22	1
3.5	N.A	N.A	Medium	3	50	2	7.63	10.36	1
0	N.A	N.A	Medium	3	0	3	4.56	4.23	1
1.25	N.A	N.A	Medium	3	25	3	7.51	10.01	1
4.25	N.A	N.A	Medium	3	50	3	8.77	10.09	1
0	N.A	N.A	Medium	3	0	4	4.26	3.95	1
1.5	N.A	N.A	Medium	3	25	4	6.7	9.03	1
4.5	N.A	N.A	Medium	3	50	4	8.47	9.57	1
0	0.5	N.A	Medium	6	0	0	10.51	9.07	2
2.5	3.5	N.A	Medium	6	25	0	8.28	11	2
5	8	N.A	Medium	6	50	0	5.76	9.78	2
0.5	1	N.A	Medium	6	0	1	9.18	8.5	2
3.5	4.5	N.A	Medium	6	25	1	7.67	9.72	2
6	8.5	N.A	Medium	6	50	1	6.03	9.48	2
0.5	1.5	N.A	Medium	6	0	3	8.62	9.84	2
1.5	4	N.A	Medium	6	25	3	7.12	8.48	2
8	13	N.A	Medium	6	50	3	6.47	9.7	2
0.5	2.5	N.A	Medium	6	0	6	8.91	9.55	2
6	10	N.A	Medium	6	25	6	7.82	9.7	2
10	16	N.A	Medium	6	50	6	6.03	10.6	2
1	7	12	Medium	9	0	0	9.15	10.43	3
7	12	16	Medium	9	25	0	7.44	9.18	3
15	22	25	Medium	9	50	0	6.94	10.93	3
1.5	4	6	Medium	9	0	1	8.61	9.09	3
8	14	18	Medium	9	25	1	8.92	10.88	3
16	26	27	Medium	9	50	1	6.53	10.84	3
1.5	6	9	Medium	9	0	3	9.24	10.37	3
8	20	22	Medium	9	25	3	8.78	10.14	3
18	26	28	Medium	9	50	3	6.96	10.77	3
2	12	15	Medium	9	0	6	10.12	10.59	3
8	22	25	Medium	9	25	6	9.11	10.32	3
24	35	38	Medium	9	50	6	6.68	9.91	3
3	20	26	Medium	9	0	9	9.63	9.03	3
8	26	32	Medium	9	25	9	7.59	9.6	3
26	34	38	Medium	9	50	9	6.71	10.03	3

Table 3: Results of the model analysis for medium clay cases.

higher surchage values up to 75 kN/m^2 compared with 50 kN/m^2 for medium clay. For excavation to 6 m depth, the stiff clay can also accommodate surcharge load to 75 kN/m^2 if excavation executed in two steps.

The horizontal distance in this case increased to 4.25 m compared with 8.0 m for medium clay at the same conditions of loading (50 kN/m^2) and ground water depth (6 m). For 75 kN/m^2, H increased to 8m. For excavation depth 9 m in three steps, H was 12 m compared with 26 m in medium clay at surcharge load 50 kN/m^2. At 75 kN/m^2, H increased to 17 m. It can be also noted that the horizontal distance wasn't affected significantly by the ground water level in stiff clay compared with medium clay [21-24].

Excavation in very stiff clay

For excavation in very stiff clay without lateral supporting system, analysis results for the same fourteen cases are presented in Table 5. For excavation depth 3 m the horizontal distance H was 0.0 for all cases of loading up to surcharge load. It is obvious that in very stiff clay the excavation up to depth of 9 m lateral supporting systems are not required q was 150 kN/m^2. For excavation depth 6 and 9 m H was zero for loads 0, 25, 50, 75, 100, and 125 kN/m^2 while it was 0.25 and 0.5 m for q of 150 kN/m^2 at dw of 3 and 6 m respectively.

Results and Discussion

Soil models representing three types of clay were designated based on its cohesion shear strengths. These models are medium, stiff, and very stiff clay soils. Every numerical model was analyzed many times by changing the values of the parameter affecting on the soil behavior including water depth and surcharge stress beneath the neighbored building. The results of this parametric study can be summarized as shown in Table 6 as follows:

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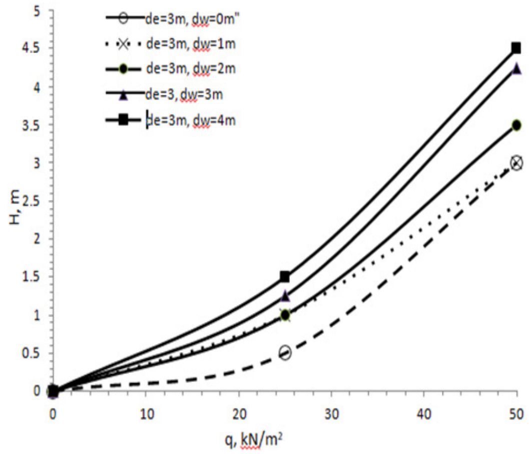

Figure 4: Safe horizontal distance for excavation depth 3.0 m in medium clay *vs.* surcharge load.

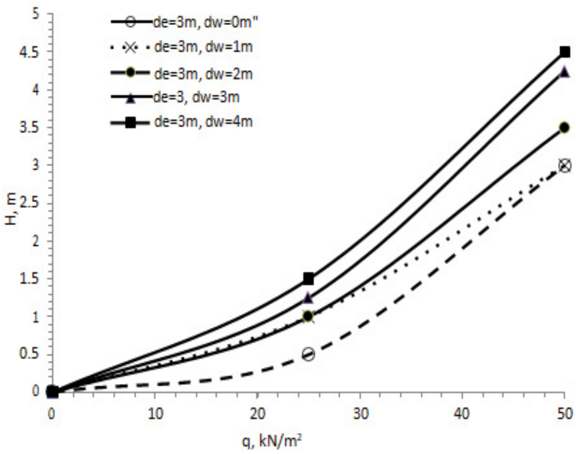

Figure 5: Safe horizontal distance for excavation depth 6.0 m in medium clay *vs.* surcharge load.

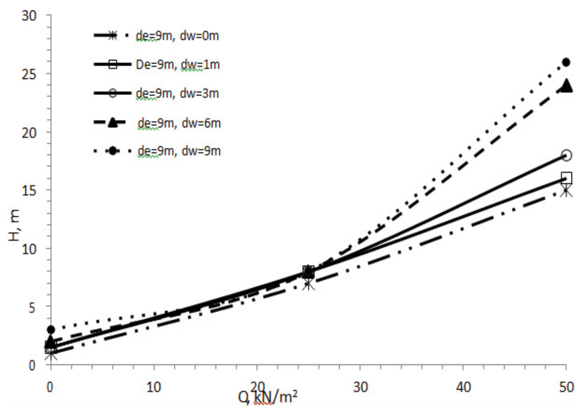

Figure 6: Safe horizontal distance for depth of excavation 9.0 m in medium clay *vs.* surcharge load.

H, m	Stiffn-ess	de, m	q, kN/m2	dw, m	sv, mm	sh, mm	excav. steps	H, m	Stiffn-ess	de, m	q, kN/m2	dw, m	sv, mm	sh, mm	excav. steps
0	Stiff	3	0	0	1.01	0.8	1	0.25	Stiff	6	0	3	6.28	6.02	2
0.25	Stiff	3	25	0	6.56	8.63	1	1.75	Stiff	6	25	3	7.55	10	2
1	Stiff	3	50	0	5	10	1	4.25	Stiff	6	50	3	7.22	10.1	2
0	Stiff	3	0	1	2.02	1.8	1	7.25	Stiff	6	75	3	5.03	10.1	2
0.25	Stiff	3	25	1	6.43	8.33	1	0.25	Stiff	6	0	6	6.92	6.52	2
1	Stiff	3	50	1	4.83	9.06	1	2	Stiff	6	25	6	7.33	9.44	2
2.5	Stiff	3	75	1	3.67	9.74	1	4.25	Stiff	6	50	6	6.45	10.2	2
0	Stiff	3	0	2	2.3	2.12	1	8	Stiff	6	75	6	5.35	10.1	2
0.25	Stiff	3	25	2	6.81	8.79	1	0.5	Stiff	9	0	0	8.76	7.7	3
1.25	Stiff	3	50	2	4.75	9.25	1	3.5	Stiff	9	25	0	7.73	10	3
2.5	Stiff	3	75	2	3.87	9.7	1	7.5	Stiff	9	50	0	6.25	10.3	3
0	Stiff	3	0	3	2.02	2.04	1	0.5	Stiff	9	0	1	8.56	7.93	3
0.25	Stiff	3	25	3	7	9.04	1	3.5	Stiff	9	25	1	7.73	9.94	3
1.25	Stiff	3	50	3	4.86	9.02	1	7.5	Stiff	9	50	1	6.32	10.2	3
2.5	Stiff	3	75	3	4.19	10.1	1	13	Stiff	9	75	1	5.22	10.3	3
0	Stiff	3	0	4	2.17	2	1	0.5	Stiff	9	0	3	9.9	9.56	3
0.25	Stiff	3	25	4	6.96	8.99	1	4.5	Stiff	9	25	3	7.62	10	3
1	Stiff	3	50	4	5.38	9.73	1	9	Stiff	9	50	3	6.22	10	3
2.5	Stiff	3	75	4	4.17	10.1	1	14.5	Stiff	9	75	3	5.27	10.2	3
0	Stiff	6	0	0	4.98	4.17	2	1.5	Stiff	9	0	6	8.96	10.2	3
1.5	Stiff	6	25	0	7.5	10.1	2	3.5	Stiff	9	25	6	7.42	9.95	3
4	Stiff	6	50	0	6.88	10	2	9	Stiff	9	50	6	8.4	9.77	3
0	Stiff	6	0	1	6.3	5.8	2	1.75	Stiff	9	0	9	9.33	10.4	3
1.5	Stiff	6	25	1	7.72	10.1	2	4.7	Stiff	9	25	9	7.76	9.8	3
4.25	Stiff	6	50	1	5.45	9.84	2	12	Stiff	9	50	9	6.88	10.3	3
7.5	Stiff	6	75	1	4.41	9.86	2	17	Stiff	9	75	9	5.57	10.3	3

Table 4: Results of the model analysis for stiff clay cases.

H, m	Stiffnes	de, m	q, kN/ m2	dw, m	sv, mm	sh, mm	No. of excav. steps	H, m	Stiffne s	de, m	q, kN/ m2	dw, m	sv, mm	sh,mm	No. of excav. steps
0	V Stiff	3	0	0	0.0003	0.00028	1	0.25	V Stiff	6	150	1	5.64	10	2
0	V Stiff	3	25	0	1.02	1.18	1	0	V Stiff	6	0	3	1.27	1.3	2
0	V Stiff	3	50	0	1.52	2.09	1	0	V Stiff	6	25	3	2.15	2.5	2
0	V Stiff	3	75	0	1.9	2.93	1	0	V Stiff	6	50	3	3.06	3.9	2
0	V Stiff	3	100	0	2.66	4.18	1	0	V Stiff	6	75	3	4.23	5.6	2
0	V Stiff	3	125	0	3.45	5.49	1	0	V Stiff	6	100	3	5.5	7.6	2
0	V Stiff	3	150	0	4.27	6.85	1	0	V Stiff	6	125	3	7.03	10	2
0	V Stiff	3	0	1	0.0004	0.00042	1	0.25	V Stiff	6	150	3	4.31	8.3	2
0	V Stiff	3	25	1	1.05	1.26	1	0	V Stiff	6	0	6	1.42	1.6	2
0	V Stiff	3	50	1	1.57	2.15	1	0	V Stiff	6	25	6	2.34	2.7	2
0	V Stiff	3	75	1	2.02	3.05	1	0	V Stiff	6	50	6	3.35	4.1	2
0	V Stiff	3	100	1	2.74	4.24	1	0	V Stiff	6	75	6	4.48	5.8	2
0	V Stiff	3	125	1	3.57	5.57	1	0	V Stiff	6	100	6	5.78	7.8	2
0	V Stiff	3	150	1	4.43	6.98	1	0	V Stiff	6	125	6	7.34	10	2
0	V Stiff	3	0	3	0.0005	0.00051	1	0.5	V Stiff	6	150	6	5.04	9.9	2
0	V Stiff	3	25	3	1.11	1.35	1	0, 0	V Stiff	9	0	0	1.61	1.5	3
0	V Stiff	3	50	3	1.62	2.23	1	0, 0	V Stiff	9	25	0	2.8	3	3
0	V Stiff	3	75	3	2.09	3.13	1	0, 0	V Stiff	9	50	0	4.02	4.9	3
0	V Stiff	3	100	3	2.83	4.32	1	0	V Stiff	9	75	0	5.41	7	3
0	V Stiff	3	125	3	3.64	5.64	1	0	V Stiff	9	100	0	7.15	9.6	3
0	V Stiff	3	150	3	4.58	7.13	1	0.25	V Stiff	9	125	0	5.99	10	3
0	V Stiff	3	0	6	0.0004	0.00045	1	1	V Stiff	9	150	0	4.38	10	3
0	V Stiff	3	25	6	1.08	1.31	1	0	V Stiff	9	0	1	1.8	1.8	3
0	V Stiff	3	50	6	1.63	2.23	1	0	V Stiff	9	25	1	2.98	3.3	3
0	V Stiff	3	75	6	2.2	3.24	1	0	V Stiff	9	50	1	4.27	5.2	3
0	V Stiff	3	100	6	2.79	4.3	1	0	V Stiff	9	75	1	5.65	7.3	3
0	V Stiff	3	125	6	3.56	5.56	1	0	V Stiff	9	100	1	7.56	10	3
0	V Stiff	3	150	6	4.53	7.1	1	0.25	V Stiff	9	125	1	5.97	10	3
0	V Stiff	6	0	0	0.0009	0.00087	2	0.5	V Stiff	9	150	1	4.77	10	3
0	V Stiff	6	25	0	1.86	2.16	2	0	V Stiff	9	0	3	2.12	2.3	3

Citation: Bakr RM (2019) The Impact of the Unsupported Excavation on the Boundary of the Active Zone in Medium, Stiff and Very Stiff Clay. J Civil Environ Eng 9: 327. doi: 10.4172/2165-784X.1000327

0	V Stiff	6	50	0	2.73	3.54	2	0	V Stiff	9	25	3	3.31	3.7	3
0	V Stiff	6	75	0	3.74	5.15	2	0	V Stiff	9	50	3	4.59	5.6	3
0	V Stiff	6	100	0	4.88	6.98	2	0	V Stiff	9	75	3	6.25	7.8	3
0	V Stiff	6	125	0	6.17	9.01	2	0	V Stiff	9	100	3	7.14	9.5	3
0.25	V Stiff	6	150	0	5.32	9.78	2	0.5	V Stiff	9	125	3	5.64	10	3
0	V Stiff	6	0	1	1.08	1.08	2	3	V Stiff	9	150	3	4.57	10	3
0	V Stiff	6	25	1	1.96	2.32	2	0	V Stiff	9	0	6	2.46	2.9	3
0	V Stiff	6	50	1	2.9	3.75	2	0	V Stiff	9	25	6	3.72	4.2	3
0	V Stiff	6	75	1	3.8	5.23	2	0	V Stiff	9	50	6	5.1	6	3
0	V Stiff	6	100	1	5.01	7.11	2	0	V Stiff	9	75	6	6.83	8.5	3
0	V Stiff	6	125	1	6.57	9.46	2								

Table 5: Results of the model analysis for very stiff clay cases.

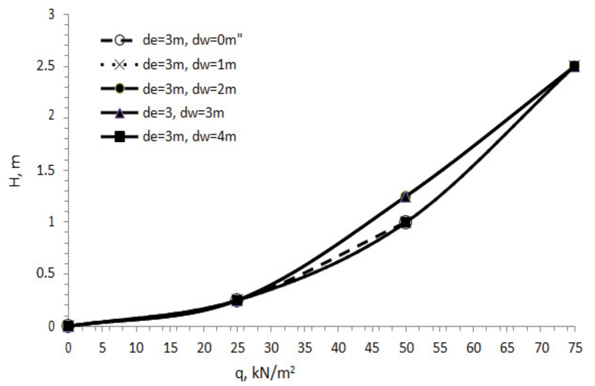

Figure 7: Safe horizontal distance for excavation depth 3.0 m in stiff clay *vs.* surcharge load.

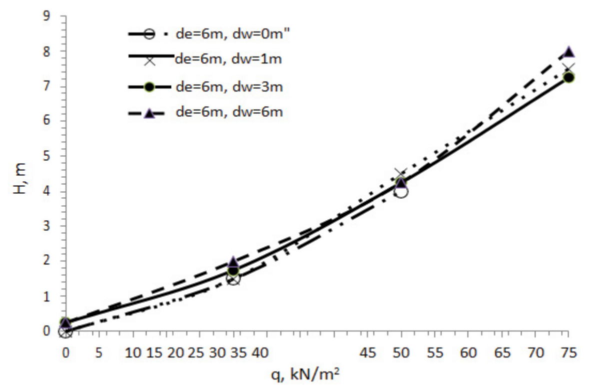

Figure 8: Safe horizontal distance for excavation depth 6.0 m in stiff clay *vs.* surcharge load.

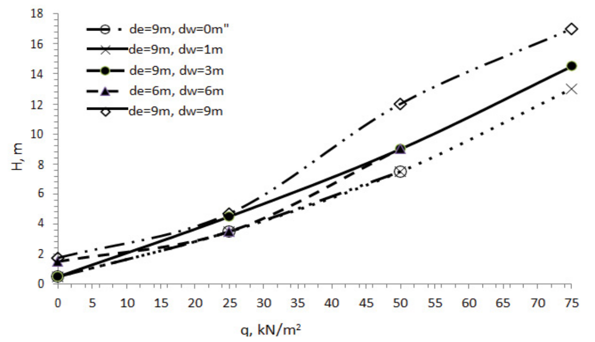

Figure 9: Safe horizontal distance for excavation depth 9.0 m in stiff clay *vs.* surcharge load.

Clay type	Excavation depth			q, kN/m²
	3 m	6 m	9 m	
Medium	1.5	6	8	25
	4.5	10	26	50
	25	2	4.7	25
Stiff	1.25	4.5	12	50
	2.5	8	17	75
	0	0	0	25
	0	0	0	50
Very Stiff	0	0	0	75
	0	0	0	100
	0	0	0.5	125
	0	0.5	NA	150

Table 6: Summary of results for horizontal distance H in meter m.

The results demonstrate that reliable and consistent predictions of soil deformations. In the very stiff soil the excavation may be executed up to 3, 6, and 9m depth for surcharge stress 100, 125, and 150 kN/m² respectively without supporting system. Due to its high shear resistance for q not more than 100 kN/m². For higher values of q this type of clay needs a minimum lateral distance 0.5m for q less than 125 kN/m² and in equilibrium when q reaches 150 kN/m². On the other hand the excavation in medium clay needs relatively large lateral distance 1.5, 6, and 8m for stress 25 kN/m² and 4.5, 10, and 26m for stress 50 kN/m² for excavation depths 3, 6, and 9 m respectively. The excavation in stiff clay the horizontal distance needed is less that required for medium clay and ranges from 0.25 to 2.5 m for q ranges from 25 to 75 kn/m² for excavation 3m. for excavation depth 6 m, H ranges from 2 to 8 m for q ranges from 25 to 75 kN/m². While for 9 m excavation depth, this distance ranges from 4.7 to 17 m for the same range of stress. Economically, it is possible to say that excavation in medium clay is feasible up to a depth of 3 m for stress not exceeding 25 kN/m². For stiff clay feasible excavation depth is 3 m for stress up to 75 kN/m² and 6 m for stress less than 25 kN/m². For very stiff clay the excavation is feasible for depth 3 m for stress up to 150 kN/m², depth 6m up to 125 kN/m², and depth 9 m up to stress 100 kN/m².

Conclusions

The following conclusions can extracted from this study:

1. The cost of the common used retaining systems ranges from 250 to 1500 USD per square meter.
2. In absence of adjacent buildings the excavation can be executed to a depth of 9 m without lateral supporting system in the very stiff clay soil.
3. In presence of adjacent buildings to the construction site, the excavation can be also executed without lateral supporting system such that the following horizontal distance left.
4. Further field investigations are needed to calibrate the results of the numerical study.

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