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Geotechnical study and analysis of the stability of pit walls

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Abstract. One of the main problems and challenges in the field of geotechnical is excavation and conservation of existing buildings in the vicinity of them. In the case of non-compliance with the appropriate procedures in order to protect deep and the slopes are being built, there is the possibility of causing irreparable damage. Therefore, it is necessary to drill before using the support system side, a secure and stable environment be created to protect the pit walls. There are many methods for stabilizing ring. Such as by nailing, micro-piles, bracing and anchors, retaining structures truss and so on. The purpose of this paper is to examine the range of geotechnical and geological project is a monument in the city of Ardabil. According to the geotechnical characteristics of the site excavation pit wall stability analysis using the software limit equilibrium Geostudio / Slope.w and finite element software Plaxis 2.D.

Discussion stability of deep excavation and the use of armed guards to protect the walls of the hollow structures with increasing levels of underground parking have always been important. The overall stability is seen in deformation and deformation of both side walls and the deepening meeting. As well as the stability of the structure in the vicinity of the ring depends on how the plan is calculating prestressing forces. Soil shear strength, hardness wall, angle of inclination, the vertical distances and forces the interlock and control stress and deformation pattern directly on the side wall of the pressure distribution of the wall will affect them. [1] Equations used to calculate the earth pressure and work overload has been proposed. It is known that such relationships Rankine and Coulomb are mentioned. Because of the deformation of the wall, pressure distribution is affected. The use of static pressure is applied to the soil the assumption that the system does not side wall with no deformation. This assumption requires that the wall system with high rigidity and high force applied by prestressing. But designed according to the needs of flexible anchoring system is impractical. [1]

When the wall in the middle layer has good resistance parameters, the high pressure side is near the location of checks. And only a small amount around the area has been fixed. Trapezoid pressure curve in the figure following the "Posh apparent earth pressure" and called for the wall inhibitory accepted. And with good approximation the shape and magnitude of pressure after the wall will be estimated. (FHWA GEC. 4 1999)

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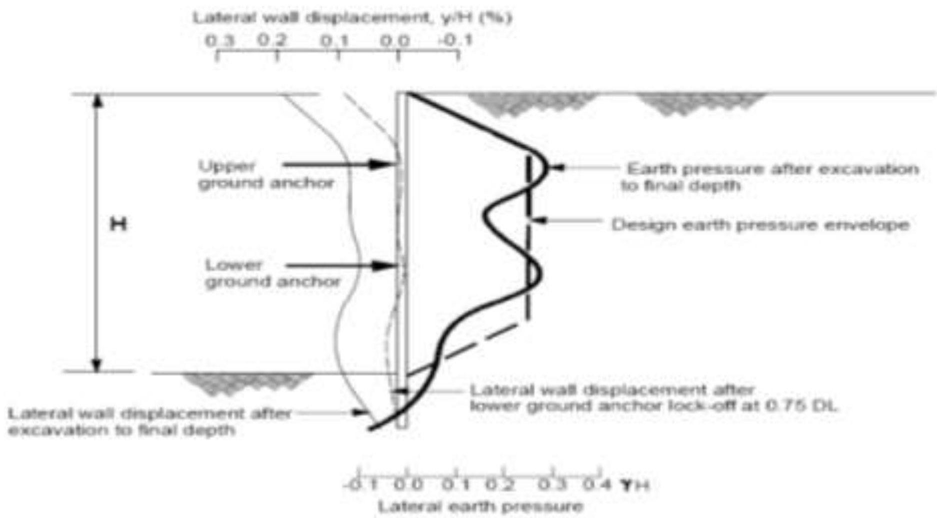


Figure 1. Adverse movements and pressure side wall of the design.

1. BULK PRESSURE

Apparent pressure charts, graphs that are experimental. The first in 1967 by Terzaghi and pack and pack in 1969 for loading a conservative strategy in the field of internal armed were introduced. Diagram for homogeneous profiles such as: (1) load of gravel drainage (2) loading no drainage in clay cracked hard to hard (3) loading no drainage in clay is soft to medium defined.

Terzaghi external pressure charts and Peck (1967) and Peck (1969) in the calculation of the containment wall friction have been proposed. [1] The figure of the reinforced system of internal bezel back analysis of strategies has been developed. And for friction control systems developed and conservative design forces are calculated. Terzaghi external pressure charts and pack in terms of form rectangular or trapezoid, respectively. [1] The maximum width of the chart with a p-value is shown on it. (Fig. 2)

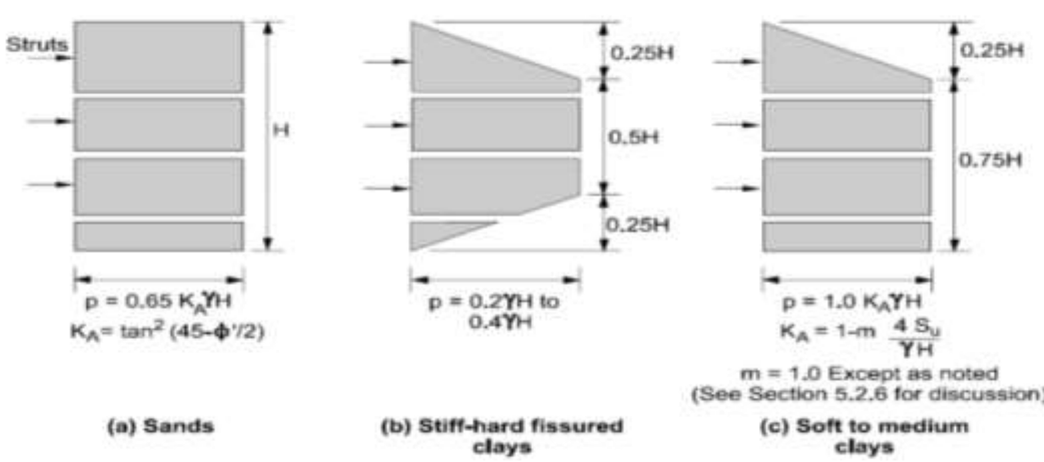


Figure 2. Diagram of external pressure and pack Terzaghi.

1.1. Bulk pressure for stiff clay to hard

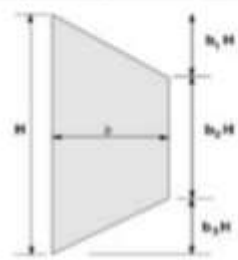
So far, researchers have associated with the distribution of ground pressure cracked clay hard to have a say. The various relationships within the chart for the maximum pressure in this type of soil are introduced. Terzaghi and pack relations are more practical. (Table 1) in the clay ground external pressure associated with the number Ns stability is defined as:

Untrained shear strength of clay under the floor of the pit, and H is the depth of excavation. U is the density of clay, S Figure 2 is shown. The external pressure cap was introduced to clay. One for the stability of the relatively is low numbers (clay, hard and hard) and one for numbers relatively is high stability (clay, soft to medium).

Table 1: Summary of external pressures trapezoid cap for temporary excavation of clay in the soil hard so hard [1]

Reference	b_1	b_2	b_3	Range of maximum pressure ordinate, p	Total load
Terzaghi and Peck (1967)	0.25	0.50	0.25	$0.2\gamma H - 0.4\gamma H$	$0.15\gamma H^2 - 0.30\gamma H^2$
Schnabel (1982)	0.20	0.60	0.20	$0.2\gamma H$ (1)	$0.16\gamma H^2$
Winter (1990)	0.20	0.60	0.20	$0.2\gamma H - 0.32\gamma H$ (1)	$0.16\gamma H^2 - 0.26\gamma H^2$
Ulrich (1989)	0.25	0.50	0.25	$0.2\gamma H - 0.4\gamma H$	$0.15\gamma H^2 - 0.30\gamma H^2$
FHWA-RD-75-130 (1976)	0	1.0	0	$0.15\gamma H - 0.30\gamma H$	$0.15\gamma H^2 - 0.30\gamma H^2$
This work (2)	0.17 (3)	0.66	0.17 (4)	$0.2\gamma H - 0.4\gamma H$	$0.17\gamma H^2 - 0.33\gamma H^2$

Notes: (1) Assumes $\gamma = 19.6\ kN/m^3$
(2) Diagram for multiple levels of ground anchors
(3) Assumes $H_1 = H/4$ (see figure 27)
(4) Assumes $H_{n+1} = H/4$ (see figure 27)



-Geological and geotechnical studies

Location: Yadman, Ardebil

3.1 General Geology of Ardebil

Marine sediments unconformity represents the upper bound of the Katangayi event. However, this event does not have the same intensity everywhere. Therefore, in Alborz - Azerbaijan rocks of Precambrian to appear at the same slope on Kahar formation was observed. But a sudden change in lithology interface there. Some Precambrian high, tuck in the region has created important. And local deformations caused by the angle at some point. The main evidence to prove this was not the Precambrian sedimentary and infrakambrin is obvious between sets. And massive volcanic activity over the highest part of the

Precambrian sub-volcanic rocks and granites are believed to be linked. Vertical movements of the Paleozoic Cambrian cause abrupt change in lithology or stratigraphy is not. In the upper Triassic before, an important movement in Azerbaijan took place and ended the Paleozoic stable platform. Some of the early Cretaceous uplift with little ambiguity detected. The eastern edge of the region has gradually subsides. And the deep sea environment changed with pelagic sediments of the Late Cretaceous to Eocene submarine volcanic accumulated at the bottom. Much of the area is covered by rocks and Tertiary sediments. It should be noted that since the beginning of Oligocene uplift of the fold, its effects can be seen more or less throughout Iran. Qom marine deposits in parts of central and north-eastern and western end of Ardabil are deposited. Authority thick red Balayider 2700 meters south of the lower part, including salt, gypsum and anhydrite is. After Azeri plateau phase of the Oligocene granitic affected. And it was hard and strong, like a mini craton acted territory of Azerbaijan. And about 20 million years ago, at the Talish Mountains and the Lesser Caucasus, faults transformed, fused Caucasus region of Zagros thrust fault zone is connected to the building. So intense tectonic movements for compression approximately north - south of the mini craton to the north Caucasus realize and out of the water. Continental sediments and conglomerates control the same time in South Caucasus and Armenia. Rhyolitic and dacitic rocks of lava and basalt are more in the upper elevations.

-2- Project Location of Yadman

Location in the city of Ardabil, Boulevard intersection with Motahari Avenue pool is located. Figure 2 shows the location of the project.

Figure 2. Location memorial project in Ardebil.

3.3 PROJECT statistics

Location in an area of 4,300 square meters of multi-purpose twin-tower project in three basement floors and two 23-storey blocks will be built on it (Figure (3-24)). Due to the presence of groundwater at the level of 4/6 m and depth of excavation to the level of 11/60 m southeast of the project area has a small portion of the balance 13/60 meters to seal the pit of deep soil mixing system (Deep Soil Mixing) and high-pressure injection (Jet Grouting) were considered. In order to complete the stabilization process control method investments (Anchoring) is used.

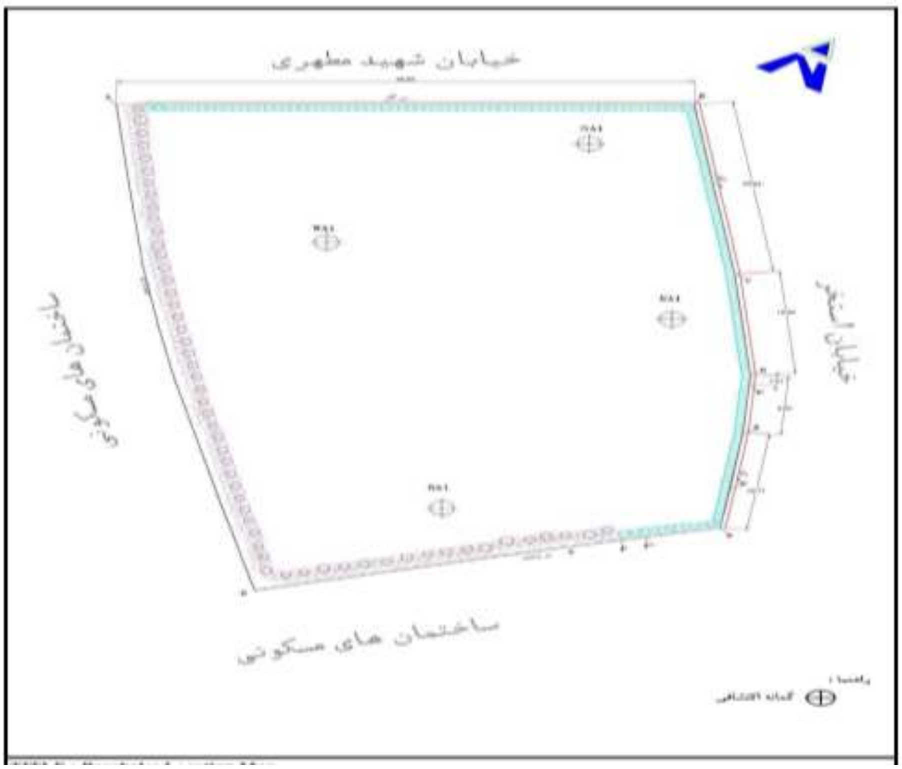


Figure 3. Dimensions and location of drill holes Location.

3.4 Geotechnical studies

3-4-1- field operation

Field operations include drilling four boreholes machine that their details in Table 2 are presented.

All boreholes except Borehole drilling is continuing WA1 Core-making method, the method wash - periods have been excavated.

Table 2. View of drilled holes.

Bore	Align the holes (m)	Water level (m)	Align the holes of the ground (m)	Drilling method
NA	40.32	4.65	3.00	Washing - periodical
SA	39.38	4.65	3.00	Washing - periodical
EA	40.45	4.65	3.00	Washing - periodical
WA	40.45	4.65	3.00	Washing - periodical

-4-2- Laboratory studies

In all speculations Standard Penetration Test (SPT) in accordance with standard ASTM D1586 standard sampling by counting the number of blows to penetrate 45 cm long by 76 cm in the fall of the hammer weight and height is 63/5 kg.

• Standard Penetration Test (Standard Penetration Test, SPT)

Standard Penetration Test (SPT) in boreholes BH3 and BH4 in accordance with standard ASTM D1586 standard sampling by counting the number of blows to penetrate 45 cm by 5/63 kg weight and height of the fall of the hammer is 76 cm. SPT changes to the diagram in Figure 4 are provided.

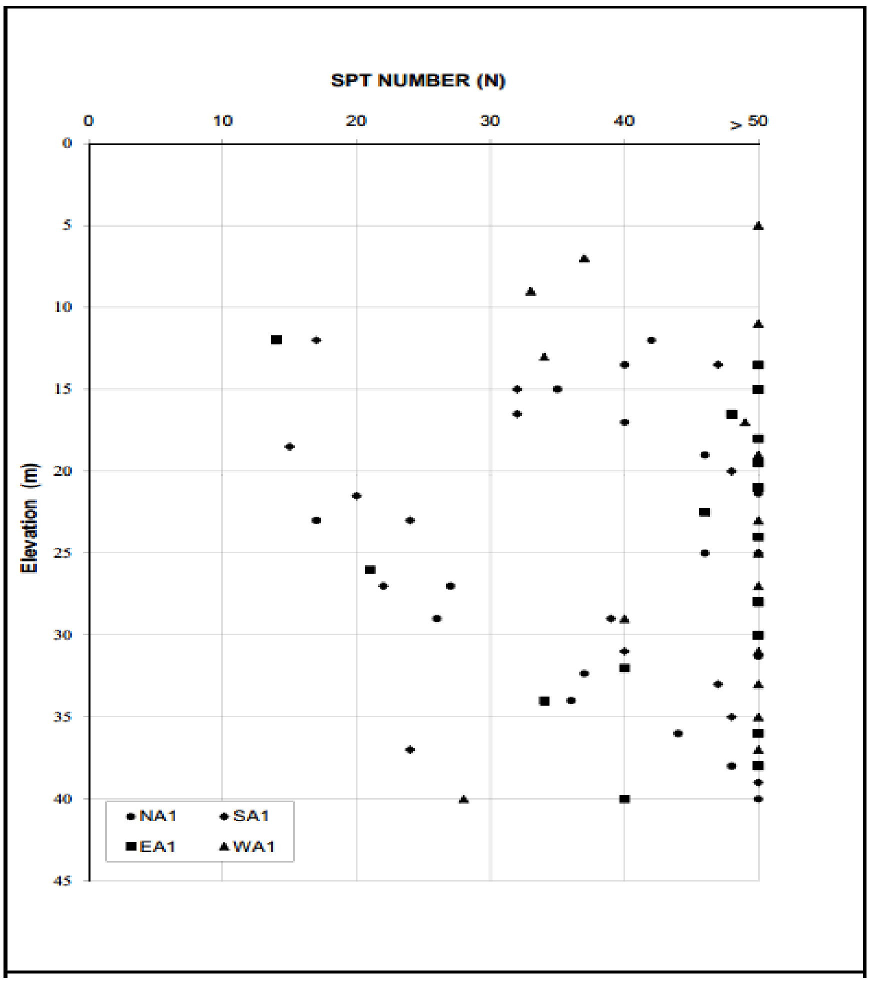


Figure 4. Graph changes to the SPT.

• Permeability ()

LefrancTest field permeability test times by falling (Falling Head) in the holes were WA1 holes (Table 3).

Table 3. Results for soil permeability Lefranc Test for the location soil.

TEST No.	DEPTH (m)	TEST METHOD	PERMEABILITY (cm/sec)		
			Rising Head	*Falling Head*	*Constant Head*
1	1.00–1.50	G		Kh=2.82E–3	
2	2.50–3.00	G		Kh=4.06E–3	
3	4.50–5.00	G		Kh=4.50E–5	

• Subsurface

According to the results of drilling, laboratory experiments and theoretical description of the specimens, materials included stream sediment and subsurface location. And generally rock aggregate acid igneous rocks.

Building up to the level of 12 meters, generally coarse, gray and density are in the category of Very Dense up. This layer classification system USCS with symbols SC, GC, GM and SM has shown (Figure 5).

The balance of 12 meters to end speculation, most of sub-surface layers of silt, sand and silt with layers containing materials are coarse. Thatdepends on classification system USCS with symbols GC, GM / GC, ML / CL, ML and SC / SM are detected. The color of this layer is mainly brown and less gray. The fine layer of consistency in the category of mid-layers Stiff and Hard to Very Dense category are coarse.

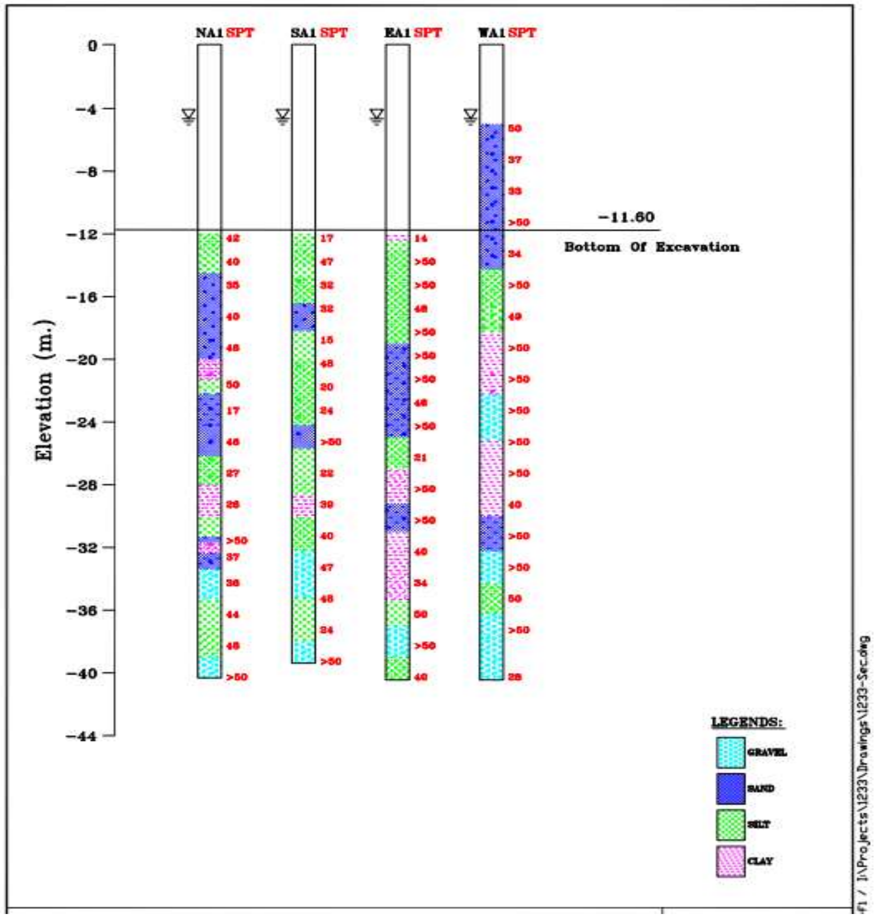


Figure 5. The profile of geotechnical boreholes.

4. Design and calculation of the stabilization and sealing martyr memorial site

4.1 Geotechnical parameters of the soil

Geotechnical parameters based on the results of Standard Penetration Test (SPT) and geotechnical studies conducted laboratory tests and field observations in the first and second phases of the project as described in the table (4) and (5) for both short-term and long-term provided . Groundwater levels in the first stage of the ground at a depth of 2 meters at a depth of 1.4 meters above normal and the second phase of the normal ground level. In all calculations of groundwater levels at ground level is considered normal.

Table 4. Short-term geotechnical parameters.

Layer No.	Soil Type	Depth (m)	$(kN/m^3)\gamma$	$(kg/cm^2)c_u$	$E(kg/cm^2)$	ν
1	Clay	0-6	25	0.6	725 C_u= 400	0.4
2	Sand	6-20	25	0.2-0.6	1000 C_u= 250-500	0.4
3	Sand	20-30	25	0	500	0.3
4	Sand	30-40	25	0.8-0.8	1000 C_u = 700-900	0.4
5	Clay	40<	Inevitable meeting			

Table 5. Long-term geotechnical parameters.

Layer No.	Depth (m)	(kN/m³)γ	(kN/m³) c'	(degree) φ'
I	0-10	25	0.13	24
II	10-20	25	0.12	22
III	20-30	25	0.08	35

4-2- Length required checks and external pressures

4-2-1- During checks

This assumption is used to calculate the inhibition of the sliding wedge angle relative to the angle of the horizon is located. (Figure 3), thus inhibiting the free length of the friction is calculated by the following equation [1]:

Good height H and H again within the control of the floor is hollow. Fixed length control width for maximum soil pressure Figure 8 Metro case is considered 13.5 meter.

Network configuration control friction between vertical and horizontal boreholes with a diameter of 150 mm Subway is 3 meter. Figure 3 shows a cross-sectional structure is shown in the Guardian.

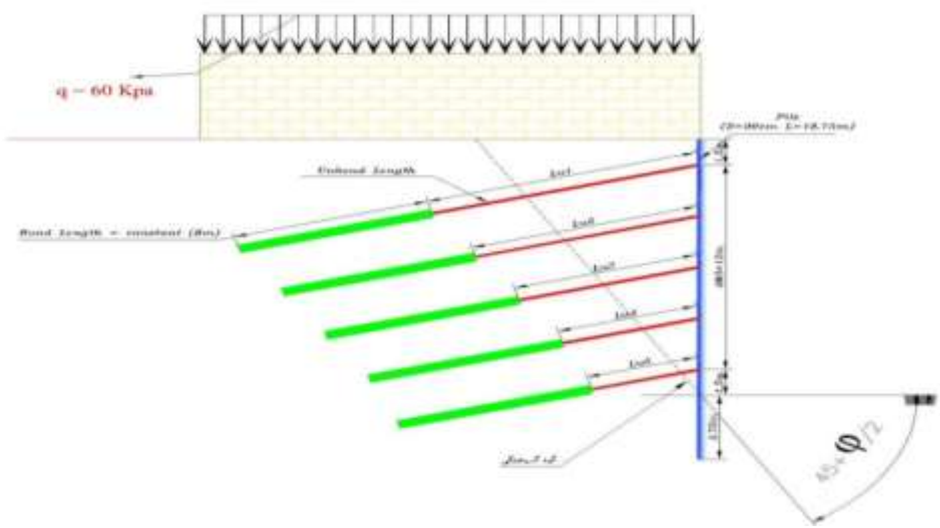


Figure 6. Make up checks.

2.4. 2. Bulk pressures and forces of prestressing checks

Given the pressure distribution of the trapezoid mentioned in paragraph 2 of the pressure exerted on the walls of Bulk pressure and pressure will be overhead. In Figure 7, the pressure distribution of the burden and pressure is portrayed Bulk:

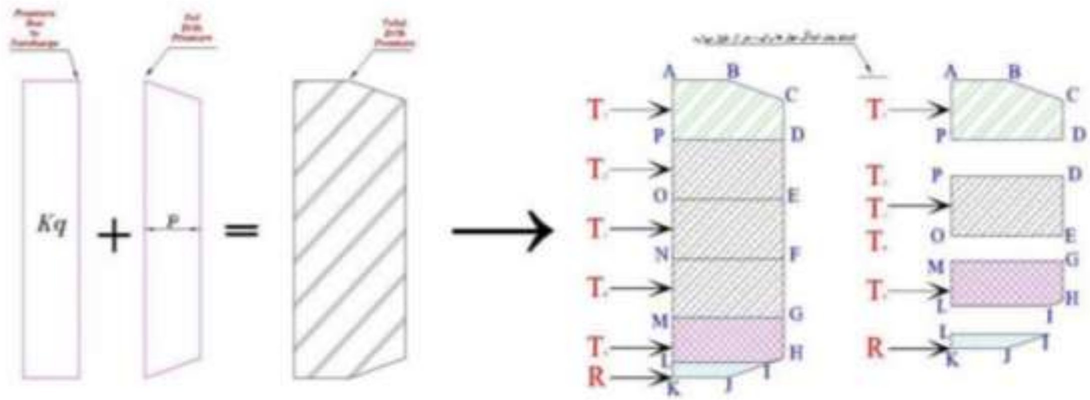


Figure 7. Diagram calculated prestressing force per unit length of the containment wall (perpendicular to the page).

Table 6 summarizes the results of the checks and they are pre-stressing force.

Table 6. The results of the checks and pre-stressing force.

		Free length (m)	Tangly length (m)	control Angle	Harness tensioning force (kN)	The number of field control
P= 0.2 yh	First control	11.1	8	15	644.39	5
	Second control	9.3	8	15	732.91	5
	Third control	7.5	8	15	732.91	5
	forth inhibitor	5.7	8	15	732.91	5
	Fifth control	4.5	8	15	544.12	4
P= 0.4 yh	First control	11.1	13.5	15	1086.97	7
	Second control	9.3	13.5	15	1264.01	9
	Third control	7.5	13.5	15	1264.01	9
	IV inhibitor	5.7	13.5	15	1264.01	9
	Fifth control	4.5	13.5	15	936.91	7

5. Analysis of stability and deformation

Based on the above assumptions and the geometric model is introduced, the software limit equilibrium wall Geostudio / Slope.w and finite element software Plaxis 2.D were analyzed. It was horizontal. But the pattern in relation to the vertical deformation and foundation differential settlement of adjacent structures were observed. Figures 8 and 9 results in software Plaxis 2.D shows the wall. The use of high pressure to force diagram of prestressing was very conservative. The exercise of this power requires the use of cables with more courses (9-7 courses) is. Such a plan is not economically justified, the safety factor of the wall, using the upper limit of the pressure profile in comparison with the least confidence in the proposed regulations 1/35) FHWA temporary state of steady-state 1.5) is conservative plan This confirms. Using the geometric model of the intermediate wall chart was modeled pressure. And some of the results are shown in Table 5 or above. Wall design and data analysis results indicate that the stability of the wall. The use of physical pressure within the maximum width of the chart is more logical. And stresses calculated by the area more reasonable levels more leads.

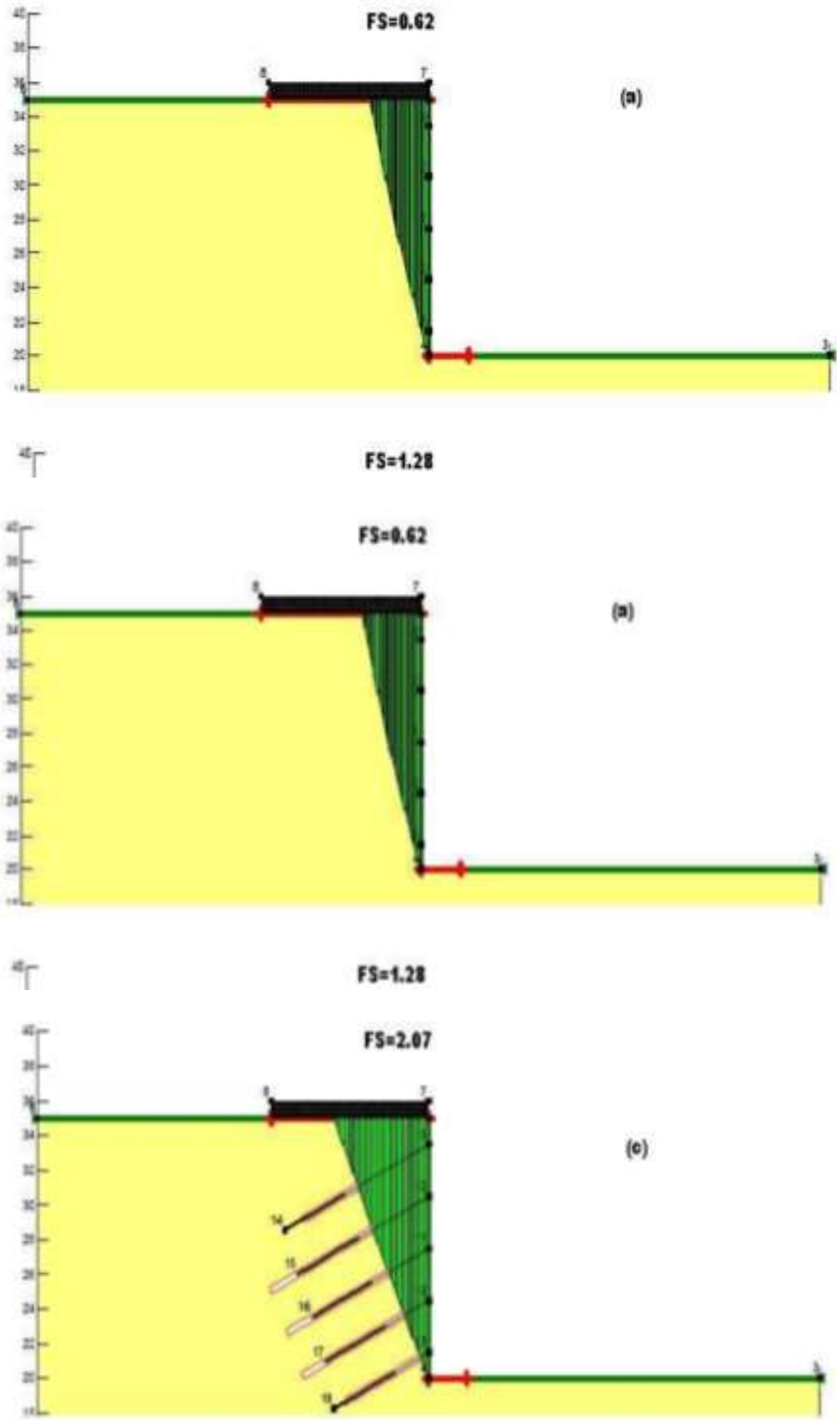


Figure 8. Coefficients to ensure the stability of the wall using software Spencer a) - Geostudio / Slope .w) reliability without reinforcement (b) ensured the safety factor in case $p = 0.4\gamma H$.

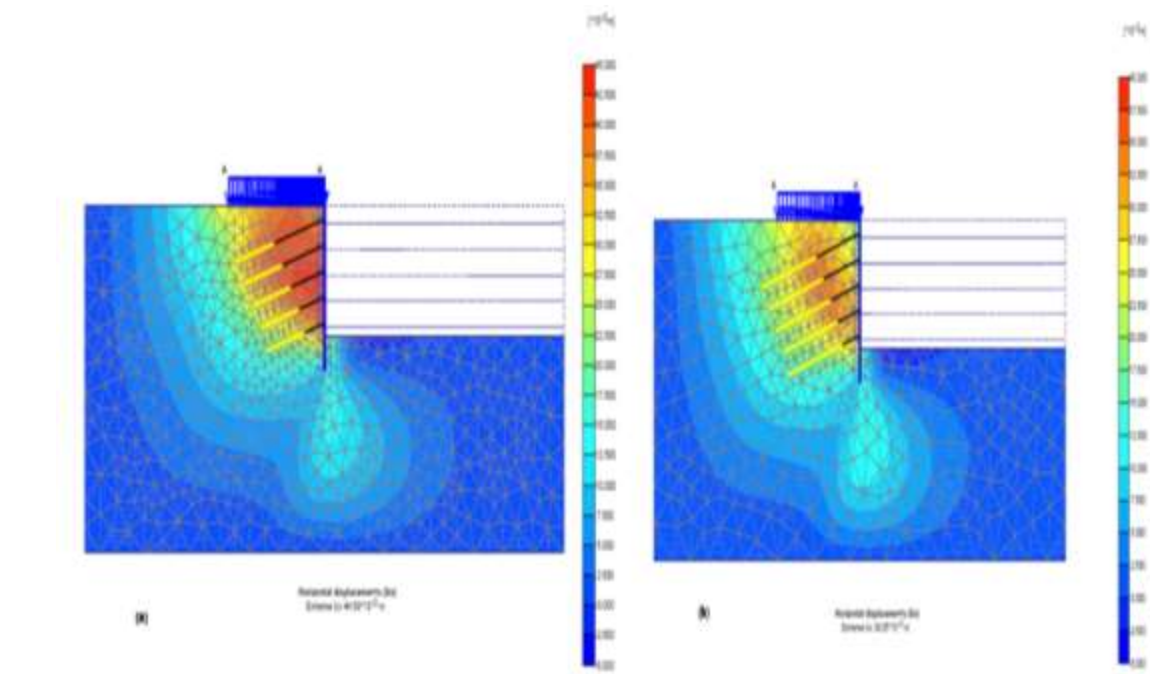


Figure 9. The horizontal deformation: $p = 0.4\gamma H$ **(b)** $p = 0.2\gamma H$ **(a)**.

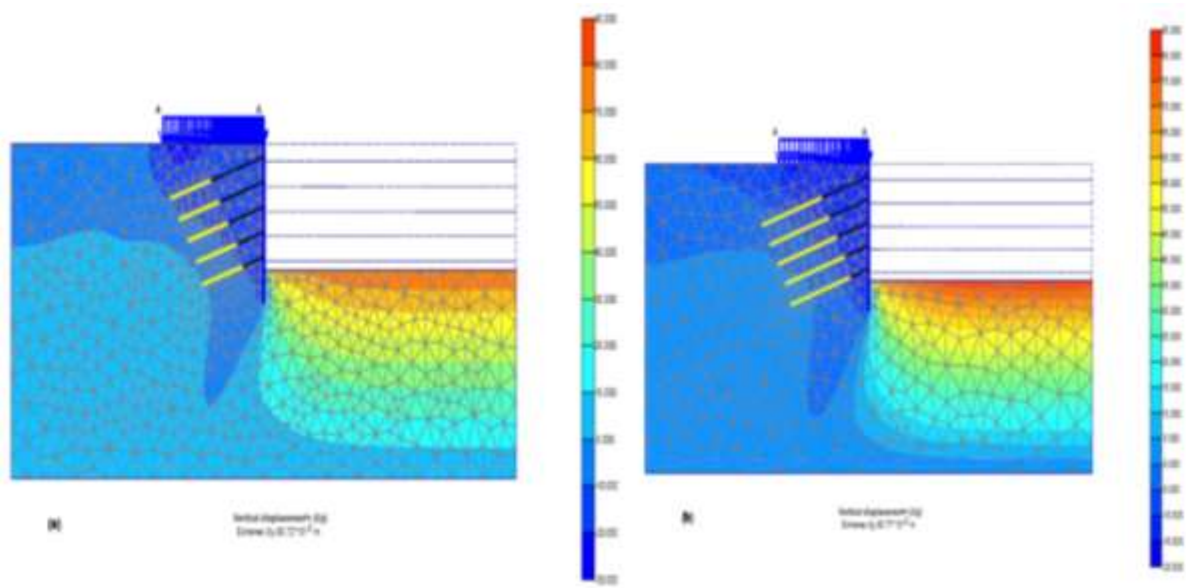


Figure 10. Distribution of vertical deformation $p = 0.4\gamma H$ **(b)** $p = 0.2\gamma H$ **(a)**.

The maximum width of the pressure diagram	Maximum lateral deformation	Change the shape of the lateral edge of the ring	Foundation differential settlement of adjacent structures	Confidence stability
P= 0.2 yh	44.59	37.43	-2	1.28
P= 0.225 yh	44.05	36.51	-9	1.37
P= 0.25 yh	44	34.95	-9	1.42
P= 0.4 yh	39.05	19.25	-8	2.07

Conclusion

In order to study geotechnical drilling at the project site were 4 holes. Physical and mechanical tests were performed on them. The results are as follows:

- The level of groundwater in the project area at a depth of 30.1 meters.

- The type of soil, consisting primarily of fine clay and silt is (CL and ML).

- Relativedensity of soil to a depth of 40 meters is equal to 20 kN m.

- Uniformity coefficient greater soil depth increases gradually. And 5 /. 8 /. Kg cm square is located.

- Modulus of elasticity increases with increasing depth. In shallow and deep as 30 meters to 375 square centimeters is equal to 800 kg Merced.

- The single-core test results soil power gradually becomes deep. As low as 44/1 deep in the depths of up to 0.7 Kg cm square.

Good Location information modeling design software stability Geostudio / Slope.w and Plaxis 2.D indicates that the upper limit of external pressure charts and packs Terzaghi very conservative. As long FHWA GEC1/51994isfriction is used. Finally, a reasonable amount for prediction of stress before stress in these soils is recommended. But the exact calculation of these forces is subject to deformation and stability analysis using software introduced. And ensure the safety of the excavation project.

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