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Case study importance in geotechnical engineering - educational approach

A.Popa, V. Farcas & N. Ilies
Technical University of Cluj Napoca, Romania

ABSTRACT: For Geotechnical Engineering study, to agree theoretical principles and practical experience has a great importance. Current principles, even if they are verified by experience, can give many non-concordant situations between design and real comportment. To form technical and engineering routine from faculty is important to have theoretical knowledge, but also to have knowledge about factors that can lead to ultimate limit states. A great number of examples are necessary to form our students, so that to show scientific mind in engineering theory, but also to explain the phenomenon in earth massive.

Considering these principles utilized for our future engineers, this paper presents some aspects about some retaining structures behavior in deep excavations case. The paper shows also some causes of ultimate limit states occurrence. By two case studies there are presented the importance of geotechnical studies but also the great importance of design geotechnical parameters in design activities.

1 INSTABILITY CAUSES FOR SHEET PILE WALL IN CLUJ NAPOCA – BAUMAX PLATFORM

1.1 *General information; geomorphologic and geotechnical conditions*

Analyzed site is a Commercial complex Baumax, a platform located on the west side of DN1 – Cluj Bucuresti. The platform is artificial subgrade on the north side of Feleac Hill, having North Slope. The whole site has now an alternance between slopes and platforms, having terrace aspect. Most part of terraces made in the last 10 years to make commercial buildings, deposits or office buildings. In the site area it was initially two slopes terrain: one on north south – north direction and another on west – est. direction, and so for site platform it was necessary some height variable earthworks, on west – east direction.

From geological point of view the site characterized by succession of sandy layers, clays and Sarmatian marls. In the sandy Sarmatian complex often appear lime concretion, having variable dimensions (“trovanti”).

Geotechnical study made on the site showed follow stratification for the boreholes on sheeting height:

- Topsoil, dark brown, having organic materials, on 0.80m (F2, F5) – 0.70m(F8) height,
- Brown clay, plastic consistent, contractile, having organic materials, on 0.90m (F5) – 0.70m (F8) height. This layer is missing in F2 borehole (down stream Turzii Street), w_p=17-17.7%, w_L=36 – 42%, I_p=18.3%, I_c=0.47, e=1.05 – 0.89, γ=17.7 – 17.8kN/m^3, φ=16^0, c=18kPa, $M_{2\text{-}3}$=5550 – 5000kPa, U_L=145 – 125%, C_V=110 – 100%, w_S=6.5%, MO=4.3 – 2.4%,
- Yellow clay, plastic consistent – straight, contractile, having sandy layers, lime fragments and sandstone concretions, on 1.00m (F2) – 0.50m (F5) – 4.50m (F8) height, w_p=20%, w_L=54%, I_p=34 - 31%, I_c=0.72, e=0.75, γ=18kN/m^3, φ=16^0, c=20kPa, $M_{2\text{-}3}$=6250kPa, U_L=114%, C_V=96%, w_S=12%,
- Yellow – brown clay, plastic consistent, having sandy zones and lime fragments, from 1.80m height (F2) and 2.30m height (F5). In F8 borehole the layers are not the same, w_p=18.2%, w_L=59 – 51%, I_p=40.7 – 33.4%, I_c=0.61 – 0.82, e=0.89 – 0.892, φ=16^0, c=20kPa, $M_{2\text{-}3}$=6700 - 7150kPa, U_L=110%, C_V=111%, MO=4.3%.

Ground water table is located between 1.40m (F8) and 3.80m (other boreholes), appealing like infiltrations in sandy layers or in contraction fissure in the clay and it is low carbonic aggressively.

Clayey package is expansive clay having high contractions and expansions (active or very active ground). Superior clay layer influenced by infiltration water has expansion cracks leading to low plasticity. This layer has also high void ratio e≥1.1. According to Romanian standard NP 074/2002, the soil is difficult or medium difficult to bear a foundation. Considering the values for Edometric deformation modulus, superior layer is a very compressive soil.

Geotechnical characteristics given in geotechnical study are probable characteristic values of geotechnical parameters, but the determination results are not given. According to NP 074 sustaining walls oblige to research the ground on minimum three times excavation height, but the research is only on 6 meters. Geotechnical study gives not any information about design geotechnical characteristic (C_V) needed for calculations in cohesion less ground. It can appreciate than on short term, behind and under excavation, the ground is on undrained state, which imposes to evaluate cohesion in undrained conditions.

1.2 *Constructive elements for diaphragm wall, made from bored piles.*

To sustain the slope resulted by cutting the platform, on the south side, it was proposed and executed a diaphragm wall, from reinforced concrete piles, interspaced. All excavation alignment was dividing in 21 sectors, each sector having 5 piles, 620mm diameter. Excavation height varies between 1.778 – 4.854m. On the superior part the piles have a girder having 1.00m height and 1.10m width. Bored piles were pinned in the ground under the excavation onto variable height ensuring 4.20 – 9.00m pile length. On the south side the diaphragm wall adjoin a reinforced concrete cantilever wall. The wall has 2.20m and sustains a ground platform located at 1.60m from the superior part of the wall (~0.60m). The wall footing has ~0.40m height and it cleaves to the diaphragm wall (between the sheet pile wall and cantilever wall there is a shed, 25mm). All along the wall there are two sheds (between 6 and 7 sector and 15 and 16 sector). Sheet wall is making by 620mm diameter piles, 1.00m interspaced, reinforced by 12Φ20, PC 52, Φ5/150mm and Φ5/250mm strap. Concrete covering is 80mm. For monitoring purpose there were positioned three PVC tubes Φ62mm on the pile height. Concrete class is C20/25.

The degradation seen on the wall was, Figure 1:

– After finishing excavation (2-4 days, September 2006) the piles located between final sectors (8 - 22) start to lean, in two hours horizontal displacement of the girder was between ~15cm on marginal sectors to 1 cm on pile no. 39. To impede the displacement earth filling was made to realize a ground slope adjoining the wall (onto head girder). In the same period was made excavation on the bottom of the wall to relocate a gas pipe (Φ200mm). Earth cut having ~2.00m height and 1,50m width produced on ~40m wall length, in 10 hours, 2,5cm horizontal displacement of the piles (pile no. 100 to 108). Displacement developments continue during September 2006 – March 2007, reaching final value of 5.4cm. To stop the phenomenon earth cut was filled and it executes a slope (~1.50m) on the wall bottom.
– In the present pile 1 reached 15cm horizontal displacement (amortized in 2 days after beginning), pile 110 reached 5.4cm horizontal displacement (having displacements with reduced values on a long period). Earth cut for the gas pipe moved in the corner area of the property.

a.

b.


c.

Figure 1. Degradation on the sheet pile wall.

Sheet pile wall monitoring works:

– Verifying component elements integrity by continuous velocity logging,
– Bearing test to determinate horizontal bearing capacity of the pile,
– Bearing test to determinate critical horizontal load for the pile.

Test results lead to following conclusions:

1 To verify piles concrete continuity and compression strength 33 tests were made using continuous velocity logging. Three piles have surface discontinuities (P54, 64 and 74) at 1.50÷2.20m depth. Three piles have discontinuities in current sections (P1, 13, 15) at 4.00÷4.75m depth. Three piles have bottom discontinuities (P 68, 91, 93) at 7.25÷9.00m depth. Ulterior discovering of the piles indicates high surface discontinuity of the piles on which segregation and discontinuity appears on pile depth having clay intercalations (Picture 1). The cause is tube concreting interruption and concreting directly into the hole, thus existing zones with fallen bore walls. There are 68 piles with accentuate segregation and 40 piles which does not have segregations in concrete section.

2 Measurements made by continuous velocity logging indicating medium compression strength R_c=19.106 – 33.938N/mm^2 for most part of the tested piles. 5 piles have compression strength R_c=14.598 – 17.261N/mm^2 which is lower than medium (corresponding for C12/5 concrete class). Quality certificates emitted by the producer indicate C20/25 concrete class for all the executed piles. This fact may be principally because of discontinuities along he piles.

3 Critical horizontal force is R_{cr}=97.50kN, resulting a displacement on the superior part Δ=10mm and a bend θ=0.0023. Strength test results prove that bearing capacity on horizontal load is R=46.66kN. Maximum allowed displacement is Δ_{max}=4.83 – 9.63mm.

Some appreciations on the design elements.

According to NP 113 – 04 and SR EN 1538 – 2002 there are some appreciations to make on the project:

- In the project between the piles axes was adopted 1000mm, but the maximum value for 600mm diameter piles is 700mm,
- Reinforcing rate for the pile is p=0.5% towards 0.0025A_c, longitudinal reinforcement is 12Φ20 and it respects minimum conditions required. Concrete covering, 80mm, is higher than recommended 60mm. The strap Φ10/ 150 – 250mm is bigger than recommended Φ8/ 150 – 250mm. Concrete class C20/25 is higher than minimum recommended (C12/15), but considering water aggressiveness(low carbonic aggressiveness) SR EN 206 – 1 recommend C30/37(exposure class XA1).

1.3 *Wall checks on service limit state.*

To do the verification on service limit state it will consider loads in the service state, where the most important load is earth pressure.

Wall check was made in three variants.

1 Considering active earth pressure behind the sheet pile wall

2 Considering earth cut in front of the wall

3 Considering active earth pressure behind the sheet pile wall before / after earth cut in front of the wall.

Because the characteristic geotechnical values for design are missing, some relations from specialty literature were used to provide characteristic geotechnical values to count up.

To determinate design values according to Eurocode 7 (SR EN 1997 – 1- 2004) partial safety coefficients have the values $\gamma_{sc}=\gamma_{s\varphi}=\gamma_{ssv}=1$. Using those coefficients the design values or geotechnical characteristics are:

Layer	γ_d	Φ'_d	C'_d	C_{ud}
	kN/m^3	^{0}	kPa	kPa
(2) brown clay	17.8	13.5	18	18 - 15
(4) yellow – brown clay	18	12.5	20	42.8 - 56

Hypothesis I.

In checking sheet pile wall stability has been used limit equilibrium method (Blum method) for cohesive grounds. In calculus it consider the ground under ground table in effective stress state, due to the possibility of water infiltration trough contractive cracks and the terrain below ground table - for short term, in total parameters. The scheme used for the wall in 1 – 40 piles sector is the one corresponding to piles in the area with the lower earth cut, Figure 2. On the wall height, earth pressure is negative (without pressure) due to the effect of effective cohesion (c'=18kPa). Earth pressure given by the surcharge effect (cantilever wall and ground) consider by the two components p_1=46kN/m^2 and p_2=14,58kN/m^2, according to Romanian standard NP 113 – 04. From the moment equation it can obtain the fixed support length.

In case A, t=3.63m < t_{ef}=3.92m and in case B, t=1.10m < t_{ef}=3.92m, Figure 2.

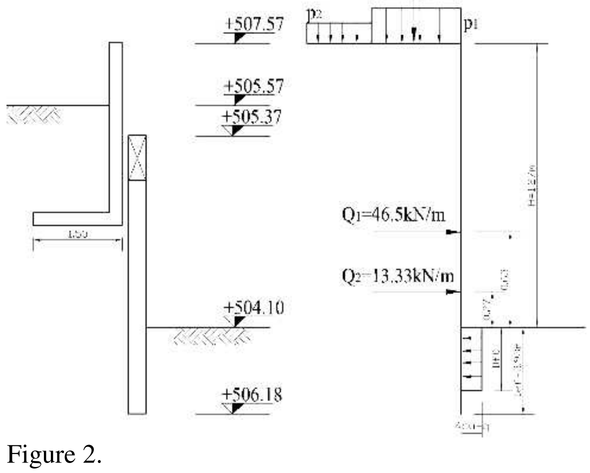

Figure 2.

Hypothesis II (piles 40 - 110).

For the wall bottom, having the bigger earth cut height (P110) calculus check scheme is given in Figure 3.

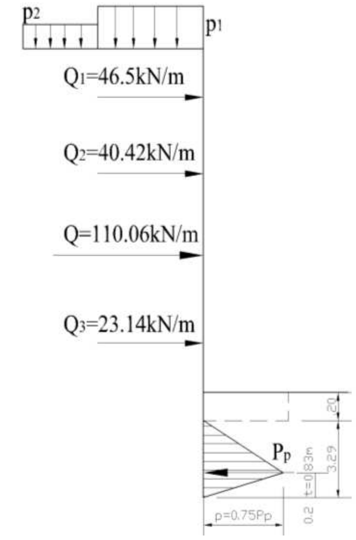

Figure 3.

In case A, t=7.43m >t_{ef}=5.29m and in case B, t=3.10m < t_{ef}=5.29m. For more than 2 m earth cut height and 2 m width situation, because that lateral support is missing rotation movement get started which lead to redistribution of reaction diagram on the wall. Diagram of resistive efforts became from a rectangular one, a triangular one on the fixed support length, Figure 4. It resulted overturning of the wall until the remobilization of the entire passive resistance of the ground in front of the wall.

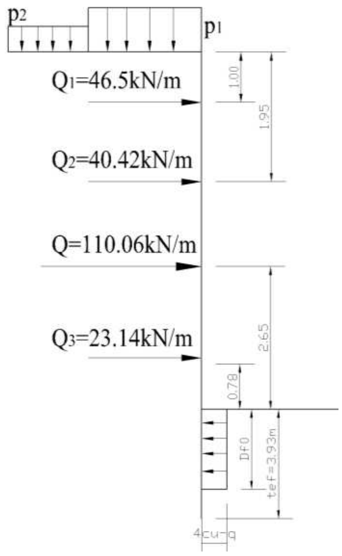

Figure 4.

1.4 *Conclusions*

According to the verification done utilizing Eurocode 7 and NP 13 – 04 we have the following conclusions:

1 Concrete class according to continuous velocity logging tests is C12/15, even if in the project is C20/25. This discrepancy between site measurements and the ones from testing bulletin can be explained by low quality of concrete compaction. The demand of a superior concrete class towards the minimum one from the standard is impose by the exposure class (XA1)

2 Discovering sheet pile wall reveal a poor technical state of the superior zone, with a high concrete segregation. Segregated surface is superior to the 250cm limit, appearing not only on the exterior side of the pile, but in ~20÷30% of the pile diameter. It supposes that the concrete has the same quality on the uncovered interior side. Poor state of the concrete makes that the segregated zone not to participate on the assumption of shear forces. More affected piles are the ones executed in the last stage (80 - 108), but also the intermediary ones (20 - 60), casing cementation being interrupted and concreting directly in the concrete form. It considers that in the zones of maximum bending moment concrete quality is fine for the majority of piles.

3 In situ tests for horizontal forces direct to the value of bearing capacity on horizontal forces R_{or}=47.77kN. Comparing this value to the earth pressure in exterior zones of the wall it results that for the zone 1 – 5: R_a=59.83kN> R_{or}=47.77kN and for the zone 105 – 110: R_a=110.06> R_{or}=47.77kN. The value of bearing capacity must be related to general stability of the wall, resulting that for the design values for shearing characteristics the values calculated in A variant are more credible.

4 Verifications done on the marginal sectors in which were observed displacements over the admitted value for pile sheeting wall droved to the next conclusions:

– On the wall area containing no. 80 – 108 piles, where earth works were done to locate a gas pipe, the cause for wall inclination is the elimination of passive earth pressure in front of the wall. The effective fixed support length could ensure wall stability only in B variant, in which design cohesion value is superior ($C_{u,\ medium}$=49.5kPa), this situation being close to the characteristic values obtained for borehole F5, located in the interested area. Earth cut execution lead to a significant decrease of passive earth reaction and it create the possibility to loss the wall stability.

– For the wall in the minimum earth cut (no. 1 – 20 piles), the verifications lead to the conclusion that fixed support length in A case is closer to the necessary one. However, comparing horizontal bearing capacity obtained in situ, it results that design values for geotechnical characteristics (cohesion obtained in undrained conditions) are lower than estimated ones. In addition, this happened because the geotechnical study is incomplete, and so we have wall instability, because of a smaller fixed support length than necessary one.

5 Some recommendations according to Romanian standard NE 012 – 99: will consolidate all the

piles in the areas having segregations on a bigger area than minimum recomnded. Consolidation works will be made by cleaning the pile body of segregated concrete and consolidation by casing grouting (grouting epoxidic raisin) and concrete coating.

6 So that the visible side of the sheet pile wall to be suitable, after consolidation a reinforced concrete diaphragm wall will be executed in front of the piles(~7÷10cm).

7 In the piles no. 1 – 20 area, where horizontal displacements were observed during earthworks, some long term consolidation works are proposed:

- To execute post stressed anchorages (P_a=250kN), fixed in the wall by metallic profiles,
- To execute some earth backfills or reinforced concrete counterfort having the weight equal to the existing earth backfill. It will be executed before earth cut, by cased earthworks and concreting.
- In the wall area where the displacements are reduced (piles no. 80 - 108), is recommended to execute some consolidation works like:
  - post stressed anchorages or
  - Reinforced concrete counterfort at the wall end (pile 108 area) to undertake supplementary earth pressure.

2 VERIFICATION OF THE STABILITY FOR A SHEET PILE WALL (TANGENT PILES) ON KAUFLAND -ZALAU

2.1 *General information*

The site is located on the left side of Zalau River, on a hill having the slope on west – east direction, between Sf. Vineri Church and the River Canal. On the north side of the site there is a recreation area and on the south area there is a kindergarden and flat – buildings, Figure 5.

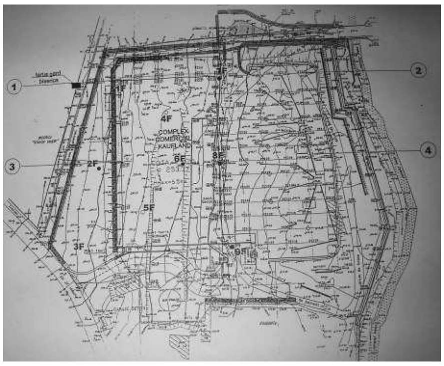

Figure 5.

The terrain in the area has small slopes and close to the site there is a road and a platform. Close to the platform some surface land slides were observed. On the hill area there are counter slopes zones, where because of the water coming from the hill some small lakes have formed.

To execute the commercial building it was necessary to execute a platform having height difference to the hill about 4.60m. For slope sustaining a sheet pile wall was designed (Φ880mm), tangent piles, tied on the superior part by a girder (0.90x1.00).

6 boreholes (2005) and 3 boreholes (2006) made ground examination and also samples taked out from site and laboratory tests. According to NP 074 the wall is in the second geotechnical category which imposes to satisfy some conditions about ground examination: boreholes made at 5.00m depth along the wall and minim three times wall height examination height.

According to geotechnical study 3 boreholes have been made along the wall, driven by ~8 - 9m and the ground has inclined stratification on homogenous base of marl clay. Based on boreholes and laboratory tests follow stratification was established along the wall:

- Ground filling, ~20cm,
- Clayey silt and gravel filling, bricks, ~0.50m thickness (on borehole F1), ~1.40m (F3), this layer is missing in central area of the wall (F2), I_p=18.3÷18.4%, I_c=0.72÷0.73, e=0.68 – 0.72, γ=19.8 – 20kN/m^3, φ=17 - 18^0, c=15 - 17kPa, M=6250 – 6666kPa, U_L=70%,,
- Silty clay, brown clayey silt, plastic consistent, 1,70cm thickness on F1, 2,10 on F2 up to 1.00m on F3, I_p=28.9÷33.1%, I_c=0.65÷0.82, e=0.70 – 0.80, γ=19.3 – 19.4kN/m^3, φ=14 - 15^0, c'=30 - 40kPa, M_{2-3}=6250 – 8333kPa, U_L=80 - 150%,
- Brown silty sand melted with gravel, rock fragments, having ~30cm thickness, C=14%, G=14%, S=49%, w=19%
- Yellow brown silty clay, very stiff consistency, ~60cm thickness, I_p=28.9÷33.1%, I_c=0.65÷0.82, e=0.70 – 0.80, γ=19.3 – 19.4kN/m^3, φ=14 - 15^0, c'=30 - 40kPa, M_{2-3}=6250 – 8333kPa, U_L=80 - 150%,
- Slope base is a package of marly clay, brown, very stiff consistency, I_p=32.9÷59.9%, I_c=0.85÷0.98, e=0.59 – 0.74, γ=19.4 – 20.7kN/m^3, φ'=15 - 20^0, c=25 - 50kPa, M_{2-3}=6666 – 9090kPa, U_L=150 - 210%,

Ground water table is located in F2 at -3.00m, but it can rise up to -1.00m from ground table and in F3 from -2.60m to -1.40m. Underground water has very low sulphatic aggressiveness, but very high carbonic aggressiveness.

Soil layers are in humid or saturated state (S_r>0.85 – 0.90). Silty clay layers are active or very active expansive grounds. All the layers have I_c>0.65, lower values having only the layers close to permeable silty sand. Clay layers have between them permeable layers, of silty sand or gravel, where ground water table is located.

For design, there were used follow geotechnical characteristics:

1 Earth filling, h=1.10, γ=17kN/m^3, φ'=18^0, c'=0kPa, I_c=0.6,
2 Silty clay, h=2.90m, γ=19.3kN/m^3, φ'=15^0, c'=35kPa, I_c=0.73,
3 Marly clay, h=2.90m, γ=20kN/m^3, φ'=18^0, c'=37kPa, I_c=0.92,
4 Sandy marl, h=3.10m, γ=20,5kN/m^3, φ'=19^0, c'=40kPa, I_c=0.93.

There are not information about the decay state of the marly clay, but the variation of plasticity index proves that there are some damaged marly clay layers (high values for I_p), expecting decrease of shear undrained resistance.

It not exist any information about shear characteristics, but is considered that the slope behind the wall has drained conditions and in front of the wall has undrained conditions. Taking into consideration this aspects, the slope behind the wall is calculated using effective efforts (φ'and c') (active state) and the massif by the wall bottom is calculated in total efforts (C_V) (in passive state).

2.2 *Behavior of sheet pile wall*

To sustain the slope for the platform (platform Kaufland) in the upper part a sheet pile wall was made. Piles are using auger pile technology, cased in place piles and they are tangent piles. Pile diameter is 880mm and pile height is 12.00m. Concrete class utilized is C20/25, but because of water aggressiveness and exposure class it should utilize C30/37 and so the conditions for concrete class aren't complete.

Toward initial project, where the piles had spaces between them, 12cm, sheet pile wall executed on the same superior level of the raft foundation, but the piles bottom has fixed support length of minimum 6m.

After concreting the wall, some piles on a second line were executed, every two piles, tied by the first line by girders having the same dimensions like the one tying pile heads. When sheet pile wall was finished and also the platform above horizontal displacements were observed having values between 0.20÷0.50cm and 5÷6cm (value measured from 25.10.2006 – 14.03.2007), which shows a ~1cm displacement by month. Concluding movement evolution, higher horizontal displacements are in the middle part of the wall, where the displacement is 5.5÷6.1cm. Those values are higher than admitted for the sheet pile wall head, Δ_{ad}=0.5÷1.0cm.

2.3 *Verification on service limit states*

To do the verification on service limit state it will consider loads in the service state, where the most important load is earth pressure. Because above the construction there is a slope, the surcharge was not consider (q=10kN/m^2). The verification considered, except the resistance condition, unacceptable displacements of the wall and surrounding terrain, which can influence also the foundation system of the plant.

Wall check was made in three variants.

1 Considering active earth pressure behind the sheet pile wall
2 Considering the force from an earth slide on a preexisting sliding surface.

The values of geotechnical characteristics are:

Layer	γ_d	φ_d	C'
	kN/m^3	^{0}	kPa
(2) Silty clay	19.3	15	35
(3) Marly clay	20.0	18	37
(4) Sandy marl	20.0	17	16

There is not any information about the way that they were determined, but they correspond to the characteristic values. To determine design characteristics partial safety coefficients were adopted: $\gamma_{sc}=\gamma_{s\varphi}=\gamma_{ssu}=1$.

In checking sheet pile wall stability has been used limit equilibrium method (Blum method) for cohesive grounds.

A variant.

k_a – active earth pressure coefficients were determinate according to SR EN 1997 – 1 – 2007 (EC 7). On the surface level p_{a1}=0 and is not negative because taking in consideration water infiltration. The value of undrained shear characteristics C_v=50kPa, to take into consideration the decay of the marl base layer. From moment equation it result D=0.60m and D_f=0.75m <<6.33(existing value) and the capable moment of the pile is bigger than maximum moment,Figure 6.

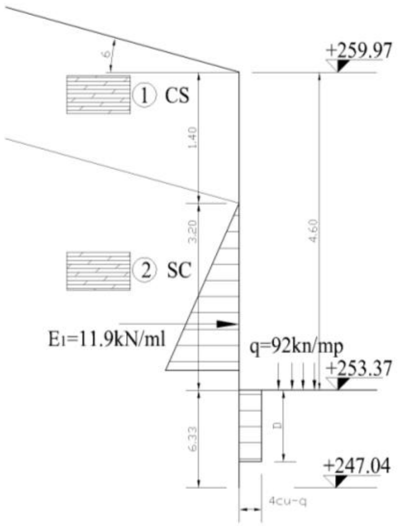

Figure 6.

B variant.

It consider that the superior layer having on base a sandy layer, with water infiltrations, has a plastic sliding on a existing sliding surface. The length of the ground package that slides is L≅20m, unto F2 borehole. Considering the equilibrium equation on the sliding surface earth pressure is W_h=83.6kN/ml. From the moment equation can conclude that D=3.54 and D_f=4.25<6.33. Maximum moment value is for T=0, M_{max}=440.60kNm< M_{cap}=473kNm. The wall displacement, for the pile bottom, for the section having free support is Δ=15.67mm<<$\Delta_{existent}$=60mm, Figure 7.

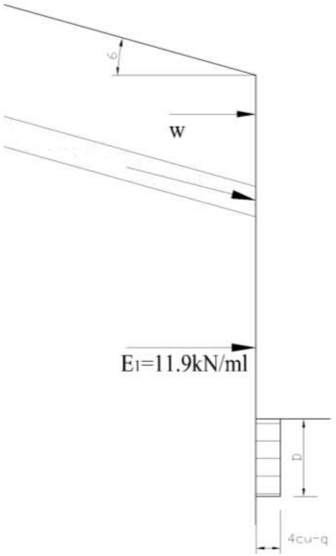

Figure 7.

For the B hypothesis, considering undrained design characteristics covering probable cases (A hypothesis is for medium probability to occur) there are values for fixed support length and maximum moment in the piles lower than existing ones. Maximum displacement value for the sheet pile wall exceeds design probable value considering the pile fixed on base, Δ_{ex}=60mm>>$\Delta_{calculated}$=15.7mm

This phenomenon occur because of the unfavorable influence of the water infiltrated into the ground from the earth cut base(marly clay), leading to the decay of the marly clay and decreasing of shear characteristics ($S_{uu}\cong C_u$).

Maximum value for displacements necessary to mobilize passive pressure is according to SR EN 1997, V_p/h=3.1cm or considering other proposals V_p =47.8mm. Because the pile is an element having satisfied the condition D=αD=3.48>2.5, horizontal earth admitted pressure on the pile side can be calculated like in (1):

$$p_z = \eta_1\eta_2\left(\frac{4}{\cos\varphi}\right)(\gamma \cdot z \cdot tg\varphi + c) = 197.26kPa \qquad (1)$$

at p_{ef}=135kPa, depth z=1.50m.

If the displacement on the superior part of the wall, Δ=60mm, results that at the depth to verify lateral pressure there is a displacement Δ_1=3.81cm, which gives to the ground a pressure (2), Figure 8.

$$p_z = m \cdot z \cdot \Delta \cdot B_p = 135.5kPa \qquad (2)$$

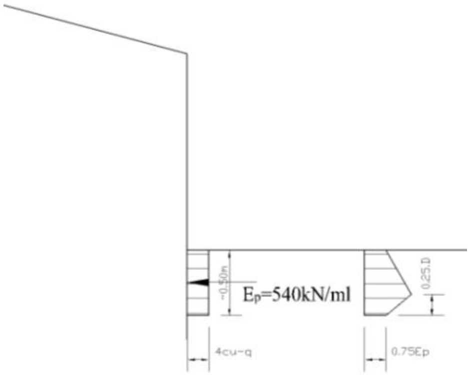

Figure 8.

The wall displacement make the pressure diagram to transform from uniform distributed on fixed support length ($4C_u$=q=108kPa) one, into a triangular one, p_p=174kPa, k_p=1/2k_p≈1.3 an $P_{p=}$130,5 kPa at 1.25m from the bottom.

By the wall displacement passive earth reaction decreased significant toward the calculated displacement, P_p=325kN/ml <540kN/ml, which can produce continuous movement of the wall.

2.4 *Conclusions*

Analyzing the displacement causes of the sheet pile wall, follow conclusions can be relived:

1 Wall displacement for Kaufland platform is due to the water infiltrations produced into the ground on the platform level. The presence of water infiltrations trough permeable saturated layers(silty sand with gravel and rock fragments) situated at 5.05 – 5.55m level, produce water movement to the base layers (marly clay, with sand layers). Those layers are decaying and shear undrained resistance is deceasing (S_{uu}=C_u).

2 Verifications made using the data from geotechnical study for the two hypotheses, considering earth pressure on the wall lead to the conclusion that main strains are given by the plastic landslide of the covering clayey silt layer, on an existing surface located in the base layers. Even considering unfavorable hypothesis sustaining system should ensure good behavior on general slope stability. The fact that general slope stability is not guarantee, by the wall continuous displacement, on constant velocity, confirm the presence of some disconcerting factors favorable to an ultimate limit state of the system(wall inclination).

3 This displacement change the design scheme by decreasing earth passive pressure in front of the wall and the transformation of pressure diagram from a uniform distributed one into a triangular one, reducing the value for passive pressure.

4 Considering the piles as stiff elements (D>2.5), pressure value given to the ground in elastic fixed support area of the pile hasn't reached yet the admitted horizontal pressure value, considering

drained characteristics. This safety coefficient could decrease by the decay of the ground, because water infiltrations trough permeable layer close to the pile.

5 To ensure wall stability there are some urgent solutions to adopt:

- Wall consolidation, by grouted post-stressed anchorages, to ensure the sustaining system stability,
- Elimination of water infiltration by a drainage system located above the wall, which to eliminate the possible exfiltration trough the wall to the earth cut base. Except the possibility to execute continuous draining trenches, can be utilized draining solutions, without open earth cut: siphon drains or electro-pneumatic drains.

3 CONCLUSIONS

One of the most important features about sheet pile wall and pile foundations is the retaining capacity of those structures. The lost of this capacity may be due to loss of information on the design stage or to construction faults whose effects might be amplified in time. Taking into account all this facts and the ones presented above, the students have the opportunity to decide by themselves about the importance of the knowledge of geotechnical characteristics in design stage and the consolidation solutions chosen. This might give them self-confidence and form them the ability to take right decisions like geotechnical engineers.

4 REFERENCES

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