TIRANA- DURRES OLD ROAD LANDSLIDE STABILIZATION

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Abstract

Generally, the most important factor of destabilization of natural slopes is the high level of underground water table. It is significant the fact, that many slopes have failed during the winter, after heavy rainfall. Drainage is often a crucial measure due to the important role played by pore-water pressure in reducing shear strength. Because of its high stabilization efficiency in relation to cost, drainage of surface water by superficial ditches, and underground water by networks of trench drains, is one of the most widely used, and generally the most successful stabilization method. The following study is related to a problematic sliding zone in Shijak-Arapaj area. The landslide in question obtains a specific importance because of its location is closed to the national roadway Tirane - Durres. The landslide has been active for years now. The average presence of a creeping movement of soil masses with a 0.25 m/year velocity. To be able to understand the geological conditions of the area there are drilled four boreholes at 15-20 m depth , there is performed a geological survey and there is installed a plastic pipe system to be able to estimate the movement dynamics of soil masses and to monitor the underground water level. In the article below is described the optimization of a drainage system, to achieve the stabilization of the zone. Disturbed and undisturbed samples are taken for the determination of physical and mechanical properties of the layers encountered in this zone. The geological map and the longitudinal geological profile of the sliding zone are compiled. In this study are determined the active sliding planes. Also are determined the physical and mechanical properties of all the geological layers, the residual parameters in the sliding planes for all types of soils and rocks encountered in this zone. Based on these data, will be carried out the calculation of the stability of the sliding zone. An approximate method for designing trench and counterfort drains has been developed by Hutchinson using finite element analyses and assuming two dimensional steady-state flow. This method gives very good solution in case of limited inclination slope. The method consists in the design of longitudinal deep trench drains, which will be installed in the incline. By means of these drains is enabled the underground water table lowering, and consequently the reduction of pore water pressure. Longitudinal trench drains are executed by means of special machine. Generally the width of the trenches is 0.5-1m and the depth varies 7-8m or deeper if it is possible and feasible.

Keywords: *Slope Stabilitys, Landslide Stabilization, Deep Drainage*.

Introductıon

Landslides are characteristic phenomena in hills and mountainous regions. In Albania, due to the dominance of mountainous relief, this geological and geotechnical phenomenon has a considerable extension. An important landslide can be found near Shijak in the center of Albania. This study is extensively focused on the ways for the stabilization of this zone

Geological and geotechnical study of the sliding zone

This landslide has been active for years. For its stabilization, several times, are carried out engineering proceedings but without any positive evident impact. The chronological development of the sliding is as following: During 2002: we have the presence of a creeping movement of soil masses with a 2m/year velocity. After an intervention, aiming the reduction of the weight of the moving body, the zone gained a temporary stability.
During 2004: the creeping movement of the sliding body reappears with a 0.5m/year velocity. Again there were applied some measures (as the construction of two retaining walls, and the embankment of the road) but without any final satisfying result. From 2004 and constantly we have centimeter movement of the sliding body with a velocity 8- 12cm/year.
For understanding the geological conditions of this zone are drilled four boreholes at 15-20 m depth, is performed a geological survey, is installed a plastic pipe system in order to estimate the movement dynamics of soil masses, and to monitor the underground water level. Disturbed and undisturbed samples are taken for the determination of physical and mechanical properties of the layers encountered in this zone. The geological map and the longitudinal geological profile of the sliding zone are compiled.
a) Site Location, Geomorphologic and Geological Structure.
The sliding zone is located at the km 28+ –29+, at the second part of the project Tirane-Durres Road. It is situated on the left side of the Erzen River flow. The inclination of the valley has different slopes in different levels. These slopes are conditioned by the geological formation. The parts composed of Sandstone are steeper than the parts composed of Mudstone. However, there are some parts almost horizontal, which are created by Erzen River. These represent the old terraces of this river. The incline is composed of Mudstone and Sandstone, which are intensively weathered in surface. These deposits are covered by 5-10 m colluvium deposits. The weathered zone depends on the composition of the rocks. Mudstone beds get weathered more easily while Sandstone ones are more resistible. The weathered zone reaches 4-5.00 m below the colluviums deposits. From the ground surface towards the deepness the quality of the rock gets better. These rocks are interrupted by two discontinuity systems.
One is in the direction of the stratification and the other one perpendicular to the stratification. These discontinuities enable the ground surface water to penetrate deeply and the weathered zone advances deeper. Erzen River Retaining Walls Existing Perimeter.
b) The Causal Factors Leading to the Loss of Slope Equilibrium.
From the study performed in this sliding zone and in some other slight ones encountered in the zone it comes out that they occurred as a result of the following reasons:

i) The deposits faced in the incline are rocks of low resistance against *weathering* phenomena.
ii) The present colluvium deposits, which are composed of silty clay with gravels, when *fully saturated with water*, lose their mechanical properties and start to move taking with them the most weathered part of the beds of Mudstone and Sandstone.

iii) The phenomenon of *erosion by the surface water* is present. During massive rainfalls water streams are created at the inclined surface, which erode parts of colluviums deposits and the most weathered parts of the rocks.

iv) The equilibrium of the slope is affected also from the *erosion that Vjosa River* creates at the toe of the slope. This is one of the main factors that has induced the sliding.

v) *The presence of the underground water during the most part of the year.* As described above there are beds of sandstone and mudstone in the geological composition of this zone. The beds of sandstone have high water permeability. The water enters at the outcropping parts at the highest points of the relief and it comes out as springs at the lowest altitudes. Beds of mudstones are practically impermeable. That's why the water springs come out at the contact between mudstone and sandstone. These waters saturate the masses of colluvium deposits and the most weathered part of the rock formations. As a result the weight of the soil masses is on one hand increased and on the other hand the mechanical properties of soils are decreased. Also the shear strength of the soils is considerably reduced by the pore-water pressure. The fact that the sliding zone is more active during the winter signifies that the stability of the slope is strongly influenced by the underground water table level. From the ''in situ' measurements derives that the fluctuation of the water table level is small. In spring the underground water table level is 1.5m deep, while in September (after the driest period of the year) is 2.5m under the ground level.

Figure 1. Photos from the existing road landslide.

c) Geological, Engineering and Geotechnical Conditions of the Sliding Zone.

In this study are determined the active sliding planes. Also are determined the physical and mechanical properties of all the geological layers, the residual parameters in the sliding planes for all types of soils and rocks encountered in this zone. Based on these data, will be carried out the calculation of the stability of the sliding zone. The sliding plans, identified by the boreholes are marked in the geological profile. The movement of the layer no.2, no.3 and no.4 is more evident and they move with respect to each other; the movement is visible at the ground surface. The movement of layer no.6 and no.7 is slower and it seems as it has reached equilibrium.

Table 1. Physical and Mechanical Properties of the Layers.

layer	Visual description of Layers	Clay Fraction	Silt fraction	Sand fraction	Grain fraction	W_L %	W_P %	W %	γ_0 KN/m^3	γ KN/m^3	Void ration ε	φ	C_r KPa
1	Medium dense beige to grey GRAVEL with sand; the gravel is rounded	-	-	-	-	--	--	-	-	-	-	-	-
2	Firm brown to beige silty CLAY with gravel; the gravel is angular	29.70	42.50	16.90	10.90	41.50	23.20	24.60	2.7	1.9	0.78	15	15
3	Weak grey to beige MUDSTONE with fissures, very weathered	31.60	40.80	14.70	12.90	42.70	23.50	23.50	2.72	1.98	0.76	15	15
4	Firm to stiff brown to beige silty CLAY with sand containing a little gravel, the gravel is angular.	24.70	38.50	25.90	10.90	39.80	23.40	24.80	2.70	1.90	0.78	16	15
5	Firm to stiff brown to beige silty CLAY with sand containing a little gravel, the gravel is angular	21.70	41.70	18.50	18.10	38.80	22.30	24.70	2.71	1.92	0.79	17	25
6	Weak beige to grey SANDSTONE with weak matrix. Very weathered rock	9.40	32.90	47.90	8.80	28.60	23.40	22.40	2.7	2.18	0.7	23	5
7	Weak beige to grey MUDSTONE and SANDSTONE	35.20	42.70	22.10	_	43.60	22.10	14.60	2.73	2.34	0.48	26	165

d) Mechanism of rupture.

Apparently, according to the geological and geotechnical study we are dealing with a translational slide which is characterized by some well determined failure surfaces in the interior of the slide's body, with elasto-palstic behavior. This is an active consequent (follows the bedding planes) landslide. Different layers move downwards and regarding each other. For analyzing the stability of the slope, the residual parameters of shear strength of the soils, will be used.

e) The measures that must be taken for the stabilization of the sliding zone.

Based on the reasons that we analyzed above, the physical and mechanical properties of the layers that are identified in the landslide and on the mechanism of rupture are recommended the following measures

For stabilization of the zone:

- Lowering the underground water level by means of drainage through sub horizontal ditches, which will reach to the most possible depth.
- Removing the surface water from the sliding body by means of a perimeter concrete ditch. Such a ditch already exists, but it is fissured and instead of removing it gathers the atmospheric water and put it in the sliding body in concentrated way.
- Building some river barrages in order to protect the slope from the erosion of Erzen River.
- Adjusting the soils created from different excavations at the road, which in some cases have enabled the reactivation of some parts of this sliding.

- It is recommended to reconsider the existing engineering measures at both retaining walls, which have not been constructed according to technical conditions.

Hutchinson's method for the deep seated instability improvement

This method gives very good solution in case of limited inclination slope. The method consists in the design of longitudinal deep trench drains, which will be installed in the incline. By means of these drains is enabled the underground water table lowering, and consequently the reduction of pore water pressure. Longitudinal trench drains are executed by means of special machine. Generally the width of the trenches is 0.5-1m and the depth varies 7-8m or deeper if it is possible and feasible. Ideally, the drain should penetrate the shear surface (such drains are referred to as "counterfort" drains) and in addition to the improvement in stability as a result of reduced pore water pressure on the slip surface, some additional restraint is achieved by the replacement of the weak slipped material by the stronger material in the drain. The trench is back-filled with suitable free draining material and in its lower part is placed a perforated plastic pipe, which accelerates the process of evacuation of water. Measures to prevent clogging must be incorporated in the design.

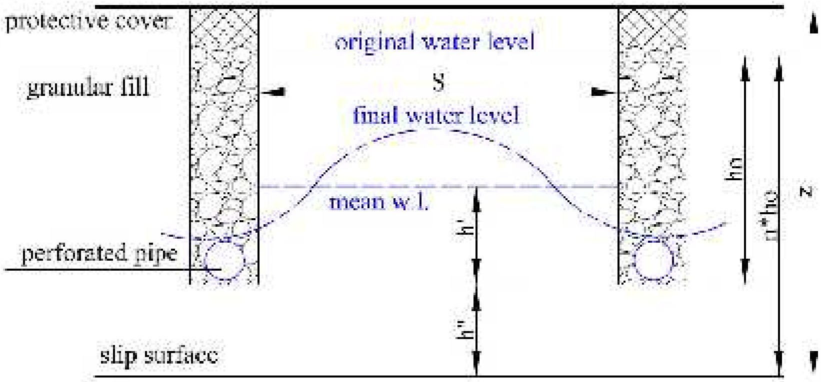

Figure 1. Effect of trenches in the underground water table.

An approximate method for designing trench and counterfort drains has been developed by Hutchinson using finite element analyses and assuming two-dimensional steady-state flow. Hutchinson used the results of the analysis to define the average efficiency of the drains (η) and to relate the efficiency to the ratio s/ho where's' is the spacing of the drains and 'ho' is the depth of the drains beneath the groundwater level (see Fig.2). η is given by:

$$\eta = \frac{h_0 - h'}{h_0} \qquad (3.1)$$

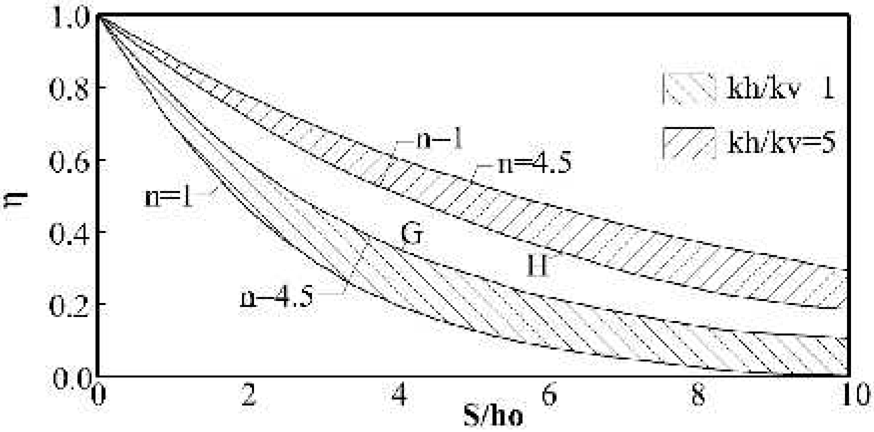

Figure 2. Efficiency of drains.

Is suggested that lines G and H can be taken as reasonable lower and upper bounds for the design of drains. For the purposes of preliminary design, curve G can be regarded as a conservative lower bound. The factor of safety can be assessed by the following formula:

$$F_s = \frac{C' + (\gamma z - \gamma_w h_u)\cos^2 \beta tg\alpha}{\gamma z \sin\beta \cos\alpha} \quad (3.2)$$

If C'=0, then

$$F_s = \left(\frac{\gamma z - \gamma_w h_u}{\gamma z}\right)\frac{tg\varphi'}{tg\beta} \quad (3.3)$$

Where:
C' and ϕ' - are shear strength parameters on the shear surface.
γ - is the bulk density of soil.
γ_W- is the specific weight of water.
β - is the inclination of the failure surface.
h_u - is the height of the water column above the failure surface.
z - is the depth of the failure surface.

These two formulas can be applied for two situations

- With the original water level $h_u^h = n.h_0$
- With the reduced water level (mean w.l.) h_r^u. So we can evaluate the factor of safety before and after the installation of the trench drains in the slope. To determine the reduced water table level we have to define the efficiency of the drains η . This can be done in relation of the ratio "S/h_o" and "$n = \frac{h_u^h}{h_0}$ in the graph at (Fig.2).

$$S_{0,}\eta = f\left(\frac{S}{h_0}, n = \frac{h_u^h}{h_0} = 1\right)$$

So we will use the line H (n=1) for defining the values of efficiency (η), assuming that the slip surface is located just in the base of the trench drain.
With the defined value of the efficiency η we are able to determine the reduced water level above the failure surface:

$$\begin{aligned} &h_u^g\ h' + h'' \\ &h' = h_0\ (1.\ \eta) \qquad (3.4) \\ &h'' = nh_0 = h_0\ \ (n\text{-}\ 1) \end{aligned}$$

And instead of h we put h_u^r in the formula (3.1 and/or 3.2) to obtain the factor of safety after the installation of the trench drains in the slope.

The optımızatıon of the deep trench draıns system

In the longitudinal geological profile are considered some "columns" (C-1, C-2, C-3, C-4, C-5) (see Fig.3) in which will be checked the stresses at the sliding planes that the "column" intersects and the factor of safety will be also calculated. In this way in each "column" will be determined the weakest point (the point with the minimum of factor of safety in high water table level conditions).

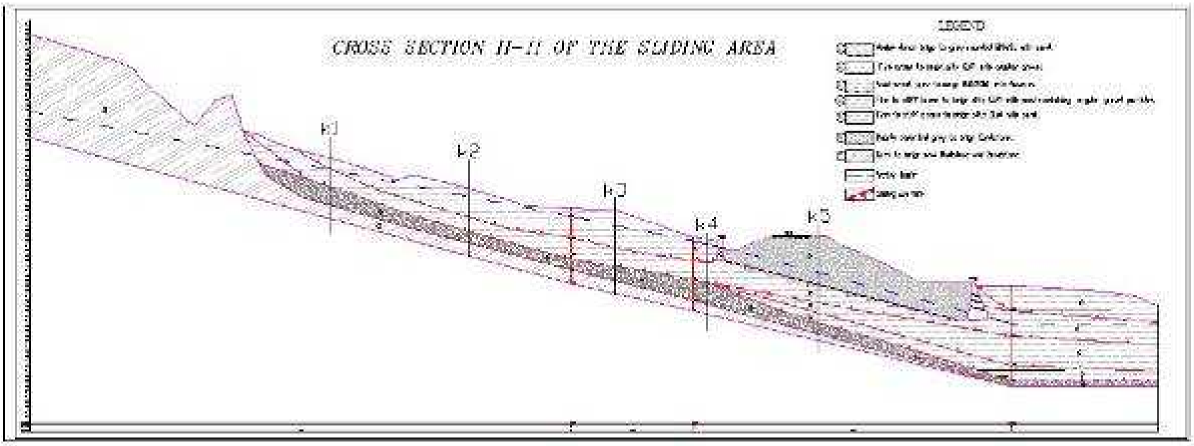

Figure 3. “Columns” for checking the stability.

Based on the weakest point of each “column”, will be assessed the needed depth and distance between the trench drains and also the respective underground water table level reduction, to achieve the required factor of safety. According to the “in situ” monitoring, the underground water level, is 1.5m beneath the ground level.
In each “column” are determined the heights of different geological strata (h_i) and the inclinations (β_i) of the sliding plans intersected by the “column”.

COLUMN 1 : h_1=2,17m $\beta_1=23^o$, h_2=3.48m $\beta_2=15^o$, h_3=3,64m $\beta_3=15^o$

COLUMN 2 : h_1=5,56m $\beta_1=15^o$, h_2=2,83m $\beta_2=15^o$, h_3=2,43m $\beta_3=14^o$

COLUMN 3 : h_1=7,32m $\beta_1=12^o$, h_2=3,73m $\beta_2=10^o$, h_3=3,01m $\beta_3=13^o$

COLUMN 4 : h_1=4,40m $\beta_1=2^o$,h_2=4,10m $\beta_2=10^o$,h_3=0,00m $\beta_3=15^o$, h_4=0,00m $\beta_4=21^o$

COLUMN 5 : h_1=9,90m $\beta_1=14^o$, h_2=3,44m $\beta_2=10^o$, h_3=1,98m $\beta_3=15^o$, h_4=1,92m $\beta_4=16^o$, h_5=2,39m $\beta_5=14^o$

Assessment of geometrical input data.
For calculating of the factor of safety for the point “A” will be applied the formula:

$$F_s = \frac{C'(\gamma_{ave} z - \gamma_w h_u)\cos^2 \beta tg\varphi}{\gamma z \sin\beta \cos\beta}$$

$z=h_1+h_2$ is the depth of point “A”.
h_u=z-1.5m is the depth of point “A” beneath underground water table level.
$\beta=\beta_2$ is the angle of inclination of the second sliding plan where the point ‘A’ belongs to.

$$\gamma_{ave} = \frac{\gamma_1 h_1 - \gamma_2 h_2}{h_1 + h_2}$$ is the average unit weight of the soil above point ‘A’.

All the calculations are presented in the Table 2. In this table are calculated the factors of safety in two cases; 1. in conditions of high underground water table level (1.5m beneath the ground level) ‘F_{S1} and 2. in conditions of absence of underground water in the slope ‘F_{S2}’. (See Table.2) Comparing the two factors of safety comes to light the negative impact of the underground water in the slope. The weakest point of each column is marked with red color. The trench drain system will be optimized based on these weakest points.

Table 2. Assessment of the weakest point.

Column	**Strata**	**C_r**	**φ_r**	**γ**	**h_i**	**γ_{ave}**	**γ_w**	**h_w**	**z**	**β**	**F_{s1}**	**F_{s2}**
1	2	15	15	19	2.17	19.00	9.81	0,67	2.17	23	1.54	1.64
	3	15	15	19.8	3,48	19.49	9.81	4.15	5.65	15	1,18	1.54
	6	5	23	21.8	2.64	20.40	9.81	7.79	9.29	15	1.05	1.69
Column	**Strata**	**C_r**	**φ_r**	**γ**	**h_i**	**γ_{ave}**	**γ_w**	**h_w**	**z**	**β**	**F_{s1}**	**F_{s2}**
2	2	15	15	19	5.56	19.0	9.81	4.06	5.56	15	1.19	1.57
	3	15	15	19,8	2,83	19.27	9.81	6.89	8.39	15	0.95	1.37
	6	5	23	21.8	2.43	19.84	9.81	9.32	10.82	14	1.08	1.80
Column	**Strata**	**C_r**	**φ_r**	**γ**	**h_i**	**γ_{ave}**	**γ_w**	**h_w**	**z**	**β**	**F_{s1}**	**F_{s2}**
3	2	15	15	19	7.32	19.0	9.81	5.82	7.32	12	1.27	1.79
	3	15	15	19,8	3,73	19.27	9.81	9.55	11.05	10	1.26	1.93
	6	5	23	21.8	3.01	19.81	9.81	12.56	14.06	13	1.11	1.92
Colum	**Strata**	**C_r**	**φ_r**	**γ**	**h_i**	**γ_{ave}**	**γ_w**	**h_w**	**z**	**β**	**F_{s1}**	**F_{s2}**
4	2	15	15	19	4.40	19.00	9.81	2.95	4.40	2	10.13	12.76
	3	15	15	19,8	4.10	19.38	9.81	7.06	8.50	10	1.41	2.05
	4	15	16	19	0	19.38	9.81	7.06	8.50	15	0.99	1.43
Column	**Strata**	**C_r**	**φ_r**	**γ**	**h_i**	**γ_{ave}**	**γ_w**	**h_w**	**z**	**β**	**F_{s1}**	**F_{s2}**
	3	15	15	19.8	3.44	20.69	9.81	3.44	13.34	10	1.65	1.84
	4	15	16	19	1,98	20.47	9.81	5.42	15.32	15	1.08	1.26
	5	25	17	19.2	1.92	20.33	9.81	7.34	17.24	16	1.12	1.34

DRAINAGE OPTIMIZATION

Column	Options	Depth of drainage(h)	Width of drainage (b)	Depth of Underground Water (hw)	Depth of drainage from U.W Surface. ho=h-hw	Depth of Sliding plane from ground zero (z)	Depth of sliding plane from max.of U.W (hs=z-hw)	Depth Ratio n=hs/ho	Largesia midis kanaleve gjatesore (S)	Ratio S/ho	Drainage Eficensy η=f(n=1,S/ho)	Drainage Depth from Min.U.W Surface. h'=ho(1-η)	Distance from drainage to Sliding Plane h''=z-h	Distance from Sliding plane to Min.U.W Surface hu=h'+h''	γ_{mes}	γ w	φ m b	C m b	β	Fs
1	1	8	1	1,5	6,5	5,65	4,15	0,64	10	1,54	0,772	1,48	-2,35	0,00	19,49	10	15	15	18	1,288
	2	8	1	1,5	6,5	5,65	4,15	0,64	15	2,31	0,668	2,16	-2,35	0,00	19,49	10	15	15	18	1,288
	3	7	1	1,5	5,5	5,65	4,15	0,75	10	1,82	0,737	1,45	-1,35	0,10	19,49	10	15	15	18	1,281
	4	7	1	1,5	5,5	5,65	4,15	0,75	15	2,73	0,629	2,04	-1,35	0,69	19,49	10	15	15	18	1,236
2	1	8	1	1,5	6,5	8,39	6,89	1,06	10	1,54	0,772	1,48	0,39	1,87	19,27	10	15	15	15	1,255
	2	8	1	1,5	6,5	8,39	6,89	1,06	15	2,31	0,668	2,16	0,39	2,55	19,27	10	15	15	15	1,214
	3	7	1	1,5	5,5	8,39	6,89	1,25	10	1,82	0,737	1,45	1,39	2,84	19,27	10	15	15	15	1,196
	4	7	1	1,5	5,5	8,39	6,89	1,25	15	2,73	0,629	2,04	1,39	3,43	19,27	10	15	15	15	1,159
3	1	8	1	1,5	6,5	14,06	12,56	1,93	10	1,54	0,772	1,48	6,06	7,54	19,81	10	23	5	13	1,423
	2	8	1	1,5	6,5	14,06	12,56	1,93	15	2,31	0,668	2,16	6,06	8,22	19,81	10	23	5	13	1,378
	3	7	1	1,5	5,5	14,06	12,56	2,28	10	1,82	0,737	1,45	7,06	8,51	19,81	10	23	5	13	1,359
4	1	8,5	1	1,5	7	8,5	7	1,00	9	1,29	0,806	1,36	0	1,36	19,38	10	17	25	21	1,184
	2	8,5	1	1,5	7	8,5	7	1,00	10	1,43	0,784	1,51	0	1,51	19,38	10	17	25	21	1,177
	3	8,5	1	1,5	7	8,5	7	1,00	15	2,14	0,691	2,16	0	2,16	19,38	10	17	25	21	1,145
	4	8	1	1,5	6,5	8,5	7	1,08	10	1,54	0,772	1,48	0,5	1,98	19,38	10	17	25	21	1,154
5	1	3,5	1	9,9	3,5	15,32	5,42	1,55	15	4,29	0,48	1,82	1,92	3,74	20,47	10	16	15	15	1,134
	2	4,5	1	9,9	4,5	15,32	5,42	1,20	15	3,33	0,56	1,98	0,92	2,90	20,47	10	16	15	15	1,163
	3	5,4	1	9,9	5,4	15,32	5,42	1,00	15	2,78	0,622	2,04	0,02	2,06	20,47	10	16	15	15	1,191
	4	5,4	1	9,9	5,4	15,32	5,42	1,00	10	1,85	0,731	1,45	0,02	1,47	20,47	10	16	15	15	1,211

The procedure of the optimization is presented in the table nr.3 and the selected alternatives for the deep trench drain system design are in colored background.

Conclusıons and recommandatıons

In the table nr.2 (of the weakest point assessment) we can see that in the upper part of the incline ("column 2") we have sliding activity between the strata nr.3 and nr.6. In the middle part of the slope ("column" 3), the situation is presented better. Here more active is the plan of contact of layers nr.6 and nr.7. In the "column" 4 we have the most critical point of the entire landslide. Here the sliding activity is evident between layers nr.4 and nr.5, nr.5 and nr.6. In the table nr.3 (of the trench drain system optimization) can be seen that in the upper part (“columns” 1, 2, 3) of the landslide are required 8m deep trench drains every 15m, in the “column 4” are required 8.5m deep trench drains every 15m and under the roadway body are required 8m deep trench drains every 15m to achieve the stability of the sliding zone. In “column 5” is assumed that the water level is just in the contact between the road embankment and layer nr.3. At “column 4” we have the presence of the counterfort effect of the trench drain because the body of the drain reaches the sliding surface. The failure was clearly triggered by the high underground water table level. In these cases the dewatering of the ground is an inevitable remedial measure. The method used for improving the stability conditions of the sliding zone is in accordance with its mechanism of rupture. For preventing the clogging of the trench drains, a geocomposite liner will be applied at its slats. Also, for accelerating the process of evacuation of water, a perforated plastic pipe Φ300mm will be placed at the bottom of the trench drain. To gather the atmospheric surface water and to prevent its intrusion in the slide’s body, will be constructed a perimeter reinforced concrete ditch. For further improvement of safety of the zone, an intervention at the existing retaining walls is foreseen.

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