Assessment of embankment stability on organic soils using Eurocode 7

Évaluation de la stabilité des remblais sur sols organiques en utilisant l'Eurocode 7

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ABSTRACT: A problem of the stability assessment of stage-constructed embankment on soft organic subsoil is discussed. Calculations were performed for the first and the third stage of embankment construction at the Antoniny site and for the failure test. The analysis contains measured and corrected shear strength values of organic soils obtained in the virgin and consolidated organic subsoil by staged construction. The statistical analysis of the field vane test results was performed to obtain mean values, standard deviations and probability distribution. Derived, characteristic and design values of undrained shear strength of organic soils were determined. The stability analysis according to design approaches of Eurocode 7 was carried out using mean values of undrained shear strength of organic soils. Additionally, the stability analysis was carried out using undrained shear strength reduced by 0.5 and 1.0 standard deviation. Based on probabilistic stability analyses the safety factor and the reliability index β were calculated.

RÉSUMÉ: Le problème présenté est celui de l'évaluation de la stabilité d'un remblai experimental construit par étapes sur un sous-sol organique. Les calculs ont été effectués pour la première et la troisième étape de la construction du remblai sur le site d'Antoniny et pour l'essai de rupture. L'analyse contient les valeurs mesurées et corrigées des valeurs de résistance au cisaillement des sols organiques obtenus dans le sous-sol vierge et consolidé par les étapes de construction. L'analyse statistique des résultats d'essais sur le terrain a été réalisée pour obtenir des valeurs moyennes, l'écarts-type et la loi statistique de distribution. Les valeurs, caractéristiques de la résistance au cisaillement non drainé des sols organique ont été déterminées pour la conception. L'analyse de stabilité selon les approches de l'Eurocode 7 a été réalisée à l'aide des valeurs moyennes de la résistance au cisaillement des sols organiques. L'analyse de stabilité a été réalisée également en utilisant la résistance au cisaillement réduite de 0.5 et 1.0 écart-type., Les acteurs de sécurité et de fiabilité de l'indice b ont été calculés sur la base de l'analyse probabilistique de la stabilité.

KEYWORDS: organic soils, undrained shear strength, embankment stability, Eurocode 7, characteristic and design values.

1 INTRODUCTION

Geotechnical problems associated with embankments located on organic soils are more difficult than on soft mineral soils mainly due to their higher compressibility and very low virgin shear strength (Hartlen and Wolski 1996). The variability in the value of soil properties is a major contributor to the uncertainty in the stability of an embankment located on soft and highly compressible subsoil. This means that even construction of an embankment of only several meters height may lead to failure. As a result, the methods of design and construction that are normally used in other soft soils, may not be adequate in organic soils.

However, in many countries there are large areas with organic soils where different kinds of embankments have to be constructed. In such cases, the prediction of soil behaviour and the selection of a proper design method become an important as well as a difficult engineering task. In practice, embankment construction by stages utilizing the change in shear strength due to consolidation process is often selected solution. For such a stage constructed embankment, time and money can be saved if the embankment stability is accurately predicted.

Stability analysis according to Eurocode 7 can be performed using following Design Approaches: DA1 (Combination 1), DA1 (Combination 2), DA2 or DA3. Many countries as well as Poland accepted in 2010 Design Approach DA3 for design of slopes. In particular Design Approaches are applied different partial factors to characteristic values of parameters (Frank et al. 2004, Bagdahl 2005, Frank 2007, Bond and Harris 2008, Van Seters and Janses 2011, Orr 2012).

To assess the variability in the value of soil properties the statistical analysis should be performed. Statistical analysis allows to obtain mean values, standard deviations and probability distribution. Thus, characteristic values of geotechnical parameters used in calculations can be obtained.

The characteristic value X_k used in geotechnical calculations corresponding to a 95% confidence level that the actual mean value, X_m is greater than this value, is given by Schneider (1999):

$$X_k = X_m(1 - k_n V) \quad (1)$$

where:
k_n - the factor, depending on the type of statistical distribution and the number of test results,
V - variation coefficient.

Schneider (1999), on the basis of calculations, has shown that a good approximation to X_k is obtained when $k_n = 0.5$; i.e. if the characteristic value is chosen as one half a standard deviation below the mean value, as in the following equation:

$$X_k = X_m - 0.5SD \quad (2)$$

In this paper a problem of the assessment of embankment stability on soft organic subsoil is discussed based on the Antoniny test site. This is an adequate example to perform stability analysis and research of organic subsoil under stage constructed embankment because each case of embankment construction, from loading of virgin subsoil to failure test can be considered.

2 DESCRIPTION OF THE ANTONINY SITE

The Antoniny test site is located in north-western Poland in the Noteć river valley. In 1980s the Department of Geotechnical Engineering of Warsaw University of Life Sciences in co-operation with the Swedish Geotechnical Institute performed extensive field and laboratory investigations.

At this test site the embankment was constructed in three stages on organic subsoil from 1983 to 1987. The structure was then brought to failure by increasing the height of the fill (Wolski et al. 1988, 1989). The schedule of construction is presented in Figure 1.

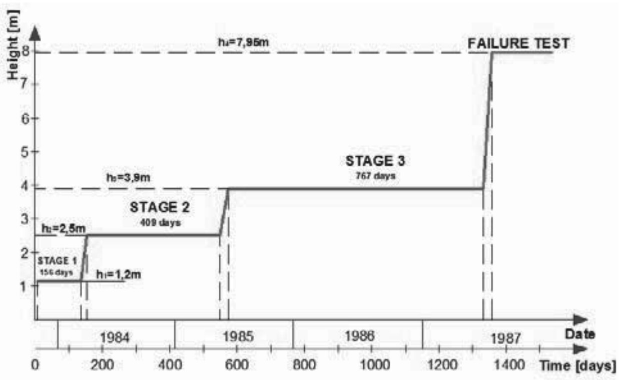

Figure 1. Construction schedule of embankment at the Antoniny site (Wolski et al. 1988).

At the Antoniny site the virgin subsoil consists of an amorphous peat layer and a calcareous-organic soil layer called "gyttja" underlain by a sand layer. The organic subsoil, 7.8 m thick, consists of 3.1 m peat and a 4.7 m gyttja. The results of the index properties of organic soils are summarized in Table 1 and Table 2. Based on the index properties the peat layer was divided into two layers: the first one being fibrous peat from the ground surface to depth 1.0 m and the second one amorphous peat below 1.0 m to depth 3.1 m. The gyttja layer was divided into three layers, the first one being calcareous-organic gyttja extending from 3.1 m to 4.5 m and the second and third layer calcareous gyttja from 4.5 m to 6.8 m and below 6.8 m, respectively. The ground surface is covered with grass vegetation. In the first layer of peat is an abundance of cracks and root channels.

Table 1. Index properties of peat at the Antoniny site (Wolski et al. 1988, 1989).

Properties	Peat	
	Fibrous	Amorphous
Water content w_n [%]	420-450	310-340
Unit density ρ [t/m^3]	1.05-1.1	1.05-1.1
Specific density ρ_s [t/m^3]	1.4	1.45
Liquid limit w_L [%]	-	305-310
Organic content [%]	8-85	65-75
$CaCO_3$ content [%]	5-10	10-15

Table 2. Index properties of gyttja at the Antoniny site (Wolski et al. 1988, 1989).

Properties	Gyttja		
	Calcareous -organic	Calcareous	Calcareous
Water content w_n [%]	130-140	105-110	110-115
Unit density ρ [t/m^3]	1.25-1.30	1.35-1.40	1.40-1.45
Specific density ρ_s [t/m^3]	2.2	2.3	2.4
Liquid limit w_L [%]	100-110	80-90	90-100
Organic content [%]	15-20	8-10	5-7
$CaCO_3$ content [%]	65-75	80-85	85-90

In the virgin subsoil the static ground water level was present in peat layer at the depth of 0.2 m below the ground surface. In the underlying sand at the 7.8 m depth the water pressure is artesian and has been measured to correspond to a water head of 1.0 to 1.5 meters above the ground surface. The high ground water level, combined with the artesian pore water pressure and with relatively low bulk densities of organic soils resulted in the effective vertical stresses in organic soils being only a few kPa and almost constant in depth.

The preconsolidation pressure obtained from oedometer tests is higher than the initial values of effective vertical stresses, which shows that organic soils are overconsolidated with an overconsolidation ratio, OCR, decreasing from 5 to 2 with depth. However, in first stage the effective stress was only smaller than the initial preconsolidation pressure. During staged construction the effective vertical stress exceed the initial preconsolidation pressure several times.

3 UNDRAINED SHEAR STRENGTH

3.1 *Undrained shear strength from field vane tests*

The field vane test is relatively simple and quick in situ method of shear strength measurement of organic soils. In order to evaluate undrained shear strength (τ_{fu}) from vane shear tests, the measured values of shear strength (τ_{fv}) have to be corrected using correction factor (μ).

In this paper the correction factors evaluated according to Swedish Geotechnical Institute method (Larsson et al. 1984) and average correction factors which was determined based on laboratory tests: triaxial compression (TC), triaxial extension (TE) and direct simple shear (DSS) (Lechowicz 1992). Values of correction factors are shown in Table 3.

Table 3. Correction factors for field vane tests obtained at the Antoniny site: $\mu(w_L)$ – correction factor recommended by SGI, μ(lab) – correction factor based on laboratory tests.

Type of soil	$\mu(w_L)$	μ(lab)
Peat (0-3.1 m)	0.50	0.51
Gyttja 1 (3.1-4.5 m)	0.70	0.56
Gyttja 2 and 3 (4,5-7,8 m)	0.80	0.61

Mean values of corrected shear strength were calculated based on early corrected values of measured shear strength. Measured, corrected and mean values of undrained shear strength in the virgin organic subsoil before loading are shown in Figure 2.

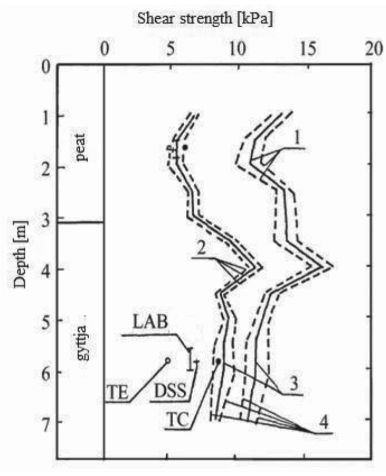

Figure 2. Profile of undrained shear strength based on field vane tests at the Antoniny site: 1 - results of field vane tests, 2 - values corrected according to SGI, 3 - mean values, 4 - mean values ± standard deviation; TC - triaxial compression test, TE - triaxial extension test, DSS - direct simple shear test, LAB - mean value from laboratory tests.

Mean values of corrected shear strength (τ_{fu}) in the virgin organic subsoil before loading and in the consolidated organic subsoil before failure test are presented in Tables 4 and 5.

Table 4. Mean values, standard deviations and variation coefficients of undrained shear strength before loading at the Antoniny site (Lechowicz 1992, Batory 2004).

Layer	Undrained shear strength τ_{fu} [kPa]	Standard deviation SD [kPa]	Variation coefficient V
peat 1	12.8	2.40	0.19
peat 2	6.4	0.71	0.11
gyttja 1	7.7	1.12	0.15
gyttja 2	7.0	0.49	0.07
gyttja 3	7.5	0.92	0.12

Table 5. Mean values, standard deviations and variation coefficients of undrained shear strength before the failure test at the Antoniny site. A - under embankment crest, B - under embankment slope, C - outside of embankment (Lechowicz 1992, Batory 2004).

Zone	Layer	Undrained shear strength τ_{fu} [kPa]	Standard deviation SD [kPa]	Variation coefficient V
	peat 1	28.0	2.40	0.09
	peat 2	24.5	0.71	0.03
A	gyttja 1	20.2	1.56	0.08
	gyttja 2	18.7	1.38	0.07
	gyttja 3	22.3	2.62	0.12
	peat 1	18.0	1.40	0.08
	peat 2	15.7	1.11	0.07
B	gyttja 1	13.2	2.12	0.16
	gyttja 2	12.6	1.84	0.15
	gyttja 3	13.4	1.29	0.10
	peat 1	12.5	2.40	0.19
	peat 2	7.7	0.73	0.09
C	gyttja 1	10.3	1.53	0.15
	gyttja 2	9.9	1.23	0.12
	gyttja 3	10.2	0.91	0.09

Undrained shear strength values obtained by the field vane tests show a considerable increase in undrained shear strength due to the loading and subsequent consolidation. The highest strength increase was measured under the centre of the embankment and the increase was most evident in the peat layer. A smaller increase in undrained shear strength was obtained under the slope of embankment, while the measured shear strength values under the toes of slopes and outside the embankment remained practically unchanged.

3.2 *Statistical analysis of the field vane test results*

The statistical analysis of the field vane test results was performed to obtain mean values, standard deviations and probability distribution. For statistical analysis of shear strength normal distribution was used. Additionally, for comparison also other type of statistical distribution were tested (Lechowicz et al. 2004).

The statistical analysis was carried out using program Statgraphics Plus 4.1 (Batory 2004). In this program goodness-of-fit test is performing by the Kolmogorov-Smirnov test for a 95% confidence level.

The mean values, standard deviations and variation coefficients of corrected shear strength for each geotechnical layer are shown in Tables 4 and 5.

4 STABILITY ANALYSIS

4.1 *Selection of shear zones and design parameters*

In stability analysis performed for the first stage the same parameters for whole geotechnical layers were used. For the third stage and the failure test the organic subsoil was divided into three different shear zones: A - under embankment crest, B - under embankment slope, C - outside of embankment.

Division of organic subsoil into shear zones is presented in Figures 3 and 4.

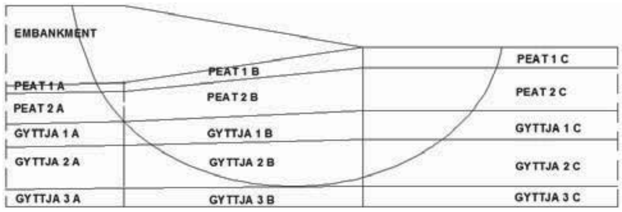

Figure 3. Division of organic subsoil into shear zones for the third stage at the Antoniny site.

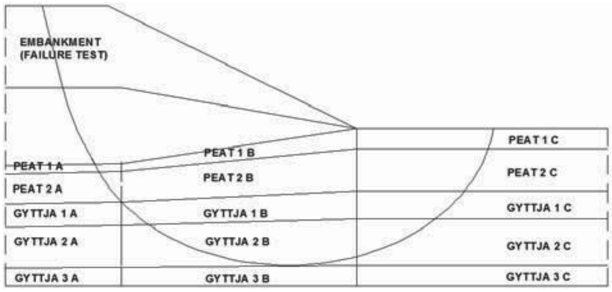

Figure 4. Division of organic subsoil into shear zones for the failure test at the Antoniny site.

In the stability analysis mean values of undrained shear strength were used. The characteristic values of undrained shear strength determined as mean values reduced by half and one standard deviation were also used. In stability analysis design values of undrained shear strength were received by using partial factors (γ_M) to mean and characteristic values.

In Design Approach DA1(C1) partial factor recommended by Eurocode 7 is equal to 1.0 so values of design parameters used in calculations are the same as characteristic ones. In DA1(C2) and DA3 recommended partial factor (γ_M) on undrained shear strength is equal to 1.25, so design parameters used in stability analysis are reduced by that factor.

4.2 *Calculation results*

The stability analysis was carried out for the first and the third stage as well as for the failure test of the embankment at the Antoniny site. The safety factor was calculated using the GeoSlope program with the use of Bishop's simplified method. The calculations were performed according to Eurocode 7 using Design Approaches: DA1(C1), DA1(C2), DA3 and also using probabilistic method. The stability analysis was carried out for subsoil divided into geotechnical layers and zones presented in Figures 3 and 4. Results of stability analysis performed according to Eurocode 7 of embankment at the Antoniny site are shown in Table 6.

Calculations performed for the first stage of embankment at the Antoniny site prove that the embankment was stable. Both for calculations carried out according to Eurocode 7 and using probabilistic method the safety factor was much higher than required. For DA1(C1) safety factor was equal to 1.82 assuming mean values while for DA1(C1) and DA3 was equal to 1.46. For comparison, safety factor using probabilistic method and normal distribution was equal to 1.52 assuming mean values of undrained shear strength. In this case reliability index β = 4.7. Using characteristic values of undrained shear strength evaluated based on Schneider's recommendation the safety factor for DA1(C1) was equal to 1.71, and for DA1(C2) as well as DA3 was equal to 1.39. For characteristic values reduced by one SD the safety factor was equal to 1.60 for DA1(C1) and 1.28 for DA1(C2) as well as DA3. Obtained values are less than permissible for this class of construction.

Table 6. Results of stability analysis performed according to Eurocode 7 at the Antoniny site.

Stage	Mean and characteristic values of undrained shear strength	Safety factor F	
		DA1(C1)	DA1(C2), DA3
1	$\bar{\tau}_{fu}$	1.82	1.46
	$\bar{\tau}_{fu}$ - 0.5SD	1.71	1.39
	$\bar{\tau}_{fu}$ - SD	1.60	1.28
3	$\bar{\tau}_{fu}$	1.41	1.12
	$\bar{\tau}_{fu}$ - 0.5SD	1.33	1.07
	$\bar{\tau}_{fu}$ - SD	1.26	1.01
Failure test	$\bar{\tau}_{fu}$	0.75	0.60
	$\bar{\tau}_{fu}$ - 0.5SD	0.71	0.57
	$\bar{\tau}_{fu}$ - SD	0.67	0.54

Calculation results show that after construction the first stage of embankment with a height of 1.2 m exists a large reserve of safety. In connection with it, for the embankment with a height of 1.7 m additional calculations were carried out using mean values of undrained shear strength. Then, safety factor was equal to 1.16 while β = 3.4. Using logistic type of distribution which better characterized distribution of shear strength the safety factor was close to one and β = 2.2. Performed calculations shows that the first stage of embankment could be constructed higher.

The stability analysis after consolidation of the third stage of embankment performed according to Eurocode 7 (Table 6) and probabilistic method assuming mean values of undrained shear strength shows that construction was stable. The safety factor using probabilistic method was equal to 1.48 and β = 8.9. Taking into consideration partial factors for undrained shear strength (DA1(C2) and DA3) the safety factor was insufficient.

During failure test the thickness of the fill was increased from 3.9 m to a final 7.95. The safety factor determined according to Eurocode 7 was less than permissible (Table 6) both using mean values of undrained shear strength and designed ones. Also calculations using probabilistic method prove that the construction was unstable.

It is important to pointed out the fill constructed in the failure test had approximately the shape of a truncated pyramid. The simplified Bishop's method modified to three-dimensional analysis to estimate the safety factor for the test fill during the failure test was used. Half of the sliding body of the fill was divided into cylindrical part with semi-ellipsoidal end. The three-dimensional calculations of stability for the final stage in the failure test indicate that the factor of safety was close to 1. The two-dimensional calculations carried out for the cross-section of the cylindrical part yielded a safety factor of about 0.75.

Performed stability analysis shows, that in case of Design Approach DA1(C1) the embankment stability in organic soils during construction can be assessed using the characteristic values of undrained shear strength determined according to Schneider's recommendation based on mean values and standard deviation of undrained shear strength derived from field vane tests.

Experience shows that in case of Design Approaches DA1(C2) and DA3 the assessment of embankment on organic soils based on the design values of undrained shear strength evaluated using partial factor γ_M = 1.25, characteristic values of τ_{fu} from Schneider's recommendation, mean values of τ_{fu} determined from undrained shear strength derived after correction of measured values of τ_{fv} introduce too much safety for embankment during construction. The calculation results show that after third stage the safety factor form DA1(C2) and DA3 approaches is closed to one while in real case the embankment height was increased almost double, from 3.9 to 7.95 m in failure test.

5 CONCLUSIONS

The example of the determination of derived, mean, characteristic and design values of undrained shear strength of organic soils obtained from field vane tests for stage-constructed embankment at the Antoniny site was presented.

Performed stability analysis shows, that Design Approach DA1(C1) for stability assessment of embankment on organic soils during construction using the characteristic values of undrained shear strength evaluated based on Schneider's proposal from mean values and standard deviation of undrained shear strength derived from field vane test is recommended.

Experience indicates that Design Approaches DA1(C2) and DA3 introduce too much safety for the embankment on organic soils during construction. In this case the mean values of undrained shear strength evaluated based on corrected values from field vane tests, as characteristic values are recommended.

6 ACKNOWLEDGEMENTS

This research was supported by Grant N N506 218039 from the National Science Centre. Cracow, Poland.

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