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Stability of the Front Embankment of Lake Druzno in Nowe Dolno

Bartłomiej Brzeziński[1], Dominika Iskra-Świercz[2], Andrzej Olchawa[3]*

[1]University of Applied Sciences in Elbląg, Poland

https://orcid.org/0000-0002-0268-0551

[2]University of Applied Sciences in Elbląg, Poland

https://orcid.org/0000-0002-0432-151X

[3]University of Applied Sciences in Elbląg, Poland

https://orcid.org/0000-0002-6211-4375

**corresponding author's e-mail: b.brzezinski@ans-elblag.pl*

Abstract: The article presents calculations of the estimated safety factor F and the overdesign factor, ODF, for the front embankment body of the Lake Druzno reservoir in Nowe Dolno. The rehabilitation of low flood-protection levees in the Elbląg Żuławy region was almost exclusively performed using various local organic soils obtained from riverbeds or borrow ditches. Using soils with drastically different properties precludes homogenizing shear strength parameters within any embankment cross-section. This feature prevents calculating the stability factor (F) using assumed physical and strength parameters with standard analytical methods or numerical software. In situ shear strength testing was conducted on the potential slip surface of the embankment in the village of Nowe Dolno, located within the Lake Druzno basin in the Elbląg Żuławy region, specifically on Polder No. 76. The stability factor (F) was estimated by adapting the Fellenius method to incorporate hydrodynamic forces. The calculated values of F = 1.76 and ODF = 1.18 indicate adequate embankment stability.

Keywords: stability of low flood embankments, Fellenius' method, levee rehabilitation, organic soils

1. Introduction

The Vistula Fenlands form a flat plain elevated slightly above sea level. Also called the Vistula delta plain, the Żuławy region developed about five thousand years ago through the gradual accumulation of river sediments in a sea bay (Augustowski, 1976; Kondracki, 1998). Concurrently, a spit was formed, which over time isolated a portion of the bay, resulting in the creation of the Vistula Lagoon. The depression areas formed within the Żuławy region constitute approximately 27% of its total surface area. A substantial part of the region is its largest depression zone extending across the Lake Druzno basin, i.e., the Elbląg Żuławy.

Lake Druzno represents a remnant of the former Vistula Lagoon. The lake extends about 9 kilometers long, forming a north-to-south arc with a width of up to 1700 meters. It occupies the deepest sections of the depression situated between the Elbląg Upland and the Iława Lake District Upland. The entire lake area, along with its rich aquatic and marsh vegetation, is protected as a nature reserve. The lake bed is elevated higher than the land areas situated behind the levees. The total length of all the embankments surrounding Lake Druzno is approximately 49 kilometers (Fac-Beneda, 2013).

In the depression area of the Elbląg Żuławy, the flood-protection levees function as dams because they permanently retain water. The current water management and drainage system lacks a uniform structure. The flood protection system comprises levees, pumping stations, and channels. Drainage from higher-elevation areas is gravity-driven, whereas excess water from depression and near-depression areas is removed by pumping stations. The extensive drainage system primarily discharges large volumes of water during intense precipitation and high water stages in spring and summer. This need arises from the limited capacity of natural outflow through the main watercourses (Byczkowski, 1999; Cebulak, 2001; Kaniecki, 2004; Majewski & Walczykiewicz, 2010).

The network of ditches and channels in the Żuławy region is extremely dense, with 1 kilometer of natural watercourses and over 10 kilometers of man-made canals and ditches per square kilometer of surface area. The total length of the channels is estimated at over 3,000 kilometers, while the length of lateral ditches reaches tens of thousands of kilometers. The numerous rivers across the Vistula Fenlands territory should also be considered. They exhibit a minimal gradient with a water level close to sea level. The channels of these rivers are mostly canalized and integrated into the drainage system. Their outflows have been artificially regulated, and the majority of these watercourses are enclosed by flood-protection embankments (Majewski & Walczykiewicz, 2010).

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Within the Żuławy region's complex drainage system, the depression areas constitute distinct hydrological units. The drainage ditch network forms a closed system. Polders have been artificially isolated and protected by levees, and their functioning depends entirely on the drainage infrastructure. Nevertheless, despite these supporting facilities, the dams are subjected to supplementary hydrodynamic forces, and their magnitude increases with the rising water retention height. Increased water heads occur annually due to intensive runoff from watercourses into Lake Druzno, which serves as a catchment basin. Additionally, the backwater effect from the Vistula Lagoon, which expands through the Elbląg River into Lake Druzno, tends to pose a particular hazard. In addition, stormwater runoff from the Elbląg Upland areas further contributes to higher water levels. This is why flood-protection embankments are such a critical component of the flood defense system (Cyberski & Mikulski, 1976). The levee bodies in this area are composed of soils characterized by variable organic matter content as well as fluctuating physical and shear strength parameters. The primary parameter for evaluating a levee's safe operational lifespan is its factor of safety, F. Calculating the exact value of F using conventional methods is virtually infeasible. This limitation arises from the high heterogeneity of the soils used during levee rehabilitation, combined with the properties mentioned above. Therefore, F can only be estimated based on in-situ strength testing, embankment geometry, maximum water head, and the shape of the phreatic line of the water seeping through the body (Abramson et al., 2002; Borys & Mosiej, 2008; Chalfen, 2007; Pisarczyk, 2015; Wiłun, 2000).

This paper presents the calculations of the estimated factor of safety, F, for the frontal levee body of Lake Druzno, located in Nowe Dolno within Polder 76.

2. Characteristics of Flood Levees in the Low-lying Areas of the Elbląg Żuławy

Nearly all levees in this region are classified as Class III and IV hydraulic structures. They were constructed prior to 1939 and have undergone multiple rehabilitations since 1945. The levee embankments in the village of Nowe Dolno were constructed using local organic soils, such as alluvial soils, gyttja, and occasionally peat (Olchawa, 2003). The safe operational lifespan of embankments constructed from organic soils is significantly shorter compared to embankments of identical design constructed from mineral soils. The technical operational lifespan of embankments constructed from mineral soils is at least 100 years, whereas embankments made of organic soils maintain their properties for no longer than approximately 30 years. This phenomenon results from degradation of the structural material, driven by mineralization of organic matter and subsequent alterations in soil structure. The rapid degradation of the construction material, which reduces the technical performance of such embankments, requires continuous monitoring and regular stability analyses. An alternative approach in the rehabilitation and modernization of levee bodies involves the application of mineral materials, seepage cutoff walls, and specialized compaction techniques (Borys, 1993; Borys et al., 2002; Borys & Rycharska, 2004; Brzeziński et al., 2023; Cheng et al., 2022; Falaciński et al., 2005; Iravanian & Bilsel, 2016; Mioduszewski, 1991; Myślińska, 2001; Olchawa, 2003; Pisarczyk, 2015).

The soil used in the levee rehabilitation works originated from various locations, resulting in highly variable properties along individual sections. As noted, the spatial heterogeneity of soil properties within the embankment body, combined with their variation over time, precludes applying strength parameters determined via conventional triaxial compression tests. Given this limitation, it must be recognized that under such conditions, only an estimation of the factor of safety, F, is feasible. This value can be calculated based on an assumed slip surface geometry and shear strength of the soil obtained from in-situ testing (e.g., field shear strength tests) (Head, 2006; Lambe & Whitman, 1978; PN-EN 1997-1 Eurocode 7; Olchawa & Borko, 2008-I; Olchawa & Borko, 2008-II; Sobczak, 1975).

3. Selection of the Location for Stability Calculations

The frontal levee of Lake Druzno in the village of Nowe Dolno, spanning chainage 1+000 to 1+800, was modernized and rehabilitated twice over the last two decades and was commissioned for operation after meeting the requirements of the PN-B-12095:1997P standard. The now-defunct Provincial Board of Drainage and Water Facilities granted permission to conduct detailed investigations at a cross-section located at chainage 1+400. Beneath the embankment base, organic soils in the form of sandy muds and clayey muds extend to a depth of up to 12 meters (Olchawa, 2003). At this cross-section, testing was performed to determine the compaction degree and bulk unit weight of the soil within the embankment body, supplemented by field strength testing using a manual light dynamic penetrometer (SD-10 – DPL).

The values of the compaction degree were calculated by comparing the dry density of soil in the embankment with the maximum dry density of the same soils obtained from the conventional Proctor compaction test in accordance with the PN-88/B-04481 standard. The moisture content of the soil samples retrieved from the

levee body indicated that compaction had occurred at a moisture content higher than the optimum moisture content (Pisarczyk, 2015).

Fig. 1. View of the embankment on Polder 76 at cross-section 1+400 km in the village of Nowe Dolno, Markusy Municipality

4. Proposal to Determine the Estimated Value of the Stability Index of Embankment

Using the "Konstruktor" numerical software by Intersoft, the most critical slip surface corresponding to the minimum factor of safety (F = Fmin) was determined. Given the embankment's cross-sectional geometry and the shear strength parameters of its construction material, the factor of safety F can be calculated using the Fellenius method, also known as the Swedish method (Equation 1, Figure 4). This method assumes that the shear strength of the soil along the slip surface depends on the normal stresses acting on that surface. These stresses result from the soil's self-weight. The factor of safety (F) is the ratio of the moments resisting the landslide to the driving moments causing it. This relationship can be expressed by the following formula (Equation 1) (Czyżewski et al., 1973; Fellenius, 1936; Obrycki et al., 2007; Konderla, 2007).

$$F = \frac{\sum(W_i' \cdot cos\alpha_{i)} . tg\varphi' + A_i \cdot c_i'}{\sum W_i' \cdot sin\alpha_i + P_s \cdot r} \quad (1)$$

where:
$P_s = Vg \cdot \gamma_w$ – seepage velocity vector,
Vg – the volume of the water between the failure surface and the groundwater level,
r – moment arm of seepage force,
$W' = \gamma \cdot h_1 + (\gamma_{SR} - \gamma_w) \cdot h_2$,
γ – bulk unit weight of soil,
γ_{sr} – the unit weight of saturated soil,
φ, – the angle of friction in terms of effective stress of the soil in the i -slice,
c – the cohesion o in terms of effective stress of the soil in the i -slice,
α – angle of slope (see Fig. 4).

On the left side of the vertical axis, the value of a is positive; on the right, it is negative (See Fig. 3).

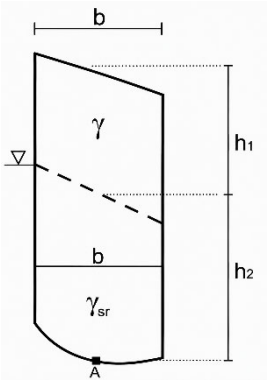

Fig. 2. Scheme for the calculation of the bulk unit weight of soil (the black square represents the location where the field shear strength test was performed)

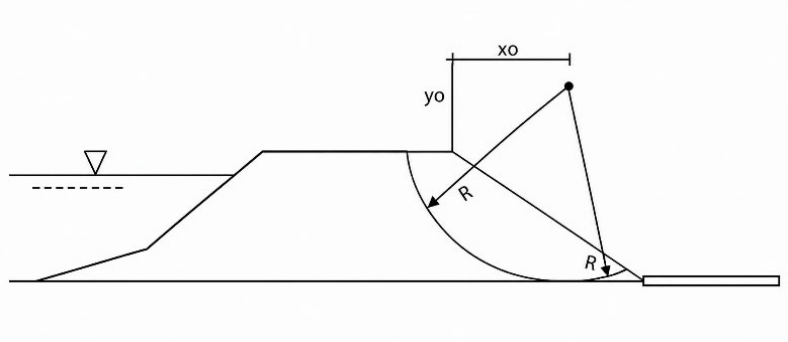

Fig. 3. The cylindrical sliding surface that was generated using numerical software

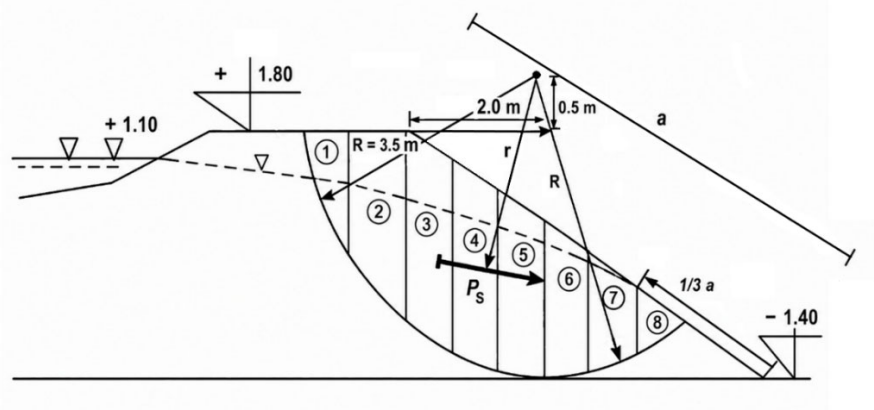

Fig. 4. Scheme for calculating the stability of an embankment, taking hydrodynamic forces into account

The product of the normal stresses and the base area of the slice determines the normal force, which represents the component perpendicular to the slip surface originating from the slice self-weight, W_i. In this analysis, the interactions between adjacent slices are neglected. The driving moment inducing the displacement of the soil mass section can be represented as the sum of the products of the weights W_i and their corresponding arms, determined relative to the center of the cylindrical surface. As noted earlier, the magnitude of the normal force acting on the slip surface is determined without accounting for lateral interactions between slices. In reality, interactions between individual slices influence both the distribution of normal forces along the slip surface and the shear resistance generated by the displacing soil mass. However, the method under consideration omits these inter-slice effects.

The Swedish method, also referred to in the literature as the Fellenius method, assumes that each slice can be evaluated independently, without considering interactions between them. The number of designated slices and their respective widths can be chosen arbitrarily. However, an insufficient number of slices reduces the accuracy of the results. In engineering practice, two primary categorization methods are most commonly used. The first approach assumes the same slice width, which simplifies calculating soil weights, the values of angles α, and the distances from the center of the cylindrical surface.

The second approach assumes dividing the entire semicircle of radius R into a predetermined number of n slices. Most frequently n = 20, with the width of each slice equal to $\frac{2R}{n}$. This configuration ensures identical values of the angles α across all analyzed cases, regardless of the location of the slip surface center or the value of the radius R. At the same time, it reduces the time required to determine the sinα and cosα functions. It should also be emphasized that this method allows for the inclusion of the influence of seepage forces on slope stability without significant degradation in the accuracy of the final results (Abramson et al., 2002; Cheng & Lau, 2014; Duncan & Wright, 2005; Morgenstern & Price, 1965).

Vertical VST of τ_u = Su were performed at each depth interval, with spacing between measurement points being b = R/8. The field vane shear test was used to measure the undrained shear strength Su. The Su measurements were taken at each vertical slice. Given this method of calculation, formula (1) simplifies to the form:

$$F = \frac{\sum A_i \cdot Sui}{\sum W' \cdot sin\alpha_i + \frac{r \cdot P_S}{R}} \tag{2}$$

where:
A_i – length of the surface in the i-th strip,
Sui – designed shear strength in non-drained conditions,
$W' = \gamma \cdot h_1 + (\gamma_{sat} - \gamma_w) \cdot h_2$,
α – the inclination angle,
P_s – filtration velocity vector,
r – the filtration velocity vector arm,
R – radius of cylindrical slip surface.

4.1. Application of the Method in Practical Calculations

The first step is to generate a potential slip surface based on the geometry of the embankment body, as well as the assumed strength parameters and bulk unit weights. Divide the distance between the upper and lower edges of the surface into 8–10 slices. In the middle of each slice, at the depth of the generated slip surface, vane shear tests (VST) are to be performed to determine the τu values. Shear strength values of Su = τu · μ, where μ ≈ 0.8, are to be adopted for the calculations. The subsequent procedure applies formula 2 and the geometric data shown in Figure 4 and Table 1. The phreatic line can be determined from the water level in control wells or using generally accepted methods that assume the phreatic line exits at one-third of the slope length measured from the toe.

5. The Results of the Shear Test

In-situ shear strength testing was performed by rotating a DPL 10 vane probe in the middle of each slice at the depth of the potential slip surface. The measurement was carried out in accordance with the device manufacturer's instructions. A rectangular vane with a diameter of 55 mm and a height of 110 mm was used. The undrained shear strength, τf, is calculated from the torque resistance along the horizontal ends and vertical sides based on the following equation:

$$\tau_u = \frac{M_{max}}{\frac{\pi D^2 H}{2} + \frac{\pi D^3}{6}} \tag{3}$$

where:
τ_u – the undrained shear strength [kPa],
M_{max} – maximum torque [Nm],
D – diameter of the blade [m],
H – height of the blade [m].

The correction factor μ for the S analysis was adopted from Lechowicz and Szymański (2002). The values of the factor μ were estimated based on a comparison between the undrained shear strength measured in a triaxial test and the undrained shear strength measured using a vane shear test. The estimated average value of μ is 0.72.

6. Estimating the Stability of the Embankment

Table 1 summarizes the data used to calculate the driving moment (causing the landslide). Table 2 summarizes the values used to calculate the resisting (retaining) moment.

Table 1. Stability analysis results for the embankment at chainage 1+400 km in Nowe Dolno, Markusy municipality

No*	h_1	h_2	γ_{sat}	γ	$\gamma'=\gamma_{sat}-\gamma_w$	$\gamma \cdot h_1$	$\gamma' \cdot h_2$	b	W'	sinα	$h_2 \cdot b \cdot \gamma_w$	$W' \cdot \sin\alpha$
	[m]	[m]	[kN/m^3]	[kN/m^3]	[kN/m^3]	[kPa]	[kPa]	[m]	[kN]	[1]	[kN]	[kN]
1	0.6	0.9	19	18.6	9.19	11.16	8.27	0.5	19.43	0.97	4.41	18.85
2	1	1.5	18.9	18.7	9.09	18.70	13.64	0.85	32.34	0.74	14.72	23.93
3	0.9	2	19	18.6	9.19	16.74	18.38	0.7	35.12	0.48	13.73	16.86
4	0.6	2	18.6	18.2	8.79	10.92	17.58	0.75	28.50	0.31	14.72	8.84
5	0.4	2	18.4	18	8.59	7.2	17.18	0.75	24.38	0.11	14.72	2.68
6	0.2	1.9	18	17.8	8.19	3.56	15.56	0.7	19.12	-0.11	13.05	-2.10
7	0	1.7	17.9	17.5	8.09	0	13.75	0.8	13.75	-0.31	13.34	-4.26
8	0	0.7	17.7	17.1	7.89	0	5.52	0.5	5.52	-0.45	3.43	-2.49
											∑92.12	**∑62.31**

*Number of the slice

Table 2. Summary of data up to the moment of counteracting the slip (holding) in the cross-section of embankment 1+400 km in the town of Nowe Dolno, Markusy commune

No	A_i	$Su = \mu \cdot \tau_u$	Ai·Sui
	[m^2]	[kPa]	[kN]
1	1.68	48.02	80.67
2	0.86	41.01	35.27
3	1.03	36.04	37.12
4	0.87	33.89	29.48
5	0.78	29.37	22.91
6	0.85	25.23	21.45
7	0.85	18.09	15.38
8	0.9	12.03	10.83
∑			**253.11**

Calculation of the stability index value.

$$F = \frac{\sum A_i \cdot Sui}{\sum W' \cdot \ sin\alpha_i + \frac{r \cdot P_s}{R}} = \frac{253.11}{62.31 + \frac{3.1 \cdot 92.12}{3.5}} = 1.76$$

Stability calculation according to Eurocode 7, approach 2 (PO 2).
Combination: A1 + M1 + R2

Table 3. Partial coefficients for geotechnical parameters and earth resistance (for slope)

γ_G	$\gamma_{\varphi'}$, $\gamma_{c'}$, γ_{cu}	$\gamma_{R;e}$
1.35	1.0	1.1

Over–Design Factor calculated according to:

$$ODF = \frac{F}{\gamma_G \cdot \gamma_{R;e}} = \frac{1.76}{1.35 \cdot 1{,}1} = 1.18 \quad (4)$$

where:
F – stability indicator,

γ_G – partial coefficient permanent action (unfavorable),
$\gamma_{R;e}$ – partial coefficient of earth resistance.

7. Conclusion

Based on the calculations and analyses performed, the following conclusions can be drawn:

1. Despite using local organic soils for modernization and rehabilitation, the estimated embankment factor of safety (F) and the Over-Design Factor (ODF) indicate safe operation of the structure.
2. The calculated ODF of 1.19 indicates that the safety requirement is met with slight overdesign.
3. The proposed method can be used to assess the ongoing stability of flood embankments, especially those approaching the end of their design life.
4. High factors of safety indicate that organic soils can be used to construct flood embankments classified as class III and IV hydrotechnical structures. However, meeting the required design criteria demands appropriate engineering techniques. The proposed method is the only applicable approach for stability analysis of embankment bodies characterized by such highly varied soil properties.
5. However, it should be noted that organic soils in embankments degrade relatively quickly, and the factors of safety decrease during operation.
6. The fundamental disadvantage of the method is the procedure for generating the potential slip surface, as it is always necessary to input the shear strength for the entire cross-section into the numerical software. However, these values are only determined in the subsequent step, namely during field testing.

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