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INDUSTRIAL WASTE AS AN EFFECTIVE MATERIAL FOR DAM CONSTRUCTION

1. Geotechnical criteria for tailings utilization in dam building and in building of other hydrodevelopment facilities

To be able to completely utilize the full supply of tailings in hydrodevelopment constructions it is necessary to know the tailings‘ specific properties perfectly well. Only after that the dams and other facilities can be designed without any danger or risk [5, 8, 11]. The problem is that only a few representative physico-mechanical values are known and these are not statistically processed enough for application of the borderline state design method. Statistical data authentication is absolutely essential when designing for example high embankments, dams etc. Unfortunately, the main problem is connected with the specification of the physico-mechanical properties of the tailings supply. Due to extensive size of grains and petrographical and strength in homogeneity of particular fractions, it is not possible to use the method of model grain size distribution curve. The results of consistency and compressibility tests even after model similarity adjustment are not congruent with the tailings values. As for tailings, model similarity in compressibility tests was not proved. The sample was consisted of graded tailings fractions (0.4 mm, 0.32 mm) and fraction mixture (5−16 mm). The best model similarity to genuine tailings is approximately 1.2 with natural aquosity of about 6.0–7.2% [3, 8].

Some of the experts state that our laboratory apparatuses for determination of tailings strength when using classified fine tailings fraction did not prove its worth even if the similarity model of the curve shape was preserved. Final results were similar in quality but they only described some proceeding processes in tailings during the activity of sliding stress. To gain more exact laboratory findings it is inevitable to use the Imperial College London type sliding joint, constructed right after Alberfan accident. For the designing of constructions and frames built of these materials, deep knowledge of the tailings mechanics is vital.

It is also essential to realize that some of the tailings frames or constructions (mud pit embankments, special purpose embankments etc.) are going to be built in the undermined

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areas therefore they have to satisfy all the requirements of ČSA 73 0039 "Utility designing in undermined areas". In the Ostrava Region a lot of breakdowns of these constructions took place due to not observing the instructions. The input mechanical values must be standardized according to Field Engineering values as required by The Coal Board.

2. Embankments of coal processing plants mud pits

Mud pit embankments and dams belong to hydrodevelopment facilities that are generally designed as homogeneous earth dams with the earth of different grain size. According to the function and the method of slurry deposition, mud pits can be divided into:

— Permanent mud pits with continuous slurry deposition that means that the stored slurry is not exploited and the mud pit is decontaminated immediately after filling.
— Cyclic mud pits with slurry deposited temporarily which means that it forms part of a cyclic mud pits system. Its operation phases are: hydraulic filling, sludge dewatering and exploitation.
— Securing dam pits serving as a backup for the central cleaning system, for example in case of the central mud pit deactivation because of subsidence, sewage purification etc.
— Intercepting dam pits; a reservoir for water collection, water quality adjustment by recirculation [12], possibly for a tertiary treatment before the reservoir drawdown into a receiving stream.

A section through a mud pit is shown in Figure 1.

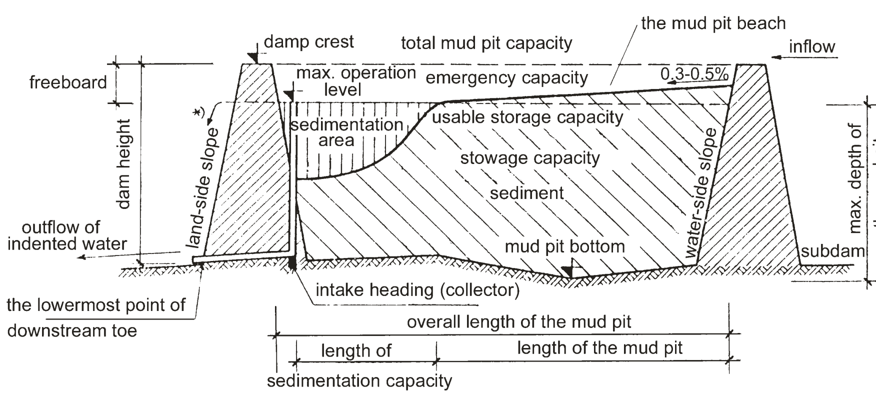

Fig. 1. Drawing describing a mud pit of a coal processing plant [12].
Maximum mud pit depth — vertical distance between the lowermost point of the mud pit bottom and the maximum operation level (local ditches excluded). Dam height — vertical distance between the dam crest and the lowermost point of the downstream toe*) crest overall heading that can complement or even substitute the intake heading

Mud pit dams are designed as homogeneous constructions made of mine spoils with various grain size distributions or, they are optionally made of loose ash filling. When designing waterside slope, sealing of the mud pit dams with spontaneous slurry infiltration was considered.

The dam frame has to satisfy the following demands:

- filtration stability with secure and controllable flow drainage,
- static and deformation stability including the stability of the lying wall,
- security against dam infusion and dam fracture,
- the intended life span in consideration of mud pit dam utilization.

As listed above, tailings comprise rocks of unequal apparent density of solid particles. This density depends on proportion of carbon rocks. Bulk density depends on compactness $\gamma = 1.55–2.05\ Mg{\cdot}m^{-3}$. Relative bulkage coefficient of opened up tailings is $K = 1.25–1.32$, when exploited from the mine dump it is only $K = 1.04–1.15$. The coefficient of loose tailings rock fluctuates between $1.10^{-3}\ m{\cdot}s^{-1}$ and $1.10^{-7}\ m{\cdot}s^{-1}$ (compact tailings). It depends on the tailings petrographical structure, grain size distribution and age. Angle of internal friction is min. φ = 36%. Physico-mechanical properties of tailings rock are listed in Table 1 and Figures 2 and 3.

TABLE 1

Target physico-mechanical properties of tailings

Target value	Tailings		
	mine spoils up to 100 mm[1)]	mine tailings up to 250 mm[2)]	up to 150 mm with the smooth[3)]
Apparent density γ_n [$Mg{\cdot}m^{-3}$]	1.55	1.60	1.63
Natural aquosity W [%]	5.0	5.0	5.0
Apparent density of dry tailings γ_d [$Mg{\cdot}m^{-3}$]	1.48	1.57	1.55
Particle-size distribution [mm]	0.05–100	0.002–250	0.002–150
Relative bulkage (tailings rock from soil dump) k_n max	<1.26	<1.24	<1.24
Relative bulkage (tailing rock from the spoilbank) k'_n	1.07	1.04	1.06
Apparent density after compaction γ'_d [$Mg{\cdot}m^{-3}$][*)]	1.86	1.95	1.92
Angle of internal friction Φ'	36–28°	54–36°	38–32°
Consistency (effective) c' [MPa]	0.00–0.01	0.00	0.01
Poisson's ratio	0.20	0.20	0.20
Modulus of deformation after compaction E_{def} [MPa][**)]	>45.0	>50.0	>45.0

[*)] Natural aquosity $W \rightarrow 5\%$.
[**)] Average attainable value in relation to apparent density of tailing rock γ'_d after compaction.
[1)] Tailings rock from a JIG processing plant.
[2)] Tailings rock predominantly from fieldwork.
[3)] Tailings rock from preparation partially treated (grain 0.002–150 mm) or tailings rock from a slime processing plant (grain 0.5–max. 200 mm).

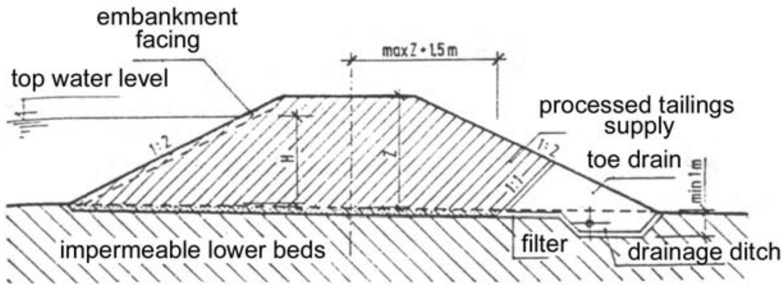

Fig. 2. Homogenous dam on impermeable beds upstream dam sealing

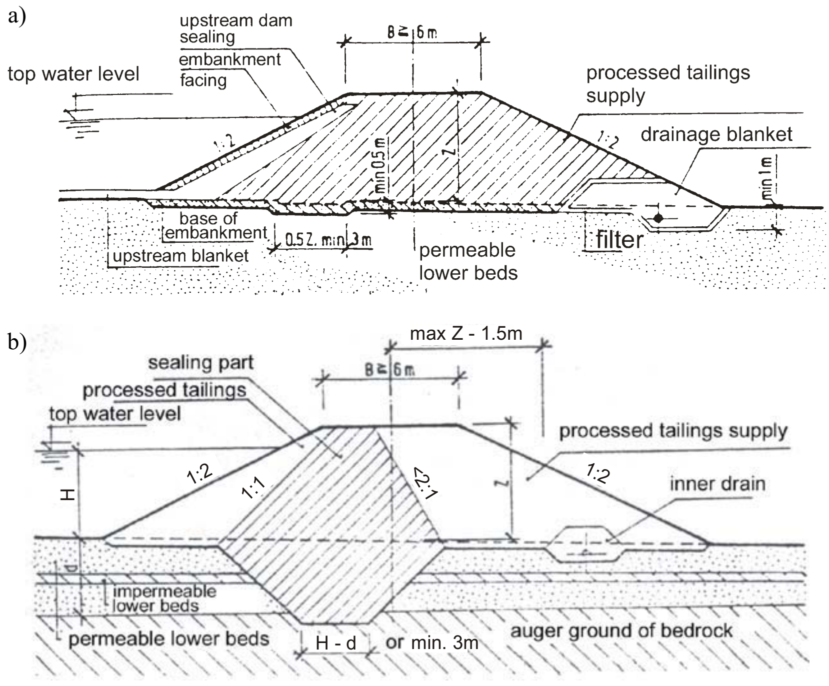

Fig. 3. Non-homogenous dam on:
a) permeable lower beds (foundation alteration); b) permeable lower beds

3. Fill dams in the undermined areas and their influence on foundation soil deformation

Primary assumption, when designing and constructing dams are in undermined areas, is proper knowledge of surface consequences of mining operations. The authors of this paper have done a proper analysis of the upper layer's normal and shear strain in Karviná-Orlová Region. In this region, mining intensity was substantial. Because of the technical reasons normal and shear strain of the upper layer has been analysed in dam crest Křivý důl only.

In Table 2 the extreme values of total positive (tension) and negative (compaction) deformation of dam crest are depicted. It was being monitored during 5 years [11]. In this figure also extreme values of the dam crest normal deformations during the first month are listed. These values characterize the kinetics of normal deformations in undermined areas.

Extreme values of total shear strain of the embankment terrain upper layer (inclinations) during the observation period describe shear strain kinetics. The values are introduced in Table 3.

TABLE 2

Extreme values of the dam crest total transformation in the course of monitoring

For period	Type of normal transformation	Value of transformation
For the entire period [mm/m]	+ tensile stress	
	max. 1	2.30
	max. 2	2.29
	max. 3	1.40
Per month [mm/m]	max. 1	1.30
	max. 2	1.00
	max. 3	0.75
For the entire period [mm/m]	− compression stress	
	max. 1	2.40
	max. 2	2.30
	max. 3	2.20
Per month [mm/m]	max. 1	1.20
	max. 2	0.80
	max. 3	0.75

TABLE 3

Extreme values of total top layer shear strain and dam total shear strain in the course of observation

Terrain surface inclination (top layer shear strain) measuring in situ		Dam crest	Terrain in "Stonava" location		
			polygon 46–75	polygon 1–24	polygon 91–98
For the entire period [mm/m]	max. 1	30.00	31.67	32.00	33.00
	max. 2	25.20	28.00	28.33	25.00
	max. 3	24.50	25.67	24.00	18.00
For one month [mm/m]	max. 1	6.20	4.67	10.33	3.67
	max. 2	4.00	3.67	6.60	2.60
	max. 3	2.30	2.30	6.33	2.33

The influence of undermining on tailings dam deformation, strain and load capacity is possible to examine with finite elements computational method. When applying this method, computer programs operating with the material constitutive models help to compute the values. These models describe soil mechanical behaviour, e.g. a relation between strain and deformation, in a complicated case a relation between strain and deformation dependent on time. Moreover, they describe accepted hypothesis of failure mode during the top limit load. Utilization of constitutive models takes into account strain development (strain history) in particular items of the examined constructions and its environment. After this manner it enables to observe plasticity incidence of certain areas.

In this case [11], the mathematical model of soil was able to comprehend 12 types of strain development as well as 3 types of fracture. This model is an accretion non-linear model with plastic effects. Calculated stepped lagging modified different pseudoplastic parameters of deformation E_t and μ_t.

The type and state of stress (compaction or tension, exacerbation or relief) and the type of fracture (by sliding or by tension) of particular elements has been taken into consideration as well.

On the basis of analysis of actual terrain surface mobility (due to undermining) the terrain surface type alternations have been elaborated. Dam computational models were loaded with these types of alternations. Consequences of this undermined terrain surface movements are illustrated in Figures 4 and 5.

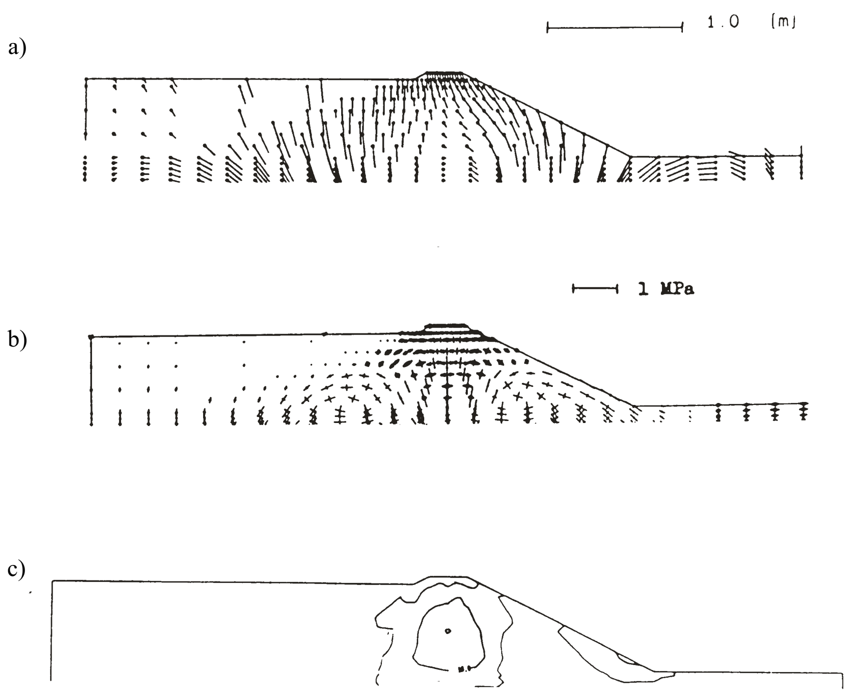

Fig. 4. Tailings dam built on medium yielding lower beds (*Eo* = 10 MPa).
Due to undermining extensive outer downthrows of dam footing bottom appeared:
a) vector of shift; b) axes of stress ellipses; c) isolines of maximum shear stress (unit — 10 KPa)

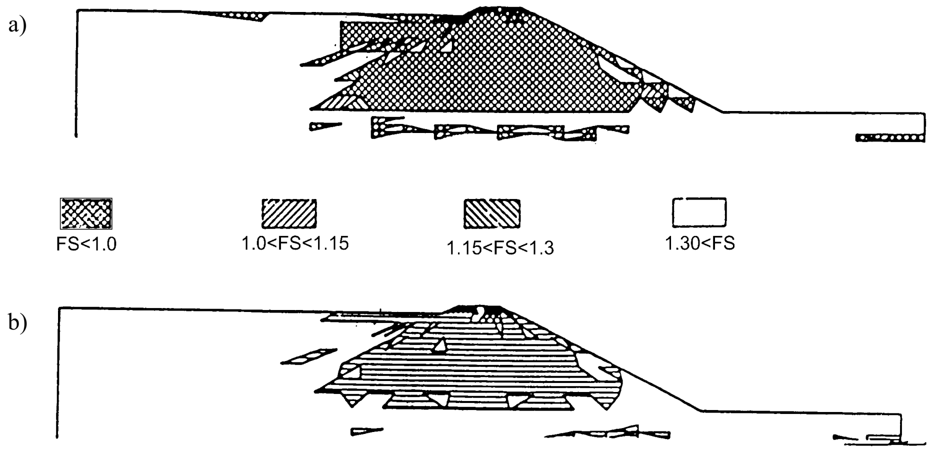

Fig. 5. Tailings dam built on medium yielding lower beds (*Eo* = 10 MPa).
Due to undermining extensive outer downthrows of dam footing bottom appeared:
a) level of shear strength mobilization; b) localities of tensile strain

The analyses and the figures listed above implicate that the tailings dams and embankments on certain conditions can resist successfully the footing bottom downthrow when it is:

— a rectangular downthrow,

— a central downthrow,

— a small-scale outer downthrow.

It can resist the dam footing bottom downthrow as well provided that it is one of the following types of downthrow: an extensive outer downthrow, a central heaving and an extreme outer downthrow and a central slide.

It has also been proved that tailings dams and embankments in undermined areas can resist dam footing bottom movements — for example one-sided inclination and a downthrow in the middle of the dam. On the contrary, dams and embankments are very perceptive of lower bed movements such as toe downthrows or shear loading in the middle of the dam.

Tailings fill embankments hardly resist tension strain of material caused by these types of lower beds' movements [11].

If tension strain appears in an undermined area, it is recommended to reinforce the upper layer of the foundation soil, optionally to reinforce the dam or the embankment frame with a geonetting, a geograting or with geocells.

4. Tension in critical sliding surfaces of dams and embankments

The process of shear stress τ_1 along the tailings dam's sliding surface is depicted in Figure 6. There is a descent stability method according to Peterson applied and the finite elements method too provided that the lying wall of the dam is in an idle condition.

The tailings dam stability, e.g. the borderline stability situation assessment in undermined areas was specified by the following methods:

— computing method of slices according to Peterson (circular sliding surfaces), not considering the influence of lateral forces affecting the walls of the slice;
— method of slices according to Bishop (circular sliding surface) taking into account the influence of lateral forces affecting the walls of the slice;
— method of slices according to Woldt taking account of the influence of lateral forces affecting the walls of the slice;
— computing method of finite elements enabling to ascertain the actual state of stress along the whole examined sliding surface.

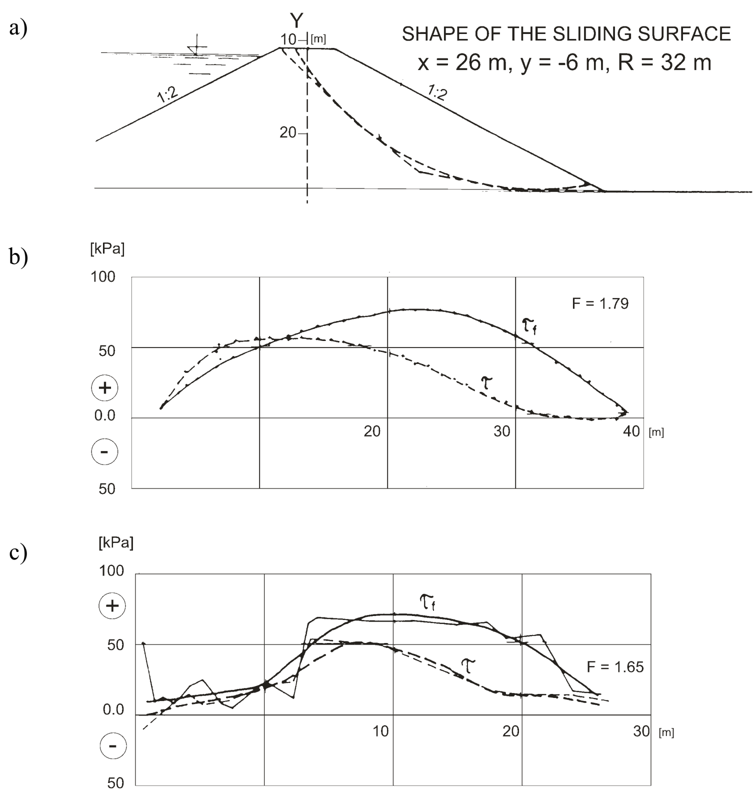

Fig. 6. The comparison of the tailings dam stability computing methods:
a) tailings dam; b) distribution of shear stress (τ) and of limit shear stress (τ_1) along the sliding surface for the stability analysis according to Petersen's method (without lower beds movement);
c) distribution of shear stress according to the method of finite elements (without lower beds movement)

The influence of undermining on dam stability and the results from all the methods of slices can be modified only by alternation of input geotechnical parameters of the dam lying beds (Tab. 4).

The finite elements method can (to a certain limit of the lying beds' deformation) specify the consequences of the lying beds' movements on the state of stress along the whole examined sliding surface. General disadvantage of this method is high demand on computing technology and time consumption.

As mentioned previously, strong bracing synthetic textiles especially in the under-mined areas, increase the dam face stability. World top bracing textiles have tensile strength of 800 $KN{\cdot}m^{-1}$. Japanese experiments with these bracing textiles in seismic areas were so successful that bracing of tailings dams with the textiles seems to be meaningful even in undermined areas.

TABLE 4

Alternative possibilities of strain state and strain development and the development of material defects

$\sigma_1 \leq 0$; $\sigma_3 \leq 0$	stress, isotropy, exacerbation
$\sigma_1 > 0$; $\sigma_3 \leq 0$	stress + current, anisotropy
$\sigma_1 > 0$; $\sigma_3 > 0$	current, isotropy, exacerbation
$\sigma_1 < 0$; $\sigma_3 < 0$	$\mathrm{abs}\left(\sigma_{oct}^{(n)}\right) < \mathrm{abs}\left(\sigma_{oct}^{(n-1)}\right)$ stress, isotropy, discharge
$\sigma_1 > 0$; $\sigma_3 \leq 0$	$\mathrm{abs}\left(\sigma_{oct}^{(n)}\right) < \mathrm{abs}\left(\sigma_{oct}^{(n-1)}\right)$ stress + current, anisotropy, discharge
$\sigma_1 > 0$; $\sigma_3 > 0$ $\Delta_s < 0$	$\mathrm{abs}\left(\sigma_{oct}^{(n)}\right) < \mathrm{abs}\left(\sigma_{oct}^{(n-1)}\right)$ current, isotropy, discharge
$\sigma_1 < 0$; $\sigma_3 \leq 0$	fracture by shear, stress
$\sigma_1 > 0$; $\sigma_3 \leq 0$	fracture by shear, by current, stress + current
$\sigma_1 > 0$; $\sigma_3 > 0$	fracture by current, current

Figure 7 shows an elementary state of lower beds of tailings dam and Figure 8 demonstrates the process of shear stress distribution τ and τ_1 according to the computing method named above.

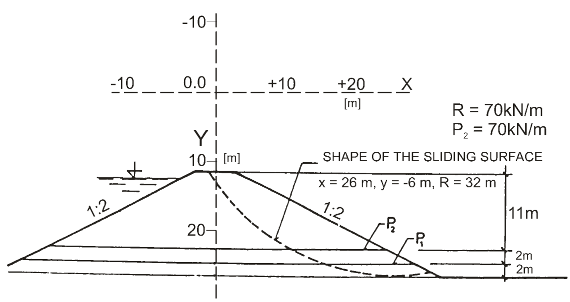

Fig. 7. Tailings dam. Elementary state of lower beds

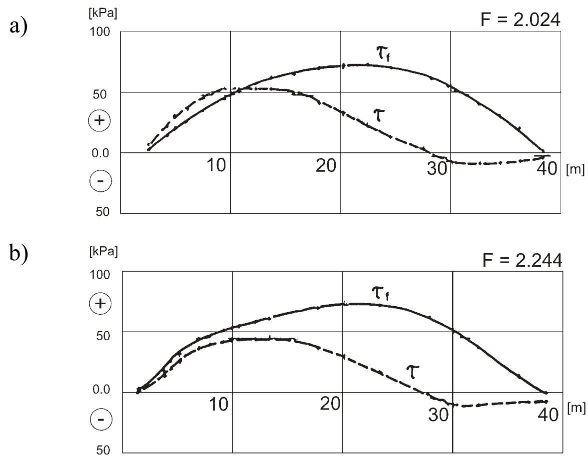

Fig. 8. Tailings dam. Distribution of shear stress τ and τ_1 according to:
a) Petersen, b) Woldt

5. Summary

The possibility of utilization of tailings in dam construction appears to be real. The above is said even if very scant results verify this idea. In cases where higher load bearing capacity and stability of dams would be required, this publication might help in developing new specific construction procedures for stabilization and hardening of this material (e.g. with comminuted blast-furnace slag, lime, cement or with the use of bracing synthetic textiles).

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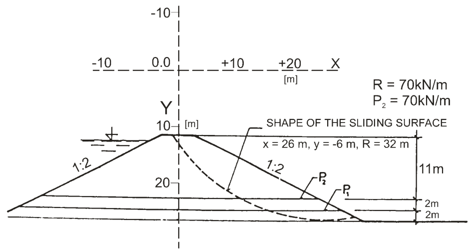
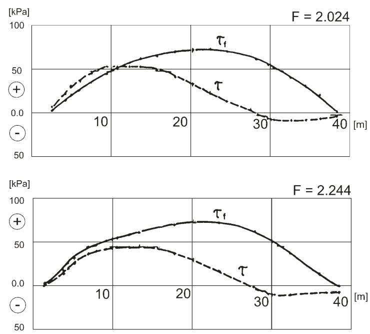

-10
-10
0.0
+10
+20
X
[m]
R = 70kN/m
P2 = 70kN/m
Y
10
[m]
SHAPE OF THE SLIDING SURFACE
x = 26 m, y = -6 m, R = 32 m
1:2
1:2
20
P2
P1
11m
2m
2m


[kPa]
100
F = 2.024
τf
50
τ
0.0
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20
30
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[m]
50
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100
F = 2.244
τf
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τ
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50