GEOTECHNICAL ANALYSIS OF AN EMBANKMENT BASE FAILURE

By

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1. Introduction

The dynamic development of the town Szolnok imposed to construct a new road and a bridge across the *Zagyva* River in order to link the different parts of the town.

Seventeen years later a base failure occurred in the flood area section of the embankment.

The damaged road section was reconstructed, nevertheless some months later an embankment base failure occurred again at the same place.

The repeated failure of the embankment proved that the measures taken previously were inadequate to prevent embankment base failure.

This case study aims at detecting causes, collecting the experience and drawing conclusions with a view on a satisfactory design of the reconstruction as well as on avoiding further damage.

2. Description of the damage

On the spot of the actual road and bridge, a wooden bridge of low bearing capacity existed until 1958. The wooden bridge and the low embankment became periodically flooded, imposing to be replaced.

The new road and the 56 m long, reinforced concrete bridge over the river were inaugurated in 1958. Conditions before and after construction are shown in *Fig. 1*.

After construction, the $\sim$8.7 m high embankment behind the abutment settled. In 1970 the settlement reached a maximum of $\sim$0.4 to 0.5 m in relation to the construction condition.

On October 23, 1975 a base failure occurred in a section of the embankment after withdrawal of the flood. Photograph of the base failure is seen in *Fig. 2*.

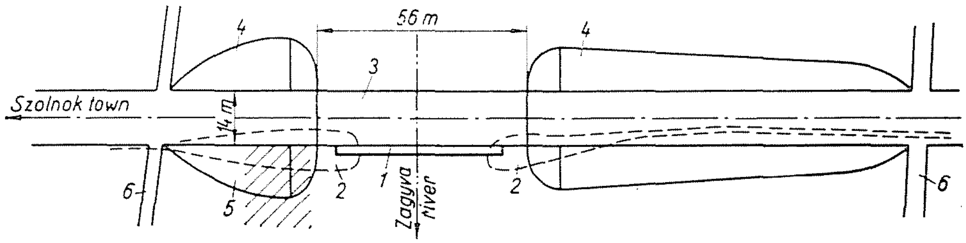

Fig. 1. Lay-out sketch: 1 — wooden bridge; 2 — old embankment to the wooden bridge; 3 — reinforced concrete bridge; 4 — embankment sections joining the reinforced concrete bridge; 5 — spot of base failures in 1975 and 1977; 6 — levee-dam of the Zagyva river

a

b

Fig. 2. The July 23, 1975 base failure: a — spot of the base failure; b — surface upheaval before the embankment foot

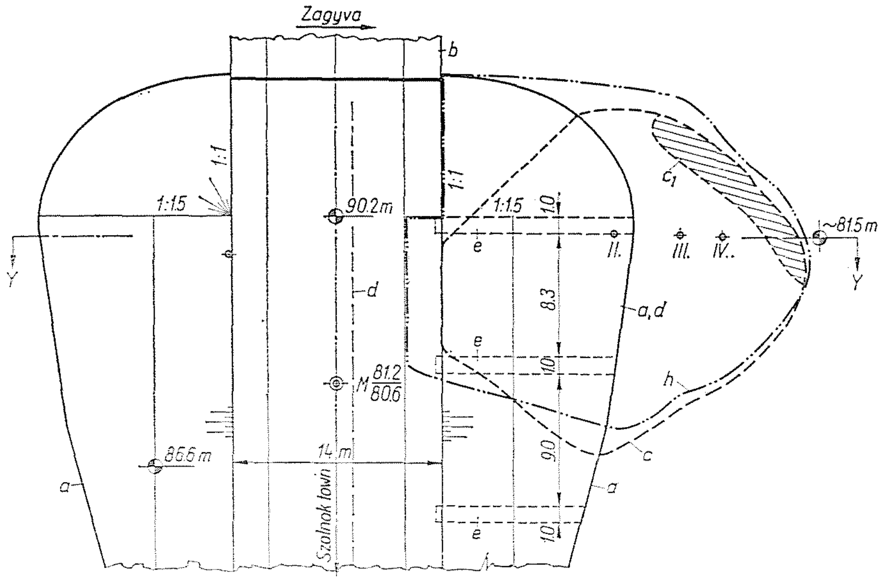

Fig. 3. Lay-out sketch for the failed embankment section: a — embankment constructed in 1958; b — the new reinforced concrete bridge; c — boundary of the base failure, July 24, 1975; c_1 — surface upheaval; d — boundary of the reconstructed embankment section; e — buttress drains; h — boundary of the repeated base failure after reconstruction (May 31, 1977); *M*, I, II; III; IV — boreholes

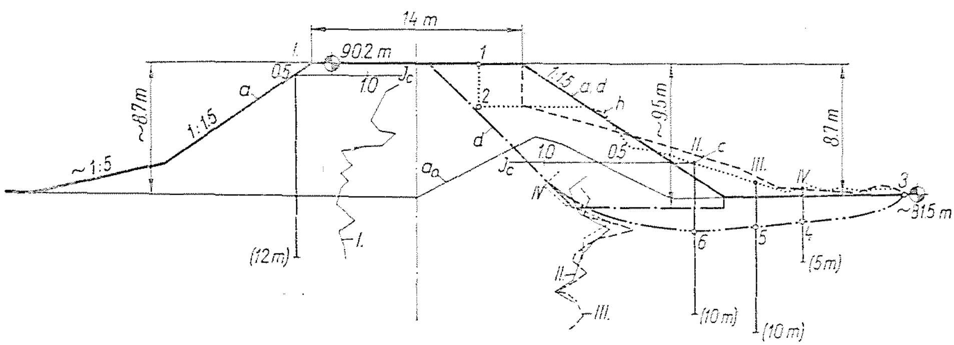

Fig. 4. Elevation Y—Y of failed embankment section: a_0 — site and embankment before construction of the reinforced concrete bridge ; 2, 3, 4, 5, 6 — the points of sliding surface (other symbols see in Fig. 3)

The layout plan and the elevation of the damaged section are shown in *Figs 3* and *4* together with data of the site and the embankment before construction as well as other data involved.

It became evident from the construction data that the old embankment leading to the wooden bridge was not pulled down under the new section of the embankment in 1958.

The slopes of the embankment, built in 1958, were clad with concrete facing up to the 86.5 m level.

The flood level of *Zagyva* River was ~87.8 m due to the backwater of *Tisza* River.

After the base failure the design seen in *Figs 3* and *5* was carried out. (In *Fig. 5* also the boundary lines of the two discussed base failures are plotted.) It was tried to reinforce the failed embankment section, reconstructed according to *Figs 3* and *5*, by buttress drains. The buttress drains were subsequently constructed in braced trenches inside the completed embankment.

For quick drainage of the water seeping into the embankment, earthenware tubes were placed at the invert of the drains leading to the open air.

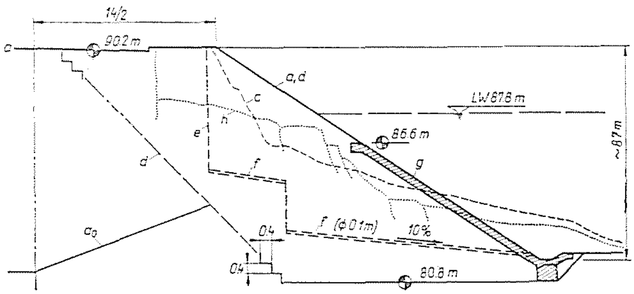

Fig. 5. Proposal for reconstruction of the 1975 base failure: f — earthenware tubes; g — quarry stone masonry facing; LW — flood level maximum (other symbols see in Figs 3 and 4)

The jointed quarrystone facing of the slope was again raised to the 86.6m elevation.

The reconstruction works were completed in November 1976.

After the high water level in December 1976, the slope facing and the filling material between the buttress drains showed in one section a vertical and horizontal displacement of ten centimeters' order.

The flood lasted long and the water level only sunk to ~80.0 to 81.0 m after May 10, 1977. At this time the water level was already ~0.5 to 1.0 m below the lowest point of the embankment foot. Following this a base failure under the embankment occurred on May 24. *Fig. 6* is a photograph made of the base failure on May 31.

At the time the picture was taken, water was seeping at the foot of the embankment from the remoulded and dilated material at several places. In the $Y — Y$ section of the base failure at the farthest points from the embankment, still a very slow movement was observed. (During one or two hours the surface heaved by some ~5 to 6 cm and a crack of ~2 to 3 mm appeared.) After these precedences, the Department of Geotechnique undertook analysis of the base failure causes.

3. Soil exploration and soil mechanics tests

For the design of the bridge and the road 8 boreholes 5 to 20 m deep were carried out in 1954—55. These explorations did not analyse strength properties of the ~2 to 3 m deep layer under the ground level, and gave no data.

Following the base failure in 1975, 23 further boreholes were made. Unfortunately also these soil mechanic tests were deficient, even the consistency limits were only estimations; shear strength parameters were assumed from

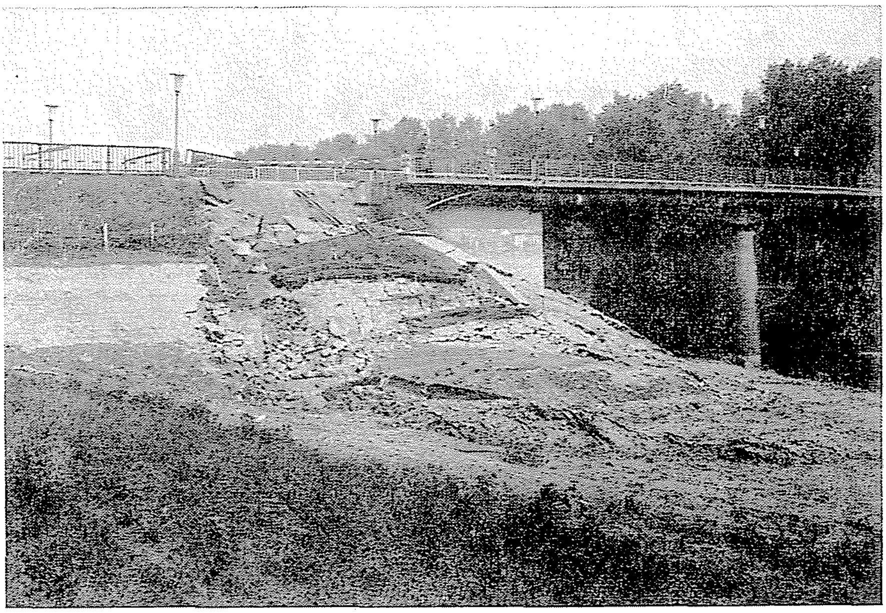

Fig. 6. Photograph of the May 24, 1977 base failure

estimated data. Because of these circumstances, further tests and also the design of the reconstruction became uncertain and to a certain degree arbitrary.

(In *Figs 3* and *4* only those are plotted among the many boreholes, which are actually referred to in the following, in connection with test data.)

Following the repeated base failure, further tests were carried out to obtain information on the physical properties, with undisturbed samples, taken from pits.

The explorations showed that under the ground surface there were ~9 to 12 m thick strata of soft fat clays of mosaic structure, overlaying mixtures of silts and fine sands to the tested depth.

Typical subsoil conditions of the site are shown by the borehole section in *Fig. 7*. The location of boring *M* shown in *Fig. 7* is seen in *Fig. 3*.

Subsoil and embankment material gradings have been compiled in *Fig. 8*. In 1958, the backfill behind the abutment was constructed of the inundation

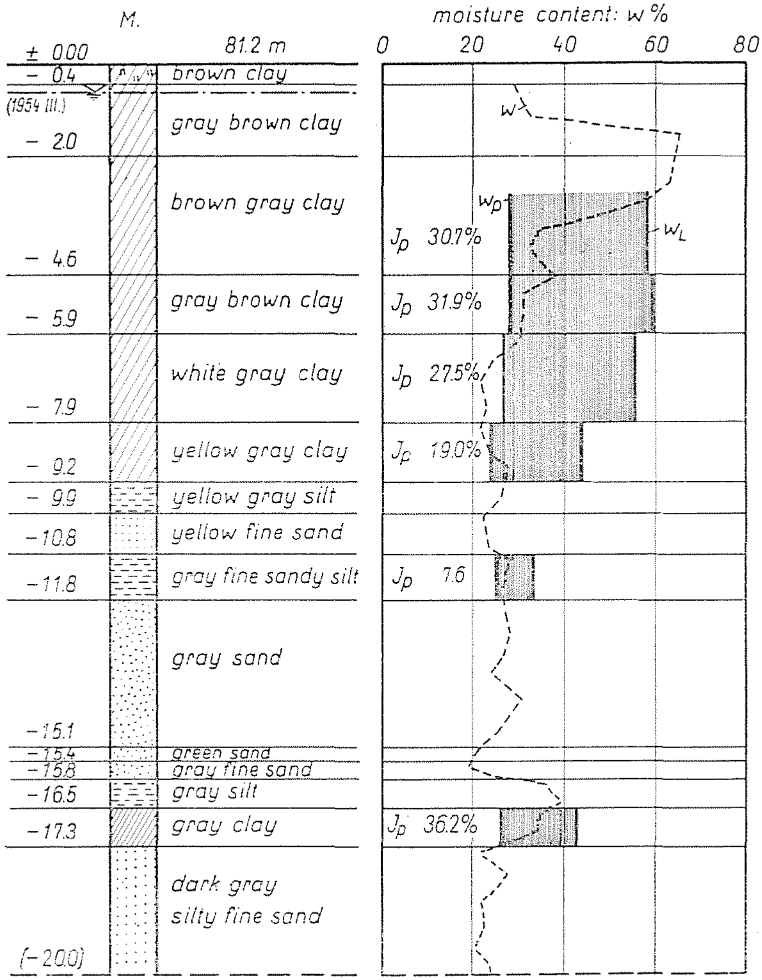

Fig. 7. Borehole section *M* (March 1954)

area material of the *Zagyva* River. This is clearly seen from the distribution of data related to the fill and subsoil in *Fig. 8*. The 1975 reconstruction of the failed section applied, however, a mosaic-structured clay of medium plasticity.

Changes in plasticity index and water content of the subsoil have been plotted in *Fig. 9*. These results also proved that properties of the soft fat clay of mosaic structure ~3 to 4 m thick were rather unfavourable for an embankment foundation, already before the construction.

Changes in phase composition of the different soil types are seen in *Fig. 10*.

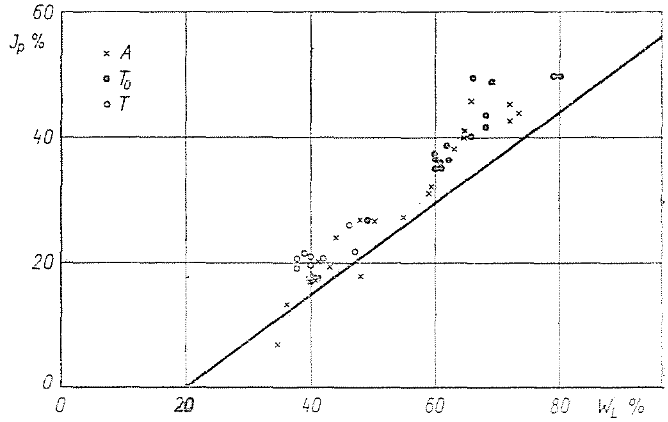

Fig. 8. Consistency limits of the tested soils. *A* — samples of the subsoil; T_0 — samples of the old embankment constructed in 1958; *T* — filling material used in the reconstruction of the 1975 base failure

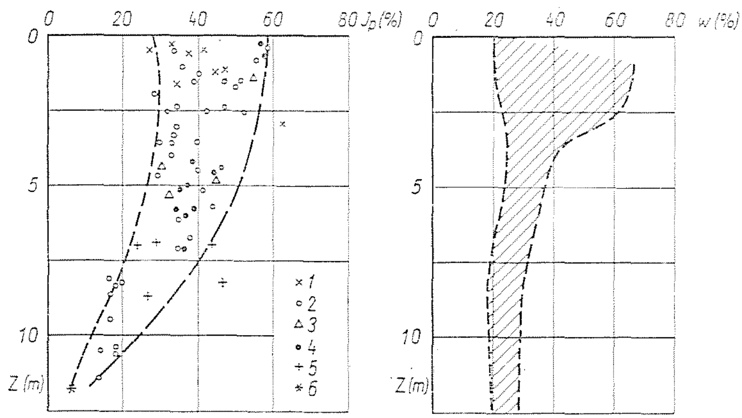

Fig. 9. Changes with depth of the plasticity index and the water content: 1 — yellow, drab clay; 2 — gray, green-gray rust-marbled clay; 3 — brown, olive-drab clay; 4 — dark gray, black clay; 5 — white, white-gray clay; 6 — drab sandy silt

Shear strength values of pit samples were determined partly in triaxial tests, and partly in direct shearing tests.

Triaxial tests were carried out both on samples in the original condition and on saturated samples in an undrained shear test. The axial deformation velocity of the samples was 1 mm/min.

The direct shearing tests were made on samples in the original state, resulting in shearing deformation velocity of 2 mm/min.

Two typical results of the direct shearing tests have been plotted in *Fig. 11.*

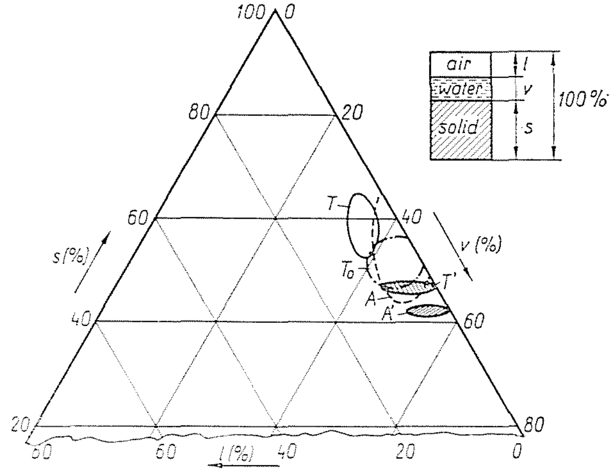

Fig. 10. Phase composition of the soils *A'*, *T'* — saturated dilated subsoil and embankment material (other symbols see in Fig. 8)

The shearing strength parameters (Φ, c) vs. total stresses are seen in *Fig. 12*. Physical characteristics of soil samples used in the strength tests are shown in *Table 1*.

Table 1

Results of shearing strength tests

Symbol		Soil type	Consistency limits (%)			Phase composition (%)			Shearing strength parameter values	
			W_L	W_p	I_p	s	v	l	$\Phi°$	C (kN/m²)
T	1	yellow, gray sandy clay	42	20	22	59	35	6	2	71.5
	2	with lime pockets							17	40
A	3	yellow, gray, mosaic clay	51	24	27	53	43	4	8	20
	4	with organic pockets	69	28	41	54	42	4	3	25.5
	5	olive gray clay	91	37	54	45	47	8	17	17
	6	drab	77	30	47	45	45	10	11	18
	7	mosaic clay	62	27	35	51	41	8	14	17.5
A'	8	drab mosaic clay	76	31	45	42	52	6	13	2 5
	9					43	54	2	10	8.5
	10					43	48	9	6	16
T'		yellow, gray sandy clay with lime pockets	47	23	24	47	38	15	10	16

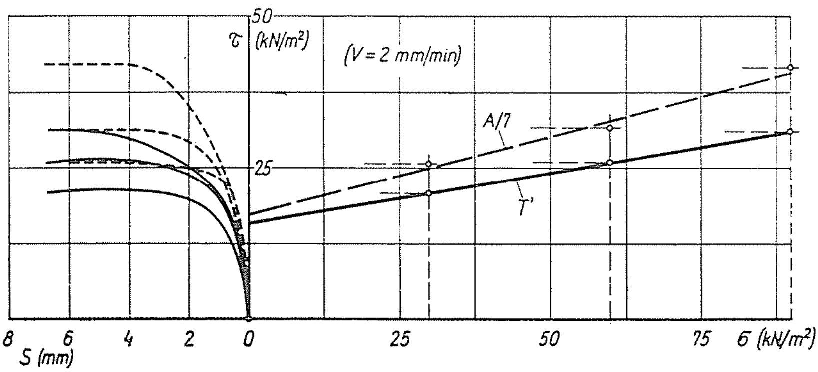

Fig. 11 Typical strength results of subsoil and embankment material (symbols 7, T' are those in Table 1 and in Fig. 8)

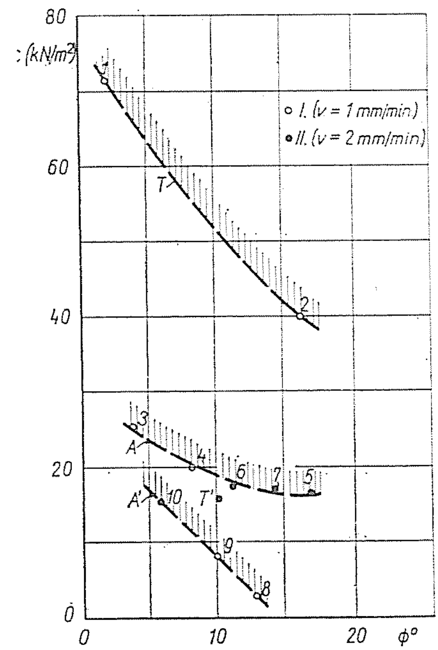

Fig. 12. Shearing strength parameters of the analyzed soils; I — triaxial compression; II — direct shearing tests (other symbols see in Table 1 and Fig. 8)

The test results prove that the shearing strength of the subsoil is very low, the cohesion in the case of $\Phi \approx 0$ being as low as $c \approx 25$ to 35 kN/m^2.

The embankment material had an important strength during reconstruction, but soaking and expansion drastically reduced the shearing strength.

To analyse the properties of the embankment material used for reconstruction also unconfined compressive tests have been carried out, with results plotted in *Figs 13* and *14*.

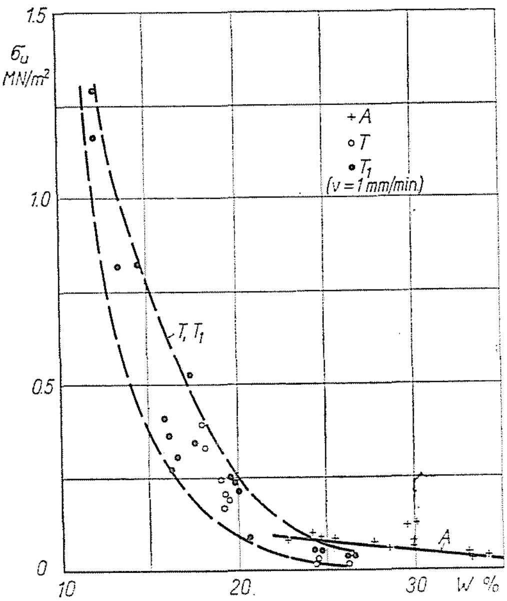

Fig. 13. Unconfined compressive strength vs. water content. T_1 — Filling material samples for Proctor tests (Symbols A, T see in Fig. 8)

Fig. 13 shows the change of unconfined compressive strength of the subsoil A and the embankment material T as a function of water content.

Strength of the embankment material used for the reconstruction was very sensitive to water; with increasing water content its unconfined compressive strength abruptly diminished.

Water sensitivity tests of the embankment material are also illustrated in *Fig. 14.* These were *Proctor* tests made on two unconfined compressive samples of embankment material each with different water contents. One sample was subjected to unconfined compressive test σ_u with the original water content w.

The second sample remained in the sampling cylinder, on its two free surfaces filter stones were placed and the sample was submerged in water. After some days of saturation the sample was tested for unconfined compressive strength σ_u and its water content was determined again ($w' = w + \Delta w$).

The results have been plotted in *Fig. 14.* Line 1 in Fig 14a shows the relationship between the initial water content w and the unconfined compressive strength σ_u'. In the same figure the rate of water content increase Δw is shown by line 2. *Fig. 14b* indicates the rate of strength decrease (σ'_u/σ_u) upon water absorption. The figure also shows the data of the *Proctor* test.

The experiment allowed of course not only water absorption but also a small volume change ($\sim$0 to 5%).

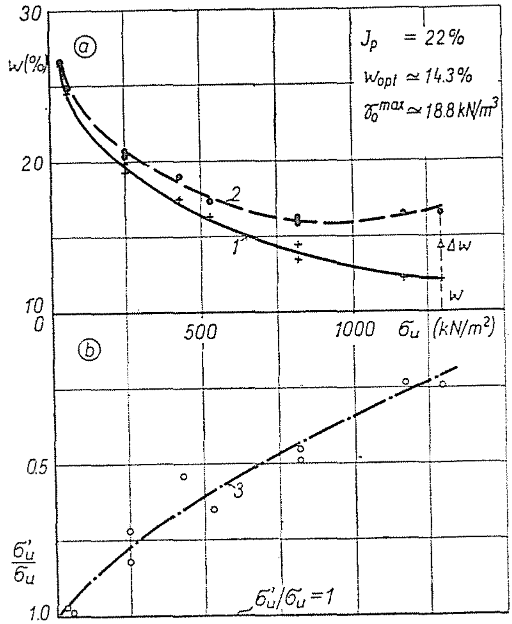

Fig. 14. Unconfined compressive strength vs. water content

Data in *Fig. 14* show a small increase in water content to be accompanied by an important drop of the unconfined compressive strength because it caused also movement of the solid skeleton (see KÉZDI, 1976).

From the discussed test results it became obvious that the stability of the slope reconstructed according to the design shown in *Figs 3* and *5* was still very low. Outlets of the drains and of the earthenware tubes did not only lead out the water, but in case of inundation they led it in, likely to contribute to the decrease of shearing strength. Test results in *Figs 13* and *14* proved that the soil absorbed quickly water, a small water content change and swelling markedly reduced the shearing strength. At the same time the embankment material got rid slowly and with difficulty of the absorbed water. Shearing strength began to diminish along the boundary surfaces of the buttress drains.

In the first inundation after the construction with increase of the water level the filling between the stone facing and the rock buttresses was first displaced both vertically and horizontally by tens of centimeters.

Beside the discussed soil mechanics tests, also the ground water conditions of the site must be mentioned.

In case of low level of the river, the ground water is seeping across the surface in direction of the river bed at the foot of the levee-dam of the *Zagyva* (at an elevation of ~81 to 83 m). This seepage persists even in the dryest summer months. Therefore at the foot of the levee-dam there are soaked, muddy areas in varying patches.

In boring *M*, the ground water level was ~0.6 m below the ground level before the construction, in March 1954.

Observation data in borings after the base failure refer to important rise of the water levels. Water level data from borings carried out after the failure were used to plot isohypses seen in *Fig. 15*. The isohypses are referred to the ground level before 1958 (actually the surface of the levee-dam of the

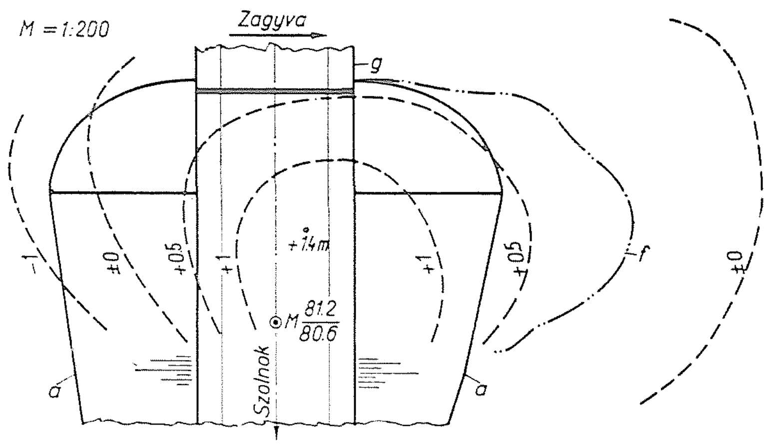

Fig. 15. Isohypses of the ground water level for the low river water level following base failure according to measurements in summer and autumn 1976 and 1977

Zagyva). Water levels below and above the surface of levee-dam are affected by negative and positive sign, respectively.

The maximum water level change was ~2 m. Data in *Fig. 15* illustrate changes of probable pressure conditions of the site against conditions before construction. The isohypses refer to the period after the base failure, when the water seeped from the broken embankment material through cracks into the open, at a low river water level.

Immediately after the rapid drawdown of the river water level the pressures were probably much higher, because the sound slope facing hindered the quick passing of the water.

It should be noted that in the area covered by the embankment, the water supply and the changes in pressure conditions might be influenced also by the water flow through the boreholes and beside the open caisson foundation of the bridge abutment.

Unfortunately there are no measured data for the exact relation between the ground water level change and the river water level fluctuation.

4. Stability tests

Actual base failures are suitable for approximating the maximum shearing strength value provided the position of the sliding surface is known, because here the condition $\nu \simeq 1$ is really fulfilled.

Surface line and extension of the 1975 base failure have been plotted in *Figs 3, 4* and *5*. The exact depth of the sliding surface beneath the ground level was, however, not exactly known. From the tests carried out, it can be stated beyond doubt that the sliding surface followed the embankment slope existing in 1954 and penetrated before the slope foot to a depth of $\sim$1.5 to 2.5 m. This is most likely from the relative consistency data I_c of the borings.

Course of the base failure repeated after reconstruction could be approximately established from the surface form and the relative consistency data shown in *Fig. 4*. (See soil mass outlined by points *1, 2, 6, 5, 4* and *3* in *Fig. 4*.)

The displacement boundary line in *Fig. 4* can be replaced approximately by a circular arch between points *2, 6, 5, 4* and *3* as shown in *Fig. 16*.

The first stability test involved the condition $\Phi = 0$ and $\nu = 1$, according to the well-known moment principle:

$$\nu = \frac{M_s}{M_r} = \frac{rL_i c}{G_i a_i} = 1 .$$

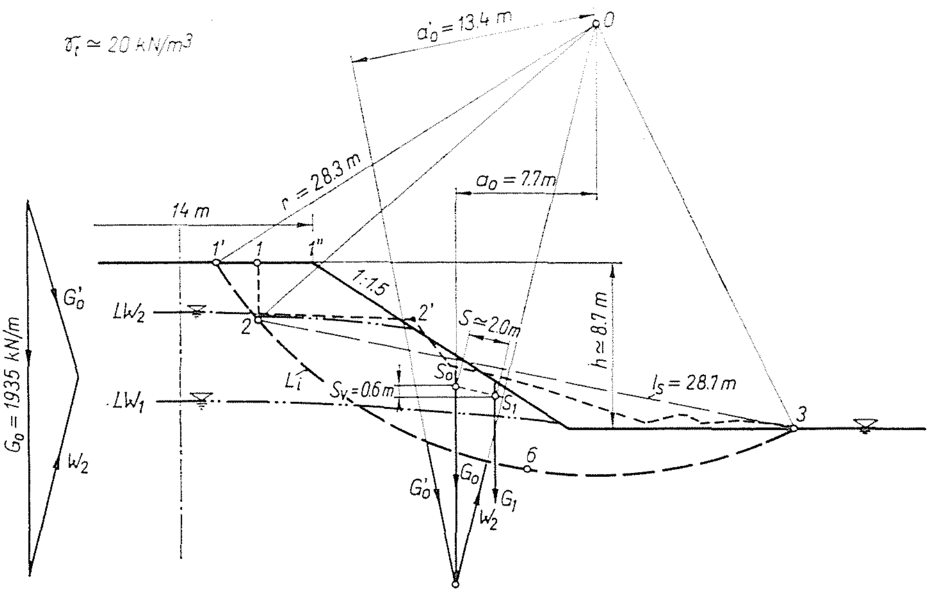

Fig. 16. Stability analysis of slope and computation data

Test results have been compiled in *Table 2* assuming different ground water levels and soil masses. The foregoing equilibrium condition yields the necessary cohesion:

$$c = \frac{G_i a_i}{r L_i} .$$

Table 2

Stability test results

Boundary of the sliding mass	Ground water level	Resultant mass force G_i, G'_i (kN/m)	Distance between the resultant mass force and the centre of gravity a_i, a'_i (m)	Arc length L_i (m)	Necessary cohesion ($\nu = 1$) $C_i = G_i a_i/L_i r$ (kN/m²)
1′-1″-3-1′	—	2020	7.9	34.5	19.5
1-1″-3-2-1	—	1935	7.7	30.5	20.5
1-1″-3-2-1	LW_1	1540	9.8	30.5	20.8
1-1″-3-2-1	LW_2	1110	13.4	30.5	20.5
2-2′-3-2	—	1940	5.7	30.5	15.2
2-2′-3-2	LW_1	1380	8.0	30.5	15.2
2-2′-3-2	LW_2	960	11.5	30.5	15.1

Particulars of the computation principle are shown in *Fig. 16* for the soil mass limited by points *1, 1'', 3, 2* both reckoning with, and omitting water pressure. For convenience, lines *2, 3* indicate only the resultants of water pressure.

The resultant water pressure acting on the sliding surface was obtained by plotting drawdown surfaces LW_1 and LW_2 in *Fig. 16*. LW_1 was assumed according to data in *Fig. 15*. LW_2 involved the following assumptions:

a) The drawdown surface is identical with the flood level (~87.8 m) in the embankment axis.

b) The intersection of the slope surface and the drawdown surface is identical with the top of the slope facing (~86.6 m). (For the sake of completeness the computation included the case of drawdown surface below the sliding surface.)

Based on positions of the original slope, the displaced slope and the sliding surface, the shearing strength can also be calculated from the assumption:

$$M_k = M_b$$

where

$M_k = s_v G_0$ (external virtual work)

$M_b = s l_s c$ (internal virtual work)

s_v and s are distances between centres of gravity S_0 and S_1. S_0 is the centre of gravity of the mass limited by points *1, 1'', 3, 2, 1;* and S_1 is the centre of the mass limited by points *2, 2', 3, 2* omitting the mass force due to water movement.

From the condition equation for $\Phi = 0$ and $\nu = 1$,

$$c = \frac{s_v G_0}{s 1_s} = 20.3 \text{ kN/m}^2 .$$

During the movement the external virtual work ($M_k = s_v G_0$) was spent for deforming, breaking up and expanding the mass, because the heat generated can be considered in the given case as zero (see KÉZDI, 1976).

The results of the two calculations yielded
for $\Phi = 0$, an average value of
$c = 20.3$ to 20.8 kN/m^2

at the beginning of the base failure. In the new situation following the base failure, for $\nu = 1$, $c \approx 15.2$ kN/m^2 giving the average safety against further movement:

$$\nu = 20.5/15.2 \simeq 1.35.$$

The values obtained from the equilibrium analysis ($\Phi = 0$, $c = 20.3$ to 20.8 kN/m^2) are in an apparent contradiction to data of actual soils A, A' and T' in *Fig. 12*.

Namely from the data presented above it is obvious that the triaxial tests and the direct shearing tests refer to sudden loading.

In case of the slope, however, the first movement occurred 17 years after construction. Under prolonged and alternating loads also the physical properties of clays are known to change, the shearing strength to diminish (see SKEMPTON, 1948, 1964). Under constant loading only the fundamental shearing strength of the soil related to deformation velocity $d\varepsilon/dt \to 0$ can be taken in account.

The stability analysis for $\Phi = 0$ yielded $c \simeq 20.3$ to 20.8 kN/m^2, about 30% lower than the laboratory tests under short-time loading.

This decrease is due to load changes acting on the subsoil — flood level fluctuations — and to many other effects.

Conclusions

According to the investigations the base failure on July 23, 1975 was due to the combination of the following adverse effects:

The embankment built in 1958 was constructed on the embankment leading to the wooden bridge in a way that the right-side slope involved the possibilities of a potential sliding surface and of water movement along the boundary surface.

The subsoil is made up of soft, fat clays of mosaic structure, of low shearing strength according to short-time loading tests under laboratory conditions. The embankment behind the bridge abutment was constructed of the same material.

Shearing strength of the mosaic-like clays near the surface and the clays forming the embankment markedly change upon phase movement and structural change due to external effects.

Construction of the embankment stopped the seepage of water from the levee-dam resulting in further unfavourable change of the embankment material.

The shearing strength of mosaic-structured clays in the same phase condition much decreases also in time, due to external influences — water level fluctuation, rain, thermal variations.

In design the slope inclination was assumed in ignorance of these effects and the slope had not the specified safety within the planned lifetime. The shearing strength of the mosaic structure clays gradually decreased to the ultimate equilibrium value. Unfortunately, the reconstruction proposal after the base failure in 1975 contained several mistakes, leading again to a base failure, occurring on May 24, 1977.

In the course of the reconstruction, only part of the soil layer accommodating the base failure sliding surface was removed. Namely in the environment of the embankment foot the new embankment foundation plane was probably higher than the plane of the sliding surface of the base failure.

This circumstance thus held again the possibility of a potential sliding surface in the vicinity of the embankment foot as confirmed by the close coincidence of the boundaries of both base failures and by the recent examination results (change of I_c values in boreholes II, III and IV).

The mosaic-structured clay of medium plasticity used for reconstruction lost much of its shearing strength because of the phase movement due to water absorption.

The buttress drains allowed both outward movement of the groundwater and inward flow of flood water.

After rise of the flood level a possibility of rapid water absorption and phase movement was given along boundary surfaces of the reconstructed embankment — interface between old and new embankment, buttress drains — decisively decreasing the shearing strength. This is proved by the facing and embankment movement of tens of centimeter order almost simultaneous to the flood level rise after the construction.

The mentioned causes necessarily started the repeated base failure as seen regularly by the discussed analyses.

After having analysed the causes, the Department of Geotechnique cooperated in the reconstruction designs to be reported later.

Summary

In the town *Szolnok* a new road and bridge were constructed across the *Zagyva* River in 1958. A base failure occurred in the flood area section of the embankment in 1975. The damaged road section was reconstructed, but some months later an embankment base failure occurred again at the same place. This case study aims at detecting the causes, collecting the experiences and drawing the conclusions.

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