The Stoegersbachdamm road embankment

La route Stoegersbachdamm en remblai

W.K.HAZIVAR, Civil Engineer, Geotechnical Consultant, Wien, Austria

SYNOPSIS: In the course of a bypass road a stream and its adjacent swampland of some 300 m width had to be crossed on an embankment up to 18 m high. Two covered bridges are embedded partly in the fill, partly in the subsoil. Very insufficient preliminary soil investigations made calculations as settlement prediction and stability analysis unreliable. Measures taken to improve an up to 10 m thick zone of very soft subsoil were plastic drains and rammed gravel piles. The estimates for soil parameters and settlement behaviour were subsequently updated and corrected by monitoring the embankment during filling works geodetically, by inclinometers and settlement gauges and by additional soil tests.

1 INTRODUCTION

The B 50 feeder road to the A 2 motorway was planned in 1982. Earthworks began in early 1986, filling started in November 1986 and was finished in May 1988. The road shall be open for traffic by end of October 1988. The embankment has an overall length of some 400 m. Near both ends of the embankment, but still on the swampland two covered bridges are placed. Through one of them (obj. 17/39) also runs a regional sewer with a gradient of only 4 ‰.

2 PROBLEMS

The main problems which had to be considered for construction of the embankment and for design of stabilizing measures were:

- stability during construction, especially with respect to excess pore pressure
- stability in final state
- settlement behaviour with respect to total amount of settlement, differential behaviour of the areas of the bridges including their immediate vicinity and of the regular dam profile, not influenced by the bridges and their foundation
- time dependent course of settlement, of special importance for the Stögersbach bridge, containing the sewer.

3 SUBSOIL CONDITIONS

The subsoil conditions are shown in a generalized longitudinal section of the embankment (Fig. 1). The topmost 8 to 10 m consist of very soft to pulpy sandy silts and loose fine sands. This soft zone is underlain by medium dense sands, partly also gravel, and stiff clayey silts. The groundwater table varies between surface and 1 m below surface. Static penetrometer tests showed penetration rates of 50 - 80 mm/s down to a depth of 8 m. The soundings (heavy standard probe) showed only 4 blows per meter in

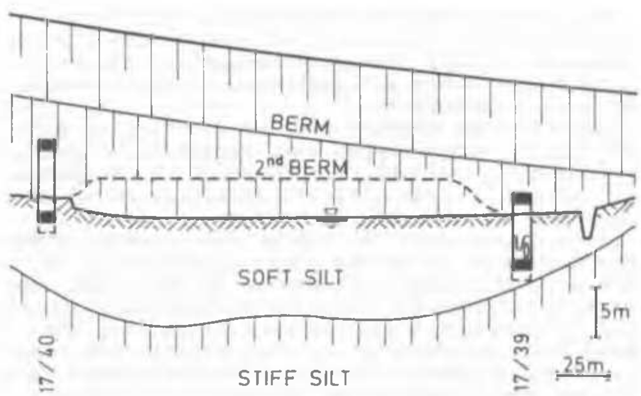

Fig. 1. Longitudinal section of embankment and subsoil

the minimum, n_{10} constantly over 10 blows was only found in depths greater than 11 m.

Table 1 gives the soil parameters, on the left side for the initial investigations, on the right side for tests carried out on samples taken after one year of consolidation under an overburden of 5 m.

Table 1. Soil parameters

	range	mean	s.dev	range	mean	s.dev
w_n %	23 - 59	40,2	9,6	32 - 37	34,5	3,5
I_c	0,38-0,72	0,54	0,17	0,53-0,65	0,59	0,08
1) kN/m²	85 - 100			100 - 120		
2) φ °	24,0-31,0	27,5	5,0			
2) φ_r °	19,5-31,0	23,5	11,0			
E_s kN/m²	6500	6500	0			

1) Unconfined compressive strength
2) angle of internal friction (Vienna routine shear test)

Permeabilty was calculated from the grain size distribution by the formula

$$k = \left(\frac{268}{\frac{d_{60}}{d_{10}} + 3,4} + 55\right) * d^2_{10}$$

From the first tests k = $1,0*10^{-5}$ cm/s was found. From the second test series vertical permeability $k_v = 4,0*10^{-7}$ cm/s was calculated. Horizontal permeability was assumed to $k_h = 2,0*10^{-6}$ cm/s, 5 times k_V. These values proved to agree quite well with the observed consolidation behaviour.

4 DESIGN OF EMBANKMENT, ALTERNATIVES

Since it was impossible to enter the swampland with heavy equipment, it was decided to let the subsoil in place. Subsoil improvement by plastic drains and gravel piles should enable the fill.

The boundary conditions imposed by the subsoil and by the task of placing two embedded bridges at the dam's base led to the dismissal of other conventional methods for filling embankments on soft ground.

Full excavation and replacement would have ment moving and disposing of some 300000 m³ waste material. It was dismissed because of the high groundwater table and interlayed instable sands of high permeability.

Displacement methods such as blasting or displacing by wedge-shaped fill seemed to be not sufficiently reliable for founding the bridges, or, if carried out after the bridges, could severely damage the new structures.

Pile foundations for the bridges seemed inadequate because of excess load concentration on the bridges. Great differences in settlement behaviour between the soft founded dam and the rigid bridges would clearly be reflected to the road surface. Therefore a semi-soft foundation on improved subsoil was found most suitable. The bridges were designed to be able to cope with settlements in an order of some 20 to 40 cm. The sewer in obj. 17/39 was to be finished as late as possible. Fig. 2 shows the settlement profiles of obj. 17/39 and 17/40.

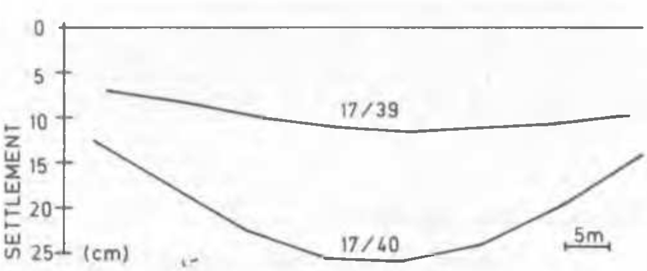

Fig. 2. Settlement profiles of obj. 17/39 & 17/40

5 EMBANKMENT FILL

The fill is made up by alternating layers of sandy silt and fine gravel. The slopes are inclined by 1:2 with one berm of 4 m in half of the height. See fig. 3. Stability was calculated with the slice method (DIN 4084). For stability analysis a maximum pore pressure of 10 % of the dam's weight for the subsoil and of 10 % of the overburden within the fill was taken into account.

Table 2. Safety requirements and soil parameters

		φ (°)	c (kN/m²)
initial safety factor: 1,1	fill	26	10
	subsoil	10	25
final safety factor: 1,3	fill	30	0
	subsoil	26	0
or:	fill	26	10
	subsoil	26	0
ultimate safety factor without cohaesion: 1,0	fill & subsoil	26	0

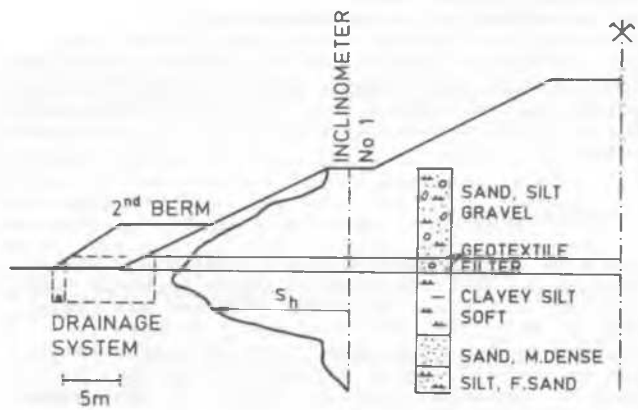

Fig. 3. Embankment profile; additional stabilizing measures, characteristic displacements

6 SUBSOIL IMPROVEMENT

Embankment design was carried out in three steps:

- profile and inclination of slopes were chekked with regard to the filling material to meet the demands as shown above
- the maximum permissible thickness of a new fill layer was calculated
- the measures for subsoil improvement were layed out such as to permit a reasonable filling rate.

The maximum permissible thickness of each fill step was calculated by means of a simplified analysis (Fellenius method). A polygonal slip surface was assumed, which was enlarged proportionally to the height of fill already reached. Pore pressure was assumed to be constant in the fill and to decrease linearly along the slip surface in the underground. The maximum permissible thickness had to meet two criteria:

- a safety factor of ≥ 1,3 with a degree of consolidation U = 90 % in dam and subsoil before applying a new load step
- a safety factor of ≥ 1,15 was required, assuming the new load step would cause 100 % excess pore pressure in fill and subsoil, together with a degree of consolidation for the last load step as above.

So step by step new layers were placed. This calculation gave a maximum permissible load step of Δh = 0,8 m, variing (increasing) a little with the total height already reached.

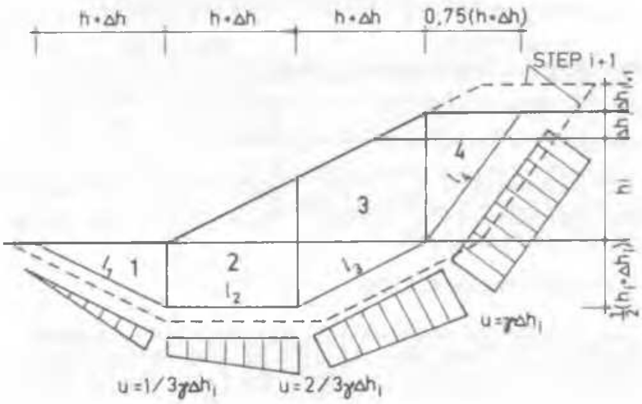

Fig. 4. Simplified stability analysis for maximum load step

For instance water pressure for slice 3 is:

$$W_3 = \gamma_w \cdot l_3 \cdot 0{,}25(h + \Delta h) + u_{wi}$$

$$\text{with } u_{wi} = (1 - U) * (\gamma \cdot \Delta h \cdot \tfrac{1}{2}(1 + \tfrac{2}{3}) l_3 + u_{wi-1})$$

Subsoil improvement was done by rammed gravel piles under and in the vicinity of the bridges. The piles Ø 65 cm were spaced in a triangular grid 1,5/1,5 m. With this spacing an improvement factor of >2,5 for the modulus of compression could be achieved. In reality the improvement factor was significantly higher, since intensive compaction and drainage of the subsoil surrounding the gravel piles took place, an effect that is not taken into account in the considerations of Priebe (1970).

In the areas off the bridges plastic drains (MEBRA drains) were set in a 2,0/2,0 m square grid. Necessary time to reach a degree of consolidation of U = 0,9 was calculated according to Feuerlein (1965):

$$U(\rho,t) = \frac{c}{2} \cdot \left(\ln\frac{\rho}{\rho_i} + \frac{\rho_i^2}{2} - \frac{\rho^2}{2}\right) \cdot e^{-\frac{c\varkappa}{4\pi}}$$

$$\text{with } c = \frac{1 - \rho_i^2}{-\frac{1}{2}\ln\rho_i - \frac{3}{8} + \frac{\rho_i^2}{2} - \frac{\rho_i^4}{8}}, \quad \varkappa = t \cdot \frac{4\pi \bar{E} k}{\gamma_w l_x^2}$$

With $k = 1{,}0 * 10^{-5}$ cm/s, E_s = 6500 kN/m², ν = 0,33 resulted t = 9,6 h. Such little time let the measures seem being oversized. But taking into account a great uncertainty in the value of permeability, which could be smaller by a factor of 10, and also in the compression modulus E_s, which might only be half the size, the necessary time becomes 20-fold, i.e. 8 days. Also no disturbance of the surrounding soil (smear effect) by placing the drains is taken into account. Therefore pore pressure measurement was deeply recommended, but the authority did not approve. So the maximum filling rate was reduced from 80 to 60 cm per week. On aggregate 19300 m gravel piles and 74000 m MEBRA drains were used.

7 INCREASED FILLING RATE

During first half of 1987 filling works had to

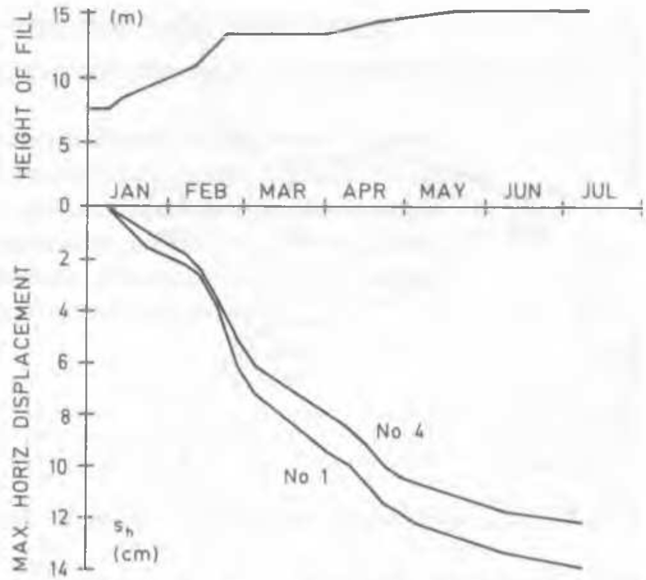

Fig. 5. Max. horizontal displacement (Incl. 1 & 4)

be interrupted due to accessibility of the central part of the embankment while the bridges were constructed. To keep up with the time plan filling speed should be increased to 1,2 m/week for the second half of height. The long time of consolidation under small load (some 4 m overburden) seemed to make this possible. Anyway close monitoring of the embankment was imposed: 4 inclinometer borings, placed in the berm, and 6 settlement gauges (double and triple type) were scanned once a week. The embankment base was geodetically surveyed weekly, a sight line on both sides of the base was controlled daily. When full filling speed of 1,2 m/week was reached in February 1988, the horizontal displacements increased dramatically. The geodetic survey showed upheaval near the base of over 1 cm. See fig. 5.

Filling was stopped immediately. Since almost no decrease of speed of the horizontal displacements could be observed after one week, a system of gravel drainage cuts, 4 m deep with a groundplan like a comb, was carried out. These filter cuts were layed out with geotextiles and filled with crushed rock. Revised stability analysis showed that it was necessary to enlarge the base by 6 m, such creating a second berm of 4 m height over the drainage cuts. (Fig. 3). After completion of these securing measures filling was taken up again on April 1st and finished on May 15th. Filling speed was limited to 50 cm/week. Horizontal displacement amounts to 14 cm at maximum at beginning of July, but the deformation speed has already significantly declined.

8 SETTLEMENT AND CONSOLIDATION

First calculations led to an estimate of 80 cm maximum settlement, based on E_s = 6500 kN/m² for the top soft layer. Measured settlements with only 4 m fill already amounted to over 40 cm. Therefore soil parameters had to be revised. New calculations with E_s = 3000 kN/m² led to a maximum settlement of at least 1,20 m. Up to beginning of July 106 cm were reached, the calculated

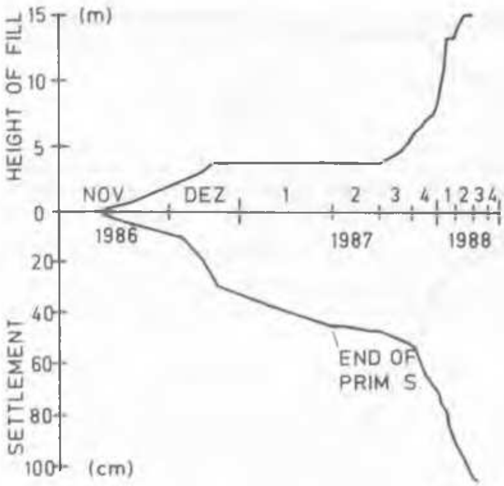

Fig. 6. Log. time versus settlement, gauge No 10

value for the location of the gauge is 118 cm. Agreement is quite well, especially when considering that the settlement gauge was installed after the base filter layer.

The diagram time versus settlement (fig. 6) indicated the end of primary settlement after some 100 days for the first 4 m of fill. A Somewhat shorter period for primary settlement (60 days) has been observed for the last fill stage. This is confirmed by the fact that the drainage system seized to discharge water by mid of July. This shorter period can be explained by two reasons: First a higher modulus of compression (increased by consolidation progressing) causes shorter calculatoric consolidation periods. Second the width of the last 3 m of fill on top of the embankment is small compared with the dam's base and so is the additional load compared with the great mass already filled.

Finally the consolidation by plastic drains was recalculated according to Barron (Schneider, 1986), both for old and new soil parameters and for one- and twodirectional drainage. This theory takes into account a zone of disturbance around the drains caused by stitching. The necessary time to reach U_h = 0,9 was calculated from

$$\bar{U}_h = 1 - e^{-\frac{8 \cdot c_h \cdot t}{d_e^2 \cdot F}} \qquad \text{with}$$

$$F = \ln\frac{d_e}{d_r} - 0{,}75 + 0{,}64\, l^2 \frac{k_h}{q_w} + \frac{k_h}{k_v} \cdot \ln\frac{d_r}{d_w}$$

equivalent circle diameter d_e = 2,26 m
diameter of disturbed zone d_r = 0,15 m
characteristic drain length l = 3 or 6 m
transmissivity of drain $q_w = 1*10^{-6}$ m³/s
horiz./vertical permeability k_h/k_v = 5
equivalent drain diameter d_w = 2(b+t) = 66mm

The results for the different boundary conditions are compared in table 3.

This clearly shows that problems had to be expected even in case of a constant filling rate of 60 cm/week. Therefore the horizontal displacements are to be interpreted as an indication of a critical state - safety factor around 1,0. Additional measures would have been unavoidable even in case of no increased filling speed.

Table 3. Time required to reach a degree of consolidation U_h = 0,9 (hours, days)

old parameters		new paramaters	
drainage		drainage	
onedirect.	twodirect.	onedirect.	twodirect.
84 h	50 h	515 h	442 h
3,5 d	2,1 d	21,5 d	18,4 d

old parameters: $k_h = 1*10^{-5}$ cm/s E_s = 6500 kN/m²
new parameters: $k_h = 2*10^{-6}$ cm/s E_s = 3000 kN/m²

9 FILTER PERMEABILITY

The minimum permeability required for the embankment's base was checked by the water pressure to discharge the observed flow of 0,1 l/s:

$$h_w = \frac{L.Q}{2.k.d}$$

with $Q = 5{,}56*10^{-7}$ m³/s, L = 46 m, d = 1,2 m. If you limit the permissible pressure with the thickness of the filter layer (1,0 - 1,2 m = h_w), with $k = 1*10^{-5}$ m/s results h_w = 1,06 m. A permeability which is very low for a filter layer is sufficient for drainage.

CONCLUSIONS

This report shows the problems which can arise from a great variation in soil parameters. From today's view can be said that without close survey of filling works the dam would have failed by base failure because of excess pore pressure. Especially variation in permeability and consolidation most easily exceeds common safety margins for stability. So thorough observation during filling is at any rate necessary to minimize the risk. Furthermore it must be possible to carry out quickly changes in design and additional measures if necessary. In this particular case the high displacements could only be permitted because shear strength showed no significant drop with high deformations.

REFERENCES

Feuerlein, P. (1965). Die Konsolidation planparalleler, unendlich ausgedehnter und wassergesättigter Tonschichten unter starren Lastflächen mit Hilfe von Sanddrains. Bautechnik 4/1965, 120-125.

Grundbau Taschenbuch (1), 1980, 3. ed., 215-228. W. Ernst & Sohn, Berlin, München, Düsseldorf.

Hansbo, S. (1981). Consolidation of fine grained soils by prefabricated drains. Proc. X ICSMFE, (3), 677-682, Stockholm.

Priebe, H. (1976). Abschätzung des Setzungsverhaltens eines durch Stopfverdichtung verbesserten Baugrundes. Bautechnik 5/1976, 160-162.

Schneider, H. et al. (1986). Polyfelt TS - Bemessung und Praxis. Polyfelt AG, Linz.

Vertical Drains (1982). Thomas Telford Ltd., London.