Design and measurements of a reinforced steep slope under motorway Nuernberg – Berlin

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ABSTRACT: In connection with the extending of BAB A9 into a 6 lane motorway, it surmounts an approx. 180m high terrace with 6 different geological formations in the section of the Hienberg ascent. This article reports about the difficult geotechnical circumstances and the special construction measures in respect of the construction and design of a geogrid reinforced soil structure, which is up to 15 m high with an approx. 8 m high embankment on top on a steep slope. The results of strength- and deformation measurements gained by substantial measurement- and control measures are a further central point of the report.

1 THE MOTORWAY PROJECT

Concerning the traffic project of the German unification No. 12: A9 Berlin – Nuernberg, the widening into a 6-lane motorway between the boarder Thuringia / Bavaria and Nuernberg was executed, due to the rapidly increasing volume of traffic caused by the unification of Germany. The approx. 9.5 km long motorway section includes the Hienberg ascent between junction Hormersdorf and junction Schnaittach. In this section the BAB A9 surmounts the terrace between the Franconian mountain region Alb and Rednitzbecken. Because of the steep mountain slopes and the difficult geological situation, the motorway section was divided in two separate roadways and runs separately along the Eastern and Western flank of the hill for about 3.6 km. In order to follow the plans (max. longitudinal slope 4.5%, min curve radius 500m), a new selection of road for a length of approx. 8.4 km was necessary.

During the substantial pre-investigations and the planning procedure (spring 1991 until spring 1993), the possible selection of routes were carefully examined (concentration route west, concentration route east, divided route) and assessed regarding their advantages and disadvantages. Because of the difficult topographical, geological, hydrogeological and structural circumstances, a divided route according to the route of the old motorway was chosen. Immediately after the conclusion of the plan statement procedure in spring 1995, the construction of the bridges Simmelsdorf and Schnaittach and in spring 1996 the construction of the road section started. Already in July 1998 the main construction works had been finished.

In the course of the construction work a reinforced earth construction with a length of 250 m and a height of 15m with an approx. 8 m high embankment on top of it in the direction Nuernberg-Berlin (mountain drive) was built.

The faculty for foundations, ground mechanic and rock mechanic of the Technical University Munich under the chairmanship of Professor Dr. R. Floss were substantially involved by establishing the expert's opinions.

The "Joint Venture Geotechnik Hienberg", consisting of the regional industrial inspectorate Bavaria, and Professor Floss were appointed to execute the performance and quality safety investigations.

2 GEOLOGICAL AND HYDROGEOLOGICAL OVERVIEW

In the section of the Hienberg the motorway crosses the typical stratigraphic sequence of the Southern German hill countries. Layers of Jura, Cretaceous and of the tertiary and quaternary period were found.

The most important stratigraphic sequences and thickness of beds as well as their altitude together with the reinforced earth construction are shown in the scheme drawing figure 1.

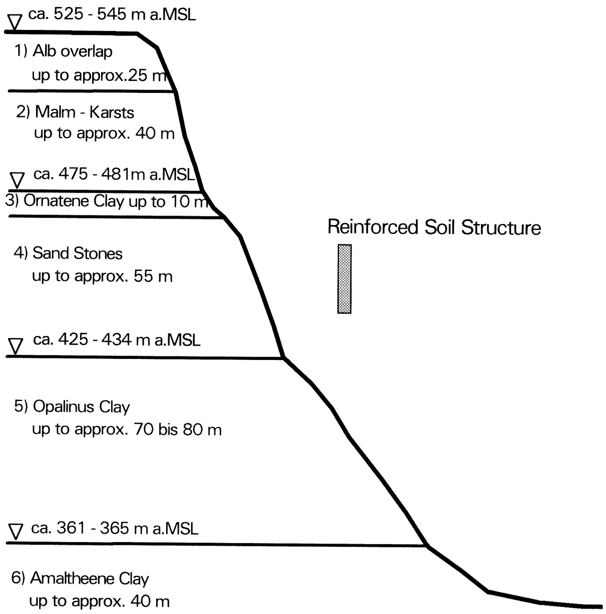

Figure 1. Schematised geological stratigraphic sequence with the altitude of the surface of the reinforced earth body

The strongly rugged surface of the malm karst , which is fissured and interstratified by filled gaps and valleys, is mainly overlapped by variegated sand /interlayered clay beddings (quartz-felspathic sand), clays and marls of the Cretaceous as well as limestone material, loess loam, clay and silt of the Alb overlap (layer 1).

The less strongly karstified malm karsts (layer 2) are found firstly as cleaved up to strongly cleaved, thin-shaly up to thickly bedded, beige-grey, hard, smooth up to shelly breaking limestones. Thin clay layers are interbedded in their bed joints. Beneath it there are mostly cleaved, bedded up to thickly bedded, partly even massive grey, mostly hard limestones.

The layers of the ornatene clay (layer 3), of the opalinus clay (layer 5) and of the amaltheene clay (layer 6) are bedded, mostly cleaved, varying hard and partly carbonate-bounded clay stones and clays of hard consistence, which show varying high contents of fine sand and different weather stages (non-regular stratigraphic sequence of hart clay clods and clay slabs and half stiff and stiff, partly soft clays) up to distinctively plastic residual earth. Gypsum crystals as well as calcareous deposits can be found sometimes on the cleaving surface of water carrying cleaves of the hard clays and clay stones. These rocks show great differences in their compactness according to their composition, their inner bindings and their degree of weathering. The approx. 8 m to 13 m thick, mostly friable weathering aggregates and the distinctively plastic weathering clays react very sensitive to water: they can become slippy and can build flat, up to 8 sloping planes of sliding. The same applies to the mostly bedded sand stones (layer 4), which are cleaved in a different strong way, and which are sometimes platy or stratified and on which surface high-plastic, only a few centimetre thick black-grey layers of clay are interstratified.

These sand stones are also variably hard rocks of different quality. Additionally, interstratifications of clay stones with interlayers of sand stone and non-regular flaser beddings can be found. Between the few hard flaser clays and hard sandy ashlars , different transition beds can be found. Open, water-routing, horizontal and vertical cleaves were found in the sand stones. The firmness of the sand stones, which have the general tendency to get crushed can differ from being hard to very friable.

The weathering layers of the sand stones mostly consist of partly clayish fine sands and medium sands with interstratified sand stone clods.

The Hienberg's stratums have a slide dip to North East. Differences in elevation of the individual layers, possibly caused through fault zones, which run in East-West direction, have been noticed during geological surveys in the region of Schnaittachtal. Only regional existing stratum slopes of partly more than 30 refer to existing, further layer changes through tectonics or much deeper slope movements, for example at the depression of ground in the region of the hill side bridge and of the reinforced earth construction.

There are gravely brook depositions of the Schnaittach, which are overlaid with cohesive flood plain deposits at Schnaittachtal.

The hydrogeological situation is mainly characterised by the different water routings in the cleaved mountains and in the cleaved rock beds. The malm is strongly cleaved and karstified, through which it is able to quickly outlet the rain water. Normally, layers of clay and of marl in malm rocks, ornatene clay, layers of clay and flaser clay in ferruginous sandstone, opalinus clay, amaltheene clay as well as clayish layer of talus are impermeable beds. Groundwater and joint water carrying layers were found in deep malm karst, in sand stones and in brook gravels at Schnaittachtal. Also, in cleaved opalinus and amaltheene clay, a partly strong water onrush and confined ground water was noticed. At the Western and the Eastern slopes of the Hienberg, the surfaces of the above mentioned impermeable beds form line of springs. Their spring water outlets are partly used for the water supply of the surrounding communities.

3 REINFORCED EARTH CONSTRUCTION AT THE MOUNTAIN DRIVE

3.1 *General*

On the mountain drive between km 55+775 and km 56+050, the terrain surface slopes away diagonally to the road direction (natural ground and existing, old embankments) with an incline of up to approx. 28° towards the horizontal line in the East. The gradient of the new road is up to approx. 7 m above the existing motorway surface, so that massive embankments became necessary.

The steep slopes between the dam slopes, which have a normal incline of 1:1,5 and the original surface, would make very long slope lines necessary. Additionally, because of the uncertain conditions of foundation under the existing old embankment (danger of landslide because of the additional, heavy weight of the new embankments), a removal of the old embankments up to the natural existing earth layers was planned, and therefore it was decided to use a steep slope reinforced with geogrids with a frontal incline of 60° in this section. A good optical impression could be gained by maintaining the terrain and the forest, by landscaping the front and by bringing the reinforced earth construction into line with the variable terrain topography.

3.2 *The Construction of the Reinforced Earth Body*

The construction of the reinforced earth body at its top level as well as the adjoining filter-, bottom sealing- and soil improvement layers are shown in figure 2.

As fill material for the reinforced soil structure silty/clayish fine and medium sand from the existing sand stone was used and its shear resistance depending on the degree of compaction was established with triaxial tests. As reinforcement element, the Geogrid Tensar SR110 was installed, which is approved for this use by the German Authority for Construction Technique DIBt. In the slope surface, the geogrids were wrapped around and connected with the above geogrid layer by using a bodkin, so that a closed cushion was built.

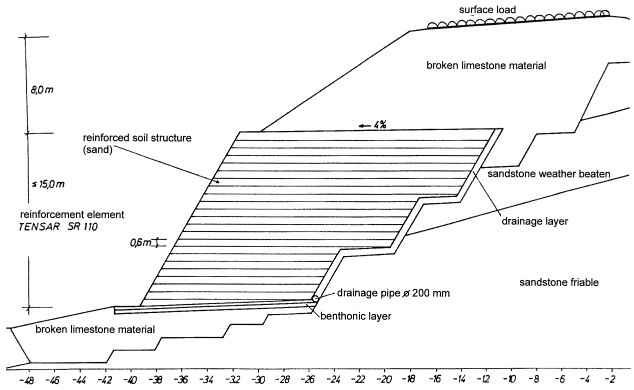

Figure 2. Structure of the reinforced earth body with motorway embankment

The installation, the filling and the control of the used product were executed according to the requirements of the approval (license No. Z-20.1-102) Especially the degrees of compaction (according to the approval requirements) of D_{Pr} > 97% and the deformation modulus, E_{V2} > 45 MN/m^2 (SU, ST) and 80 MN/m^2 (SE,SI, SW) according to the sand materials used were proved. The control of compaction was based on the regulations of ZTVE-StB 94/97. The grading of the sand had been examined continuously.

For the sealing layer of the bento gravel (broken limestone material with bentonite), suitability checks regarding shear resistance and permeability were carried out before.

The reinforced soil structure is completely founded on friable sand stone or on soil improvement layers of broken limestone, which is able to take weight. The good, continuing interlocking of the soil improvement layer with the broken limestone and the sealing layer as well as between the bento gravel layer and the above filter layer are ensured through an intermediate layer of coarse limestone gravel.

The stability of the filter between the existing ground (sand) / sandstone and the limestone gravel, which is used as a filter layer, is ensured by using a mechanical bounded, geotextile separation and filter non-woven.

Because of the slopes, the terrain was filled in front of the nadir of the reinforced soil structure up to a height of approx. 2 m above the footpoint of the reinforced steep slope.

3.3 *Stability Analysis*

3.3.1 *Methods of calculation and safety definitions*

Arithmetical investigations regarding the external and internal stability of the reinforced soil structure were necessary. Hereby a search was made regarding the situation of the unfavourable slip line with the smallest safety factor (outside and inside of the reinforced soil structure). By preference, the arithmetical investigation of the internal stability of the reinforced soil structure was carried out by using straight slip lines and for the external stability by using circular slip lines.

Different methods of calculation were used, for example the method of Bishop (circular slip lines) and according to Janbu (straight slip lines) and rigid block mechanisms (sliding wedge method), which differ from each other regarding the fault mechanism and the safety definition. While the method of Bishop (DIN 4084) refers do the safety of the shear parameters of the ground according to Fellenius' safety definition, the safety at the rigid block mechanism as required by the general approval for use of DIBt (appendix 7 of the approval) refers to the comparison between the retention was necessary for the horizontal balance, as well as for the allowable and available reinforcement elements. The latter are restricted by the design value of the tensile strength and pull out resistance of the geogrid. Because of the comparatively high imposed load of the embankment, the tensile strength of the geogrid is decisive in this particular case.

3.3.2 *Design strength analysis of the geogrid*

Sliding wedge method:

The analysis of the internal stability according to the approval certificate of DIBt (sliding wedge method), a value of

$$\text{perm. } F = F_{B,N} / A_1 / A_2 / \gamma_M =$$
$$= 110 / 2.40 / 1.05 / 1.75 = 24.94 \text{ kN/m}$$

for the approved tensile strength of the geogrid was fixed. (Diminishing factors A_1 for creep, and A_2 for installation damage as well as arithmetical safety ratio γ_M are according to the approval certificate).

The reduction factor A_2 for the installation damage was tested with a trial embankment which resulted in a value of $A_2 = 1.03$ for the used, slightly gravely and slightly stony sand material.

Bishop method:

A value of

$$F = F_{B,N} / A_1 / A_2 =$$
$$= 110 / 2.40 / 1.05 = 43.65 \text{ kN/m}$$

for the long-term tensile strength of the geogrid was used for the calculations using the method of vertical slices according to Bishop and Janbu.

This value shows the characteristic long-term tensile strength of the product, which results from the nominal value of the short-time tensile strength (95% confidence level) under consideration of the reduction factors A_1 and A_2 as approved by the DIBt.

With these values of the strength of the geogrid, safety ratios for the relevant sliding bodies were calculated based on the method of vertical slices according to Bishop and Janbu, which show a sufficient safety level corresponding to the results of the calculation according to the sliding wedge method as approved.

Method of vertical slices (according to EBGEO):

In order to further secure the design values of the geogrid, the highest cross sections at construction stage km 55+825, km 55+837.5 and km 55+850 were carried out according to new partial safety concept of DIN V 1054-100 and DIN V 4084-100 (method of vertical slices).

The limit state 1C (for external stability) and load case 1 (for permanent use) were examined. The partial safety values γ and the design strength of the geogrid $F_{B,D}$ were fixed as follows:

permanent loads:

$\gamma_G = 1.00$

temporary loads, unfavourable:

$\gamma_{Qsup} = 1.30$

soil resistance:

$\gamma_\varphi = 1.25$

$\gamma_c = 1.60$

resistance of the reinforcement (geogrid):

$\gamma_B = 1.40$

$$F_{B,D} = F_{B,KO} / (A_1 / A_2 / A_3 / \gamma_B)$$
$$= 110 / 2.4 / 1.05 / 1 / 1 / 1.4 = 31.18 \text{ kN/m}$$

The limit state 1B (internal stability) and load case 1 for the cross section at construction stage km 55+825 were examined. The chosen partial factors of safety γ and the design strength of the geogrid $F_{B,D}$ were fixed as follows:

permanent loads:

$\gamma_{Gsup} = 1.35$

temporary loads, unfavourable:

$\gamma_{Qsup} = 1.50$

resistance of the reinforcement (geogrid):

$\gamma_B = 1.40$

$F_{B,D} = F_{B,KO} /(A_1 /A_2 /A_3 / \gamma_B)$

$= 110 /2.4 /1.05/ 1 /1 /1.4 = 31.18$ kN/m

With these values, the limit state equation for both limit sates were fulfilled, e.g. a sufficient level of safety according to the new partial safety concept exists.

The following table 1 shows the values of strength of the used geogrid, the design strengths of the geogrid, fixed with the different methods of calculation and the required minimum factor of safety. Additionally, column 3 and 4 of the chart show, which parts of the short-time tensile strength corresponds to the design strength of the geogrid and the corresponding strains at short term load.

Table 1.

Nominal Value of short-time tensile strength: F_{BN} = 110 kN/m (95% confidence level)			
Method of calculation/ Required FOS	Design strength of the geogrid [kN/m]	corresponds to a part of the short-time tensile strength of [%]	corresponding strain of material during short-time tensile strength test approx. [%]
Method of vertical slices acc. to Fellenius η = 1.4 (DIN 4084)	43.7 characteristic long-term tensile strength	40	2.1
Sliding wedge acc. to approval certificate Appendix 7/FOS against rupture η = 1.75	24.9 characteristic long-term tensile strength reduced with γ_M = 1.75	23	1.0
Method of vertical slices/ safety acc. to EBGEO: partial FOS of ground and reinforcements	31.12 characteristic long-term tensile strength reduced with γ_B = 1.4	29	1.3

We have to add, that a tensile strength in the geogrid of F = 10 kN/m is necessary for an equilibrium (η = 1.0) according to the Bishop method (DIN 4084) using safe shear strengths of the fill materials. This strength equals to approx. 9% of the short time tensile strength of the geogrid and is reached at the short time tensile strength experiment at a strain of 0.3%.

Examples of the calculation results by using the method of vertical slices are shown in figures 3 and 4.

3.3.3 *FOS against sliding according to DIN 1054 and FOS against bearing failure according to DIN 4017*

Comparative calculations on a random basis according to the German guideline (EBGEO) under the existing geometrical profiles and site conditions have shown, that at a sufficient safety against sliding according to DIN 4084, a sufficient safety against sliding according to DIN 1054 and a safety against bearing failure according to DIN 4017 is also given.

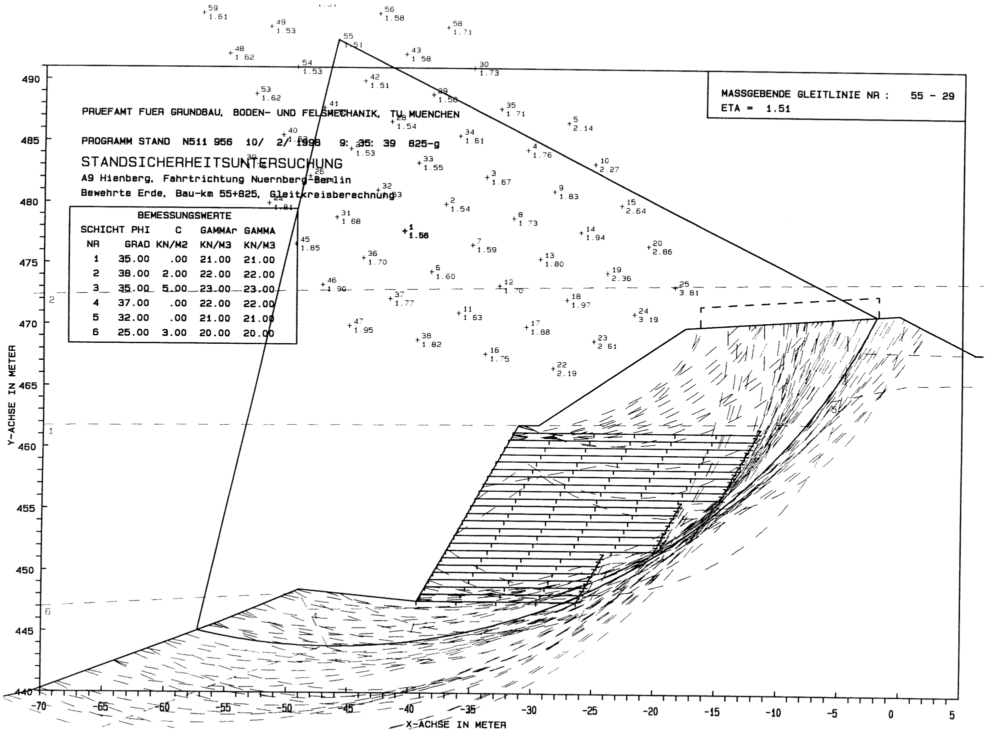

Figure 3. Example for stability investigations with circular slide lines (Method of vertical slices according to Bishop)

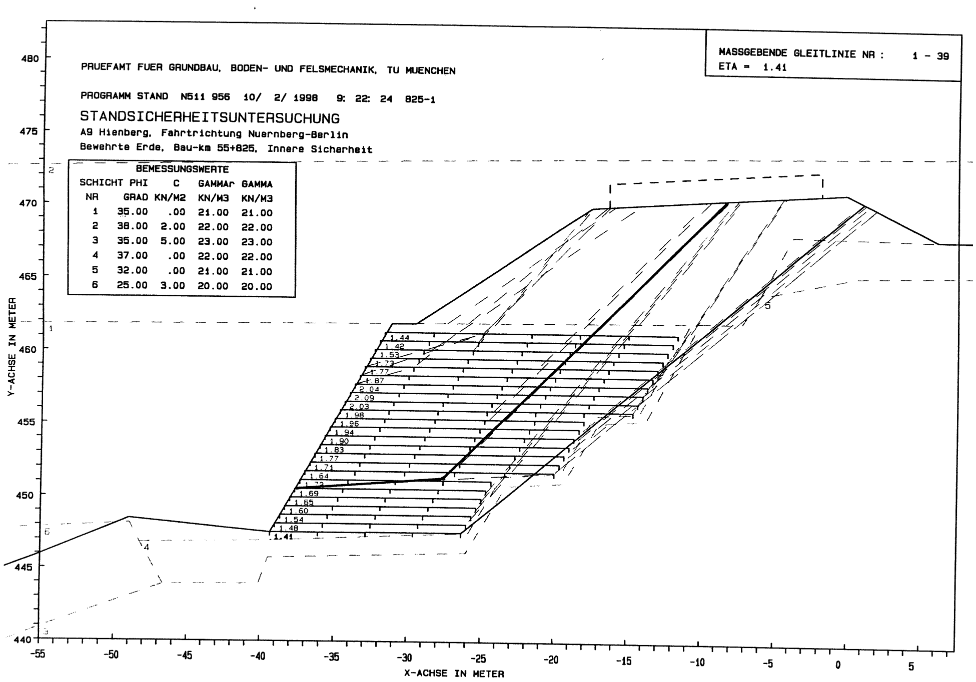

Figure 4. Example for stability investigations with straith lines (Method of vertical slices according to Janbu)

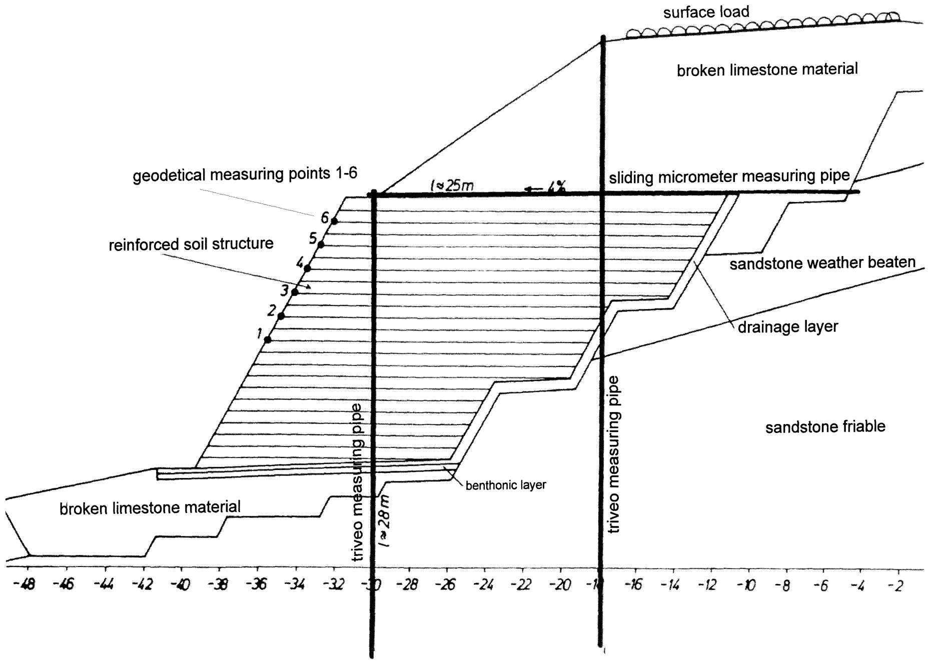

Figure 5. Measuring section with measuring equipment for earth pressure and extensions

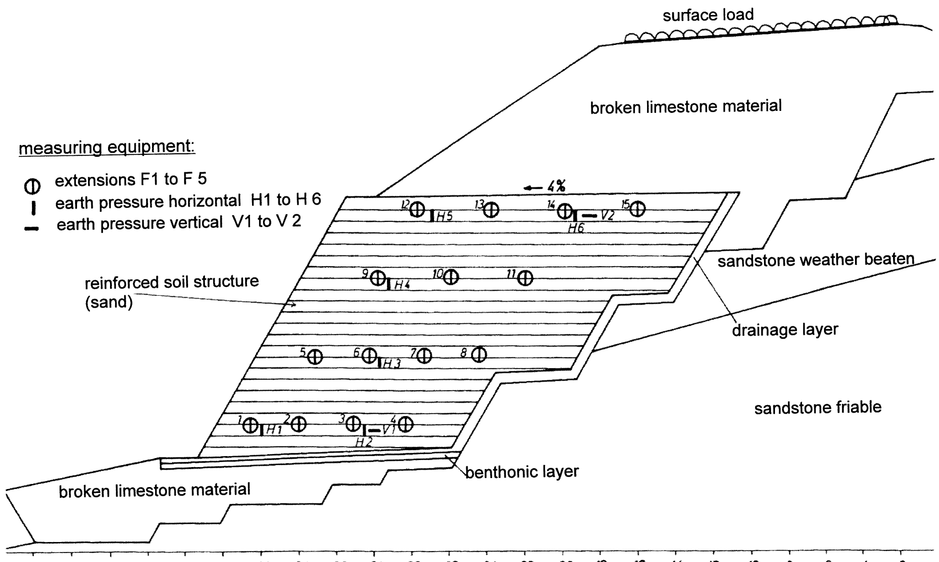

Figure 6. Measuring section with measuring pipes for directional deviation and deformation

3.4 *Method of observation*

The method of observation of the construction during the construction works was according to the new design guidelines based on the partial safety concept (DIN V 1054-100 and DIN V 4084-100) as well as on the seize and meaning of the site.

Instruments for measuring earth pressures and strains of the geogrid had been already installed on the geogrids when building the reinforced soil structure at km 55+850 and the measuring procedure was observed (figure 5). Additionally, fixed measuring marks (No. 1 to No. 6) were fitted under the 60 sloped surface of the reinforced earth body and measured geodetically (figure 6).

For controlling the deformation in the reinforced soil structure, developed during the filling of the reinforced soil structure, one Triveo-measuring pipe and a gliding micrometer measuring pipe each were installed at the jointing point to the embankment at construction stages km 55+847 and km 56+000 and corresponding measuring were carried out in tight sequences (Figure 6). Finally, after finishing of the embankments in the before mentioned cross sections, one additional Triveo measuring pipe each was installed from the embankment shoulder for long-term observations. The location of the described measuring instruments as well as the bottom edges of the measuring pipes are shown in figures 5 and 6.

With this measuring equipment it was possible to permanently observe the deformations of the construction and the interactions between construction ground and construction itself. An automatic measuring system with data storage was connected to the described measuring equipment for earth pressure and deformation of the geogrid. The measured data can be called up via a mobile modem at each point of time, which makes it possible to continuously observing the construction also in the future.

3.5 *Measured results of the measuring section at km 55+850*

The strains in the geogrid measured during the set up of the reinforced earth body (F1 until F4) together with the earth pressures V1 (vertical) and H2 (horizontal) in the section of the lowest measuring system No. 1 (figure 5) depending on time, are shown in figure 7 as an example.

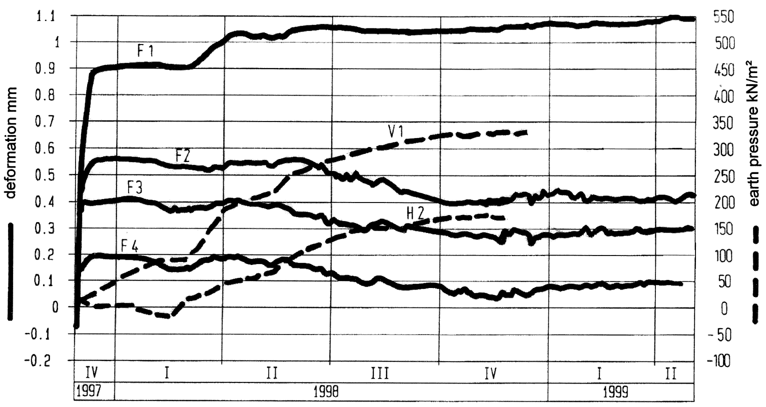

Figure 7: Strains of geogrids F1 – F4 and earth pressures V1 and H2 during construction of the reinforced soil structure (until May 1998) and under the increasing weight caused by the embankment (May until June 1998); locations of measuring points according to figure 5

Comparable to these measured results are the results of the other geogrid layers which were equipped with measuring instruments. They showed the maximum strain near the surface of the slope (compare: measuring instruments F1, F5, F9 F12 in figure 5). Small deformations of the grid were noticed at increasing distances from the slop surface (compare: measured results F1 – F4 in figure 7) The maximum measured extensions until finishing of the reinforced earth body in May 1998 regarding the measuring section of 300 mm each amounted to approx. $\varepsilon = 0.33\%$.

The strains in the geogrid, which increased in March 1998 and from May 1998 onwards, are caused by the restart of the construction work after the winter break and caused by the increasing weight of the reinforced soil structure through the above 8m high embankment. Significant increases of the strains in the geogrid caused through the embankment are only noticed at the measuring instrument F1, where up to then maximum strains of the geogrid of a total of approx. ε = 0.36% were measured. At the geogrids no creep-strains were noticed, under constant weight between July 1998 (opening to the traffic) and May 1999.

The measured, vertical earth pressures V1 and V2 corresponded very well to the existing heights of embankment at the time of the measuring.

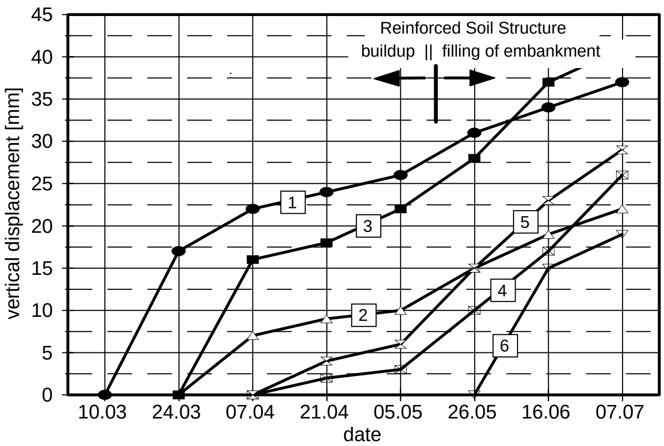

Figure 8: Geodetic measurement (measuring points 1 to 6 according to Figure 6), vertical displacement of the face of the reinforced soil structure.

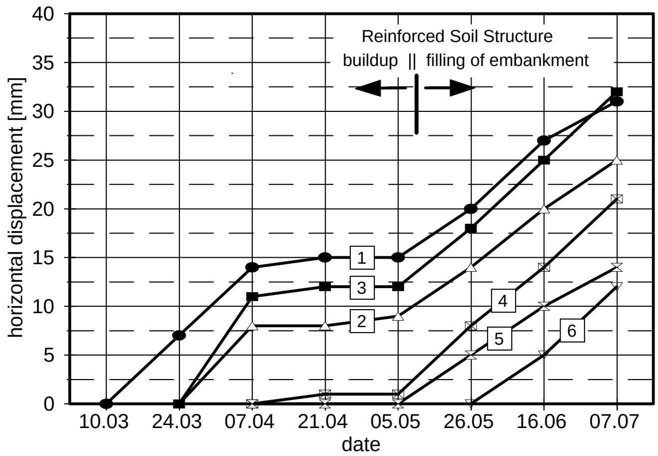

Figure 9: Geodetic measurement (measuring points 1 to 6 according to Figure 6), horizontal displacement of the face of the reinforced soil structure.

The vertical (figure 8) and the horizontal displacement (figure 9) at the face of the reinforced soil structure as well as deformations (figure 10), measured with the Triveo- and the Inclinometer were lower values than expected. During the filling of the embankment, the reinforced soil structure, the horizontal displacement measured geodetically at the front of the reinforced soil structure were bigger than the ones in the Triveo measuring pipe where a displacement of about 5mm were measured in the area of the berm. The same applies to the settlement within the area of the berm, which was measured during the filling of the embankment.

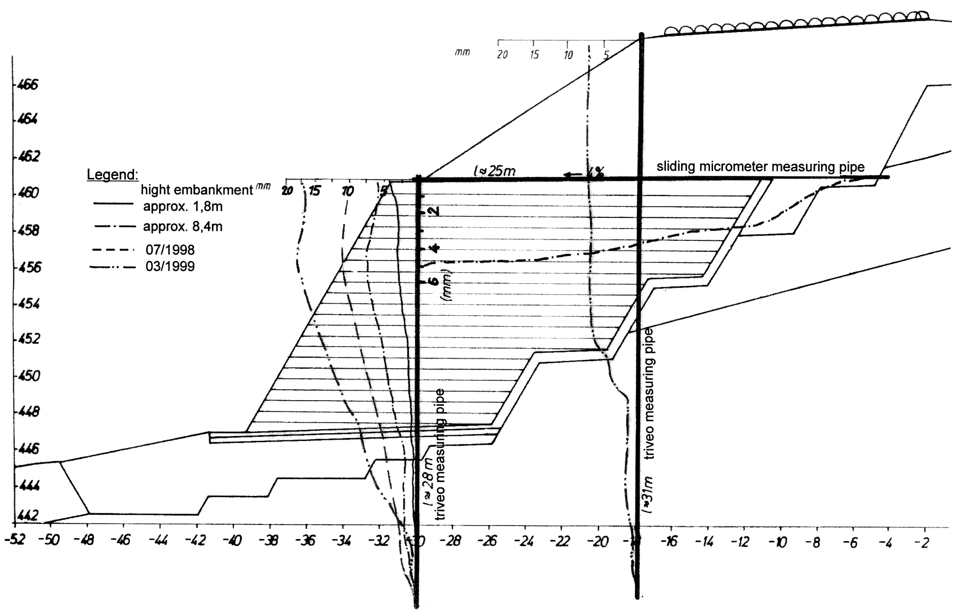

Figure 10: Measurement of the horizontal displacement by using the Triveo measuring pipe and measurements of the extensions within the base of the dam by using a sliding micrometer measuring pipe at the base to the dam embankment (top of reinforced soil structure, berm) during the build up of the embankment on the reinforced soil structure, and in March 1999 – 8 months after opening to the traffic.

A settlement of about 10 mm were measured with the Triveo measuring pipe, and of about 18 mm were measured geodetically (measuring point 6 in figure 8). The reason for that might be, that the measuring were not carried out at the same point of time.

Since finishing the construction, time-depending horizontal displacement of about 11 mm were measured in the cross section km 55+850. According to the present measuring results, these deformations occurred outside of the reinforced soil structure within the existing mountain. This indicates a movement of the entire reinforced soil structure (monolithically connected through the reinforcements) on the existing underground, which must be observed further.

4 CONCLUSIONS

The reinforced soil structure at the Eastern slope of the Hienberg and its large embankment were finished by the end of June 1998. Up to now, the measured results, measured by using measuring- and controlling instruments for stress and deformation, have been lower than expected, so that we can assume that an additional safety reserve exists. This shows once more, that extraordinary constructions in difficult slope locations can be economical erected by using geogrid-reinforced soil structures. The measurements will continue for a longer period of time, in order to observe the long-term behaviour of the construction.