Experiences on stabilisation of landslide in South Black Forest

H.-G. Kempfert & D. Zaeske
Institute of Geotechnique, University of Kassel, Germany

M. Stadel
Kempfert + Partner GmbH, Kassel, Germany

ABSTRACT: This paper presents the experiences on a redevelopment of a landslide in South Black Forest. A big part of a hillside was sliding with velocities up to 90cm/year downstream and endangered a building located above the creep zone. To reduce the movement of the soil masses, a doweling of the hillside was executed. The redevelopment consists of a serie of piles, that are embedded in the stable underground and anchored backwards at the pile caps. Due to the support at both ends of the pile, the load distribution is advantageous and enables an economic design of the piles. The analytical formulations used to determine the forces in the dowels will be reflected and the success of the redevelopment will be verified by results of measurements and calculations with the FEM.

1 INTRODUCTION

In Spring 1991 a landslide in South Black Forest was realized. The area affected by the landslide is located uphill the country road No.132 between the villages Badenweiler and Sehringen. The moving soil mass on the hillside covers an area about 80 m in North-South and 50 m in West-East direction and creeps in dependence of the intensity of precipitation with creep velocities between 10 and 90 cm/year. The resulting movement of the hillside became so large, that a redevelopment was necessary, in particular, a building on the hillside was in danger to suffer damages.

The concept of the redevelopment provides constructional reinforcements to reduce the creep movement of the bed load on an acceptable degree. This report describes the landslide that appeared before the redevelopment was executed. The effectiveness of the redevelopment will be analyzed with numerical calculations with the FEM and compared with present measurements.

2 LANDSLIDE

The concerned area is situated between the country road 132 in the west at a height of 497 m NSL and a hospital in the east at 522 NSL. The hillside has an inclination of 13°, the situation is illustrated in Figure 1. Simplifying, the formation has a structure of two layers: a loose soil layer, which forms the creeping soil, and clay stones, which forms the deeper stable underground.

The thickness of the loose soil layer is between 9 and 16 m and is made of clayey soil in the upper 4 to 5 m, which has less coarse grain particles with mainly stiff, partially plastic or semi-solid consistency. The deeper section of the loose soil layer down to the claystone consists of loam and/or debris with fine fractions of clay, sand-clay or sand-silt mixtures. The loam and/or debris is partial soaked and softened due to water supply. The stable underground is formed by clay stone of the opalinus-clay (Braunjura alpha). The stratigraphical sequence is a serie of uniform dark-grey foliated clay stones, which could not be divided by petrography. The clay stone in the upper 3 m is softened to greybrown clay in semi-solid and solid consistency. Several slickensides were determined in the clay stone, which indicate significant tectonic pretensions by the mountain.

The hillwater was found in different levels. It flows above the watertight layer of claystones in the creeping bed load to downstream.

The movement of the bed load is recorded by inclinometer - measurements since October 1991. The measurements indicate that the creeping bed load demarcates itself legible from the stable underground.

The movement of the soil mass takes place on a thin sliding plane, that is located more or less parallel to the surface of the hillside. The depth of the sliding plane for a characteristic profile of the hillside is illustrated in Figure 2.

The boundary line of the creeping soil upstream is

observed by detected gaps in the soil close underneath the area of the hospital. The sliding plane is located at a depth of 11 to 12 m, partially up to 15 m and has an almost constant inclination of 10° to 13°. The area affected by the landslide extends downstream until 200 or 300 m behind the country road. Before the redevelopment was made in Dec. 1995, the recorded creep velocities of the moving soil mass were between $v_o = 10$ and 90 cm/year.

The reason for the increasing and decreasing rates of creep velocities is the seasonal changing of the amount of precipitation, which influences the hill water conditions and the groundwater flux, however this effect can not be quantified exactly. The total lateral displacement of the road has reached a value of about 1,0 m since the measurements started in Oct. 1991. The consequences of the displacements in the area of the road are shown in Figure 3.

3 REDEVELOPMENT MEASURE

3.1 Concept of the measure

The concept of the redevelopment was to stabilize the creeping soil masses by security measures with the destination to reduce the forward movement as far as no danger for the hospital appears in foreseeable future.

The redevelopment consists of 15 tied-back piles with a diameter of 120 cm at spacing of 4 m, each pile has a length of 20 m and is embedded in the stable underground, they are placed about 20 m from the country road in the hillside. The pile caps are tied backwards with two injection anchors, which have an inclination of 25° from horizontal. In ground level, all caps are connected with a girder, that has a total length of 60 m.
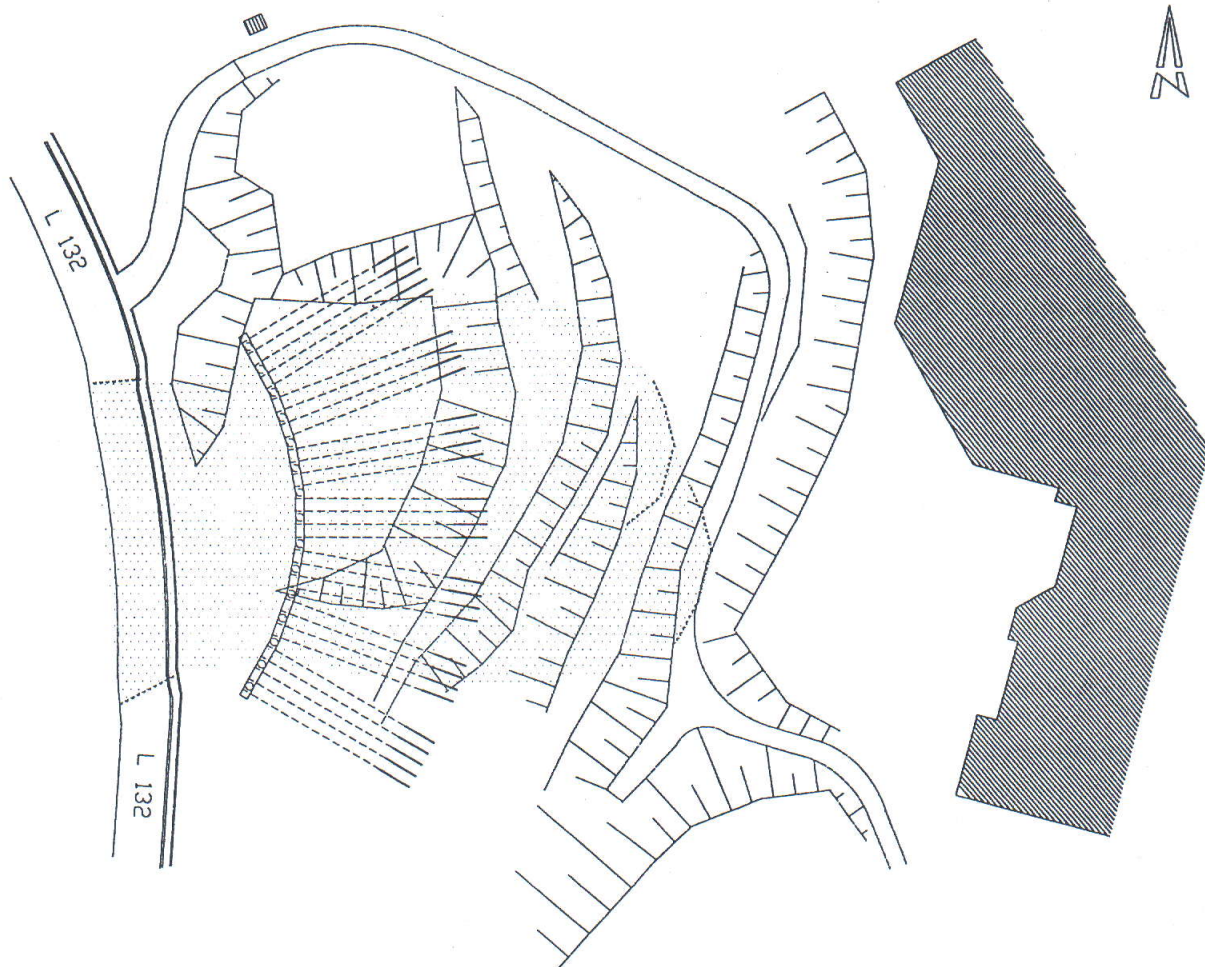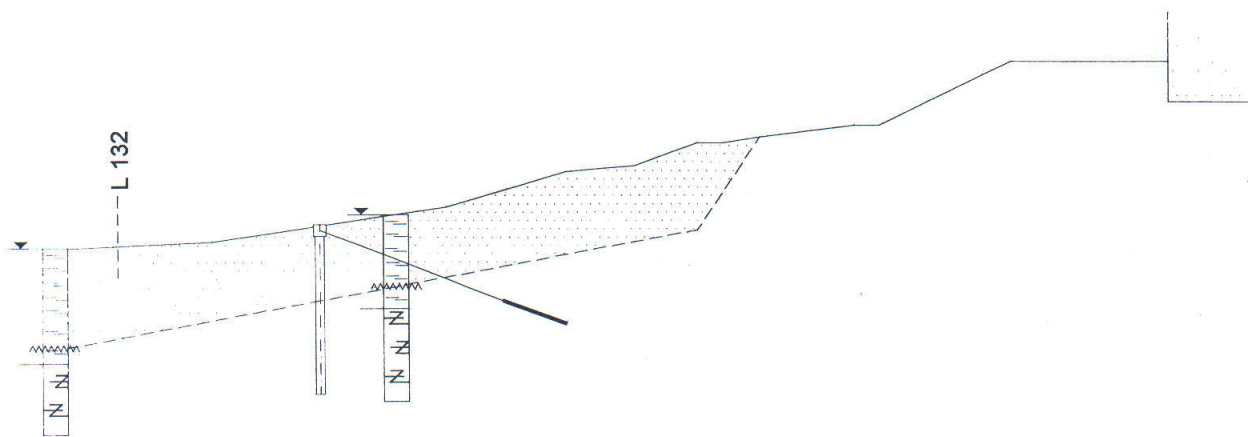

Figure 1. Plan of the redevelopment

Figure 2. Profile through the hillside

The girder is used for the absorption of the anchoring forces and for compensation of the different bearing capacities of the piles. Due to a better bearing behaviour of the construction and a more suitable integration in the existing nature, the piles were disposed in arched configuration. The anchoring of the pile caps was necessary, because the sliding plane is located in a relative deep horizon, the stressing of only one end restrained piles would become too large for the chosen pile diameter. In the hillside, a drainage-system was also installed to reduce the hydraulic forces caused by the hill-water.

3.2 Design of the redevelopment

To estimate the forces absorbed by the structural elements of the redevelopment, two different analytical formulations were considered. The first assumption is based on the state of equilibrium between acting and resisting forces in the sliding plane. The stability against sliding of the bed load is defined by the ratio of the resisting forces and the acting shearing forces along the slide direction.

$$\eta = \frac{G' \cdot \cos\beta \cdot \tan\varphi + H}{G' \cdot \sin\beta + S} \tag{1}$$

where G' is the effective gravity load of the soil, β the inclination of the sliding surface, φ the angle of shear friction in the sliding plane, H the tangential resisting forces and S the hydraulic thrust of the hill-water per metre.

Originating from the assumption that the landslide occured before the redevelopment has a safety factor of 1,0 and from the known location of the sliding surface, the decisive angle of shear friction in the sliding plane was determined. Water level located 5 to 7 m above the sliding plane was also taken into account. Using the established angle of shear friction, the necessary resisting force to increase the safety factor from 1,0 to 1,15 was calculated.

The second formulation avoids the difficulties in determination of absolute values for the calculation like the effective angle of shear friction in the sliding plane or the hill-water conditions to simulate the equilibrium state correctly. The required resistance force along the sliding surface to reduce the creep movement can be obtained from a logarithm toughness law for soil (Kreuter&Lippomann, 1993)

$$H = l \cdot \tau_0 \cdot I_{va} \cdot \ln\left(\frac{v_0}{v_1}\right) \tag{2}$$

where l is the length of the creeping soil body, τ_0 the average shear stress along the slide plane, I_{va} the viscosity index of the soil, v_0 the initial creep velocity and v_1 the aimed creep velocity.

The average shear stress in the sliding plane can be determined by the relation

$$\tau_0 = \gamma_r \cdot h \cdot \sin\beta \cdot \cos\beta \tag{3}$$

with γ_r = weight of the saturated soil of the bed load and h = depth of the sliding plane.

Assuming, that in future no unforeseen and unfavourable conditions will occur, which will lead to greater creep velocities than 90 cm/year, the target of the redevelopment was to reduce the creep velocity to a value smaller than 0,5 cm/year.

Both formulations lead almost to the same value for the necessary resisting force, that is about 350 kN/m. For the design of the construction, the calculated resisting force was converted to a line load acting on the piles above the sliding surface. In account of the anchored pile caps, the load was assumed to be distributed uniformly along the pile. The part of the piles beneath the sliding plane was idealized as a subgrade reaction with constant modulus of subgrade in horizontal direction. The acting forces in the structural elements of the redevelopment, determined at this system, enabled the design of the pile, the girder and the anchors.
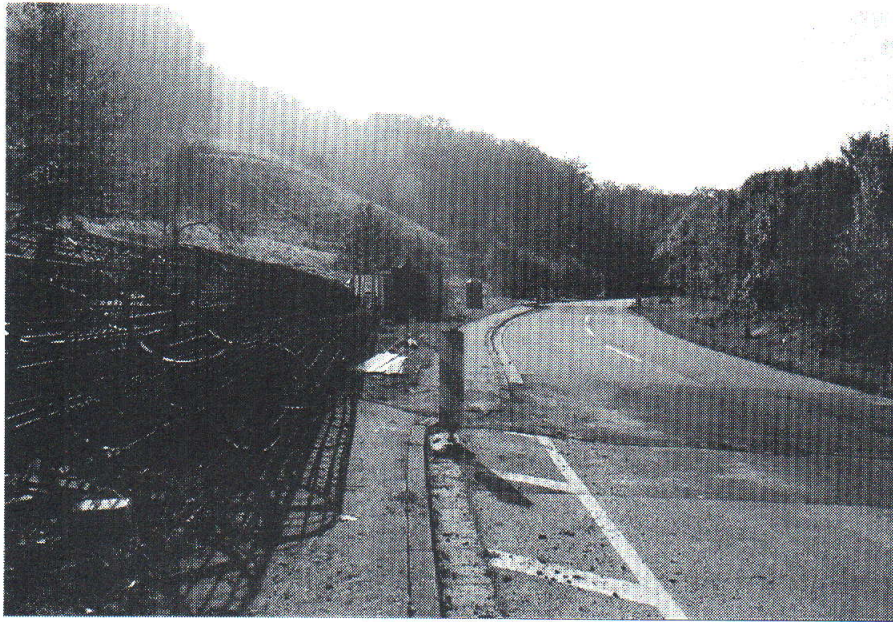
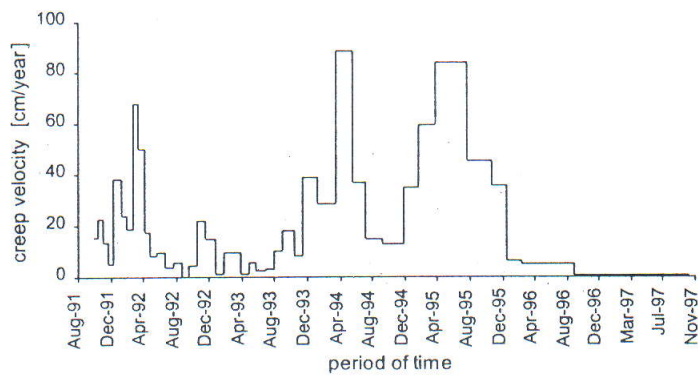Figure 3. Situation at the road before the redevelopment

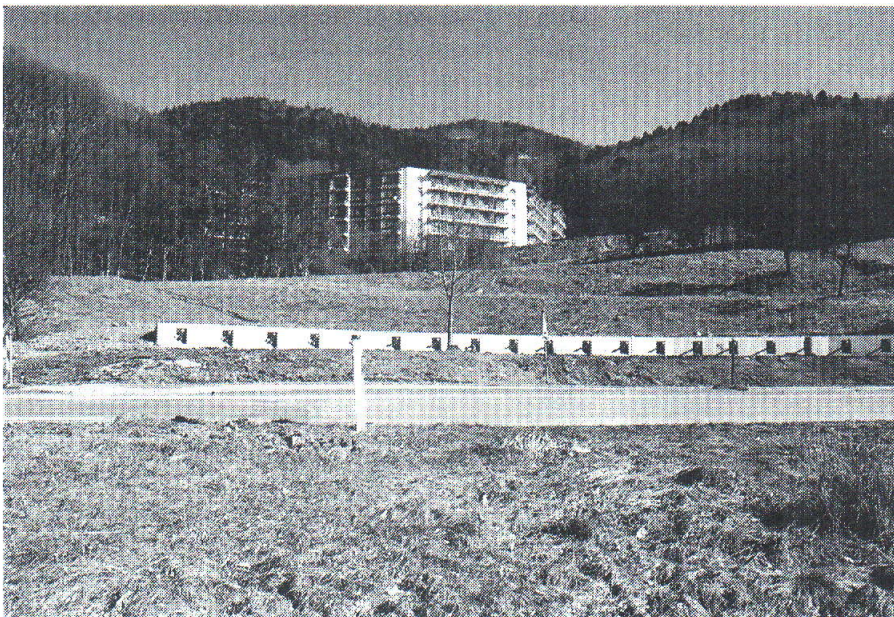Figure 4. Measured velocities of the creeping soil mass

Figure 5. Redevelopment measure today

4 NUMERICAL EXAMINATIONS

The effectiveness of the redevelopment will be compared with the results of numerical calculations with the FEM. Only a small section of the hillside has been considered in the calculations. Using the properties of symmetry, a section of the construction was extracted and translated in a three-dimensional model for the FEM, see Figure 6.
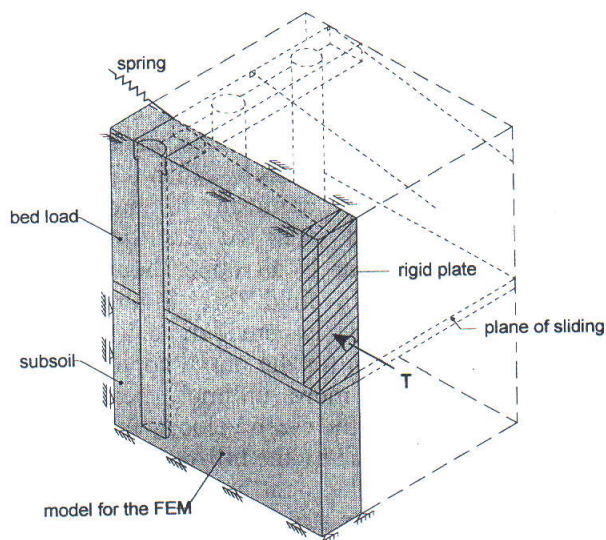

Figure 6. Model for the FEM

The tied-back piles are simulated by only half a pile in the model, whereas the connection between the pile and the girder is assumed to be fixed. The anchor is simulated by a spring with an external load in the size of half of the value of the prestressing. Boundary conditions are defined with free displacement in direction of the creep movement for the bed load and fixed for the stable underground. Between the bed load and the subsoil a thin layer is included to simulate the sliding surface. The FEM-model is build up with second-order hexahedral solid elements. To determine sliding of the soil along the surfaces of the pile and the girder special contact elements are implemented, they allow relative displacements and cracking between soil and pile. The material behaviour for the bed load and the subsoil is assumed to be elastic, hence, the elements in the plane of sliding have viscous behaviour by decreasing shear modulus with time.

The pushing force from the hillside is simulated by a horizontal load acting on a rigid plate at the end of the hill-section. This plate ensures a continuos movement of the bed load, the same as observed by inclinometer measurements. The total shearing force along the sliding surface is calculated on the base of equation (3) and yields the total load T, acting on the rigid plate (Figure 6). To obtain correct values for the time depended shear relaxation modulus, in a first calculation the pile and the girder were removed by replacing the material parameters of the concerned elements with the values of the surrounding soil. The viscous damping coefficients were calibrated, in such a way, that a creep velocity of 90 cm/year was achieved, which corresponds to the maximum value before the redevelopment. The material parameters are shown in table 1

Table 1. Material parameters for the FEM

section	Young's modulus	Poisson's ratio
bed load	5500 kN/m^2	0.35
subsoil	55000 kN/m^2	0.30
pile / rig. plate	$3 \cdot 10^6$ kN/m^2	0.00

The final Model including the redevelopment was analyzed in two steps. First, the external load is applied instantaneously in a simple static step. In a second calculation step the load was held constant and the shear relaxation effect was applied in 20 time increments, where each increment represents a duration of five days, so a total time of t = 100 days was inspected. Figure 7 shows the deformed mesh after the second calculation step at t = 100 days.
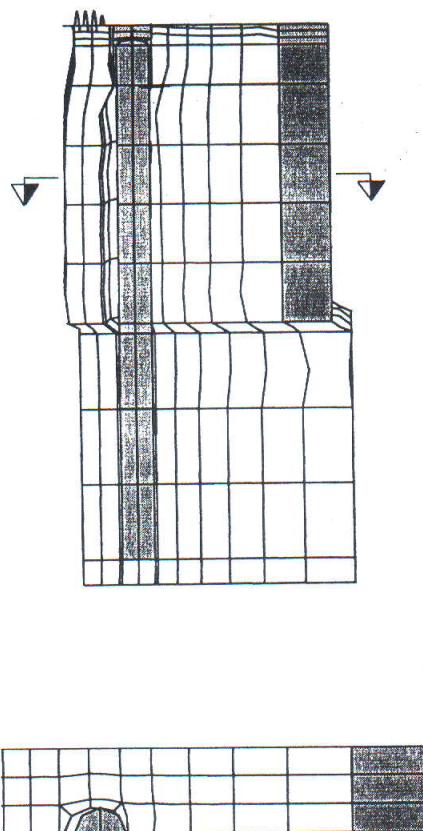

Figure 7. Deformed mesh

The displacement of the rigid plate, which represents the movement of the bed load, is illustrated in Figure 8. After the redevelopment was completed, the slope failure decreases due to retardation, which is a time-depend process. To judge the effectiveness of the redevelopment, Figure 8 also shows the creeping displacement in the initial state correspondending to $v_o = 90/365 = 0,247$ cm/day.
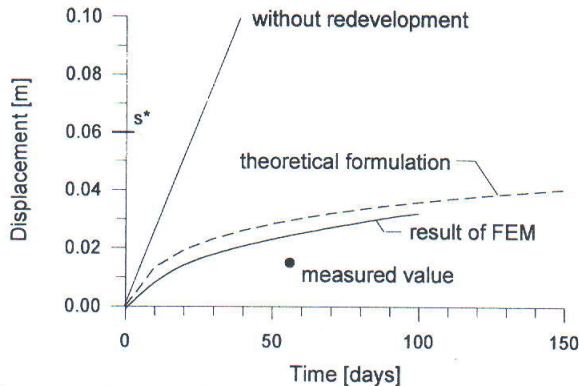

Figure 8. Creeping displacements of the bed load

For the calculation with the FEM, the material behaviour for the bed load and the subsoil is assumed to be linear elastic without any limit of admissible stresses. Factual the stresses increases with strains only until a defined yielding pressure is reached (Winter, 1980). The yielding pressure is given by several authors in the range of $2,5...7,5 \cdot c_u \cdot d$, where c_u is the undrained shear-strength and d the diameter of the pile. The point of time when the yielding pressure is attained depends on the relative displacement between the pile and the surrounding soil. The magnitude of the relative displacement, s^*, necessary to mobilize the yielding pressure can calculated by the equation Winter's (1980):

$$s^* = \varepsilon_u \cdot d \qquad (4)$$

ε_u is the limit strain of the soil and is supposed with $\varepsilon_u = 5\%$ for the bed load. The total time t^* until the yielding pressure occurs, can be calculated by (Schwarz, 1987) from

$$t^* = \frac{s^*}{v_o \cdot \ln(v_o / v_1)} \cdot \left(\frac{v_o}{v_1} - 1\right) \qquad (5)$$

In this case, after $t^* = 839$ days, the pressure on the pile does not increase further and a constant creeping velocity of $v_1 = 0,5$ cm/year is present. If the pile, connected resistant to bending with the girder, is assumed to be nearly undisplaceable, which agrees with the results of the FEM, the motion of the bed load, shown in Figure 8 displays also the relative displacement between pile and soil. In account of the regarded duration of 100 days for the numerical calculation, the assumption of elastic behaviour for

the bed load is admissible. The increment of displacement before the yielding pressure is reached, can be derivated as a function of time t by

$$s = \frac{s^*}{\ln(v_o / v_1)} \cdot \ln\left[1 + \frac{v_o \cdot \ln(v_o / v_1)}{s^*} \cdot t\right] \qquad (6)$$

The results of the theoretical formulation are shown in Figure 7.

Both the results of the FEM and equation (6) leads to larger displacements, than indicated by the inclinomter-measurements, where 53 days after the redevelopment was done a movement of only 1,5 cm was detected. Next inclinometer results are presented for this profile after 255 and 487 days and show very small additional displacements less than 0,3 cm. This is tendentiously conform to anticipation, because the supposed value of the initial creep velocity of 90 cm/year is an upper bound of the measured velocities in field (see Figure 4); the factual mean annual is smaller. Due to the pessimistic assumption for the initial conditions, the calculation leads to more untimely results. The seasonal variations of the creep velocity in the initial state are primary caused by the hill-water conditions. The executed redevelopment includes a drainage system, that reduces the streaming potential, thus the shearing force decreases. This effect does not enter into the theoretical calculations of FEM or formulation (6).

CONCLUSIONS

The redevelopment reduces the sliding of the hillside significantly. The measurements after the redevelopment indicate very small movements of the bed load, whereas analytical methods forecast the success of the measure slight untimely. Since the redevelopment was executed two years ago (Dec. 1995), the creep velocity reduces continuously over a time interval of about 839 days and approaches a limit velocity of about 0,5 cm/year to be attained in spring 1998.

REFERENCES

Kreuter, H. & Lippomann, R. 1993. Planung und Überwachung von Verdübelungsmaßnahmen zur Sanierung kriechender Hänge. Tiefbau, Ingenieurbau, Straßenbau 7: 491-493.

Winter, H. 1980. Bemessung von Pfahlgründungen und Hangverdübelungen auf Fließdruck. Vorträge Baugrundtagung Mainz 1980: 563-593

Schwarz, W. 1987. Verdübelung toniger Böden. Veröffentlichung des Institutes für Bodenmechanik und Felsmechanik der Universität Fridericiana in Karlsruhe. Heft 105