Stabilization of the Outenberg Hill in Geraardsbergen

R. De Paepe
Ministry of Public Works, Brussels, Belgium

M. Wallays
Research and Geotechnics, Franki, Liège, Belgium

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Secretary General, Ministry of Public Works, Brussels, Belgium

M. Wallays
Manager Research and Geotechnics, Franki, Liège, Belgium

SYNOPSIS. A road, winding across the "Oudenberg" hill at Geraardsbergen, Belgium, and built partly in embankment and in excavation, suffered from three landslides from 1937 to 1966. Bored piles were used to nail the surface layers to the stable substratum, subsurface drainage to limit the fluctuations of the water table and a 220 m long low viaduct founded directly on bored piles to re-establish the traffic. Since the end of the works no additional movement appeared.

PROBLEMS TO BE SOLVED

In Geraardsbergen, on the western side of the "Oudenberg" hill, which stretches from approximately el.+17 to over +100, the Guillemin boulevard winds between el.+35 and +70 in the steepest area of the hill, where surface slope varies between 10 and 12.5°. The hill, of tertiary age, consists of very fine micaceous sands and silty clays (Yd, lower Eocene) resting on a layer of clay (Yc). Above it, fine sand with occasional sandstone beds (Pld) is found. From available geological data, the separation between the layers is approximately situated at el.+55 for Yd and Pld and el.+23 for Yd and Yc.Yc layer stretches down to el.-10. When the sliding equilibrium is close to 1, this downwards succession of layers with decreasing permeability favours the formation of slides in long raining periods. The road, carried out in 1932, is laid out partly on embankment in the upper part (ref.A in fig.1 and 2) situated at the North of "Pavillon Bellevue" and partly in cut (ref.B in fig.1 and 2) in the downward area. In the hair-pin section (fig.1), the road progressively passes from a mainly filled to an in cut cross-section.
Already, in 1937, a large slide occurred and stretched for at least 200m, the drop in height being more than 3m locally. The cross-sections were re-established by means of fills intersected with transverse drains and a bottom drainage blanket made using stone wastes protected by straw. The repaired slopes received a stone facing over a large extent. In 1945, after

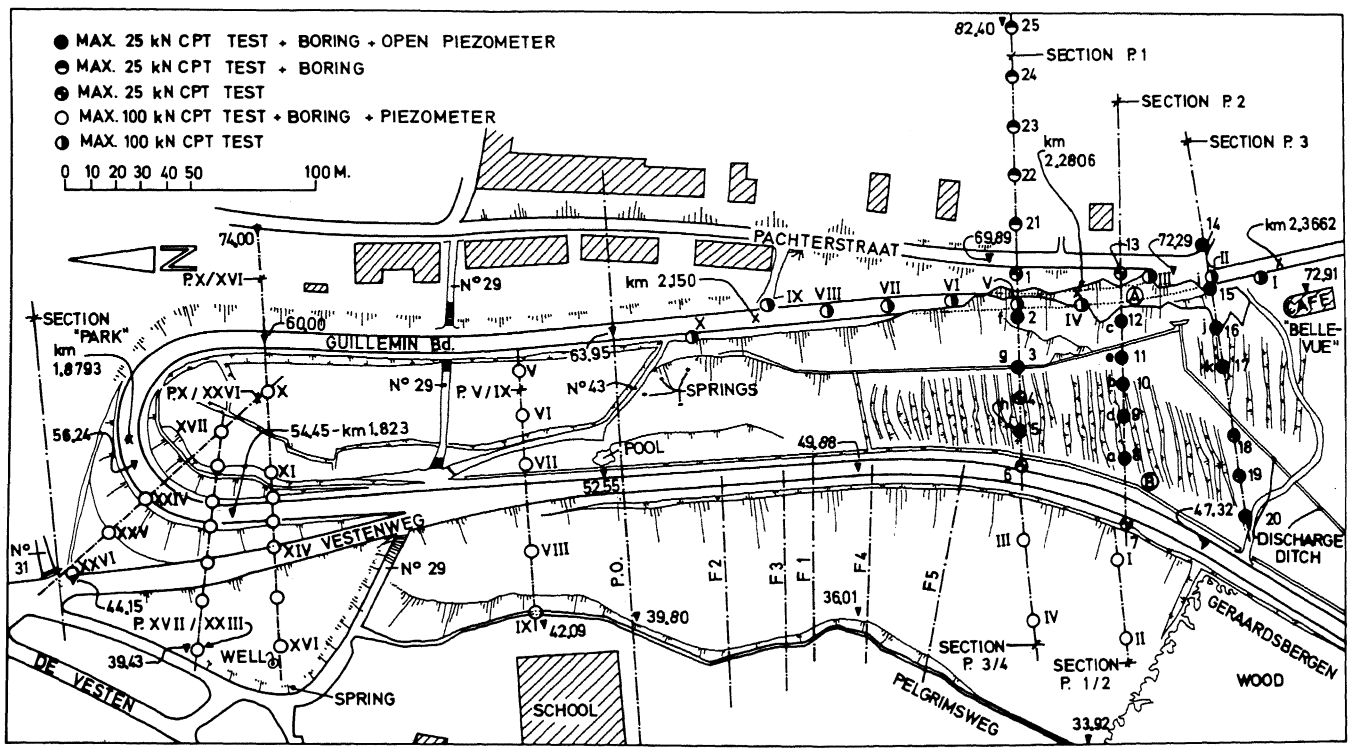

Figure 1

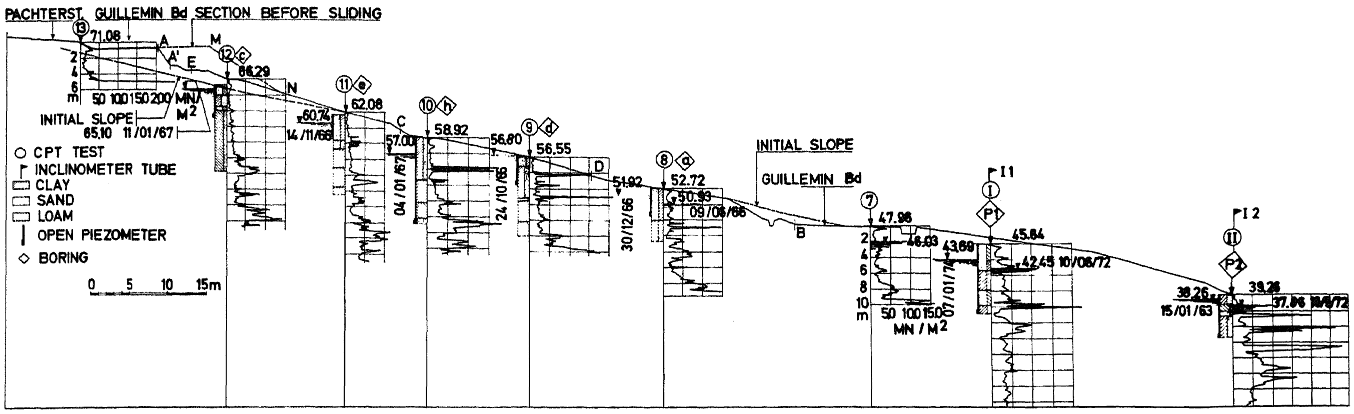

Figure 2

torrential rains, new slides occurred in the bend of the hair-pin for 57m extent with a 1.90m maximum drop, and in the upper arm of the boulevard for about 100m extent with 1.25m to 2.25m maximum drops. The initial cross-sections were once more re-established with drainage pipes installed at the bottom, and the downward slope covered with a concrete stone masonry facing. Moreover, the water discharge system along the whole new linking was improved, and in particular, a large water discharge ditch was installed below "Pavillon Bellevue" (fig.1).

The stability of the hill has always been considered as precarious, since after each incident the responsible Road Service in Ghent improved the drainage of the soil and the water discharge. The Service had also installed the series of more or less deep drainage ditches in the zone AB in fig.1. In Feb. 1966, another slide (fig.3 and fig.1) affected the upper area of the boulevard to an extent of 180 m approximately with a maximum drop of 3.20m. Half-hill (ref.B in fig.1 and fig.2), the bulge of soil formed by heaving the side-path of the boulevard shows that at the spot of section P2, the slide had spread over 80m approximately in the direction of the steepest slope. Thus, within approximately 30 years, three successive slides affected the same area of the hill.

The Ministry of Public Works consulted Prof. De Beer, who insisted not to change the relief resulting from the 1966 sliding, and proposed the solution to re-establish the traffic and stabilize the hill. Franki was entrusted with the study which has been carried out in close collaboration with Prof. De Beer. The Road Service was responsible for the supervision of the works, which were executed by Franki.

SOIL INVESTIGATION AND SHEAR STRENGTH PARAMETERS

With a view to the stabilization works, 25 borings with undisturbed and disturbed samplings and installation of open piezometers and 25 max. 25KN CPT tests were carried out along the three sections P1, P2 and P3 approximately parallel with the line of the steepest slope (fig.1 and fig.2). For the foundation of the low viaduct, 10 max. 100kN CPT tests were carried out (fig.1).

A detailed survey of the hill revealed that its stability was also precarious along the whole extent of the boulevard. In the Vestenweg (fig.1), the 20-30° inclination of the trees showed the significant creep occurred. Below the lower part of the boulevard, the concave shape of some poplars towards the top of the hill evidenced the slower creep of the soil and thus a slightly better local stability of the hill.

Around the hair-pin, 22 borings with undisturbed and disturbed samplings, installation of open piezometers and casings for inclinometer measurements and 22 max 100kN CPT tests were located in the four marked sections PV-IX, PX-XVI, PXVII-XXIII and PX-XXVI in fig.1, and below the slide zone 4 borings and 4 max. 100kN CPT tests in the sections P1/2 and P3/4.

From the analysis of the data, the soil consists of relatively fine mixed strata made up of clayey sand, sandy and silty clay. A well-defined stratification does not appear. The area over +65 to +70 is sandier and has sandstone beds.

The water level generally appears between the surface of the soil and 2 to 3 m depth. In the upper strata, no water was found above +66.6. Depending on rain and the position of the piezometers in the hill, the piezometric level fluctuates from 0.75m to 1.75m depth. In some places, the piezometric level is situated at more than 1m above the ground surface, e.g. piezometers j and h. Springs arose locally (fig.1).

The shear strength parameters obtained from 37 cell tests are :

- central value : $\varphi' = 30°$, $c' = 0$ (1)
- average value : $\varphi' = 30°$, $c' = 0{,}9\ kN/m^2$ (2)

and from 45 consolidated undrained triaxial tests (127 specimens) with pore pressure measurement :

-regression line : $\varphi' = 35°$ $c' = 4{,}2\ kN/m^2$ (3)
-line on the safe side : $\varphi' = 31{,}4°$ $c' = 0$ (4)

When the samples are distributed according to the soil nature, very close mean values are obtained, so finally it was decided to consider all the triaxial samples together.

Alternatively, the shear strength parameters of the soil can be approximately deduced by studying the stability of the slope AMNCDB (fig.2) which actually slipped, those along which sliding has not occurred, (AMNC and AMND) an

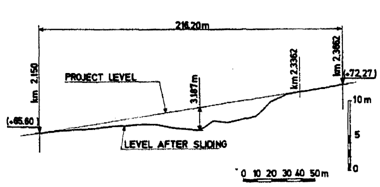

Figure 3

the one along which the slide has stabilized (AA'NCDB). For these different slopes, the safety factor s against sliding respectively satisfies the conditions (5) to (8). The

$$s_{AMB} \leq 1;\ s_{AMC} > 1;\ s_{AMD} > 1;\ s_{A'NB} > 1 \quad (5)(6)(7)(8)$$

calculation was made for the section P2, obviously the most critical one, considering, because of lack of knowing the piezometric levels during sliding, 6 positions of the phreatic surface among the highest ones observed, and assuming series of 4 values of φ' and c'. The safety factors have been determined according to Bishop's method (1955). The four conditions are simultaneously fulfilled when c' ⩾ 7kPa. Hence, the adopted shear strength parameters :

$$\varphi' = 19° \qquad c' = 7\ kN/m^2 \qquad (9)$$

These values are much lower than the ones deduced from the laboratory tests. Three reasons can explain this apparent relatively large discrepancy :

1. it is likely that during sliding, the phreatic surface occupied a higher position than the six ones taken for the calculation.
2. because the sliding is a progressive phenomenon, the shear strength is only partly mobilized during it.
3. the slip surface is not necessarily circular but fits the soil in order to develop as much as possible in the areas of low shear strength.
   So, the stabilization has been studied with the values (9).

PRINCIPLES OF THE STABILIZATION

Basic Principles

The stabilization of the hill has been carried out by using two main methods :
- by increasing the safety against sliding by nailing by means of bored piles the surface layers of the hill to the stable substratum.

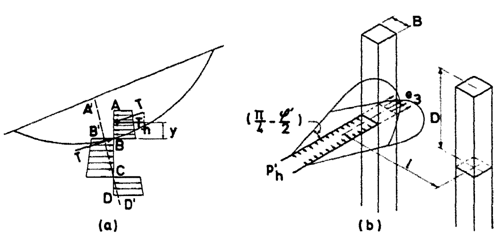

Figure 4

- by limiting the fluctuations of the water table by means of sub-surface drainage.

The nailing has been studied so that after the works, the safety factor against sliding, defined relevant to the shear strength parameters, reaches at least s = 1.25 for the highest phreatic surface observed and s = 1.10 assuming that the phreatic surface falls with the surface of the soil. On the other hand, the sub-surface drainage is foreseen to prevent the phreatic surface from rising on the average to less than one meter depth.
Consequently, as long as the drainage works well, the safety factor is close to s = 1.25 and in case of widespread clogging, it still remains s = 1.10. In order to avoid this clogging as best possible, the criteria for composition of the drains have been strictly applied.

The bored pile was preferred to the displacement pile to avoid the generation of excess pore waterpressures.

Method of Calculation of the Nailing

Shear force to be resisted by the piles

When the safety factor s_o against sliding without any pile is close to 1, because of creep phenomena, the soil above the potential slip surface tends to go through limited deformations compared with the substratum. The piles oppose the relative displacement resulting from it through the stabilizing force T (fig.4a). The safety factor against sliding becomes $s > s_o$.
Nowadays, because of the impossibility to determine the contact pressures, the piles are dimensioned from the rupture theory. The slide is supposed to occur and the necessary force T to obtain a determined value s of the safety factor (1.25 or 1.10) is estimated. In a first approximation, T is supposed to act at the intersection with the pile along the tangent to the slide surface.

Pressures on the piles.

The displacement of the mass situated above the slip surface (fig.4a) provokes the dragging of the pile from the position ABCD to the position A'B'CD'. The directions of the differences of the passive and active pressures have been represented in the figure.
The passive-active resultants have been estimated by means of three different methods :

1- By Brinch Hansen's method (1961), when the spacing ℓ between the piles compared to their diameter B is large enough in order that no interference appears between the adjacent piles. The resultant pressure e_1, the force q_1 by unit length and the average stabilizing pressure p_1 on the soil caused by the piles are given respectively by the equations (10) to (12), k_{qr} and k_{cr} being factors

$$e_1 = k_{qr}.(\gamma'D) + k_{cr}.c'_r \qquad (10)$$

$$q_1 = e_1.B = k_{qr}.(\gamma'D).B + k_{cr}.c'_r.B \qquad (11)$$

$$p_1 = \frac{1}{\ell} = k_{qr}(\gamma'D).\frac{B}{\ell} + k_{cr}.c'_r.\frac{B}{\ell} \qquad (12)$$

depending on the reduced friction angle φ'_r, which increase with the relative depth D/B.

2- In the second method, the piles are assumed close enough to admit that they form a continuous wall. In this case, in Brinch Hansen's method, the pressure factors to introduce are k_{qro} and k_{cro} for D/B = 0.
The equations (13) and (14) respectively give

$$p_2 = k_{qro}.(\gamma'D) + k_{cro}.c'_r \qquad (13)$$

$$q_2 = k_{qro}.(\gamma'D).\ell + k_{cro}.c'_r.\ell \qquad (14)$$

the values of p_2 and q_2 in this case.

3- The third method due to Prof. De Beer (De Beer, Wallays, 1970) determines (fig.4b) the passive pressures e_3 acting on the pile from Jaky's failure surface (1948). The minimum horizontal pressure p'_h on both downdrag planes is assumed to be linked to the at rest pressure by Rankine's formula. e_3 is estimated from the equations (15) and (16). This method implicitly assumes that the

$$e_3 = tg^2\left(\frac{\pi}{4}+\frac{\varphi'}{2}\right)e^{2\pi tg\varphi'}(p'_h + c'cotg\,\varphi') - c'cotg\varphi' \qquad (15)$$

$$p_h' = (1-\sin\varphi')(\gamma' D)\,tg^2(\frac{\pi}{4} - \frac{\varphi'}{2}) - 2c'\,tg(\frac{\pi}{4} - \frac{\varphi'}{2}) \quad (16)$$

adjacent piles do not interfere between themselves and that the pressures are calculated at relatively great depths. q_3 and p_3 are calculated by taking into account a safety factor s_3.

$$q_3 = \frac{1}{s_3}.e_3.B \quad (17)$$

$$p_3 = \frac{q_3}{\ell} = \frac{1}{s_3}.\frac{B}{\ell}.p_3 \quad (18)$$

The pressure factors introduced in these three calculation methods correspond to the case when the surface of the soil is horizontal. By approximation, they are applied in the case of a sloping surface.
In order to make sure that the calculated rupture pressures are lower than those, which would arise in case of an effective sliding, safety factors are introduced.
For the first two methods (Brinch Hansen), the rupture pressures are calculated with reduced values φ_r' and c_r' by introducing the reducing factors $F_\varphi = 1.2$ on $tg\varphi'$ and $F_c = 1,5$ on c'.

$$\varphi_r' = arctg\frac{tg19°}{1,2} = 16° \quad (19)$$

$$c_r' = \frac{7}{1,5} = 4,67 \text{ kN/m}^2 \quad (20)$$

In De Beer's method, the reduction affects the calculated pressure by introducing:
$s_3 = 2$ (21)
In each case, the lowest value among p_1, p_2 and p_3 is adopted for the pressures at the depths where they are exerted.

Maximum bending moment and necessary length of embedment of the piles.

Once the resulting pressures on the piles known, it is easy to calculate the heights AB, BC and CD (fig. 4a) on which they exert. The resultant of the unit forces q on AB equals the horizontal projection T_h of T. The heights BC and CD are determined by the transversal equilibrium ($\Sigma T = 0$) and rotation ($\Sigma M=0$) conditions applied at the base D of the nailing pile. Hence, the maximum bending moment and the theoretical length of embedment BD of the pile.
The stabilizing moment around the centre of the slide surface is lower than the one considered first, because T_h acts at a certain height y above the point B (fig.4a) and the lever arm of T_h at most equals the one of T. The involved slight decrease of the safety factor must be balanced by a corresponding increase of T_h.
The piles were uniformly reinforced because of the uncertainty concerning the distribution of the pressures which actually act on the piles.
For safety reasons, the theoretical length of embedment of the piles (BD fig.4a) has been increased by 20 per cent as a rule. The slip surface, along which is found the highest value for the stabilizing force T, is not necessarily the one which requires the greatest length for the piles. Deeper slip surfaces at the spot of the row of piles can require smaller stabilizing forces but may lead to a greater length of the piles. So, the length of the piles has been determined by considering different slip surfaces giving values among the lowest for the safety factor, and the greatest among the so-calculated lengths has been kept back.

CONCLUSIONS OF THE STUDY, REPAIRING AND STABILIZATION WORKS

The traffic has been re-established by means of a 220.5m long viaduct (fig.5). Its width is 15 m including the footpaths. The viaduct consists of 7 sections and is founded on 50 rows of 3 bored piles 0.81m diameter ending upwards in a column. The spacing between the rows is 4.50m and between the piles in the rows 4.30m.

Figure 5

The main data resulting from the study are summarized in table I.

Slide Section - Zone where the Slide Occurred

The slope to be stabilized stretches between the downward row of the foundation piles for the viaduct (point E, fig.6) and the lower part of the boulevard (point B). Nevertheless, the piles must prevent a slide extending to the foot of the hill (point G). In these cases, are obtained, after sliding and before installation of piles, for the slope EB, the safety factor $s_o^{meas} = 1.19$ for the phreatic surface measured on 13.12.66 (the most unfavourable) and $s_o^{surf} = 0.95$ for the phreatic level coinciding with the ground surface, and for the slope EG, $s_o^{meas} = 1.18$ and $s_o^{surf} = 1.01$. The potential slip surfaces for the phreatic level coinciding with the ground surface are represented in fig.6. These s_o values underline how sensitive to the variations of the phreatic surface the safety factor is, the 13.12.66 phreatic surface being only situated as a whole at about

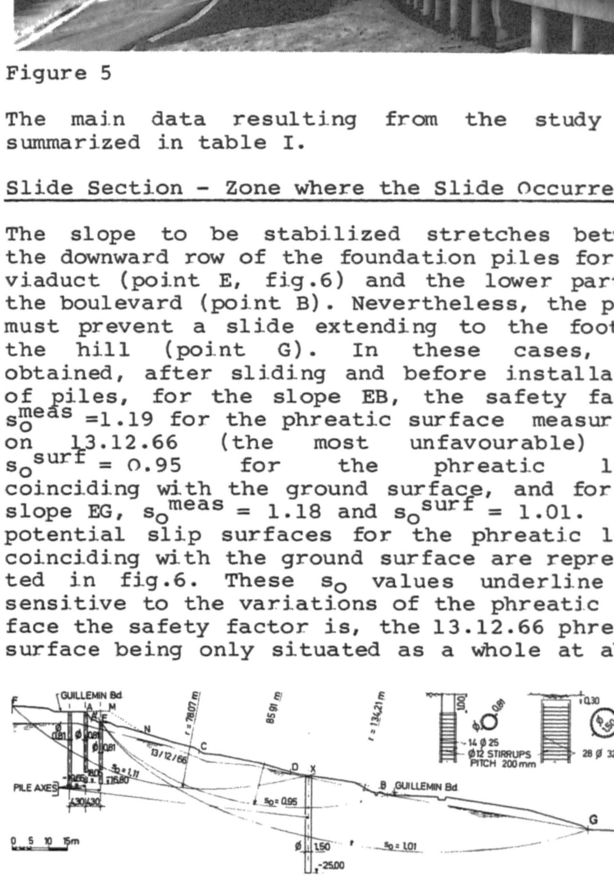

Figure 6

1.50m depth. The row of piles has been approximately located at the vertical where the slip surface EB for the case of the phreatic level coinciding with the ground surface shows a horizontal tangent. The given conditions $s \geqslant 1.2$ and 1.10 are satisfied when T = 281 kN/m B = 1.50m diameter piles have been installed a ℓ= 4.50m. Their length, required to prevent slide starting from B or G (fig.6), is 25 m an the maximum bending moment is 1740 kNm.
The values 35 MPa for the concrete cubu strength and 400 MPa for the reinforcement stee were considered for all the piles installed.
The allowable stresses were limited respectivel to $\sigma_c' = 10$ MPa and $\sigma_s = 240$MPa. In the slide

Table I

Area	slide section											Hair-pin			
Zone	slidden			downwards the slide											
Section	P2			F2	F3	F1	F4	F5	P3/4	P1/2	Park	P-X XXVI	PXVII-XXIII	PX-XVI	PV-IX
Potential slip surface (fig.6)	EB	EG	from F to X												
A. WITHOUT PILES															
s_o for phreatic surface															
- on 13.12.1966	1.19	1.18	1.31	-	-	-	-	-	1.23	1.52	-	1.29	0.99	1.05	1.19
- at the ground surface	0.95	1.01	1.11	1.07	0.92	0.86	0.82	1.00	1.14	1.31	1.09	1.05	0.87	0.87	1.01
B. WITH PILES															
Diameter (m)	1.50		3x0.81	1.50	1.50	1.50	1.50	1.50					1.50	1.50	1.50
Spacing (m)	4.50		4.50	4.50	4.00	3.00	3.00	4.50					2.50	1.75	2.00
Stabilizing force T(kN/m^2)	281		3x80	116	326	579	418	235					550	835	950
s for phreatic surface															
- on 13.12.1966	1.24		1.43	1.25	1.25	1.25	1.25	1.25					1.25	1.25	1.25
- at ground surface	1.08		1.25	1.10	1.10	1.10	1.10	1.10					1.10	1.10	1.10
Max. bending moment(kNm)	1740		380	480	2650	3730	2710	2260					3020	4970	3860
Embedment (m)	4.70		(vert. load)	3.19	6.76	7.20	7.04	5.75					7.06	10.57	6.94
Theoretical length (m)	24.7			17.5	20.0	24.5	20.0	18.0					23.0	29.6	37.1
Reinforcement															
-percentage (%)	1.25		1.25	0.7	1.71	2.84	1.77	1.49					2.06	4.55	3.06
-bars:number,diameter(mm)	27ø32		13ø25	20ø28	24ø40	40ø40	25ø40	21ø40	-	-	-	-	29ø40	64ø40	43ø40

zone, a lower limit of 1.25 per cent has been introduced for the reinforcement ratio, in order that the condition of the fine fissuration is approximately fulfilled.
The required safety factor against sliding for the piles sustaining the viaduct must be larger than respectively 1.25 and 1.10 in order to limit the possible deformations. Before installation of the piles, $s_o^{meas} = 1.31$ and $s_o^{surf} = 1.11$ are calculated. After installation of the piles, $s^{surf} = 1.25$ has been required. For the 13.12.66 phreatic surface, it corresponds to $s^{meas} = 1.43$.

Slide Section - Zone Downwards the Slide and Hair-Pin Area.

The hazard of a slide in the lower zone of the hill downwards the boulevard (BG, fig.6) and in the hair-pin area was studied by considering the sections P1/2, P3/4, F2 to F5 (fig.1) on one hand and PV-IX, PX-XVI, PXVII-XXIII, PX-XXVI and "Park" on the other hand.
In the zone downwards the slide, s_o^{surf} passes by a minimum 0.82 in the section F4. The nailing has been limited at the section P3/4 but the sub-surface drainage was installed up to the poplar plantation, whose limit cuts the section P1/2 (fig.1). The pile spacing and their actual length were arranged in order to obtain a continuous variation.

In the hair-pin area, the values $s_o^{meas} = 1.05$ and 0.99, obtained in the sections PX-XVI and PXVII-XXIII, are very close to 1 and are well in accordance with the important creep deformations observed. From the inclinometer measurements in the casing XIV located in the Vestenweg (fig.1) between February and August 1973, the upper layer of the ground, 19m thick, has crept regularly and more or less uniformly. At the end of this time, the downward relative movement of the ground surface was about 30 mm. The nailing was limited at the outlet of the Vestenweg into the Vesten, which is close to the lower point of the section PX-XXVI, but the sub-surface drainage was extended to the way n°31 (fig.1) in order to cover the area of the sections PX-XXVI and "Park". The necessary stabilizing force T varies between 550 and 950 kN/m. With 1.75m, 2.00m, and 2.50m spacings, the maximum bending moment grows up from 3020 to 4970 kNm, so that the reinforcement ratio varies between 2.06 and 4.55 per cent. The piles to be installed theoretically every 1.75m were placed in triangular pattern in order to bring the spacing side to side from 0.25m to 0.50m, and so to facilitate the seepage water run-off. The safety factor s_o^{meas} in the sections PXVII-XXIII and PX-XVI is close to 1, and thus close to the safety factor of the section P2 before sliding. But no slide occurs in sections PXVII-XXIII and X-XVI. The comparison in both cases of the necessary stabilizing forces T, the spacings and pile reinforcements, underlines how large is the expense to be accepted in order to prevent a soil mass at the limit of the equilibrium from sliding.

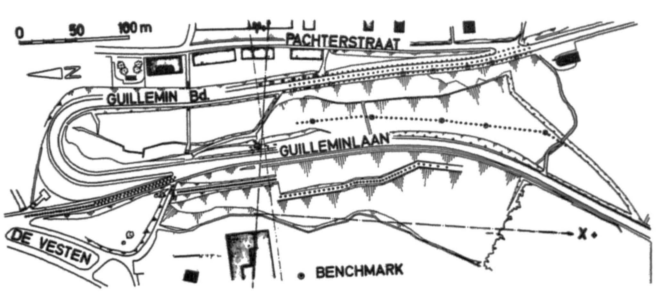

Figure 7

Summary of the Works of Stabilization by Nailing

Fig.7 reproduces the location of the bored piles used. The reinforcement works in the Oudenberg hill concern a width of about 500m and a measured length along the line of the steepest slope of 200m approximately and a level difference of about 40m. 150 piles, 0.81m diameter and 20.9m average length and 213 piles 1.50m diameter and 27.2m average length were installed.

Sub-surface Drainage

The sub-surface drainage system consists of a network of 1.25 m depth draining trenches (fig.8a), filled with fine sand and located every 4,50m along the line of the steepest slope of the hill. The drains consist of 160mm

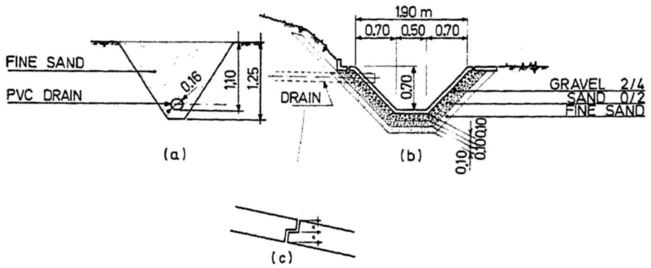

Figure 8

diameter PVC tubes with a network of longitudinal slits. They are lined with a 10 mm thick fiber-glass draining layer not only on the common sections but also on the connectors, elbows and the upper ends. The fiber-glass is used to prevent entry of the fine sand and as a protection against the rodents. At the lower end, a stainless steel lattice-work jacket is fitted on the PVC tube and concreted into the concrete ditch elements. This arrangement assures the drainage and prevents the rodents from penetrating inside the tubes. The jacket material has been chosen in order to be child-proof.
The discharge ditches (fig.8b) are made of reinforced concrete elements. The joints are arranged (fig.8c) in order to facilitate the clogging of the joints by fine particles swept along by the surface waters. The elements of the ditch rest on a draining blanket made of three layers in order to prevent piping. The used materials for the draining trenches and blankets were determined by strictly complying with the criteria for composition of the draining layers. Pipes were only used where ditches could not be accepted.
Fig.9 gives a plan view of the drainage and discharge networks. The total length of the draining trenches is 10,167 m.

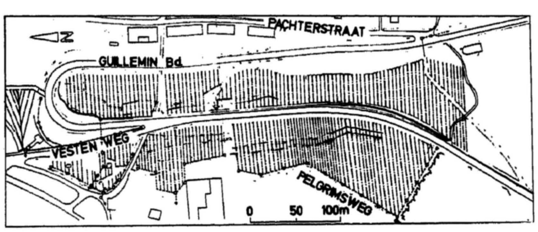

Figure 9

TOPOGRAPHICAL SURVEY SINCE THE END OF THE WORKS

Ten benchmarks were installed at the points indicated in fig.7, five on the 0.81m piles of the downward line sustaining the viaduct and five on the line of the 1.50m piles in the slide zone. From reference measurements taken in February 1977, the displacements parallel to the horizontal axes X and Y were surveyed by five measurements stretching until March 1981. In fig.10 are graphically represented the extreme and central values of the displacements ΔX and ΔY versus time. No trend can be noted from these figures. Only secondary factors such as the ambient temperature, sunshine, and unavoidable reading errors influenced the measurements. Until now, the piles have carried out their function as stabilizing nails for the hill.

CONCLUSIONS

The Oudenberg hill is in precarious equilibrium, in its steepest part, about 40m high. The relief alterations due to the laying out of the road has made this situation worse. The relatively large variations of the ground water surface as a result of rain and the downward decreasing permeability of the successive soil layers,

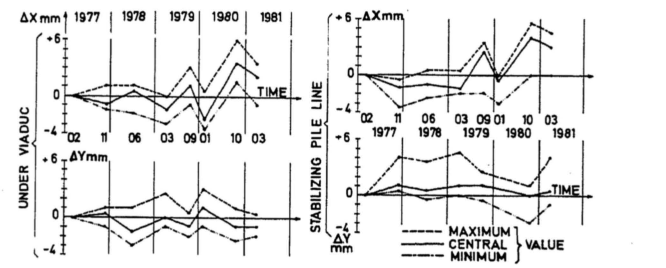

Figure 10

provoke, from time to time, relatively large slides when the water table surface rose too much. Such accidents successively occurred in 1937, 1945 and 1966. In 1966, in order not to re-establish the fill supporting the road, a 220m long low viaduct was built. In addition, in the hair-pin section, the fill which was damaged by the sweeping down of the materials has been replaced (De Paepe, Wallays, 1982).
The hill has been stabilized thanks to the nailing of the surface layers to the substratum, and by a sub-surface drainage of the whole unstable area. The study has been carried out using a safety factor of about 1.25. In case of widespread clogging of the draining system it neverthless will remain at least 1.10. The works were carried out from 1971 to 1977 and the topographical survey shows that no additional movement appeared since then.

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