the isostatic lines illustrated by these curves, the approximate center of the critical circle was more readily ascertained. On the basis of these analyses, the 80-foot cut was selected as the most economical section for the pumping plant.

CONCLUSIONS.

The results of the field investigations and laboratory testing indicated that although the materials were sedimentary, lenticular, clayey sands and sandy clays with some pockets of sand and clay, the friction values were fairly uniform and the tan φ values were closely grouped about 0.50. The cohesion varied considerably and a minimum cohesion value of 450 pounds per square foot was selected for the slope analysis.

The sections shown in Figure 1 were determined by the stability analysis. Under the design conditions, the slopes have a safety factor of 1.4 to 1.5. These slopes were in close agreement with those recommended by the field after they had inspected existing cuts (up to 50 feet deep) in the general vicinity. As a result of the analyses, field studies, and economical considerations, this design of the 80-foot cut was adopted and construction, which is now under way, will be completed in the spring of 1948.

-o-o-o-o-o-o-o-

IV c 11 FAILURE OF A BIG RETAINING WALL AT " WIENFLUSS " CREEK-SHORE IN VIENNA. VI

Dr. R.F. TILLMANN
(Vienna, Austria)

INTRODUCTION.

A part, about 163 meters long, of the left shore retaining wall (about 10 meters high) at "Wienfluss"-creek in Vienna was displaced outward and deformed by pressure of clay-soil and fill at wall-back. Cause of that failure was erosion by ground water seapage in consequence of deteriorations of concrete lining of creek-bottom by bombholes. During repairwork slope of earthcut behind the wall slid down after a year of stability in an extent of about 50 meters in length, drawing with it two big flats at its top. That secondary earth movement occurred within the reach of influence of an artesian well. Safety against sliding had been computed following Prof. Fellenius to be at least 1,7 (-and was lateron reckoned at the base of observed rather logarithmic sliding-cylinder to be at least 1,5-), using soilmechanical characteristics found by testing "undisturbed" clay-samples drawn out of normal drillholes. So unstability of that terrain could not be recognised by soil mechanical methods adopted. Secondary sliding may have been effected by a lubricating layer between two layers of normal clay, that intermediate layer not having been detected by boring. On the other hand resistance of clay soil may have been found too great because of an eventual compression of the samples during sampling work.

The regulated bed of "Wienfluss"-creek in the inner districts of Vienna is bordered at left side by the retaining wall here in question. At right hand a lower wall is lining the creek-bed and at the same time the cut, containing municipal electric railway which runs along "Wienfluss"-creek in that part of the city. The second of that creek dealt with in this report is situated between 2 bridges and shown in figures 1 and 2, the latter photo having been taken before Vienna had been attacked by bombers. Sole of creek-bad is lined with concrete, about 0,6 m thick. Both shore walls are built up in concrete at the base and in stone masonry at the top, foundation of them not being deeper than 1 m below creek-bottom.

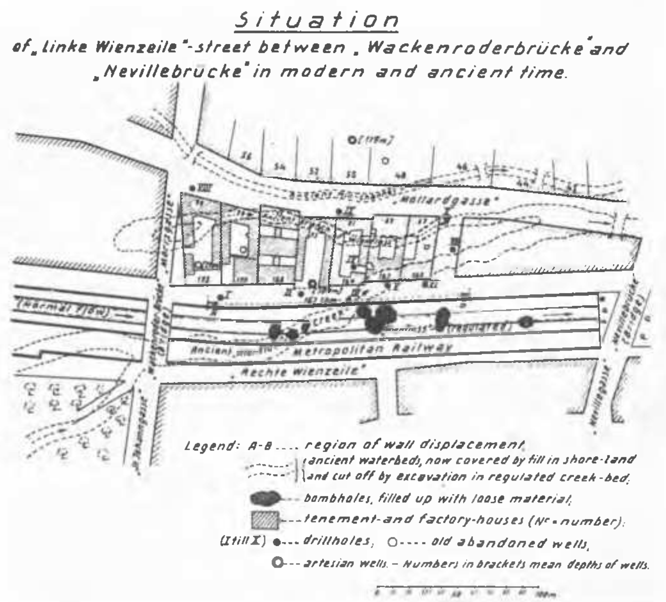


FIG.1

On April 23rd 1946 the above mentioned wall almost suddenly moved outside simultaneously tilting creek-ward. Dimensions of that retaining wall and the extent and manner of its displacement are to be seen in figure 3. The photos in figures 4 and 5 illustrate by view the immediate effects of that wall displacement. Figure 6 shows the steadily diminishing progress of wall movement after its first big step.

In order to investigate the cause of that disastrous phenomenon the topographic conditions (see figure 1) and the characteristics of the underground in situ had to be studied. 10 drillholes (I till X) and two investigationshafts were at once sunk down. The results drawn from them could be used for designing the geological profiles shown in figures 7 and 8. It may be recognised from these figures and the geological map of Vienna that the underground of the "Wien-

Upstream view of "Wienfluss"-creek, taken from "Nevillebrücke" in 1941.

FIG.2

Downstream view of "Wienfluss"-creek, taken from "Wackenroderbrücke" in April 1946, immediately after displacement of left shore wall.

FIG.4

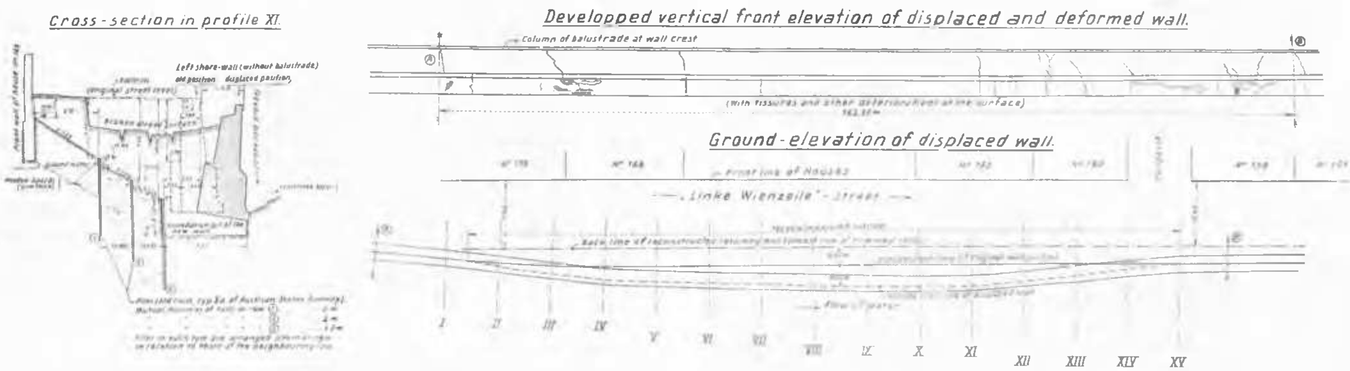


FIG.3

Downsettled "Linke Wienteile"-street on back of displaced creek wall. View downstream, taken in April 1946 immediately after wall displacement.

FIG.5

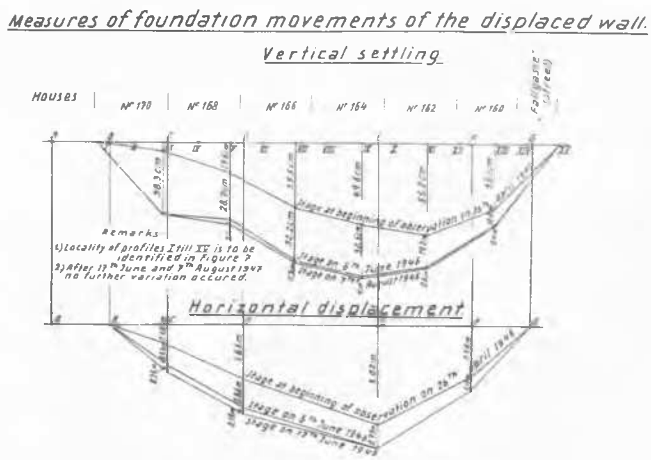


FIG.6

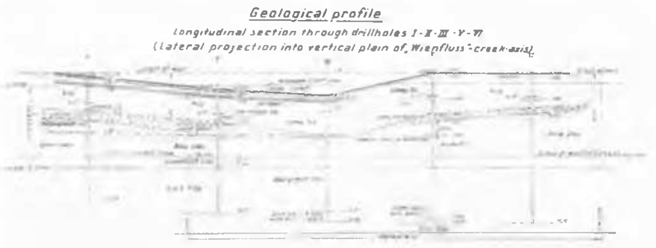


FIG.7

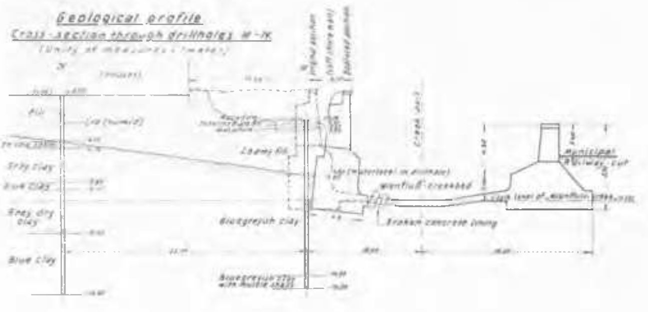


FIG.8a

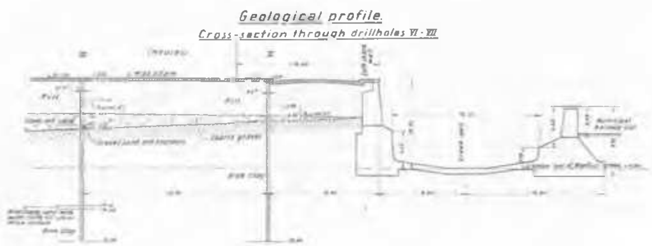


FIG.8b

fluss"-creek within the region in question is for most part clay (coloured generally blue,but partly grey and yellow) of the tertiar age,pertaining to the pontic stratum, bearing at its top loamy gravel and fill of divers sandy material, rubbish etc. The surface of the clay-ground is uneven and has in general a flat inclination towards the old "Wienfluss"-creek bed, the trace of which could be drawn from old city plans (see figure 1).

Special attention had to be payed to ground water conditions. Besides of water comming from above, seaped through the loose fill and gravel and stopped at the top surface of the clay,there is to be found artesian water in important depths below terrain. Two of them are opened by old abandoned wells (see figure 1). The bottom of the artesian well within the premises of house Nr. 166 is estimated to be about 100 m below ordinar surface level of "Linke Wienzeile"-street. That well exists since about 100 years and is no more needed for a long period passed. It was filled up many years ago. Through that permeable fill, through loose material outside of the brick masonry and voids of it, ground water has been rising till near terrain and so imbuing the surrounding soil. So clay ground in the immediate neighbourhood of that well, which seems to have been of great influence on soil stability, became soft with a tendency of yielding in lateral direction. Earth pressure in consequence was increased by hydrodynamic pull of running artesian water in the clay (flowing pressure). That nearly horizontally acting force had formerly been of about parallel direction to the regulated "Wienfluss"-creek, because transversal movement of groundwater was then hindered by the solid bed-lining of that creek, plunged as well as a vessel into the rather homogeneous mass of clay.
Hydraulic grade of groundwater flow under such conditions had been very low, so that disturbances of soil equilibrium in consequence of hydrodynamic pressure could not occur.
Conditions essentially altered, when in 1944 Vienna was attacted by bombs. The "Wienfluss"-creek bed was then in many points covered with bombholes. For the creek-stretch here being of interest the damages of creek-sole, caused by bombs, are designed in figure 1. Reduced working conditions during the second world war did not allow to repair at once that damaged parts of creek-bed. So bomb-funnels only were filled up with loose material. By that way groundwater, comming in part from top surface of clay and on the other hand from the deep horizon of artesian wells, had a short lateral opportunity to escape into the creek. Transversal hydraulic grade of groundwater movement towards the creek-water level being high, speed of underground flow increased to such extent, that fine particles of the clay in the groundfloor of wall base were spilled out, so that by and by during a period of 2 years, that clay took a tendency to flow out, frictional resistance being at the same time extremely reduced. Weight of fill and wet clay with its horizontal outside component as well as flowing pressure of groundwater on the one hand, diminished weight of the shore wall(by underpressure of seapage water) and reduced sole friction on the other hand, effected together the afore described wall displacement and deformation. The advance of the retaining wall towards the creek-axis was accompanied by crushing concrete lining slab of the creek-sole, resistance of which having been considerably reduced by bomb-damages. It is surprising that the retaining wall in question, being at least 50 years old, has shown only a few important cracks, in spite of a horizontal deflection till nearly 4 m (i.e. about $2\frac{1}{2}$ % of length of the deformed wall!). Concrete and stone masonry since had proved to be of plastic behaviour within a wide range of stresses. Street surface behind the wall had settled for about $3\frac{1}{2}$ m at the same point where maximum of horizontal wall displacement was observed; so that volumetric equilibrium of backfill before and after wall movement evidently was granted.

Vienna municipal building office (department on bridge building and reclamation service) immediately after that catastrophe started repairwork by engaging 3 Vienna constractors,head of them being "A. Porr A.G.".- First all earth material resting on the wall back had to be removed in order to avoid further sliding of the retaining wall. A further progress of wall travelling would have been of disastrous consequences, i.e. breaking down of the damaged wall into the creek-bed, obstructing of the latter by concrete and masonry blocks and so provoking danger of inundation, interruption of municipal railway function etc. The photo in figure 9 shows the front of the deformed retaining wall and the creek-sole damages as evident during excavation work. The cut (see photo in figure 10) had an average slope ratio of about 1:2 and was secured against transversal outside movement by 3 rows of rammed steel piles (old railway rails) arranged in lines, about parallel to creek-axis. At the bottom of the cut is to be seen the level of water, seaped in part out from the slope, but to a greater extent from the creek-bed. That water has been removed by

Outside front view of the displaced shore wall, taken downstream from a roof at right shore on 27th May 1946.

FIG.9

View of excavation behind the displaced wall, taken upstream from bottom of the trench on 3rd August 1946.

FIG.10

nearly continuous pumping. The output of the artesian well next house Nr. 166 is rather constant and amounts to about 1,2 liters per second. It has been conducted by gravity through a pipline into the creek during repairwork of the wall. The area for that work had been closed up against the creek by means of sheetpiling (see photo in figure 11).

Stability of the aforementioned earth slope, at the top-line of which stood a row of flats, some of them being tenement houses of 3 till 6 floors, had to be investigated by adopting soil-mechanical methods. To that purpose, first of all, mechanical qualities of clay-soil were

Downstream view of sheetpiling, taken on 3rd August 1946.

FIG.11

stated by testing 17 undisturbed samples(drawn out of the drillholes I till VII and 2 sampling shafts, from points 7 till 17 m below street surface) in the Soil-mechanical Laboratory of Technical University of Vienna (Director: Prof.Dr. O.K. Fröhlich). Cardinal test results are as follows.

1. General data.

Natural water content, related to dry soil) w_n = 0,19 till 0,29, average 0.26;

in the middle depths values of w_n were found to be generally greater than in the deep ones.

Voids content ---------- n = 0.38 till 0.45, average 0.42; in the upper and middle depths values of n were found to be generally greater than in the deep ones.

Voids ratio $\varepsilon = \frac{n}{1-n}$ = 0,61 till 0,82, average 0,72.

Atterbergs characteristics:

Flowing limit ------- F = 58 till 84, average 75%
Rolling limit ------- A = 21 till 29, - 25%
Plasticity.. PI = F - A = 36 till 59, - 49%

That clay may therefore be qualified to be of half-solid consistency and a high degree of plasticity.

Weights per unit:

γ_c = 2,75 tons/m³ - specific gravity of dry clay substance.

$\gamma_{ch} = \gamma_c(1-n) + w_n = 2{,}75(1-0{,}42) + 0{,}20 = 1{,}60 + 0{,}20 = 1{,}80$ tons/m³ for natural humid clay.

$\gamma_{cw} = 1{,}60 + 1{,}0\,n = 1{,}60 + 0{,}42 = 2{,}0$ tons/m³ for wet clay.

$\gamma_f = 1{,}70$ tons/m³ for fill material (average)

2. Compressibility.

17 compression tests with confined lateral expansion resulted in ε/p -diagrams (p = pressure per 1 cm²), from which the following characteristics (for an average pressure of about 2 kg/cm² (kilograms per square centimeter)) could be drawn ($\varepsilon = 0{,}72$):

$a_m = \left(\frac{d\varepsilon}{dp}\right)_m = 0{,}014$ cm²/kg -- coefficient of compression,

$E_m = (1+\varepsilon_m)/a_m = 121$ kg/cm² -- modulus of compression.

From compression tests with unconfined lateral expansion of 52 cylindric samples (diameter 2,6 cm, height 3,9 cm) the following values of compressive strength were derived:

σ_c = 1,4 till 6,0, average 3,25 kg/cm^2 .

3. Shearing strength τ

$\tau = C + tg\,\rho \cdot p$ following Coulomb.

C = cohesive resistance = $\sigma_c \cdot \dfrac{1 - \sin\rho}{2\cos\rho}$

the latter relation having been derived from Mohr's diagram of stresses (strengths).

ρ = angle of internal friction. That angle can be reckoned out of the sliding angles, to be observed in the compression tests following 2.).

Results in the actual case: ρ = 2° till 29°, av. 14°.

C_m (average, for σ_c = 3,25 kg/cm^2, ρ = 14°, $tg\,\rho$ = 0,249) = $3{,}25 . \dfrac{1 - 0{,}242}{2.\ 0{,}97}$ = 1,27 kg/cm^2 =

= 12,7 tons/m^2.

c_{min} (minimum, for σ_c = 1,40 kg/cm^2, ρ = 29°, $tg\,\rho$ = 0,555) = $1{,}40\ \dfrac{1 - 0{,}486}{2.\ 0{,}874}$ = 0,42 kg/cm^2 =

= 4,2 tons/m^2.

4) For earth pressure, computation sliding angle (= angle of total transversal resistance) has been determined by tests to be ψ = 24°.

Safety (γ) against sliding of the slope in the cut, a typical cross section of which is schematically designed in figure 12, was computed following the usual methods as described by general terms in the same figure. Sliding area has been assumed to be a circular cylinder-following the regula of Prof.Fellenius (Sweden) - with central point m and including the edges B and T . By substituting the characteristics of soil, as found by soil mechanical tests, and the dimensions to be taken from the cross section, the moments decident for stability resulted from the formulae (figure 12) as follows: (t m = tons x meter):

Ad I.) M_A = 721,23 + 85,00 + 631,15 +
+ 292,72 = ----------------------1730,10 tm

Ad II.) $M_{R,m}$(for C_m = 12,7 tons/m^2, ρ = 14°)=
=(12,7.14,20 + 0,249.80,93),18,30 +
+(12,7.1,70 + 0,249.44,0,3413).18,30 +
+ 552,50 + 184,03 ----------------4870,03 tm

$M_{R,min}$ (for C_{min} = 4,2 tons/m^2, ρ = 29°) =
= (4,2.14,20 + 0,555.80,93).18,30 +
+ (4,2.14,20 + 0,555.80,93).18,30 +
+ 552,50 + 184,03 ----------------2966,53 tm.

Ad III.) The degree of safety against sliding therefore was found to be:

$\gamma_m = \dfrac{4870{,}03}{730{,}10}$ = 2,82 (average)

$\gamma_{min} = \dfrac{2966{,}53}{730{,}10}$ = 1,72 (minimum)

That result seemed to have proved sufficient stability of the slope and security enough for the houses on its top. Notwithstanding that favourable prognose the aforementioned houses were thoroughly observed in the meantime during repairwork of the retaining wall was going on. Those houses had all been damaged by bombing more or less, and therefore all existant fissures and other injuries were conciously stated and noted before starting of wall repairwork and since continuously inspected. At the same time fix points of the houses were observed by levelling. Results of that inspection may be drawn from figure 13. It shows a nearly periodically varying vertical movement of the house Nr. 168 with amplitudes of 1 till 4 mm during about a year of observation. These movements were evidently concordant with variations of meteorological conditions. No tendency of monotonous settling of that house in eventual consequence of disturbances of stability could

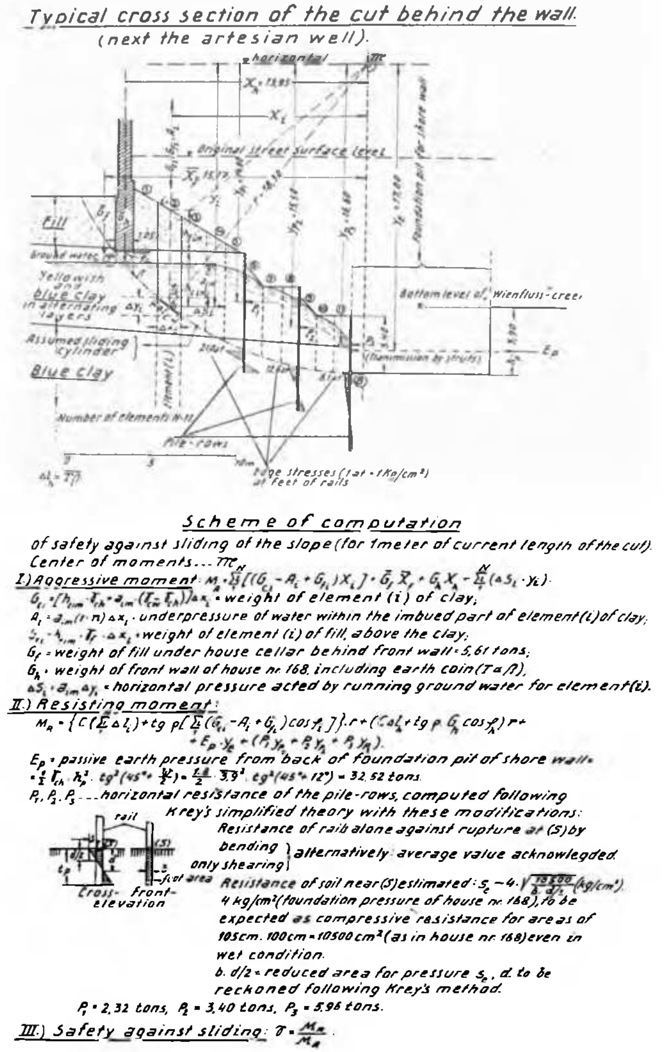


FIG.12

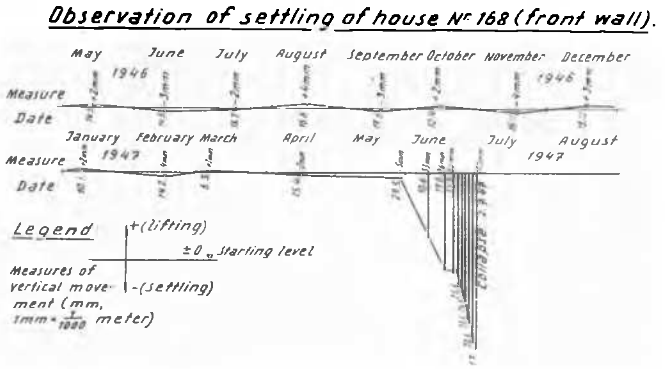


FIG.13

be observed till beginning of May 1947.

Reconstruction of the retaining wall was first intended to be performed by sectional underpinning, rolling back and adjusting the displaced wall by means of steel roll sets, to be inserted, and connecting the old replaced masonry with the new basement, in order to save material, time and money. By lack of labourers and timber, as well as by frequent outfall of electric current progress of work was considerably reduced with respect to the original program. So winter calamities intervened and effected a further detainment of work, at the same time damaging the clay ground and the old concrete masonry of the displaced wall by frost. Clay walls in pits, excavated under some parts of the old retaining wall in order to underpin it, began to break out locally, a huge block of frozen concrete fell down, etc. Under such conditions the first program had to be abandoned with respect to dangers in connection with it and to the high cost to spend, if it should be fulfilled in safety. In spring 1947 therefore a new rebuilding plan was decided for further realisation. That project conceives the erection of a completely new concrete shore-wall after having demolished the old one. Sole level and size of foundation block hereby in the same as in the first plan. (See figure 14). An advanced stage of demolition of the old wall and construction of the new concrete-foundation is shown by the photo in figure 15. One may easily recognise in that picture individual concrete blocks arranged between the creekward raft foundation of the wall and the retaining construction at the foot of cut-slope, in order to sustain the latter construction.

On April 10th 1947 fissures in the earth slope of the cut next house Nr. 166 were observed for the first time. The dangered slope was then locally sustained near the artesian well by timber-stouts. Towards the end of May 1947 new fissures were stated in the houses Nr.166 and 168 and since that time a precipitous increase of settling of the two houses was observed. (See figure 13). After a short rest between June 17th and 23rd the houses in question were continuously moving with acceleration towards their collapse, water seapage from the cut having considerably increased in the meantime. Biggest vertical component of movement observed was 165 mm. During that destructive process the houses also were tilting creekward, especially towards the artesian well (next Nr. 166). Wide cracks in the house walls announced the catastrophe. (See figure 16). This latter began on July 2nd at 9h.45 min.a.m. with a foregoing secondary terrain sliding and the consecutive cracking down of a part of house Nr.166. (See figure 17). Immediately after, a new part of the cut slid down, training with it the front wall of house Nr. 168 with roof and ceilings. (See figure 18).

Dimensions of that secondary earth movement may be taken from figure 19. One may recognise that the sliding cylinder, which could be stated after removal of the ruins of the collapsed houses and the loose material, has been of somewhat different form, when compared with the assumed circular one as designed in figure 12. The bottom edge of the sliding area seems to be identical with the limit between blue and yellowish clay. Within the reach of that secondary sliding the piles (rammed rails) also had moved. Immediately before the catastrophe the topside row (1) of piles went for a short distance downward without essential inclination, whereas the middle (2) and bottomside row (3) showed a considerable creekside tilt. After collapse of the cut-slope there could be stated, that in each

Typical cross-section of former and of reconstructed wall.

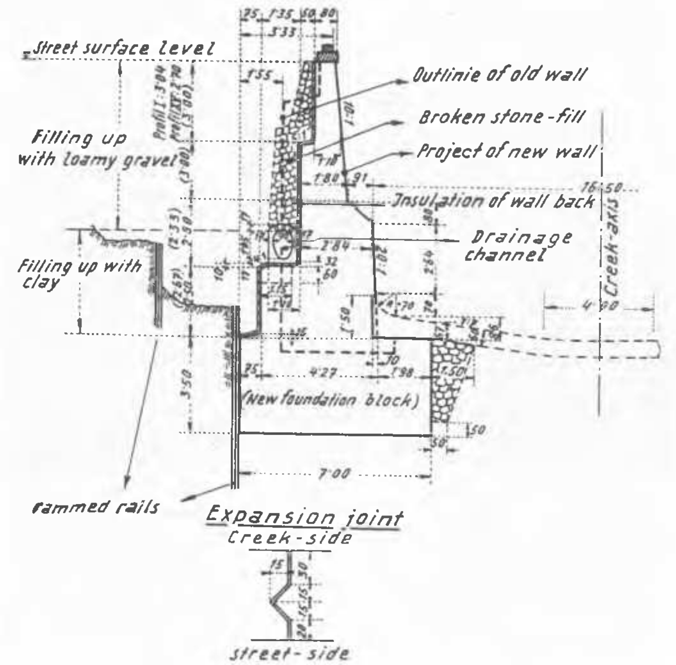


FIG.14

Downstream view of stage of repairwork on 2 nd July 1947.

FIG.15

row of piles several rails had broken about at their intersection with the observed sliding cylinder.

Rupture of that rails seemed to have been caused by shearing. The trial of checking the aforedescribed phenomenon by soil-mechanical computation, essentially following the method as explained in figure 12 (-exept profile of sliding area which here is resembling a logarithmic line-) leads to these results (see gen-

Upstream View of the houses nr. 166 and 168 July 2 nd, 1947, 9 h. 10 min. a.m., immediately before collapse of them.
(Pay attention to the wide crack in house nr. 168 and to its forward inclination).

FIG.16

Front view of houses nr. 166 and 168 after first stage of their cracking down.
(July 2nd July 1947, 9h 45 min. a.m.)

FIG.17

Full collapse of front-part of house nr. 168.
(July 2nd, 1947, 10 h 30 min a.m.)

FIG.18

eral terms of calculation in figure 19):

$$M_A = 684{,}00 + 38{,}30 + 635{,}00 + 91{,}40 = 1448{,}70 \text{ tm.}$$

$$M_{R,min} = 1436{,}50 + 321{,}10 + 323{,}20 + 120{,}47 = 2201{,}27 \text{ tm.}$$

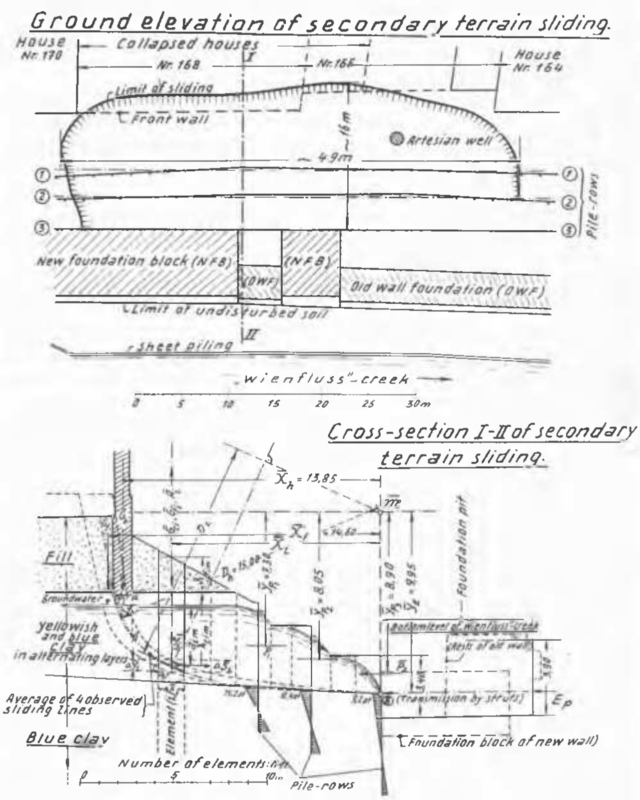
$$\bar{\delta}_{min} = \frac{2201{,}27}{1448{,}70} = 1{,}52$$ (minimal degree of safety against sliding here to be expected).

The central point π of moments could here be chosen arbitrarily.

From the aforementioned results may be derived, that the secondary catastrophe cannot be sufficiently explained by the soil-mechanical methods adopted.

It may be so, that between the yellowish and the blue clay in drillhole II a very thin lubricating layer existed, which could not be detected by boring. By such an intermediate layer shearing resistance can be extremely reduced. In order to avoid mistakes comming from that source one would have to carefully sink down shafts for investigation instead of boreholes in spots suspicious for sliding tendency. Indeed, besides the high cost of such shafts, they may be dangerous because of unavoidable disturbances of equilibrium of the cut, during excavation of them.

Another possibility to explain the phenomenon would be the assumption of deformations of the so-called "undisturbed" samples during sampling work, as has already been shown by Dr.L. Bendel (Luzern, Switzerland). There is performed a certain compression of the soil sample.when inserting the sampling cylinder, so that the sample to test seems to be more consistent than the soil in natural situ. Therefore resistance against rupture by shearing may be stated too high. The only remedy against that disadvantage would be to essentially increase dimensions of drillholes. Till now inner diameters of boring



Scheme of computation. (Compare figure 12).

$$\bar{M}_A = \sum_1^N [(\bar{G}_{ci} - \bar{A}_i + \bar{G}_h)\bar{X}_i] + \bar{G}_f'\bar{X}_f' + \bar{G}_h\bar{X}_h + \sum_1^N (\Delta\bar{S}_i \cdot \bar{Y}_i).$$

$$\bar{M}_R = \{C.\sum_1^N (\Delta\bar{l}_i . D_i) + tg\rho \sum_1^N [(\bar{G}_i - \bar{A}_i + \bar{G}_{fi})\cos\bar{\varphi}_i . D_i]\} + (C.\Delta\bar{l}_h + tg\rho.\bar{G}_h\cos\bar{\varphi}_h).D_h + E_P.\bar{y}_E + (\bar{P}_1.\bar{y}_{P_1} + \bar{P}_2.\bar{y}_{P_2} + \bar{P}_3.\bar{y}_{P_3}). \quad \bar{\gamma} = \frac{\bar{M}_R}{\bar{M}_A}.$$

$\bar{G}_f' = 2{,}62$ tons, $\bar{G}_h = 45{,}9$ tons, $E_P = 32{,}52$ tons.
$\bar{P}_1 = 3{,}33$ tons, $\bar{P}_2 = 3{,}97$ tons, $\bar{P}_3 = 7{,}22$ tons.

FIG.19

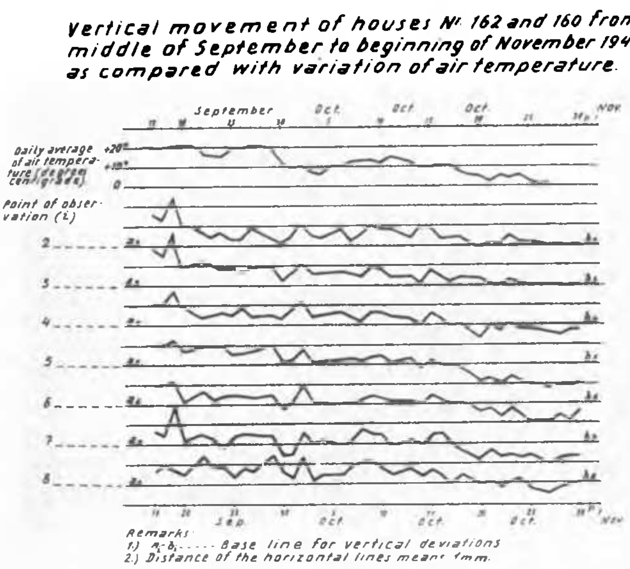


FIG.20

tubes here generally used for such purposes have been 15, 18, 22 cm. Wider drillholes would furthermore facilitate taking samples for proper shearing tests, the normal boreholes being too narrow for such samples.

After collapse of the houses Nr, 166 and 168 the behaviour of still standing houses nr. 162 and 160 has been observed with extraordinar care. The results of precise levelling of fix points on front walls of these houses (see figure 20) show distinctly a perfect stability of the flats and a rather synchronous character of their little vertical movements with variations of air temperature.

In order to exclude for the future any danger by groundwater, the project on reconstruction of the shore-wall has been completed by a plan providing security against imbuing the clay-soil with artesian water. Ground water level is intended to be controlled by an outlet tube, discharging into the "Wienfluss"-creek, so that it cannot rise above the highest admissible horizon of soil-water behind the new retaining wall. In cases of floods in the "Wienfluss" the mouth of the aforementioned tube is to be closed up automatically and groundwater to be pumped into the creek. From the upper horizon of groundwater (above clay surface) drainage is secured by a channel at the wall-back. By that way protection of the retaining wall against any harm comming from groundwater may pass as guaranteed.

SUMMARY, (CONCLUSIONS).

Investigation of clay ground with respect to safety-computation against sliding should since be made -if possible- by means of carefully excavated and secured shafts instead of boring narrow drillholes to that purpose. By that way thin lubricating layers are easier to be found. When taken "undisturbed" samples of soil, diameters of sampling cylinders should be essentially wider than those having been till now generally used.

By that way not intended disturbances of "undisturbed" samples and errors in determination of soil resistance can surely be avoided. Such an increase of dimensions of samples supposes of course a corresponding increase of drillhole diameters. Wider drillholes furthermore enable the soil-mechanist to draw undisturbed samples for proper shearing tests.

IVc12

TREATMENT OF FROST SLOUGHING SLOPES

KENNETH S. LANE
Chief, Soils & Geology Br., Garrison Distr., Corps of Engineers,
Bismarck, N. Dakota

SUMMARY

Sloughing of slopes during frost melting often requires expensive maintenance for removal of soil which has moved down to partly cover the area at base of the slope. Examples are given and movement explained as a flow of loose and wet soil. From successful experience in New England, surfacing such frost reactive slopes with a thin blanket of pervious soil or cinders is considered an adequate preventative.

Sloughing of slopes during the frost melt period is a frequent source of annoyance and maintenance in regions experiencing frost action. The movements considered here are generally shallow and are most pronounced under highly frost reactive conditions. In the usual form of a frost slough, the top 6 to 30 inches of surface soil moves down the slope and develops to a flow, filling any ditch present and spreading over the ground at toe of the slope. In central New York State such flows have covered highways to a depth of several feet, often with an annual frequency; whence, one of the treatments adopted has been expensive widening of cuts to provide a 15 to 25 foot shoulder to partially contain future flows. The following two cases are good examples; pictures of others have been presented by Mullis 1).

Fig. 1 shows a frost sloughing slope on the access railroad to Westover Airfield, near Springfield, Mass. This cut is about 30 feet deep through approximately 5 feet of sand underlain by varved silt (silt, fine sand and lean clay in thin interstratifications). It was originally constructed in 1940 with a 1 on 1½ slope from back of the ditch. During its first spring melt period, the silt sloughed down to fill the ditch. One or two years later a more extensive slough flowed about 2 feet thick across the tracks. At time of the photograph, in the Spring of 1946, the cut had been widened considerably by further sloughing and mainte-