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Meyerhof, G. G.

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THE MECHANISM OF FLOW SLIDES IN COHESIVE SOILS

BY

G. G. MEYERHOF

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THE MECHANISM OF FLOW SLIDES IN COHESIVE SOILS

by

PROFESSOR G. G. MEYERHOF, D.Sc., Ph.D., A.M.I.C.E.

SYNOPSIS

The Paper outlines the geological factors and soil conditions at sites of flow slides, and indicates the similarity of the physical properties of cohesive soils in the affected regions of Canada, Norway, and Sweden. The causes and characteristics of flow slides are discussed, and the observed mechanism is used for stability analyses. The proposed methods of analysis are applied to flow slides, and the estimates are compared with observations.

L'article donne un aperçu des facteurs géologiques et des conditions du sol aux lieux de glissements de terrains et il fait remarquer la similitude des propriétés physiques des terrains cohésifs dans les régions affectées du Canada, de la Norvège, et de la Suède. On y discute les causes et les caractéristiques des glissements de terrains et le comportement observé est utilisé comme base pour des analyses de stabilité. Les méthodes d'analyse proposées sont appliquées aux glissements de terrains, et les estimations sont comparées avec les observations.

INTRODUCTION

Since the beginning of this century an increasing number of flow slides on natural slopes have been reported from eastern Canada, the north-eastern districts of the United States of America, and the central and southern coastal districts of Norway and Sweden. Most of these slides occurred on river terraces with a practically flat plateau and gently sloping banks of post-glacial clays and silts. The deposits forming the slopes were very sensitive to disturbance and flowed in blocks to a flat gradient leaving behind a large and frequently elongated depression.

The mechanics of such slides are influenced by geological factors, soil and ground-water conditions, and causes producing instability of the slopes. Coastal flow slides in sands by spontaneous liquefaction will not be considered in this Paper, which is restricted to slides in cohesive soils.

GEOLOGY

The geological history of the areas affected by most flow slides is very similar. The regions were originally covered by great Pleistocene ice masses which removed loose material and depressed the land. Subsequently (about 10,000 years ago) the glaciers melted and soil was deposited in the salt water of the invading sea. At the same time isostatic uplift raised these deposits above sea level and they were subjected to chemical and physical changes depending on the conditions in the particular area.

In eastern Canada most flow slides have occurred in the valleys of the St Lawrence and Ottawa rivers and their tributaries where clays and silts had been laid down in the Champlain Sea and were then raised generally up to about 300 ft above sea level (Hurtubise, Gadd, and Meyerhof, 1957).

In north-eastern U.S.A. conditions were similar to those of eastern Canada, and apart from areas underlain by marine or estuarine clays, some flow slides have taken place in varved clays laid down in glacial lakes (Sharpe, 1938).

In Norway the fjords and valleys near Trondheim (west coast) and Oslo (south coast) have been most affected where marine clays were deposited on the shores of the Atlantic Ocean and subsequently lifted up to about 600 ft (Holmsen, 1953). Similarly in Sweden the valleys near Göteborg (west coast) and Stockholm (east coast) were the scene of numerous slides in clays laid down along the Baltic Sea and later raised up to about 300 ft above sea level ; this uplift is still continuing with a maximum of about 1 ft/century (Wenner, 1951).

These sediments were generally consolidated under their own weight without reduction of overburden (normally consolidated) but some Canadian and Swedish deposits were slightly over-consolidated (effective over-consolidation pressure up to about 1 ton/sq. ft). As a result of weathering and desiccation the upper layer of many deposits changed to a stiff crust which may contain fissures. The thickness of this crust with increased shearing strength varied from a few feet to about 20 ft on the sites of flow slides. Leaching reduced the salt content of the pore-water and, like other chemical changes, affected the physical properties of the clays.

SOIL CONDITIONS

The sites on which flow slides occurred were mainly underlain by soft to firm extra-sensitive or quick clays, silty and varved clays and, to a lesser degree, silts. The water-table was generally close to the surface. In a few cases excess pore-water pressures are believed to have existed before the slides, especially in stratified soils. Sometimes the soils were covered by a thin overburden of cohesionless material in which the water-table was located and prevented desiccation. The average soil properties of some typical sites of flow slides in various areas are given in Table 1.

The clays and silty clays contained a great amount of illite whilst the silts were dominated by the rock-forming minerals, mainly quartz and felspar. The marine or estuarine clays had a low plasticity and were characterized by a liquid limit of about 25–60, plastic limit of 15–25 and a plasticity index of about 10–35. The clays were generally inactive (activity of 0·2–0·6) and had a water content close to or considerably above the liquid limit (liquidity index from about 1 to nearly 3). The sensitivity of the soils ranged from very high to very quick (10–100 and occasionally more) ; Canadian and Swedish deposits were frequently less sensitive and the corresponding Atterberg limits were somewhat higher than the Norwegian ones (Table 1).

These physical properties are consistent with the considerable reduction of the pore-water salt content by leaching, frequently to less than one-tenth of the original value. This may decrease the undisturbed shearing strength to about two-thirds of the original value and the remoulded strength to that of a viscous liquid so that the sensitivity is much increased (Bjerrum, 1954) ; (Skempton and Northey, 1952). The clays and silty clays below any stiff crust had an undisturbed shearing strength from about 0·1–0·4 ton/sq. ft with up to 5% strain to failure in undrained compression tests. The ratio of shearing strength to effective overburden pressure varied from about 0·1–0·5 corresponding to an angle of shearing resistance of about 5° to 20° (Table 1).

CAUSES AND CHARACTERISTICS OF FLOW SLIDES

In view of the relatively low strength and recent origin of the sediments, they are readily eroded by water. This process which seems to be the most important cause of flow slides is still going on. Thus the height and slope of banks may increase steadily leading to instability, which is aggravated by undercutting of the toe by a river. In addition, percolating water may leach the salt from the pore-water of marine or estuarine clays and thus produce a gradual deterioration of the shearing resistance, which can be further reduced by an increase of the pore-water pressures. Apart from these natural causes construction operations, such as overloading of the slope or lowering of the water-table, infiltration of water and shock have also initiated slides of slopes which were already close to failure in their natural condition.

Before an initial slip takes place, ground movements and tension cracks at the top of the potential rupture surface were sometimes observed. The characteristics of typical flow slides in various areas are given in Table 2, which shows that the height involved in an initial slip increased with the average undisturbed shearing strength of the soils and ranged from about 20–60 ft with a slope of about 10°–35°. The initial length of the bank affected was usually a few hundred feet long and increased roughly with the height of the bank. The inclination of the top surface of the bank varied from zero to about 5°.

Once an initial slip has developed, the shearing strength along the rupture surface is decreased to the residual (ultimate) strength, which for sensitive clays is less than three-quarters and for quick clays less than one-half of the maximum (peak) strength. Further slipping and disturbance will reduce the residual strength to the remoulded strength, which may be only a few per cent of the undisturbed strength in quick clays and saturated silts. The sliding body will thus consist of an intact slice, which may break into blocks, with a surface layer of negligible shearing strength offering practically no resistance to movement so that the material is carried away aided frequently by water flow from a river. Thus, hardly any counterbalance or stability is provided to the exposed bank, which accordingly slips in turn by a simple slide.

Through this process of successive slips a large area of bank is rapidly transformed into a flow slide, the slide material consisting of intact blocks flowing away by lubrication of their remoulded surfaces in a thick slurry until stability is reached by piling up of debris or approach to more resistant strata. The general slip surface was usually located some 5–20 ft, i.e., about one-quarter to one-half the height of the bank, below the level of the toe of the initial slip and ran approximately parallel to the top surface of the bank since the critical height of successive slips remained sensibly unchanged under fairly constant soil conditions. The average thickness of the debris ranged from about one-quarter to the full height of the initial bank and the average inclination of the debris varied from about $\frac{1}{2}°$–2° (Table 2). After a period of rest the remoulded material stiffened by thixotropic hardening and consolidation but generally never regained its original strength.

Flow slides can be propagated either retrogressively (receding), which is the most frequent mode of failure when initiated by stream erosion at the toe of the bank, or progressively (advancing), which occurs when failure is initiated on a steeper or more heavily loaded rear part of the slope. In exceptional cases practically simultaneous flow of the whole area (sheet flow) can occur, especially when failure is due to a weaker layer or excessive pore-water pressures. Irrespective of the mode of propagation, the shearing strength is completely mobilized on the rupture surface of any elemental or successive slip, and it is usually successively mobilized along the base of the whole slide.

The type of movement is generally rotational for every individual slip and translational for the whole slide, which is under longitudinal tension when retrogressive and under longitudinal compression when progressive. The rate of translational movement of the slide material varied from about 1–3 miles/hour, and the slides lasted usually from about one-half to several minutes. Some 100,000 cu. yd of material were involved in small slides and a few million cubic yards in the larger slides, while the corresponding area of depression or scarp ranged from a few acres to about 100 acres.

Since the initial slip is a local one (i.e., of three-dimensional characteristics), subsequent slips occur at the sides as well as at the rear of the original crater so that the scarp is progressively widened with increasing distance from the mouth in the typical form of a "bottle-neck" slide (Fig. 1). The width at the mouth varied usually from one-quarter to once the maximum width of the slide whilst the length of the depression was from one-third to three-times the width of the slide. The depth of the depression ranged from about 20–50 ft depending on the undisturbed shearing strength of the soil (Table 2).

ANALYSIS OF STABILITY

The mechanics of flow slides indicates that three stages can generally be distinguished: an initial slip, successive slips, and final stability (Figs 1 and 2). The characteristics of the initial slip in very sensitive soils are similar to those of an insensitive material, and a stability analysis can therefore be made using the full shearing strength on a circular rupture surface

Table 1
Average properties of soils at sites of flow slides

Location	Soil type	Natural water content (%)	Liquid limit	Plastic limit	Acti-tivity	Pore-water salt content (per mille)	Shearing strength (ton/sq. ft)	Sensi-tivity	Ratio Shear strength / Eff. pressure	Reference
Aserumvannet (Norway)	Soft quick clay (N)	62	35	22	0·21	*c.* 0	0·1	200	*c.* 0·16	Bjerrum (1954) and *per. com.*
Bekkelaget (Norway)	Soft quick clay (N)	39	26	17	0·20	2·5	0·1	80	0·12	Eide and Bjerrum (1955)
Desbiens (Canada)	Firm quick stratified silty clay (O)	50	46	26			0·45	>40		Hurtubise, Gadd, and Meyerhof (1957)
Hawkesbury (Canada)	Soft quick clay (O)	70	58	26	0·5	*c.* 0	0·4	*c.* 30	*c.* 0·6	Eden (1956)
Nicolet (Canada)	Firm e.-s. stratified clay (O)	70	55	22	0·45	*c.* 1	0·25	10	*c.* 0·5	Hurtubise and Rochette (1956) and *per. com.*
St Thuribe (Canada)	Firm quick silty clay	44	33	21	0·33		0·4	*c.* 150		Peck, Ireland, and Fry (1951)
Säve (Sweden)	Soft e.-s. clay (O)	65	*c.* 55	*c.* 25			0·2	11	*c.* 0·3	Caldenius (1946), Cadling and Odenstad (1950)
Sköttorp (Sweden)	Firm quick varved silty clay (N)	66	54	24	0·57	*c.* <1	0·4	35	0·5	Odenstad (1951), Jakobson (*per. com.*)
Surte (Sweden)	Soft quick clay (O)	70	60	25	0·52	<1	0·2	25	*c.* 0·25	Jakobson (1952) and *per. com.*
Ullensaker (Norway)	Soft quick clay (N)	32	26	19	0·18	2	0·15	40	0·11	Bjerrum (1955)

Note.—e.-s. = extra-sensitive ; N = normally consolidated ; O = slightly over-consolidated

Table 2

Characteristics and results of stability estimates of flow slides

(For soil properties and references see Table 1)

Location type and date	Initial slip			Successive slips				Final stability						
								Depression			Ratio final / initial height	Debris		
	Height of bank (ft)	Average inclination of bank	Estimated factor of safety	Average depth of rupture surface (ft)	Average inclination of top surface	Estimated factor of safety of individual slips	Estimated inclination for simultaneous slip	Max. length (ft)	Max. width (ft)	Max. height (ft)		Average thickness (ft)	Average inclination of surface	Estimated inclination of surface
Bekkelaget (Oct. 1953) S(P)	c. 40	30°	1·1	20	3·5°	0·6	3·5°	500	600	40 *	c. 1·0*	20	0·5°	0·1°
Hawkesbury (Dec. 1955) R	47	20°	c. 1·0	c. 50	1·4°	0·7	9°	500	1450	20	0·45	c. 30	1·3°	0·7°
Nicolet (Nov. 1955) R	30	25°	1·1	40	0·5°	0·7	7°	600	400	25	0·75	18	1·5°	1·7°
St Thuribe (May 1898) R	c. 35		c. 1·1	c. 40	0·5°	0·7	c. 10°	2800	1500	28	c. 0·8	8	0·5°	0·3°
Säve (Jan. 1945) R	35	32°	0·9	40	c. 5·0°	0·5	6·5°	170	200	20	0·6	25	1·4°	1·0°
Sköttorp (Feb. 1946) R	c. 65	32°	0·8 –1·3	80	0·0°	0·5	6°	1200	850	30	c. 0·5	55	1·7°	0·5°
Surte (Sept. 1950) P	c. 40	9°	0·9	70	2·0°	c. 0·5	3·3°	2300	1200	28	c. 0·7	70	0·5°	0·3°
Ullensaker (Dec. 1953) R	20	10°	c. 1·0	23	2·5°	0·6	3·5°	600	550	18	0·9	4·5	1·9°	1·0°

Note.—P = progressive, R = retrogressive, S = simultaneous
* Restraint from firm base.

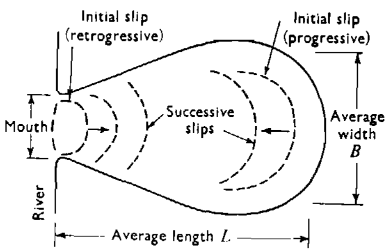

Fig. 1. Plan of typical flow slide

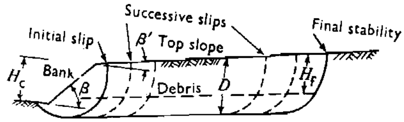

Fig. 2. Longitudinal section of typical retrogressive flow slide

involving failure of the slope, toe, or base (Terzaghi, 1943). Since such slopes have stood practically unchanged for a long time, the initial slip should be analysed in terms of effective stresses making allowance for any pore-water pressures and using shear parameters of drained tests with reduction of the minor principal stress. As such tests are difficult to carry out on very sensitive soils and in normally consolidated clays the shearing strength at a given failure water content is generally independent of the drainage conditions during test, an analysis in terms of total stresses using undrained test results (e.g., vane tests) is more convenient. Such an analysis is approximately correct where pore-water pressures in a slope at failure are small, but it gives a theoretical rupture surface different from the real one. For preconsolidated soils, for instance in the stiff crust, this method frequently overestimates the strength and the more elaborate procedure is to be preferred (Bjerrum, 1955).

For simple slopes with horizontal base and top surfaces and uniform soil conditions, the critical height H_c of an initial slip (Fig. 2) can be estimated from (Terzaghi, 1943):

$$H_c = cN_s/\gamma \qquad (1)$$

where c = average shearing strength on rupture surface,

γ = average unit weight of soil

and N_s = stability factor depending on inclination β of slope, depth to hard stratum and angle of shearing resistance ϕ ($\phi = 0$ for saturated clays and clayey silts).

For normally consolidated or slightly over-consolidated clays the greater strength at depth leads to a relatively shallow slip surface, even for flat slopes, and the stability can readily be estimated by a graphical circular arc analysis.

The results of such analyses for some flow slides are given in Table 2 and generally give theoretical factors of safety from 0·9–1·1 at failure. In view of the uncertainties involved in an analysis of initial slips in very sensitive soils, a minimum factor of safety of 1·5 would seem to be advisable in stability estimates, and full allowance should be made for future increase of height or inclination of the slope (e.g., by erosion and undercutting) and decrease of shearing strength (e.g., by leaching and pore-water pressures). If the estimated factor of safety is found to be adequate, the danger of an initial slip—and therefore of any successive slips—is likely to be small.

The characteristics of successive slips in very sensitive soils vary with the mode of propagation whether by a series of individual slips (either retrogressive or progressive) or by a simultaneous slip over a wide area. The corresponding analyses can be made in terms of total stresses and undrained shearing strength because any dissipation of pore-water pressures during successive slips is negligible. The stability of individual retrogressive slips is conveniently estimated from a circular arc analysis, as already mentioned, on the conservative assumption that no material of the preceding slip above toe level is available as a counterbalance (Bjerrum, 1955), so that a lower limit of the factor of safety is obtained. Since the inclination of the face of such successive slips is steeper than that of an initial slip and the height is practically unchanged or even slightly greater during the early stages of flow slides, it follows that the stability of successive slips is likely to be much smaller than that of the initial slip. Thus if it is assumed that successive slips have a vertical face and a height equal to that of the initial bank, the theoretical factors of safety of individual slips are given in Table 2, and range from about 0·5–0·7. These estimated factors of safety would be of the order of unity if the average inclination of the face of successive slips is the same as that of the corresponding original bank. The stability of individual progressive slips can be estimated as for the initial slip on the assumption of local slope failure.

Simultaneous slip over a wide area of a relatively flat slope can be analysed on the assumption of a plane rupture surface parallel to the slope (sheet flow), which gives an upper limit of the factor of safety for successive slips. For a uniform slope of great extent in purely cohesive soil the critical inclination β' (Fig. 2) is then given by (Terzaghi, 1943):

$$\sin 2\beta' = 2c/\gamma D \qquad (2)$$

where D = average depth to hard stratum, and other symbols as before. Thus with good approximation for a flat slope ($\beta' \leqslant 10°$):

$$\beta' = c/\gamma D \qquad (3)$$

The effect of the active pressure at the top edge and the passive pressure at the bottom edge of a slip increases the critical inclination, as shown previously (Jakobson, 1952) on the assumption of plane end surfaces. The Author has modified this analysis for curved and thus continuous end surfaces, and making an additional allowance for the shearing resistance along the sides of the slide he obtained with good approximation for $D/L \leqslant 0{\cdot}2$:

$$\beta' = \frac{c}{\gamma D}\left(1 + \frac{4D}{L}\right)\left(1 + \frac{2D}{B}\right) \qquad (4)$$

where B = average width of slip area, and L = average length of slip area (Fig. 1).

For normally consolidated soils with a water-table at ground level:

$$\frac{c}{\gamma D} = \frac{\gamma' c}{\gamma p}$$

where p = effective overburden pressure ($= \gamma' D$) and γ' = submerged unit weight of soil, so that substituting into equation (4):

$$\beta' = \frac{\gamma' c}{\gamma p}\left(1 + \frac{4D}{L}\right)\left(1 + \frac{2D}{B}\right) \qquad (5)$$

or, very approximately:

$$\beta' = c/2p \qquad (6)$$

Where a stiff crust exists on the clay, the critical inclination given by the above equations is governed by the minimum shearing strength and the corresponding depth below the crust because the rupture surface will be located at that depth.

Application of the above analyses to some flow slides is given in Table 2, and shows that the estimated and observed inclinations for simultaneous slip agree in the one case (Bekkelaget) where simultaneous slip had actually taken place. For the other flow slides the estimated inclinations for simultaneous slip are considerably greater than the observed inclinations and the corresponding theoretical factors of safety ranged from 1·3–1·6, and more for practically horizontal top surfaces. The overestimates may be expected because the shearing strength was successively mobilized along the base of the whole slide through a series of individual successive slips.

Final stability of a flow slide is reached, in the absence of an approach to more resistant strata, when the debris has piled up sufficiently high to prevent successive slips. At that instance dissipation of pore-water pressures is negligible so that the analysis can be made in terms of total stresses and undrained shearing strength; moreover, the resistance of the debris is governed by the remoulded strength which for highly sensitive clays is very small. Since the face of the intact bank is practically vertical and the inclination of the top surface is generally small, the critical final height above mean debris level (Fig. 2) is, approximately:

$$H_f = 3{\cdot}8c/\gamma \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

where c = average shearing strength of bank above mean debris level.

If the soil conditions of the initial and final slips are the same:

$$H_f/H_c = 3{\cdot}8/5{\cdot}5 = 0{\cdot}7 \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

where H_c = critical height of initial slip ($\beta \leqslant 53°$) with horizontal top surface.

If the inclination of the final bank is subsequently reduced to that of the initial slip, the final factor of safety of the bank will become approximately $1/0{\cdot}7 = 1{\cdot}4$.

Under the above conditions the minimum mean height of debris required for final stability of the slide is thus about 30% of the height of the initial bank. The observed final heights of craters of flow slides (Table 2) are found to vary from about one-half to the full height of the original bank, which is roughly in accordance with the above analysis. The final inclination of the debris can be estimated from equations (2) to (6) using the remoulded shearing strength and depth of the soil. The observed average inclinations of debris would give some support to this analysis, as shown in Table 2; however, the estimated inclinations are almost invariably too small, especially for quick clays, which may be due to the neglect of viscous resistance during the relatively rapid flow of the debris.

CONCLUSION

Although flow slides have occurred on natural slopes of cohesive soils in widely separated regions, the affected areas consisted mainly of river terraces and gently sloping banks of similar geological history and similar physical properties. The deposits were normally consolidated or slightly over-consolidated marine or estuarine post-glacial clays and silts and lacustrine varved clays. They were generally inactive, had a low plasticity, and a water content close to or considerably above the liquid limit; the soils below any stiff crust or thin cohesionless overburden containing the water-table were soft to firm with a low strain to failure and were extra-sensitive or quick through considerable leaching of salt or excess pore-water pressures.

Three stages can usually be distinguished in the mechanics of flow slides: an initial slip, successive slips and final stability. Most flow slides were caused by water erosion increasing the height and slope of banks and by leaching of salt or pore-water pressures reducing the shearing resistance of soils. Construction operations and artificial causes initiated some slides of banks which were already close to failure. The initial slip in very sensitive clays can be analysed by the customary methods for insensitive soils using total stresses and undrained shearing strength, except for any stiff crust. The results of such analyses generally give factors of safety of about unity at initial slip of flow slides.

On account of the high sensitivity, the material involved in an initial slip flows away in a slurry and the exposed bank fails by successive slips at the sides and rear of the crater to form a bottleneck-type slide. Propagation of failure is either retrogressive, when initiated by stream erosion at the toe, or progressive, when initiated on a steeper or more heavily loaded rear part of the slope, along a rupture surface with a base roughly parallel to the top surface of the bank. An analysis of successive slips on the assumptions of a vertical bank and no counterbalance from debris gives a lower limit of the factor of safety at failure as shown by application to successive slips of flow slides. On the other hand, an analysis on the basis of simultaneous slip along a rupture surface parallel to the slope gives an upper limit of the factor of safety because of successive mobilization of shearing strength by individual slips, as shown by the flow slides analysed; in one case, however, simultaneous slip had actually occurred and the estimated and observed inclinations agreed.

Final stability is usually reached by an approach of the slide to more resistant strata or piling up of debris. An approximate analysis of the latter case has been given some support by observations of the final height of craters and average inclination of debris of flow slides. It may therefore be concluded that the present methods of analysis enable a fair estimate to be made of the stability of flow slides in cohesive soils and that a minimum factor of safety of 1·5 would seem to be adequate to cover uncertainties after full allowance for the worst anticipated conditions in practice.

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