Studies of Several Dam Failures on Clay Foundations

Études de Plusieurs Cąs de Rupture de Barrages sur Fondations d'Argile

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Summary

This paper presents the results of two investigations made on the failures of clay embankments on saturated highly plastic clay foundations. The studies indicate that conventional quick shear test results applied in a total stress analysis can give safety factors greater than actually exist. Effective stress analyses utilizing estimated pore pressures may also give safety factors greater than unity for failure conditions.

Sommaire

Ce rapport présente les résultats de deux études de rupture de barrage en terre construits sur fondations d'argile grasse. En se servant des valeurs de la résistance au cisaillement, obtenues par essais rapides, dans une analyse de contraintes totales, le coefficient de sécurité indiqué est plus grand qu'en réalité. Mais dans les analyses de contraintes effectives, lorsqu'on utilise des valeurs de pression d'eau interstitielles, le coefficient de sécurité peut être plus grand que l'unité à la rupture.

Introduction

The General Reporter on Session 8 of the Third International Conference pointed out that the $\phi = 0$ method of stability analysis has proved to be reliable for a considerable number of cases involving saturated clay soils, particularly in Europe. However a number of references were mentioned where this method gave results on the unsafe side. The following summaries describe two cases in Western Canada which gave results on the unsafe side in both the total stress analysis and the effective stress analysis using the circular arc. When movements occurred on these two projects after pre-construction studies had indicated adequate factors of safety, a much more thorough investigation was undertaken.

The detailed laboratory test procedures are described by CASAGRANDE and WILSON (1953) and by CASAGRANDE and RIVARD (1956). However it should be pointed out that the triaxial tests with pore pressure measurements were loaded at a slower rate than is considered normal for tests without pore pressure measurements. Furthermore the vector curve failure criteria (CASAGRANDE and RIVARD, 1956) were used rather than the maximum deviator stress. Both the above tend to give lower strength values than conventional procedures.

Seven Sisters Dikes

The first stage of the Seven Sisters Falls Hydroelectric Project, completed in 1931, required the construction of earth dikes with a maximum height of 7 to 10 ft. The second stage, completed in 1949, involved raising the dikes approximately 14 ft. as shown in Fig. 1.

During the second stage of construction some minor slides and settlement occurred at a few locations. These were the result of steep construction slopes or other unusual conditions and consequently did not cause undue concern for the overall stability since it was assumed that the foundation would increase in strength as a result of consolidation. However, during the

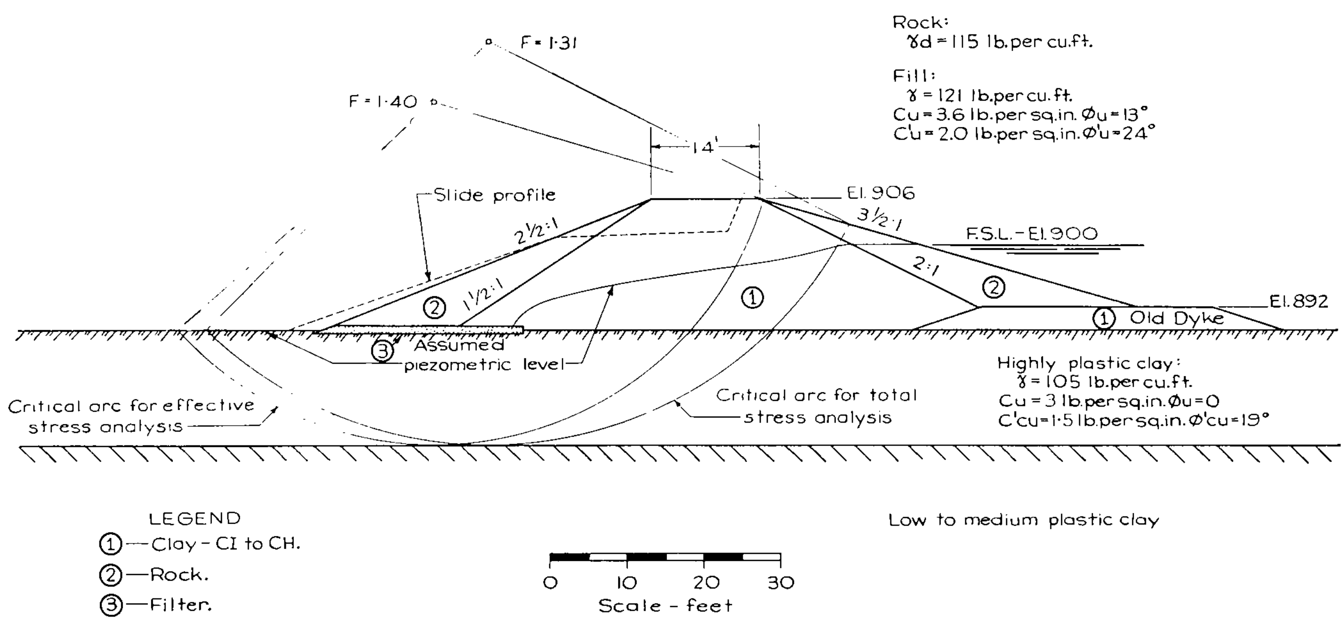

Fig. 1 Seven Sisters—typical cross-section and critical arcs
Seven Sisters—coupe et courbes critiques de glissement

period from four months after construction was completed until July 1956, a total of 13 slides occurred in a 3·5 mile length even though the dikes had an estimated factor of safety of at least 1·5.

Description of slides—All the slides that have occurred since the completion of construction are very similar in nature. The first indication of a slide is generally the formation of a crack on the waterside of the dike crest. Initially there is no sign of vertical displacement but as the movement progresses settlement occurs and a crack up to 1 ft. wide develops over a length of 100 to 400 ft. During a period of several weeks a scarp 1 to 6 ft. high generally forms between the dike centre line and the waterside shoulder. In some cases there is an upheaval on the landside toe but generally these signs are subtle and difficult to observe because of the rough terrain. Observations of continuous cores from the slide areas have not disclosed the position of the sliding surface.

None of the slides that have occurred to date have been of a serious nature since there was no immediate threat of a breach. All slides have been stabilized by the addition of a berm, the most economical being approximately one-quarter the height of the dike and 40 to 50 ft. wide. In some cases sand drains have been used.

Foundation—The foundation consists of highly plastic clay to depths of 10 to 20 ft., underlain by a clay of low plasticity or bedrock. The highly plastic clay has the average properties shown in Table 1. It exhibits a nugget structure to a depth of approximately 3 ft. and from 3 to 7 ft. laminations or fissures oriented in the horizontal direction are evident. This is thought to be a result of the formation of thin ice lenses within the clay. Below a depth of 7 ft. this phenomenon has not been observed but some stratification and silty layers are evident.

Table 1

Average properties of foundation and dike materials
Propriétés moyennes des matériaux pour fondations et digues

Material	*Property*	*Average*	*Range*	*Number of tests*
Highly plastic foundation clay	Water content	45	19–66	551
	Liquid limit	85	50–121	191
	Plastic limit	26	16–40	191
	Wet density lb./cu. ft.	104	92–117	90
Low to medium plastic foundation clay	Water content	21	14–39	105
	Liquid limit	31	23–40	50
	Plastic limit	13	11–19	50
Fill material	Water content	26	10–49	188
	Liquid limit	55	19–114	78
	Plastic limit	20	11–38	78
	Wet density lb./cu. ft.	123	105–140	55

It has been found, by averaging water contents of the foundation clay over various depths, that most of the failures occurred in areas where the average water content of the 0 to 8 ft. layer exceeded 45 per cent.

The shear strength of the foundation as listed in Table 2 was obtained by the following methods:

(1) Laboratory unconfined compression tests conducted on specimens from 3 in. diameter thin-walled tube samples;

(2) Field unconfined compression tests conducted on specimens obtained from 1 in. diameter thin-walled tube samples;

(3) Field vane borer.

The results in Table 2 include unconfined compression tests obtained from chunk samples which were taken in the zone exhibiting the nugget and fissured structure.

Table 2

Average shear strength of foundation and fill in critical areas
Résistance au cisaillement moyenne des fondations et remblais dans les zones critiques

Material	*Method of test*	*Property*	*Average*	*Range*	*Number of tests*
Highly plastic foundation clay	Laboratory unconfined from 3 in. ϕ samples	Shear strength lb./sq. in.	9·6	5·9–14·2	40
		Water content	45·1	30·2–57·7	40
	Field unconfined from 1 in. ϕ samples	Shear strength lb./sq. in.	6·8	3·7–12·6	20
		Water content	43·5	23·7–55·9	20
	Vane borer	Shear strength lb./sq. in.	12·8	4·0–26·0	25
Highly plastic foundation clay, depth 0–8 ft.	Laboratory unconfined from chunk samples	Shear strength lb./sq. in.	4·9	2·8–7·7	19
		Water content	44·4	42·1–46·7	19
Fill material	Laboratory unconfined from 3 in. ϕ samples	Shear strength lb./sq. in.	18·0	12·6–42·4	11
		Water content	25·2	11·9–34·7	11
	Field unconfined from 1 in. ϕ samples	Shear strength lb./sq. in.	13·1	6·0–19·8	6
		Water content	24·7	11·5–55·3	6
	Vane borer	Shear strength lb./sq. in.	14·5	7·1–20·9	11

In an attempt to determine the effect of the fissured structure upon the strength a number of direct shear tests were conducted with the fissures oriented parallel to the failure plane and at 45 degrees to the failure plane. No significant variation in the strength was observed.

Quick and consolidated quick triaxial tests with pore pressure measurements, believed to be representative of the foundation strength for the typical stability analyses presented, are shown in Fig. 2. The sensitivity of the clay lies between 1 and 2.

The low to medium plastic clay underlying the highly plastic clay has average properties as shown in Table 1. The average shear strength as determined from unconfined compression tests was 23 lb./sq. in. at an average water content of 15 per cent.

Fill—The dikes were constructed of medium to highly plastic clay with water contents 5 to 10 per cent above the Standard Proctor optimum water content. The density was, in general, greater than 90 per cent Standard Proctor density. Tables 1 and 2 show the average water contents, limits, densities and shear strength for the dikes.

The Mohr envelopes for quick triaxial compression tests with pore pressure measurements are shown on Fig. 3. The strength envelope assumed is shown as a dashed line.

Stability analyses—A cross-section with the most critical arcs for both total and effective stress analyses is shown in Fig. 1. Using the strengths believed to be representative of this area a factor of safety of 1·31 was obtained by the total stress method and 1·40 by the effective stress method.

The factor of safety for each of the remaining 12 failure areas

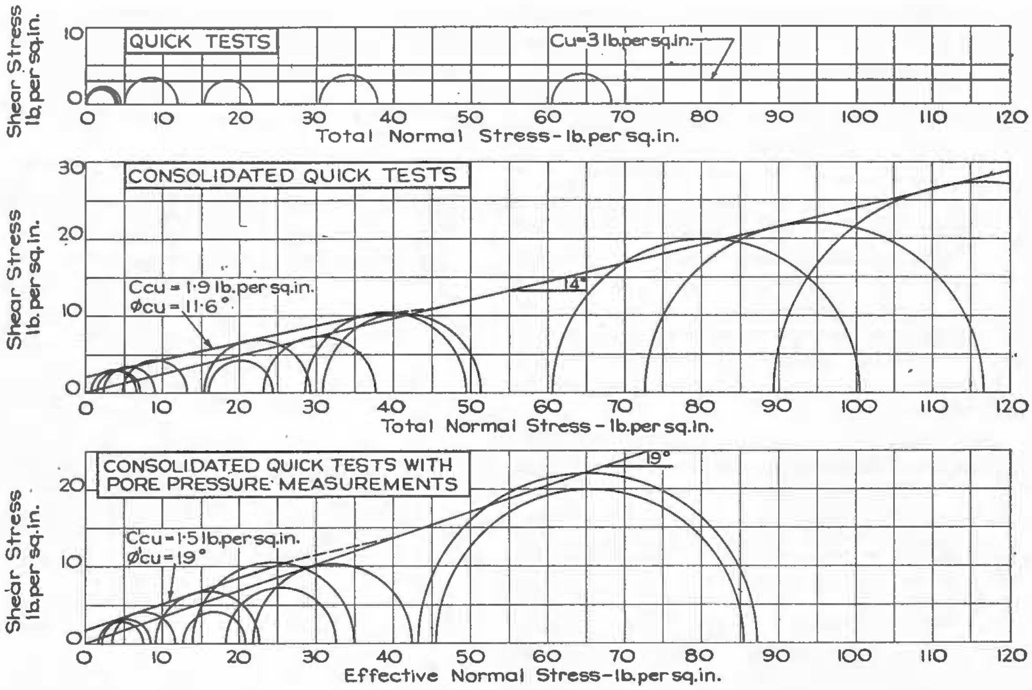

Fig. 2 Seven Sisters—foundation material strength envelopes
Seven Sisters—courbes intrinsèques, materiaux de fondation

has been computed by the total stress method of analysis. The strengths used for the foundation clay have been taken from the water content strength relationship developed from several failure areas. Where the water content profiles have indicated more than one layer appropriate strengths have been chosen for each layer. The factors of safety obtained ranged from 0·9 to 2·1, the average being 1·5.

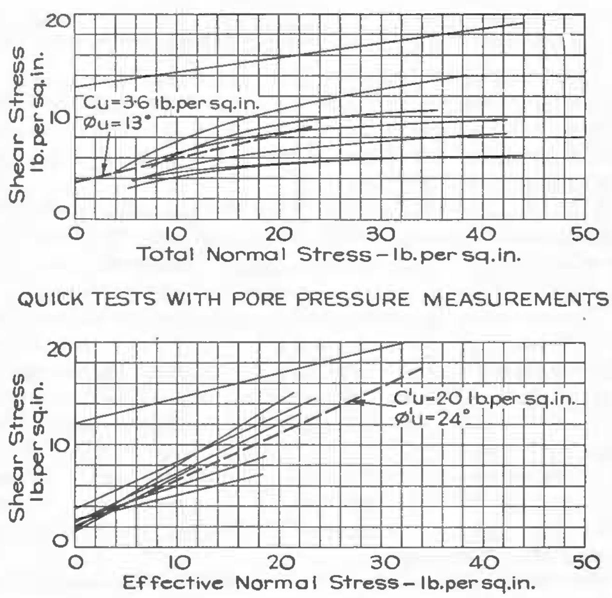

Fig. 3 Seven Sisters—embankment material strength envelopes
Seven Sisters—courbes intrinsèques, materiaux de barrage

In an attempt to explain the high factors of safety, a correlation between the occurrence of slides and the precipitation was attempted, pore pressures were measured and the creep effect (Casagrande and Wilson, 1953) investigated. None of these appeared to have any significant effect on the stability.

North Ridge Dam

The major portion of this 70 ft. high dam was constructed in a 10 week period between late June and mid-September 1953. When it was nearing completion (Fig. 4a) a foundation movement occurred. The fill construction was immediately stopped and loading berms added at the toes. An investigation, including more detailed drilling and testing, was conducted and additional observational apparatus was installed. The dam was successfully completed in 1956 with no further modifications in design.

Description of movement—The visible indications of the movement were the appearance of cracks, a slight bulging of the fill, and an overthrust at the downstream toe. The widest cracks were found to be 11 ft. deep along the upstream slope and 22 ft. deep along the downstream berm. They were up to 6 in. wide at the top and became narrower with depth. They were essentially vertical and there was no evidence of vertical displacement across them. The ground surface for 60 ft. beyond the toe heaved and moved outward about 3 ft. The movement along the surface of the downstream berm (El. 3332) was 4·4 ft. horizontally and 0·7 ft. downward. The measured foundation settlement during the movement was 1·8 ft., near the centre line.

Embankment material—The embankment was constructed of lean to medium plastic clay of glacial origin: LL = 25 to 35, PL = 14 to 21 and PI = 10 to 18. The Standard Proctor dry density was 114 lb. per cu. ft. at a water content of 14·5 per cent. The average water content and dry density recorded during construction were 14·4 per cent and 110·5 lb./cu. ft. respectively. The lower 15 ft. was somewhat wetter and less dense than average.

Density tests taken in pits dug after the movement averaged 113 lb./cu. ft. dry but the average for the lower 15 ft. was much higher, i.e. 118 lb./cu. ft. The water content of the fill samples ranged from 12 to 16 per cent with an average of 14·6 per cent.

The quick and consolidated quick strength of fill samples showed considerable variation with a few per cent change in water content. The quick strength of the embankment corresponding to the estimated average water content of the fill was $c_u = 8{\cdot}5$ lb./sq. in. and $\phi_u = 13$ degrees. The quick triaxial tests with pore pressure measurements were used to obtain an effective strength envelope. The effective strength showed much less variation with water content and a reasonable strength value appeared to be

$$c'_u = 6{\cdot}0 \text{ lb./sq. in and } \phi'_u = 29 \text{ degrees}$$

Foundation—The foundation consists of 10 to 19 ft. of fine silty sand and 35 to 50 ft. of highly plastic clay overlying a fine to medium sand. The transition zone between the upper sand layer and the clay has thin layers or lenses of silt and sand and the upper half of the clay has fragmented layers having the appearance of 'cottage cheese'. Piezometers installed immediately after the movement revealed high pore pressures only in the clay, particularly the upper portion. These pressures dissipated slowly until by March 1956 they were generally below the fill surface (Fig. 4a).

Laboratory tests on foundation samples gave the following results.

Upper sand layer: $\gamma_d = 97{\cdot}5$ lb./cu. ft. at $w = 22{\cdot}7$ per cent. Shear strength, $c_d = 0$, $\phi_d = 29$ degrees.
Highly plastic clay layer: $\gamma_d = 83{\cdot}5$ lb./cu. ft. at $w = 36{\cdot}5$ per cent, LL = 72, PL = 21.
Quick shear strength, $c_u = 8{\cdot}0$ lb./sq. in. $\phi_u = 0$ degrees.
Consolidated quick shear strength, $c_{cu} = 4{\cdot}3$ lb./sq. in. $\phi_{cu} = 14$ degrees.

Effective shear strength from consolidated quick shear strength tests with pore pressure measurements $c'_{cu} = 3{\cdot}6$ lb. per sq. in. and $\phi'_{cu} = 21{\cdot}8$ degrees.

Creep tests (Casagrande and Wilson, 1953) at constant water content indicate strength values as low as 75 per cent of the quick strength, but no allowance has been made for this in the following analyses.

Stability studies—Stability analyses were made on the embankment cross-section as it existed at the time of movement. Using quick shear strengths and total stresses the minimum factor of safety on a circular arc (M15—Fig. 4a) was 1·23. By assuming a 22 ft. deep vertical crack (dry) this value was reduced to 1·20. An analysis was also made using effective stresses and the effective strength of fill and foundation obtained from triaxial tests with pore pressure measurements. The total normal pressure on the portion of the arc through the fill was reduced by the estimated pore pressure as determined from consolidation tests on typical samples (Hilf, 1948; Shannon, 1955). The factor of safety obtained, assuming no increase in effective pressure in the foundation clay due to the weight of fill, was 1·14.

The composite surface of sliding (sliding block) method of analysis was also used because the movement appeared to be lateral rather than rotational. A thin horizontal cohesionless layer ($c' = 0$; $\phi' = 29$ degrees) was assumed to exist in the upper part of the foundation clay. Analyses were made for various pore pressure conditions in the foundation clay and along the horizontal cohesionless layer. The strength of the foundation clay and the embankment were based on the effective strengths as determined by the triaxial tests with pore pressure measurements. The total stress in the fill was reduced by the estimated pore pressure as determined from consolidation tests on typical samples. The calculated factor of safety with 100 per cent uplift pressure in the foundation clay and along the

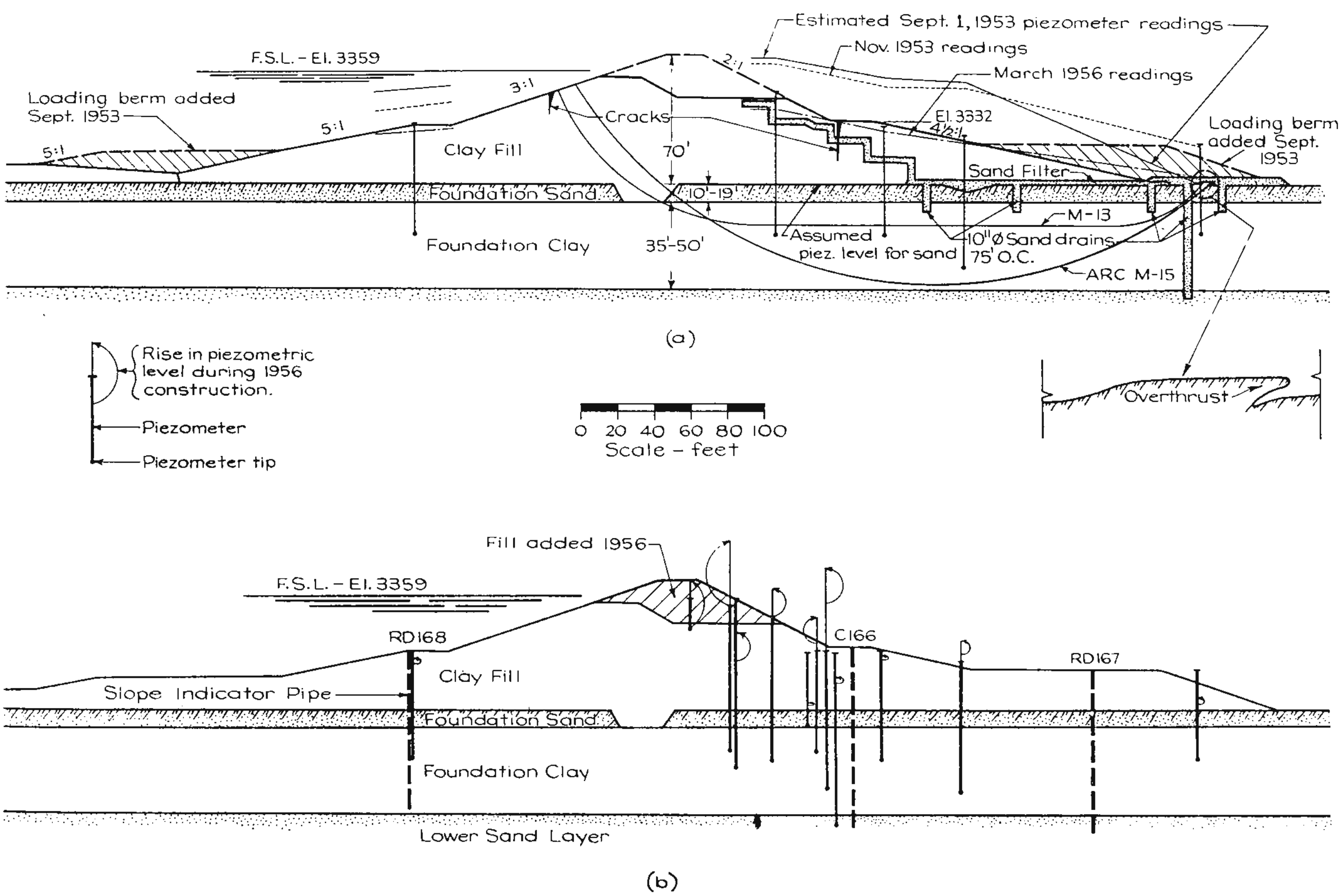

Fig. 4 North Ridge—embankment cross-sections
North Ridge—coupe du barrage

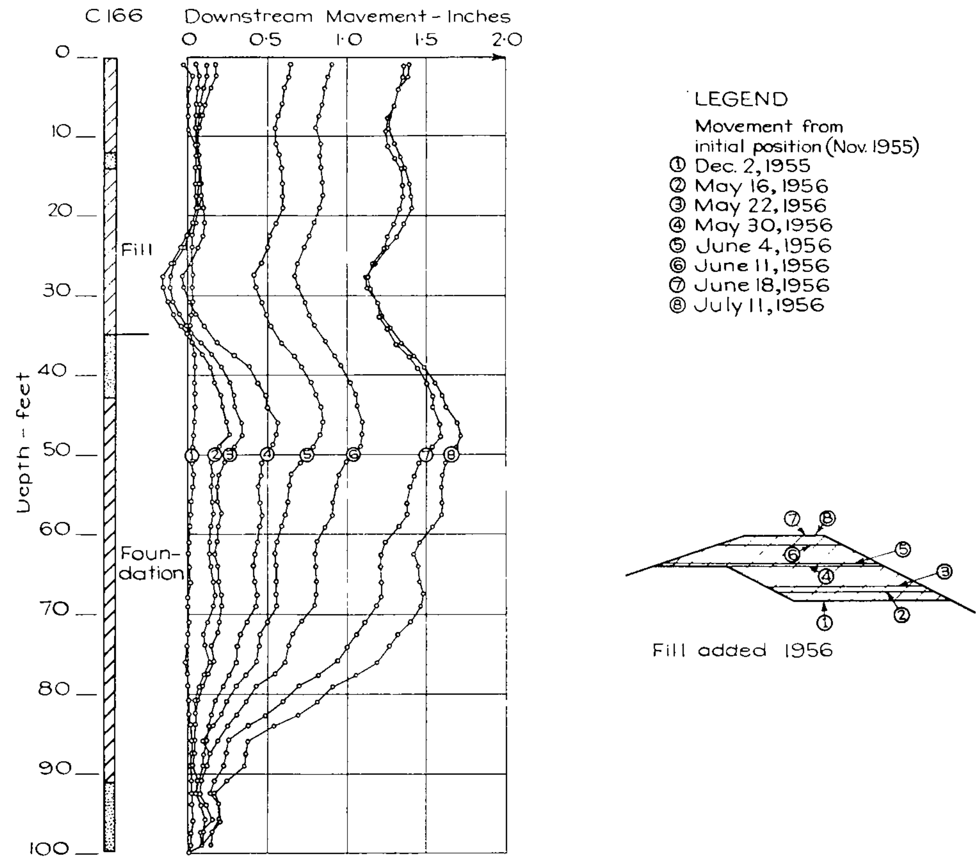

Fig. 5 North Ridge — movement determined by slope indicator

North Ridge — mouvement déterminé par appareil d'inclinaison

assumed cohesionless layer, due to the weight of fill, was less than unity (M13—Fig. 4a). The computed factor of safety was 1·06 for piezometric conditions estimated to have existed just before the movement began (1–9–1953—Fig. 4a).

These calculations show that, if a cohesionless undrained layer exists in the foundation, failure is more probable along a composite surface of sliding than on an arc. The existence of such a layer is considered unlikely but not impossible. The computed factors of safety along a composite surface of sliding through the clay foundation without a cohesionless layer are approximately the same for the effective stress analysis as those obtained for a circular arc.

Stability studies were also made for the final design cross-section shown in Fig. 4b. The factor of safety calculated on a circular arc using a total stress analysis was found to be 15 per cent greater than at the time of movement. The computed factor of safety along a composite surface of sliding, based on effective stresses and with the piezometric level of the foundation clay at the estimated 1–9–1953 level, was 1·4.

Completion of construction—The embankment (except slope protection) was completed in June 1956. Frequent readings were taken on test apparatus and typical changes in pore pressure are shown in Fig. 4b.

Previous to this final stage of construction three plastic slope indicator pipes were installed at the drill hole locations shown in Fig. 4b. By means of an apparatus developed by Wilson (1953) the lateral movement of the plastic pipe was determined. The maximum lateral movement occurred in hole C166 and typical readings have been plotted in Fig. 5.

Conclusions

For the cases analysed, using the data obtained when movement occurred, the computed factors of safety varied within the limits of 0·9 to 2·1 for both the total stress and effective stress methods of analyses. The computed safety factors would have been higher for both methods of analyses if the maximum deviator stress criterion for shear failure had been used, or if the results of vane tests or unconfined compression tests on tube samples had been applied in the total stress method of analysis.

The reasons for the discrepancy between computed and actual safety factors have not been explained. However, the following would appear to warrant further consideration: the possibility of progressive failure having occurred, the compatibility of the stress–strain characteristics of the foundation and fill material, the representativeness of the samples used for strength tests (obtained from a limited number of slide areas), the relationship of laboratory rates of strain to field rates, and the applicability of the shear failure criterion used.

Until such time as the stability of dams and foundations involving the more plastic clays can be predicted with greater reliability observations of movement and pore pressure must be relied upon to warn of critical conditions.

The wholehearted co-operation of Professor A. Casagrande and the Winnipeg Electric Company is gratefully acknowledged.

References

Arthur, H. G. (1951). Design of the earth embankment Falcon Dam and power plant. *U.S. Bureau of Reclamation Technical Memorandum*, No. 642, p. 15

Casagrande, A., and Wilson, S. D. (1953). Effects of stress history on the strength of clays. *Soil Mechanics Series* No. 43, Harvard University

— and Rivard, P. J. (1956). *Strength Investigation of Highly Plastic Clays*. To be published by Harvard University Department of Engineering

Hilf, J. W. (1948). Estimating construction pore pressures in rolled earth dams. *Proc. 2nd International Conference on Soil Mechanics and Foundation Engineering*, Vol. 3, p. 234

Shannon, W. L. (1955). Measurements of lateral movements in soils. *Ninth Canadian Soil Mechanics Conference*, Associate Committee on Soil and Snow Mechanics, National Research Council, Ottawa