Induced Failure of an Instrumented Clay Slope

Rupture Provoquée d'une Pente d'Essais en Argile

R.J. MITCHELL Professor of Civil Engineering, Queen's University, Kingston, Canada
D.R. WILLIAMS Project Engineer, E.B.A. Engineering Consultants Ltd., Edmonton, Canada

SYNOPSIS Piezometers, surface extensometers, lateral deflection meters and distortion gauges were installed in a natural slope composed of a sensitive fissured clay. These instruments were continuously monitored while the local ground water pressures were increased by recharge wells. Slope movements developed as the water levels in the slope were gradually increased and rotational failures occurred at a calculated safety factor very close to unity. The results are presented and analyzed with emphasis on the usefulness of slope distress monitoring instruments.

INTRODUCTION

The Champlain Sea deposits of Eastern Canada are well known for large rapid earthflows and summary papers on this natural phenomenon have been published by Eden and Mitchell (1973) and by Mitchell and Klugman (1979). Less catastrophic single and multiple rotational landslides are more common in natural slopes in these deposits and result annually in substantial property losses. Many older urban and rural developments, such as buildings, roadways and services, exist dangerously close to the crests of natural slopes — in some cases the slope crest has progressively approached these developments through a process of cyclic erosion and slope instability (Williams et al, 1979). Natural slope stabilization is often not economically warranted but a reliable method of monitoring slope distress and a landslide warning system would be valuable for evacuation of personnel and valuable equipment. Such systems could also be used in earthflow terrain since these destructive earthflows are thought to be preceded, in many cases, by smaller local natural slope failures (Tavenas et al, 1971; Eden et al, 1971).

Analysis of numerous landslides in the sensitive Champlain Sea clays in the Ottawa area has demonstrated that these slopes fail by mass dilation of the closely fissured structure inherent in these clays (Eden and Mitchell, 1970) and inclinometer installations have indicated that significant pre-failure slope movements can be expected (Mitchell and Eden, 1972). The field instrumentation and test work reported in this paper was initiated to obtain greater detail with regard to the failure mechanism and to evaluate slope movement instruments as potential indicators of slope distress.

SLOPE MONITORING

Stability analysis with suitable soil strength and ground water pressure data is generally adequate in delineating high risk and low risk situations. Natural slopes often fall into the high risk category, having calculated safety factors close to unity. Statistical analysis of data can be used in an attempt to quantify the risk and piezometers can be monitored to provide a basis for calculating seasonal variations in the safety factor. None of these efforts can provide a direct indication of slope distress. Tension cracks and toe bulging are visual indicators of slope distress but these can develop years in advance, or may not be observed at all in advance of failure. Microseismic or acoustic monitoring devices have been successfully used in granular soils but fine grained soils (particularly at a high liquidity index) have low emittivity (Koerner et al, 1977). In any event, acoustic emission monitoring must be combined with experienced interpretation to confirm slope distress and this method of monitoring may not be ideally suited for public warning systems. Slope movement monitoring was selected as being most applicable to the Champlain Sea clay slopes.

INSTRUMENTATION

A natural slope oversteepened by toe erosion was selected for instrumentation. This slope was located on Department of National Defence (DND) Canada property along the Ottawa River in Ottawa and about 500 m downstream from a landslide, pictured on Figure 1, which had occurred in 1967. The interest and co-operation of DND personnel made this an ideal location for a large scale field experiment. Some physical properties of this clay are given in Table 1 and its behaviour in laboratory tests has been detailed by Mitchell (1970).

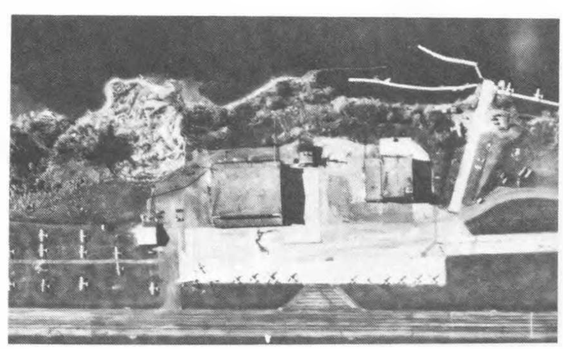

Fig. 1 Air Photo of 1967 Landslide

TABLE 1

Properties of the Clay at the Test Site

Undrained shear st. C_u, kPa	Precons. pressure P'_c, kPa	Water content %	Atterberg limits, % w_P	w_L	Sensitivity St.
70	220	62-70	25	66	8-10

Electric piezometers and tiltmeters were prefabricated and calibrated in the laboratory and installed in boreholes in the slope. The piezometers were fabricated using commercial transducers and were supplied with flushing and field calibration leads as shown on Figure 2. These were installed in 100 mm diameter boreholes with bentonite seals above and below the approximately 300 mm height of clean saturated sand which surrounded the tip. Geonor type open standpipe piezometers were also used and were installed in the same manner.

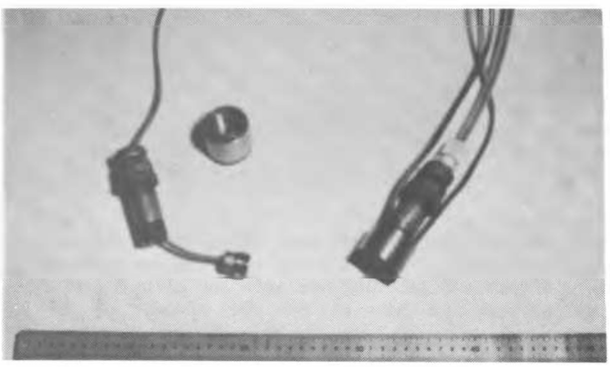

Fig. 2 Electric Piezometer Tip

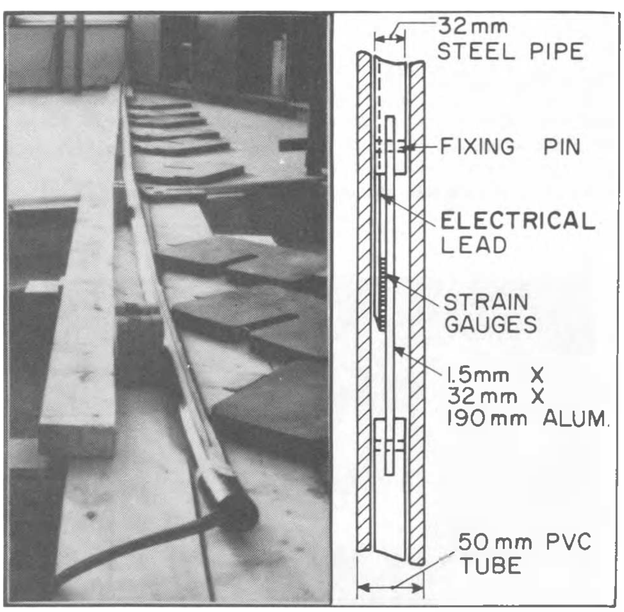

Fig. 3 Lateral Deflection Meter (LDM)

Figure 3 shows a photograph of a lateral deformation meter (LDM) during laboratory calibration and a sketch of the LDM joint assembly. These instruments consisted of a series of 32 mm O.D. rigid steel pipe sections connected by strain gauged flexible aluminum strips (see Figure 3) and encased in a snugly fitting flexible PVC tube which was sealed at the lower end. The flexible joints were located at 0.8, 2, 3, 4 and 5 m from the top of the LDM. These instruments are similar, in principle, to commercial borehole deflectometers. Each flexible joint had two active and two dummy 120 Ω electrical resistance strain gauges connected in the form of a Wheatstone bridge and protected by rubberized epoxy sealing compound and 'heat shrink' insulator tubing. All joint assemblies were individually calibrated prior to assembling the LDM and the calibrations were checked by subjecting the assembled units to known deflections (as shown on Figure 3) prior to installation in the ground. These instruments were installed in hand augered 50 mm diameter holes to provide a close fit between the PVC tube and the borehole walls. The tolerances were so close that grease had to be used to reduce friction during assembly of the LDM and during installation in the field. The shear strain meters were simply a single joint LDM sealed at both ends as shown on Figure 4.

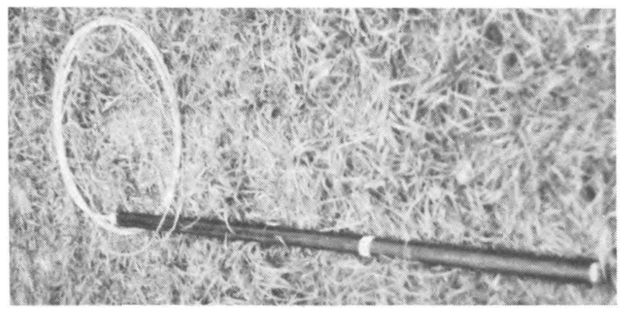

Fig. 4 Shear Strain Meter

Figures 5 and 6 show the locations of the various instruments. It is noted from Figure 5 that this slope is naturally benched and this benched characteristic, noted to be common to slopes along this section of the Ottawa River, is thought to result from the processes of erosion and regression (Williams et al, 1979).

All electric instruments were connected through a junction box located on the bench of the slope and the shielded leads were connected to the field trailer through a shallow buried plastic tube. The field trailer housed a programmable multi-channel data acquisition system and was supplied with mains power. Six volt direct current was supplied to the electric instruments from a 'power pack' converter. Recharge wells, consisting of 100 mm diameter perforated plastic pipe connected by a header, were supplied with water by a centrifugal pump with an intake from the Ottawa River.

FIELD TESTS

All instruments had been installed by 28 July, 1978 and continuous monitoring (printouts every six hours) was carried out for 20 days before ground water recharge was initiated. A natural ground water drawdown condition was known to exist in this area (Jarrett and Eden, 1970) and the average pore water pressure ratio,

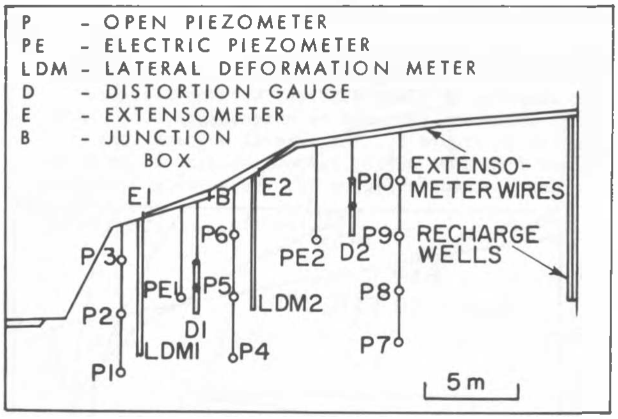

Fig. 5 Instrument Locations: Profile

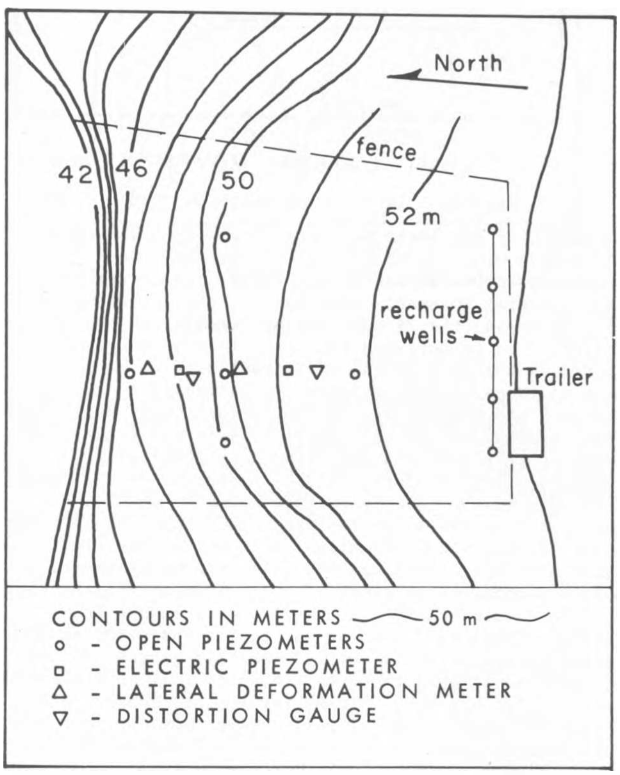

Fig. 6 Instrument Locations: Plan

r_u , for the slope was found to be zero, giving a calculated safety factor of 1.6 for the slope using the Bishop simplified equation in a computer analysis with strength data obtained by Schell (1978) for this site as $c' = 12$ kPa , $\phi' = 33°$. These strength values are identical to those found by Eden and Mitchell (1971) for the 1967 landslide site shown on Figure 1.

Ground water recharge between 8 August and 2 September 1978 was an intermittent process due to problems with the pump intake but the ground water pressures gradually increased over this period to give an average r_u value of about 0.12. Continued recharge for several days increased this r_u value to a temporary peak of about 0.35 and a tension crack appeared in the ground surface near the location of the junction box (see Figure 5) — the surface of the lower bench had experienced a total lateral movement of about 50 mm but the tension crack was less than 5 mm in width and there was no discernible vertical displacement across this crack. These observations indicate a general distortion of the slope rather than a rotational slip. The above noted results are displayed graphically on Figure 7.

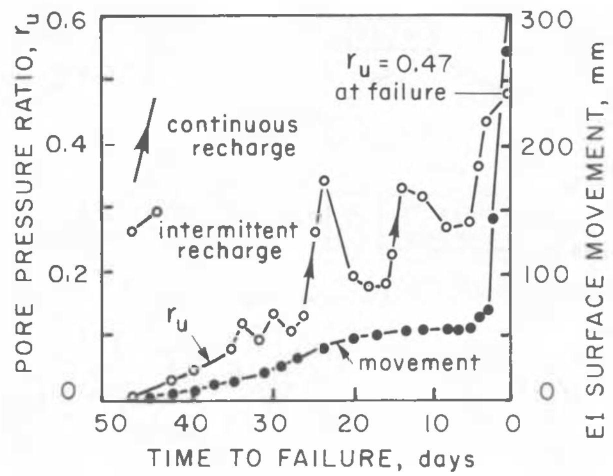

Fig. 7 Slope Movement and r_u

Following this evidence of slope distress, the recharge was reduced and the average r_u value was maintained at about 0.2 for several days by operating the pump on timer intervals. As shown on Figure 7, the lateral slope movements ceased and the slope did not become active again until the average r_u value exceeded the previous peak of 0.35.

The ground water pressure response to recharge was surprisingly rapid and this is considered to be due to the relatively high permeability of the weathered horizon. The soil response to increased r_u , as measured by slope movements, appears to be slow at higher safety factors but was fairly rapid as failure was approached (see Figure 7). Intermittent recharge was intended to simulate natural springtime conditions and the only substantial assistance received from the weather occurred when 51 mm of rain fell during the 24 hour period preceding the slope failure. The slope failed at 0500 h on 2 October 1978 and most of the instruments maintained communication with the data acquisition system right up to the time of failure.

Figure 8 shows the phreatic surface and calculated safety factors at three stages of the field test. For stability analysis, r_u values were obtained from piezometer readings as $r_u = u/\gamma h_s$ for each computed slice of height h_s — the average r_u values noted on Figure 8 are obtained by averaging all r_u values for the lower two thirds of the critical circle.

Figures 9 and 10 illustrate the three rotational slips that were observed on the day following the induced landslide. The field failure agrees closely with computed critical circles; for this benched slope, a range of analytical circles have closely similar safety factors. Further details on the analysis are presented by Williams (1979).

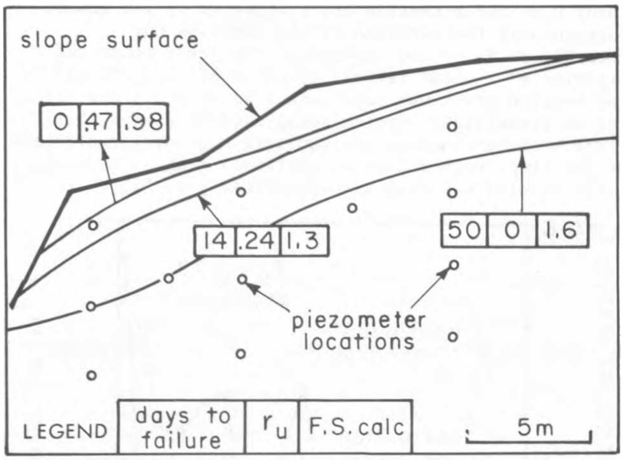

Fig. 8 Phreatic Surface, r_u , and Factor of Safety

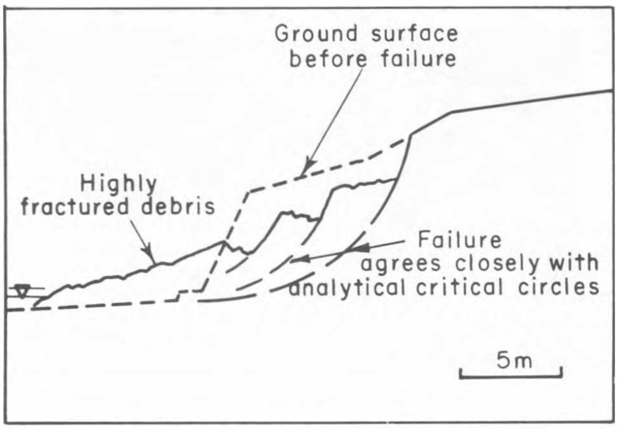

Fig. 9 Profile of Observed Landslide

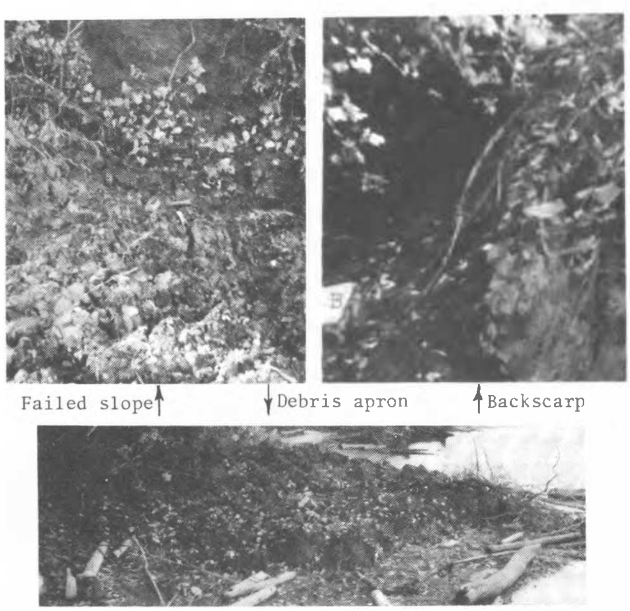

Fig. 10 Failed Material

DISPLACEMENT MONITORING RESULTS

The main purpose of this paper is to evaluate the displacement monitoring instruments with respect to their adaption as slope distress warning systems. Surface movements measured by extensometer E1 were presented on Figure 7. The lateral deformations recorded from LDM1 during various stages of the field tests are plotted on Figure 11. Measurable downslope

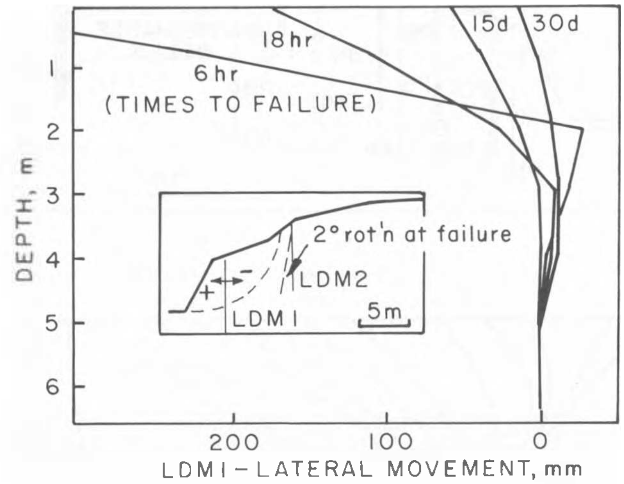

Fig. 11 Lateral Deformations, LDM1

movements of the instrument were registered between 30 and 3 days prior to failure as the ground water pressures increased and the safety factor decreased. Major lateral movements developed in the upper 3 metres of soil during the 24 hour period immediately preceding failure. It is believed that this form of movement is caused by volume increase (dilation) within the soil mass as it approaches failure. Dilation at shear stresses above about 80% of the failure shear stress has been observed in drained triaxial tests and shear box tests on this fissured sensitive soil (Eden and Mitchell, 1970; Jarrett, 1972). Extensometer E2 and LDM2 showed very little surface movement up to two days prior to the slope failure. LDM2 was not anchored below the toe slope elevation; during the final 24 hours preceding the landslide this instrument showed a positive rotation of close to 2° (see inset, Figure 11). The rotation of LDM2 is considered to result from lateral stress reduction as the toe of the slope moved laterally toward the river. Distortion gauges D1 and D2 were positioned close to the analytical locations of critical failure arcs. These instruments cannot detect solid body translation or rotation but measure only local angular distortion (shear strain) within the soil mass. Figure 12 shows a plot of the measured shear strains. Both of these gauges reflected the development of shear strain within the slope about 8 days after the ground water recharge was initiated. This is considered to result from the increased stress ratio imposed on the soil by the reduction in the average effective stress. Following this initial loading reaction, gauge D2 showed a continued angular distortion which confirmed the deformation pattern that developed in LDM2 (i.e. a positive angular rotation). The reaction of gauge D1 is considered to be further evidence of mass dilation in the zone of shear rather than the formation of a rotational sliding surface. About 30 hours prior to failure, however, an increased rate of angular distortion was measured by gauge D2 .

The calibrations and measurement accuracies of the displacement monitoring instrumentation are discussed

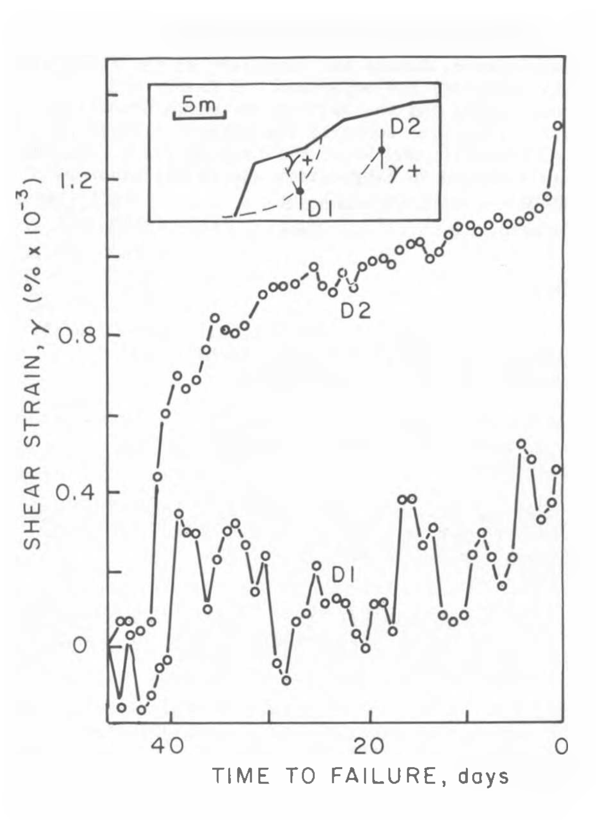

Fig. 12 Distortion Gauge Results

by Williams (1979) who concludes that these accuracies are sufficient to determine an angular rotation of $\pm 1 \times 10^{-6}$ radians or a shear strain of $\pm 1 \times 10^{-4}$ % . Thus while some of the scatter on Figure 12 may be due to measurement inaccuracies, these types of instruments are considered to be capable of providing an early indication of slope distress.

WARNING DEVICES

Where there is concern for the long term stability of a slope in a fissured clay, warning devices can be employed to provide a direct indication of slope distress. In some cases an early warning would be desired in order that remedial measures could be undertaken. Standard inclinometer tubes, which may be useful in other types of materials or other types of failures (Gould and Dunnicliff, 1971) are not considered accurate enough to provide an early indication of slope distress in stiff fissured clays. The fact that the Champlain Sea clays are sensitive should not alter this consideration since the sensitive microstructure does not break down until a slope failure has developed. Such cases could be considered as short term monitoring situations since a decision with regard to the need for remedial action should be made following (or during) one wet season of monitoring. The lateral deflection meter (LDM), or other monitoring devices of similar accuracy, would appear to be capable of providing sufficient information on slope distress (or the absence of distress) to allow this decision to be made. Monitoring and interpretation requires geotechnical expertise and accuracy should be considered more important than long term durability in selecting the monitoring system. During the fairly steady creep stage between 15 and 30 days prior to failure (see Figure 7) LDM1 showed an average angular distortion of about 8×10^{-3} radians (shear strain of about 0.8%). This angular distortion rate of 0.05 percent per day would certainly be considered indicative of the need for immediate remedial action. Ideally, instrumentation should be sufficiently accurate to determine whether the distortion rate is increasing or decreasing over a short time span. Simple distortion gauges with one flexible joint (such as described in this paper) are extremely accurate but the performance of this type of gauge is dependent on the installation location and type of failure. With reference to the results shown on Figures 11 and 12 it is apparent that distortion gauges should be installed close to the slope surface rather than close to the analytical critical circle in these strongly fissured clay slopes. The opposite might be expected in stiff unfissured materials where solid body rotational shear might begin to develop at an early stage of slope distress.

In many cases where natural slopes are a threat to the safety of personnel or mobile equipment, a warning system may be required for evacuation purposes only. In these cases reliability is more important than accuracy: the instrument should be maintenance free over long periods of time and should sound an alarm only in case of imminent failure. Figure 13 shows a normally-off battery powered instrument, which also provides a visual reading for inspection purposes, that is considered to be appropriate as a reliable public warning device. The data on Figures 7 and 11 show clearly that a direct measurement of surface (or near surface) lateral displacement on the lower

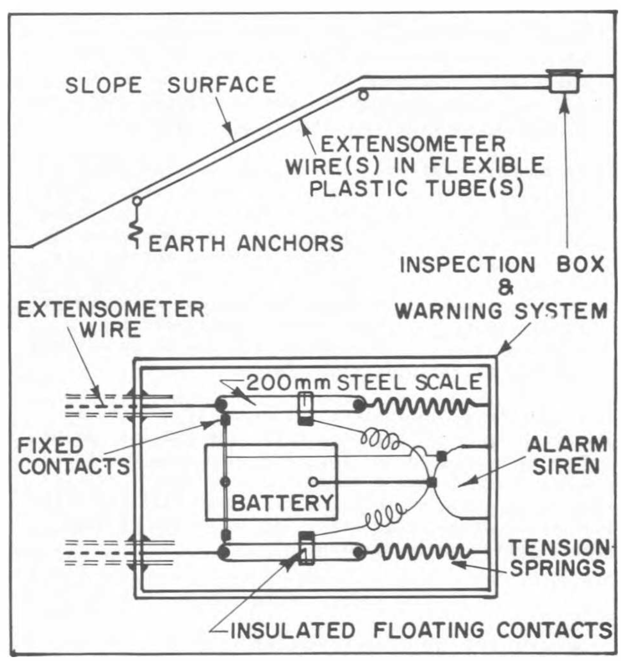

Fig. 13 Suggested Warning System

slope surface would be superior to any deeper measurements in these fissured clay slopes. Extensometer wires attached to earth anchors and encased in flexible plastic tubing are connected to an inspection box and warning device located above the crest of the slope in the proposed system. Tensioning springs and steel scales are provided to allow the slope movement to be visually inspected and an insulated sliding contact on the steel scale would be adjusted to establish the allowable immediate displacement. A reliable battery would energize the warning device if rapid slope movement exceeded the allowable limit. The field test indicates that an allowable limit in the order of 100 mm would provide a warning several hours in advance of a slope failure in the Ottawa area fissured clay slopes. Periodic inspection and adjustment for seasonal creep in the slope would be the only maintenance requirements and a record of adjustments could also provide a basis for long term evaluation of progressive movements. The extensometer system could be buried to a depth such that temperature effects (including frost heave) could be largely eliminated.

CONCLUSIONS

This paper outlines the methods employed in instrumentation and in inducing a failure in a natural slope in a stiff fissured clay. The results of this full scale field test are presented with special emphasis on the use of displacement or deformation monitoring equipment in stability evaluations. The detailed conclusions resulting from this work are:

1. Deformations in marginally stable long term slopes develop in sympathy to increases in ground water pressure.

2. The simplified Bishop analysis gave a safety factor of unity for the conditions at failure.

3. Electrical strain-gauged lateral deformation meters were able to measure slope movements after the calculated factor of safety was reduced below about 1.2 and these instruments are considered useful for providing an early warning of slope distress.

4. Approaching failure, the maximum displacements in the slope were found to be at ground level. This observation is considered to be a result of volume increase in the soil mass as the maximum shearing resistance of this strongly fissured material is developed within a zone of shear. On the basis of this observation, a simple mechanical extensometer arrangement, connected to a normally-off d.c. alarm system, is recommended for use as an advance warning of imminent slope failure.

While the above conclusions are directly applicable only to the area of the test site, the results can be used to provide guidance for slope distress monitoring in other stiff fissured materials. Different results might be expected in softer soils or in stiff brittle soils where progressive softening in a zone of high shear stresses could develop.

ACKNOWLEDGEMENTS

The authors wish to acknowledge: the Department of National Defence, Canada for provision of the field site and site services; the Department of Supply and Services, Canada and the National Research Council of Canada for financial support; the technical staff of Queen's University and Royal Military College, Kingston who were involved in fabrication and installation of instruments.

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