57. PORE PRESSURE MEASUREMENTS IN SENSITIVE CLAY SLOPES

Project 740052

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Introduction

General

The sensitive clays of eastern Canada are susceptible to slope instability. The determination of the degree of stability of a slope requires an appropriate anayltical model which adequately represents the slope failure mechanism. Input factors into such a model are: slope geometry, stratigraphy, strength parameters for each soil layer, and an estimate of the pore pressures acting along a potential failure plane. Obtaining the actual pore pressure distribution in a slope requires the installation and monitoring of a series of piezometers – a procedure which is both time-consuming and expensive. On a regional scale it is impractical to instrument every potentially critical slope; therefore the slopes investigated in Ottawa Valley were classified into two groups according to their stratigraphy. Typical slopes of each group were monitored, and the data can be extrapolated with a reasonable amount of confidence to other slopes in the region.

Classification of Clay Slopes and Soil Descriptions

The division of sensitive clay slopes into two groups or geological settings was based on data collected during a drilling and mapping project covering Ottawa Valley (Fransham et al., 1976). Typical profiles of the two settings are shown in Figure 57.1. The main difference between the two is the existence of a sand and a clay and silt layer on the surface of setting type A as compared to setting type B which has desiccated clay as an upper layer. The sand acts as a groundwater reservoir thereby maintaining a high water table and saturated conditions in the underlying clay. The desiccated crust is fissured and somewhat more permeable than the intact clay but cannot store a significant volume of water to maintain saturated conditions near the surface. The red-grey colour banded clay and the massive grey clay are not thought to be sufficiently layered to create an anisotropic permeability. On the other hand the banded silt and clay and the varved clay can be expected to have a coefficient of horizontal permeability higher than the vertical permeability.

The till ranges from sandy to silty. Because it is a mixture of sand, silt, and boulders, a relatively low coefficient of permeability can be expected due to its low porosity. During drilling operations open fractures were encountered in the bedrock in some holes, and all the drilling water was lost. The drill rig pump has a capacity of 500 gpm, thus the effective permeability of the fractured bedrock must be relatively high.

All irregularities on the bedrock surface have been ignored for the purpose of this study. This is not the general case in the field where the bedrock surface is undulating and contains many scarps and valleys. A flat bedrock surface does not represent too serious a simplification as some knowledge of the bedrock profile has to

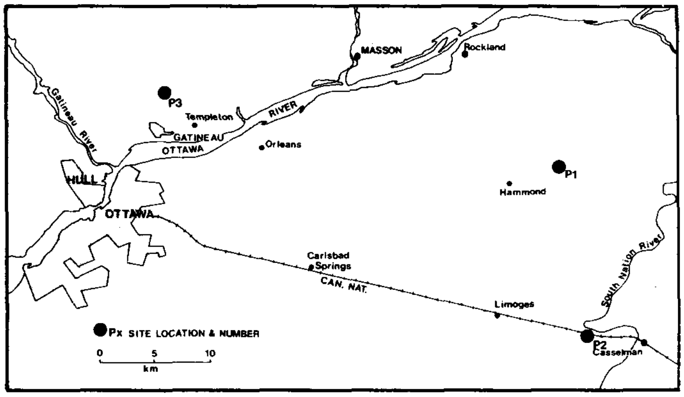

Figure 57.1. Location of instrumented slopes.

be established before a stability analysis can be carried out. The pore pressure profile can be modified assuming that the bedrock is a significant aquifer with permeabilities much higher than the overlying clay. Before an estimate of the pore pressures is made, judgment also is needed concerning the relative influence on the pore pressure distribution of: slope geometry, thickness of overburden, location of any groundwater discharge, and depth of erosion of streams at the toe of the slope.

Site Locations

Setting Type A

Two sites (P1 and P2) of this geologic setting were instrumented with piezometers; their locations and that of the third site are shown in Figure 57.2. Nine piezometers were installed at location P1: four were at the crest of the slope, three at the midslope, and two at the toe of the slope. At each of the three piezometer locations perforated pipes were driven to a depth of 3 m and served to measure the surface water table elevations. Ten piezometers were installed at site P2: one group of five piezometers was near the crest of the slope, and another group of five piezometers was about 200 m back from the crest. The slope profiles, piezometer elevations, mean pressure heads for each piezometer, and the site stratigraphy for sites P1 and P2 are shown in Figures 57.3 and 57.4 respectively.

Setting Type B

Site P3 is located on the Quebec side of Ottawa River. Nine piezometers were installed on the slope: three at the crest, three at the midslope, and three at the toe. The site profile and mean pressure heads are shown in Figure 57.5. Because a few problems were encountered at this site, the data for site P3 are not as complete as for the other two sites. The piezometers were installed in 1974,

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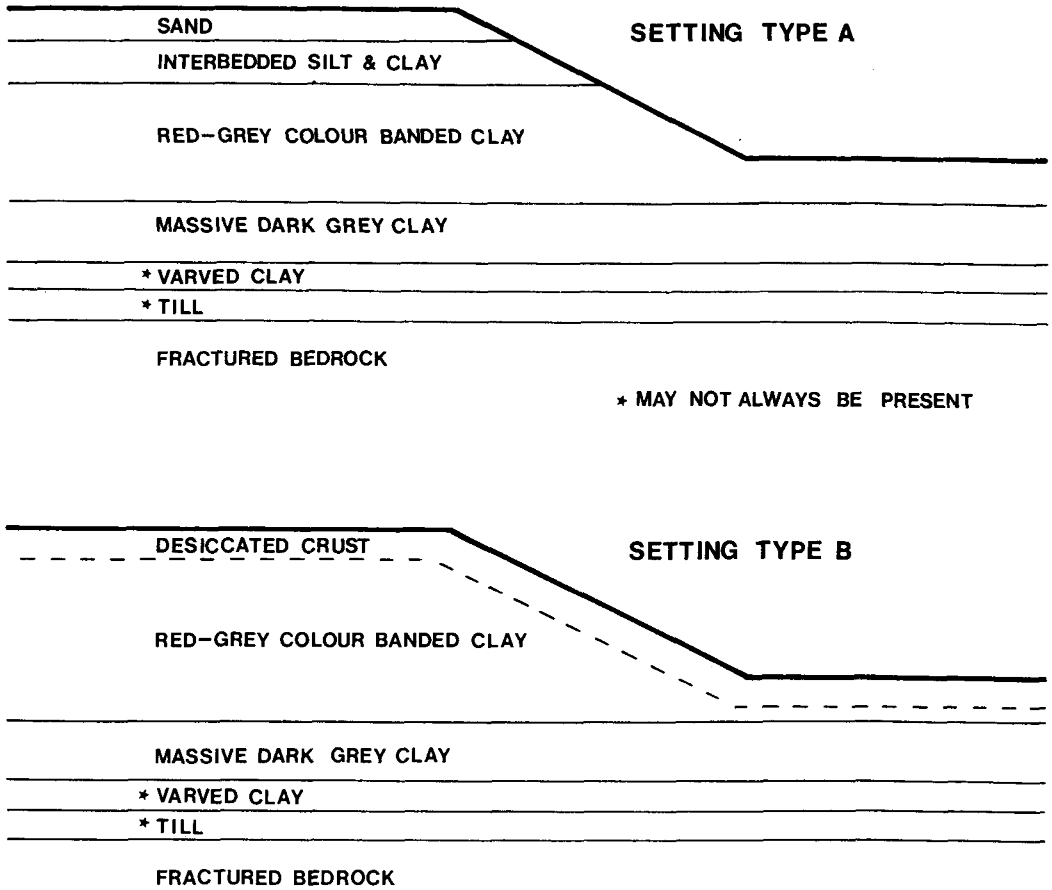

Figure 57.2

Geological settings of sensitive clay slopes.

and the set on the midslope subsequently had become obscured with soil and grass. These piezometers were not located until late in the field season, and only one set of readings was obtained. One of the piezometers at the crest became blocked and was useless for any further readings. Accessibility to the toe of the slope was poor, and readings were difficult to obtain.

Instrumentation

General

Considerable research has been done on the design of piezometers and the relative benefits of each design (Lindberg, 1965; Hvorslev, 1951). Hvorslev (1951) stated that "the total volume or flow of water required for equalization of differences in hydrostatic pressure in the soil and in the measuring device depends primarily on the permeability of the soil, the type and dimensions of the device, and on the hydrostatic pressure difference." The volume of water required to equalize the pressure difference, therefore, should be kept to an absolute minimum and the intake screen should be large so that the water involved in the equalization can be drawn from a greater volume of soil. It is also important that no seepage occurs along the piezometer-soil interface otherwise the pressure head will be influenced by seepage to or from strata other than the one in which the piezometer is set.

Piezometer Design

The piezometer is essentially a well point grouted into the soil or rock at the required depth. A schematic diagram of the piezometer design is shown in Figure 57.6A. The design is much the same as that described by Brooker et al. (1968) with the exception of the procedures described below, which were followed to ensure that no seepage occurred along the soil-piezometer boundary. A 13 cm-diameter borehole was drilled to the required depth; unless the soil was sufficiently stiff, steel casing was used to support the walls of the hole. A known volume of silica sand was poured down the hole to form a sand filter between the clay and the well point. A 5 cm-diameter well point was attached to a steel pipe of similar diameter, and a large rubber stopper was placed at the top of the well point. The stopper acted as a seal between the casing and the piezometer and helped to carry any sand down the hole that might have become lodged in the side of the casing. The rubber also kept the bentonite balls, which were added above the rubber seal, from falling down around the well screen. A steel washer placed between the stopper and the bentonite balls gave the rubber a certain amount of rigidity and prevented it from being displaced. A similar steel washer welded 30 cm above the rubber stopper confined the bentonite balls in the vertical direction and allowed the balls to expand laterally to form a good seal between the piezometer and the soil. Once the piezometer was at the required depth a cement-bentonite grout was pumped down between the

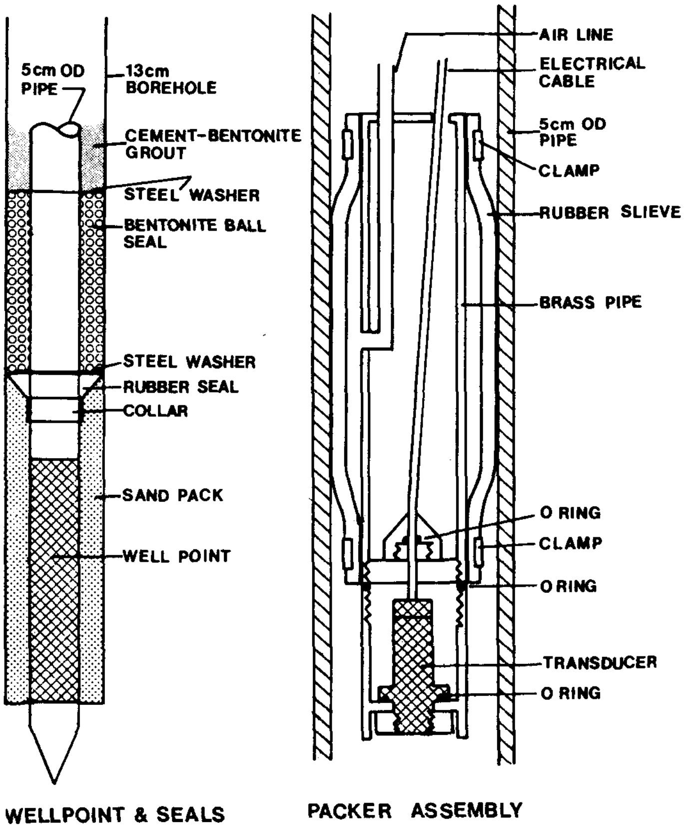

Figure 57.3. Piezometer design.

casing and the piezometer. The casing was then removed, and the hole was filled to the top with grout leaving only the 5 cm pipe showing above ground level. The entire piezometer then was flushed out with clean water, thereby removing any soil which may have entered the piezometer during installation.

Pore Pressure Measurement

An inflatable packer with an electrical transducer (see Fig. 57.6B) was lowered into the piezometer to a depth just above the well screen. The packer then was inflated by air pressure resulting in an instantaneous increase in pore pressure due to the volume of water displaced by the packer. The pressure then decreased to some equilibrium value which equals the pore pressure in the surrounding soil mass. The time required for equalization depends on the permeability of the soil or rock, the geometry of the piezometer, and the response of the packer to the inflation pressure.

Figure 57.7 shows two examples of pressure equalization curves. The bedrock in this case is highly fractured and therefore is permeable. The pressure-time curve decreases rapidly to the equilibrium pressure in six minutes. For the relatively impermeable clay, the decrease is much slower, requiring at least an hour before equilibrium is reached. Experiments are now underway to determine if some relationship can be obtained which will relate the pressure equilibrium curves to the permeability of the soil.

The advantage of this measuring system is that once the packer is inflated it seals off the rest of the pipe from the soil. Because the volume of water required to activate the pressure transducer is small, the response time for changes in the pore pressure in the soil should be rapid.

The main disadvantage of this system is the volume of electronics which are part of the recording unit. A schematic diagram of the apparatus is shown in Figure 57.8. An attempt was made to reduce the size of the generator to 300 watts; however, the voltage was too erratic to allow for accurate readings. The Hewlett Packard Chart recorder, measuring about 15 cm on each side, is about the smallest usable machine available. Some saving in bulkiness may be had by eliminating the DC voltmeter and obtaining a smaller ampliifer. These reductions in size would be marginal and do not represent a major improvement in the equipment design. A more useful change would be to use a pneumatic transducer which would eliminate the need for any electronic equipment. The entire apparatus then could be reduced to a small read-out box and a cylinder of gas to pressurize the packer. Another disadvantage of this system is the length of time required to obtain one reading in clay; generally about one hour is required before equalization is reached therefore restricting the number of piezometers that can be read in a given time period. It would be far more efficient to have several packers, which could be inflated, thereby increasing the number of readings per day. Pneumatic transducers are relativley inexpensive, and the cost for three packer assemblies would be about the same as one electrical transducer packer. Considering the savings in cost and size of the equipment, it would be preferable to change the system over to a series of pneumatic transducer packers.

Results and Discussions

General

The general case of downdrainage in slopes of Ottawa Valley has been reported by various investigators (Jarrett and Eden, 1970; Lefebvre et al., 1976) and was found in the slopes instrumented for this study. For stability calculations the maximum recorded value of pore

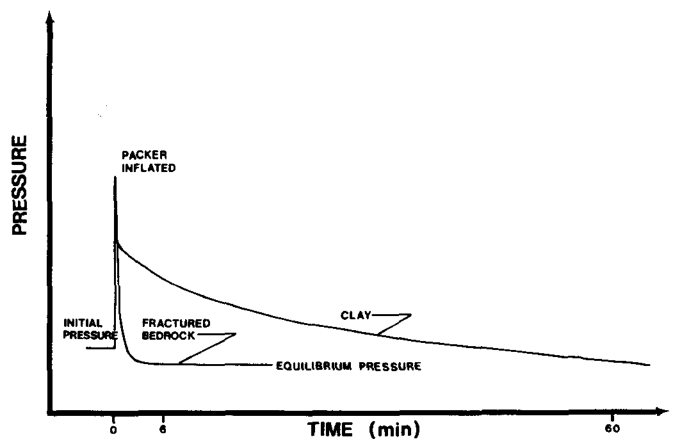

Figure 57.4. Pressure-time curves for clay and fractured bedrock

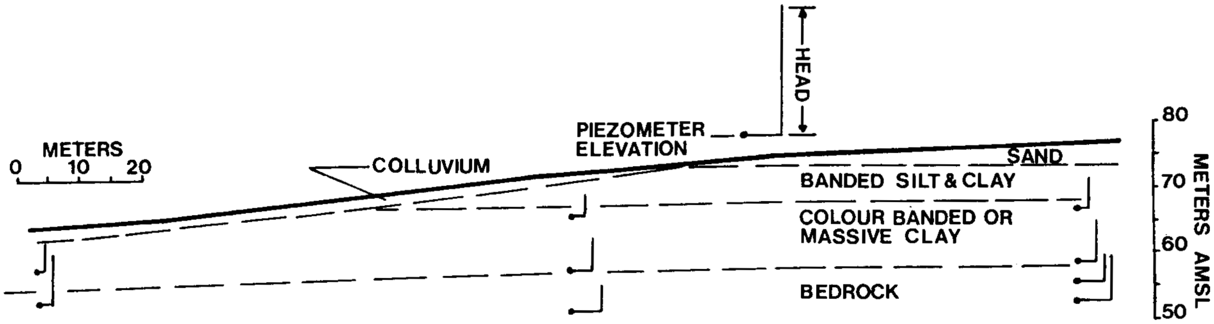

Figure 57.5. Site Pl, profile and mean heads.

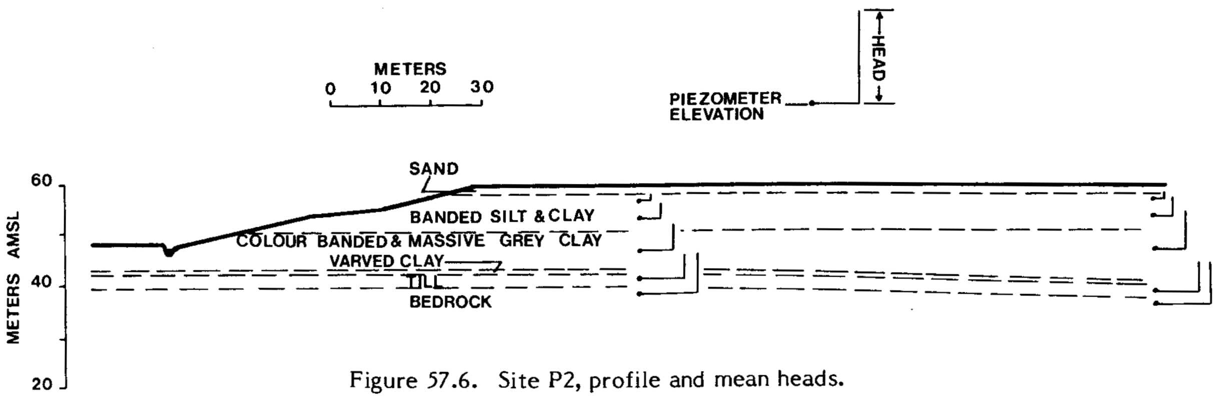

Figure 57.6. Site P2, profile and mean heads.

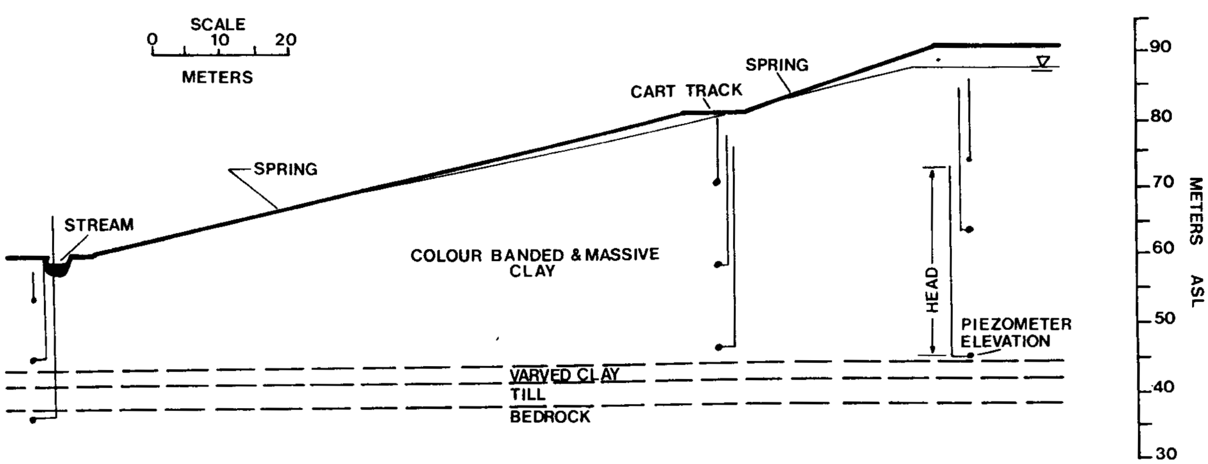

Figure 57.7. Site P3, profile and mean heads.

pressure is critical. A statistical approach is being formulated which will allow for a better estimate of the pore pressure variability and the probability of attaining a maximum value. Only the mean pore pressure will be discussed here so that the general trends of the pore pressure with depth can be seen.

Type A

Figures 57.3 and 57.4 illustrate the main results for the two slopes falling into this group. The difference in vertical gradient is of the order of 1 and in horizontal gradient is about 0.025. This difference in gradients results in flow being almost vertical downward, with only a minor component in the horizontal direction. The results from the piezometer nest at the toe of site P1 shows that the vertical gradients are smaller at the toe of the slope than at the crest of midslope. In wet periods the pore pressures at the toe of the slope approach hydrostatic. No piezometers were installed at the toe of site P2. Because the horizontal gradients are small, however, if an extrapolation were made to the toe area, the pore pressures likely would be hydrostatic and the heads would be close to the level of the small stream at the toe.

Type B

The pore pressure distribution at this site is somewhat different from that at the previous two sites. Downdrainage again was identified at the crest of the slope; however, the toe had artesian pressures. This type of distribution is similar to the neoclassical flow patterns in slopes which have been described by Patten and Hendron (1974). The artesian pressures dissipate near the toe if the depth of river erosion at the base of the slope intersects the fractured bedrock surface (Scott et al., 1976).

Groundwater Development with Time

It is interesting to speculate on the history of groundwater development in Ottawa Valley and how the pore pressure distribution has changed with time. During its early stages Ottawa River meandered and cut several broad channels in Champlain Sea sediments. At this stage the slopes were submerged completely, and no groundwater flow took place. As the land rose, the water level in the channels dropped and mainly hydrostatic conditions prevailed in the portion of the slope above river level. In the final stage, which is in effect today, the rivers and streams have eroded down to bedrock and are able to maintain a lower piezometric surface in the bedrock than in the upland plains. Major rivers are the ultimate local groundwater discharge, and their surface elevations have a strong influence on the piezometric elevation in the surrounding bedrock and clay.

Influence of the Pore Pressure Distribution on the Stability of Natural Slopes

The critical stage for slope instability in the history of Ottawa Valley was when the groundwater profile was essentially hydrostatic. Before this time the slopes were submerged completely and were somewhat stable.

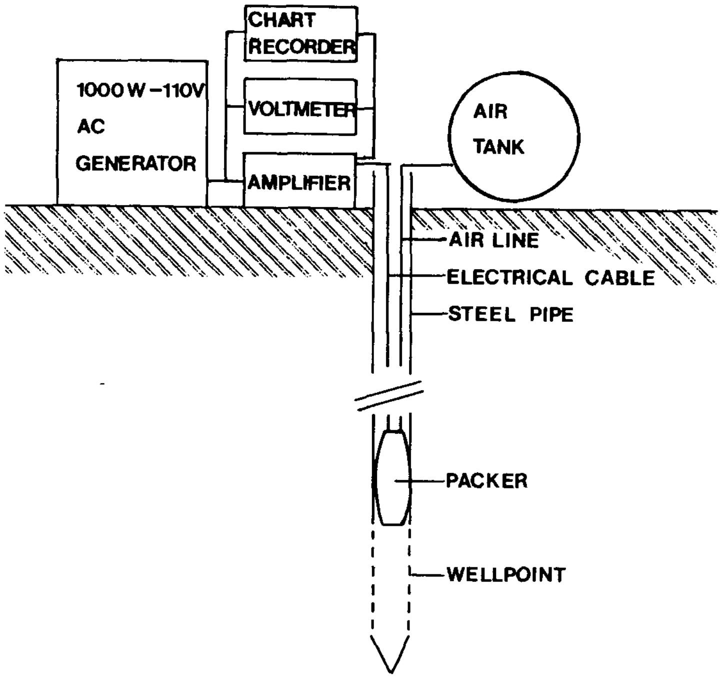

Figure 57.8. Equipment schematic.

The slopes are subjected to downdraining which adds to the overall stability. During periods of prolonged wetness the pressures in the upper part of the soil profile approach hydrostatic, thereby reverting to conditions favourable for instability; however, the clay has been overconsolidated due to the increase in effective stresses resulting from the downward flow. Thus the clays may be somewhat stronger than they were before consolidation. The increase in strength with time may not allow for the development of the extemely large slides like those which have occurred in the past. Groundwater fluctuations, however, may trigger smaller slides like the South Nation River Slide of 1971. The moments resisting failure along the critical failure circle will be dependent partly on the pore pressure distribution in the slope. As the portion of the slope that is subject to near hydrostatic conditions increases, therefore, the probability of failure also increase. This would explain why most slope failures occur during spring when the water table is high and pore pressures are maximum.

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