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GROUNDWATER FLOW IN EASTERN OTTAWA

BY

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Groundwater flow in eastern Ottawa[1]

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A mile-long line of piezometers were installed at various depths in a clay terrace system east of the City of Ottawa. Measurements indicated a significant downward piezometric gradient through the clay layer to the underlying bedrock. The downward flow of water through the clay affects both existing effective stresses and the stress history of the clay deposit.

On a installé à diverses profondeurs une série de piézomètres sur une distance d'un mille dans l'argile des terrasses fluviales à l'est d'Ottawa. Les mesures ont indiqué un gradient piézométrique fortement décroissant au travers des couches d'argile jusqu'au roc sous-jacent. L'écoulement des eaux au travers de l'argile influence à la fois contraintes actuelles et l'évolution des contraintes affectant le dépôt d'argile.

[1]NRCC 11422.

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The discovery of an ancient earth flow in a river terrace to the east of Ottawa led to an investigation of slope stability in neighboring areas (Crawford and Eden 1967). These studies were undertaken to try to correlate present-day methods of analysis with known cases of slope failure. During these and allied investigations in this area a downward flow of the groundwater became apparent. The hydraulic pressure gradient necessary to maintain this downward flow radically affects any estimate of the *in situ* effective stresses, which in turn are of paramount importance in slope stability problems. It was therefore decided to investigate the nature of this downward flow.

Study Area

The area studied is located along the south bank of the Ottawa River. At this location, the river has cut into postglacial clay sediments which are up to 160 ft (49 m) thick. The clay is underlain by a thin layer of glacial till and then by bedrock which may be either shale or limestone (Wilson 1946). A contour plan of the study area (Fig. 1) shows two flat plains divided by a slope 90 ft (27 m) high. The landslide scar is seen in the southeast part of the plan. Figure 3 is an aerial photograph of the same region.

Piezometric Instrumentation

It was decided at the outset to install a line of piezometers in the clay, southward from the river to the height of land at the Montreal Road where the clay beds thin out into the till or rock formations. In case drainage was not perpendicular to the river, some piezometers were offset from the general line to give an indication of the lateral drainage. All these piezometers were the "Geonor" type having a porous bronze tip with a plastic stand-pipe of $\frac{3}{8}$ in. (0.95 cm) diameter extending to the surface. An electrical probe was lowered down the stand-pipe to determine the water level.

A section taken along the line of the piezometers showing their positions is illustrated in Fig. 2. Piezometers 11 and 12 are those offset from the line. They are plotted on this section in the position they hold when joined perpendicularly to the main line. Figure 1 gives the plan positions of the installations. Two rockpoints were installed at the locations shown in Figs. 1 and 2. These points are merely small diameter casings (approximately 2 in. (5 cm) diameter) driven and sealed into the upper layers of the rock. A core was then drilled out through the casings for about 5 ft (1.5 m) to allow water into the stand-pipe, permitting measurement of the piezometric elevation in the rock. The elevations of all the piezometers and rockpoints are listed in Table I.

Pore-water Pressure Observations

Measurements of the piezometric levels have now been made in all the instruments for at least 1 y, and some have been observed for considerably longer periods. Table I lists the maximum and minimum piezometric elevations measured in each stand-pipe. It will be noted that the piezometers near the slope have large variations between maximum and minimum levels, whereas those in the flat areas tend toward constant values.

Discussion

The piezometric observations are useful for the analysis of the groundwater flow regime in the slope between the flat areas as outlined in the paper by Crawford and Eden (1967), or for the analysis of the downward piezometric gradient in the flat areas. The following discussion deals with this latter aspect.

It may first be helpful to define the component parts of groundwater pressures. From Bernoulli's equation one can relate the total hydraulic head at a given point to a particular datum elevation in the following way.

Total Head = Pressure Head

+ Elevation Head + Velocity Head.

Pressure head is the water pressure at any point considered and in the soil mechanics sense is the pore pressure as measured in an open stand-pipe. Elevation head is that due to the position of the point considered with respect to the datum level. Velocity head is that due to the velocity of the water. In the case of groundwater this term may generally be ignored.

It follows that where general groundwater flow is discussed then changes in the total head

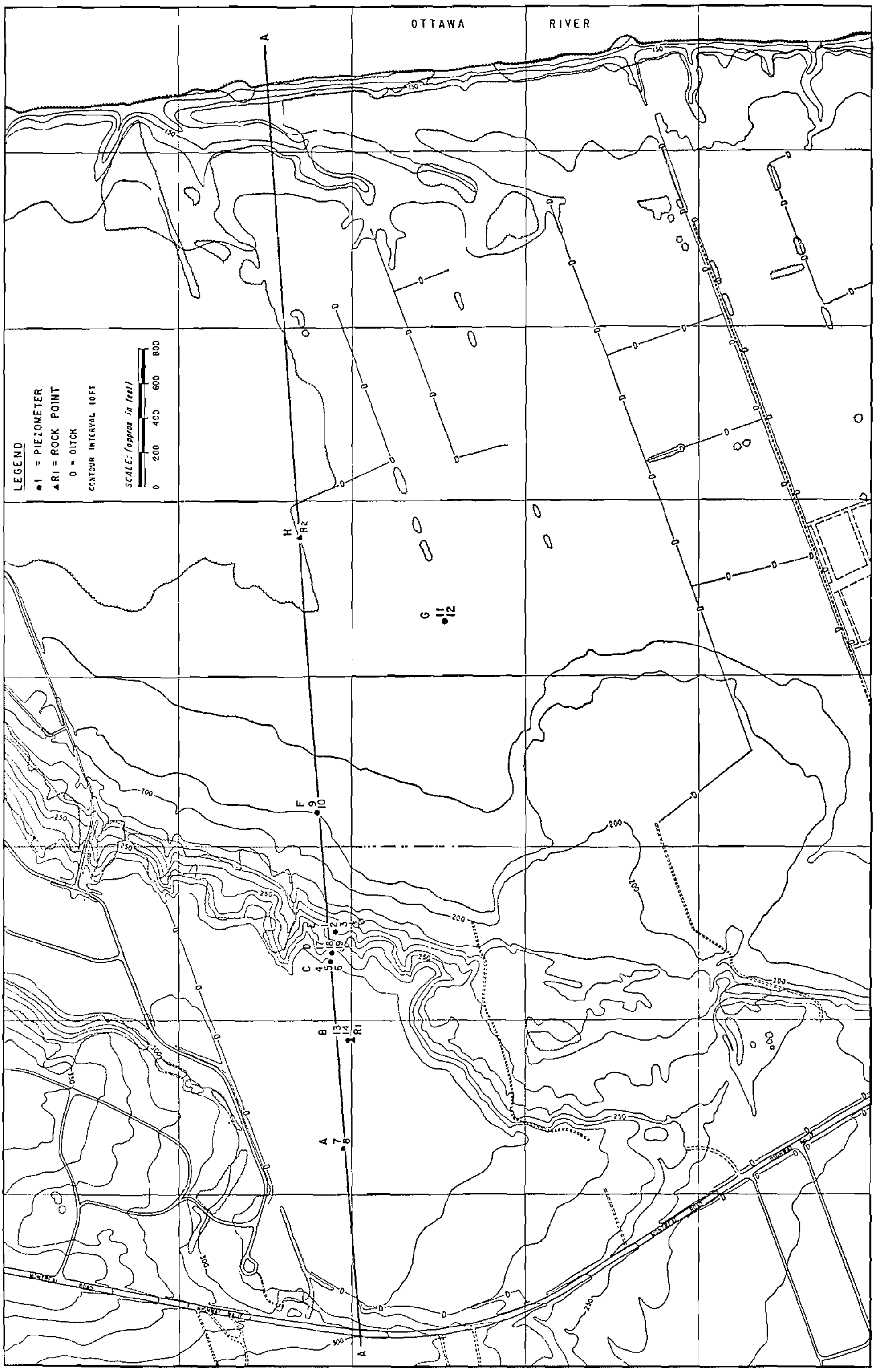

FIG. 1. Map of study area showing piezometer locations.

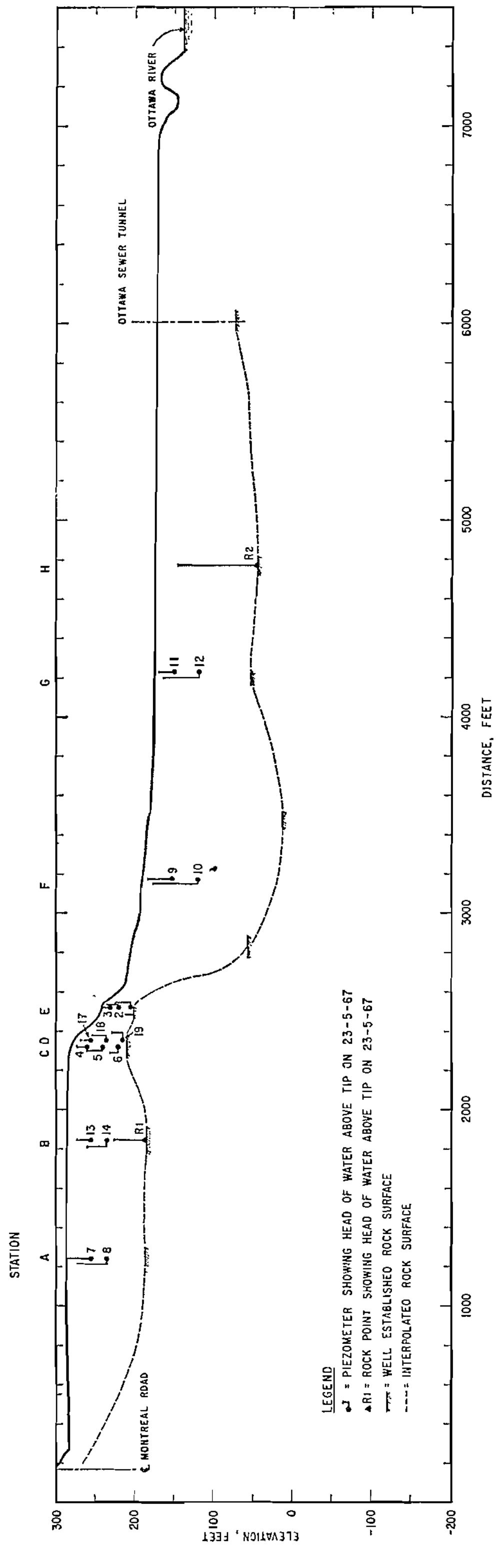

FIG. 2. Section along main line of piezometers.

FIG. 3. Aerial photo of study area (courtesy City of Ottawa).

are considered. Where effective stresses and related soil properties are concerned, the pressure head (pore pressure) at the point in question is the parameter used.

The results from the piezometers at stations A, B, F, and G reveal a substantial total head loss in a downward direction. Figure 4 gives typical values of the pressure heads in piezometers 7 and 8 at station A. They are plotted from their respective elevations and show that in the 20-ft (6-m) vertical distance between their tips there is a loss in total head of 7.2 ft (2.2 m) of water. A comparison of the pressure heads between piezometers 8 and 14, which

TABLE I
Details of piezometric installations

Station	Piezometer	Surface elevation (ft)	Tip elevation (ft)	Maximum piezometric elevation measured (ft)	Minimum piezometric elevation measured (ft)	Difference ft	Difference m
A	7	287	257	281.3	279.3	2.0	0.61
	8	287	237	273.9	272.7	1.2	0.37
B	13	284	255	272.4	270.2	2.2	0.67
	14	284	235	261.7	259.7	2.0	0.61
	R1	284	185	228.6	227.4	1.2	0.37
C	4	280	260	279.2	265.2	14.0	4.27
	6	280	240	259.4	250.8	8.6	2.62
	5	280	220	233.0	226.3	6.7	2.04
D	17	276	256	273.8	262.7	11.1	3.38
	18	276	236	254.7	246.1	8.6	2.62
	19	276	216	228.7	221.9	6.8	2.07
E	3	240	230	239.5	237.7	1.8	0.55
	2	240	220	238.5	235.7	2.8	0.85
	1	240	205	225.6	221.8	3.8	1.16
F	9	190	152	185.5	181.3	4.2	1.28
	10	190	120	177.6	176.8	0.8	0.24
G	11	175	150	173.7	170.2	3.5	1.07
	12	175	118	164.9	163.5	1.4	0.43
H	R2	174	45	148.2	—	—	—

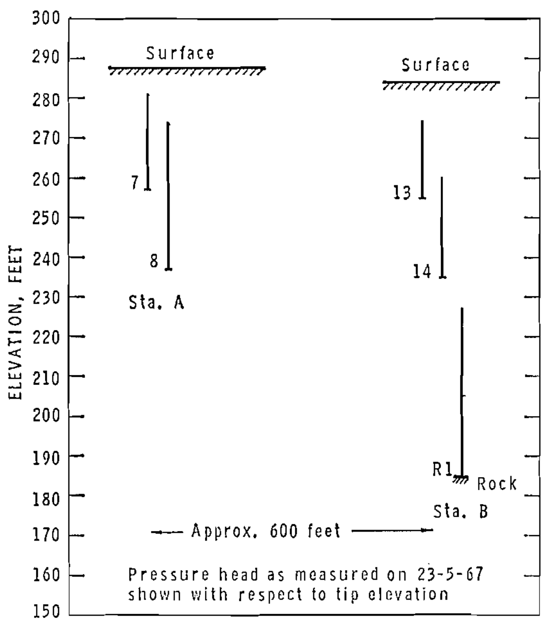

FIG. 4. Typical values of pressure head of water on upper flat area.

are at similar elevations but at stations A and B respectively, shows there is a loss towards the river of only 11.3 ft (3.4 m) in a distance of approximately 600 ft (180 m) horizontally. The total head gradient is thus about 20 times greater in the vertical direction than in the horizontal. Similar relative conditions exist between the piezometer groups on the lower plain at stations F and G.

It is known[2] that in this area the interface between the rock and the clay is a water-bearing stratum. It is believed that cracks and fissures in the upper part of the bedrock provide a good drainage system towards the river. None of the observations has cast doubt on the river as the final drainage sink. Indeed at the deepest piezometer, rockpoint R2, the total head is at least 10 ft (3 m) above mean river level.

It can be seen from Fig. 5 that there is an appreciable difference in the relationship of pressure head to depth between the upper and lower flat areas. The important factor in this relationship is the pressure head at bedrock which has a dominating influence on the pore pressures in the clay. The pressure head at R1 is relatively low, due to the configuration of the bedrock which drops sharply a short distance north of R1. At this point, therefore, the water passing along the rock does not need a

[2]G. C. McRostie 1966, personal communication.

TABLE II

Computed stresses

Groundwater condition	Vertical effective stress lb/ft² (kg/cm²) H = 50 ft	(15 m)	H = 100 ft	(30 m)
Hydrostatic groundwater	1880	(.92)	3760	(1.83)
Lower flat area	2380	(1.16)	4950	(2.42)
Upper flat area	3380	(1.65)	7320	(3.58)

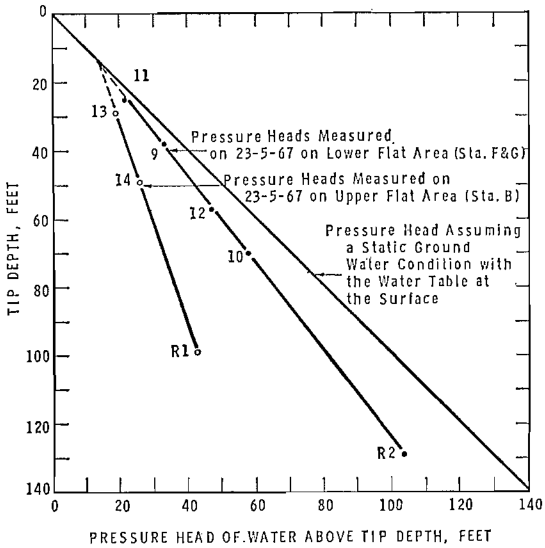

FIG. 5. Relationship of groundwater pressures and depth below surface.

large pressure head as the pressure gradient necessary for movement is provided by the loss in elevation head in the water-bearing strata.

In many practical engineering situations, *in situ* effective stresses are calculated assuming hydrostatic conditions below the water table. Equation [1] is a general relationship for the vertical effective stress σ' at a point X in a uniform soil mass, which is H ft below the ground surface.

[1] $$\sigma' = H\gamma_{\text{wet}} - h\gamma_{\text{w}},$$

γ_{wet} = Unit weight of soil,

γ_{w} = Unit weight of water,

h = Pressure head of water at X.

If the groundwater table is at the surface and hydrostatic conditions prevail then $h = H$. If, however, as is the case described in this report, there is a downward flow of water, the hydrostatic condition does not exist and $h < H$. Figure 5 shows the difference between the general assumption of hydrostatic pressure and the measured pressure conditions. Using values of h obtained from Fig. 5, numerical examples of the effective stresses calculated under the varying conditions are given in Table II for $\gamma_{\text{wet}} = 100$ lb/ft³ (1.60 g/cm³).

The differences obtained in effective stress values are large and increase with depth. At the 100-ft (30 m) depth in the upper flat area the calculated effective stress value is almost twice that obtained assuming hydrostatic conditions. This increase in effective stresses due to downward flow increases the stability of the slope between the upper and lower plains. On the other hand, in making settlement estimations, great difficulty could be encountered. For example, if the preconsolidation of the subsoil is determined from consolidation tests and the *in situ* effective stresses are calculated assuming hydrostatic pore pressures (when in fact a downward drainage condition existed) then the additional loading that could be applied without exceeding the preconsolidation pressure would be overestimated with the possibility of unexpected settlements.

In this case the downward flow of water may have influenced the stress history of the clay in the area. The lower flat area is a terrace of the Ottawa River. Some overburden has therefore been removed by the erosive action of the river; the extent of this erosion has been tentatively estimated by Crawford and Eden (1965) using measured preconsolidation pressures and assuming hydrostatic conditions. It is now believed that part of this calculated overconsolidation may be caused by a downward drainage which may have existed before and during the erosion process.

Conclusions

(1) The river acts as the drainage sink for the area, with subsurface drainage to the river through the upper rock layers.

(2) The porewater pressures in the clay are directly dependent on those in the underlying bedrock.

(3) The downward flow of water in the clay strata can radically affect both the stress history of the material and present-day stress analyses. The possibilities of such a flow should, therefore, be considered during design work.

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