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Groundwater and drawdown in a large earth excavation[1]

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In deep excavations below the water table, the transient and steady state drawdown conditions are often of interest. Particularly in moderately pervious soils, the stability of the slopes, the excavation methods, and equipment are affected by the groundwater response in the excavation. The extent of the drawdown in the area beyond the excavation must also be determined.

For a site under construction by Ontario Hydro, a test excavation to determine the drawdown response in the irregularly stratified soils is described. Analyses of the data by the finite element method and by the theory of aquifer tests are presented. The effect of recharge on the steady state and transient drawdown is discussed.

For the main excavation, approximate methods to determine the rate of drawdown, the quantity of seepage, and the steady state drawdown position with recharge from infiltration or deep seepage are outlined.

Dans les excavations sous la nappe phréatique, la connaissance des conditions transitoires et permanentes de rabattement présente un fréquent intérêt. En particulier, dans les sols faiblement perméables, la stabilité des pentes, les méthodes et les équipements d'excavation dépendent de la réaction de la nappe phréatique dans l'excavation. L'étendue du rabattement dans la zone voisine de l'excavation doit aussi être déterminée.

L'article décrit une excavation expérimentale réalisée sur un site de construction de l'Hydro-Ontario, pour étudier le phénomène de rabattement dans des sols irrégulièrement stratifiés. Les analyses des mesures par la méthode des éléments finis et par la théorie des essais d'aquifère sont présentées. L'effet d'une recharge sur les états de rabattement transitoire et permanent est discuté.

On présente les méthodes approximatives utilisées pour déterminer dans l'excavation réelle, la vitesse de rabattement, le débit d'infiltration et la position permanente de la nappe après rabattement compte tenu d'une recharge par infiltration ou par écoulement profond.

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Introduction

In a large excavation below the water table, the effects of groundwater and seepage are important when considering the stability of the slopes, construction planning, equipment, and methods. Also, major changes in the water table may be caused by the drawdown in a deep, wide excavation. The extent of these changes in the area surrounding the excavation must be determined.

Following the general description of a site under development by Ontario Hydro, the investigation of the groundwater response in a test excavation on the site is discussed and the application of the data derived from the test pit to the study of seepage in the main excavation is presented. Transient and steady state conditions analysed by the finite element method are compared with the results from the theory of aquifer tests.

Description of the Site and the Proposed Excavation

The site is being developed for a nuclear power plant on the north shore of Lake Ontario near Bowmanville about 70 km east of Toronto. The location is part of the area covered in a detailed hydrogeological study by Singer (1974). In Fig. 1 is a plan of the general layout of the major features of the works—the excavation, the cooling water forebay, the powerhouse, and vacuum building.

A cross section generalized from details from boreholes drilled during various stages of investigations of the site is presented in Fig. 2. The overburden can be considered in three main formations: (i) an upper stratum, about 3–10 m thick, of lacustrine sand and silts or silty till; (ii) an interglacial series of fine sand and layered clayey silt up to about 30 m thick; and (iii) a lower basal silty till about 3 m thick.

At the shoreline the upper till or lacustrine soil as well as the interglacial fine sands and clayey silt are visible in steep bluffs that rise 10–20 m above the mean lake level, elev. 74.7 m (Canadian geodetic

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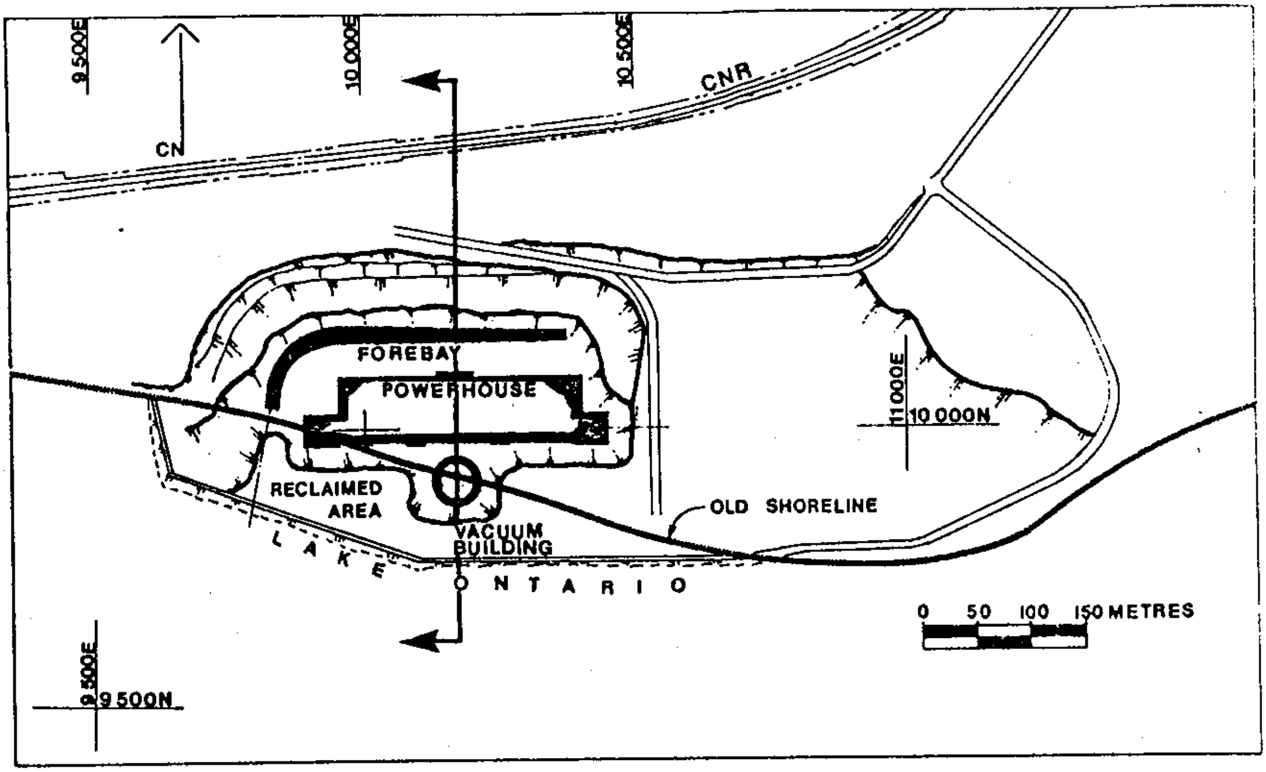


FIG. 1. Darlington Generating Station—site layout.

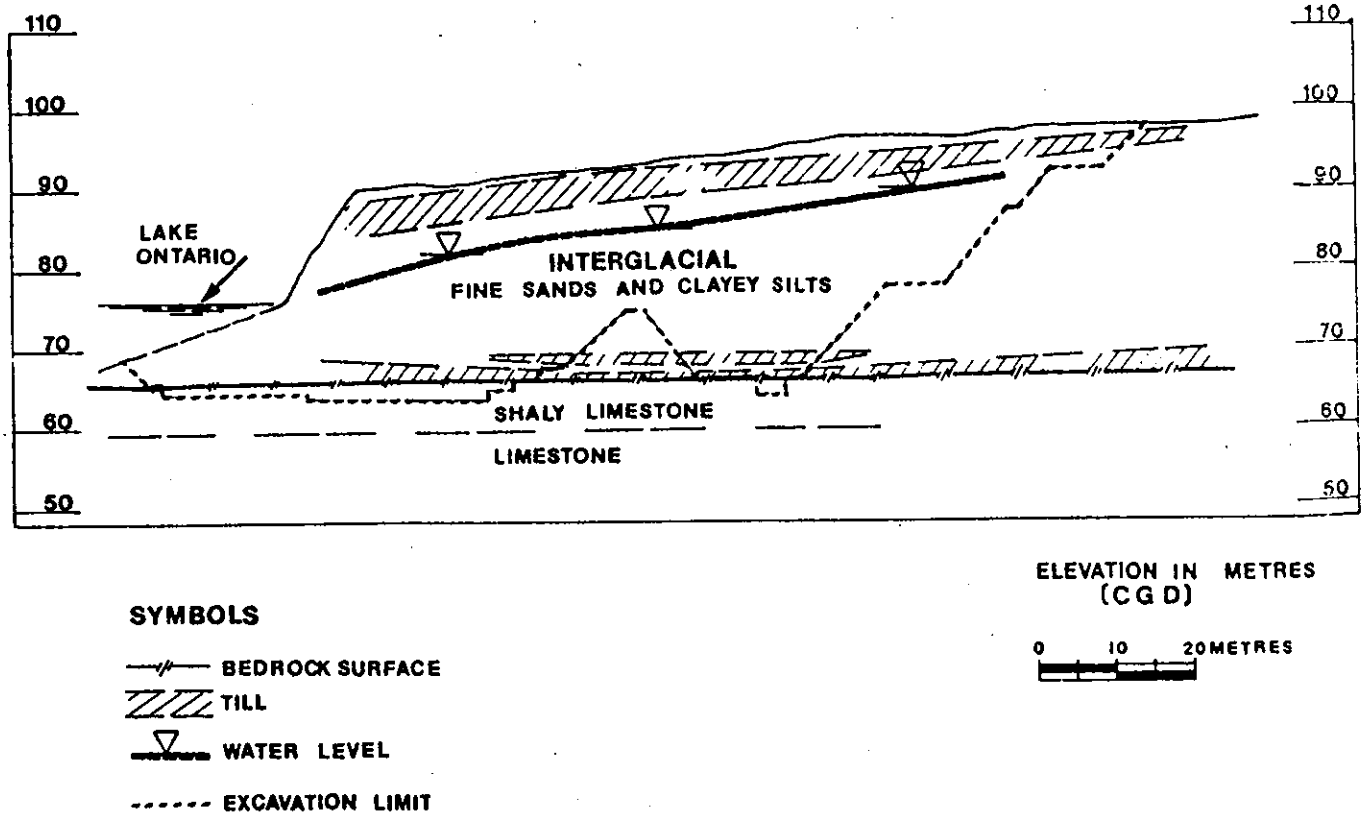


FIG. 2. Darlington Generating Station—section.

datum). From the shore cliff the ground surface rises gradually to the north to about elev. 106 m near the excavation limits and to higher values further north. The bedrock surface is at around elev. 66 m and is fairly flat. The bedrock consists mainly of Ordovician limestone of the Trenton Group. The upper 7–9 m of

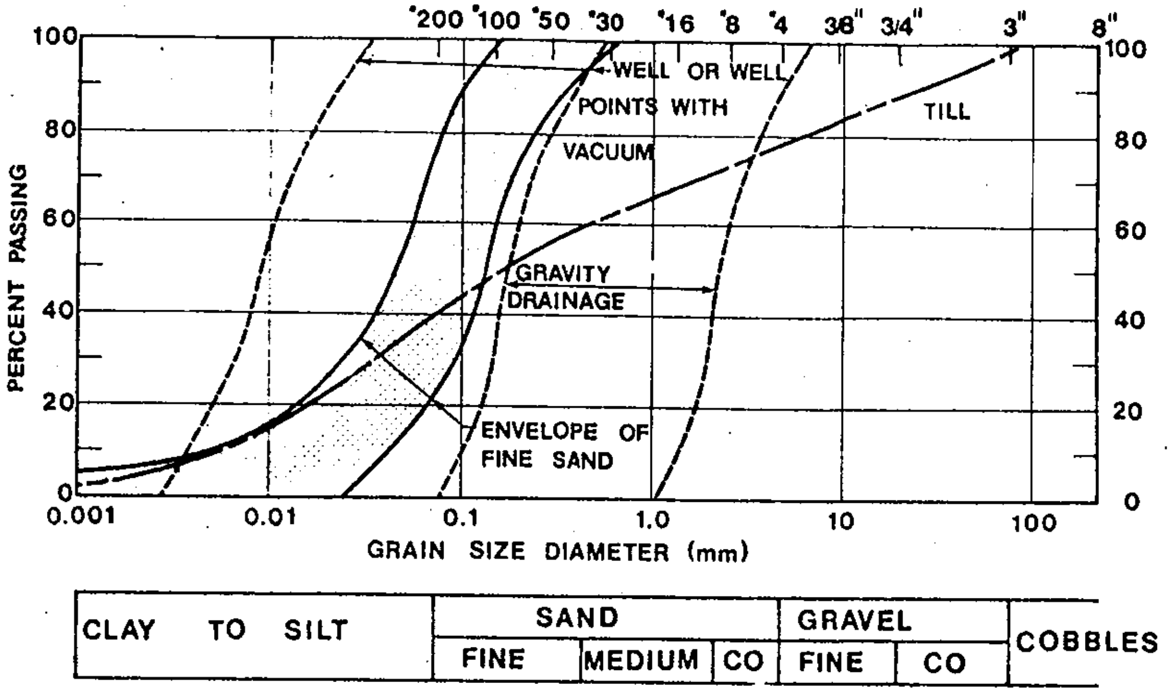


FIG. 3. Typical grading curves.

bedrock consists generally of dark brown to black thin to medium bedded fossiliferous shaly limestone. Up to about 0.6 m of weathered rock occurs on the surface at some locations; the remainder of the rock mass is generally free of any major open joints or fissures.

In Fig. 3 are shown typical grain-size curves for the upper till and the fine sand strata. Based on the exceedingly high standard penetration resistance in the soils, with *N* values exceeding 100, the deposits are in a very dense or hard state. From laboratory and borehole tests, the mean coefficient of permeability of the upper and lower tills and interglacial clayey silts is in the range of 10^{-7} to 10^{-8} m/s and that of the fine sand is of the order of 10^{-5} to 10^{-6} m/s. Where exposed in natural slopes at the shoreline or in excavations, the soil deposits do not exhibit any secondary structures or more pervious zones that may have any influence on the average permeability. From the appearance of the fine sand strata, it is assumed that the deposit is homogeneous and isotropic with respect to permeability. Water pressure tests in the boreholes in the bedrock indicate the rock mass to be generally watertight with a permeability coefficient probably less than 10^{-9} m/s.

The groundwater conditions were observed over several years in observation wells with tips in the bedrock and also with tips in the overburden. The water levels in the overburden are, in some locations, under excess hydrostatic pressure and produce artesian flow. Also, the water levels measured in the overburden are about 4 m higher than the water level in bedrock. The elevation of the water table in the overburden near the north limits of the site excavation is at around elev. 90 m. The water table surface slopes to the south at a gradient of about 2% and, at local exit points, the water flows from the face of the bluffs from exposed fine sand strata.

The proposed excavation for site grading and foundations covers an area of about 800 m by 300 m. The soil excavation will be taken to bedrock at around elev. 66 m and will remain open for about 5 years for construction of the foundations after which the site will be backfilled to around elev. 78 m. The forebay for the cooling water system is located near the toe of the north slope of the excavation about 250 m from the shoreline.

At the north side of the site excavation, the temporary drawdown of the water table will be about 24 m but the permanent changes at the slope location will be about 12 m. Because of the pervious fine sands in the interglacial complex, the control of groundwater and seepage was considered a major problem in the preliminary engineering stage. For example, Fig. 3 shows the fine sand grading falling in the limits in which gravity drainage is too slow and special treatment of well point systems may be required. A field drawdown test was considered useful for evaluation of the groundwater response during and after the excavation for site grading and foundations.

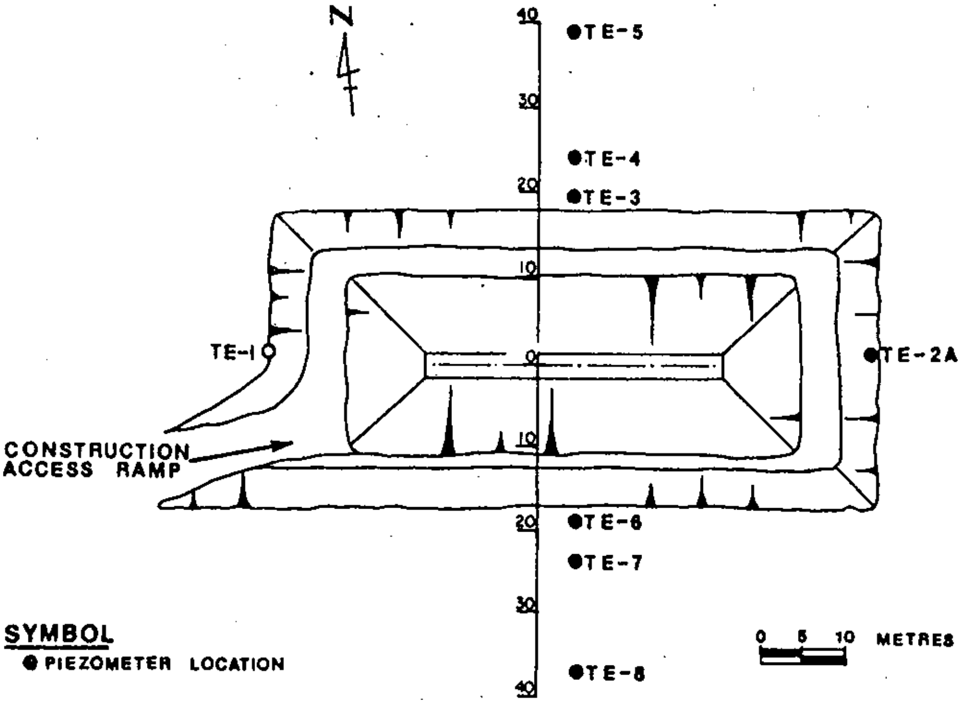


FIG. 4. Darlington Generating Station—plan of test excavation.

The Test Excavation

Based on data from the borehole investigations, preliminary studies of the groundwater control requirements indicated major problems with installation and testing of deep wells or well points. As an alternative a test excavation could provide data for drawdown evaluation and access for *in situ* loading tests for undisturbed block samples for laboratory work. Direct observation of the slopes and of the performance of construction equipment particularly for disposal of the soil could also be made.

A location for the test excavation was selected where the surficial soils and upper till were about 3 m thick and minimum stripping was required to reach the interglacial deposits. Also, at this location borehole piezometers showed the initial groundwater table to be close to the ground surface with artesian conditions thus providing a suitable background for data for studying the drawdown during the test excavation.

Observations of the water levels in boreholes from earlier site investigations were continued and rainfall measurements were started in July 1976. Also, eight boreholes, TE1 to TE8, were drilled to confirm the stratigraphy and to install additional piezometers. Mende-type piezometer tips were used. The 450 mm long porous plastic cylinder filled with fine gravel was attached to a 38 mm diameter plastic riser pipe and placed in the borehole. The annular space between the riser and the wall of the borehole was filled with clean medium to coarse sand except for the upper 3 m section, which was sealed with a bentonite–cement grout up to the ground surface. In addition to the piezometers immediately around the test excavation, other boreholes drilled on the site for previous investigations were available for water level observations.

At the location selected, the ground surface was at around elev. 88 m, gently sloping to the east. The test excavation, as shown in Fig. 4, was about 80 m long by 36 m wide. The depth ranged between 7 and 9 m. Side slopes in the upper 3 m and above elev. 84.8 m were at 1 to 1 and, below, in the interglacial strata, they were about 2 to 1. The bottom of the pit was at around elev. 80.3 m and about 3 m wide.

The test excavation was started on 3 August 1976 and was carried out in two main stages (shown in Fig. 5) with a backhoe and dump trucks as follows: (i) stripping of topsoil, surficial material and till, and the upper level of the interglacial sands to elev. 84.8 m; and (ii) the excavation of the interglacial fine sands and clayey silt below elev. 84.8 m to around elev. 80.3 m.

After stripping of the topsoil was completed on 6 August, ditches for surface runoff control were completed on the north, south, and east sides of the area and a sump was excavated at the northeast corner with the bottom at around elev. 84 m on 10 August. Several small boils were observed in the interglacial fine sand in the bottom of the sump. The seepage ranged between 4 and 9 L/min.

The excavation of the upper stage from 16 to

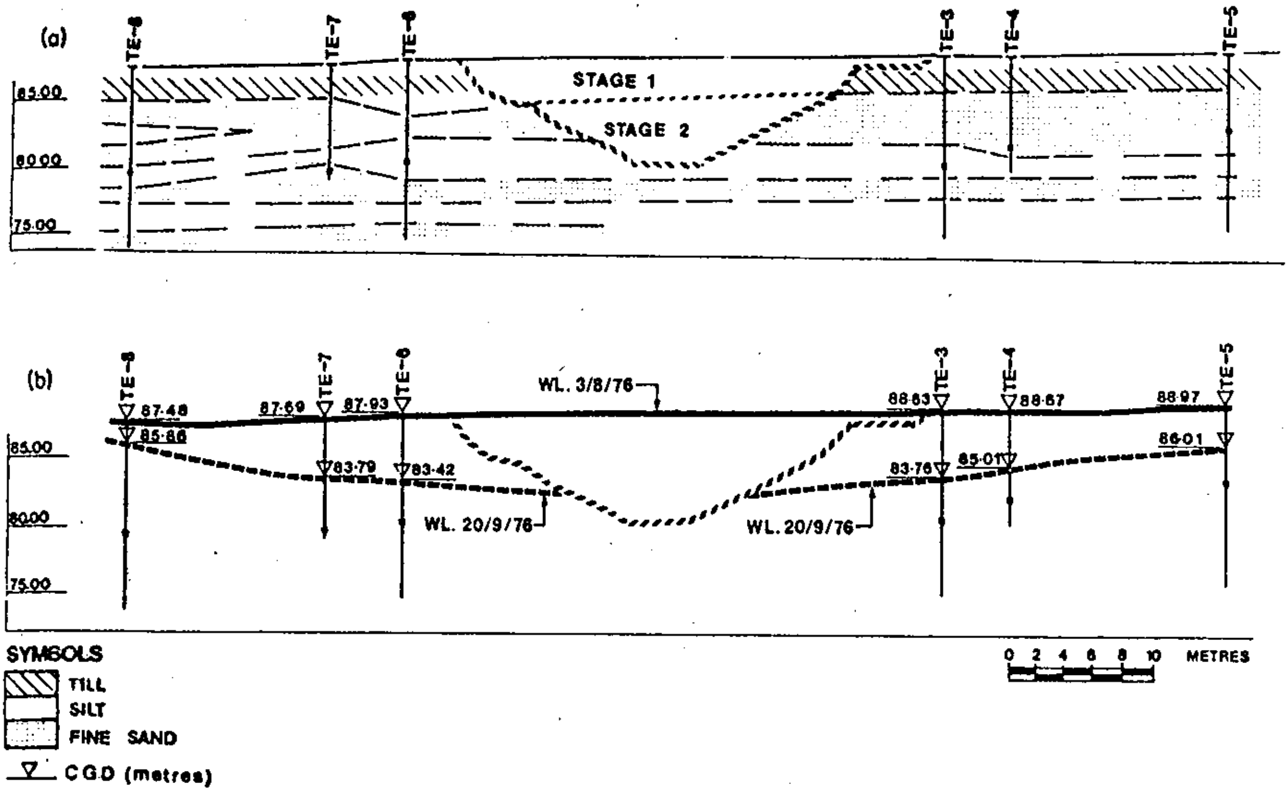


FIG. 5. (*a*) Section—soil stratigraphy. (*b*) Groundwater levels.

23 August was carried out working from west to east. Numerous temporary small boils were observed in the bottom and pumping developed under truck traffic. The ramp and parts of the bottom were lined with crushed stone or rockfill about 0.5 m thick to provide a trafficable surface.

Stage 2 excavation was started on 24 August. Working from the east end, the soil, mainly the interglacial fine sand and clayey silt, was excavated to final width and depth with side slopes of about 2 to 1 and to the bottom with a sump at around elev. 80 m. Boils of moderate size developed in the bottom soon after completion of the sump.

Fine sand and silt from seepage erosion and piping caused frequent silting of the pumps, the flow from which now ranged between 27 and 36 L/min. Groundwater levels around the excavation continued to drop while digging progressed towards the west.

Immediately after excavation, the slope appeared generally stable. Within a few hours, a surface of seepage developed and erosion of the fine sand began. A short section of the slope was lined with crusher run stone, 50 mm maximum size and 0.3 m thick, to observe the effects of seepage erosion control by this means. This treatment was successful in stopping the erosion of the soil while allowing seepage from the slope.

The excavation was completed on 10 September. Observations of the groundwater levels and pumping were continued to 1 October when pumping was stopped.

Results and Analysis

Water levels in the observation wells near the test excavation were observed at least twice each day, and at other locations daily readings were taken. Typical drawdown–time values are plotted in Figs. 6 and 7 in which rainfall measurements for the period are also shown.

During the first stage of the excavation, August 3–24, a small seepage of about 6 L/min was pumped. An initial drawdown was noted. The drawdown increased rapidly after the second stage was started and progressed with the excavation generally reaching an equilibrium state soon after the excavation was completed. The lag in response in piezometers TE3 and TE8 shown in Figs. 6 and 7 is probably caused by local variations in the soil permeability and by irregular stratification.

The drawdown cone, which developed around the excavation, was of a slightly asymmetrical elliptical section. Extrapolation of the plot of the drawdown against the log of the distance from the excavation indicated the average radius of influence of the excavation to be about 200 m.

During the second stage, August 24 to September

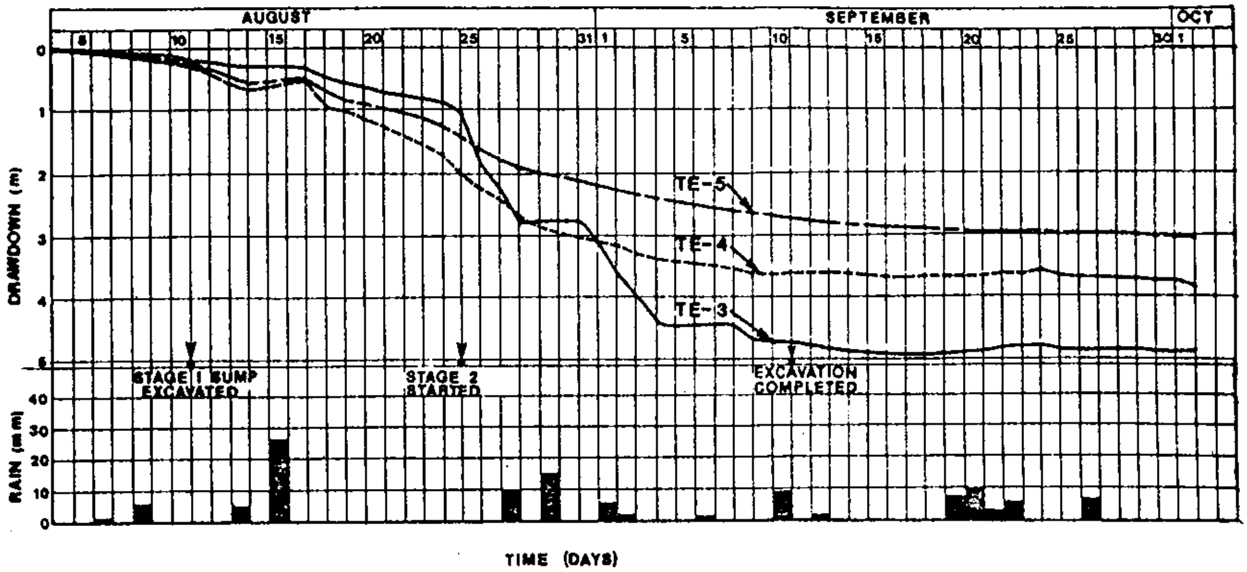


FIG. 6. Drawdown–time curves.

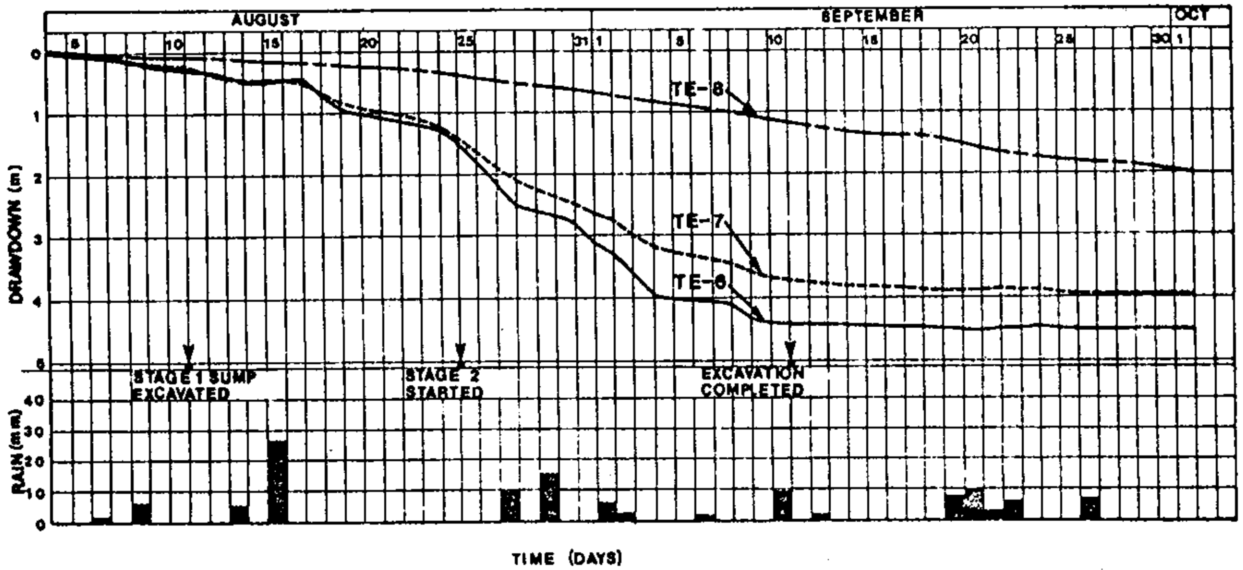


FIG. 7. Drawdown–time curves.

10, the volume of pumping from the bottom of the excavation increased to between 27 and 36 L/min with an average of 32 L/min. Because of the increase in drawdown and flow during the second stage of the test excavation, the effect of the first stage is considered negligible after the excavation is deepened. The zero drawdown for the second stage is thus taken as the water level position on August 24. Drawdown against time values are shown in Fig. 8.

Finite Element Approach

To aid in the interpretation of these field observations, and to verify the hydromechanical characteristics of the multilayer system, a finite element simulation of seepage and drawdown around the test excavation was carried out. The soil stratigraphy in Fig. 5 was idealized to give a finite element grid as shown in Fig. 9. The finite element formulation and solution of steady state seepage problems are well documented (e.g., Zienkiewicz 1971; Desai and Abel 1972). The approach adopted for simulating transient drawdown around the test excavation was similar to that described in France *et al.* (1971) and Zienkiewicz (1971). With this approach the drawdown process is divided into an arbitrary number of stages, the state of flow within each being considered as momentarily steady. The free surface is still one of zero excess pressure, but is no longer a streamline. The velocity vectors are thus computed for each element, along with the nodal heads and quantities of flow (Lee

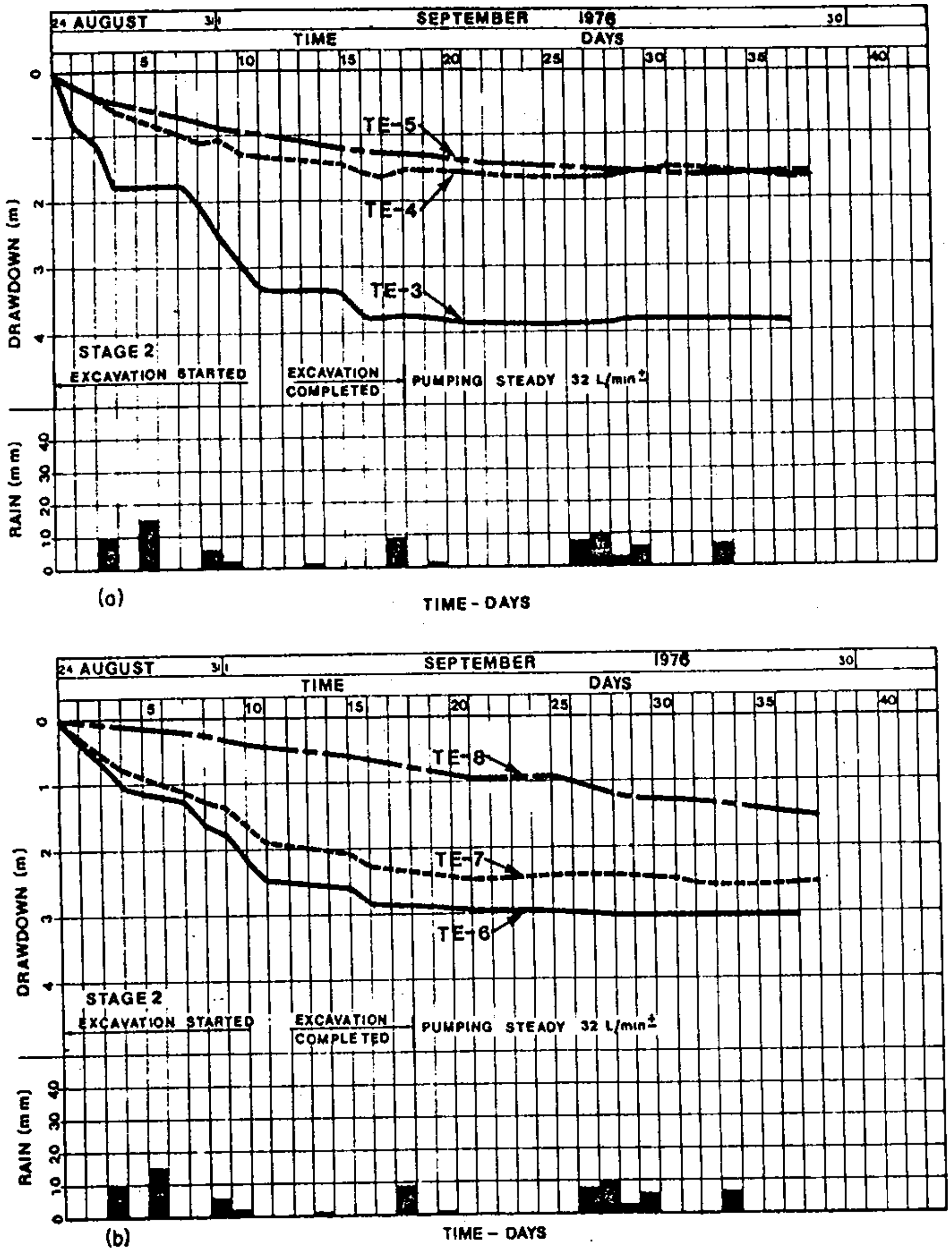


FIG. 8. Stage 2 drawdown.

1976, 1977). At each stage, the normal true velocity of the free surface (V_{nt}), which is the rate of drawdown, can be determined as

$$[1] \quad V_{\mathrm{nt}} = V_{\mathrm{ns}}/S_{\mathrm{y}}$$

where V_{ns} is the component of Darcy velocity normal to the free surface, and S_{y} is the specific yield or drainable porosity.

Beginning with its pre-excavation position, the location of the free surface at the end of a predetermined time interval can be deduced from [1] above. The finite element grid is then revised to account for the reduction in the region of flow as a result of drawdown. Figure 9 shows the finite element grids used in simulating the two stages of the test excavation.

The following observations and deductions were made from the results of this finite element analysis.

(1) In a multilayer system consisting of alternating layers of fine sand and clayey silt or dense till, seepage occurs primarily in the relatively pervious fine sand strata. As illustrated in Fig. 10 *a* and *b*, the direction of flow is predominantly horizontal in the fine sand

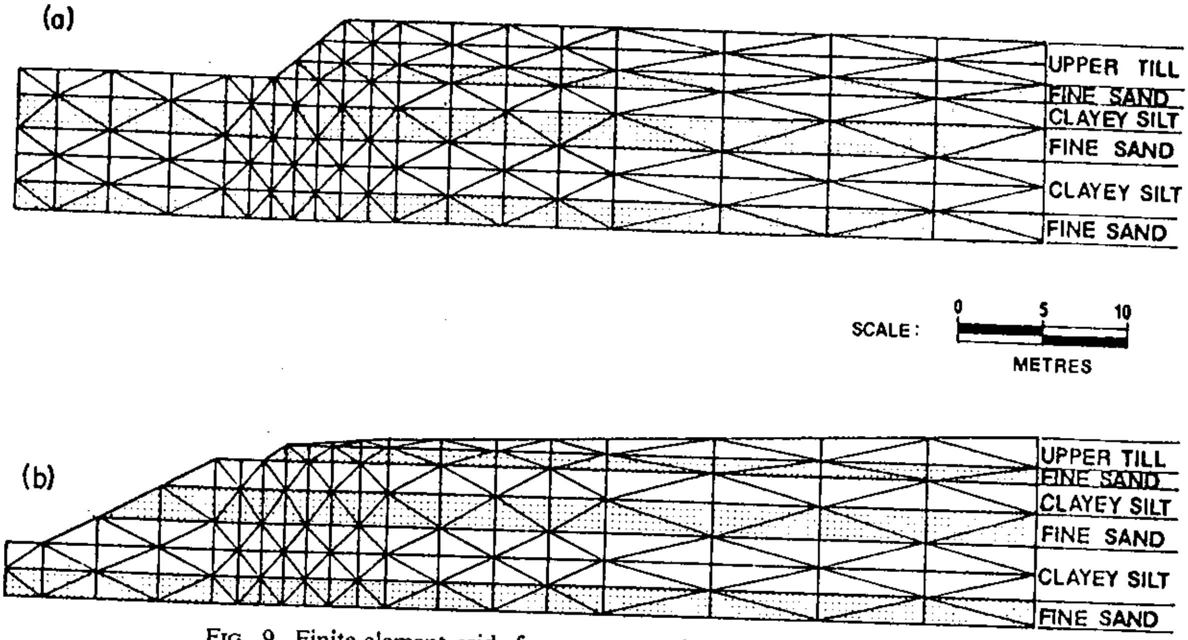
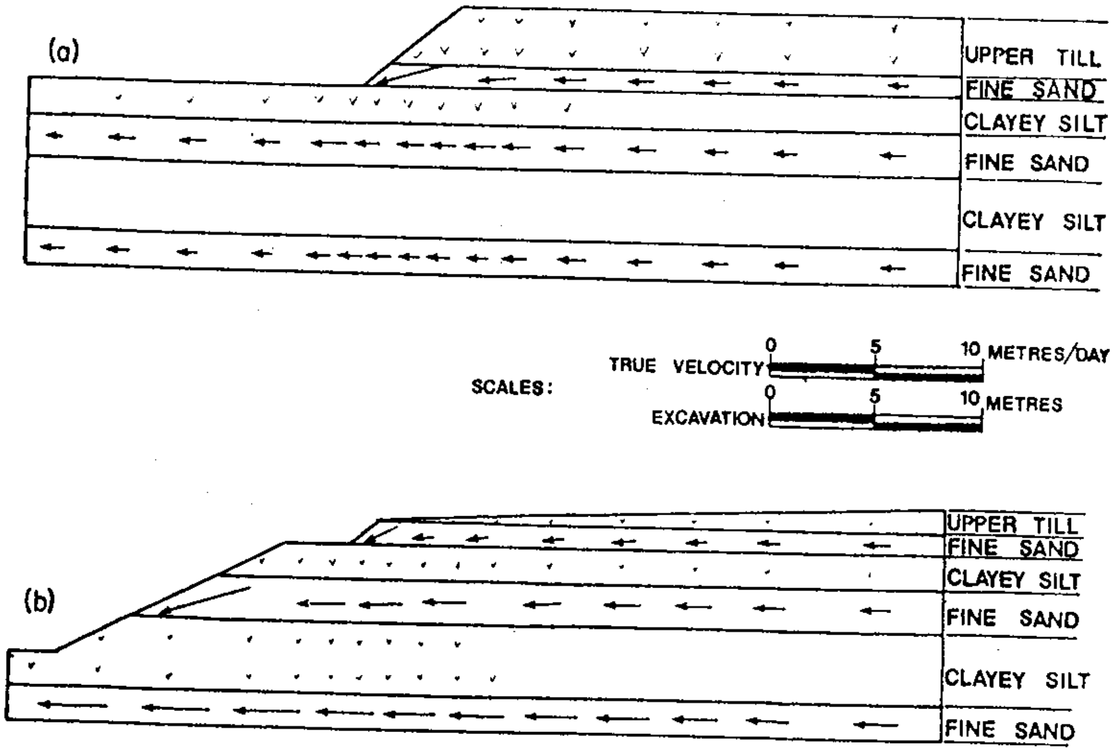


FIG. 9. Finite element grids for seepage study: (*a*) stage 1; (*b*) stage 2.



NOTE: True velocities in impervious layers generally too small to be drawn to scale. Only directions of flow are indicated

FIG. 10. Distribution of true velocity: (*a*) stage 1; (*b*) stage 2.

strata and vertically downward in the less pervious strata around the excavation. These flow directions indicate the shortest possible routes for water particles to reduce their hydraulic potentials in these two types of materials. As a result, there is a "stepped" appearance in the equipotential lines obtained (Fig. 11).

(2) A permeability coefficient of the order of $2\text{–}4 \times 10^{-6}$ m/s can be backfigured for the fine sand strata, based on the quantities of flow pumped out of the excavation and earlier stated.

(3) Based on the rates of drawdown depicted in Figs. 6 and 7, a range of 0.014–0.02 can be backfigured for the specific yield of the fine sand strata.

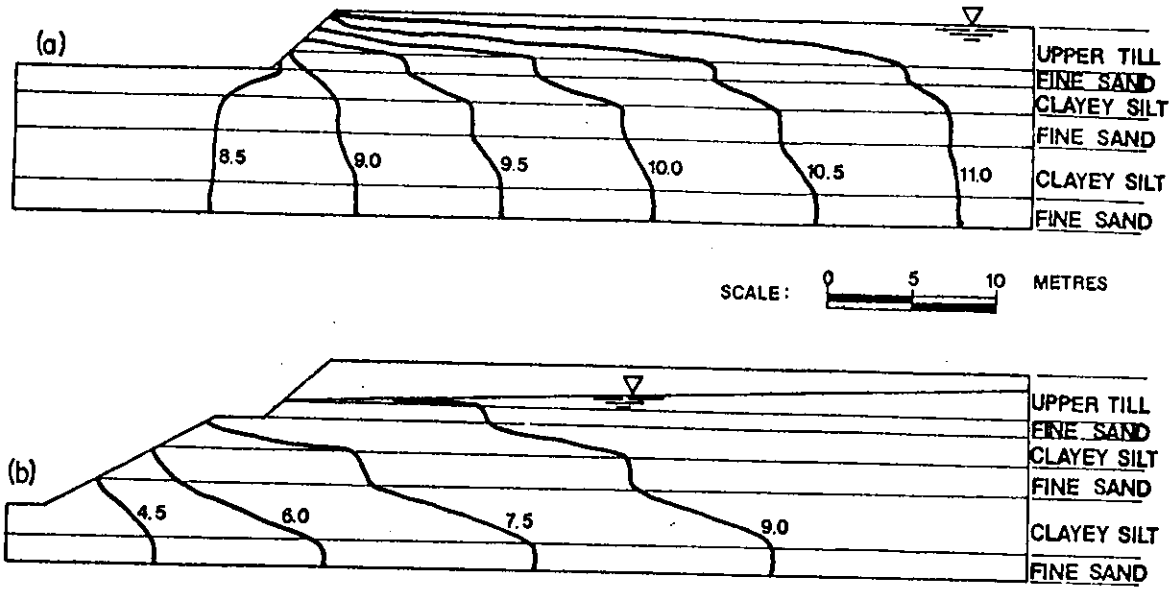


FIG. 11. Equipotentials (in metres) at ends of two stages of excavation: (*a*) stage 1; (*b*) stage 2.

giving an average of the order of 0.015. The corresponding values for the upper till and the clayey silt deposits are between 0.002 and 0.007. There is a tendency for the specific yield to increase towards the excavation surface, reflecting a further reduction in the confinement effect immediately around the excavation.

Analytical Approach

Because of the variability of the soil strata and the detailed input required by the finite element approach for each situation to be studied, approximate analytical methods by drawdown theory were also used. Because the test excavation was carried out in one lift during the second stage for removal of the interglacial soil, the test excavation is envisaged as a large diameter well. After completion of the excavation, the three-dimensional effect on the flow becomes negligible and the steady state seepage may be considered as a two-dimensional system.

The analyses for the quantity of seepage for the completed excavation and for the transient drawdown during the second stage are presented below.

Equivalent Well Radius

The excavation may be considered as an equivalent well with a radius given by (Mansur and Kaufman 1962):

$$[2] \quad r_w = (2/\pi)(LB)^{1/2}$$

where L is the length and B is the width of the excavation. Thus, for average values of $L = 64$ m and $B = 18$ m, $r_w = 23.5$ m.

Active Zone

The active zone (Sheikov and Zel-brandt 1964) is the thickness of soil below the bottom of the excavation through which seepage occurs. This thickness is equal to the depth of the bottom below the initial water table or the depth from the bottom to an impervious boundary, whichever is less.

For the final depth of the test excavation, the interglacial strata extend to more than 7.5 m below the bottom and thus the active zone, equal to 7.5 m, extends to around elev. 77 m. Between the ground surface and elev. 77 m, the total thickness (M) of the pervious fine sands is about 5 m as shown in Fig. 5.

Quantity of Seepage: Completed Test Excavation

The drawdown–time plots in Figs. 6 and 7 show an equilibrium or steady state position soon after completion of the excavation. It is inferred from this that a recharge by both vertical and horizontal flow occurs and causes the drawdown to stabilize. The equilibrium may also be caused, to some extent, by partial penetration effects both in the excavation and in the observation wells (Hantush 1962).

Assuming confined flow conditions, the quantity of seepage (Q) may be calculated by (Peterson 1957):

$$[3] \quad Q = \frac{2kM(H - h_w)}{\ln \frac{r'}{r_w} - \frac{n}{2}}$$

The radius of influence (r') from the drawdown–distance plot is estimated to be 200 m. The thickness (M) of the pervious strata is 5 m. The drawdown ($H - h_w$) at the excavation is the depth from the initial water table to the bottom of the excavation, about 7.5 m. The value of n is the proportion of total discharge replenished by vertical seepage and is equal to unity for a fully recharging system or to zero where there is no recharge. Thus for $k = 3 \times 10^{-6}$ m/s and $r_w = 23.5$ m, Q is estimated to be 26 L/min, which compares well with the measured value of about 32 L/min.

Drawdown Rate: Second Stage

In considering the transient drawdown, it is assumed that the effects of partial penetration of the excavation and of the observation wells into the aquifers are negligible and that vertical recharge from precipitation and upward seepage from underlying strata in the active zone are the main cause of the equilibrium or steady state conditions. The analysis of this stage is then based on the theory of drawdown when pumping from leaky artesian aquifers (Hantush 1956).

At steady state, the maximum drawdown (S_m) in a fully penetrating well is given by:

$$[4] \quad S_m = \frac{Q}{2\pi T} K_0\left(\frac{r}{\beta}\right)$$

where Q is the discharge from the well; T is the transmissibility $= kM$ of the pervious deposits of thickness M and permeability k; $K_0(r/\beta)$ is the modified Bessel function of the second kind and of zero order of the value of r/β (given in tables by Hantush 1956); r is the distance from the well; and β is the leakage factor, which reflects the performance of the less pervious or semi-confining strata.

In Table 1, the maximum drawdown for the second stage is used to determine the value of $K_0(r/\beta)$ and r/β.

The values of r/β are relatively high and indicate that the steady state drawdown is caused not only by recharge but also by the effects of partial penetration. The above values of r/β are thus used in studying the transient drawdown conditions.

The transient drawdown (s) for a leaky aquifer and fully penetrating conditions is given by:

$$[5] \quad s = \frac{Q}{4\pi T} W\left(u, \frac{r}{\beta}\right)$$

where $u = r^2S/4Tt$ (for radial flow conditions to a well); S = storage coefficient; T = transmissibility $= kM$; t = time elapsed from start of pumping; and $W(u, r/\beta)$ = well function for leaky systems from tables (Hantush 1956).

The storage coefficient is an important parameter in transient drawdown studies. For unconfined flow situations, the storage coefficient is equal to the drainable porosity or specific yield particularly at later stages of drawdown. In confined or artesian flow, the storage coefficient is a relatively low value.

TABLE 1. Stage 2: steady state drawdown—estimate of r

Observation well No.	r (m)	Max. drawdown (m)	$K^0(r/\beta)$	r/β
TE3	20	3.8	0.68	0.68
TE6	20	3.0	0.53	0.9
TE4	25	1.7	0.31	1.2
TE7	25	2.5	0.44	0.97
TE5	40	1.5	0.27	1.3
TE8	40	1.3	0.23	1.5

The analysis by the finite element method was used to back-calculate the specific yield for which an average value of 0.015 was obtained. The recharge during the stage 2 drawdown suggests a semi-confined or leaky aquifer situation and the storage coefficient is therefore assumed equal to the specific yield or 0.015.

The theoretical drawdown is then calculated from [5] with r/β values from Table 1 and $W(u, r/\beta)$ from tabulated values (Hantush 1956). The observed drawdown is divided by the term $Q/4\pi T$ to give field values of $W(u, r/\beta)$, which are plotted with the theoretical values in Figs. 12 and 13. There is a fair agreement between the observed and calculated values if the distortion in observed values caused by about 25 mm of rain during the drawdown period is taken into account.

From the foregoing analysis, it is concluded that the use of the soil parameters developed in the various phases of the investigations and the appropriate drawdown theory will yield estimates that are reasonable. The flow conditions may be considered as a semi-confined or unconfined system and the storage coefficient of 0.015 appears to account for the leaky effect of the less pervious strata adequately.

Drawdown and Seepage in the Excavation

Analytical methods for determining the steady state and transient drawdown in a reservoir or channel bank and in subsurface drainage systems give approximate results sufficiently accurate for practical purposes (Ferris *et al.* 1962; Moody 1962; Jenab *et al.* 1969; Streltsova 1975). The boundary conditions created by an assumed instantaneous large excavation may be considered similar to those caused by drawdown in a reservoir or channel bank, solutions for which may thus be adopted. These methods do not usually include the effects of recharge to the water table from infiltration or deep seepage, which result in an equilibrium condition or steady state.

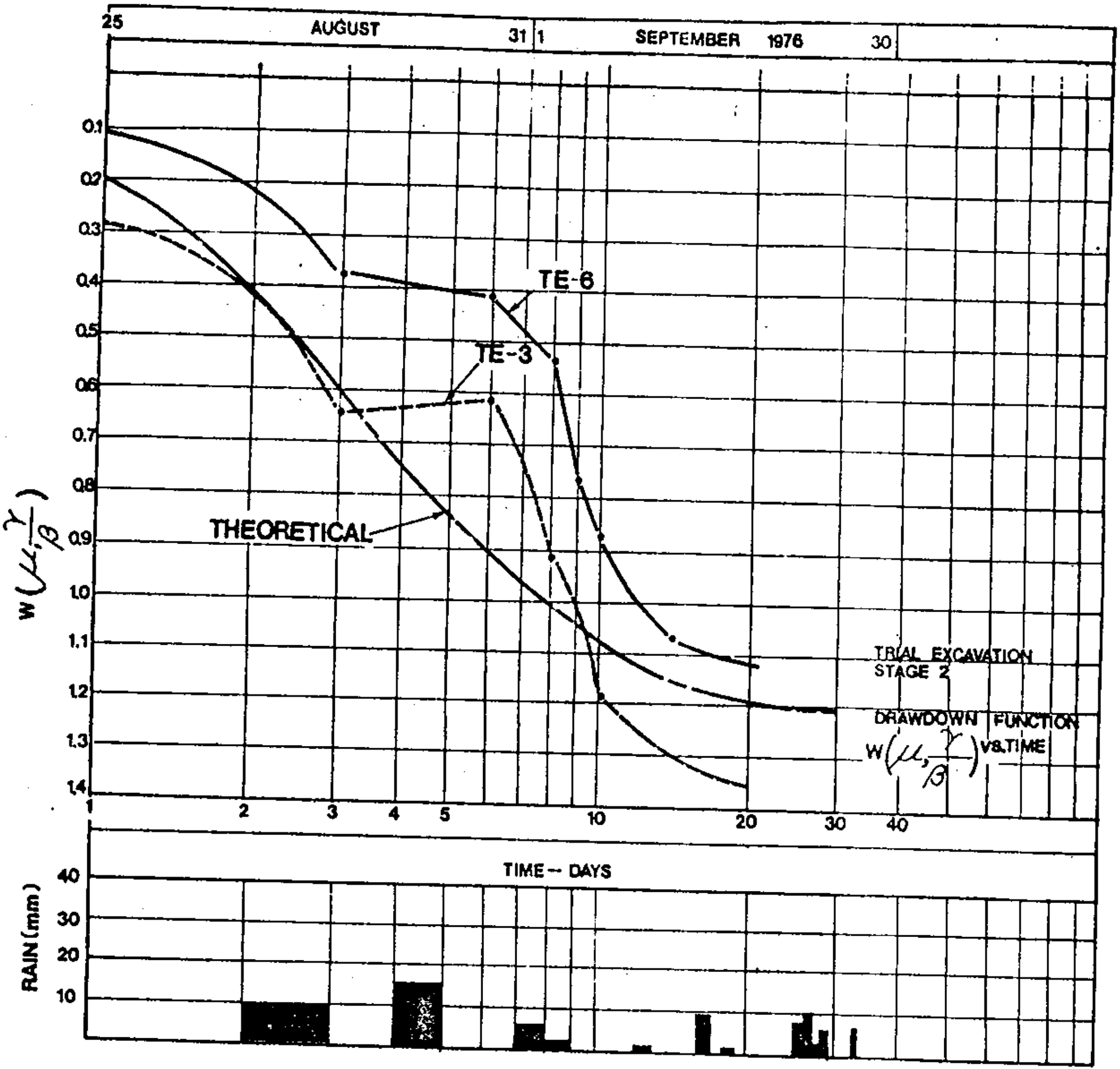


FIG. 12. Well function – time curves.

An approximate solution is attempted herein to estimate the extent of the groundwater changes with recharge from precipitation or deep seepage.

Drawdown without Recharge

When there is no recharge, the drawdown (s) at any time (t) and distance (x) from the slope is given by:

$$[6] \quad s = S_0 D(u)$$

$$[7] \quad u^2 = x^2 S/4Tt$$

(for flow to a line sink such as a channel, drain, or long slope) where S_0 is the drawdown at the slope face; $D(u)$ is a function of u (values for which are tabulated in Ferris *et al.* 1962); and S and T are the coefficients of storage and transmissibility of the pervious strata. The coefficient of storage may be considered equal to the drainable porosity for sufficiently large values of t, particularly if the conditions of flow are unconfined (Streltsova 1975).

Also, the quantity of seepage (Q) from the slope is given by:

$$[8] \quad Q = S_0(ST/\pi t)^{1/2}$$

The quantity of seepage thus decreases in time.

In using this analysis for the site, the effect of stratification or the thickness of the less pervious silt and clay strata in the site excavation is ignored. It is assumed that these less pervious soils allow sufficient leakage or are discontinuous enough to permit drainage into the fine sand strata. This assumption is justified by the response of the water levels in the boreholes around the test excavation.

The transmissibility $T = kM$ depends on the total

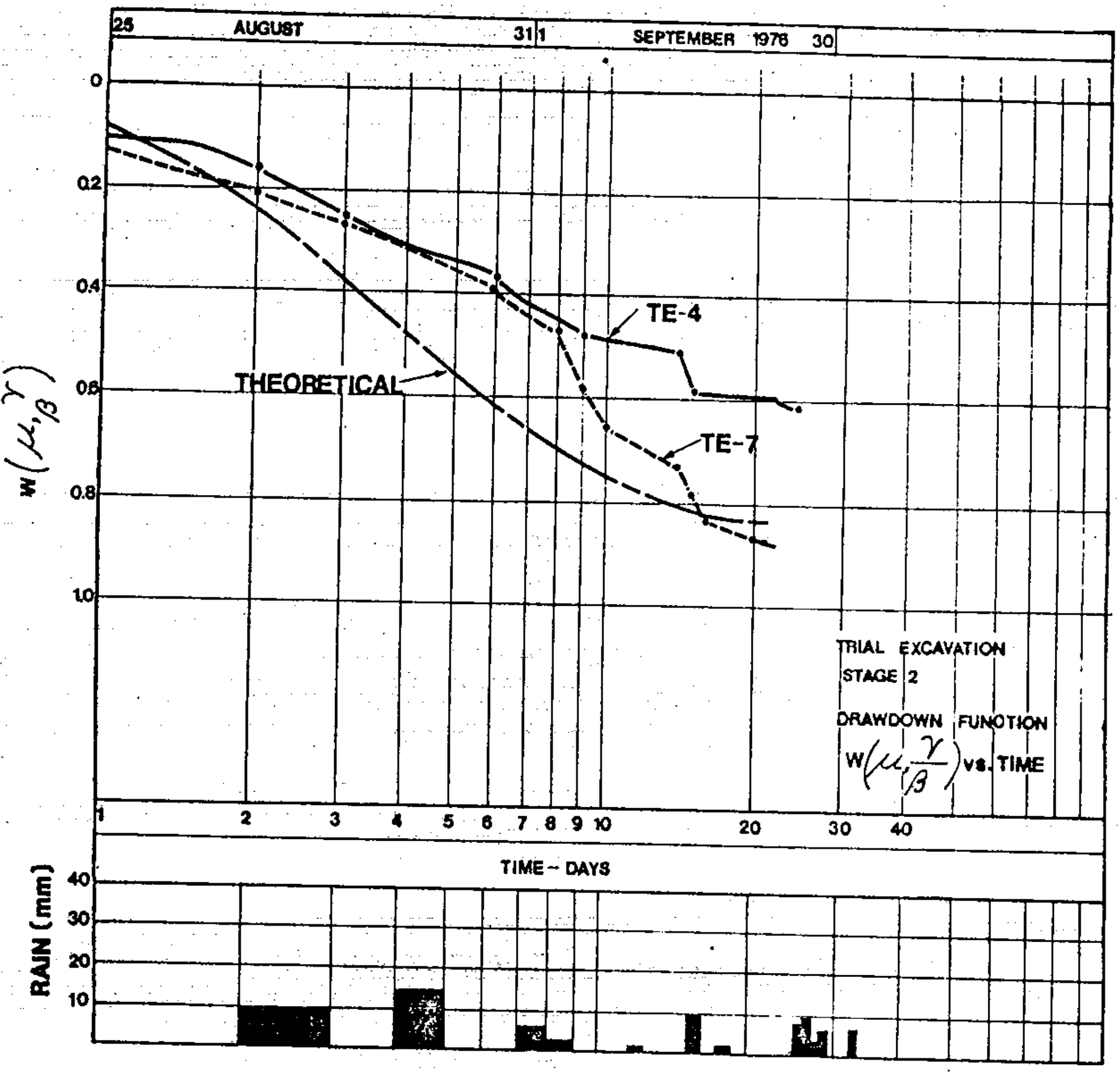


FIG. 13. Well function – time curves.

thickness (M) of the sand strata, which ranges between 3 and 10 m with an average value of about 6 m. The drainable porosity, S, is assumed to be 0.015 for the fine sand deposits as back-calculated from the test excavation.

Near the location of the main excavation slopes, the existing water table ranges between elev. 90 and 95.4 m. For general site grade at around elev. 78 m, the drawdown (S_0) at the slope is about the same as the depth of excavation below the water table, 12–17.4 m. For additional excavation to bedrock at elev. 66 m, S_0 will be increased to 24–29 m.

In Fig. 14 are shown typical values of the drawdown ratio $D(u) = s/S_0$ plotted against the distance (x) from the slope on a logarithmic scale for values of t = 32, 128, and 512 days and for M equal to 7.6 m. In Fig. 15, for various values of M and for a distance (x) equal to 15 m, the typical variation in $D(u)$ with time (days) is shown.

From these figures it is seen that the drawdown ratio increases with time and with values of M. After a period of about 100 days or more, the difference in drawdown ratio for the range of values of M is small. For earlier times the difference in $D(u)$ is generally appreciable. Values of $D(u)$ for intermediate times (t) or thickness (M) can be obtained by interpolation or from other typical curves.

Because no recharge is considered in the above analysis, the drawdown increases indefinitely with time at any distance. However, recharge may occur from vertical infiltration from precipitation or from deep seepage and a steady state condition is eventually attained.

Recharge Rate and Infiltration

In a few locations, the water levels in boreholes on

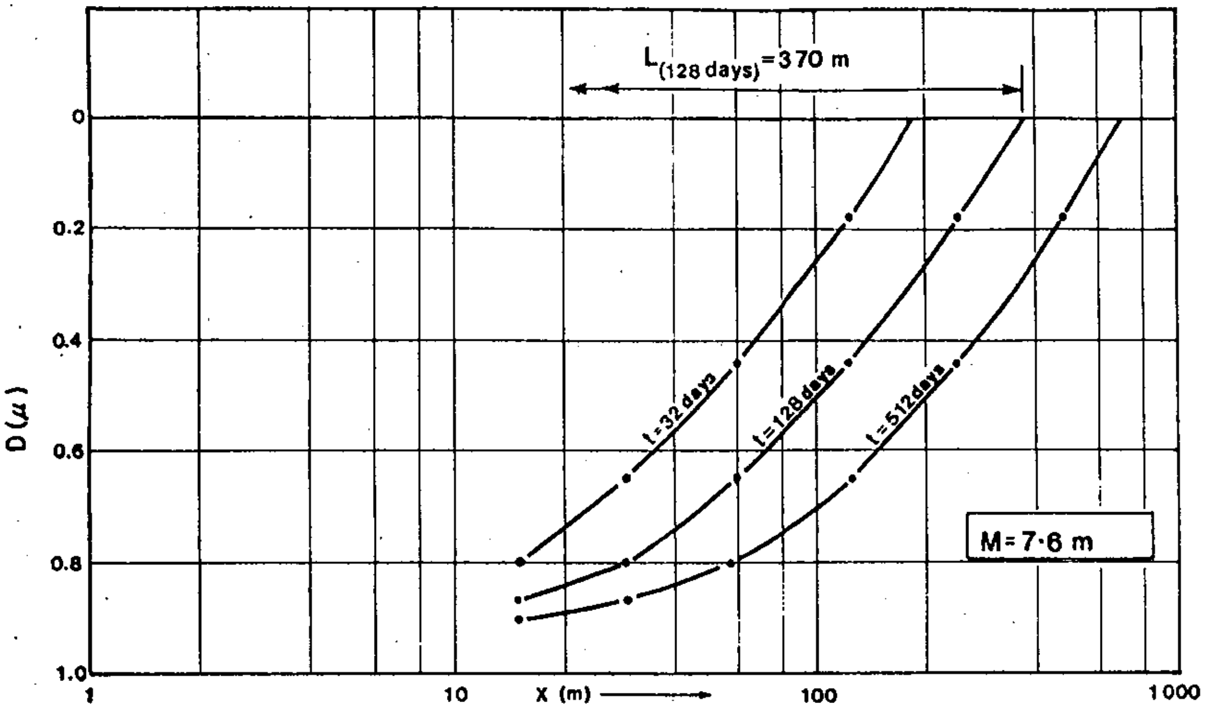


FIG. 14. Drawdown ratio ($D(u)$) vs. distance (x) from the slope.

the site in summer are about 6 m higher than those observed in winter. Generally, however, the fluctuation is much smaller, about 0.6 m. These changes reflect the effect of recharge from precipitation, which is estimated below.

(i) Singer (1974) considers that the flow region extends to about 15 m below the bedrock surface. In the analysis discussed herein, the effect of the seepage in the bedrock is neglected because of the low permeability of the rock mass. Singer estimates from base flow analyses that the recharge rate (R) for the Bowmanville Creek area (immediately east of the site) is about 9.55 in./year or 6.7×10^{-4} m/day. For an annual precipitation of about 825 mm, the recharge from infiltration is about 30%. A value of 23% is estimated for the area further east of the site.

(ii) Borehole water levels are plotted in Fig. 16 against the distance (x) from the shoreline on the logarithmic scale. An approximate position for the water table may be obtained from the drawdown form for a subsurface drain or a channel given by Maasland (1959) or by Ferris *et al.* (1962):

[9] $h = Rx(2a - x)/2kM$

where h is the water surface height above lake level; R is the recharge rate; k and M are the permeability and thickness of the draining strata; and a is the distance to a position where the water table is assumed horizontal. For a range of a = 300–610, R is estimated to vary between 1.5×10^{-4} and 4.5×10^{-4} m/day.

(iii) For an average radius of influence in the test excavation of about 200 m and a pumping rate of about 46 m^3/day, the recharge rate is estimated to be 3.6×10^{-4} m/day. From the above ranges a value of $R = 4.5 \times 10^{-4}$ m/day is conservatively assumed for the site.

Effects of Recharge

The effects of recharge are assumed to be negligible when the quantity of seepage (Q) from a slope is relatively high. The quantity decreases in time and a steady state position is reached, when, for a unit width of slope:

[10] $RL = Q$

where L is the recharge section or the distance from the slope to the zero drawdown point for the elapsed time.

Both RL/S_0 and Q/S_0 are straight lines when plotted on log–log paper against time (t) and this provides a convenient means of estimating the time to steady state (t_e). The procedure is shown in Fig. 17 for M = 7.6 m and S_0 between 6 and 24 m.

For the assumed value of R, the time for steady state (t_e) for values of M between 3 and 10 m are shown in Fig. 18. The following points may be noted.

1. The time to steady state is proportional to the drawdown at the slope.

2. The time to steady state is maximum for M equal to about 7.6 m and is slightly lower for smaller and larger values of M. No explanation could be found for this variation.

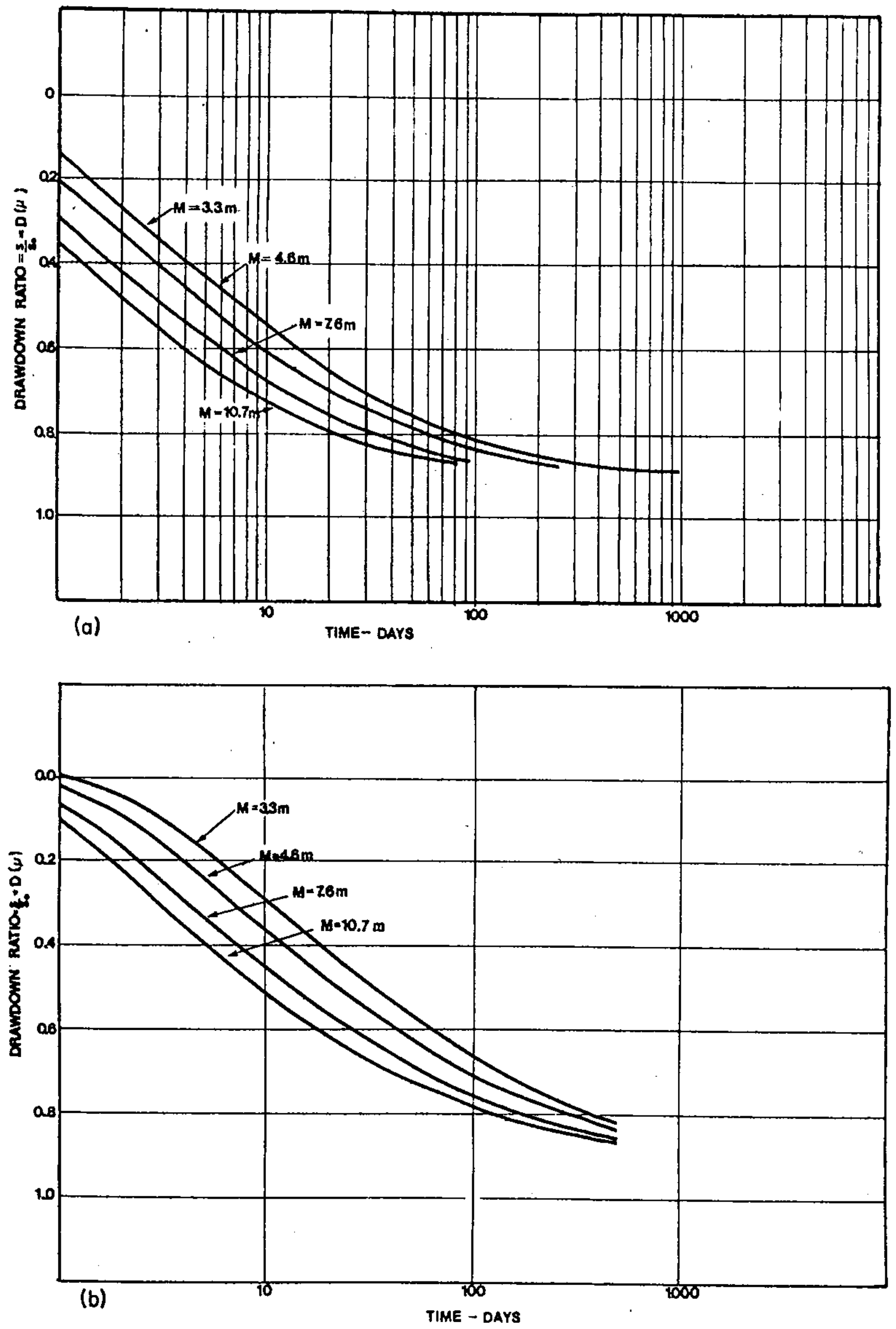


FIG. 15. (*a*) Drawdown ratio (s/S_0) for x = 15 m vs. time. (*b*) Drawdown ratio (s/S_0) for x = 30 m vs. time.

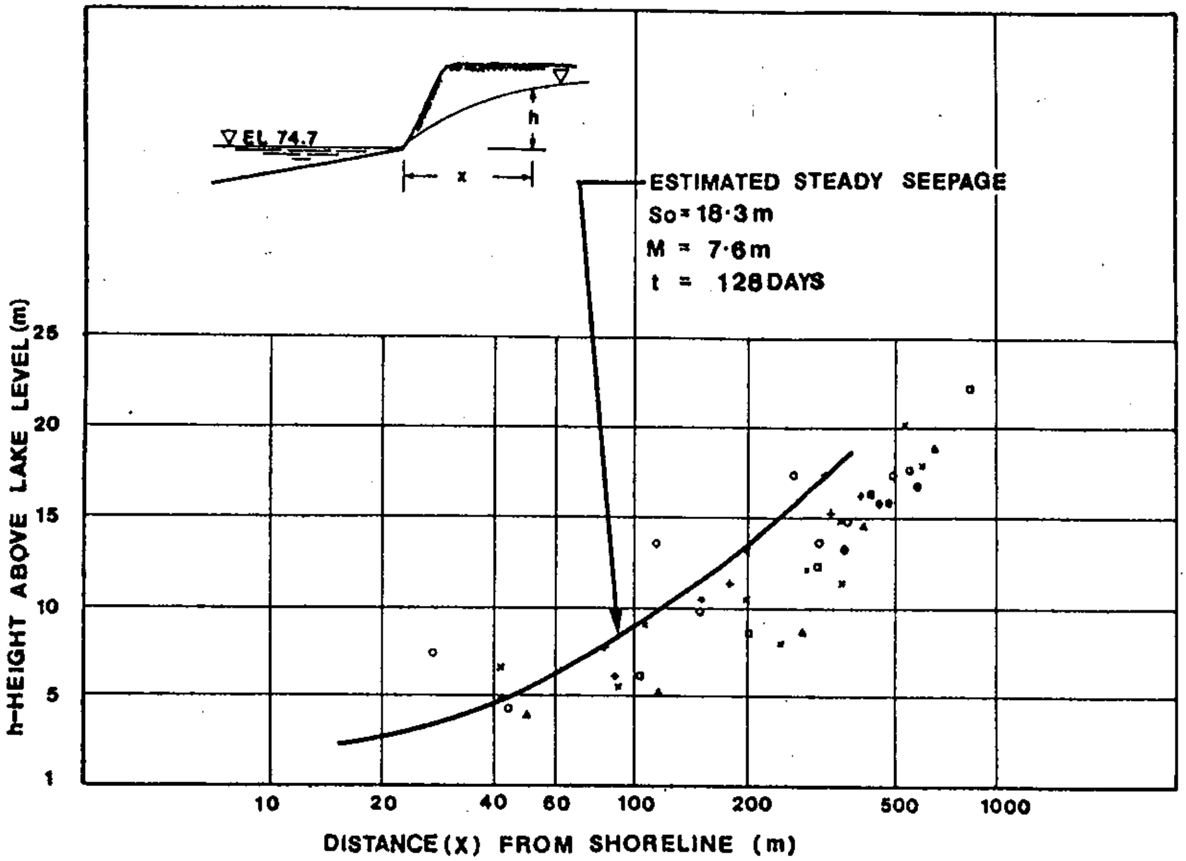


FIG. 16. Borehole water levels vs. distance from shoreline.

3. The time for steady state for $S_0 = 18$ m and $M = 7.6$ m is about 120 days. These values of S_0 and M are approximately the average values for the existing water table and shoreline conditions and, from [8], the quantity of seepage is estimated at 164 L per metre width per day.

Singer (1974) estimates the range to be 2.11–7.74 L/s per kilometre (0.12–0.44 cfs per mile) length of shoreline or 180–670 L per metre width per day, with the higher values in summer and fall.

4. Based on the time to steady state (t_e) of about 120 days, the water table position is estimated by means of [3] and [4] for $S_0 = 18$ m and $M = 7.6$ m. The estimated position shown in Fig. 16 is generally close to the upper limit of the measured borehole water levels.

5. Also shown in Fig. 18 are the data for the test excavation for which $S_0 = 7.6$ m and $M = 5$ m. The estimated number of days to equilibrium is 42, which may be compared with the period of about 38 days during which the excavation was in progress.

The transient drawdown rate in the slope given by [6] does not include the effect of recharge from vertical flow. The equivalent average recharge rate of 4.5×10^{-4} m/day is smaller than the average permeability of the soil strata and the associated water table recession rate is assumed to be rapid enough not to have any effect, except locally or after a heavy rainfall, on the drawdown. This assumption is justified by the small general variation in borehole water levels over several seasons and the fluctuations around the test excavation.

Polubarinova-Kochina (1962) gives an alternative approximate solution for drawdown with constant vertical recharge for a horizontal drain. For unconfined flow:

$$[11] \quad L_t = H\sqrt{\frac{k\,(1 - e^{-\alpha t})}{2R}}$$

where

$$[12] \quad \alpha = 6R/SH$$

and where R = recharge rate; S = drainable porosity; L_t = distance from the slope to zero drawdown after time t; and H = the drawdown at the slope.

The length L_t tends to a limit L_F for large values of t given by

$$[13] \quad L_F = H(k/2R)^{1/2}$$

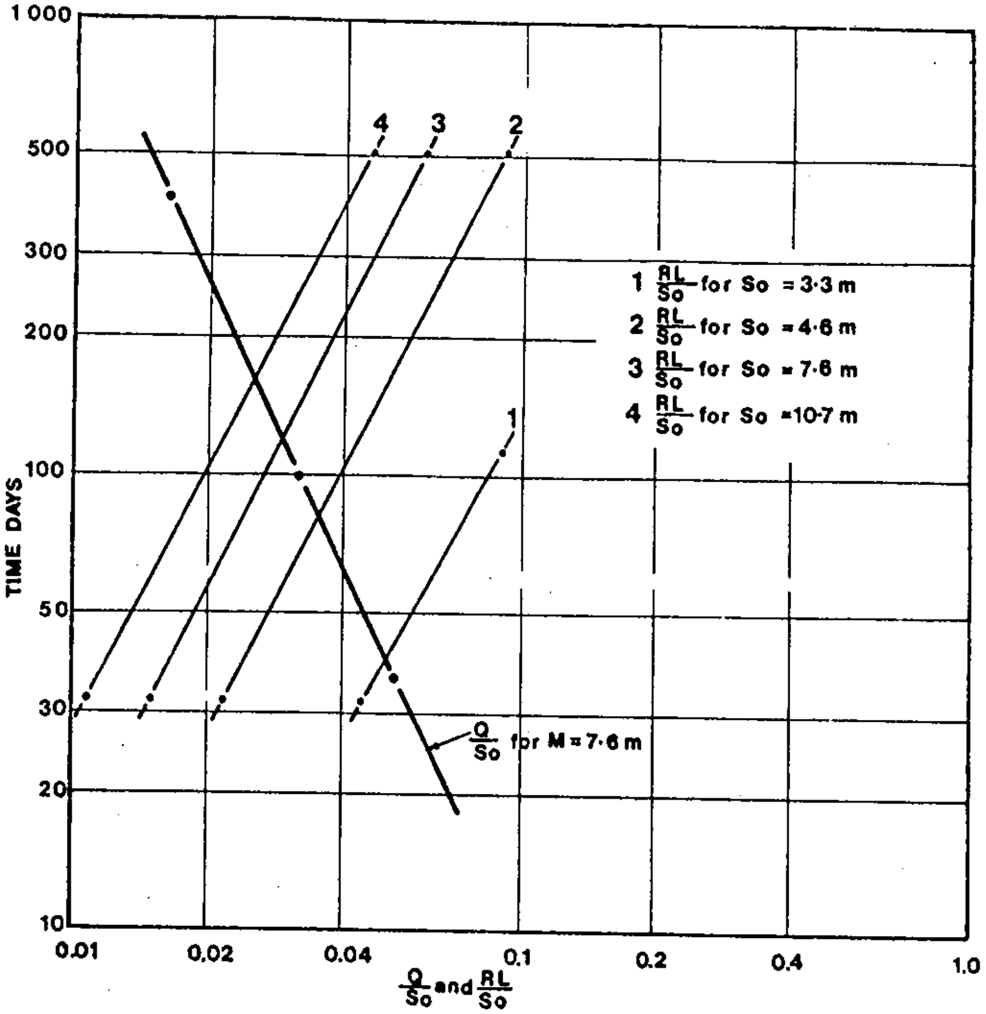


FIG. 17. Determination of steady state time (t_e).

For a practical value of L_t of about 90% of L_F, it may be shown that the term αt in [11] and [12] will give a linear relationship between t and H. This result confirms the linear relationship given in Fig. 18, particularly for a value of $L_t = 0.85L_F$.

Also, for the existing water table, L_F calculated by [13] for $H = 18.3$ m is about 310 m, which may be compared with about 370 m obtained from Fig. 14 for a drawdown period of about 128 days.

Exit Conditions

At the surface of the slope, the exit point and surface of seepage may be determined by [14] due to Kashef (1969):

$$[14] \quad \frac{kD}{Q} = \frac{\cot B}{1 - \frac{2}{3}\sin^2 B}$$

where D is the height of the exit point above the toe of the slope; Q is the quantity of flow at steady state; and B is the slope angle in degrees.

The presence of an impervious stratum within the exit cannot be ignored since it will result in an actual exit point higher than that given by [14], which, in turn, will influence the position of the drawdown surface.

Application to Design and Construction

The methods described above were used in determining the drawdown and seepage in the main excavation to establish the background for stability analysis, construction planning, slope protection, and drainage requirements.

In considering the groundwater problems in the main works, the test excavation has shown that the groundwater table may be lowered by deep ditches in advance of excavation. Erosion of the silts and fine sand by seepage is a major problem, which will necessitate special attention to the surface protection treatment. The fine sand, even after drainage, still retains a large percentage of moisture and the trafficability of the material will be generally poor.

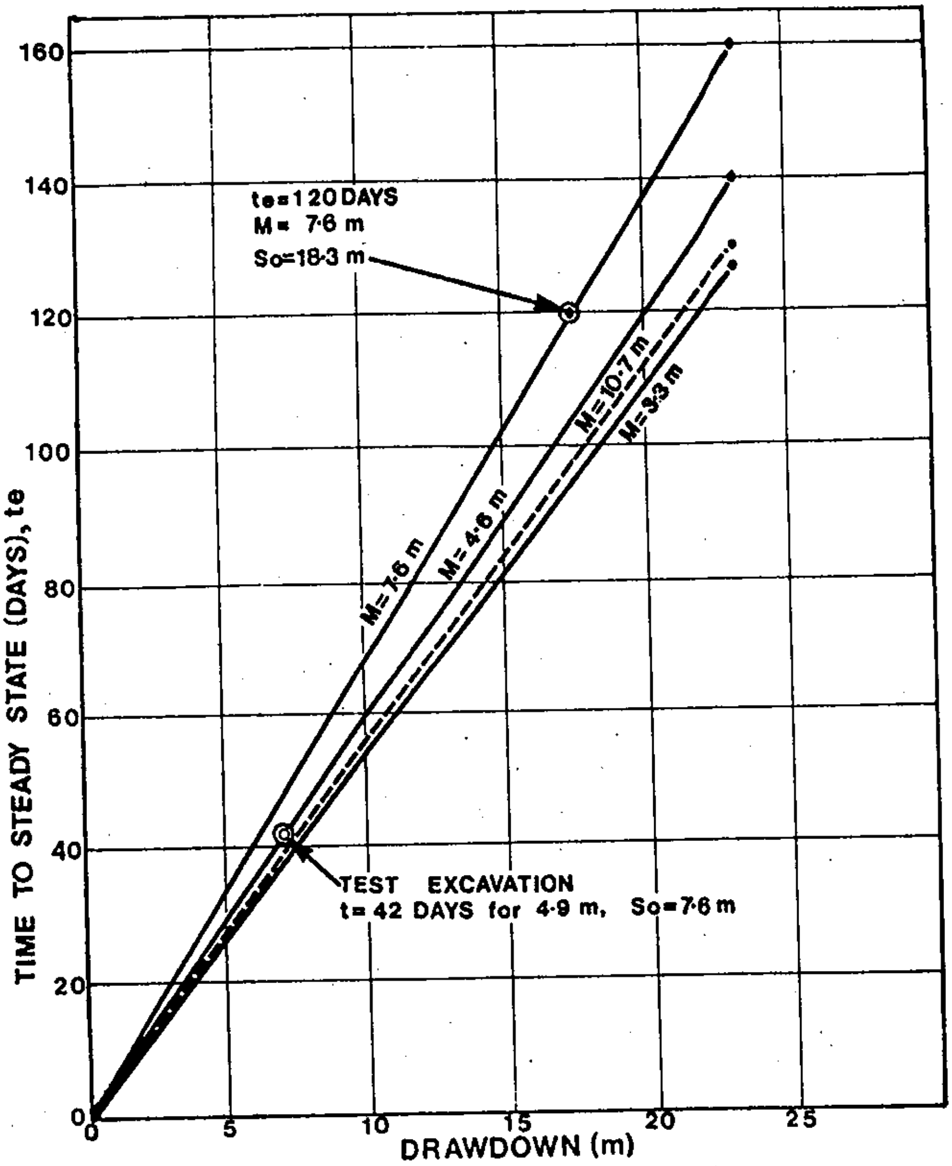


FIG. 18. Time to steady state vs. drawdown at slope.

The steady state condition attained by the water surface is dependent on recharge from infiltration and deep seepage. The drawdown caused by the excavation is unlikely to extend more than about 400 m from the slope when the excavation below the water table is about 22 m deep.

Summary and Conclusions

A test excavation has proved useful for determining the drawdown and seepage characteristics in semi-confined fine sand deposits in conditions in which conventional pumped wells may be of limited usefulness.

The analyses of the data by the finite element method and by the theory of aquifer tests produce approximate results, which are generally compatible with the observations. The effect of recharge from vertical flow must be taken into account in evaluating the data and in determining the effects of excavation below the water table. The approximate methods of analyses are outlined to indicate the main parameters to be considered. In transient drawdown studies the specific yield or drainable porosity is an important parameter, which should be determined by field tests. In the fine sand strata, a specific yield of 0.015 appears to give acceptable results for the conditions at the site considered.

At the time of writing, the main excavation is in progress above the water table. Deep ditches at select locations are planned as part of the groundwater control as the excavation continues. It is intended to observe the groundwater response and to compare the results with those obtained from the analyses presented.

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