Investigation of the water table in a tidal beach: final report

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FINAL REPORT

INVESTIGATION OF THE WATER TABLE IN A TIDAL BEACH

Harrison, Boon, Fang, Fausak, and Wang

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GLOUCESTER POINT, VIRGINIA 23062

OCTOBER 1971
FINAL REPORT

"INVESTIGATION OF THE WATER TABLE IN A TIDAL BEACH"

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Principal Investigator

I. Instrumentation for Measurement of Water Table Fluctuations
   by John D. Boon, III, and W. Harrison

II. The Beach Water Table as a Response Variable of the System
    by L. E. Fausak

III. Changes in Foreshore Sand Volume: Role of Fluctuations in Water Table and Ocean Still Water Level
     by W. Harrison

IV. One-dimensional Finite Element Analysis of the Groundwater Flow
    by W. Harrison, C. S. Fang, and S. N. Wang

V. Two-dimensional Finite Element Analysis of the Groundwater Flow
    by C. S. Fang, S. N. Wang, and W. Harrison

SPECIAL SCIENTIFIC REPORT NO. 60

Virginia Institute of Marine Science
Gloucester Point, Virginia 23062

Dr. William J. Hargis, Jr.
Director

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October 1, 1971

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INSTRUMENTATION FOR MEASUREMENT OF BEACH WATER-TABLE FLUCTUATIONS

John D. Boon, III, and W. Harrison

ABSTRACT

A suitable system for acquisition of water-level data from the water table of a sandy ocean beach consists of 1) a 10.16-cm (4-in.) I.D. slotted PVC well pipe with non-slotted upper section, housing a float and float wire, 2) a 3.8-cm (1.5-in.) O.D. galvanized steel pipe, housing a counterweight and counterweight line, and 3) a float-gage system consisting of a differential pulley connected to a 10-turn, 5 K-ohm precision potentiometer, both mounted on a covered wooden platform at the top of the pipes some 1 m above the beach surface. A low-voltage power source connects to the potentiometer which gives an output in the 0-100 mv range, corresponding to the elevation of the water table [one turn = 30.48 cm (1.00 ft.) or 10 mv]. The response of such a float-pulley system is considered accurate to ±3.0 mm, and it exhibits a high degree of precision. Details of fabrication and installation of the monitoring system are given together with selected portions of an analog plot of output from 11 to 13 wells that operated continuously for 31 days.

INTRODUCTION

The water table in a sandy beach at Virginia Beach, Virginia, was expected to rise and fall from place to place in response to a variety of factors, including local oceanic tides, storm-wave flooding, rainfall, and fluctuations in atmospheric pressure. Thus, it was necessary that a water-level monitoring system be capable of covering a range of more than 2 m, to handle storm-flooding and storm-surge conditions in addition to measuring tidal fluctuations (in the maximum range of 104 cm). A compact, quick-response system was needed, one incorporating a well pipe that could pass fluid freely without undue "lag" effects or dampening of small-scale fluctuations. Because no equipment was available commercially that could satisfy all of the special requirements, it was necessary to develop the system described herein. The system is comprised of the following major components: an electromechanical device consisting of a differential pulley and a precision potentiometer,
a slotted PVC pipe, a counterweighted float, and steel counterweight pipe. The final system configuration was based on suggestions by Mr. George Smoot of the U. S. Geological Survey, Water Resources Division.

SYSTEM COMPONENTS

PVC Well Pipe For Float

A 3-m section of slotted, 10.16-cm (4-in.) I.D., PVC well pipe, joined to a 1.0-m section of solid PVC pipe, formed the well for the float. The pipe slot size was #18, which restricted sand-size material from entering the walls of the pipe, yet permitted water to enter and exit freely. The solid PVC section was intended for the above-ground support of the gage mechanism where it also served to prevent wind and wave runup from entering the well. The PVC pipe was obtained from Gator Sales, Inc., Box 15020, Baton Rouge, La., 70815.

Steel Well Pipe For Counterweight

A 4-m section of 3.8-cm (1.5-in.) galvanized steel pipe was jetted into the beach so that its side touched the side of the PVC well pipe. In this configuration the steel pipe could house a counterweight for the float-pulley system astride the top of the PVC pipe. The tops of both pipes were squared off level with one another.

Differential Pulley and Precision Potentiometer

A plexiglas (acrylic, type G) pulley, consisting of two drums of differing diameters, was machined. The larger drum accommodated a 0.397-mm (1/64-in.) stainless steel stranded wire within a helical, v-shaped groove (Fig. 1). The smaller drum, which had a smooth surface, accommodated a light monofilament line. This light line was wound in a direction opposite to the stainless steel wire of the large drum. The pulley had a stainless-steel shaft mounted on ball-bearing supports attached to an aluminum frame. The pulley shaft was coupled to a 10-turn 5 K-ohm precision potentiometer, which itself had a ball-bearing shaft. The potentiometer was a Spectrol Model 800 with a linear tolerance of ±0.025%, manufactured by Spectrol Electronics Division of the Carrier Corporation, San Gabriel, California. The grooved
FIG. 1. Schematic of float-pulley system.
drum of the differential pulley was precision machined (Fig. 2) to accept 30.48 cm (1.00 ft.) of stainless-steel wire per revolution. A low-voltage input to the potentiometer than gave an output in the 0-100 mv range. This output corresponded to the level of the water table, at an approximate rate of 30.48 cm per 10 mv. This rate was adjustable via other electronic components in the remote data-acquisition center.

The frame containing the pulley and potentiometer was screwed into a wooden platform; this platform was in turn fixed by brackets to the top of the PVC pipe. Holes were cut in the platform to permit passage of the float wire and counterweight lines. A hard plastic cover was placed over the platform and fixed by screws. All edges and joints were caulked to effect a tight weather seal around the components.

Float and Counterweight

The gage float was made from a 8.255-cm-diameter float of plastic foam of the type used to support fishing nets. The center hole of the float was plugged, an eyelet was screwed in, and the insertions were sealed with epoxy. A swivel-link connection was made to the float wire at the point of its attachment to the eyelet. A small trim weight was added beneath the float to provide tension and to position the water line on the side of the float. The counterweight consisted of approximately 198 gm (7 oz.) of lead, cast in a thin cylindrical form. It was connected to the monofilament line and placed in the counterweight well.

FIELD INSTALLATION

Thirteen well units were placed in the beach in the positions shown on Figure 3. At each installation point the slotted, 3-m-long PVC pipe was first jetted into the beach. This was accomplished by inserting a length of slightly smaller diameter, solid PVC pipe within the slotted PVC pipe. A steel pipe was then inserted within the solid PVC pipe and water was pumped down the steel pipe. The solid PVC pipe acted as a return-flow "casing" for advancing all three pipes into the sand. Water was pumped down the steel pipe at a rate sufficient to flush sand grains up and over the top of the solid PVC pipe. A baffle ring prevented sand from running back down between the two PVC pipes. The pipes were advanced slowly, the tip of
FIG. 2. Top view of pulley showing position relative to PVC, steel pipes. (Dimensions are in inches).
FIG. 3. Plan and profile views of the Fort Story water-table study site, showing typical beach and water-table profiles, water-table monitoring wells (1-13), profiling stations (A-Z), and groundwater sampling probes (I-IV).
the steel pipe leading the PVC pipes by a few cm. Success in such an operation requires a portable high-volume water pump. A gasoline-engine-powered Jabsco pump was used. Some sticking of the slotted PVC pipe was experienced during the pipe-jetting process.

Water for jetting was obtained directly from the ocean for the seawardmost wells; a tank truck had to be used for a supply of water for jetting the more landward wells (Fig. 3, nos. 1-10).

After emplacement and prior to packing sand around the top, the slotted PVC section was trimmed so as to be about one foot below the sand surface. A solid PVC section was then added and the entire upper portion of the well was aligned vertically with a carpenter's level. The steel pipe for the counter-weight was then jetted into the beach, its side touching the PVC pipe. The two pipes were snugged together tightly with stainless-steel wire and the pulley-potentiometer system was then added. It was found that sand rose up only a few cm in the bottoms of the pipes after termination of jetting. Thus, capping of the bottom ends was unnecessary.

Electrical cables were strung out to each well unit on cross braces fixed just below the instrument platform of each well. A single waterproof multi-conductor cable entered the base of each instrument platform. The wires connected the potentiometers to a low-level, digital data acquisition system (DATUM, Inc., Model 120-115). Power for the potentiometer input circuit was obtained from a power supply in the DATUM system that operated off line voltage. Output signals were amplified and digitized by the DATUM unit and then recorded on a computer-compatible magnetic tape recorder (DIGIDATA No. 1339-800).

Water levels were determined every 10 or 15 minutes. Four several-hour-long gaps in data acquisition were experienced, owing to failures in one of the circuit boards of the DATUM system. A storm on August 21-23 damaged wells 12 and 13 (Fig. 3). With these two exceptions, the entire system functioned continuously for 31 days.
RESULTS

The results of this investigation have been published by Harrison and Fausak (1970, Appendix III) in the form of a computer listing of digital values for water level at the 13 wells. L. E. Fausak programmed the digital data for a Calcomp plotter. The portions of the Calcomp printout that appear in Figures 4-8 were selected to give the reader an idea of the reliability of the instrumentation.

Figure 4 shows some obviously spurious data, that were obtained at about 10,000 minutes (elapsed time), for wells 2 and 4. The anomalies should be discarded and the gaps bridged by interpolation. The significance of other sharp oscillations in the output signals, however, may be much more difficult to interpret (for example, Fig. 5, 14,750 minutes).

Fluctuations in water level due to surf action are evident in wells 11, 12, and 13 during non-storm conditions (Fig. 4, 2800 to 3100 minutes) around the time of high tide. Such fluctuations are common at many more of the wells during the storm flooding that began at roughly 15400 minutes (Fig. 6). The spurious signals from wells 12 and 13, the wells that were damaged by the storm, are evident in Figures 7 and 8 (uppermost two traces).

CONCLUSION AND RECOMMENDATIONS

With the exception of certain failures in the electronics of the DATUM system, the instrumentation met the needs of this study. Installation and removal of water wells requires manpower and machinery that go beyond the needs of casual beach studies. The electronic and water-well equipment alone cost about $18,000 in 1969. The cost of support equipment cannot be estimated because surplus materials and vehicles were used.

For most studies of water-table fluctuations, far less than 13 water wells are needed to adequately sample over a distance such as that (83 m) which was investigated in this study. Future students of beach water-table fluctuations or groundwater flow can use the data of this study as a guide when designing their measurement systems. In general, it should be possible to use less wells and, perhaps, simpler and less-expensive recording equipment.
FIG. 4. Water table elevation at the 13 wells of Figure 3, as a function of time. (Pre-storm conditions).
FIG. 5. Water table elevations as a function of time. (Pre-storm conditions).
FIG. 6. Water table elevations as a function of time.
(Onset of storm and storm conditions).
FIG. 7. Water table elevations as a function of time. (Post-storm conditions).
FIG. 8. Water table elevations as a function of time. (Post-large storm and onset of small storm conditions). Wells 12 and 13 are broken and give spurious signals (uppermost two traces).
ACKNOWLEDGMENTS

We thank Dr. David DeVries for supervising installation of the well pipes and Mr. David Tyler for assistance in the field. Mr. William Hale assisted in assembly of the float-pulley and electrical systems. Dr. W. G. MacIntyre supervised the mating and operating of the electronic instrumentation.

REFERENCE

ABSTRACT

Fluctuations of the water table of a tidal marine beach over a 30-day period are examined. The relative importance of variations in still water level, swash runup distance, distance of a sampling station from the shoreline, and atmospheric pressure are determined for rising half-tide cycles in each of 13 water table monitoring wells spaced along a transect perpendicular to the shoreline.

Results showed that tidal oscillations exert the strongest influence in all except the two seawardmost wells and in the most landward well. Distance from the shoreline is the most important variable in the seawardmost wells because of the exponential decay of the input wave and the resultant large range of water-table fluctuations near the beach face. Atmospheric pressure becomes the dominant variable influencing water-table fluctuations in the most landward well, due to the relatively slight contribution of the tide and wave inputs. The amplitude of the water-table fluctuations decreases exponentially in a landward direction and the lag time of the input wave increases linearly with distance from the shoreline. The time lag is found to be approximately 60 minutes per 18 meters of beach penetrated.

INTRODUCTION

The primary factors responsible for the size and configuration of beaches are the type and quantity of sediment available, and the action of the wind-generated surface waves which break on the shore. The source materials are generally constant for a given beach and change only very gradually; the waves striking the beach are responsible for the rapidly changing, short-term, and often cyclic, variations in beach characteristics. The interaction between the waves and the beach, however, is regulated by the position and pressure head of the beach water table, which in turn primarily a function of wave input, tidal level, and permeability of the sand body. This paper will focus attention upon the water table of a specific tidal marine beach and attempt to elucidate some interactions of beach process variables and
fluctuations of the beach water table.

The role of the water table in beach processes was noted by Bagnold (1940), who recognized that sand was deposited on an unsaturated beach due to the energy loss accompanying percolation of the swash into the dry sand. Subsequent work by Grant (1948) and Emery and Foster (1948) established that a high water table promotes erosion of the beach face, and that conversely, a low water table may result in pronounced accretion on the foreshore. Issacs and Bascom (1949) examined the water tables of ten Pacific beaches and recognized the damping of the tidal wave as it passes through the sand body. Duncan (1964), Strahler (1964), and Geise (1966) have shown that zones of erosion and deposition migrate up and down the foreshore in response to the relative positions of the water table and still water level. The relationship of water table position and the movement of sand on the foreshore was defined in quantitative terms by Harrison (1969), who found that the strongest predictors of foreshore erosion and deposition were (1) the breaker steepness, and (2) the ratio of the hydraulic head of the water table to the swash runup distance. Given the demonstrated importance of the water table to beach stability, the present study was undertaken in order to investigate the response of the water table to waves, tides, and atmospheric conditions.

The site chosen for the study is a marine beach located on the seaward side of Cape Henry, Virginia (Figs. 1 and 2), adjacent to the mouth of Chesapeake Bay. The beach is composed of quartzose sand with an average median diameter of 0.41 mm. The average width of the beach, from shoreline to dune ridge, is approximately 60 meters; the average slope is approximately 6°. The offshore zone is gently shoaling and has few bottom irregularities. Figure 3 shows a typical beach profile with the location and spacing of the water table monitoring wells and profile station-markers. The mean range of the astronomical tide is 0.85 m, the spring range is 1.04 m. The nearshore current system is influenced by the ebb and flood of the tide through the adjacent mouth of Chesapeake Bay, but the dominant direction of flow is northerly, due to the normal southerly wave approach and to a persistent clockwise eddy located south of Cape Henry. The mean breaker height for the study period ranged from 0.6 to 1.0 m.

The 30-day data collection phase of the study can be divided into three periods, based on weather and sea state conditions (see Fig. 4). The initial ten days
FIG. 1. Regional location map.

FIG. 2. Location of the Fort Story study site.
FIG. 3. Plan and profile views of the Fort Story study site, showing typical beach and water table profiles, and the spatial distribution of profile stations (A-Z), water table monitoring wells (1-13), and water table sampling probes (I-IV).
FIG. 4. Wave and tide data for the study period.
were characterized by the normal long, low swell
approaching from the south; 3.05 inches of rain fell
on the 14th of August, but was not accompanied by a
significant change in the sea state. A three-day
period of high, short-period waves generated by a
local storm followed; rainfall during this period was
slight, however, with a total of 0.15 inches. The
remaining 17 days saw the return of the normal summer
sea conditions; two periods of significant rainfall
occurred during this latter period, one of 0.45 inches
and one of 0.34 inches, both on September 9th.

PROCESS-RESPONSE MODEL

An intuitive appraisal of the driving forces
behind fluctuations of the water table yields the
following: 1) the ocean tides will be expected to be
the major forcing function, 2) surf conditions should
be of considerable importance, 3) distance inland from
the ocean free surface has been shown to be a deter­
minant of the water table excursion (e.g., Emery and
Foster, 1948; Issacs and Bascom, 1949), and 4) a small
but significant factor might be found in atmospheric
pressure fluctuations, with regard to both the rhythmic
atmospheric tides and to progressive changes associated
with the local weather conditions.

These factors were investigated within the
conceptual framework of the general process-response
model (Krumbein, 1963), in which a given state of the
beach is considered to be a response to any number of
geologic processes. Transformed into a linear
mathematical model, the relationship becomes:

\[ Y = f(X_1, X_2, X_3, \ldots, X_n) \]

where \( Y \) is designated the response element and \( X_1 \) to
\( X_n \) are termed the process elements. In terms of the
present study

\[ W = f(T, X, D, P) \]

where

\[ W = \text{the change in elevation of the water table}
   \text{for a rising half-tide cycle} \]

\[ T = \text{the tidal range for a rising half-tide}
   \text{cycle} \]

\[ X = \text{the horizontal distance from a well to a}
   \text{point on the foreshore one half the vertical}
   \text{distance between the preceding low and the} \]
succeeding high still water levels, measured at time of mid-tide

$D =$ the vertical distance between high tide still water level and a horizontal line representing the average position of the swash at its highest level

$P =$ the change in atmospheric pressure over the period of the rising half-tide cycle.

The variables are depicted graphically in Figure 5.

A sequential linear multiple regression analysis was chosen to determine the relative importance of the four process elements determining the magnitude of the water-table fluctuations. This method was developed by Krumbein (Krumbein, Benson, and Hempkins, 1964) and is reviewed and utilized by Harrison and Krumbein (1964). The method consists essentially of first performing a simple regression analysis of the dependent variable against the independent variables, taken one at a time. Regressions are then run using all possible pairs, triplets, etc., until all possible combinations of the variables have been exhausted. The advantages of this method are that interrelationships among the dependent variables themselves become apparent, and that data redundancy, i.e., the degree to which the same information is found in two or more variables, can be determined. Such an approach also may be used to rank the "independent" variables, taken singly and in combination, in order of the relative importance.

In addition, simple linear regression analyses were performed to derive a mathematical expression for the decrease in amplitude of water table fluctuations as a function of distance from the shoreline, and for the lag time of the input tide wave as a function of the distance from the shoreline.

For purposes of simplification, only those factors responsible for a rise in water table level were investigated. Furthermore, only periods of normal conditions were used in the regression analyses; no periods of rainfall or wave overtopping were included. The rising half-tide cycles used are indicated by heavy block lines on the tide curve of Figure 4.

The regression variables were derived from five measured variables of the beach-ocean-atmosphere system. The symbols used for the measured and derived variables, with their dimensions, sampling intervals, and error estimates, are summarized in Table 1.
FIG. 5. Diagrammatic representation of the variables $W$, $T$, $X$, $D$, and $P$ used in the multiple regression analysis.
Table 1.- Measured and Derived Variables, with Their Symbols, Sampling Frequency, Range, and Estimate of Accuracy.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Sampling frequency</th>
<th>Range in values</th>
<th>Estimate of accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>(E_w)</td>
<td>Elevation of groundwater table</td>
<td>10 and 15 minutes</td>
<td>0.291 to 1.999 m above msl</td>
<td>±0.01 m</td>
</tr>
<tr>
<td>(E_t)</td>
<td>Elevation of tidal plane</td>
<td>continuous</td>
<td>-0.55 to 1.27 m above msl</td>
<td>±0.05 m</td>
</tr>
<tr>
<td>S</td>
<td>Position of limit of swash</td>
<td>hourly and at high, low, and mid-tides</td>
<td>G to Z</td>
<td>±0.25 m</td>
</tr>
<tr>
<td>(E_b)</td>
<td>Elevation of beach profile station</td>
<td>high, low, and mid-tides</td>
<td>-0.940 to 3.693 m above msl</td>
<td>±0.05 m above water</td>
</tr>
<tr>
<td>P</td>
<td>Atmospheric pressure</td>
<td>continuous</td>
<td>29.550-30.285 in. Hg.</td>
<td>±0.005 in. Hg.</td>
</tr>
<tr>
<td>X</td>
<td>Distance of well from datum</td>
<td>Derived</td>
<td>6.83 to 72.62 m</td>
<td>±0.20 m</td>
</tr>
<tr>
<td>D</td>
<td>Swash height above still water level</td>
<td>Derived</td>
<td>0.04 to 0.56 m</td>
<td>±0.25 m</td>
</tr>
<tr>
<td>W</td>
<td>Change of (E_w) for each rising half-tide</td>
<td>Derived</td>
<td>0.003 to 0.870 m</td>
<td>±0.005 m</td>
</tr>
<tr>
<td>T</td>
<td>Tidal range</td>
<td>Derived for each rising half-tide</td>
<td>0.46 to 0.04 m</td>
<td>±0.05 m</td>
</tr>
</tbody>
</table>
The following assumptions were made: (1) the beach is internally homogeneous in texture and sediment characteristics; (2) changes in temperature, and therefore in the density and viscosity of the sea and ground waters, were not significant; and (3) that the angle of wave approach did not significantly alter the swash runup distance.

RESULTS

General Water-Table Dynamics

Time-dependent fluctuations of the water table for each of the 13 wells are represented by the curves in Figures 6 and 7. These are selected, but typical, parts of the total 30-day record. Figure 6 shows the changes in water level for normal, low-breaker conditions and will be used to illustrate several features of water table dynamics. Of particular interest are the following points:

1) the rise of the water table is generally more rapid than the fall. This phenomenon is due to the seaward-directed head gradient of the groundwater which, on the rising tide, contributes water to a given volume of the beach in addition to the seawater that is being added. Thus, at a point near the foreshore there is a rapid rise due to the addition of water from both directions, while the rate of fall is lessened due to the continuing addition of new, seaward-flowing groundwater;

2) at times of higher high tides, the water-table elevations on the foreshore exceed those on the backshore, resulting in a landward sloping water table. A landward flow of sea water might therefore be expected. Note also that such a slope reversal is uncommon on the lower high tide maxima;

3) the time required for the passage of the damped tidal wave through the sand prism is clearly seen. The time of maximum water table elevation increases for each well in a landward direction. The time required for the wave to pass from the shoreline to well number 1, a distance of approximately 56 meters, is on the order of 4.5 to 5.0 hours;

4) the decrease in amplitude of the tide wave as it passes through the beach is clearly seen. The range of the water table fluctuations in well number 1 is normally less than 5 centimeters; the range of the

-24-
FIG. 6. Time series plot of water table fluctuations for a period of normal wave and tide conditions.
ocean tides is 0.7-0.8 meters. The decay of the tide wave approximates the exponential amplitudinal decay rate which would be expected from classical wave mechanics; and

5) the levels of the water table between certain wells are characteristically closer than between other wells, resulting in the pairing of the traces for wells 1 and 2, 4 and 5, 7 and 8, and 9 and 10. The underlying reasons for these pairings are not clear; there is apparently no relationship, however, between the pairs and the depth of the water table below the beach surface or the distance between wells. Since the phenomenon is stationary in both time and space, it is most likely due to some feature of the sand prism itself, such as the distribution of sand size, sorting, or packing.

In contrast to the normal conditions discussed above, Figure 7 shows the fluctuations in groundwater level for a period of two full tidal cycles during wave overtopping accompanying a local storm. The increase in the water table elevations is due in part to the increase in still water level (the storm surge), in part to the storm waves which reached heights of two meters and overtopped the berm crest, and in part to the decrease in atmospheric pressure accompanying the disturbance. The cumulative effect is a rapid rise in groundwater level; the restoration of the water table to its pre-storm position is seen to be much slower than the rise, taking approximately nine days to return to its original equilibrium position.

The flattening of the trough of the curve for well number 13 near the righthand edge of the figure is due to damage sustained to the well mechanism during the period of high waves; shortly thereafter well 13 ceased to function entirely, followed a short time later by well number 12.

Regression Analyses

The means and standard deviations of the data set used in the sequential multiregression analyses are presented in Table 2. The values for the tide range (T), the swash height (D), and pressure change (P) remain largely the same, since these are not peculiar to an individual well. The 15 sets of measurements obtained for wells 12 and 13 are for the pre-storm period, prior to the failure of these two wells.
FIG. 7. Time series plot of water table fluctuations for a period of storm conditions.
Table 2.- Means and Standard Deviations for $W$, $T$, $X$, $D$, and $P$ for Each of the 13 Wells.

<table>
<thead>
<tr>
<th>Well no.</th>
<th>n</th>
<th>$W$</th>
<th>$T$</th>
<th>$X$</th>
<th>$D$</th>
<th>$P$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$n$</td>
<td>$\bar{x}$</td>
<td>$s$</td>
<td>$\bar{x}$</td>
<td>$s$</td>
<td>$\bar{x}$</td>
</tr>
<tr>
<td>1</td>
<td>28</td>
<td>0.033</td>
<td>0.020</td>
<td>0.81</td>
<td>0.17</td>
<td>59.37</td>
</tr>
<tr>
<td>2</td>
<td>28</td>
<td>0.045</td>
<td>0.023</td>
<td>&quot;</td>
<td>&quot;</td>
<td>50.17</td>
</tr>
<tr>
<td>3</td>
<td>28</td>
<td>0.063</td>
<td>0.029</td>
<td>&quot;</td>
<td>&quot;</td>
<td>43.67</td>
</tr>
<tr>
<td>4</td>
<td>28</td>
<td>0.089</td>
<td>0.036</td>
<td>&quot;</td>
<td>&quot;</td>
<td>37.28</td>
</tr>
<tr>
<td>5</td>
<td>29</td>
<td>0.107</td>
<td>0.042</td>
<td>&quot;</td>
<td>&quot;</td>
<td>33.95</td>
</tr>
<tr>
<td>6</td>
<td>29</td>
<td>0.135</td>
<td>0.051</td>
<td>&quot;</td>
<td>&quot;</td>
<td>30.75</td>
</tr>
<tr>
<td>7</td>
<td>29</td>
<td>0.164</td>
<td>0.061</td>
<td>&quot;</td>
<td>&quot;</td>
<td>27.52</td>
</tr>
<tr>
<td>8</td>
<td>29</td>
<td>0.183</td>
<td>0.069</td>
<td>&quot;</td>
<td>&quot;</td>
<td>24.30</td>
</tr>
<tr>
<td>9</td>
<td>29</td>
<td>0.236</td>
<td>0.089</td>
<td>&quot;</td>
<td>&quot;</td>
<td>21.11</td>
</tr>
<tr>
<td>10</td>
<td>29</td>
<td>0.284</td>
<td>0.110</td>
<td>&quot;</td>
<td>&quot;</td>
<td>17.97</td>
</tr>
<tr>
<td>11</td>
<td>29</td>
<td>0.360</td>
<td>0.148</td>
<td>&quot;</td>
<td>&quot;</td>
<td>14.90</td>
</tr>
<tr>
<td>12</td>
<td>15</td>
<td>0.391</td>
<td>0.135</td>
<td>0.89</td>
<td>0.11</td>
<td>16.08</td>
</tr>
<tr>
<td>13</td>
<td>15</td>
<td>0.517</td>
<td>0.191</td>
<td>&quot;</td>
<td>&quot;</td>
<td>12.82</td>
</tr>
</tbody>
</table>
The results of the regression analyses for each of the 13 wells are given in Tables 3 and 4. It should be noted that wells 12 and 13 have only 15 samples each; the analyses for these two wells, therefore, lack adequate numbers of observations, and the results are not directly comparable to wells 1 through 11. They are tabulated insofar as they may serve as indicators of trends on the foreshore.

Table 3 lists the predictor equations derived for each well and serves to show the signs of correlation for the process variables. Tidal range (T) and swash height above still water level (D) are positively correlated; i.e., an increase in the magnitude of each independent variable causes an increase in the elevation of the water table in each of the 13 wells. The distance of a well from the foreshore (X), and change in atmospheric pressure (P), on the other hand, are negatively correlated; an increase in either of these variables results in a decrease in the level of the water table in a given well. The correlations are consistent throughout the data set, except for (P) in well number 13, which, as stated above, lacked a sufficient number of data points.

The total percent reductions of the sums of squares of W accounted for by T, X, D, and P taken together are given in Table 4. Also given are the individual contributions of T, S, D, and P, presented as percentages of the total sums of squares reduction for all four process variables; these values are plotted in Figure 8. The results show that the four process variables chosen account for a greater per cent of the variability in the water table fluctuations near the shoreline than they do on the backshore. They show also that the effects of atmospheric pressure on water-table fluctuations are relatively more significant near the dune line, but become less so in a seaward direction; the distance of an individual well from the shoreline and the swash height above still water level become relatively more important toward the shoreline, while the relative importance of the tidal range becomes less.

**DISCUSSION**

The equations listed in Table 3 are the results of linear multiple regression analyses. It is known, however, that nonlinear relationships exist between the water table fluctuations and certain of the independent variables; e.g., water table changes and distance from...
Table 3.- Regression Equations for $W$ for the 13 Wells.

<table>
<thead>
<tr>
<th>Well no.</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$W = -0.0206 + 0.0787(T) - 0.0005(X) + 0.0729(D) - 0.3301(P)$</td>
</tr>
<tr>
<td>2</td>
<td>$W = -0.0014 + 0.0899(T) - 0.0009(X) + 0.0703(D) - 0.2707(P)$</td>
</tr>
<tr>
<td>3</td>
<td>$W = -0.0070 + 0.1342(T) - 0.0013(X) + 0.0704(D) - 0.2825(P)$</td>
</tr>
<tr>
<td>4</td>
<td>$W = -0.0009 + 0.1706(T) - 0.0020(X) + 0.0949(D) - 0.2603(P)$</td>
</tr>
<tr>
<td>5</td>
<td>$W = +0.0143 + 0.1904(T) - 0.0027(X) + 0.1079(D) - 0.2278(P)$</td>
</tr>
<tr>
<td>6</td>
<td>$W = +0.0165 + 0.2288(T) - 0.0034(X) + 0.1286(D) - 0.2273(P)$</td>
</tr>
<tr>
<td>7</td>
<td>$W = +0.0236 + 0.2703(T) - 0.0044(X) + 0.1497(D) - 0.2097(P)$</td>
</tr>
<tr>
<td>8</td>
<td>$W = +0.0113 + 0.3009(T) - 0.0051(X) + 0.1735(D) - 0.2323(P)$</td>
</tr>
<tr>
<td>9</td>
<td>$W = +0.0272 + 0.3663(T) - 0.0072(X) + 0.2190(D) - 0.1968(P)$</td>
</tr>
<tr>
<td>10</td>
<td>$W = +0.0117 + 0.4409(T) - 0.0092(X) + 0.2685(D) - 0.1509(P)$</td>
</tr>
<tr>
<td>11</td>
<td>$W = -0.0669 + 0.5874(T) - 0.0120(X) + 0.4479(D) - 0.1757(P)$</td>
</tr>
<tr>
<td>12</td>
<td>$W = +0.0086 + 0.4336(T) - 0.0093(X) + 0.5776(D) - 0.6278(P)$</td>
</tr>
<tr>
<td>13</td>
<td>$W = +0.2880 + 0.4121(T) - 0.0175(X) + 0.2793(D) + 0.1360(P)$</td>
</tr>
</tbody>
</table>
Table 4.- Contributions of T, X, D, and P to Explanation of Variation in W.

<table>
<thead>
<tr>
<th>Well no.</th>
<th>n</th>
<th>Total % Red. in SS</th>
<th>T</th>
<th>X</th>
<th>D</th>
<th>P</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>28</td>
<td>50.69</td>
<td>26.94</td>
<td>22.61</td>
<td>22.35</td>
<td>28.08</td>
</tr>
<tr>
<td>2</td>
<td>28</td>
<td>48.92</td>
<td>29.09</td>
<td></td>
<td>22.20</td>
<td>23.39</td>
</tr>
<tr>
<td>3</td>
<td>28</td>
<td>57.85</td>
<td>35.65</td>
<td>23.78</td>
<td></td>
<td>19.91</td>
</tr>
<tr>
<td>4</td>
<td>28</td>
<td>62.64</td>
<td>35.01</td>
<td>24.90</td>
<td>20.85</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>29</td>
<td>65.21</td>
<td>33.82</td>
<td>25.71</td>
<td>21.45</td>
<td>19.00</td>
</tr>
<tr>
<td>6</td>
<td>29</td>
<td>65.33</td>
<td>33.64</td>
<td>25.86</td>
<td>21.58</td>
<td>18.90</td>
</tr>
<tr>
<td>7</td>
<td>29</td>
<td>66.70</td>
<td>32.84</td>
<td>26.59</td>
<td>21.70</td>
<td>18.84</td>
</tr>
<tr>
<td>8</td>
<td>29</td>
<td>67.42</td>
<td>32.26</td>
<td>26.93</td>
<td>21.93</td>
<td>18.96</td>
</tr>
<tr>
<td>9</td>
<td>29</td>
<td>67.32</td>
<td>30.45</td>
<td>27.27</td>
<td>22.38</td>
<td>18.98</td>
</tr>
<tr>
<td>10</td>
<td>29</td>
<td>67.26</td>
<td>29.63</td>
<td>27.61</td>
<td>22.40</td>
<td>18.88</td>
</tr>
<tr>
<td>11</td>
<td>29</td>
<td>71.40</td>
<td>28.70</td>
<td>27.95</td>
<td>24.08</td>
<td>19.24</td>
</tr>
<tr>
<td>12</td>
<td>15</td>
<td>(85.75)</td>
<td>(23.54)</td>
<td></td>
<td>(28.68)</td>
<td>(27.52)</td>
</tr>
<tr>
<td>13</td>
<td>15</td>
<td>(64.55)</td>
<td>(24.47)</td>
<td></td>
<td>(30.76)</td>
<td>(23.35)</td>
</tr>
</tbody>
</table>
FIG. 8. Relative importance of T, X, D, and P in the reduction of the sums of squares of W.
the shoreline (Fig. 9). A deterministic equation for fluctuations of the water table must account for these nonlinear dependencies; predictive equations based on multiregression analysis for each well, on the other hand, need not account for the nonlinearity if the data are such that they may be satisfied by a linear expression. Scatter diagrams for individual wells indicated that the latter method could be used here. The data plotted in Figure 9 represent the combined data from the 13 wells; these data were in fact analyzed in 13 individual regression analyses, and resulted in predictor equations for water table fluctuations at 13 points on the beach.

The reliability of the predictor equations listed in Table 3, as indicated by the per cent sums of squares accounted for, ranges from fair in the backshore area to good on the foreshore; the sums of squares accounted for generally increases in a seaward direction. It may be concluded from these observations that water-table fluctuations in the backshore and dune areas are significantly influenced by variables not taken into account in the present study. The major factor believed to be of significance in the backshore is the groundwater pressure head, as influenced by local weather conditions in the supply or charging area.

The major forcing function, as shown by the regression analyses, is the action of the tides. The progressive tide wave evident on the free surface is propagated into the sand prism; the amplitude and period of the tidal oscillations at the foreshore are essentially the same as those in open waters; as the wave form passes into the beach, however, and is propagated in the water table, certain fundamental changes take place, which are functions of the porosity and permeability of the sand, of the pressure gradient encountered, and of the amplitude of the tidal fluctuations. The amplitude of the wave is rapidly reduced, as is presented in Figure 9, in which the rise of the water table over a rising half-tide is plotted against distance from shore. A wide range of water level fluctuations are observed at the foreshore, corresponding to the various amplitudes of the input tidal wave; the water table fluctuations are reduced to an essentially constant value of about two to three centimeters at a distance of 60 to 65 meters from the shoreline. The least-squares curve for the data is represented by the following equation:

\[ W = f(1/X) = -0.04 + (0.52/X) \]

which resulted from a regression of \( W \) on \( 1/X \). Even
FIG. 9. The amplitude of the water table fluctuations as a function of distance from the shoreline.
though $1/X$ resulted in a somewhat better fit, an exponential function would physically be more appealing.

The greater variation of the post-storm data exhibited in Figure 9 reflects the greater variability of the tidal amplitudes following the storm (see Fig. 4). A plot of the ratio of the water table rise and the tidal amplitude, $W/T$, versus distance from the shoreline (Fig. 10) effectively eliminates that variation and shows the rise of the water table as a function of both tidal amplitude and distance from the shoreline.

Lag times of the input tide waves are presented as a function of distance from the shoreline in Figure 11; regression lines are shown for the lag of the high water crest, the low water trough, and for all observations combined. The data are from the sample periods as listed in the appendix. The plot indicates an average lag of the wave of approximately one hour for each 18 meters of beach penetrated; the lag is represented by the equation

$$L = 50.38 + 3.27X$$

where

$L = \text{lag time in minutes}$

$X = \text{distance from shoreline in meters}$.

The lag time determined here is somewhat less than those observed by Emery and Foster (1948), who found that the wave lags from one to three hours at a distance of from 20 to 40 feet (approximately 6 to 13 meters) from the shoreline.

The slope of the low water regression line is less steep than the high water line, indicating that the high water crest is propagated through the sand prism somewhat slower than the low water trough; this differential lag phenomenon can also be seen in the time series plot of water table elevation (Fig. 6) by comparing the travel times of the crests and troughs between wells 13 and 1. The lag differential is another manifestation of the seaward-directed pressure head of the ground water, which also causes the water table to rise more rapidly than fall, as was discussed earlier.

The increase in lag time appears to be generally linear throughout the range studied. However, the lag time at the shoreline should, by definition, be
FIG. 10. The ratio of the rise of the water table and the rise in still water level as a function of distance from the shoreline.
FIG. 11. Lag time of the input tide waves, as a function of distance from the shoreline.
zero, whereas the regression analyses show initial lag times of 38 to 59 minutes. The discrepancy may be due to measurement error; it may on the other hand, be due to a somewhat modified propagation mechanism at work in the first 10-15 meters of the beach. Such an assumption would imply that the rate of energy dissipation is greater during the first 10 to 15 meters of the beach, followed by a lesser rate of dissipation for the backshore area.

The tidal forces predominate throughout the backshore area; near the foreshore, the tidal forces are subordinate to the swash height and the distance from the shoreline; the effects of both die off rapidly in a landward direction. The relative importance of each of the four process variables is graphically depicted in Figure 8.

The effect of rainfall on the level of the water table appeared to be very slight during the study period. The largest period of rainfall for which water table data were obtained amounted to 0.83 inches (2.11 cm). The effect on the water table can be seen (Fig. 12) as an increase of approximately 1.5 cm in water table level; the increase is consistent for all wells in the transect. The rapid and large increase in water table level (Fig. 7) is due mainly to the action of the high waves and the storm surge and possibly to atmospheric pressure effects; atmospheric pressure suddenly dropped 0.735 inches of Hg just prior to the increase in water column of about 18 cm.

A possible source of noise in the data which may account for some part of the low sums of squares reductions may be found in changes in beach configuration which were not due to changing wave or water table conditions. Such changes were observed in the long-shore passage of sand waves, a phenomenon which belies the original assumption of a two-dimensional beach. Such changes are illustrated in Figure 13, which shows the position of the shoreline plotted against time for each rising half-tide of the study period. The fluctuations of the shoreline are quasi-periodic and have a period of six to seven days; no correspondence is seen between the changes in shoreline position and any changes in the wave or tidal characteristics. Similar features are not uncommon along the middle-Atlantic coast and have been investigated by Sonu and Russel (1966) and Dolan (1970) at Nags Head, North Carolina, where they are quite pronounced. Future beach studies along the mid-Atlantic coast will necessarily have to consider the beach as a three-dimensional feature.

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FIG. 12. Effect of rainfall on the level of the water table.
FIG. 13. Position of the shoreline as a function of time for the 30-day study period.
CONCLUSIONS

Quantitative and qualitative consideration has been given to the variations in the level of the water table of a marine beach and to the factors responsible for these variations.

The following conclusions have been drawn from the study:

(1) The tidal fluctuations of the free ocean surface are the major forcing function with regard to water table elevational changes.

(2) The distance from the shoreline (and thus from the source of energy input) is a more important determinant of the level of the water table near the foreshore than is the oscillation of the tidal plane.

(3) The input tide wave decreases rapidly in amplitude upon entering the beach, and dies off exponentially until a semi-constant value of two to three centimeters is attained approximately 60 meters from the shoreline.

(4) The input tide wave exhibits a lag time which increases landward at about the rate of one hour per 18 meters of beach penetrated.

(5) Rainfall did not alter the water table level to a significant degree during the study period.

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CHANGES IN FORESHORE SAND VOLUME ON A TIDAL BEACH: 
ROLE OF FLUCTUATIONS IN WATER TABLE 
AND OCEAN STILL-WATER LEVEL 

W. Harrison 

ABSTRACT 

A 30-day-long time series of observations of variables in the beach-ocean-groundwater system was made at Virginia Beach, Virginia, during August and September, 1969. Results of linear multiregression analysis of the data show that the change in ocean still water level, SWL, is the single most important variable influencing changes in quantity of foreshore sand, Qf, over intervals of half-tidal-cycle to tidal-cycle length. An index of groundwater head, I, is the next strongest predictor of Qf, followed closely by the number of swash events, S, over the time interval in question. Predictor equations that are presented for Qf are the strongest yet obtained and when one of the equations was tested upon a completely independent set of data it explained 40 percent of the observed variability. 

The importance to Qf of change in ocean SWL and groundwater head is apparent when it is realized that, statistically, the following equation explains 67 percent of the variability in quantity of foreshore sand eroded or deposited over a tide-cycle interval: 

\[
Q_f = -0.9762 - 0.04857(\Delta h) + 0.0021636(I_2)
\]

where \(Q_f\) is in m\(^3\), \(\Delta h\) is the net change (in cm) in SWL relative to the SWL at initial low water, and \(I_2 = (Y_{10} - Y_{11}) x\), where \(Y_{10}\) and \(Y_{11}\) are the MSL elevations of the water table, in mm, in two wells, and \(x\) is the distance of well 11 from the foreshore surface in m. (The value of \(x\) ranged between 2.6 and 12.0 m, the distance between wells 10 and 11 was 3.1 m, and subscript 2 stands for time of rising half tide). 

INTRODUCTION 

The earliest suggestion of the importance of swash percolation and groundwater flow to beach dynamics seems to have been made by Bagnold (1940). His laboratory experiments were followed by the field studies of Grant (1946, 1948), Emery and Foster (1948), Issacs and Bascom (1949), and Duncan (1964). Although
these studies indicated the importance of water-table fluctuations to beach foreshore changes, they did not quantify the processes involved. Harrison (1969) made some progress toward quantification of the relationship between changes in the water-table and foreshore sand volume. He performed multiregression analysis on a 26-day-long time series of observations collected in 1966 from a beach at Camp Pendleton, Virginia (Fig. 1). The groundwater head was found to be one of the strongest predictors of deposition or depletion of foreshore sand, over a tidal-cycle interval. "Groundwater head" was expressed as the vertical distance between the water-table outcrop on the foreshore and the still-water level in front of the breaking waves.

The present study deals first with data for the beach-ocean-groundwater system that were gathered in 1969 from a beach at Ft. Story, Virginia (Fig. 1). A 30-day-long time series of observations of the variables listed in Table 1 was obtained to document pertinent interactions in the system. Thirteen wells for monitoring the water table (Fig. 1, 1-13), and 26 pipe-stations (Fig. 1, A-Z) for monitoring changes in beach elevation, were positioned along an 83-m-long transect oriented perpendicular to the shoreline and extending seaward from the edge of the foredune. Four multi-tube probes (Fig. 1, I-IV) were installed for extracting small amounts of groundwater for tests of salinity and to facilitate dye tests of flow characteristics. Salinity was determined at the site using a Goldberg refractometer (Behrens, 1965).

The water wells consisted of 1) a #18-slotted PVC pipe, 102-mm in diameter, jetted into the beach to a depth of 3.5 m and 2) a 32-mm-O.D. steel pipe jetted to a depth of 5 m and touching the PVC pipe. A float-pulley system mounted on the pipes drove a potentiometer which provided a DC output voltage that corresponded (linearly) to the instantaneous water level. Water-table elevations were recorded at the site on computer-compatible magnetic tape (after A/D conversion). Details for this and the other measurement systems, as well as the complete time-series of measurements for the variables of Table 1, appear in Harrison and Fausak (1970).

The gently-sloping, quartz-sand beach at Ft. Story (Fig. 1) has a representative porosity of about 34 percent (medium grain diameter of pit samples ranged between 0.37 and 0.59 mm). The mean range of the astronomical tide is about 0.85 m for Cape Henry, the spring range is 1.1 m, and tide is semidiurnal.
FIG. 1. Plan and profile views of the Ft. Story study site, showing typical beach and water-table profiles, beach profiling stations (A-Z), wells for monitoring water table (1-13), and groundwater sampling probes (I-IV). Inset shows locations of the 1969 and 1966 studies, at Ft. Story and Camp Pendleton, respectively, Virginia Beach, Virginia.
TABLE 1. Measured and Derived Variables for the Beach-Ocean-Groundwater System at Ft. Story, Virginia Beach, Virginia, for the Period 10 August through 9 September, 1969.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Sampling Frequency</th>
<th>Range in Values</th>
<th>Estimate of Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_b$</td>
<td>Elevation of beach surface</td>
<td>High, low, and mid-tide levels</td>
<td>-0.940 to 3.693 m (MSL)</td>
<td>±0.005 m above water, ±0.020 m below water</td>
</tr>
<tr>
<td>$E_t$</td>
<td>Elevation of tidal plane</td>
<td>Continuous</td>
<td>-0.55 to 1.27 m (MSL)</td>
<td>±0.05 m</td>
</tr>
<tr>
<td>$E_w$</td>
<td>Elevation of water table</td>
<td>Either every 10 or every 15 minutes</td>
<td>0.291 to 1.999 m (MSL)</td>
<td>±0.003 m</td>
</tr>
<tr>
<td>$H_b$</td>
<td>Mean height of 50 successive breaking waves</td>
<td>High, low, and mid-tide levels</td>
<td>0.19 to 1.30 m (MSL)</td>
<td>±0.10 m</td>
</tr>
<tr>
<td>2$h_o$</td>
<td>Rise or fall in ocean SWL over half-tide cycle</td>
<td>Derived from $E_t$</td>
<td>-125 to +128 cm</td>
<td>±0.5 cm</td>
</tr>
<tr>
<td>$I$</td>
<td>Groundwater head index</td>
<td>Derived (high, low, and mid-tide levels)</td>
<td>+1408 to -1234</td>
<td>±3</td>
</tr>
<tr>
<td>$m$</td>
<td>Slope of the foreshore</td>
<td>Derived (high, low, and mid-tide levels)</td>
<td>4.0° to 11.0°</td>
<td>±0.5°</td>
</tr>
</tbody>
</table>

(Continued)
TABLE 1. (Continued)

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Sampling Frequency</th>
<th>Range in Values</th>
<th>Estimate of Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>p</td>
<td>Barometric pressure</td>
<td>Continuous</td>
<td>29.550 to 30.285 in. of Hg.</td>
<td>±0.005 in Hg.</td>
</tr>
<tr>
<td>r</td>
<td>Rainfall</td>
<td>Hourly during storms</td>
<td>Trace to 2.22 cm</td>
<td>±1.0 mm</td>
</tr>
<tr>
<td>s</td>
<td>Position of swash limit</td>
<td>Hourly, and at high, low, and mid-tide levels</td>
<td>Sta. G to Sta. Z (Fig. 1)</td>
<td>±0.25 m</td>
</tr>
<tr>
<td>$S$</td>
<td>Total number of swash events over a time interval</td>
<td>Derived from $T_b$</td>
<td>2150 to 4861 (1/2 cycle)</td>
<td>±5</td>
</tr>
<tr>
<td>$T_b$</td>
<td>Mean period of 50 successive breaking waves</td>
<td>High, low, and mid-tide levels</td>
<td>3.96 to 13.20 sec.</td>
<td>±0.02 sec.</td>
</tr>
<tr>
<td>x</td>
<td>Horizontal distance from well no. 11 to point on foreshore</td>
<td>Derived from $E_b$ and $E_t$</td>
<td>2.6 to 12.0 m</td>
<td>±0.03 m</td>
</tr>
<tr>
<td></td>
<td>1/2 vertical distance between successive extremes in SWL</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(Continued)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Sampling Frequency</th>
<th>Range in Values</th>
<th>Estimate of Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Δh</td>
<td>Net change in ocean SWL over a tidal cycle</td>
<td>Derived from $2h_o$</td>
<td>-13 to +16 cm</td>
<td>±0.5 cm</td>
</tr>
<tr>
<td>ΔQ_f</td>
<td>Change in quantity of foreshore sand over a time interval</td>
<td>Derived from $E_b$</td>
<td>-2.74 to +3.37 m$^3$ for $\frac{1}{2}$ tide cycle; -1.95 to +1.70 m$^3$ for entire cycle</td>
<td>±0.04 m$^3$</td>
</tr>
</tbody>
</table>
with a slight diurnal inequality.

Low, long-period swells characterized the first ten days of the 30-day study period. They were followed by three days of high, short-period waves that, combined with a storm surge, resulted in flooding of the backshore (Fig. 1) for two full tidal cycles. Nine days were required for the groundwater level to return to its pre-storm position. The remaining 17 days of the study had a wave climate similar to the first 10 days. Rainfall amounted to 10.2 cm during the 30-day period. The greatest hourly rainfall was 2.22 cm, an event that caused an increase of about 1.5 cm in the water level at all wells. The effects of rainfall, as well as sudden fluctuations in atmospheric pressure, can be ignored in what follows.

Figure 2 shows the groundwater salinity structure at Ft. Story for the summer beach (17 August 1969); that is, the normal condition prior to the first "northeaster" of the season. A pronounced salinity gradient is in evidence beneath and to the seaward side of the berm crest. The saline front is seen to move slightly seaward, at low water (1733 hrs., Fig. 2), from the previous high-water position (1200 hrs.). Most of the beach foreshore fluctuations occur either above or seaward of the salt front. Saltier ocean water infiltrates the berm and finds its way into the groundwater at high tide. The seaward-directed head gradient causes the front to move seaward as the tide falls.

The first northeast storm sent ocean water over the berm and onto the backshore. Groundwater salinity increased markedly (Fig. 2, 27 Aug. 69). Flushing of the saline water could be gaged by the distance moved by the 18% isoline -- about 8 m -- in the 87.5 hours between 1430 hours on 27 August and 0600 hours on 1 September.

MODEL FOR FORESHORE VOLUME CHANGE

Figure 3 shows the change in foreshore sand volume, over a half-tidal cycle, plotted as a function of the change in volume of groundwater in the beach. Both the change in groundwater volume and the change in foreshore sand volume were computed from measured changes in cross-sections plotted for low-tide and high-tide conditions. The groundwater volume change was for a unit distance along the shore that extended landward from the foreshore for a distance of about
FIG. 2. Salinity structure of the beach groundwater before (17 Aug. 69) and after (27 Aug. 69) storm wave flooding of backshore.
33 m, to well no. 6 (Fig. 1). The "foreshore" for the Ft. Story data extended from the limit of uprush at high water to the significant change in slope (at the toe of the foreshore) that is visible at low water. For the Camp Pendleton data, mentioned below, the inshore margin of the breaking waves was taken as the seaward limit of the foreshore. (Thus, absolute spatial location of the foreshore changed somewhat with each tide cycle).

As indicated by Figure 3, there is a clear relationship that can be stated: "as the volume of groundwater increases (water table rises), the foreshore is depleted of sand; as it decreases (water table falls), sand is deposited on the foreshore." The relationship is clear but the underlying process-response mechanism is not.

The groundwater fluctuations in the Ft. Story and Camp Pendleton beaches were produced by fluctuations of the ocean still water level (SWL) and, intuitively, one would expect changes in the quantity of foreshore sand (ΔQf) to be determined, to a large extent, by SWL oscillations.

In modeling the relationship between ocean level and foreshore volume change it is desirable to work with a minimum number of variables, each of which has intuitive physical significance. Imagine a perfectly calm sea whose surface is rising and falling with the frequency of a diurnal tide and amplitude of 1 m. The waveless, slowly-fluctuating sea surface will transfer no grains up or down the foreshore owing to the absence of swash-backwash. The beach water table will rise and fall with tidal periodicity, however.

Now let the energy of breaking waves and swash currents become available for grain transport. As the tide plane falls progressively from its highest level, one would expect sand to be put into suspension in the breaker zone and transported up the foreshore by each successive swash. Much of the sand transported up the foreshore would be deposited because it takes less energy to keep a grain in motion than it does to erode one; that is, the breaker zone is the primary energy source for grain suspension, and the swash zone is the region of upslope and downslope transfer. The still-water level determines the locus of sand deposition by the swash. Throughout a falling tidal half-cycle, therefore, sand will be deposited on the foreshore. The foreshore profile will rise relative to what it was at the time of high water. (At high
FIG. 3. Change in volume of foreshore sand at Ft. Story versus change in groundwater volume over a rising or falling half-tidal cycle.
water, the breaker zone will be located part way up the foreshore and will lower the profile by sending sand both land and seaward from the zone of maximum dissipation of turbulent energy).

As the still-water level rises from its lowest position, the breaker zone moves up the foreshore. This results in a net lowering of the foreshore profile because, even though some of the eroded foreshore sand is moved upslope and redeposited, a like portion is lost to the offshore side of the breaker zone. Owing to the width of the breaker zone, sand moving to its offshore side will, by the definition of "foreshore" adopted here, be lost from the foreshore profile.

The foregoing describes a symmetry of sand transfer that has been alluded to by others (LaFond, 1939; Strahler, 1964; and Schwartz, 1967) and, indeed, if $\Delta Q_f$ is plotted as a function of the change in ocean SWL, a remarkably symmetrical relationship emerges. The Camp Pendleton data foreshore volume change ($\Delta Q_f$) are plotted in Figure 4 as a function of change in tide level, over a half-tidal cycle. As expected, foreshore depletion occurs almost exclusively during the rising half-tidal cycle; foreshore deposition occurs during the falling half-cycle. The Ft. Story data (Fig. 4) reinforce this finding, the only significant exception to the rule being the large loss of foreshore sand that occurred during a falling half-tidal cycle that coincided with the first storm-wave over-topping of the summer berm. For the rising-tide condition, fully 84 percent of the data from both beaches conform to the rule; 89 percent of the data conform for the falling-tide condition.

Strahler studied an equilibrium beach at Sandy Hook, N.J., and observed (1964, Fig. 9) essentially the same relationship in a series of consecutive beach profiles made over an entire tidal-cycle. The fundamental relationship between ocean SWL fluctuations and foreshore sand-volume changes holds not only for an equilibrium beach, however. As indicated here (Fig. 4), it holds for sandy beaches undergoing overall retreat or advance, for waves as diverse as locally-generated, steep storm waves or long-period hurricane forerunners, and for beaches differing somewhat in mean grain-size and mean foreshore slope. The basic data for the two beaches studied, obtained in August and September of 1966 and 1969, are found in time-series reports for Ft. Story (Harrison and Fausak, 1970) and Camp Pendleton (Harrison, et al., 1968).
FIG. 4. Change in volume of foreshore sand at Ft. Story and Camp Pendleton (Fig. 1) versus change in ocean still-water level over a rising or falling half-tide cycle.
The relationship of the oceanic SWL to the outcrop of the water table bears special attention. If the SWL is relatively high, the uprush portion of the swash may percolate into the unsaturated sand above the water table outcrop, losing potential energy as it infiltrates. The backwash will not have the same kinetic energy, therefore, as the uprush, leading to sand deposition on the upper foreshore. If the SWL is relatively low, the seaward-directed head gradient will produce lift forces on the sand grains of the lower foreshore, downslope of the outcrop of the water table. These lift forces will assist the backwash in downslope transport of sand.

REGRESSION ANALYSIS

Quantification of the relationship between the ocean SWL and the water-table outcrop is difficult, owing to the problems related to maintenance of wells in the swash zone during storms. It will be assumed that the difference in the head (Fig. 5, Δy) between the two seawardmost wells that remained undisturbed throughout the study (nos. 10 and 11, Fig. 1), is the best descriptor of this relationship. It will also be assumed that Δy adequately reflects a) the swash percolation at high ocean SWL and b) the hydraulic forces exerted on the lower foreshore by the seaward-flowing groundwater at low ocean SWL.

An additional consideration now enters the picture. The foreshore surface moves landward and seaward, relative to the seawardmost well, over several tidal cycles. Δy, being sensitive to the distance from the foreshore at which it is measured, must be corrected for variable x (Fig. 5). The following "head index" will be used, therefore: \( l = (Δy) x \), or \( l = (y_{10} - y_{11}) x \), where \( y_{10} \) and \( y_{11} \) are the MSL elevations of the water table in wells 10 and 11, in mm, and \( x \) is in meters. When \( y_{11} > y_{10} \), Δy is negative (landward slope).

The following expressions are now formulated for sequential (linear) multiregression:

\[
ΔQ_f = f(I_1, I_2, I_3, 2h_o, S, \overline{H}_b)_R \quad (1)
\]

\[
ΔQ_f = f(I_3, I_4, I_5, 2h_o, S, \overline{H}_b)_F \quad (2)
\]

\[
ΔQ_f = f(I_1, I_2, I_3, I_4, I_5, Δh, S, \overline{H}_b)_C \quad (3)
\]
FIG. 5. Definition sketch for variables d, $h_o$, x, y, and $\Delta y$. 
where \( \Delta Q_f \) = change in quantity of foreshore sand (m\(^3\)), \( I \) = the index of head in the beach water table between wells 10 and 11, \( \pm 2h_0 \) is the range in rise or fall of the ocean still-water level over a rising or falling half-tidal cycle (in cm), \( \Delta h \) is the net change in ocean still-water level over a complete tide cycle (in cm), \( S \) = the total number of swash events over the interval of interest, \( H_b \) = the mean breaker height (in m) over the interval, the subscripts \( R \), \( F \), and \( C \) refer to rising, falling, and complete (low-water to low-water) tidal cycles, and the subscripts 1 through 5 denote times during a tidal cycle corresponding to initial low water, rising half-tide level, high water, falling half-tide level, and final low water, respectively. Total swash events \( S \) are assumed equal to the number of breakers occurring over the time interval.

The results of the regression analyses are given in Table 2. They show that, although the groundwater head index \( (I) \) is overshadowed by the change in ocean SWL \( (2h_0 \) or \( \Delta h \)), the head index at various times is, nevertheless, a strong predictor of \( \Delta Q_f \). Total swash \( (S) \) is the next strongest predictor after \( I \). Somewhat unexpectedly, \( H_b \) turned out to be the weakest of all predictors.

To the author's knowledge, a predictor equation such as (10) has never before been tested on field data obtained from a different beach, when the data from both beaches were gathered using identical measurement techniques.

Predictor equations for the regressions follow:

**Rising half-tide cycle:**

\[
\Delta Q_f = 0.1758 - 0.015193(2h_0) \tag{4}
\]

\[
\Delta Q_f = 1.633 - 0.017471(2h_0) - 0.00040402(S) \tag{5}
\]

\[
\Delta Q_f = 2.935 - 0.014474(2h_0) - 0.00052881(S) - 0.0016273(I_1) \tag{6}
\]

**Falling half-tide cycle:**

\[
\Delta Q_f = 0.0479 - 0.015193(2h_0) \tag{7}
\]

\[
\Delta Q_f = 0.4209 - 0.022052(2h_0) + 0.00060146(I_3) \tag{8}
\]

\[
\Delta Q_f = 3.360 - 0.03484(2h_0) + 0.0011271(I_3) + 0.00056068(S) \tag{9}
\]
### TABLE 2. Selection of Predictors by Regressing Equations 1, 2, and 3.

<table>
<thead>
<tr>
<th>Condition</th>
<th>N</th>
<th>Regression</th>
<th>Variable</th>
<th>Sign of correlation</th>
<th>R²</th>
<th>F Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rising half-tide cycle</td>
<td>38</td>
<td>1</td>
<td>2ho</td>
<td>-</td>
<td>0.39</td>
<td>6.47</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>S</td>
<td>-</td>
<td>0.51</td>
<td>6.30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>I₁</td>
<td>-</td>
<td>0.58</td>
<td>*</td>
</tr>
<tr>
<td>(Equ. 1)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Falling half-</td>
<td>34</td>
<td>1</td>
<td>-2ho</td>
<td>+</td>
<td>0.30</td>
<td>3.21</td>
</tr>
<tr>
<td>tide cycle</td>
<td></td>
<td>2</td>
<td>I₃</td>
<td>+</td>
<td>0.36</td>
<td>*</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>S</td>
<td>+</td>
<td>0.45</td>
<td>*</td>
</tr>
<tr>
<td>(Equ. 2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Complete tide cycle</td>
<td>27</td>
<td>1</td>
<td>Δh</td>
<td>-</td>
<td>0.55</td>
<td>10.60</td>
</tr>
<tr>
<td>(Equ. 3)</td>
<td></td>
<td>2</td>
<td>I₂</td>
<td>+</td>
<td>0.67</td>
<td>*</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>S</td>
<td>-</td>
<td>0.68</td>
<td>*</td>
</tr>
</tbody>
</table>

*Not valid because of interdependence of 2ho or Δh, and I.
Complete tide cycle:
\[ \Delta Q_f = 0.16682 - 0.051067(\Delta h) \]  \hspace{1cm} (10)
\[ \Delta Q_f = 0.019762 - 0.04857(\Delta h) + 0.0021636(I_2) \]  \hspace{1cm} (11)

To the author's knowledge, a predictor equation such as (10) has never before been tested on field data obtained from a different beach, when the data for both beaches were gathered using identical measurement techniques. This is possible now because the Ft. Story data (Harrison and Fausak, 1970) for \( \Delta Q_f \), \( 2h_0 \), and \( S \), from which equation (10) was developed, were obtained with the same measurement techniques as were used at Camp Pendleton (Harrison, et al., 1968), some 9 km to the south (Fig. 1).

\( \Delta h \) emerges as a most significant predictor for sandy beaches like the one studied because \( R^2 \) for the regression of \( \Delta Q_f \) predicted by (10), on the observed values of \( \Delta Q_f \), is 0.40, where \( N = 30 \). That is, equation (10), developed from the Ft. Story data, and explaining 55 percent of the variation in \( \Delta Q_f \) there, explains 40 percent of the total variation observed in \( \Delta Q_f \) at Camp Pendleton. Mean sand-grain diameter at the two beaches is approximately the same.

**DISCUSSION AND CONCLUSIONS**

Schwartz (1967) analyzed Bruun's (1962) thesis that sea-level rise is the main cause of shore erosion and, after applying his findings (Schwartz, 1968) to three different time and space scales, concluded that sea-level rise is the "common denominator underlying shore erosion on any scale." The multiregression analyses of the present study confirm Schwartz's finding for the rising half-tide situation on sandy beaches; that is, \( +2h_0 \) is the strongest predictor (Table 2) of \( \Delta Q_f \), and \( \Delta Q_f \) is nearly always negative (Fig. 4) when \( 2h_0 \) is positive. The regression analyses also suggest that \( -2h_0 \), falling SWL, is the major variable determining foreshore buildup, on the time scale of 6 to 12 hours.

As mentioned previously, this symmetry of foreshore volume change in response to oscillating SWL had been noticed by many others before Schwartz.
(See Schwartz's thorough summary in his 1968 paper). As also mentioned above, however, oscillation of the oceanic SWL, by itself, will not result in beach volume changes. Energy must be available for transport of sand grains and, as a further condition, if the index for available energy is taken as the number of swash events \( S \) per unit time, then the diminution of augmentation of that swash energy, by swash percolation into the upper foreshore or groundwater flow through the lower foreshore, must also be taken into account.

Equations 4—9, therefore, are more adequate statements of the processes occurring over a rising or a falling half-tidal cycle than are statements based upon fluctuating SWL alone. Also, the strength (in \( R^2 \)) of each of the empirical predictor equations is greater than the strength of those previous predictor equations (cf. Harrison, 1969), developed from various other independent variables in the beach-ocean-atmosphere system. Thus, some progress has been made in the attempt to reduce to the lowest possible number the independent variables influencing \( \Delta Q_f \) in a significant way. The result of testing equation (10) on an independent set of data indicates that \( \Delta Q_f \) can be reasonably well predicted, over a tidal-cycle interval, knowing only \( \Delta h \). It was unfortunate that variable \( I_2 \) (Table 2 and equ. 11) could not be determined accurately from the Camp Pendleton time series. Had this been possible, equation 11 could have been tested on the independent data from the Camp Pendleton beach study.

The best test of an empirical equation that contains interdependent variables \( (X_i) \) is to apply it to a set of independent data and examine the strength (in \( R^2 \)) of the resulting distribution of observed versus predicted values.

It is hoped that future studies of the dynamics of sandy tidal beaches will be so designed that the variables of Table 1 can be quantified in the same fashion as was done for the present study. In this way, the equations can be tested and improved.
ACKNOWLEDGMENTS

I thank Maurice L. Schwartz and Robert J. Byrne for critical review of the manuscript. Paul Bullock computed the beach volume changes at Camp Pendleton and Mark Stewart made the groundwater salinity determinations at Ft. Story.

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GROUNDWATER FLOW IN A SANDY TIDAL BEACH.
1. ONE-DIMENSIONAL FINITE-ELEMENT ANALYSIS

W. Harrison, C. S. Fang, and S. N. Wang

ABSTRACT

A 31-day-long time series of observations of beach water-table and tidal fluctuations was obtained from 13 wells along a profile perpendicular to the shoreline at Virginia Beach, Virginia. Finite-element techniques were applied to solve the one-dimensional, unsteady-state, nonlinear equation for groundwater movement. For the finite-element analysis, the semi-infinite mass (unconfined aquifer) had to be replaced by a finite mass. The boundary conditions were found from the field data by directly solving the flow equation with a finite-difference technique. The finite-element method, utilizing the variational principle, provided a reasonable solution and afforded economy in computer time. Field data were compared with the corresponding finite-element solution. Results indicate general accuracy of the methodology.

INTRODUCTION

Several workers (e.g., Grant, 1948; Emery and Foster, 1948; and Duncan, 1964) have shown, in a qualitative way, the importance of the slope of the beach water table, and its elevation above tide level, to the stability of sandy foreshore slopes. The goal of this work was to carefully document fluctuations in a beach water table and then develop models of the fluctuations for application to the foreshore stability problem. The present study was directed toward determining the feasibility of a one-dimensional model, using finite-element techniques.

The finite-element method of analysis was developed by the aircraft industry about 15 years ago to calculate stress and strain in complicated aircraft structures. It was not extensively applied to engineering or mechanical problems until two textbooks—the first edited by Zienkiewicz and Holister and the second authored by Zienkiewicz and Cheung (1967)—became available. These furnished considerable background, application, and research material on the finite-element method.
Extension of the method to cover unsteady-state, free-surface flow has been slow in development, with difficulties encountered in the case of triangular elements. The primary limitation of the finite element method is that it requires access to large computers for solutions to problems of any degree of complexity. Where such computers have been made available, the finite-element method has been widely applied in a variety of engineering fields. The method was first applied to the analysis of fluid flow in porous media by Taylor and Brown (1967). Recently, Neuman and Witherspoon (1970) applied the method to the problem of transient groundwater flow. Guymon (1970) applied finite-element methods to solve the one-dimensional, unsteady, diffusion-convection equation. The numerical technique employed variational principles, and triangular elements were very helpful in that study.

A 31-day-long time series of observations of the variables listed in Table 1 was obtained to document pertinent interactions in the beach-ocean-groundwater system. Thirteen wells for monitoring the water table (Fig. 1, 1-13) and 26 pipe-stations (Fig. 1, A-Z) for monitoring changes in beach elevation were positioned along an 83-m-long transect oriented perpendicular to the shoreline and extending from the edge of the foredune to the low-water line. Four multi-tube probes (Fig. 1, I-IV) were installed for extracting small amounts of groundwater for tests of salinity and to facilitate dye tests of flow characteristics. The wells consisted of 1) a #18-slotted PVC pipe, 102-mm in diameter, jetted into the beach to a depth of 3.5 m and 2) a 32-mm-O.D. steel pipe jetted to a depth of 5 m and touching the PVC pipe. A float-pulley system mounted on the pipes drove a potentiometer which provided a DC output voltage that corresponded (linearly) to the instantaneous water level. Water-table elevations were recorded at the site on computer-compatible magnetic tape (after A/D conversion). Details for this and the other measurement systems, as well as the complete time-series of measurements for the variables of Table 1, appear in Harrison and Fausak (1970).

The gently-sloping, quartz-sand beach that was studied (Fig. 1, "study site"), has a representative porosity of about 34 percent (median grain diameter of pit samples ranged between 0.37 and 0.59 mm). The mean range of the astronomical tide is about 0.85 m for Cape Henry, the spring range is 1.1 m, and the tide is semidiurnal with a slight diurnal inequality.
TABLE 1. Measured and Derived Variables for the Beach-Ocean-Groundwater System at Ft. Story, Virginia Beach, Virginia, for the Period 10 August through 9 September, 1969.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Sampling Frequency</th>
<th>Range in Values</th>
<th>Estimate of Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_b$</td>
<td>Elevation of beach surface</td>
<td>High, low, and</td>
<td>-0.940 to 3.693 (MSL)</td>
<td>±0.005 m above water, ±0.020 m below water</td>
</tr>
<tr>
<td></td>
<td></td>
<td>mid-tide levels</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$E_t$</td>
<td>Elevation of tidal plane</td>
<td>Continuous</td>
<td>-0.55 to 1.27 m (MSL)</td>
<td>±0.05 m</td>
</tr>
<tr>
<td>$E_w$</td>
<td>Elevation of water table</td>
<td>Either every 10 or every 15 minutes</td>
<td>0.291 to 1.999 m (MSL)</td>
<td>±0.003 m</td>
</tr>
<tr>
<td>$H_b$</td>
<td>Mean height of 50 successive breaking waves</td>
<td>High, low, and</td>
<td>0.19 to 1.30 m (MSL)</td>
<td>±0.10 m</td>
</tr>
<tr>
<td></td>
<td></td>
<td>mid-tide levels</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$m$</td>
<td>Slope of the foreshore</td>
<td>Derived (high, low, and mid-tide levels)</td>
<td>4.0° to 11.0°</td>
<td>±0.5°</td>
</tr>
<tr>
<td>$p$</td>
<td>Barometric pressure</td>
<td>Continuous</td>
<td>29.550 to 30.285 in. of Hg.</td>
<td>±0.005 in Hg.</td>
</tr>
<tr>
<td>$r$</td>
<td>Rainfall</td>
<td>Hourly during storms</td>
<td>Trace to 2.22 cm</td>
<td>±1.0 mm</td>
</tr>
</tbody>
</table>

(Continued)
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
<th>Sampling Frequency</th>
<th>Range in Values</th>
<th>Estimate of Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>s</td>
<td>Position of swash limit</td>
<td>Hourly, and at high, low, and mid-tide levels</td>
<td>Sta. G to Sta. Z (Fig. 1)</td>
<td>±0.25 m</td>
</tr>
<tr>
<td>$\bar{T}_b$</td>
<td>Mean period of 50 successive breaking waves</td>
<td>High, low, and mid-tide levels</td>
<td>3.96 to 13.20 sec.</td>
<td>±0.02 sec.</td>
</tr>
</tbody>
</table>
FIG. 1. Plan and profile views of the Fort Story Study site (inset), showing typical beach and water-table profiles, the spatial distribution of profile stations (A-Z), water-table monitoring wells (1-13), and groundwater sampling probes (I-IV).
Low, long-period swells characterized the first ten days of the 30-day study period. They were followed by three days of high, short-period waves that, combined with a storm surge, resulted in flooding of the backshore (Fig. 1) for two full tidal cycles. (Nine days were required for the groundwater level to return to its pre-storm position). The remaining 17 days of the study had a wave climate similar to the first 10 days. Rainfall amounted to 10.2 cm during the 30-day period. The greatest hourly rainfall was 2.22 cm, an event that caused an increase of about 1.5 cm in the water level at all wells. The effects of rainfall, as well as sudden fluctuations in atmospheric pressure, can be ignored in what follows.

WATER TABLE RESPONSE CHARACTERISTICS

It is commonly assumed (see Chow, 1964, p. 13-37) that for an unconfined aquifer connecting with the ocean, the tide wave will damp exponentially as it is propagated inland, so that, if the water table fluctuations are small relative to the saturated thickness, the amplitude \( y \) at any distance \( x \) inland is

\[
y = h_0 e^{-x(\pi S/\tau_0 T)^{\frac{1}{2}}} \sin \left( \frac{2\pi t}{\tau_0} - x \cdot \frac{\pi S}{\tau_0 T} \right)
\]

where \( y = h_0 \sin \omega t \) at \( x = 0 \), and \( y = 0 \) at \( x = \infty \); \( \tau_0 = \) tide period, \( T = \) coefficient of transmissibility, \( t = \) time, and \( S = \) the storage coefficient. A seaward-directed head gradient is almost invariably present in beach aquifers and the beach water table is also affected by tidal forces. Field data indicate that these two factors significantly affect water-table fluctuations. The overall effect is for the water table to rise too rapidly during rising tide and fall slowly during falling tide. Equation 1 cannot be used, therefore, to model the tide-wave response of the water table in a natural beach.

To gain insight into the response characteristics of the beach water table Fausak (1970) performed the following linear multiregression analysis on the water table elevation data for each well:

\[
2y = f(2h_0, d, x, p)
\]
The results of the regression analysis (Fig. 3) indicate the significance of the tidal forcing function throughout most of the 50-meter width of instrumented beach. As could be expected, the variable x, or the distance of a well from the foreshore, is the most significant factor in water table fluctuations for wells closest to the ocean. The regression analysis suggests that positive water table fluctuations in wells 12 and 13, closest to the shore, are as strong a function of x and d as they are of the rise in the tide, $2h_0$. At the landwardmost well (Fig. 3, no. 1), the increase in water level is as much a function of p as it is of $2h_0$. In general, however, Figure 3 indicates that we may feel confident in using the tidal fluctuation as the primary forcing function when attempting to model water table fluctuations between wells 1 and 13. A certain amount of noise (unexplained variability) will be present in the output of any model, especially for the seawardmost end of the water table, due to the unaccounted effects of the swash and the variable distance of the wells from the foreshore surface as sand is eroded and deposited. These variables contribute noise which is independent of that due to the two factors mentioned earlier. The total noise contributed by all of the foregoing effects will be lumped into the "drainage velocity," V, in what follows. This is because the groundwater head is the most significant factor of all of the noise effects.

GROUNDWATER FLOW EQUATION

A differential equation of groundwater movement was derived in accordance with the following assumptions:

1) the flow is one-dimensional, 2) the density of the fluid is constant, 3) Darcy's law is valid, 4) the groundwater occurs in a homogenous sand body, and
FIG. 2. Definition sketch for variables $y$, $x$, $d$, $h_o$, and $p$ used in the regression analysis.
FIG. 3. Relative importance of $2h_0$, $x$, $d$, and $p$, as indicated by percent reduction in sums of squares ($R^2$) of $2y$ (after Fausak, 1970, fig. 8).
5) the beach sand drains instantaneously.

Based on the principle of conservation of mass, the continuity equation of groundwater within a homogenous sand body can be written as

\[- \frac{\partial}{\partial x} (K_h \frac{\partial h}{\partial x}) + f \frac{\partial h}{\partial t} - V = 0 \]  \hspace{1cm} (3)

where \(V = V(x,t)\) is a function describing the bottom mass flux of the groundwater

\(x = \) direction normal to the shoreline
\(f = \) porosity
\(t = \) time
\(K = \) hydraulic conductivity
\(h = \) total head

Applying the variation principle to equation (3), for conditions at a particular instant, the functional of the minimizing function of \(h\) for each element is:

\[ I_m = \int_{x_n}^{x_{n+1}} \left[ \frac{1}{2} K_h \left( \frac{\partial h}{\partial x} \right)^2 + \left( f \frac{\partial h}{\partial t} - V \right) h \right] dx \] \hspace{1cm} (4)

for all inner elements, and

\[ I_1 = \int_{x_1}^{x_2} \left[ \frac{1}{2} K_h \left( \frac{\partial h}{\partial x} \right)^2 + \left( f \frac{\partial h}{\partial t} - V \right) h \right] dx \] \hspace{1cm} (5)

\[ + \left[ K_h \left( \frac{\partial h}{\partial x} \right) \delta h \right]_{x=0} \]

and

\[ I_M = \int_{x_N}^{x_{N+1}} \left[ \frac{1}{2} K_h \left( \frac{\partial h}{\partial x} \right)^2 + \left( f \frac{\partial h}{\partial t} - V \right) h \right] dx \] \hspace{1cm} (6)

\[ - \left[ K_h \left( \frac{\partial h}{\partial x} \right) \delta h \right]_{x=L} \]

for boundary elements.

In deriving the above equations, the assumption was made that equation (3) could be treated as a Sturm-Liouville problem (Weinstock, 1952), when applying the variational principle; i.e., \(K_h\) in the
term \( \frac{\partial^2}{\partial x^2} (K_h \frac{\partial h}{\partial x}) \) is taken as function of \( x \) only (Volker, 1969).

The boundary was applied to the prescribed value of \( h \) for each time step in this study; therefore, equations (5) and (6) are identical to equation (4).

The general concept of the method is to imagine the surface subdivided into a group of subassemblies or elements which are interconnected only at the element joints. Thus, the 1-dimensional region is divided into many subregions (Fig. 4).

By assuming that the linear function of \( h \) passes through two end points of each element, and representing equation (4) in local-element coordinates, the contribution over the \( m \)th element for the (\( n-1 \))th node is given by

\[
\frac{\partial I^m}{\partial h_{n-1}} = \frac{3k^m}{4y^m} h_{n-1}^2 - \frac{k^m}{2y^m} h_{n-1} h_n - \frac{k^m}{4y^m} h_n^2
\]

\[
+ \frac{f^m y^m}{3} \frac{\partial h_{n-1}}{\partial t} + \frac{f^m y^m}{6} \frac{\partial h_n}{\partial t} - \frac{v^m y^m}{2}
\]

where \( y^m \) is the length of the \( m \)th element, subscripts indicate nodal counters, and superscripts indicate element counters.

Considering the over-all contribution on nodal point \( n \) due to all elements, and setting equal to zero:

\[
\frac{\partial I}{\partial h_n} = \frac{\partial}{\partial h_n} \left( \sum_{m=1}^{M} r^m \right) = \sum_{m=1}^{M} \frac{\partial I^m}{\partial h_n} = 0
\]

That is:

\[
\frac{\partial I^{m+1}}{\partial h_n} + \frac{\partial I^m}{\partial h_n} = 0
\]
FIG. 4. Definition sketch for elements and nodes.
Substituting equation (8) into equation (9), and expressing the results in matrix form,

\[
\begin{bmatrix}
- \frac{k^m}{4y^m} \left( \frac{3k^m}{4y^m} + \frac{3k^{m+1}}{4y^{m+1}} \right) & - \frac{k^{m+1}}{4y^{m+1}} \\

\end{bmatrix}
\begin{bmatrix}
h_{n-1}^2 \\

h_n^2 \\

h_{n+1}^2 \\

\end{bmatrix}
\]

\[
+ \begin{bmatrix}
- \frac{k^m}{2y^m} & - \frac{k^{m+1}}{2y^{m+1}} \\

\end{bmatrix}
\begin{bmatrix}
h_{n-1} \\
h_n \\
h_{n+1} \\

\end{bmatrix}
\]

\[
+ \begin{bmatrix}
\frac{\varphi y^m}{6} & \frac{\varphi y^{m+1}\varphi y^{m+1}}{3} \\

\frac{\varphi y^{m+1}\varphi y^{m+1}}{6} \\

\end{bmatrix}
\begin{bmatrix}
\frac{\partial h_{n-1}}{\partial t} \\

\frac{\partial h_n}{\partial t} \\

\frac{\partial h_{n+1}}{\partial t} \\

\end{bmatrix}
\]

\[
+ (-v^m y^m/2 - v^{m+1} y^{m+1}/2) = 0
\]

Equation (10) represents a system of non-linear functions of \( h_{n-1}, h_n, h_{n+1} \) for every node.

In order to handle easily, equation (10) is defined as:

\[
f = (h_2, h_3, \ldots, h_{n-1}, h_n, h_{n+1}, \ldots, h_N) = 0 \quad (11)
\]

where \( n = 2, 3, 4, \ldots, N \), but \( n \neq 1 \), and \( n \neq N+1 \).

Thus:

\[
f_n = A_1 h_{n-1}^2 + A_2 h_n^2 + A_3 h_{n+1}^2 + A_4 h_{n-1} h_n \\
+ A_5 h_n h_{n+1} + A_6 h_{n-1} + A_7 h_n + A_8 h_{n+1} + A_9 = 0 \quad (12)
\]
Relating equations (10) and (12), the coefficients (A's) are:

\[ A_1 = -\frac{k^m}{4y^m} \]
\[ A_2 = 3\frac{k^m}{4y^m} + 3\frac{k^{m+1}}{4y^{m+1}} \]
\[ A_3 = -\frac{k^{m+1}}{4y^{m+1}} \]
\[ A_4 = -\frac{k^m}{2y^m} \]
\[ A_5 = -\frac{k^{m+1}}{2y^{m+1}} \]
\[ A_6 = \frac{f^m y^m}{6\Delta t} \]
\[ A_7 = (\frac{f^m y^m}{3} + \frac{f^{m+1} y^{m+1}}{3}) \frac{1}{\Delta t} \]
\[ A_8 = \frac{f^{m+1} y^{m+1}}{6\Delta t} \]
\[ A_9 = (-\frac{v^m y^m}{2} - \frac{v^{m+1} y^{m+1}}{2}) \]

\[ -\frac{f^m y^m}{6\Delta t} h_{n-1} \]
\[ - (\frac{f^m y^m}{3} + \frac{f^{m+1} y^{m+1}}{3}) \frac{h_n}{\Delta t} \]
\[ -\frac{f^{m+1} y^{m+1}}{6\Delta t} h_{n+1} \]

**NUMERICAL ANALYSIS**

**Basic steps.** - The basic steps in the formulation of the finite-element method can be summarized as follows:

1) development of the element-equation coefficient matrix,
2) generation of the matrix for the entire system,
3) calculation of nodal head values due to imposed forces and boundary conditions, and
4) calculation of the water-table elevation in each element, from the nodal displacement.

The Newton-Raphson method was chosen to solve the system of nonlinear equations given by equation (10). Fang (1968) has given a detailed description of this method. Only a brief discussion will be given here.
Denote the kth approximation of iteration for the actual roots \([h_i] \ i = 1, 2, \ldots, I\), as \([h_i^{(k)}]\), then equation (11) may be expanded in Taylor series about \([h_i^{(k)}]\), truncating higher order other than first-derivative terms,

\[
F_i(h_i^{(k+1)}) = F_i(h_i^{(k)}) + \sum_{j=1}^{I} \frac{\partial F_i(h_i^{(k)})}{\partial h_j} [h_j^{(k+1)} - h_j^{(k)}] \tag{13}
\]

If \(h_i^{(k+1)}\) are found close to the actual root, then \(F_i(h_i^{(k+1)}) = 0\). Equation (13) represents a system of linear equations that can be expressed in matrix form as

\[
[J] \begin{bmatrix} h_{j}^{(k+1)} - h_{j} \end{bmatrix} = - \begin{bmatrix} F_i(h_i^{(k)}) \end{bmatrix} \tag{14}
\]

where \([J]\) is the Jacobian matrix.

The Jacobian matrix has the form

\[
[J] = \begin{bmatrix}
\frac{\partial F_1}{\partial h_1} & \frac{\partial F_1}{\partial h_2} & 0 \\
\frac{\partial F_2}{\partial h_1} & \frac{\partial F_2}{\partial h_2} & \frac{\partial F_2}{\partial h_3} \\
\frac{\partial F_3}{\partial h_1} & \frac{\partial F_3}{\partial h_2} & \frac{\partial F_3}{\partial h_3} \\
& \ddots & \ddots \\
& 0 & \frac{\partial F_{I-1}}{\partial h_{I-2}} & \frac{\partial F_{I-1}}{\partial h_{I-1}} & \frac{\partial F_{I-1}}{\partial h_I} \\
& & \frac{\partial F_{I}}{\partial h_{I-1}} & \frac{\partial F_{I}}{\partial h_I}
\end{bmatrix}
\]

It should be noted that each row of the Jacobian matrix contains, at most, three non-zero elements. For large \(I\), the matrix is sparse and one can take advantage of the large number of zeros. The modified
subroutine from the IBM System 360 subroutine package entitled SIMQ takes advantage of the large number of zeros in the matrix of coefficients by using the Gaussian elimination method which gives a rapidly convergent procedure with the generalized Newton-Raphson iteration method.

That is, all the non-zero elements of \([J]\) are placed on the diagonal region. There are only three non-zero elements for each row, except for the first and final row where there are two non-zero elements.

The iterative steps may be summarized as follows:

1) \([h_i(k)]\) is found. (It must be a close approximation to the root, otherwise the equation may not converge). For the first iteration, the value of the previous time step is used as an approximation; if \(k > 0\), the result of the previous iteration is used.

2) Compute

\[ \Delta h_j = h_j^{(k+1)} - h_j^{(k)} \]  

by solving the system of linear equations (14).

3) Find \(h_i^{(k+1)}\), using a reasonable convergent criterion.

**Boundary conditions.**—The prescribed values of \(h_i\) obtained from the field experimental data are applied to the boundary nodal points, \(n = 1\) and \(n = N+1\).

The most confused boundary condition occurs on the bottom side of the control region. It is difficult to handle the drainage velocity function, \(V(x,t)\), in proper mathematical form because this boundary is a function of both space and time, due to the combined effects of the tide, waves, foreshore changes, and rainfall.

**Fourier series analysis for drainage velocity.**—Assume that \(V(x,t)\) is constant over each interval \(x_i \leq x \leq x_{i+1}\), and can be calculated from actual field data by the Eulerian finite-difference formulation, where \(i = 1, 2, \ldots, 12\), and the \(x_i\) are the coordinates of the wells.
From the results of the Eulerian finite-difference method, we find that \( V(x,t) \), \( x_1 \leq x \leq x_{i+1} \), has much fluctuation from time to time. To simplify the Fourier series analysis, we take the time average first.

\[
\bar{V}(x,t) = \frac{1}{N} \sum_{n=1}^{N} V(x,t_n) \quad x_1 \leq x \leq x_{i+1} \quad t_1 \leq t \leq t_n \quad (16)
\]

In our example, we take 100 time cycles (1 cycle = 15 minutes) as the period. Taking the time average for each 20 cycles, three average values in each average interval give a more accurate Fourier series.

The average value of two different intervals is taken as the value at the point of discontinuity. We then find the Fourier series

\[
\bar{V}(x,t) = \sum_{n=1}^{N} a_n \cos \frac{2\pi nt}{T} + b_n \sin \frac{2\pi nt}{T} \quad (17)
\]

for each well interval. Results are given in Table 2. The Fourier series coefficients are used in the subroutine to find the drainage velocity for each time step.

APPLICATION OF THE METHOD

The analytical method was applied to the field data as follows (refer to Fig. 5). The prescribed values of \( h \) were applied at \( C \) and 0 for every time step. The drainage velocity \( V(x,t) \) was applied on the bottom boundary \( OC \). Also, the following data were used in the example: total nodes = 21, porosity = 34%, hydraulic conductivity = 0.014 cm/sec, length of each element = 250 cm, time increment for each step = 15 minutes.

RESULTS AND CONCLUSION

The finite-element method was applied to solve the nonlinear equation for treating the complicated case of beach water-table fluctuations. The finite-element method, based on the variational principle, provides an accurate solution with an economy of computer time. A compromise decision was made as to
TABLE 2. Fourier Series Coefficients for Each Well Interval. ($x_i$ = well coordinates; $i = 1, 2, ..., 13$).

<table>
<thead>
<tr>
<th>interval coeff.</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
</tr>
</thead>
<tbody>
<tr>
<td>$x_1 \leq x_2$</td>
<td>a 0.2060E-04</td>
<td>-0.2555E-04</td>
<td>0.3933E-04</td>
<td>-0.1161E-04</td>
<td>0.7523E-04</td>
<td>-0.2118E-05</td>
</tr>
<tr>
<td></td>
<td>b 0.0000E 00</td>
<td>-0.4224E-05</td>
<td>-0.1677E-05</td>
<td>0.6999E-05</td>
<td>-0.1689E-04</td>
<td>-0.9095E-05</td>
</tr>
<tr>
<td>$x_2 \leq x_3$</td>
<td>a 0.3689E-04</td>
<td>-0.1483E-04</td>
<td>0.7028E-04</td>
<td>-0.2182E-04</td>
<td>0.1646E-04</td>
<td>0.3183E-05</td>
</tr>
<tr>
<td></td>
<td>b 0.0000E 00</td>
<td>-0.6800E-05</td>
<td>-0.2002E-04</td>
<td>0.6440E-05</td>
<td>-0.2374E-04</td>
<td>-0.1631E-04</td>
</tr>
<tr>
<td>$x_3 \leq x_4$</td>
<td>a 0.3666E-04</td>
<td>-0.1003E-04</td>
<td>0.1005E-03</td>
<td>-0.4412E-04</td>
<td>0.2276E-04</td>
<td>0.3765E-05</td>
</tr>
<tr>
<td></td>
<td>b 0.0000E 00</td>
<td>-0.8123E-05</td>
<td>-0.5320E-04</td>
<td>0.1999E-05</td>
<td>-0.3054E-04</td>
<td>-0.2445E-04</td>
</tr>
<tr>
<td>$x_4 \leq x_5$</td>
<td>a 0.4971E-04</td>
<td>-0.9646E-05</td>
<td>0.1261E-03</td>
<td>-0.6833E-04</td>
<td>0.2999E-04</td>
<td>0.6248E-05</td>
</tr>
<tr>
<td></td>
<td>b 0.0000E 00</td>
<td>-0.7289E-05</td>
<td>-0.8863E-04</td>
<td>0.8633E-06</td>
<td>-0.3150E-04</td>
<td>-0.2679E-04</td>
</tr>
<tr>
<td>$x_5 \leq x_6$</td>
<td>a 0.6418E-04</td>
<td>-0.8761E-05</td>
<td>0.1453E-03</td>
<td>-0.9076E-04</td>
<td>0.3359E-04</td>
<td>0.6758E-05</td>
</tr>
<tr>
<td></td>
<td>b 0.0000E 00</td>
<td>-0.7440E-05</td>
<td>-0.1177E-03</td>
<td>-0.2665E-05</td>
<td>-0.3154E-04</td>
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<tr>
<td>$x_6 \leq x_7$</td>
<td>a 0.5345E-04</td>
<td>-0.7978E-05</td>
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<td>-0.1162E-03</td>
<td>0.3628E-04</td>
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<td>b 0.0000E 00</td>
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<td>-0.1470E-03</td>
<td>-0.2576E-05</td>
<td>-0.2888E-04</td>
<td>-0.2791E-04</td>
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<td>$x_7 \leq x_8$</td>
<td>a 0.7328E-04</td>
<td>-0.4882E-05</td>
<td>0.1783E-03</td>
<td>-0.1286E-03</td>
<td>0.4556E-04</td>
<td>0.1265E-04</td>
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<td>b 0.0000E 00</td>
<td>-0.1082E-05</td>
<td>-0.1770E-03</td>
<td>-0.8028E-05</td>
<td>-0.2667E-05</td>
<td>-0.2772E-04</td>
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<tr>
<td>$x_8 \leq x_9$</td>
<td>a 0.8153E-04</td>
<td>0.7425E-05</td>
<td>0.2038E-03</td>
<td>-0.1508E-03</td>
<td>0.5643E-04</td>
<td>0.1902E-04</td>
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<tr>
<td></td>
<td>b 0.0000E 00</td>
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<td>-0.2246E-03</td>
<td>-0.1760E-04</td>
<td>-0.2682E-04</td>
<td>-0.3179E-04</td>
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<td>$x_9 \leq x \leq 10$</td>
<td>a</td>
<td>0.1070E-03</td>
<td>0.1214E-04</td>
<td>0.2185E-03</td>
<td>-0.1908E-03</td>
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<td></td>
<td>b</td>
<td>0.0000E 00</td>
<td>-0.6813E-05</td>
<td>-0.2331E-03</td>
<td>-0.2555E-04</td>
<td>-0.1887E-04</td>
<td>-0.2813E-04</td>
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<td>$x_{10} \leq x \leq 11$</td>
<td>a</td>
<td>0.1245E-03</td>
<td>0.8425E-05</td>
<td>0.2222E-03</td>
<td>-0.2390E-03</td>
<td>0.5734E-04</td>
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<td>b</td>
<td>0.0000E 00</td>
<td>-0.1079E-04</td>
<td>-0.3439E-03</td>
<td>-0.2934E-04</td>
<td>-0.5645E-06</td>
<td>-0.1428E-04</td>
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<td>$x_{11} \leq x \leq 12$</td>
<td>a</td>
<td>0.1214E-03</td>
<td>0.2163E-04</td>
<td>0.2230E-03</td>
<td>-0.2915E-03</td>
<td>0.6012E-04</td>
<td>0.2998E-04</td>
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<tr>
<td></td>
<td>b</td>
<td>0.0000E 00</td>
<td>-0.2396E-04</td>
<td>-0.4261E-03</td>
<td>-0.3836E-04</td>
<td>0.2684E-04</td>
<td>0.5779E-05</td>
</tr>
<tr>
<td>$x_{12} \leq x \leq 13$</td>
<td>a</td>
<td>0.7925E-04</td>
<td>-0.7718E-04</td>
<td>0.1126E-03</td>
<td>-0.4094E-03</td>
<td>0.2125E-04</td>
<td>0.3222E-04</td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>0.0000E 00</td>
<td>-0.1256E-04</td>
<td>-0.4542E-03</td>
<td>0.3179E-04</td>
<td>0.1628E-03</td>
<td>0.1422E-03</td>
</tr>
</tbody>
</table>
FIG. 5. Definition sketch for application of finite-element model to field data.
the assumed position of the boundaries. In the finite-element analysis, the semi-infinite mass (unconfined aquifer) was replaced by a finite mass. The drainage velocities were found from the field data by directly calculating the pertinent differential equation with a finite-difference technique. Then, a Fourier function was used to describe the mean regional drainage-velocity characteristics and the beach water-table's response to the input tidal fluctuation. Comparing the results (Fig. 6) between this procedure and the field data indicates that the finite-element method is accurate enough to solve the problem of fluctuations of the beach ground-water table.

The fact that small differences exist between the field data and the theoretical results can be explained as due to the effects of variables x and d (equation 2) mentioned earlier. Also of importance will be effects due to capillarity and groundwater density gradients. (These effects possess a complex relationship to space and time). Density gradients in the groundwater will be due to variations in water temperature and salinity (cf. Jansson, 1967). As shown in Figure 7, the groundwater salinity for the summer beach (17 Aug. 69) ranged from near zero at well 3 to 26‰ (parts per thousand) at well 13 at time of high water (1200 hrs). At low tide (1733 hrs) the saline groundwater had migrated slightly seaward. At the end of a storm that sent ocean water over the berm and onto the backshore, however, salinity of the groundwater increased markedly (Fig. 7, 27 Aug. 69). (Flushing of the saline water can be gaged by the distance moved by the 18‰ isoline—about 8m—in the 87.5 hours between 1430 hrs on 27 August and 0600 hrs on 1 September). The point here is that a normal, horizontal density gradient in the beach groundwater changed to a vertical density gradient, as a result of storm flooding.

The procedure developed in this study can be applied to noneven elements and nonhomogenous materials. It makes it possible to predict water-table fluctuations in any sandy, two-dimensional tidal beach knowing only the drainage velocity (obtained from two wells) and the predicted fluctuations in ocean level.

The Fortran computer programs used for the one-dimensional analysis are described and listed in a special section following the References.
FIG. 6. Comparison between field data (solid circles) and computer results (open circles).
FIG. 7. Salinity structure of the beach groundwater before (17 Aug. 69) and after (27 Aug. 69) storm wave flooding of backshore.
ACKNOWLEDGMENTS

We thank Drs. John Bredehoeft and George Pinder for critical review of the manuscript.

REFERENCES


FORTRAN PROGRAMS FOR THE ONE-DIMENSIONAL, FINITE-ELEMENT ANALYSIS

Main Program & Subroutine DRANV

Following this description the reader will find the main program and a subroutine, DRANV, which was designed to find the bottom drainage velocity with Fourier coefficients for drainage velocity. Definitions of all of the variables can be found from the comment cards in the programs. Also included are comment cards describing the purposes of various sections of the programs. All data that are needed should be prepared in card form; no other input device is used. This program takes 2.38 minutes CPU time, per 100 cycles, in mode 50/IBM 360. Input cards are listed in the following groups:

1) First card: FORMAT(216, 5E12.4)

   (1) NPOIN: total number of nodal points
   (2) NELEM: total number of line elements
   (3) DT: time increment of time step
   (4) TL: total time to quit
   (5) XK: hydraulic conductivity
   (6) POROS: porosity
   (7) DX: length of even line elements

2) Second card: FORMAT(E12.4)

   (1) EPS: the convergent criteria of Newton-Raphson method

3) 3rd to 15th card: FORMAT(I6, 2E12.4)

   Each card contains three initial well's characteristics defined as:

   (1) I: integer counter to specify well number
   (2) XW(I): x-coordinate of well No. 1
   (3) YW(I): water table of well No. 1

4) 16th to (15+ NPOIN)th card: FORMAT(I10,12X,E12.4)

   Each card contains two initial nodal characteristics defined as:

   (1) I: integer counter to specific node number
   (2) Y(I): water tables of node I
5) $(16 + \text{NPOIN})^{th}$ to $(39 + \text{NPOIN})^{th}$ card: 
\text{FORMAT ( A7, 6E12.4) }

This group, containing 24 cards, can be thought of as 12 sets, each set consisting of two cards. Fourier cosine coefficients of drainage velocities are punched on the first card, which may be described as:

(1) \text{CDUMY: variables for A-field, to be identified as Fourier Cosine coefficients}
(2) \text{A(I,1): first Fourier cosine coefficient of well number I}
(3) \text{A(I,2): second Fourier cosine coefficient of well number I}
(4) \text{A(I,3): third Fourier cosine coefficient of well number I}
(5) \text{A(I,4): fourth Fourier cosine coefficient of well number I}
(6) \text{A(I,5): fifth Fourier cosine coefficient of well number I}
(7) \text{A(I,6): sixth Fourier cosine coefficient of well number I}

Fourier sine coefficients are punched on the second card which may be described as:

(1) \text{SDUMY: Variables for A-field, to be identified as Fourier sine coefficients}
(2) \text{B(I,1): first Fourier sine coefficient of well number I}
(3) \text{B(I,2): second Fourier sine coefficient of well number I}
(4) \text{B(I,3): third Fourier sine coefficient of well number I}
(5) \text{B(I,4): fourth Fourier sine coefficient of well number I}
(6) \text{B(I,5): fifth Fourier sine coefficient of well number I}
(7) \text{B(I,6): sixth Fourier sine coefficient of well number I}

6) $(29 + \text{NPOIN})^{th}$ card on: \text{FORMAT( 4F10.4) }

Each card contains left- and right-hand-side boundary conditions obtained from field data. The total number of cards in this group is equal to the total number of time steps desired:

(1) \text{Y(I): water table for left-hand-side boundary}
(2) \text{Y(NPOIN): water table for right-hand-side boundary}
(3) The third and fourth variable of each card are dummy variables.
C PROGRAM OF ONE-DIMENSIONAL UNDERGROUND WATER FLOW, SOLVED BY FINITE
ELEMENT METHOD WITH EVEN SPACING, AND NEWTON-RAPSHON METHOD, WRITTEN
BY SCL-NAN WANG, UNDER CNR GRANTS.

NPCINT=TOTAL NUMBER OF NODAL POINT
NLEPFT=TOTAL NUMBER OF FINITE ELEMENT
DT=TIME INCREMENT
TL=TIME TO QUIT
XK=HYDRAULIC CONDUCTIVITY
PCRCS=PCRCSITY

DX=THE LENGTH OF ELEMENT M
EPS=THE CONVERGENT CRITERIA OF NEWTON-RAPSHON METHOD

A1= CF THE TERM Y(N-1)**2 IN THE NONLINEAR FUNCTION F(N)=0
A2= CF THE TERM Y(N)**2 IN THE NONLINEAR FUNCTION F(N)=0
A3= CF THE TERM Y(N+1)**2 IN THE NONLINEAR FUNCTION F(N)=0
A4= CF THE TERM Y(N-1)*Y(N) IN THE NONLINEAR FUNCTION F(N)=0
A5= CF THE TERM Y(N)*Y(N+1) IN THE NONLINEAR FUNCTION F(N)=0
A6= CF THE TERM Y(N+1) IN THE NONLINEAR FUNCTION F(N)=0
A7= CF THE TERM Y(N) IN THE NONLINEAR FUNCTION F(N)=0
A8= CF THE TERM Y(N+1) IN THE NONLINEAR FUNCTION F(N)=0
A9=THE CONSTANT TERM IN THE NONLINEAR FUNCTION F(N)=0

F(I)=NONLINEAR FUNCTION F(N) AT EACH NODAL POINT

Y(I)=THE HEIGHT OF FREE SURFACE MEASURED FROM DATUM LINE AT NODES
YW(I)=THE HEIGHT OF FREE SURFACE MEASURED FROM DATUM LINE AT WELLS

XW(I)=X-COORDINATES OF EACH WELL

YCLC(I)=ORIGINAL VALUE OF Y(I) AT EACH TIME STEP FOR COMPARING
WITH EACH ITERATED VALUE OF Y(I)

NITER=COUNT OF TOTAL NUMBER OF ITERATION IN NEWTON-RAPSHON
METHOD AT EACH TIME STEP WHEN Y(I) REACHES CONVERGENCE

NLEPS=COUNT OF TOTAL NUMBER OF NODAL WHICH SATISFIES THE
CONVERGENT CRITERIA AT EACH ITERATION

YP(I)=EQUIVALENT TO Y(I)

NPRINT=CONTROL OF NUMBER OF PRINTING

A(I,J)=FOURIER COSINE COEFFICIENTS

B(I,J)=FOURIER SINE COEFFICIENTS

DIMENSION Y(21),YCLC(21),F(19),FDY(19,19),A9(21),V(20),
YP(21),YW(13),XW(13)
DIMENSION A(12,6),B(12,6)
DIMENSION VW(12)
COMMON DX,NPCII,A,B,VW
EQUIVALENCE (Y(1),YP(1))

IR=5
IW=6

1 FCRMAT(216,5E12.4)
2 FCRMAT(2X,6HNPCII=,I6,2X,6NLEPFT=,I6,2X,3HDT=,E12.4,2X,3HDX=,E12.4)
3 FCRMAT(110,12X,E12.4)
4 FCRMAT(E12.4)
5 FCRMAT(5X,6HNITER=,I6,2X,6NLEPS=,I6,2X,4HEPS=,E12.4)
6 FCRMAT(110,2X,E12.4)
7 FCRMAT(1H0,2X,2HT=,E12.4)
8 FCRMAT(4F10.4)
9 FCRMAT(16,5E12.4)
10 FCRMAT(16,E12.4)
11 FCRMAT(8X,1H1,6X,4+Y(I),9X,6HY(I+1),9X,6HY(I+2),9X,
WRITE(IW,21)
REAC(IR,1) NPCIN,NELEM,CT,TL,XK,PORCS,DX
WRITE(IW,2)NPCIN,NELEM,CT,TL,XK,PORCS,DX
REAC(IR,4) EPS
WRITE(IW,14)EPS
REAC(IR,16)(1,XW(I),YH(I),I=1,13)
DC 55 I=1,NPCIN
55 REAC(IR,3) I,Y(I)
WRITE(IW,27)
WRITE(IW,11)
NCIV=NPCIN/6
NMAX=NCIV*6
DC 56 I=1,NPCIN,6
IF(1-NMAX) 211,211,210
211 WRITE(IW,12)I,Y(I),Y(I+1),Y(I+2),Y(I+3),Y(I+4),Y(I+5)
GC TC 56
210 DC 212 J=I,NPCIN
212 WRITE(IW,13) J,Y(J)
56 CCNTINUE
NPC11=NPCIN-1
NPC12=NPCIN-2
DDT=1.0/CT
T=0.0
NCD=NPCI1/5
NMA=NCD*5
NMA1=NMA+1

KNCwING THE RELATION BETWEEN NUMBERING SYSTEM OF ELEMENTS AND
C
C NUMBERS, WE EXPRESS THE COEFFICIENTS OF NONLINEAR FUNCTION IN TERMS
C OF NUMBER SYSTEM OF NCCALS
C
A1=-0.25*XK/CX
A2=1.5*XK/CX
A3=A1
A4=2.0*A1
A5=A4
DCT=PR=0.166667*ECT*PORCS

-92-
A6 = CCTPR * CX  
A7 = 4.0 * A6  
A8 = A6  
DC 91 I = 1, NPCIN, NPCII
91 A9(I) = 0.0  
DC 58 I = 2, NPCII  
AA = 0.0  
BB = A6 * Y(I - 1)  
A9(I) = AA - BB  
AA = A8 * Y(I + 1)
58 A9(I) = A9(I) - BB - AA  
DC 61 I = 1, NPCI2  
DC 62 J = 1, NPCI2
62 FDY(I, J) = 0.0  
V(I) = C.0  
61 F(I) = 0.0  
V(NPCI1) = 0.0

C READ FOURIER COEFFICIENTS OF WELL DRAINAGE VELOCITY
C
DC 50 I = 1, 12  
READ (IR, 16) CLMY, A(I, 1), A(I, 2), A(I, 3), A(I, 4), A(I, 5), A(I, 6)  
50 READ (IR, 16) SLMY, B(I, 1), B(I, 2), B(I, 3), B(I, 4), B(I, 5), B(I, 6)  
WRITE (IW, 28)  
DC 290 I = 1, 12  
WRITE (IW, 18) CLMY, A(I, 1), A(I, 2), A(I, 3), A(I, 4), A(I, 5), A(I, 6)  
WRITE (IW, 18) SLMY, B(I, 1), B(I, 2), B(I, 3), B(I, 4), B(I, 5), B(I, 6)  
290 CONTINUE

C PRISCRIBE BOUNDARY OF Y AT EACH TIME STEP
C
NPRII=0
400 T = T + CT  
NITER=0  
YCLC(NPCIN) = Y(NPCIN)  
DC 116 I = 1, NPCII
116 YCLC(I) = Y(I)  
READ (IR, 8) Y(1), Y(NPOI), CYB1, DYB2  
NRECV = NRECV + 1  
IF (T - 2.0 * CT) 280, 280, 281
280 IF (NRECV - 1) 283, 282, 283
C FINISH DRAINAGE VELOCITY BY CALLING SUBROUTINE DRANW
282 CALL DRANW(T, XW, V)  
283 IF (NRECV - 2) 285, 284, 285  
284 NRECV = 0  
GC TC 285
281 IF (NRECV - 1) 287, 286, 287  
286 CALL DRANW(T, XW, V)  
287 IF (NRECV - 5) 285, 288, 285  
288 NRECV = 0  
285 CONTINUE  
DC 110 I = 2, NPCII
110 XV1 = V(I - 1)  
XV2 = V(I)  
AA = -0.5 * XV1 * CX - 0.5 * XV2 * CX  
C FINISH BEST FIRST APPROXIMATION VALUE SET FOR Y(I)
AA = Y(NPCI) - YCLC(NPCI)

XN = NPCI
BB = XN * CX
CC = I
DC = DC
II = I + 2, NPCI

250 Y(I) = YCLC(1) + AA * HE * XII * CX * C.5

C FIND RESIDUE AND JACOBIAN MATRIX

300 DC = 66
I = 2, NPCI


2 + AA * Y(I) + A8

FCY(I-1, I-1) = 2.0 * A9

FCY(I-1, I) = 2.0 * A10

FCY(I, I) = 2.0 * A11

GTC = 66

63 BB = 2.0 * A12

FCY(1, 2) = BB

64 CONTINUE

I = NPCI


* AA * Y(I) + A8

FCY(I-1, I-1) = 2.0 * A9

FCY(I-1, I) = 2.0 * A10

FCY(I, I) = 2.0 * A11

NDIV = NPCI / 6

NMAX = NDIV

CALL SIMC(FCY, F, NPCI, KS)

NLEPS = 0

DC = 71
I = 1, NPCI

A = ABS(F(I))

AA = AA / YCLC(I+1)

IF (AA - EPS) 72, 72, 71

72 NLEPS = NLEPS + 1

71 CONTINUE

DC = 76
I = 1, NPCI

Y(I+1) = Y(I+1) - F(I)

IF (NLEPS - NPCI) 77, 78, 77

77 CONTINUE

NITER = NITER + 1

IF (NITER < 5) 300, 400, 300

78 CONTINUE

C C FIND YW(I) CF WELLS FROM Y(I) BY LINEAR INTERPOLATION

C YW(I) = YP(I)

AA = YP(NPCI) - YP(NPCI)

YW(I) = YP(NPCI) - 77.9 * AA / CX

DC = 221
I = 2, 12

CC = 222
J = 1, NPCI

AA = XW(I)

FJ = J

X = FJ * CX

BB = X

IF (AA - BB) 223, 223, 222

223 AA = XW(I) - X + CX

BE = X - XW(I)

CC = AA * YP(J+1) + BB * YP(I)
YW(I) = CC/(AA + BB)
        GC TC 221
222 CONTINUE
221 CONTINUE
        NPRIN = NPRIN + 1
        IF(NPRIN - 10) 226, 225, 226
225 CONTINUE
        WRITE(IW,7) T
        WRITE(IW,5) NITER, NLEPS, EPS
        WRITE(IW,25)
        WRITE(IW,26)
        WRITE(IW,9) (I, YW(I), YW(I+1), YW(I+2), YW(I+3), YW(I+4), I = 1, 10, 5)
           I = 11
        WRITE(IW,15) I, YW(I), YW(I+1), YW(I+2)
        NPRIN = 0
226 CONTINUE
        IF(T - TL) 201, 202, 202
201 CONTINUE
        DC B5 I = 2, NPCI1
           AA = 0.0
           BB = A6*Y(I - 1)
           A9(I) = AA - BB
           BB = A7*Y(I)
           AA = A8*Y(I + 1)
           85 A9(I) = A9(I) - BB - AA
        GC TC 400
402 WRITE(IW,22)
        STCP
202 STCP
        END
SUBROUTINE CRNV(T,XW,V)
C SUBROUTINE CRNV FOR FINDING DRAINAGE VELOCITY OF WELLS AND EVEN ELEMENTS
C WITH GIVEN FOURIER COEFF. OF WELL DRAINAGE VELOCITY
C ALL UNITS SHOULD BE IN C.G.S. UNIT SYSTEM
C INPUT A(I,J) = FOURIER CCSINE COEFFICIENTS OF WELL DRAINAGE VELOCITY
C B(I,J) = FOURIER SINE COEFFICIENTS OF WELL DRAINAGE VELOCITY
C XW(I) = X-COORDINATES OF 13 WELLS
C DX = LENGTH OF EVEN ELEMENTS
C T = CURRENT TIME
C NPCII = TOTAL NC. OF NODEAL POINTS MINUS 1
C OUTPUT VW(I) = DRAINAGE VELOCITY OF WELLS
C V(I) = DRAINAGE VELOCITY OF EVEN ELEMENTS
C A(I,J) = FOURIER CCSINE COEFFICIENTS
C B(I,J) = FOURIER SINE COEFFICIENTS
C PERCC = 100*15*60 SECONDS, WHERE 100 MEANS 100 CYCLES, 15 MEANS 15
C MINUTES PER CYCLE, 60 MEANS 1 MINUTE EQUAL TO 60 SECONDS
DIMENSION VW(12),V(20),A(12,6),B(12,6),XW(13)
COMMON CX,NPCII,A,B,VW
C FIND DRAINAGE VELOCITY BETWEEN TWO WELLS BY FOURIER SERIES APPROXIMATI
CC=2.0*3.1416/(15.0*60.0*1C0.0)
DC 51 I=1,12
52 SUM=A(I,1)
DC 52 J=2,6
53 AA=A(I,J)
54 BB=B(I,J)
55 FJ=J-1
56 DD=FJ*CC*T
57 SUM=SUM+AA*CC*COS(CC)+BB*SIN(CC)
58 VW(I)=SUM
C FIND DRAINAGE VELOCITY FOR EACH ELEMENT WITH EVEN LENGTH
X=0.5*DX
DC 60 I=1,NPCII
DC 61 J=1,12
63 V(I)=VW(J)
64 BB=X+C.5*DX-XW(J+1)
65 CC=(C.5*DX+AA)*VW(J)+BB*VW(J+1)
66 V(I)=CC/CX
67 V(I)=0.5*VW(J+1)+C.5*VW(J)
68 AA=X-XW(J+1)-C.5*DX
69 IF(AA) 67,61,61
70 BB=X-XW(J+1)+C.5*DX
71 CC=XW(J+1)-X+C.5*DX
72 V(I)=(BB*VW(J+1)+CC*VW(J))/CX
73 V(I)=0.5*VW(J+1)+C.5*VW(J)
74 GC TC 69
75 AA=X-XW(J+1)-C.5*CX
76 IF(AA) 67,61,61
77 BB=X-XW(J+1)+C.5*CX
78 CC=XW(J+1)-X+C.5*CX
79 V(I)=(BB*VW(J+1)+CC*VW(J))/CX
80 GC TC 69
81 CONTINUE
82 X=X+CX
83 CONTINUE
RETURN
ENC
INPUT DATA FOR ILLUSTRATED EXAMPLE

NPCIN  NELEM  CT  TL  XK  POROS  DX  EPS
21  20  9.000E+02  9.000E+04  1.4000E-02  0.3400E+00  2.5000E+02

1.0000E-03

INITIAL DATA OF 13 WELLS

<table>
<thead>
<tr>
<th>I</th>
<th>XW(I)</th>
<th>YW(I)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0000E+00</td>
<td>1.0590E+02</td>
</tr>
<tr>
<td>2</td>
<td>1.0000E+03</td>
<td>1.0390E+02</td>
</tr>
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RESULTANT WATER TABLES OF ILLUSTRATED EXAMPLE

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-100-
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**HEIGHT OF FREE SURFACE AT EACH WELL**

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**TIME T = 0.9000E 05**

**NITER= 1  NLEPS= 19  EPS= 0.1000E-02**

**HEIGHT OF FREE SURFACE AT EACH WELL**

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Fortran Program for Obtaining Fourier Coefficients for Drainage Velocities

This program contains one main program and three subroutines whose purposes can be determined from the comment cards. All variables are defined in the program. Input is in the form of punched cards, which can be described in the following groups:

1) First card: FORMAT( 4E12.4, I6)

(1) DT: time increment
(2) POROS: porosity
(3) XK: hydraulic conductivity
(4) TL: total time to quit (end of run)
(5) NWELL: total number of wells

2) 2nd to ( 1 + NWELL)th card: FORMAT (I6, 2E12.4)

Each card has three initial well characteristics defined as:

(1) I: integer counter to specify well number
(2) XW(I): x-coordinate of well no. 1
(3) YW(I): water table of well no. 1

3) (2 + NWELL)th to (1 + NWELL + NSTEP)th card: FORMAT (I5, 12F5.1)

Each card contains fourteen variables as:

(1) I: integer to specify time step
(2) Y(I,1): water table elevation of well no. 1 at time step 1
(3) Y(I,2): water table of well no. 2 at time step 1

. 

. 

. 

(14) Y(I,13): water table of well no. 13 at time step 1.
C PROGRAM TO FIND FOURIER COEFFICIENTS OF WELL DRAINAGE VELOCITY
C
C INPUT
C
DT=TIME INCREMENT
PORCS=POROSITY
XK=HYDRAULIC CONDUCTIVITY
NWELL=TOTAL NUMBER OF WELLS, SHOULD BE GREATER THAN 3
TL=TIME TO QUIT
XW(I)=X-COORDINATE OF EACH WELL
YW(I)=Y-COORDINATE OF EACH WELL AT TIME T
YW(N)=NEW WATER TABLES OF WELLS
C
C OUTPUT
C
A(IJ)=FOURIER COSINE COEFF. OF DRAINAGE VELOCITY
B(IJ)=FOURIER SINE COEFF. OF DRAINAGE VELOCITY
C
DEFINITION OF OTHER VARIABLES
NSTEP=TOTAL NC. OF TIME STEP
YW(N)=Y-COORDINATE OF EACH WELL AT TIME T+DT
V(IJ)=BOTTOM DRAINAGE VELOCITY AT TIME STEP I, ELEMENT J
VW(I)=SUMMARY V(IJ) FOR CALLING FDRAV
C
DIMENSION Y(101,13),V(100,12),A(6,2),B(6,2),
*YW(13)
C
IR=5
IW=6
1 FORMAT(I5,13F5.1)
4 FORMAT(4E12.4,16)
5 FORMAT(I6,2E12.4)
C
READ(IR,4) DT, PORCS, XK, TL, NWELL
READ(IR,5) (I, XW(I), YW(I), I=1,NWELL)
NSTEP=TL/LT+0.1
DO 50 I=1,NSTEP
READ(IR,1)(I, Y(I,1), Y(I,2), Y(I,3), Y(I,4),
*Y(I,7), Y(I,8), Y(I,9), Y(I,10), Y(I,11), Y(I,12), Y(I,13)
50 CONTINUE
C
L=NSTEP
NJ=NWELL-1
DO 70 I=2,L
DO 71 JJ=1,NWELL
71 YWN(JJ)=Y(I, JJ)
70 CONTINUE
C
FIND WELL DRAINAGE VELOCITY BY EULERIAN FINITE DIFFERENCE METHOD
C
CALL FDRAV(DT, PORCS, XK, XW, YW, YWN, V, NWELL)
DO 72 JJ=1,NJ
72 V(I-1, JJ)=YW(JJ)
DO 73 JW=1,NJ
73 WRITE(IW,7) JW, V(JW)
70 CONTINUE
C
FIND FOURIER COEFF. OF WELL DRAINAGE VELOCITY
C
L=NSTEP-1
CALL FORAV(L, 5, NWELL, 21, 5, 0., 0., 1., 0., V, A, B)
END
SUBROUTINE FDRAV(CT, POROS, XK, XW, YW, YWN, VW, NWELL)

PROGRAM TO FIND THE TABULATED VALUE OF DRAINAGE VELOCITY
BY EULERIAN FINITE DIFFERENCE METHOD

V(I) = DRAINAGE VELOCITY OF ELEMENT WITH EVEN SPACING
VW(I) = DRAINAGE VELOCITY OF ELEMENT SEPARATED BY WELLS

DIMENSION VW(NJ), XW(NJ), YW(NJ), YWN(NJ)
NJ = NWELL - 1
FDT = 0.5 * POROS / DT
DO 50 I = 1, NJ
AA = FDT * (YW(NJ) + YWN(I) - YW(I) - YW(I+1))
IF(I-1) 52, 51, 52
52 BB = -XK * YW(I) * (YW(I+1) - YW(I-1))
DXW = 1.0 / (XW(I+1) - XW(I-1))
BB = BB * DXW
IF(I-NJ) 53, 54, 53
53 CC = -XK * YW(I+1) * (YW(I+2) - YW(I))
DXW = 1.0 / (XW(I+2) - XW(I))
CC = CC * DXW
GO TO TC 55
54 CC = -XK * YW(I+1) * (YW(I+1) - YW(I))
DXW = 1.0 / (XW(I+1) - XW(I))
CC = CC * DXW
GO TO TC 55
51 BB = -XK * YW(I) * (YW(I+1) - YW(I))
DXW = 1.0 / (XW(I+1) - XW(I))
BB = BB * DXW
CC = -XK * YW(I+1) * (YW(I+2) - YW(I))
DXW = 1.0 / (XW(I+2) - XW(I))
CC = CC * DXW
55 DXW = 1.0 / (XW(I+1) - XW(I))
VW(I) = AA - DXW * (BB - CC)
50 CONTINUE
DC 75 I = 1, NWELL
75 YW(I) = YWN(I)
RETURN
END
SUBROUTINE FORAV(NTSPS,NAVER,NWELL,LL,MM,TS,TSTEP,VW,A,B)

C PROGRAM TO SMOOTH THE FLUCTUATION OF DRAINAGE VELOCITIES BY TAKING AVERAGE
C OF EACH WELL DRAINAGE VELOCITIES EVERY NAVER TIME STEP
C
C PR & PR1 = 8 CHARACTERS FOR IDENTIFICATION
C NTSPS = NC. OF TIME STEPS
C NAVER = NC. OF TIME STEPS TO AVERAGE
C NWELL = NC. OF WELLS (12 OR 13)
C LL = NC. OF TABULATED DATA POINTS DESIRED (FOURIER ANALYSIS IS PERFORMED ON THESE POINTS)
C BE INTEGER, OTHERWISE PROGRAM STOP
C MM = NC. OF FOURIER COEFFICIENTS TO GENERATE MINUS ONE
C TS = START TIME FOR PLOT ROUTINE
C TSTEP = TIME INCREMENT FOR PLOT ROUTINE
C VW(IJ)=GIVEN ARRAY TO BE PERFORMED FOURIER ANALYSIS
C SVW(IJ)=ARRAY OF VW(IJ) AFTER TAKING AVERAGE EVERY NAVER TIME STEPS
C IRPT=(NUMBER OF POINT INTERPOLATED ON EACH AVERAGE REGION)-1
C = BETTER GREATER THAN OR EQUAL TO ONE
C
C EXAMPLE
C
C NPSPS=120
C NAVER=5, THEN
C IN=120/5+1=25=TOTAL NUMBERS OF AVERAGE REGION PLUS ONE
C IF, NUMBER OF POINT INTERPOLATED ON EACH REGION IS 3, THEN
C LL=24*(3+1)+1=97
C AND, IRPT=3-1=2
C
C DIMENSION SVW(5,2),VW(13,2),A(6,2),B(6,2)
C
C IW=6
51 FORMAT(110)
52 FORMAT(2110,E12.4)
NJ=NWELL-1
IN= NTSPS/NAVER +1
DO 23 I=1,NJ
23 SVW(1,I)=VW(1,I)
KMJ=1
DO 6 J=2,IN
DO 5 I=1,NJ
5 SVW(J,I)=0.0
DO 47 I=1,NAVER
KMJ = KMJ +1
IF(KMJ- NTSPS)11,11,12
11 KMJ = KMJ -1
12 DO 47 L=1,NJ
47 SVW(J,L) = SVW(J,L) + VW(KMJ,L)
DO 48 I=1,NJ
48 SVW(J,I) = SVW(J,I)/NAVER
6 CONTINUE
IRPT=(LL-IN)/(IN-1)
IRPT=IRPT-1
WRITE(IW,51) IRPT
DO 21 J=1,NJ
DO 21 I=1,NTSPS
21 WRITE(IW,52) I,J,VW(I,J)
DO 22 J=1,NJ
DO 22 I=1,IN

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22 WRITE(IW,52) I,J,SVW(I,J)
    CALL TABFR(NJ,NTPS,LL,MM,TS,TSTEP,IRPT,VW,SVW,A,B)
    RETURN
    END
SUBROUTINE TAEFR(KRO, NTSPS, L, M, TS, TSTEP, IRPT, VWW, SVW, A, B)

C PROGRAM TO PERFORM FOURIER ANALYSIS FOR AVERAGE DRAINAGE VELOCITY

L SHOULD BE CCD INTEGER
NTSPS CAN BE EITHER EVEN OR ODD INTEGER
WELLS(J) = ARRAY OF SVW(IJ) AFTER SETTING INTERPOLATION ON EACH AVERAGE REGION
VW(I) = 1-D DUMMY ARRAY OF WELL(J) FOR CALLING SUBROUTINE FORIT
FC(I) = FOURIER COSINE COEFF.
FS(I) = FOURIER SINE COEFF.
V(I) = ARRAY OF FOURIER SERIES APPROXIMATION

DIMENSION VW(13), FS(6), FC(6), WELLS(2,13), T(13), V(13), VWW(13,2), *SVW(5,2), A(6,2), B(6,2)

10 FORMAT('ON NCT .GT. OR = TO M')
11 FORMAT('ONM .LT. 0')
19 FORMAT('ONUMBER OF TABULATED VALUES NOT ODD')
21 FORMAT('WELL NUMBER ', I3, 'SINE COSINE FOURIER COEF.')
56 FORMAT(110, E12.4)
57 FORMAT(2110, E12.4)

C
IR=5
IW=6
C
C GENERATE TIME SCALE
C
DO 20 I=1, L
20 T(I) = TS + (I-1)*TSTEP
M = M+1
F = (L-1)/2.0
NCR = 3.141592/F
N = F
C
CHECK, IF VWW(IJ) HAS ODD NUMBER OF TABULATED DATA, STOP PROGRAM
C
IF(N-F)18, 17, 18
18 WRITE(IW, 19)
STOP
C
LET FIRST AND FINAL DATA POINTS OF WELLS(JI) AND VWW(IJ) EQUAL
C
DO 1 J=1, KRO
WELLS(J,1) = VWW(1,J)
1 WELLS(J,L) = VWW(NTSPS,J)
C
SETUP FIRST POINT OF EACH AVERAGE REGION
C
I=2
NSVW=1
34 NSVW=NSVW+1
DO 3 J=1, KRC
3 WELLS(J,1) = SVW(NSVW,J)
DO 41 J=1, KRC
WRITE(IW,56) NSVW, SVW(NSVW,J)
41 WRITE(IW,56) I, WELLS(J,1)
IF(IRPT)30, 30, 31

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SETUP 2ND TO (IRPT)TH POINT OF EACH AVERAGE INTERVAL

31 DO 32 K=1,IRPT
   I=I+1
   DO 33 J=1,KRD
   33 WELLS(J,I)=WELLS(J,I-1)
   WRITE(IW,56) (I,WELLS(J,I),J=1,KRD)
   32 CONTINUE

SETUP INDEX TC SKIP TO NEXT AVERAGE INTERVAL

30 I=I+2
   IF(I-L-1)34,35,35

TAKE AVERAGE VALUE AT EACH JUMP DISCONTINUITY POINT BETWEEN TWO ADJACENT AVERAGE REGIONS

35 IRPT=IRPT+2
   KK=L-IRPT
   IK=IRPT+1
   DO 36 I=IK,KK,IRPT
   37 J=1,KRD
   37 WELLS(J,I)=(WELLS(J,I-1)+WELLS(J,I+1))/2.0
   36 CONTINUE

FIND FOURIER COEFF. OF EACH WELL

DO 61 I=1,KRD
   DO 61 J=1,L
   WRITE(IW,57) I,J,WELLS(I,J)
   LN=(L-1)/2
   MM=MM+1
   DO 5 K=1,KRD
   5 VW(J)=WELLS(K,J)
   DO 6 J=1,L
   6 VW(J)=WELLS(K,J)

SUBROUTINE FORIT IS IBM STANDARD SCIENTIFIC SUBROUTINE FOR FOURIER ANALYSIS

CALL FORIT(VW,LN,M,FC,FS,IER)

CHECK

IF(IER-1)7,8,9
8 WRITE(IW,10)
   GO TO 12
9 WRITE(IW,11)
12 STCP
7 CONTINUE
   WRITE(IW,63) K
   WRITE(IW,21)(I,FS(I),FC(I),I=1,MM)
   DC 26 I=1,MM
   A(I,K)=FC(I)
26 B(I,K)=FS(I)

FIND RESULTANT ARRAY OF FOURIER SERIES APPROXIMATION

DC 13 I=1,L
   G=FNCR*(I-1)
   SUM=FC(I)
DO 14 J=2,MM
14 SUM = SUM + (FC(J) * COS((J-1)*G) + FS(J) * SIN((J-1)*G))
V(I) = SUM
CONTINUE

C PLOT RAW DATA AND COMPUTED VALUES, IF DESIRED

5 CONTINUE
RETURN
END
ABSTRACT

Two-dimensional finite-element techniques are described which model closely the complicated fluctuations observed in the water table of an ocean beach. Use of triangular elements permits specification of more realistic boundary conditions than was possible with the line elements of the one-dimensional model; also, results for the region close to the ocean compare more favorably with the field data than was the case with the one-dimensional, finite-element model.

INTRODUCTION

The object of this study was to remedy deficiencies of the one-dimensional groundwater flow model (Harrison, Fang, and Wang, 1971) and to examine the efficacy of a two-dimensional, finite-element model that uses triangular elements. The use of finite-element methods to attack boundary-value field problems was anticipated by Zienkiewicz and Cheung (1965). Later, Zienkiewicz and Cheung (1967) gave detailed analyses of the theory as well as examples of application of the finite-element method. The application of general variational principles to the groundwater flow equation did not occur until Newman and Witherspoon's studies (1970b, 1971). Application of this method has been limited to steady flow (Newman and Witherspoon, 1970a) until now. To model the movement of beach groundwater, where a free surface is involved, requires complete solution of the unsteady equation. Studies by Javandel and Witherspoon (1969), and France, et al. (1971) were helpful in this aspect of application of the finite-element method.

As mentioned in the one-dimensional model for groundwater flow in a sandy tidal beach (Harrison, et al., 1972), the hydrostatic assumption is critical over the region near the ocean boundary where the effects of tidal forces and seaward-directed head gradient are important. A two-dimensional finite-element model was necessary for modeling the effects of tidal fluctuations in this region.
EQUATIONS OF GROUNDWATER FLOW WITH A FREE SURFACE

Unsteady flow in an elastic porous medium was first studied by Theis (1935). Theis (1938) also introduced the concepts of storage coefficient and aquifer transmissibility.

The equation for unsteady groundwater flow through uniformly thick, horizontally compressible sand was derived by Jacob (1940) as:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = \frac{S}{T} \frac{\partial h}{\partial t}$$

where the storage coefficient $S$ is defined as:

$$S = \rho gb(\alpha + n\beta)$$

and $T = \text{transmissibility}, \rho = \text{fluid density}, \quad g = \text{gravitational acceleration}, \quad b = \text{uniform thickness of aquifer}, \quad \alpha = \text{vertical compressibility of the medium}, \quad n = \text{porosity of the medium}, \quad \beta = \text{compressibility of the liquid}, \quad \text{and } h = \text{the piezometer head above the datum, defined as:}$

$$h = z + \frac{1}{g} \int_{P_o}^{P} \frac{dP}{\rho(P)}$$

where $P$ is pressure head.

The equation for three-dimensional groundwater flow was also derived by Jacob (1950) as:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = \frac{S_s}{K} \frac{\partial h}{\partial t}$$

where

$$S_s = \rho g(\alpha + n\beta)$$

is called the specific storage, and defined as the volume of water in a unit volume that the aquifer releases from storage under a unit head decline, and $K$ is hydraulic conductivity. Theis's storage coefficient is equal to the product of the specific storage and the thickness of the aquifer, if the aquifer is homogeneous and uniformly thick.
By considering the conservation of mass in a control volume, Dewiest (1966) rederived the equation of groundwater flow as:

\[
\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} - 2 \rho \beta g \frac{\partial h}{\partial z} = \frac{S^*}{K} \frac{\partial h}{\partial t} \tag{6}
\]

where DeWiest's specific storage was defined as:

\[
S^* = \rho g [(1-n) \alpha + n \beta] \tag{7}
\]

Cooper (1966) improved DeWiest's equation of flow by changing the vertical coordinates to deforming coordinates (\(z'\))

\[
\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial (z')^2} - 2 \rho \beta g \frac{\partial h}{\partial z'} = \frac{S_s}{K} \frac{\partial h}{\partial t} \tag{8}
\]

In practical problems, the fourth term on the left hand side of Equ. (8) is always neglected without introducing any significant error. In all problems, then, the flow equation can be written as:

\[
\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial (z')^2} = \frac{S_s}{K} \frac{\partial h}{\partial t} \tag{9}
\]

Therefore, Jacob's equation, Equation (4), for three-dimensional flow appears to be exact if one considers the fixed coordinates to be deforming coordinates.

Hence, the governing partial differential equation of an isotropic, homogeneous porous medium in two dimensions can be represented by

\[
K \left( \frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} \right) = S_s \frac{\partial h}{\partial t} \tag{10}
\]

For the beach groundwater flow problem the initial and boundary conditions can be specified as follows.

**Initial condition:**

\[
h(x,y,0) = h_0(x,y) \tag{11}
\]

**Boundary condition:**

\[
j(x,o) = j_s(x) \tag{12}
\]
Boundary conditions:

A prescribed head on the left boundary, \( A_1 \)
\[
h = H(t), \quad (13)
\]
and a prescribed flux at the bottom boundary, \( A_2 \)
\[
K \frac{\partial h}{\partial y} = -V(x,t), \quad (14)
\]
where \( V \) is defined as positive downward, and
\( (x,t) \) represents the equation of the free surface. (See Fig. 1).

Two conditions must be satisfied on the free surface
\[
\gamma = h \quad (15)
\]
\[
K \left( \frac{\partial h}{\partial x} n_x + \frac{\partial h}{\partial y} n_y \right) = (1 - S_y \frac{\partial h}{\partial t}) n_y \quad (16)
\]
where \( S_y \) and \( I \) are the specific yield and downward infiltration through the porous medium, and \( n_x \) and \( n_y \) are \( x\)- and \( y\)-directional cosines of unit outward normal along the free surface.

Before employing finite-element analysis, the variation principles must be applied to find the minimum value of the functional for a particular function \( h \); thus (Newman and Witherspoon, 1970b and 1971),
\[
\Omega = \iint_K \left[ \frac{1}{2} K \left( \frac{\partial h}{\partial x} \right)^2 + \frac{1}{2} K \left( \frac{\partial h}{\partial y} \right)^2 + S_y h \frac{\partial h}{\partial t} \right] dxdy
\]
\[
+ \int_{A_2} V h \, dA - \int_{FS} h \left(1 - S_y \frac{\partial h}{\partial t} \right) n_y \, dS
\]

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FIG. 1. Definition sketch for triangular elements and nodes, initial conditions, and boundary conditions.
Assume the flow region is divided into many triangular elements, each element shown as in Figure 1, where i, j, and m represent the first, second, and third nodes of an element e. Then o' is the centroid of element e and x' and y' are the element coordinates through the centroid, assuming no transformation angle. The total head, h, within each triangular element can be uniquely defined (linearly) by:

\[ h = \alpha_1 + \alpha_2 x + \alpha_3 y \]  

(18)

Substituting the coordinates and the total heads for the three nodes of each element into Equation (18), h can be represented in terms of coordinates and total head at three nodes, in matrix form, as:

\[ h = [N] [h]^e \]  

(19)

where

\[ [h]^e = \begin{pmatrix} h_i \\ h_j \\ h_m \end{pmatrix} \]  

(20)

and

\[ [N] = [N_i N_j N_m] \]  

(21)

where

\[ N_i = \frac{1}{2 \Delta} (a_i + b_i x + c_i y), \quad i = i, j, m, \text{ and } \Delta \]

is the area of the triangular element. Letters a, b, and c, with subscripts i, j, and m, are short notations for:

\[ a_i = x_j y_m - x_m y_j \]

\[ b_i = y_j - y_m \]  

(22)

\[ c_i = x_m - x_j \]
and the corresponding coefficients for each element are obtained by a cyclic permutation of the subscripts in the order $i, j, \text{and} \; m$.

Similarly, time derivatives within each element can be represented by

$$
\begin{bmatrix}
\frac{\partial h}{\partial t}
\end{bmatrix} = [N] \begin{bmatrix}
\frac{\partial h}{\partial t}
\end{bmatrix}^e
$$

where

$$
\begin{bmatrix}
\frac{\partial h}{\partial t}
\end{bmatrix}^e = \begin{bmatrix}
\frac{\partial h}{\partial t}^i \\
\frac{\partial h}{\partial t}^j \\
\frac{\partial h}{\partial t}^m
\end{bmatrix}
$$

Dividing the whole region into many small elements, the minimum function of the overall region can be expressed by the summation of all the element parts of a functional, whose minimizing functions are approximate solutions to the problems, as

$$
\Omega = \sum_{e=1}^{K_1} \Omega_{FS}^e + \sum_{e=K_1+1}^{K_2} \Omega^e + \sum_{e=K_2+1}^{M} \Omega_{A_2}^e
$$

where $\Omega_{FS}^e$, $\Omega^e$, and $\Omega_{A_2}^e$ are the element minimum functionals of the elements along the free surface, of the inside region, and along the boundary of prescribed flux, respectively; $K_1$ is the total number of elements free surface, $(K_2 - K_1) =$ the total number of inner elements, $(M - K_2) =$ the total number of elements along the prescribed flux boundary, and $M =$ the total number of elements in the whole region.

In Equation (17), the integral along the prescribed flux boundary, $\int_{A_2} VhdA$, existed only in the element minimum functionals $\Omega_{A_2}^e$, and the integral along the free surface vanished, except in $\Omega_{FS}^e$; therefore, the element minimum functionals can be rewritten as:
\[ \Omega^e = \iint_{R^e} \left[ \frac{1}{2} K^e \left( \frac{\partial h^e}{\partial x} \right)^2 + \frac{1}{2} K^e \left( \frac{\partial h^e}{\partial y} \right)^2 \right. \\
+ S^e_s h^e \left( \frac{\partial h^e}{\partial t} \right) \] dx dy \]  
(26)

\[ \Omega_{A2}^e = \iint_{R^e} \left[ \frac{1}{2} K^e \left( \frac{\partial h^e}{\partial x} \right)^2 + \frac{1}{2} K^e \left( \frac{\partial h^e}{\partial y} \right)^2 \right. \\
+ S^e_s h^e \left( \frac{\partial h^e}{\partial t} \right) \] dx dy + \int_{A2} V^e h^e dA \]  
(27)

\[ \Omega_{FS}^e = \iint_{R^e} \left[ \frac{1}{2} K^e \left( \frac{\partial h^e}{\partial x} \right)^2 + \frac{1}{2} K^e \left( \frac{\partial h^e}{\partial y} \right)^2 \right. \\
+ S^e_s h^e \left( \frac{\partial h^e}{\partial t} \right) \] dx dy \\
- \int_{FS} h^e \left( i^e - S^e_y \frac{\partial h^e}{\partial t} \right) n_y^e dS \]  
(28)

where the superscript \( e \) indicates the parameters of an element \( e \) under consideration, and \( R^e \) means to take the area integral of the element being considered.

If \( h \) is defined uniquely and continuously throughout the region, then the functional can be minimized with respect to all nodal values of the total head, \( h_1 \); that is,

\[ \frac{\partial \Omega}{\partial h_1} = \sum_{e=1}^{K_1} \frac{\partial \Omega_{FS}^e}{\partial h_1} + \sum_{e=K_1+1}^{K_2} \frac{\partial \Omega_{A2}^e}{\partial h_1} \]

\[ + \sum_{e=K_2+1}^{M} \frac{\partial \Omega_{A2}^e}{\partial h_1} = 0 \]  
(29)
If there are $N$ nodes in the whole region, then Equation (29) becomes a linear system of $N$ equations at any particular time step; for example,

$$
[P] [h]_t + [Q] \left( \frac{\partial h}{\partial t} \right)_t = [R] \quad (30)
$$

where $[P]$ and $[Q]$, called the overall matrices, are $N \times N$ square matrices. $[h]_t$ is an $N \times 1$ row matrix formed by the total head of all nodes at the particular time step being considered, and so is $\left( \frac{\partial h}{\partial t} \right)_t$. $R$ is an $N \times 1$ constant row matrix obtained due to the existence of the prescribed flux and the infiltration flux.

When the central finite-difference approximation for $\left( \frac{\partial h}{\partial t} \right)_t$ is made as

$$
\left( \frac{\partial h}{\partial t} \right)_t = - \left( \frac{\partial h}{\partial t} \right)_{t-\Delta t} + \left( [h]_t \right)_{t-\Delta t} \quad (31)
$$

then Equation (30) becomes

$$
[D] [h]_t = [E],
$$

where

$$
[D] = [P] + \frac{2}{\Delta t} [Q] \quad (33)
$$

$$
[E] = [R] + [Q] \left( \frac{2}{\Delta t} [h]_{t-\Delta t} + \left( \frac{\partial h}{\partial t} \right)_{t-\Delta t} \right)
$$

and, $\Delta t$ is the increment of the time step. Then, the total head of all nodes $h_i$, $i = 1, 2, \ldots, N$, should be found from Equation (32), instead of Equation (30). If there are just $N'$ nodes ($N' < N$) with unknown total head, which are numbered first, then, only the first $N'$ non-redundant equations in Equation (32)
useful. The prescribed values of all given \( h_i \) should be substituted into Equation (32), and the constant terms moved to the left hand side of the equation to get a new linear system of \( N' \) equations as

\[
[D'] [h]_t = [E']
\]

where

\[
D'_{ij} = D_{ij}, \quad i = 1, 2, \ldots, N' \]
\[
j = 1, 2, \ldots, N'
\]

\[
E'_{i} = E_{i} - \sum_{j=N'+1}^{N} D_{ij} h_j
\]

\[
i = 1, 2, \ldots, N'
\]

In order to achieve the purpose of obtaining overall matrices and the total heads of all nodes shown in the above procedures; the terms,

\[
\frac{\partial \Omega_{FS}}{\partial h_i}, \quad \frac{\partial \Omega}{\partial h_i}, \quad \text{and} \quad \frac{\partial \Omega_{A2}}{\partial h_i}
\]

obtained first as expressed in equation (29). Equation (29) is the source of element matrices and the reason why element matrices must be found before overall matrices.

The detailed formulation of the element matrices can be obtained in (Zienkiewicz and Cheung, 1967). The results are as follows:

For inner elements:

\[
\left[\frac{\partial \Omega}{\partial h}\right]_e = \left[\begin{array}{c}
\frac{\partial \Omega}{\partial h_i} \\
\frac{\partial \Omega}{\partial h_j} \\
\frac{\partial \Omega}{\partial h_m}
\end{array}\right] = [BC] [h]^e + [SKN] \left[\frac{\partial h}{\partial t}\right]^e
\]
For inner elements along the boundary of prescribed flux:

\[
\frac{\partial \Omega}{\partial h}^e_{A2} = [BC] [h]^e + [SKN] \left[ \frac{\partial h}{\partial t} \right]^e
+ \frac{1}{2} V^e \Delta x^e \begin{pmatrix}
1 \\
1 \\
0
\end{pmatrix}
\]  

(37)

and for elements along the free surface:

\[
\frac{\partial \Omega}{\partial h}^e_{FS} = [BC] [h]^e + [SKN] \left[ \frac{\partial h}{\partial t} \right]^e
+ \frac{S_y^e}{6} (x_j - x_m) \begin{pmatrix}
0 & 0 & 0 \\
0 & 2 & 1 \\
0 & 1 & 2
\end{pmatrix}
\left[ \frac{\partial h}{\partial t} \right]^e
\]  

- \frac{\kappa^e}{2} (x_j - x_m) \begin{pmatrix}
0 \\
1 \\
1
\end{pmatrix}
\]

(38)

where

\[
[BC] = \frac{K^e}{4\Delta} \begin{pmatrix}
b_i b_i b_j b_i b_i b_m \\
b_j b_i b_j b_j b_j b_m \\
b_m b_i b_m b_j b_m b_m
\end{pmatrix}
\]

\[
+ \frac{K^e}{4\Delta} \begin{pmatrix}
c_i c_i c_j c_i c_i c_m \\
c_j c_i c_j c_j c_j c_m \\
c_m c_i c_m c_j c_m c_m
\end{pmatrix}
\]  

(39)
and

\[
[SKN] = \begin{bmatrix}
\int_{R_e} N_i N_i \, dx \, dy & \int_{R_e} N_i N_j \, dx \, dy & \int_{R_e} N_i N_m \, dx \, dy \\
\int_{R_e} N_j N_i \, dx \, dy & \int_{R_e} N_j N_j \, dx \, dy & \int_{R_e} N_j N_m \, dx \, dy \\
\int_{R_e} N_m N_i \, dx \, dy & \int_{R_e} N_m N_j \, dx \, dy & \int_{R_e} N_m N_m \, dx \, dy
\end{bmatrix}
\]

(40)

and \( \Delta x \) is the length of the side of an element along the prescribed flux boundary.

In deriving the above element matrices, we numbered the nodes of elements along the free surface and the boundary of the prescribed flux in the manner shown on Fig. 1. Also, it was assumed that \( h \) varied linearly from one node to the other.

**BOUNDARY CONDITIONS**

Mathematical modeling of groundwater flow using the Jacob's equation requires a knowledge of hydraulic conductivity and specific storage, as well as appropriate boundary conditions. The lack of precise measurements of these two geohydrologic parameters will cause an uncertainty in preparing mathematical models. Freeze and Witherspoon (1968) tried to estimate the parameters, by a trial-and-error process of matching calculated and measured data at various points. Kleinecke (1971) attempted to employ linear programming to achieve the same purpose. Hydrologic records over large regions were needed in order to use even the simplest forms of boundary conditions.

There are two types of boundaries: prescribed-head, and prescribed-flux boundaries. Such boundaries of simple form are used in most studies, but for the beach groundwater problem, if a finite region is selected, portions of the boundaries do not possess hydrologic characteristics. Both the bottom boundary and the ocean boundary (right-hand boundary, Fig. 1) are complicated functions of space and time, due to the seaward-directed head gradient and tidal fluctuations. Therefore, it is necessary to replace the semi-infinite, unconfined aquifer with a finite
region. The assumption of the existence of the hydrostatic condition, required for the one-dimensional model on the ocean-side boundary (Harrison, et al., 1971), was somewhat weak; it may be replaced here by a prescribed boundary condition (see below).

The "drainage velocity" V(x,t), calculated (Harrison, et al., 1971) from the field data by the finite-difference method, was used to impose the effects of tide on each element of the bottom boundary. If there is any infiltration in the system it will also be limped into this term.

For the present two-dimensional model, the landward (left) boundary was assumed to be hydrostatic. The ocean boundary was approximated by imposing a uniform horizontal flux along the boundary of each element (Fig. 1).

This horizontal flux could be approximated by Darcy's Law as

\[ U^e = -\frac{k^e}{\Delta x} (h_1 - h_2) \]  

(41)

where point 1 is any point on the boundary, and \( \Delta x \) is selected as small as possible. Because point 2 has the same altitude as point 1, and because Equation (19) holds for any element; then, the uniform boundary flux becomes

\[ U^e = -\frac{k^e}{\Delta x} [b] [h]^e \]  

(42)

Effects of this flux must be considered, as shown in Equation (28), to be expressed implicitly in terms of unknown nodal heads before substituting given nodal heads into Equation (32). The same procedure is also followed for the left boundary, whether a prescribed head or a hydrostatic condition is imposed.
FREE SURFACE

The most difficult problem with the free-surface boundary for the beach groundwater problem lies in treating the free-surface as a moving boundary. All free-surface elements are needed to recalculate element matrices at every time step, otherwise the accuracy of the model will be decreased. The iteration method was chosen for recalculating the free-surface elements. The cumulative change of the free-surface leads to changes in node-element configuration. It is necessary, therefore, to reset nodes and elements or shift nodes during each successive time step.

For simplification, shifting was restricted to vertical coordinates; thus,

\[ y_{N_i} = y_i + (y_{N_j} - y_j) \]

where \( y_{N_i} \) and \( y_i \) represent the y-coordinate of node \( i \) under consideration, before and after shifting, \( y_{N_j} \) and \( y_j \) represent the new and old y-coordinates of free-surface node \( j \), which is directly above the inner node \( i \). Then, the new total head \( h_i' \), after a shift, can be found from equation (19) as (Fig. 2):

\[
h_i' = \frac{1}{4\Delta(e)} \left[ (a_i + b_i x_i + c_i y_{N_i}) h_i + (a_j + b_j x_i + c_j y_{N_i}) h_j + (a_m + b_m x_i + c_m y_{N_i}) h_m \right] (e) + \frac{1}{4\Delta(e+1)} \left[ (a_i + b_i x_i + c_i y_{N_i}) h_i + (a_k + b_k x_i + c_k y_{N_i}) h_k + (a_m + b_m x_i + c_m y_{N_i}) h_m \right] (e+1)
\]

where the superscripts \( e \) and \( e+1 \) correspond to transformed elements \( e \) and \( e+1 \), respectively.
FIG. 2. Definition sketch for shifting vertical coordinates.
APPLICATION OF METHOD

To facilitate computer programming, nodes on the free surface were numbered first (nodes 1 to 19, Fig. 3). Then all nodes with unknown total head were numbered (nodes 20 to 69), after which nodes with given head were numbered (for example, nodes 70 to 74). In like manner, the elements along the free surface were numbered first, from element 1 to element 40, in the element-numbering system. Elements along the free surface should be smaller than other elements to obtain accurate results.

Once the node-element configuration is decided, data cards for such a numbering system should be prepared. Initial total head and coordinates are also needed. Starting time was chosen as 0645 EDT, August 11, 1969 (see Harrison and Fausak, 1970).

The computer program continuously seeks the element matrices, based on Equations (39) and (40), before obtaining the overall matrices equation (Equ. 30); then, it finds time-derivatives of total head at time \( t - \Delta t \) before reaching Equation (32). The prescribed values of \( h \), obtained from the field data (Harrison and Fausak, 1970), are now applied on the nodes of the landward-side boundary (Fig. 3, nodes 71 to 74), where the hydrostatic state was assumed to exist. A prescribed value of \( h \) is also applied at the upper node (node 70) of the ocean-side boundary, where the hydrostatic condition does not exist.

The prescribed head values substituted into Equation (32) to obtain a linear system of simultaneous equations, as Equation (34). The equations were solved by the elimination method, using the largest pivotal divisor, to obtain the total head for all nodes at any time step under consideration. The program then proceeded to consider the effects of the moving free surface. The results of each time step were used as initial values for the next time step. The necessity of resetting the node-element configuration was checked every 5 cycles. This procedure was followed as long as desired.

Fourier coefficients for bottom drainage velocities were read at initial setup, a subroutine being called to find the drainage velocities for the coefficients given at each time step. The following data were also read in: total nodes = 74, total elements = 110, porosity = 34%, hydraulic conductivity = 0.014 cm/sec, specific storage = 0.003125 1/cm, and
FIG. 3. Definition sketch for numbering nodes and elements.
time increment for each step = 15 minutes.

The storage coefficient may be considered equal to the specific yield for an unconfined flow with a free surface (Chow, 1964). Since only field tests of porosity and hydraulic conductivity were made, the Figure 13-2, in Chow (1964), was chosen to find the specific yield. This was found to be 25% for the given porosity. The specific storage was taken as 0.003125 l/cm, for an average flow region assumed to be 80 cm.

RESULTS AND DISCUSSION

As shown in Figure 4, the two-dimensional finite-element method has provided an accurate solution for groundwater flow with respect to complicated beach water-table fluctuations. A compromise decision was made relative to the assumed positions of the boundaries, the value of the specific storage, and the average drainage velocity. A Fourier series was used to describe the mean regional drainage-velocity characteristics and the beach water-table's response to the input tidal fluctuations. Comparison of the results (Fig. 4) for the two-dimensional case, the one-dimensional case, and the field data indicates that the two-dimensional finite-element method is more accurate for modeling the fluctuations of the beach groundwater table than is the one-dimensional method.

Even though one-dimensional field data were used as the boundary condition for the two-dimensional case, the results still exhibit less fluctuations, after many time steps, near the ocean (right) side boundary (Fig. 4) where the effects of tidal fluctuations are large. The small discrepancies (Fig. 4) can be further reduced by using smaller elements over this region, since no matter what combinations of element sizes are used, a system of linear matrix equations will finally result from the two-dimensional finite-element method. For the one-dimensional case, a system of nonlinear functions was obtained; the equations were solved by the Newton-Raphson iteration method (Harrison, et al., 1971). The Newton-Raphson method, after our testing, seems restricted to even-length elements; otherwise, the iteration solution would easily become divergent.

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FIG. 4. Comparison between field data (solid circles), one-dimensional model results (open circles), and two-dimensional model output (triangles).
The fact that small differences exist between the field data and the theoretical results can be explained as due to the effects of variables in Equation (2) of Harrison, et al. (1971, p. 1315). The effects of capillarity and groundwater density gradients are probably also important. It also seems certain that if the specific storage can be measured precisely, the model results will more closely parallel the field data.

COMPUTER PROGRAM

A description and listings of the FORTRAN computer programs used in the finite-element analysis are given in a section following the References.

REFERENCES


———, The significance and nature of the cone of depression in groundwater bodies, Econ. Geol., (33), 889-920, 1938.


FORTRAN PROGRAM
FOR TWO-DIMENSIONAL, FINITE-ELEMENT ANALYSIS

Following this description are the main program, subroutine DRANV (which was designed to find the bottom drainage velocity with given Fourier coefficients of drainage velocity), and subroutine SIMQM (which was obtained by modifying IBM standard scientific subroutine SIMQ, to eliminate an underflow problem). Definitions of all variables can be found on the comment cards of the programs. Also included are comment cards describing the purposes of various portions of the programs. Only punched cards are used for input. This program takes 47.82 minutes CPU time per 100 cycles in MODE 50/IBM 360. Input cards are listed according to the following groups:

1) First card: FORMAT (3I6, 4E12.4)

This card contains three integer variables, four real variables in sequence as:

1st variable: NPART
2nd variable: NPOIN
3rd variable: NELEM
4th variable: DT
5th variable: XK
6th variable: TL
7th variable: POROS

2) Second card: FORMAT (14I5)

This card contains 14 integer variables in sequential order as:

1st variable: NFSF
2nd variable: NFSL
3rd variable: NBSF
4th variable: NBSL
5th variable: MFSF
6th variable: MFSL
7th variable: MBSF
8th variable: MBSL
9th variable: NLEFT
10th variable: NRIGT
11th variable: NBOND
12th variable: LITER
13th variable: NST
14th variable: INNER
3) 3rd card: FORMAT (6E12.4)

Contains six real variables as:

1st variable: SYE (0 in this study)
2nd variable: FIE (0 in this study)
3rd variable: YLENS
4th variable: FLEPS
5th variable: FDCON
6th variable: FD

4) 4th to (3 + NPOIN)th card: FORMAT (I10, 3F10.4)

Each card of this group contains four variables as:

1st variable: I = integer to specify nodal number
2nd variable: X(I)
3rd variable: Y(I)
4th variable: H(I)

5) (4 + NPOIN)th card: FORMAT (I10)

Contains variable NCARD only

6) (5 + NPOIN)th to (4 + NPOIN + NELEM)th card: FORMAT (4I10)

Each card of this group contains:

(1) 1st variable: I = integer to specify element number
(2) 2nd variable: NOD1(I)
(3) 3rd variable: NOD2(I)
(4) 4th variable: NOD3(I)

7) (5 + NPOIN + NELEM)th card: FORMAT (I10)

Contains variable NCARD only

8) (6 + NPOIN + NELEM + INNER)th card: FORMAT (5I5)

Each card of this group contains

(1) 1st variable: I = integer specify nodal number
(2) MNUM (I,1)
(3) MNUM (I,2)
(4) MNUM (I,3)
9) \((7 + \text{NPOIN} + \text{NELEM} + \text{INNER})_{\text{th}}\) to \((30 + \text{NPOIN} + \text{NELEM} + \text{INNER})_{\text{th}}\) card:

\text{FORMAT (A7, 6E12.4)}

Contains 12 sets in this group. Each set has 2 cards. The first card is composed of Fourier cosine coefficients as:

(1) 1st variable: \text{CDUMY} = \text{variable for A-field to identify as Fourier cosine coefficients}
(2) 2nd variable: \text{A(I,1)}
(3) 3rd variable: \text{A(I,2)}

\vdots

(7) 7th variable: \text{A(I,6)}

Similarly, a second card is composed of Fourier sine coefficients punched in same sequence.

10) \text{FORMAT (I6, 2E12.4)}

This group contains 13 cards, each card has:

(1) 1st variable: \text{I} = \text{integer to specify well number}
(2) 2nd variable: \text{XW(I)} = \text{x-coordinates of well I}
(3) 3rd variable: \text{YW(I)} = \text{water table of well I}

11) \text{FORMAT (I6, 6E12.4)}

This card contains three variables as:

(1) \text{NPOV}
(2) \text{DXV}
(3) \text{SSTA}

12) \text{FORMAT (4F10.4)}

The total number of cards in this group is equal to the total number of time steps desired. Each card contains left and right-hand-side boundary conditions obtained from field data.

(1) 1st variable: \text{YLEFT}
(2) 2nd variable: \text{YRIGT}
(3) 3rd and 4th variables of each card are dummy variables.
/* PROGRAM TO SOLVE GROUND WATER PROBLEM WRITTEN BY SOU-NAN WANG,
DEPT. OF GEOMORPHOLOGY
UNDER GRANT CAR
TW-DIMENSIONAL UNSTEADY CASE

Npart=TOTAL NUMBER OF PARTITIONS
NPCIN=TOTAL NUMBER OF NODEAL POINTS
NELEM=TOTAL NUMBER OF ELEMENTS
DT=TIME INCREMENT CF EACH TIME CYCLE
TL=TIME CUIT
BELTA=THE COMPRESSIBILITY CF THE FLUID
RHC=CENSITY CF FLUID CONSIDERED
GY=GRAVITY ACCELERATION
PORCS=POROSITY
SSTA=VERTICAL COMPRESSIBILITY OF THE SOLID OR
=X THE SPECIFIC STORAGE
XK =X THE HYDRAULIC CONDUCTIVITY OF HOMOGENEOUS, ISOTROPIC SAND
NFSF =FIRST NC. CF FREE SURFACE NODE
NFSL =FINAL NC. CF FREE SURFACE NODE
NBSF =FIRST NC. CF BOTTOM SIDE NODE
NBSL =FINAL NC. CF BOTTOM SIDE NODE
MFSF =FIRST ELEMENT ALONG FREE SURFACE
MFSL =FINAL ELEMENT ALONG FREE SURFACE
MBSF =FIRST ELEMENT ALONG BOTTOM SIDE
MBSL =FINAL ELEMENT ALONG BOTTOM SIDE
NLEFT=NCAL NC. CF FIRST NODE ON LEFT BOUNDARY
NRIGT=NCAL NC. CF FIRST NODE ON RIGHT BOUNDARY
NCNC=TOTAL NUMBER OF NODE ON EACH BOUNDARY
NSHFT=CURRENT COUNTER TC CHECK WHETHER OR NOT TO SHIFT NODEAL COORDINATES
WHEN NSHFT=NST
NST =TOTAL NC. CF TIME STEP ALLOWED UNTIL CHECKING THE SHIFT CF NODEAL
COORDINATES
NITER=CURRENT NC. CF ITERATION TO CORRECT THE PCISON CF FREE SURFACE
LITER=TOTAL NC. CF ITERATION ALLOWED TO CORRECT PCISON CF FREE SURFACE
NCCNV=TOTAL NC. CF NODES ON FREE SURFACE WHICH ARE SATISFIED CONVERGENT
CRITERIA AT EVERY ITERATION.
SYE =X THE SPECIFIC YIECF OF THE POROS MEDIUM
FIE =X THE NET AVERAGE RATE OF INFILTRATION
YLENS =IF ANY ABSOLUTE VALUE OF (Y(I)-YFCD(I)) ALONG FREE SURFACE GREATER
THAN YLENS, SHIFT COORDINATES
X(I)=X-COORDINATES OF NODES W. R. TO COMMON COORDINATES AT ANY TIME STEP
Y(I)=Y-COORDINATES OF NODES W. R. TO COMMON COORDINATES AT TIME STEP N
YCLD(I)=Y-CCORD. CF FREE SURFACE AT TIME STEP N-1
YFC(LD(I)=Y-CCORD. CF FREE SURFACE AT FIRST TIME STEP CF AFTER SHIFTING
NCCAL COORDINATES
Y(UMY(I)=CUMY Y-CCORD. CF INNER NODES FOR CORRECTION CF Y(I) CLE TC
SHIFTING CCORD.
YLC =Y-CCORD. CF FIRST NODE ON LEFT BOUNDARY AT FIRST TIME STEP AFTER
SHIFTING
YRC =Y-CCORD. CF FIRST NODE ON RIGHT BOUNDARY AT FIRST TIME STEP AFTER
SHIFTING
YLC(I)=Y-CCORD. CF FIRST NODE ON LEFT BOUNDARY AT TIME STEP N-1

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C YKCLL = Y - CCC4E. CF FIRST NCCE ON RIGHT BOUNDARY AT TIME STEP N-1
C YLEFT = Y - CCC4C. CF FIRST NCCE ON LEFT BOUNDARY AT TIME STEP N
C YRIGHT = Y - CCC4D. CF FIRST NCCE ON RIGHT BOUNDARY AT TIME STEP N
C H(I) = PIOLANCETER. CF NCCE ON BOUNDARY AT TIME STEP N
C H(I) = PIOLANCETER. CF INNER NCCEs FOR CORRECTION DUE TO SHIFTING CCRD.
C YOLC = NUMBER OF CARCS READ IN FOR THE PREVIOUS SET, USED FOR CHECKING
C YOL1(K) = FIRST NCCE (THAT, IS, NODE I) OF THE TRAIEEE ELEMENT K
C YOL2(K) = SECOND NCCE (THAT, IS, NODE J) OF THE TRAIEEE ELEMENT K
C YOL3(K) = THIRD NCCE (THAT, IS, NODE M) OF THE TRAIEEE ELEMENT K
C BC1(IJ) = CCLLME 1 OF ROW J ON FIRST PART OF ELEMENT MATRIX OF ELEMENT I
C BC2(IJ) = CCLLME 2 OF ROW J ON FIRST PART OF ELEMENT MATRIX OF ELEMENT I
C BC3(IJ) = CCLLME 3 OF ROW J ON FIRST PART OF ELEMENT MATRIX OF ELEMENT I
C SKN1(IJ) = CCLLME 1 OF ROW J ON SECOND PART OF ELEMENT MATRIX OF ELEMENT I
C SKN2(IJ) = CCLLME 2 OF ROW J ON SECOND PART OF ELEMENT MATRIX OF ELEMENT I
C SKN3(IJ) = CCLLME 3 OF ROW J ON SECOND PART OF ELEMENT MATRIX OF ELEMENT I
C BCOl(IJ) = CORRECTING MATRICE OF BC1(IJ) DUE TO CHANGE OF FREE SURFACE
C BCO2(IJ) = CORRECTING MATRICE OF BC2(IJ) DUE TO CHANGE OF FREE SURFACE
C BCO3(IJ) = CORRECTING MATRICE OF BC3(IJ) DUE TO CHANGE OF FREE SURFACE
C O(IJ) = LUMY MATRICE TO AVOID DESTROYING P(IJ) IN COMPUTATION
C O(IJ) = LUMY MATRICE TO AVOID DESTROYING R(I) IN COMPUTATION
C HT(I) = TIME DERIVATIVE OF H(I)
C H(I) = DRAINAGE VELOCITY OF ELEMENTS ALONG BOTTOM BOUNDARY
C H(I) = DRAINAGE VELOCITY OF ELEMENTS ALONG BOTTOM BOUNDARY
C MNP1(IJ) = NUMBERING INDEX OF SOME ELEMENTS AROUND AN INNER NCCE WHICH IS NEEDED TO CONSIDER THE CORRECTION OF H(I) AFTER SHIFTING NODAL CCRD.
C MNP1 = TOTAL NC. OF INNER NCCEs
C FL, AN XT = LUMY VARIABLES
C A(IJ) = FOURIER CCSINE COEFF. OF BOTTOM DRAINAGE VELOCITY OF WELL ELEMENT I
C E(IJ) = FOURIER SINE COEFF. OF BOTTOM DRAINAGE VELOCITY OF WELL ELEMENT I
C VCLM(Y(IJ) = LUMY VELOCITY FOR CALLING SUBROUTINE DRANV
C DXV = LENGTH OF BCTTCM ELEMENT FOR CALLING SUBROUTINE DRANV
C NPCV = NC. OF BCTTCM NCCEs FOR CALLING SUBROUTINE DRANV
C BFLX(IJ) = LUMY VARIABLES FOR CONSIDERING THE EFFECTS OF HORIZONTAL FLUX ON OVER-ALL MATRICES
C

DIMENSION X(74), Y(74), NC01(110), NC02(110), NC03(110), YCLD(19),
* H(74), BC1(110,3), BC2(110,3), BC3(110,3), SKN1(110,3), YFCLD(19),
* SKN2(110,3), SKN3(110,3), P(74,74), Q(74,74), D(74,74), R(74),
* VC1(40), E(74), F(74), BC1(40,3), BC02(40,3), BC03(40,3), RCLD(19),
* SK1C(40,3), SK2C(40,3), SK3C(40,3)

DIMENSION MNP(38,4), YCLMY(74), HDMY(74), BFLX(4,3)

DIMENSION A12,6, E12,6, VCLM(10), XW(13), YW(13)

CMMNCN XW, NPCV, A, E
1 FCRMAT(316,4, E12,4)
2 FCRMAT(1H,6,6, NPCV = 15,2X,6, HPOIN = 15,2X,6, HNELEM = 15,2X,3, HCT =
* E12,4,2X,3, HXX = E12,4,2X,3, HHT = E12,4,2X,6, HPORES = E12,4, //)
3 FCRMAT(110,310,4)
4 FCRMAT(100X,111,6,4, X(1),6,4, Y(1),6,4, H(1),6,4, H(1) //)
5 FCRMAT(110)
6 FCRMAT(1HO,2X,59H INITIAL DATA OF COORDINATES AND TOTAL HEAD CF
* ALL NCCEs //)
7 FCRMAT(4110)
8 FCRMAT(6, E12,4,3X)
9 FCRMAT(1H0,2X,75H THE NUMBERS OF THREE NCCEs IN TRIANGULAR ELEMENT
* IN COUNTERCLOCKWISE SENSE//
10 FORMAT (9X,1H1,3X,7HNOG(1),3X,7HNOG2(1),3X,7HNOG3(1)//
11 FORMAT (2X,216,2X,2E12.4)
12 FORMAT (10X,2HT=,E12.4)
13 FORMAT (6E12.4)
14 FORMAT (16,3X,E12.4)
15 FORMAT (5110)
16 FORMAT (A7,6E12.4)
17 FORMAT (16,2E12.4)
18 FORMAT (515)
19 FORMAT (4F10.4)
20 FORMAT (E12.4)
21 FORMAT (16,6E12.4)
22 FORMAT (2X,5C: FOURIER COEFFICIENTS OF BOTTOM DRAINAGE VELOCITY)
23 FORMAT (2X,5C:)
24 FORMAT (2X,5C:)
25 FORMAT (2X,5C:)
26 FORMAT (1H0,2X,15H: PRINT MNLM(1)///)
27 FORMAT (9X,1K1,4C: MNLM(1,1) MNLM(1,2) MNLM(1,3) MNLM(1,4)///)
28 FORMAT (1H0,2X,5C:)
29 FORMAT (5X,1H1,7X,5H: COORDINATES OF 13 WELLS)///
30 FORMAT (5X,1H1,7X,5H:)
31 FORMAT (1H0,2X,5H:)
32 FORMAT (1H0,2X,4H: TOTAL REAC OF NODES AT TIME T I=1 TO 66///)
33 FORMAT (5X,1H1,7X,5H:)
34 FORMAT (1H0,2X,4H:)
35 FORMAT (1H0,2X,4H:)
36 FORMAT (1H0,2X,4H:)
37 FORMAT (1H0,2X,4H:)
38 FORMAT (1H0,2X,4H:)
39 FORMAT (1H0,2X,4H:)
40 FORMAT (5H: STCPA)
41 FORMAT (5H: STCPC)
42 FORMAT (5H: STCPE)

C C
IR=5
C
IH=6
C
REAL AND PRINT DATA REQUIRED
C
REAL(IR,1)NPAR,INPCIN,NELEM,DT,XK,TL,POROS
WRITE(IH,2)NPAR,KPOIN,NELEM,DT,XK,TL,POROS
REAL(IR,22)NFSF,NFSL,NFSF,NFSL,MFSF,MFSF,MBSF,MBSF,MBSF,MBSF,MBSF,MBSF,MBSF,MBSF,MBSF,MBSF
*NCAL,LITER,NST,IANER
REAL(IR,13)SYE,FIE,YLNS,FLEPS,FDCON,FD
C
REAL COORDINATES OF NCAL W.R. TO COMMON COORDINATES
C
REAL INITIAL NCAL VALUE OF H(I)
C
WRITE(IH+6)
WRITE(IH+4)
DC 51 I=1,INPCIN
REAL(IR,3)X(I),Y(I),H(I)
$51 \text{WRITE(IW,3)}1,X(I),Y(I),I(1)$
$\text{REAL(IR,5)NCARC}$
$\text{IF} (\text{NCARC-PCIN}) 53,52,53$
$53 \text{WRITE(IW,46)}$
$\text{STCP}$
$52 \text{CONTINUE}$
$C$
$\text{READ THE NUMBERS OF THREE NODAL IN TRIANGLE ELEMENT IN COUNTERCLOCKWISE}$
$\text{SENSE AND PROPERTIES OF CF ELEMENT}$
$C$
$\text{WRITE(IW,9)}$
$\text{WRITE(IW,10)}$
$\text{DC 57 I=1,NELEM}$
$\text{REAL(IR,7)}1,NCD1(I),NCC2(I),NPC3(I)$
$57 \text{WRITE(IW,7)}1,NCD1(I),NCC2(I),NPC3(I)$
$\text{REAL(IR,5)NCARC}$
$\text{IF} (\text{NCARC-NELEM}) 59,58,59$
$59 \text{WRITE(IW,48)}$
$\text{STCP}$
$58 \text{CONTINUE}$
$C$
$\text{REAL DC 56 I=1,INNER}$
$\text{REAL(IR,20)}1,MNUM(I,1),MNUM(I,2),MNUM(I,3),MNUM(I,4)$
$56 \text{CONTINUE}$
$C$
$\text{REAL DC 54 I=1,12}$
$\text{REAL(IR,18)}1,CUMLY, A(I,1),A(I,2),A(I,3),A(I,4),A(I,5),A(I,6)$
$54 \text{REAL(IR,18)}1,SCUMY, B(I,1),B(I,2),B(I,3),B(I,4),B(I,5),B(I,6)$
$\text{WRITE(IW,26)}$
$\text{DC 55 I=1,12}$
$\text{WRITE(IW,18)}1,CUMLY, A(I,1),A(I,2),A(I,3),A(I,4),A(I,5),A(I,6)$
$55 \text{WRITE(IW,18)}1,SCUMY, B(I,1),B(I,2),B(I,3),B(I,4),B(I,5),B(I,6)$
$C$
$\text{REAL DC 53 I=1,13}$
$\text{REAL(IR,19)}1,XW(I),YW(I),I=1,13)$
$\text{WRITE(IW,29)}$
$\text{WRITE(IW,30)}$
$\text{WRITE(IW,19)}1,XW(I),YW(I),I=1,13)$
$C$
$\text{MCBS=NC. CF ELEMENTS ALONG BOTTOM BOUNDARY}$
$C$
$\text{MCBS=MBSL-MBSF+1}$
$\text{DCT=1.0/DT}$
$C$
$\text{FOR AN UNCONFINED AQUIFER, THE STORAGE COEFF. EQUAL TO THE SPECIFIC YIELD,}$
$\text{THEREFORE SSA OBTAINED ABOVE HLDLS ONLY FOR CONFINED AQUIFER, MAKE}$
$\text{THE CORRECTION AS SSA=CF.CC3125, SEE HYDROLOGY HANDBOOK BY CHW}$
$C$
$\text{REAL(IR,19) NPCV,CXV,SSTA}$
$C$
$\text{SSKI=SSTA/XK}$
$\text{WRITE(IW,31) NFSF,NFSL,NBSL,MFSF,MFSL,MBSF,MBSL,NLEFT,}$
$\text{*NRIGT,NBCNC,LITER,AST,INNER,SYE,FIE,YLENS,FLEPS,FCON,SSTA}$
$-137-$
T=0.0
NPRINT=0
NKV=0

C INITIALIZE THE OVER-ALL MATRIX FOR THE INNER ELEMENTS

150 NSFFT=0
DC 60 I=1,NFSL
60 YFCLC(I)=Y(I)
YLC=Y(LEFT)
YRC=Y(RIGHT)
DC 73 I=1,NPCIN
DC 71 J=1,NPCIN
P(I,J)=0.0
71 C(I,J)=0.0
YLUMY(I)=0.0
HYUMY(I)=0.0
E(I)=0.0
R(I)=0.0

C INITIALIZE THE CORRECTING MATRICES OF OVER-ALL MATRICES FOR EACH ITERATION
DUE TO EFFECT OF CHANGING FREE SURFACE

C
DC 63 I=1,NFSL
63 RCLC(I)=0.0
DC 64 I=1,NFSL
DC 64 J=1,3
BCC1(I,J)=0.0
BCC2(I,J)=0.0
BCC3(I,J)=0.0
SKNC1(I,J)=0.0
SKNC2(I,J)=0.0

64 SKNC3(I,J)=0.0
DC 1022 I=1,4
DC 1022 J=1,3
1022 BFLX(I,J)=0.0

C FORMATION OF ELEMENT STIFFNESS MATRICES
OF INNER ELEMENTS WHICH ARE NOT ADJACENT TO THE FREE SURFACE
C
C Ni=Since use the same program loop to calculate element matrix of both
C INNER AND FREE SURFACE ELEMENT, THEREFORE, USE Ni AS CONTROL INTEGER
C TO SKIP COMPUTER JOB TO APPROPRIATE LOOP
C
C NJ=SAME PURPOSE AS Ni EXCEPT TO CONTROL WHETHER OR NOT TO FIND ALL
C PROPERTIES AT TIME STEP N-1
C
N=NFSL+1
NN=NELEE
Ni=O
NJ=O
201 DC 61 I=N,NN
C TRANSFROM CCCRCINATES OF THE THREE NODALS OF TRIANGLE ELEMENT TC
C NEW CENTRIC CCCRCINATES SYSTEM
C
I1=NCC1(I)
I2=NCC2(I)
I3=NCC3(I)
AA=0.33333*(X(I1)+X(I2)+X(I3))
BB = 0.33333 * (Y(I1) + Y(I2) + Y(I3))

XN1 = X(I1) - AA
XN2 = X(I2) - AA
XN3 = X(I3) - AA
YN1 = Y(I1) - BB
YN2 = Y(I2) - BB
YN3 = Y(I3) - BB

FINC VALUES REQUIRED FOR ELEMENT STIFFNESS MATRICES

A1 = XN2 * YN3 - XN3 * YN2
A2 = XN3 * YN1 - XN1 * YN3
A3 = XN1 * YN2 - XN2 * YN1
B1 = YN2 - YN3
B2 = YN3 - YN1
B3 = YN1 - YN2
C1 = XN3 - XN2
C2 = XN1 - XN3
C3 = XN2 - XN1

XX2 = XN1 * XN1 + XN2 * XN2 + XN3 * XN3
YY2 = YN1 * YN1 + YN2 * YN2 + YN3 * YN3
XY = XN1 * YN1 + XN2 * YN2 + XN3 * YN3

DELTa = XN2 * YN3 + XN3 * YN1 + XN1 * YN2 - YN1 * XN2 - YN2 * XN3 - YN3 * XN1
DELTa = 0.5 * DELTA
OCELT = 1.0 / DELTA
OCEL = 0.25 * OCELT

F11 = OCELT * (A1 * A1 + 0.08333 * B1 * B1 * XX2 + 0.08333 * C1 * C1 * YY2 + 0.16667 * E1 * XY)
F33 = OCELT * (A3 * A3 + 0.08333 * B3 * B3 * XX2 + 0.08333 * C3 * C3 * YY2 + 0.16667 * B3 * C3 * XX2)

AA = B1 * C2 + C1 * B2
F12 = OCELT * (A1 * A2 + 0.08333 * AA * XY + 0.08333 * B1 * B2 * XX2 + 0.08333 * C1 * C2 * YY2)

CONTINUE IF (A1) 202, 202, 203
FINC THE OVER-ALL MATRIX FCR THE INNER ELEMENTS

202 N=PFSL+1
NN=NELEM
206 DO 72 I=N,NN
   I1=ACC1(I)
   I2=ACC2(I)
   I3=ACC3(I)
   P(I1,I1)=P(I1,I1)+8C1(I,1)
   P(I1,I2)=P(I1,I2)+8C2(I,1)
   P(I1,I3)=P(I1,I3)+8C3(I,1)
   P(I2,I1)=P(I2,I1)+8C1(I,2)
   P(I2,I2)=P(I2,I2)+8C2(I,2)
   P(I2,I3)=P(I2,I3)+8C3(I,2)
   P(I3,I1)=P(I3,I1)+8C1(I,3)
   P(I3,I2)=P(I3,I2)+8C2(I,3)
   P(I3,I3)=P(I3,I3)+8C3(I,3)
   Q(I1,I1)=Q(I1,I1)+SKN1(I,1)
   Q(I1,I2)=Q(I1,I2)+SKN2(I,1)
   Q(I1,I3)=Q(I1,I3)+SKN3(I,1)
   Q(I2,I1)=Q(I2,I1)+SKN1(I,2)
   Q(I2,I2)=Q(I2,I2)+SKN2(I,2)
   Q(I2,I3)=Q(I2,I3)+SKN3(I,2)
   Q(I3,I1)=Q(I3,I1)+SKN1(I,3)
   Q(I3,I2)=Q(I3,I2)+SKN2(I,3)
   Q(I3,I3)=Q(I3,I3)+SKN3(I,3)
72 CONTINUE
   IF(NKV) 1005,1005,133
1005 CONTINUE

FINC CONSTANT MATRICES DUE TO EFFECTS OF BOTTOM DRAINAGE VELOCITY
AT INITIAL TIME

CALL CRANV(T,XW,VCLMY)

CCNVERT DRAINAGE VELOCITY FROM VDUMY (I) INTO V(I), THIS PART CNLY HCLDS CN
THE PARTICULAR NCEF-ELEMENT CONFIGURATI0N CONSIDERED

NN=MBS-3
NE=1
74 DO 1=2,NN,4
V(I)=VDUMY(NE)
V(I+1)=-VDUMY(NE+1)
V(I+2)=0.0
V(I+3)=0.0
NE=NE+2
1 CONTINUE

V(I+4)=-VDUMY(NE)
V(I+5)=-VDUMY(NE+1)
V(I+6)=0.0
NE=0
75 DO 75 I=MBSF,MBSL
   NE=NE+1
   I1=ACC1(I)
   I2=ACC2(I)
   R(I1)=R(I1)-C.5*V(NE)*(X(I2)-X(I1))
   R(I2)=R(I2)-C.5*V(NE)*(X(I2)-X(I1))
75 -140 -
EFFECTS OF HORIZONTAL FLUX ON LEFT HAND SIDE BOUNDARY WHERE HYDROSTATIC CONDITON IS PRESUMELY EXISTED. THEREFORE, COMBINE ALL EFFECTS BY

DYE=THE TOTAL LENGTH OF THE WHOLE LEFT BOUNDARY

I=FSF
DYE=Y(NLEFT)
NFLUX=1

1012 I1=NCC1(1)
I2=NCC2(1)
I3=NCC3(1)
AA=0.33333*(X(I1)+X(I2)+X(I3))
BB=0.33333*(Y(I1)+Y(I2)+Y(I3))
XN1=X(I1)-AA
XN2=X(I2)-AA
XN3=X(I3)-AA
YN1=Y(I1)-BB
YN2=Y(I2)-BB
YN3=Y(I3)-BB
B1=YN2-YN3
B2=YN3-YN1
B3=YN1-YN2
DELTA=XN2*YN3+XN3*YN1+XN1*YN2-YN1*XN2-YN2*XN3-YN3*XN1
DELTA=0.5*DELTA
DELTA=0.25/DELTA
AA=XK*DYE*DELTA

P(I1,I1)=P(I1,I1)+AA*B1-BFLX(NFLUX,1)
P(I1,I2)=P(I1,I2)+AA*B2-BFLX(NFLUX,2)
P(I1,I3)=P(I1,I3)+AA*B3-BFLX(NFLUX,3)
P(I2,I1)=P(I2,I1)+AA*B1-BFLX(NFLUX,1)
P(I2,I2)=P(I2,I2)+AA*B2-BFLX(NFLUX,2)
P(I2,I3)=P(I2,I3)+AA*B3-BFLX(NFLUX,3)
BFLX(NFLUX,1)=AA*B1
BFLX(NFLUX,2)=AA*B2
BFLX(NFLUX,3)=AA*B3
GO TO (1013,1C14,1C15,1C16),NFLUX

EFFECTS OF HORIZONTAL FLUX ON RIGHT HAND SIDE BOUNDARY WHERE HYDROSTATIC CONDITON IS EXISTED, THEREFORE, THE EFFECTS OF INDIVIDUAL ELEMENT MUST BE CONSIDERED SEPERATELY

1013 I=40
DYE=Y(69)-Y(70)
NFLUX=NFLUX+1
GO TO 1012

1014 I=99
DYE=Y(68)-Y(69)
NFLUX=NFLUX+1
GO TO 1012

1015 I=110
DYE=Y(67)-Y(68)
NFLUX=NFLUX+1
GO TO 1012

1016 CONTINUE

THE FOLLOWING PROGRAM REQUIRED FOR EACH TIME CYCLE

133 CONTINUE
T=T+CT
WRITE(IW,12)T
NPRINT=NPRINT+1
SETUP CLC COORDINATES OF FREE SURFACE BEFORE ITERATION

DC 76 I=1,NFSL
76 YCLC(I)=Y(I)

YLC= Y(LEFT)
YRCLC= Y(RIGHT)

NITER=0

130 NITER=NITER+1
NI=1
N=FSF
NN=FSL

SETUP ELEMENT MATRICES OF ELEMENTS ALONG FREE SURFACE

GC T0 201

CORRECT ELEMENT MATRICES DUE TO EFFECT OF FREE SURFACE

203 DC 80 I=N,NN
I2=ACC2(I)
I3=ACC3(I)
IF(I.EQ.1) GC T0 84
IF(I.EQ.FSFL) GC T0 84
IF(I2.NFSL) 81,81,80
IF(I3.NFSL) 84,84,80
84 CC=X(I2)-X(I3)

DC=(CC*SYE)/6.0

SKN2(I,2)=SKN2(I,2)+2.0*CC
SKN3(I,2)=SKN3(I,2)+CC
SKN2(I,3)=SKN2(I,3)+CC
SKN3(I,3)=SKN3(I,3)+2.0*CC
R(I2)=R(I2)+C.5*FIE*CC
R(I3)=R(I3)+C.5*FIE*CC

CONTINUE

DC 87 I=1,NFSL
R(I)=R(I)-RCLC(I)
RCLC(I)=R(I)

SETUP COVER-ALL MATRICES DUE TO ELEMENTS ALONG FREE SURFACE

DC 88 I=1,FSFL
I1=ACC1(I)
I2=ACC2(I)
I3=ACC3(I)
P(I1,I1)=P(I1,I1)+BC1(I,1)-BC01(I,1)
P(I1,I2)=P(I1,I2)+BC2(I,1)-BC02(I,1)
P(I1,I3)=P(I1,I3)+BC3(I,1)-BC03(I,1)
P(I2,I1)=P(I2,I1)+BC1(I,2)-BC01(I,2)
P(I2,I2)=P(I2,I2)+BC2(I,2)-BC02(I,2)
P(I2,I3)=P(I2,I3)+BC3(I,2)-BC03(I,2)
P(I3,I1)=P(I3,I1)+BC1(I,3)-BC01(I,3)
P(I3,I2)=P(I3,I2)+BC2(I,3)-BC02(I,3)
P(I3,I3)=P(I3,I3)+BC3(I,3)-BC03(I,3)
Q(I1,I1)=Q(I1,I1)+SKN1(I,1)-SKNO1(I,1)
Q(I1,I2)=Q(I1,I2)+SKN2(I,1)-SKNO2(I,1)
Q(I1,I3)=Q(I1,I3)+SKN3(I,1)-SKNO3(I,1)
Q(I2,I1)=Q(I2,I1)+SKN1(I,2)-SKNC1(I,2)
Q(I2,I2)=Q(I2,I2)+SKN2(I,2)-SKNC2(I,2)
\[ Q(I_2, I_3) = C(I_2, I_3) + SKN_3(I_2) - SKN_03(I_2) \]
\[ Q(I_3, I_1) = C(I_3, I_1) + SKN_1(I_3) - SKN_01(I_3) \]
\[ C(I_3, I_2) = C(I_3, I_2) + SKN_2(I_3) - SKN_02(I_3) \]
\[ Q(I_3, I_3) = C(I_3, I_3) + SKN_3(I_3) - SKN_03(I_3) \]

\[ HCC_1(I, 1) = HC_1(I, 1) \]
\[ BCC_2(I, 1) = BC_2(I, 1) \]
\[ BCC_3(I, 1) = BC_3(I, 1) \]
\[ BCC_1(I, 2) = BC_1(I, 2) \]
\[ BCC_2(I, 2) = BC_2(I, 2) \]
\[ BCC_3(I, 2) = BC_3(I, 2) \]
\[ BCC_1(I, 3) = BC_1(I, 3) \]
\[ BCC_2(I, 3) = BC_2(I, 3) \]
\[ BCC_3(I, 3) = BC_3(I, 3) \]

CONTINUE

IF(NJ) 204, 204, 205

C FIND PARTIAL DERIVATIVE OF TIME AT TIME \( (T-\Delta T) \)

204 CONTINUE

NN=NPCIN/5
NB=NN*5-1
NCV=NN*5+1

C ANC REAC B. C. AT TIME \( T \)

REAC(IR, 23) YLEFT, YRIGT, DLMY, DUMY
DC 101 I=1, NPCIN
AA=0, 0
DC 102 J=1, NPCIN
AA=AA-P(I, J)*H(J)

102 CONTINUE

E(I)=E(I)+AA*R(I)

101 CONTINUE

C FIND PRESCRIBED TIME-DERIVATIVES OF BOUNDARY AT TIME \( (T-\Delta T) \)

AA=(YLEFT-YLCLC)/CT
BB=(YRIGT-YRCLC)/CT
NN=NLEFT+NBCNC-1

DC 146 I=NLEFT, NN

HT(I)=AA
HT(NRIGT)=BB
NFI=NRIGT-1
DC ': !' =1, NFI
DC 149 J=NRIGT, NPCIN

149 E(I)=E(I)-C(I, J)*HT(J)

148 CONTINUE

C SETUP DUMMY MATRICES \( C(IJ) \) FOR ARRAY AND SIMQ TO AVOID DESTROYING OF
C MATRIX \( C(IJ) \)

DC 145 I=1, NPCIN
DO 145 J=1,NPCIN
145 D(I,J)=C(I,J)
CALL ARRAY(2,NFI,NFI,NPCIN,NPOIN,C,D)
CALL SIMCM(C*E,NFI,0)

DC 106 I=1,NFI
106 HT(I)=E(I)

C FIND CONSTANT MATRICES DUE TO EFFECTS OF BOTTOM DRAINAGE VELOCITY
C READ DRAINAGE VELOCITY AT TIME T
C CALL CRANV(T,XW,VCUMY)
C
C CONVERT DRAINAGE VELOCITY FROM VDUMY(I) INTO V(I), THIS PART ONLY HOLDS FOR
C THE PARTICULAR NODE-ELEMENT CONFIGURATION CONSIDERED

NN=MCBS-3
NE=1
V(1)=0.0
DO 107 I=2,NN,4
V(I)=VCUMY(NE)
V(I+1)=VCUMY(NE+1)
V(I+2)=0.0
V(I+3)=0.0
NE=NE+2
107 CONTINUE
V(I+4)=VCUMY(NE)
V(I+5)=VCUMY(NE+1)
V(I+6)=0.0
DO 104 I=NBSF,NBSL
104 R(I)=0.0
R(NLEFT+NBCNC-1)=C*0
R(NRIGHT-NBCNC+1)=C*0
NE=0
DO 105 I=MBSF,MBSL
105 R(I)=0.0
R(NLEFT+NBCNC-1)=C*0
R(NRIGHT-NBCNC+1)=C*0
NE=NE+1
I1=NCC1(I)
I2=NCC2(I)
R(I1)=R(I1)-0.5*V(NE)*(X(I2)-X(I1))
R(I2)=R(I2)-0.5*V(NE)*(X(I2)-X(I1))

C EFFECTS OF HORIZONTAL FLUX ON LEFT HAND SIDE BOUNDARY WHERE HYDROSTATIC
C CONDICTION IS PRESUMED ESISTED. THEREFORE, COMBINE ALL EFFECTS BY
C DYE=THE TOTAL LENGTH OF THE WHOLE LEFT boundary
C
I=FLSF
DYE=Y(NLEFT)
NFLUX=1
1017 I1=NCC1(I)
I2=NCC2(I)
I3=NCC3(I)
AA=0.33333*(X(I1)+X(I2)+X(I3))
BB=0.33333*(Y(I1)+Y(I2)+Y(I3))
XN1=X(I1)-AA
XN2=X(I2)-AA
XN3=X(I3)-AA
YN1=Y(I1)-BB
YN2=Y(I2)-BB

-144-
YN3 = Y(I3) - BB
B1 = YN2 - YN3
B2 = YN3 - YN1
B3 = YN1 - YN2
DELT = XN2*YN3 + XN3*YN1 + XN1*YN2 - YN1*XN2 - YN2*XN3 - YN3*XN1
DELTA = 0.5*DELTA
DDEL = 0.25/DELTA
AA = XK*DYE*CCEL
P(l1,l1) = P(l1,l1) + AA*B1 - BFLX(NFLUX,1)
P(l1,l2) = P(l1,l2) + AA*B2 - BFLX(NFLUX,2)
P(l1,l3) = P(l1,l3) + AA*B3 - BFLX(NFLUX,3)
P(l2,l1) = P(l2,l1) + AA*B1 - BFLX(NFLUX,1)
P(l2,l2) = P(l2,l2) + AA*B2 - BFLX(NFLUX,2)
P(l2,l3) = P(l2,l3) + AA*B3 - BFLX(NFLUX,3)
BFLX(NFLUX,1) = AA*E1
BFLX(NFLUX,2) = AA*E2
BFLX(NFLUX,3) = AA*E3
GC TC (1018,1C19,1C20,1C21),NFLUX

C EFFECTS OF HORIZONTAL FLUX ON RIGHT HAND SIDE BOUNDARY WHERE AC HYDRO-
C STATICS IS EXISTED THEREFORE THE EFFECTS OF INDIVIDUAL ELEMENT MUST BE
C CONSIDERED SEPERATELY

C 1018 I=40
DYE = Y(69) - Y(7C)
NFLUX = NFLUX + 1
GC TC 1017

1019 I=90
DYE = Y(6d) - Y(69)
NFLUX = NFLUX + 1
GC TC 1017

1020 I=110
DYE = Y(67) - Y(68)
NFLUX = NFLUX + 1
GC TC 1017

1021 CONTINUE

C DC110 I=1,NPCIN
BB = HT(I) + 2.0*CCT*P(I)
110 HT(I) = BB

C SETUP PRESCRIBED PIEZOMETER HEAD OF BOUNDARY AT TIME T

C NN = NLEFT + NBCNC - 1
DC 114 I = NLEFT,NN
114 H(I) = YLEFT
Y(NLEFT) = YLEFT
H(NRIGT) = YRIGT
Y(NRIGT) = YRIGT

C TC FIND H(I) AT TIME T

C 205 CONTINUE
DC 111 I=1,NPCIN
DG 111 J=1,NPCIN
111 D(I,J) = P(I,J) + 2.0*CCT*Q(I,J)
DC 112 I=1,NPCIN
AA = 0.0
DG 113 J=1,NPCIN
AA = AA + C(I,J) * HT(J)
113 CONTINUE
E(I)=AA*R(I)

112  C  CONTINUE
C
C  SUBSTITUTING  PR-DESCIBED VALUE FOR SOME H(I)
C
NFI=NRIGT-1
DC 116 I=1,NFI
DC 117 J=NRIAR,T,NPCIN
117  E(I)=E(I)-C(I,J)*J(J)
116  CONTINUE
C
CALL ARRAY(2,NFI,NFI,NPCI,NPCI,U,C)
CALL INUP(C,E,NFI,KS)
DC 118 I=1,NFI
118  H(I)=E(I)
IF(NPRINT-10) 1009,1008,1CC9
1008  C  CONTINUE
WRITE(Iw,32)
WRITE(Iw,33)
DO 92 I=1,66+t
92  WRITE(Iw,8) T(I),T(I+1),H(I+2),H(I+3),H(I+4),H(I+5)
C
C  CORRECT THE POSITION OF FREE SURFACE
C
EPS=CONVEGENT CRITERIA
FCCCN=COUNTING FACTOR TO GET CONVEGENT CRITERIA
FLEPS=LENGTH OF BOTTOM BOUNDARY
C
1009  NCCNV=O
AA=YRIGT-YRCLC
BB=YLEFT-YLCLC
AA=ABS(AA)
BB=ABS(BB)
IF(AA.GE.BB) GO TO 141
EPS=BB*FCCCN
GO TO 142
141  EPS=AA*FCCCN
142  CONTINUE
C
NEXT CARD CAN BE CHANGED, NOW ( EPS.GE.4.5 ) BE HSCEEN
C
IF(EPS-4.5) 144,143,143
144  EPS=4.5
143  DC 120 I=NFSF,NFSL
CC=(X(I)+50.C)/FLEPS+0.4
CC=CC+EPS
AA=ABS(H(I)-YCLC(I))
IF(AA-EPS) 121,122,122
122  AA=T(I)-YCLC(I)
IF(AA) 123,124,124
124  H(I)=YCLC(I)+C.05*CC
GO TO 120
123  H(I)=YCLC(I)-C.05*CC
GO TO 120
121  NCCNV=NCCNV+1
120  CONTINUE
DC 125 1=NFSF,NFSL
125  Y(I)=T(I)
IF(wh sinh-10) 1011,1010,1011
1010 CONTINUE
WRITE(1W,34)
93 WRITE(1k,8) Y(1),Y(1+1),Y(1+2),Y(1+3),Y(1+4),Y(I+5)
WRITE(1w,35)
WRITE(1w,39)Y(19),Y(67),Y(71),FY,FD,FD
NPRINT=0
1011 CONTINUE
IF(NITER>:-10) 126,127,127
127 WRITE(1k,50)
126 IF(NCCNV=AFSL) 129,128,128
129 NJ=1
126 IF(T=T) 132,131,131
132 NJ=0
1000 NS=FT=NSHFT+1
1 IF(NS=FT=EC,AST) GC TC 134
1 GC TC 133
C
C IF ANY NODE CAN F.S. SHIFTING TOO MUCH
C
134 DC 135 I=1,NFSL
135 CONTINUE
AA=ABS(Y(I)-YFCLC(I))
IF(AA,CT,YLENS) GC TO 139
135 CONTINUE
NS=FT=0
GC TC 133
C
C THE FOLLOWING PROGRAM CAN BE APPLIED TO SPECIAL NODE-ELEMENT
C CONFIGURATION CONSIDERED IN THIS PROGRAM
C
C SHIFTING THE NODES OF SECOND ROW
C
139 N=NFSL+1
136 NE=NFSL
138 DC 137 I=N,NN
137 CY=Y(NF)-YFCLC(NF)
138 YY(I)=Y(I)+CY
139 YY(I)=Y(I)+CY
C
C SHIFTING THE NODES OF THIRD ROW
C
N=NN+1
137 NE=NE+1
136 NE=NFSL/2
138 DC 137 I=NN,NN
137 CY=Y(NF)-YFCLC(NF)
138 YY(I)=Y(I)+CY
139 YY(I)=Y(I)+CY
C
C SHIFTING THE NODES OF FOURTH ROW
C
N=NN+1
137 NE=NE+2
136 NE=NFSL/2-1
138 DC 137 I=NN,NN
137 CY=Y(NF)-YFCLC(NF)
138 YY(I)=Y(I)+CY
139 YY(I)=Y(I)+CY
-147-
NE=NE+2

THE FOLLOWING PROGRAM IS USED TO CORRECT H(I) DUE TO SHIFTING THE CCORD.

CORRECT H(I) CF NCES ON SECOND ROW
N=NFSL+1
N+2*NFSL
NJ=1

312 DC 299 JJ=N,N
M1=NUM(JJ-NFSL,1)
M2=NUM(JJ-NFSL,2)
M3=NUM(JJ-NFSL,3)
M4=NUM(JJ-NFSL,4)
I1=NCC1(M1)
I2=NCC2(M1)
I3=NCC3(M1)
IF(YUMY(JJ).LT.Y(11)) GO TO 306
NI=1

310 AA=0.33333*(X(11)+X(12)+X(13))
BB=0.33333*(Y(11)+Y(12)+Y(13))
WN1=X(11)-AA
WN2=X(12)-BB
WN3=X(13)-BB

FINC VALUES REQUIRED FOR ELEMENT STIFFNESS MATRICES

A1=XN2*YN3-XN3*YN2
A2=XN3*YN1-XN1*YN3
A3=XN1*YN2-XN2*YN1
B1=YN2-YN3
B2=YN3-YN1
B3=YN1-YN2
C1=YN3-XN2
C2=YN1-XN3
C3=YN2-XN1
XX2=XN1*XN1+XN2*XN2+XN3*XN3
YY2=YN1*YN1+YN2*YN2+YN3*YN3
XY=XN1*YN1+XN2*YN2+XN3*YN3
DELT=YN2*YN3*YN1+YN2*YN1*YN2*YN2*YN3*YN3
DELT=0.5*DELT
CDELT=1.0/DELT
FN1=0.5*CDELT*(A1+B1*(X(JJ)-AA)+C1*(YUMY(JJ)-BB))
FN2=0.5*CDELT*(A2+B2*(X(JJ)-AA)+C2*(YUMY(JJ)-BB))
FN3=0.5*CDELT*(A3+B3*(X(JJ)-AA)+C3*(YUMY(JJ)-BB))

302 HI=FN1+F(11)+FN2+F(12)+FN3+F(13)
304 GC TC(300,301,305,301),NI
300 I1=NCC1(M1)
I2=NCC2(M1)
I3=NCC3(M1)
NI=2
GC TC 310
303 H2=FN1+F(11)+FN2+F(12)+FN3+F(13)
GC TC 304
301 HUMY(JJ)=0.5*(H1+F2)

-148-
GC TC 299
306 I1=NCC1(M12)
I2=NCC2(M12)
I3=NCC3(M12)
N1=3
GC TC 310
305 I1=NCC1(M13)
I2=NCC2(M13)
I3=NCC3(M13)
N1=4
GC TC 310
299 CONTINUE
GC TC(311,313),NJ
C CORRECT H(I) CF THE NCCES CN FOURTH ROW
311 N=NN+1
NN=NN+NFSL/2
N=NN+1
NJ=2
NN=NN+NFSL/2-1
GC TC 312
C CORRECT H(I) CF THE NCCES CN THIRD ROW
313 CONTINUE
N=NFSL+1
NN=2*NFSL
N=NN+1
NN=NN+NFSL/2
DC 314 JJ=NN+1
MI1=MAM(JJ-NFSL+1)
MI2=MAM(JJ-NFSL+2)
MI3=MAM(JJ-NFSL+3)
I1=NCC1(M13)
I2=NCC2(M13)
I3=NCC3(M13)
IF(YCUMY(JJ).LT.Y(I1)) GO TO 315
NI=1
318 AA=0.33333*(X(I1)+X(I2)+X(I3))
BB=0.33333*(Y(I1)+Y(I2)+Y(I3))
XN1=X(I1)-AA
XN2=X(I2)-AA
XN3=X(I3)-AA
YN1=Y(I1)-BB
YN2=Y(I2)-BB
YN3=Y(I3)-BB
C FIND VALUES REQUIRED FOR ELEMENT STIFFNESS MATRICES
C
A1=XX2*YN3-XN3*YN2
A2=XX3*YN1-XN1*YN3
A3=XX1*YN2-XN2*YN1
B1=YN2-YN3
B2=YN3-YN1
B3=YN1-YN2
C1=XX3-XN2
C2=XX1-XN3
C3=XX2-XN1
XX2=XX1*XN1+XX2*XN2+XN3*XN3
YY2=YN1*YN1+YN2*YN2+YN3*YN3
XY=XX1*YN1+XX2*YN2+XN3*YN3
DELTAX=XX2*YN3+XN3*YN1+XN1*XN2-YN1*XN2-YN2*XN3-YN3*XN1
DELTAY=DELTAY
DELTA=0.5*DELTAY
DELTA=1.0*DELTAY
-149-
\[
FN1 = 0.5 * CCEL \times (A1 + B1 \times (X(JJ) - AA) + C1 \times (YDUWY(JJ) - BB))
\]
\[
FN2 = 0.5 * CCEL \times (A2 + B2 \times (X(JJ) - AA) + C2 \times (YDUWY(JJ) - BB))
\]
\[
FN3 = 0.5 * CCEL \times (A3 + B3 \times (X(JJ) - AA) + C3 \times (YDUWY(JJ) - BB))
\]

GC TC (316, 315, 316), NI
\[
H1 = FN1 \times H(11) + FN2 \times H(12) + FN3 \times H(13)
\]

GC TC (317, 321, 322), NI
\[
H2 = FN1 \times H(11) + FN2 \times H(12) + FN3 \times H(13)
\]

GC TC 314
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 320
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 318
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 314
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 320
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 318
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 314
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 320
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 318
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 314
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 320
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]

GC TC 318
\[
H2UMY(JJ) = 0.5 \times (H1 + H2)
\]
WRITE(IW,37)
DC 338 I=1,72,6
338 WRITE(IW,8) Y(I),Y(I+1),Y(I+2),Y(I+3),Y(I+4),Y(I+5)
WRITE(IW,8) Y(73),Y(74),FC,FC,FC,FD
NKV=1
1001 GC TC 150
131 STCP
ENC

-151-
SUBROUTINE DRAV(V, XW, V)
C SUBROUTINE DRAV FOR FINDING DRAINAGE VELOCITY OF WELLS AND EVEN ELEMENTS
C WITH GIVEN FOURIER COEFFICIENTS OF WELL DRAINAGE VELOCITY
C ALL UNITS SHOULD BE IN C.G.S. SYSTEM
C PERIOD = 100*15/60, WHERE 100 MEANS 100 CYCLES, 15 MEANS 15 MINUTES PER
C CYCLE, 60 MEANS 1 MINUTE EQUAL TO 60 SECONDS
C INPUT A(I,J) = FOURIER COSINE COEFFICIENTS OF WELL DRAINAGE VELOCITY
C B(I,J) = FOURIER SINE COEFFICIENTS OF WELL DRAINAGE VELOCITY
C XW(I) = X-CORDINATES OF 13 WELLS
C CX = LENGTH OF EVEN ELEMENTS
C T = CURRENT TIME
C NPCIl = TOTAL NO. OF NODAL POINTS MINUS 1
C OUTPUT
C V(I) = DRAINAGE VELOCITY OF EVEN ELEMENTS
C A(I,J) = FOURIER COSINE COEFFICIENTS
C B(I,J) = FOURIER SINE COEFFICIENTS
DIMENSION VW(12), V(20), A(12, 6), B(12, 6), XW(13)
CC=2.0*3.1416/(15.0*60.0)
DC 51 I=1,12
SU=A(I,1)
DC 52 J=2,6
AA=A(I,J)
BB=B(I,J)
FJ=J-1
DD=FJ*CC*T
52 SUM=SUM+AA*COS(CC)+BB*SIN(CC)
51 VW(I)=SUM
C FIND DRAINAGE VELOCITY BETWEEN TWO WELLS BY FOURIER SERIES APPROXIMATION
CC=2.0*3.1416/(15.0*60.0)
DC 51 I=1,12
SU=A(I,1)
DC 52 J=2,6
AA=A(I,J)
BB=B(I,J)
FJ=J-1
DD=FJ*CC*T
52 SUM=SUM+AA*COS(CC)+BB*SIN(CC)
51 VW(I)=SUM
C FIND DRAINAGE VELOCITY FOR EACH ELEMENT WITH EVEN LENGTH
X=0.5*CX
DC 60 I=1,NPCI1
DC 61 J=1,12
IF(X-XW(J+1)) V(I)=VW(J)
AA=XW(J+1)-X
IF(AA-0.5*CX) 64,69,69
64 BB=X+0.5*CX-XW(J+1)
CC=(0.5*DX+AA)*VW(J)+BB*VW(J+1)
V(I)=CC/ADX
GO TO 69
65 V(I)=0.5*VW(J+1)+0.5*VW(J)
GO TO 69
66 AA=X-XW(J+1)-0.5*CX
IF(AA) 67,65,65
67 BB=X-XW(J+1)+0.5*CX
CC=XW(J+1)-X+0.5*CX
V(I)=(BB*VW(J+1)+CC*VW(J))/DX
GO TO 69
61 CONTINUE
69 X=X+CX
60 CONTINUE
RETURN
END
SUBROUTINE SIMQM(A, B, N, KS)
DIMENSION A(1), B(1)
C FORWARD SOLUTION
TCL=0.0
KS=0
JJ=-N
65 I=1,NPCI1
IF(JJ<0) 66,65,65
66 X*=X
65 JJ=JJ-1
GO TO 66
61 CONTINUE
RETURN
END
00
65 J=1,N
JJ=JJ+N+1
BIGA=0
IF=JJ-J
DO 30 I=J,N

C SEARCH FOR MAXIMUM COEFFICIENT IN COLUMN
I=IT+I
IF(ABS(BIGA)-ABS(A(IJ))) 2C,30,30
20 BIGA=A(IJ)
IMAX=I
30 CONTINUE

C TEST FOR PIVOT LESS THAN TOLERANCE (SINGULAR MATRIX)
IF(ABS(BIGA)-TOL) 35,35,4C
35 KS=1
RETURN

C INTERCHANGE ROWS IF NECESSARY
40 II=J+N*(J-2)
IT=IMAX-J
DO 50 K=J,N
II=II+K
IT=II+IT
SAVE=A(IK)
A(IK)=A(IK)
A(IK)=SAVE

C CIVIC EQUATION BY LEADING COEFFICIENT
50 A(IK)=A(IK)/BIGA
SAVE=E(IMAX)
H(IMAX)=E(JK)
B(JK)=SAVE/BIGA

C ELIMINATE NEXT VARIABLE
IF(J=N) 55,7C,55
55 LCS=N*(J-1)
DC 65 IX=JY,N
IXJ=IJS+IX
IT=J-IX
DO 60 JX=JY,N
IXJX=N*(JX-1)+IX
IXJX=IXJX+IT
A(IJX)=A(IJX)-A(IJX)*A(IJX)
C FCLLCWING THREE CARDS ARE USED FOR ELIMINATING UNDERFLOW
AA=AES(A(IJX))
IF(AA-1.0E-3C) 100,100,60
100 A(IJX)=0.0
60 CONTINUE
B(IK)=B(IK)-(B(JK)*A(IK))
C FULLCWING THREE CARDS ARE USED FOR ELIMINATING UNDERFLOW
BU=AES(B(IK))
IF(EE-1.0E-3C) 101,101,65
101 B(IK)=0.0
65 CONTINUE

C BACK SOLUTION
70 NY=N+1
80 J=1,NY
IA=IT-J
IB=N-J
IC=N
DO 80 K=1,J
B(IE)=B(IE)-A(IA)*B(IE)
IA=IA-N
-153-
80 IC=IC-1
RETURN
END

//GC.SYSAIN CC *
VALUES OF PARAMETERS REQUIRED

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INITIAL DATA OF COORDINATES AND TOTAL HEAD OF ALL NODES

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-157-
C  MNUM(I,J), NUMBERING INDEX OF SOME ELEMENTS AROUND A INNER NODE WHICH IS
NEEDED TO CONSIDER THE CORRECTION OF H(I) AFTER SHIFTING NODAL COORDINATES

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1COS 0.2606E-04 -0.2555E-04 0.3933E-04 -0.1161E-04 0.7523E-05 -0.2118E-05
1SIN 0.0000E 00 -0.4224E-05 -0.1677E-05 0.6999E-05 -0.1689E-04 -0.9095E-05
2COS 0.3689E-04 -0.1483E-04 0.7028E-04 -0.2182E-04 0.1646E-04 0.3183E-05
2SIN 0.0000E 00 -0.6800E-05 -0.2002E-04 0.6440E-05 -0.2374E-04 -0.1631E-04
3COS 0.3666E-04 -0.1003E-04 0.1005E-03 -0.4412E-04 0.2776E-04 0.3765E-05
3SIN 0.0000E 00 -0.8123E-05 -0.5320E-04 0.1999E-05 -0.3054E-04 -0.2445E-04
4COS 0.4971E-04 -0.9646E-05 0.1261E-03 -0.6833E-04 0.2999E-04 0.6248E-05
4SIN 0.0000E 00 -0.7289E-05 -0.8633E-04 -0.3150E-04 -0.2679E-04
5COS 0.6418E-04 -0.8761E-05 0.1453E-03 -0.9076E-04 0.3359E-04 0.6758E-05
5SIN 0.0000E 00 -0.7440E-05 -0.1177E-03 -0.2665E-05 -0.3154E-04 -0.2828E-04
6COS 0.5345E-04 -0.7978E-05 0.1637E-03 -0.1162E-03 0.3628E-04 0.6731E-05
6SIN 0.0000E 00 -0.3818E-05 -0.1470E-03 -0.2576E-05 -0.2888E-04 -0.2791E-04
7COS 0.7328E-04 -0.4882E-05 0.1783E-03 -0.1286E-03 0.4556E-04 0.1265E-04
7SIN 0.0000E 00 -0.1082E-05 -0.1770E-03 -0.8028E-05 -0.2667E-04 -0.2772E-04
8COS 0.8153E-04 -0.7425E-05 0.2038E-03 -0.1508E-03 0.5643E-04 0.1902E-04
8SIN 0.0000E 00 -0.1136E-05 -0.2246E-03 -0.1760E-04 -0.2682E-04 -0.3179E-04
9COS 0.1070E-03 0.1214E-04 0.2185E-03 -0.1908E-03 0.5860E-04 0.2009E-04
9SIN 0.0000E 00 -0.6813E-05 -0.2831E-03 -0.2555E-04 -0.1887E-04 -0.2813E-04
10COS 0.1245E-03 0.8425E-05 0.2222E-03 -0.2390E-03 0.5734E-04 0.2059E-04
10SIN 0.0000E 00 -0.1079E-04 -0.3439E-03 -0.2934E-04 -0.5645E-04 -0.1428E-04
11COS 0.1214E-03 0.2163E-04 0.2230E-03 -0.2915E-03 0.6012E-04 0.2998E-04
11SIN 0.0000E 00 -0.2396E-04 -0.4261E-03 -0.3836E-04 0.2684E-04 0.5779E-05
12COS 0.7925E-04 -0.7718E-04 0.1126E-03 -0.4094E-03 0.2125E-04 0.3222E-04
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RESULTANT WATER TABLES OF ILLUSTRATED EXAMPLE

C TIME T= 0.9000E 04

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C TIME T= 0.1800E 05

HEIGHT OF NONES ALONG FREE SURFACE

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C TIME T= 0.2700E 05

HEIGHT OF NONES ALONG FREE SURFACE

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HEIGHT OF NONES ALONG FREE SURFACE

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HEIGHT OF NONES ALONG FREE SURFACE

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HEIGHT OF NONES ALONG FREE SURFACE

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HEIGHT OF NONES ALONG FREE SURFACE

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C TIME T= 0.9000E 05

HEIGHT OF NONES ALONG FREE SURFACE

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### Investigation of the Water Table in a Tidal Beach

Fluctuations in the beach water table at Virginia Beach, Virginia, were studied to better understand the causes and consequences of the oscillations. Special instrumentation for monitoring the water-table surface was designed and is described in the report. The major environmental variables responsible for the fluctuations were evaluated by multiregression analysis in which the dependent variable was the rise of the water table in a well (over a half-tidal cycle) and the independent variables were atmospheric pressure, the rise in ocean still-water-level (SWL), the distance of the well from the foreshore, and a measure of the contribution of the uprush to the water table.

One and two-dimensional finite-element models were constructed to model the groundwater flow. Results show general accuracy of the methods used. Descriptions of and listings for the FORTRAN computer programs are given.

Changes in foreshore sand volume, as a result of fluctuations in the water table, changes in head gradient, and oscillations in the oceanic SWL were also studied. Predictor equations for changes in quantity of foreshore sand over intervals of one-half and entire tidal cycles are developed. Tests of the equations on independent data indicate that they are more reliable than any developed heretofore.
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