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Model Studies of the Botany Basin Aquifer

by

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AN INVESTIGATION INTO THE POSSIBILITY OF SEA WATER INTRUSION INTO THE BOTANY BASIN AQUIFER BY MEANS OF AN HYDRAULIC FLOW MODEL.

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PREFACE

The research described in this report was carried out by Mr. A.J. Hall as a part requirement for the award of the degree of Master of Technology. The investigation was directed by Mr. A.F.S. Nettleton, Senior Lecturer in Civil Engineering, and the work was carried out in the Geo-hydrology and Soil Engineering Laboratory of the Department of Water Engineering.

The report makes a significant contribution to the evaluation, in a quantitative manner, of the hydro-geological characteristics of the Banksmeadow area of the Botany Basin aquifer.

The Appendix was prepared from data collected by Mr. M.A. Cornell whose original thesis was also a part requirement for the award of the degree of Master of Technology.

SUMMARY

Groundwater studies using a Hele-Shaw, or Viscous Flow Analogy model, were made of a cross-section of the northern end of Sydney's Botany Basin. The section modelled is not typical of the Basin as a whole, but represents an area in the suburb of Banksmeadow where heavy pumping occurs. This section also follows the deepest part of the aquifer, and hence it is located in an area where salt water intrusion is most likely to occur. In order to study the effects of fresh water outflows on the sea water - fresh water interface, under conditions of varying water table slopes and varying amounts of pumping from the aquifer, a total of 150 test runs were made on the model.

Lack of field data, particularly relating to the geological structure of the Basin, necessitated the assumption of a geologically homogeneous aquifer. There must consequently be some doubt as to the validity of the seaward fresh water outflows, and thus the fresh water-salt water interfaces obtained.

Theoretical fresh water-salt water interfaces are also presented and compared with those obtained from the model. In all theoretical considerations the transition zone between the fresh and salt water was not considered as this could not be simulated on the model,

A brief report on an investigation of sea water pollution in this same area by chemical means is presented in the Appendix to this publication.

The natural unpumped underflow into Botany Bay through the northern shoreline is estimated to be of the order of 25 million gallons per day, and this figure can be compared with Griffin's (1962) estimate of 8 million gallons per day for the pumpage from the entire basin.

1. THE BOTANY BASIN

The Botany Basin is situated on the southern boundary of the City of Sydney. Figure No. 1 shows the northern section of the Basin, and indicates the recreation, and/or open spaces, as well as the drainage intake areas suggested by Griffin (1962). In the north-western corner it is densely populated and heavily industrialised, whilst in the eastern section there are large, open recreational areas over the north-eastern corner and along the Lachlan Swamps intake area.

Attracted by the relatively flat, cheap building sites, (convenient both to the city and to sea access through Botany Bay), and by the chance of obtaining good quality groundwater for only the cost of bore sinking and pumping, many industries, which are large users of water, have been established within the Botany Basin. These range from wool scourers and tanneries to paper manufacturers and chemical and food processors; the latter seeking, and obtaining, water of comparable quality to that available from Sydney's normal reticulation system.

The aquifers consist of unconsolidated deposits of Quaternary age overlying relatively impervious Triassic Hawkesbury Sandstone and Wianamatta Shales. The sediments vary in thickness from a few feet at the periphery to a maximum of approximately 250 feet. Hawkesbury Sandstone, which encircles the Basin, is covered along the eastern coastal sector by recent aeolian sands and, to the west and south, is capped by Wianamatta Shales.

Drilling logs show that the sand beds of the aquifer also contain lenticular beds of clay, peat, gravel and ferruginous cemented sands at varying depths. Partial cementation of the sand in some places has occurred through secondary deposition of either organic matter, carbonates, or iron oxide. These "consolidated" horizons are usually less than a few feet thick and the resulting formation is popularly termed "Waterloo Rock."

The average annual rainfall over the northern section of the Basin was approximately 44 inches for the period 1935 to 1959. Pan evaporation figures at The Sydney Weather Bureau, Observatory Hill, are approximately 40 inches per year. However, this latter value is not directly applicable to the Botany Basin area; but allowing for the higher temperatures and windier conditions, it is suggested that the potential free water surface evaporation is very probably not less than the annual rainfall in this area.

2. THE MODELLED SECTION

The location of the modelled section, and bores for which some data are available, are shown in Figure No.2. The section runs approximately N.N.E. from a point 10 chains west of Government Pier to the centre of Kingsford shopping area. Its direction and position were chosen as a result of consideration of the data presented in Figures Nos. 2 to 5.

The section follows a line along which the heaviest pumping occurs. Bore Nos. 232, 233 and 234 lie close to the section, and together form a group of wells capable of pumping 30,000 g.p.h. These are represented by Drain No.3 in the section detail of Figure No.6. The drain depth of 115 feet is the average of the lower portions of the casings of these three bores. Drain No. 1 represents the group of bores viz. Nos. 216, and 225 to 231. Heavy pumping also occurs near the middle of the section with Bore Nos. 173, 174, 194, 196, 198 each having recorded pumping rates of 11-14,000 g.p.h. Drain Nos. 2 and 4 were placed on the top of the aquiclude to demonstrate, if required, the effect of pumping from fully penetrating wells.

The base rock contours shown in Figure No. 3 reveal that the aquiclude slopes S.S.W. in the direction of the section. The modelled section does not represent average conditions in the Basin as a whole, but is restricted essentially to the Banksmeadow area. However, being located along the line of heaviest pumping, and where the aquifer is thickest, it covers an area of lower outflow velocities. It consequently includes the most likely zone in which the greatest inland intrusion of salt water might occur.

The topographical contours (Figure No.4) show the section to be in the middle of a low depression. The ground surface possibly slopes more in a north-south direction and thus cuts across the model section. Figure No. 5 suggests that the unpumped water levels fall to the south-west, but these water levels are most variable and rather unreliable. No standard techniques, to allow for either recovery after pumping or the drawdown effects of other pumping bores in the area, had been used in their estimation. Considering all factors, the chosen section is as close as can be estimated to the actual direction of ground water flow.

The model section profiles (Figure No.6) were drawn from the contour of maps of Figure Nos. 3 to 5. From Drain No. 4 seaward, a straight line was assumed as no data were available. (Should the bed rock profile rise, the fresh water-salt water interface would still not be affected as the toes of all interfaces studied were inland from Drain No. 4).

The assumed phreatic surface lines shown in Figure No. 6 cover the widest possible range of hydraulic gradients which may exist in the aquifer. In test Series 11 a rise of 50 feet over the 8,350 feet of the modelled land area was considered. This represented a slope of 1 in 167 which is a flatter slope than the 1 in 120 suggested by Griffin (1962). Assuming a steady state unpumped condition and a straight line phreatic surface at 1 in 167, over a quarter of a mile of the low land situated 60 chains from the shoreline would be flooded to a depth of five feet. In the Basin the phreatic surface would probably be much closer to the higher ground surface at the upper end of the model, and flooding would then only occur in extreme wet weather when heavy pumping of the bores between the 80 to 100 chain shoreward distrances did not provide sufficient withdrawal from storage. Pumping from the bores represented on the model by Drain No. 1 would have a similar, but lesser, effect.

A section, parallel to and a half a mile west of the modelled section would pass under the Lachlan Swamps and associated dams. In 1949 a firm of consulting engineers (Griffin 1962) estimated the level of the water in a dam near Gardener's Road to vary between reduced levels of 47 and 16 feet. When the reversed slope of the phreatic surface (which would occur in the vicinity of the dam at this low level) is considered, the maximum slope would approximate the hydraulic gradient of test Series 1 (1 in 238) and the minimum that of test Series 111 (1 in 427). An average maximum hydraulic gradient for this northern part of the Basin would probably lie between the gradients of Series 1 and II.

For model purposes the profile of the aquifer was assumed to be homogeneous. If more information had been available regarding the position and permeabilities of the different layers this data could have been included in the model. Some allowance was made for non-homogeneity in using Shiel's (1942) value for the coefficient of permeability of 230 feet per day as a horizontal permeability figure. Although this was the only value available its magnitude seems reasonable when compared with the Tomago Sands which contribute to Newcastle's water supply. Griffin (1962) recorded a permeability of 270 feet per day for these sands which he showed to be slightly coarser than the Botany sands.

Evidence collected by Griffin (1962) also indicated the presence of impervious or semi-impervious lenses, but there were insufficient data to enable the definition of any lenses or their inclusion in the model. The absence of these layers in the model must affect the derived fresh water-salt water interfaces.

3. THE VISCOUS FLOW HELE-SHAW MODEL

3.1 INTRODUCTION

The mechanics of the movement of fluids through porous media is most complex from both mathematical and laboratory points of view. Non-linear differential equations of unsteady flow, inadequate definition of the free surface, a variable zone of partial saturation, variable permeability and apparent rather than actual velocities are some of the difficulties confronting investigators in this field. Frequently model studies, and particularly analogies, can circumvent some of these problems (Todd, 1954).

The viscous flow analogy model consists essentially of two closely spaced parallel plates which form a narrow channel or capillary interspace. The analogy is based upon the similarity between the differential equations which describe the potential field of flow of water through saturated porous media, and the equations for laminar flow of a viscous fluid through the capillary interspace between two parallel plates.

The first model of this kind was developed by Hele-Shaw (1897, 1898). These two papers are descriptive. Stokes (1898) was the first to develop the mathematical theory of the model, while Drachler (1936) was the first to recognise its possibilities in ground water flow studies. Since 1940, the model has been used extensively by workers in the Hydrological Laboratory of the Government Institute for Water Supply, The Hague, Netherlands (e.g. Krul and Leifrinck, 1946; Santing, 1951). It has been used at the Israel Institute of Technology, Haifa and the Water Planning for Israel Centre, Tel-Aviv, and also in the U.S.A. at the University of California (Todd, 1954, 1955, 1959).

The viscous flow analogy can be applied to almost any two dimensional flow in the vertical plane of porous media, and both steady and unsteady flows can be studied. Suitable time scales can be chosen by varying the interspace and the viscosity of the fluid. For unsteady flows various grades of oil are commonly employed. In the case of steady flows tap water can be used to reduce the time required for steady state equilibrium conditions to be reached, and easily observed streamlines can be produced by injecting dyes. The model section, itself, can also be modified to take into account non-homogeneity and anisotropy. However, there are several important limitations, e.g. phenomena such as dispersion and unsaturated flow caused by the granular nature of porous media cannot be studied. As the model is two-dimensional openings in the model represent horizontal drains, and thus the three-dimensional effects of cones of depression, or local upconing in the vicinity of a single pumping well, cannot be directly investigated. Again, because of the model scales usually required, the anisotropy of the model is often greater than the prototype. It can thus, be seen that experimental results are not always directly applicable to field conditions.

3.2 HELE-SHAW MODEL THEORY

The mathematical basis of the model is founded upon the Navier - Stokes' equations for a viscous incompressible fluid, the equation of continuity and Darcy's Law. A complete resume of the mathematics is given by Hall (1964) and in the interest of continuity a brief review is included.

The average velocity ($\bar{u}$) in the "x" direction (horizontal) is given by the expression

$$\bar{u} = -\frac{1}{12} \cdot \frac{gb^2}{v} \cdot \frac{\partial h}{\partial x}$$

where g = acceleration due to gravity

b = width of the model capillary interspace

h = piezometric head

v = kinematic viscosity

Similarly, the average velocity ($\bar{w}$) in the "z" direction (vertical) is

$$\bar{w} = -\frac{1}{12} \cdot \frac{gb^2}{v} \cdot \frac{\partial h}{\partial z}$$

If $\bar{u} = -K_m \cdot \frac{\partial h}{\partial x}$, which is an expression analogous to Darcy's Law, then

$$K_m = \frac{1}{12} \cdot \frac{gb^2}{v}$$

where K_m = hydraulic conductivity of the model interspace

Jacob (1950) derived the expressions from which the model scales may be calculated. The more important scale factors are:-

(a)

$$\frac{K_{xr}}{K_{zr}} = \frac{K_{zp}}{K_{xp}} = \left(\frac{x_r}{z_r}\right)^2$$

where K = hydraulic conductivity

x, z, = subscripts denoting directions of co-ordinate axes

m, p, r = subscripts denoting model dimensions, prototype dimension and ratio of model to prototype dimensions.

(b)

$$t_r = \frac{n_r . z_r}{K_{zr}} = \frac{n_r . x^2_r}{K_{xr} . z_r}$$

where t = time

n = effective porosity and

$n_m = 1$

$n_p = 0.15$

(c)

$$Q_{xr} = \frac{K_{xr} . b_r . z^2_r}{x_r}$$

$$Q_{zr} = K_{zr}.\, b_{r}.\, x_{r}$$

where Q = discharge

(The symbol "K" for hydraulic conductivity is identical with that of "k" which is the coefficient of permeability).

Where areas of open sea water must be simulated there are two further important considerations. Firstly, the same volume scale must be applicable to both the narrow capillary interspace of the model itself and the enlarged interspace which represents the sea i.e.

(a)

$$\bar{b}_r = \frac{b_r}{n_p}$$

where $\bar{b}_r$ = ratio of model width of enlarged interface to prototype width. Secondly, where two fluids are used such as sea water and fresh water, then

(b)

$$\left(\frac{\rho s - \rho f}{\rho f}\right)_m = \left(\frac{\rho s - \rho f}{\rho f}\right)_p$$

where ρ_f, ρ_s are respectively the specific gravities of fresh and sea water.

The ratio $\dfrac{\rho f}{\rho_s - \rho_f}$ is given the symbol "δ" and is numerically equal to 40 in this investigation.

3.3 PROPORTIONING OF THE BOTANY MODEL

The horizontal scale was determined by the necessity to model a section from the shore-line to intakes in the vicinity of the Lachlan Swamps. A scale of 1 in 1500 was chosen. At a vertical scale of 1 in 1500, a drawdown of one foot in the field would be represented by 1/40th of an inch on the model; and this was considered to be the smallest vertical scale factor, which could be conveniently used.

Although the model itself remains isotropic, because of the model distortion, the flow condition actually modelled represents a condition of anisotropy in the prototype. The degree of anisotropy is given by:-

$$\frac{K_{zp}}{K_{xp}} = \frac{k_{zp}}{k_{xp}} = \frac{x_r^2}{z_r^2} = \left(\frac{500}{1500}\right)^2 = \frac{1}{9} .$$

This ratio of permeabilities is not excessively high in the light of comments previously made regarding the geology of the area, and Maasland (1957) has reported tests on sands ranging in anisotropy from 1 to 1/42. Although the direction of normal ground water flows are usually close to horizontal, with only a relatively small vertical component, the approximation of horizontal flow certainly does not remain valid in the immediate vicinity of pumping wells or the seaward interface.

The prototype horizontal permeability (k_{xp}) was assumed to be 230 ft./day (Refer to Section No. 2). It should be again emphasised that the accuracy of this figure is debatable, but it is the best estimate available.

Piezometer tappings were not used to determine the precise phreatic surface. As mentioned earlier, an accurate estimate of the true water table surface under steady state conditions is impossible to obtain, and accordingly all model flows were related to an average hydraulic gradient measured over the modelled section. Heights at the fresh water inflow end of the model were adjusted to the desired hydraulic gradient by varying the elevation of the constant head apparatus. The capillary rise was assumed constant, and the actual measurements of hydraulic gradient were made away from locations, such as the shoreline interface, where the surface capillary rise could be affected by the flow condition in the immediate vicinity.

Vertical replenishment was not considered because of the inadequacy of data. Furthermore, Griffin (1962) suggested that little rainfall replenishment occurred in the section being modelled. Only steady state conditions were reproduced and, consequently, tidal effects were neglected.

A check calculation of the Reynolds Number for the maximum discharge used showed that the flow condition was well within the laminar limit. (R_n = 13)

The model fluids used were actual sea water and fresh tap water.

The construction of the model is shown by Figure No. 7.

4. RESULTS OF MODEL TESTS.

Each series of tests was carried out under a constant "hydraulic gradient"

(i) viz:

(a)	Series I	-	"i" = I in 238
(b)	Series II	-	"i" = I in 167
(b)	Series III	-	"i" = I in 427

The steady state unpumped condition was repeated at least 8 times and the maximum deviation from the mean discharge was less than 5%.

Figure No. 8 is a typical plot of the steady state condition for intrusion of the sea water "wedge" at a given pumpage from one drain (or in prototype - bore). The curve of best fit for each individual test was either linear or slightly curved to degree illustrated by Figure No. 8. The variation in curvature can be attributed to the existence, or otherwise, of "upconing" at a drain and the change in elevation of the aquiclude floor.

Figure No. 9 is a similar plot for equal discharge from the two nominated "pumping" drains. The trend of data was similar for both the Series 1 and Series II experiments. As would be expected, where the flow rates are higher, the salt water encroachment into the aquifer is not as great.

Basically, the effect of the hydraulic gradient is to produce a seaward flow which pushes back the intruding sea water "wedge" and the greater the gradient the less the intrusion.

Tests at the low gradient of Series III were difficult to reproduce and the results showed somewhat more scatter than that shown in Figure No. 8. Testing was confined to Drain No. I in this Series because sea water intrusion had already occurred in the unpumped state.

The results of all tests are summarised in the following table:

TABLE I

Series		I	II	III
Hydraulic Gradient		I in 238	I in 167	I in 427
Unpumped Seaward outflow per ft. run		59 g.p.h.	77 g.p.h.	40 g.p.h.
% of unpumped seaward outflow at first intrusion of sea	Drain No. No. 1	80	98	55
	" 2	40	70	-
	" 3	55	60	-
	" 1 & 3	53	67	-
Distance of toe of intruding sea water "wedge" from shoreline - feet.	No pumping	1650	1300	2650
	Drain No. 1	3100	2950	3450
	" 2	2600	2600	-
	" 3	3450	1850	-
	" 1 & 3	2400	2200	-

As the steady state discharge of the aquifer itself is unknown, and the pumpage from the aquifer only vaguely known, results can only be expressed as percentages of the actual unpumped model flows. It is hoped that the range of hydraulic gradients used might, in the future, be found to cover the range of measured field gradients.

From the results of Series II it can be estimated that the "average" thickness of aquifer over the profile modelled is 215 ft. The "average" thickness as measured directly on the model is 175 ft. The correlation is not unreasonable when it is considered that the actual thickness varies from approximately 95 ft. to 250 ft.

Figures Nos. 10, 11 and 12 show the interface profiles for each test Series and drainage point. The differing upconing effects between Drain No. 1 and Drain No. 3 can be seen, as also can the pronounced upconing for Drain No. 3, at high hydraulic gradients. Where Drains Nos. 1 and 3 are being pumped together the upconing into Drain No. 3 is not very noticeable as Drain No. 1 is also pulling the interface inland. The effects of upconing, and the varying slope of the aquiclude profile, are reflected in the unequal spacing of the interface profiles even though equal pumping increments are plotted. This is particularly noticeable in Figure No. 10(a) and No. 11(a) for Drain No. 1 at the higher hydraulic gradients.

5. THEORETICAL INTERFACES

The modelled section showed a relatively constant aquifer thickness (D = 240 ft) for approximately 1300 ft. inland from the shoreline, and a comparison of theoretical interface calculations has been based on this assumed "uniform" condition. The profile for the steady state unpumped condition of Series II is used as a measure of the accuracy of the theoretical formulae.

All theories are shown as graphical plots to natural scale in Figure No. 14.

5.1 GLYBEN-HERZBERG

Reference should be made to Figure No. 13 for nomenclature. This is a theory based on pure hydrostatics and the depth to the interface below sea level is given by

$$h_s = \left(\frac{\rho_f}{\rho_s - \rho_f}\right) . h_f = \delta . h_f = 40 . h_f .$$

The interface is linear and the slope is only a function of the hydraulic gradient. In Figure No. 14 two interfaces are shown. One (1 in 167) represents the assumed hydraulic gradient, and the other (I in 200) represents the actual difference in elevation between the fresh water intake and sea level. (Refer Figure No. 15).

5.2 DUPUIT PARABOLA

Reference should be made to Figure No. 13 for the definition of the axes. The interface equation is derived from a substitution of the Glyben-Herzberg expression into Darcy's Law under conditions which assume horizontal flow.

The equation of the interface parabola is:-

$$y^2 = \frac{2. Q. \delta}{k} . x.$$

and the length of the horizontal projection of the interface is given by:-

$$L_p = \frac{K.D^2}{2.Q.\delta}$$

At the shoreline there is the inconsistency that at x = 0, y also = 0 and thus Q is indeterminant.

5.3 NOMITSU ET AL(1927)

Reference should again be made to the nomenclature of Figure No. 13, The equation of the phreatic surface ABC is derived as:-

$$(y')^2 = \text{constant} - \left(\frac{2.\,Q.\,x.}{k}\right) . \left(\frac{\rho_s - \rho_f}{\rho_s}\right).$$

and the constant can be evaluated by measuring, in the field, y' at say

$$\begin{aligned} y' &= y'_o \quad \text{at } x = o \\ &= y'_L \quad \text{at } x = L \end{aligned}$$

The equation of the interface $A^I B^I C^I$ becomes:-

$$y^2 = y_o^2 + \left(\frac{y_L^2 - y_o^2}{L}\right).\, x.$$

At $y = y' = 0$, the velocity of discharge approaches infinity. However, if $(y'_o)^2$ and $(y_o)^2$ are small compared with $(y')^2$ and $(y)^2$ the following approximate equations can be used in the vicinity of the shoreline.

$$(y')^2 \approx (y'_L)^2 . \frac{x}{L}$$

$$y^2 \approx y_L^2 . \frac{x}{L}$$

where $y = \delta . y'$

and $$\frac{k.\,\delta}{Q} \approx \frac{2.L}{(y'_L)^2}$$

$$\frac{k}{Q} \approx 2.L.y^{-2}$$

In the model precise measurements were not made at the shoreline and this method was not pursued further.

5.4 GLOVER (1959)

A close representation of the flow conditions near a shoreline can be obtained by modifying a solution developed by Kozeny (1953) for the flow of ground water under gravity forces.

The interface between the fresh water and sea water can be plotted from the expression:-

$$y^2 = \frac{2.\,Q.\,\delta.\,x.}{k} + \frac{Q^2.\,\delta^2}{k^2}$$

This equation is identical with the Dupuit equation except that it has a different origin.

The width of the gap through which the fresh water excapes to the sea is:-

$$x_o = -\frac{Q.\,\delta}{2\,k}$$

and the depth to the interface through the shoreline is:-

$$y_o = \frac{Q.\,\delta}{k} = -\,2.\,x_o$$

A non-dimensional flow net can be constructed for this potential flow pattern, (Glover 1959), and the whole interface can be transposed once x_o and y_o are established.

The Glover interface for an isotropic medium is shown plotted on Figure No. 14.

5.5 BEAR & DAGAN (1962)

These workers have made use of conformal mapping and hodograph techniques which have, in part, been developed by Lamb (1932), Muskat (1937), Engelunde (1957), Henry (1959) and Vallentine (1959).

The problem treated by Bear and Dagan was essentially an interface development in an isotropic confined aquifer with horizontal flow. However the effect of anisotropy can be included in the analysis by modifying the various parameters (Maasland 1957). The mathematical treatment and the equations developed are too complex for inclusion here, and calculations are normally made from graphs relating parameters such as

$$\frac{Q.\delta}{k.x}\ , \quad \frac{Q.\delta}{k.y}\ , \quad \frac{Q.\delta}{k.D}$$

A comprehensive summary of this paper, which is not readily available, is given by Hall (1964).

Dagan and Bear partly summarise their results by plotting the parameters

$$\frac{\pi.\ k.\ D}{Q.\delta} \quad \text{and} \quad \frac{\pi.\ k.\ L_p}{Q.\delta}$$

i.e. relating the depth of aquifer D with the overall projected length of the interface L_p. It is of interest to note, however that if the Dupuit parabola

$$y^2 = \frac{2.Q.\delta}{k} . \quad (L_p - x)$$

is assumed, then providing the parameter $\frac{\pi.k.D.}{Q.\delta} > 6$ errors in the interface position are only of the order of 3%.

For the isotropic condition modelled, the value of $\frac{\pi.\ k.\ D}{Q.\ \delta}$ was 14.6 and consequently the "exact" hodograph analysis and the Dupuit modification (also the Glover analysis) gave virtually identical interfaces.

However, the model itself is distorted in scale and an anisotropic prototype condition is actually being modelled. Because the model is isotropic regarding permeability, the equivalent permeability is given by:-

$$k_e = \sqrt{k_x .\ k_z}$$

The model ratio of horizontal (k_x) to vertical (k_z) permeability is 9, and consequently the "k_x" value of 230 ft/day assumed in the field becomes an "equivalent istropic" prototype permeability of 77 ft/day. The vertical field permeability becomes 26 ft/day.

The appropriate value of $\frac{\pi.k_e.D.}{Q.\delta}$ now becomes 4.8, and from a mathematical viewpoint, an exact solution is probably worthwhile. The anisotropic "exact" and the Dupuit modification solutions are shown in Figure No. 14.

5.6 SUMMARY

The following table summarises the more important calculations.

TABLE 2

Type of Analysis	Width of Gap through which fresh water escapes -"x_o"-(ft)	Depth to Interface through shoreline "y_o" = (ft)	Overall Projected Length of Interface "L_p" -(ft)
Model (anisotropic field condition)	91	80	1390
Glyben-Herzberg	0	0	1000
Dupuit Parabola (assumes horizontal flow, k = 230 ft/day)	0	0	557
Glover (isotropic)			
k = 230 ft/day	26	52	557
k = 77 ft/day	78	156	Profile not calculated.
Bear & Dagan Hodograph (isotropic)			
k = 230 ft/day	26	52	557
k = 77 ft/day	78	156	Profile not calculated.
Bear & Dagan Hodograph (Anisotropic field condition) exact solution	233	146	573
Dupuit approx.	233	156	557

6. COMPARISON OF THEORETICAL AND MODEL INTERFACE PROFILES

The theoretical profiles are summarised in Figure No. 14 and all can be seen to differ markedly from the model interface profile. Calculations were based on the seaward outflow obtained on the model and, in the first instance, the aquifer was assumed isotropic. The parabola based on the Dupuit assumption, and all Glyben-Herzberg interfaces, pass through the origin (or edge of the bay), but from the model studies it can be seen that this cannot be the case. Assuming vertical outflow from the bed of the bay adjacent to the origin, the parabolas derived from Glover's complex variable theory and Bear & Dagan's hodograph methods (approximated by use of the Dupuit parabola) will coincide, and be displaced seaward in the case of the isotropic aquifer.

Since the Hele-Shaw model represents an anisotropic condition, the isotropic interface theories can be only expected to be reproduced on the model where the flow is horizontal. This is certainly not true near the outflow face where adjustments for anisotropy might be expected to yield more accurate profiles. Using the same model outflow discharge the resulting interface was now found to be displaced to seaward by the degree ofanisotropy (1/9), but the overall interface length remained virtually unchanged.

To obtain a comparison between the hodograph theory and the model interface profile, the curve for the anisotropic case which coincided with the model profile curve at the shoreline was calculated. The modelled and theoretical profiles were found to closely correspond for discharges within the range 30.5 to 35 g.p.h.

These calculations emphasise that anisotropy must be considered when studying fresh water - salt water interfaces; although, in this case, the curves which approximated the model interface were based on a discharge of approximately half that measured on the model. This difference might suggest serious errors in model discharge calculations. However, assuming the model theory to be correct, errors on individual runs were within 8%, and, as the unpumped seaward discharge is the average of eight measurements, the error in the accepted figure is probably within 5%. The error due to flow in the "seepage" face above M.S.L., even if flow is assumed in the capillary zone, is also negligible.

Another source of possible error lies in the approximations of the Navier-Stokes' equations in developing the model theory. However, since no turbulent flow was encountered, most approximations must be valid, and no doubts, regarding their validity, have been raised by other workers using this type of model.

The principal reasons for the incompatibility of modelled and theoretical profiles must lie in the assumption of horizontal inflow to the interface area, and uniform velocity distribution within the section. Observations of dyed groups of particles showed that the fluid in the vicinity of the top streamline (Figure No. 7) had a velocity of only approximately 80 - 90% of that flowing in the vicinity of the bottom streamline. As well as this non-uniform velocity distribution, observations of the same streamlines showed the model flow to be nowhere truly horizontal.

The hodograph theories of Bear & Dagan assumed a confined aquifer with constant discharge (and because of geometry - constant velocity) across any vertical section. In the model, the ratio of the average velocity of flow at the fresh water intake to that at the vertical section through the interface-aquiclude junction is 2.5. There is thus retardation, and this involves the application of a force system according to Newton's 2nd law of motion. This force will act in the opposite direction to the flow, but rough calculations would indicate that this force system would be much too small to cause of itself a loss in head at the intake of about 37 feet of water. (This latter figure represents the head loss which would result from a reduction in flow from 77 g.p.h. to 31 g.p.h.)

A comparison of the assumed water surface profile with that obtained from an average model run (Figure No. 15) shows that the model water surface is quite flat in the critical fresh water - salt interface area. Furthermore, the more refined theories considered here are all based on the assumption of no flow in the salt water region. This seemed to be essentially correct in the model, although some slight circulatory underflow did occur. Thus, when considering a static interface and the potential at the interface, the observed flat hydraulic gradient must tend to produce a flatter and longer interface than would be obtained for a steeper water surface.

The Glyben-Herzberg straight-line interface is not applicable under any conditions, but it does raise the question of what is the real phreatic surface slope. Figure No. 15 shows the assumed slope of Series II as I in 167. This was based on the assumption that the rise in the water surface due to surface tension was constant along the length of the model, but as streamlines are curved in a similar manner by similar effects, it is even conceivable that the phreatic surface near the shoreline may be slightly concave upwards. Piezometer tappings would be required to determine the true gradient. The installation of piezometer tappings would also show if the departure from a straight line phreatic surface actually does occur near the middle of the model. Furthermore, at a distance of 55 inches inland from the shoreline on the model the rise in the bed-rock profile is reflected by a similar rise in the water surface profile. Only further investigation can evaluate the effect of these variations of phreatic surface on the interface, but it must be emphasised that the theories which have been discussed do ignore these modelled phenomena.

7. CONCLUSIONS

1. It becomes very evident that there are insufficient field data, both from the geological and hydrological aspects, to accurately model any part of the Botany Basin. Until this basic data becomes available there is no reason to attempt greater accuracy than that already achieved for this selected profile.

2. Model tests suggest that the unpumped seaward flow of fresh water into Botany Bay is of the order of 60 to 75 g.p.h. per foot of shoreline at the deepest section of the modelled area.

3. If an average outflow of say 50 g.p.h. per foot of shoreline is assumed, and the shoreline is taken as 3 miles long, the total underflow is of the order of 25 million gallons per day.

4. Studies of the fresh water - salt water interface show that, for bores penetrating the aquifer to 100 feet, and more than half a mile inland, the entire unpumped seaward flow can be possibly withdrawn before salt water intrusion will occur; providing that the ground water storage is not being simultaneously reduced elsewhere and the drawdowns in the pumping area are not large.

5. Alternatively, bores at 1/4 and 1/2 mile from the shoreline and pumping equal amounts, can pump approximately 55% of the seaward flow before salt water intrusion would occur; subject to the same provisos as in (4).

6. As far as the groups of bores represented by drains No. 1 and 3 are concerned, it would appear from the limited data available (Griffin '62) that there is no immediate danger of salt water intrusion. (Chemical studies (Cornell, 1964) have been undertaken to further check this possibility, and are reported briefly in the attached Appendix).

7. It should be realised that, if the pumpages referred to in (4) and (5) were obtained, then very little fresh water could be pumped from shallower depths above the interface at, say, 1/4 mile from the shoreline.

8. Interface theories, even when quite refined, did not agree with the modelled interface, and it is suggested that the differences are largely attributable to the non-uniform geometry of the aquifer which results in a change of seepage velocity with elevation and time, and also a serious departure from horizontal flow.

9. The necessity for model studies, as a check upon theoretical analyses, is emphasised.

10, When reliable information is available regarding hydraulic gradients and permeabilities in the field, more detailed attention would be required in the reproduction of model gradients. Further, by a suitable choice of interspace width, liquids, and vertical scales the capillary rises in model and prototype can be made to correspond.

8. ACKNOWLEDGEMENTS

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10. APPENDIX

Recognition of Sea Water Intrusion by Chemical Analyses.

The major constituents of a typical sea water as determined by Rankama and Sahama (1950) are listed below:-

Typical Ions		Concentration (p.p.m.)
Cl	-	18,980
Na	-	10,560
SO_4	-	2,560
Mg	-	1,272
Ca	-	400
K	-	380
HCO_3	-	142

Of all the major chemical constituents of native ground water (and possibly contaminants) it is considered that only chloride probably remains chemically inert, and thus the amount of this constituent present in a contaminated water can be assumed to indicate the proportions in which the native water and the contaminant have been mixed. However, the source of the contaminant is usually not so easily identified.

In the case of sea water contamination, because of the modifications which sea water itself may undergo as it passes through the ground or through marine and/or estuarine sediments, the observed changes in composition of ground waters may be quite different from those that would occur by simple mixing of the two waters. Furthermore, the absence of such changes cannot be taken as conclusive proof that salt water encroachment has not occurred. The modifications in composition which sea water may undergo on passing through soil or sediment can be divided into three main categories:-

(a) Base Exchange Changes, i.e. those modifications which affect the proportions between positively charged ions (cations).

(b) Sulphate and Bicarbonate Changes i.e. those modifications which affect the proportions between the negatively charged ions (anions).

(c) Solution and Precipitation Effects i.e. those modifications which result in changes of equal significance to both cations and anions.

These physico-chemical reactions may be briefly described as follows:

(a) Base Exchange

If the base exchange capacity and amount of soil, the amount of water and the total amount of each base which are involved in a base exchange reaction are known, it is possible to estimate the final equilibrium concentrations of cations in the pore water and the relative proportions between the bases in the soil. These complete data are seldom, if ever, available in considering problems of salt water intrusion. Consequently, changes in the ratios between different cations observed in ground water cannot usually be used as valid criteria of the presence of sea water contamination; nor can the absence of such changes be taken as evidence that contamination has not occurred.

(b) Alteration due to Sulphate Reduction.

Oil field waters, which are usually considered to be fossil sea water trapped in marine sedimentary rocks, often contain very little sulphate whereas sea water contains sulphate to the extent of approximately one seventh by weight of chloride. This removal of sulphate is caused by the reduction of the sulphate ion in the presence of organic material and bacteria. For example, a schematic equation involving glucose could be:-

$$3\,Na_2SO_4 + C_6H_{12}0_6 \xrightarrow[\text{Bacteria}]{} 3H_2S + 6Na.\,HCO_3$$

Since sulphate reduction can take place wherever anaerobic conditions exist, it would be anticipated that sea water entering an aquifer through a layer of marine and/or estuarine sediments will lose a part of its sulphate content and gain a corresponding amount of bicarbonate.

(c) Alteration due to Solution and Precipitation.

Under certain conditions the sulphate concentration may increase upon contact with the aquifer, and sea water can dissolve approximately three times as much calcium sulphate as it actually does contain. Thus sea water in contact with gypsum beds will gain as much as 75% of its initial sulphate, and its calcium content may be tripled. If the pH is about 8, calcium carbonate will then be precipitated and still more sulphate will be dissolved. The precipitation of calcium carbonate may also accompany the process of sulphate reduction, particularly if this process takes place at high pH, and that part of the bicarbonate produced is converted into carbonate. It is generally supposed that at a pH of 8 or more (and at temperatures greater than 20°C) normal sea water is saturated with calcium carbonate, and consequently an increase of calcium or bicarbonate will tend to produce precipitation.

It is therefore evident that an increase in the amount of chloride present in a ground water must be considered as the most reliable indicator of the first stage of sea water intrusion. Unfortunately, small rises in the chloride level may be due to a temporary increase in total dissolved salts, and a ratio of chloride to some other ion must normally be introduced to minimise the effects of changes in concentration. For this purpose Revelle (1941) suggested the chloride-bicarbonate ratio. Bicarbonate is usually one of the most abundant ions in ground waters, but it only occurs in sea water in the ratio I : 135 of chloride. Even though sulphate reduction, and calcium carbonate precipitation, may alter the bicarbonate content in intruded sea water, there is usually such a large difference in the chloride-bicarbonate ratio in sea water and ground water that these variations only affect the ratio, as an index of sea water intrusion, in a minor manner.

Contamination by sewage and/or industrial wastes can also raise the chloride-bicarbonate ratio of a ground water, and hence, in the case of the Botany Sands where such contamination is suspected, other means must be used to recognise sea water intrusion. e.g. some bores produce water of 2000 p.p.m. chloride content with zero bicarbonates (i.e. pH is less than 4.8).

To determine the point at which it can be said that a sample of water is contaminated by a certain compound a "normal" concentration of this compound must be stated. The results of the seventy-five analysis carried out by the M.W.S.D.B., in 1942 have been used to arrive at a "normal" concentration of chloride and nitrate in the Botany Basin ground waters. Accepting the premise that pollution in the Botany Basin is getting progressively worse these results, which are the earliest available, should give the most reliable figures.

From the total of seventy-five samples analysed, sixty-seven, which are not obviously unusually high in chloride and/or nitrate, can be selected.

The mean and standard deviation of these sixty-seven sets of results are:-

(a) Chloride	-	mean 60 p.p.m. standard deviation 31 p.p.m.
(b) Nitrogen (as nitrate)	-	mean 0.26 p.p.m. standard deviation 0.28 p.p.m.

Assuming that 95% of determinations will fall within two standard deviations of the mean, it is concluded that a sample containing chloride in excess of 122 p.p.m. or nitrate nitrogen in excess of 0.82 p.p.m. (i.e. nitrate in excess of 3.6 p.p.m.) is contaminated.

For the purpose of recognising sea water intrusion into the Botany Basin aquifer the minor constituents of sea water were also considered viz:-

Concentrations of Some Minor Sea Water Constituents (p.p.m.)

Bromine	-	65
Nitrogen	-	0.03 to 0.9
Barium	-	0.05 (max. possible in presence of SO_4)
Borate	-	25

Of these constituents bromine, barium and borate are considered to be virtually chemically inert when in contact with an aquifer, and the concentrations of bromine and borate have previously been used to investigate connate brines and sea water contamination.

In this study of the Botany Basin aquifer bromine was used to identify the presence of sea water, and nitrogen was used to show that certain bores containing high concentrations of chloride were contaminated by sources other than sea water.

The following table lists the results of analyses of samples from bores close to the coastline in the Banksmeadow area, and also of an exceptionally high chloride content. The column headed "Expected Bromide" has been computed by assuming sea water intrusion and a normal maximum chloride level of 122 p.p.m. The chloride-bromide ratio of sea water is approximately 290 : 1.

Bore No.	Ion Concentrations (p.p.m.) NO_3 - N	Cl	Br.	Expected Br.
a	7.9	1930	0	6.2
b	0.5	1630	-*	5.6
c	1.7	338	0	0.4
d	0.3	438	0	0.7
e	1.3	104	0	0.0
f	2.2	302	0	0.6
g	0.1	309	0	0.6
h	3.2	67	0	0.0

* The sample from bore No. (b) was collected and analysed by Griffin (1962). Since this sample was collected the bore has been shut down because its chloride content is too high for the water to be useful. Thus it was not possible for the author to collect and analyse further samples from this bore.

The table clearly demonstrates that, using the bromide-chloride ratio as an index of sea water intrusion, there is no sea water intrusion into the Banksmeadow section of the Botany Basin aquifer at the level used for commercial pumping. It can also be seen that, of the eight samples listed in the table, five of them are contaminated by nitrate which, because of the low nitrogen content of sea water, did not originate from sea water intrusion. There is no direct evidence to prove that the chloride and nitrate contaminations, where present in the bores listed above, came from the same source; but it seems reasonable to assume that since the bores are obviously exposed to nitrate, a typical surface pollutant, the chloride contamination also came from surface waters.

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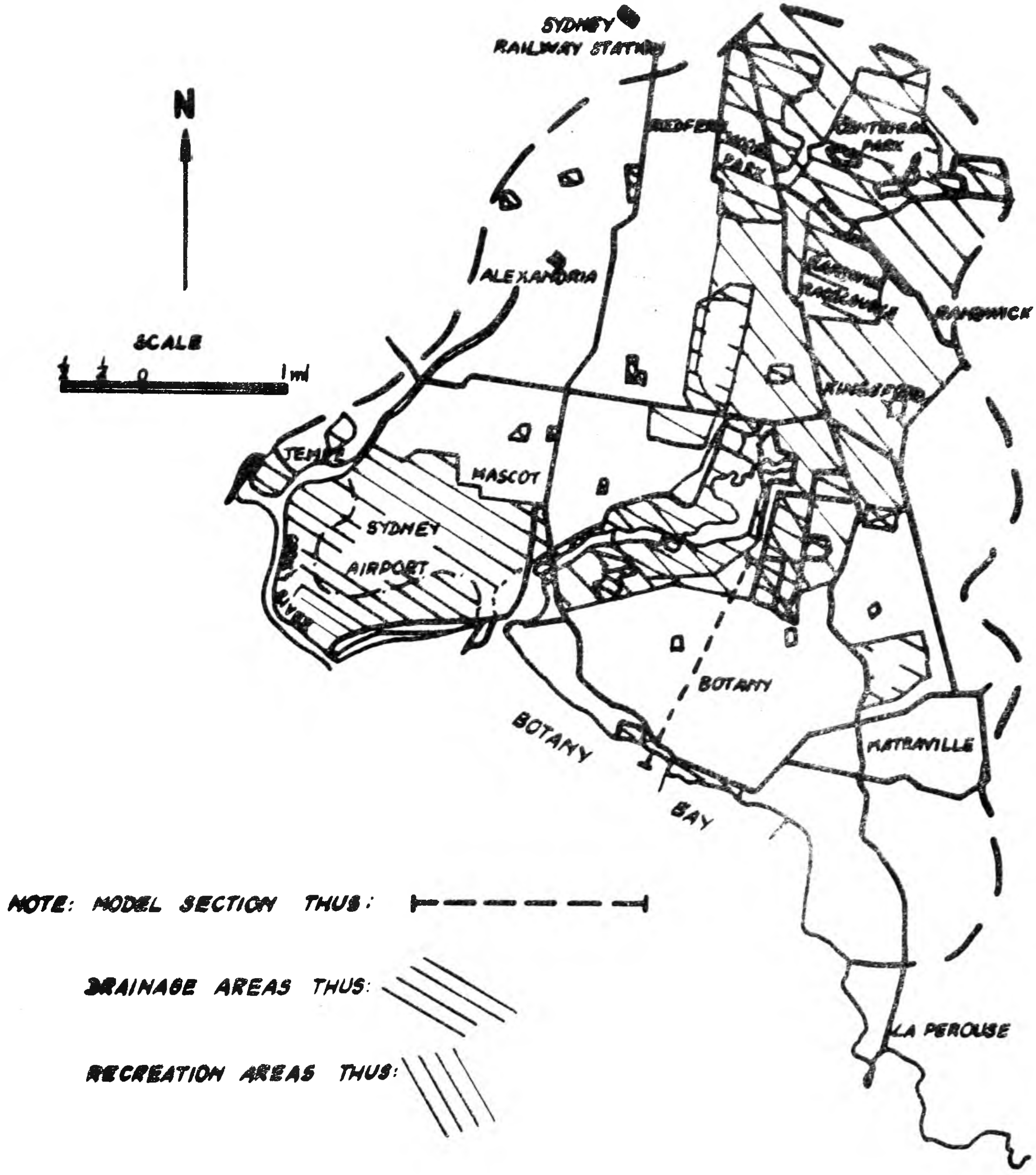


FIG 1.

NORTHERN SECTION OF BOTANY BASIN.

AFTER GRIFFIN, 1962.

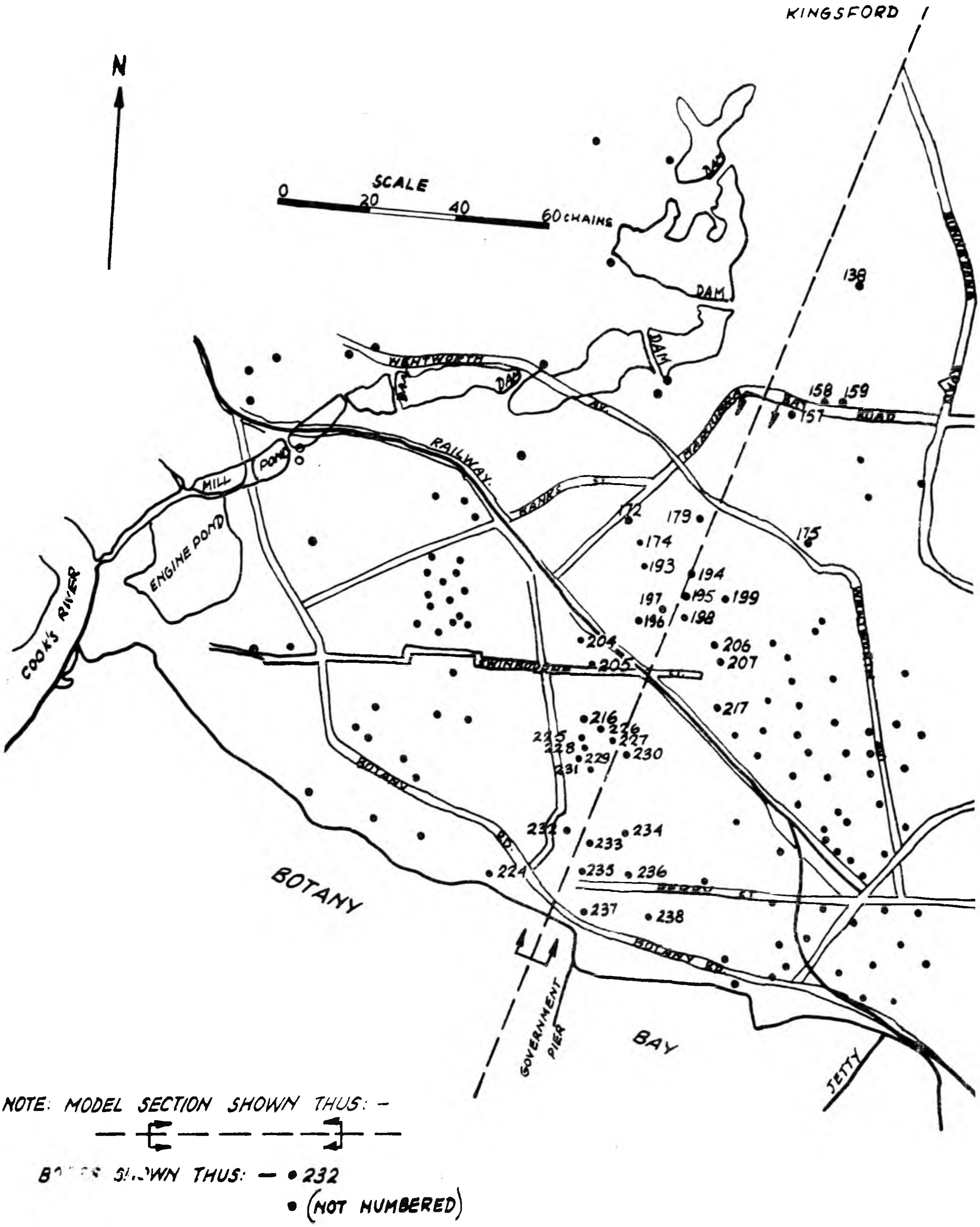


FIG. 2

LOCALITY MAP OF MODEL SECTION

AFTER GRIFFIN 1962

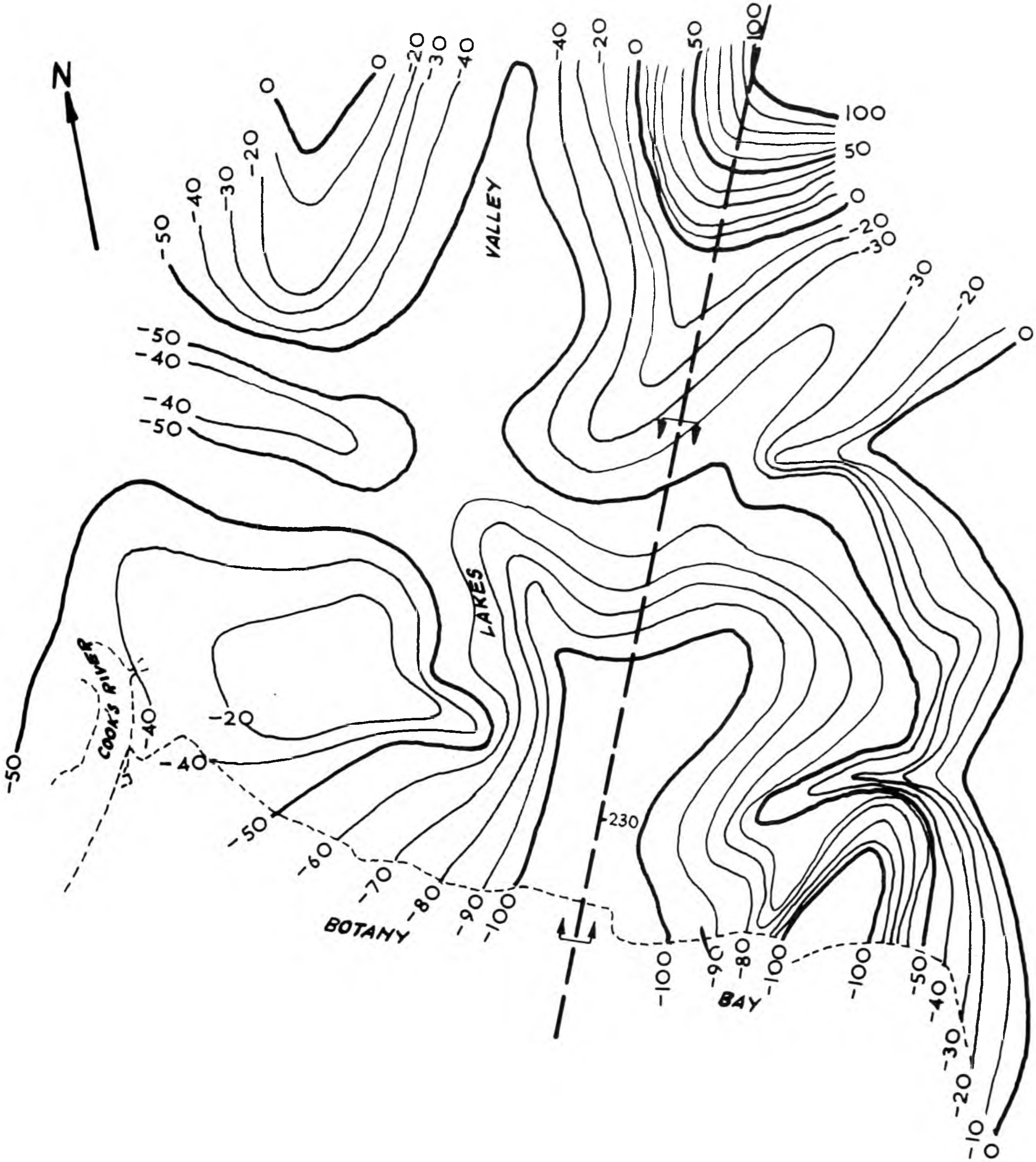


FIG. 3

BASE ROCK CONTOURS OF MODEL AREA - ft.

AFTER GRIFFIN, 1962.

STANDARD DATUM - SYDNEY.

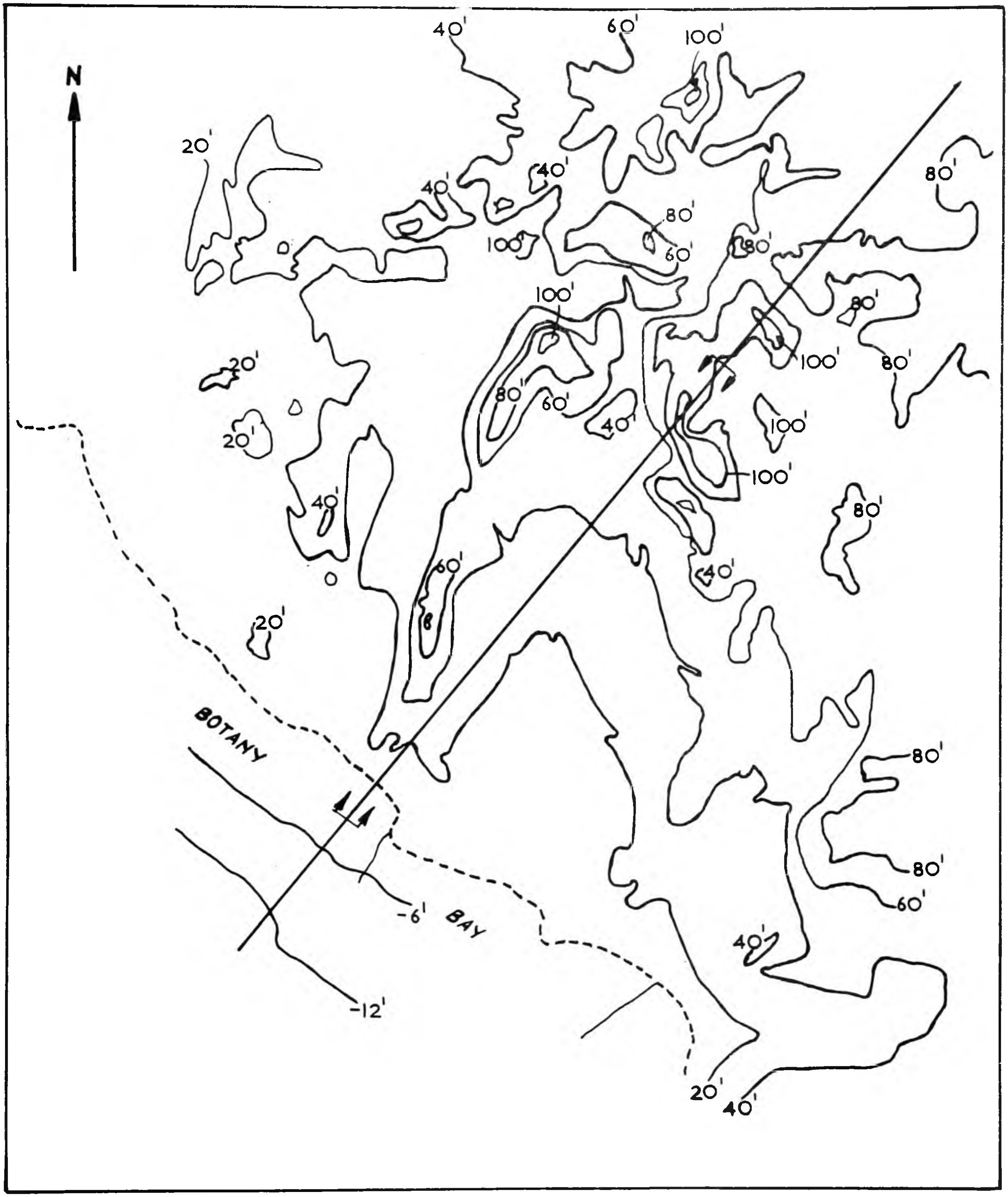
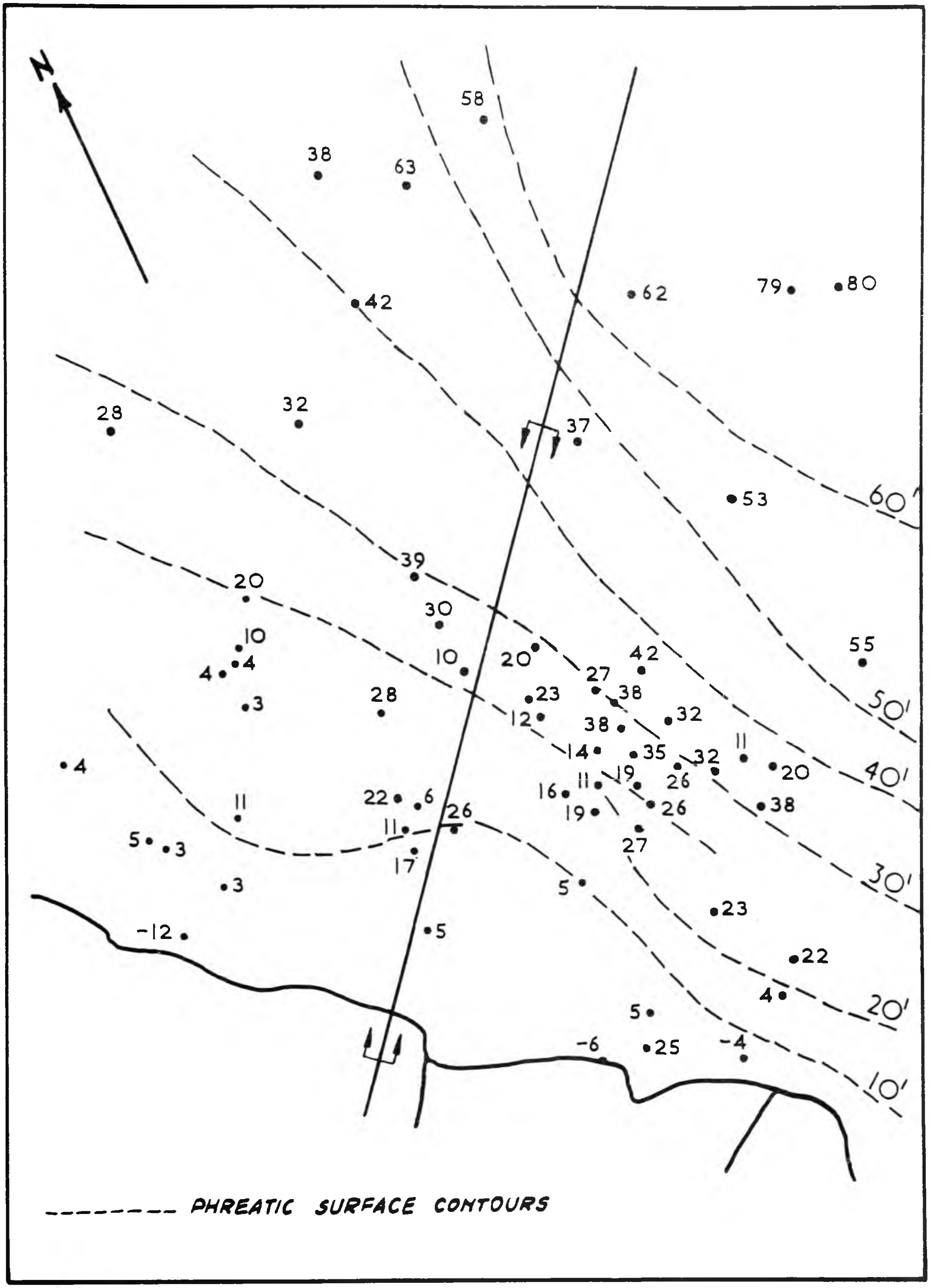


FIG. 4

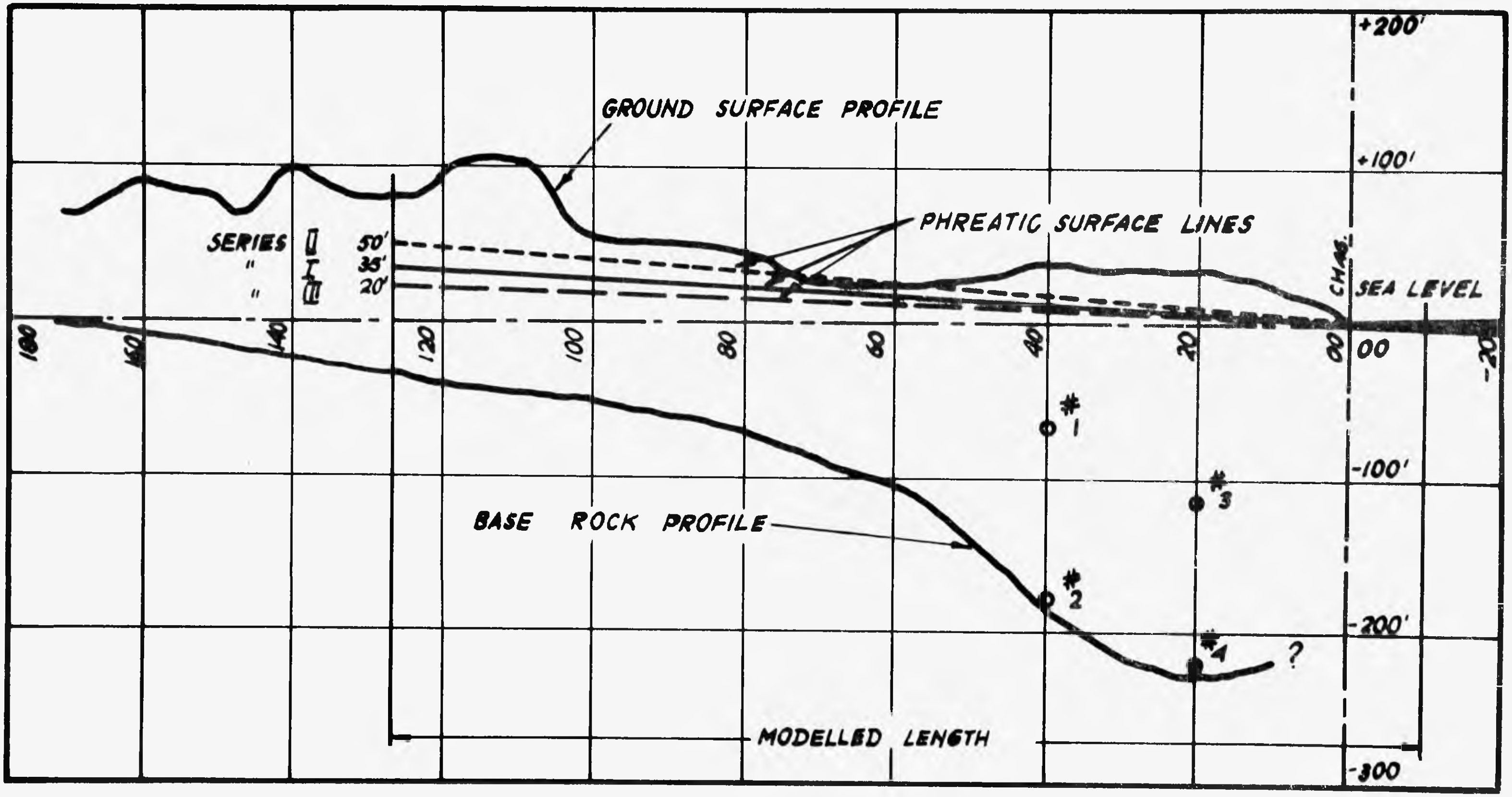
TOPOGRAPHICAL CONTOURS OF MODEL AREA - ft.

AFTER GRIFFIN, 1962.



"UNPUMPED" STATIC WATER LEVELS - ft.

FIG. 5



SCALES: VERT. 1 UNIT = 100 ft.
HORIZ. 1 UNIT = 20 CHAS.

MODEL SECTION PROFILES

FIG. 6

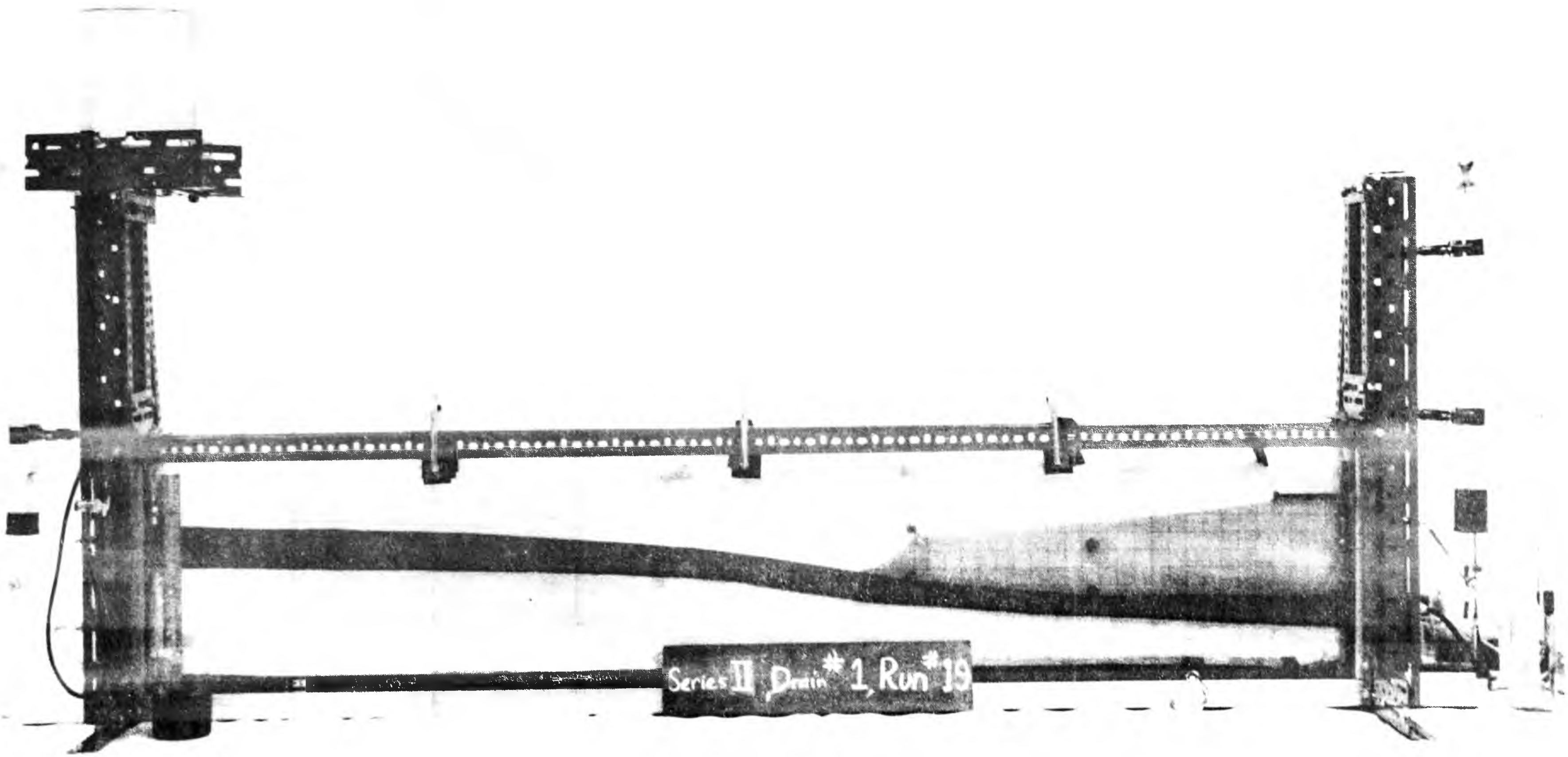
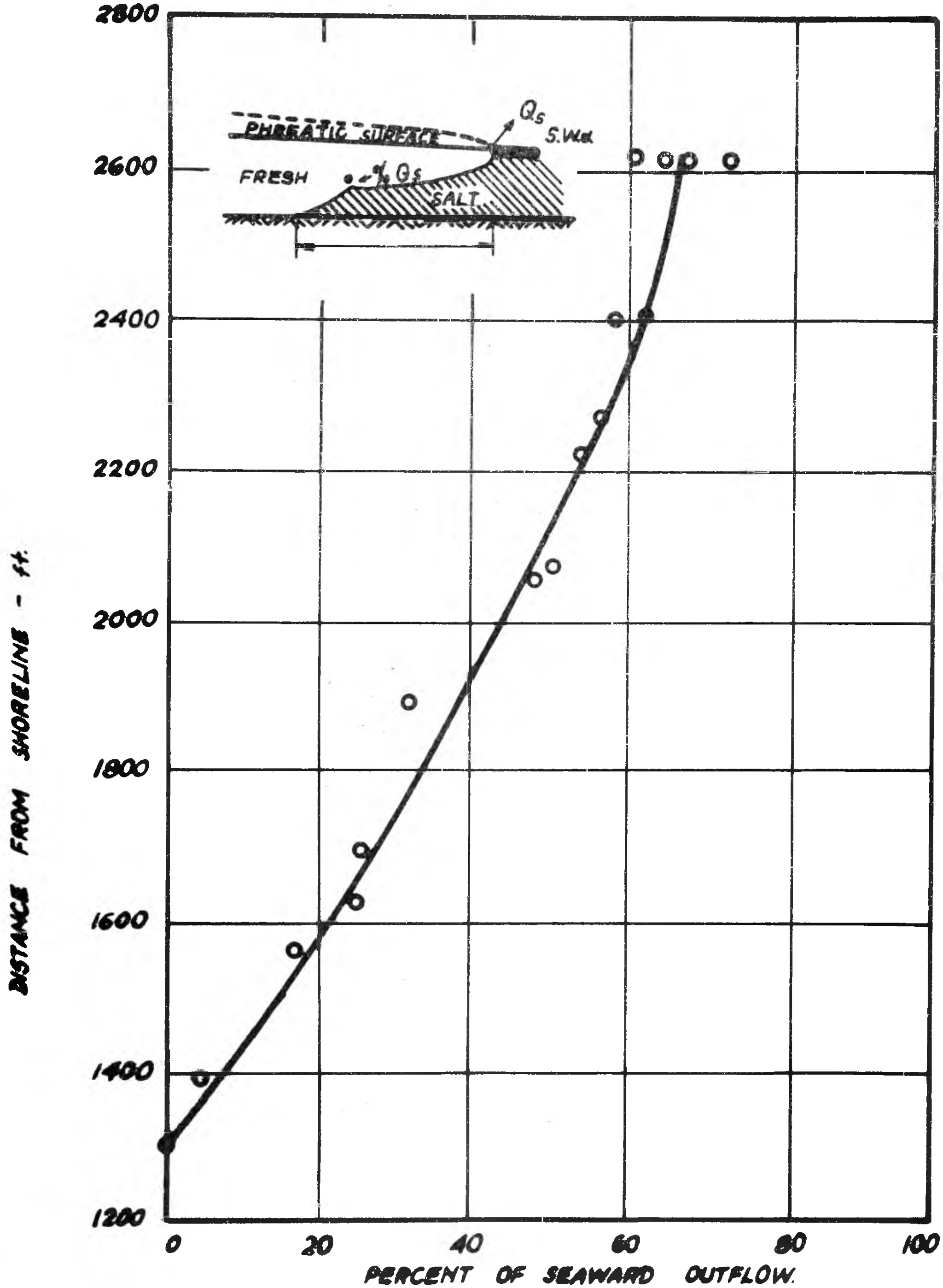


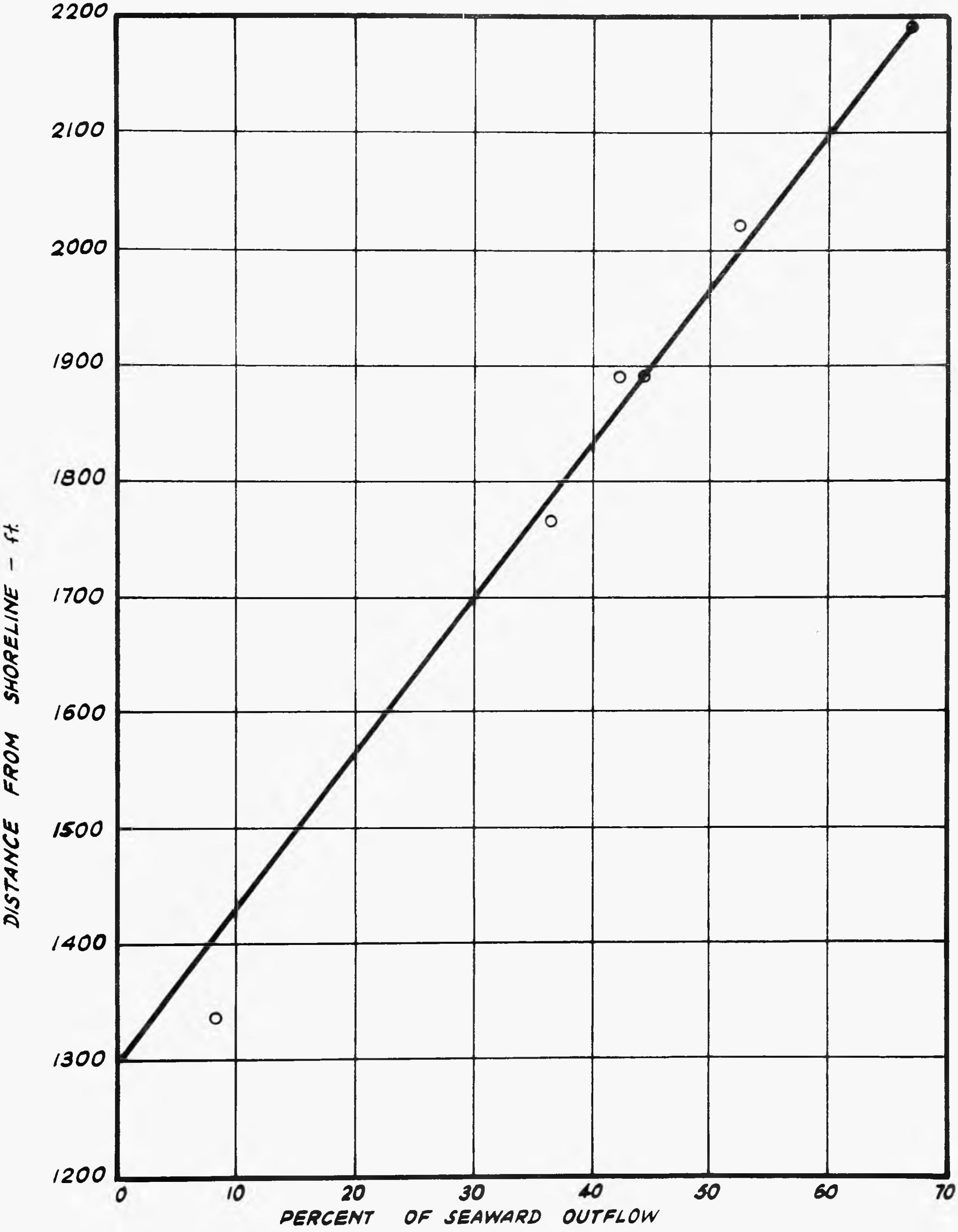
Fig. 7. Viscous Flow Analogy Model: Hele-Shaw Model.



TOE OF INTERFACE AT DIFFERENT PUMPING RATES.

FIG. 8.

SERIES II - DRAIN #2



TOE OF INTERFACE AT DIFFERENT PUMPING RATES.

FIG. 9.

SERIES II – DRAINS 1 & 3 PUMPED AT EQUAL RATES.

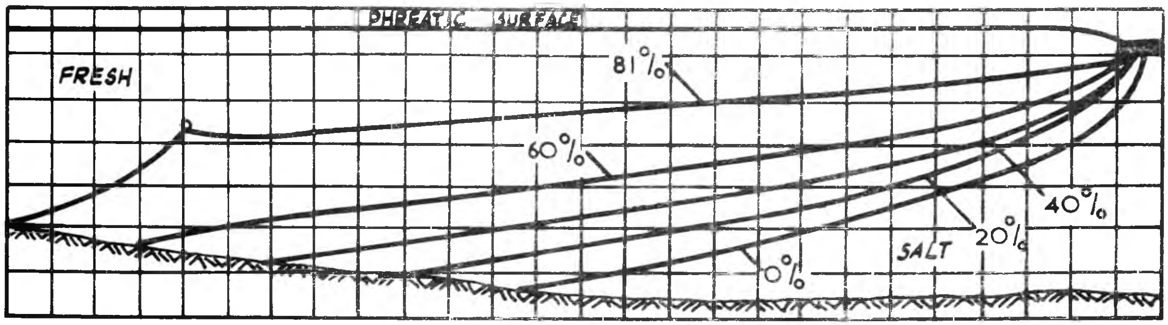


FIG. 10(a) DRAIN #1

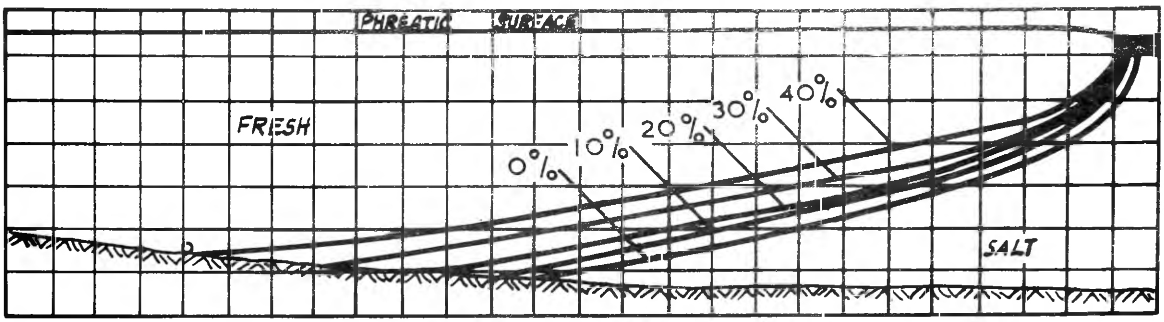


FIG. 10(b) DRAIN #2

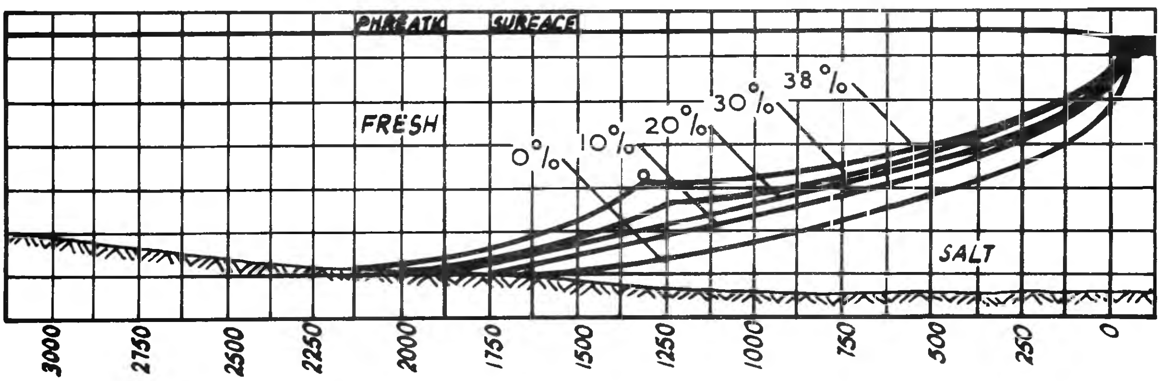


FIG. 10(c) DRAIN #3

FRESH WATER – SALT WATER INTERFACE PROFILES.

SERIES I, P = PERCENT OF SEAWARD OUTFLOW.

SCALES: 1 DIVISION = 1 in. ON MODEL AND 41 2/3 ft. VERT. AND 125 ft. HORIZONTALLY ON PROTOTYPE.

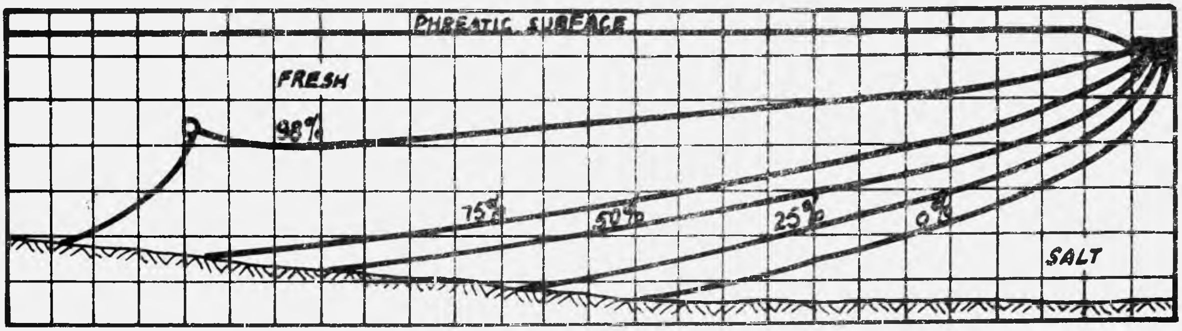


FIG. 11(a) DRAIN #1

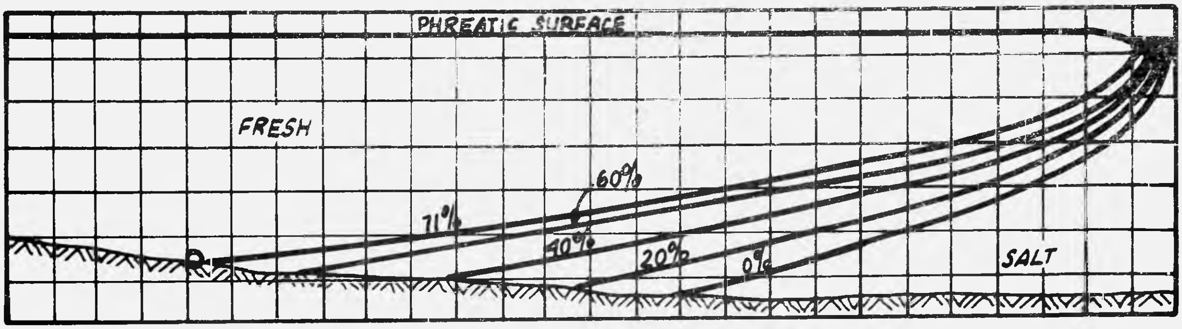


FIG. 11(b) DRAIN #2

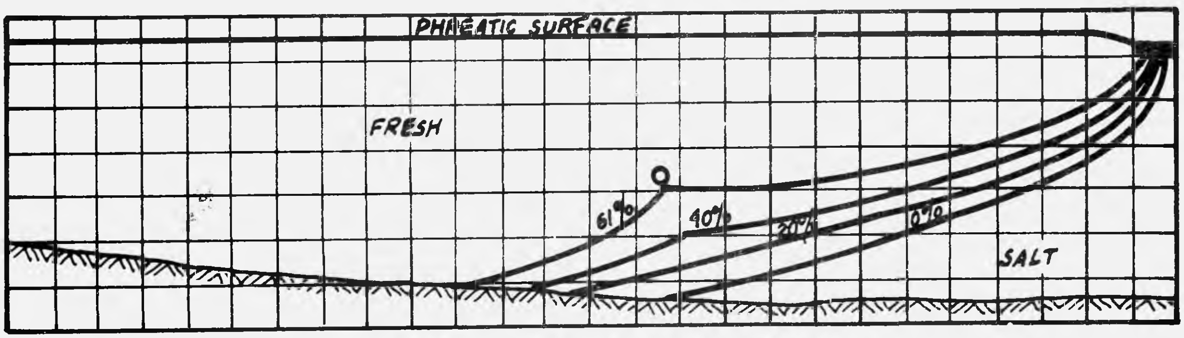


FIG. 11 (c) DRAIN #3

FRESH WATER-SALT WATER INTERFACE PROFILES

SERIES II, P-PERCENT OF SEAWORD OUTFLOW.

SCALES: 1 DIVISION = 1 in. ON MODEL AND $41\frac{2}{3}$ ft. VERT. AND 125 ft. HORIZONTALLY ON PROTOTYPE.

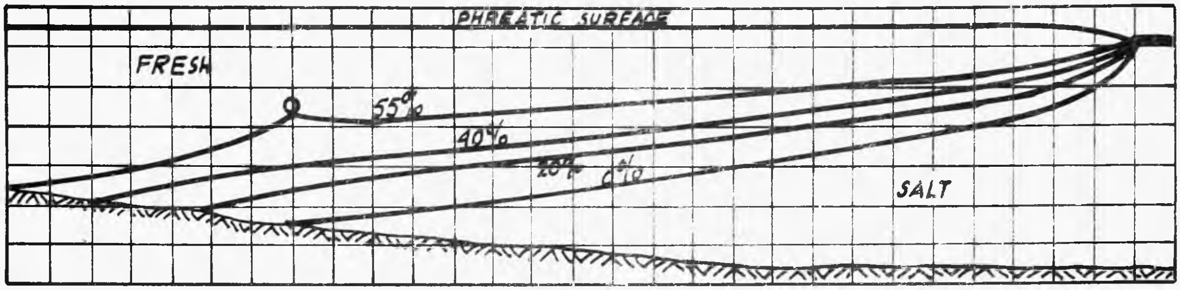


FIG. 12 (a) SERIES III, DRAIN #1

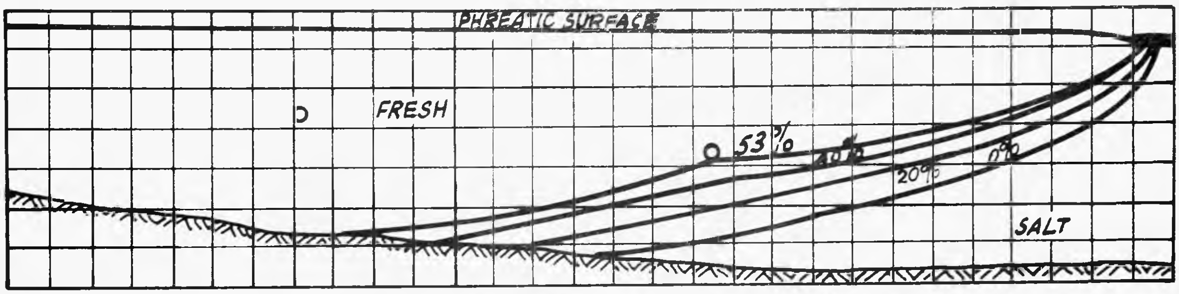


FIG. 12 (b) SERIES I, DRAIN #1 & 3

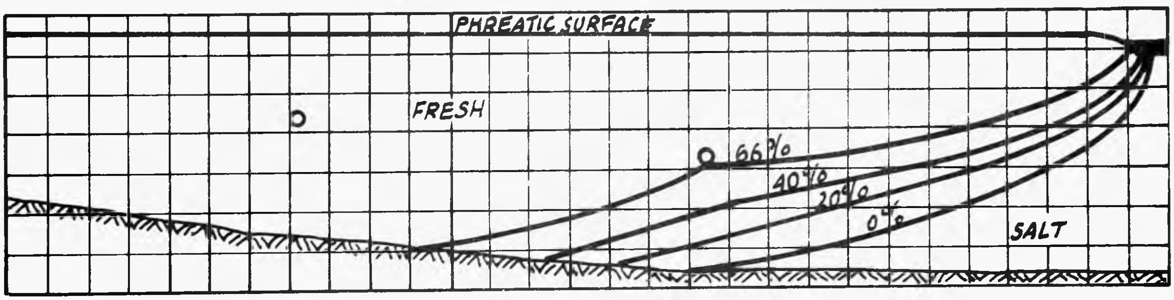
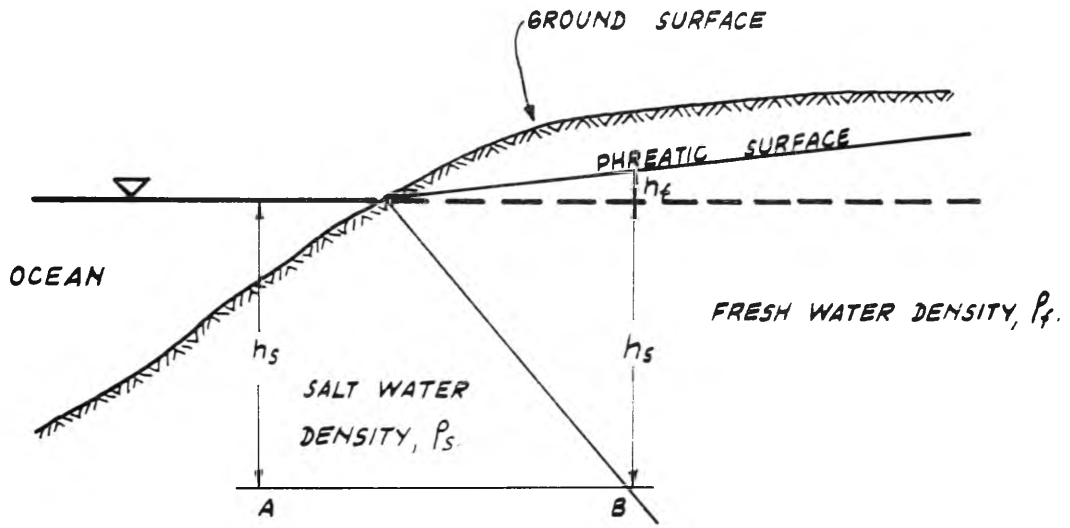


FIG. 12 (c) SERIES II, DRAIN #1 & 3

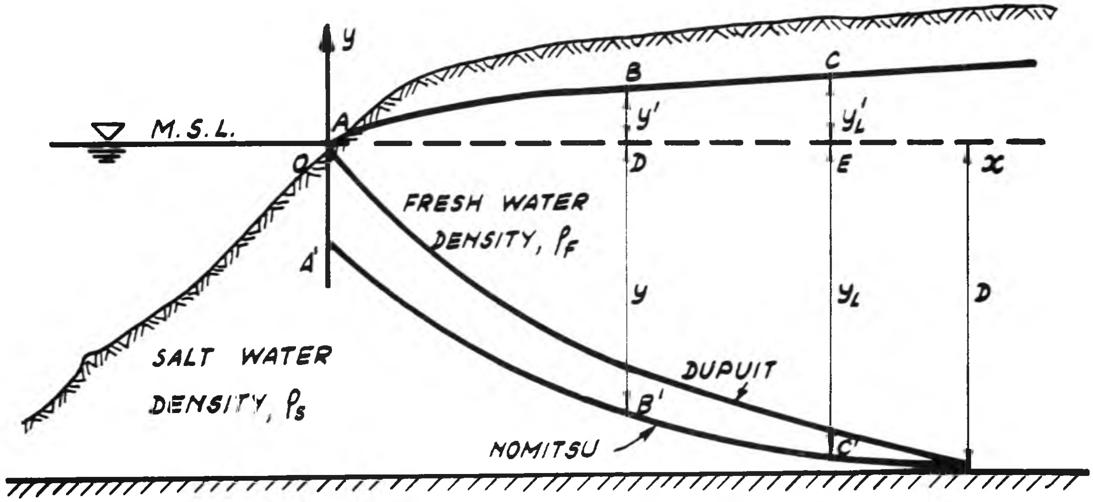
FRESH WATER – SALT WATER INTERFACE PROFILES.

P = PERCENT OF SEAWARD OUTFLOW.

SCALES: 1 DIVISION = 1 in. ON MODEL AND $41\frac{2}{3}$ ft. VERT. AND 125 ft. HORIZONTALLY ON PROTOTYPE.

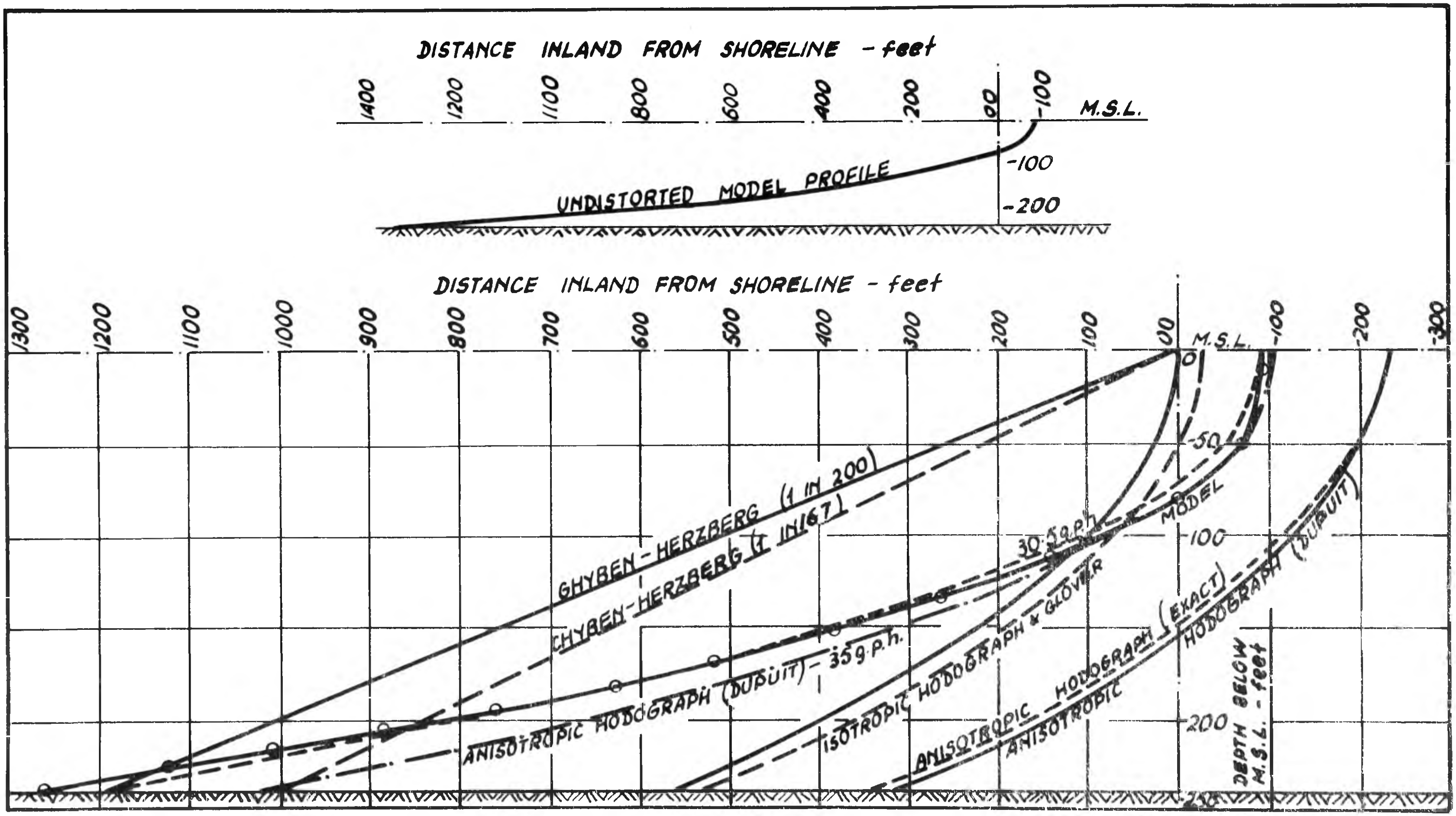


GHYBEN-HERZBERG RELATION



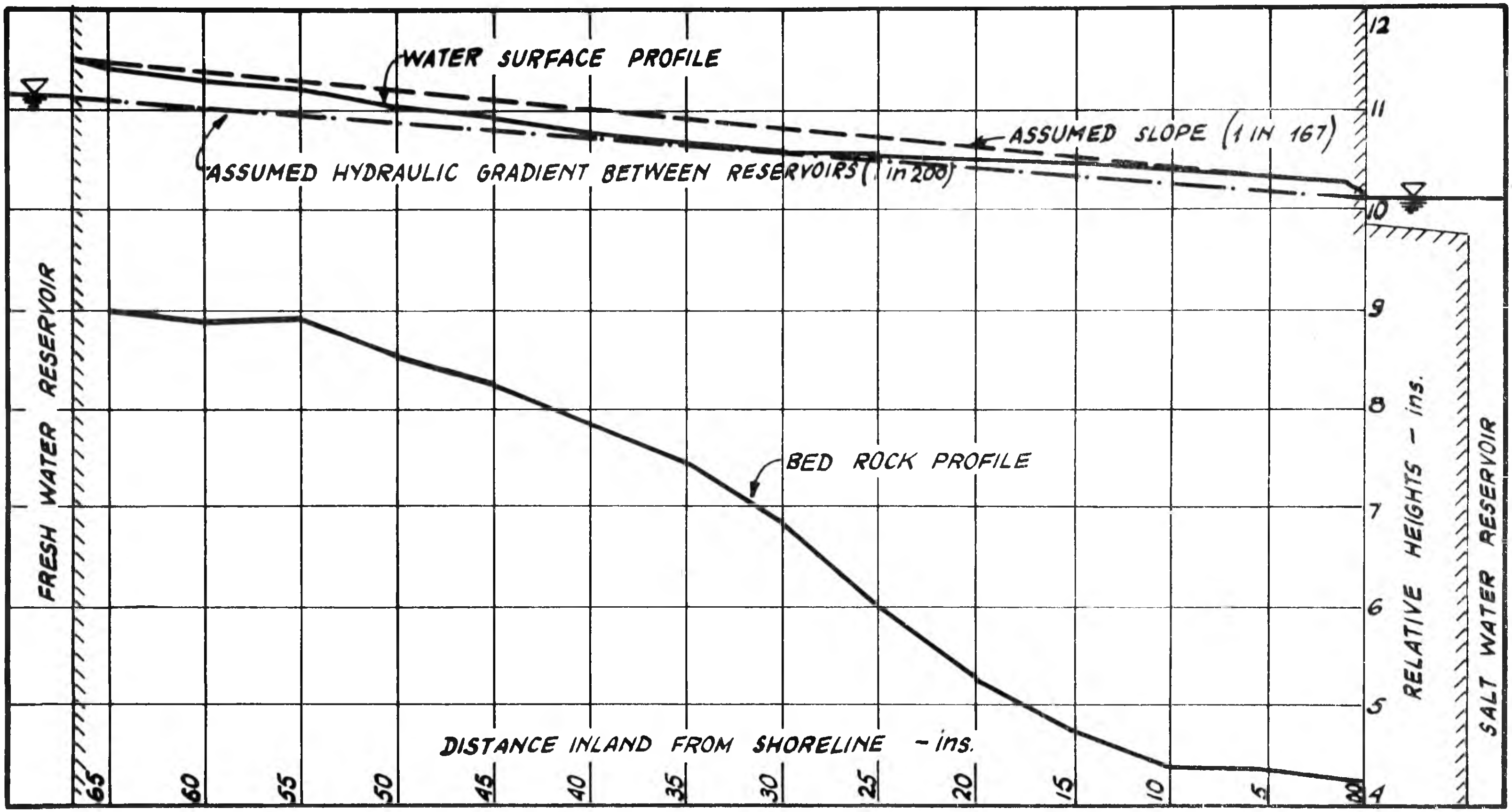
NOMITSU & DUPUIT PARABOLIC RELATION

FIG. 13



THEORETICAL AND MODEL INTERFACE PROFILES.

FIG. 14



WATER SURFACE & BED ROCK PROFILES.

FIG. 15