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ROYAL OBSERVATORY, HONG KONG

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STORM SURGE INVESTIGATIONS AND THE USE OF VERTICALLY INTEGRATED HYDRODYNAMICAL MODELS

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1. THE STORM SURGE PHENOMENON

Storm surges are complex manifestations of the exchange of energy between the atmosphere and the hydrosphere in which some of the atmospheric energy contained in winds and air pressure variations is absorbed by the sea. The classical and still the most popular explanation is that the air-sea energy transfer produces strong currents. In the open sea, these currents are associated with gradients in the surface elevation and in time will decay by the action of dissipative forces such as eddy viscosity and sea-bed friction. However, where the current is impeded by the presence of submarine barrier such as a continental shelf or by a coastline, more of the kinetic energy of the sea tends to be converted into potential energy. Abnormal elevations of the sea may then occur, with disastrous results if the coast is low-lying. More recently, storm surges have been variously regarded : as the uplift arising from a deep column of swirling water which shrinks when compensating for the vorticity loss due to bottom friction (9); as long gravity wave and edge wave effects (13); or simply as the process of continuous accretion of sea water that piles up in front of and on the right side of the storm's track. Although from individual case studies, evidence in support of each of the three interpretations of storm surge generation can be cited, investigators have been cautious in stating the exact mechanism of the phenomenon. Nevertheless, much effort has been channelled into the pressing problem of developing models which will enable accurate forecasts to be made of the height of storm surge levels at locations frequented by storms.

2. HISTORICAL STORM SURGE DISASTERS

Historically, storm surges have brought disastrous inundations and have been responsible for the majority of deaths in coastal areas affected by storms. In 1953, 1800 lives were lost in the Netherlands as a result of a surge more than 3 metres high. In 1959, surges from Typhoon 'Vera' were responsible for nearly 5,000 deaths in Japan. During Hurricane Camille of 1969 in the U.S.A. peak surges reached nearly 8 metres. In November 1970, more than 200,000 people were drowned during a cyclone in Bangladesh. Nor have high storm surges been confined to ocean coasts. The Baltic, a virtually enclosed sea was affected in 1924, when a 4-metre high surge flooded Leningrad. In 1928, surges were generated in Lake Okeechobee in the United States resulting in a 2,000 death roll.

3. SIGNIFICANCE OF STORM SURGES IN HONG KONG

In Hong Kong, the storm surge phenomenon is important both for the safety of the public and also in economic terms. With more than 1 million residents housed on reclaimed and low-lying areas, Hong Kong is threatened by the possibility of storm surge floods whenever a tropical cyclone approaches. Two of the better documented storm surge disasters occurred during a typhoon on September 2, 1937 and during Typhoon Wanda, on September 1, 1962. In the former event, 11,000 lives were lost and several villages in Tolo Harbour were destroyed. In the latter case, 127 died, 3,000 huts submerged and hundreds of hectares of farmland were inundated. Thus accurate operational storm surge forecasting is of great concern in Hong Kong. To meet the pressing need for housing from Hong Kong's ever increasing population, Hong Kong Government has undertaken large scale reclamation. Over 1,000 hectares of Tolo Harbour have been or are being reclaimed creating two New Towns of Sha Tin and Tai Po. The vast area of reclamation required a careful choice of basic reclamation formation level - one that has to be low enough to be economically acceptable but high enough to make flooding a rare event. As storm surge flooding is an overriding consideration when deciding on reclamation levels, realistic assessment of storm surge risks is indispensable for all major coastal engineering projects in Hong Kong.

4. FACTORS AFFECTING STORM SURGE LEVELS

Storm surges, taken to be the departures of sea-levels from the predicted astronomical tides, are influenced by many factors. These include :

1) Large scale parameters of the storm :

   (a) the central pressure

   (b) the distance of closest approach

   (c) the translational speed

   (d) the path of the storm

   (e) the size of the storm.

2) Coastal parameters :

   (a) sea-bed topography

   (b) coastline configuration.

and 3) Local factors :

   (a) local wind effects

   (b) river discharges

   (c) seiching effects

   (d) rainfall runoff

   (e) tidal effects.

5. EMPIRICAL METHOD OF STORM SURGE FORECASTING IN HONG KONG

The number of factors, together with the mathematical difficulties arising from non-linear terms in the fundamental equations describing the storm surge propagation generally preclude analytical solutions. Prior to attempts in numerical modelling, empirical forecasting equations were formulated to forecast peak surges in Hong Kong. In 1967, T.T. Cheng established the following regression equations relating the peak surge height to local meteorological observations. (2)

(a) $S = 0.045G - 0.43$

(b) $S = 0.087W_{10} - 0.61$

(c) $S = 0.089W_{60} - 0.45$

(d) $S = -0.085P + 87.16$

where S = Storm surge (in feet) at North Point

G = Maximum gust at the Royal Observatory in knots

W_{10} = Maximum 10 minutes mean wind at the Royal Observatory in knots

W_{60} = Maximum 60 minutes mean wind at the Royal Observatory in knots

P = Minimum sea-level pressure in millibars.

In 1976, H.F. Chan (1) extended this empirical approach and gave a further series of relationship between peak surges and storm parameter :-

(a) $S = 0.104 \times (1009.5 - P_c)$

(b) $S = 2.17 - 0.0029D$

(c) $S = 0.79 + 0.08V_s$

(d) $S = 0.088W_{60} - 0.75$

(e) $S = 0.00217W_{60}^2 + 0.43$

where P_c is the minimum sea-level pressure near the storm centre

D is the distance of nearest approach

V_s is the translational speed of the storm.

6. EMPIRICAL METHOD OF ASSESSING STORM SURGE RISKS IN HONG KONG

Prior to the development of Storm Surge Numerical Models, the assessment of storm surge risks in Hong Kong followed three approaches :-

(a) Field surveys — Long-residing residents were requested to recall the high water level marks during a sequence of historical storms and statistical inferences were then made. An example is the feasibility study of developments at Cheung Chau and Peng Chau undertaken by an engineering consultant firm (12).

TABLE 1. SURVEY OF STORM DATA FROM LOCAL RESIDENTS

Location	Storm	Observer	High water (m)
Tai O	Oct. 13, 1974	School teacher	3.5 C.D.
Tai O	Oct. 13, 1974	U.S.D. worker	3.7 C.D.
Sai Kung Town	Wanda, 1962	Fisherman	3.4 C.D.
Mui Wo (Chung Hau)	Oct., 1974	Villager	3.8 C.D.
Peng Chau(East Beach)	Wanda, 1962	Housewife	3.3 C.D.
Cheung Chau Wan	Wanda, 1962	Householder	3.5 C.D.

(b) Statistical Analysis of Historical Storm Surge data — a series of storm surge level in Hong Kong was compiled and was fitted to a Gumbel probability distribution for specific return periods. Results of such analysis has been summarised as follows :

TABLE 2. RETURN PERIODS OF WATER LEVELS IN HONG KONG USING GUMBEL'S METHOD DIRECTLY ON THE HIGHEST ANNUAL WATER LEVELS

Location	Tolo Harbour (1954-1973)	North Point (1950-1974)
Water Level (metre)	Return Period (year)	Return Period (year)
3.0		3
3.5		91
4.0	7	323
5.0	33	
6.0	180	
7.0	800	

(c) Joint Probability Analysis — the probability of various astronomical tidal levels and the probability of various storm surge levels were computed separately and statistically cross-multiplied. In 1975, P. Peterson gave the following table as results on this type of analysis :- (8)

TABLE 3. PREDICTED WATER-LEVELS - OCCURRENCE & PROBABILITIES

Location		Tolo Harbour			North Point		
Surge (A) Height	Normal Tide (B)	Prob.of (A)	Prob.of (B)	Combined Prob.	Prob.of (A)	Prob.of (B)	Combined Prob.
(m)	(m)						
4.0	any tide	0.017	1.0000	0.0170	0.00011	1.0000	0.00011
3.9 - 4.0	tide 0.1	0.003	1.0000	0.0030	0.00004	1.0000	0.00004
3.8 - 3.9	0.2	0.003	0.9991	0.0030	0.00005	0.9991	0.00005
3.7 - 3.8	0.3	0.004	0.9959	0.0040	0.00006	0.9959	0.00006
3.6 - 3.7	0.4	0.004	0.9891	0.0040	0.00008	0.9891	0.00008
3.5 - 3.6	0.5	0.004	0.9755	0.0039	0.00010	0.9755	0.00010
3.4 - 3.5	0.6	0.005	0.9628	0.0048	0.00014	0.9628	0.00013
3.3 - 3.4	0.7	0.006	0.9411	0.0056	0.00017	0.9411	0.00016
3.2 - 3.3	0.8	0.007	0.9180	0.0064	0.00025	0.9180	0.00023
3.1 - 3.2	0.9	0.007	0.8763	0.0061	0.0004	0.8763	0.00035
3.0 - 3.1	1.0	0.010	0.8143	0.0081	0.0005	0.8143	0.00041
2.9 - 3.0	1.1	0.010	0.7310	0.0073	0.0006	0.7310	0.00044
2.8 - 2.9	1.2	0.012	0.6241	0.0075	0.0007	0.6241	0.00044
2.7 - 2.8	1.3	0.013	0.5349	0.0070	0.0009	0.5349	0.00048
2.6 - 2.7	1.4	0.015	0.4534	0.0068	0.0013	0.4534	0.00059
2.5 - 2.6	1.5	0.02	0.3787	0.0075	0.0018	0.3787	0.00068
2.4 - 2.5	1.6	0.02	0.3103	0.0062	0.0023	0.3103	0.00071
2.3 - 2.4	1.7	0.02	0.2541	0.0051	0.0035	0.2541	0.00089
2.2 - 2.3	1.8	0.02	0.2016	0.0040	0.004	0.2016	0.00081
2.1 - 2.2	1.9	0.03	0.1522	0.0046	0.005	0.1522	0.00076
2.0 - 2.1	2.0	0.03	0.1042	0.0031	0.006	0.1042	0.00063
1.9 - 2.0	2.1	0.03	0.0630	0.0019	0.010	0.0630	0.00063
1.8 - 1.9	2.2	0.04	0.0317	0.0013	0.012	0.0317	0.00038
1.7 - 1.8	2.3	0.04	0.0127	0.0005	0.015	0.0127	0.00019
1.6 - 1.7	2.4	0.04	0.0023	0.0001	0.020	0.0023	0.00005
1.5 - 1.6	2.5	0.05	0.0000	0.0000	0.025	0.0000	0.00000
TOTAL PROBABILITY				0.1288			0.00940
RETURN PERIOD				7.8 years			106 years

7. SOME SHORTCOMINGS OF EMPIRICAL METHODS

Although statistical approaches to storm surge prediction and surge risk assessment is simple from the computational point of view, little insight is gained into the physics of the air-sea interaction with their application. Regression equations provide peak surge forecasts but no account has been taken of the surge profile with time. Another complication is that, whilst peak surges correlate highly with particular meteorological parameters, a multiple regression, of these parameters combined greatly reduces the significance of the predicted levels. In Hong Kong, the fundamental reason for the preference of the full numerical dynamical treatment of the storm surge problem over the empirical method, lies in the rapid and drastic changes in the shoreline due to reclamation. Even bottom topography is continuously modified with the construction of more cross harbour tunnels.

8. NUMERICAL MODELS FOR FORECASTING OPEN COAST STORM SURGES — SPLASH I & II

Recently, great advances have been made in the development of Open Coast Storm Surge models. In general, these models either simulate the movement of a model storm along/across the open coast and compute the propagation of water movement under the influence of the prescribed storm pressure and wind field (e.g. Jelesnianski Splash Model)(4) or utilise pressure and wind outputs from atmospheric models and proceed onto the water movement computations (Heaps & Bank's North Sea Model) (3).

In this note only the Jelesnianski Splash Model is described. Splash, the operational U.S. Weather Service's numerical storm surge model, forecasts storm surge levels both in space and in time. This model consists of a storm travelling across a rectangular basin of variable depth. The storm surge driving forces from pressure and wind, are determined from a tropical storm model. The driving forces are applied to the storm surge equations. Numerical calculations are generally made for a grid distance of 6.5 km (4 miles) with 2.5 minutes time interval between computations. The Splash model can be used for two types of storms: those that cross the coast (Splash I) and those that travel along the coast but do not actually landfall (Splash II). For storms expected to cross the coast the following information is incorporated in the model :-

(a) the landfall position of the storm

(b) central pressure of the storm

(c) storm speed

(d) storm direction of movement

(e) radius of maximum wind, which is a measure of storm size.

For storms that do not landfall, five storm positions at 6 hourly intervals are supplied to the model.

The results of the model computation — the expected storm surge — are displayed by programmed computer outputs.

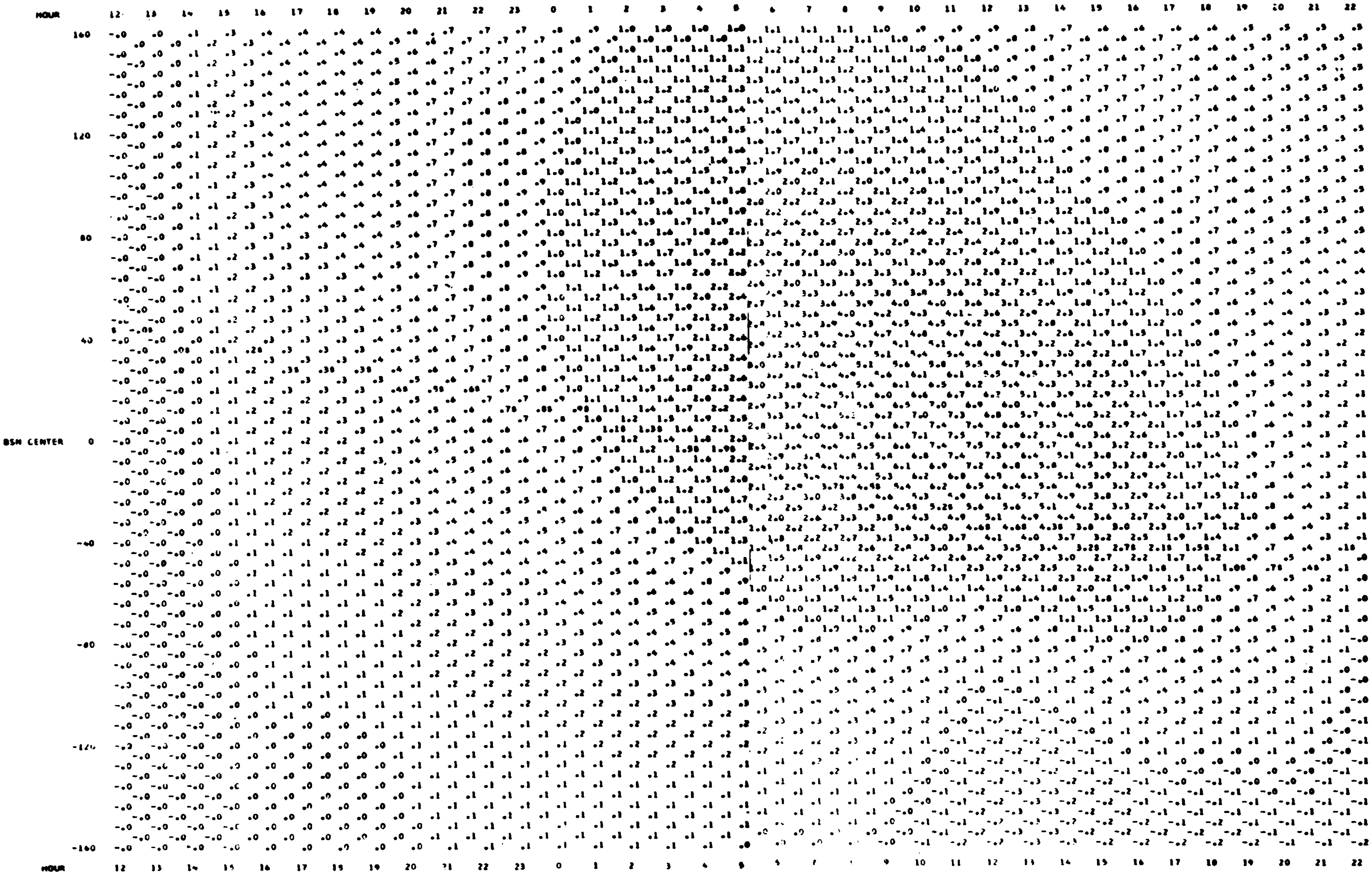
A sample is given in Figure 1.

THIS IS A SPACE-TIME PLOT OF THE COASTAL SURGES

THE COMPUTATIONS HAVE BEEN INITIALIZED FOR TUE, THE 20 OF AUG, 1968 AT 11 HOUR

GROWTH TIME OF STORM TO MATURITY, AFTER INITIALIZATION, IS 4.2 HOURS

NEAREST APPROACH TIME OF STORM TO BASIN CENTER IS 11 HOURS.
PRIOR TO THIS TIME THE COMPUTED SURGE VALUES MAY BE SERIOUSLY IN ERROR DUE TO INITIALIZATION PROCEDURES
THIS CAN EASILY HAPPEN IF THE STORM IS INITIALIZED ON THE CONTINENTAL SHELF, IN SHALLOW WATER, CLOSE TO THE COAST.
EXAMPLE 1.
SOMETIMES THERE ARE EXCEPTIONALLY LARGE SURGES PRIOR TO 11 HOURS DUE TO TRANSIENT OVER-REACTION OF THE SEA.
IF THIS APPEARS BETWEEN 12 TO 17 HOURS, AND IF DURING THIS PERIOD
1. STORM DISTANCE ABEAM OF COAST IS SMALLER THAN STORM DIAMETER
2. STORM IS MOVING TOWARD THE COAST
THEN THE INITIALLY LARGE SURGE MAY HAVE A SIGNIFICANT + ERROR.
EXAMPLE 2.
SOMETIMES THERE ARE SMALLER THAN EXPECTED SURGES PRIOR TO 11 HOURS DUE TO TRANSIENT UNDER-REACTION OF THE SEA.
IF THIS APPEARS BETWEEN 12 TO 17 HOURS, AND IF DURING THIS PERIOD
1. STORM DISTANCE ABEAM OF COAST IS SMALLER THAN STORM DIAMETER
2. STORM IS MOVING AWAY FROM THE COAST
THEN THE SMALLER THAN EXPECTED SURGE MAY HAVE A SIGNIFICANT NEGATIVE ERROR.
NOTE--THE COASTAL HIGH WATERS LAG THE STORM. HENCE, FOR A FAST MOVING STORM
THE COMPUTED SURGE MAY BE TOO SMALL AT TIME OF TERMINATION OF COMPUTATIONS.



HOUR	12	13	14	15	16	17	18	19	20	21	22	23	0	1	2	3	4	S
PRES DROP	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0
STRM SIZE	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
MLS ABEAM	268.4	257.1	245.7	234.4	223.0	211.7	200.3	189.0	177.6	166.3	154.9	143.6	132.2	120.9	109.5	98.1	86.7	75.3
STRM SPD	11.7	11.7	11.7	11.7	11.7	11.7	11.7	11.7	11.7	11.7	11.7	11.7	11.8	11.9	12.0	12.1	12.2	12.3
MAX WIND	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5	87.5

THE MAX-WIND IS A STORM-SCALE-WIND, THAT IS, THE WIND AVERAGED AROUND THE CIRCLE OF HIGHEST WINDS FOR A STATIONARY STORM.
THE FASTEST-MILE-WIND IS ABOUT 1.3 TIMES FASTER.

THE SYMBOL, S, LISTS APPROXIMATE HOURLY STORM POSITIONS, ABEAM OF THE COASTLINE.

Figure 1 Sample splash printout

A brief background of the hydrodynamical formulae of the Splash Programme is given below :-

Basic Equations of the Splash Open Coast Model.

$$\frac{\partial U}{\partial t} = - gD\ (x, y)\ \frac{\partial h}{\partial x} + fV + T^x\ (x, y, t) \qquad (1)$$

$$\frac{\partial V}{\partial t} = - gD\ (x, y)\ \frac{\partial h}{\partial y} - fU + T^y\ (x, y, t) \qquad (2)$$

$$\frac{\partial h}{\partial t} = - \frac{\partial U}{\partial x} - \frac{\partial V}{\partial y} \qquad (3)$$

where D = depth of basin from the equilibrium surface

f = Coriolis parameter

T^x, T^y = force components per unit mass

h = height of disturbed surface fluid from the undisturbed free surface

U = transport in the x direction

V = transport in the y direction

g = acceleration due to gravity

For a circular storm :

$$\frac{1}{p_a}\ \frac{\partial p}{\partial r} = \frac{V_c^2}{r} \qquad (4)$$

$$V = K\ V_c \qquad (5)$$

where p_a = air density

p = pressure

r = distance from storm centre

V_c = cyclostrophic wind

V = surface wind

K = proportionate constant

Also

$$V = V_R\ \left(\frac{r}{R}\right)^{3/2}, \qquad 0 \leqslant r < R \qquad (6)$$

$$V = V_R\ \left(\frac{R}{r}\right)^{1/2}, \qquad r \geqslant R \qquad (7)$$

where V_R = maximum wind

R = radius of maximum wind

and

$$\frac{\partial p}{\partial r} = p_a\ \left(\frac{V_R}{K}\right)^2 \frac{r^2}{R^3}, \qquad 0 \leqslant r < R \qquad (8)$$

$$\frac{\partial p}{\partial r} = p_a\ \left(\frac{V_R}{K}\right)^2 \frac{R}{r^2}, \qquad r > R \qquad (9)$$

$$p_\infty - p_o = \frac{4}{3}\ p_a\ \left(\frac{V_R}{K}\right) \qquad (10)$$

$$K = 2\ V_R \sqrt{p_a/3\ (p_\infty - p_o)} \qquad (11)$$

where p_o = pressure at centre

p_∞ = pressure at great distance from centre

9. NUMERICAL MODEL FOR COMPUTING NEAR SHORE WATER LEVELS AND MOVEMENTS

To adequately simulate the features of a storm, open coast models such as Splash, must necessarily adopt large grid sizes in the numerical computation. As such, open coast models cannot cope with surge height variation near the shores of complex embayments and estuaries. This is the realm of study of the coastal engineers. After W. Hansen, hydrodynamical-numerical models have been developed by modellers to analyse and predict tides, and storm surges on small grid sizes of 1 kilometre or under. These models, referred to as 'Bay models', generally neglect atmospheric pressure variations, but can simulate the propagation of water movement shoreward given the seaward water level variations. One such model is the 'Vertically Integrated Hydrodynamical-Numerical Model' by the Oceanography Department of the U.S. Navy Environmental Prediction Research Facility. A full description of an adapted version of this type of Bay Model is given on the next page (11).

10. THE IDEA OF COUPLING THE OPEN COAST MODEL TO BAY MODELS

The two types of storm surge numerical models have been idenpendently developed by two scientific disciplines : the meteorologists and the coastal hydraulic engineers.

The meteorologists' major concern is in finding out how the raised sea underneath a storm propagates along a continental shelf as the storm moves on a prescribed track. The entity under computation is the height of the sea-level above the astronomical tide i.e. the storm surge level. The area covered by a storm is large. Because of the coarse grid size, coastline geography is greatly simplified and the storm surge computations often end at a location that is tens of kilometres from the shore.

The coastal engineers whose tasks are to design formation levels for waterfront structures, are concerned with the effects of shoaling in surge levels plus astronomical tides. Using numerical schemes with grid lengths of half a kilometre or less, these models simulate the coastline geography closely. They can provide computer-simulated tide gauge records at locations right up to the shore, given inputs of water level variations at a location tens of kilometres seaward.

Ideally, if the two categories of hydrodynamical models can be merged into one, for any storm track, the propagation of the raised sea will be depicted in its entirety from near the storm centre, across the continental shelf, to the shores of the coast. However, having been developed for separate objectives, the two types of hydrodynamical models compute different entities (storm surge levels versus total water levels) and use grid sizes an order of magnitude apart. Merging models constitute a presently insurmountable task.

However, analogous to nesting atmospheric models, it is possible to couple these models. The output of the Open Coast Model in the form of storm surge variations with time at an interface location can be easily added to the predicted astronomical tidal variations. The resultant water-level variations at the interface location can then be treated as inputs to the Bay Model which will then describe the propagation of these raised sea-levels shorewards to locations within bays and inlets.

Such non-interactive coupling of the Open Coast Model to the Bay Model was attempted by staff of the Storm Surge Unit of the Royal Observatory, Hong Kong. Details of the methodology and limitations when applying the technique of coupling an Open Coast Model to a Bay Model to evaluate storm surge risks at locations within embayment are given in the Royal Observatory paper 'Report on the Tolo Harbour Storm Surge Numerical Modelling Investigations (6).

11. DEVELOPMENT OF THE BAY MODEL AT THE ROYAL OBSERVATORY

Open Coast Surge Models such as the Splash Program require computer capabilities beyond that of the Royal Observatory's computer which had a core size of 32K. On the other hand, the Bay Models, requiring about 20K core for a basin with 10x10 grid points, are amenable to adaptation and development using a computer of 32K. Thus, the adaptation of the Bay Model for use in Hong Kong has been the subject of attention for several storm surge investigations over the past few years. The following is a detailed description of the Bay Model currently in use by the Observatory for storm surge studies.

(a) Basic Hydrodynamical Equations

In the single-layer model, two momentum equations, in the X direction and in the Y direction (Equations (1) & (2)) and a continuity equation (Equation (3)) are used :-

$$\frac{\partial U}{\partial t} + fV - \nu\nabla^2 U + \frac{r}{H} U\sqrt{U^2 + V^2} + g\frac{\partial \zeta}{\partial x} = \frac{\tau^x}{H} \qquad (1)$$

$$\frac{\partial V}{\partial t} - fU - \nu\nabla^2 V + \frac{r}{H} V\sqrt{U^2 + V^2} + g\frac{\partial \zeta}{\partial y} = \frac{\tau^y}{H} \qquad (2)$$

TERMS: A B C D E F

$$\frac{\partial \zeta}{\partial t} + \frac{\partial}{\partial x}(HU) + \frac{\partial}{\partial y}(HV) = 0 \qquad (3)$$

TERMS: G H I

where
- x, y = space co-ordinates
- t = time
- U, V = components of current velocity
- ζ = surface elevation
- H = total depth
- τ^x, τ^y = components of wind stress
- g = acceleration due to gravity
- f = Coriolis parameter
- r = friction coefficient (bottom stress)
- ν = coefficient of horizontal eddy viscosity
- ∇^2 = Laplacian

$$\tau^x = \lambda W_x \sqrt{W_x^2 + W_y^2} \qquad (4)$$

$$\tau^y = \lambda W_y \sqrt{W_x^2 + W_y^2} \qquad (5)$$

where
- λ = coefficient of friction (drag coefficient)
- W_x, W_y = wind speeds

In the momentum equations :

Term A	is the acceleration of water currents.
Term B	is the Coriolis acceleration.
Term C	is the eddy viscosity retardation.
Term D	is the bottom stress.
Term E	is the acceleration due to the difference in water elevations.
Term F	is the wind stress.

In the continuity equation :

Term G	is the time rate of change of water elevation.
Terms H & I	are the convergence or divergence of water mass resulting from the water currents U & V.

(b) The algorithm for solving the hydrodynamical equations

By neglecting the lesser effects of the Coriolis acceleration, eddy viscosity, the basin algorithm for solving the three hydrodynamical equations numerically can be simply illustrated as follows :-

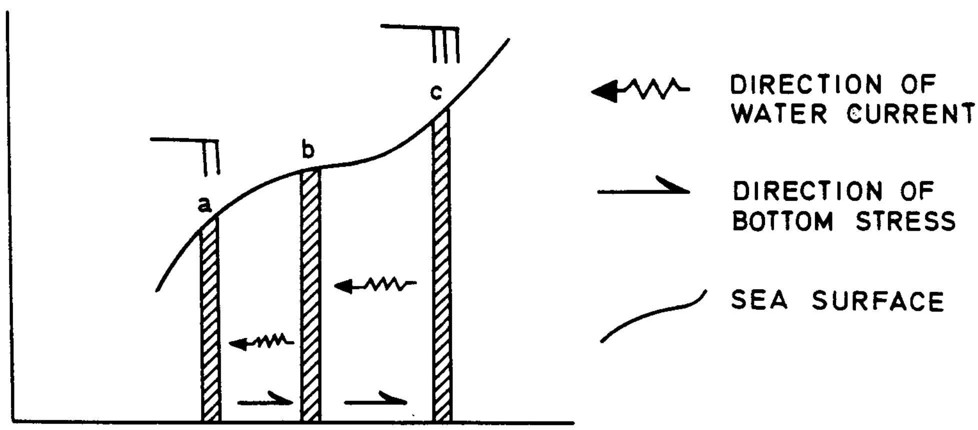


FIGURE 2. GRAPH ILLUSTRATING THE PROCEDURE

Water elevation at c, being higher than at b, will force a water current to flow towards b. This current is aided by wind and retarded by bottom friction. The magnitude of the water currents in X & Y directions, Ucb and Vcb can be computed from equations (1) & (2). Similarly, higher elevations at b will cause a current to flow towards a. Equations (1) & (2) again enable computations of currents Uba, Vba. If one now applies the continuity equation for point b, the convergence of water mass at b can be evaluated because Ucb, Vcb, Uba & Vba are all known. With convergence computed, the continuity equation then gives the change in water elevation at b over a certain duration of time. Hence, if a time step is specified, new water elevation at b can be computed from the old elevation by adding the rate of change multiplied by the time step.

Clearly therefore, given a grid net, a water elevation field will generate a new current field after solving the momentum equations; a new current field will in the next time step generate a new water elevation field by applying the continuity equation. This process of repetitive alternate generation of elevation and current whilst stepping forward in time constitutes the algorithm for the numerical solution of the basic hydrodynamical equations.

(c) The finite-difference formulations

In solving the three equations numerically, the usual finite-difference approach is used. The finite-difference approximations for equations (1) through (3) are :

$$\varsigma^{t+\tau}(n,m) = \bar{\varsigma}^{t-\tau}(n,m) - \frac{\tau}{\ell}\Big\{H_u^t(n,m)\,U^t(n,m) - H_u^t(n,m-1)\,U^t(n,m-1) + H_v^t(n-1,m)\,V^t(n-1,m) - H_v^t(n,m)\,V^t(n,m)\Big\} \quad (6)$$

$$U^{t+2\tau}(n,m) = \Big\{1 - \Big[2\tau r/H_u^{t+2\tau}(n,m)\Big]\sqrt{\bar{U}^t(n,m)^2 + V^{*t}(n,m)^2}\Big\}\,\bar{U}^t(n,m) + 2\tau f V^{*t}(n,m) - \frac{\tau g}{\ell}\Big\{\varsigma^{t+\tau}(n,m+1) - \varsigma^{t+\tau}(n,m)\Big\} + 2\tau X^{t+2\tau}(n,m) \quad (7)$$

$$V^{t+2\tau}(n,m) = \Big\{1 - \Big[2\tau r/H_v^{t+2\tau}(n,m)\Big]\sqrt{\bar{V}^t(n,m)^2 + U^{*t}(n,m)^2}\Big\}\,\bar{V}^t(n,m) - 2\tau f U^{*t}(n,m) - \frac{\tau g}{\ell}\Big\{\varsigma^{t+\tau}(n,m) - \varsigma^{t+\tau}(n+1,m)\Big\} + 2\tau Y^{t+2\tau}(n,m) \quad (8)$$

The averaged velocity and water elevation (sea level) components are :

$$\bar{U}^t(n,m) = \alpha U^t(n,m) + \frac{1-\alpha}{4}\Big\{U^t(n-1,m) + U^t(n+1,m) + U^t(n,m+1) + U^t(n,m-1)\Big\}. \quad (9)$$

$\bar{V}^t(n,m)$ and $\bar{\varsigma}^t(n,m)$ are analogous to $\bar{U}^t(n,m)$ above.

(The factor α can be interpreted as a "horizontal viscosity parameter." Its normal value is 0.99).

$$U^{*t}(n,m) = \frac{1}{4}\Big\{U^t(n,m-1) + U^t(n+1,m-1) + U^t(n,m) + U^t(n+1,m)\Big\} \quad (10)$$

$V^{*t}(n,m)$ is analogous to the $U^{*t}(n,m)$ above.

The time step is 2τ. The total depth (H_u, H_v) is computed as :

$$H_u^{t+2\tau}(n,m) = h_u(n,m) + \frac{1}{2}\Big[\varsigma^{t+\tau}(n,m) + \varsigma^{t+\tau}(n,m+1)\Big] \quad (11)$$

(d) The grid net

As illustrated earlier, water currents flow between grid points where water elevations are computed. Thus a staggered grid system is employed. The grid net, shown in Figure 3, consists of three different sets of grid points : (1) the water elevation points (z) at the intersection of the grid; (2) the point for u-velocity component to the right and (3) the v-velocity component below the corresponding z point. Each of these three points has the same co-ordinate designation (n,m).

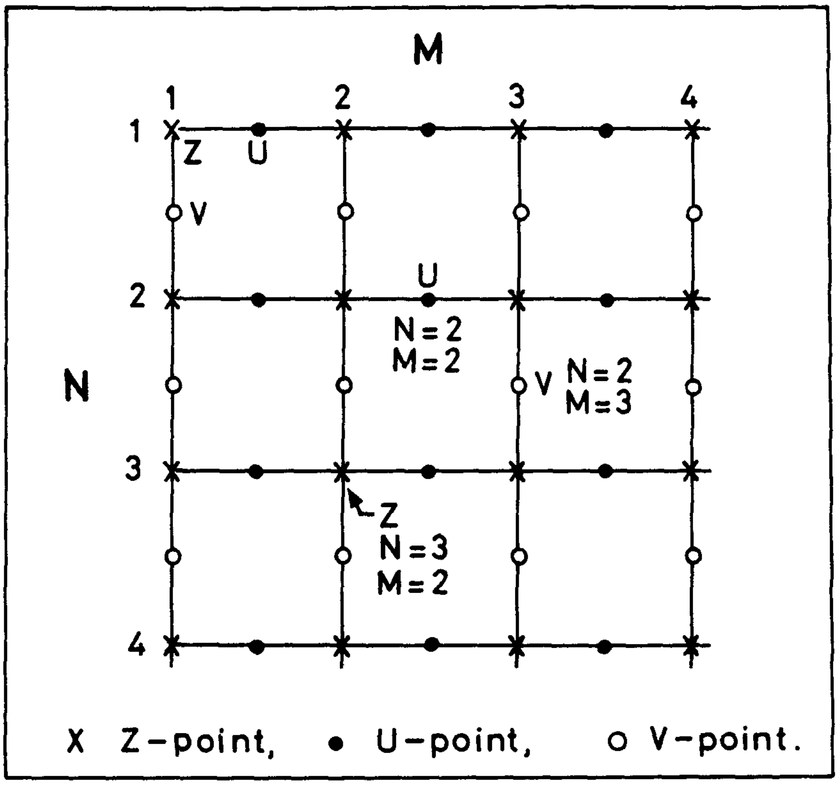


FIGURE 3. THE GRID NET

(e) Size of grid & stability criteria

The selection of the size of the area and the distance between grid points is usually based on requirements of details and accuracy, and availability of computer core memory.

The maximum length of the time step is determined by the grid length and maximum depth found in the area according to the Courant-Friedrichs-Lewy criteria :

$$\Delta t < \frac{L}{\sqrt{2gH_{max}}}$$

where Δt is time step (seconds); L is grid length (centimetres); H_{max} is the maximum depth in the area of computation (centimetres); g is the gravitational constant.

Any attempt to increase the time step above the criterion has resulted in the 'blow up' of computations (computational instability).

(f) Input of depths

In the momentum equations and the continuity equations, depth values of the sea bottom at each grid point are required. Three sets of depth values must be specified. The first set is symbolic depth (sea and land table) at z points : the z points over land are designated 0; at the boundary on the coast, -1 (immediately outside the boundary line); over the water 1; and at the input boundary 2. The second and third sets of depth values are read off from the Admiralty Chart or from soundings of hydrographic surveys. They are read at u and v points respectively.

(g) Boundaries

There are two types of boundaries normally encountered in the numerical simulation of storm surges. One is the water-land interface, while the other is the designated open water boundary which is the artificial termination of the grid system. Various boundary conditions can be imposed. In the Observatory Bay Model, the time variation of the combined surge and tide levels are invariably imposed onto the boundary to force currents into the bay under consideration.

(h) Initialisation

Unlike atmospheric models, the initialisation of the Bay Model is simple. Both the water elevations and the currents for all grid points may be set to zero and computation commences by inputting the water elevations at the input boundary only. In the event, a lead up time will be required for the water elevation and currents to attain equilibrium after which the Bay Model will respond faithfully to the applied tidal forcings.

(i) The bottom stress

Along with the wind stress and the smoothing parameter, the bottom stress parameter is one of the three important parameters that must be specified for the Bay Model.

The effects of bottom stresses are highly complicated. It is known (9) that an Ekman-type spiral due to bottom friction in a steady flow can act in the same direction as the wind stress adding to the setup. It is also found that the total bottom drag in non-steady flow normally acts against the wind stress and hence the setup may be less than in the absence of friction.

Different ways have been tried to compute the effects of bottom friction. In these trials, the main differences in results occur in shallow water. In deep water, the bottom stress becomes small. In the Observatory Bay Model, the bottom friction is considered to always act against the direction of current and is set in the magnitude ranges between 0.1×10^{-2} and 0.3×10^{-2}.

(j) The wind stress

The wind stress is a primary parameter in any storm surge model. Unfortunately, it is still far from clear how the surface drag is related to the wind over water. Munk (7) stated the drag is composed of two parts (i) the pure stress, being quardratic in velocity, (ii) the form drag, proportional to the cube of velocity.

Comparison of computed water setup with measured values and hindcasting of storm surges indicated that a typical drag coefficient would be of the order of 3×10^{-3}. Direct measurement of momentum fluxes over water suggested much smaller values. Timmerman (10) gave the following summary of the survey of wind drag coefficients.

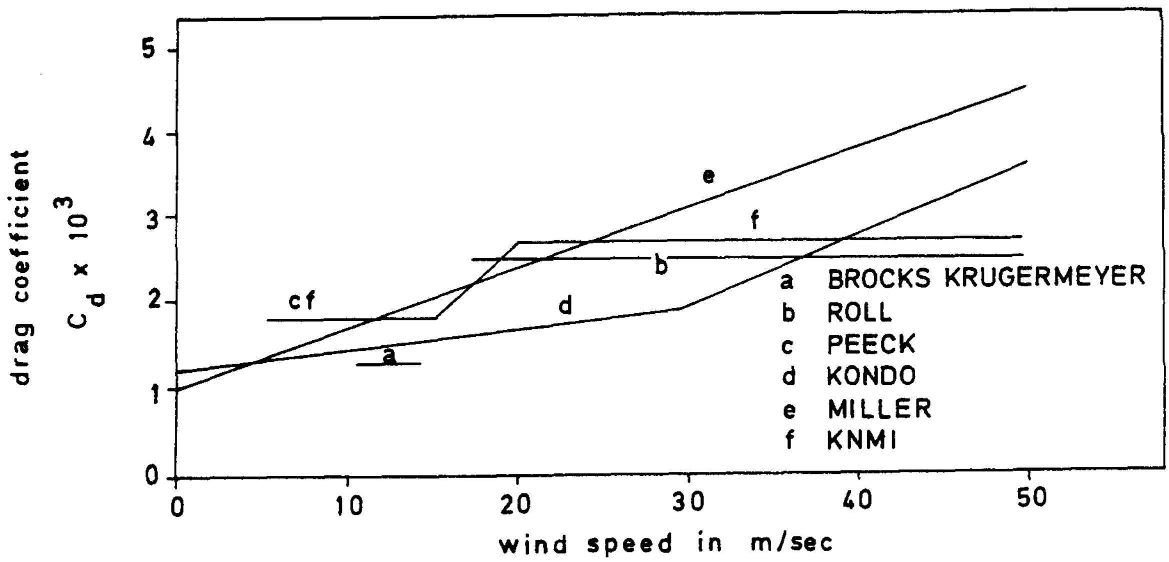


FIGURE 4. SURVEY OF DRAG COEFFICIENTS

In the Observatory Bay Model, a value of wind drag = 0.0018 is used for wind speeds up to 14 m/sec; a value of wind drag = 0.0027 is used for wind speeds 20 m/sec or more; a linear interpolation of the wind drag coefficients is used for wind speeds between the two values.

(k) The smoothing parameter

The smoothing coefficient is used to account for horizontal viscosity. In the Model, a value from 0.90 to 0.998 is used. This coefficient is also used as a tuning factor. When a lower value (e.g. 0.90) is used, the current speed is decreased making the water more viscous and generally delaying the occurrence of high and low tides.

12. SENSITIVITY TESTS ON THE BAY MODEL

In order to gain insight into the relative effects on the storm surge levels when individual parameters of the Bay Model vary, a series of Bay Model runs was performed on the simplest hypothetical basin with one and only one parameter varying at a time. The basin used is a rectangular basin with uniform depth for all grid points, being enclosed on three sides and having one seaward inputting boundary.

The results of the series of runs are summarised in Figures 5 to 12. Some of the main findings are listed below :

(i) When the wind drag coefficient, bottom friction and the smoothing parameter are kept constant, the increase in water elevation at a grid point when imposing a 50 m/sec wind onto the basin is on average 3.9 times the increase in water elevation when a 25 m/sec wind is imposed. (Figure 5)

(ii) When the direction of the imposing wind is changed from on-shore to off-shore, other parameters being kept constant, the wind effects on the computed water levels are for all practical purposes reversed. (Figure 6)

(iii) Cross wind effects on computed water-levels are small although water does pile up slightly to the right of the wind due to the Coriolis force. (Figure 7)

(iv) When the wind drag coefficient is increased from 0.32×10^{-2} to 0.48×10^{-2}, an increase of 50 percent, the computed water-levels due to this increase in drag coefficient are on average 48 percent higher. (Figure 8).

(v) When the bottom friction is increased from 0.1×10^{-2} to 0.2×10^{-2}, other parameters being kept unchanged, the computed water-levels are on average decreased by less than 10 percent. When the profile of the computed water-level variation is plotted, the peaks and troughs are found to be slightly delayed with the increase in bottom friction. (Figure 9)

(vi) When the smoothing parameter is changed from 0.99 to 0.90, other parameters being kept constant, the computed water elevations are on average about 4 percent lower. The peaks and troughs of the computed water profile are also slightly delayed in a similar way to the case for increasing bottom friction. (Figure 10)

(vii) The seaward boundary of the numerical basin is an artificial termination of the bay in the Bay Model runs. The distance from this boundary to the shore of the bay generally governs the period of the seiching.

In the sensitivity runs, when the basin length has been doubled, results show that seiching tends to be more pronounced, the corresponding periods of the seiching are also doubled. (Figures 11 & 12).

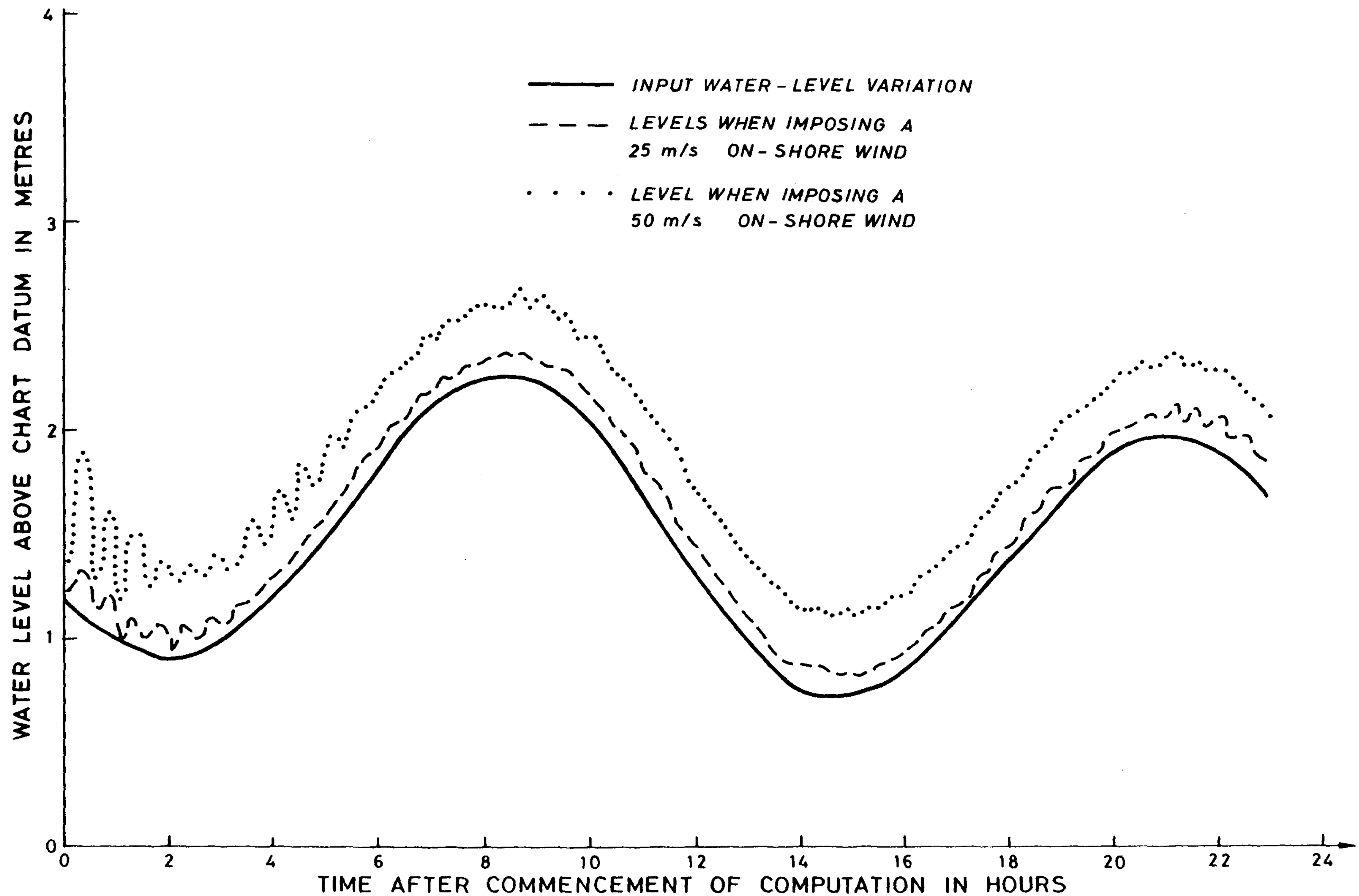


Figure 5 Model output water-levels when varying wind speed

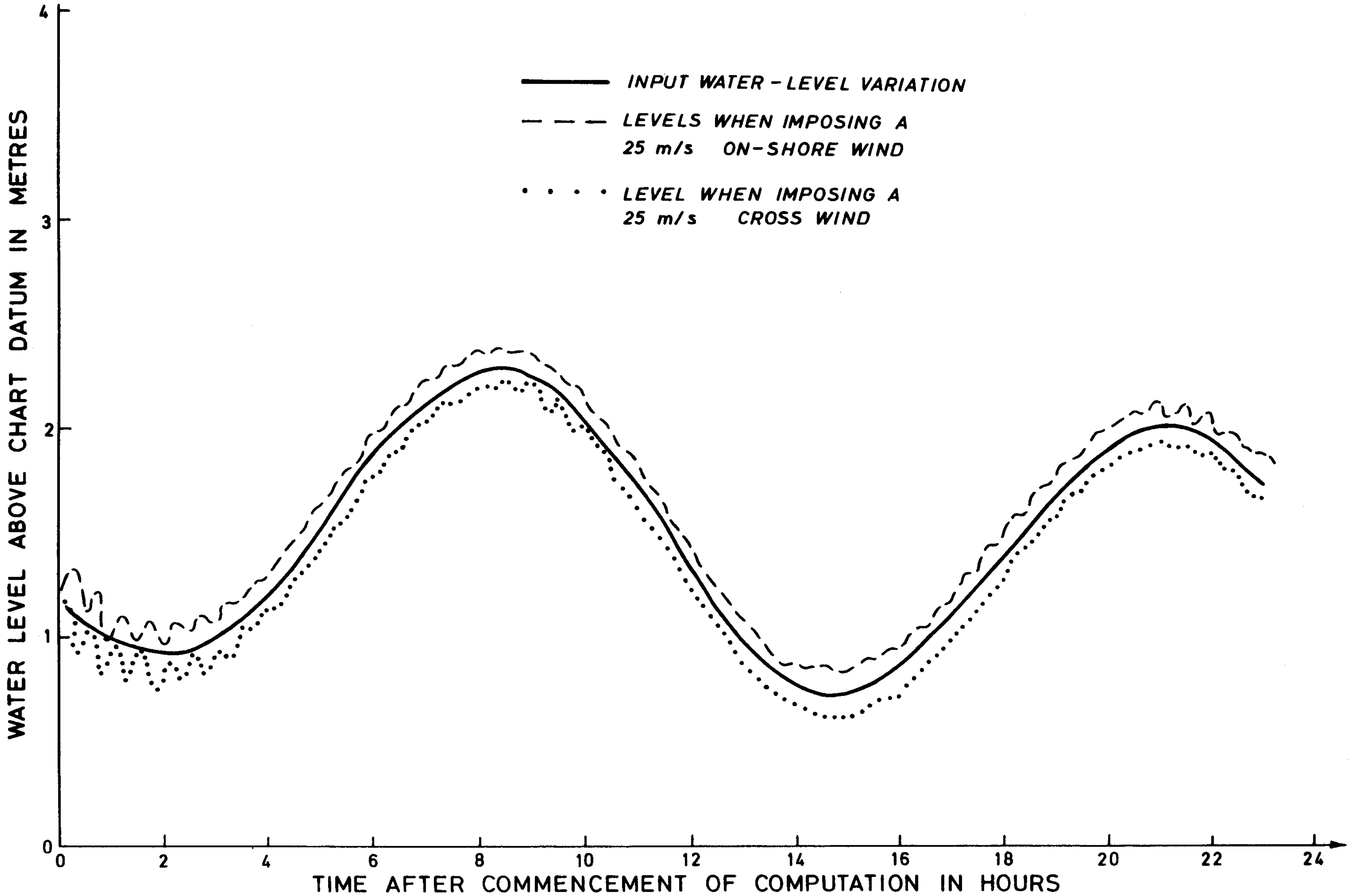


Figure 6 Model output water-levels when varying wind direction

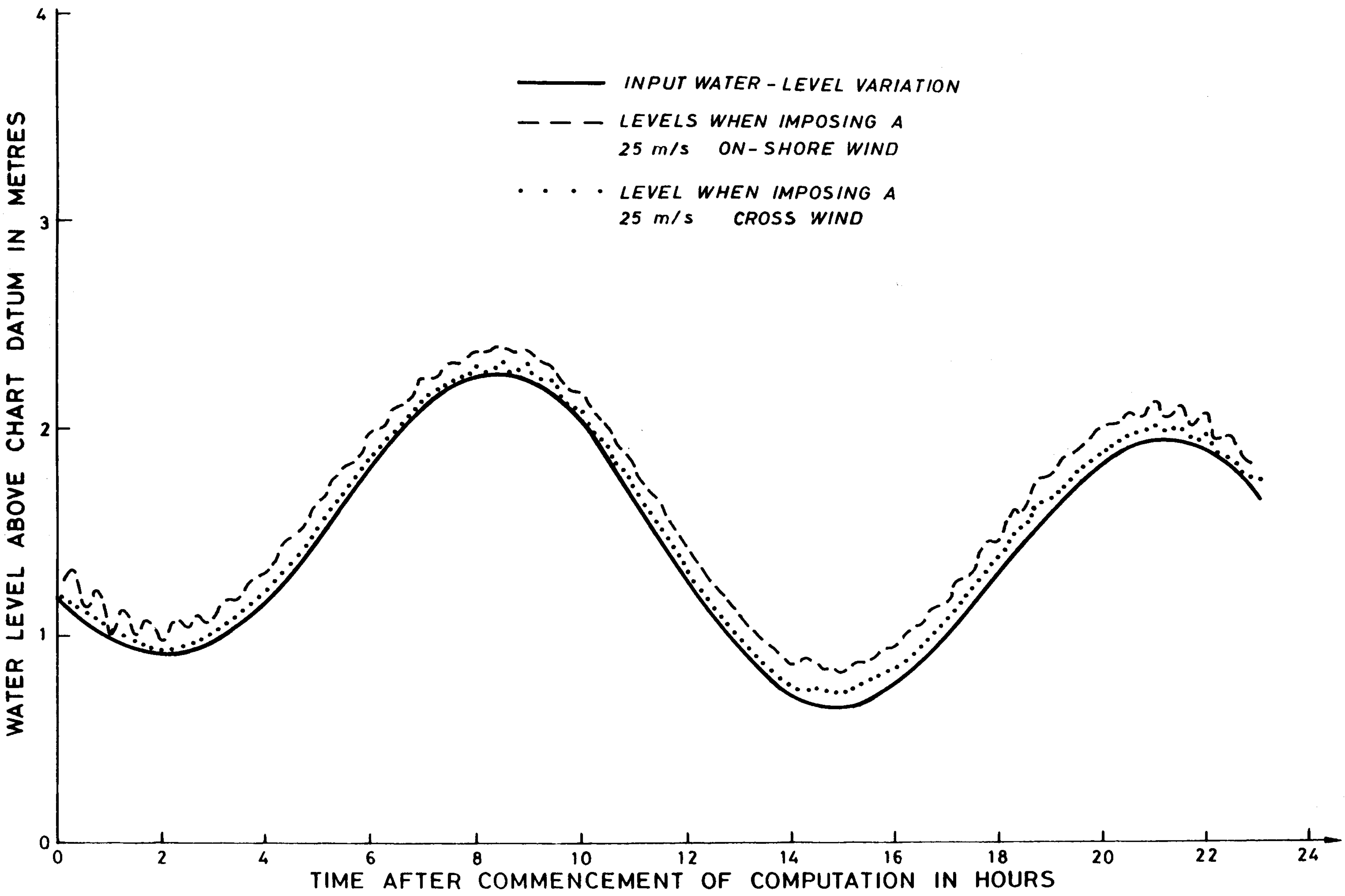
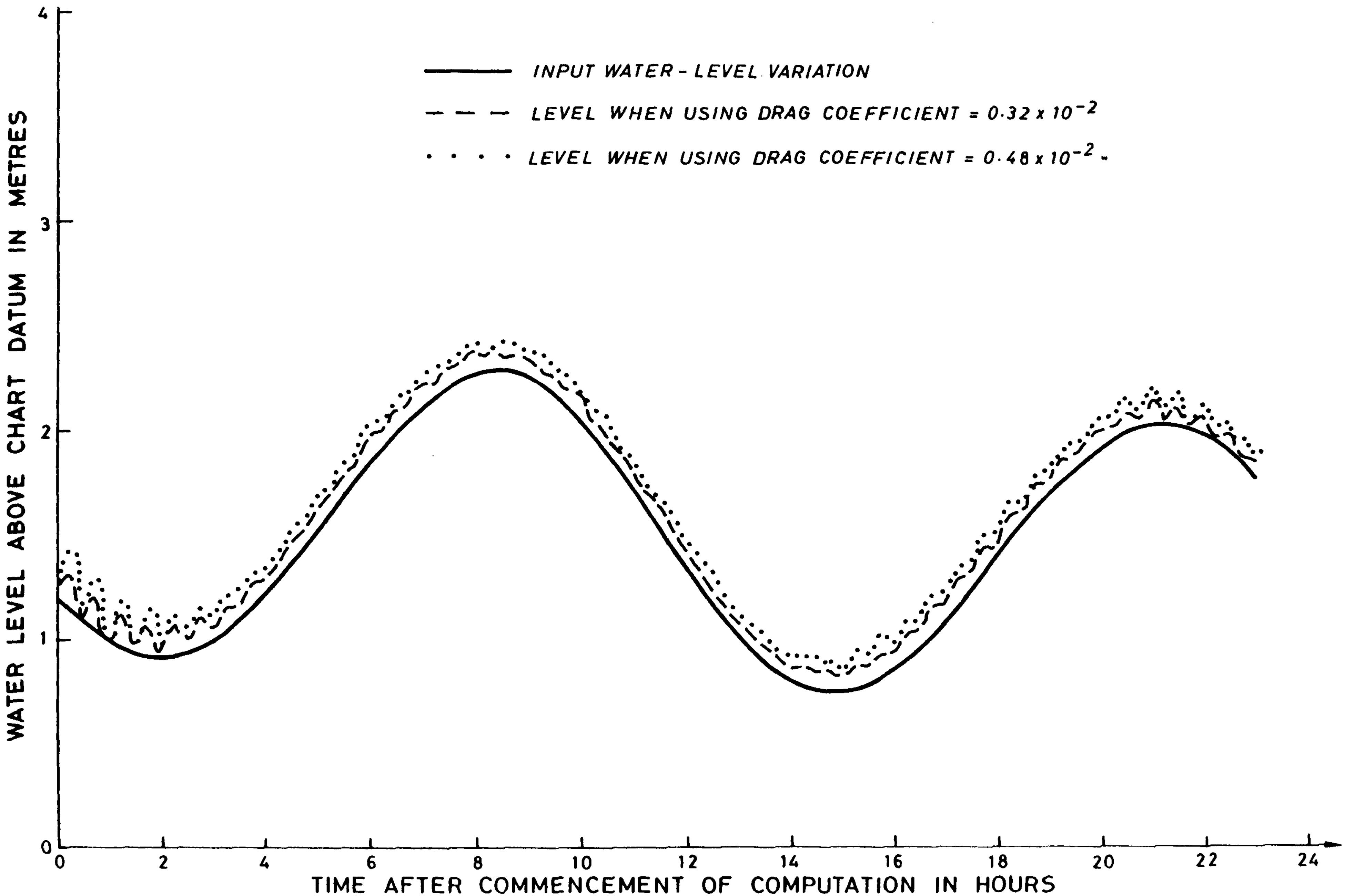


Figure 7 Model output water-levels for cross wind situation

Figure 7 Model output water-levels for cross wind situation



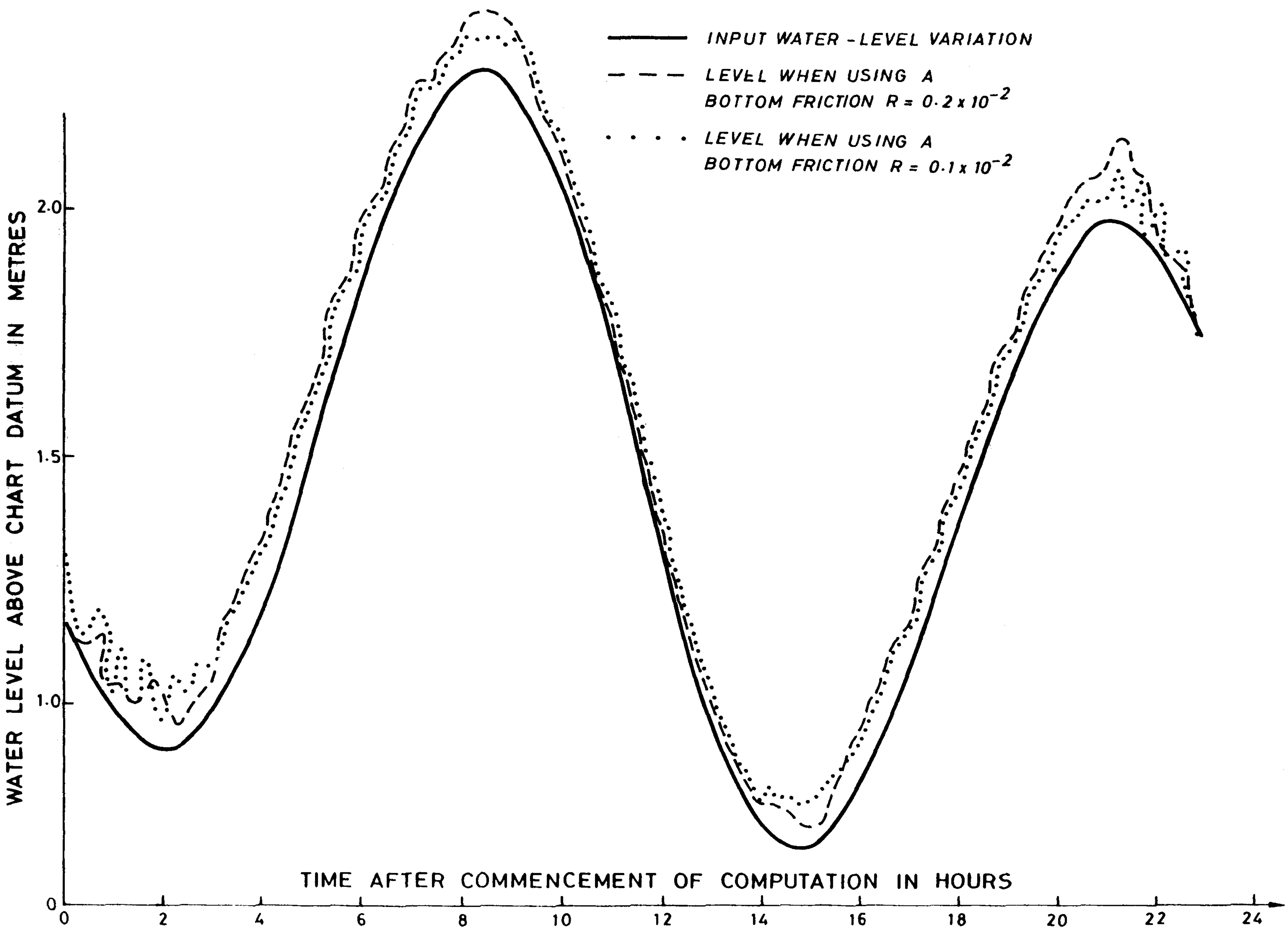


Figure 9 Model output water-levels when varying the bottom friction

Figure 9   Model output water-levels when varying the bottom friction

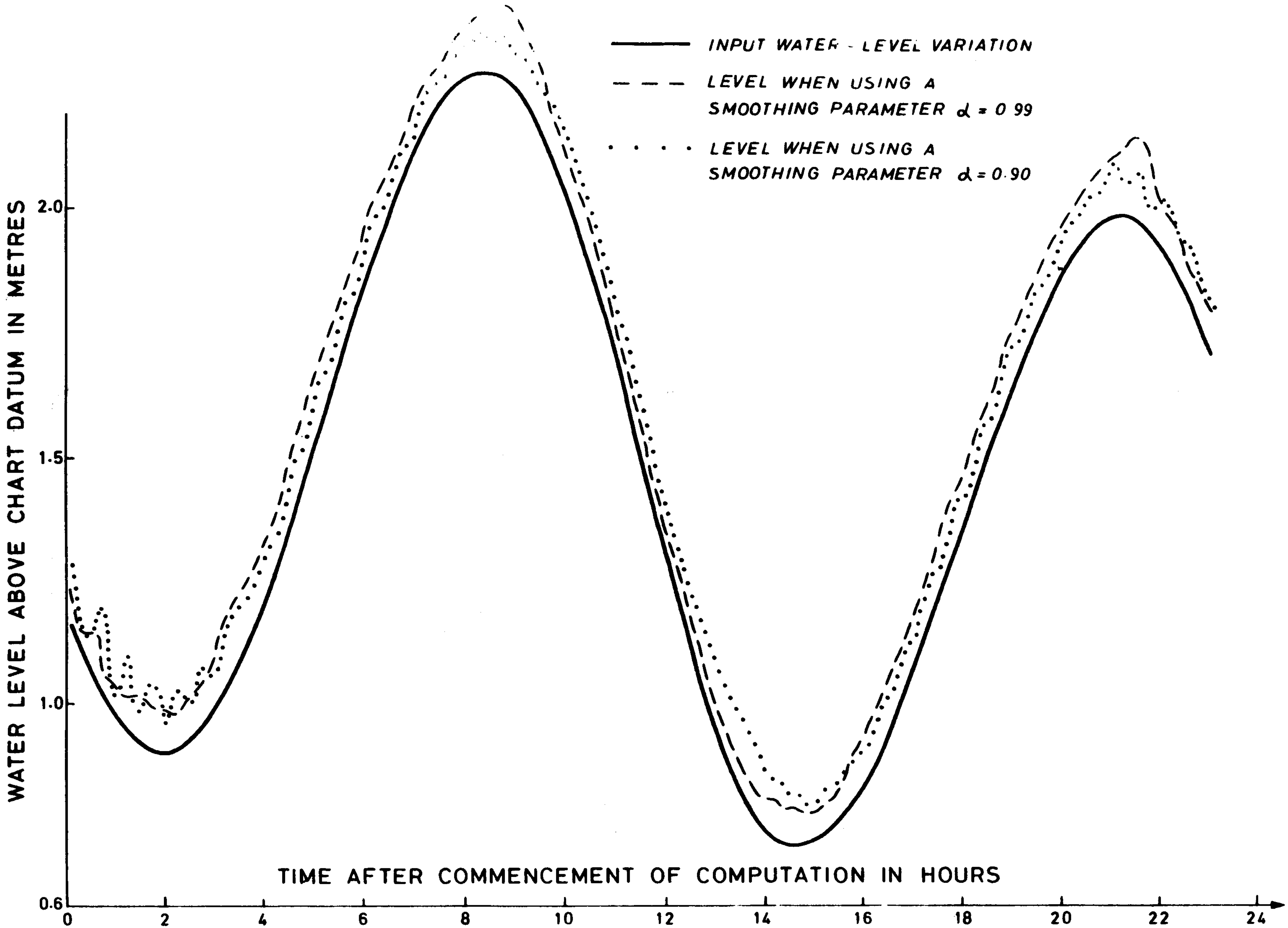


Figure 10   Model output water-levels when varying the smoothing parameter

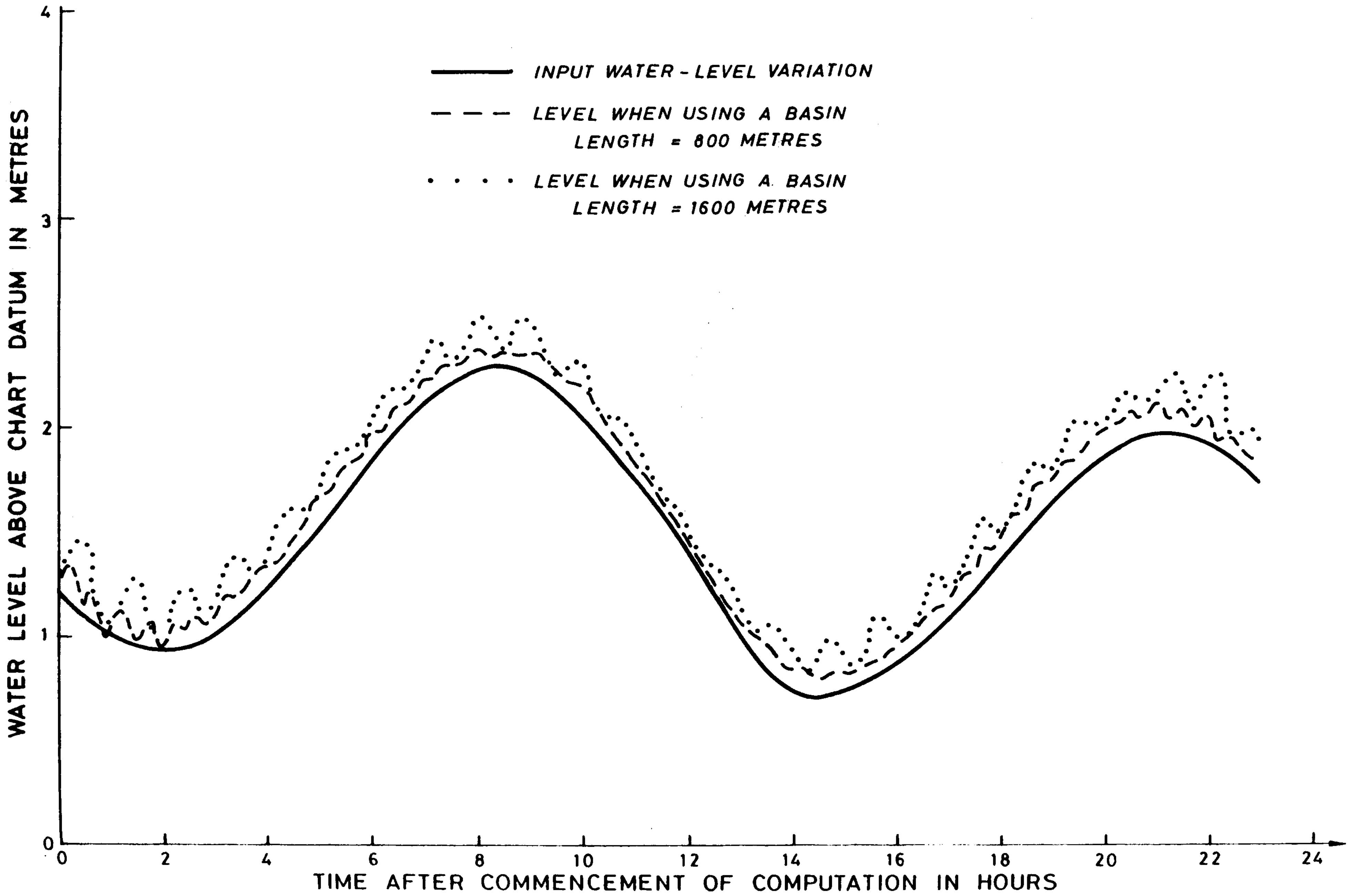


Figure 11 Model output water-levels when varying the basin length from seaward boundary to shore with 25 m/s on-shore wind

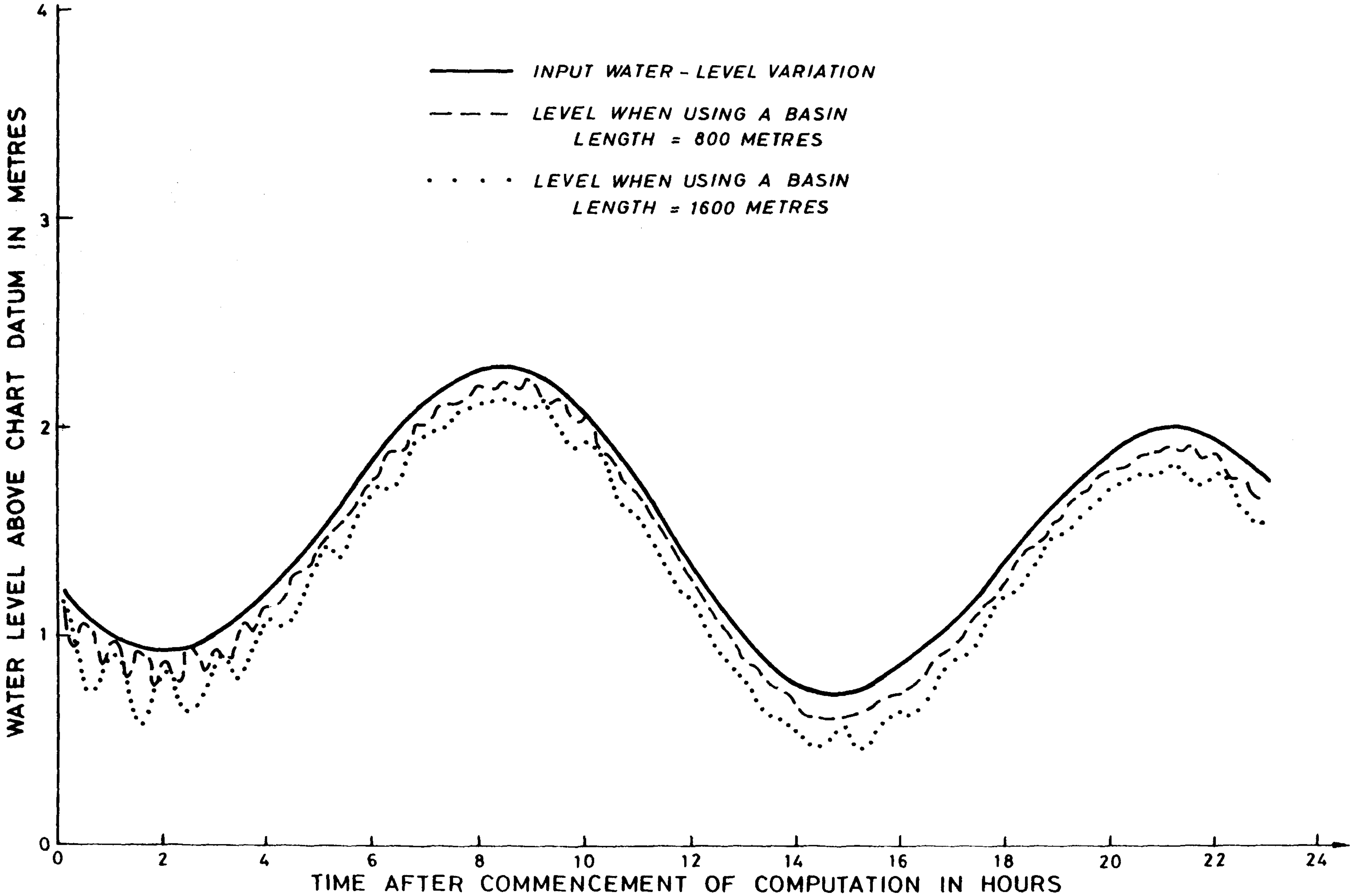


Figure 12 Model output water-levels when varying the basin length from seaward boundary to shore with 25 m/s off-shore wind

13. LIMITATION OF THE BAY MODEL

The Bay Model is a numerical model and thus is subjected to all the limitations that are inherent with simulation modelling. In particular, the finite difference scheme employed can cause computational dispersion of phase speeds. The over-idealisation of the coastline and bathymetry may at times be significant. The boundaries are not variable and no allowance is made for overtopping. Errors in wind stress or bottom stress parameterisation would yield poor results. Smoothing usually leads to amplitude damping. Simplification due to vertical integration and incomplete boundary specification reduce the accuracy of the simulation etc. It is therefore important that the users be well aware of these limitations before applying the Bay Model in their respective investigations.

14. APPLICATION OF THE BAY MODEL ON VARIOUS STORM SURGE INVESTIGATIONS

Despite the many imperfections, the Bay Model, being an analytical method is still considered superior to the statistical or regressional approaches. It is far cheaper and easier to realise than a physical model. In Hong Kong, the Bay Model has been applied to look into sea flooding risks in connection with a number of major engineering projects. These include :

The Tolo Harbour Storm Surge Investigation.
The Hebe Haven Marina Storm Surge Study.
The Storm Surge Risks for the Lantau Fixed Crossings, the North Lantau Development and a new Power Station at Tap Shek Kok.
The Storm Surge Study for a New Power Station in Lamma Island.
The Yuen Long Industrial Estate Storm Surge Study.

Each of the above projects will be briefly described. The emphasis will be on the basin used, the parameterisation and the special features in the Bay Model utilisation.

15. THE TOLO HARBOUR STORM SURGE INVESTIGATION

In 1977, a detailed storm surge investigation was carried out to establish (1) whether changes in the Tolo Harbour coastline resulting from the construction of the Plover Cove Reservoir and the planned large scale reclamations at Sha Tin & Tai Po could seriously alter the distribution and amplitude of the peak storm surge levels near Sha Tin & Tai Po, (2) to examine whether the adopted formation levels for Sha Tin & Tai Po differ significantly from mathematical modelling findings. For the investigation, a tide gauge was sited at Kau Lau Wan (see Figure 13). 9 different coastlines were used in order to detect the difference caused by different planned reclamation configurations, one of these is shown in Figure 14. The Bay Model was calibrated by using observed water level variations at Kau Lau Wan and Tai Po Kau during the passages of Typhoon Elsie, October 14-15, 1975 and Typhoon Elaine, October 29-30, 1974. Figures 15 & 16 show the comparison between the observed and the computed water levels at Tai Po Kau.

In the investigation, outputs of Splash Open Coast surge heights for 93 historical storms with Kau Lau Wan tides added are used to couple non-interactively to the Bay Model to generate return period of various flooding levels for Sha Tin & Tai Po. A full description of the investigation is contained in (6). The computational aspects of the Bay Model runs are given below :

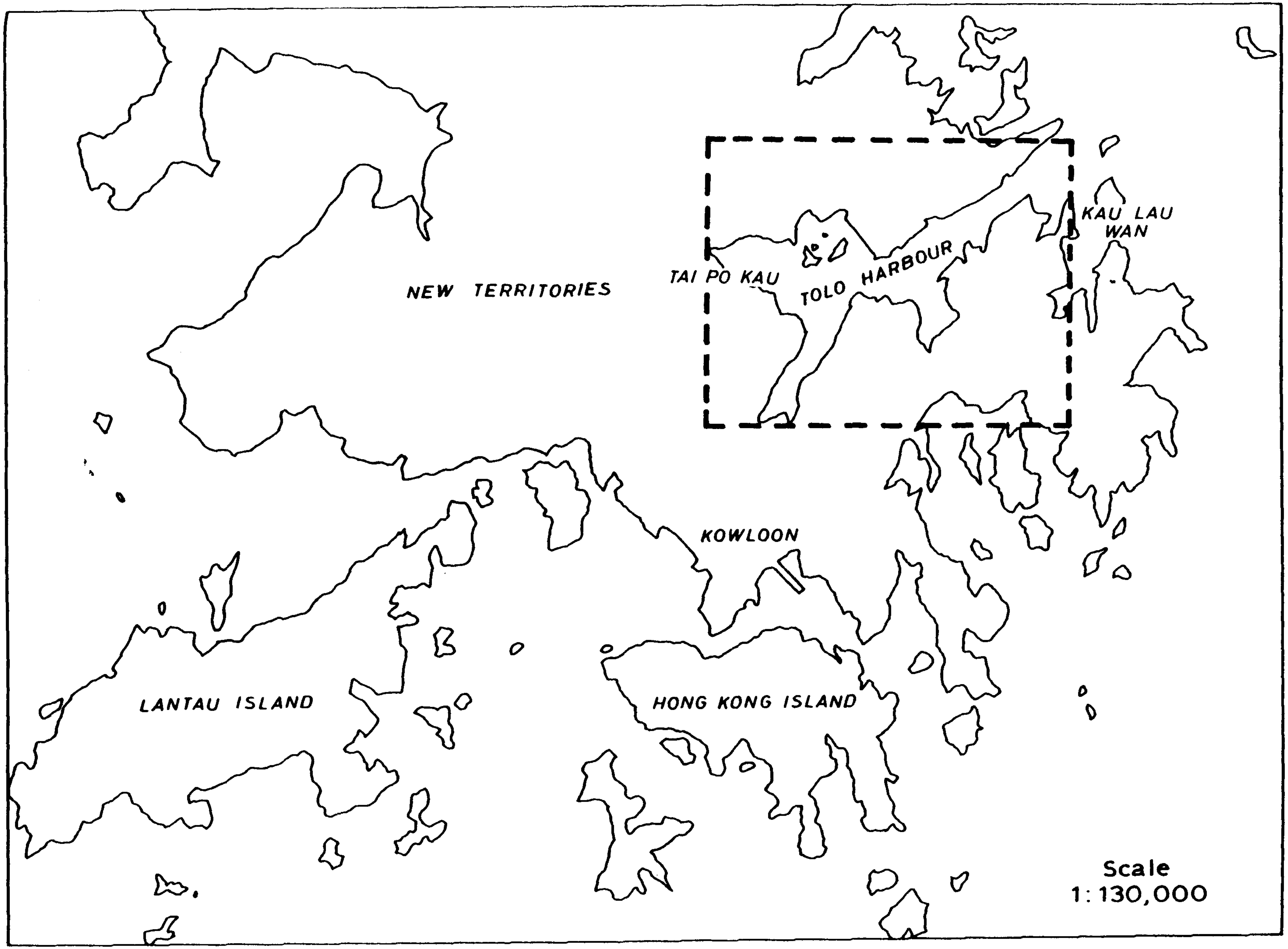


Figure 13 Map showing the area investigated in the Tolo Harbour storm surge modelling study

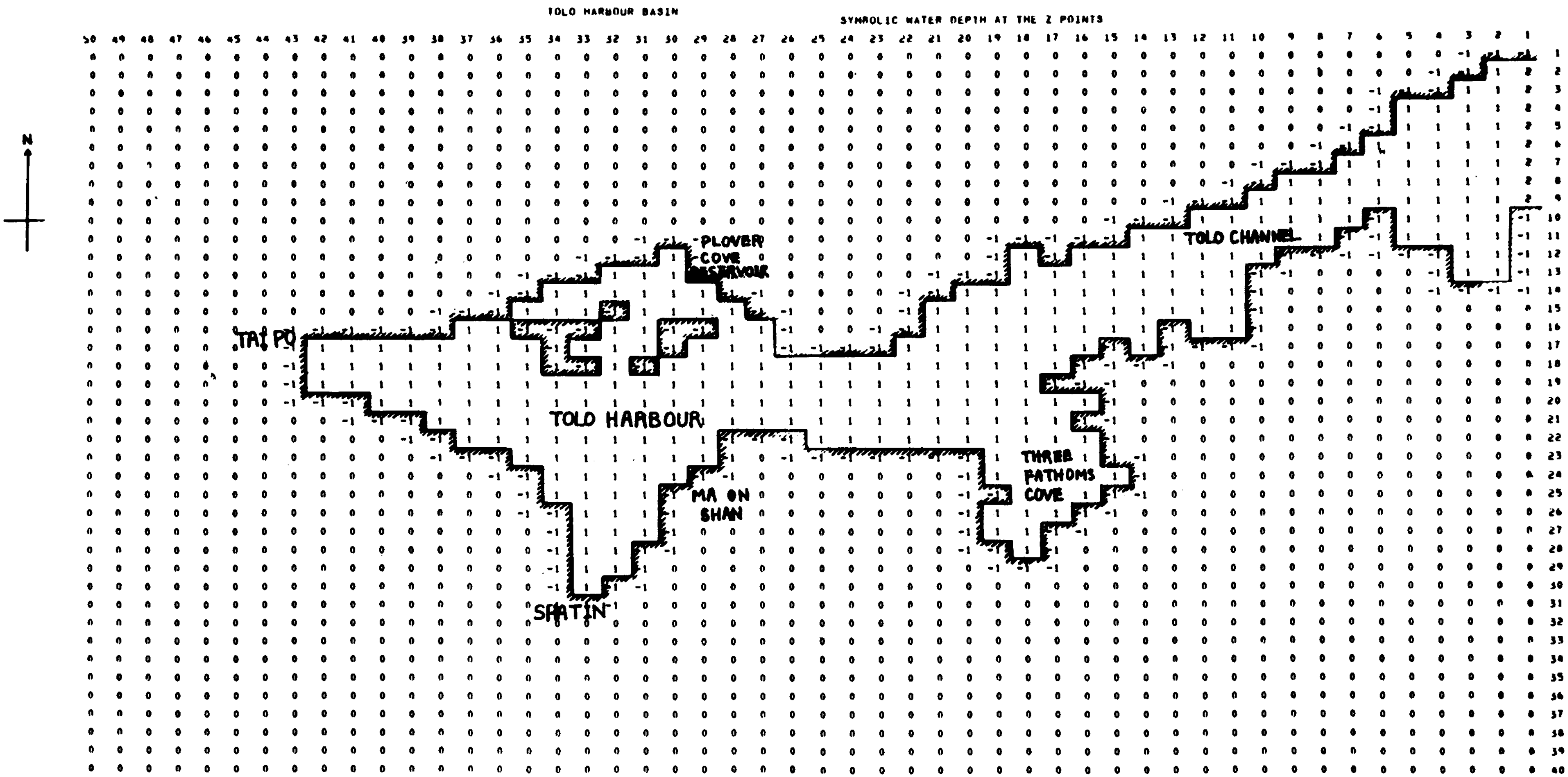


Figure 14 Fine grid of Tolo Harbour, sample spatial printout
(grid length = ¼ mile)

Number of basins coded	:	9 (representing 9 stages of reclamation)
Number of grid points	:	40 x 50
Grid length	:	402.3 metres
Time step	:	18 seconds
Drag coefficient	:	0.0024
Smoothing parameter	:	0.998
Bottom friction	:	0.1×10^{-2}
Boundaries	:	One input boundary across the entrance to Tolo Harbour
Input water levels	:	Surge heights from Splash coupled to Kau Lau Wan tides
Input winds	:	Vector mean of Waglan and Observatory winds

16. THE HEBE HAVEN MARINA STORM SURGE STUDY

Figure 17 shows the location of Hebe Haven. It is an enclosed bay within the Port Shelter area. The feature of this application of the Bay Model is that coupling of models has been performed twice. The outputs of Splash surge heights with modified Kau Lau Wan tides were used as inputs to a coarse grid Bay Model basin. The result of the coarse grid Bay Model run for the location of Sha Tsui was used as input to a fine grid Bay Model basin to obtain water levels for locations inside Hebe Haven. There was no tide gauge in the Port Shelter area to tune the Bay Model. The already calibrated Bay Model was used with no parameters changed. The Bay Model runs were found to compute current values closely resembling those given in a tidal stream map for the Port Shelter area. The salient computational aspects of the Bay Model runs are given below :

(i) Coarse Grid Basin

Number of grid points	:	21 x 20
Grid length	:	600 metres
Time step	:	24 seconds
Drag coefficient	:	0.0024
Smoothing parameter	:	0.998
Bottom Friction	:	0.1×10^{-2}
Boundaries	:	One input boundary across Port Shelter near Bluff Island
Input water levels	:	Splash surge heights adding to modified Kau Lau Wan tides
Input winds	:	modified Waglan winds

(ii) Fine Grid Basin

Number of grid points	:	26 x 17
Grid length	:	100 metres
Time step	:	6 seconds
Input water levels	:	Output water levels for Sha Tsui from coarse grid run
Boundaries	:	One input boundary across Hebe Haven entrance near Sha Tsui
Other parameters	:	same as for coarse grid run

Comparison between observed and computed water- levels at Tai Po Kau during Typhoon Elsie from 0000H October 14, to 0000H October 15, 1975

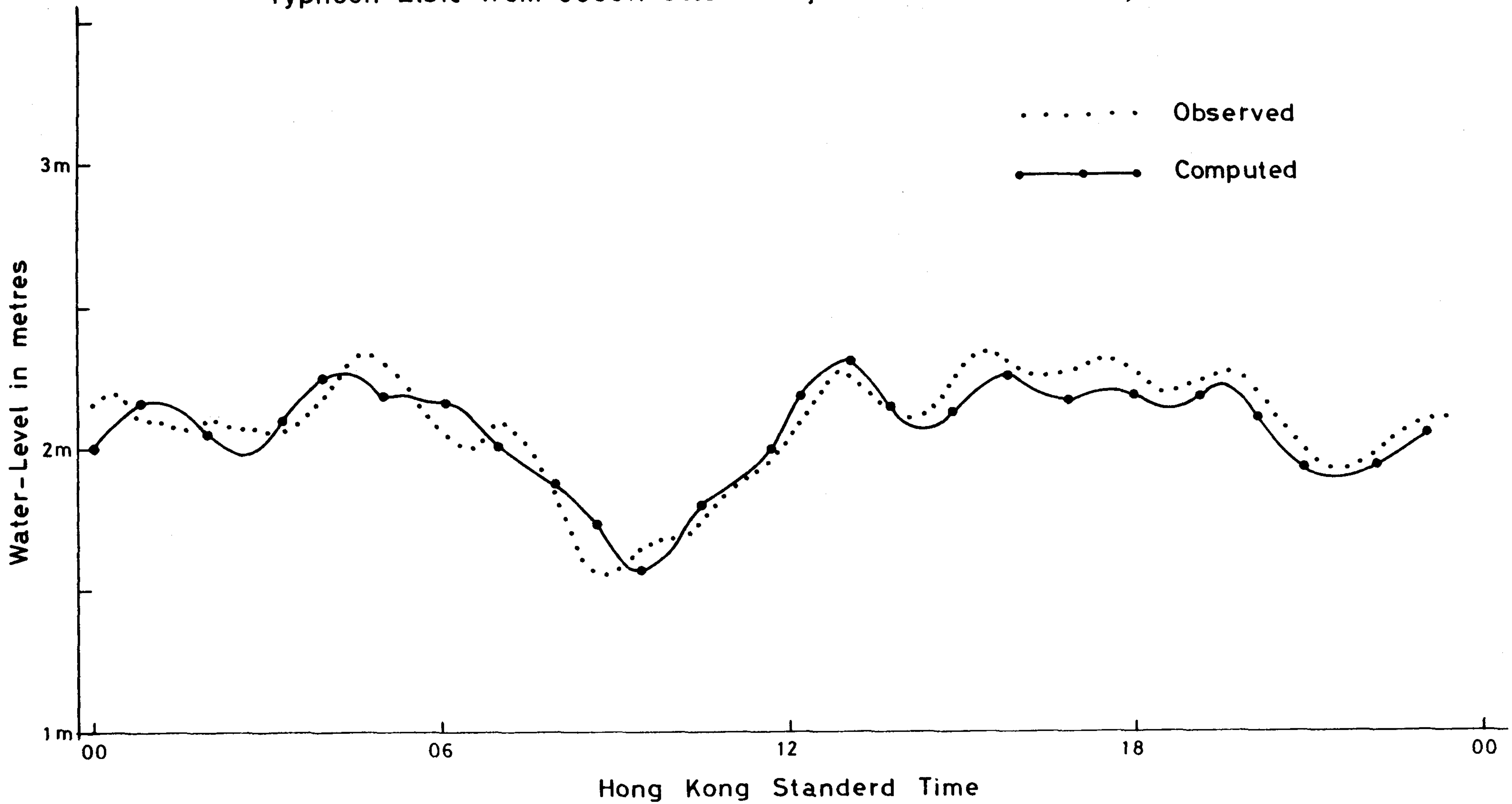


Figure 15 Comparison between observed and computed water-levels at Tai Po Kau during Typhoon Elsie from 140000 H to 150000 H October 1975

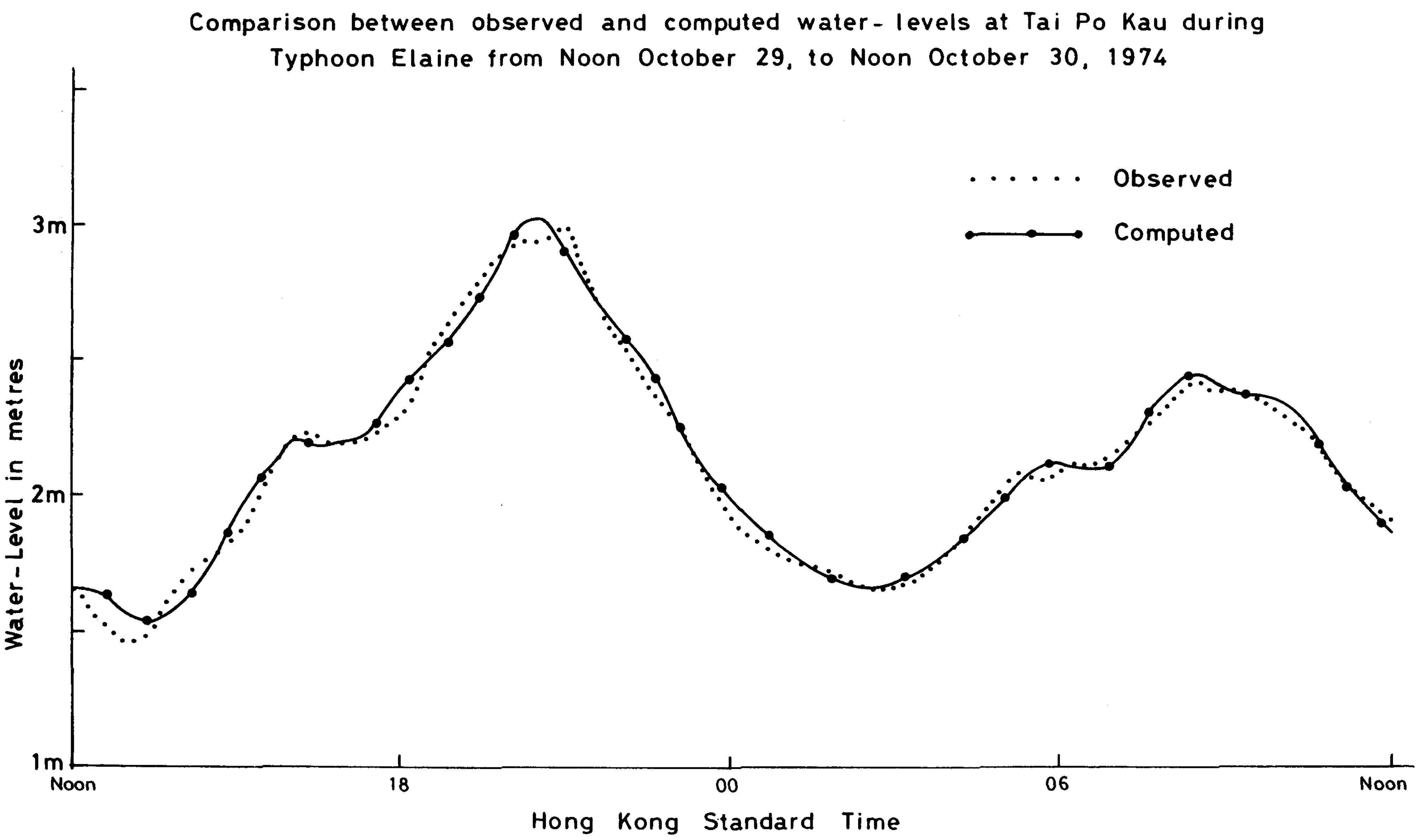


Figure 16 Comparison between observed and computed water-levels at Tai Po Kau during Typhoon Elaine from 291200 H to 301200 H October 1974

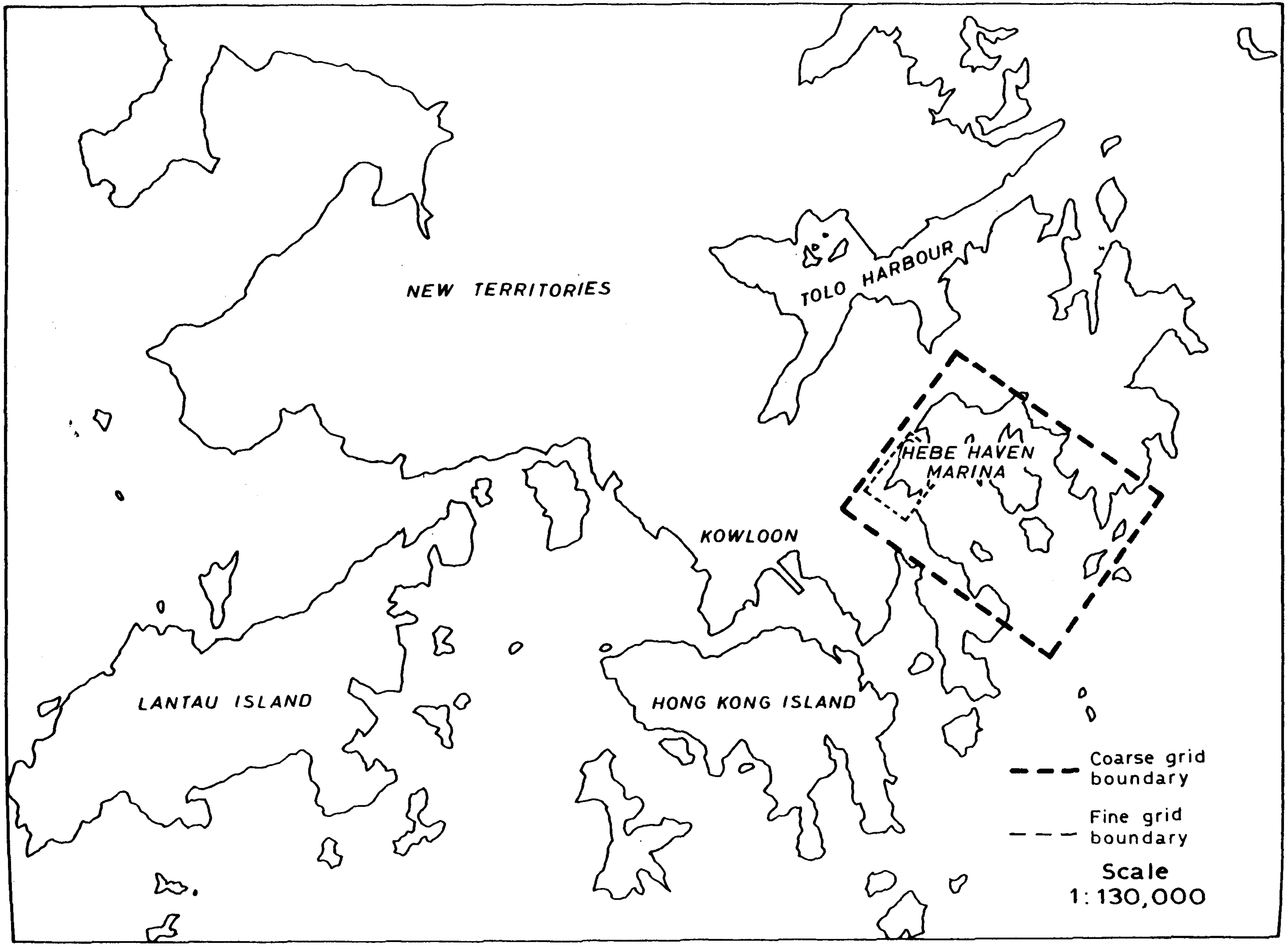


Figure 17 Map showing the area investigated in the **Hebe Haven storm surge modelling study**

HEBE HAVEN COARSE GRID (GRID LENGTH = 600 METRES)

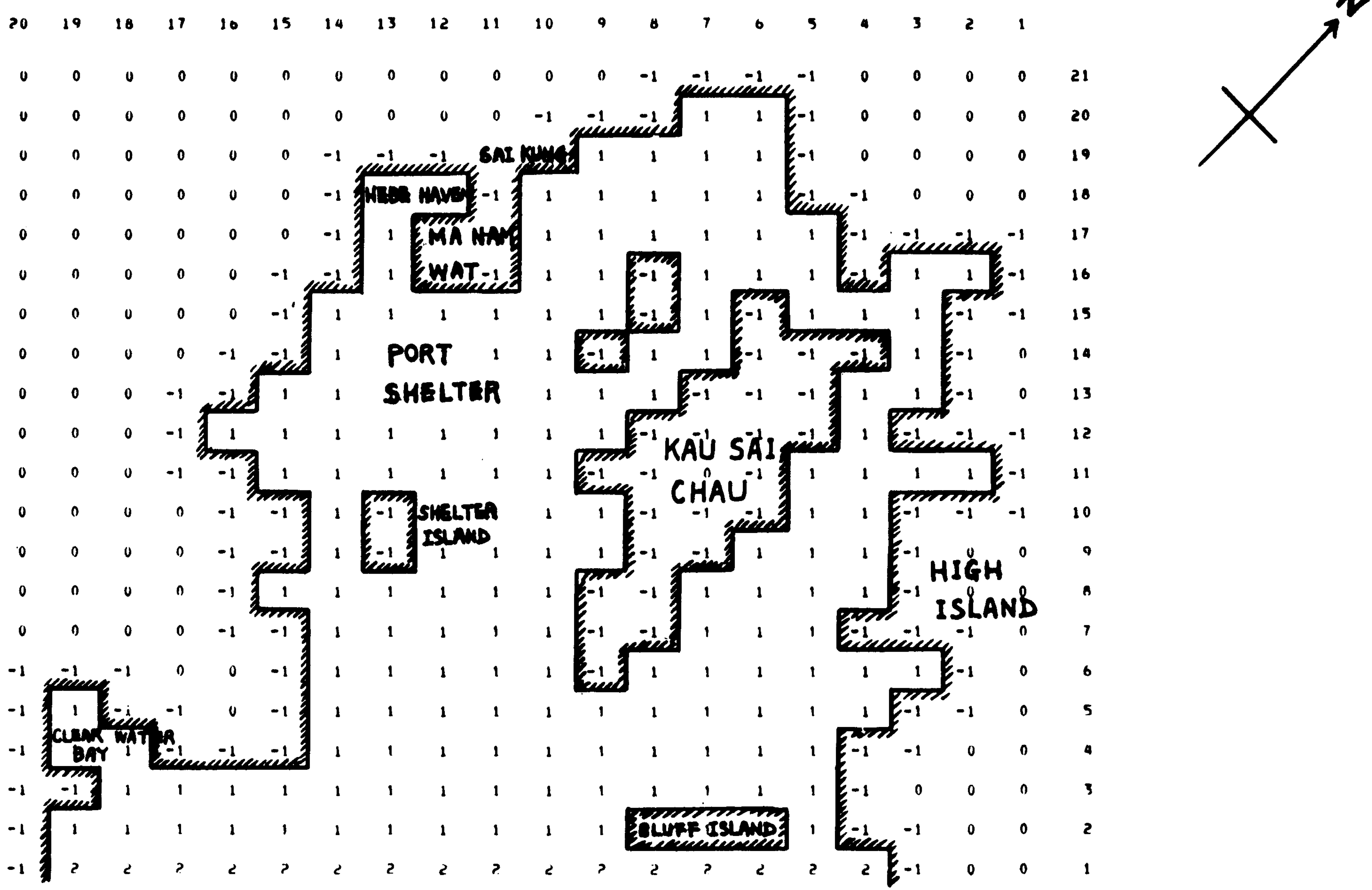


Figure 18 Layout of the Hebe Haven coarse grid basin

HEBE HAVEN FINE GRID BASIN (GRID LENGTH = 100 METRES)

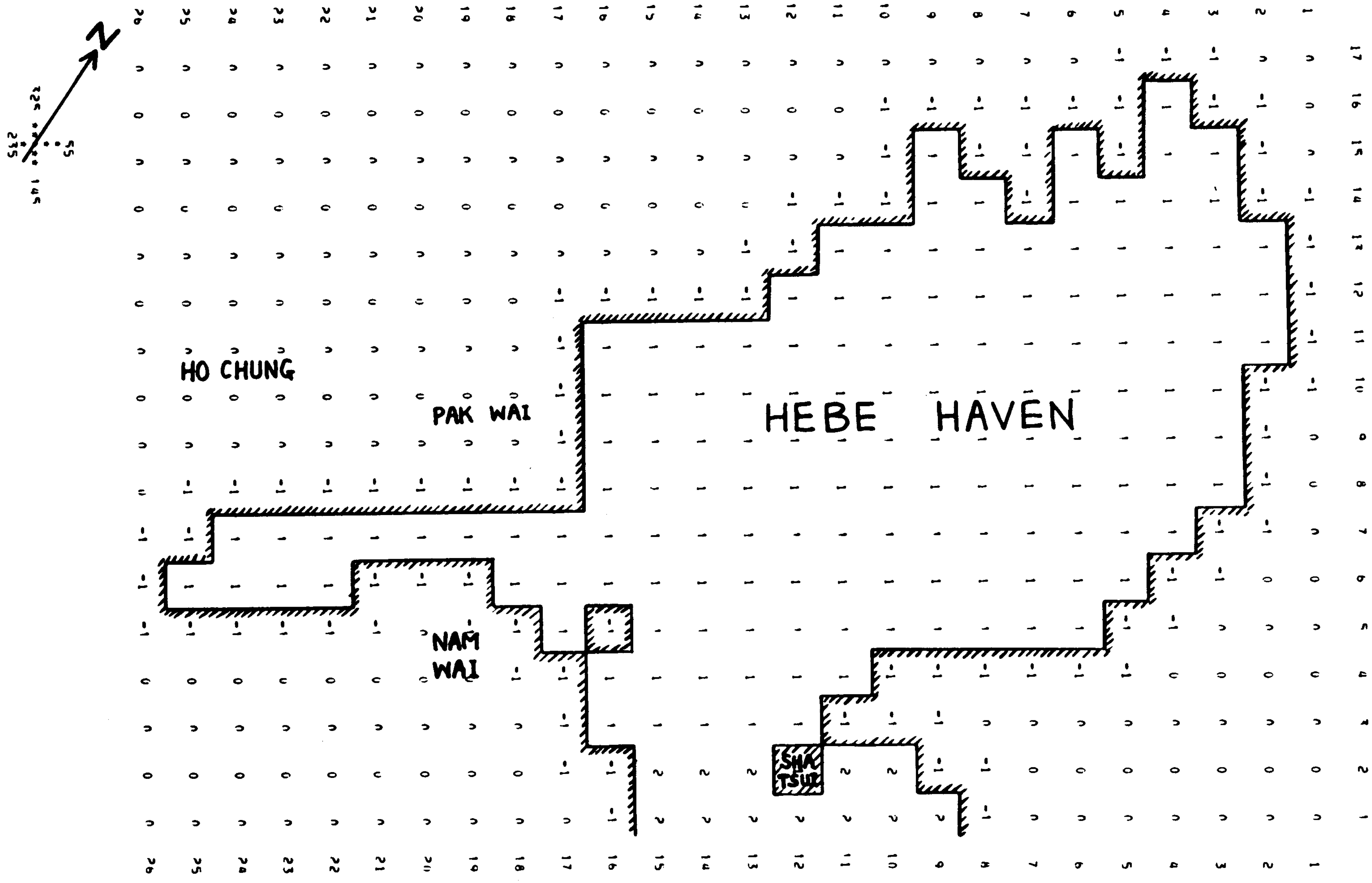


Figure 19   Layout of the Hebe Haven fine grid basin

17. THE NORTH LANTAU STORM SURGE INVESTIGATION

The area under consideration covers the Victoria Harbour, the northern part of the West Lamma Channel, Kap Shui Mun, the sea area near the Brothers Island and Urmston Road off Castle Peak (Figure 20). The basin was designed to provide answers for locations near the Ma Wan Channel for the Lantau Fixed Crossings Project; for locations near the shore of North Lantau for the North Lantau Development Project and for locations near Tap Shek Kok for the new China Light and Power Company Power Station. The main feature of this Bay Model application lies in the increased number of boundaries used. There was no need to use Splash surge heights as observed North Point water-levels can be used as tidal forcings which propagate westwards. The Bay Model was calibrated in this instance using the Chi Ma Wan recorded water-levels and current values over different sea areas during various tidal cycles. It was found necessary to slightly displace the Ma Wan Island to the northeast because of computer core size limitations. Key aspects of the computation were as follows :

Number of basin coded	:	One
Number of grid points	:	21 x 30
Grid length	:	1271 metres
Time step	:	45 seconds
Drag coefficient	:	0.0018 for wind speed below 15 m/sec 0.0027 for wind speed above 20 m/sec linear interpolation between 15 & 20 m/sec
Smoothing parameter	:	0.998
Bottom friction	:	0.1×10^{-2}
Boundaries	:	4 boundaries north-south across Victoria Harbour, east-west across Lamma Channel, north from southwest Lantau and east-west across entrance to Deep Bay
Input water levels	:	North point water-levels
Input winds	:	Royal Observatory winds

18. THE LAMMA ISLAND STORM SURGE INVESTIGATION

The investigation was conducted for the Hong Kong Electric Company in connection with a new power station to be sited on the west side of the Lamma Island. Actual float trackings were carried out on a number of occasions off west Lamma from which current data were derived. The Bay Model was closely calibrated to these set of current data.

For this investigation :

Number of basin coded	:	One
Number of grid points	:	24 x 24
Grid length	:	925 metres
Time step	:	30 seconds
Drag coefficient	:	0.18×10^{-2}
Smoothing parameter	:	0.99
Bottom Friction	:	0.3×10^{2}
Boundaries	:	4 boundaries
Input water levels	:	North Point water-levels
Input winds	:	Cheung Chau winds

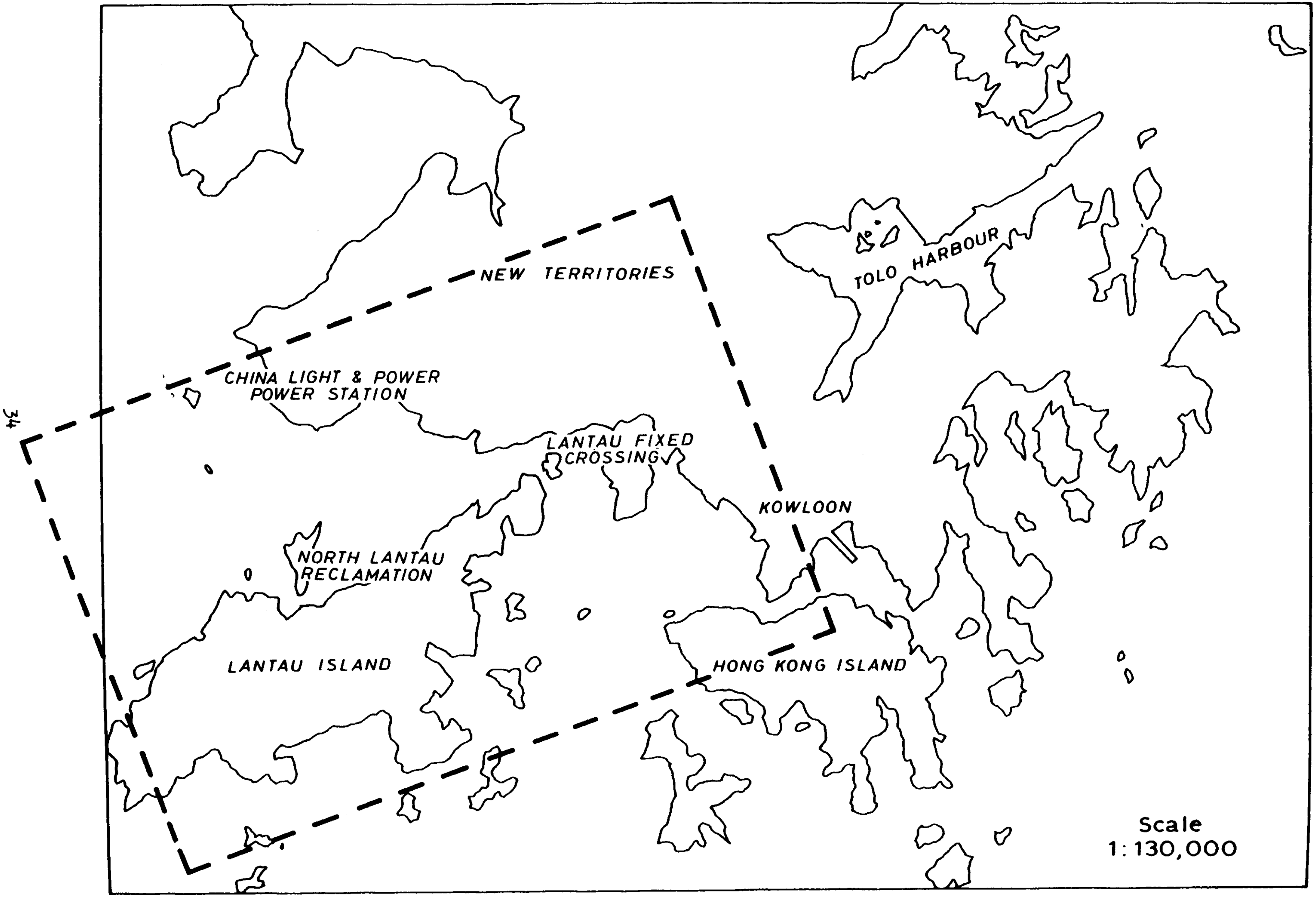


Figure 20 Map showing the area investigated in the north Lantau development storm surge modelling study

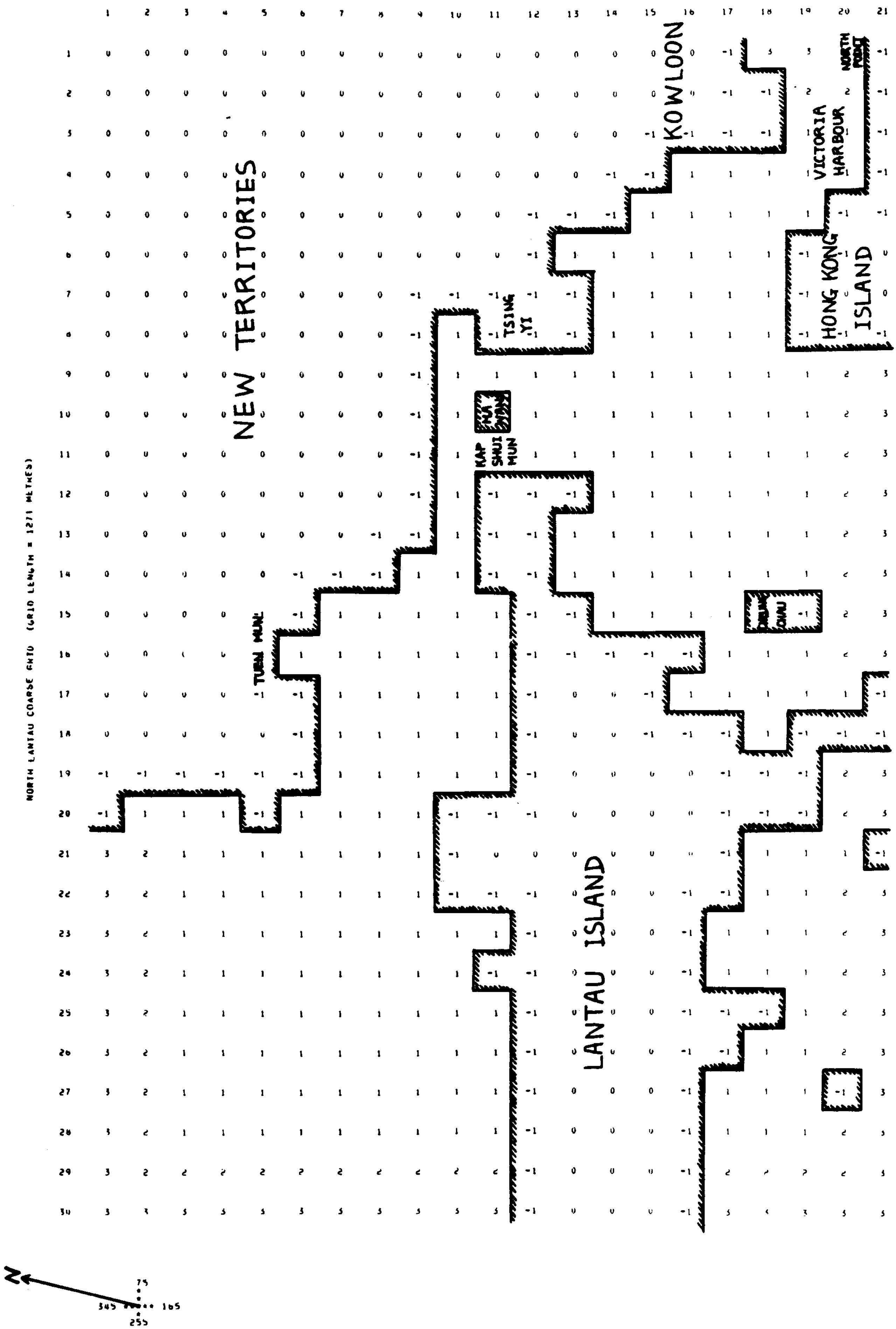


Figure 21 Layout of the north Lantau basin

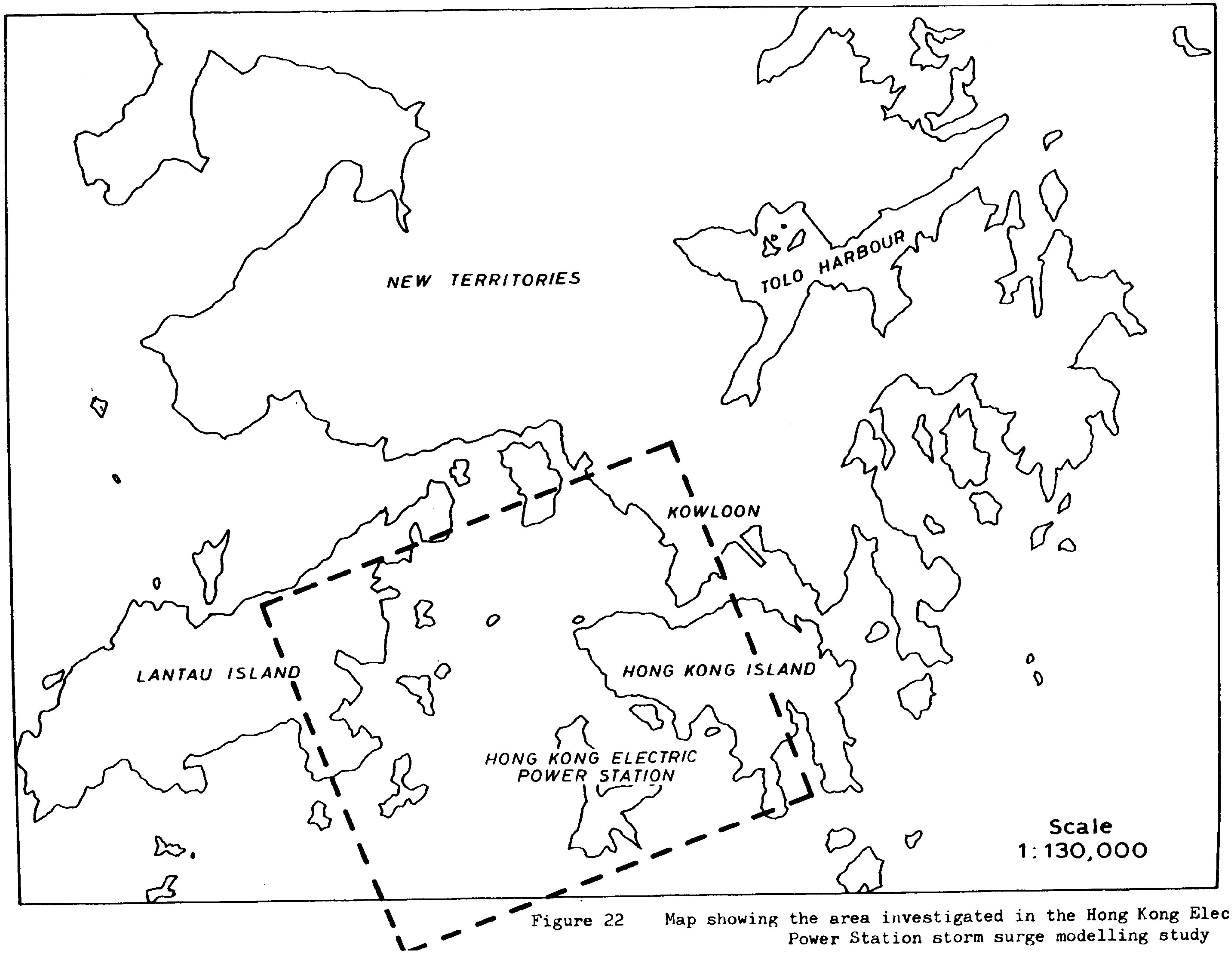


Figure 22 Map showing the area investigated in the Hong Kong Electric Power Station storm surge modelling study

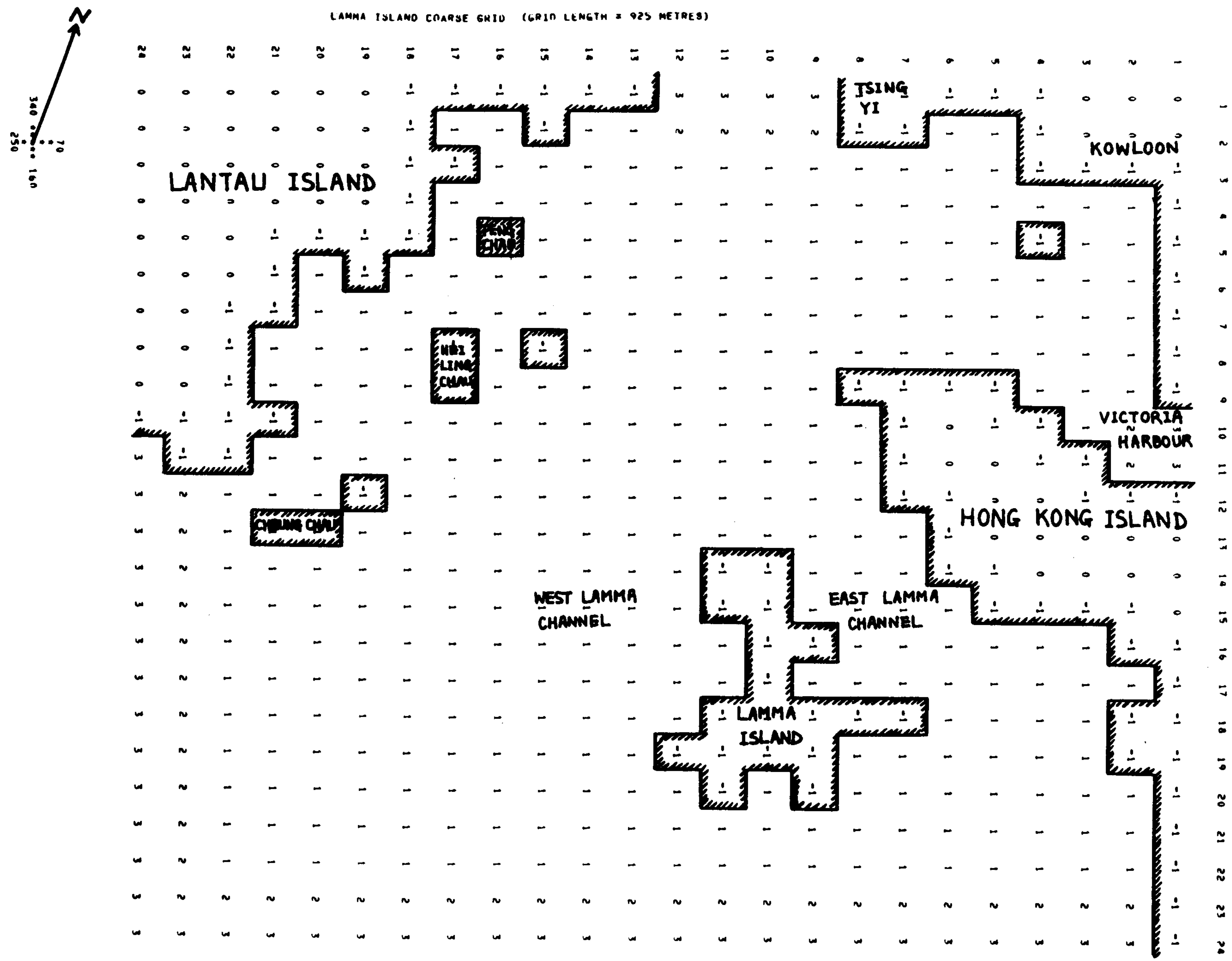


Figure 23   Layout of the Lamma basin

19. THE YUEN LONG INDUSTRIAL ESTATE STORM SURGE INVESTIGATION

Figure 24 shows the location of the basin which covers Deep Bay and part of the Pearl River Estuary. Deep Bay is noted for its shallow waters and the very late tides that are often 2½ to 3 hours behind that of North Point's. The Model was calibrated using Tsim Bei Tsui tides. A relatively large bottom friction was used. The smoothing parameter was reduced to 0.98. For this investigation, the general impression is that closer agreement can be attained if the rate of Pearl River discharge is more accurately known. Details of the various parameters are as follows :

Number of basin coded	:	One
Number of grid points	:	15 x 23
Grid length	:	1271 metres
Time step	:	60 seconds
Drag coefficient	:	0.0018 for wind speed below 15 m/sec, 0.0027 for wind speed above 20 m/sec, linear interpolation for in between speed
Smoothing parameter	:	0.98
Bottom friction	:	0.3×10^{-2}
Boundaries	:	3 boundaries
Input water levels	:	Output from the North Lantau Bay Model run and Nei Ling Ting tides that have been adjusted to the right tidal epoch from North Point tides.
Input winds	:	Royal Observatory winds

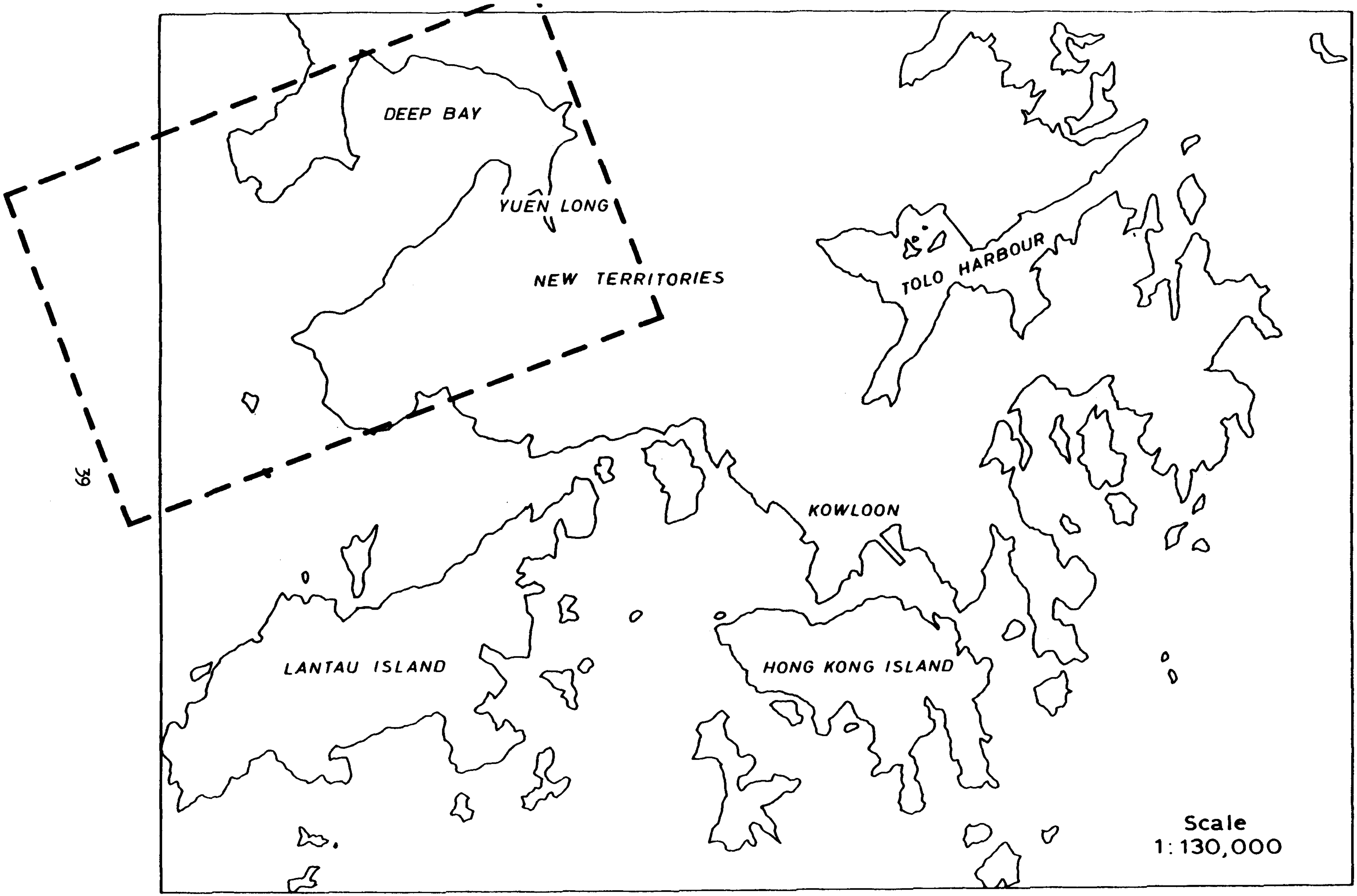
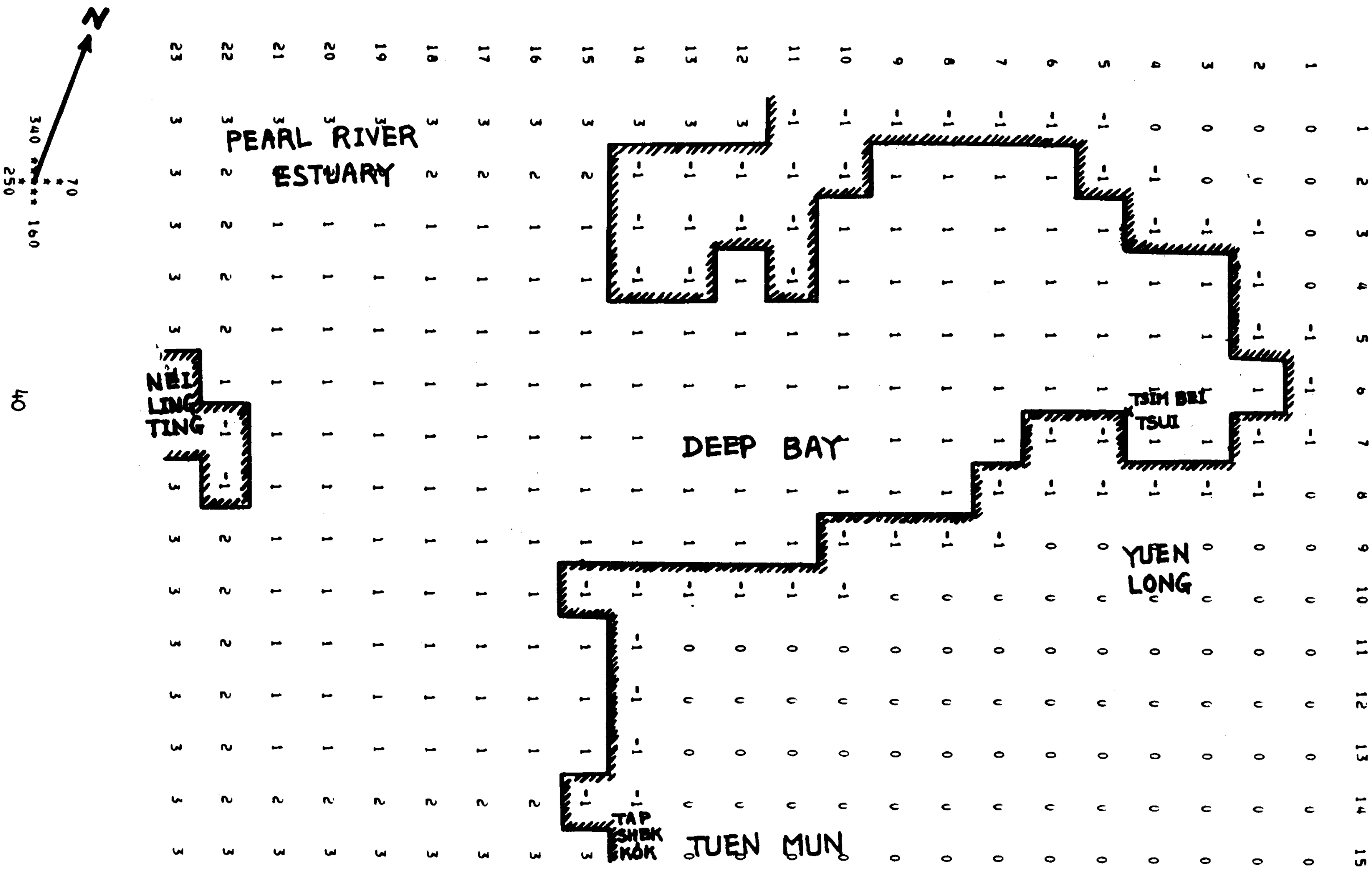


Figure 24 Map showing the area investigated in Yuen Long
Industrial Estate storm surge modelling study

DEEP BAY COARSE GRID (GRID LENGTH = 1271 METRES)



	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	0	0	0	0	-1	-1	-1	0	0	0	0	0	0	0	0
2	0	0	-1	-1	-1	1	-1	-1	0	0	0	0	0	0	0
3	0	0	-1	1	1	1	1	-1	0	0	0	0	0	0	0
4	0	-1	-1	1	1	[illegible]	1	-1	[illegible]	0	0	0	0	0	0
5	-1	-1	1	1	1	1	-1	-1	0	0	0	0	0	0	0
6	-1	1	1	1	1	1	-1	-1	0	0	0	0	0	0	0
7	-1	1	1	1	1	1	1	-1	-1	0	0	0	0	0	0
8	-1	1	1	1	1	1	1	1	-1	0	0	0	0	0	0
9	-1	1	1	1	1	1	1	1	-1	0	0	0	0	0	0
10	-1	-1	1	1	1	1	1	1	-1	-1	0	0	0	0	0
11	-1	-1	-1	-1	1	1	[illegible]	1	1	-1	0	0	0	0	[illegible]
12	3	-1	-1	1	1	1	[illegible]	1	1	-1	0	0	0	0	[illegible]
13	3	-1	-1	-1	1	1	[illegible]	1	1	-1	0	0	0	0	0
14	3	-1	-1	-1	1	1	1	1	1	-1	-1	-1	-1	-1	[illegible]
15	3	2	1	1	1	1	1	1	1	-1	1	1	1	-1	3
16	3	2	1	1	1	1	1	1	1	1	1	1	1	2	3
17	3	2	1	1	1	1	1	1	1	1	1	1	1	2	3
18	3	2	1	1	1	1	1	1	1	1	1	1	1	2	3
19	3	[illegible]	1	1	1	1	1	1	1	1	1	1	1	2	3
20	3	[illegible]	1	1	1	1	1	1	1	1	1	1	1	2	3
21	3	[illegible]	1	1	1	1	1	1	1	1	1	1	1	2	3
22	3	2	2	2	2	1	-1	-1	2	2	2	2	2	2	3
23	3	3	3	3	3			3	3	3	3	3	3	3	3

Figure 25 Layout of the Deep Bay basin

20. OTHER CAPABILITIES OF THE BAY MODEL

Besides storm surge investigations, the Bay Model can be used to analyse and predict tides, compute real time currents, compute the transport and diffusion of pollutants and perform drift computations. In the case of oil spills or other flotsam on the sea surface, successfully computing the size and future movements of the oil patches would be a valuable spin off from the Model. Likewise the computation of drifts would greatly facilitate any search and rescue missions.

As currents are available from the models in small time steps at each grid point, it is possible to evaluate the dispersion by means of the three additional equations : The general diffusion formula, the basic dispersion formula and the Fickian equation.

The general diffusion formula is :

$$\frac{\partial^2 S}{\partial x^2} + \frac{\partial^2 S}{\partial y^2} - \frac{1}{A}\frac{\partial S}{\partial t} = 0$$

The basic dispersion formula :

$$\frac{\partial S}{\partial t} = Y - \frac{S}{n} - \frac{\partial}{\partial x}(U_x + pS_x) - \frac{\partial}{\partial y}(U_y + pS_y)$$

The Fickian equation :

$$\frac{\partial S}{\partial t} = Y - \frac{S}{n} - K\nabla^2 S - \frac{\partial}{\partial x}(S_u U) - \frac{\partial}{\partial y}(S_v V)$$

where : Y - addition (release); n - decay; S - concentration;
$pS_{x,y}$ - concentration velocity component; t - time;
K (coefficient) = $\beta a v_r$ (a - depth; $v_r = u^2 + v^2$; $\beta = 0.003$);
S_u, S_v - concentration gradients in u and v direction ; and
A - diffusion coefficient (Austausch coefficient).

These equations have already been incorporated into the Observatory's Bay Model.

21. MULTILAYER MODELS

The main objection to the single layer vertically integrated model is that it is difficult to visualise how the wind stress at the sea surface can transcend downwards undiminished to the sea bottom as would be the case with the single layer treatment. Nor can it be accepted that the bottom stress will act on the entire column of the sea which has been vertically integrated. Also, it is not at all uncommon to find that the surface current and the current deep down flow freely in opposite directions. To better simulate these real situations, two or more layers in the numerical model are necessary. In general, a two layer Bay Model will require 6 equations, a three layer model, 9 equations etc. The amount of computations will be increased about tenfold by introducing one more layer. However, the attainment of greater accuracy with a multilayer model should far outweigh the expense of the additional computer time. Efforts to develop such models and to eradicate some of the limitations of the current model will continue.

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