韓國 迎日湾 松島 海水浴場의 海岸線變化 및 漂砂移動率에 關한 數値 시뮬레이션

李　重　雨*

A Numerical Simulation of the Shoreline Change and Sediment Transport with Shore Structures at Songdo Beach Youngil Bay, Korea

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TABLE OF CONTENTS

CHAPTER
I. INTRODUCTION
II. MATHEMATICAL FOUNDATION
2.1. Backgrounds
2.2. Basic Assumptions
2.3. Equations for Sediment Transportation
III. DESCRIPTION OF THE NUMERICAL MODEL
3.1. Explicit Model
3.2. Implicit Model
3.3. Constraint of the Shoreline Movement in the Model
IV. BREAKING WAVE FIELD
V. MODEL SIMULATION
VI. CONCLUSIONS
REFERENCES

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Abstract

Two numerical models of the shoreline change and sedimdnt transport rates, explicit and implicit, are simulated with shore structures such as breakwaters, a jetty, groins and a seawall. The applied study area is Songdo Beach, Youngil Bay, Korea since it has all the shore structures mentioned above.

The two models investigate the beach line changes and sediment transport rates for the beach before design of three groins with and without an offshore breakwater. In order to estimate the shoreline changes after three groins were built, the beach response inside the three groin compartments and the offshore barrier are also investigated.

The simulations based on the initial shoreline conditions surveyed by the Hydrographic office, Korea in 1979 and 1984. The breaking wave characteristics are introduced into the models by calculation from the empirical equations and modification from the numerical and hydraulic model test results developed for waves behind an offshore breakwater.

The numerical simulation describes well the tendencies of the sand transport and shoreline changes affected by wave diffraction behind a detached breakwater and by interruption of sand transport at three groins.

(要　　約)

防波堤, 突堤, 防砂堤, 防潮堤 等 海岸의 構造物設置에따른 海岸線變化및 標砂移動率의 分析을 위해 두개의 數値모델 (Explicit 및 Impicit)을 開發하였다. 本硏究의 適用地域은 上記의 海岸構造物을 現實的으로 포함하고 있는 韓國의 東海岸에 位置한 迎日湾의 松島 海水浴場을 選擇하였다.

먼저, 두모델로 海岸에 設置되어있는 3個의 防砂堤를 無視하여 海岸에서 떨어진 海域에서 離岸堤의 設置有無에 따른 海岸線變化와 標砂移動率을 시뮬레이션한 後, 3個의 防砂堤를 考慮하여 같은 시뮬레이션을 實行하였다.

本硏究에 使用된 初期海岸線에 관한 資料는 1979年및 1984年의 2回에 걸친 韓國 水路국의 調査資料이다.

碎波特性에 관한것는 여러硏究를 通해 公開된 離岸堤後方水域에 대한 水理모델과 數値모델 資料및 그修正資料를 使用하였다.

開發된 數値모델은 離岸堤에 의한 波의回折과 3防砂堤의 標砂移動의 制限에 따른 海岸線의 變化와 標砂移動率의 傾向을 잘說明하고 있다.

CHAPTER I. INTRODUCTION

It is an important problem in coastal engineering to protect shoreline changes around some coastal strucrtures and to consider shore protection methods when some coastal structures are constructed.

At Youngil Bay in Korea, according to the comprehensive national econmic and physical development plan, industrial companies including Pohang Steel Company (POSCO), traffic facilities, port and harbors are expanding.

The port of pohang (see Figures 1.1 and 1.2) is presently functioning mainly as a commercial port (Pohang New Harbor) in relation with POSCO and partly as a fishery port (Pohang Old Harbor). In addition to these, it also includes Songdo Beach which gives a rest place for the civilians and a resort area for the tourists.

The dredging work in connection with the construction of Pohang New Harbor to assist the industrial complex like POSCO since 1968, the bottom profile near the beach area between two harbors have been changed signiflicantly year by year and still changing even though the construction was finished. This was indicated in a series of technical reports, Pub. No. 1011 by the Hydrographic Office, Republic of Korea (Korean) and the Journal of the Oceanological Society of Korea.

Moreover, the beach width, due to several storms, was reduced to less than 20m in particular area and eroded to the structure on the berm in southern part of the beach around 1979 and an accretion near the breakwater of the New Harbor was reported. These are well shown in Figure 1.3 based on the data by the Hydrographic Office, Republic of Korea both in 1979 and 1984. By looking at 2m and 5m depth contour, the left side of the beach is becoming shallower and the right side, deeper. Thus, it is easily known that the sand are moving right to left direction since 1979.

At the present time 3 groins are in this erosion area and the beach was partly recovered by artificial works as Figure 1.4.

The numerical model is a valuable tool for assessing beach changes caused by structures on the coast. It strengthens the coastal engineer's ability to predict the effects of a coastal engineering design. In particular, numerical models allow representation of the very important impacts of time-varying wave conditions.

Herein, the shoreline changes at Songdo Beach, before and after construction of groins, will be studied by using a numerical shoreline change model with some basic assumptions.

After confirming the shoreline changes under some prototypes of wave characteristics and time scale, the shoreline with construction of a detached breakwater for different locations will be simulated.

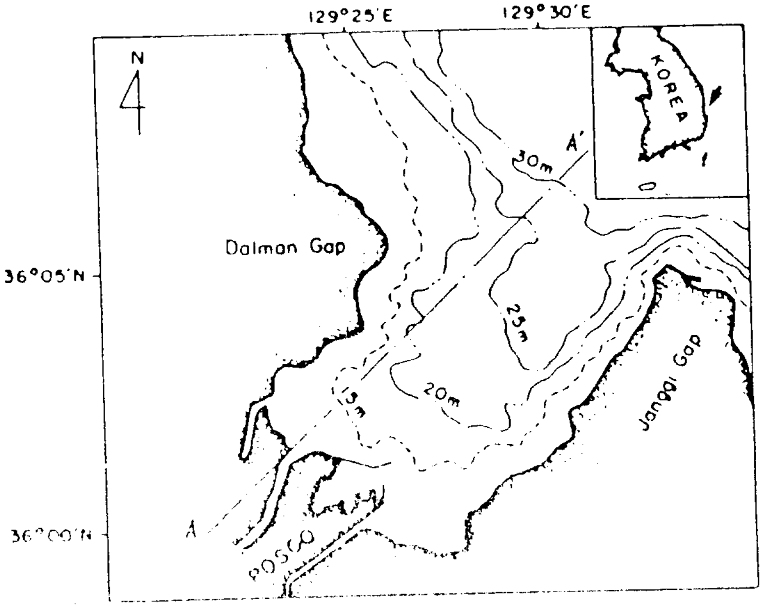

Figure 1.1 Location Map of the Study Area

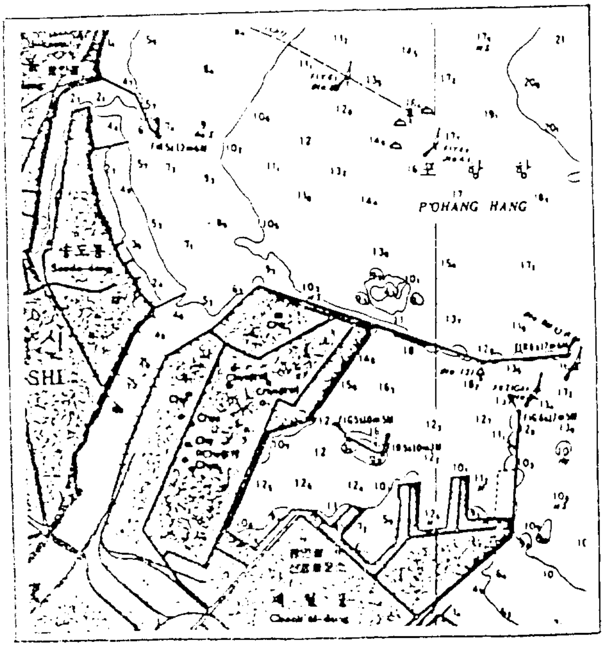

Figure 1.2 Pohang New and Old Harbor and Songdo Beach

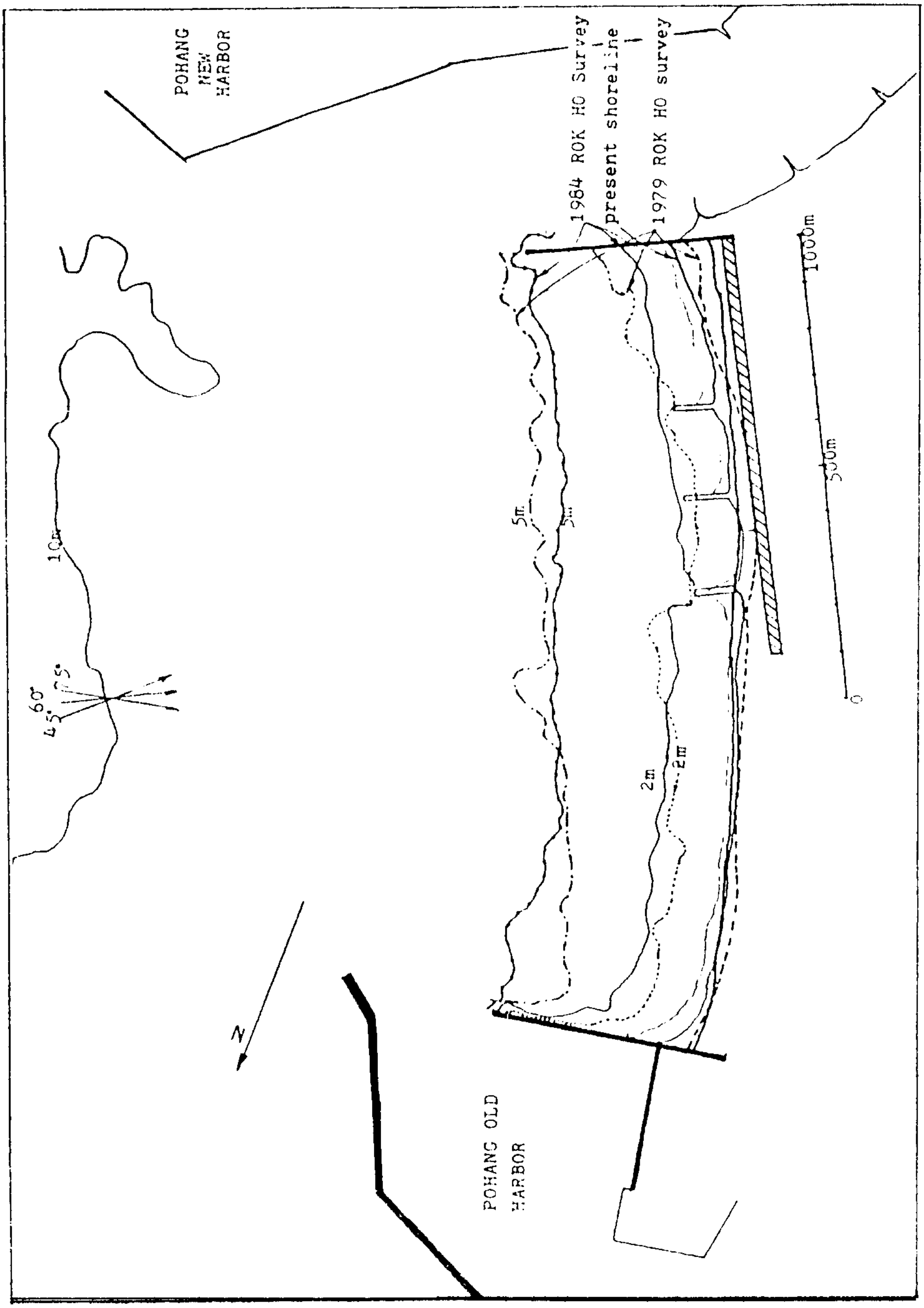

Figure 1.3 Shoreline Changes at Songdo Beach

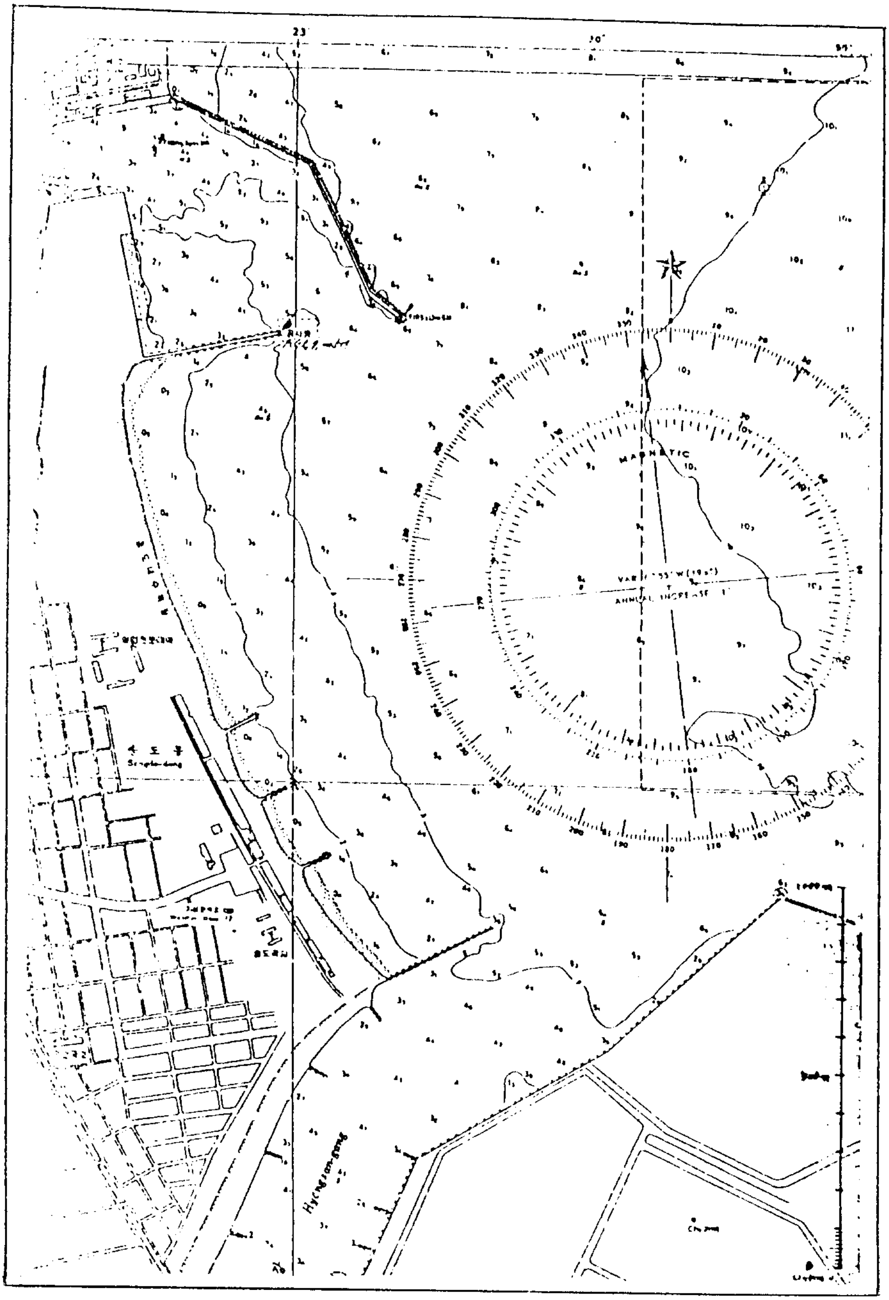

Figure 1.4 Detail of Songdo Beach(H.O. Survey, R.O.K., 1984)

CHAPTER II. MATHEMATICAL FOUNDATION

2.1. Backgrounds

Numerical models for shoreline changes has been widely applied in recent years since Pelnard-Considere (1954).

Despite the large number of applications of this type of model, representation of the impact of a structure as a boundary condition near the shoreline has received little attention.

Usually, shoreline models treat the breaking waves bring sediment transport on-offshore and along the shore and finally are dissipated on the beach. A structure such as seawall, revetment, groin, coastal dike, breakwater, storm surge barrier, bulkhead and house, and rocky coastal cliff, prevents the sediment behind it from entering the littoral system but modifies the sediment transport rate along the beach.

Additionally, standing waves reflected from the walls can cause local scour that may temporarily increase transport along shore or offshore, until a new, steeper equilibrium profile is achieved.

Introduction of a structure to the shoreline model, hydraulically or numerically, has been conducted by engineers associated with coastal engineering in many countries because of the man-made environmental condition.

For example, in 1983 more than 25% of Japanese coastal line (greater than 8500km) was protected by seawalls, coastal dikes, armored blocks, and similar structures.

Literatures about the behavior of the sediment transportation in front of the shore structure have been published continuously.

Hashimoto et al. (1971) discussed the longshore sediment transport in front of an armored seawall by blocks and Komar (1977, 1983) made a jetty blocking one direction littoral drift model. Perlin and Dean (1987) and Everts (1983) gave prediction of beach planforms with littoral controls between two groins.

Numerical models simulating beach changes due to the presence of an offshore breakwater have been developed by Hashimoto (1974) and Sasaki (1975). Again, Perlin (1979) presented beach planforms in the lee of detached offshore breakwater without any stucture at shore side.

Ozasa and Brampton (1980) treated the loss of berm in front of a seawall numerically. Tanaka and Nadaoka (1982) and Hanson and Kraus (1985, 1986) discussed a procedure of shoreline adjustment on numerical model.

Mizumura (1982) tested shoreline changes with hydraulic model and compared this with field measurements. Matsuoka and Ozasa (1983) utilized the wave deformation process on prediction model for shoreline change. There are more literatures about the shore erosion problems but all above are related to the horizontal changes.

In this study a shoreline change model by the finite difference method will be discussed based on some basic assumptions described in following section.

2.2. Basic Assumptions

In the following, we consider a long shoreline with parallel bottom contours. For simplification, linear shallow water wave theory is employed in immediately seaward from the surf zone. It is assumed that the beach is plane and constant slopping and is acted on by a monochromatic wave train.

A numerical model for shoreline changes also needs various assumptions at shore boundary which include three general principles :

a) the shoreline in front of a structure cannot recede landward of the structure,

b) the preservation of sand volume, and

c) the preservation of the direction of sediment transport alongshore in accordance with the natural direction of the local transport.

In the model wave reflection, sea bottom scouring, settling, flanking and collapse of the structure such as a seawall is only primarily functioning to protect the land behind it and it is not known with certainty whether the structure promotes the growth of the beaches in front of them, even the hydraulic model test indicated that a rough-faced slopping and permeable structure would promote recovery by wave energy dissipation.

The most important assumption made in this model is an equilibrium of the beach profile that the vertical beach profile remains unchanged and only moves seaward or shoreward until the shore structure in parallel to itself.

Thus, the governing equation for the shoreline position can be obtained from the continuity equation for beach sediment with assumption of cohesionless sand as shown in Figure 2. 1.

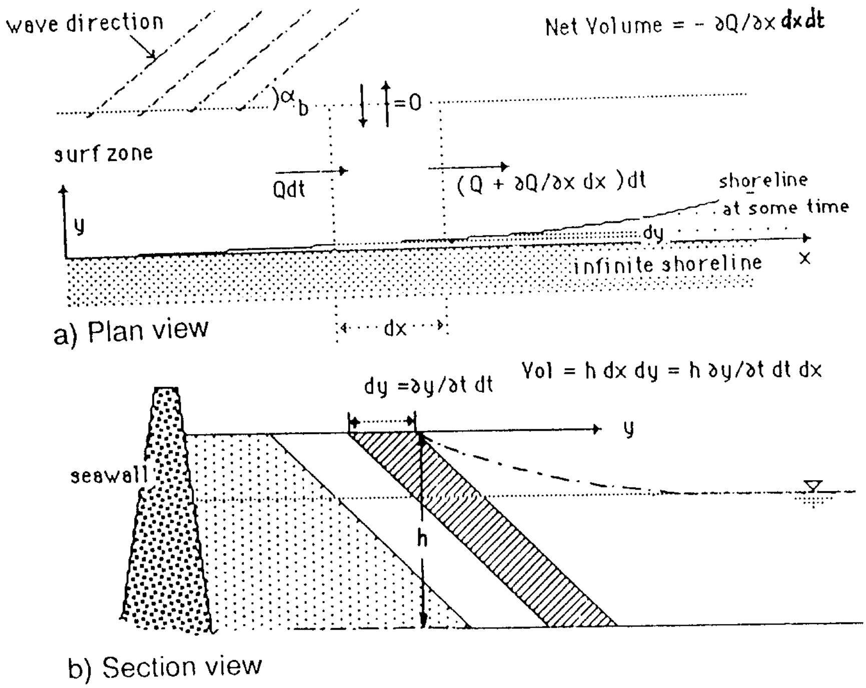

Figure 2. 1 Continuity of Beach Sediment

2.3. Equations for Sediment Transportation

Sediment transport and the resultant shoreline change depend on the local wave conditions, beach planform, boundary conditions and possible constraints such as the one due to a shore structure. The governing differential equation for beach sediment continuity is

$$\frac{\partial y}{\partial t}+\frac{1}{h}\frac{\partial Q}{\partial x}=0, \tag{2.1}$$

where y=the shoreline position (m)
x=the longshore coordinate (m)
t=the time (sec)
h=a representative depth beyond which is assumed no sediment transport occurs (m)
Q=the longshore sediment transport rate (㎥/sec).

In order to solve Equation (2.1), information on shore boundary conditions and the longshore sediment transport rate must be introduced to the system. Two locations of the shoreline (1979 and 1984 H.O. Survey) and two lateral ends of the beach (breakwater and jetty) are given as Figure 2.2 based on Figure 1.4.

S.P.M. describes about CERC formula that longshore transport rate Q depends on the longshore component of energy flux in the surf zone approximated by assuming conservation of energy flux.

The energy flux per unit length of wave crest is

$$P=\frac{1}{8}\rho g H^2 C_g, \tag{2.2}$$

where ρ=mass density of seawater, 1020 Kg/㎥
g=acceleration of gravity, 9.81 m/sec^2
C_g=group velocity (m/sec)
H=wave height (m).

If the wave crests make an angle α with the shoreline, the energy flux in the direction of wave advance per unit length of beach is

$$P\cos\alpha=\frac{1}{8}\rho g H^2 C_g \cos\alpha \tag{2.3}$$

and the longshore component of energy flux is expressed as

$$\begin{aligned}P_l=P\cos\alpha\sin\alpha&=\frac{1}{8}\rho g H^2 C_g\cos\alpha\sin\alpha\\&=\frac{1}{16}\rho g H^2 C_g\sin 2\alpha\end{aligned} \tag{2.4}$$

Using significant wave height the longshore component of wave energy flux at the breaker line is

$$P_{ls}=\frac{1}{16}H_{sb}^2 C_{gb}\sin 2\alpha_{bs}, \tag{2.5}$$

where subscript b is breaker value and s is significant wave height, and α_{bs} is the breaking wave crest angle relative to the shoreline. The angle we need in the equation of sediment transport is the angle between the wave crest at depth h and the instantaneous shoreline at some time, (t.) Because the shoreline is not parallel to the x-axis in this study, α_{bs} is different with the breaking wave crest angle relative to the x-axis, α_b.

$$\alpha_{bs}=\alpha_b-\tan^{-1}\left(\frac{\partial y}{\partial x}\right). \tag{2.6}$$

According to shallow water wave approximation, we have

$$C_{gb}=\sqrt{gh_b}=\sqrt{\left(g\frac{H_b}{\gamma}\right)}, \tag{2.7}$$

where γ is the ratio of wave height to water depth at breaking, approximately 0.78.

The empirical relationship between the logshore component of energy flux P_{ls} and the immersed weight of sand moved I_l along a straight shoreline as expressed by Inman and Bagnold (1963) is

$$I_l = KP_{ls} = (\rho - \rho) g a' Q, \qquad (2.8)$$

where K=dimensionless coefficient, depends on wave height used in wave energy term, 0.39 for H_s (Watts, 1953 and Caldwell, 1956) or 0.77 for H_{rms} (Komar and Inman, 1970)

ρ=mass density of sand, 2650kg/㎥ for quartz sand

a'=sand porosity (volume of solids/total volume), approximately 0.6.

Thus, the volumetric sediment transport rate becomes

$$Q = \frac{KP_{ls}}{(\rho_s - \rho) g a'}$$

$$= \frac{KH_{sb}^2 C_{gb} \sin 2\alpha_{bs}}{16 \quad (s-1) a' 1.416^{5/2}}$$

$$= \frac{KH_{sb}^{5/2} \sqrt{\frac{g}{\gamma}} \sin 2\alpha_{bs}}{16(s-1) a' 1.416^{5/2}}, \qquad (2.9)$$

in which s is the ratio of sand density to water density$\left(\frac{\rho_s}{\rho}\right)$, 2.5854 and the factor $1.416^{5/2}$ converts the root mean square wave height to significant wave height.

CHAPTER III. DESCRIPTION OF THE NUMERICAL MODEL

The advantages of numerical modeling include the capability to readily incorporate many parameters such as complicated structure geometry, changing wave direction and wave height, wave diffraction behind structure, etc.

Used numerical scheme in this study is the finite difference method. The beach is discretized along the shoreline at a constant interval ($\triangle x$) and the position of the shoreline iscalculated at constant time interval ($\triangle t$). The discretized beach area is shown in Figure 3.1.

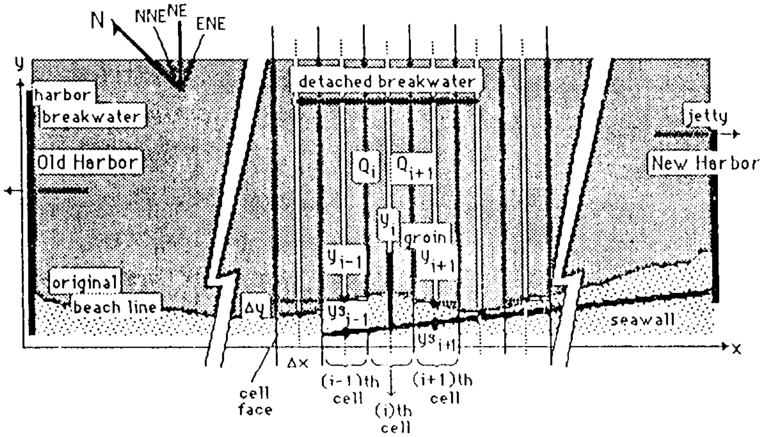

Figure 3.1 Shoreline Representation in a Finite Difference Model

The x-axis which starts from the harbor breakwater of Pohang Old Harbor is divided into N (68) calculation cells by N+1 (69) cell faces (solid vertical line) for case I. For case II, N=336, in order to compare the stability of the solution. The y-axis runs perpendicular to the beach line, directed NNE to the open sea and includes Q and y points which define the cell face and the center of each cell, respectively.

Both Q and y points are discretized by a constant cell width ($\triangle x$=25m and 5m for cases I and II, respectively). Hence, the distance between Q_i and y_i point is $\frac{\triangle x}{2}$ = 12.5 m or 2.5 m Two limits of the lateral boundary are specifide at Q_1 as the old Harbor breakwater and at Q_{N+1} as the jetty at the mouth of Hyongsan River.

A seawall denoted by y_{si} and a detached breakwater which will be introduced into the model are decided to start and end at the center of a cell. Groins are represented as lines at the center of each cell becase of the relative width to the cell width.

Two nemerical models, explicit and implicit, will be introduced and illustrated in following sections.

3.1. Explicit Model

The standard explicit scheme utilizes a time-marching space in which the beach line orientation and the calculated sand transport rate held fixed over one time step from $j\triangle t$ to $(j+1)\triangle$t. The changes in shoreline position are calculated based on this information.

The finite difference equations for Equations (2.9) and (2.1) are

$$Q_i^{j+1}=\frac{K}{16(s-1)a'\,1.416^{5/2}}\left(H_{sb}^{5/2}\right)^j\left(\sqrt{\frac{g}{r}}\right)^j\sin 2\alpha_{sb}, \quad (3.1)$$

$$y_i^{j+1}-y_i^j=-\frac{\triangle t}{h^j\triangle x}\left(Q_{i+1}^{j+1}-Q_i^{j+1}\right) \quad (3.2)$$

and Equation (2.6) for the breaking wave crest angle relative to the x-axis is

$$\alpha_{sbi}^j=\alpha_{bi}^j-\alpha_{zi}^j \quad (3.3)$$

where

$$\alpha_{zi}^j=\tan^{-1}\frac{y_{i+1}^j-y_i^j}{\triangle x}.$$

The superscripts denote the time level at which the variable is evaluated. Quantities at the jth time step are known. The subscripts denote the cell level.

This model is convenient because only immediately neighboring values of Q_i and y_i are involved, and the implementation only involves the present beach line position and sand transport rates.

3.2. Implicit Model

The implicit scheme is based on the same equations as the explicit model.

Compared to the straightforward development for the explicit scheme, representation of the seawall constraint in the implicit scheme is very complex and increases the execution time of the computation. However, for the implicit model they are solved for all of the cells, simultaneously and thus, greater numerical stability results.

In this method the finite difference form of Equation (2.1) is expressed as

$$y_i^{j+1}-y_i^j=-\frac{1}{2}\left\{\frac{\triangle t}{h^{j+1}\triangle x}\left(Q_{i+1}^{j+1}-Q_i^{j+1}\right)+\frac{\triangle t}{h^j\triangle x}\left(Q_{i+1}^j-Q_i^j\right)\right\} \tag{3.4}$$

In order to give equal weight to two adjacent time levels the mean values are introduced.

The time evolution of the shoreline position y_i in the implicit scheme is compared with that of the explicit scheme in Figure 3. 2. As the shoreline change rate $\frac{\partial y}{\partial t}$ is constant in time $\triangle t$, the shoreline change over $\triangle t$ presents a straight line mode.

As Equation (2. 9) includes higher order term, more simplified expression is introduced. Linearized expression of Equation (2. 9) was first used by Perlin and Dean (1978)

and this was extended by Ozasa and Brampton (1980), and Kraus and Harikai (1983), considering a systematic change in breaking wave height alongshore by wave diffraction.

Hanson and Kraus (1985, 1986) contributed to improve this expression under the seawall constraint. In this report the model is based on a procedure developed by Kraus and Harikai and improved by Hanson and Kraus. For details on derivation of the equation these references should be consulted.

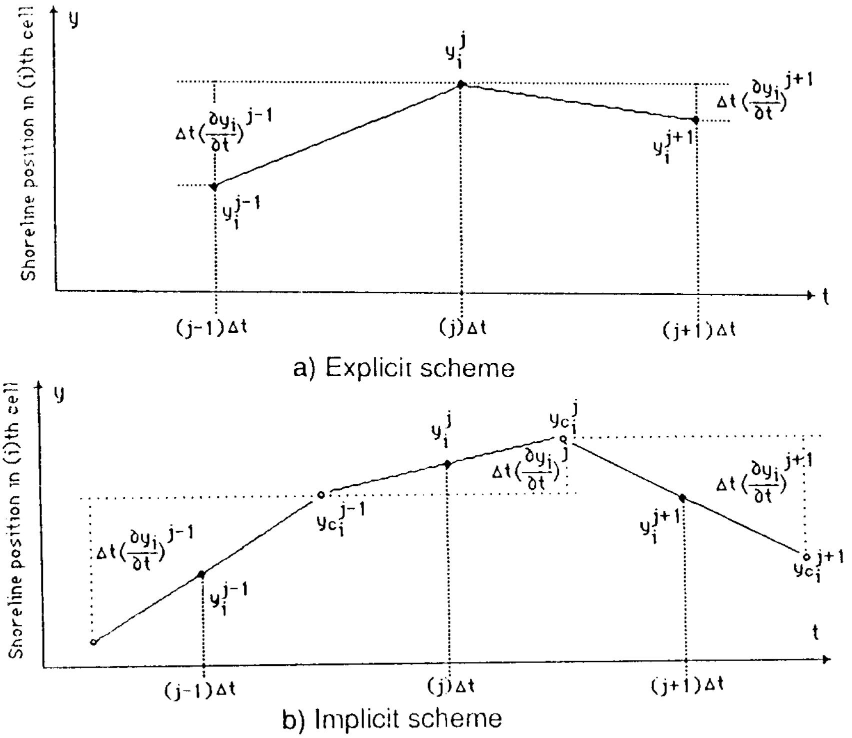

Figure 3. 2 Comparison of the Time Evolution of a Representative Shoreline position Coordinate.

The final expression of the sand transport rate along the beach is

$$Q_i^{j+1}=E_i^{j+1}Q_{i+1}^{j+1}+F_i^{j+1}, \qquad (3.5)$$

where

$$E_i^{j+1}=\frac{\triangle tEP_i^{j+1}}{2h_j\triangle x\{1+\triangle tEP_i^{j+1}(2-E_{i-1}^{j+1})2h_j\triangle x\}}$$

$$EP_i^{j+1}=\frac{K}{16(s-1)a'1.416^{5/2}\triangle x}\left(H_{sb}^{5/2}\right)^j\left(\sqrt{\frac{g}{\gamma}}\right)^j 2\cos 2\alpha_b\cos^2\alpha_z$$

$$F_i^{j+1}=\frac{FP_i^{j+1}-EP_i^{j+1}\left(y_{ci}^j-y_{ci-1}^j\right)+\frac{\triangle t}{2h^j\triangle x}EP_i^{j+1}F_{i-1}^{j+1}}{1+\frac{\triangle t}{2h^j\triangle x}EP_i^{j+1}\left(2-E_{i-1}^{j+1}\right)}$$

$$FP_i^{j+1}=\frac{K}{16(s-1)a'1.416^{5/2}}\left(H_{sb}^{5/2}\right)^j\left(\sqrt{\frac{g}{\gamma}}\right)^j\sin 2\alpha_b\{(2\cos^2\alpha_z)^2-1\}$$

$$y_{ci}=y_i+\frac{\triangle t}{h^j\triangle x}\left(Q_{i+1}^j-Q_i^j\right)$$

3.3. Constraint of the Shoreline Movement in the Model

The lateral boundaries, the harbor break-water and the jetty at the mouth of Hyoungsan River, and groins (if applicable) are assumed impermeable barriers as $Q=0$ and $\frac{\partial Q}{\partial x}=0$.

In addition to this, the shoreline along the beach backed by a structure as described in section 2.1 cannot recede behind the structure. Thus, the sand transport rates should be modified along the beach in this case.

Depending on the breaking wave condition the sand transport rate categorizes three conditions in front of the seawall such as regular, minus and plus areas (see Figure 3.3)

At two conditions as b) a regular minus cell and c) a minus cell, the transport rate should be adjusted to allow calculation cells in contact with the seawall to transfer sand in parallel to the structure so as to preserve direction of transport and conserve total sand volume.

Hanson and Kraus (1985) showed a correction to the transport rates at such a cell as equal proportion of the original rates as following equation.

$$\frac{Q_i}{y_i^j-y_i^{j+1}}=\frac{\triangle Q_i}{y_i^j-y_{si}^j} \qquad (3.6)$$

where Q_i moves the shoreline from y_i^j to y_i^{j+1}, $\triangle Q_i$ is a corrected transport rate and it moves the shoreline from y_i^j to y_{si}^j.

Figure 3.4 shows the relationship of Equation (3.6) and this equation is also to be introdced into two models for this study to recalculate the transport rate and adjust the shoreline position, accordingly.

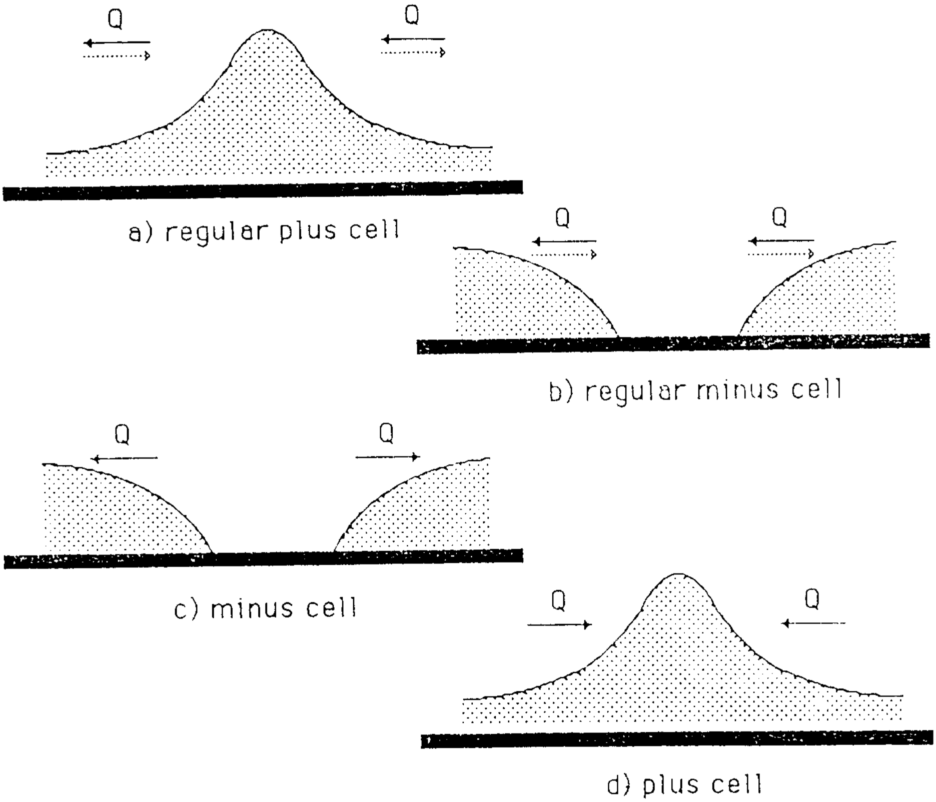

Figure 3.3 Definition Sketch of Sand Transport

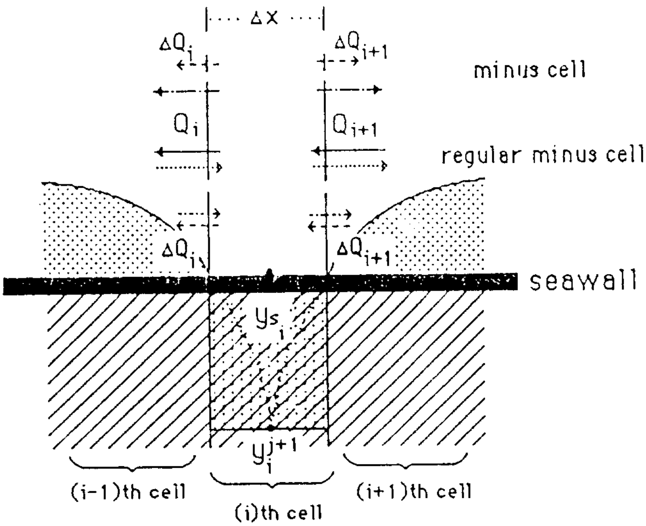

Figure 3.4 Conceptual Diagram for Shoreline and Sand Transport Rate Corrections

CHAPTER IV. BREAKING WAVE FIELD

In order to calculate the longshore sediment transport along the beach, the breaking wave field, specifically height and angle along the beach, must be obtained from a computer program such as a combined refraction and diffraction program or from the statistical analysis of the field measurements.

As this was beyond the scope of this study, the empirical equations were used for the model.

Le Mehaute and Koh (1967) presented an approximation equation to get a breaking condition from the deep water wave height and period,

$$H_b = 0.76 H_o \left(\frac{H_o}{L_o}\right)^{-1/4} m^{1/7}. \tag{4.1}$$

where H_o and L_o are deep water wave height and wave length and m, the average bottom slope.

The waves are assumed to break where the wave height becomes 0.78 times the water depth. After breaking, the transition of wave height can be expressed by Horikawa and Kuo's approximtion equation (1966),

$$H = h\left(0.5 + 0.3e^{-0.11\frac{\Delta x}{h_b}}\right) \tag{4.2}$$

where h, the water depth referring to mean water level, Δx, the distance from the breaking point, and h_b, the water depth at the breaking point.

This approximation is valid for the bottom of moderate bed slopes less than 0.033. Youngil Bay has much less (about 0.0125) than this limit throughout the bay along the cross sectiom A-A' in Figure 1.1.

An example of calculated breaking wave height and breaking water depth by above equations for a 100 year design typhoon wave (H_s=14.8m, T_s=15.6sec) at the study area is shown in Figure 4.1.

The second diagram in Figure 4.1 represents the depth profile and one wave length from the shore for periods of 6, 8, 10, 12, 14, 16, 18 and 20 seconds by Hunt's formula (1979).

The final introduced breaking wave height at the depth of 5m to the models is 2.44m.

As per Chu's analysis (1976) of wind climte at Youngil Bay, it was recorded that NNE wind among the wind directions over 10 m/sec of wind speed was 40.2% of all directions and N, NNE and NE winds, 67.8%, whereas W, WSW, and SW winds were only 25.3%.

With this information it is assumed that the significant breaking waves are approaching from NE, NNE and ENE directions.

Wave characteristics in the lee of an offshore breakwater which will be introduced into the models are based on the calculations by Wiegel (1962) and the results of a numerical model by Copeland (1984). The relative wave heights behind the breakwater for three different directions shown in Figure 4.2 are partly from the numerical model test and partly from the calculation.

The significant breaking wave height and angle to the x-axis are collected at various points behind the breakwater as Figure 4.3

and these are introduced into the model in three groups as per the breaking wave directions.

For case II, we need more data points than case I.

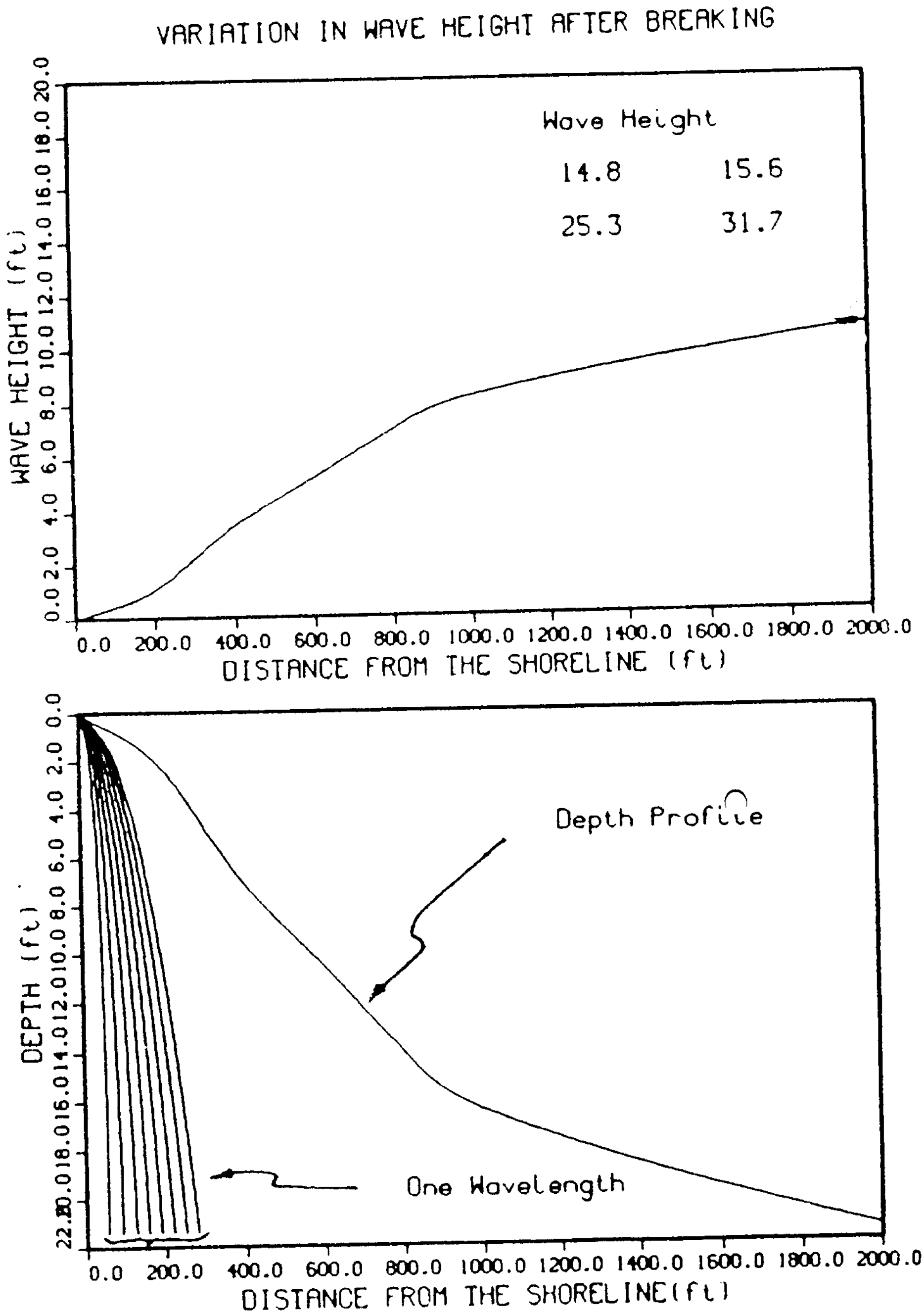

Figure 4. 1 Variation in wave Height after Breaking

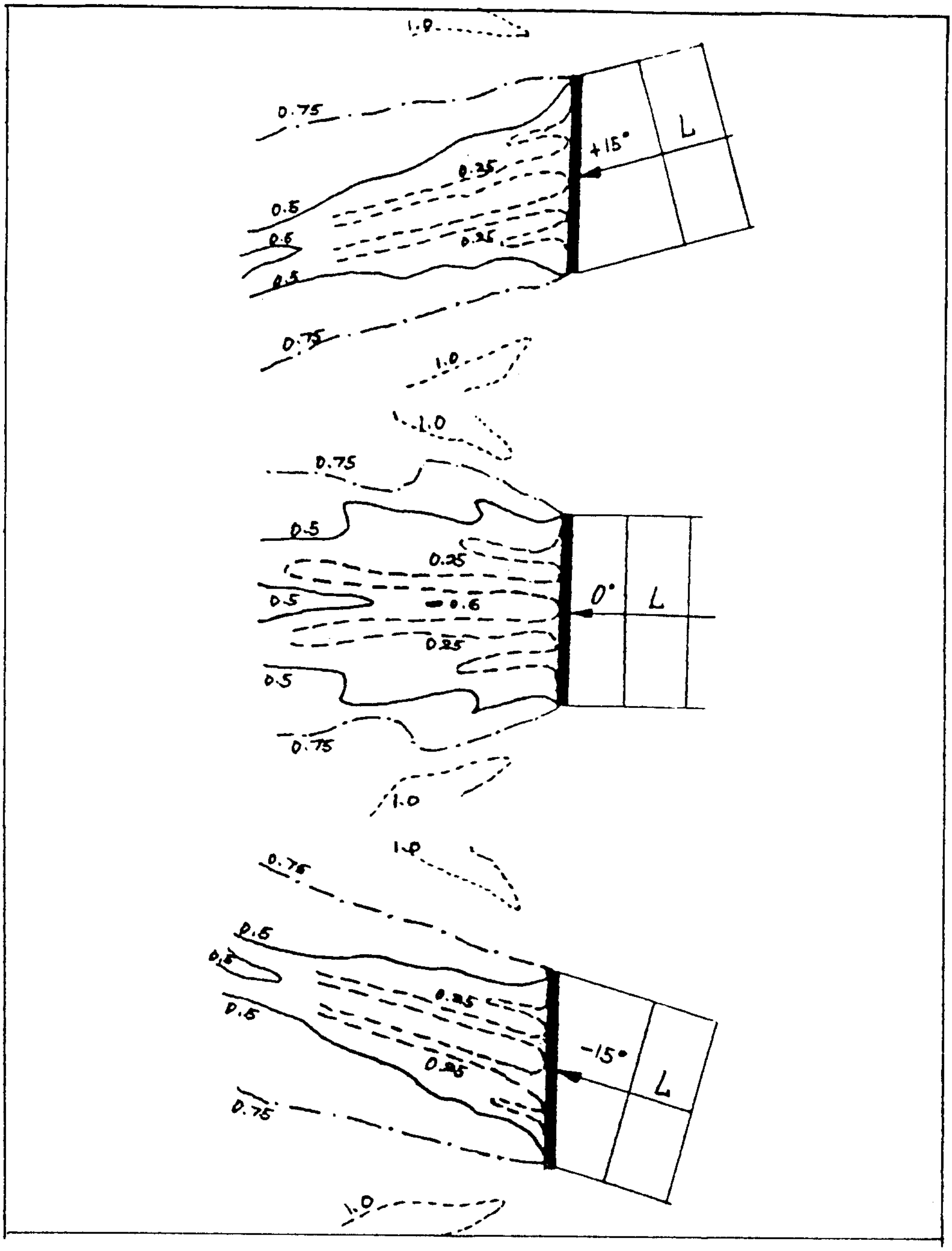

Figure 4.2 Contours Significant Breaking Wave Heights Predicted for a Perfectly Reflecting Off-shore Breakwater of Length 3 x Wavelength

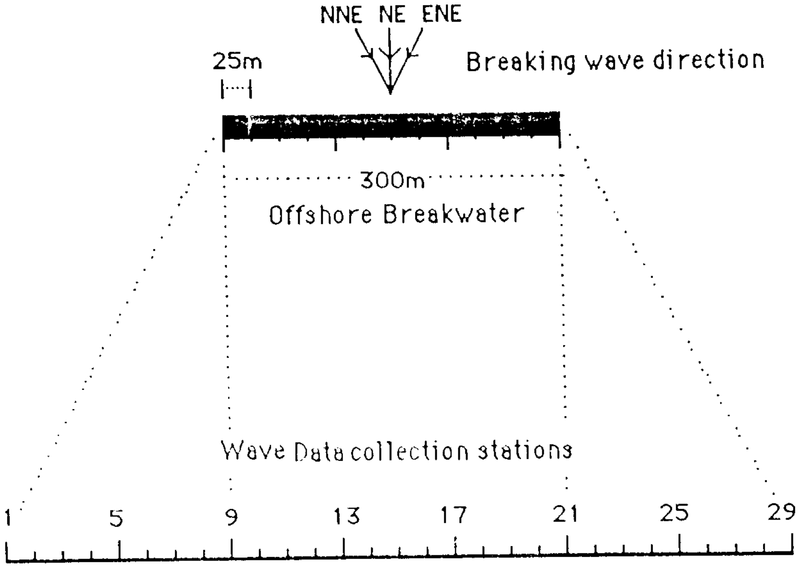

Figure 4.3 An Example of Data Collection Points behind an Offshore Breakwater for Significant Breaking Wave

CHAPTER V. MODEL SIMULATION

In this chapter results for several test cases will be given. The initial shoreline is classified as a 1979 shoreline before a storm, a 1979 shoreline after storm, and a 1984 shoreline with three groins. The time step $\triangle t$ is important to the models because it is related to the stability of the model and the numerical accuracy of the solution.

The stability parameter introduced by Kraus and Harikai (1983) can be expressed as

$$R_s = \frac{K}{8(s-1)a' \, 1.416^{5/2}} H_{sb}^{5/2} \left(\sqrt{\frac{g}{\gamma}} \right)_b \frac{\triangle t}{h_b (\triangle x)^2} \qquad (5.1)$$

and all parameters were well described in Chapter II.

The explicit model showed twice faster than the implicit model for 6 hours time step. The implicit model took larger time steps while preserving reasonable numerical accuracy, whereas the explicit model showed unstable for case I. For case II, the explicit method showed stable solution, too.

Simulations were made with three 6 day representative waves such as waves directed mormal (NE) to the beach, -15°(NNE) and 15°(NNE), successively. Shoreline changes and sediment transport rates for both models were plotted on same sheet along with original shoreline and transport rate used.

In order to find out the effect of the groins, first, the beach line in 1979 before a storm was used for simulation and the one in 1984 with three groins was included later.

Computer the shoreline by the numerical models with the surveyed shoreline in 1979 after storm in Figure 5.1. it was found that the erosion at the right side of the beach was caused by the severe storm waves directed from NNE at first and later by the waves directed from NNE and E.

Even with the breaking waves normal to the x-axis the sand moves from right to left direction between at the beginning of the seawall and the jetty of the right lateral boundary.

As shown in Figures 5.5 through 5.8, the three groins greatly affect to the beach formation and sand transport rate. Moreover, the offshore breakwater changes this situation, depending on the location.

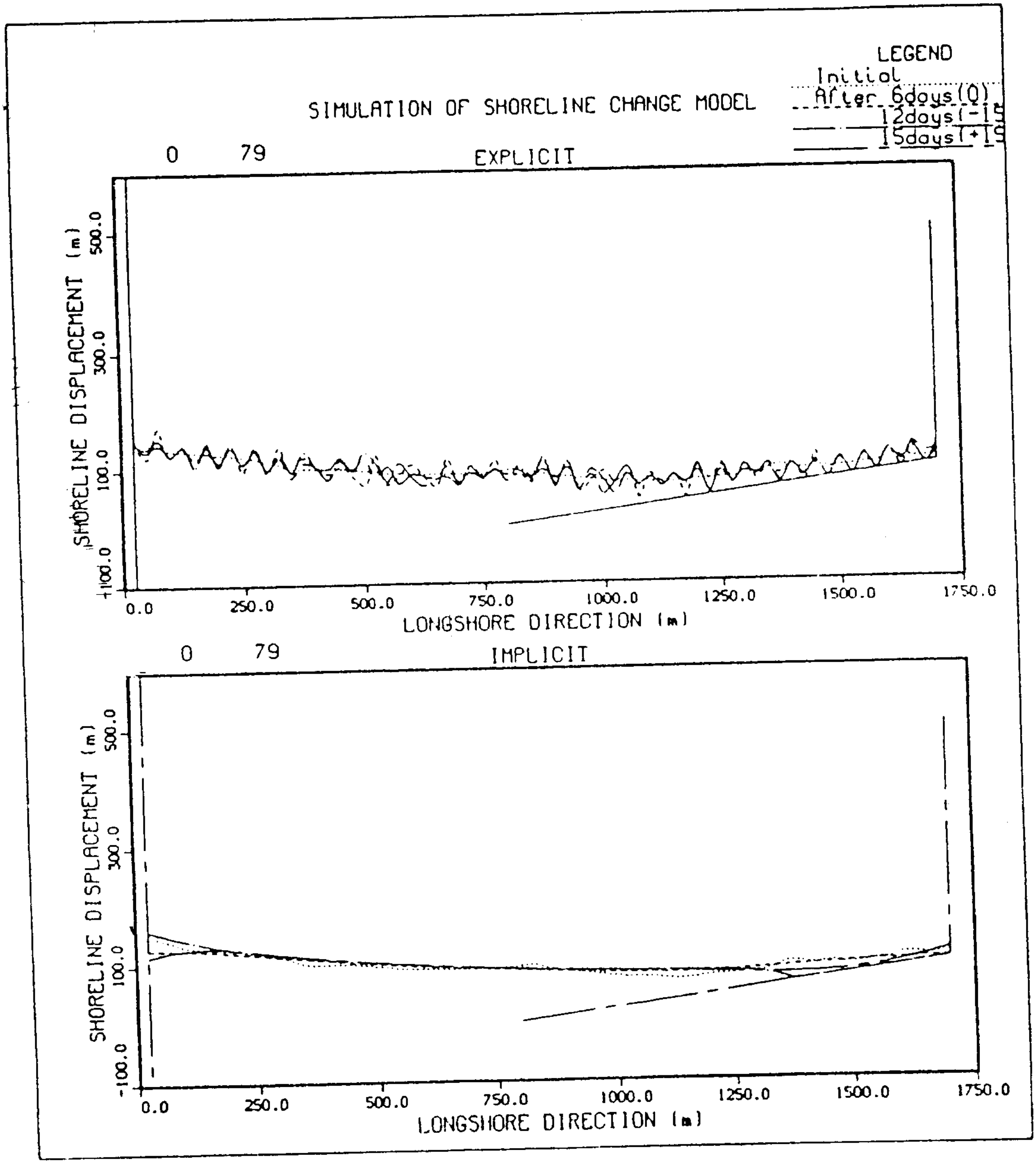

Figure 5. 1 Simultion of Shoreline Change Model with the 1979 Shoreline before a Storm and without an Offshore Breakwater-Case I

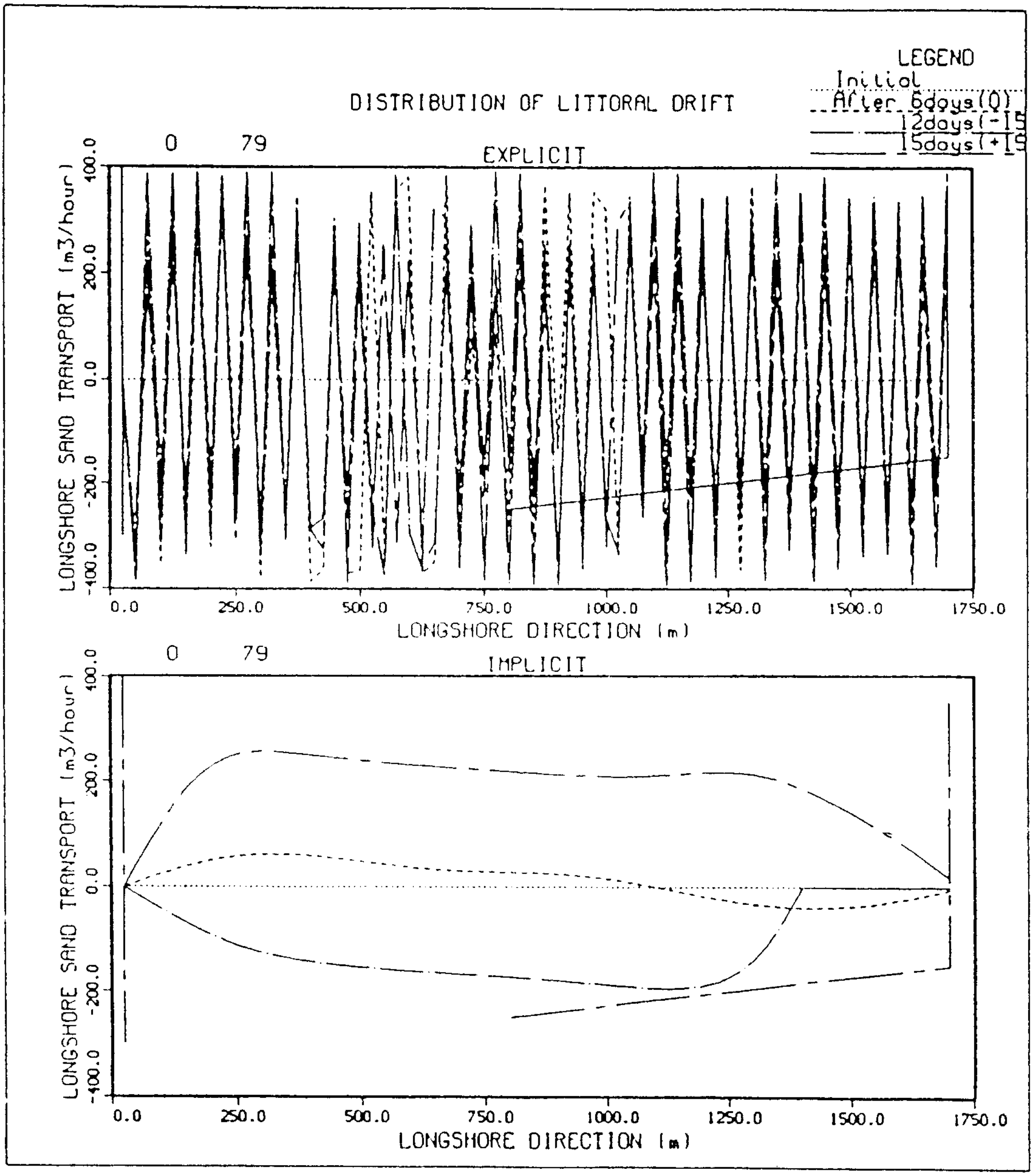

Figure 5.2 Distribution of Littoral Drift with the 1979 Shoreline before a Storm and without an Offshore Breakwater-Case I

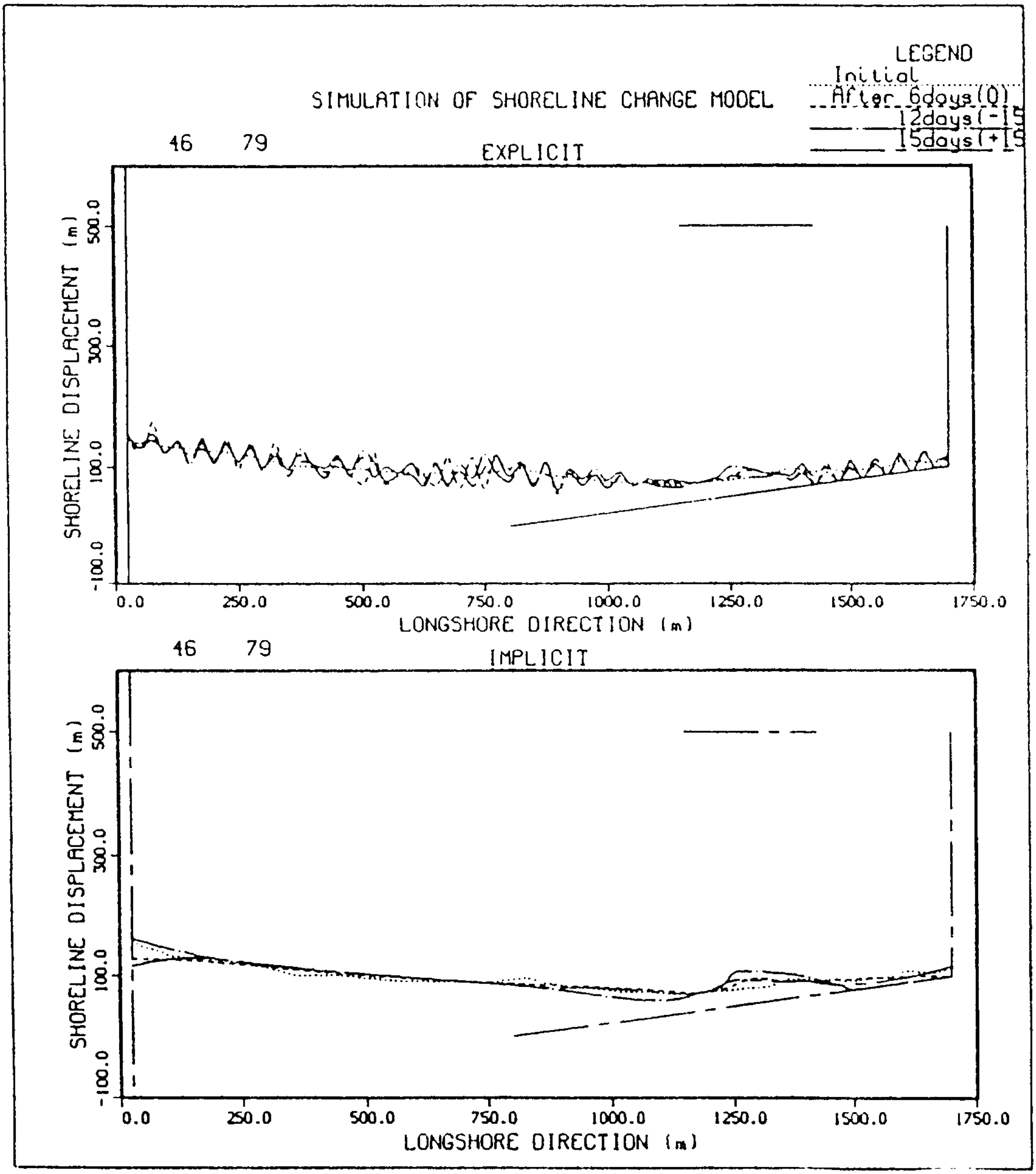

Figure 5.3 Simulation of Shoreline Change Model with the 1979 Shoreline before a Storm and an Offshore Breakwater–Case I

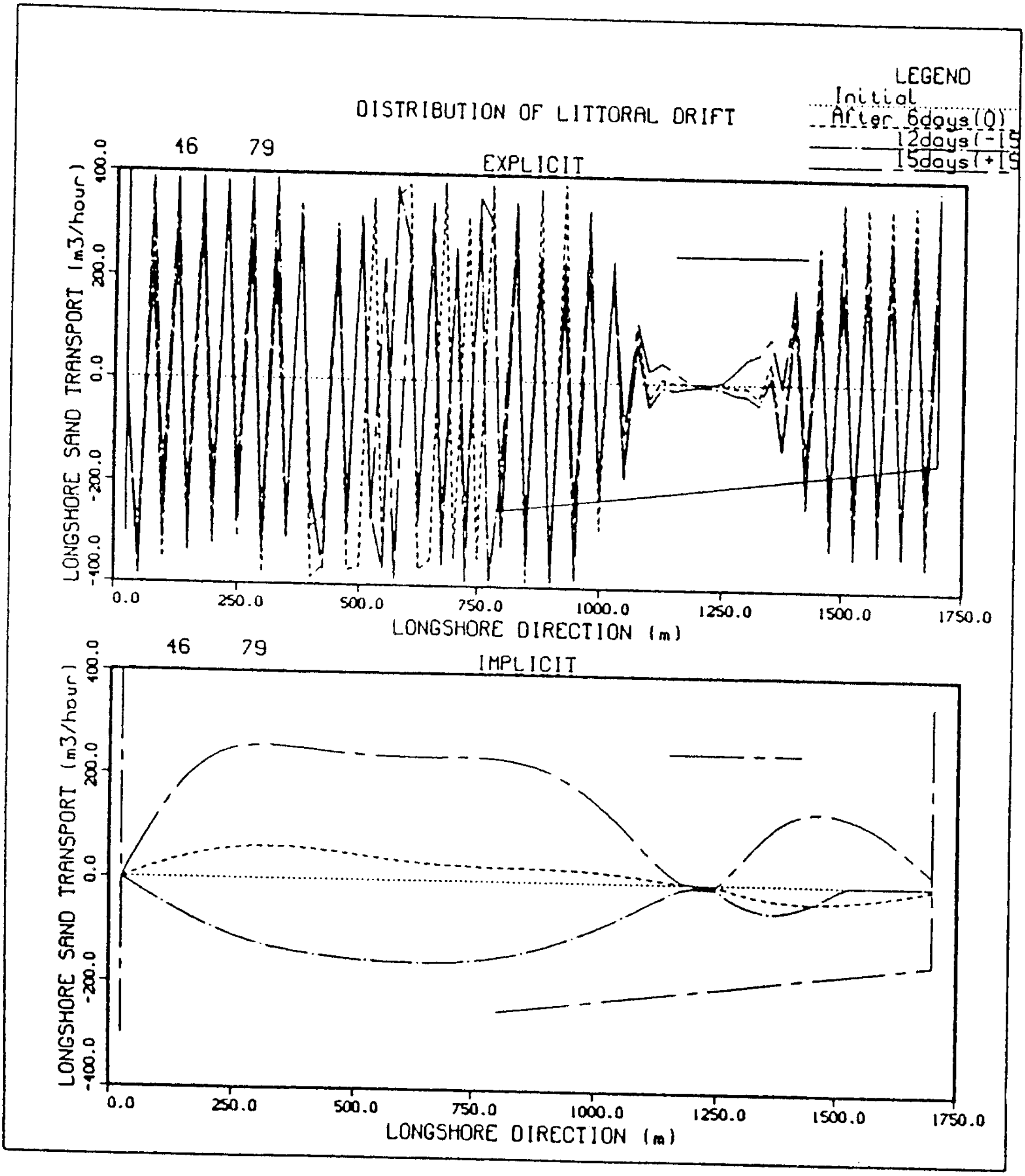

Figure 5.4 Distribution of Littoral Drift with the 1979 Shorline before a Storm an Off-shore Breakwater–Case I

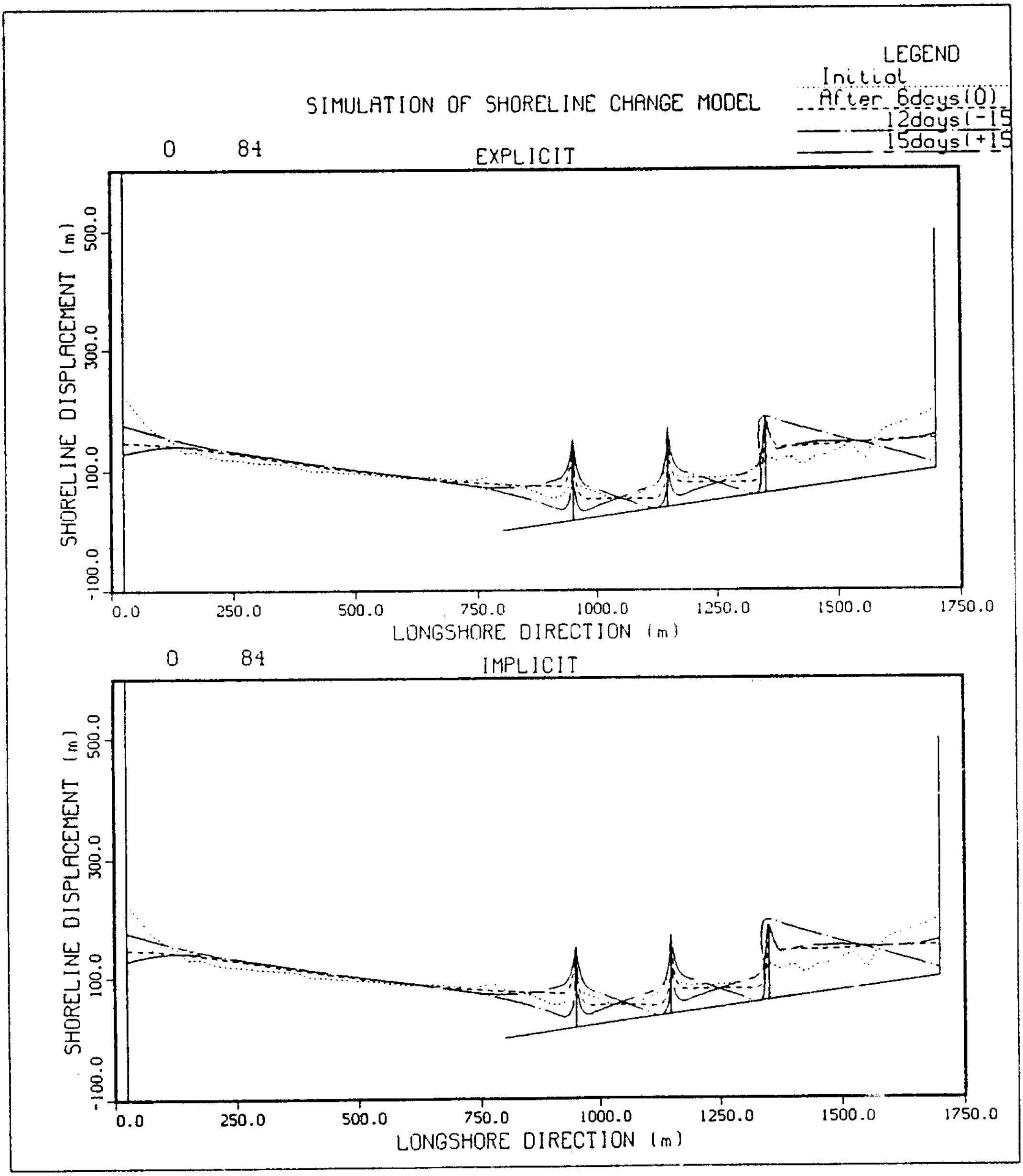

Figure 5.5 Simulation of Shoreline Change Model with the 1984 Shoreline before a Storm and without an Offshore Breakwater–Case II

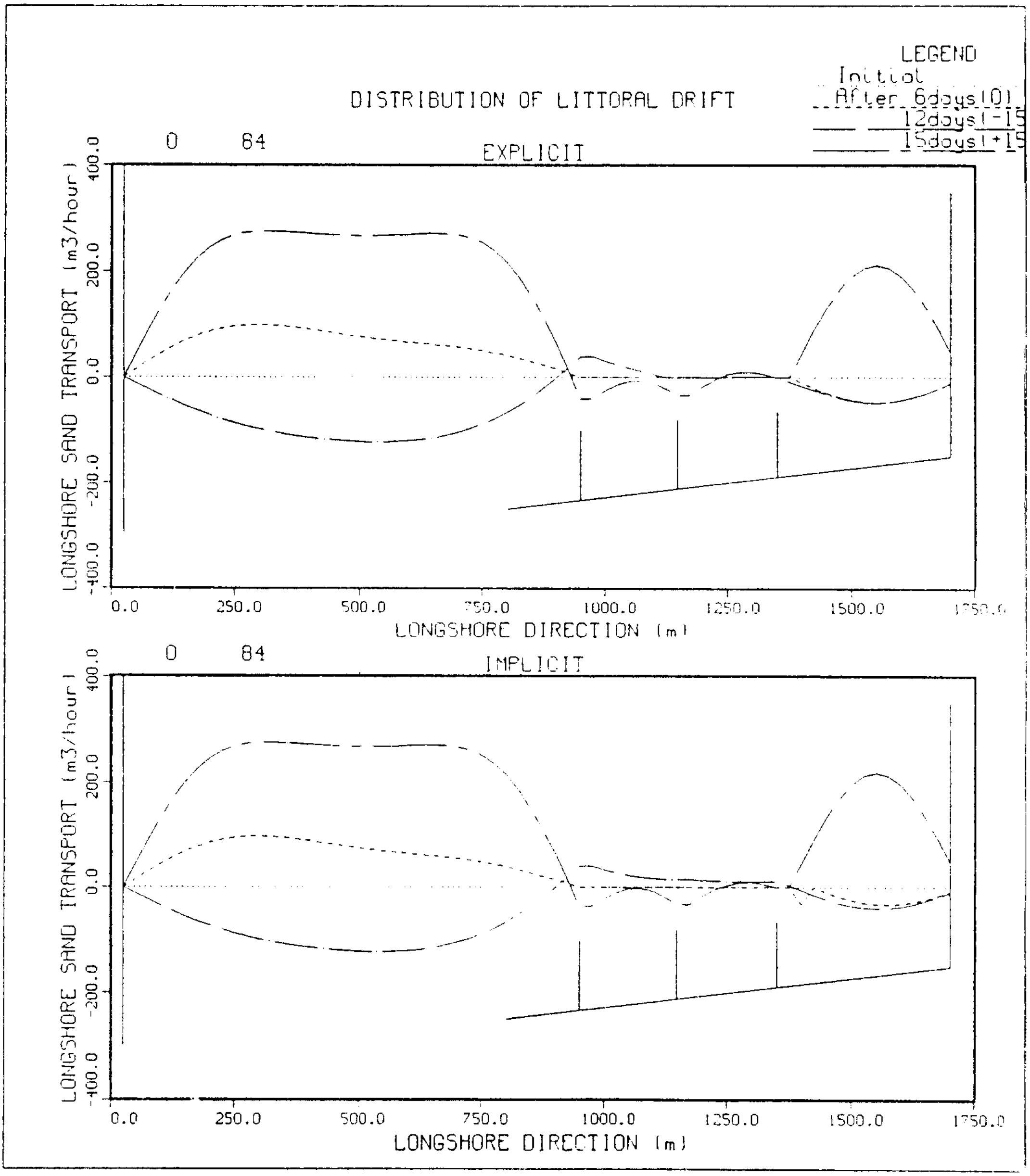

Figure 5.6 Distribution of Littoral Drift with the 1984 Shoreline before a Storm and without an Offshore Breakwater-Case II

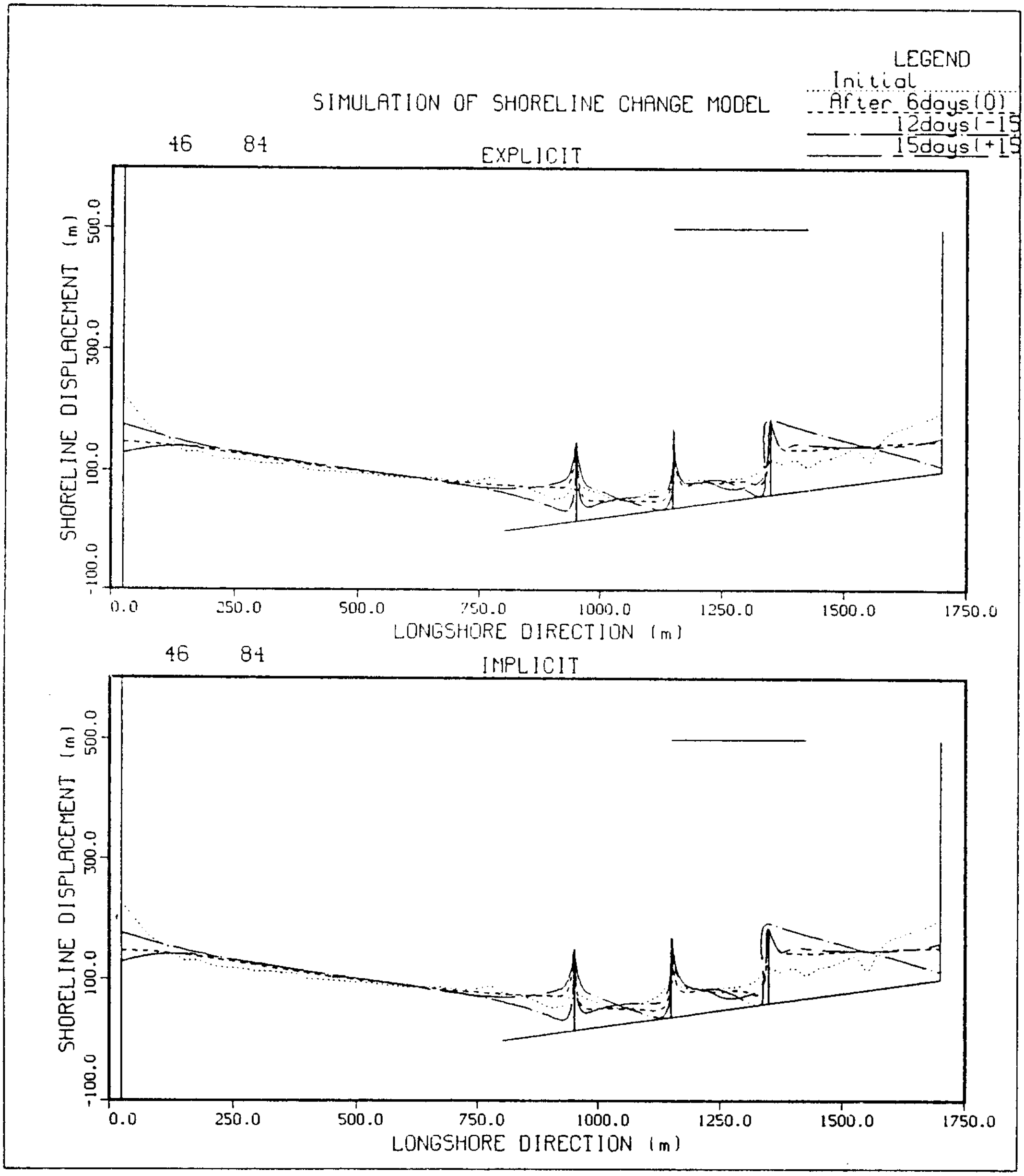

Figure 5.7 Simulation of Shoreline Change Model with the 1984 Shoreline before a Storm and an Offshore Breakwater-Case II

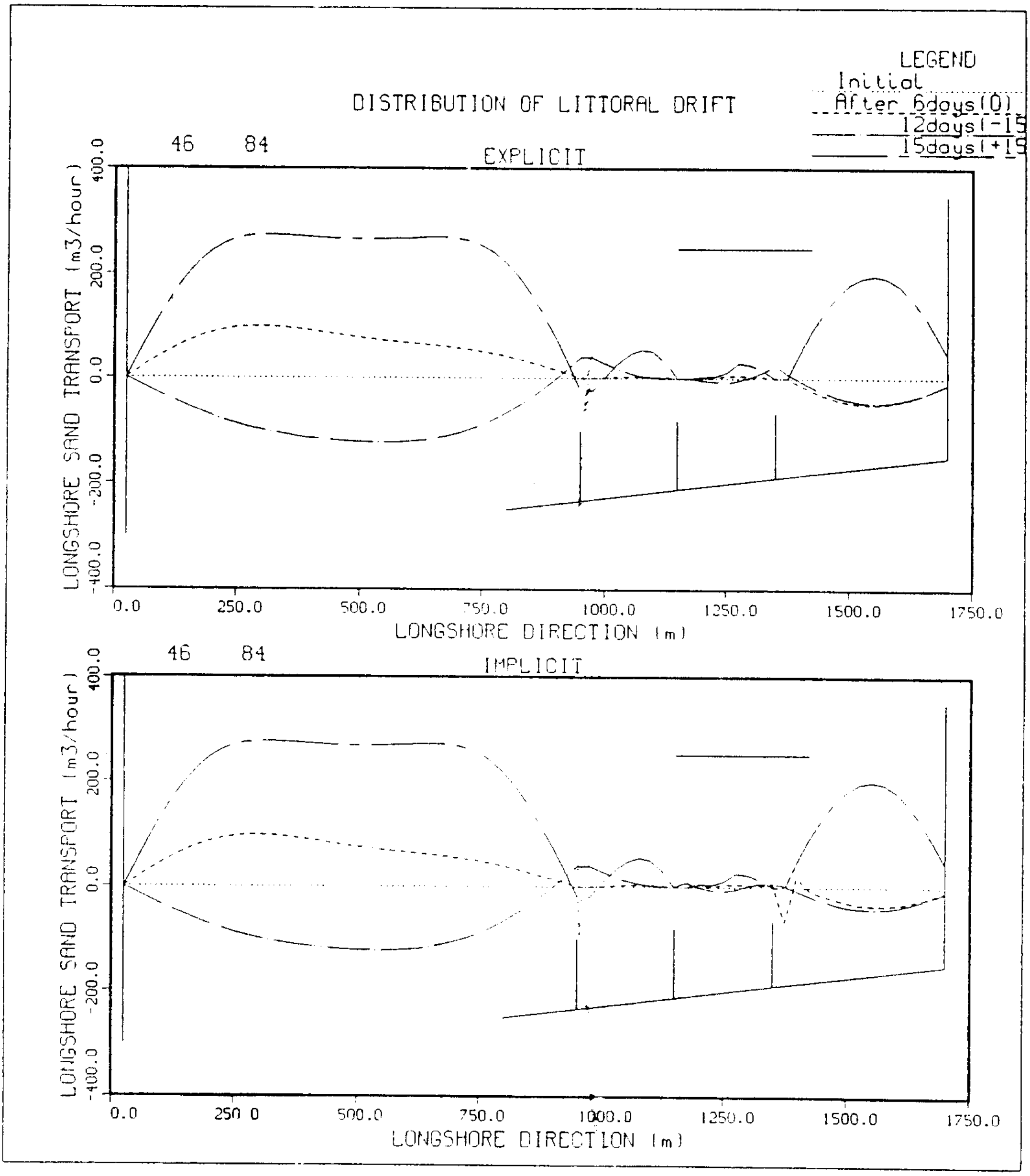

Figure 5.8 Distribution of Littoral Drift with the 1984 Shoreline before a Storm and an Offshore Breakwater-Case II

CHAPTER VI. CONCLUSIONS

The governing differential equation for beach sand continuity and equations related to the sediment transport rate are presented in two different finite difference forms : an explicit solution algorithm and an implicit algorithm.

Both algorithms include the seawall constraint and various boundaries, such as two lateral boundaries : harbor breakwater and jetty, three groins, and an offshore breakwater.

Investigated by two models are the beach line changes and sediment transport rates for the beach before design of three groins with and without an offshore breakwater. In order to estimate the shoreline changes after the groins were built, it was also invesigated the beach response inside three groin compartments and the effect of an offshore barrier based on the original beach line by the Hydrographic Office survey in 1984.

These results are in accordance with the general tendencies in the several references, even though the conditions are more complicated. However, since the field measurment of wave and sand parameters to the study area are not available at present time, a comparison with the real world could not be made. More improvements on two models needed in future study are

a) a consideration of beach slopes milder and steeper than the assumed equilibrium slopes,

b) an inclusion of the mechanics of onshore-offshore sediment transport,

c) a consideration on the distribution of longshore sediment transport across the surf zone along with the longshore current,

d) an inclusion of the field measurement of wave characteristics and sand transport parameters,

e) quantification of the sand transport processes through a permeable shore structure, and

f) an inclusion of a sand source from the Hyongsan River.

An alternative to the six considerations above is to develop a multi-line model, especially in the vicinity of structures.

REFERENCES

1) Bailard, J. A., "Simple Models for Surfzone Sediment Transport," Naval Civil Engineering Laboratory, NCEL Technical Note TN-1740, Cali. December 1985.

2) Borah, D. K. and Balloffet, A. "Beach Evolution Caused by Littoral Drift Barrier," Coastal Structures '83, 1983, pp. 690-702.

3) Chu, K. S., "The Seiches at pohang Harbor," Journal of the Oceanological Society of Korea, Vol. 11, No. 2, December 1976, pp. 51-56.

4) Copeland, G. J. M., "A Practical Alternative to the Mild-slope Wave Equation," Coastal Engineering, Vol. 9, 1985, pp. 125-149.

5) Dally, W. R., "Detached Breakwaters for Shore Protection," Department of the Army, US Army Corps of Engineers, Coastal Engineering Research Center Tech. Rept. CERC-86-1, Washington, January 1986.

6) Deigaard, R., Fredsoe, J. and Hedegaard, I. B., "Mathematical model for Littoral Drift," J. Waterway, Port, Coastal and Ocean Engineering, ASCE, Vol. 112. No. 3, May 1983, pp. 673-689.

7) Everts, C. H., "Shoreline Changes Downdrift of a Littoral Barrier," Coastal Structures '83, ASCE, 1983, pp. 673-689.

8) Gerritsen, F. "Beach and Surf Parameter in Hawaii," University of Hawaii Sea Grant College Program, 1978.

9) Hanson, H. and Kraus, N. C., "Seawall Constraint in Shoreline Numerrcal Model," J. Waterway, Port, Coastal and Ocean Engineering, ASCE, Vol. 111, No. 6, November 1985. pp. 1079-1083.

10) Hanson,. H. and Kraus, N. C., "Seawall Boundary Condition in Numerical Models of Shoreline Evolution," Department of the Army, US Army Corps of Engineers, Washington, April 1986.

11) Hoppen, H. G. H. Ten and Bakker, W. T., "Erosion Problems of the Dutch Island of Goeree," Coastal Research Department, Rijkswaterstaat, W-74. 026, 1974.

12) Hydrographic Office, Republic of Korea, Technical Reports pub. No. 1101-1970, 1972.

13) Komar, P. D., "Computer Models of Shoreline Changes," in CRC Handbook of Coastal Processes and Erosion (ed. Komar, P. D.), CRC Press, Inc., Boca Raton, F. L., 1983. pp. 205-216.

14) Kraus, N. C. and Harikai, S., "Numerical Model of the Shoreline Change at Oarai Beach," Coastal Engineering, No. 7, 1983, 00. 1-28.

15) Massie, W. W. (ed)., "Coastal Engineering Vol. II : Harbor and Beach Problems," Delft University of Technology, DELFT, Fall 1980.

16) Matsuoka, M. and Ozawa, Y., "Application of a Numerical Model to Prediction of Shoreline Changes," Coastal Structures, ASCE, 1983, pp. 646-659.

17) Mimura, N., Shimizu, T. and Horikawa, K., "Laboratory Study of the Influence of Detached Breakwater on Coastal Change," Coastal Structrure '83, 1983, pp. 740-752.
18) Mizumura, K., " Shoreline Change Estimates near Oarai, Japan," J. Waterway, Port, Coastal and Ocean Division, ASCE, Vol. 108, No. WW1, February 1982, pp. 65-80.
19) Ozasa, H. and Brampton, A. H., "Mathematical Modelling of Beaches Backed by Seawalls," Coastal Engineering, Vol. 4, No. 1, 1980, pp. 47-63.
20) Park, H. I., Chung, J. Y. and Oh, I. S., "Numerical Experiment of the Seiche in Youngil Bay and Pohang New Harbor, Korea," J. Oceanological Society of Korea, Vol. 21, No. 4, December 1986, pp. 245-258.
21) Perilin, M., "Predicting Beach Planforms in the lee of a Breakwater," Coastal Structures '79, Vol. II, ASCE, 1979, pp. 792-808.
22) Sargent, E. F. and Birkemeier, W. A., "Application of the Dutch Method for Estimating Storm-Induced Erosion," Coastal Engineering Research Center Instruction Report CERC-85-2, 1985.
23) U.S. Army Corps of Engineers, Waterway Experiment Station, "Shore Protection Manual," Vickburg, Vol. I & II, 1984.
24) U.S. Army Corps of Engineers, Waterway Experiment Station, "Regional Coastal Processes Numerical Modeling System Report 1," Technical Report CERC-86-4, March 1986.
25) Van de Graaff, J. and Van Overeem, J., "Evaluation of Sediment Transport Formulae in Coastal Engineering Practice," Coastal Engineering Vol. 3, 1979, pp. 1-32.
26) Vellinga, P., "Beach and Dune Erosion during Storm Surges," Waterloopkundig Laboratorium Delft Hydraulics Laboratory, Pub. No. 276, August 1982.
27) Vellinga, P., "Predictive Computational Model for Beach and Dune Erosion during Storm Surges," Waterloopkundig Laboratorium Delft Hydraulics Laboratory, Pub. No. 294, February 1983.
28) Wiegel, R. L., "Diffraction of Waves by Semi-infinite Breakwater," J. Hydraulic Division, Proceedings of the American Society of Civil Engineers, Vol. 00, No. HY1, January 1962, pp. 27-44.