This chapter details the methods involved in collection and analysis of the data used in the study. It is followed by the brief description of the selected models to estimate hurricane parameters, offshore wave, nearshore wave characteristics, sediment transport rate and shoreline change.

### 3.1 Data collection and analysis

#### 3.1.1 Hurricane

The data of hurricanes crossed the coastline from $14^0\text{N}$ to $17^0\text{N}$, in the vicinity of the Hoian coastline, between 1945 and 2003 were extracted from the National Weather Service, USA (www.weather.unisys.com/hurricane). This page provides access to a wealth of hurricane information including charts on the track of the storm plus a text based table of tracking information. The features such as velocity of forward motion and wind speed were considered. To classify of hurricanes the Saffir/Simpson hurricane scale was used in the present study (Table 3.1)

<table>
<thead>
<tr>
<th>Type</th>
<th>Category</th>
<th>Pressure (mb)</th>
<th>Wind speed (knots)</th>
<th>Surge (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depression</td>
<td>TD</td>
<td>-</td>
<td>&lt; 34</td>
<td>-</td>
</tr>
<tr>
<td>Tropical Storm</td>
<td>TS</td>
<td>-</td>
<td>34 - 63</td>
<td>-</td>
</tr>
<tr>
<td>Hurricane</td>
<td>1</td>
<td>&gt; 980</td>
<td>64 - 82</td>
<td>4 - 5</td>
</tr>
<tr>
<td>Hurricane</td>
<td>2</td>
<td>965 - 980</td>
<td>83 - 95</td>
<td>6 - 8</td>
</tr>
<tr>
<td>Hurricane</td>
<td>3</td>
<td>945 - 965</td>
<td>96 - 112</td>
<td>9 - 12</td>
</tr>
<tr>
<td>Hurricane</td>
<td>4</td>
<td>920 - 945</td>
<td>113 - 134</td>
<td>13 - 18</td>
</tr>
<tr>
<td>Hurricane</td>
<td>5</td>
<td>&lt; 920</td>
<td>&gt; 134</td>
<td>&gt; 18</td>
</tr>
</tbody>
</table>

#### 3.1.2 Wind

Wind data at six hourly intervals during September 1997 to July 2000 collected from NCEP/NCAR data (Kalney et al., 1996; Tolman, 1998) was used in the study. This reanalysis global wind data in $2.5^\circ \times 2.5^\circ$ grid was extracted and linearly interpolated for the $1^\circ \times 1^\circ$ grid size over South China Sea. The wind
characteristics for offshore of Hoian coast was extracted from the point of coordinate: 109°E, 16°N.

3.1.3 Tide

For the present study, tide data collected at Danang Station at hourly interval for a period of one month in September 1997 was used for the tidal analysis. Tidal Analysis Software Kit (TASK-2000) of Permanent Service for Mean Sea Level, Proudman Oceanographic Laboratory, U.K (Bell et al., 1998) was used for estimating the tidal constituents for the study region.

3.1.4 Current

Current velocity in and around the Thubon River was measured using current meters (Model DNC-2M and BMM) suspended from a vessel. The current meters use a rotor and magnetic compass to measure speed and direction respectively. Longshore current velocity and direction were measured by releasing neutrally buoyant floats in the surf zone and noting the distance covered in 2 minutes. In the present study currents was measured at different stations in and around the Thubon River mouth at each survey times. For measurement of current velocity in the offshore region and river flow the current meters were deployed at surface, mid-depth and bottom at each station. The locations of current measurement are shown in Figure 1.2 and Figure 3.5.

3.1.5 Wave

There are several ways of obtaining wave information. However, each source has its own inherent limitations. For example, in-situ wave measurement provide only spot samples and they are very expensive; remote sensing provide global coverage data, but good algorithms are required to derive wave parameters; models predict wave parameters, but they should be provided with accurate input parameters and boundary conditions. In this context, space borne measurements along with numerical modeling offer an effective way to
supplement the conventional synoptic network, and contribute substantially to coastal and offshore activities.

Statistical wind data from 1975 to 1984, and wave data from 1961 to 1982 based on the ship observed wind and wave data compiled from Vietnamese daily weather report for offshore of Central Vietnam coast was used to describe the seasonal distribution of wind and wave characteristics off Hoian coast. The wave characteristics such as significant wave height ($H_s$) and wave period were recorded using submerged pressure gauge Model AWH16M-1 (Plate 3.1).

Plate 3.1 The submerged pressure gauge (Model AWH16M-1) used for measuring wave characteristics in the Hoian area (on 22nd May 1998)

The measurement was carried out with burst interval of 60 minutes, measure interval of 0.5 s. The wave direction with respect to north was obtained by the visual observation. In the present study wave characteristics were measured at different stations in and around the Thubon River mouth at each survey times. The locations of wave measurement are shown in Figure 1.2.
3.1.6 Sediment transport

Suspended sediment concentrations in the surf zone were measured by the segmented plastic tubes (called bamboo poles). These bamboo poles were constructed and deployed according to that described by Schoonees (1991). Also streamer traps were used for measuring the sediment transport rate. Configuration of the trap was similar to that described by Kraus (1987). The opening of the trap was circular, and the diameter of the opening was 0.035 m. The filter cloth mesh opening size was 90 μm. The frame was deployed at the significant breaker point with three streamers at 0.1 m, 0.25 m and 0.4 m layer above the bed (Plate 3.2).

Plate 3.2 The streamer traps used for measuring sediment transport rate in the surf zone along Hoian coastline (on 22nd May 1998)

Measurement was carried out for 5 minutes duration of 2 hourly intervals for the period of 3 days for each station of each survey time. The total longshore transport rate was estimated following procedure described by Kumar et al. (2003a). Beach sediment samples were collected every survey time close to
waterline and the sieve analysis was carried out to study the grain size distribution.

Suspended sediment concentration was collected using a 5-l Niskin bottle. The water samples were collected from surface, mid-depth and bottom at each station. Samples were then filtered through pre-weighed filter paper of pore size 0.45 μm and diameter of 47 mm. Filtration was carried out at 25 cm. Hg vacuum and the volume of filtration varied between 500 – 1000 ml. After filtration the filters were rinsed with distilled water to remove residual salt and dried at 70 °C. The initial and final readings were taken in milligram.

3.1.7 Bathymetry

Bathymetry of South China Sea was taken from 'ETOPO' bathymetry data set of the National Geophysical Data Center, Colorado, USA (www.ngdc.noaa.gov), which covers the region between 0°N to 25°N and 100°E to 120°E and the space of 1 degree latitude/longitude has been extracted for the required region. The bathymetry of the Hoian area was taken from the hydrographic map with scale 1:100,000 published in 1980 by the Vietnamese Navy.

Bathymetry in and around Thubon River mouth was taken from the data collected during the National Project KHCN0608 in 1998 (Trinh, 2000) using an Echo sounder model F – 840 coupled with precise positioning GPS model GPS –722. Echo sounder is an instrument that works on a principle of sound waves. It is made up two parts, a transducer and a recording unit. A conical shaped acoustic beam is sent from an array of transducers. This beam reaches the sub-bottom which reflects back and is received by the same transducers. Systematic study was done through parallel traverses, spaced sufficiently close enough to portray the minute possible details of the area in and around the Thubon River mouth.
3.1.8 Shoreline positions

Shoreline positions in and around the Thubon River mouth in 1965 were extracted from the hydrographic map with scale of 1:50,000, issued in 1967 by U.S. Navy (collected data from 1965). Shoreline positions during different survey periods: September 1997, August 1998, August 1999, January 2000, July 2000, September 2001 were taken from National Project KHCN-0608 (Trinh, 2000; Trinh et al., 2003), and a limited shoreline positions during August 2003 was taken from Technical Report of the Indo-Vietnamese joint report and recommendations made during the exploratory visit of Indian delegation to the Institute of Oceanography, Nhatrang, Vietnam (ION, 2003). Shoreline positions were surveyed using a GPS (Model Fuso FGP-722) along the line of mean high water.

3.2 Computation methods

3.2.1 Hurricane wave

3.2.1.1 Young’s model

The Young’s model (Young, 1988) was used in the estimation of wave characteristics for the hurricanes considered. The input parameters to the model were the radius of maximum wind speed for the storm, \( R \); together with the maximum wind speed, \( V_{\text{max}} \), and the speed of forward motion, \( V_{\text{fm}} \). The output was the maximum significant wave height (\( H_s \)) and spectral peak period (\( T_p \)) within the storm. The JONSWAP fetch-limited growth relationships (Hasselmann et al., 1973) given below are used in the Young’s model.

\[
\frac{gH_s}{V_{\text{max}}^2} = 0.0016 \left( \frac{gF}{V_{\text{max}}^2} \right)^{0.5}
\]

\[
\frac{gT_p}{2\pi V_{\text{max}}} = 0.045 \left( \frac{gF}{V_{\text{max}}^2} \right)^{0.33}
\]

Where,
\( V_{\text{max}} \) = the 10-m wind velocity (m/s)
\[ g = \text{the acceleration of gravity (m/s}^2\text{)} \]
\[ F = \text{the fetch length (m)} \]

The speed of forward motion, \( V_{fm} \), is extracted from the data set. The equivalent fetch, \( F \), is a function of both \( V_{max} \) and \( V_{fm} \) and dependence on \( R \).

For given \( V_{fm} \), \( V_{max} \), and \( R \), an effective radius \( R' \) can be defined using a parametric model as follows:

\[ R' = 22.5 \times 10^3 \log R - 70.8 \times 10^3 \]  
(3.3)

Where both \( R \) and \( R' \) have units of meters. Using \( R' \), \( V_{fm} \), and \( V_{max} \), the equivalent fetch \( F \) is determined as follows:

\[ \frac{F}{R'} = aV_{max}^2 + bV_{max} V_{fm} + cV_{fm}^2 + dV_{max} + e V_{fm} + f \]  
(3.4)

Where,
\[ a = -2.175 \times 10^{-3} \]
\[ b = 1.506 \times 10^{-2} \]
\[ c = -1.223 \times 10^{-1} \]
\[ d = 2.190 \times 10^{-1} \]
\[ e = 6.737 \times 10^{-1} \]
\[ f = 7.980 \times 10^{-1} \]

3.2.1.2 SPM model

According to SPM (1984) for a slow moving hurricane, the deep-water significant wave height (\( H_s \)) and period (\( T_s \)) are given by,

\[ H_s = 5.03 e^{\frac{R_{RP}}{4700}} \left( 1 + \frac{0.29\alpha V_{fm}}{\sqrt{U_R}} \right) \]  
(3.5)

\[ T_s = 8.6 e^{\frac{R_{RP}}{9400}} \left( 1 + \frac{0.145\alpha V_{fm}}{\sqrt{U_R}} \right) \]  
(3.6)

Where,
\[ U_R = \text{the maximum sustained wind speed (m/s)} \]
\[ \alpha = 1.0 \text{ (for a slowly moving hurricane)} \]
\[ \Delta P = P_n - P_0 \], where \( P_n \) is the normal pressure of 760 mm of mercury, and \( P_0 \) is the central pressure of the hurricane.
3.2.1.3 Radius of maximum wind speed

Under hurricane conditions, a parameter called the radius of maximum wind, R, defined as the radial distance from the storm center to the region of maximum wind speed, has been used extensively in deep water wave studies (SPM, 1984). Simpson and Riehl (1981) described R as a well-formed inner ring of maximum wind which encircles the eye at a variable distance average perhaps 50 km, while Anthes (1982) states that the typical R = 40 km. More precisely, however, according to Harris (1958), R should be obtained through the following equations whenever pertinent data are available:

\[
\frac{P - P_0}{P_n - P_0} = \exp\left(-\frac{R}{r}\right) \tag{3.7}
\]

or

\[
R = r \ln \left(\frac{P_n - P_0}{P - P_0}\right) \tag{3.8}
\]

Where,

- \(P\) = pressure at a point located at a distance \(r\) from the storm center;
- \(P_0\) = central pressure; \(P_n\) = pressure at the outskirts of the storm.

According to Hsu and Yan (1998), to provide a more precise value of R as a function of hurricane classification so that better deep water significant wave height and period can be estimated, since the real time values of R is not always readily available. In order to do this, a comprehensive data set having both values of central pressure and R is employed. The data set is further analyzed according to the Saffir/Simpson hurricane classification. The result is shown in Table 3.2.

If the real time hurricane data such as \(P\), \(P_0\), \(P_n\), and \(r\) are not available and a quick estimate of R is needed, the composite mean for all R studied shown in the last row in the Table 3.2 (47 km) may be used.
Table 3.2 Results of mean and standard deviation for the radius of maximum wind as a function of Saffir – Simpson hurricane scale based on the data set provided in Simpson and Riedl (1981)

<table>
<thead>
<tr>
<th>Hurricane category</th>
<th>Central pressure $P_0$(mb)</th>
<th>Mean $R$ (km)</th>
<th>Standard deviation $R$ (km)</th>
<th>Number of hurricanes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$\geq 980^*$</td>
<td>34</td>
<td>13</td>
<td>5</td>
</tr>
<tr>
<td>2</td>
<td>965 - 979</td>
<td>46</td>
<td>22</td>
<td>15</td>
</tr>
<tr>
<td>3</td>
<td>945 - 964</td>
<td>51</td>
<td>26</td>
<td>27</td>
</tr>
<tr>
<td>4</td>
<td>920 - 944</td>
<td>48</td>
<td>40</td>
<td>11</td>
</tr>
<tr>
<td>5</td>
<td>$&lt; 920^**$</td>
<td>19</td>
<td>19</td>
<td>1</td>
</tr>
<tr>
<td>Composite***</td>
<td>909 - 993</td>
<td>47</td>
<td>27</td>
<td>59</td>
</tr>
</tbody>
</table>

Notes: $^*$: $P_0 \approx 986 - 993$ mb; $^**$: $P_0 \approx 909$ mb; $^***$: all 59 hurricanes between 909 and 993 mb were taken into account regardless of their $P_0$ value.

3.2.1.4 Design wave height

The design wave height for the hurricane condition for different return periods was obtained by fitting a two-parameter Weibull distribution (Goda and Kobune, 1990)

$$H_s = H_c \left( \ln \left( \frac{K_p}{r} \right) \right)^{\gamma}$$ (3.9)

Where,

$H_s$ = significant wave height (m)

$K_p$ = return period (years)

$\tau$ = time interval of observation

$H_c$ and $\gamma$ = Weibull distribution parameters
3.2.2 Offshore wave

The offshore wave climate off Hoian was computed using WAM (Wave Modeling) model (WAMDI Group, 1988; Guenther et al., 1992). WAM describes the evolution of a two-dimensional ocean wave spectrum without additional ad hoc assumptions regarding the spectral shape. It is the third generation wave model and computes the 2-d wave variance spectrum through integration of the transport equation. The model runs on a spherical latitude-longitude grid for an arbitrary region of the ocean.

3.2.2.1 Formulation of the model – deep water case

The evolution of the two-dimensional ocean wave spectrum $F(f, \theta, \phi, \lambda, t)$ with respect to frequency $f$ and direction $\theta$ (measured clockwise relative to true north) as a function of latitude $\phi$ and longitude $\lambda$ on the spherical earth is governed by the transport equation

$$\frac{\partial F}{\partial t} + (\cos \phi)^{-1} \frac{\partial}{\partial \phi} (\phi \cos \phi F) + \frac{\partial}{\partial \lambda} (\lambda F) + \frac{\partial}{\partial \theta} (\theta F) = S$$  \hspace{1cm} (3.10)

Where $S$ is the net source function describing the change of energy of a propagating wave group and

$$\dot{\phi} = \frac{d\phi}{dt} = vR^{-1} \cos \theta$$  \hspace{1cm} (3.11)

$$\dot{\lambda} = \frac{d\lambda}{dt} = v \sin \theta (R \cos \phi)^{-1}$$  \hspace{1cm} (3.12)

$$\dot{\theta} = \frac{d\theta}{dt} = v \sin \theta \tan \phi R^{-1}$$  \hspace{1cm} (3.13)

represent the rates of change of the position and propagation direction of a wave packet traveling along a great circle path.

Here, $v = g/4\pi f$ denotes the group velocity.
\( g \) = the acceleration of gravity

\( R \) = the radius of the earth

The Equations 3.10 - 3.13 apply for waves in water of infinite depth.

The generalization of the standard Cartesian geometry transport equation to the spherical geometry form (3.10) from the energy conservation equation

\[
\frac{\partial \hat{F}}{\partial t} + (\cos \phi) \frac{\partial}{\partial \phi} (\phi \cos \phi \hat{F}) + \frac{\partial}{\partial \lambda} (\lambda \hat{F}) + \frac{\partial}{\partial \theta} (\theta \hat{F}) = S
\]

for the spectral density \( \hat{F} \) (\( f, \theta, \phi, \lambda \)) with respect to the four-dimensional phase space \( (f, \theta, \phi, \lambda) \).

Here \( \hat{F} \) is related to the normal spectral density \( F \) with respect to a local Cartesian frame \((x,y)\) through

\[
\hat{F} = FR^2 \cos \phi
\]

Substitution of (3.15) into (3.14) yields (3.10).

The structure of the transport Equations (3.10) – (3.13) carries over to the finite depth. However, modifications need to be introduced in the expression for the group velocity, in the refraction Equation (3.13), and in the form of the source function.

The source function for the deep water case may be represented as a superposition of the wind input - \( S_{in} \), nonlinear transfer - \( S_{nl} \), and white capping dissipation source function - \( S_{dis} \)

\[
S = S_{in} + S_{nl} + S_{dis}
\]

(3.16)

The wind input source function was adopted from Snyder et al. (1981). However, following Komen et al. (1984), their relation was scaled in terms of the friction velocity \( \nu \), rather than the wind speed \( U_5 \) at 5 m height:

\[
S_{in} = \beta F
\]

Where,
\[
\beta = \max \left\{ 0, 0.25 \frac{\rho_a}{\rho_w} \left( \frac{28 \frac{\mu}{c} \cos \theta - 1}{\mu} \right) \right\} \omega 
\]

and \( \omega = 2\pi f \), \( \rho_a (\rho_w) \) = density of air (water).

The dissipation source function is based on the form

\[
S_{ds} = -3.33 \times 10^{-5} \bar{\omega} (\omega / \bar{\omega})^2 (\bar{\alpha} / \alpha_{pm})^2 F 
\]

proposed by Komen et al. (1984).

Where,

\[
\bar{\omega} = E^{-1} \iint F(f, \theta) \alpha d\omega d\theta 
\]

denotes the mean frequency,

\[
E = \iint F(f, \theta) d\omega d\theta 
\]

is the total energy (surface elevation variance), \( \alpha \) is an integral wave steepness parameter defined by

\[
\bar{\alpha} = E \bar{\omega}^4 g^{-2} 
\]

and

\[
\alpha_{pm} = 4.57 \times 10^{-3} 
\]

is the theoretical value of \( \bar{\alpha} \) for a Pierson-Moskowitz spectrum.

- The nonlinear source function \( S_{nl} \) was represented by the discrete interaction operator parameterization proposed by Hasselmann et al. (1985). This retains the basis form of the exact nonlinear transfer expression,

\[
S_{nl}^{\text{exact}} (k_4) = \int \omega_4 \sigma_\delta (k_1 + k_2 - k_3 - k_4) \times \delta (\omega_1 + \omega_2 - \omega_3 - \omega_4) \times n_1 n_2 (n_3 + n_4) - n_3 n_4 (n_1 + n_2)] dk_1 dk_2 dk_3
\]

where,

\( n_i = F(k_i) / \omega_i \) = the action spectrum

\( \sigma (k_1, k_2, k_3, k_4) \) = the coupling strength of a resonantly interacting wave number quadruplet \( k_1, k_2, k_3, k_4 \).

The model contains 25 frequency bands on a logarithmic scale, with \( \Delta f / f = 0.1 \), spanning a frequency range \( f_{\text{max}} / f_{\text{min}} = 9.8 \) and 12 directional bands (30° resolutions). The frequency units can be selected arbitrarily. In all hind cast studies the frequency interval extended from 0.042 to 0.41 Hz.
Beyond the high-frequency limit $f_{hf}$ of the prognostic region of the spectrum, an $f^4$ tail is added, with the same directional distribution as the last band of the prognostic region,

$$F(f, \theta) = F(f_{hf}, \theta) \left( \frac{f}{f_{hf}} \right)^4 \text{ for } f > f_{hf}$$

(3.25)

The high-frequency limit is set as

$$f_{hf} = \min\{f_{max}, \max(2.5 \bar{f}, 4f_{rms})\}$$

(3.26)

Thus, the high-frequency extent of the prognostic region is scaled for young waves by the mean frequency and for more developed wind seas by the "wind frequency" $f_{rms}$. A dynamic high-frequency cut-off, $f_{hf}$ rather than a fixed cutoff at $f_{max}$ is necessary to avoid excessive disparities in the response time scales within the spectrum.

### 3.2.2.2 Formulation of model – shallow water case

To generalize the deep-water transport Equation (3.10) to shallow water, the source function (3.16) needs to be extended to include an additional source function $S_{bf}$ representing the energy loss due to bottom friction and percolation. The other terms of the transport equation must also be suitably modified to allow for the dependence on the depth $D$ of the finite depth dispersion relation

$$\omega = (gk \tanh kD)^{1/2}$$

(3.27)

Specifically, the following changes were made:

- The additional bottom friction term was taken from the JONSWAP study (Hasselmann et al., 1973)

$$S_{bf} = -\frac{\Gamma}{g^2 \sinh^3 kD} F$$

(3.28)

With $\Gamma = \text{constant} = 0.038 \text{ m}^2 \text{ s}^{-3}$

- The infinite depth group velocity $v = \frac{1}{2} \omega / k$ in the propagation equations (3.11) – (3.13) was replaced by the corresponding expression for finite depth $D,$
\[
\nu = \frac{\partial \omega}{\partial k} = \frac{1}{2} \left( \frac{g \tanh kD}{k} \right)^{1/2} \left( 1 + \frac{2kD}{\sinh kD} \right)
\]  
\hspace{1cm} (3.29)

- Refraction due to variations of the water depth
\[
\dot{\theta}_D = \frac{1}{kR} \frac{\partial \omega}{\partial D} \left( \sin \theta \frac{\partial D}{\partial \phi} - \cos \theta \frac{\partial D}{\partial \lambda} \right)
\]  
\hspace{1cm} (3.30)

### 3.2.2.3 Numerical implementation

Different numerical techniques and time steps were used to integrate the source functions and the advective terms of the transport equation.

- Implicit integration of the source functions:

The implicit second-order, centered difference equations (leaving out the advection terms) are given by

\[
F_{n+1} = F_n + \frac{\Delta t}{2} \left( S_{n+1} + S_n \right)
\]  
\hspace{1cm} (3.31)

where \( \Delta t \) is the time step and the index \( n \) refers to the time level.

- Propagation:

Two alternative propagation schemes were implemented in the model:

\( + \) First-order upwind scheme

\[
F_j^{n+1} = F_j^n - \sum \frac{\Delta t}{\Delta x_k \cos \theta_j} \left[ (u \cos \phi F^n)_j - (u \cos \phi F^n)_{k+} \right]
\]  
\hspace{1cm} (3.32)

\( + \) Second order leapfrog scheme

\[
F_j^{n+1} = F_j^{n-1} - \sum \frac{\Delta t}{2 \Delta x_k \cos \phi_j} \left[ (u \cos \phi F^n)_{k+} - (u \cos \phi F^n)_{k-} \right] + \text{diffusion} \hspace{1cm} (3.33)
\]

in equations (3.32) and (3.33) the index \( n \) refers to the time level and the indices \( k_-, k_+ \) to the neighboring grid points in the upstream and downstream propagation directions, respectively, relative to the reference grid point \( j \). The index \( k \) runs over the three propagation directions \( \lambda, \phi, \theta \), and \( u_k, \Delta x_k \) denote the velocity component \( (\lambda, \phi, \theta) \) and grid spacing, respectively, in the relevant direction.
Input data for WAM model was bathymetry over South China Sea and wind data sets. The influence of currents on the wave is not considered in the present study. The computational domain cover the region between 0°N to 25°N and 100°E to 120°E with resolution of 1° x 1°. The grid system for WAM model over South China Sea is shown in Figure 3.1
3.2.3 Nearshore wave

The wave characteristics in the Hoian area was computed using SWAN (Simulating WAves Nearshore) which is a third – generation wave model (Booij, et al., 1999; Ris, et al., 1999; Holthuijsen et al., 2003) with which realistic estimates of wave parameters in coastal areas, lakes and estuaries from given wind, bottom, and current conditions can be obtained.

3.2.3.1 Action balance equation

The evolution of the wave spectrum is described by the spectral action balance equation, which, for Cartesian coordinate, is (Hasselmann et al., 1973)

\[
\frac{\partial N}{\partial t} + \frac{\partial}{\partial x}(C_x N) + \frac{\partial}{\partial y}(C_y N) + \frac{\partial}{\partial \sigma}(C_\sigma N) + \frac{\partial}{\partial \theta}(C_\theta N) = \frac{S}{\sigma}
\]  

(3.34)

The first term in the left hand side of (3.34) represents the local rate of change of action density (N) in time, the second and third term represent propagation of action in geographical space (with propagation velocities C_x and C_y in x and y space, respectively). The fourth term represents shifting of the relative frequency due to variations in depths and currents (with propagation velocity C_\sigma in \sigma space). The fifth term represents depth-induced and current-induced refraction (with propagation velocity C_\theta in \theta space). The term S = S(\sigma,\theta) at the right hand side of the action balance equation is the source term in term of energy density representing the effects of generation, dissipation and nonlinear wave-wave interactions.

3.2.3.2 Wind input

Transfer of wind energy to the waves is described with the resonance mechanism of Phillips (1957) and the feedback mechanism of Miles (1957). The corresponding source term for these mechanisms is commonly described as the sum of linear and exponential growth:
\( S_{in}(\sigma, \theta) = A + BE(\sigma, \theta) \) \hspace{1cm} (3.35)

In which A (linear growth) and B (exponential growth) depend on wave frequency and direction and wind speed and direction. The effects of currents are accounted by using the apparent local wind speed and direction.

*\( > \) Linear growth by wind:

\[ A = \frac{1.5 \times 10^{-3}}{g^2 2\pi} \left[ U, \text{max}[0, \cos(\theta - \theta_c)] \right] H \] \hspace{1cm} (3.35a)

Where

\[ H = \exp \left( - \left( \frac{\sigma}{\sigma_{pm}^*} \right)^4 \right) \text{ with } \sigma_{pm}^* = \frac{0.13g}{28U} \]

in which,

\( \theta_c \) = wind direction

\( H \) = the filter

\( \sigma_{pm}^* \) = peak frequency of the fully developed sea state according to Pierson and Moskowitz (1964).

*\( > \) Exponential growth by wind:

According to Komen et al. (1984)

\[ B = \max \left[ 0, 0.25 \frac{\rho_a}{\rho_w} \left[ 28 \frac{U_s}{C_{ph}} \cos(\theta - \theta_c) - 1 \right] \sigma \right] \] \hspace{1cm} (3.35b)

in which,

\( C_{ph} \) = the phase speed

\( \rho_a, \rho_w \) = density of air and water, respectively.

### 3.2.3.3 Wave energy dissipation

The dissipation term of wave energy is represented by the summation of three different contributions: white capping \( S_{ds,w}(\sigma, \theta) \), bottom friction \( S_{ds,b}(\sigma, \theta) \), and depth-induced breaking \( S_{ds,b}(\sigma, \theta) \).

- White capping is primarily controlled by the steepness of the waves as follow,
\[ S_{ds,w}(\sigma, \theta) = -\Gamma \sigma \frac{k}{k} E(\sigma, \theta) \quad (3.36) \]

Where, \( \Gamma \) = steepness dependent coefficient

\( k \) = wave number

\( \bar{\sigma}, \bar{k} \) = mean frequency and mean wave number, respectively.

- Bottom-induced dissipation may be caused by bottom friction, bottom motion, percolation, or backscattering on bottom irregularities. For continental shelf seas with sandy bottoms, the dominant mechanism appears to be bottom friction, which can generally be represented as

\[ S_{ds,b}(\sigma, \theta) = -C_{\text{bottom}} \frac{\sigma^2}{g^2 \sinh^2(kd)} E(\sigma, \theta) \quad (3.37) \]

In which, \( C_{\text{bottom}} \) = bottom friction coefficient

- Depth-induced wave breaking is still poorly understood and little is known about its spectral modeling

\[ S_{ds,br}(\sigma, \theta) = \frac{S_{ds,br,tot}}{E_{\text{tot}}} E(\sigma, \theta) \quad (3.38) \]

In which, \( E_{\text{tot}} \) = total wave energy

\( S_{ds,br,tot} \) = the rate of dissipation of \( E_{\text{tot}} \) due to depth-induced wave breaking.

The value of \( S_{ds,br,tot} \) depends critically on the breaking parameter \( \gamma = H_{\text{max}}/d \) (\( H_{\text{max}} \) is the maximum wave height in the local water depth \( d \)), in SWAN a constant value \( \gamma = 0.73 \).

### 3.2.3.4 Nonlinear wave-wave interactions

In deep water, quadruplet wave-wave interactions dominate the evolution of the spectrum. They transfer wave energy from the spectral peak to lower frequencies (thus moving the peak frequency to lower values) and to higher frequencies (where the energy is dissipated by white capping). In very shallow water, triad wave-wave interactions transfer energy from lower frequencies to higher frequencies, often resulting in higher harmonics.
The discrete triad approximation is used in SWAN in each spectral direction:

\[ S^-_{n3}(\sigma, \theta) = S^+_{n3}(\sigma, \theta) + S^{-+}_{n3}(\sigma, \theta) \]  \hspace{1cm} (3.39)

with

\[ S^+_{n3}(\sigma, \theta) = \max \left \{ 0, \alpha_{EB} 2 \nu c \omega e^J \sin(\beta) \left[ E^2(\sigma / 2, \theta) - 2 E(\sigma / 2, \theta) E(\sigma, \theta) \right] \right \} \]  \hspace{1cm} (3.40)

\[ S^+_{n3}(\sigma, \theta) = -2 S^+_{n3}(2\sigma, \theta) \]  \hspace{1cm} (3.41)

where, \( \alpha_{EB} \) is a tunable proportionality coefficient

\[ \beta = -\frac{\pi}{2} + \frac{\pi}{2} \tanh \left( \frac{0.2}{U_r} \right) \]  \hspace{1cm} (3.42)

where, \( U_r = \text{Ursell number} \)

\[ U_r = \frac{g}{8 \sqrt{2 \pi^2}} \frac{H \bar{T}^2}{d^2} \]  \hspace{1cm} (3.43)

where, \( \bar{T} = \frac{2 \pi}{\sigma} \), \( J = \text{the interaction coefficient} \)

\[ J = \frac{k_{\sigma J}^2 (gd + 2c_{\sigma J}^2)}{k_o d \left( gd + \frac{2}{15} gd^3 k_o^2 - \frac{2}{5} \sigma^2 d^2 \right)} \]  \hspace{1cm} (3.43a)

\subsection{3.2.3.5 Numerical implementation}

The integration of the action balance equation has been implemented in SWAN with finite difference schemes in all five dimensions (time, geographic space, and spectral space). The numerical propagation schemes in SWAN are implicit as follow:

\[
\left[ \frac{N_{i+1, n} - N_{i, n-1}}{\Delta t} \right]_{i, j, l, m, p} + \left[ \frac{[c_x N]_{i, n} - [c_x N]_{i, n-1}}{\Delta x} \right]_{i, j, l, m, p} + \left[ \frac{[c_y N]_{i, j+1, n} - [c_y N]_{i, j, n-1}}{\Delta y} \right]_{i, j, l, m, p} \\
+ \left[ \frac{(1-\nu)[c_\sigma N]_{i, j+1, n+1} + 2\nu[c_\sigma N]_{i, j+1, n} - (1+\nu)[c_\sigma N]_{i, j+1, n-1}}{2\Delta \sigma} \right]_{i, j, l, m, p} \\
+ \left[ \frac{(1-\eta)[c_\theta N]_{i, j+1, n+1} + 2\eta[c_\theta N]_{i, j+1, n} - (1+\eta)[c_\theta N]_{i, j+1, n-1}}{2\Delta \theta} \right]_{i, j, l, m, p} = \frac{S^-_{n3}}{\sigma} \]  \hspace{1cm} (3.44)
The SWAN model accounts for shoaling, refraction, generation by wind, white-capping, triad and quadruplet wave-wave interactions, bottom friction, and depth-induced wave breaking. The input data for SWAN model was wave boundary conditions, which was taken from the results of WAM model at location of $109^\circ E$, $16^\circ N$ (Figure 3.1), and local wind fields. The influence of currents on the wave is not considered in the present study.

Size of the computational grid was $45 \times 47.75$ km, with resolution of $250 \times 250$ m, which covers the area from $108.24^\circ E$ to $108.645^\circ E$ and from $15.695^\circ N$ to $16.124^\circ N$ (Figure 3.2). The spectral frequency ($f$) range was $0.052 - 1$ Hz with $\Delta f = 0.01$ Hz and range in direction ($\theta$) was $0$ to $360^\circ$ with $\Delta \theta = 10^\circ$.

Figure 3.2 Grid system for SWAN model in the Hoian area (Grid spacing: $DX = DY = 250$ m)
3.2.4 Sediment transport

3.2.4.1 CERC method

The longshore sediment transport rate is usually estimated from an empirical equation relating the longshore energy flux in the breaker zone to the longshore transport rate (Komar and Inman, 1970). This is known as Coastal Engineering Research Center (CERC) formula (SPM, 1984). Almost in all studies, the longshore transport rate has been attempted to relate with longshore energy flux in the form,

\[ Q = K \cdot P_l \]  

(3.45)

Where, \( Q \) = volume rate of longshore sediment transport

- \( K \) = constant
- \( P_l \) = longshore component of wave power at breaking = \( P \cos \alpha_b \sin \alpha_b \)
- \( P \) = wave power at breaking = \( (E C_n)_b \)
- \( \alpha_b \) = angle between wave crest and shoreline
- \( E \) = wave energy
- \( C_n \) = group celerity

and suffix \( b \) denotes at breaker zone.

Using the value of \( P_l \), Eqn. (3.45) can be rewritten as,

\[ Q = K (E C_n) \cos \alpha_b \sin \alpha_b \]  

(3.46)

The volume rate of longshore transport is related to immersed weight transport rate (Inman and Bagnold, 1963) as,

\[ I = (\rho_s - \rho) g (1- \rho) Q \]  

(3.47)

Where, \( I \) = immersed weight sediment transport rate

- \( \rho_s \) = density of the sediment
- \( \rho \) = density of sea water
- \( \rho \) = porosity of the sediment
- \( g \) = acceleration due to gravity (9.81 m/s\(^2\))

Komar and Inman (1970) made extensive field measurements and proposed the following approach for the prediction of longshore transport. The immersed
weight rate takes into consideration of the density of sediment and permits the I
and P to be of the same unit.

\[ I = K \left( E C_n \right)_b \cos \alpha_b \sin \alpha_b \]  

(3.48)

Where, \( K = \) dimensionless constant \( = 0.77 \) and the root mean square wave
height \( (H_{rms}) \) is used in the Eqn. (3.48).

Using Eqn. (3.47) in Eqn. (3.48) (Perlin, 1979),

\[ Q = \frac{0.77 \rho g}{16(\rho_s - \rho)(1 - \rho)g} H_b^2 C_b \sin 2\alpha_b \]  

(3.49)

Using the following values in the above equation,

\( \rho = 1025 \ \text{kg/m}^3, \ \rho_s = 2650 \ \text{kg/m}^3, \ p = 0.4, \) and the significant wave height instead
of \( H_{rms} \) \( (H_s = 1.414 \ H_{rms}) \), volume rate of longshore transport rate \( (Q) \) in \( \text{m}^3/\text{year} \)
is,

\[ Q = 0.79775 \times 10^6 (H_s)^2 C_b \sin 2\alpha_b \]  

(3.50)

Using the shallow water approximations, \( C_b = \left( g d_b \right)^{0.5} \) and \( d_b = 1.28 H_b \) in the
above equation,

\[ Q = 2.8269 \times 10^6 H_b^{2.5} \sin 2\alpha_b \]  

(3.51)

The above equation does not consider the wave period. At the present study
another form of the SPM equation (SPM, 1984) taking into account the wave
period is as given below.

\[ Q = K A \frac{\rho^2 g^2}{64 \pi} T H_b^2 \sin 2\alpha_b \]  

(3.52)

Where,

\( Q = \) volume of longshore transport rate in \( \text{m}^3/\text{day} \)

\( K = \) dimensionless constant relating sand transport to longshore energy flux \( = 0.39 \)

\[ A = \frac{1}{(\rho_s - \rho)g(1 - \rho)} \]  

(3.53)

\( T = \) wave period in s

\( H_b = \) breaking wave height in m
### 3.2.4.2 Van Rijn formula

Sediment transport rate was also calculated using Van Rijn formula (Van Rijn, 1990 and 1991), for a fine sand bed (50 to 500 µm) the total sediment transport rate can be computed from the vertical distribution of fluid velocities and sediment concentration, as follow:

\[
q_t = \int_0^{h+} U C dz
\]  
\[(3.54)\]

in which:

- \( q_t \) = total sediment transport rate (kg/m/s)
- \( U \) = local instantaneous fluid velocity at height \( z \) above bed (m/s)
- \( C \) = local instantaneous sediment concentration at height \( z \) above bed (kg/m\(^3\))
- \( h \) = water depth (to mean surface level - m)
- \( \eta \) = water surface elevation (m)

Defining: \( U = u + \bar{u} \) and \( C = c + \bar{c} \)  
\[(3.55)\]

in which:

- \( u \) = time and space-average fluid velocity at height \( z \) (m/s)
- \( c \) = time and space-average concentration at height \( z \) (kg/m\(^3\))
- \( \bar{u} \) = oscillating fluid component (including turbulent component)
- \( \bar{c} \) = oscillating concentration component (including turbulent component)

Substituting Eq. (3.55) in Eq. (3.54) and averaging over time and space, yields:

\[
\bar{q}_t = \int_0^h u c dz + \int_0^h \bar{u} \bar{c} dz = \bar{q}_c + \bar{q}_w
\]  
\[(3.56)\]

Where,

- \( \bar{q}_c = \int_0^h u c dz \) = time-averaged current-related sediment transport rate
- \( \bar{q}_w = \int_0^h \bar{u} \bar{c} dz \) = time-averaged wave-related sediment transport rate
Computation of time-average concentration profiles

For plane bed conditions with sheet flow, the concentration profile can be obtained from numerical integration of the time-averaged convection-diffusion equation:

\[ \frac{d\bar{c}}{dz} + \frac{\alpha_s}{W_{s,m}} \frac{dc}{dz} = 0 \]  

(3.57)

where:

- \( W_{s,m} \) = particle fall velocity of suspended sediment in a fluid-sediment mixture.
- \( \alpha_s \) = sediment mixing coefficient.
- \( \bar{c} \) = time-averaged concentration at height \( z \) above the bed.

Wave-related sediment transport

The time-averaged transport rate over half the wave period in non-breaking waves \( \bar{q}_w \) (m\(^2\)/s) as follow,

\[ \bar{q}_w = \alpha_1 \ a \ c_a \ \frac{\hat{U}_\delta}{\delta_w} = \alpha_2 \ d_{50} \ \frac{\hat{U}_\delta}{\delta_w} \ T^{1.5} \ D_s^{-0.3} \]  

(3.58)

where,

- \( T \) = dimensionless bed-shear stress parameter
- \( D_s \) = dimensionless particle parameter
- \( \hat{U}_\delta \) = peak value of near-bed orbital velocity
- \( d_{50} \) = median particle diameter of bed material
- \( a = \delta_w \) = reference level (= wave boundary layer thickness)
- \( c_a \) = reference concentration in near bed region
- \( \alpha_2 \) = calibration coefficient = 0.03

The net time-average wave-induced transport rate in asymmetrical oscillatory motion is given by:

\[ \bar{q}_{w,net} = \bar{q}_{w,max} - \bar{q}_{w,min} = \alpha_1 d_{50} D_s^{-0.3} \left[ \frac{\hat{U}_{\delta,max} T_{max}^{1.5} - \hat{U}_{\delta,min} T_{min}^{1.5}}{T_{max}} \right] \]  

(3.59)
Current-related bed-load transport

\[ \bar{q}_{b,c} = 0.25 u'_{c} d_{50} T^{1.5} D^{-0.3} \]  
(3.60)

where,

- \( \bar{q}_{b,c} \) = time-average bed-load transport (m\(^3\)/s)
- \( u'_{c} \) = current-related grain bed-shear velocity (m/s)
- \( d_{50} \) = median particle diameter of bed material (m)
- \( T \) = dimensionless bed-shear stress parameter due to current and waves
- \( D \) = dimensionless particle parameter
- \( C' = 18 \log(12h/3d_{90}) \) = grain-related Chezy coefficient (m\(^{0.5}\)/s)

Current-related suspended load transport

\[ \bar{q}_{s,c} = \int_{a}^{h} v_{R} c dz \]  
(3.61)

in which:

- \( \bar{q}_{s,c} \) = time-averaged suspended load transport (m\(^3\)/s)
- \( v_{R} \) = resultant current velocity at height \( z \) above the bed (m/s)
- \( c \) = sediment concentration at height \( z \) above bed
- \( a \) = reference level (m)
- \( h \) = water depth (m)

Total sediment transport

\[ \bar{q}_{r} = \left[ (\bar{q}_{c})^2 + (\bar{q}_{w})^2 + 2|\bar{q}_{c}|\bar{q}_{w}|\cos\phi| \right]^{0.5} \]  
(3.62)

in which:

- \( \bar{q}_{r} \) = total current-related transport rate
- \( \bar{q}_{c} \) = current-related bed load transport rate
- \( \bar{q}_{s,c} \) = current-related suspended load transport rate
- \( \bar{q}_{w} \) = net wave-relate sediment transport rate in the direction of the largest bed shear stress
\( \Phi \) = angle between current direction and wave propagation direction.

The input parameters considered in the Van Rijn formula are given below.

\( D_{50} \) = median particle size of bed = 0.175 mm
\( a \) = reference level = 0.01 m
\( RC \) = current related roughness = 0.05 m
\( RW \) = wave related roughness = 0.05 m

### 3.2.4.3 Net longshore sediment transport rate

Since the movement of longshore sediment transport is parallel to the shoreline, there are two possible direction of motion, right to left, relative to an observer standing on the shore looking out to sea. Movement from the observer's right to his left is motion towards the left, indicated by the subscript 'lt'; movement towards the observer's right is indicated by the subscript 'rt'. Gross longshore transport rate, \( Q_g \), is the sum of the amount of littoral drift transport to the right and to the left, past a point on the shoreline in a given time period (SPM, 1984).

\[
Q_g = Q_{rt} + Q_{lt}
\]  
(3.63)

Similarly, net longshore transport rate, \( Q_n \), is defined as the difference between the amounts of littoral drift transported to the right and to the left pass a point on the shoreline in a given time period.

\[
Q_n = Q_{rt} - Q_{lt}
\]  
(3.64)

The quantities \( Q_{rt} \), \( Q_{lt} \), \( Q_n \), and \( Q_g \) have engineering uses: for example, \( Q_g \) is used to predict shoaling rates in uncontrolled inlets. \( Q_n \) is used for design of protected inlets and for predicting beach erosion on an open coast; \( Q_{rt} \) and \( Q_{lt} \) are used for design of jetties and impoundment basins behind weir jetties. In addition \( Q_g \) provides an upper limit on other quantities.

Longshore transport rates are usually given in units of volume per time (cubic meters per year).
3.2.5 Shoreline change

Shoreline change caused primarily by wave action is calculated by GENESIS model. GENESIS (GENEralized model for Simulating Shoreline change) simulates changes in position of the shoreline in response to wave action and boundary conditions (Hanson, 1989; Gravens et al., 1991; Hanson and Kraus, 1991; Kraus et al., 1985) which is based on "one-line" theory.

The fundamental assumption of the one-line model is:
- The beach profile shape is constant.
- The shoreward and seaward limits of the profile are constant.
- Sand is transported alongshore by the action of breaking waves.
- The detailed structure of the nearshore circulation is ignored.
- There is a long-term trend in shoreline evolution.

2.2.5.1 Governing equation for shoreline change

Definition sketch for shoreline change calculation is shown in Figure 3.3, and the governing equation for the rate of change of shoreline position is formulated by conservation of sand volume as

\[
\frac{\partial y}{\partial t} + \frac{1}{(D_B + D_C)} \frac{\partial Q}{\partial x} = q
\]

(3.65)

Where,
\( y = \) shoreline position (m); \( x = \) distance alongshore (m); \( t = \) time interval (h)
\( D_B = \) the berm elevation (m); \( D_C = \) the depth of closure (m); \( Q = \) the longshore transport rate (m\(^3\)/s); \( q = \) the rate of source or sink of sand (m\(^3\)/s)

In order to solve Equation (3.65), the initial shoreline position over the full reach to be modeled, boundary conditions on each end of the beach, and values for \( Q, q, D_B, \) and \( D_C \) must be given.
3.2.5.2 Sand transport rate

The empirical predictive formula for the longshore sand transport rate \((Q)\) used in GENESIS is given below.

\[
Q = \left( H^2 C_g \right) b \left[ a_1 \sin \theta_{bs} - a_2 \cos \theta_{bs} \frac{\partial H}{\partial x} \right] \tag{3.66}
\]
Where,

- \( H \) = wave height (m);
- \( C_g \) = wave group speed given by linear wave theory (m/s)
- \( b \) = subscript denoting wave breaking condition
- \( \theta_{bs} \) = angle of breaking waves to the local shoreline

The non-dimensional parameters \( a_1 \) and \( a_2 \) are given by

\[
a_1 = \frac{K_1}{16(\rho_s / \rho - 1)(1 - p)(1.416)^{3/2}} \tag{3.67}
\]

\[
a_2 = \frac{K_2}{8(\rho_s / \rho - 1)(1 - p)\tan(\beta)(1.416)^{3/2}} \tag{3.68}
\]

where,

- \( K_1, K_2 \) = empirical coefficient, treated as a calibration parameters
- \( \rho_s \) = density of sand (taken to be 2650 kg/m\(^3\) for quartz sand)
- \( \rho \) = density of water (1030 kg/m\(^3\) for seawater);
- \( p \) = porosity of sand on the bed (taken as 0.4)
- \( \tan(\beta) \) = average bottom slope from the shoreline to the depth of active longshore sand transport.

The factor 1.416 is used to convert significant wave height, the statistical wave height required by GENESIS, to root-mean-square (rms) wave height.

The first term in Equation (3.66) corresponds to the “CERC formula” (SPM, 1984). The second term is used to describe the effect of the longshore gradient in breaking wave height.

### 3.2.5.3 Internal wave transformation model

The modeling system GENESIS is composed of two major submodels. One submodel calculates the longshore sand transport rate and shoreline change. The other submodel is a wave model that calculates, under simplified conditions, breaking wave height and angle alongshore as determined from wave information given at a “nearshore reference line”. This submodel is called the internal wave transformation model.
- There are three unknowns in the breaking wave calculation: the wave height, wave angle, and depth at breaking.

\[ H_2 = K_R K_S H_{ref} \]  \hspace{1cm} (3.69)

Where,

- \( H_2 \) = breaking wave height
- \( K_R \) = refraction coefficient
- \( K_S \) = shoaling coefficient
- \( H_{ref} \) = wave height at the nearshore reference line

\[ K_R = \left( \frac{\cos \theta_1}{\cos \theta_2} \right) \]  \hspace{1cm} (3.70)

Where,

- \( \theta_1 \) = wave angle at the nearshore reference line.
- \( \theta_2 \) = wave angle at the breaker depth.

\[ K_S = \left( \frac{C_{g1}}{C_{g2}} \right)^{1/2} \]  \hspace{1cm} (3.71)

Where,

- \( C_{g1} \) and \( C_{g2} \) = wave group speeds at the nearshore reference line and the initial break point, respectively. The group speed is defined as

\[ C_g = C_n \]  \hspace{1cm} (3.72)

Where,

- \( C \) = wave phase speed = \( L/T \)
- \( L \) = wavelength at the depth \( D \)
- \( n = 0.5[1+(2\pi D/L)/\sinh(2\pi D/L)] \)

- The wavelength is calculated from the dispersion relation,

\[ L = L_o \tanh \left[ \frac{2\pi D}{L} \right] \]  \hspace{1cm} (3.73)

- The equation for depth-limited wave breaking is given by

\[ H_b = \gamma D_b \]  \hspace{1cm} (3.74)

Where,

- \( D_b \) = depth at breaking
- \( \gamma \) = breaker index
\[ \gamma = b - a \frac{H_0}{L_0} \]  

(3.75)

Where,
\[ a = 5[1 - \exp(-43 \tan\beta)] \]
\[ b = 1.12/[1 + \exp(-60 \tan\beta)] \]
\[ \tan\beta = \text{beach slope} \]

- The wave angle at breaking is calculated by means of Snell's law,
\[ \frac{\sin\theta_b}{L_b} = \frac{\sin\theta_1}{L_1} \]  

(3.76)

Where,
\[ \theta_b, L_b = \text{angle and wavelength at break point} \]
\[ \theta_1, L_1 = \text{angle and wavelength at an offshore point} \]

The three unknowns, \( H_b, D_b, \) and \( \theta_b \), are obtained at interval alongshore by iterative solution of Equations 3.69, 3.74, and 3.76 as a function of the wave height and angle at the reference depth and the wave period.

3.2.5.4 Numerical solution scheme

- Referring to Figure (3.3) and the shoreline change Equation (3.65) the change in position of the shoreline can be mathematically written as
\[ \Delta y = -\frac{\Delta t}{(D_b + D_c)} \frac{\Delta Q}{\Delta x} \]  

(3.77)

where,
\[ \Delta Q = \text{difference in longshore sand transport rates at the walls of the cell} \]

- Numerical stability: the allowable grid spacing and time step of a finite difference numerical solution of a partial differential equation such as Eq. (3.65) depend on the type of solution scheme. Under certain idealized conditions, Eq. (3.65) can be reduced to a simpler form to examine the dependence of the solution on the time and space steps. The main assumption needed is that the angle \( \theta_{bs} \) in Eq. (3.66) is small. In this case, \( \sin2\theta_{bs} \approx 2\theta_{bs} \), and \( \theta_{bs} = \theta_b - \partial y/\partial x \)
Where, \( \theta_b \) = angle between wave breaking with x-axis in the fixed coordinate system, since the inverse tangent can be replaced by its argument if the argument is small.

The derivative of \( Q \) with respect to \( x \) is required (Eq. (3.65)) and, under the small-angle approximation, \( \partial Q / \partial x \sim \partial (2 \theta_b \theta_v) / \partial x \sim 2 \theta^2 \theta_v^2 / \partial x^2 \), if it is assumed that \( \theta_b \) does not change with \( x \). After some algebraic manipulation, Eq. (3.65) can be expressed as (Kraus and Harikai, 1983):

\[
\frac{\partial y}{\partial t} = (\varepsilon_1 + \varepsilon_2) \frac{\partial^2 y}{\partial x^2}
\]

(3.78)

where,

\[
\varepsilon_1 = \frac{2K_1}{(D_B + D_C)} (H^2 C_g)_b
\]

(3.79)

and

\[
\varepsilon_2 = \frac{2K_2}{(D_B + D_C)} [H^2 C_g \cos \theta_b \frac{\partial H}{\partial x}]_b
\]

(3.80)

As Eq. (3.78) is a diffusion-type equation, its stability properties are well known. The numerical stability of the calculation scheme is governed by:

\[
R_s = \frac{\Delta t (\varepsilon_1 + \varepsilon_2)}{(\Delta x)^2}
\]

(3.81)

The quantity \( R_s \) is known as the Courant number in numerical method; here it is called the stability parameter. The finite difference form of Eq. (3.78) shows that

\[
\Delta y \sim \Delta y (\Delta x)^2
\]

Equation (3.65) or Equation (3.78) can be solved by either an explicit or implicit solution scheme. If solved using an explicit scheme, the new shoreline position for each of the calculation cells depends only on values calculated at the previous time step. The main advantages of the explicit scheme are easy programming, simple expression of boundary conditions, and shorter computer run time for a single time step as compared with the implicit scheme. A major disadvantage is, however, preservation of stability of the solution, imposing a severe constraint on the longest possible calculation time step for given values.
on model constants and parameters. If an explicit solution scheme is used to solve the diffusion equation, the following condition must be satisfied (Crank, 1975)
\[ R_s \leq 0.5 \] (3.82)
Equation (3.65), of which Equation (3.78) is a special case, can also be solved using an implicit scheme in which the new shoreline position depends on values calculated on the old, as well as the new, time step. The main advantage of the implicit scheme is that it is stable for very large values of \( R_s \). The disadvantages of the implicit solution scheme are that the program, boundary conditions, and constraints become more complex, as compared with the explicit scheme.

An implicit finite difference solution scheme is used in GENESIS to solve Equation (3.65), a subscript \( i \) denotes a quantity located at an arbitrary cell number \( i \) along the beach. A prime (') is used to denote a quantity at the new time level, whereas an unprimed quantity indicates a value at the present time step, which is known. The quantities \( y' \) and \( Q' \) are not known and are being sought in the solution process; other primed quantities such as \( q' \) and \( D_B' \) refer to data at the next time step and are known.

The Crank-Nicholson implicit scheme is used (Crank, 1975) in which the derivative \( \partial Q / \partial x \) at each grid point is expressed as an equally weighted average between the present time step and the next time step:
\[
\frac{\partial Q_i}{\partial x} = \frac{1}{2} \left[ \frac{Q_{i+1} - Q_i}{\Delta x} + \frac{Q_{i+1} - Q_i}{\Delta x} \right]
\] (3.83)
Substitution of Equation (3.83) into Equation (3.65) and linearization of the wave angle in Equation (3.66) in term of \( \partial y / \partial x \) results in two systems of coupled equations for the unknown's \( y_i' \) and \( Q_i' \):
\[
y_i' = B'(Q_i' - Q_{i+1}') + y_{c_i}
\] (3.84)
and
\[
Q_i' = E_i (y_{i+1}' - y_i') + F_i
\] (3.85)
Where
\[
B' = \Delta t \left[ 2(D_s + D_e') \Delta x \right]
\] (3.86)
y_{c_i} = function of known quantities, including \( q_i' \) and \( q_i \)
Figure 3.4 shows the example of a coordinate system and grid used by GENESIS model.

\[ E_i = \text{function of the wave height, wave angle, and other known quantities} \]
\[ F_i = \text{function similar to } E_i \]

- The concerned shoreline around the Thubon River mouth is divided into three project reaches and modeled each separately. Three model reaches considered are, the first north of the Thubon River mouth (Northern shoreline reach), second south of the Thubon River mouth (Southern shoreline reach) and the third south bank of Thubon River mouth (River bank reach).

GENESIS model boundary conditions used as follows: on the left boundary of the Northern shoreline, left boundary of the River bank, and right boundary of
the Southern shoreline used "pinned-beach" boundary condition allowing sand to freely cross it from both sides (the sediment budget is balanced); on the right boundary of the Northern shoreline, right boundary of the River bank, and left boundary of the Southern shoreline used "gated" boundary condition (the boundary influences the transport rate). Figure 3.5 shows the details of GENESIS model reaches, boundary conditions, and associate nearshore reference depths.

Figure 3.5 Shoreline change model boundaries
Empirical parameters used in GENESIS are shown in Table 3.3

<table>
<thead>
<tr>
<th>Empirical Parameters</th>
<th>Northern shoreline</th>
<th>Southern shoreline</th>
<th>River bank</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transport parameter - $K_1$</td>
<td>0.15</td>
<td>0.125</td>
<td>0.3</td>
</tr>
<tr>
<td>Transport parameter - $K_2$</td>
<td>0.10</td>
<td>0.10</td>
<td>0.2</td>
</tr>
<tr>
<td>Depth of closure - $D_c$ (m)</td>
<td>6</td>
<td>6</td>
<td>5</td>
</tr>
<tr>
<td>Average berm height - $D_b$ (m)</td>
<td>2</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Effective grain size - $D_{50}$ (mm)</td>
<td>0.175</td>
<td>0.175</td>
<td>0.175</td>
</tr>
<tr>
<td>Alongshore grid spacing $\Delta X$ (m)</td>
<td>50</td>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>Input wave data time step - $\Delta t$ (hrs)</td>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
</tbody>
</table>

3.2.5.6 Computation of shoreline change by wave and river flow action

GENESIS only allows computation of shoreline change by single wave action. Therefore, in order to calculate shoreline change along River bank by the combined of river flow and wave action the following steps were carried out:

- Wave characteristics along nearshore reference depth of River bank were taken from SWAN model, and is transferred in to the breaking zone by internal wave transformation model of GENESIS (Eqs. 3.69, 3.74, and 3.76).
- River flow velocity along River bank were calculated from average monthly measured river discharge and limited measured river flow velocity during project survey periods; also surf zone width were taken during project survey periods by visual (Hung, 1995; Trinh, 2000).
- Longshore sediment transport rate was estimated by Van Rijn formula in the combined wave and river flow action (Van Rijn, 1991).
- Shoreline position change was estimated using GENESIS formula (Eq. 3.77), and all input parameters were kept similarly as previous work in computation of shoreline change under single wave action.
The influence of tidal currents on wave and sediment transport was not considered in the present study.

The overall calculation flow used in the present study is shown in Figure 3.6.

Figure 3.6 Overall calculation flow used.