CHAPTER ONE HUNDRED FORTY ONE

SHORELINE CHANGE AT OARAI BEACH:
PAST, PRESENT AND FUTURE

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ABSTRACT

Large breakwaters and groins are being constructed at Oarai Harbor, Japan. As a result the beach is significantly deforming. The first part of this paper documents past and recent shoreline change at Oarai. The general characteristics of the offshore waves, breaking waves, and longshore current pattern are described and used to explain qualitative features of the observed shoreline change. The second part presents results of numerical simulations of shoreline change at the site which occurred over different time periods. The model includes three sources of wave diffraction, a rigorous formulation of the seawall boundary condition, and sand bypassing at groins. The modeling of historical shoreline change was reasonably successful. As an exercise in investigating problems associated with prediction, the model was used to forecast the shoreline position at the site five years from now. The prediction of the wave history was the main problem encountered. A simple intuitive method was devised to estimate the probable range in variation of the wave history, and the results are discussed in connection with the shoreline forecast.

1. INTRODUCTION

Since 1977, the fishing harbor at Oarai, Ibaraki Prefecture, Japan, has been undergoing expansion for conversion to a ferry and cargo port, to be completed around 1990. Figure 1 shows past and planned construction. Extension of the harbor breakwater and construction of the large detached breakwater for improvement of navigation have resulted in considerable changes along the sandy beach to the south.

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Oarai Harbor suffers from chronic and troublesome accretion of sediment. To describe it, we can do no better than to quote from an article by Hiroi (1921): *"A small harbor at Isohama (presently Oarai), in Ibaraki Prefecture, built in 1911 - 1916, is an instance of failure to keep off the inroad of drifting sands. The breakwaters (see Fig. 1 here), when built, had enclosed a water surface of about 30 acres, nearly all of which is now dry ground. The entrance, which is 240 ft in width, had a depth of 13 ft at the low water of ordinary spring tide. The prevailing winds are from NE, raising waves which cause the travel of sand and shingle along the beach lying to the east of the harbor. Accumulation of the sand began to take place as the breakwaters progressed, and by the time the latter were completed, nearly 370,000 cu. yds. of sand were found deposited in the harbor, which makes the yearly inroad of sand 74,000 cu. yds. The harbor has since been entirely invested by sand . . ."*

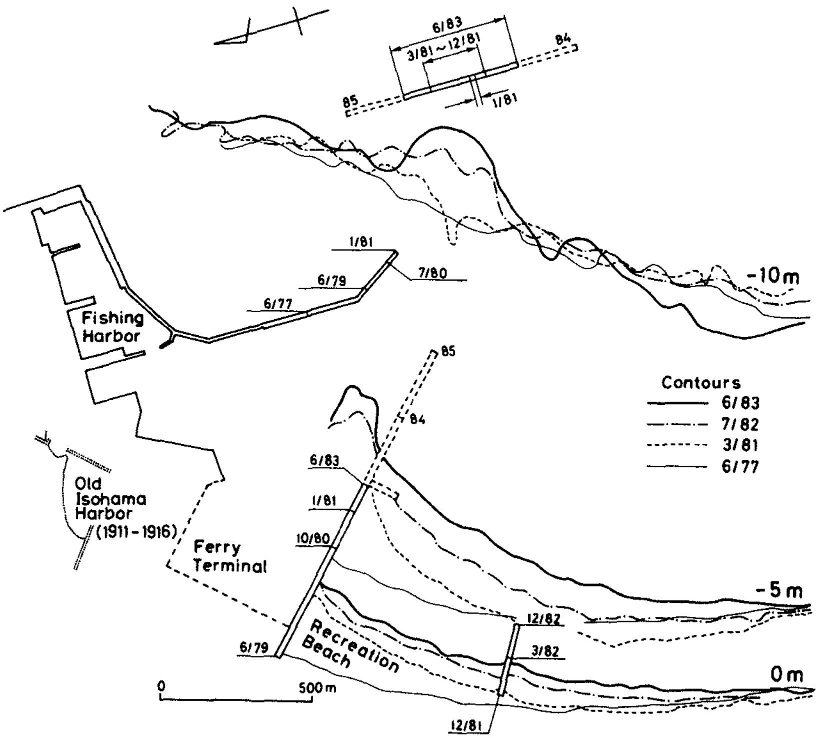


Fig. 1 Construction and contour change at Oarai.

In modern times, prior to construction of the detached breakwater and short groin, long-term sediment-related problems took the form of harbor shoaling and rapid shoreline advance at the long groin, together with erosion along the beach 1 to 3 km to the south of the harbor. Construction of the detached breakwater and short groin (Fig. 2) appear to have had a positive effect in reducing the rate of shoreline advance near the long groin. An apparent negative effect has been the gradual movement of the area of maximum erosion further south. The area of maximum erosion is presently located about 3 km to the south of the long groin, along what had once been a fairly wide sandy beach. The beach within 5 km to the south of the harbor contains two large seawalls (north seawall, approx. 1.5 km long; south seawall, 800 m long), two large impermeable groins, and numerous groups of armor blocks positioned in rows along the beach to provide shore protection and footing protection for the seawalls.

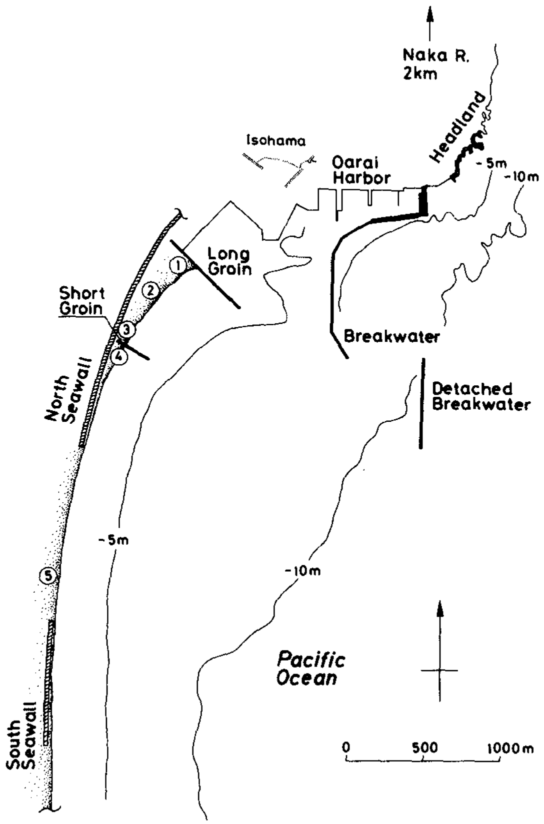


Fig. 2 Oarai harbor and the beach to the south.

The present paper, a continuation of the study of Kraus & Harikai (1983), consists of two parts. The first part presents a case study documenting the wave climate, currents, and past and present shoreline positions. The general features of the long-term shoreline change are then related to construction of the harbor breakwaters and groins. The second part presents results of simulations of the shoreline change.

2. CASE STUDY

2.1 Background

Oarai Harbor lies on a north-south-oriented coast about 150 km north of Tokyo. The area has served fishermen since ancient times owing to the shelter given by the rocky Oarai Headland against the predominant NE and ENE incident waves. The harbor has a long history of sediment shoaling. Hiroi (1921) and Ijima et al. (1961) report the complete burial of early harbor structures shortly after their construction in 1916 (Fig. 1). Using radioactive tracers, Ijima et al. (1961) found that a large portion of the infiltrating sand comes from the discharge of the Naka River, 2 km to the north. Through a series of hydraulic model experiments, Arakida et al. (1978) concluded that sand is also transported north toward the harbor due to the circulation cell produced by diffraction at the harbor breakwater tip (see also Mizumura, 1982).

2.2 Waves

The Oarai Harbor Management Office operates an ultrasonic wave gage and a strain gage, located at a depth of 20 m and connected to shore by cable. The strain gage, which measures the wave direction, does not function if the significant wave height falls below about 40 cm. Statistics for 100 waves are computed and recorded automatically at 2-hr intervals. Figure 3 gives a histogram plot of the frequencies of the significant wave height, direction, and energy flux for a 4-yr period.

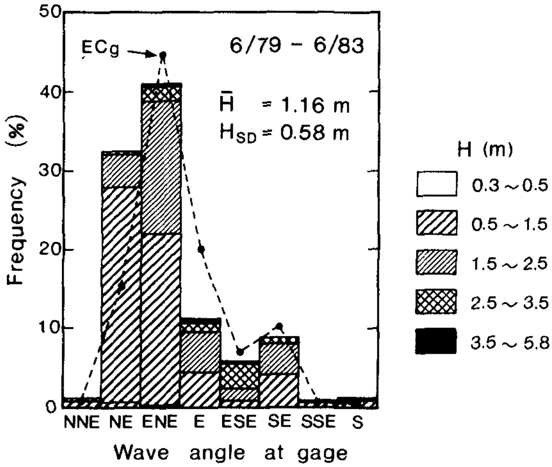


Fig. 3 Wave height, direction, and energy flux (EC_g).

A wave data set covering four years at 6-hr intervals was formed from the records. This data set, termed the RDS (Raw Data Set) consists of triplets of values of the significant wave height, direction, and period (H, Θ, T). The RDS contains gaps due to instrument down-time and limitations of the strain gage. For the numerical modeling, a data set without gaps was required. This data set, termed the CDS (Complete Data Set) was fabricated by first separating the available data from the RDS by month, giving four sets of 120 triplets, including blanks, for a 30-day month. Values were then randomly selected from the triplets available in the four sets for each month, and the gaps filled. The random selection and placement was done by computer and the results slightly 'cleaned' by hand. The respective distributions of H, Θ, and T of the CDS closely approximate those of the RDS.

As part of the NERC project (Horikawa and Hattori, 1984), the breaking wave height was measured at five locations alongshore (circled Nos. 1-5 in Fig. 2). Measurements were made weekly for periods of about 30 months (Nos. 1, 2, 3) and 18 months (Nos. 4, 5) between May 1980 and Dec. 1982. The significant breaking wave height, H_b, was measured by sighting over a graduated staff placed at the mean water shoreline; the average of ten of the higher waves was used. The frequency distributions of the average long-term breaking wave height at the five stations is shown in Fig. 4. The frequency of smaller wave heights increases with approach to the long groin, i.e., with penetration into the wave shadow zone produced by the headland and harbor structures.

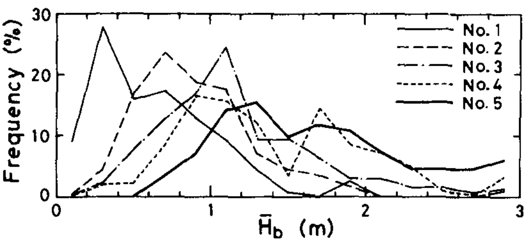


Fig. 4 Frequency distributions of significant breaking wave height alongshore (locations given in Fig. 2).

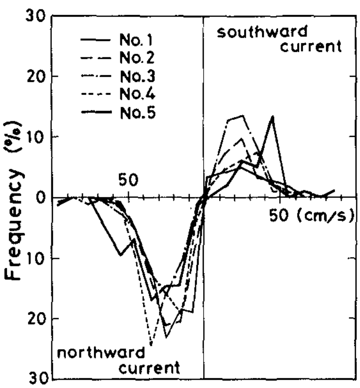


Fig. 5 Frequency distributions of longshore current magnitude alongshore.

2.3 Longshore Current and Sand Movement

The direction and speed of the longshore current were measured with floats, performed together with measurement of the breaking wave height. The frequencies according to direction are shown in Fig. 5. The longshore current is directed more often to the north than to the south and this tendency increases with approach to the long groin.

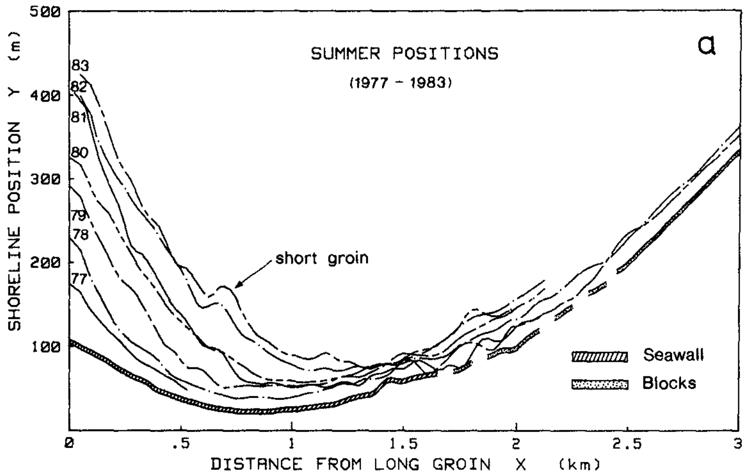
Waves incident at the site, arriving mainly from the northern sector, should produce a current moving to the south. The observed northward-moving longshore current is evidence of the large circulation cell produced by the headland and harbor structures. At some location near the edge of the diffraction shadow zone on the beach, the current reverses direction: at this location the beach will erode. The area of erosion is expected to move south with enlargement of the shadow zone due to extension of the detached breakwater. The northward-moving longshore current weakens deep inside the shadow zone, and sand is deposited there.

2.4 Shoreline Change

Shoreline positions on survey charts were digitized at 25-m intervals for plotting, analysis, and use in the shoreline simulations. The shoreline positions measured in summer and winter surveys are displayed in Figs. 6a & b. It is seen the shoreline near the long groin has steadily advanced, independent of season, if the interval of one year is selected as the time scale. Figure 6c shows the shoreline position during the four seasons of 1982. Although a general trend for accretion near the long groin is observed, the pattern of shoreline change is masked by short-term fluctuations such as those caused by sequences of storms and calm weather, and by changes in wave direction.

From Figs. 6a & b, it is seen that the north seawall served an important function in protecting the residential area behind it during the 1970's. With construction of the long groin (which blocks the northward-moving littoral drift) and extension of the harbor breakwater and detached breakwater (which increases the area of the diffraction zone where sand will be deposited), an expansive sandy beach has formed in front of the seawall. In contrast, on a recent inspection of the site in April 1984, the beach approximately 3 km south of the long groin was found to be severely eroding. Rubble mound blocks and the 800-m long south seawall in this area were being undercut and flanked.

Figure 7 plots the mean rate of change of the shoreline position for surveys taken before and after start of construction of the detached breakwater, based on 17 and 13 surveys, respectively. After start of construction, the rate of shoreline advance decreased by 50% near the long groin; the rate of advance increased somewhat in the region 1-2 km from the groin, and the formerly stable section of beach roughly centered at the 3-km mark began to erode (necessitating wider coverage in the shoreline surveys). The reduced rate of increase near the long groin is attributed to the low waves (weak longshore current) in this area deep in the shadow of the harbor breakwaters and to reduction of transport onshore through trapping by the detached breakwater. Trapping seems apparent, as judged by the change in 10-m contours in Fig. 1.



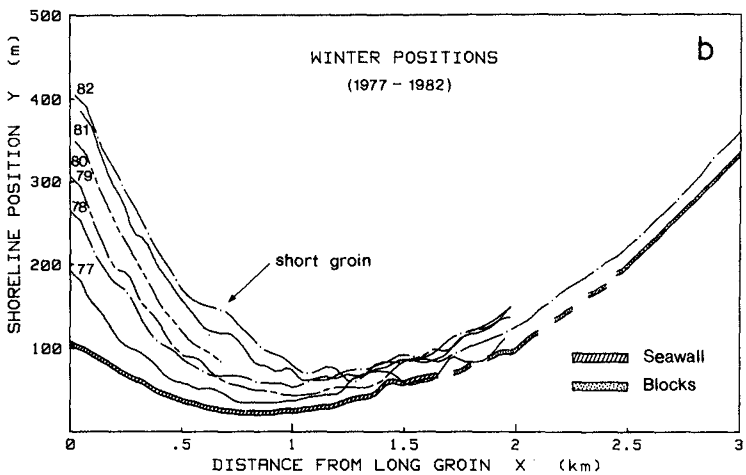


Fig. 6 Shoreline positions at Oarai Beach: (a) in the summer, (b) in the winter. (cont'd)

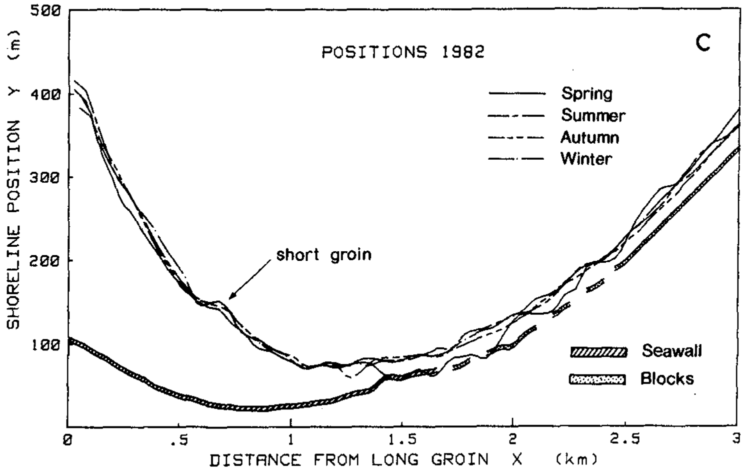


Fig. 6 Shoreline positions at Oarai Beach (c) during the four seasons of 1982.

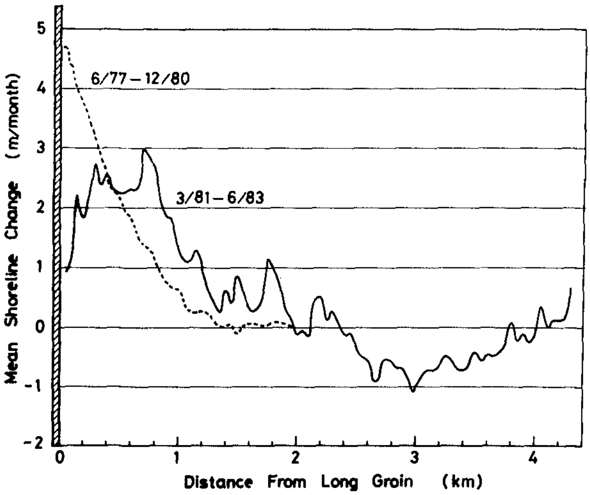


Fig. 7 Measured mean rate of shoreline change for periods before and after construction of the detached breakwater.

3. NUMERICAL MODEL

3.1 Background

The 1-line (shoreline) numerical model was used to simulate changes in the beach planform at Oarai. The purpose of the 1-line model is to reproduce the large-scale features of shoreline change that take place over a relatively long period. The temporal and spatial scales of the major types of beach change numerical models are discussed by Kraus (1983). In the present case, application of the 1-line model for periods of several months to several years is justified by the clear trend in shoreline movement shown in Figs. 6a & b.

The basic model used is that of Kraus and Harikai (1983), who modeled shoreline evolution at the site for the period between 1977 and 1980. A number of refinements to and extensions of the model were made for carrying out the present study. We plan to present a description of the numerical model in another paper. Here we simply list the major improvements and changes: (1) The calculation procedure for the breaking wave height and angle alongshore was refined (Kraus, 1983, 1984) and extended to account for three sources of diffraction -- at the tip of the harbor breakwater and at both tips of the detached breakwater. (2) A seawall boundary condition which conserves sand volume and preserves transport direction was incorporated (Hanson and Kraus, in press). (3) The angle of the groins to the shoreline was taken into account (Perlin and Dean, 1978), and a simple prescription was introduced to permit sand bypassing at the groins. The changes in the model necessitated recalibration and verification.

3.2 Shoreline Model

The governing equation for the shoreline position y is given by

$$\frac{\partial y}{\partial t} + \frac{1}{D}\left(\frac{\partial Q}{\partial x} \mp q\right) = 0 \qquad (1)$$

in which x is the longshore coordinate, t is the time, D is the depth of closure (beyond which the profile is assumed not to move), Q is the longshore sand transport rate, and q is the cross-shore transport rate onshore (-) or offshore (+). The predictive expression for the longshore transport rate is taken as

$$Q = \frac{H_b^2 C_{gb}}{16(\rho_s/\rho - 1)(1-p)} \left(K_1 \sin 2\theta_{bs} - 2K_2 \frac{\partial H_b}{\partial x} \cot\beta \cos\theta_{bs}\right) \qquad (2)$$

in which C_{gb} is the wave group velocity at the breaker line, ρ_s (ρ) is the sand (water) density, p is the sand porosity, θ_{bs} is the angle of the breaking wave crests to the shoreline and $\tan\beta$ is the beach slope. The coefficients K_1 and K_2 are treated as parameters in calibration of the model.

The first term in Eq. (2) corresponds to the CERC formula (CERC, Chap. 4, 1977) and describes the sand transport due to obliquely incident waves. The second term describes the transport due to a systematic variation in wave height alongshore. The second term has been found to be of importance for describing the shoreline change near structures, where diffraction dominates (Ozasa and Brampton, 1980; Kraus and Harikai, 1983; Mimura et al., 1983; Kraus, 1983). Gourlay (1982) gives a review of various derivations of Eq. (2).

The Shore Protection Manual (CERC, 1977, Chap. 4) recommends a value equivalent to K_1 = 0.77 for root-mean-square height. Several theoretical evaluations of K_2 have been given, resulting in different values, depending on the assumptions made (Gourlay, 1982). Experience of the authors in shoreline modeling, results of the NERC tracer experiments (Kraus et al., 1982), and other reports (Gourlay, 1982) indicate that K_1 is more likely to be in the range 0.1 to 0.6.

Considering the uncertainty in the values of K_1 and K_2 and the multitude of approximations and simplifications in the model, it is appropriate to treat K_1 and K_2 as site-specific parameters to be determined by calibration. Modeling experience indicates that the ratio K_2/K_1 lies in the range 0.5 to 1.5. The parameter K_1 acts principally as a time scale adjustment factor. The parameter K_2 controls the relative strength of the two terms in Eq. 2.

The depth of closure D in Eq. (1) also acts to adjust the time scale of shoreline movement. This characteristic depth in the model is related to the width of the beach through which sand is transported alongshore. Hallermeier (1983) has given expressions for calculating the depth of the seaward boundary where significant longshore transport and intense on-offshore transport take place. The simplest version has been found suitable for use with the shoreline model (Kraus and Harikai, 1983). It is

$$\frac{D}{H_s} = 2.28 - 10.9 \frac{H_s}{L_s} \tag{3}$$

in which H_s and L_s are the significant wave height and wavelength of waves in the shoaling zone, here assumed to be equivalent to those of the deepwater wave. In the simulations, D is recalculated at each change in the wave input.

The cross-shore transport rate, q, is the final quantity remaining to be specified before calculations can begin. Unlike the longshore transport rate, there are no predictive expressions available. Reflecting this situation, there have been only a few attempts to model shoreline change including cross-shore transport (Komar, 1973; Ozasa and Brampton, 1980; Kraus and Harikai, 1983). In the present case, we are faced with the need to account for the sand that moves around the harbor breakwater from the north. The amount, its time dependence, and its distribution are complicated functions of the discharge at the Naka River, wave conditions, and stage of construction of the detached breakwater. We described the onshore transport with the following convenient utilitarian expression:

$$q = q_o/(1 + ((x-2\cdot x_o)/x_o)^{p_o}) \tag{4}$$

in which q_o (units: m^3/s-m alongshore) is used as a fitting parameter together with K_1 and K_2.

3.3 Breaking Wave Model

The wave model for one diffraction source has already been described (Kraus, 1983, 1984). The introduction of three sources required additional assumptions to define the local breaking wave direction and wave height; we plan to present the procedure elsewhere. In essence, the wave model consists of solving the following equation:

$$\frac{H_b}{h_b} = \frac{K_D\ K_R\ K_S}{h_b} H_{tip} = \gamma \tag{5}$$

in which h_b is the depth at breaking, K_D, K_R, and K_S are respectively diffraction, refraction, and shoaling coefficients for linear waves, H_{tip} is the incident wave height at the tip of the structure and γ is a breaking wave index. A multiple iteration is performed over h_b and either one or two angles entering in K_D and K_R, depending on the level of approximation desired and run time considerations. At convergence, both the breaking wave height and angle are determined at a given point alongshore. The model was favorably compared with laboratory measurements and with measurements made at Oarai prior to construction of the detached breakwater (Kraus, 1983, 1984).

4. RESULTS OF SHORELINE SIMULATION

4.1 Recalibration of the 1-Diffraction Source Model

In order to examine the extent to which the various refinements altered the model, the model was recalibrated for the same period used by Kraus & Harikai (1983) (11 Jul 79 - 21 Feb 80) and with the same wave conditions. Best results were obtained with values of the transport parameters (K_1, K_2, q_o) of (0.1, 0.1, $4.0\cdot 10^{-6}$ m^3/s-m) as opposed to the previously determined values of (0.3, 0.4, $2.5\cdot 10^{-6}$ m^3/s-m). The reduction in K_1 and K_2 is mainly due to (1) the improved calculation of the breaking wave angle, which causes an increase in magnitude of the angle (an increase in θ_{bs} requires a decrease in K_1 to maintain the same time scale) and (2) the improved, volume-conserving seawall boundary condition. The results of this exercise demonstrate the fact that the transport parameters depend on the model used as well as on the characteristics of the particular site.

4.2 Calibration & Verification of the 3-Diffraction Source Model

For modeling periods after start of construction of the detached breakwater (Jan. 81), three sources of wave diffraction had to be used. Also, it was found during calibration runs that the distribution of onshore transport, q(x), appeared to have a different character than for the pre-1981 period. Inspection of the outputs of many runs with different combinations of transport parameters indicated a visual best fit with $K_1 = K_2 = 0.1$, $q_o = 4.5 \cdot 10^{-6}$ m^3/s-m, and $x_o = 300$ m, $p_o = 4$.

Verification runs with these and other values were made for the 2 1/4-year period from 11 Mar 81 to 15 Jun 83. Figure 8 shows results for the final shoreline position obtained with the calibrated model, together with the shorelines computed using other values of the transport coefficients. The simulated final shoreline obtained with the calibrated model agrees well with the actual shoreline up to 2 km from the long groin, but it shows too much advance along the beach 2 to 4 km from the long groin. The results of the calibrated model are far superior to those obtained using the other values of the transport coefficients.

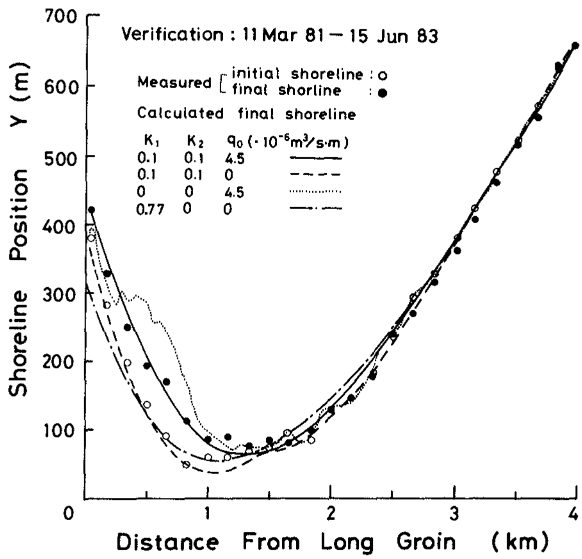


Fig. 8 Measured and calculated shoreline change for the verification interval.

The calculated mean rate of shoreline change along the beach is shown in Fig. 9. It can be compared with the measured rate for the same period, the solid curve in Fig. 7. The rate of shoreline change up to 1 km from the long groin is well reproduced. However, the location of the calculated maximum rate of erosion is at about the 1.5-km point, instead of at 3 km, and the calculation gives an essentially stable shoreline from about the 3-km point. The lack of agreement suggests the need to use a more sophisticated refraction routine for the irregular bottom topography for the region far from the harbor structures, where diffraction is no longer the dominant wave transformation.

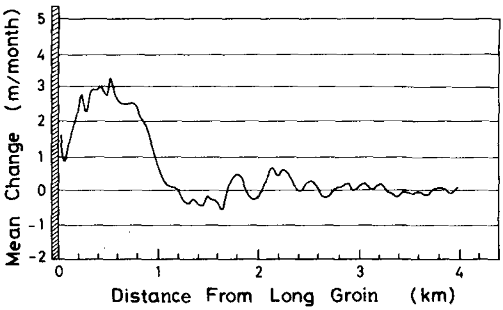


Fig. 9 Calculated mean rate of shoreline change for the verification interval 11 Mar 81 - 15 Jun 83.

Figure 10 compares calculated shoreline positions using the calibrated model and different averaging intervals of the wave input over the verification period (energy flux-weighted averages at the gage). In general, for the region near the long groin, the agreement deteriorates with increase in the averaging interval defining the representative wave conditions. The calculated shoreline in the region distant from the groin appears to be governed by the location of the eroded sector, which, in turn, is produced by the relatively fixed distribution of the breaking wave angle alongshore. Although recalibration of the model for a given averaging interval may improve the result somewhat, because of the sensitivity of the wave calculation to the incident wave angle, the result is expected to be inferior to that obtained by updating the wave information at simulated 6-hr intervals (cf., Le Mehaute et al., 1983, Kraus & Harikai, 1983).

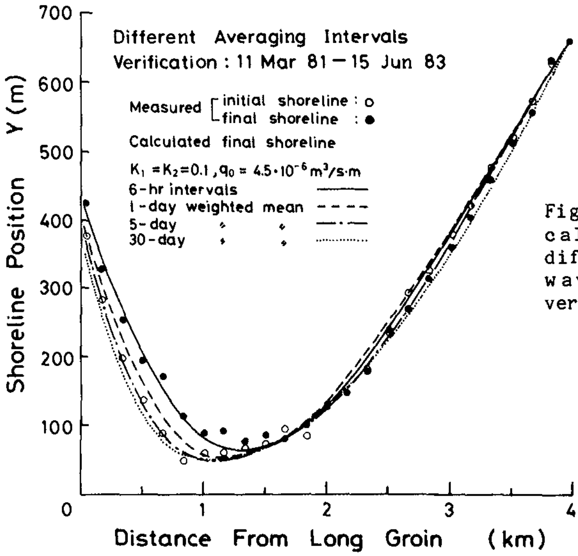


Fig. 10 Shoreline change calculated by using different time-averaged wave inputs for the verification period.

4.3 Forecast Shoreline Change

Although the verification simulation had a mixed outcome of success, it was decided to use the model to predict future shoreline change as an exercise for investigating problems associated with forecast-type simulations. Engineering-oriented forecasts of shoreline change cannot be found in the open literature, probably because of the legal aspects involved. The main technical problem is the treatment of the wave input, i.e., how the time history of the waves should be forecast.

In the present work, two additional problems emerged. First, due to active construction at the site, the construction schedule and structure positions had to be estimated for automatic update in the model. (These affect the location of the diffraction sources and bypassing at the groins.) The second problem concerned the possible trapping function of the detached breakwater of sediment that would normally move south around the harbor breakwater and then onshore. We speculate that the detached breakwater traps most of this sediment. To investigate the potential trapping function, q_o was varied in different series of forecast runs.

Treatment of the the wave time history in shoreline numerical modeling has been considered by Le Mehaute, Wang & Lu (1983) and Kraus & Harikai (1983). In the former study it was demonstrated that the results of shoreline modeling depend on both the interval at which the wave data are input and the order of the input. In the latter study it was shown that time averaging of the wave data for Oarai for intervals greater than about 5 days gave rather poor results. The acceptable interval for updating wave information in a model will depend on the steadiness of the waves at the site in question, in particular, for wave direction.

In forecasting shoreline change, the assumption is made that the time sequence of wave events and the wave conditions in the future will have much the same character as in the past. It is valuable to have a long wave record with which to accurately describe the average and time-varying wave conditions. One must also keep in mind possible long-term cycles in the wave climate (e.g., Kuhn and Shepard, 1983), as well as infrequent extreme events.

Noting that the exact time history of wave events is not predictable, Le Mehaute et al. (1983) introduced a Monte Carlo procedure to provide measures for the possible deviation. Their approach is a sound conceptual solution to the problem. However, our experience gives us the impression that direct application of the Monte Carlo method might yield a range of shoreline change which is too narrow.

Basically, we would like to estimate the maximum range of shoreline movement resulting from wave histories which do not differ "too greatly" from the past time history. The "not too great" differences should incorporate variations in both wave conditions (energy flux) and sequence of wave events. In other words, how can we estimate limits of variations in the future wave data set? We took the direct approach of manipulating the individual distributions of wave height and direction, while leaving the period distribution unchanged for simplicity.

For a simple estimation of the limits of the maximum range of deviation of the forecast waves, it was assumed that wave height and direction distributions would be shifted by a certain amount from the past distribution (given by the CDS). As a limit of the shift, we took one half of the standard deviation of the respective distributions, i.e., H was allowed to vary between $H \pm H_{SD}/2$, and similarly for Θ, where the subscript SD denotes the standard deviation. This procedure provides deviations in the energy flux. To estimate the effect of sequencing, the CDS was ordered with respect to increasing and decreasing H and Θ. In all data sets generated, only one quantity was manipulated and the other two quantities were left unchanged to form a new triplet of values of H, Θ, and T at 6-hr intervals.

The transport parameters K_1 and K_2 were held at 0.1 as determined in the calibration, but q_o was varied to produce different series of runs. The resultant shorelines for the 5-yr forecast period using $q_o = 2.0 \cdot 10^{-6}$ are shown in Fig. 11. The heavy line denoted by H, Θ, T gives the result using the CDS. The greatest deviation from the CDS prediction occurs for the runs with $H \pm H_{SD}/2$, corresponding to severe and mild wave conditions, respectively (neglecting the correlation with wave period and direction). All runs gave a shoreline advance near the groin. The amount of advance is less than that which occurred over recent 5-yr periods. Except for the run with decreasing Θ, all runs produced an eroded beach starting from about 2 km from the long groin. Other series with different q_o had much the same qualitative features.

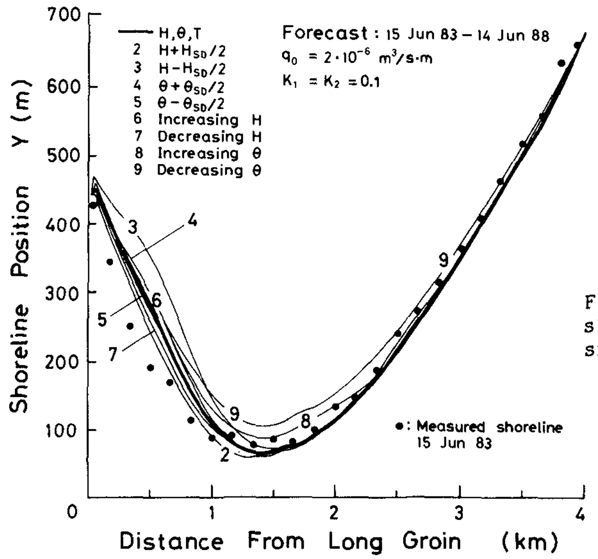


Fig. 11 Results of shoreline forecast simulations.

The forecast shoreline changes both near and far away from the long groin are in general agreement with recently observed trends at the site. However, the inability to predict the time-dependent behavior of q_o introduces a great uncertainty into the forecast, and the results should be interpreted appropriately in light of that uncertainty.

5. CONCLUDING DISCUSSION

The large harbor breakwaters being built at Oarai for the purpose of improving navigation exert a great effect on the near and distant sandy beach. The observed shoreline change can be well understood on the basis of long-term measurements of the waves and currents. The 1-line numerical model was reasonably successful in simulating the observed shoreline change in spite of the complications of multi-sources of wave diffraction, the wide area of coverage including a seawall and two groins, and problems with estimating onshore sand transport. The forecast shoreline change for the next five years was in general agreement with the present trend in evolution of the shoreline.

In order to make more accurate predictions of shoreline change for Oarai, the following four topics should be addressed. (1) The necessity of including a 2-dimensional refraction routine for the region far from the long groin should be investigated. (2) The south seawall should be included in the model and the model range should be extended well past the south seawall. (3) The trapping efficiency of the detached breakwater should be empirically estimated (as from bottom surveys, and possibly tracer experiments). (4) The expected limits of variation of the wave time history should be more objectively determined from the statistical properties of the CDS, and the joint probability distribution of wave height, direction, and period should be utilized.

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