

CHAPTER 71

ESTUARY INLET CHANNEL STABILIZATION STUDY USING A HYDRAULIC MODEL

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ABSTRACT

A hydraulic model study was conducted to determine the most feasible means of reducing shoaling into a coastal inlet channel that had been used by loaded Landing Ship Tank (LST) class vessels serving the port of Tan My near Hue, Republic of Vietnam. The ultimate objective was to increase the time during which the channel will be open to navigation by improvements in physical configuration.

An extensive field data acquisition and analysis program was executed to provide basic information for model prototype correlation. This incidentally resulted in the re-evaluation of several empirical formulae being used for prediction of littoral transport and of tidal currents. The comprehensive field measurements of shoaling in the channel area also permitted the determination of the scale effects on the quantitative results of sediment deposition in the channel and on the time scale of bottom evolution.

The experiments were conducted systematically in three phases. The three-dimensional studies for Phase I and II were conducted using a rigid bed, sand or walnut shell covered (1/250 horizontal and 1/50 vertical scale) hydraulic model of a 15 sq mi ocean-estuary land area. Periodic long-crested waves (tides, tide/river induced currents) were simulated in the model. Phase I covered qualitative evaluation of nine improvement plans, from which a "BEST PLAN," an upshore jetty to protect the existing channel, was selected. Phase II covered qualitative and quantitative evaluation of four variants of the "BEST PLAN," from which the most effective and economical variant was determined. To insure reliability of quantitative results, comprehensive experiments were conducted using three different sediments ($D_{50} = 0.41$ mm and 0.90 mm, $\gamma = 1.35$ for two types of ground walnut shells and $D_{50} = 0.22$ mm, $\gamma = 2.65$ for sands), and three model scaling criteria (Froude Scaling, "Ideal Velocity" Scaling, and Modified Froude/Ideal Velocity Scaling). Phase III covered two-dimensional model tests, in a 180 foot flume, of the stability characteristics of the proposed rubble mound type jetty on a movable bed.

The major conclusions of this study include (1) the jetty, as postulated in the "BEST PLAN," would reduce annual maintenance dredging requirements by 74 to 84 percent, (2) a systematic three phase model study such as used in this investigation is most feasible (from a time-consumption viewpoint, use of light weight materials such as ground walnut shells may be more economical than use of natural sands as model material), and (3) use of theory alone in predicting a time scale for bottom evolution should be approached with caution especially where field data are not readily available.

INTRODUCTION

This paper describes the techniques used for a particular coastal inlet stabilization study using a hydraulic model in order to obtain an optimum solution to the excessive shoaling of the existing ship channel. The channel in question is located at Tan My, on the South China Sea Coast near Hue Republic of Vietnam. The shoaling results from significant storm waves during the winter and summer monsoon seasons dominated by wind waves from the Northeast during the period from November to January. Sediment contribution from the inland rivers (Huong, Bo and O Lau) is mainly deposited in the large estuary area with only negligible wash load diverted into the offshore channel area. To maintain a 20 ft deep and 300 ft wide ship channel considerable efforts were expended to keep the channel open by dredging (1.8 million cubic yards per year). Unfortunately, the duration of the relatively safe dredging periods are prohibitively short and infrequent. It was therefore considered necessary to construct permanent barrier structures in order to protect the Tan My Channel from shoaling so that the channel can be operationally useful for most time with minimum dredging maintenance.

The study (Lee, 1970) was sponsored by the Officer in Charge of Construction Republic of Vietnam Naval Facilities Engineering Command Department of the United States Navy.

DESCRIPTION OF STUDY AREA

The subject study covered a small area (3 miles by 5 miles) of the coastal line and offshore topography, the inlet, and a portion of the estuary. The boundaries of the project area to be modeled were selected on the basis of such criteria as (a) the model should extend to sufficient depth (10 fathoms) to obtain correct refraction patterns of waves approaching the shoreline from prevailing directions, including NE, N, and NNW, around the area of interest, (b) sufficient shore length should be provided to permit cutting a new channel through the spit and to allow a natural development of longshore current by water waves and (c) the boundary condition in regard to the flow should not change as various improvement plans for the inlet are studied in the model.

OCEAN ENVIRONMENTAL DATA ACQUISITION

An extensive field data acquisition and analysis program was executed in order to provide basic information for model prototype correlation.

Waves and Swell Wave climates simulated in the model were determined from observations by Lyon Associates (1968-1969), NAVFORV Weather, Saigon, RVN from wave budget analyses by Marine Advisors (Inman and Harris, 1966), and wave hindcasting by Glenn Associates (1966). The wave rose developed from most recent field observations is shown in Fig 1(a).

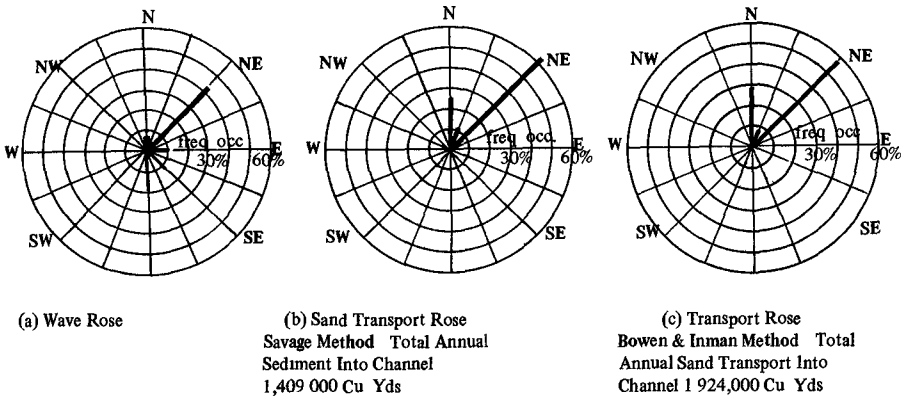


Fig 1 Wave and Sediment Transport Distribution

The following "Typical Waves" were used in the model tests for the selection of the "BEST PLAN"

	Wave height (feet)	Wave period (seconds)
Severe Storm Wave Condition	10	8
Normal Storm Wave Condition	5	5

Wave directions selected for model tests included NE, N, NNW. An analysis of the waves incident on the channel has shown that shoaling in the channel results as follows: 60% from NE, 25% from N, and 15% from NNW and other directions [Fig 1(b) and (c)]. Furthermore, the wave and swell data, wind information, and weather maps related to the period of Typhoon Bess (27 September 1968) were analyzed. It was found that the average wave climate incidentally corresponded to the Normal Storm Wave condition defined above: i.e. 5 ft high, 5 second period (from NE).

Tides and Tidal Currents The diurnal tide at Tan My was an average of 2.0 feet with a tidal period of 25 hours and 35 minutes. The semi-diurnal tide was 1.6 feet high and 12 hours and 47 minutes. Tide induced current in the Tan My entrance channel without river discharge was predicted, using the theoretical method of Keulegan (1967). The maximum velocities predicted compared reasonably well with those measured by Lyon Associates and by the U.S. Naval Oceanographic Office. The field measurements were made of tides and tidal currents at four locations as shown in Fig. 2. The Lyon's measurements involved detailed measurements of tidal current velocities covering the whole

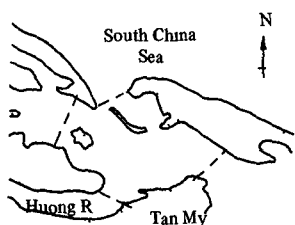


Fig. 2 Tide and Current Measurements at Four Stations

cross section at 0.2 and 0.8 of the water depth, from which the mean velocity through each section can be determined. Unfortunately, tides were not recorded continuously and the measurements at the sections were not made simultaneously. Ten series of measurements were made at each station during the 5 month period from September 1968 to January 1969. The measurements by the Naval Oceanographic Office were made simultaneously at each of the four stations but only currents at one or two points were recorded instead of numerous points at each station as done by Lyon. The measurements covered a four month period from December 1968 to March 1969. The field measurements helped verify somewhat the tide and tidal current predictions but it is not feasible to simulate such tides because the measurements include the effects of waves, river discharge, and longshore currents. Therefore, the tide simulated in the model was based on tide prediction and the tidal currents through the channel according to proper adjustments of tidal prism in the model estuary.

The maximum tide prism measured at Tan My was 47×10^6 m³/diurnal tide cycle as determined from the maximum mean tidal discharge of 1500 m³/sec measured by Lyon. This compared well with the 49×10^6 m³/diurnal tide cycle as estimated by Inman and Harris (1966).

The tide and tidal currents simulated in the model are considered adequate because the low tide range does not significantly affect the sand transport into the channel.

Hydraulic Regime Offshore Tan My Inlet For calibration of the model and study of the flow conditions in the inlet area, it was necessary to obtain current patterns. These patterns were obtained by the Naval Oceanographic Office using infra red techniques.

Huong (perfume) River Flow Based on frequency of occurrence of the Huong River flow, 500 m³/sec and 1,800 m³/sec were selected to simulate the normal and severe flood conditions. This seems to be high, but it would compensate for the omission of the contributions from the Bo and O Lau Rivers and reduce the scale effects of the use of sands as bed materials. (Flow measurements at Huong River during a 4 month period showed the maximum discharge was 1,250 m³/sec and averaged 300 m³/sec.)

Longshore Currents Offshore ocean currents upcoast of the Tan My Inlet were measured with float techniques but the measurements are not utilized because they were made too far offshore to be within the model limits. Subsequently, longshore currents of 2.0 ft/sec and 4.0 ft/sec were predicted by theory and simulated in the model for normal storm wave (5 ft high, 5 second period) and for severe storm wave (10 ft high, 8 second period) conditions respectively.

Sediment Properties. Beach samples were taken upcoast of the Tan My Inlet. The typical size distribution of sediment—the median diameter (D_{50})—was 0.41 mm. The sediment is 100 percent quartz, containing no carbonate, with a density of 2.65. The size distributions of both prototype and model sediments are shown in Fig. 3.

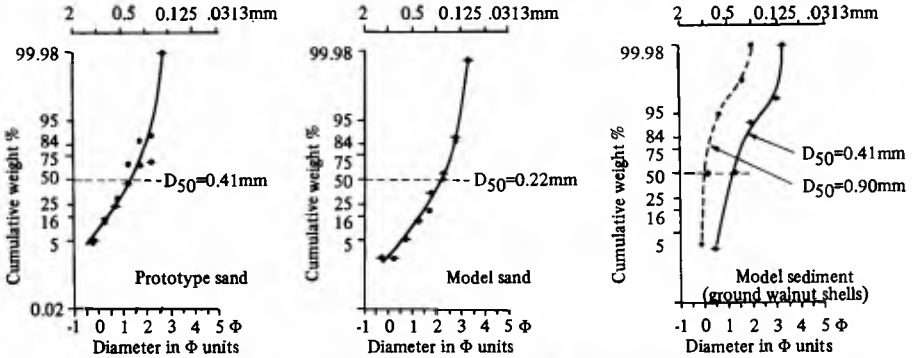


Fig. 3 Size Distribution of both Prototype and Model Sediments

Coastal Morphology. The shoreline evolution around Tan My Inlet from April through December 1968 was studied from the analyses of monthly uncontrolled aerial photographs. These photographs showed the unstable nature of the Tan My Inlet. A comparison of the coastal morphology at Tan My during August, September, and December 1968 is shown in Fig. 4. The stability parameter for Tan My is compared with other similar inlets in Table 1.

Shoaling and Dredging Information. An extensive study was made of the shoaling and dredging aspects at the Tan My existing entrance channel. Included are: annual sediment budget, sediment transport during a special storm (Typhoon Bess), bottom topography changes, and dredging studies. It was found that the total sediment transport into the existing channel amounted to 2.1 million cubic yards ($1.6 \times 10^6 \text{ m}^3/\text{year}$) as compared with 1.4 to 1.9 cubic yards per year (1.1 to $1.5 \times 10^6 \text{ m}^3/\text{year}$) predicted by theories of Savage (1959) and Bowen-Inman (1966) respectively [see Fig. 1(b) and (c)]. Based on the estimated inlet stability parameter (Bruun, 1969) the annual sediment transport would be 1.4 million cubic meters or 1.8 million cubic yards. This estimate was the basis for economic evaluation of the improvement plans.

Summary of Ocean Environment Data for Model Tests. The ocean environmental data selected for the model tests are summarized in Table 2.



Fig. 4 Coastal Morphology at Tan My Inlet

Table 1 INLET STABILITY

Inlet	Tide Prism (Ω) (m^3 /tide cycle)	Sand Transport (M) (m^3 /year)	$\frac{\Omega}{M}$	Stability*
Tan My	47×10^6			
Model		1.50×10^6	31	
Savage		1.08×10^6	43	
Bowen & Inman		1.46×10^6	32	
Dredge Record		1.61×10^6	29	
Average		1.40×10^6	34	Very Poor
Masonboro, N C (Bruun, 1966)			30	Very Poor
Ponce de Leon, Fla (Bruun, 1966)			40	Poor
Figueira da Foz, Portugal (Bruun & Gerritsen, 1960)			40	Poor
Oregon, N C (Bruun & Gerritsen, 1960)			80	Poor to Fair
* Very Poor	very much shoaling			
Poor	much shoaling			
Poor to Fair	significant shoaling			
Fair	some shoaling			
Good	neghible shoaling, $\frac{\Omega}{M} = 100$			

Table 2 SUMMARY OF OCEAN ENVIRONMENT DATA FOR MODEL TESTS

Environment	Normal Storm Condition	Severe Storm Condition
Wave and swell		
Wave height (ft)	5	10
Wave period (sec)	5	8
Wave direction	NE, N, NNW	NE, N, NNW
Tide		
Diurnal		
Tide range (ft)	2	2
Tide period or cycle	25 hr , 35 min	25 hr , 35 min
Semi Diurnal		
Tide range (ft)	1.6	1.6
Tide period or cycle	12 hr 47 min	12 hr , 47 min
Tide Prism	$47 \times 10^6 m^3$ /diurnal tide cycle	
Huong River Discharge (m^3 /sec)	500	500 and 1,800
Longshore Current (ft /sec)	1.8 2.0	3.6 4.0
Sediment Property		
Medium diameter (d_{50})	0.41 mm	0.41 mm
Density	2.65	2.65
Annual Sand Deposition into Channel (used in study)	1.8×10^6 (cu yds /year)	

THE TAN MY ESTUARY MODEL

Design of Model The model (Fig 5) has the following linear model scale ratios

- (a) Horizontal Scale $1/250 = n_1 = \lambda$
- (b) Vertical Scale $1/50 = n_d = \mu$
- (c) Model Distortion $1/5 = n_d/n_1 = \mu/\lambda = \Omega$

The vertical scale and model distortion were selected with due consideration of the scale effects, the artificial sediments to be used in the model, and prototype beach slope. In general most models of movable bed have a distortion ratio from 1.3 up to 1.8. The natural beach slope at Tan My is relatively gentle (1:60) and the equilibrium beach slope is changed with wave characteristics, thereby affecting the decision of choosing a single model distortion to satisfy all equilibrium beach slope conditions, therefore a 1.5 model distortion was selected. The selection of the vertical scale does not materially affect model results because the model has a semi movable bed rather than a movable bed. The foreshore and offshore beach slopes were maintained constant during the tests because the natural beach slope is relatively flat.

Similitude Relationship The model and prototype scale relationship for the Tan My Estuary Model for the Phase I and early Phase II tests is summarized by Lee (1970)

The following references were helpful: Fan and LeMehaute (1969), Byker (1967), Bruun, et al (1966)

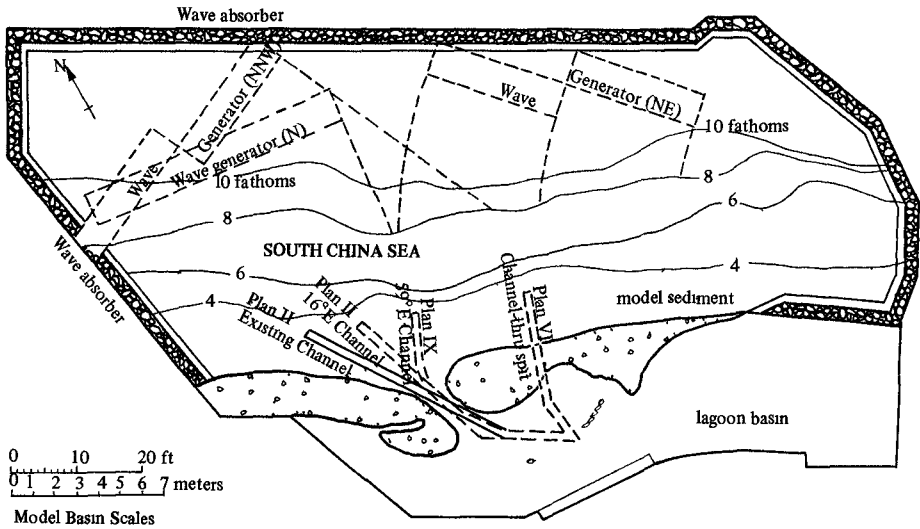


Fig 5 General Layout of Tan My Estuary Model

EXPERIMENTAL EQUIPMENT AND PROCEDURE

Ocean Environment Simulation, Measuring and Recording Apparatus

Wave Generators The wave generator for the three dimensional model tests was of the plunger type, capable of simulating long crested, periodic water waves. The wave generator for the 180 foot long wave flume was also of the plunger type, with a unique control unit. The generator is driven by a high torque, low speed hydrostatic pump motor. The hydrostatic drive is infinitely variable in speed in either direction off detent center from 1 to 60 rpm. The variable displacement hydraulic pump is driven at a constant speed by means of a 20 hp, 1750 rpm motor.

Tide Generator Tides of diurnal and semi-diurnal types were simulated by alternately pumping and draining water to and from the model basin. The outer ocean basin and inner lagoon basin were controlled separately. The tide generator was designed to reproduce tidal discharge through the inlet rather than tidal elevation differentials between the ocean and bay lagoon. The tide generator had a unique electro pneumatic flow control system which was simple and economical. It was calibrated by a cam arrangement to reproduce inlet flow by changing electrical voltage which in turn controlled the valve opening of the pipe lines.

River Flow Generator Huong River discharge was simulated by spilling water over a V weir into the estuary near the river mouth through a pumping system.

Other Measuring and Recording Systems Time-exposure photographs were used extensively to provide information on the speed and direction of currents in the inlet area and to illustrate graphically the eddy locations and current patterns. This helped evaluate the hydraulic regime of each particular improvement plan tested in the model. Current through the channel entrance was measured by timing the travel of a surface float over a known distance. Waves were measured by means of optical (Palmer, 1970) and electrode resistance type wave gauges. Tidal current and tide level variations were measured respectively and intermittently by float and point gauge techniques over the entire tide cycle to verify the outputs which had been calibrated. River flow was measured by a V shaped weir. Motion pictures were taken to document the pertinent tests of the entire study.

Experimental Procedure The following experimental procedure was followed:

1. Calibrate the three dimensional model after a series of preliminary tests to determine the most significant parameters which would affect the flow characteristics and the quantity and distribution of sediment transport and deposition in the channel. Relative effects of waves, tides, river discharge type of model sediment, artificial roughness were thoroughly investigated.
2. Conduct Phase I tests covering qualitative evaluation of nine channel improvement plans (Fig 6) from which a "BEST PLAN" (an upshore jetty to protect the existing Tan My channel) was selected.
3. Conduct Phase II tests covering the qualitative and quantitative evaluation of four variants (Fig 7) of the "BEST PLAN" from which the most effective and economical variant was determined.
4. Conduct Phase III tests in a large wave flume at a much larger scale to determine the stability characteristics of the rubble mound type jetty of the "BEST PLAN" against normal and severe storm wave actions.

MODEL STUDY AND RESULTS

Proof of Model The model base line is the quantity and distribution of sand deposited in the existing channel during a 35 day period, 9 August to 13 September 1968, which covered the period of Typhoon Bess, 31 August to 7 September. Field data on channel configuration at the beginning and end of the period were also available, as well as the quantity and location of dredging during the period. Excitation during the period was not measured. However, hindcasting procedures indicated that it likely was the Normal Storm Condition as shown in Table 1. Coincidentally, this condition also represents the average wave condition during the NE wave monsoon season. Therefore, model prototype correlation was also made of shoaling caused by normal, or long term wave action.

Because the orientation of the channel is unclear and the river flow through it during Typhoon Bess is unclear and shoaling is sensitive to these factors proper proof of the model on a quantitative basis was not possible. Therefore, only qualitative model prototype correlation was achieved during the Phase I tests. Subsequently, comprehensive tests with seven scaling criteria were conducted to relate their relationship quantitatively under Phase II tests which will be discussed later.

Furthermore, the favorable reproducibility of the bottom topography at the inlet [Fig 6(a)] and of the hydraulic regime (current pattern) in the model leads to the belief that the model prototype correlation is considered adequate with the data available for the purpose. Preliminary viewing of the imagery indicated that the eddy near the mouth of the Tan My Channel in the model appears also in the imagery [Fig 8]. The patterns appear to be consistent with a

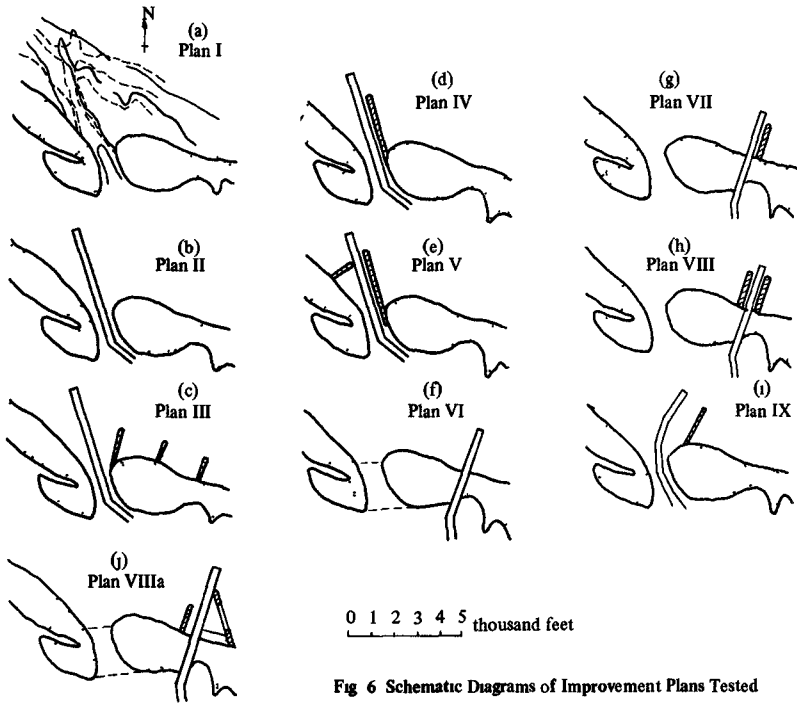


Fig 6 Schematic Diagrams of Improvement Plans Tested

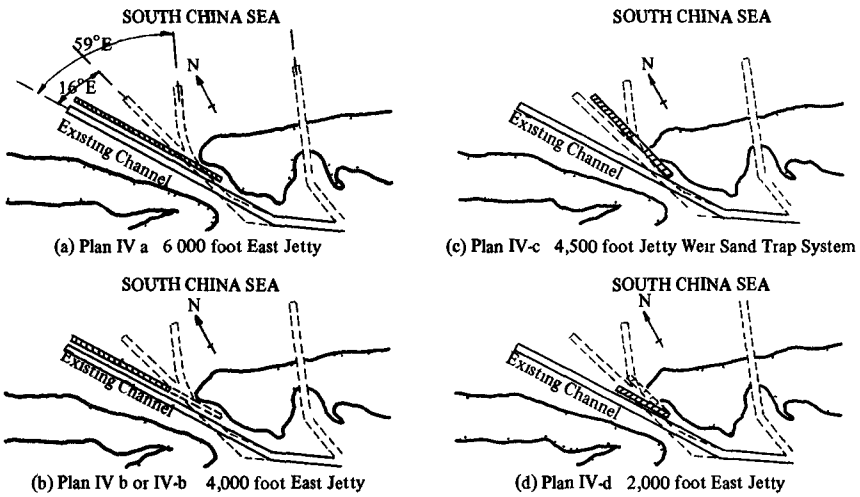


Fig 7 Schematic Diagrams of Four Variants of "BEST PLAN" Tested

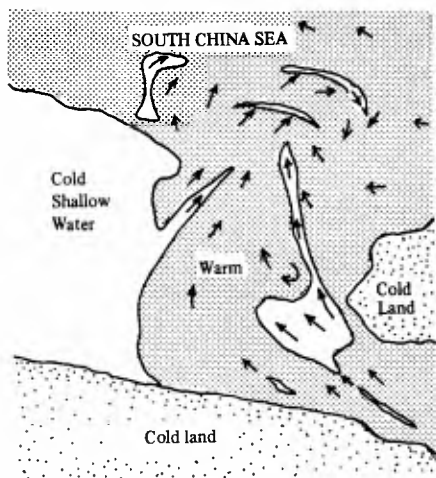
postulated, relatively small, clock-wise eddy just off the inlet (Wiemat, 1969). This favorable prototype-model relationship helped increase confidence in the model results. The distribution of peak shoaling area is consistent with actual prototype conditions observed over long periods of time under normal conditions. Also, the favorable comparability of the littoral transport (Table 3) by model, prototype, and prediction, and favorable comparability of measured and predicted tidal current (Table 4) have encouraged the preceeding of the model tests for subsequent test and evaluation of a variety of channel improvement plans.

Table 3. COMPARABILITY – LITTORAL TRANSPORT

Method/ Source	Littoral Transport (cu. yds./day)			
	(1) Normal wave NE, 5 ft., 5 sec.	(2) Storm wave NE 10 ft., 8 sec.	Ratio (2) / (1)	
Model measurement	Measured	10,500	79,900	7.57
Prototype	Calculated from dredging records	9,700	—	—
Prediction	Caldwell (1956)	10,500	53,750	5.13
"	Savage (1959)	18,500	50,000	5.90
"	Bowen & Inman (1966)	8,800	69,000	7.85
"	Bijker (1968)	6,450	48,900	7.59

Table 4. COMPARABILITY OF MEASURED & PREDICTED TIDE CURRENT

	max. mean current vel.	max. surface current vel.
Measured in model	0.093 ft./sec.	0.210 ft./sec.
Predicted by Keulegan (1967) method	0.078	0.175
Error	16%	17%



(a) Prototype Circulation Pattern
Infrared Imagery June 24, 1969



(b) Model Circulation Pattern

Fig. 8 Comparison of current circulation patterns, prototype vs. model

Phase I Qualitative Evaluation of Improvement Plans Phase I covered qualitative evaluation of nine (9) improvement plans [Fig 6] from which a 'BEST PLAN,' an upshore jetty to protect the existing channel, was designated. These sedimentation reduction plans involved four channel configurations. These were the existing channel, two channels rotated about 16° and 59° East, relative to the existing channel, and a new channel relocated through the spit about one mile upshore of the existing entrance. All these channel configurations (except the 59°E channel) were protected by groins and/or jetties upshore. In some cases downshore jetties were tested as well. (Plans tested are shown schematically in Fig 6)

The quantity and distribution of sand transport into the channel for various test plans, as measured in the model are compared in Fig 9. It is shown that Plan IV, V, or VIII provides better protection from shoaling of the navigation channel, i.e. the existing channel or a new channel cut through the spit. Their relative merits will be described below.

The four unprotected channels [Fig 5]—two associated with Plan II, existing and 16°E, and IX and VI—behave similarly as a function of their orientation relative to the direction of the incident wave when the other excitations are constant, i.e. river, longshore, and tidal currents.

Their behavior is highlighted by a dominant constant river outflow which (both aided and opposed by the wave as well as by tidal and longshore currents) tends to generate eddies up and downcoast of the entrance, this action, in turn, builds a bar on the upcoast side and scours a hole on the downcoast side.

At the channel entrance, the wave generated current (mass transport plus oscillatory) tends at significant times to transport sediment into and up the channel. Thus, the quantity of sediment (Q) deposited in the channel should be a function of the ratio of component (Py) of the wave generated power (P) parallel to the centerline of the channel proportional to P. This appears not only to be the normal case, but also roughly a linear case for Test Condition I with NE waves with $H_o/L_o = 0.40$ incident on the four channels as indicated by a plot of P_y/P versus Q/ft of channel [Fig 10] where H_o and L_o refer to deep water wave height and length respectively, even though Johnson (1965) and others indicate transport should be dominantly offshore when $H_o/L_o = 0.025$.

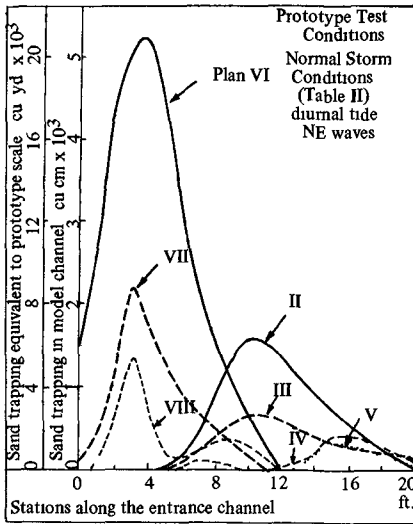


Fig 9 Comparison of Relative Merits of Various Plans for the Existing Entrance and New Entrance through the Spit

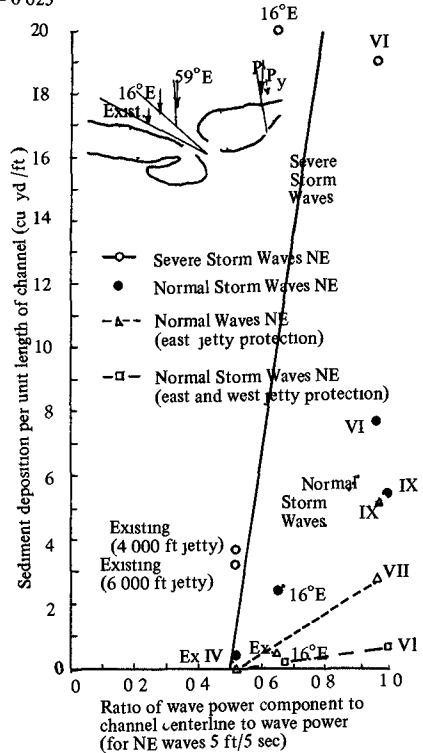


Fig 10 Correlation of Sediment Deposition in Four Channels Tested

A review of these data indicates that the existing, 16°, IX and VI channels are sediment retarders in that order of effectiveness this is attributable to the fact that the channel centerlines are 64, 49, 5, and 17 degrees, respectively, to the NE direction of the incident wave in deep water That is, the channel (existing) closest to perpendicular to the incident wave crests is the most effective sediment retarder

The four channels (two associated with Plan II—existing and 16°E—and IX and VI), when protected by a full length upshore jetty (Plans IV and VII), and partial length groin well upshore (Plan IX), behave predictably This predictability is, as in their unprotected condition somewhat a function of channel "heading" relative to the direction of incident waves The protected versions in all cases are more effectively self cleaning channels

An upshore jetty is considered entirely adequate as a channel protector The addition of a downshore jetty adds a quite small increase in effectiveness at the expense of inducing undesirable eddies at the entrance and is not recommended

It was found that the unprotected anti shoaling effectiveness of the four channels (existing, 16°E, 59°E and through the spit) varies inversely as the heading into the incident waves The existing channel with full upshore jetty is most effective (and the channel through spit the least effective) as designated "BEST PLAN

An upshore jetty weir sand trap system, such as at Masonboro Sound N Carolina, is no more effective as a total sedimentation retarder than a conventional system However, it reduces sedimentation into the channel (balance in trap), and hence merits further study only if preservation of downshore configuration is important

The reproducibility of the magnitude and distribution of sand deposited in the channel from run to run is close when the beach is stabilized, i e after two tide cycles for normal waves and four to six cycles of severe storm waves There is a variation of 1 to 12 percent, which is quite acceptable

Sand transport measured in the prototype and model compare well with those predicted theoretically by Savage (1959) and Caldwell (1956) for normal to moderate waves, and Inman (1966) for storm waves The comparison is poor for those predicted by Bijker (1968) i e 40 percent lower than those measured in the model and predicted by other methods (see Table 3)

Phase II Qualitative and Quantitative Evaluation of the 'BEST PLAN' Phase II covered qualitative and quantitative evaluation of the most effective and economical variant of the "BEST PLAN" This phase of the study involved determination of the optimum length and orientation of the jetty among four variants [Fig 7] To insure reliability of the quantitative results, a series of comprehensive experiments were conducted with sand ($D_{50} = 0.22$ mm, $\gamma = 2.65$) and two sizes of light weight, ground walnut shell material ($D_{50} = 0.41$ and 0.90 mm $\gamma = 1.35$) as model sediment Three different model-similitude criteria were used in these tests, i e, Froude Law Scaling 'Ideal Velocity' Scaling, as proposed by Bijker (1967) of Delft University, and "Modified Froude Law/Ideal Velocity" scaling, as proposed by the Look Laboratory The time scale of bottom evolution was determined experimentally for each case and compared with theory

Three versions of the "BEST PLAN" included jetties immediately upshore (east) of the existing channel, with lengths of 6,000, 4,000 and 2,000 feet, located along the channel bank a fourth version included a 4,500 foot jetty upshore of the channel bank to form a jetty weir sand trap system [Fig 7]

Three model similitude criteria were employed

Froude Criteria $n_v = n_u = n_d^{1/2}, n_H = n_d, n_T = n_d^{1/2}$

where n = scale ratio = value of model/value of prototype
 v and u designate respectively current (tidal or river) velocity, and orbital velocity (wave)
 d , H , and T designate vertical depth, wave height, and wave period

Ideal Velocity Criteria

$$n_v = \frac{n_\Delta n_D}{n_\mu}^{1/2} \quad n_{C_c}, n_u = n_v / n_{C_c} \quad n_H = n_u n_T \quad n_T = n_d^{1/2} \quad (\text{Bijker, 1967 pp 128-129})$$

where Δ , D , μ and C designate respectively relative apparent density of sediment, mean grain diameter, ripple coefficient, and resistance coefficient

Modified Froude/'Ideal Velocity' Criteria

$$n_v = \frac{n_{\Delta} n_D^{1/2}}{n_{\mu}} \quad n_{C_c}, n_u = n_d^{1/2}, n_H = n_d, n_T = n_d^{1/2}$$

Use Froude Criteria for wave characteristics, and 'Ideal Velocity' Criteria for current velocity (tidal and river flow)

The primary objectives were to

- determine the appropriate time scales of sediment deposition, frequently referred to as time scales of bottom evolution,
- determine the jetty effectiveness in each case,
- determine annual maintenance dredging requirements after the jetty is built,
- determine the effect of by passing when the jetty is built

Froude Law Scaling tends to distort sediment transportation similarity particularly in areas where oscillatory wave action is absent. However, this wave action tends to reduce the critical shear stress and/or velocity necessary for initial movement of material in the zone of littoral drift over that without wave action. For this reason, Froude Law Scaling may still be reasonable for use in wave induced sediment transportation.

Ideal Velocity Scaling relationship is established by Byker (1967) with due consideration of shear stress similitude for both tidal/river currents and wave orbital velocities toward reducing similarity distortion likely when Froude velocity scaling is used. This criterion requires that both the tidal/river and wave orbital velocities be exaggerated, hence the height of the waves and of the jetty which obstructs them must be exaggerated.

Modified Froude Law/Ideal Velocity scaling as established by the Look Laboratory is designed to reduce the wave height and model exaggerations necessary when 'Ideal Velocity' scaling is used. The tidal/river current velocities required in 'Ideal Velocity' scaling and the waves required in 'Froude Law' scaling are used in the 'Modified Froude Law/Ideal Velocity' scaling. The similitude relationship of these criteria is shown in Table 5.

Froude Law scaling criteria were applied to the tests in which sand was used as sediment. Lack of time and inadequate wave-generator capacities precluded application of two other scaling criteria to tests with sand. However, all three criteria were applied in tests using ground walnut shells as sediment material (in two grain sizes, $D_{50}=0.41$ mm and 0.90 mm, respectively). However, when larger walnut shell material was used, the desirable similitude relationship for 'Ideal Velocity' criteria and 'Modified Froude Law/Ideal Velocity' criteria were adjusted as shown in Table 5. It was impractical to perform required exaggerations in wave height and tidal/river velocities because of limitations in the wave generator.

Test Procedure Excitation was by normal and severe storm waves from the NE, on the existing channel with or without upshore east jetty protection.

Because the rate of sediment transportation and deposition of ground walnut shell material is much faster than that of sand, the test was correspondingly shorter, i.e. periods of much less than a complete tide cycle, but including portions of both ebb and flood tides. After a view of the tidal/river current pattern obtained previously with sand for Plan II, it was concluded that there are ebb currents in the channel during approximately two thirds of a tide cycle, and flood currents during the remaining one-third period. The sediment deposited in the channel during a portion of tide cycle and during operation of ebb and flood currents (for 15 and 10 minutes respectively) was measured. These test results were used to extrapolate the anticipated deposition over a complete tide cycle (on a basis of proportional contributions, two third ebb current and one third flood environment).

The time scale for each case is dependent on the amount of sediment passing over the channel. Every effort was exerted to avoid a condition in which the model channel would be filled to capacity; therefore, there was unnecessary by pass of bed load material. Tests of short duration have resulted in negligible by passing over the channel during normal and severe storm wave conditions. The time scale under no bypassing condition was used to predict sediment deposition in the channel. Furthermore, deposition by waves from N and NNW was estimated using depositions experienced in earlier tests with sands. This is considered feasible because the 'BEST PLAN' does not provide significant protection against N and NNW waves, duration is significantly short.

Table 5 MODEL SCALING CRITERIA AND MODEL SEDIMENT AS APPLIED TO RESPECTIVE CRITERIA

Parameter	Froude Scaling (1)			'Ideal Velocity Scaling (2)			Modified Froude/ Ideal Velocity Scaling (3)		
	Sands	Small Walnut Shells	Large Walnut Shells**	Sands*	Small Walnut Shells	Large Walnut Shells**	Sands*	Small Walnut Shells	Large Walnut Shells**
scale ratio	(D ₅₀ = 0.22 mm)	(D ₅₀ = 0.43 mm)	(D ₅₀ = 0.90 mm)	(D ₅₀ = 0.22 mm)	(D ₅₀ = 0.43 mm)	(D ₅₀ = 0.90 mm)	(D ₅₀ = 0.22 mm)	(D ₅₀ = 0.43 mm)	(D ₅₀ = 0.90 mm)
n	1/250	1/250	1/250	(1/250)	1/250	1/250	(1/250)	1/250	1/250
Depth	1/50	1/50	1/50	(1/50)	1/50	1/50	(1/50)	1/50	1/50
Wave height	1/50	1/50	1/50	(1/11)	1/28	(1/12)	(1/50)	1/50	1/50
Wave period	1/7.07	1/7.07	1/7.07	(1/7.07)	1/7.07	1/7.07	(1/7.07)	1/7.07	1/7.07
Current Velocity	1/7.07	1/7.07	1/7.07	(1/1.8)	1/4.5	(1/2.0)	(1/1.8)	1/4.5	(1/2.0)
Orbital Velocity	1/7.07	1/7.07	1/7.07	(1/1.6)	1/3.9	(1/1.99)	(1/7.07)	1/7.07	1/7.07
Jetty Height or Elevation	1/50	1/50	1/50	(1/11)	1/28	(1/12)	(1/50)	1/50	1/50

Note * Tests were not conducted for 'Ideal Velocity scaling and Modified Froude/Ideal Velocity scaling with sands
 ** The parameters as shown in the parenthesis represent the theoretical values required for the large walnut shells but were modified due to simulation equipment limitations. The modified criteria will enable the study of size effects on quantitative results of sediment deposition

Summary of Phase II Test Results Figure 11 was prepared to facilitate comparison between the distribution of sediment deposition along the existing channel in the model with no jetty protection and related measurements in the actual channel before and after Typhoon Bess. All deposition quantities are normalized to represent average one day deposition during Typhoon Bess, considering both volume and respective time scales as indicated. It was found that

- (a) Distribution predictions made by use of the Froude Law Criteria, based on results with large walnut shell material, tend to be fair for the Typhoon Bess period
- (b) Distribution predictions tend to be fair, under the normal wave condition over a long period of time (for example, one year), either by use of Froude Law Criteria based on results with sands, by use of the "Modified Froude Law/Ideal Velocity" scale criteria or the "Ideal Velocity" scale criteria (as modified) based on results with large walnut shell material, or by use of "Ideal Velocity" scale criteria based on results with small walnut shell material
- (c) Quantitative prediction by use of the Froude Law scaling criteria based on results with sands, tends to be fair when one considers only comparison of the model time scale of bottom evolution with theoretical value. However, it is expected that quantitative prediction should be reasonably good if the appropriate time scale of bottom evolution or deposition factor is applied for each test (based on respective scaling criteria with either sands or walnut shell material)

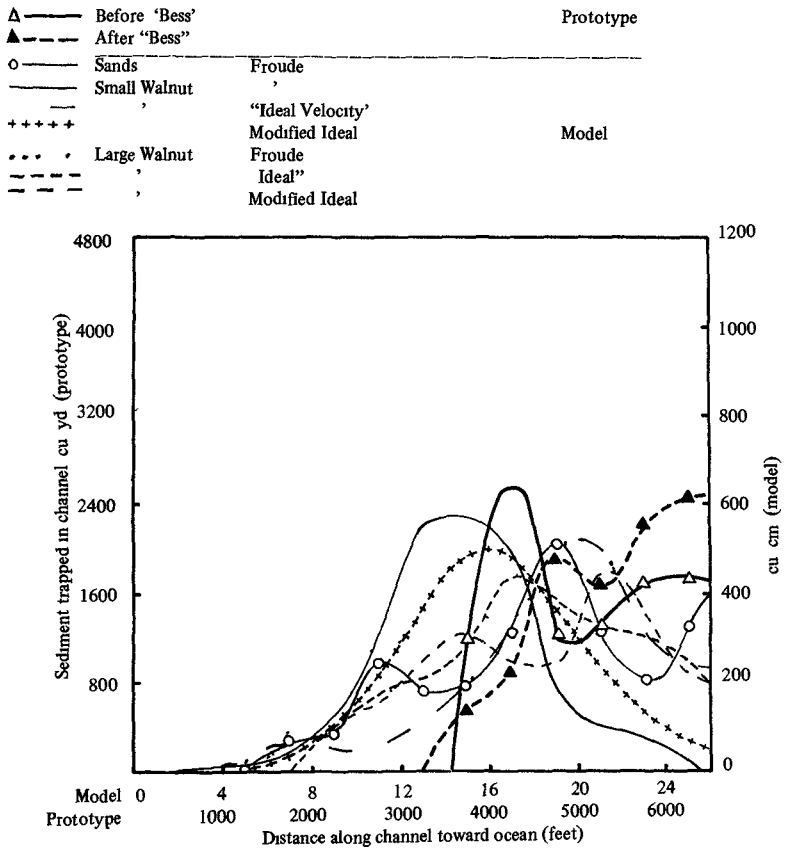


Fig 11 Model-Prototype Correlation

Time Scale Determination Time scales (the ratio of model time to prototype time for equal prototype sediment transport) were evolved for normal storm wave excitations using the three model scaling criteria and appropriate deposition in the model Measurement of the actual shoaling which occurred during Typhoon Bess is the basis for time scale determination for normal storm wave condition

As shown in Table 6, the time scales of bottom evolution vary significantly because the scale, at minimum is a function of scale factors of grain diameter, apparent density and wave height distortion However, they did not compare well with the theoretical values predicted using methods of either Manohar (1962), or Waterways Experiment Station (Fan and LeMehaute, 1969) The significant differences between theory and model results obtained with walnut-shell material raise the question of whether or not the theory is valid for the model scaling criteria used Further, the theory did not take the tide/river current scale effects into consideration in the formulation Further research is needed to determine the feasibility of using the theoretical time scale of bottom evolution in a movable bed study when no prototype quantitative data are available

Table 6 COMPARISON OF TIME SCALE OF BOTTOM EVOLUTION BETWEEN MODEL AND THEORETICAL VALUES

Time Scale of Bottom Evolution n_{tb}	Sand		Ground Walnut Shells				
	Froude	Froude	Small Grains		Froude	Large Grains	
			Ideal	Modified Froude/Ideal		Ideal (Modified)	Modified Froude/Ideal
Model	1/7.5	1/4820	1/6070	1/5400	1/3160	1/3585	1/2450
Theory *							
Manohar (1962)	1/97	1/410	1/1330	1/410	1/850	1/2750	1/2750
WES (Fan & LeMehaute 1969)	1/115	1/600	1/1940	1/600	1/1030	1/3340	1/3340

Effectiveness of the Jetty in Reduction of Maintenance Dredging Requirements It is predicted from the test results that the jetty will significantly reduce sedimentation in the existing channel (Table 7) The predicted reductions vary due to NE waves, from 82 to 99 percent when compared to sediment deposited in the unprotected channel with an average improvement of 91 percent The 87 percent reduction predicted by use of the Froude Law scale criteria (based on results with sands) is relatively conservative

Predictions of annual reduction by the jetty of sedimentation into the channel by waves from NE, N and NNW directions are also included in Table 7 (For greatest rigor, these require considerable prototype data, e.g. for calibration of the time scale of bottom evolution) Reductions in sedimentation are predicted on the order of 74 to 84 percent to the unprotected channel with an average value of 79 percent Deviations of about 5 percent may be expected between predictions based on the three scaling criteria It is concluded that the predictions (by use of the Froude Law scale criteria, based on results of tests in the model with sands) of quantities of sand deposits are adequate for engineering purposes

Table 7 PREDICTED "BEST PLAN PERFORMANCE (Based on Model Tests)

	Sand		Ground Walnut Shells				
	Froude	Froude	Small Grains		Froude	Large Grains	
			Ideal	Modified Froude/Ideal		Ideal (Modified)	Modified Froude/Ideal
Silting Reduction (NE Waves)	87%	86%	82%	93%	99%	94%	99%
Silting Reduction (all waves)	77%	76%	74%	81%	84%	81%	83%

* Manohar (1962) $n_{tb} = \lambda^2 \mu^{3/2} n_D^{-1} n_\gamma^{-1/2}$

WES (Fan & LeMehaute, 1969) $n_{tb} = \lambda^2 (n_H/\mu)^2 \mu^{3/2} n_D^{-3/4} n_\gamma^{-3/4}$

Effect of Bar By Passing Mechanics after Jetty Construction A prolonged test of the existing channel protected by a 6,000 foot long upshore East jetty (Plan IVa, Fig 7) was conducted using ground walnut shell material (small) as sediment. The purpose was mainly to study the vitally important process by which sediment bypasses around the jetty causing shoals to develop seaward of the protected channel area. These tests determine the usefulness of the improved channel system, including the equilibrium condition of updrift bar formation. By passing from the updrift to the downdrift, and bar development outside the channel entrance were also considered. The initial shoreline for the coast area updrift (East) of the jetty is considered to be the one formed by, and in equilibrium with excitation by severe NE storm waves.

The base for the study of by pass and off-entrance shoaling in which the excitation was Test Condition I was as follows: NE waves 5 ft/5 sec, and 500 m³/sec river flow, 1.8 ft/sec tidal current in one diurnal tide cycle (44 minutes in model), increments up to three tide cycles. As a result, a new equilibrium shoreline was formed as shown in Fig 12(a). Bottom profiles were taken through the beach area (Section A-A), around the channel entrance (Section B-B), and along the existing channel (Section C-C), as shown in Figure 12 (c), (d), and (e), respectively. These permit evaluation of the shoaling and by passing process as a function of time. It is interesting to note that the new equilibrium shoreline forms parallel to the incident NE wave and establishes after two tide cycles. This is equivalent to approximately 280 days in the prototype, using a time scale for bottom development of 1/4820 based on Froude Law scale criteria for no by passing.

Heavy shoaling developed upshore, and near the center of the 6,000 foot long jetty [Fig 12(c)]. A significant bar did not develop outside the channel entrance of the protected channel even after one additional tide cycle beyond the time necessary for the beach to attain equilibrium [Fig 12(d)]. This is perhaps controlled by the scouring effect of the high currents present offshore of the end of the jetty, and by the onshore component of the wave action which tended to curtail littoral drift, and caused deposition of sediments in other areas as noted in Fig 12(a). However, a bar of about 160,000 cubic yards did develop inside the channel entrance, with a peak at approximately 750 feet upstream from the end of the jetty. This bar represents about a year (420 days) of accumulation in the protected existing channel, with no dredging. It seems unlikely that the large bar would develop should the channel be maintained reasonably by periodic dredging.

Action of the waves, other than the normal NE storm waves, studied in these special tests (i.e. waves from N and NNW and severe NE storm waves) should tend to reduce, by erosion, the sediment trapped upshore of the jetty. This would act as a storage area for sediment driven into the area by NE waves, during the normal NE monsoon season. Thus, updrift of sediment into the channel should be inhibited.

As a result of these tests, it is concluded that

- 1 The effectiveness of the improved channel system (Plan IVa—6,000 foot long jetty upshore of the existing channel) can be maintained by dredging at a minimum level (21 percent of the original estimate without jetty protection)
- 2 Seasonal changes of wave action will help to maintain the effectiveness of the jetty. In particular, they will scour the upshore beach and make it a more effective trap during monsoon season. Thus, excessive dredging will not be required after the monsoon season. Delay in ship operation will be reduced to a minimum.

Findings and Conclusions of Phase II Study A 6,000 foot long jetty along the upshore bank of the existing channel is most effective and economical, with reasonable initial cost and minimum maintenance dredging requirements.

It was found that a jetty as postulated in the "BEST PLAN" would reduce the annual maintenance dredging required in the Tan My Inlet navigation channel (caused by NE wave induced shoaling) by 82 to 99 percent. This had been predicted with different model scaling criteria and types of sediment materials. A 10 percent error (5 percent on annual basis), attributable to scale effects, may be expected. No attempt was made to determine the relative merit of conflicting model scaling criteria. By consideration of the appropriate time scale of bottom evolution, one can obtain nearly the same quantitative results of annual dredging requirements. The "Ideal Velocity Scaling" with small walnut-shell material tends to predict most conservatively.

As far as the distribution of sediment deposition along the channel is concerned, none of the scaling criteria gave comparable distribution to that found in the prototype using Typhoon Bess conditions. However, model results (with sands using Froude Law scaling, with small walnut-shell material using 'Ideal Velocity' scaling, and with large walnut-shell material using "Modified Froude/Ideal Velocity" scaling or "Ideal Velocity," as modified) tend to give fair to good comparison of sediment distribution over long periods of time under normal conditions.

From the viewpoints of time consumption for a model study, the use of light weight sediment materials (such as the ground walnut shells used in this study) may be more economical than sands.

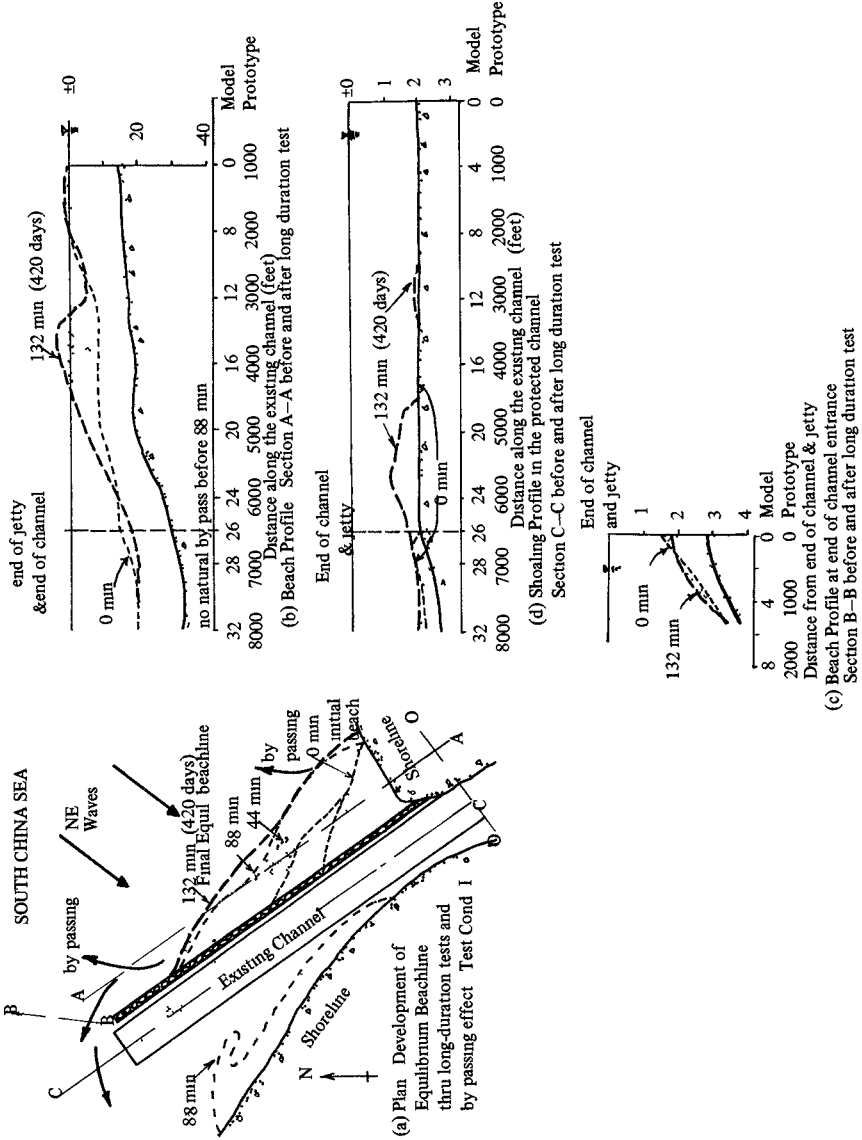


Fig 12. Prolonged Tests on By passing Characteristics of BEST PLAN IVa

Further research work on the model technology of a movable bed model is needed to clarify the uncertainties encountered in this study. Furthermore, an important point on the necessity of by pass operation should be discussed. Periodic by pass of the littoral drift that accumulated upshore of the jetty may be desirable only if the protection of the downdrift beaches from erosion is of vital importance and only when the value of the beaches is sufficient to justify the cost of such a by-pass operation. On the other hand, where preservation of the downdrift beach is not of major importance, as in the case of Tan My Inlet, the suggested "BEST PLAN" should be adequate to reduce substantially the maintenance dredging required in the navigation channel by trapping a significant portion of the predominant littoral drift on the updrift side of the jetty and by encouraging natural by passing of some of the untrapped portion.

Phase III Stability Characteristics of Rubble Mound Type Jetty of the BEST PLAN Phase III covered the two dimensional model studies which were conducted at a much larger scale in a 180 foot long wave flume. The objective here was to determine the stability characteristics of the rubble mound type jetty of the 'BEST PLAN' against storm wave action. From these tests, the jetty design was optimized. Tests and evaluations were made on the

basic designs of Fredric R Harris Inc for Jetty Sections IV and V. From these test results, the design was improved for use in this case (Fig 13).

Scale dimensions for tests were 1/20, 1/23.4 and 1/24.5. The tests were conducted with a 1/60 beach slope covered with sands (or walnut shells) as an erosible bed. Stone weight scaling followed the similitude by Hudson (1959) i.e.

$$\frac{(W_r)_m}{(W_r)_p} = \frac{(\gamma_r)_m}{(\gamma_r)_p} \left(\frac{L_m}{L_p} \right)^3 \left(\frac{(S_r)_p - 1}{(S_r)_m - 1} \right)^3$$

W_r = weight of stone

γ_r = specific weight of stone

$$S_r = \frac{\gamma_r}{\gamma_w}$$

γ_w = specific weight of water

The jetty was trapezoidal in cross section with 90 foot base, 15 foot wide crown, 24 feet high, with side slopes of 1 on 1.25. It rested in water 21 feet deep. Design configuration was rubble type rock in double layer at 6.4 or 10.3 tons each on the outside (armor), double layer at 0.3 tons each (as secondary cover), and quarry run of 2 inch size as both core and 4 foot thick base. It was intended to be overtopped by the design waves which, face on, were 14 feet high, with an 8

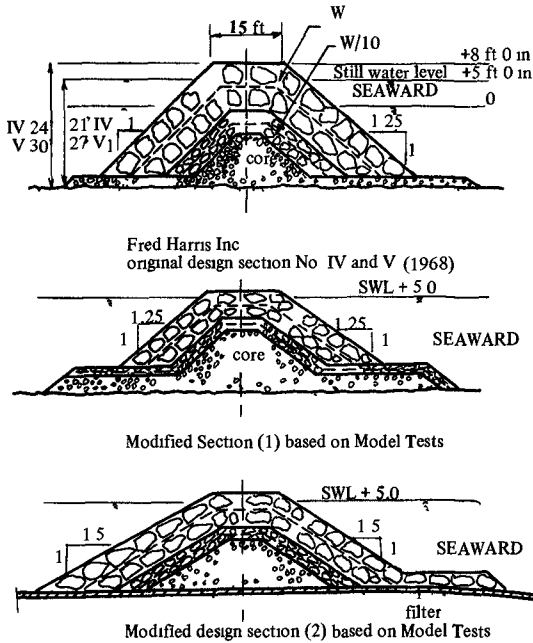


Fig 13

second period. However, toward conservatism, the jetty was tested in waves up to 20 feet high with a 10 second period.

Major findings were as follows:

1. A slope of 1:1.25 is acceptable for the 14 foot high design waves.
2. An armor stone size of 6.4 tons (for Jetty Section IV) and of 10.3 tons (for Jetty Section V) is adequate for the 14 foot high design waves under either low or high tide and storm surge conditions.

- 3 To prevent scouring under the jetty a layer of core materials should be extended beyond the armor stone toe with two layers of secondary stones placed over this layer of core material for protection An alternate for protection against sand scour is a plastic filter (such as Poly Filter X) extended in front of the jetty and under the sand with secondary stones above the sand as protection
- 4 A filter layer, preferably of plastic, is necessary in the prototype between the core material and the sand beach This could not be demonstrated in the model due to the effect of the scale, i.e., larger voids in the prototype than those in the model
- 5 Further testing of scouring depth at the jetty toe is necessary because of the discrepancies found between test results with sand and those with walnut shell material as erosible bed materials
- 6 The stability of the rubble mound jetty should be studied in a movable bed model particularly if the jetty is founded in a depth shallower than the critical depth for sand movement Otherwise the designer should specify that the jetty bottom should either extend below scour depth to be determined, or it should rest on a plastic filter
- 7 The filter system for a jetty should be designed with extreme care, particularly if the transitory currents induced by tidal and river flow along the longitudinal axis of the jetty are significant

CONCLUSIONS

The major conclusions of this study were

- 1 A 6,000 foot long rubble mound type jetty along the east bank of the existing channel [Fig 7] should be considered the 'BEST PLAN'—the most effective and economical solution of the exterior channel improvement at Tan My Inlet The rubble mound type jetty should be constructed of 6 4 to 10 3 ton armor stones with two layers of secondary stones and core materials laid underneath it A filter system must be carefully designed to avoid any failures due to erosion of the jetty foundation and toe
- 2 A semi movable bed model study of the inlet stability improvements, following the three phase experimental procedure reported herein, may be considered a feasible method by which to obtain both qualitative and quantitative results this is particularly so when sufficient prototype data are available to relate the time scale of bottom evolution
- 3 A jetty as postulated in the 'BEST PLAN' would reduce the annual maintenance dredging required in the Tan My Inlet navigation channel (caused by NE N, and NNW wave induced shoaling) by 74 to 84 percent This was predicted with different model scaling criteria and types of sediment materials By consideration of the appropriate time scale of bottom evolution, one obtained nearly the same quantitative results of annual dredging requirements 'Ideal Velocity' scaling with small walnut shell material tended to predict most conservatively

As far as the distribution of sediment deposition along the channel was concerned none of the scaling criteria gave distribution comparable to that found in the prototype using Typhoon Bess conditions However model results (with sands using Froude Law scaling with small walnut shell material using "Ideal Velocity" scaling and with large walnut shell material using 'Modified Froude/Ideal Velocity' scaling or 'Ideal Velocity,' as modified) tended to give fair to good comparison of sediment distribution over long periods of time under normal conditions

From the viewpoint of time consumption for a model study, the use of light weight sediment materials (such as ground walnut shells) may be more economical than use of sands Further research work on the model technology of a movable bed model is needed to clarify the uncertainties encountered in this study

- 4 Use of theory alone in predicting the time scale of bottom evolution should be done with caution, especially in cases where field data are not readily available for model prototype correlation This conclusion was quite obvious, at least for this study Further verification is needed
- 5 The quality of the model study is still greatly dependent on the quality and quantity of environmental data collected When adequate data are available to permit the determination of the time scale of bottom evolution the discrepancy between the results obtained with different scaling criteria and model sediment was not too large as one would expect normally Of course there are still many research problems involved in a movable bed model

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