CHAPTER 95

SCOUR AROUND MODEL PIPELINES DUE TO WAVE ACTION

by

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ABSTRACT

The size, number and application of offshore pipelines are steadily increasing. At the same time, the incidence of reported pipeline failures is also increasing. There appear to be several reasons for these failures, and they can be placed in two basic categories:

1. inadequate cover, and
2. low "specific gravity" of the pipeline.

Under the first category the depth of burial may be insufficient, the type of burial material may be inferior to the material alongside the trench, or the compaction of cover material may be inadequate. Under the second category the pipe may actually float up to the surface from the ocean bottom as material around the buried pipe liquifies.

An extensive literature search revealed that many studies were conducted by Meyers (1936), Waters (1939), Johnson (1940), W.E.S. (1940), Rector (1954), Wiegel, et al (1954), Saville (1957), Iwagaki and Noda (1962), Nayak (1972), Noda (1972), and Earattupuzha (1974). In general, two types of "equilibrium profiles" were developed in the laboratory flumes, the "ordinary" and the "storm" (sometimes referred to as summer and winter profiles). Despite numerous previous investigations, knowledge of the "scale effects" involved in equilibrium beach profiles is inadequate. Many authors have analyzed model data without stating the relation between model and prototype dimensions. In addition, many have claimed certain phenomena observed in the model to be independent of initial slope.

An extensive laboratory study was conducted to evaluate the development of underwater bars and scour patterns with the pipeline buried at various depths below the ocean bottom. Pictures of the beach profile were taken at specific time intervals through the glass wall of the wave tank.

Attempts were made to correlate equilibrium profile geometric quantities, such as depth of offshore bar, scour depth and berm height with the wave characteristics. Qualitative agreement between laboratory and natural beach profiles were demonstrated by trial and error fitting of one to the other.

INTRODUCTION

In more recent times, the energy crisis has caused acceleration of exploitation and exploration of the oil and gas reserves under the continental shelf. The most economical way of transporting oil and gas is through pipelines from offshore platforms or underwater completions to shore.

In another widespread application of pipelines, environmental considerations and regulations call for the disposal of municipal and industrial waste in deeper water, both in lakes and oceans, than previously. Here, as in all engineering design work, a better understanding of soil-pipe interaction and magnitudes of various forces acting on an unburied pipe is required before good design criteria and engineering practices can be well defined.

Experimental studies, mainly in the laboratory, but some in the field, have been conducted in many countries, notably in the U.S.A., the Netherlands and Japan.

While the petrochemical industry is evaluating pipeline failures and design practices, such technical information is generally restricted and not available. Since design engineers have been made more conscious of failures in more recent years, pipelines are sometimes over-designed and excessively expensive to construct. Over the past 10 or so years, the incidence of reported submarine pipeline failures, particularly of large diameter pipelines, has increased markedly.

The American Society of Civil Engineers appointed a Task Committee on Pipelines in the Ocean in 1969 to define the problems of pipelines in the oceans and to determine the state-of-the-art in the design and construction of pipelines in the oceans.

This effort, which has not been sponsored financially, has attempted to define the problem and review the current state-of-the-art in the areas of environmental factors, design factors and construction factors. The deficiencies were uncovered in the state-of-the-art[1], in

    (a)   structural and external pressure effects,
    (b)   depth of burial,
    (c)   economics of submarine pipelines, and
    (d)   documentation.

The current project deals with the aspects described in (a), (b), and (d).

The American Gas Association has been sponsoring a study on offshore pipelines[2], particularly those in water depths greater than 200 ft. The conclusions of the study to date indicate that

    (a)   considerable additional data are required to more fully evaluate pipeline-soil interaction, and

(b)   field measurements of storm-driven bottom particle velocities
      and accelerations should be obtained.

Recent pipeline failures include the following:

(a)   One gas line, one telephone line and two water lines have moved
      upward through silty sand at the bottom of a ship channel from
      an elevation -51 ft to elevation -41 ft. These lines were sub-
      sequently cut by maintenance dredging operations.

(b)   One 10-ft diameter steel bitumen-coated pipeline failed three
      times during construction. The failure, which occurred during
      one major and two minor storms, was probably due to liquifaction
      of the silty sand sediment around the pipe. One section of the
      pipe, weighing about 80 tons was found some 150 ft away from the
      trench in which it was placed and partially backfilled.

(c)   One 48-inch sewer outfall pipe failed during its first year of
      operation. Higher than design wave forces are thought to be
      responsible for failure. The pipeline was unburied.

(d)   One 48-inch water intake pipe was damaged and three sections of
      it displaced. The pipe was buried and an analysis is being made
      as to the possible causes of failure.

(e)   One 60-inch intake pipe at a nuclear power plant failed before
      the plant was placed in operation. Scour of sediment around the
      pipe exposed the buried pipe which was then subjected to forces
      for which the pipe was not designed.

     The information on pipe failures is difficult to obtain as construc-
tion companies or owners are reluctant to discuss or even admit failure
occurred. The problem is complicated by the fact that disputes arise as
to who is responsible for failure -- the designer or the contractor. It
is not unusual for some cases to be taken to court. It is hoped that per-
mission will be obtained to eventually summarize and document recent pipe-
line failures.

LITERATURE REVIEW

     The relationship between many variables which govern beach deforma-
tion are quite complex as was pointed out by many researchers studying
beach formation, beach erosion, scour around piers and piles, and in front
of seawalls, etc. A large number of variables and their mutual interaction
further complicate analysis of experiment data.

     Assuming a two-dimensional approach, a review of the literature re-
vealed numerous studies of model beach profiles conducted by Meyer[3] (1936),
Waters[4] (1939), Johnson[5] (1940), Rector[6] (1954), Wiegel, et al[7] (1964),
Saville[8] (1957), Iwagaki and Noda[9] (1962), Nayak[10] (1972), Noda[11] (1972),

and Earattupuzha[12] (1974). Figure 1 shows a classification scheme developed by Sunamura and Horikawa[13] (1974) for model beach profiles. Type II profile was generally observed in this study. Despite numerous previous investigations, knowledge of scale effects is inadequate.

A literature survey relating to scour and stability of submarine pipelines was conducted and reported by Ralston and Herbich[14] (1968). The present study is partially an outgrowth of previous work which was supported in part by the U.S. Corps of Engineers. An additional survey dealing with the design of offshore pipelines was conducted in 1971-73 and reported by Manley and Herbich[15] (1976).

Coastal deformations can be divided into two general groups: long-term and short-term. Long-term changes are those changes in a coastline which occur over hundreds of years and result in a general prograding or recession of the shoreline. Of more interest to the coastal engineer is the short-term change which is associated with the variable wave climate and resulting sediment motion. Wave induced sediment motion can be divided into two components: alongshore motion and motion normal to the coast. This paper is concerned with the onshore-offshore motion.

As waves progress onto a beach sediment motion occurs. The magnitude of the sediment transport depends on the wave characteristics. The resulting changes in the beach have a feedback control on the incident waves. For example, changes in depths caused by breaking waves require changes in location of the breaking waves. Intuitively, it would seem that when a given beach is subject to waves of constant characteristics for a sufficient length of time, an equilibrium state will develop. In nature, the variable meteorological conditions and resulting variable wave conditions probably seldom allow an equilibrium to be attained. In the hydraulic model, where one has control over wave parameters, such as wave height and wave period, and beach parameters, such as grain size and size distribution, the concept of the equilibrium profile can be more readily studied. Thus, in nature the equilibrium profile needs to be defined in terms of statistical averages. However, in the laboratory the equilibrium profile is defined as a stable configuration in which the oscillatory motion of a given sediment particle is about a mean position, the sorting action of waves presumably having reached an equilibrium. The net transport across any section parallel to the beach is zero.

The objective of this part of the study was to determine, through physical modeling, the effect of storm waves on buried pipelines approaching the shoreline. Scour depth and scour patterns have been evaluated in a two-dimensional wave tank. Three-dimensional effects have also been studied in a larger wave basin. The number of variables included wave characteristics such as height, period, wave crest direction, water depth, pipe burial depth, and beach slope. Preliminary results on three-dimensional effects indicate that local scour may be quite significant when pipes are placed at an angle to the approaching waves.

TYPE I

SWL

INITIAL SLOPE

TYPE II

TYPE III

ARROWS DENOTE POSSIBLE DIRECTIONS OF NET SAND
TRANSPORT.

FIGURE 1.   NEW BEACH PROFILE CLASSIFICATION.
AFTER SUNAMURA AND HORIKAWA (1974).

Offshore
Zone
Nearshore
Zone
Foreshore
Zone

break
point

beach
crest

C_{max}

wave
direction

h_c

h_t

initial slope

offshore bar

T_{max}

final profile
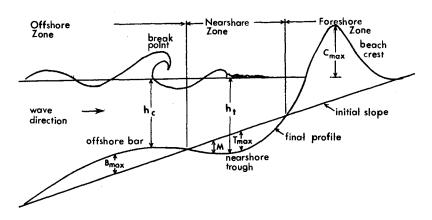
M

B_{max}

nearshore
trough

FIGURE 2.   NOTATION USED IN THE STUDY.

(FROM REFERENCE 16)

EXPERIMENTAL STUDY

Experimental studies were conducted to assess the required depth of burial for a pipeline through the surf zone. The problem was approached by using a two-dimensional beach profile. Beach profiles were allowed to come to equilibrium in a 2-ft wide wave tank. Two-dimensional profiles were recorded for a number of wave conditions and initial slopes of 1:10, 1:20, 1:30 and 1:60. The 1:60 slope study was limited because of instability of profiles associated with minor water level variations as observed by Smith[16] (1975).

A sample dimensionless plot of beach profiles for the slope of 1:20 is shown in the Appendix (Fig. A-1).

Attempts were made to correlate equilibrium profile geometric quantities, such as the maximum depth of the nearshore trough (T_{max}) and maximum height of the beach crest (C_{max}) with the wave characteristics. Figure 2 indicates the notation used in this study. It was found that wave height varies spacially and temporally in the tank making it an unsuitable independent variable for determining linear regressions. Deep water wave length (a function of wave period), however, was found to exhibit constant characteristics throughout a given test. Thus, the deep water wave length was used as the independent variable in the linear regression analysis.

Of the four attempted regressions, three were determined significant at the 5% significance level. Table I shows the correlation coefficients and regression results[16].

Table I

L_o vs. C_{max}	r	r^2	ν	$r(.05, \nu)$	significant
1:20	.94	.88	8	.632	yes
1:30	.86	.75	14	.497	yes
L_o vs. T_{max}					
1:20	.708	.502	8	.632	yes
1:30	.002	.048	10	.576	no

Where r = correlation coefficients; ν = degrees of freedom = no. of obs. -2; $r(.05, \nu)$ = tabulated values of r at 5% significance level; r^2 = % variance explained by the linear relationship of C_{max} and L_o; C_{max} = maximum height of the beach crest above the still water level; T_{max} = maximum height of the nearshore trough below the still water level, and L_o = deep water wave length.

Confidence intervals were determined for the C_{max} regression slopes and intercepts. The 95% confidence intervals for the slope and intercept of the 1:20 data are: $b_1 = .014 \pm .003$, $a_1 = .085 \pm .028$. Similarly for

1:30 data, b_2 = .025 $\pm$ .006, a_2 = .030 $\pm$ .061. Based on a 2-tailed t-test it was determined that at the 95% confidence level the populations represented by the 1:20 and 1:30 data are significantly different. Thus, for the range of tested variables, the dependence of C_{max} on L_0 is not independent on initial slope of the model beach. The failure of the T_{max} regression for the 1:30 data indicates that possibly some of the 1:30 T_{max} values had not reached equilibrium. This is in agreement with earlier observations indicating that steeper beaches reached equilibrium in a shorter time.

Hence the following relationships were obtained for a 1:20 initial slope: C_{max} = .085 + .014 L_0, for a 1:30 initial slope; C_{max} = .030 + .025 L_0, for a 1:20 initial slope: T_{max} = .001 + .075 L_0.

Qualitative agreement between laboratory and natural beaches was demonstrated by trial and error fitting. Figure 3 shows such a fit. The required distortion of scales results in an unnatural repose angle at the foreshore. In addition, the wave parameters responsible for the natural profile are unavailable. However, the trial and error method is useful in determining a general scale factor. For example, a comparison between laboratory-obtained profiles and field profiles near Sabine Pass, Texas, indicate a horizontal scale of 1 to 25 and vertical scale of 1 to 8, or a distortion of about 3 to 1.

PIPELINE STABILITY

An offshore pipeline must have sufficient horizontal and vertical stability against all environmental and imposed forces. Although environmental and gravitational forces are of primary importance, constructional and operational loadings should also be taken into account.

The gravitational forces include the weight of the pipe (either steel or concrete), weight of the corrosion protection coating, weight of the concrete coating (in case of steel pipe), and weight of the fluid in the pipeline.

The environmental forces depend on the storm severity, location of the pipeline and water depth. They are quite complex and variable. The forces acting on the pipeline include those due to storm or hurricane waves, vortex shedding and foundation strength.

Since forces due to waves may be considerable, particularly in shallow water, the most obvious solution is to place the pipeline in an excavated trench and cover it with suitable material up to the original underwater beach profile. There is no easy answer to this question as the depth of cover required will depend on a great many variables, including the wave climate, sediment size, littoral current, liquefaction potential, etc.

Buried pipelines approaching and passing through the surf zone must be placed below the storm beach profiles, or below the "winter" profiles. Such profiles must be determined prior to the design of the pipeline's profile and selection of burial depth. It has been found that in many cases

STILL WATER LEVEL

---SERIES 3 TEST 5E t=13 HRS
T=.94 SEC H_O=.23 FT

——SABINE PASS
GALVESTON CORPS OF ENGR.
PROFILE PA-2 JUNE 1969

HORIZONTAL SCALE $\dfrac{X_M}{X_P} = \dfrac{1}{25}$

VERTICAL SCALE $\dfrac{Y_M}{Y_P} = \dfrac{1}{8}$
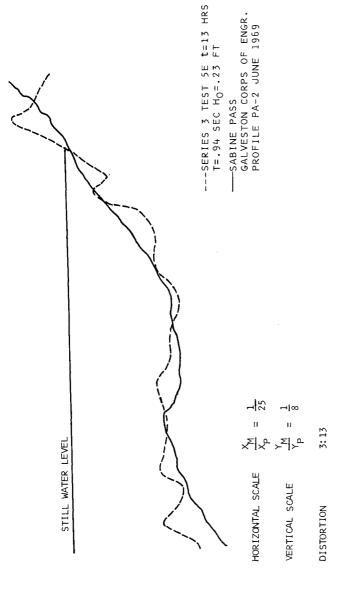
DISTORTION 3:13

FIGURE 3. COMPARISON OF EXPERIMENTAL AND NATURAL PROFILE.

offshore pipelines failed due to inadequate cover or due to the fact that
the pipeline profile generally followed the underwater beach profile ob-
served at one time of the year, sometimes only during the summer period
(or having a "summer" profile).

DEPTH OF COVER OVER PIPELINES

For safe design the pipeline should be buried under the storm beach
profiles. The actual depth of burial would depend on the storm frequency
selected, the importance of the project, and on possible environmental im-
pact should the pipe fail, etc. The forces induced by waves on the buried
pipe are generally not large, particularly as compared with unburied or
partially buried pipe. Figure 4 indicates the coefficient of drag (C_D),
coefficient of inertia (C_M) and coefficient of lift (C_L) on pipes located
above the bottom, touching bottom and partially buried (2). For example,
the forces due to drag on a pipe suspended 20 inches above the bottom are
five times as large as on the pipe partially buried (protruding 0.25 of
its diameter above the bottom). The effects of drag causing scour are
shown schematically in Figure 5.

In order to develop safe design criteria for pipe cover two types of
profile characteristic dimensions were considered:

1. (a) Maximum depth of trough below initial slope - T_{max},
    (b) Maximum height of offshore bar above initial slope - B_{max},

2. (a) Depth of trough below mean low tide - h_T,
    (b) Depth of bar below mean low tide - h_C.

Ideally, all measurements should be taken either during the storm or
as soon as practical after the storm. If no measurements are available,
laboratory developed "equilibrium" profiles may be used provided a dis-
torted scale is determined on the basis of laboratory-field comparisons.

Plots of h_T as a function of h_C have been determined from both field
and laboratory data (Figure 6). The experimental laboratory data for
beach profiles of 1 on 10, 1 on 20, and 1 on 30 are shown in Table II. A
summary of recent laboratory results indicates that beach slope affects the
ratio of h_T to h_C (Tables III and Figure 7). For example, the h_T/h_C value
for 1 on 10 slope is 1.80 with a 95% confidence interval of ± 0.21 while
the value for 1 on 30 slope is 1.35. Previously analyzed profiles are shown
in Figure 6.

Pipe Position with respect to Mudline		c_D	c_M	c_L
●	$L^* = 20''$	1.0	1.5	0.5
●	L = 3''–20''	.95	1.5	0.5
●	Pipe Touching Bottom	.75	1.5	1.0
●	L = 0.25 Diameter	.50	1.5	.85
●	L = 0.50 Diameter	.25	1.5	.75
●	L = 0.75 Diameter	.20	1.5	.20

*L = Distance from bottom of pipe to mudline.

Figure 4.   Recommended Drag (C_D), Inertial (C_M) and Lift (C_L) Coefficients for Different Pipe Positions with Respect to the Mudline (Reynolds Numbers > 200,000)

(from Reference 2)

Partial Scouring Under Fore Section   Partial Scouring Under Rear Section

Scour Zone

(a)   (b)   (c)

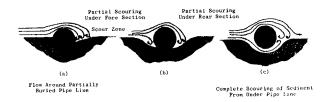Flow Around Partially Buried Pipe Line   Complete Scouring of Sediment From Under Pipe Line

Figure 5.   Drag Effects on Partially Buried Pipelines

(from Reference 2)
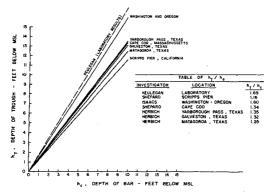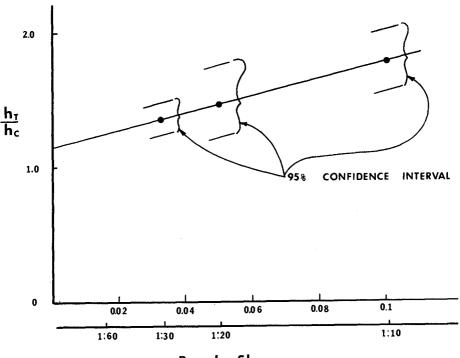
FIGURE 6. RELATIONSHIP BETWEEN DEPTH OF TROUGH AND DEPTH OF BAR

FIGURE 7. RATIO OF $\dfrac{h_T}{h_C}$ AS A FUNCTION OF BEACH SLOPE

TABLE II

EXPERIMENTAL TEST DATA

Series No. 1 Beach Slope 1:10

Test No	Depth of Trough h_T (in)	Depth of Bar h_C (in)	$\dfrac{h_T}{h_C}$	Time (hrs)
2B	3.7	1.7	2.18	13
2C	2.9	1.4	2.07	15
3C	2.2	0.9	2.44	11
4A1	3.2	1.9	1.68	5
4B1	3.3	1.8	1.83	15
4C1	3.8	1.6	2.38	13
4D1	3.2	1.8	1.88	13
4E1	2.6	1.6	1.63	13
5A1	3.6	2.4	1.50	13
5B1	3.1	2.9	1.07	13
5C1	3.8	2.0	1.90	13
5D1	3.2	2.0	1.60	13
5E1	4.2	3.1	1.35	13
6C2	8.4	3.5	2.4	7
6E2	2.4	1.2	2.00	7
7C2	3.7	2.5	1.48	7
7E2	3.3	2.8	1.18	7

Series No. 2 Beach Slope 1:20

Test No	Depth of Trough h_T (in)	Depth of Bar h_C (in)	$\dfrac{h_T}{h_C}$	Time (hrs)
1C	2.5	1.7	1.47	13
2C	1.5	0.7	2.14	13
3C	2.1	1.4	1.5	13
4C	4.3	4.1	1.05	13
5C	4.1	3.6	1.14	13
1E	3.7	1.9	1.95	13
2E	3.3	2.2	1.50	13
3E	3.4	1.9	1.79	13
4E	3.4	2.7	1.26	13
5E	3.5	3.1	1.13	13

Series No. 3 Beach Slope 1:30

Test No	Depth of Trough h_T (in)	Depth of Bar h_C (in)	$\dfrac{h_T}{h_C}$	Time (hrs)
4C	2.7	2.3	1.17	13
5C	4.5	3.4	1.32	13
4E	4.7	3.1	1.52	13
5E	4.8	3.6	1.33	13
6E	4.8	3.4	1.41	74
7E	5.9	4.4	1.34	48
8E	4.6	3.3	1.39	30

TABLE II (continued)

EXPERIMENTAL TEST DATA

Series No. 3 Beach Slope 1:30

Test No.	Depth of Trough h_T (in)	Depth of Bar h_C (in)	$\dfrac{h_T}{h_C}$	Time (hrs)
9E	4.7	3.9	1.21	65
10E	3.5	2.3	1.52	45
11E	3.6	2.2	1.64	13
12E	4.6	3.0	1.53	45
13E	4.8	4.2	1.14	3D
14E	6.4	5.5	1.16	30
15E	7.2	5.8	1.24	45

Series No. 6 Beach Slope 1:6D

Test No.	Depth of Trough h_T (in)	Depth of Bar h_C (in)	$\dfrac{h_T}{h_C}$	Time (hrs)
1X	6.4	4.8	1.33	77
1Y	6.1	5.6	1.09	77
4A	6.4	4.4	1.45	45
5A	8.2	6.6	1.24	96
6A	4.1	1.9	2.16	45
7A	5.0	3.8	1.32	30
1D	4.9	4.0	1.23	45
2D	5.5	4.8	1.15	145

TABLE III

SUMMARY OF RESULTS

Beach Slope	$\dfrac{h_T}{h_C}$	95% Confidence Interval
1:10	1.80	± 0.21
1:20	1.49	± 0.26
1:30	1.35	± 0.09

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     1975.

Table A-1[16]

1:20 Test Data

Parameter	Test									
	1E	1C	2E	2C	3E	3C	4E	4C	5E	5C
T(sec.)	1.55	1.55	1.45	1.46	1.26	1.27	1.11	1.11	.96	.96
L_o(ft.)	12.30	12.30	10.79	10.85	8.18	8.22	6.33	6.33	4.73	4.73
H_o(ft.)	.36	.35	.33	.35	.26	.27	.21	.20	.21	.21
d(ft.)	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75
t_{max}(hrs.)	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00
C_{max}(ft.)	.25	.25	.22	.25	.21	.21	.19	.18	.12	.15
T_{max}(ft.)	.17	.13	.13	.14	.12	.11	.13	.08	.11	.11

Table A-2[16]

1:30 Test Data

Parameter	1C	2C	2E	3C	3E	4C	4E	5C	6E	7E
T(sec.)	1.55	1.45	1.44	1.26	1.26	1.10	1.10	.95	1.42	1.69
L_0(ft.)	12.30	10.76	10.58	8.13	8.13	6.20	6.20	4.61	10.32	14.54
H_0(ft.)	.38	.35	.35	.27	.27	.21	.24	.22	.22	.35
d(ft.)	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75
t_{max}(hrs.)	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00	13.00
C_{max}(ft.)	*	*	.25	.17	.21	.15	.17	.14	.29	.38
T_{max}(ft.)	+	+	+	+	+	.10	.17	+	.05	.16

Parameter	8E	9E	10E	11E	12E	13E	14E	15E
T(sec.)	1.04	1.15	1.39	1.07	1.36	1.37	1.43	1.32
L_0(ft.)	5.54	6.77	9.82	5.81	9.40	9.61	10.47	5.99
H_0(ft.)	.20	.23	.26	.15	.34	.23	.32	.32
d(ft.)	1.75	1.75	1.75	1.75	1.75	1.75	1.75	1.75
t_{max}(hrs.)	30.00	65.00	45.00	13.00	45.00	30.00	30.00	45.00
C_{max}(ft.)	.15	.23	.31	.25	.33	.27	.29	.15
T_{max}(ft.)	.19	.20	.18	.15	.23	.20	.25	.24

* Beach crest equilibrium not indicated by profile sequences.

+ Nearshore trough equilibrium not indicated by profile sequences.

Test Series No. 2
Length of Test = 13 hrs.

FIGURE A-1

FIGURE A-2.   LINEAR REGRESSION OF MAXIMUM HEIGHT OF BEACH CREST ON DEEP WATER WAVE LENGTH FOR ALL 1:20 TESTS.
(FROM REFERENCE 16)

SLOPE = 1:30
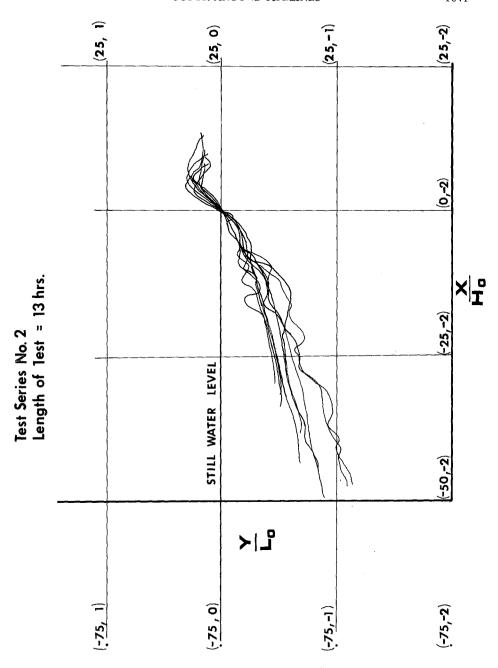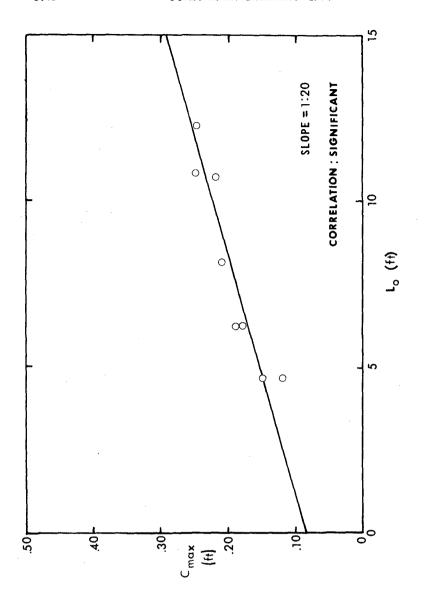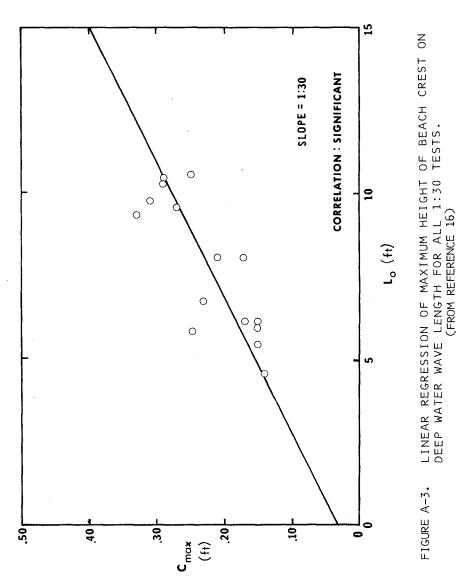
CORRELATION : SIGNIFICANT

L_o (ft)

C_{max} (ft)

FIGURE A-3.   LINEAR REGRESSION OF MAXIMUM HEIGHT OF BEACH CREST ON
DEEP WATER WAVE LENGTH FOR ALL 1:30 TESTS.
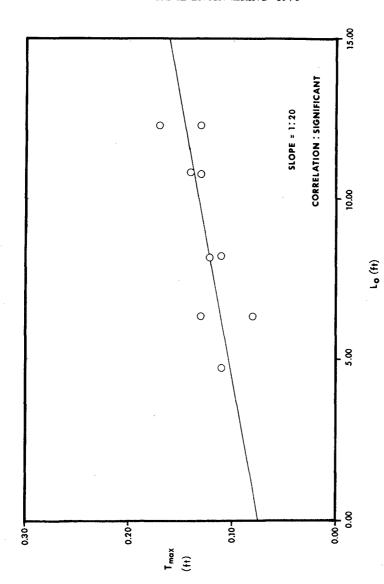(FROM REFERENCE 16)

FIGURE A–4.   LINEAR REGRESSION OF MAXIMUM DEPTH OF BEACH TROUGH ON
DEEP WATER WAVE LENGTH FOR ALL 1:20 TESTS.

FIGURE A-5. LINEAR REGRESSION OF MAXIMUM DEPTH OF BEACH TROUGH ON DEEP WATER WAVE LENGTH FOR ALL 1:30 TESTS.