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TECHNICAL REPORT H-77-20

PORT ONTARIO HARBOR, NEW YORK DESIGN FOR WAVE PROTECTION AND PREVENTION OF SHOALING

Hydraulic Model Investigation

by

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November 1977
Final Report

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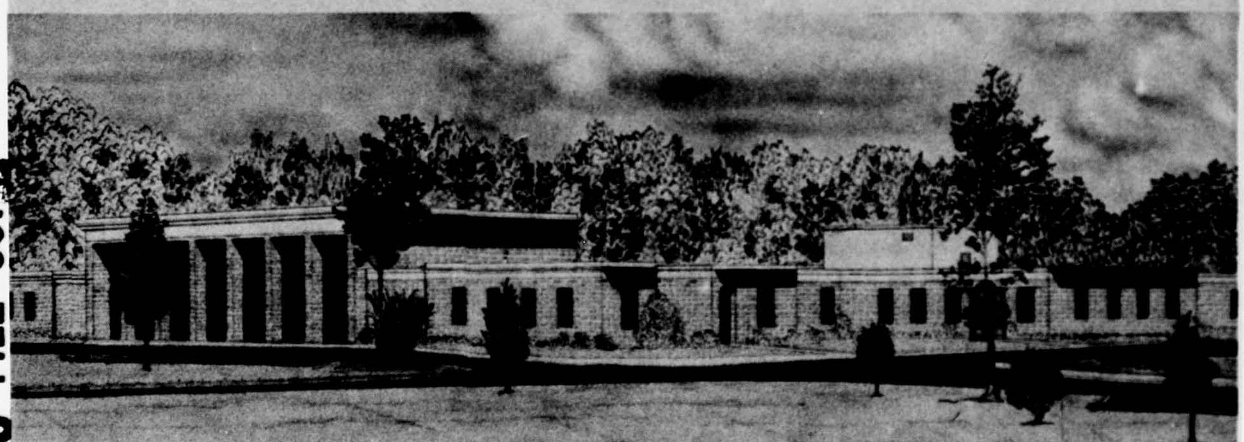

Prepared for U. S. Army Engineer District, Buffalo
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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A 1:75-scale undistorted hydraulic model of Port Ontario Harbor, New York, which included the lower 5,700 ft of the Salmon River, approximately 4,300 ft of shoreline on each side of the river mouth, Selkirk Lake, and sufficient offshore area in Lake Ontario to permit generation of required test waves, was used to investigate the arrangement and design of proposed improvements with respect to shoaling, wave protection, and riverflow conditions. Proposed improvement plans

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20. ABSTRACT (Continued).

consisted of (a) breakwaters in Lake Ontario at the mouth of the Salmon River; (b) an entrance channel; (c) an inner channel; (d) a harbor basin; and (e) recreational facilities. A 60-ft-long wave generator, a model circulation system, crushed coal tracer material, and an automated data acquisition and control system (ADACS) were utilized in model operation. It was concluded from model test results that:

a. Existing conditions are characterized by rough and hazardous wave conditions at the river entrance during periods of storm wave attack.

b. For existing conditions, wave action formed a shoal across the river mouth which interferes with navigation and the passage of riverflows.

c. Wave heights in the harbor were within the established wave-height criteria (2.0 ft at the river mouth and 0.5 ft in the mooring area) for all major improvement plans tested.

d. Of the improvement plans involving the first breakwater configuration and channel alignment tested (Plans 2-5), Plan 4 was selected as the optimum with respect to shoaling protection and construction cost.

e. Of the improvement plans involving the second breakwater configuration and channel alignment tested (Plans 7-10), Plan 7 was selected as optimum with respect to shoaling protection and construction costs.

f. Plan 4 provided better shoaling protection than Plan 7 and construction cost should be comparable.

g. Breakwaters installed for Plans 4 and 7 had little effect on water-surface elevations in the lower reaches of the river during riverflows and should not contribute to flooding in this area.

h. Reducing the crown elevation of the south breakwater from +12 ft (Plan 4) to +10 ft (Plan 11) will not increase wave heights between the structures for the wave conditions tested.

i. Sediment will probably build up along the breakwaters on both sides of the harbor for small everyday waves from 240° and 330°.

j. The larger storm waves will cause some natural bypassing of the harbor.

PREFACE

A request for a model investigation of Port Ontario Harbor, New York, was initiated by the District Engineer, U. S. Army Engineer District, Buffalo (NCB), in a letter to the Division Engineer, U. S. Army Engineer Division, North Central (NCD), dated 7 October 1975. The study was authorized by the Office, Chief of Engineers, in the 4th endorsement to this letter, dated 19 January 1976; and funds were authorized by NCB for the U. S. Army Engineer Waterways Experiment Station (WES) to conduct the study on 15 March, 7 July, and 12 October 1976.

The model study was conducted during the period November 1976-May 1977 in the Hydraulics Laboratory of WES under the direction of Mr. H. B. Simmons, Chief of the Hydraulics Laboratory; Mr. F. A. Herrmann, Jr., Assistant Chief, Hydraulics Laboratory; Dr. R. W. Whalin, Chief, Wave Dynamics Division; and Mr. C. E. Chatham, Jr., Chief, Harbor Wave Action Branch. The tests were conducted by Mr. Robert R. Bottin, Jr., Project Manager, with the assistance of Messrs. K. A. Turner, computer specialist; R. E. Ankeny, electronics technician; M. R. Tarleton, photographer; and L. D. Smith, student aid. This report was prepared by Mr. Bottin.

Prior to the model investigation, Messrs. Chatham and Bottin met with representatives of NCB and visited the Port Ontario Harbor site. During the course of the investigation, liaison between NCB and WES was maintained by means of conferences, telephone communications, and monthly progress reports.

Messrs. Larry Hiipakka and Charlie Johnson, NCD; Don Liddell, Denton Clark, Joe Hassey, Ken Hallock, Joseph Foley, and Dan Kelly, NCB; Alex Gronvall, Chief Marine Services, N. Y. State Parks and Recreation; Art Ospelt and Bruce Soule, Oswego County legislature; F. T. Carpenter, Mayor of Pulaski, N. Y.; and Joseph Heckle, local marina operator, visited WES to observe model operation and participate in conferences during the course of the model study.

COL John L. Cannon, CE, was Director of WES during the conduct of the investigation and the preparation and publication of this report. Mr. F. R. Brown was Technical Director.

CONTENTS

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	25.4	millimetres
feet	0.3048	metres
acres	4046.856	square metres
miles (U. S. statute)	1.609344	kilometres
pounds (mass)	0.4535924	kilograms
tons (2000 lb, mass)	907.1847	kilograms
square feet	0.09290304	square metres
square miles (U. S. statute)	2.589988	square kilometres
feet per second	0.3048	metres per second
miles per hour	1.609344	kilometres per hour
cubic feet per second	0.02831685	cubic metres per second
degrees (angle)	0.01745329	radians

PORT ONTARIO HARBOR, NEW YORK
DESIGN FOR WAVE PROTECTION
AND PREVENTION OF SHOALING
Hydraulic Model Investigation

PART I: INTRODUCTION

The Prototype

1. Port Ontario Harbor, New York, is located at the eastern end of Lake Ontario, at the mouth of the Salmon River, about 19 miles* northeast of Oswego Harbor and about 20 miles south of the entrance to Henderson Bay (Figure 1). The Salmon River is about 40 miles long and

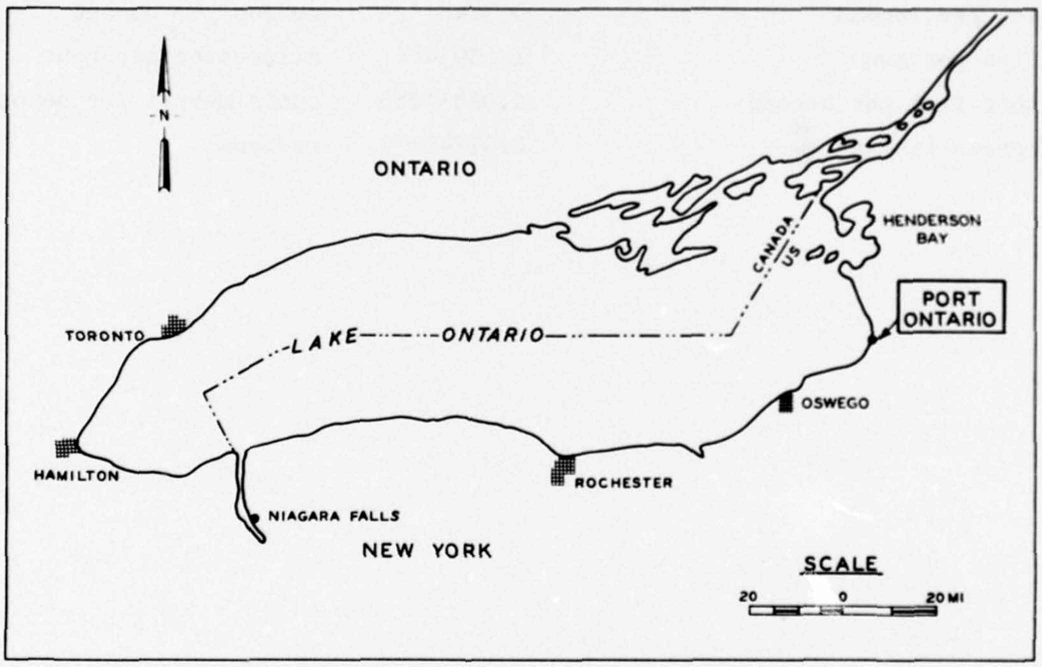

Figure 1. Project location

* A table of factors for converting U. S. customary units of measurement to metric (SI) is presented on page 3. All dimensions used in this report are given in prototype units unless otherwise noted.

drains an area of approximately 285 square miles. It has a depth of about 1 ft at its mouth during normal flows, but upstream for about 1 mile, depths vary from about 8 to 15 ft. Widths in the navigable portion of the river vary from about 100 to 500 ft. Flows in the Salmon River are regulated by a power dam located about 12 miles upstream from its mouth. There is no appreciable silting in the river channel due to the stream gradient, type of bottom materials, and controlled flows.

2. The area tributary to Port Ontario Harbor is principally recreational and agricultural, although there are several small manufacturing establishments at Pulaski, the principal trading center in the area, located on the river about 4 miles above its mouth. The settlement at the mouth of Salmon River, known as Selkirk, has about 25 permanent residences and 120 summer cottages. The village of Port Ontario, 1 mile upstream, is somewhat larger and caters to summer vacationists. Selkirk Shores State Park, under the control of the Central New York State Parks Commission, is located on the south bank of Port Ontario Harbor. It embraces 631 acres of land and consists of numerous recreational facilities. Park officials estimate the annual attendance to be approximately 80,000 people.[1]

The Problem

3. The problem at Port Ontario Harbor is the formation of a sand and cobble bar, caused mainly by littoral drift due to wave action, at the mouth of the Salmon River (Figure 2). Due to the shallow depths and constant shifting of the bar across the entrance, numerous navigational difficulties are experienced. At the end of the peak navigation season, when lake levels are normally low, the entrance channel is virtually closed to navigation.

Purpose of the Model Study

4. At the request of the U. S. Army Engineer District, Buffalo (NCB), a hydraulic model study was conducted by the U. S. Army Engineer

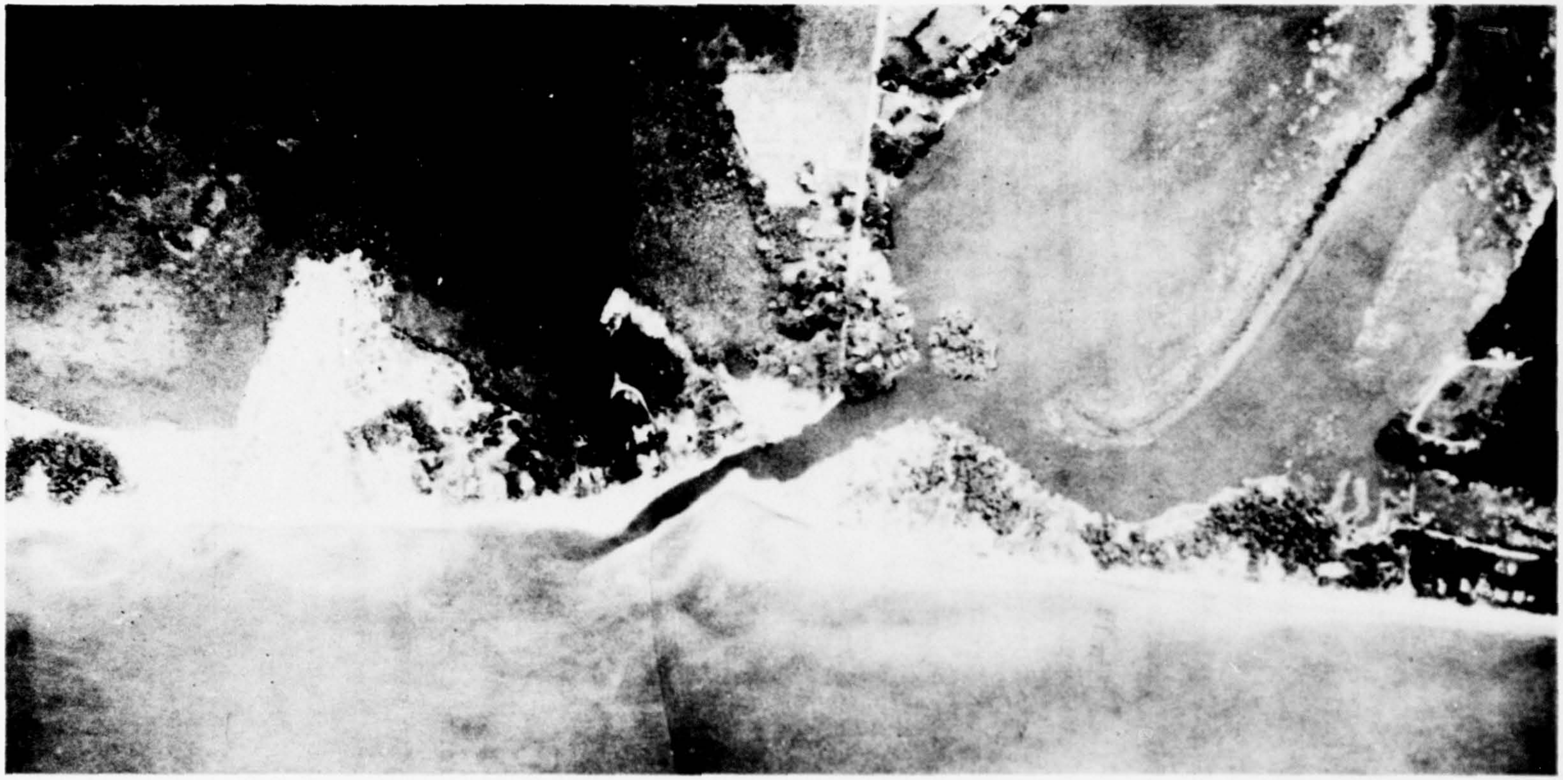

Figure 2. Aerial photograph of Salmon River mouth

Waterways Experiment Station (WES) to:

a. Study shoaling, wave action, and riverflow conditions at the harbor entrance and lower reaches of the river both with and without the proposed improvements and revisions installed in the model.

b. Develop remedial plans for alleviation of undesirable conditions as found necessary.

c. Determine whether suitable design modifications of the proposed plan could be made that would reduce construction costs significantly and still provide adequate wave and shoaling protection.

Wave-Height Criteria

5. Completely reliable criteria have not yet been developed for ensuring satisfactory navigation and mooring in small-craft harbors during attack by waves. However, for the study reported herein, NCB specified that for an inprovement plan to be acceptable, maximum wave heights in the harbor should not exceed 2.0 ft at the river mouth and 0.5 ft in the lower reaches of the river for waves occurring during the boating season (spring and summer).

PART II: THE MODEL

Design of Model

6. The Port Ontario Harbor model (Figure 3) was constructed to an undistorted linear scale of 1:75, model to prototype. Scale selection was based on such factors as:

- a. Depth of water required in the model to prevent excessive bottom friction.
- b. Absolute size of model waves.
- c. Available shelter dimensions and area required for model construction.
- d. Efficiency of model operation.
- e. Available wave-generating and wave-measuring equipment.
- f. Model construction costs.

A geometrically undistorted model was necessary to ensure accurate reproduction of short-period wave patterns. Following selection of the linear scale, the model was designed and operated in accordance with Froude's model law.[2] The scale relations used for design and operation of the model were as follows:

Characteristic	Dimension*	Model:Prototype Scale Relation
Length	L	$L_r = 1:75$
Area	L^2	$A_r = L_r^2 = 1:5625$
Volume	L^3	$V_r = L_r^3 = 1:421,875$
Time	T	$T_r = L_r^{1/2} = 1:8.66$
Velocity	L/T	$V_r = L_r^{1/2} = 1:8.66$
Roughness (Manning's coefficient, n)	$L^{1/6}$	$n_r = L_r^{1/6} = 1:2.054$
Discharge	L^3/T	$Q_r = L_r^{5/2} = 1:48,714$

* Dimensions are in terms of length and time.

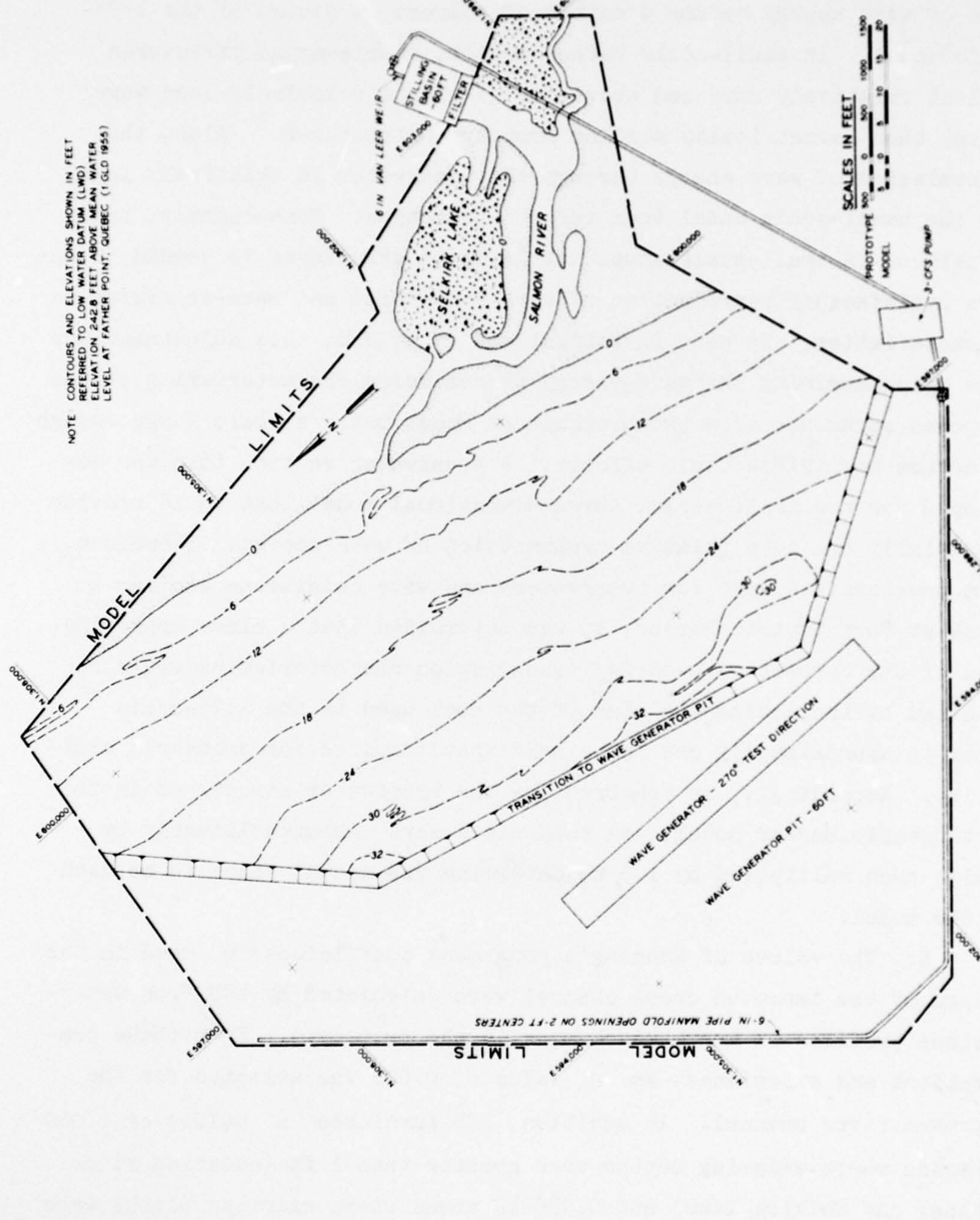

Figure 3. Model layout

7. The proposed improvement plans for Port Ontario Harbor included the use of rubble-mound breakwaters. Experience and experimental research have shown that considerable wave energy passes through the interstices of this type structure; thus, the transmission and absorption of wave energy became a matter of concern in design of the 1:75-scale model. In small-scale harbor models, rubble-mound structures reflect relatively more and absorb or dissipate relatively less wave energy than geometrically similar prototype structures.[3] Also, the transmission of wave energy through the breakwater is relatively less for the small-scale model than for the prototype. Consequently, some adjustment in small-scale model rubble-mound structures is needed to ensure satisfactory reproduction of wave-reflection and wave-transmission characteristics. In past investigations[4,5] at WES, this adjustment was made by determining the wave-energy transmission characteristics of the proposed structure in a two-dimensional model using a scale large enough to ensure negligible scale effects. A breakwater section then was developed for the small-scale, three-dimensional model that would provide essentially the same relative transmission of wave energy. Therefore, from previous findings for breakwaters and wave conditions similar to those at Port Ontario Harbor, it was determined that a close approximation of the correct wave-energy transmission characteristics would be obtained by increasing the size of the rock used in the 1:75-scale model to approximately one and a half that required for geometric similarity. Accordingly, in constructing the breakwater structures in the Port Ontario Harbor model, the rock sizes were computed linearly by scale, then multiplied by 1.5 to determine the actual sizes to be used in the model.

8. The values of Manning's roughness coefficient n used in the design of the improved creek channel were calculated by NCB from water-surface profiles of known discharges in the prototype. From these computations and experience, an n value of 0.025 was selected for the improved river channel. In addition, NCB furnished n values of 0.060 in areas where existing depths were greater than 1 ft (existing river channel and Selkirk Lake) and 0.080 in areas where existing depths were

less than 1 ft (islands and overbanks). Therefore, based on previous WES investigations and experience,[6,7] the various model areas were given finishes which would represent prototype n values of 0.025, 0.060, and 0.080.

9. Ideally, a quantitative, three-dimensional, movable-bed model investigation would best determine the effectiveness of various project plans for prevention of shoaling at Port Ontario Harbor. However, this type model investigation is difficult and expensive to conduct, and each area in which such an investigation is contemplated must be carefully analyzed. The following computations and prototype data are considered essential for such investigations;[8]

a. A computation of the littoral transport, based on the best available wave statistics.

b. An analysis of the sand-size distribution over the entire project area (offshore to a point well beyond the breaker zone).

c. Simultaneous measurements of the following items over a period of erosion and accretion of the shoreline (this measurement period should be judiciously chosen to obtain the maximum probability of both erosion and accretion during as short a time span as possible):

(1) Continuous measurements of incident-wave characteristics. Such measurements would mean placing enough redundant sensors to accurately estimate the directional spectrum over the entire project area, and in addition, would mean conducting a rather sophisticated analysis of all these data.

(2) Bottom profiling of the entire project area using the shortest time intervals possible.

(3) Nearly continuous measurements of both littoral and onshore-offshore transport of sand. These measurements would be especially important over the erosion-accretion period. A wave-forecast service would be essential to this effort to prepare for full operation during the erosion period.

In view of the complexities involved in conducting movable-bed model studies and due to limited funds and time for the Port Ontario Harbor project, the model was molded in cement mortar (fixed-bed) at an undistorted scale of 1:75 and a tracer material was obtained to determine qualitatively the degree of shoaling at the river mouth for various improvement plans.

The Model and Appurtenances

10. The model reproduced the lower 5,700 ft of the Salmon River, Selkirk Lake, approximately 4,300 ft of the Lake Ontario shoreline on each side of the river mouth, and underwater contours in Lake Ontario to an offshore depth of -32 ft with a sloping transition to the wave-generator pit elevation of -60 ft. The total area reproduced in the model was approximately 14,750 sq ft, representing about 3.0 square miles in the prototype. A general view of the model is shown in Figure 4. Vertical control for model construction was based on low water

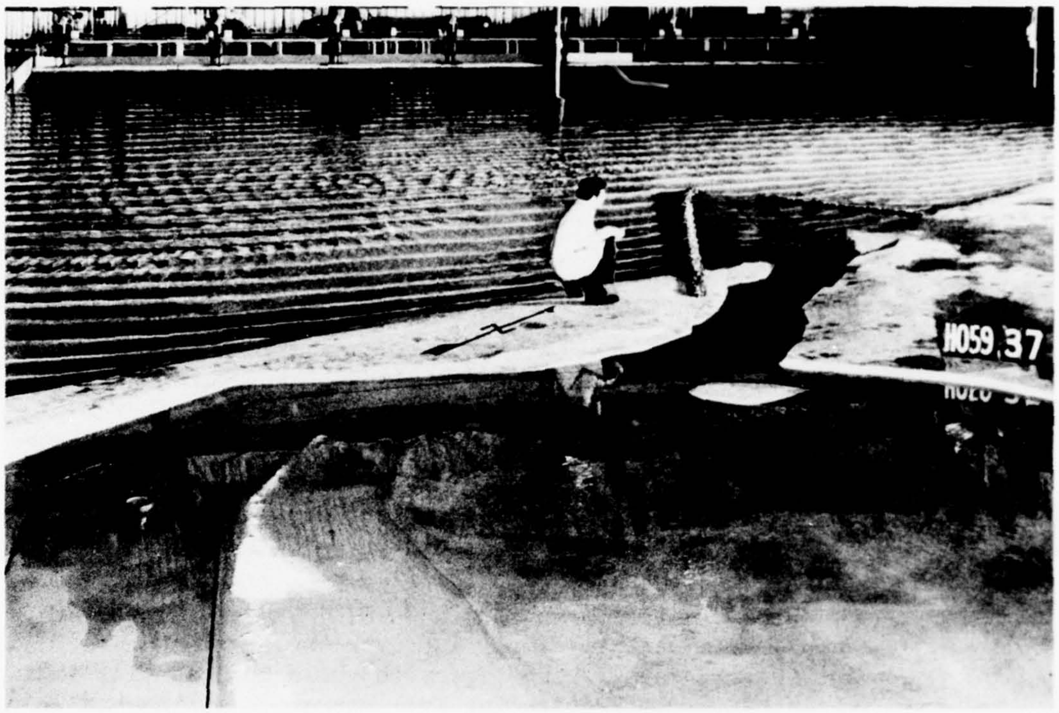

Figure 4. General view of model

datum (lwd), el 242.8* ft above mean water level at Father Point, Quebec (International Great Lakes Datum, 1955). Horizontal control was referenced to a local prototype grid system.

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* All elevations (el) cited herein are in feet referred to low water datum.

11. Model waves were generated by a 60-ft-long wave generator with a trapezoidal-shaped, vertical-motion plunger. The vertical movement of the plunger caused a periodic displacement of water incident to this motion. The length of stroke and the frequency of the vertical motion were variable over the range necessary to generate waves with the required characteristics. In addition, the wave generator was mounted on retractable casters that enabled it to be positioned to generate waves from the required directions.

12. A water-circulating system (Figure 3) consisting of a 6-in. perforated-pipe water-intake manifold, a 3-cfs pump, and a 6-in. Van Leer weir[9] was used in the model to reproduce steady-state flows through the river channel and outer harbor area that corresponded to selected prototype river discharges. The direction and magnitude of river currents were measured by timing the progress of weighted floats over known distances.

13. An Automated Data Acquisition and Control System (ADACS), designed and constructed at WES (Figure 5), was used to secure

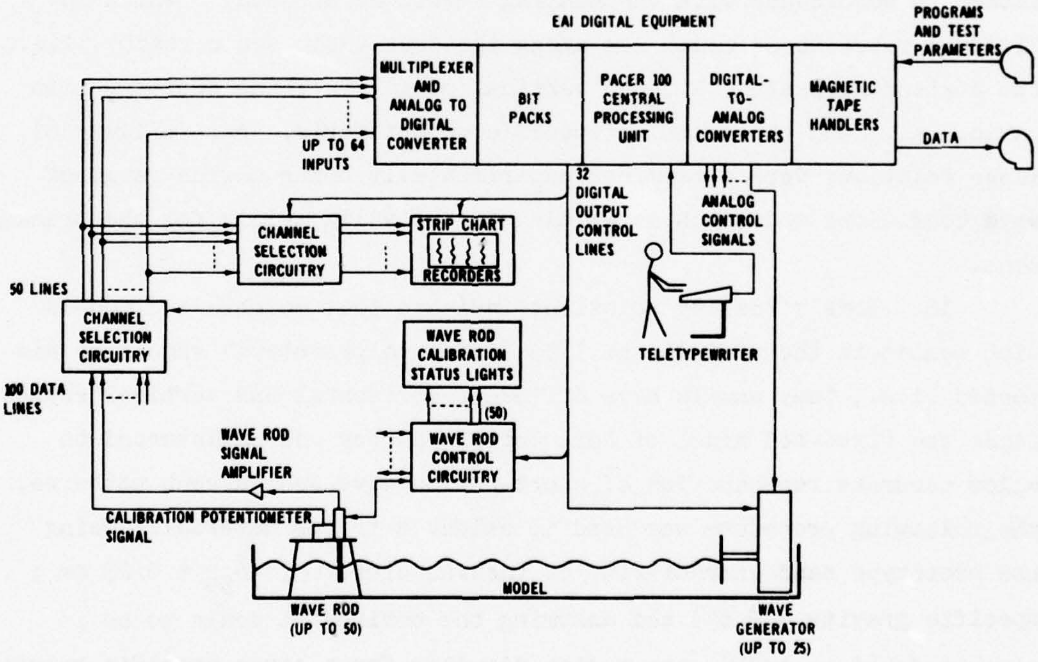

Figure 5. Automated Data Acquisition and Control System (ADACS)

wave-height data at selected locations in the model. Basically, through the use of a minicomputer, ADACS recorded onto magnetic tape the electrical output of parallel-wire, resistance-type sensors that measured the change in water-surface elevation with respect to time. The magnetic tape output was then analyzed to obtain the required data.

14. Guide vanes were placed along the wave-generator sides to ensure proper formation of the wave train incident to the model contours. In addition, a 2-ft (horizontal) solid layer of fiber wave absorber was placed around the inside perimeter of the model to dampen any wave energy that might otherwise be reflected from the model walls.

Selection of Tracer Material

15. As discussed previously in paragraph 9, a fixed-bed model was constructed and a tracer material selected to determine qualitatively the degree of shoaling at the river entrance for various improvement plans. As in previous WES investigations,[10,11] the tracer material was chosen in accordance with the scaling relations of Noda,[12] which indicate a relation or model law among the four basic scale ratios, i.e., the horizontal scale, λ ; the vertical scale, μ ; the sediment size ratio, η_D ; and the relative specific weight ratio, η_γ, (Figure 6). These relations were determined experimentally using a wide range of wave conditions and beach materials and are valid mainly for the breaker zone.

16. Noda's scaling relations indicate that movable-bed models with scales in the vicinity of 1:75 (model to prototype) should be distorted (i.e., they should have different horizontal and vertical scales). Since the fixed-bed model of Port Ontario Harbor was undistorted to allow accurate reproduction of short-period wave and current patterns, the following procedure was used to select a tracer material. Using the prototype sand characteristics (median diameter, D_{50} = 0.23 mm ; specific gravity = 2.65) and assuming the horizontal scale to be in similitude (i.e. 1:75), the median diameter for a given specific gravity of tracer material and the vertical scale were computed. The vertical

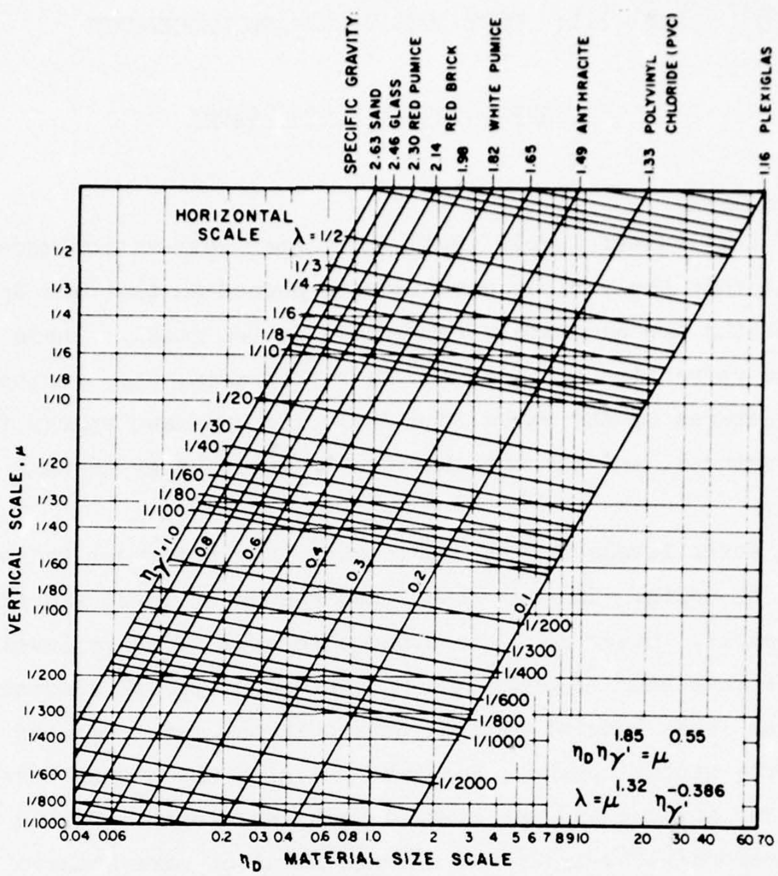

Figure 6. Graphical representation of model law (from Reference 12)

scale was then assumed to be in similitude and the tracer median diameter and horizontal scale were computed. This resulted in a range of tracer material sizes for given specific gravities that could be used. Although several types of movable-bed tracer materials were available at WES, previous investigations[10,11] indicated that crushed coal tracer more nearly represented the movement of prototype sand at the scale of 1:75 used for this study. Therefore, quantities of crushed coal (specific gravity = 1.30, median diameter, D_{50} = 0.55 mm) were selected for use as a tracer material throughout the model investigation.

PART III: TEST CONDITIONS AND PROCEDURES

Selection of Test Conditions

Still-water level

17. Still-water levels (swl) for harbor wave-action models are selected so that the various wave-induced phenomena that are dependent on water depths are accurately reproduced in the model. These phenomena include the refraction of waves in the harbor area, the overtopping of harbor structures by the waves, the reflection of wave energy from harbor structures, and the transmission of wave energy through porous structures.

18. Water levels on the Great Lakes vary from year to year and from month to month. In many locations, the water level can fluctuate daily or hourly. Since 1860, continuous records of water levels on the Great Lakes have been recorded and maintained. Typical seasonal variations of the lakes consist of high stages in the summer months and low stages in the winter months. For Lake Ontario, the higher levels usually occur in June and the lower levels in January. During the period of record (1860-1952) the average level of Lake Ontario was +2.0 ft.[13] The highest one-month average level of +4.97 ft occurred in May 1870 and the lowest one-month average level of -1.32 ft occurred in November 1934. The seasonal variation in the mean monthly level of Lake Ontario usually ranges between 1 and 2 ft, with an average variation of 1.8 ft.

19. Seasonal and longer variations in the levels of the Great Lakes are caused by variations in precipitation and other factors that affect the actual quantities of water in the lakes. Wind tides and seiches are relatively short-period fluctuations caused by the tractive force of wind blowing over the water surface and differential barometric pressures, and are superimposed on the longer period variations in the lake level. Short-period fluctuations for Oswego Harbor (19 miles southwest of Port Ontario Harbor) indicate that a rise of 1.2 ft will occur once each year.[13] Large short-period rises in local water level

are associated with the most severe storms, which generally occur in the winter when the lake level is usually low; thus the probability that a high lake level and a large wind tide or seiche will occur simultaneously is relatively small.

20. Still-water levels of +2.0 and +4.7 ft were selected for use during model testing. The lower value (+2.0 ft) represents the annual average lake level for Lake Ontario and was used for tracer tests and wave-induced current tests and while obtaining water-surface profiles and river current magnitudes. The higher value (+4.7 ft) was obtained by combining a +3.5 ft long-term level (20-year recurrence interval) with a 1.2-ft short-period rise in local water level due to wind tide (recurrence interval once per year).[13] This value was used during wave-height tests, tracer tests, and wave-induced current tests and while obtaining water-surface profiles and river current magnitudes.

Factors influencing selection of test wave characteristics

21. In planning the testing program for a model investigation of harbor wave-action problems, it is necessary to select dimensions and directions for the test waves that will allow a realistic test of proposed improvement plans and an accurate evaluation of the elements of the various proposals. Surface wind waves are generated primarily by the interactions between tangential stresses of wind flowing over water, resonance between the water surface and atmospheric turbulence, and interactions between individual wave components. The height and period of the maximum wave that can be generated by a given storm depend on the wind speed, the length of time that wind of a given speed continues to blow, and the water distance (fetch) over which the wind blows. Selection of test conditions entails evaluation of such factors as:

- a. The fetch and decay distances (the latter being the distance over which waves travel after leaving the generting area) for various directions from which waves can attack the problem area.
- b. The frequency of occurrence and duration of storm winds from the different directions.

c. The alignment, size, and relative geographic position of the navigation entrance to the harbor.

d. The alignments, lengths, and locations of various reflecting surfaces inside the harbor.

e. The refraction of waves caused by differentials in depth in the area lakeward of the harbor, which may create either a concentration or diffusion of wave energy at the harbor site.

Wave refraction

22. When waves move into water of gradually decreasing depth, transformations take place in all wave characteristics except wave period (to the first order of approximation). The most important transformations with respect to selection of test wave characteristics are the changes in wave height and direction of travel due to the phenomenon referred to as wave refraction. The change in wave height and direction can be determined by plotting refraction diagrams and calculating refraction coefficients. These diagrams are constructed by plotting the position of wave orthogonals (lines drawn perpendicular to wave crests) from deep water into shallow water. If it is assumed that waves do not break and that there is no lateral flow of energy along the wave crest, the ratio between the wave height in deep water (H_o) and the wave height at any point in shallow water (H) is inversely proportional to the square root of the ratio of the corresponding orthogonal spacings (b_o and b), or $H/H_o = K_s (b_o/b)^{1/2}$. The quantity $(b_o/b)^{1/2}$ is the refraction coefficient, K_r ; K_s is the shoaling coefficient. Thus, the refraction coefficient multiplied by the shoaling coefficient gives a conversion factor for transfer of deepwater wave heights to shallow-water values. The shoaling coefficient, which is a function of wavelength and water depth, can be obtained from Reference 14. For this study, refraction diagrams for representative wave periods from the critical directions of approach were prepared by NCB personnel. These diagrams were supplemented by additional refraction diagrams, where needed, prepared by WES.

Prototype wave data
<u>and selection of test waves</u>

23. Measured prototype wave data on which a comprehensive statistical analysis of wave conditions could be based were unavailable for the Port Ontario Harbor area. However, statistical deepwater wave hindcast data representative of this area were obtained from Reference 15, shoreline grid point 15. The numerical wind and wave models used to produce the data in Reference 15 are described in References 16, 17, 18, and 19. Reference 15 covers deepwater waves approaching from three angular sectors at the site. Due to the location of Port Ontario Harbor, deepwater waves are not possible from angle class 3 (Figure 7). Table 1 gives the significant wave heights for angle classes 1 and 2 for all seasons and for recurrence intervals of 5, 10, 20, 50, and 100 years. Table 2 shows significant wave period by angle class and wave height. The characteristics of waves used during model testing were representative of wave conditions occurring during the spring and summer (boating

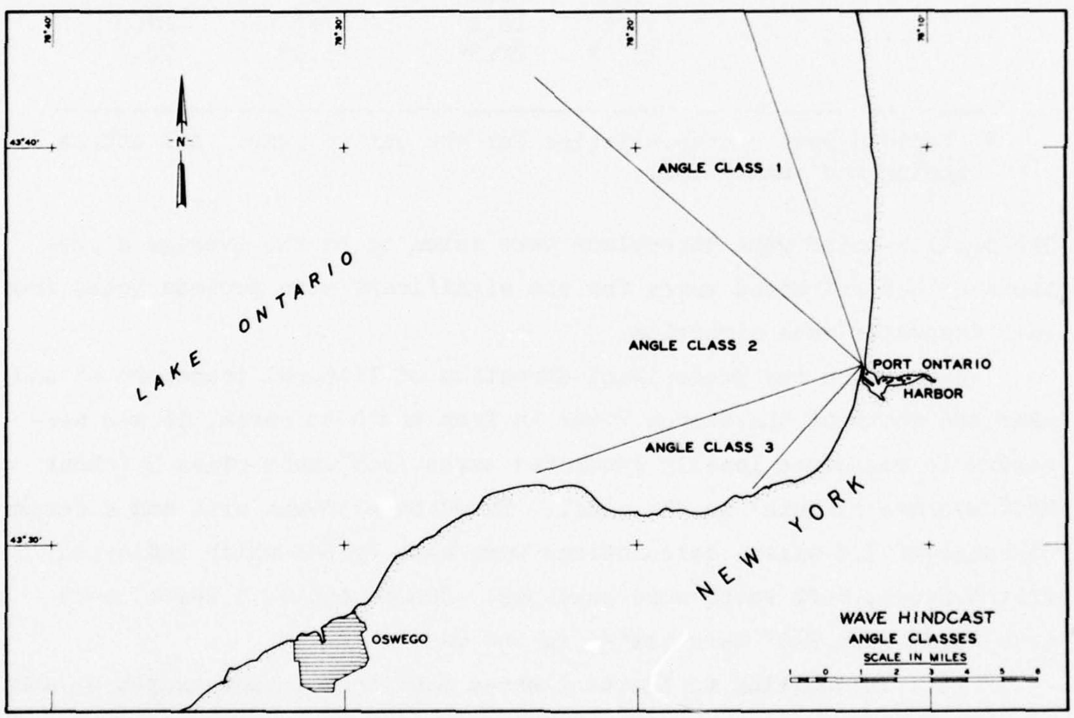

Figure 7. Wave hindcast angle classes

season). Maximum wave heights for spring and summer wave conditions were obtained for a 20-year recurrence interval. In addition, maximum wave heights for the entire year (20-year recurrence interval) were tested to aid in design of the proposed breakwaters. Model test waves were selected from Tables 1 and 2 and converted to shallow-water values by application of refraction and shoaling coefficients as shown in the following tabulation:

Deepwater Direction	Shallow-Water Direction	Period sec	Deepwater Wave Height, ft	Shallow-Water Wave Height, ft	Recurrence Interval years
330°	315°40'	5.8	6.2	4.7	20.0
		7.3*	10.8*	8.3*	20.0
300°	295°50'	6.0	6.8	6.1	0.4
		7.8	6.0	5.5	3.4
		7.8	12.1	11.1	20.0
		11.3*	22.3*	20.5*	20.0
270°	270°45'	6.0	6.8	6.1	0.4
		7.8	6.0	5.5	3.4
		7.8	12.1	11.1	20.0
		11.3*	22.3*	21.4*	20.0

* Depicts wave characteristics for the entire year. All others spring and summer only.

The shallow-water wave directions were taken to be the average directions of the refracted waves for the significant wave periods noted from each deepwater wave direction.

24. Since the predominant direction of littoral transport at and near the mouth of the Salmon River is from south to north, it was necessary to reproduce locally generated waves from angle class 3 (about 240° average azimuth) in the model. Based on a 50-mph wind and a fetch distance of 7.8 miles, calculations were made by NCB which indicated that 5.2-sec, 6-ft waves were possible. Consequently, 5.2-sec, 6-ft test waves from 240° were tested in the model.

25. In addition to the test waves mentioned in paragraphs 23 and 24, smaller everyday test waves (high frquency of occurrence) were tested from 330° and 240° to determine sediment patterns for the optimum

breakwater configuration. The characteristics of these test waves are shown as follows:

Direction	Period, sec	Height, ft
330°	5.8	3.0
	7.3	2.0
		4.0
		6.0
240°	5.2	2.0
		4.0

River discharges

26. The Salmon River flow is controlled in part by the Niagara-Mohawk power dam. The dam operators have indicated that discharges of 1,500 cfs occur approximately 30-50 percent of the time. This percentage is based on a 24-hour day over a 1-year period with continuous operation highly likely in the spring months and sporadic operation during the summer months. When the power dam is not operating, a seepage flow of 20 cfs exists through the gates. The operators indicate that the longest periods of continuous gate closure occur in the summer months and sometimes reach two days in duration. Downstream of the dam, four major tributaries contribute to the Salmon River discharge. River discharges up to 15,000 cfs (a 100-year discharge) were selected for use in the model.

Analysis of Model Data

27. Relative merits of the various plans were evaluated by:

a. Comparison of wave heights at selected locations in the harbor.

b. Comparison of wave-induced current patterns and magnitudes.

c. Comparison of tracer material movement and subsequent deposits.

d. Comparison of water-surface profiles and river current velocities.

e. Visual observations and wave pattern photographs.

In analyzing the wave-height data, the average height of the highest one third of the waves recorded at each gage location was selected. All wave heights thus selected were then adjusted to compensate for wave-height attenuation due to viscous bottom friction in the model by application of Keulegan's equation.[20] From this equation, reduction of wave heights in the model can be calculated as a function of water depth, width of wave front, wave period, water viscosity, and distance of wave travel. Wave-induced current magnitudes were obtained by timing the progress of an injected dye tracer relative to a thin graduated scale placed on the model floor.

PART IV: TESTS AND RESULTS

The Tests

Existing conditions

28. Prior to tests of various improvement plans, comprehensive tests were conducted for existing conditions. Wave-height data were obtained at the sites of the proposed breakwaters and at various locations inside the harbor (Plate 1) for the test directions listed in paragraphs 23 and 24. Wave-induced current patterns and magnitudes, shoaling patterns, and wave pattern photographs also were secured for representative waves from the four selected test directions. In addition, water-surface elevations and river current velocities were obtained for existing conditions.

Prebreakwater conditions and improvement plans

29. Tracer tests were conducted for prebreakwater conditions (Plan 1) and for nine variations in the design elements of two basic harbor configurations. These variations consisted of changes in the lengths and alignments of the breakwater structures, changes in the alignment of the entrance channel, and changes in the breakwater crown elevations. Wave-height data, wave-induced current patterns and magnitudes, wave pattern photographs, water-surface elevations, and river current velocities were obtained for prebreakwater conditions and the more important improvement plans. Prebreakwater conditions (Plans 1 and 6) consisted of the dredged channels and turning basins only and were used to establish a base with which to evaluate test results for the various breakwater plans. Brief descriptions of the test plans are presented in the following subparagraphs; dimensional details are presented in Plates 2-6. Typical breakwater sections are shown in Plate 7.

a. Plan 1 (Plate 2) consisted of a 100-ft-wide entrance channel with a bottom elevation of -8 ft extending from the -8 ft contour in Lake Ontario to an inner channel where the depth decreased to -6 ft. The 100-ft-wide, 6-ft-deep inner channel extended upstream to a 6-ft-deep harbor basin.

b. Plan 2 (Plate 3) consisted of the elements of Plan 1 with a 1,450-ft-long south breakwater with a crown elevation of +12 ft and a 360-ft-long north breakwater with a crown elevation of +9.5 ft.

c. Plan 3 (Plate 3) involved the elements of Plan 2 but 100 ft of the lakeward end of the south breakwater was removed.

d. Plan 4 (Plate 3) entailed the elements of Plan 2 with a 100-ft extension of the north breakwater.

e. Plan 5 (Plate 3) consisted of the elements of Plan 2 with a 150-ft extension of the north breakwater aligned 35° to the north of the original breakwater alignment.

f. Plan 6 (Plate 4) entailed the elements of Plan 1 except the entrance channel was aligned 25° to the south of the Plan 1 alignment.

g. Plan 7 (Plate 5) consisted of the elements of Plan 6 with a 750-ft-long south breakwater (crown el +12 ft) and a 575-ft-long north breakwater (crown el +9.5 ft). The south breakwater was joined to high ground with a 525-ft-long berm.

h. Plan 8 (Plate 5) involved the elements of Plan 7 with a 100-ft extension of the south breakwater.

i. Plan 9 (Plate 5) entailed the elements of Plan 7 but 50 ft of the lakeward end of the north breakwater was removed.

j. Plan 10 (Plate 5) consisted of the elements of Plan 7 with 100 ft of the lakeward end of the north breakwater removed.

k. Plan 11 (Plate 6) entailed the elements of Plan 4 but the crown elevation of the south breakwater was reduced from +12 ft to +10 ft.

Wave-height tests

30. Wave-height tests for prebreakwater conditions and the major improvement plans were conducted using test waves from all four test directions (i.e. 330°, 300°, 270°, and 240°). Tests involving modifications to the crown elevation of the south breakwater (Plan 11) were limited to the most critical directions of large wave approach (i.e. 300° and 270°). Wave gage locations for the various test plans are shown in Plates 2-3 and 5-6.

Wave-induced current pattern and magnitude tests

31. Wave-induced current patterns and magnitudes were determined at selected locations by timing the progress of a dye tracer relative to a known distance on the model surface. These tests were conducted for prebreakwater conditions and the best improvement plans using representative test waves from the four test directions.

Tracer tests

32. Tracer tests were conducted for prebreakwater conditions and the various improvement plans using test waves from one or more of the test directions listed in paragraphs 23 and 24. Tests involving certain proposed test plans were limited to test waves from the most critical direction with respect to shoaling in the harbor entrance (i.e. 330° and/or 240°). The major improvement plans were tested comprehensively for test waves from all four test directions (330°, 300°, 270°, and 240°). For each plan, tracer material was introduced on each side of the harbor to represent sediment from the north and south shorelines.

River current velocity and water-surface elevation tests

33. River current velocity measurements and water-surface profiles for prebreakwater conditions and the best plans of improvement were secured along the center line of the main channel at various stations for river discharges of 1,500, 3,000, 5,000, 7,500, 10,000, and 15,000 cfs at the +2.0 ft and the +4.7 ft swl's.

Test Results

34. In evaluating test results, the relative merits of each plan were based on the movement of tracer material and subsequent deposits, measured wave heights, wave-induced current patterns and magnitudes, water-surface elevations, river current velocities, or a combination of the preceding. Model wave heights (significant wave height or $H_{1/3}$), water-surface elevations, and river current velocities were tabulated to show measured values at selected locations. Water-surface elevations

also were plotted graphically to show water-surface profiles along the channel center line. Wave-induced current patterns and magnitudes were superimposed on wave pattern photographs for the corresponding plan and wave condition tested.

Existing conditions

35. Wave-height measurements obtained for existing conditions using the +4.7 ft swl are presented in Table 3. Maximum wave heights obtained for spring and summer wave conditions were 2.8 ft at the river mouth (gage 4) for 7.8-sec, 11.1-ft test waves from 300°; 0.5 ft in the lower river channel (gage 5) for 7.8-sec, 11.1-ft test waves from 300° and 270°; and 0.1 ft at the proposed mooring site (gages 8 and 9) for 7.8-sec, 11.1-ft waves from 270°.

36. Tracer tests were conducted for existing conditions using various combinations of either no river discharge or a 1,500-cfs river discharge with the +2.0 ft and +4.7 ft swl's. For test waves from each direction, spits formed across the river channel. River currents were deflected to the south as they entered the lake for test waves from 330° and 300° and to the north as they entered the lake for test waves from 270° and 240°. In instances when no river discharge was used, the spits completely closed the river entrance. Typical wave and shoaling patterns obtained for existing conditions are shown in Photos 1-11.

37. Wave-induced current patterns and magnitudes secured for existing conditions for representative test waves from the various directions using the +2.0 ft swl and a 1,500-cfs discharge are shown in Photos 3, 6, 8, and 10. Longshore currents moved from north to south for test waves from 330° and 300° and from south to north for test waves from 270° and 240°. Maximum velocities obtained lakeward of the river mouth were 3.9, 4.3, 2.7, and 4.3 fps for the 330°, 300°, 270°, and 240° directions, respectively.

38. Results of water-surface elevation and depth-averaged river current velocity measurements for existing conditions are shown in Tables 4 and 5 for the +2.0 ft and +4.7 ft swl's. Water-surface profiles plotted from the data in Table 4 are shown in Plate 8. Velocities at the river mouth ranged from 2.5 fps for the 1,500-cfs discharge to

13.3 fps for the 15,000 cfs discharge with the +2.0 ft swl. For the +4.7 ft swl, velocities ranged from 1.7 fps for the 1,500-cfs discharge to 11.8 fps for the 15,000-cfs discharge.

Prebreakwater
conditions and improvement plans

39. Results of wave-height tests with Plan 1 (prebreakwater condition) installed using the +4.7 ft swl are presented in Table 6. Maximum wave heights obtained were 5.0 ft at the river mouth (gage 4) for 7.8-sec, 11.1-ft test waves from 300°; 0.6 ft in the lower river channel (gage 5) for 5.8-sec, 4.7-ft test waves from 330° and 6-sec, 6.1-ft test waves from 300°; and 0.2 ft in the proposed mooring area (gage 9) for 5.8-sec, 4.7-ft test waves from 330°.

40. Tracer tests were conducted for Plan 1 using test waves from 330° and 240° with the +2.0 ft swl. Each test was conducted using a 1,500-cfs discharge. For test waves from 330°, a spit formed across the river entrance filling in the dredged channel. River currents were deflected to the south. Test waves from 240° also formed a spit across the river entrance which filled in the dredged channel, and river currents were deflected to the north. Typical wave and shoaling patterns obtained for Plan 1 are shown in Photos 12 and 13.

41. Results of water-surface elevation and depth-averaged river current velocity measurements for Plan 1 are presented in Tables 7 and 8 for the +2.0 ft and +4.7 ft swl's. Water-surface profiles obtained from the data in Table 7 were plotted and are shown in Plate 9. Velocities at the river mouth ranged from 1.4 fps for the 1,500-cfs river discharge to 13.0 fps for the 15,000-cfs river discharge with the +2.0 ft swl. For the +4.7 ft swl, velocities at the river mouth ranged from 1.0 to 9.5 fps for the 1,500- and 15,000-cfs river discharges, respectively.

42. The general movement of tracer material and subsequent deposits for Plans 2 and 3 are shown in Photos 14 and 15, respectively, for 5.2-sec, 6-ft test waves from 240° with the +2.0 ft swl. For Plan 2, tracer material close to the shoreline moved shoreward and was trapped by the south breakwater; however, tracer material in the breaker zone

moved around the south breakwater and was deposited lakeward of the harbor entrance. This material gradually migrated to the shoreline north of the harbor. Tracer movement for Plan 3 was similar to that obtained for Plan 2; however, material that moved in the breaker zone was deposited in the harbor entrance and slowly migrated north of the north breakwater where it was trapped in an eddy and deposited.

43. The general movement of tracer material and resulting deposits obtained with Plan 2 installed for 7.3-sec, 8.3-ft waves from 330° using the +2.0 ft swl are shown in Photo 16. Material moving along the shoreline was trapped by the north breakwater, and material moving in the breaker zone moved around the head of the south breakwater to the south. Visual observations indicated the following:

a. Tracer material placed at the head of the south breakwater (simulating deposits left for test waves from 240°) was carried to the south around the south breakwater.

b. By introducing a 1,500-cfs river discharge, tracer material moved more quickly around the south breakwater.

Tracer tests conducted for 5.8-sec, 4.7-ft test waves from 330° resulted in deposits lakeward of the harbor entrance and tracer material did not move around the south breakwater as shown in Photo 17. Observations revealed that tracer material representing deposits for test waves from 240° did not move appreciably. A 1,500-cfs river discharge did little as far as eroding the tracer material after the deposit had formed; however, the 1,500-cfs river discharge occurring in conjunction with 5.8-sec, 4.7-ft test waves prevented the tracer material from moving across the entrance as shown in Photo 18.

44. Tracer test results for Plan 4 with test waves from 330° at the +2.0 ft swl were similar to those for Plan 2, except for test conditions involving the 1,500-cfs river discharge and 5.8-sec, 4.7-ft test waves occurring simultaneously. In this case, tracer material remained north of the north breakwater and did not approach the entrance channel due to the increased river-current velocities caused by the decreased entrance opening. The general movement of tracer material and subsequent deposits for test waves from 330° with Plan 4 installed are shown in Photos 19-21.

45. Tracer movement and resulting deposits obtained with Plan 5 installed for test waves from 330° are shown in Photos 22 and 23 for the +2.0 ft swl. Deposits occurred north of the north breakwater for 5.8-sec, 4.7-ft test waves. However, visual observations revealed that deposits at the head of the south breakwater (left by test waves from 240°) would penetrate between the structures even when using the 1,500-cfs river discharge. For the 7.3-sec, 8.3-ft test waves, tracer material in the breaker zone moved around the south breakwater to the south as it did for Plans 2 and 4.

46. Based on the results of tracer tests conducted for Plans 2-5, Plan 4 was selected as the best plan for prevention of shoaling. It was, therefore, reinstalled in the model and subjected to comprehensive testing.

47. The general movement of tracer material and subsequent deposits obtained for Plan 4 for 5.2-sec, 6-ft test waves from 240° with the +2.0 ft swl are shown in Photo 24. Tracer material moving along the shoreline deposited south of the south breakwater; however, tracer material moving in the breaker zone moved around the head of the south breakwater leaving a small deposit lakeward of the entrance channel. This deposit slowly migrated to the north of the north breakwater. Visual observations revealed that a 1,500-cfs discharge had little effect on this shoaling pattern. The general movement of tracer material and deposits resulting from representative test waves from 270° and 300° are shown in Photos 25-28 for the +2.0 ft swl. Tracer material did not approach the harbor entrance for any of these test waves. To simulate the deposit left by test waves from 240°, tracer material was introduced at the head of the south breakwater for test waves from 270° and 300°. Visual observations indicated the following:

- a. The 6-sec, 6.1-ft and 7.8-sec, 11.1-ft test waves from 270° moved the tracer material north of the head of the north breakwater.
- b. The 6-sec, 6.1-ft test waves from 300° moved the tracer material north of the channel entrance.
- c. The 7.8-sec, 11.1-ft test waves from 300° moved the tracer material back around the head of the south breakwater.

A 1,500-cfs discharge, in conjunction with test waves from 270° and 300°, was beneficial in that it expedited the movement of the tracer material and/or moved the tracer material further away from the entrance.

48. Results of wave-height tests with Plan 4 installed in the model are presented in Table 9 for the +4.7 ft swl. Maximum wave heights obtained were 1.3 ft at the river mouth (gage 4) for 7.8-sec, 5.5-ft waves from 270°; 0.2 ft in the lower river channel (gage 5) for several of the test waves; and less than 0.1 ft in the mooring area (gages 8 and 9) for all the test waves.

49. Wave-induced current patterns and magnitudes secured for Plan 4 with the +4.7 ft swl are shown in Photos 29-34 for representative test waves from all test directions. Velocities ranged from 0.7 to 5.0 fps along the north breakwater, from 1.2 to 3.5 fps across the entrance channel, and from 0.2 to 4.3 fps along the south breakwater. Typical wave patterns for Plan 4 are also shown in Photos 29-34.

50. Results of water-surface elevation and depth-averaged current velocity measurements with Plan 4 installed are presented in Tables 10 and 11 for the +2.0 ft and +4.7 ft swl's. Water-surface profiles plotted from the data in Table 10 are shown in Plate 10. Velocities between the breakwaters for the +2.0 ft swl ranged from 1.5 fps for the 1,500-cfs river discharge to 11.5 fps for the 15,000-cfs river discharge. For the +4.7 ft swl, velocities between the breakwaters were 0.7 and 9.0 fps for the 1,500- and 15,000-cfs river discharges, respectively.

51. Water-surface elevations obtained for Plan 6 (prebreakwater condition for the second basic entrance configuration) are presented in Table 12 for the +2.0 ft and 4.7 ft swl's. Water-surface profiles plotted from these data are shown in Plate 11.

52. The general movement of tracer material and subsequent deposits with Plans 7-10 installed are shown in Photos 35-38 for 5.2-sec, 6.0-ft waves from 240° using the +2.0 ft swl. Tracer material moving in the breaker zone deposited around the head of the south breakwater, across the entrance channel, and around the head of the north breakwater for all the test plans. Observations revealed that a 1,500-cfs river discharge tended to erode material around the head of the north

breakwater; however, deposits at the head of the south breakwater and across the entrance channel remained.

53. The general movement of tracer material and deposits resulting from test waves from 330° are shown in Photos 39-45 for Plans 7-10 with the +2.0 ft swl. For the 5.8-sec, 4.7-ft test waves, Plans 7-9 proved adequate since they allowed no tracer material in the entrance. Tracer material moved into the entrance channel and between the breakwaters for Plan 10. For the 7.3-sec, 8.3-ft test waves, tracer material in the breaker zone moved around the heads of the north and south breakwaters and continued moving to the south for Plans 7 and 8. For Plan 9, tracer material penetrated between the breakwaters. Tracer material was placed in the entrance (simulating deposits left by 5.2-sec, 6-ft test waves from 240°) for Plans 7 and 8; and visual observations indicated the following:

a. 7.3-sec, 8.3-ft test waves moved tracer material around the south breakwater for Plans 7 and 8.

b. 5.8-sec, 4.7-ft test waves moved some of the material between the breakwaters for Plans 7 and 8; however, a 1,500-cfs river discharge prevented the tracer material from moving between the breakwaters. A 5,000-cfs river discharge moved the tracer material away from the entrance.

54. Considering results of the tracer tests described above along with construction costs, Plan 7 was selected as the best plan for this basic entrance configuration and was tested comprehensively.

55. The general movement of tracer material and subsequent deposits for representative test waves from 270° and 300° are shown in Photos 46-49 for the +2.0 ft swl. Tracer material did not approach the entrance for any of these wave conditions. To simulate deposits left by 5.2-sec, 6-ft test waves from 240°, tracer material was placed in the entrance. Visual observations revealed the following:

a. 6-sec, 6.1-ft test waves from 300° and 270° and 7.8-sec, 11.1-ft test waves from 270° moved some of the tracer material between the breakwaters.

b. 7.8-sec, 11.1-ft test waves from 300° moved the tracer material around the south breakwater to the south.

With test waves from 270° and 300°, a 1,500-cfs river discharge was beneficial in that it prevented the tracer material from moving between the breakwaters.

56. Results of wave-height tests with Plan 7 installed are presented in Table 13 for the +4.7 ft swl. Maximum wave heights obtained were 1.5 ft at the river mouth (gage 4) for 7.8-sec, 5.5-ft test waves from 270°; 0.1 ft in the lower river channel (gage 5) for all the test waves; and less than 0.1 ft in the mooring area (gages 8 and 9) for all the test waves.

57. Wave-induced current patterns and magnitudes secured for Plan 7 with the +4.7 ft swl are shown in Photos 50-55 for representative test waves from all four test directions. Velocities ranged from 1.1 to 3.5 fps along the north breakwater; from 0.7 to 3.8 fps across the entrance channel; and from 0.3 to 4.3 fps along the south breakwater. Typical wave patterns for Plan 7 are also shown in Photos 50-55.

58. Results of water-surface elevation and depth-averaged river current velocity measurements for Plan 7 are presented in Tables 14 and 15 for the +2.0 ft and +4.7 ft swl's. Water-surface profiles were plotted from the data in Table 14 and are shown in Plate 12. Velocities between the breakwaters for the +2.0 ft swl ranged from 1.1 fps for the 1,500-cfs river discharge to 10.4 fps for the 15,000-cfs river discharge. For the +4.7 ft swl, velocities between the breakwaters were 0.7 and 8.7 fps for the 1,500- and 15,000-cfs discharges, respectively.

59. Results of the wave-height tests with Plan 11 installed are presented in Table 16 for test waves from 300° and 270° with the +4.7 ft swl. Maximum wave heights obtained were 3.1 ft inside the breakwaters (gage 2) for 6-sec, 6.1-ft test waves from 300° and 1.2 ft at the river mouth (gage 4) for 7.8-sec, 11.1-ft test waves from 300°. Typical wave patterns for Plan 11 are shown in Photos 56-59 for representative test waves from 300° and 270° using the +4.7 ft swl.

60. The general movement of tracer material and deposits resulting from small test waves from 330° and 240° are shown in Photos 60-65 for Plan 11 with the +2.0 ft swl. Tracer material moved along the shoreline for the 5.8-sec, 2-ft and 7.3-sec, 2-ft test waves from 330°. For 7.3-sec, 4-ft test waves from 330°, tracer material moved alongside

the north breakwater. Material moved southerly across the entrance around the head of the south breakwater for 7.3-sec, 6-ft test waves from 330°. For 5.2-sec, 2- and 4-ft test waves from 240°, tracer material was trapped by the south breakwater.

Discussion of test results

61. Test results obtained for existing conditions revealed rough and turbulent wave conditions at the river entrance due to waves breaking on the shoal across the river mouth. Tracer tests indicated that the model accurately reproduced the sediment patterns observed in the prototype (as evidenced by visual observations and numerous aerial photographs).

62. Test results for Plan 1 (prebreakwater condition) also revealed excessive wave heights (3 to 5 ft high) at the river entrance for storm waves from the various directions. Tracer tests indicated that sediment would fill in the dredged channel and form a shoal across the river mouth.

63. A comparison of Plans 2-5 indicated that Plan 4 provides slightly better shoaling protection than the other plans. The decreased entrance opening is beneficial in that river current velocities are increased and should aid in sweeping sediment deposits out of the entrance.

64. A comparison of Plans 7-10 revealed that Plans 7 and 8 provided better shoaling protection than Plans 9 and 10. Considering construction costs, however, Plan 7 was determined to be the optimum for this channel alignment.

65. Results of wave-height tests for Plans 4 and 7 revealed that wave heights for both plans were well within the established criteria of 2.0 ft at the river mouth and 0.5 ft in the mooring area. Tracer tests indicated that Plan 4 provided better shoaling protection than Plan 7 since tracer material penetrated between the breakwaters for some test waves for Plan 7.

66. A comparison of water-surface elevations obtained both with and without breakwaters (Plan 1 versus Plan 4 and Plan 6 versus Plan 7) revealed that the breakwaters had little effect on elevations in the lower reaches of the river and should not contribute to flooding in this area.

67. Test results obtained with Plan 11 installed indicated that wave heights between the structures would not be increased by reducing the crown elevation of the south breakwater to +10 ft. Tracer tests revealed that sediment buildup along the structures may occur for smaller everyday waves from 330° and 240°. However, the larger storm waves will probably result in some natural sand bypassing around the structures protecting the entrance channel.

PART V: CONCLUSIONS

68. Based on results of the hydraulic model investigation reported herein, it is concluded that:

- a. Existing conditions are characterized by rough and hazardous wave conditions at the river entrance during periods of storm wave attack.
- b. For existing conditions, wave action formed a shoal across the river mouth which interferes with navigation and the passage of riverflows.
- c. Wave heights in the harbor were within the established wave-height criteria (2.0 ft at the river mouth and 0.5 ft in the mooring area) for all major improvement plans tested.
- d. Of the improvement plans involving the first breakwater configuration and channel alignment tested (Plans 2-5), Plan 4 was selected as the optimum with respect to shoaling protection and construction costs.
- e. Of the improvement plans involving the second breakwater configuration and channel alignment tested (Plans 7-10), Plan 7 was selected as optimum with respect to shoaling protection and construction costs.
- f. Plan 4 provided better shoaling protection than Plan 7 and construction costs should be comparable.
- g. Breakwaters installed for Plans 4 and 7 had little effect on water-surface elevations in the lower reaches of the river during riverflows and should not contribute to flooding in this area.
- h. Reducing the crown elevation of the south breakwater from +12 ft (Plan 4) to +10 ft (Plan 11) will not increase wave heights between the structures.
- i. Sediment will probably build up along the breakwaters on both sides of the harbor for small everyday waves from 240° and 330°.
- j. The larger storm waves will probably result in some natural bypassing around the harbor.

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Table 1

Wave Hindcast* Data

Recurrence Interval, year	Wave Height, ft Angle Class 1	Angle Class 2
	Winter	
5	8.9	18.0
10	9.8	19.4
20	10.5	22.3
50	13.8	23.9
100	14.1	27.2
	Spring	
5	4.9	9.8
10	5.9	11.8
20	6.2	12.1
50	7.5	13.1
100	7.9	13.8
	Summer	
5	4.9	8.5
10	5.2	8.9
20	5.9	9.2
50	7.2	9.8
100	7.5	11.8
	Fall	
5	8.2	15.4
10	9.5	17.4
20	10.8	18.4
50	12.8	21.6
100	13.4	23.3

* Hindcast taken from Reference 15, shoreline point 15.

Table 2

Significant Period by Angle Class and Wave Height

Wave Height ft	Angle Class 1	Angle Class 2
1	2.3	2.1
2	3.5	3.3
3	4.4	4.2
4	5.2	4.9
5	5.7	5.4
6	6.0	5.7
7	6.3	6.1
8	6.5	6.4
9	6.8	6.8
10	7.1	7.1
11	7.4	7.4
12	7.7	7.8
13	7.9	8.1
14	8.2	8.5
15	8.5	8.8
16	8.8	9.1
17	9.1	9.5
18	9.3	9.8
19	9.6	10.2
20	9.9	10.5
21	10.2	10.8
22	10.5	11.2
23	10.7	11.5
24	11.0	11.9
25	11.3	12.2

Table 3

Wave Heights, Existing Conditions

swl = +4.7 ft Referred to lwd = 247.5

Test Direction	Test Wave Period sec	Test Wave Height ft	Gage 1	Gage 1A	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
			Wave Height at Indicated Gage Location, ft									
330°	5.8	4.7	5.3	5.3	7.7	3.8	2.1	0.5	0.1*	0.1*	0.1*	0.1*
	7.3**	8.3	10.5	5.5	6.1	4.8	2.3	0.6	0.1	0.1*	0.1*	0.1*
300°	6.0	6.1	6.3	6.0	7.9	3.8	2.1	0.4	0.1	0.1*	0.1*	0.1*
	7.8	5.5	7.0	6.4	8.1	4.4	2.0	0.6	0.1	0.1*	0.1*	0.1*
		11.1	7.1	12.2	7.5	4.6	2.6	0.5	0.1	0.1	0.1*	0.1*
	11.3**	20.5	7.9	6.8	7.2	4.6	3.1	0.8	0.2	0.1	0.1	0.1
270°	6.0	6.1	7.8	5.7	7.6	2.6	2.3	0.3	0.1*	0.1*	0.1*	0.1*
	7.8	5.5	7.6	8.9	8.1	2.8	2.5	0.4	0.1	0.1	0.1*	0.1*
		11.1	7.9	6.7	6.6	4.1	2.8	0.5	0.2	0.1	0.1	0.1
	11.3**	21.4	7.3	6.4	6.8	4.6	3.4	0.9	0.2	0.2	0.1	0.1
240°	5.2	6.0	6.5	6.5	7.1	2.5	2.2	0.2	0.1*	0.1*	0.1*	0.1*

* Less than one tenth.
** Wave conditions for entire year, 20-year recurrence interval.

Table 4

Water-Surface Elevations, Existing Conditions

Discharge Q, cfs	Water-Surface Elevation at Indicated Station Sta 0	Sta 300	Sta 700	Sta 1000	Sta 1500	Sta 2000	Sta 2500	Sta 3000	Sta 3500	Sta 4000	Sta 4500	Sta 5000	Sta 5500	Sta 6000	Sta S.L.*
swl = +2.0 ft Referred to lwd = 244.8															
1,500	244.8	244.7	244.8	244.9	245.2	245.3	245.2	245.3	245.2	245.2	245.3	245.3	245.3	245.3	245.2
3,000	244.8	244.7	244.8	244.9	245.7	245.8	245.7	245.8	245.7	245.7	245.8	245.8	245.8	245.8	245.7
5,000	244.8	244.9	244.7	244.9	247.3	247.6	247.6	247.7	247.6	247.6	247.7	247.7	247.7	247.7	247.5
7,500	244.8	245.0	244.8	245.9	248.5	249.1	249.1	249.2	249.2	249.2	249.2	249.3	249.3	249.3	249.2
10,000	244.8	244.8	244.8	246.8	249.6	250.5	250.7	250.8	250.8	250.8	250.8	250.8	250.8	250.8	250.7
15,000	244.8	244.7	244.7	247.1	250.1	251.0	251.2	251.3	251.3	251.3	251.3	251.4	251.4	251.4	251.3
swl = +4.7 ft Referred to lwd = 247.5															
1,500	247.5	247.5	247.5	247.5	247.5	247.6	247.6	247.6	247.6	247.6	247.6	247.6	247.6	247.6	247.6
3,000	247.5	247.5	247.5	247.5	247.6	247.7	247.7	247.7	247.7	247.7	247.7	247.7	247.7	247.7	247.7
5,000	247.5	247.5	247.4	247.7	248.0	248.5	248.6	248.6	248.6	248.6	248.6	248.6	248.6	248.6	248.6
7,500	247.5	247.4	247.5	247.6	248.3	248.8	248.9	249.0	249.0	249.0	249.0	249.0	249.0	249.0	249.0
10,000	247.5	247.4	247.5	247.8	250.1	250.3	250.4	250.5	250.5	250.5	250.5	250.5	250.5	250.5	250.4
15,000	247.5	247.4	247.4	247.6	250.9	252.3	252.7	252.9	252.9	252.9	252.9	252.9	252.9	252.9	252.7

* Selkirk Lake.

Table 5

River Current Velocities, Existing Conditions

Discharge Q, cfs	Channel Center-Line Velocity at Indicated Station, fps Sta 1000	Sta 2000	Sta 3000	Sta 4000	Sta 5000	Sta 6000
swl = +2.0 ft Referred to lwd = 244.8						
1,500	2.5	0.7	0.6	0.7	0.5	0.4
3,000	4.3	1.1	1.0	1.0	0.7	0.6
5,000	8.0	1.4	1.3	1.6	1.3	1.2
7,500	10.4	2.0	1.4	1.9	1.5	1.4
10,000	11.8	3.1	1.4	1.9	1.9	1.9
15,000	13.3	4.0	1.6	2.0	2.3	2.5
swl = +4.7 ft Referred to lwd = 247.5						
1,500	1.7	0.6	0.6	0.5	0.4	0.2
3,000	2.5	0.9	0.8	0.8	0.7	0.5
5,000	4.9	1.6	1.0	1.3	1.2	0.9
7,500	6.9	2.4	1.0	1.4	1.3	1.4
10,000	10.4	3.4	1.1	1.6	1.8	1.9
15,000	11.8	4.3	1.3	2.1	2.4	2.3

Table 6

Wave Heights, Plan 1

swl = +4.7 ft Referred to lwd = 247.5

Test Direction	Test Wave Period sec	Test Wave Height ft	Wave Height at Indicated Gage Location, ft Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
330°	5.8	4.7	5.4	5.9	3.6	3.0	0.6	0.1	0.1	0.1	0.2
300°	6.0	6.1	8.7	5.0	3.9	4.0	0.6	0.1	0.1*	0.1*	0.1*
	7.8	5.5	8.2	4.7	4.7	4.5	0.4	0.1	0.1	0.1*	0.1
		11.1	7.0	6.3	5.1	5.0	0.4	0.1	0.1	0.1	0.1
270°	6.0	6.1	8.5	6.3	2.8	3.0	0.4	0.1*	0.1*	0.1*	0.1*
	7.8	5.5	7.2	8.8	3.1	3.6	0.4	0.1*	0.1*	0.1*	0.1*
		11.1	7.8	6.7	3.8	4.5	0.3	0.1	0.1	0.1*	0.1*
240°	5.2	6.0	7.1	6.0	2.5	3.9	0.2	0.1*	0.1*	0.1*	0.1*

* Less than one tenth.

Table 7

Water-Surface Elevations, Plan 1

Dis-charge Q, cfs	Water-Surface Elevation at Indicated Station														
	Sta 0	Sta 300	Sta 700	Sta 1000	Sta 1500	Sta 2000	Sta 2500	Sta 3000	Sta 3500	Sta 4000	Sta 4500	Sta 5000	Sta 5500	Sta 6000	Sta S.L.*
swl = +2.0 ft Referred to lwd = 244.8															
1,500	244.8	244.8	245.0	244.9	244.9	244.9	245.0	244.9	245.0	245.0	245.0	245.0	245.0	244.9	244.9
3,000	244.8	244.9	245.0	244.9	245.0	245.0	245.1	245.0	245.1	245.0	245.1	245.1	245.1	245.0	245.0
5,000	244.8	244.8	244.9	244.9	245.0	245.3	245.4	245.4	245.5	245.6	245.5	245.6	245.6	245.6	245.5
7,500	244.8	244.8	244.8	244.9	245.2	245.8	245.9	246.0	246.1	246.1	246.2	246.2	246.2	246.2	245.9
10,000	244.8	244.8	244.7	244.9	245.4	246.9	247.2	247.4	247.4	247.4	247.4	247.4	247.4	247.5	247.4
15,000	244.8	244.8	244.7	245.2	245.9	248.2	248.6	248.7	248.7	248.7	248.7	248.8	248.7	248.8	248.6
swl = +4.7 ft Referred to lwd = 247.5															
1,500	247.5	247.5	247.7	247.7	247.6	247.6	247.6	247.6	247.5	247.7	247.6	247.6	247.6	247.6	247.6
3,000	247.5	247.6	247.7	247.7	247.7	247.7	247.7	247.7	247.6	247.7	247.7	247.8	247.7	247.7	247.7
5,000	247.5	247.6	247.7	247.7	247.7	247.8	247.9	247.9	247.8	248.0	248.0	247.9	247.9	247.9	247.8
7,500	247.5	247.7	247.7	247.8	247.9	248.2	248.3	248.3	248.3	248.3	248.3	248.3	248.3	248.3	248.2
10,000	247.5	247.6	247.5	247.7	248.0	248.6	248.9	248.9	248.9	249.1	248.9	248.9	249.0	248.9	248.9
15,000	247.5	247.6	247.5	247.8	247.9	249.8	250.2	250.4	250.3	250.4	250.4	250.4	250.4	250.4	250.3

* Selkirk Lake.

Table 8

River Current Velocities, Plan 1

Discharge Q, cfs	Sta 1000	Sta 2000	Sta 3000	Sta 4000	Sta 5000	Sta 6000
	Channel Center-Line Velocity at Indicated Station, fps					
swl = +2.0 ft Referred to lwd = 244.8						
1,500	1.4	0.7	0.7	0.7	0.6	0.4
3,000	2.4	1.0	1.1	1.1	0.8	0.7
5,000	5.8	2.1	1.9	1.7	1.9	1.7
7,500	9.4	3.5	2.5	2.3	2.6	2.1
10,000	11.5	3.7	2.3	2.2	2.9	2.1
15,000	13.0	5.4	1.9	2.0	3.0	2.9
swl = +4.7 ft Referred to lwd = 247.5						
1,500	1.0	0.6	0.4	0.5	0.5	0.3
3,000	1.6	0.9	0.6	0.8	0.7	0.6
5,000	2.8	1.6	1.1	1.2	1.3	1.2
7,500	4.3	2.4	1.3	1.3	1.5	1.6
10,000	6.5	3.8	1.5	1.6	2.2	2.3
15,000	9.5	4.5	1.6	1.7	2.6	2.8

Table 9

Wave Heights, Plan 4

swl = +4.7 ft Referred to lwd = 247.5

Test Direction	Test Wave Period sec	Test Wave Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
			Wave Height at Indicated Gage Location, ft								
330°	5.8	4.7	7.7	3.0	3.0	1.0	0.1	0.1*	0.1*	0.1*	0.1*
300°	6.0	6.1	8.1	2.9	2.2	0.8	0.2	0.1*	0.1*	0.1*	0.1*
	7.8	5.5	9.4	2.8	2.7	1.2	0.1	0.1*	0.1*	0.1*	0.1*
		11.1	6.9	3.6	3.2	1.1	0.2	0.1*	0.1*	0.1*	0.1*
270°	6.0	6.1	7.1	0.9	3.1	0.7	0.2	0.1*	0.1*	0.1*	0.1*
	7.8	5.5	7.8	1.4	3.0	1.3	0.2	0.1*	0.1*	0.1*	0.1*
		11.1	7.5	2.7	3.1	0.9	0.1	0.1*	0.1*	0.1*	0.1*
240°	5.2	6.0	4.7	1.5	0.9	0.4	0.1	0.1*	0.1*	0.1*	0.1*

* Less than one tenth.

Table 10

Water-Surface Elevations, Plan 4

Dis-charge Q, cfs	Water-Surface Elevation at Indicated Station														
	Sta 0	Sta 300	Sta 700	Sta 1000	Sta 1500	Sta 2000	Sta 2500	Sta 3000	Sta 3500	Sta 4000	Sta 4500	Sta 5000	Sta 5500	Sta 6000	Sta S.L.*
						swl = +2.0 ft Referred to lwd = 244.8									
1,500	244.8	244.8	244.9	244.8	245.0	245.0	244.9	244.9	244.9	244.9	244.9	245.0	245.0	244.9	244.9
3,000	244.8	244.8	244.9	244.8	245.1	245.1	245.1	245.1	245.1	245.1	245.1	245.1	245.1	245.1	245.0
5,000	244.8	244.8	244.8	244.9	245.0	245.5	245.5	245.4	245.5	245.6	245.8	245.8	245.8	245.9	245.7
7,500	244.8	244.8	244.9	244.9	245.3	245.9	245.9	245.9	246.0	246.0	246.1	246.2	246.1	246.2	245.9
10,000	244.8	245.0	245.0	245.1	245.7	247.0	247.1	247.2	247.3	247.4	247.4	247.4	247.4	247.5	247.4
15,000	244.8	245.1	245.0	245.2	246.0	248.9	249.2	249.3	249.3	249.3	249.3	249.3	249.3	249.4	249.2
						swl = +4.7 ft Referred to lwd = 247.5									
1,500	247.5	247.6	247.7	247.6	247.6	247.6	247.7	247.6	247.7	247.7	247.6	247.6	247.6	247.7	247.6
3,000	247.5	247.6	247.7	247.6	247.7	247.7	247.8	247.7	247.7	247.8	247.7	247.7	247.7	247.7	247.7
5,000	247.5	247.7	247.7	247.7	247.8	248.0	248.0	248.0	248.1	248.0	248.0	248.0	248.0	248.1	248.1
7,500	247.5	247.8	247.7	247.9	247.9	248.3	248.4	248.3	248.4	248.4	248.4	248.4	248.4	248.4	248.4
10,000	247.5	247.8	248.0	248.0	248.0	248.8	249.0	249.0	249.0	249.0	249.1	249.0	249.0	249.1	249.1
15,000	247.5	247.6	247.5	247.6	247.7	250.3	250.8	250.8	250.8	250.7	250.8	250.8	250.8	250.8	250.7

* Selkirk Lake.

Table 11

River Current Velocities, Plan 4

Discharge Q, cfs	Sta 300	Sta 1000	Sta 2000	Sta 3000	Sta 4000	Sta 5000	Sta 6000
	Channel Center-Line Velocity at Indicated Station, fps						
	swl = +2.0 ft Referred to lwd = 244.8						
1,500	1.5	1.6	0.6	0.7	0.9	0.7	0.3
3,000	2.3	2.5	1.0	1.1	1.1	1.0	0.5
5,000	4.3	5.8	2.4	2.2	1.9	1.7	1.6
7,500	8.0	9.4	2.9	2.7	2.5	2.7	1.8
10,000	9.5	11.5	4.2	2.7	2.5	2.7	2.0
15,000	11.5	13.0	5.8	1.7	2.7	3.5	3.0
	swl = +4.7 ft Referred to lwd = 247.5						
1,500	0.7	0.8	0.5	0.4	0.4	0.4	0.3
3,000	1.2	1.4	0.9	0.7	0.7	0.7	0.5
5,000	2.4	2.9	1.8	1.1	1.4	1.4	1.1
7,500	3.0	3.6	2.5	1.3	1.4	1.5	1.4
10,000	4.9	5.5	3.8	1.5	1.8	2.2	1.9
15,000	9.0	9.5	4.9	1.6	2.1	3.0	3.2

Table 12

Water-Surface Elevations, Plan 6

Dis- charge Q, cfs	Water-Surface Elevation at Indicated Station														
	Sta 0	Sta 300	Sta 700	Sta 1000	Sta 1500	Sta 2000	Sta 2500	Sta 3000	Sta 3500	Sta 4000	Sta 4500	Sta 5000	Sta 5500	Sta 6000	Sta S.L.*
	swl = +2.0 ft Referred to lwd = 244.8														
1,500	244.8	244.9	244.8	244.9	244.9	244.9	244.9	244.9	244.9	244.9	244.9	244.9	244.9	244.9	244.8
3,000	244.8	244.9	244.8	244.9	245.0	245.0	245.0	245.1	245.1	245.1	245.1	245.1	245.1	245.1	245.0
5,000	244.8	244.9	244.8	245.0	245.2	245.5	245.6	245.7	245.7	245.7	245.7	245.7	245.7	245.7	245.6
7,500	244.8	244.9	244.8	245.0	245.3	246.0	246.0	246.2	246.2	246.2	246.3	246.3	246.3	246.3	246.0
10,000	244.8	244.8	244.9	245.4	246.0	247.2	247.4	247.5	247.5	247.5	247.6	247.6	247.6	247.7	247.6
15,000	244.8	244.9	244.9	245.8	246.5	248.2	248.5	248.6	248.7	248.8	248.8	248.8	248.8	248.8	248.6
	swl = +4.7 ft Referred to lwd = 247.5														
1,500	247.5	247.5	247.5	247.5	247.5	247.5	247.5	247.5	247.5	247.6	247.6	247.6	247.6	247.6	247.5
3,000	247.5	247.5	247.5	247.5	247.6	247.6	247.6	247.7	247.7	247.7	247.7	247.7	247.7	247.7	247.6
5,000	247.5	247.5	247.5	247.5	247.6	247.8	247.8	247.8	247.8	247.9	247.8	247.9	247.9	247.9	247.8
7,500	247.5	247.5	247.6	247.7	247.7	248.3	248.3	248.4	248.4	248.5	248.4	248.4	248.4	248.5	248.4
10,000	247.5	247.5	247.7	247.7	247.7	248.6	248.8	248.9	248.9	249.0	248.9	248.9	248.9	249.0	248.9
15,000	247.5	247.6	247.8	248.0	248.1	249.5	249.8	249.9	249.9	250.1	249.9	250.0	250.0	250.1	249.9

* Selkirk Lake.

Table 13

Wave Heights, Plan 7

swl = +4.7 ft Referred to 247.5

Test Direction	Test Wave Period sec	Test Wave Height ft	Gage 1	Gage 2	Gage 3	Gage 4	Gage 5	Gage 6	Gage 7	Gage 8	Gage 9
			Wave Height at Indicated Gage Location, ft								
330°	5.8	4.7	8.4	4.1	2.6	0.6	0.1	0.1	0.1*	0.1*	0.1*
300°	6.0	6.1	7.8	3.7	2.6	0.7	0.1	0.1	0.1*	0.1*	0.1*
	7.8	5.5	8.4	3.8	2.2	0.8	0.1	0.1*	0.1*	0.1*	0.1*
		11.1	8.8	3.5	2.4	0.5	0.1	0.1*	0.1*	0.1*	0.1*
270°	6.0	6.1	7.6	3.4	3.1	1.0	0.1	0.1*	0.1*	0.1*	0.1*
	7.8	5.5	8.7	4.0	2.2	1.5	0.1	0.1*	0.1*	0.1*	0.1*
		11.1	8.5	3.9	2.5	0.8	0.1	0.1	0.1*	0.1*	0.1*
240°	5.2	6.0	7.4	2.0	1.9	0.4	0.1	0.1*	0.1*	0.1*	0.1*

* Less than one tenth.

Table 14

Water-Surface Elevations, Plan 7

Dis-charge Q, cfs	Water-Surface Elevation at Indicated Station														
	Sta 0	Sta 300	Sta 700	Sta 1000	Sta 1500	Sta 2000	Sta 2500	Sta 3000	Sta 3500	Sta 4000	Sta 4500	Sta 5000	Sta 5500	Sta 6000	Sta S.L.*
	swl = +2.0 ft Referred to lwd = 244.8														
1,500	244.8	244.8	244.8	244.9	244.9	244.9	244.9	245.0	244.9	245.0	245.0	244.9	245.0	245.0	244.9
3,000	244.8	244.8	244.9	244.9	245.0	245.1	245.1	245.1	245.0	245.1	245.1	245.1	245.1	245.1	245.0
5,000	244.8	244.9	245.0	245.1	245.2	245.5	245.6	245.8	245.8	245.8	245.8	245.8	245.9	245.9	245.6
7,500	244.8	244.9	245.0	245.2	245.4	245.9	246.0	246.1	246.2	246.2	246.2	246.2	246.3	246.3	246.0
10,000	244.8	244.9	245.0	245.5	246.1	247.3	247.5	247.6	247.6	247.7	247.7	247.7	247.8	247.8	247.8
15,000	244.8	244.9	245.0	245.9	246.6	248.3	248.7	248.8	248.8	248.9	248.9	248.8	248.9	248.9	248.7
	swl = +4.7 ft Referred to lwd = 247.5														
1,500	247.5	247.5	247.5	247.5	247.6	247.6	247.6	247.6	247.6	247.6	247.6	247.6	247.6	247.6	247.5
3,000	247.5	247.5	247.5	247.6	247.6	247.7	247.7	247.7	247.7	247.7	247.7	247.7	247.7	247.7	247.6
5,000	247.5	247.6	247.6	247.6	247.7	247.9	247.9	248.0	248.0	247.9	247.9	248.0	247.9	248.0	247.9
7,500	247.5	247.6	247.6	247.7	247.9	248.3	248.3	248.4	248.4	248.4	248.4	248.4	248.4	248.4	248.3
10,000	247.5	247.6	247.7	247.7	248.4	249.0	249.1	249.2	249.2	249.2	249.2	249.2	249.2	249.2	249.1
15,000	247.5	247.6	247.7	247.9	248.9	249.5	249.8	250.0	250.0	250.0	250.0	250.0	250.0	250.0	249.8

* Selkirk Lake.

Table 15

River Current Velocities, Plan 7

Discharge Q, cfs	Channel Center-Line Velocity at Indicated Station, fps Sta 300	Sta 1000	Sta 2000	Sta 3000	Sta 4000	Sta 5000	Sta 6000
	swl = +2.0 ft Referred to lwd = 244.8						
1,500	1.1	1.4	0.6	0.6	0.6	0.6	0.4
3,000	2.0	2.5	1.0	1.1	1.1	0.9	0.6
5,000	4.3	5.2	2.1	1.9	1.6	1.7	1.7
7,500	6.9	8.0	2.7	2.5	2.3	2.5	1.2
10,000	8.7	10.4	4.0	2.0	2.2	2.4	1.6
15,000	10.4	11.5	4.9	1.6	3.0	3.0	2.2
	swl = +4.7 ft Referred to lwd = 247.5						
1,500	0.7	0.9	0.5	0.4	0.4	0.5	0.3
3,000	1.4	1.6	0.8	0.7	0.8	0.8	0.5
5,000	2.7	3.2	1.8	1.2	1.2	1.4	0.9
7,500	3.2	4.3	2.7	1.4	1.5	1.7	1.2
10,000	5.8	6.9	4.0	1.5	1.7	2.1	1.7
15,000	8.7	10.4	4.7	1.5	1.9	2.2	2.5

Table 16

Wave Heights, Plan 11

swl = +4.7 ft Referred to lwd = 247.5

Test Direction	Test Wave Period sec	Test Wave Height ft	Wave Height at Indicated Gage Location, ft Gage 1	Gage 2	Gage 3	Gage 4
300°	6.0	6.1	5.5	3.1	1.7	0.8
	7.8	5.5	6.0	2.9	1.9	0.8
		11.1	7.7	3.0	2.9	1.2
270°	6.0	6.1	4.2	1.7	1.9	0.7
	7.8	5.5	5.5	2.4	2.2	0.6
		11.1	6.0	2.4	2.3	0.8

Photo 1. Typical wave and shoaling patterns for existing conditions; 5.8-sec, 4.7-ft waves from 330°, 1,500-cfs discharge, swl = +2.0 ft lwd

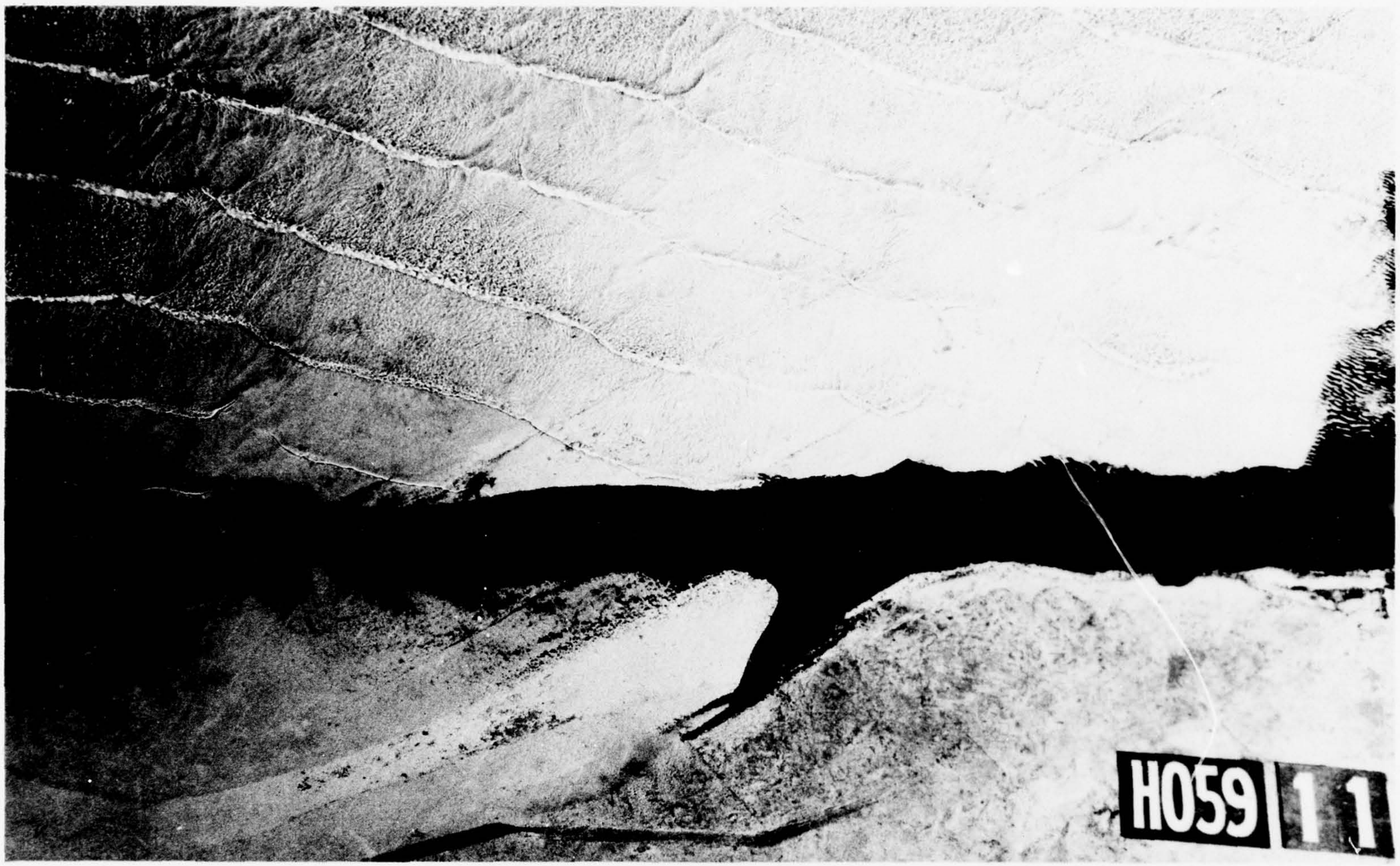

Photo 2. Typical wave and shoaling patterns for existing conditions; 7.3-sec, 8.3-ft waves from 330°, swl = +2.0 ft lwd

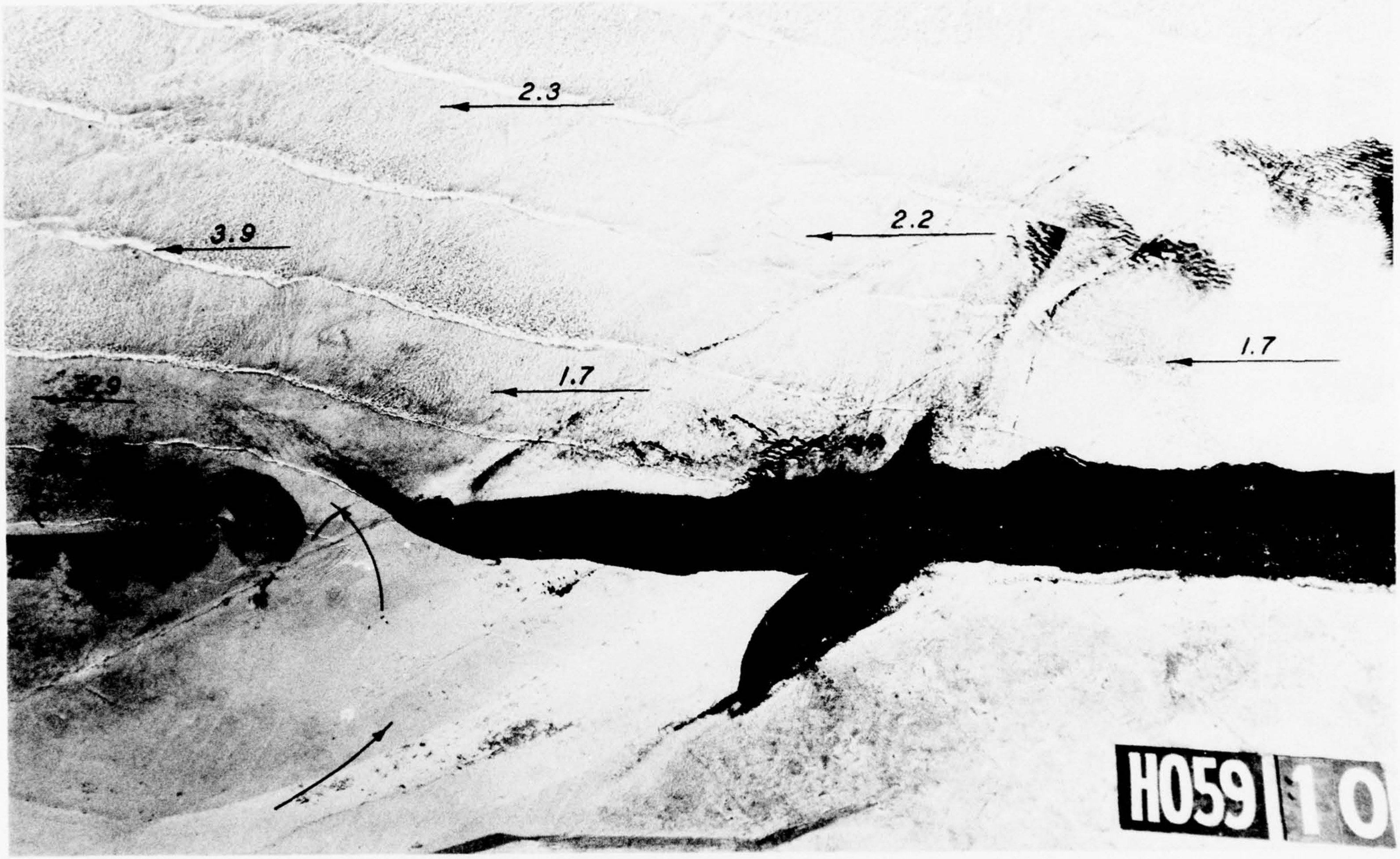

Photo 3. Typical wave, shoaling, and current patterns and current magnitudes (prototype ft per sec) for existing conditions; 7.3-sec, 8.3-ft waves from 330°, 1,500-cfs discharge, swl = +2.0 ft lwd

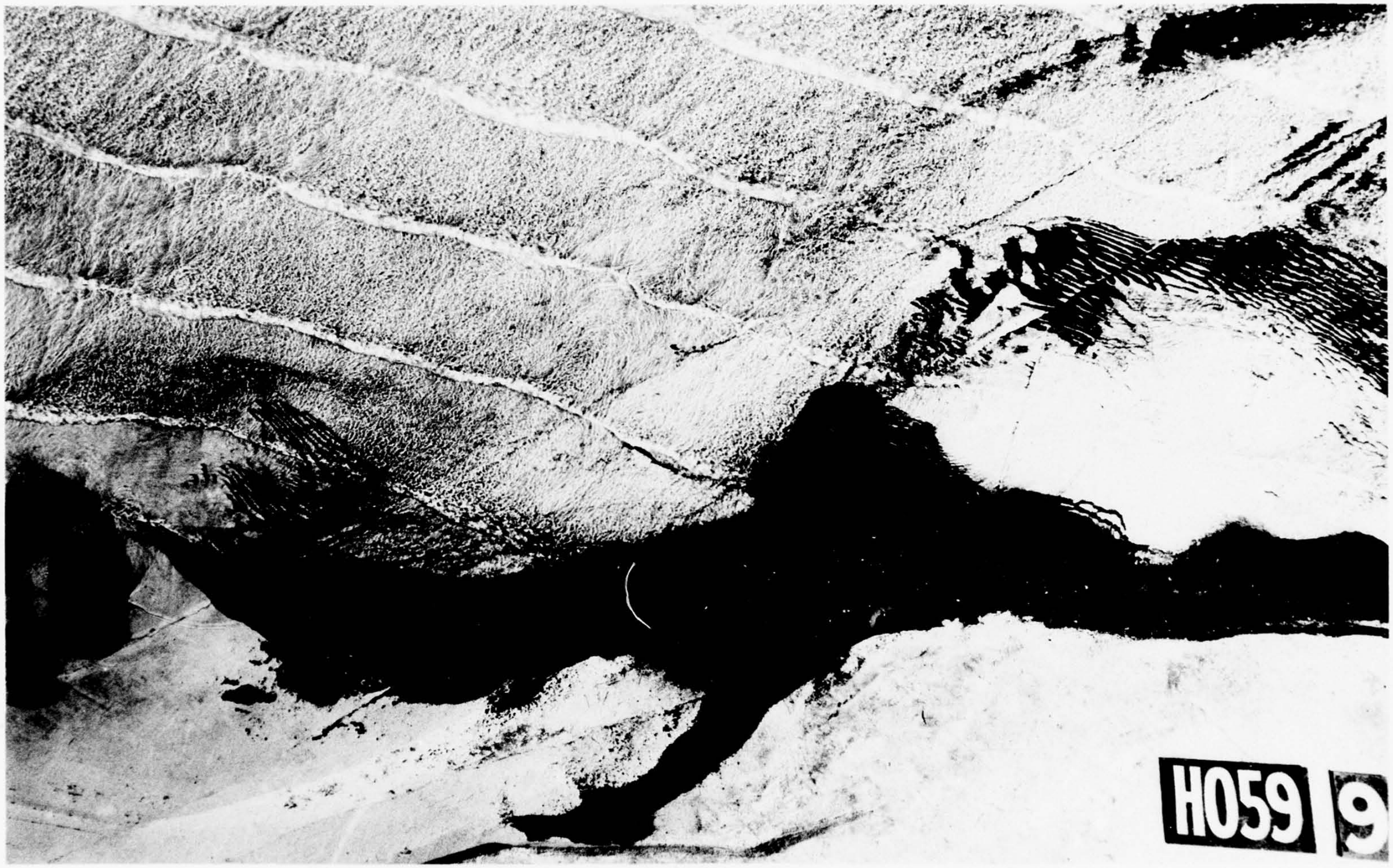

Photo 4. Typical wave and shoaling patterns for existing conditions; 7.3-sec, 8.3-ft waves from 330°, 1,500-cfs discharge, swl = +4.7 ft lwd

Photo 5. Typical wave and shoaling patterns for existing conditions; 6-sec, 6.1-ft waves from 300°, 1,500-cfs discharge, swl = +2.0 ft lwd

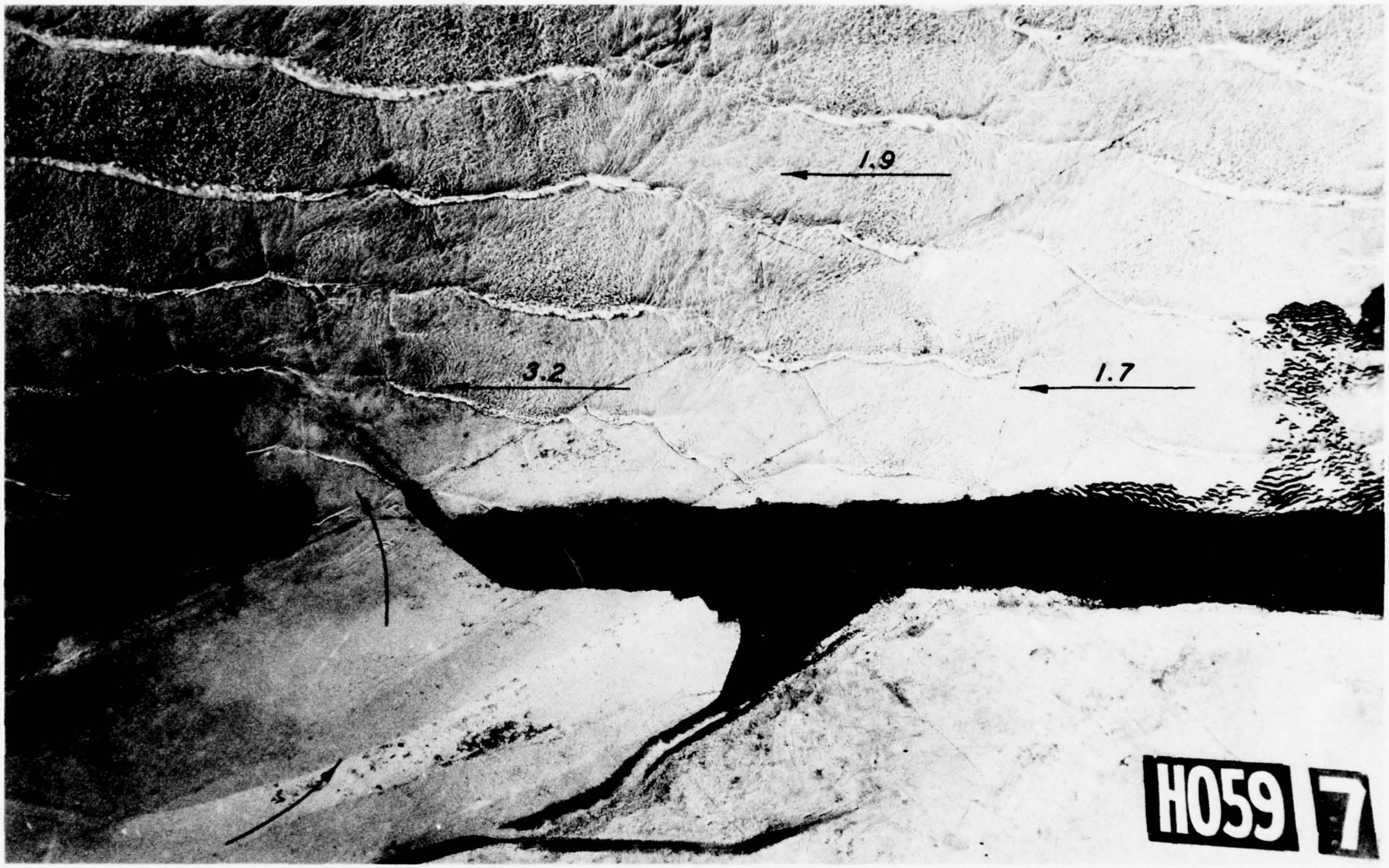

Photo 6. Typical wave, shoaling, and current patterns and current magnitudes (prototype ft per sec) for existing conditions; 7.8-sec, 11.1-ft waves from 300°, 1,500-cfs discharge, swl = +2.0 ft lwd

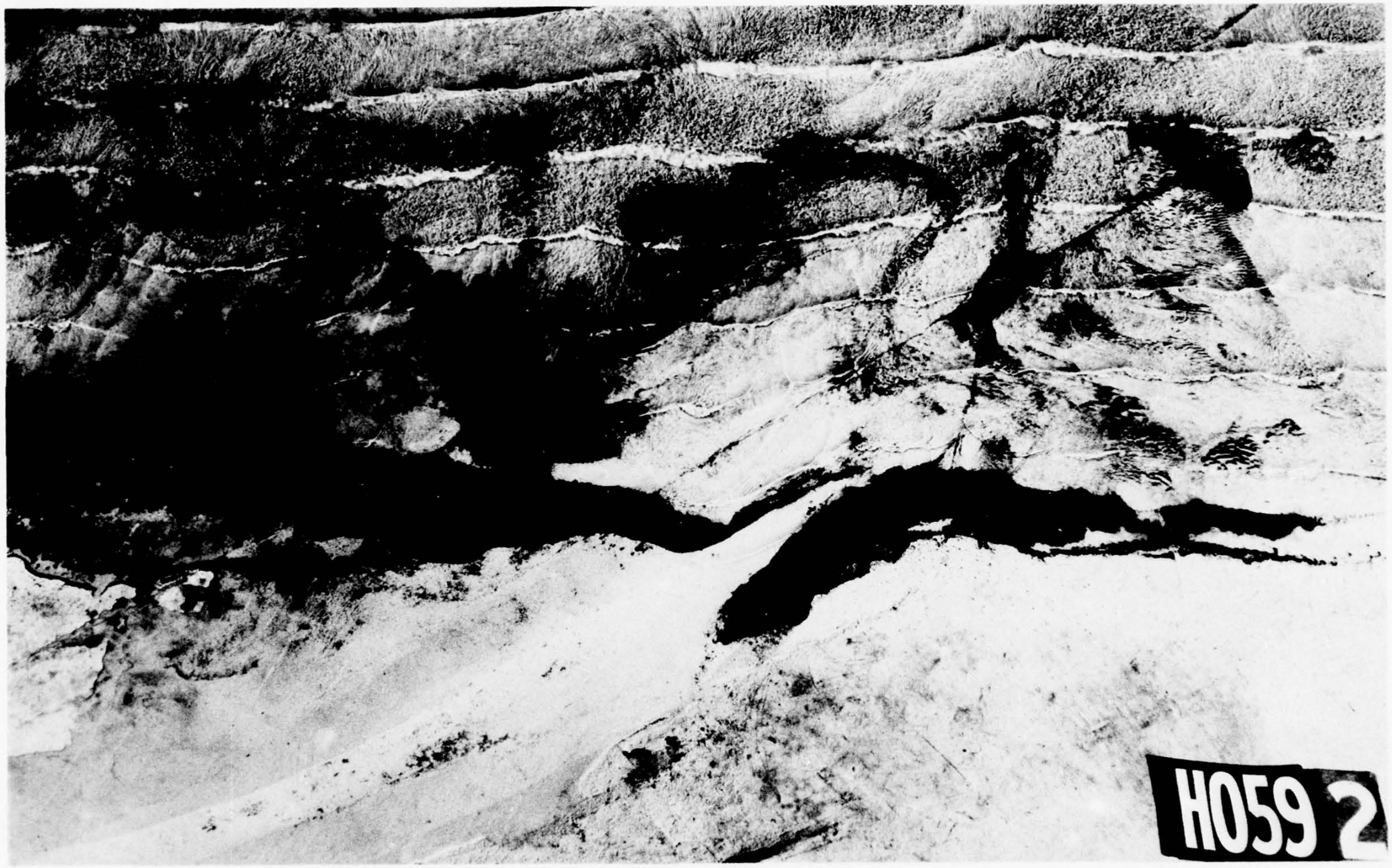

Photo 7. Typical wave and shoaling patterns for existing conditions; 6-sec, 6.1-ft waves from 270°, 1,500-cfs discharge, swl = +2.0 ft lwd

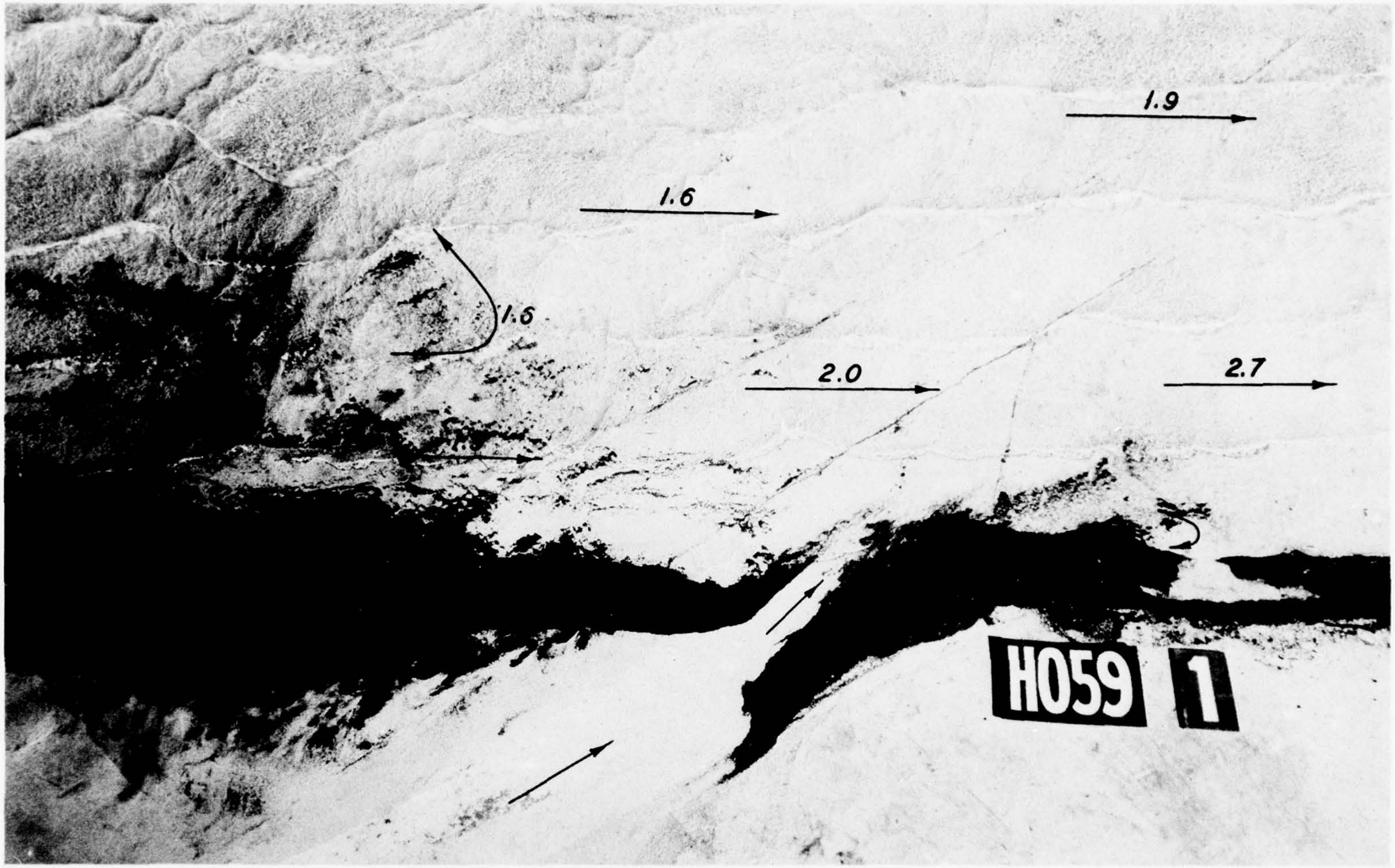

Photo 8. Typical wave, shoaling, and current patterns and current magnitudes (prototype ft per sec) for existing conditions; 7.8-sec, 11.1-ft waves from 270°, 1,500-cfs discharge, swl = +2.0 ft lwd

Photo 9. Typical wave and shoaling patterns for existing conditions; 5.2-sec, 6-ft waves from 240°, swl = +2.0 ft lwd

Photo 10. Typical wave, shoaling, and current patterns and current magnitudes (prototype ft per sec) for existing conditions; 5.2-sec, 6-ft waves from 240°, 1,500-cfs discharge, swl = +2.0 ft lwd

Photo 11. Typical wave and shoaling patterns for existing conditions; 5.2-sec, 6-ft waves from 240°, 1,500-cfs discharge, swl = +4.7 ft lwd

Photo 12. Typical wave and shoaling patterns for Plan 1; 7.3-sec, 8.3-ft waves from 330°, 1,500-cfs discharge, swl = +2.0 ft lwd

Photo 13. Typical wave and shoaling patterns for Plan 1; 5.2-sec, 6-ft waves from 240°, 1,500-cfs discharge, swl = +2.0 ft lwd

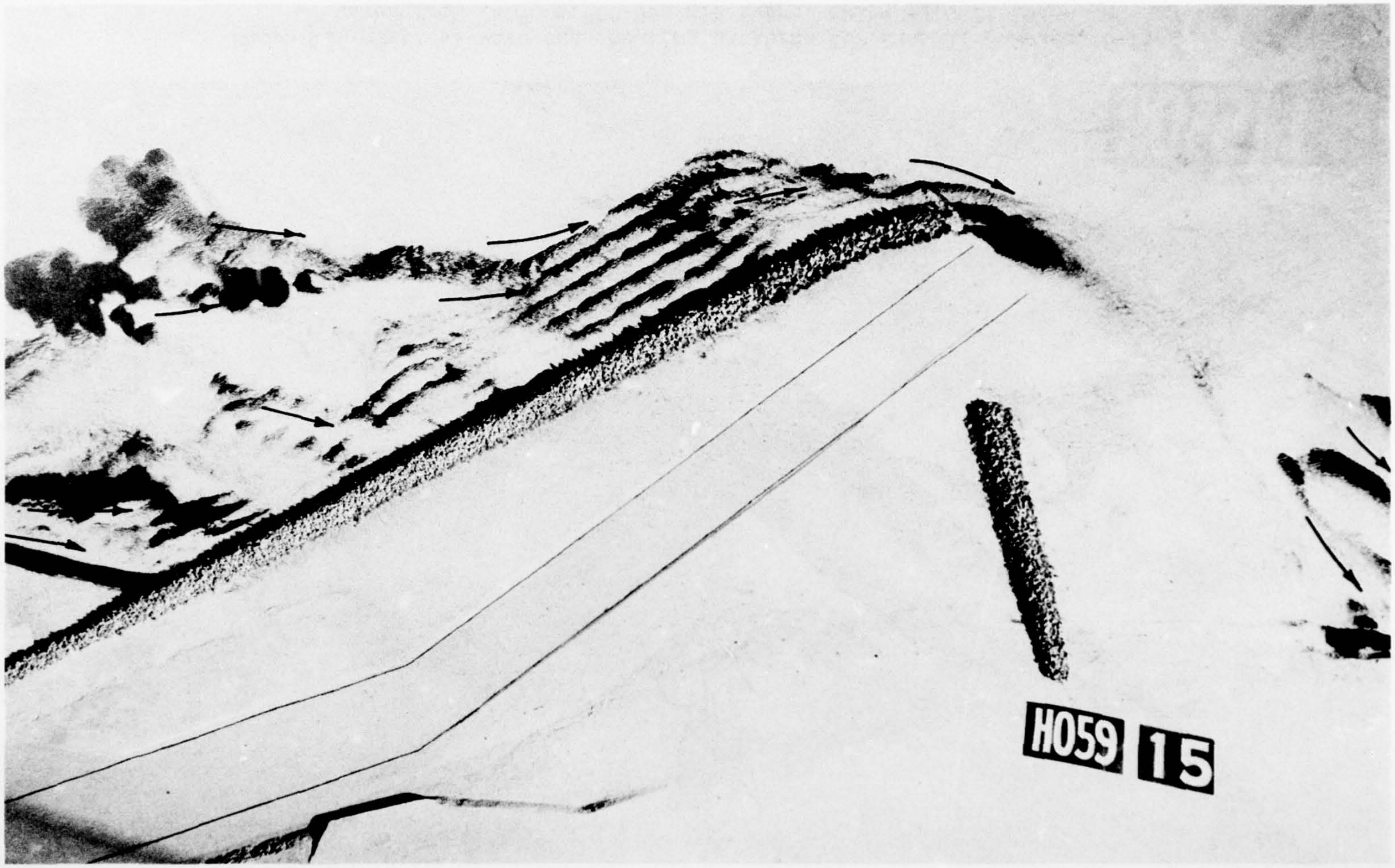

Photo 14. General movement of tracer material and deposits resulting from 5.2-sec, 6-ft waves from 240° with Plan 2 installed; swl = +2.0 ft lwd

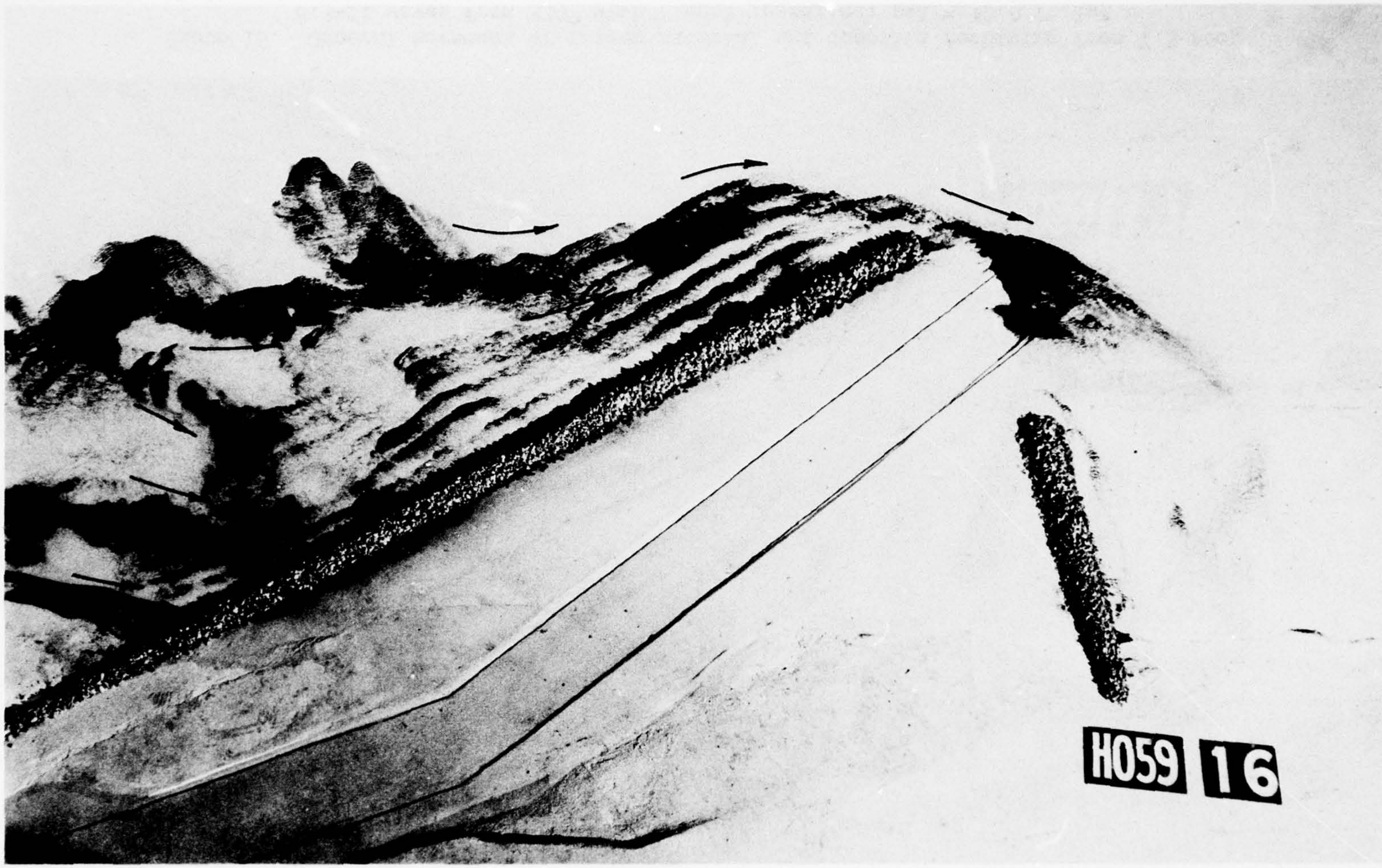

Photo 15. General movement of tracer material and deposits resulting from 5.2-sec, 6-ft waves from 240° with Plan 3 installed; swl = +2.0 ft lwd

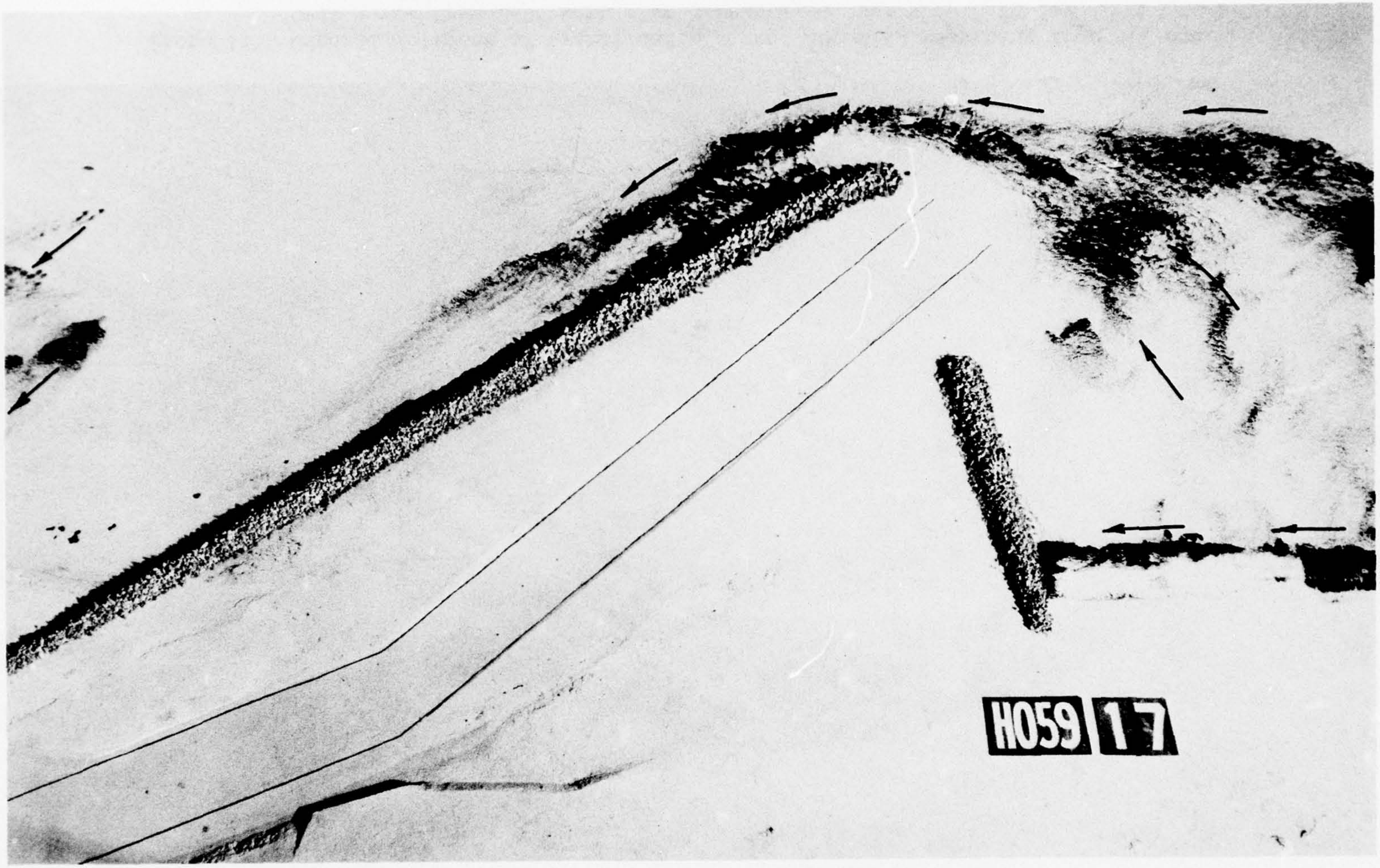

Photo 16. General movement of tracer material and deposits resulting from 7.3-sec, 8.3-ft waves from 330° with Plan 2 installed; swl = +2.0 ft lwd

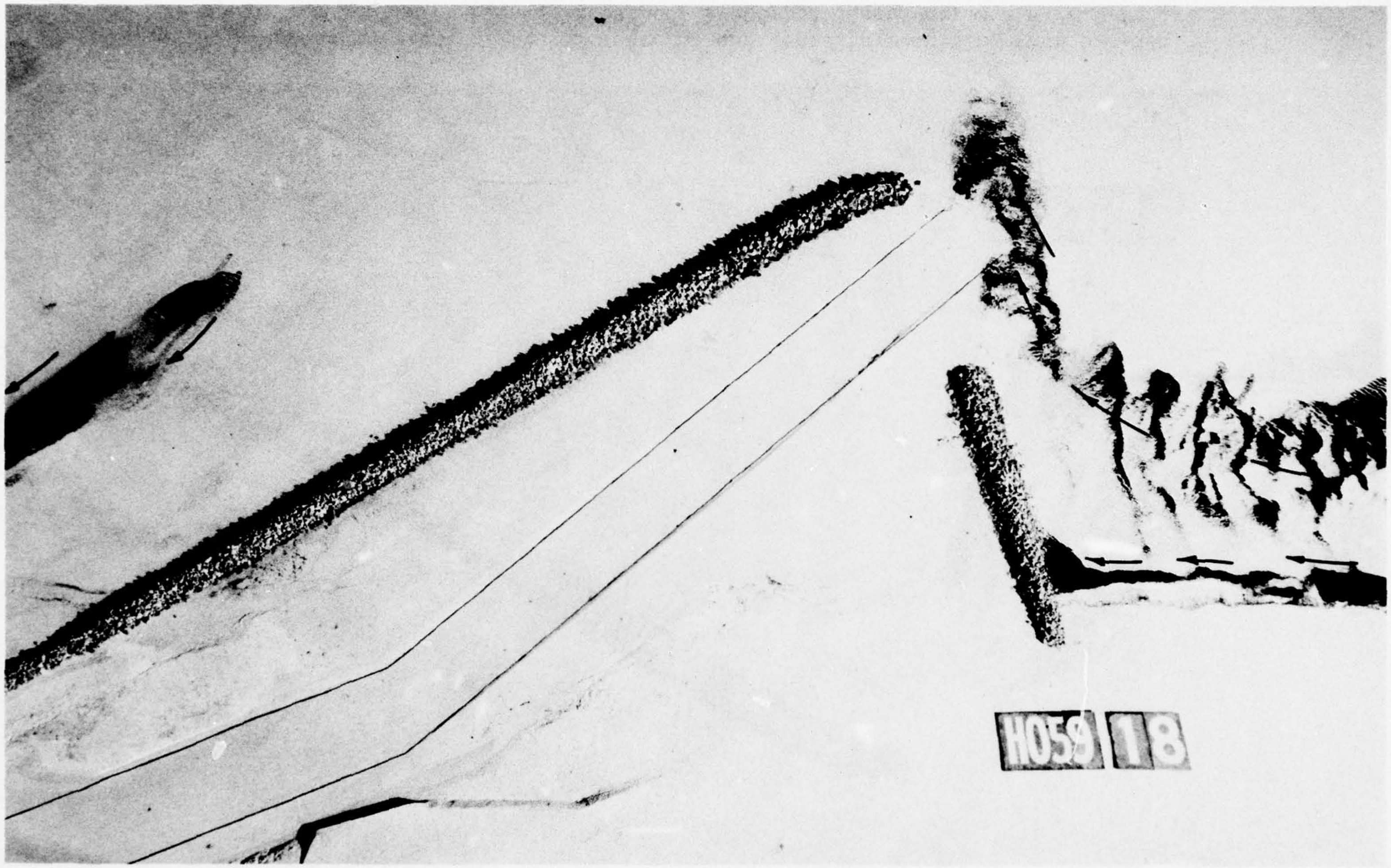

Photo 17. General movement of tracer material and deposits resulting from 5.8-sec, 4.7-ft waves from 330° with Plan 2 installed; swl = +2.0 ft lwd

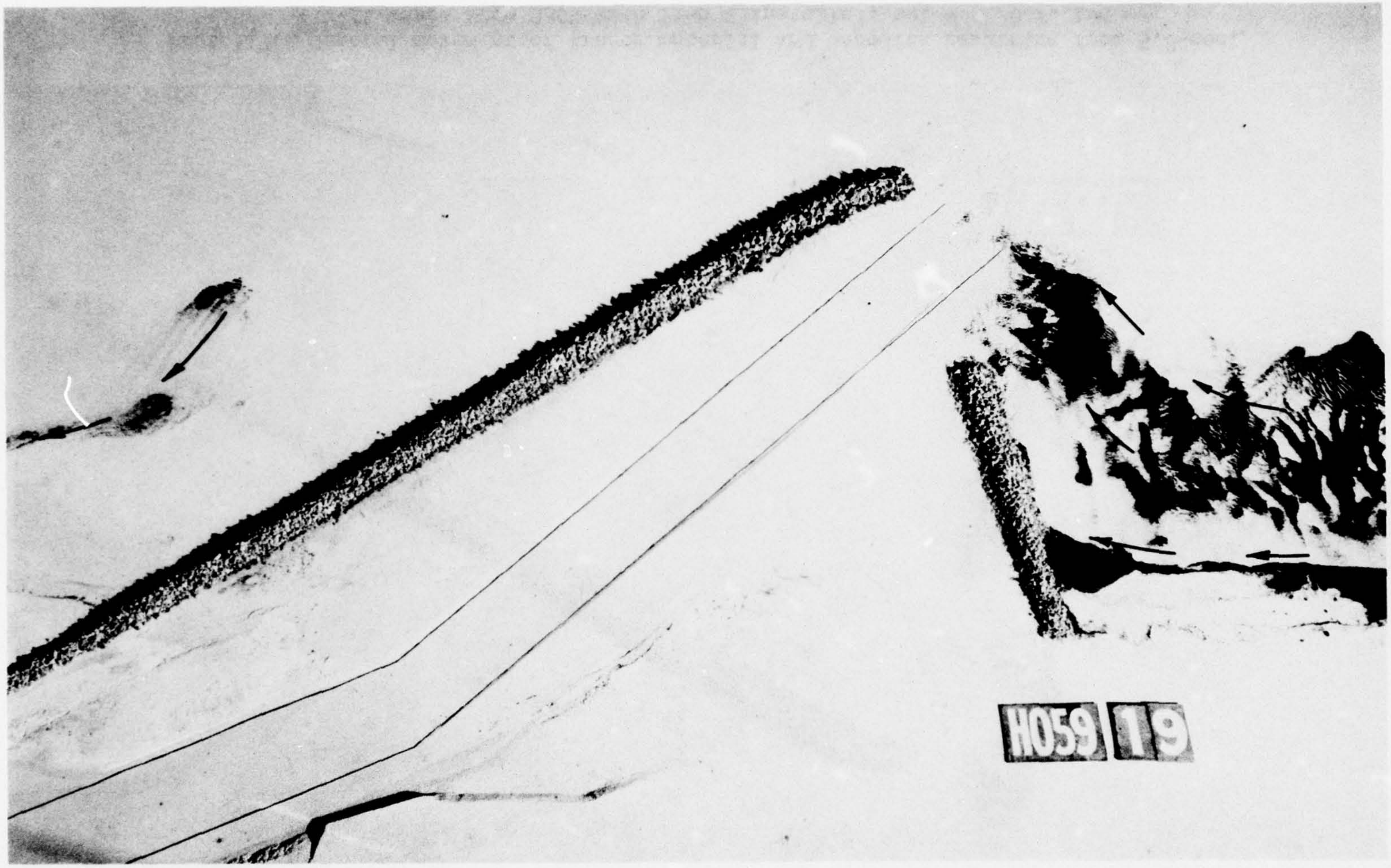

Photo 18. General movement of tracer material and deposits resulting from 5.8-sec, 4.7-ft waves from 330° with Plan 2 installed; 1,500-cfs discharge, swl = +2.0 ft lwd

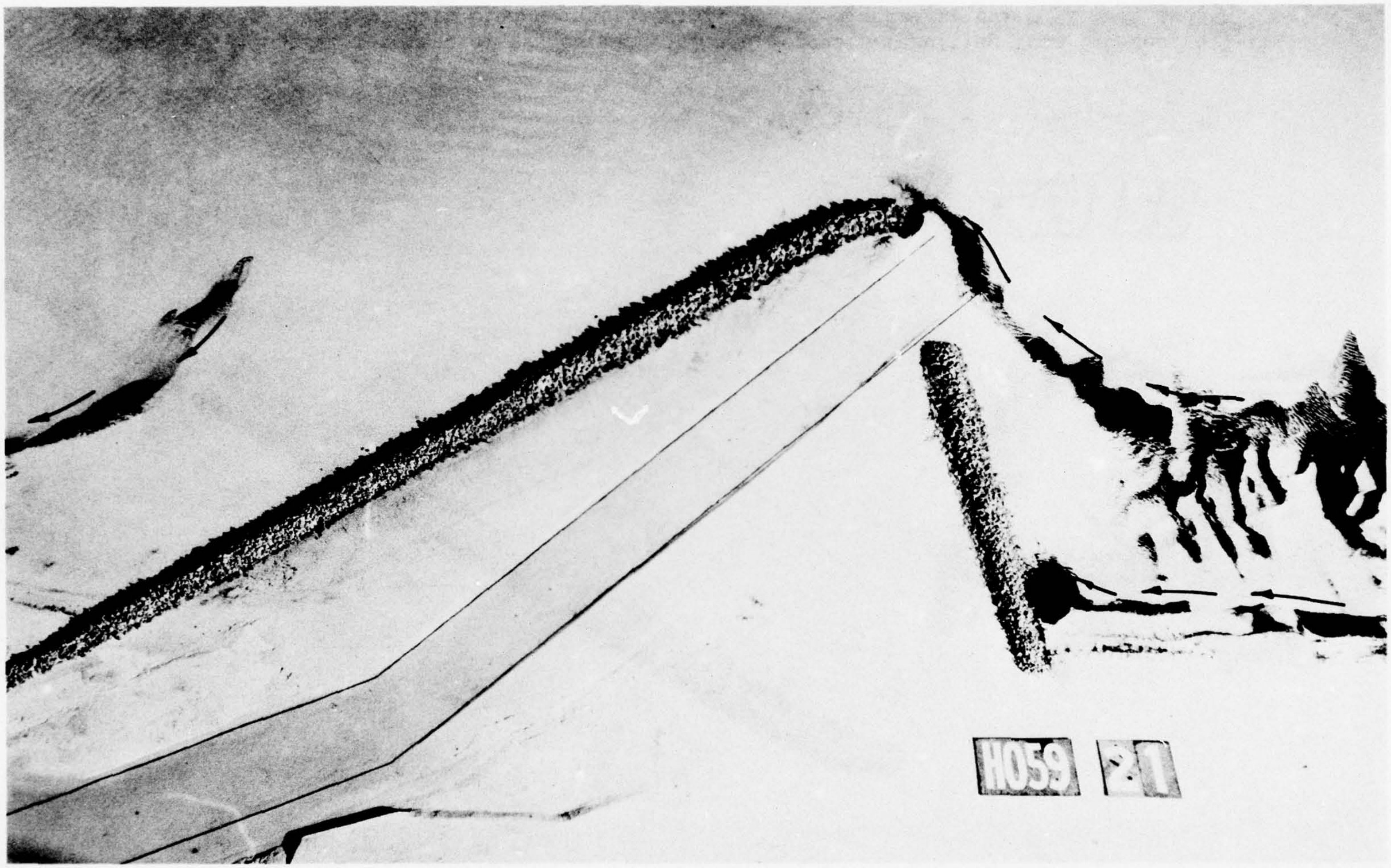

Photo 19. General movement of tracer material and deposits resulting from 5.8-sec, 4.7-ft waves from 330° with Plan 4 installed; swl = +2.0 ft lwd

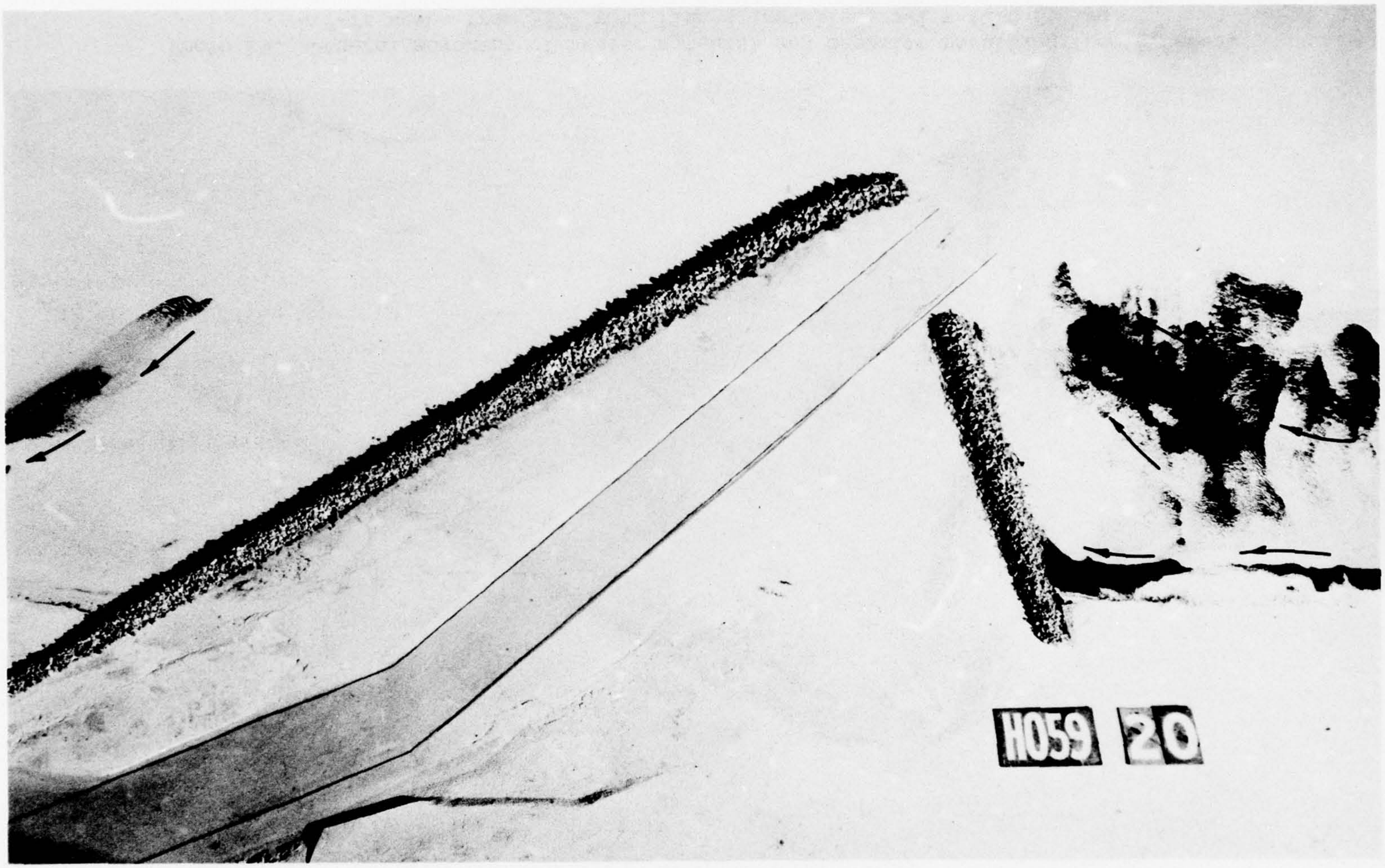

Photo 20. General movement of tracer material and deposits resulting from 5.8-sec, 4.7-ft waves from 330° with Plan 4 installed; 1,500 cfs-discharge; swl = +2.0 ft lwd

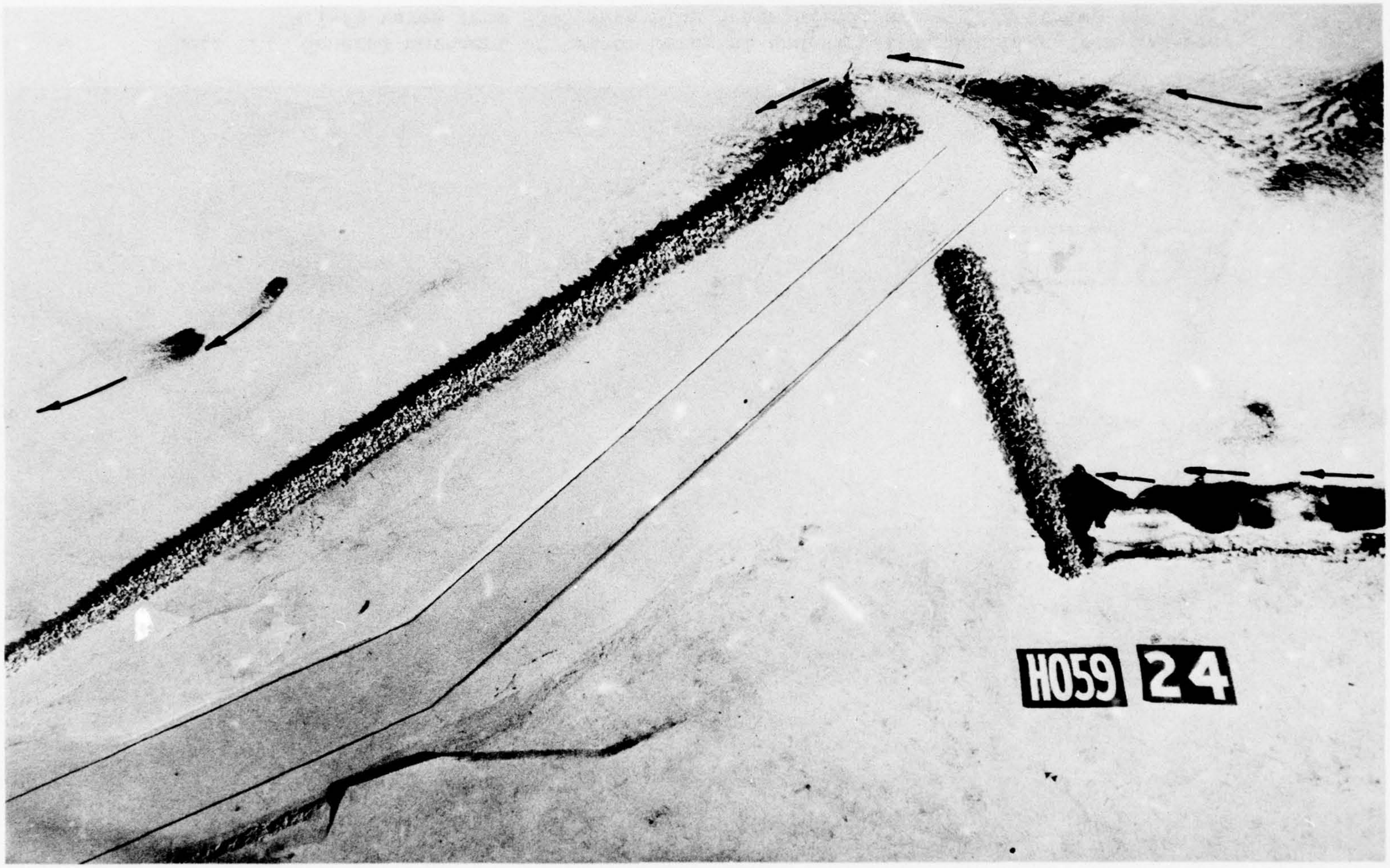

Photo 21. General movement of tracer material and deposits resulting from 7.3-sec, 8.3-ft waves from 330° with Plan 4 installed; swl = +2.0 ft lwd

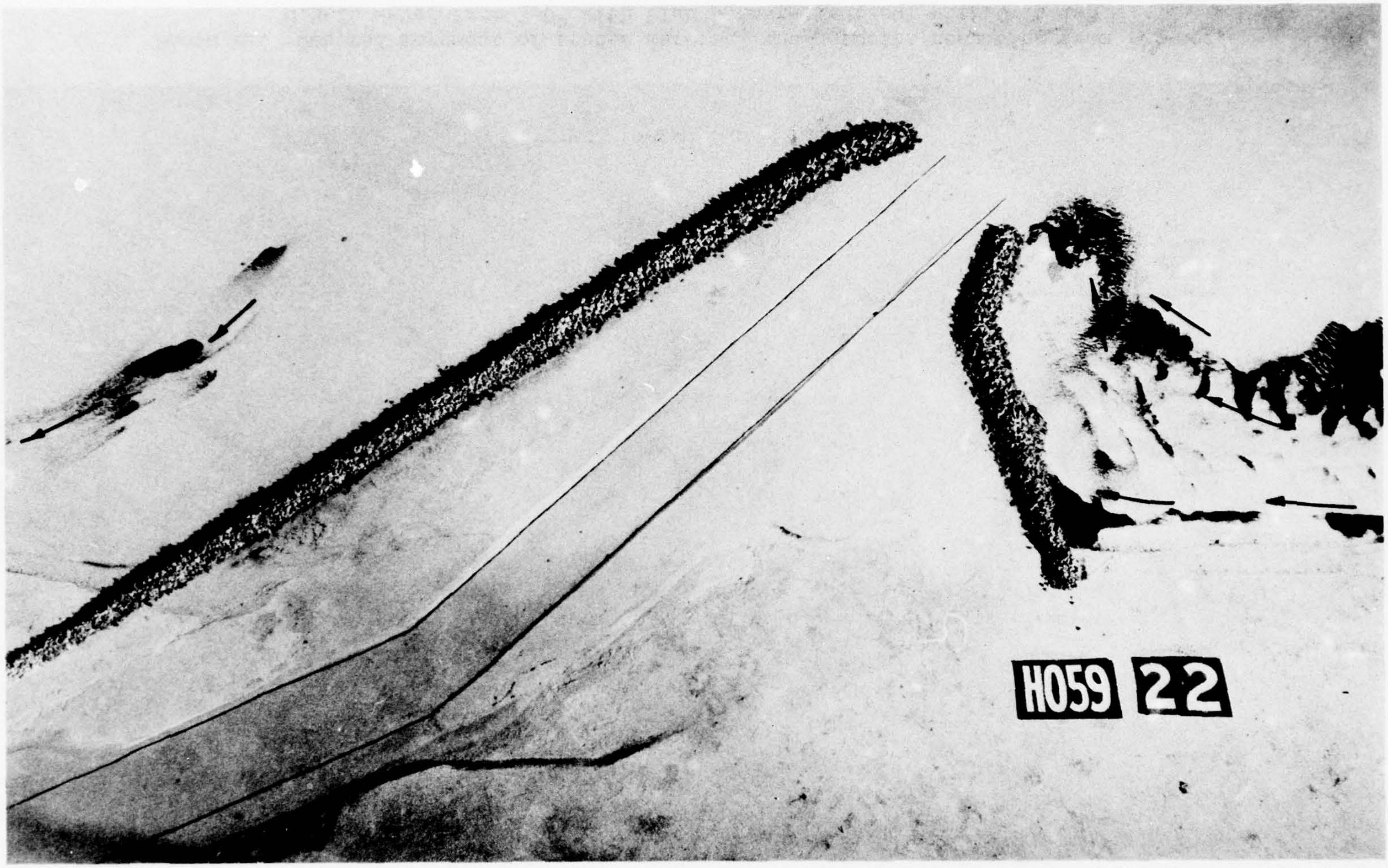

Photo 22. General movement of tracer material and deposits resulting from 5.8-sec, 4.7-ft waves from 330° with Plan 5 installed; swl = +2.0 ft lwd

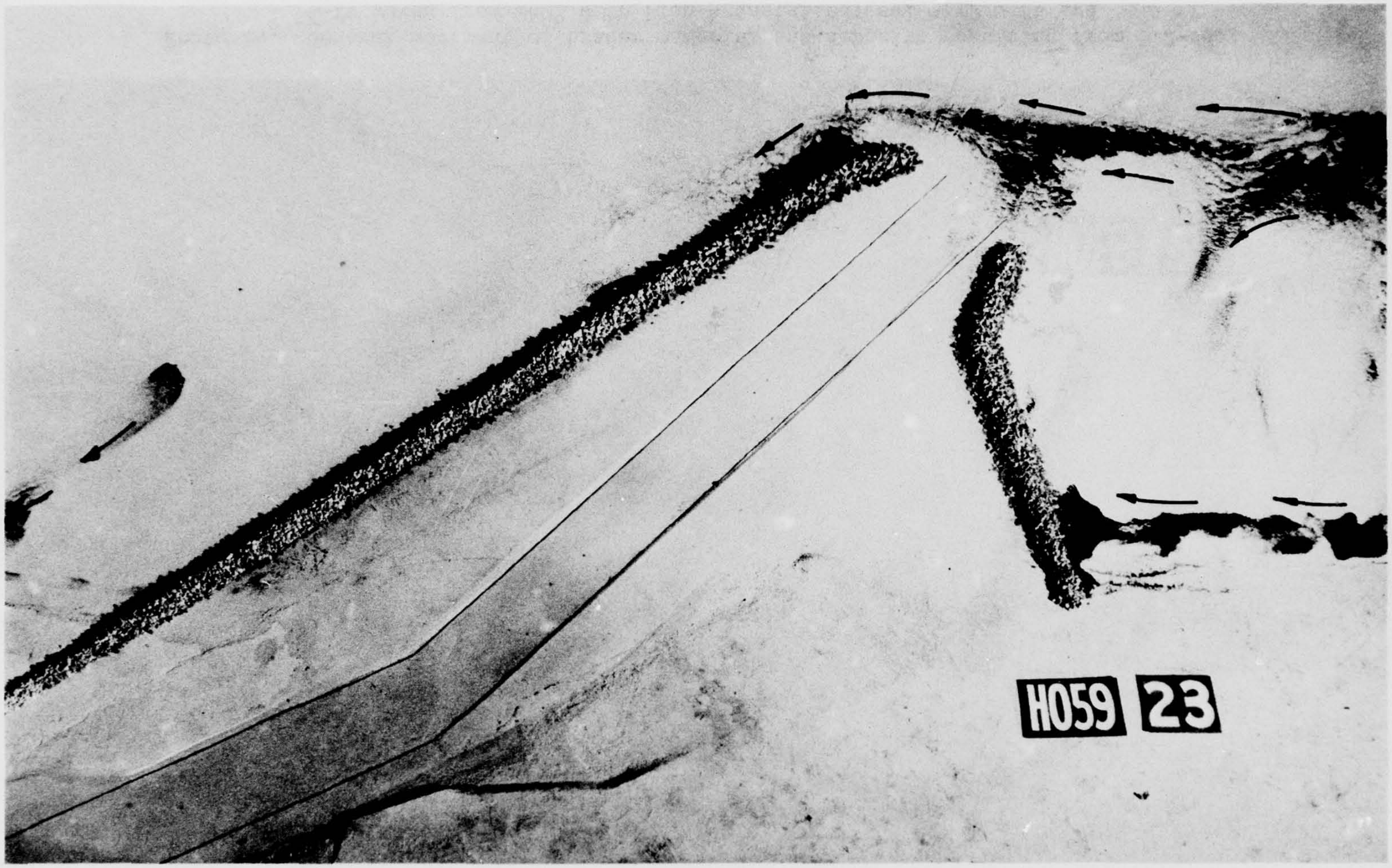

Photo 23. General movement of tracer material and deposits resulting from 7.3-sec, 8.3-ft waves from 330° with Plan 5 installed; swl = +2.0 ft lwd

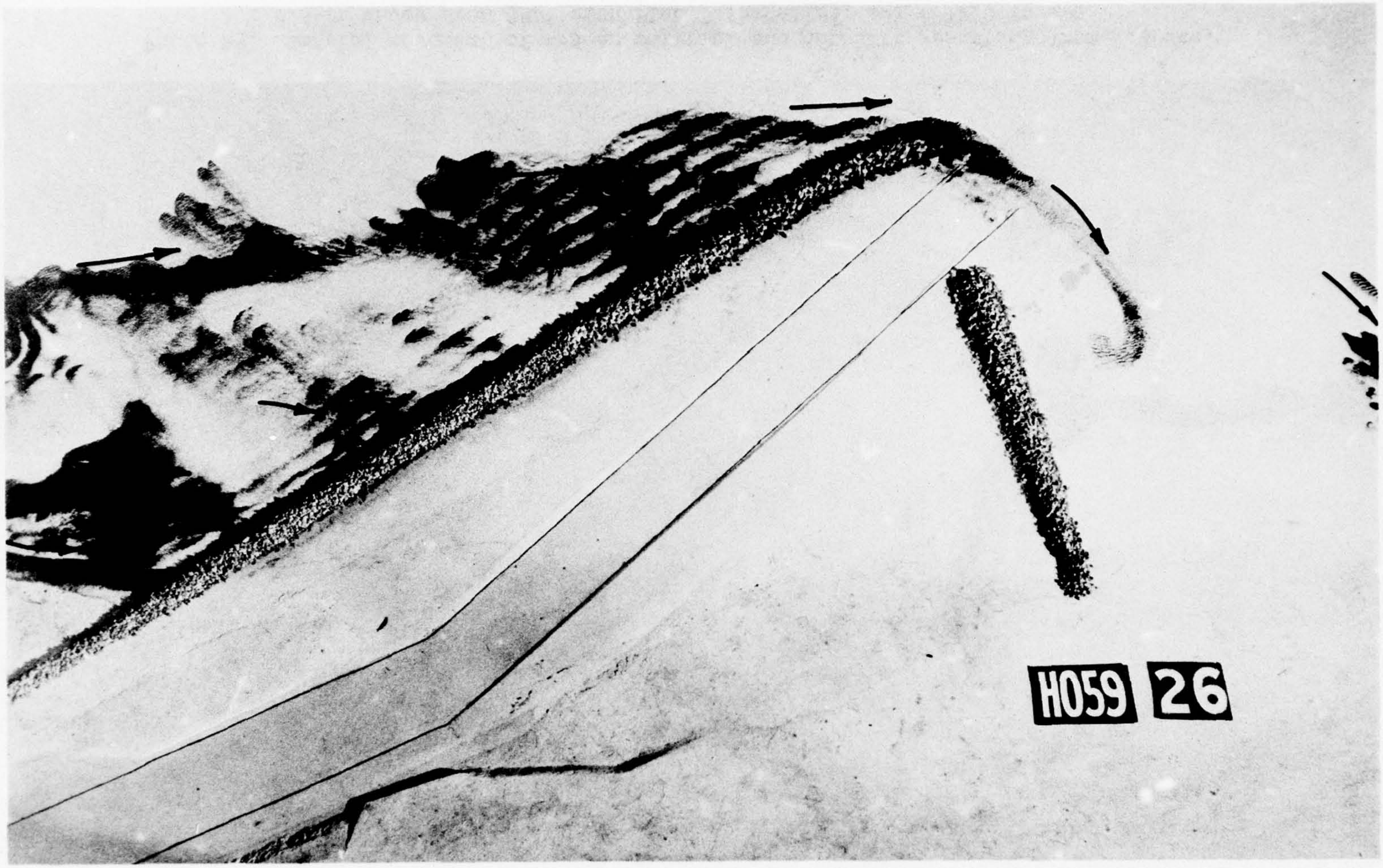

Photo 24. General movement of tracer material and deposits resulting from 5.2-sec, 6-ft waves from 240° with Plan 4 installed; swl = +2.0 ft lwd

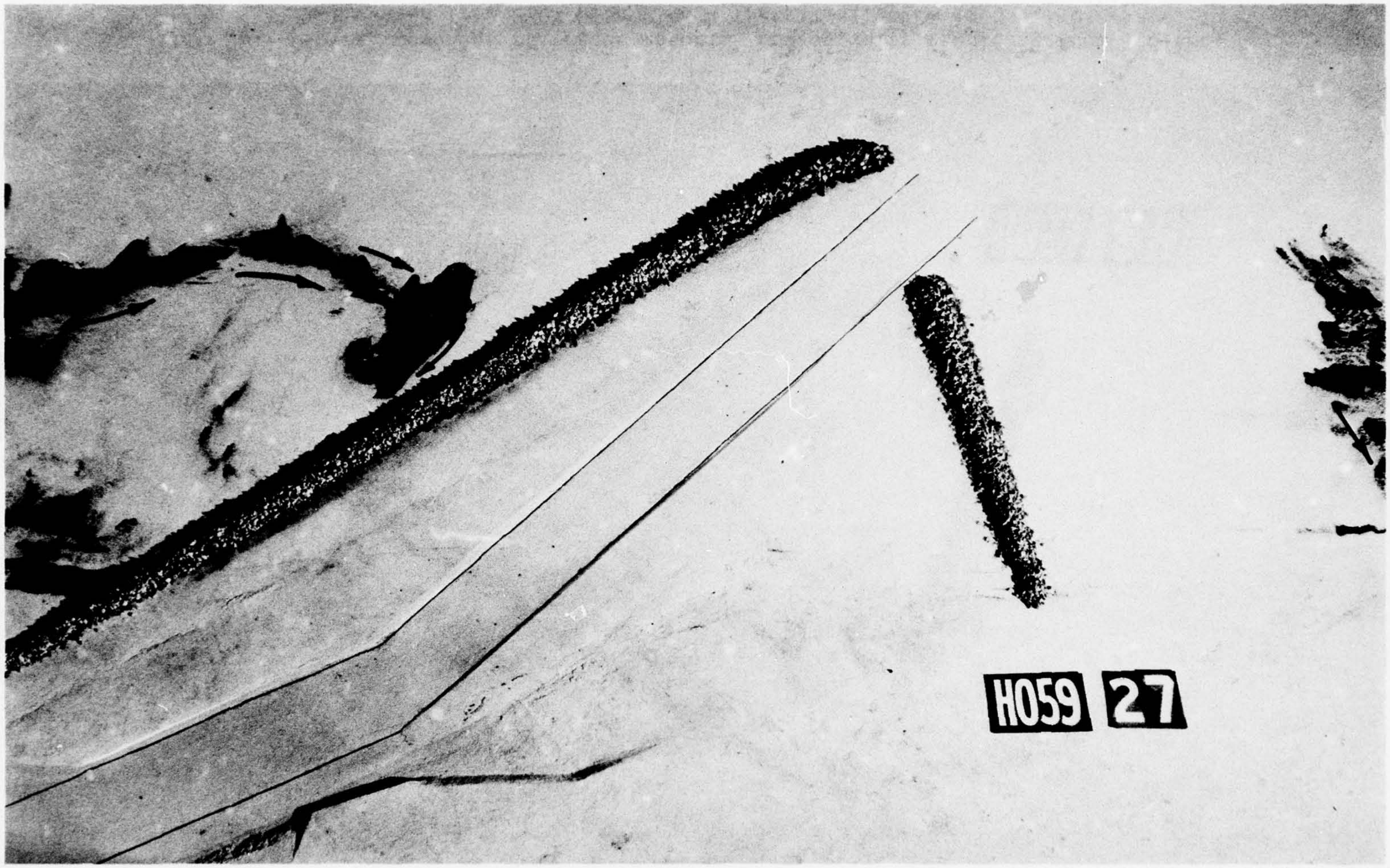

Photo 25. General movement of tracer material and deposits resulting from 6-sec, 6.1-ft waves from 270° with Plan 4 installed; swl = +2.0 lwd

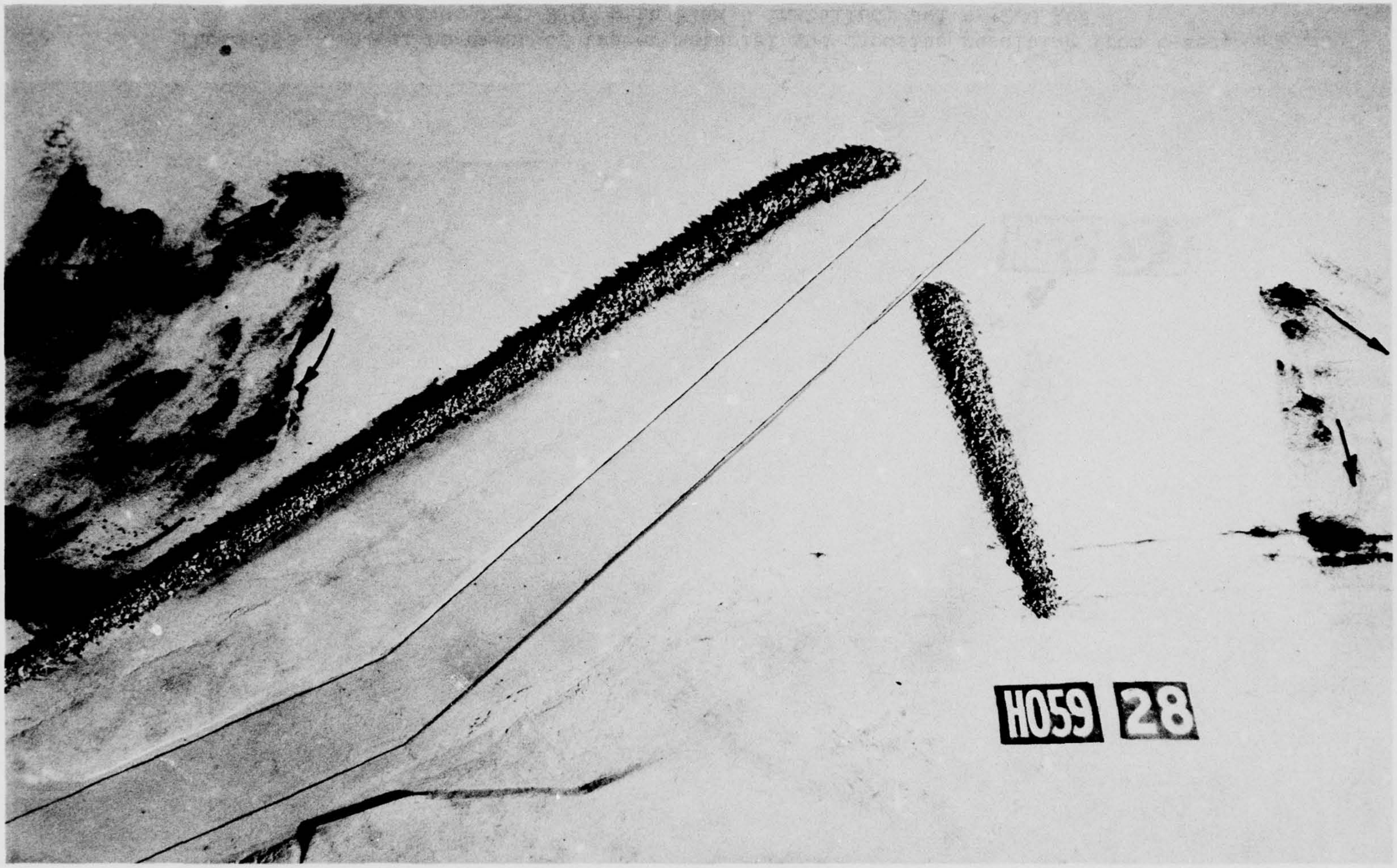

Photo 26. General movement of tracer material and deposits resulting from 7.8-sec, 11.1-ft waves from 270° with Plan 4 installed; swl = +2.0 ft lwd

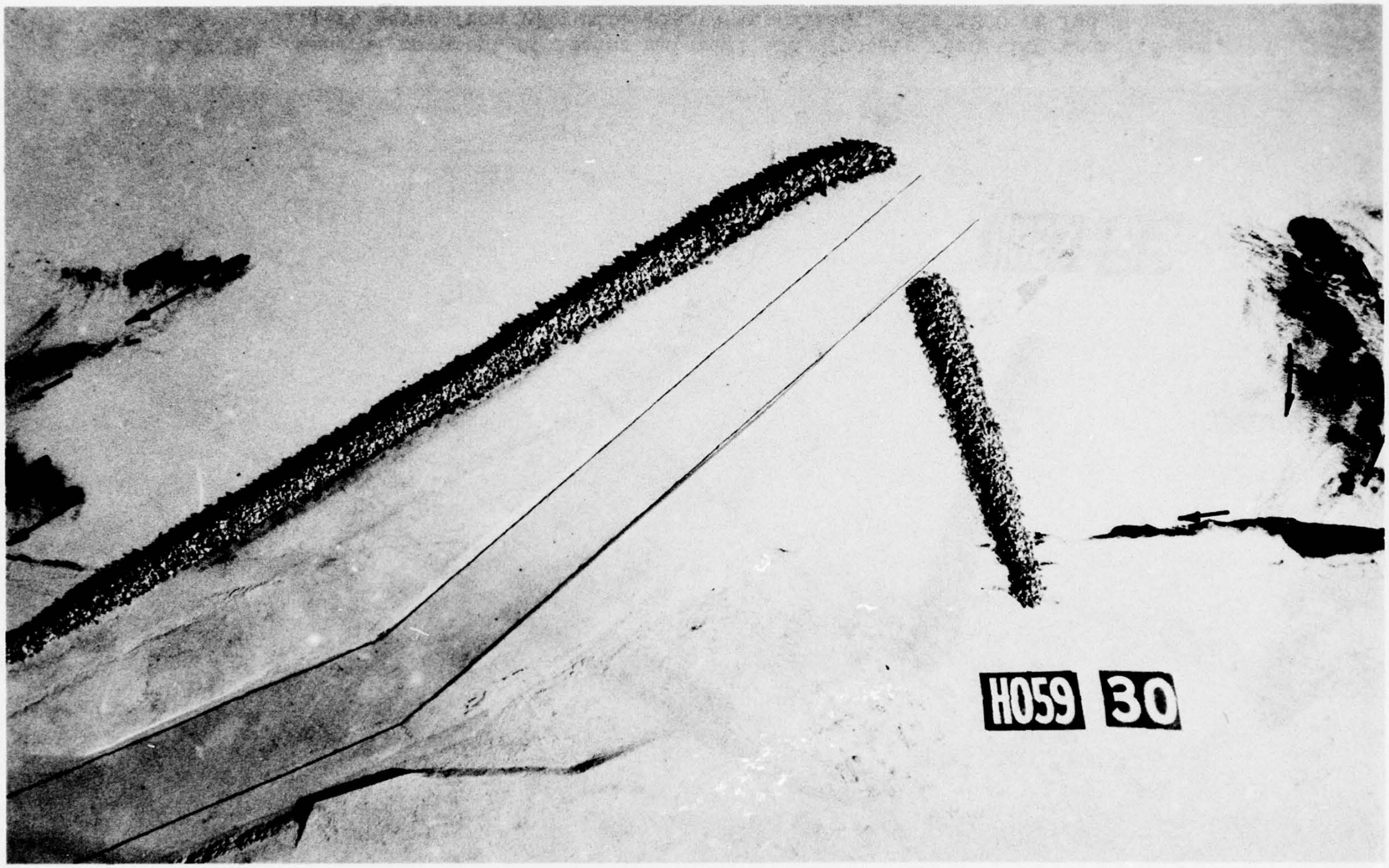

Photo 27. General movement of tracer material and deposits resulting from 6-sec, 6.1-ft waves from 300° with Plan 4 installed; swl = +2.0 ft lwd

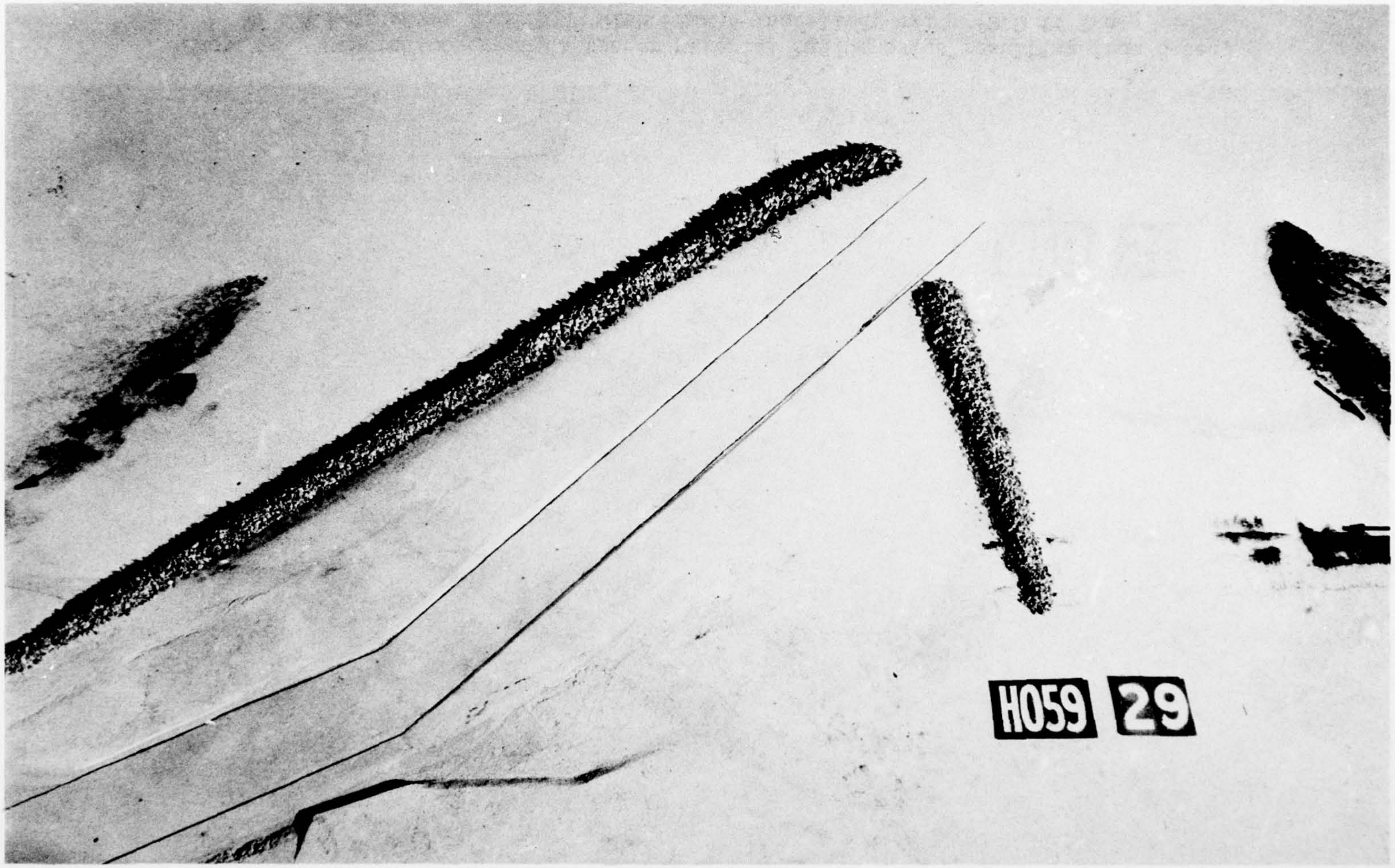

Photo 28. General movement of tracer material and deposits resulting from 7.8-sec, 11.1-ft waves from 300° with Plan 4 installed; swl = +2.0 ft lwd

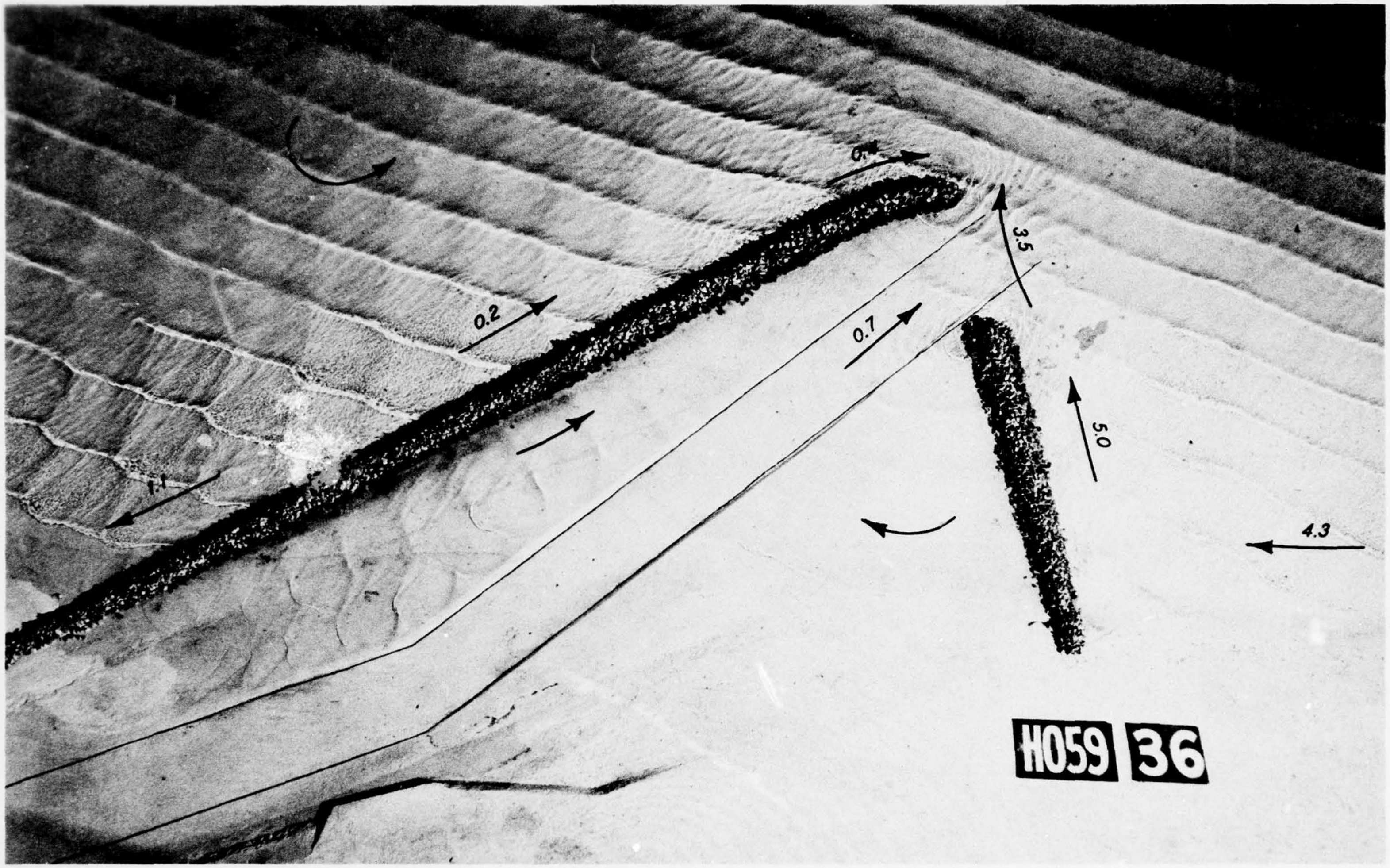

Photo 29. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 4; 5.8-sec, 4.7-ft waves from 330°; swl = +4.7 ft lwd

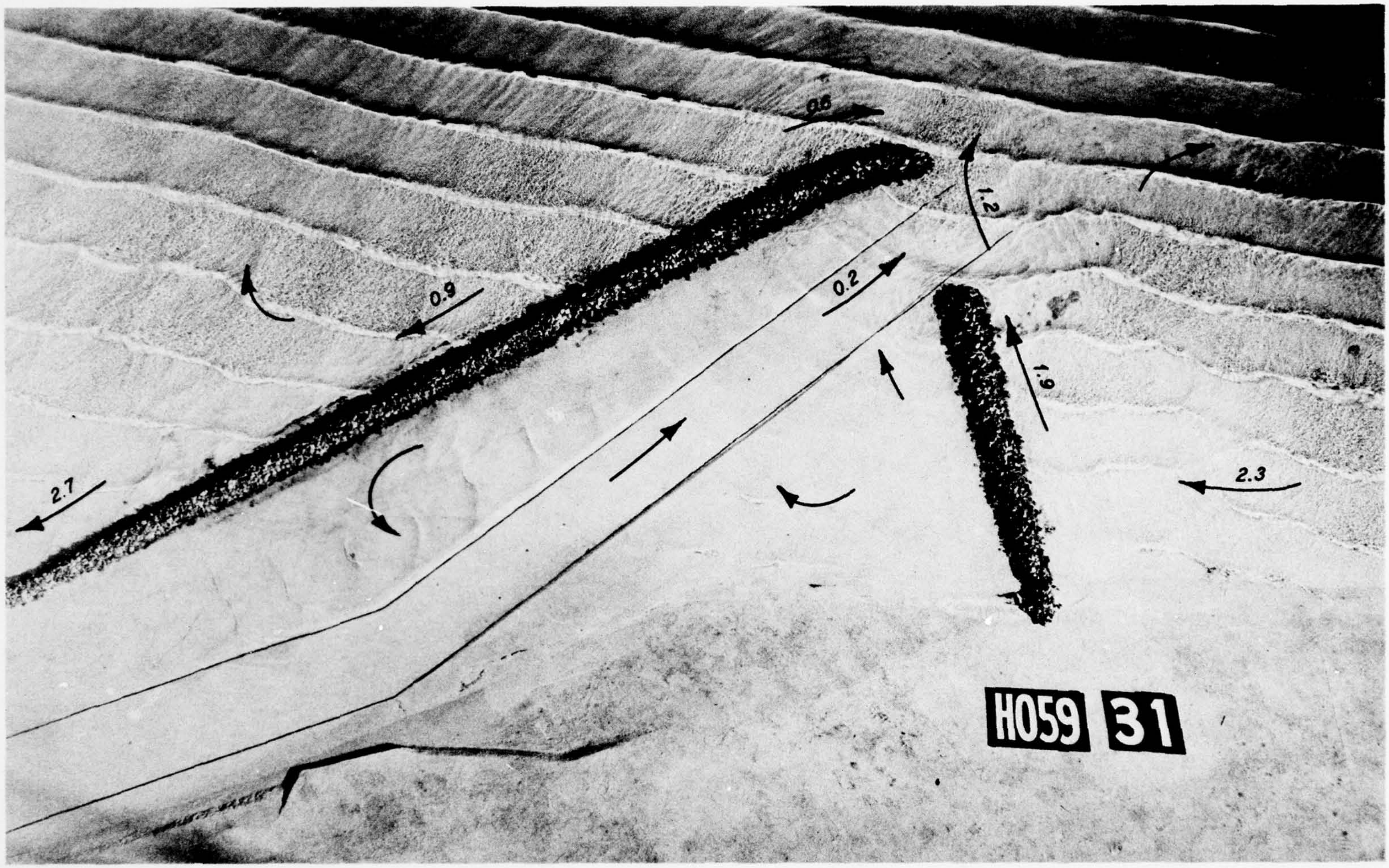

Photo 30. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 4; 6-sec, 6.1-ft waves from 300°; swl = +4.7 ft lwd

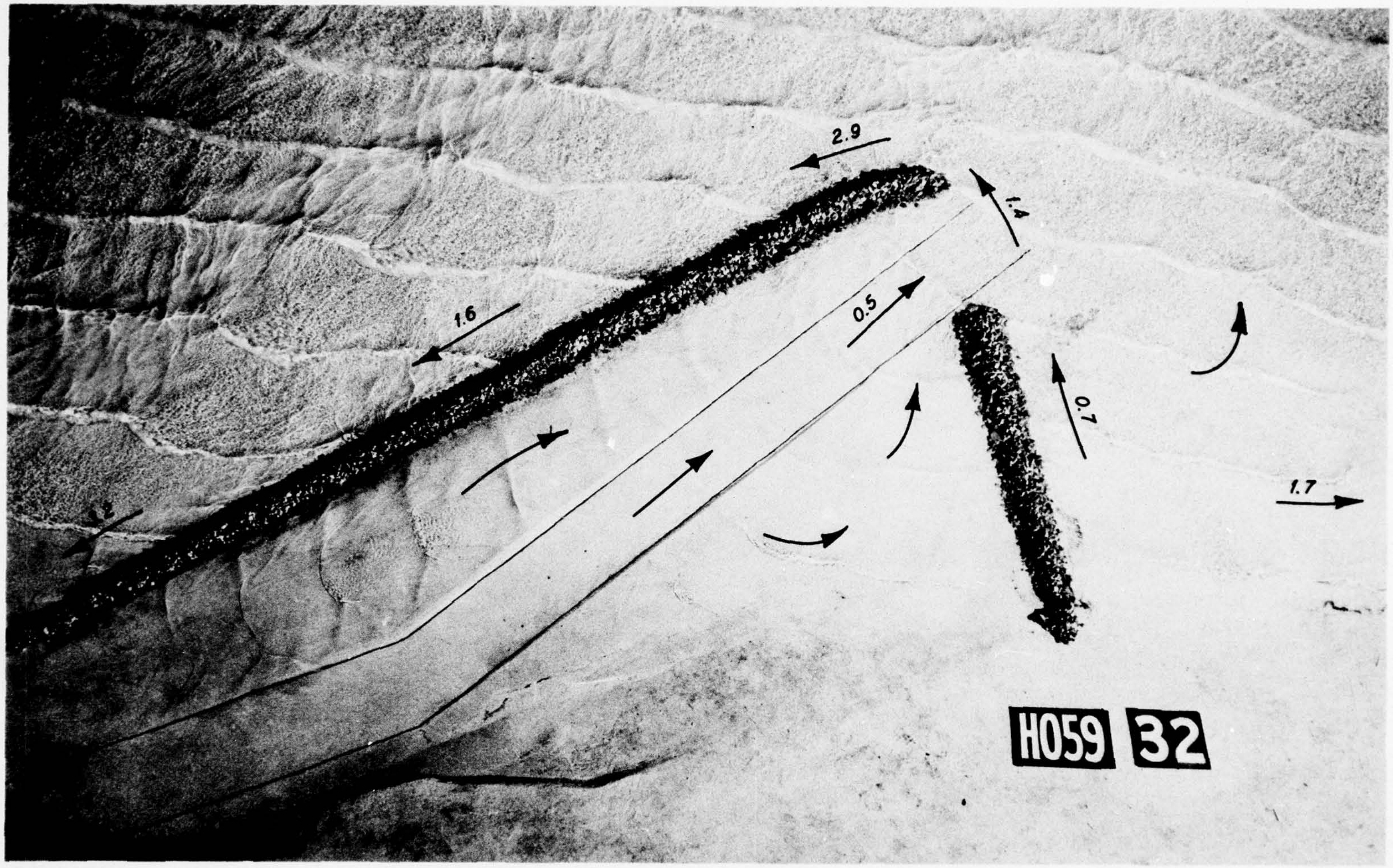

Photo 31. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 4; 7.8-sec, 11.1-ft waves from 300°; swl = +4.7 ft lwd

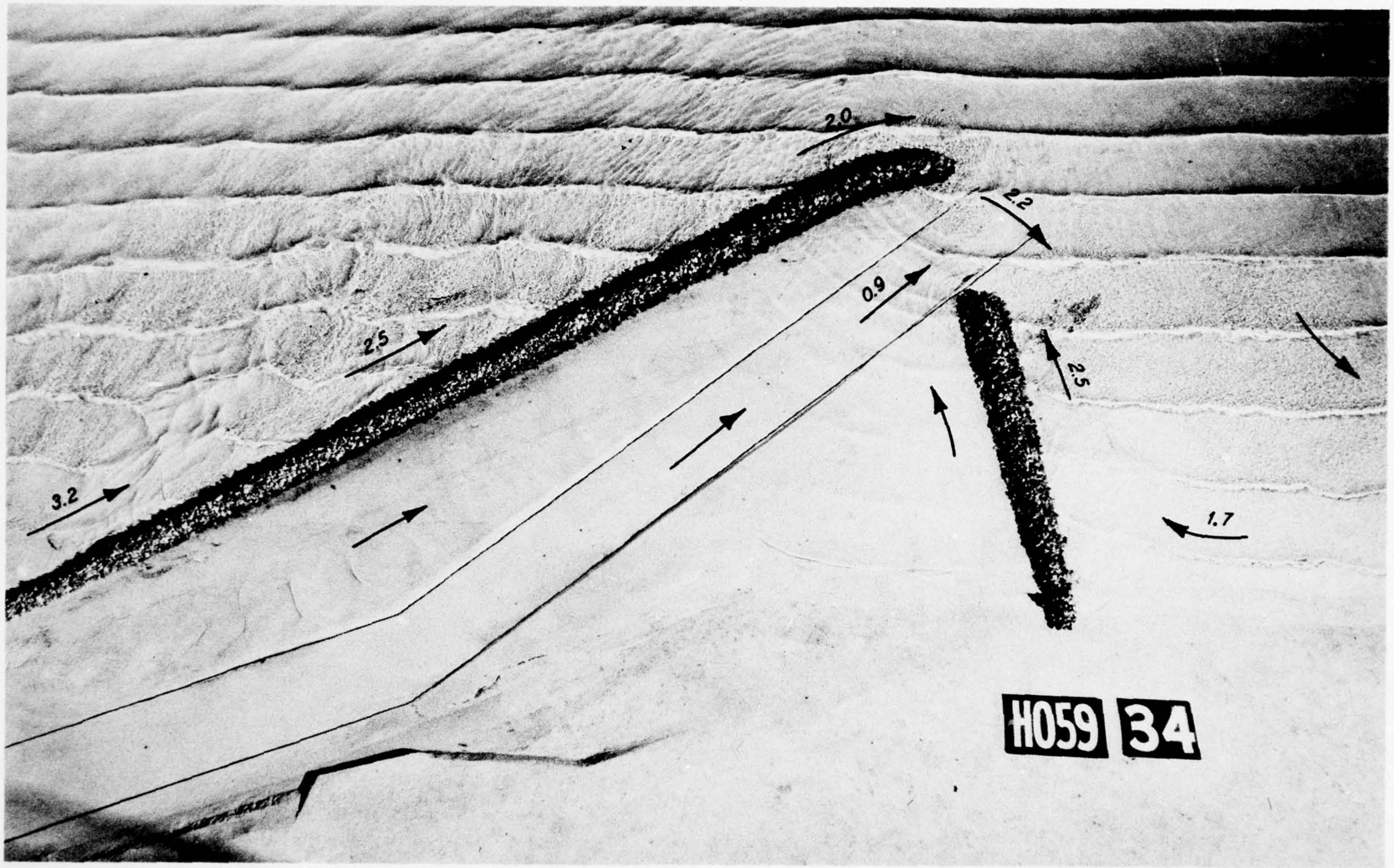

Photo 32. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 4; 6-sec, 6.1-ft waves from 270°; swl = +4.7 ft lwd

Photo 33. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 4; 7.8-sec, 11.1-ft waves from 270°; swl = +4.7 ft lwd

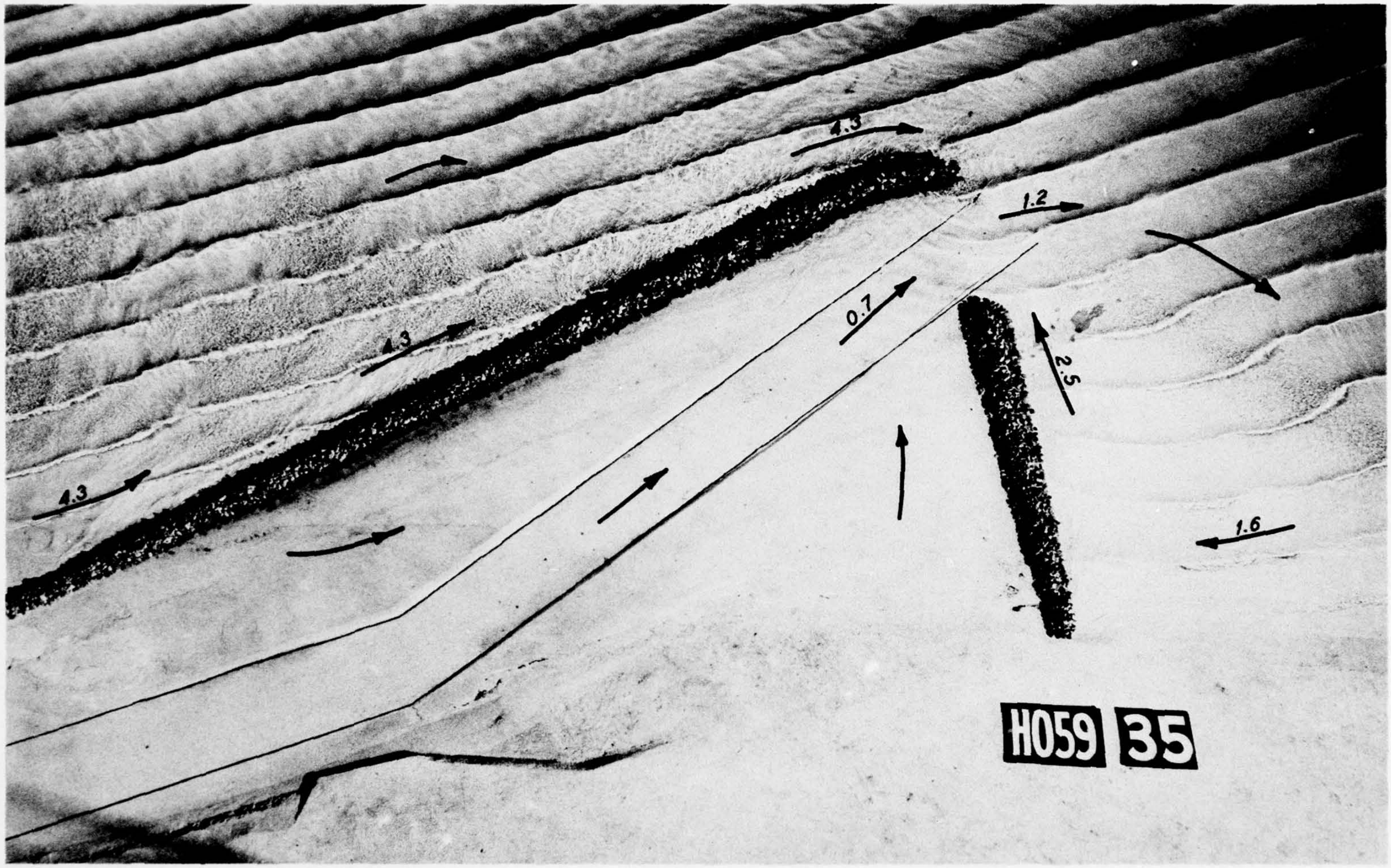

Photo 34. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 4; 5.2-sec, 6-ft waves from 240°; swl = +4.7 ft lwd

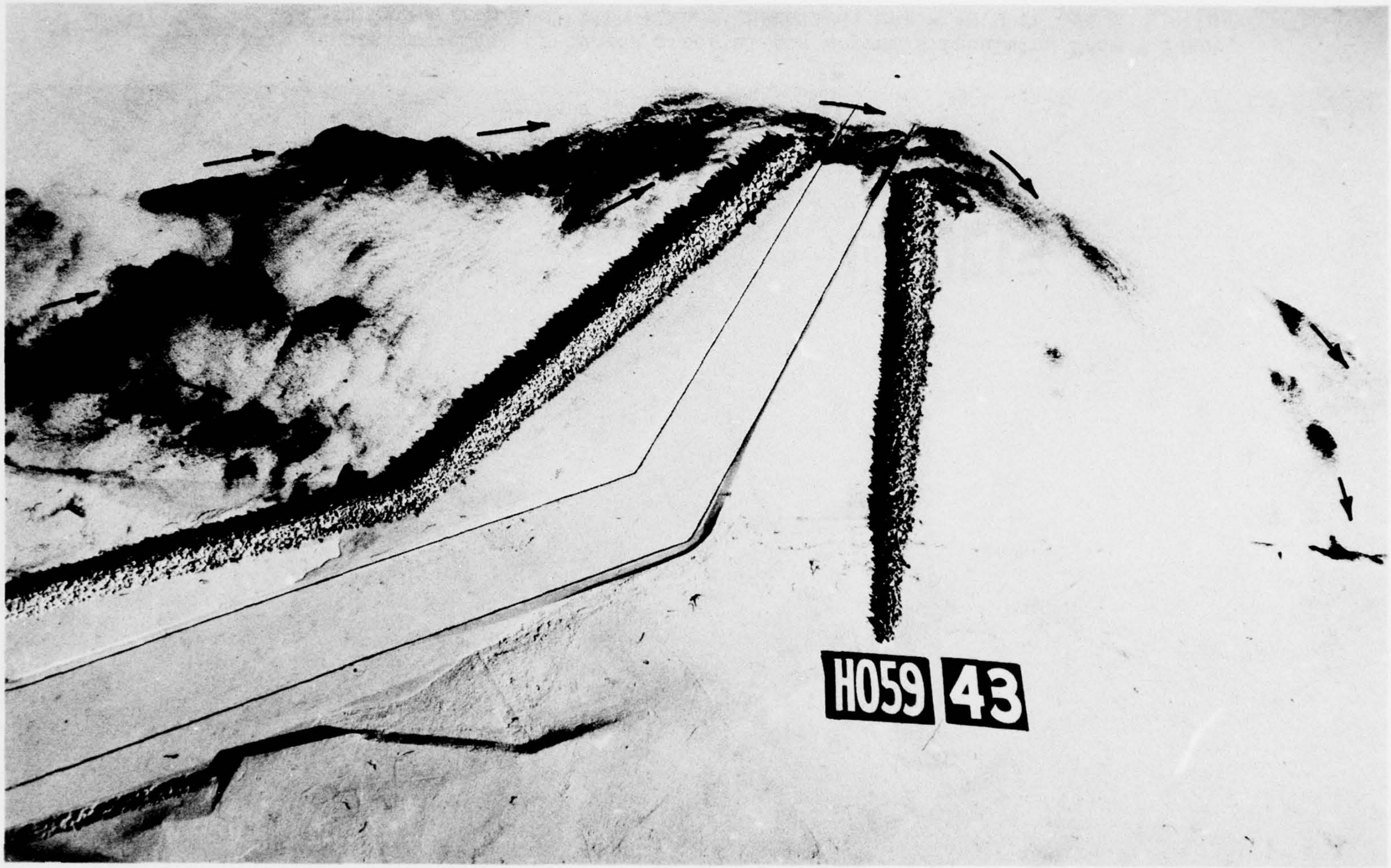

Photo 35. General movement of tracer material and deposits resulting from 5.2-sec, 6-ft waves from 240° with Plan 7 installed; swl = +2.0 ft lwd

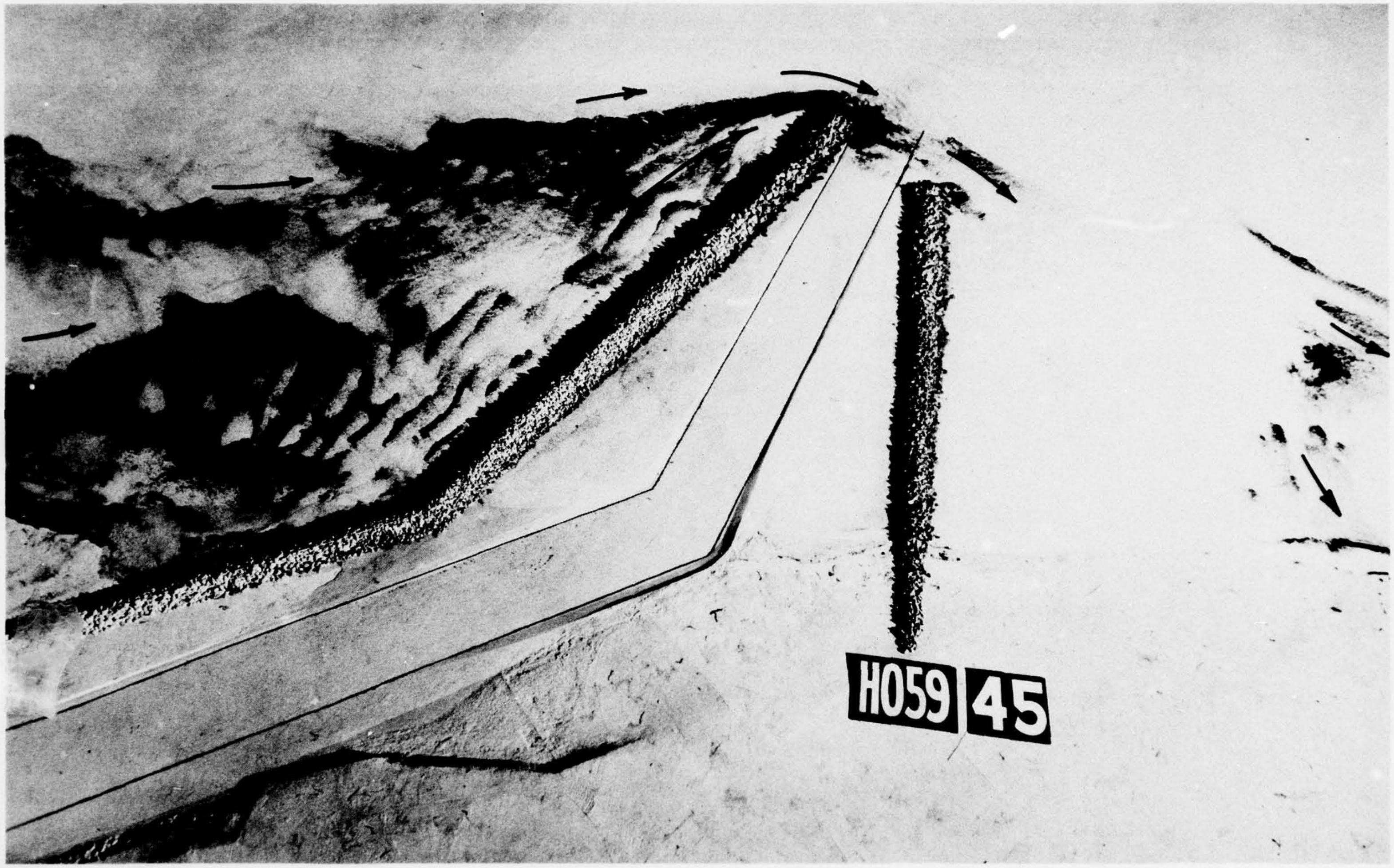

Photo 36. General movement of tracer material and deposits resulting from 5.2-sec, 6-ft waves from 240° with Plan 8 installed; swl = +2.0 ft lwd

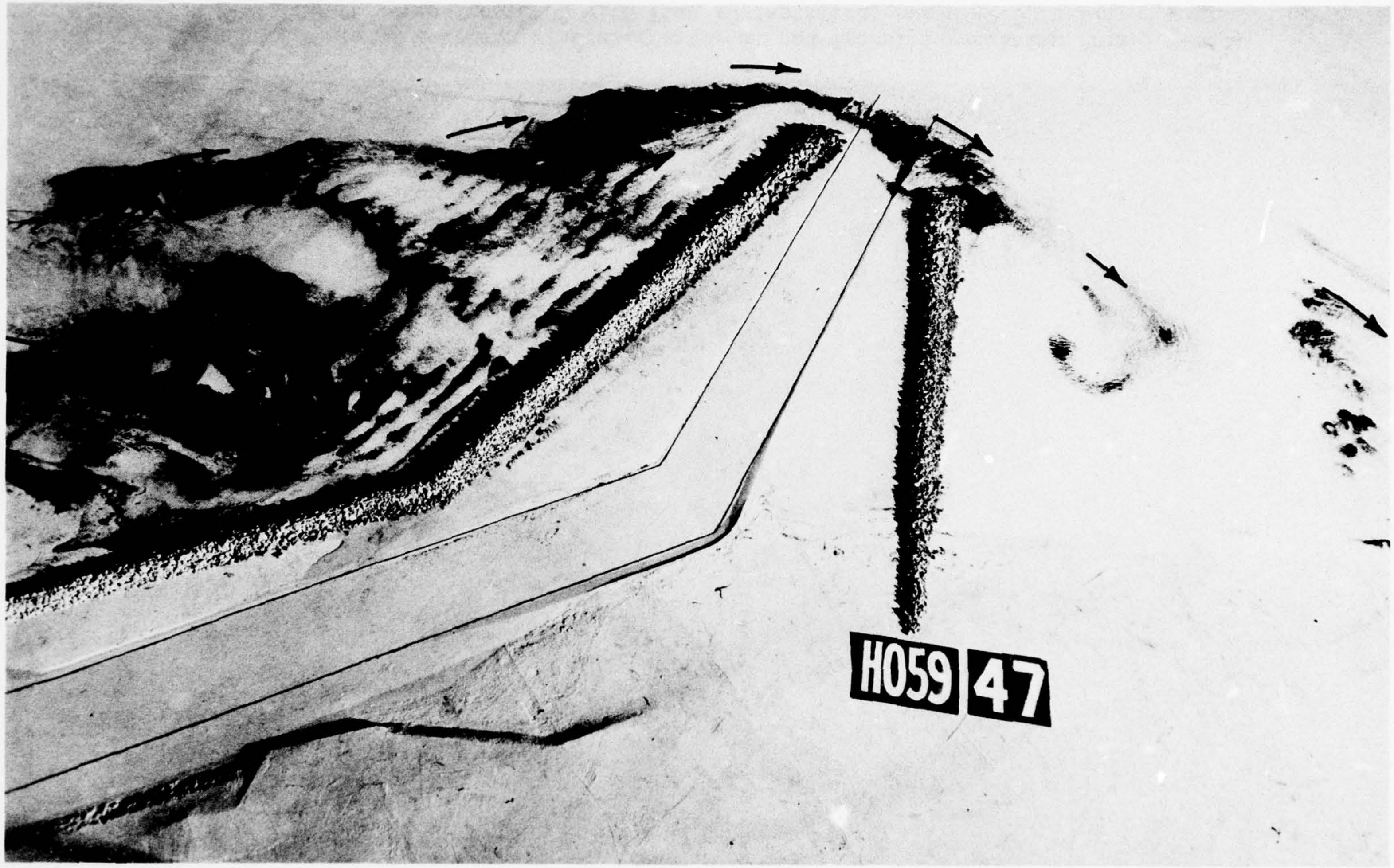

Photo 37. General movement of tracer material and deposits resulting from 5.2-sec, 6-ft waves from 240° with Plan 9 installed; swl = +2.0 ft lwd

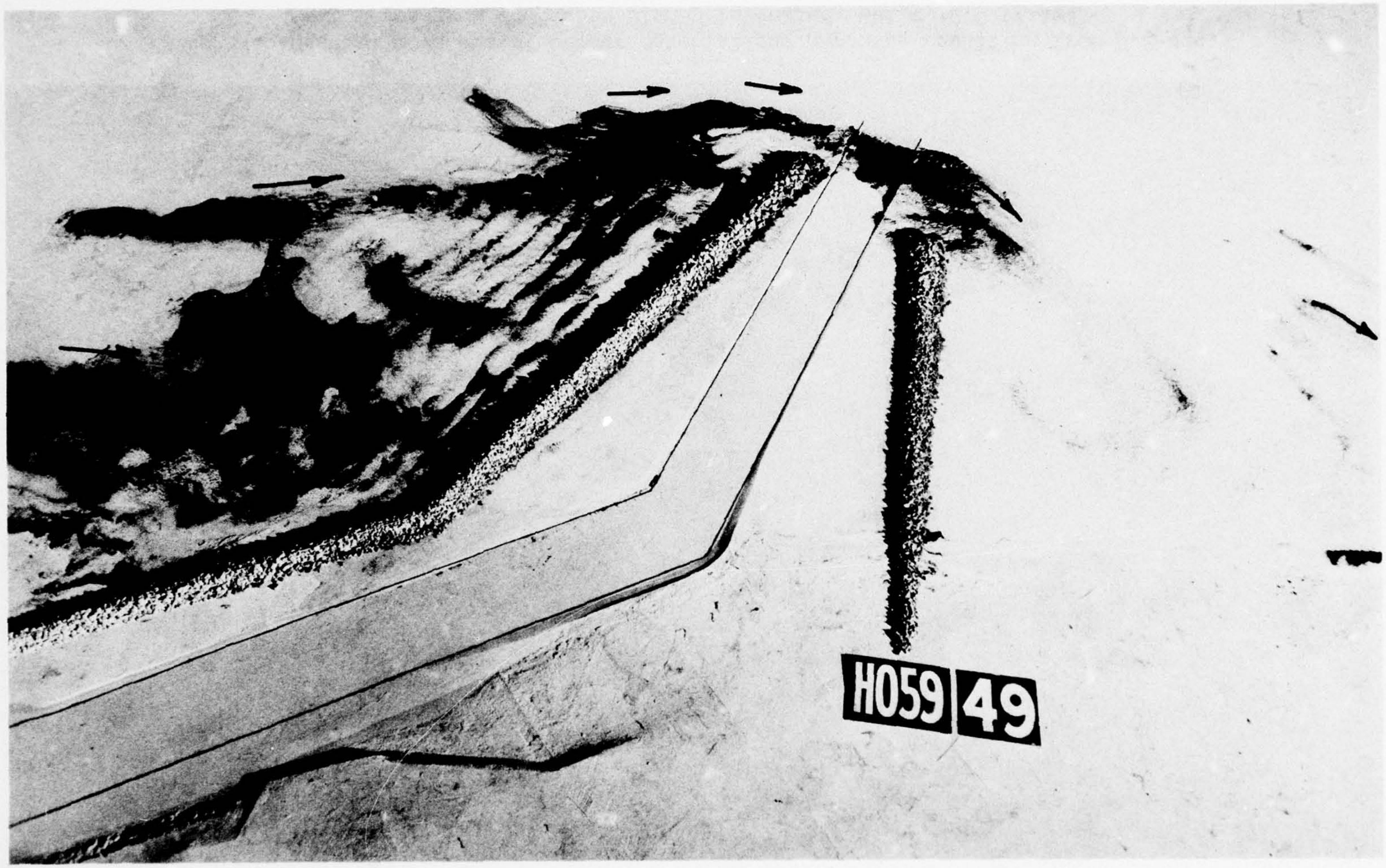

Photo 38. General movement of tracer material and deposits resulting from 5.2-sec, 6-ft waves from 240° with Plan 10 installed; swl = +2.0 ft lwd

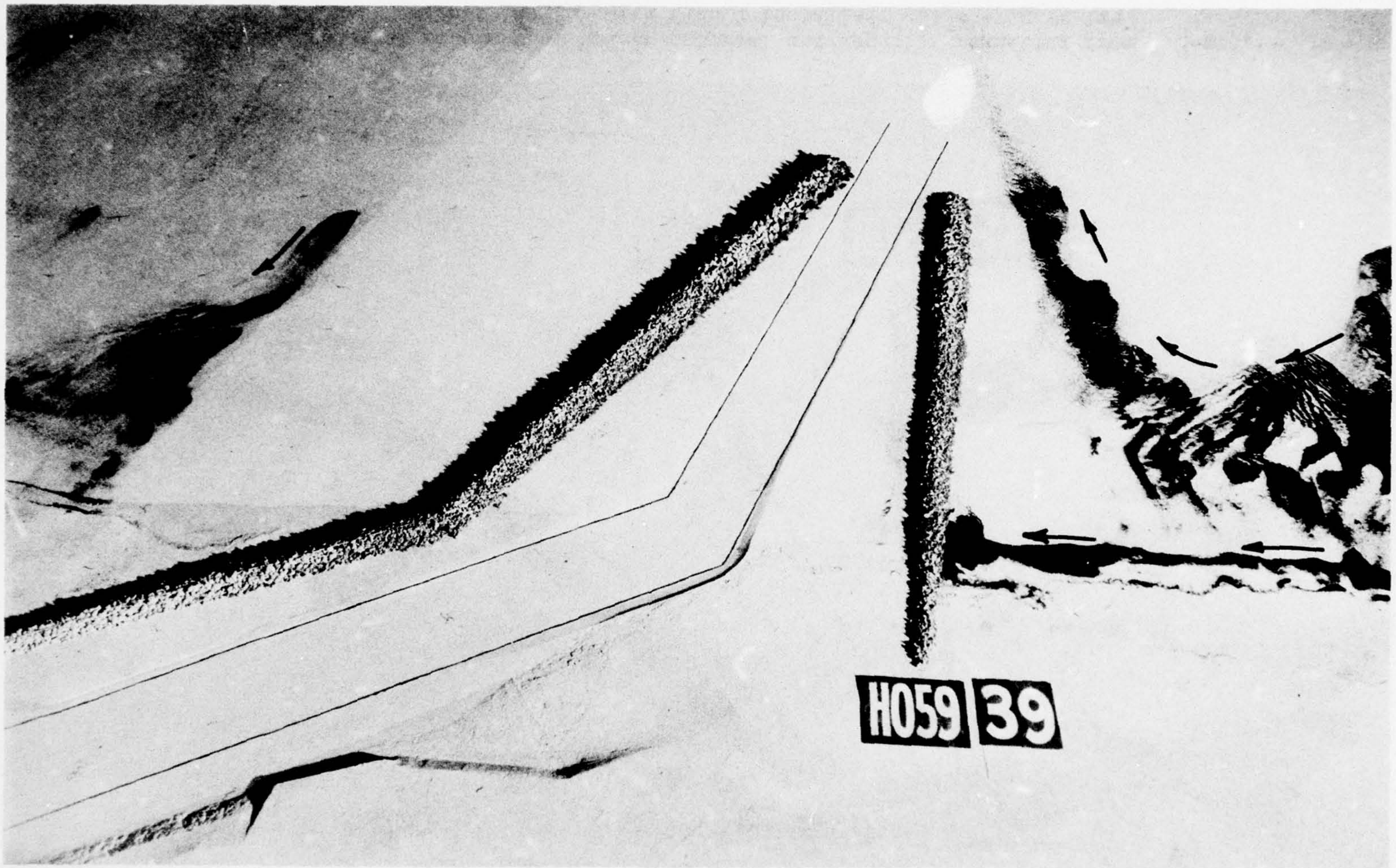

Photo 39. General movement of tracer material and deposits resulting from 5.8-sec, 4.7-ft waves from 330° with Plan 7 installed; swl = +2.0 ft lwd

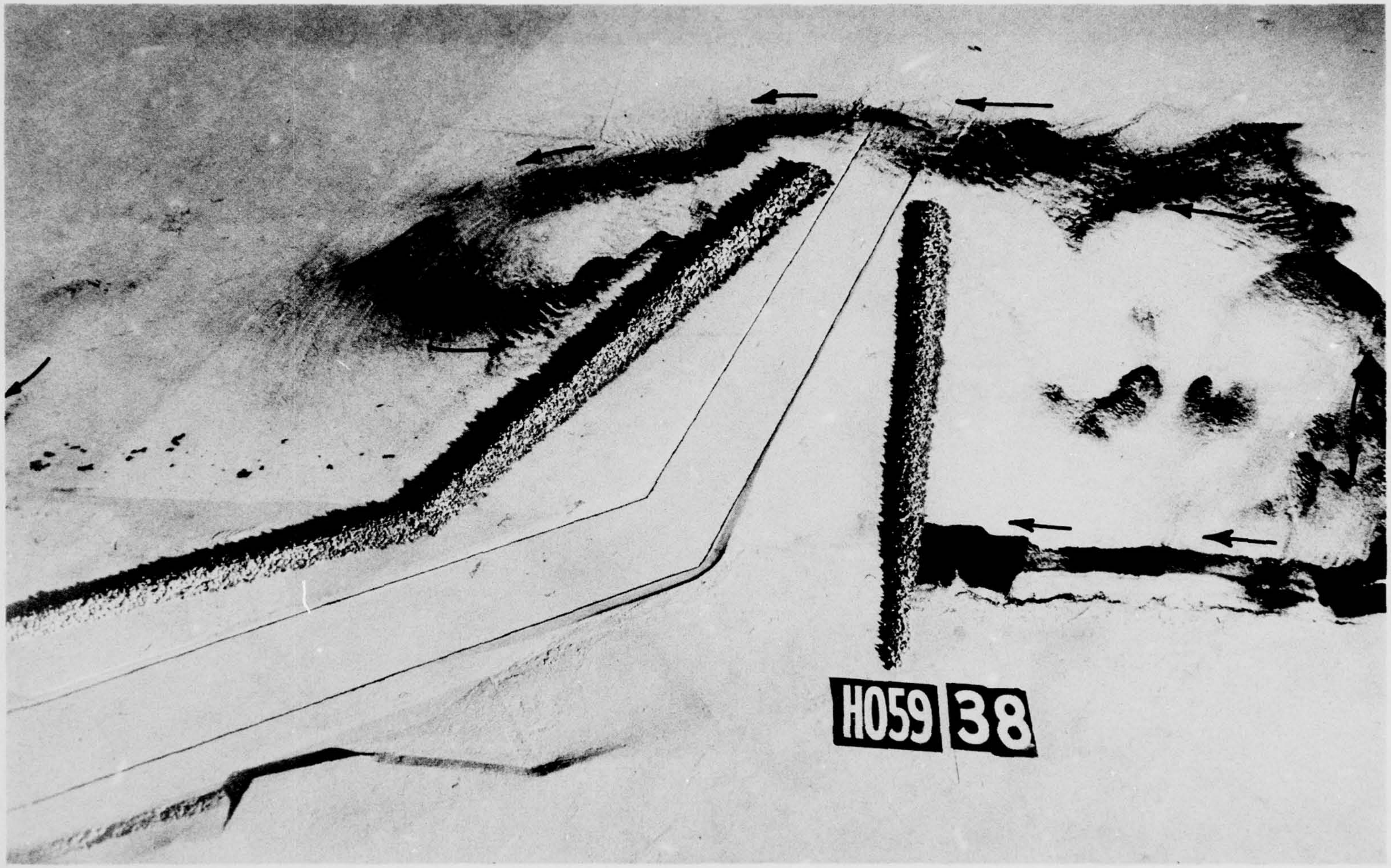

Photo 40. General movement of tracer material and deposits resulting from 7.3-sec, 8.3-ft waves from 330° with Plan 7 installed; swl = +2.0 ft lwd

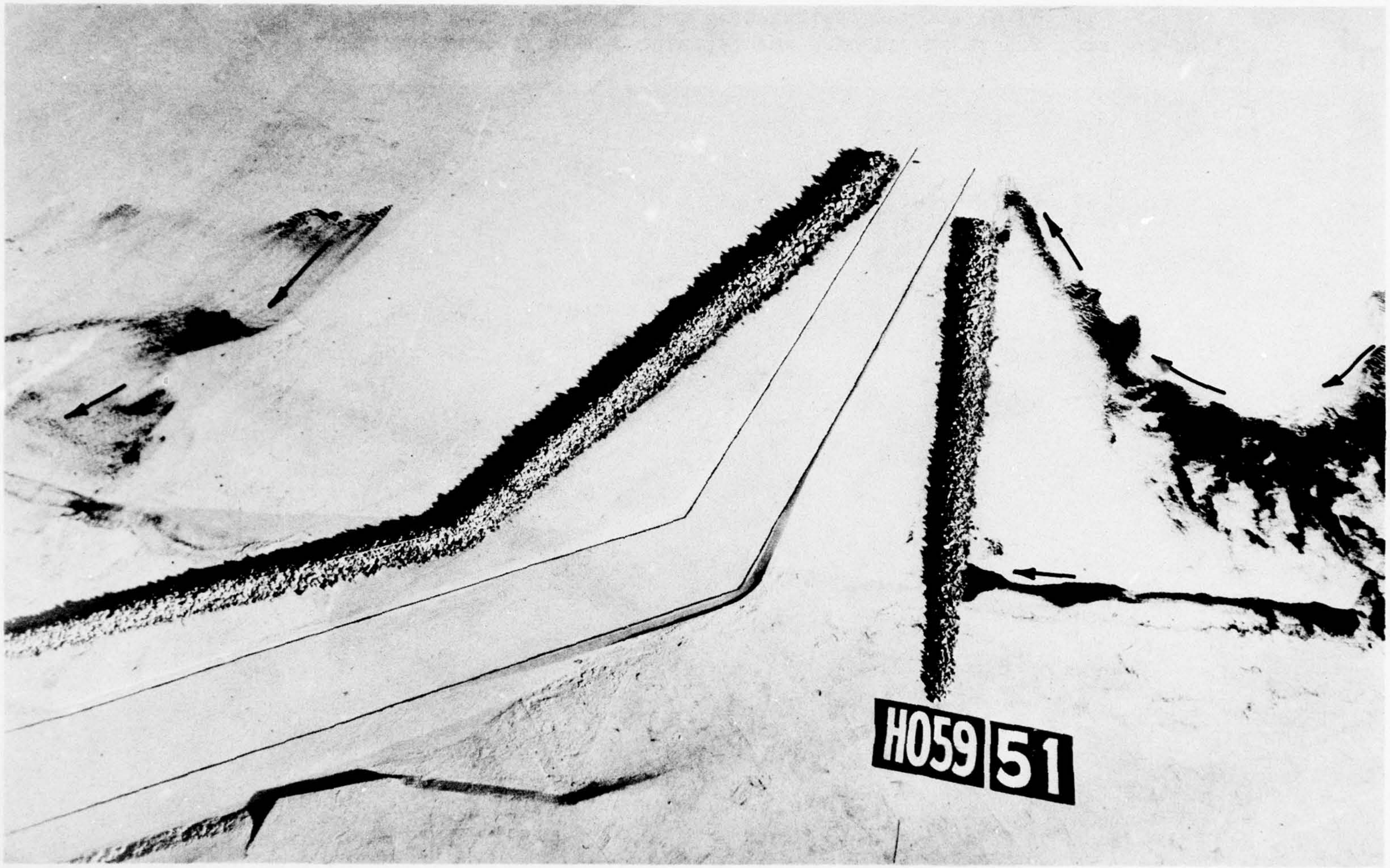

Photo 41. General movement of tracer material and deposits resulting from 5.8-sec, 4.7-ft waves from 330° with Plan 8 installed; swl = +2.0 ft lwd

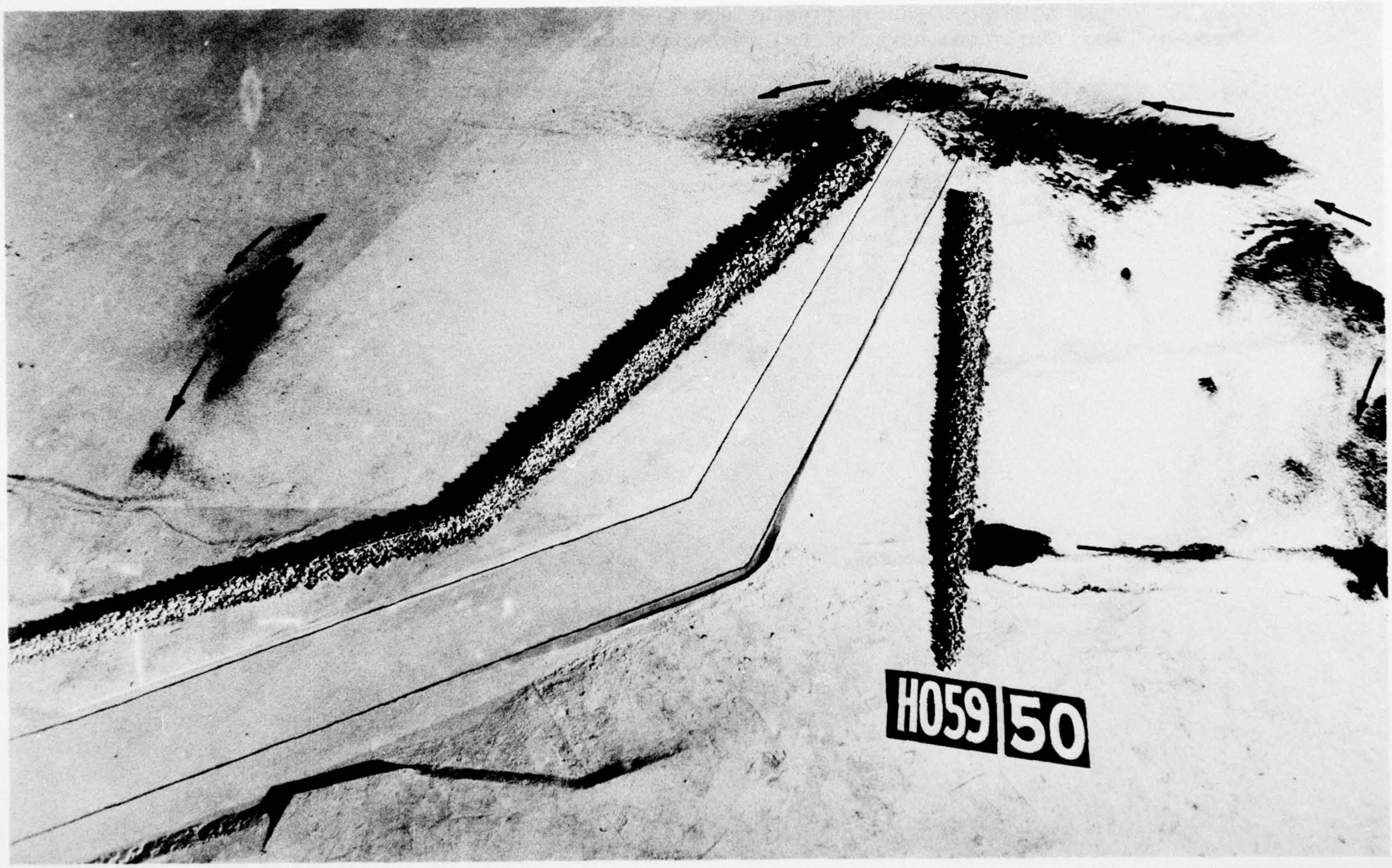

Photo 42. General movement of tracer material and deposits resulting from 7.3-sec, 8.3-ft waves from 330° with Plan 8 installed; swl = +2.0 ft lwd

AD-A049 246 ARMY ENGINEER WATERWAYS EXPERIMENT STATION VICKSBURG MISS F/G 8/6
PORT ONTARIO HARBOR, NEW YORK, DESIGN FOR WAVE PROTECTION AND P--ETC(U)
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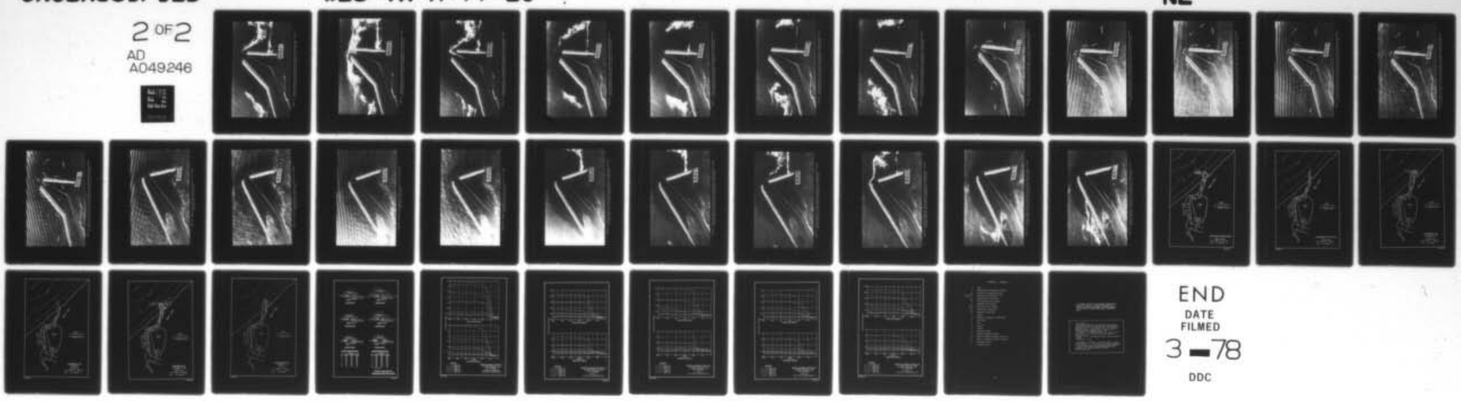

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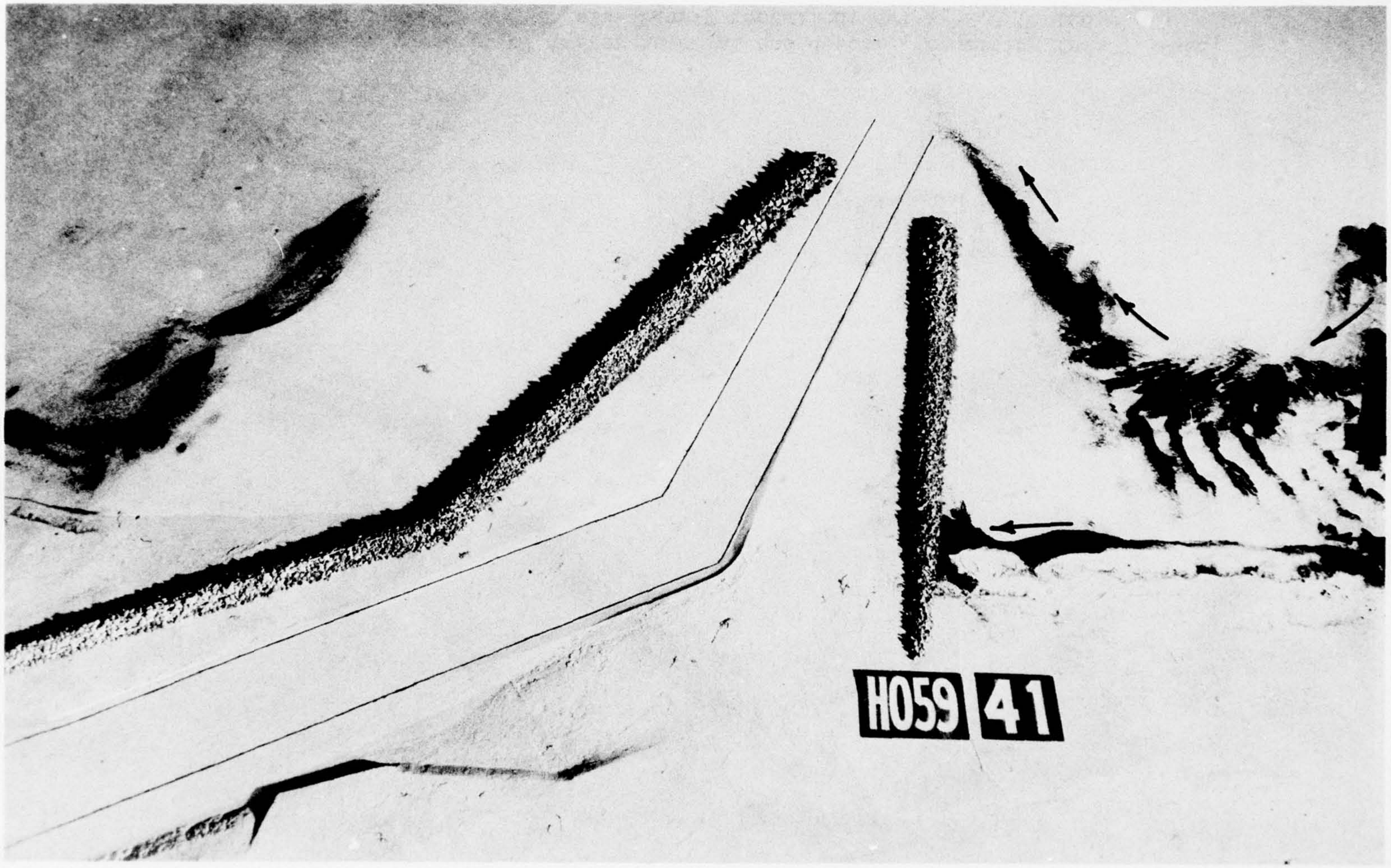


Photo 43. General movement of tracer material and deposits resulting from 5.8-sec, 4.7-ft waves from 330° with Plan 9 installed; swl = +2.0 ft lwd

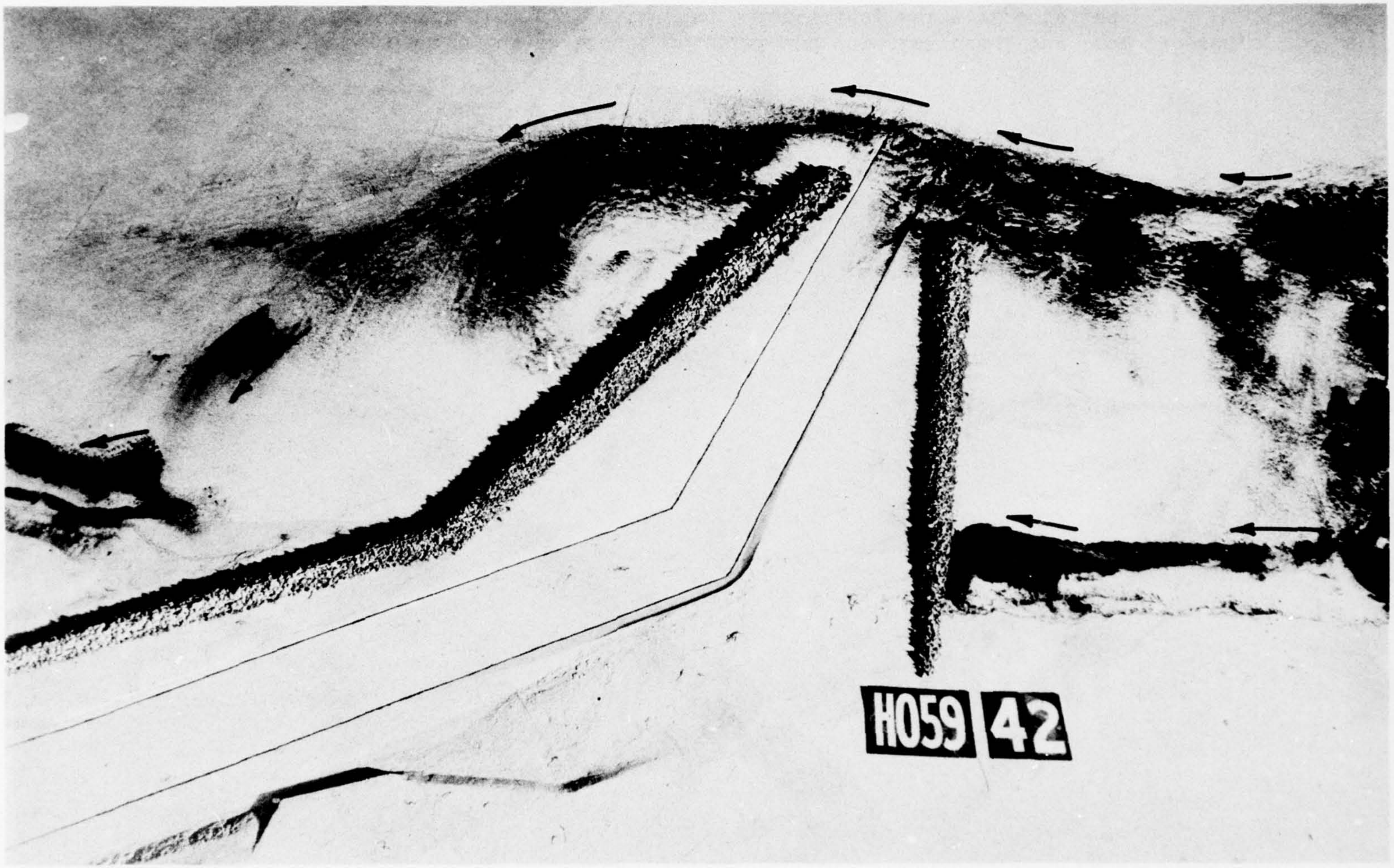


Photo 44. General movement of tracer material and deposits resulting from 7.3-sec, 8.3-ft waves from 330° with Plan 9 installed; swl = +2.0 ft lwd

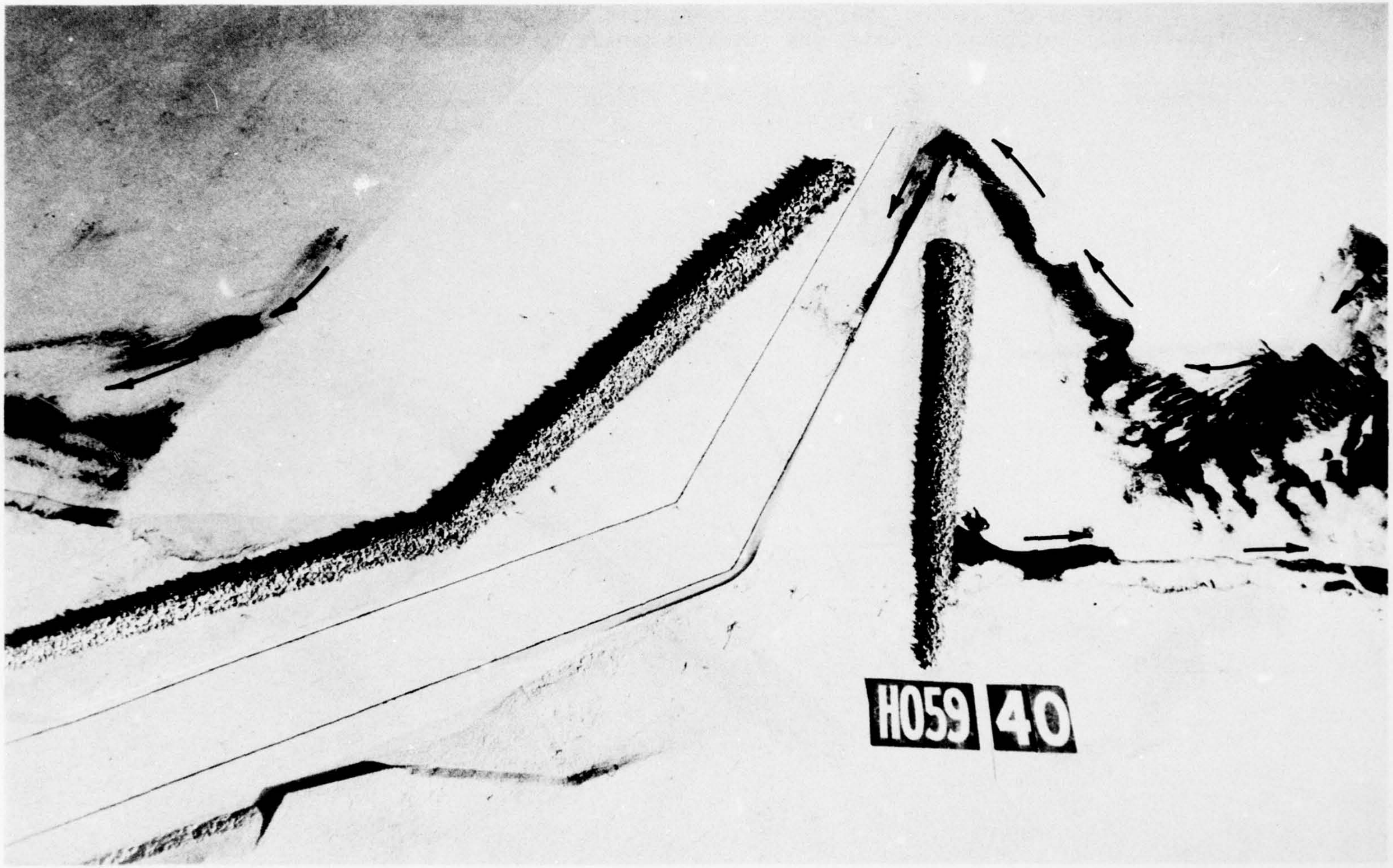


Photo 45. General movement of tracer material and deposits resulting from 5.8-sec, 4.7-ft waves from 330° with Plan 10 installed; swl = +2.0 ft lwd

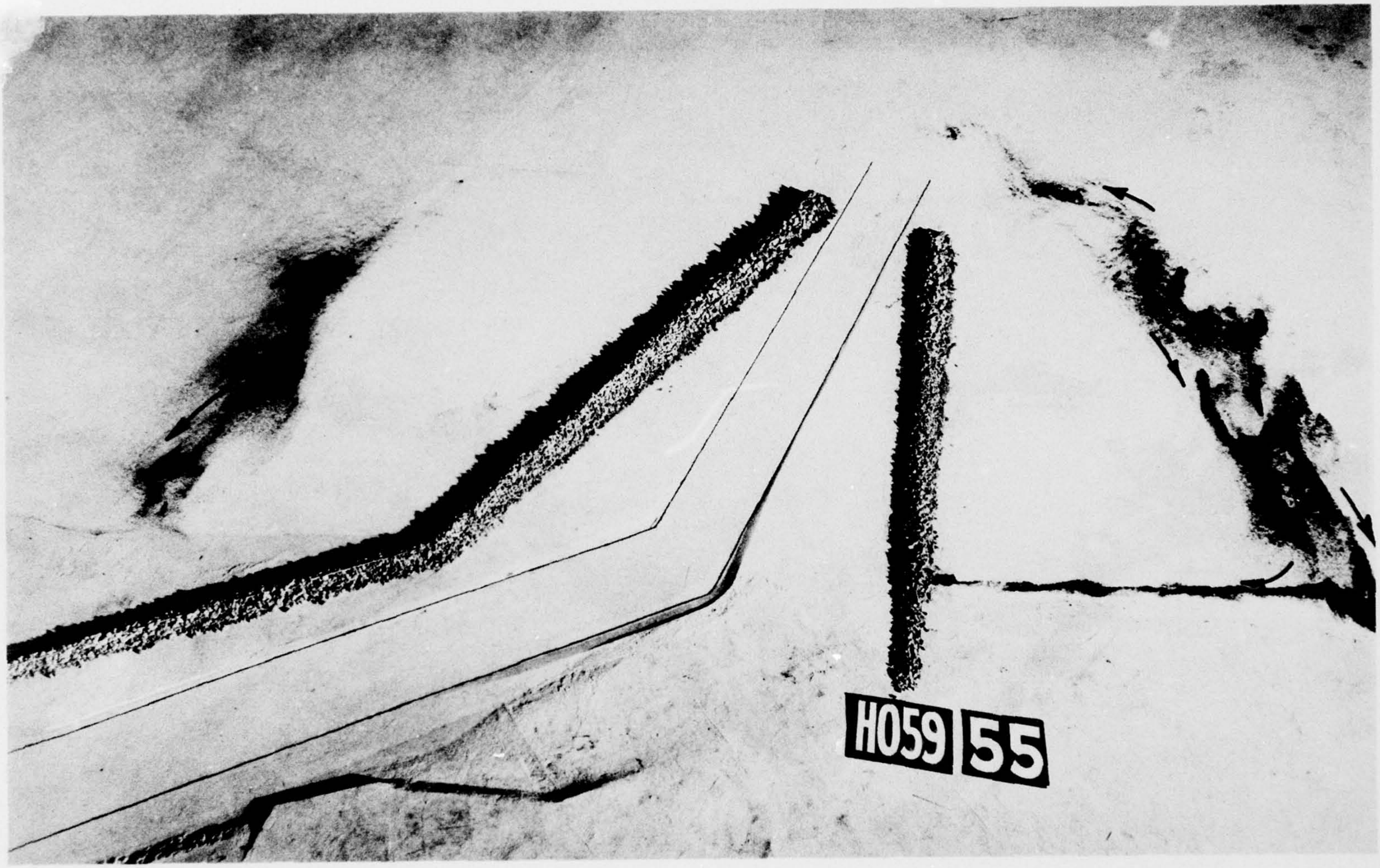


Photo 46. General movement of tracer material and deposits resulting from 6-sec, 6.1-ft waves from 300° with Plan 7 installed; swl = +2.0 ft lwd

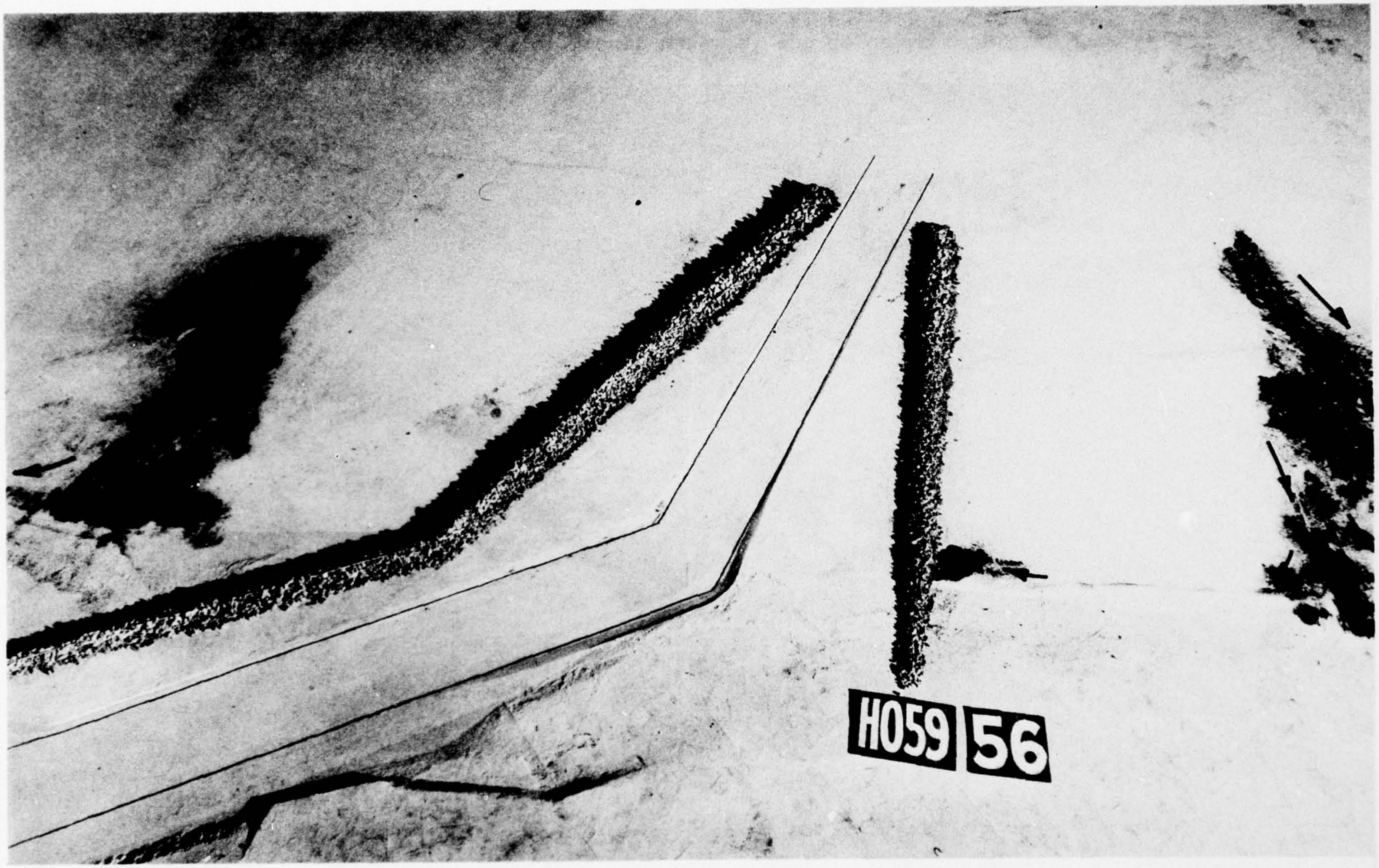


Photo 47. General movement of tracer material and deposits resulting from 7.8-sec, 11.1-ft waves from 300° with Plan 7 installed; swl = +2.0 ft lwd

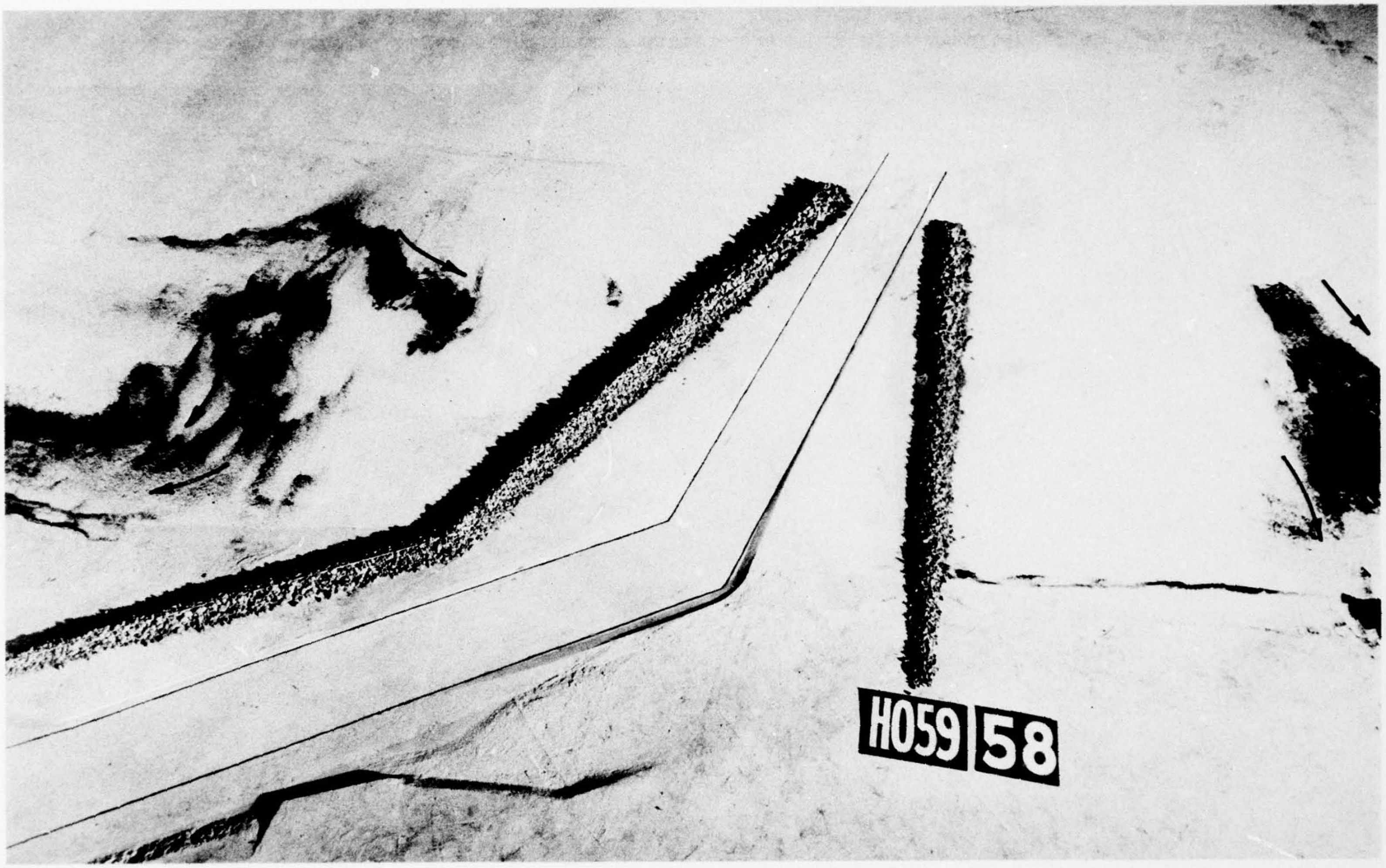


Photo 48. General movement of tracer material and deposits resulting from 6-sec, 6.1-ft waves from 270° with Plan 7 installed; swl = +2.0 ft lwd

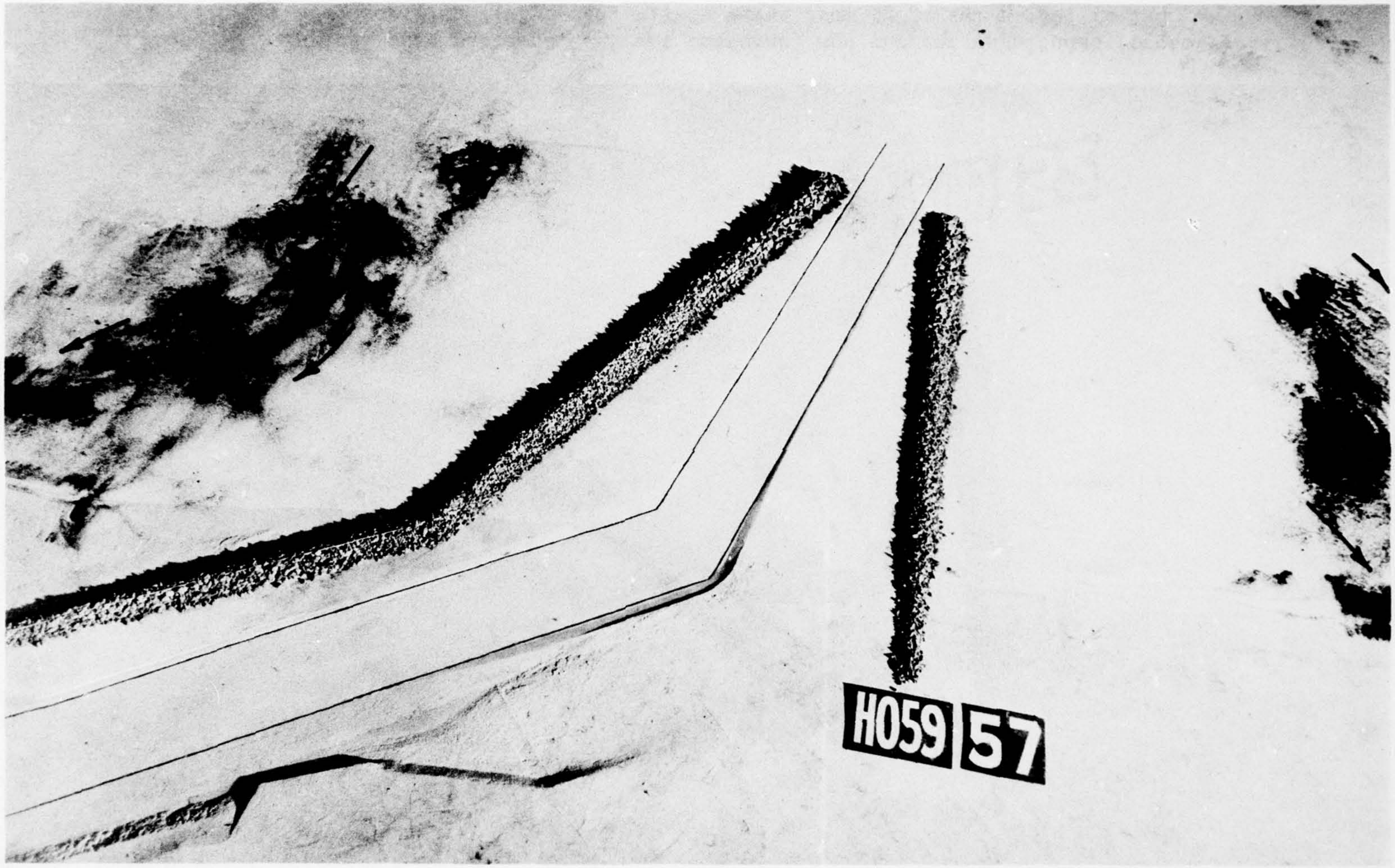


Photo 49. General movement of tracer material and deposits resulting from 7.8-sec, 11.1-ft waves from 270° with Plan 7 installed; swl = +2.0 ft lwd

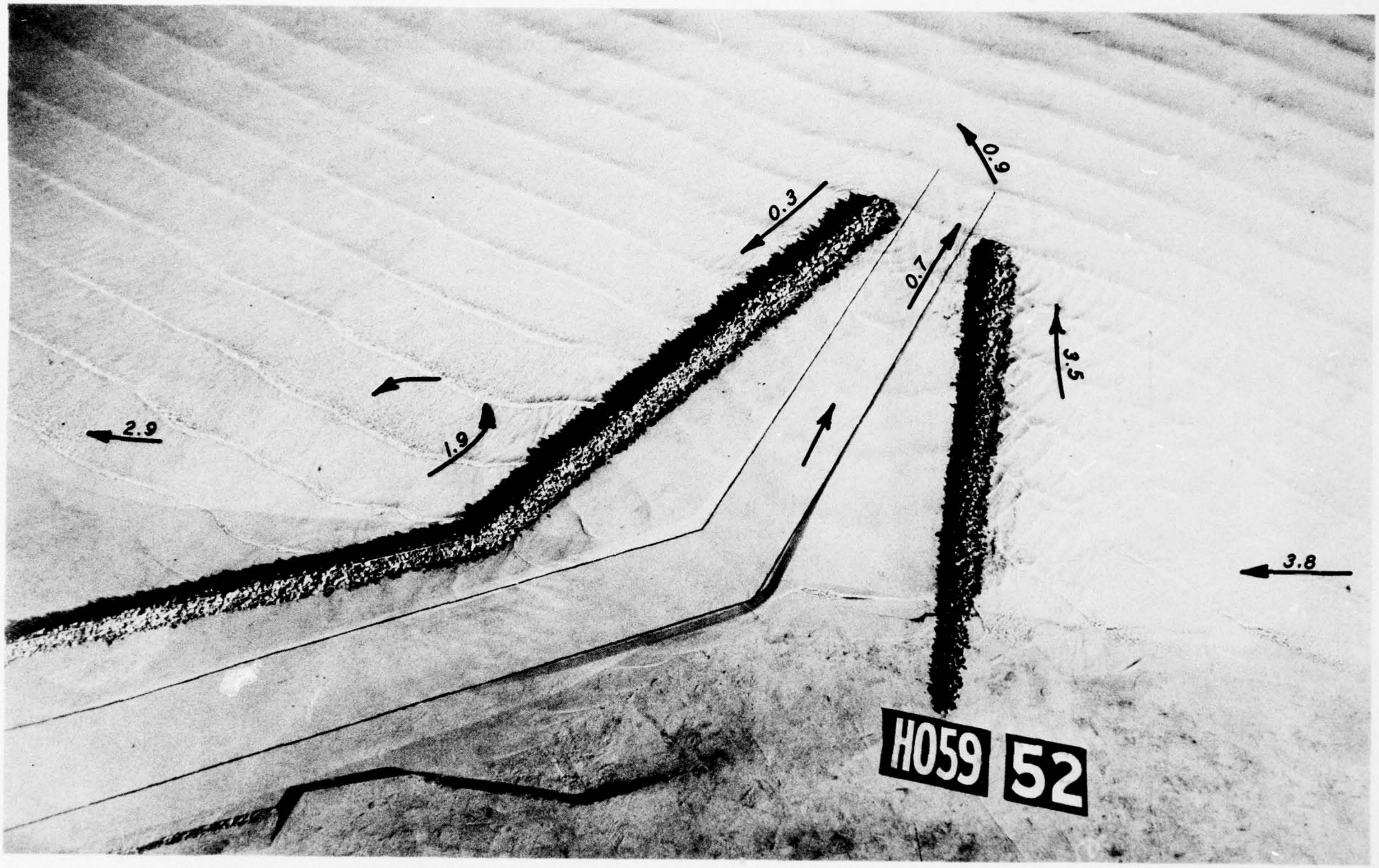


Photo 50. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 7; 5.8-sec, 4.7-ft waves from 330°; swl = +4.7 ft lwd

Photo 51. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 7; 6-sec, 6.1-ft waves from 300°; swl = +4.7 ft lwd

Photo 52. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 7; 7.8-sec, 11.1-ft waves from 300°; swl = +4.7 ft lwd

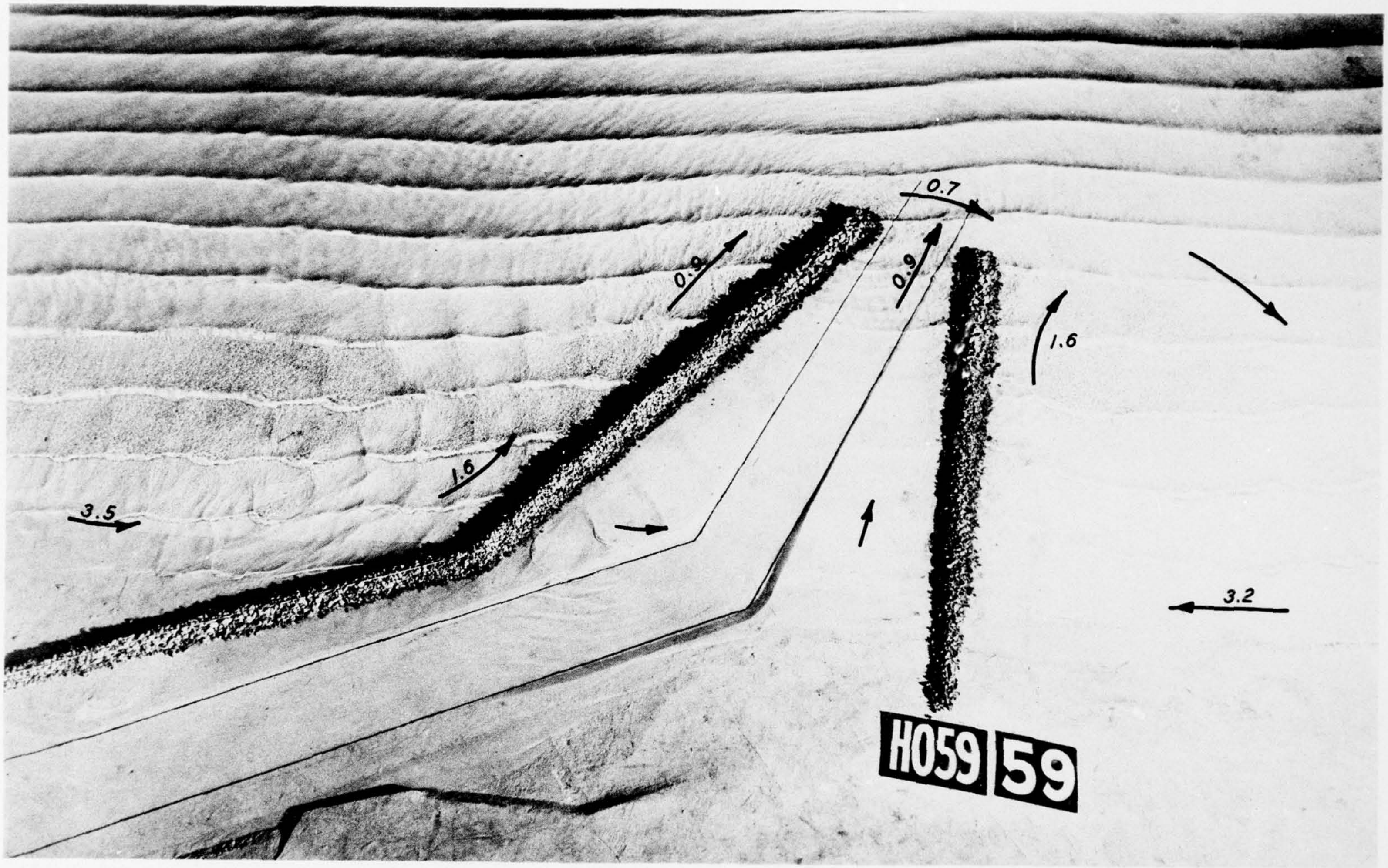


Photo 53. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 7; 6-sec, 6.1-ft waves from 270°; swl = +4.7 ft lwd

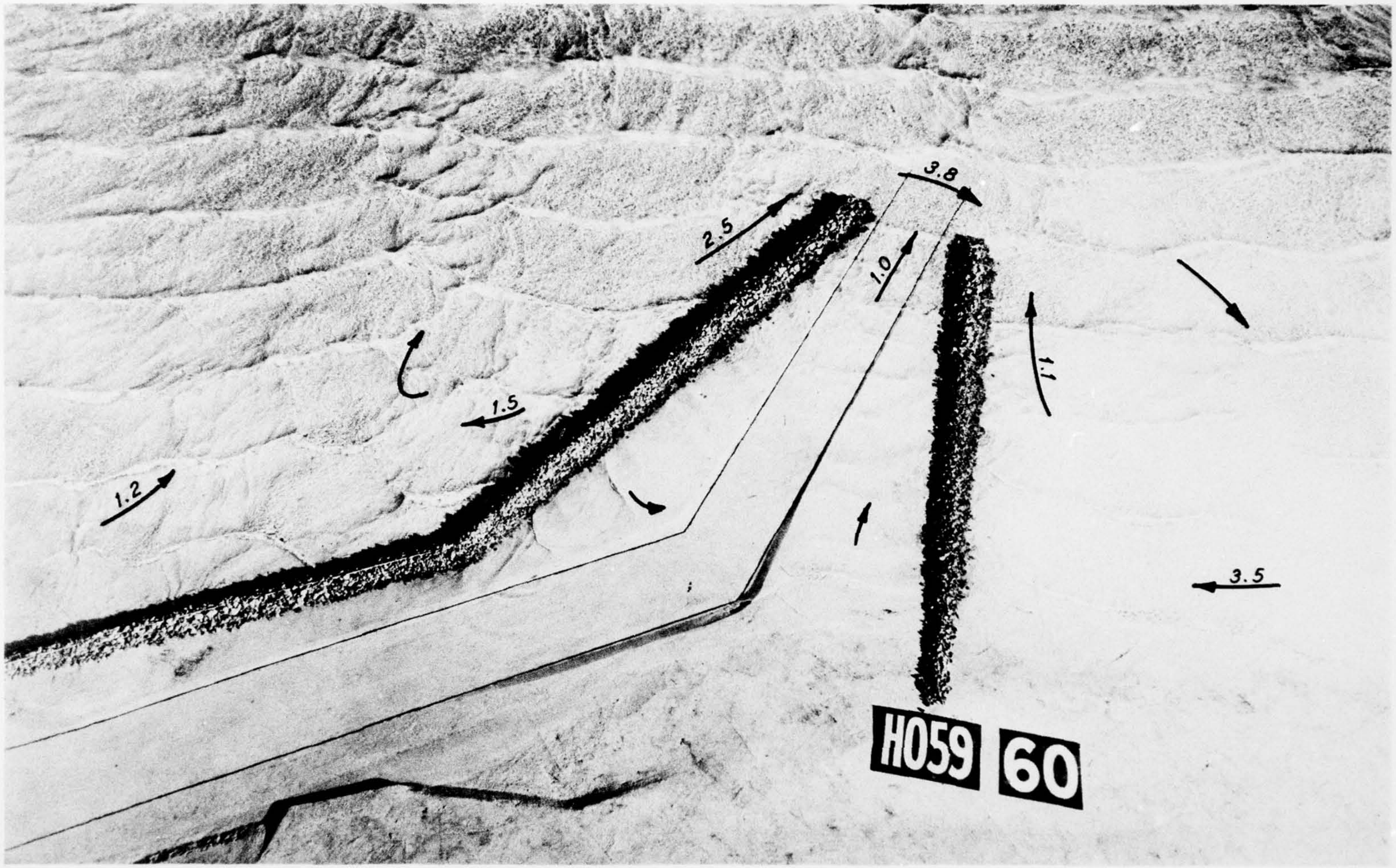


Photo 54. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 7; 7.8-sec, 11.1-ft waves from 270°; swl = +4.7 ft lwd

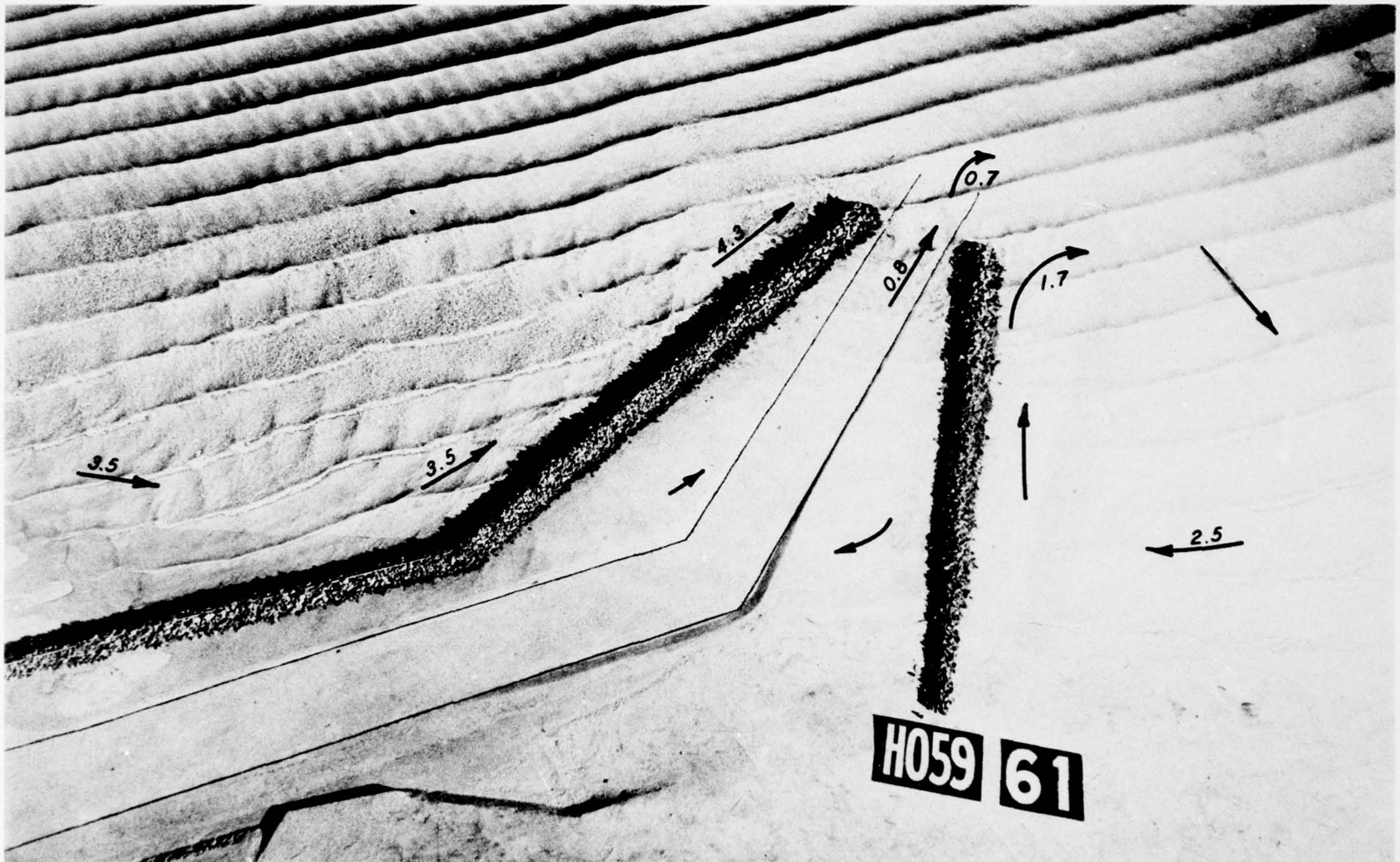


Photo 55. Typical wave patterns, current patterns, and current magnitudes (prototype ft per sec) for Plan 7; 5.2-sec, 6-ft waves from 240°; swl = +4.7 ft lwd

Photo 56. Typical wave patterns for Plan 11; 6-sec, 6.1-ft waves from 300°; swl = +4.7 ft lwd

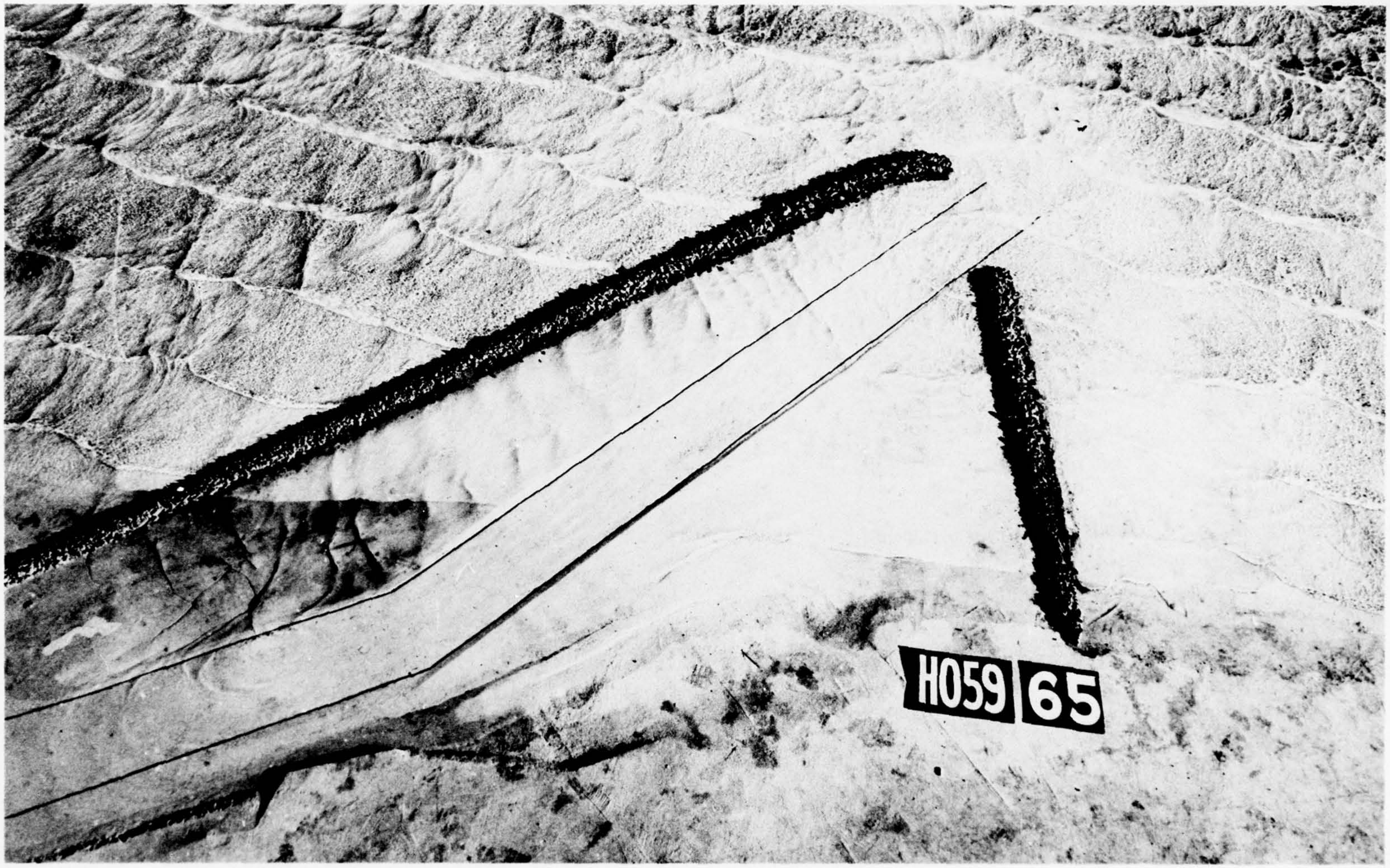


Photo 57. Typical wave patterns for Plan 11; 7.8-sec, 11.1-ft waves from 300°; swl = +4.7 ft lwd

Photo 58. Typical wave patterns for Plan 11; 6-sec, 6.1-ft waves from 270°; swl = +4.7 ft lwd

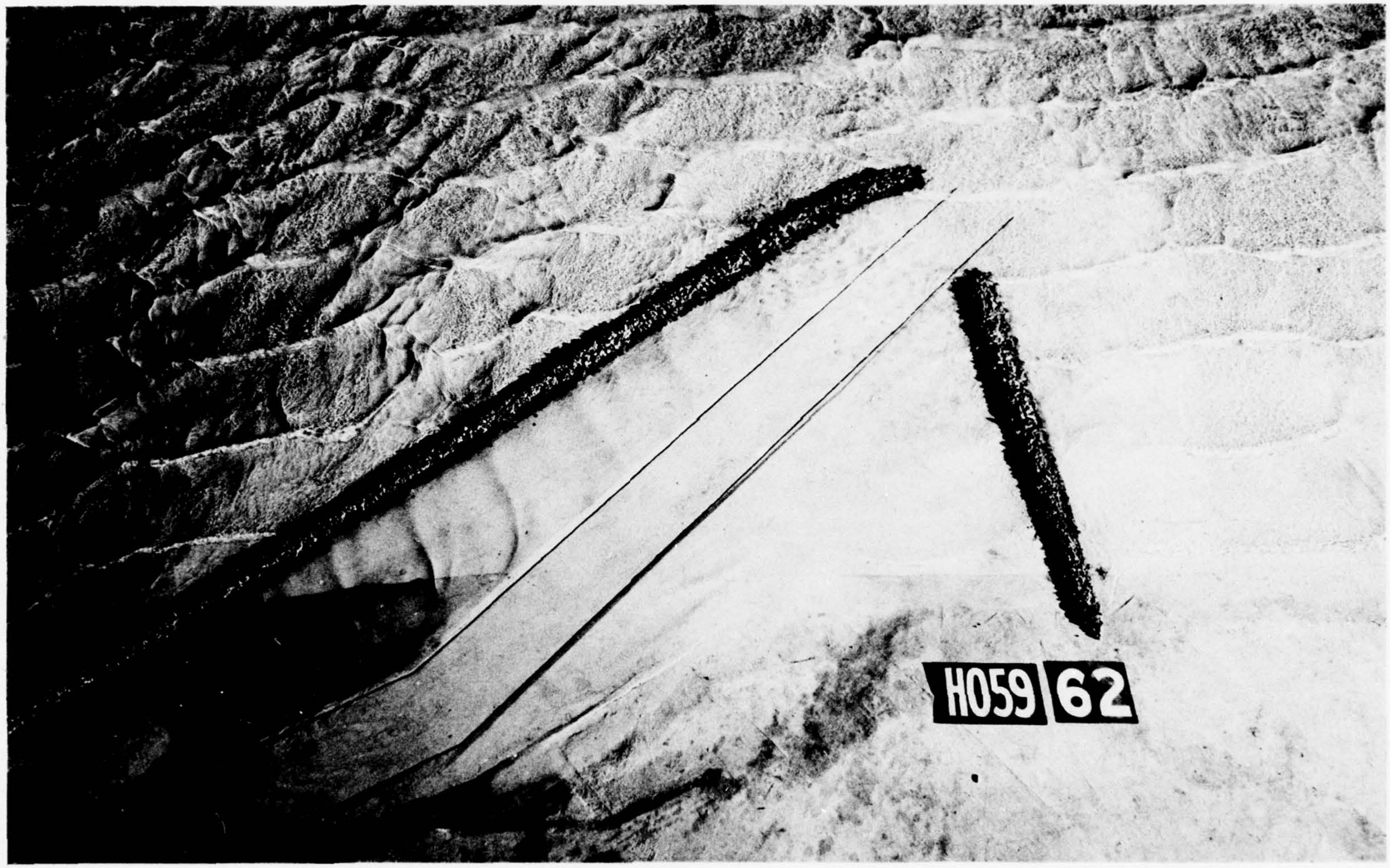


Photo 59. Typical wave patterns for Plan 11; 7.8-sec, 11.1-ft waves from 270°; swl = +4.7 ft lwd

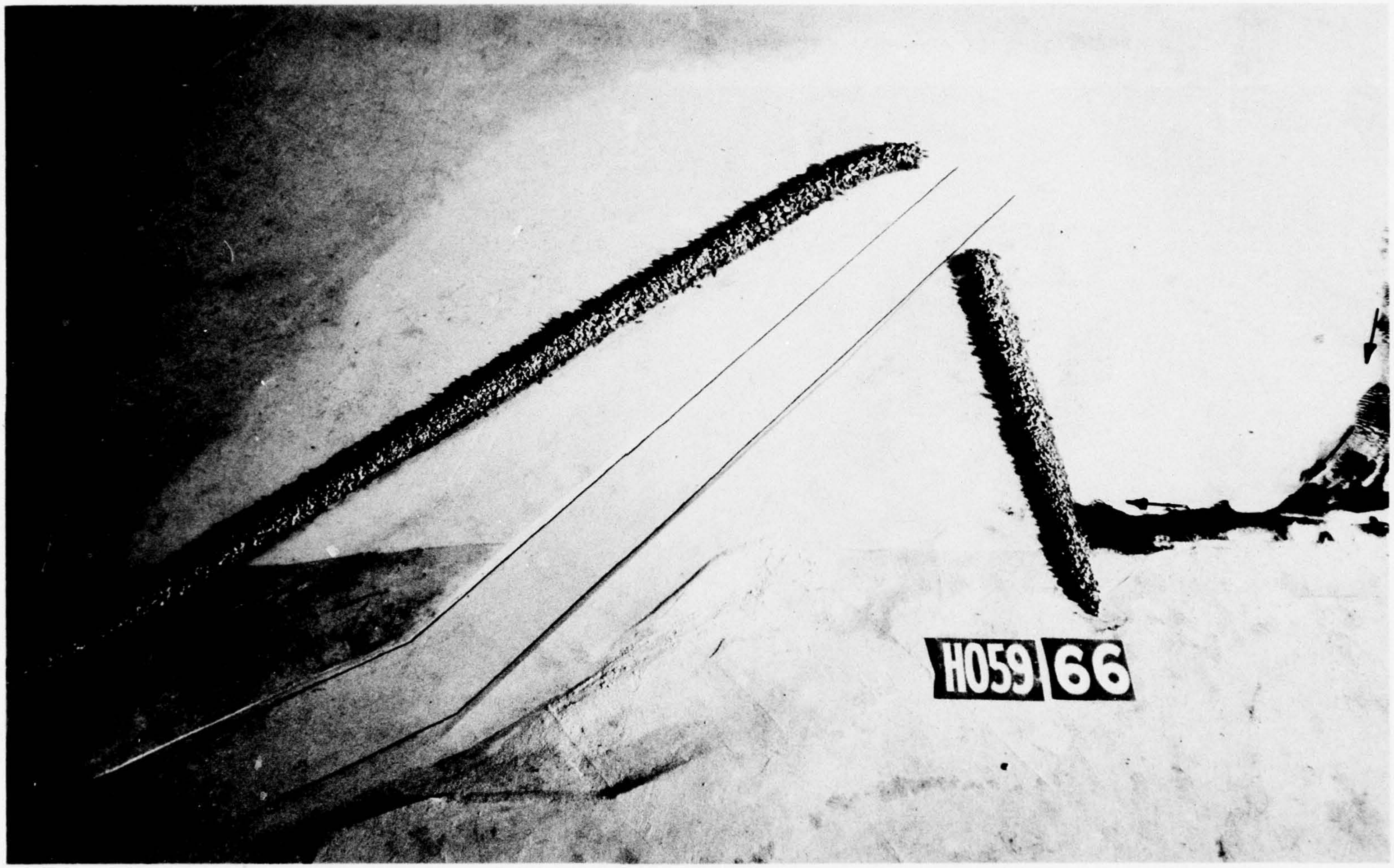


Photo 60. General movement of tracer material and deposits resulting from 5.8-sec, 3-ft waves from 330° with Plan 11 installed; swl = +2.0 ft lwd

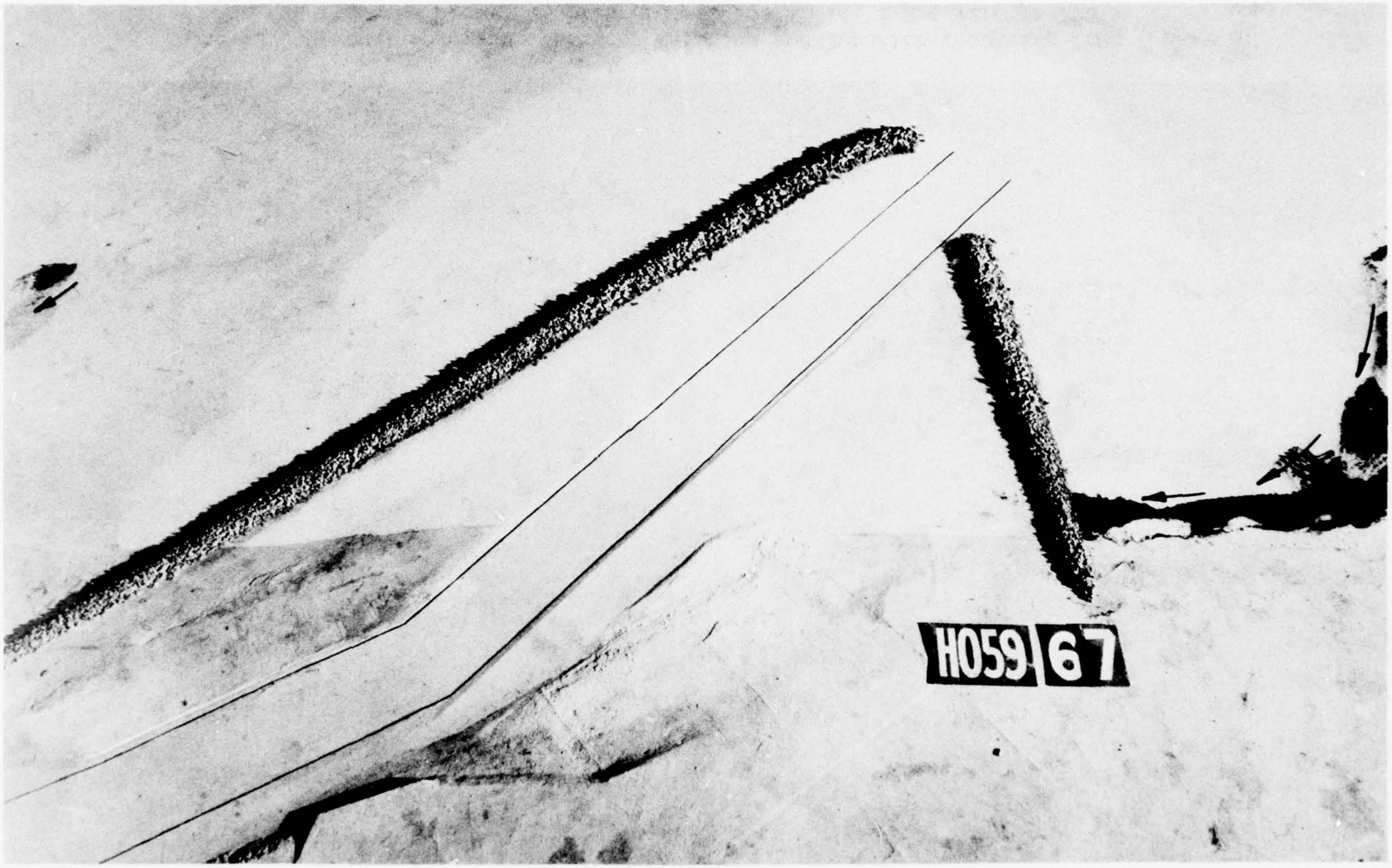


Photo 61. General movement of tracer material and deposits resulting from 7.3-sec, 2-ft waves from 330° with Plan 11 installed; swl = +2.0 ft lwd

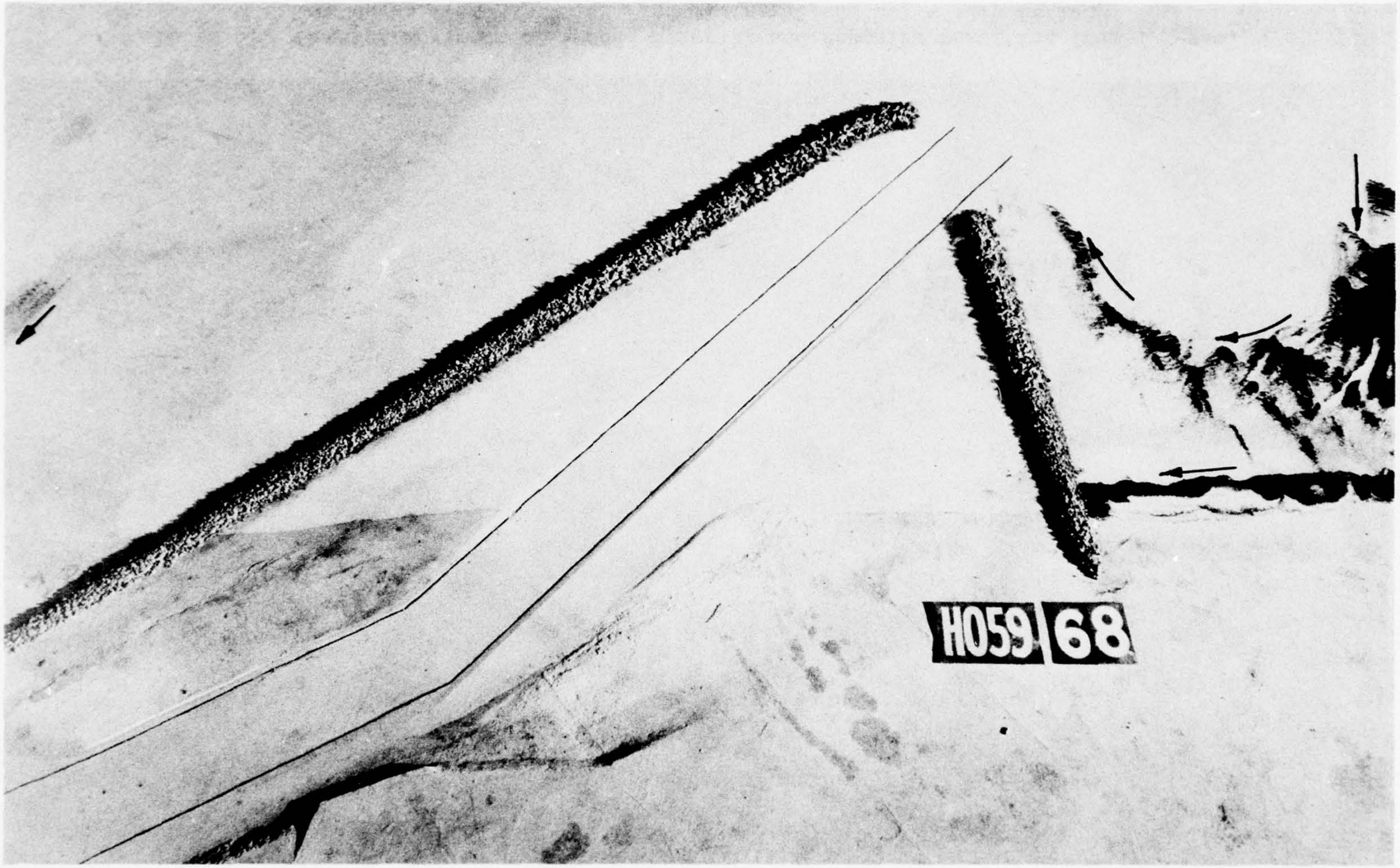


Photo 62. General movement of tracer material and deposits resulting from 7.3-sec, 4-ft waves from 330° with Plan 11 installed; swl = +2.0 ft lwd

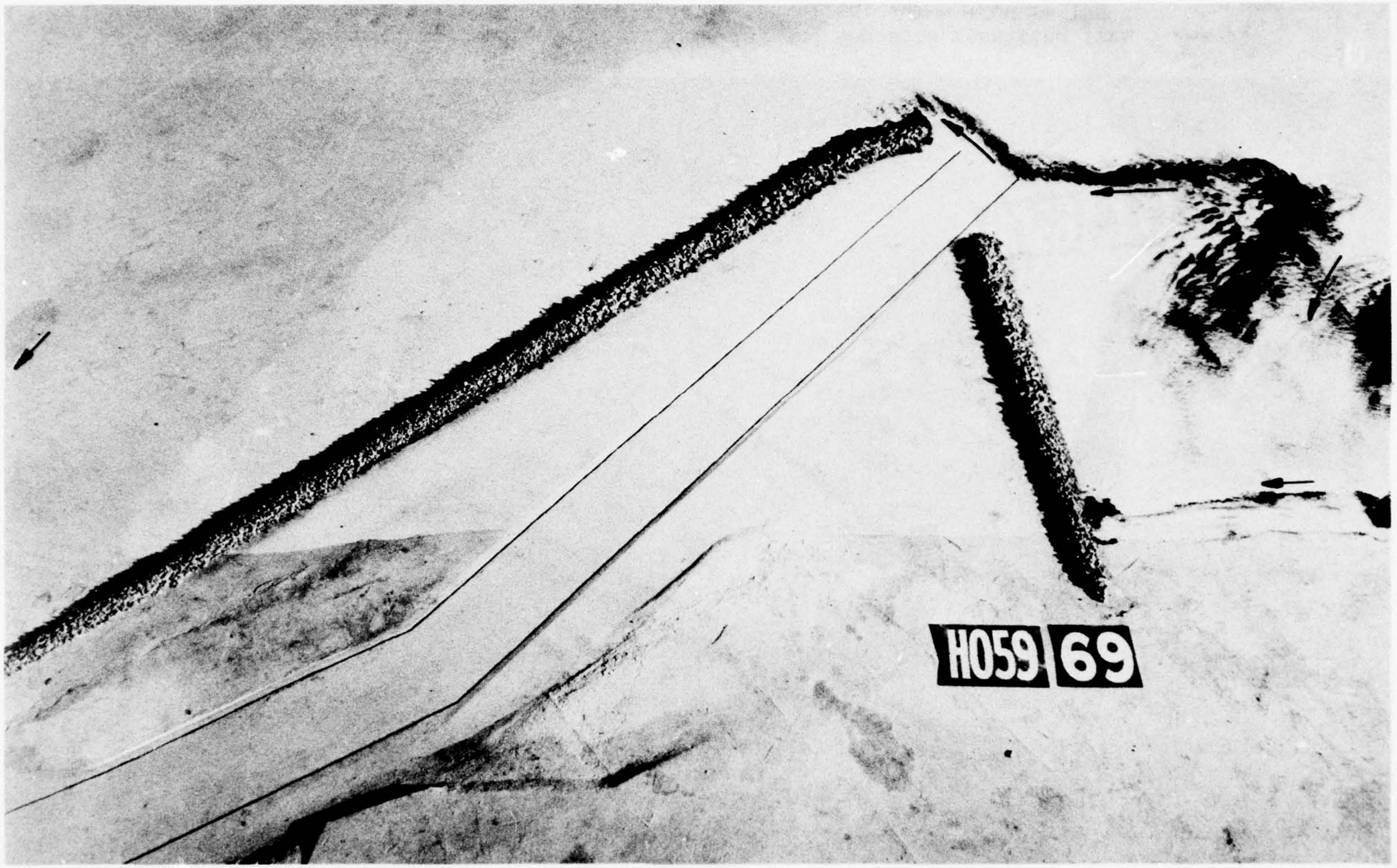


Photo 63. General movement of tracer material and deposits resulting from 7.3-sec, 6-ft waves from 330° with Plan 11 installed; swl = +2.0 ft lwd

Photo 64. General movement of tracer material and deposits resulting from 5.2-sec, 2-ft waves from 240° with Plan 11 installed; swl = +2.0 ft lwd

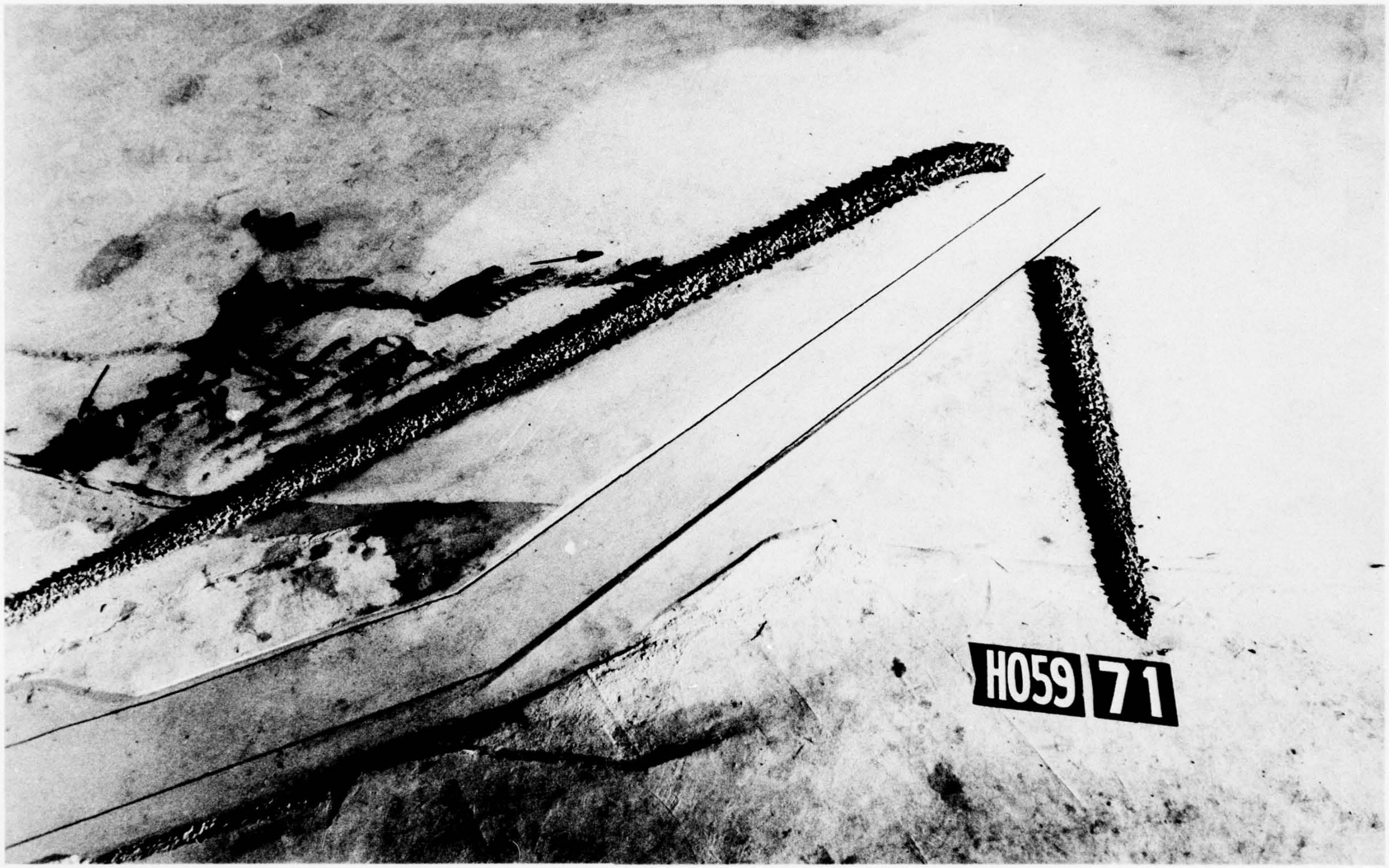


Photo 65. General movement of tracer material and deposits resulting from 5.2-sec, 4-ft waves from 240° with Plan 11 installed; swl = +2.0 ft lwd

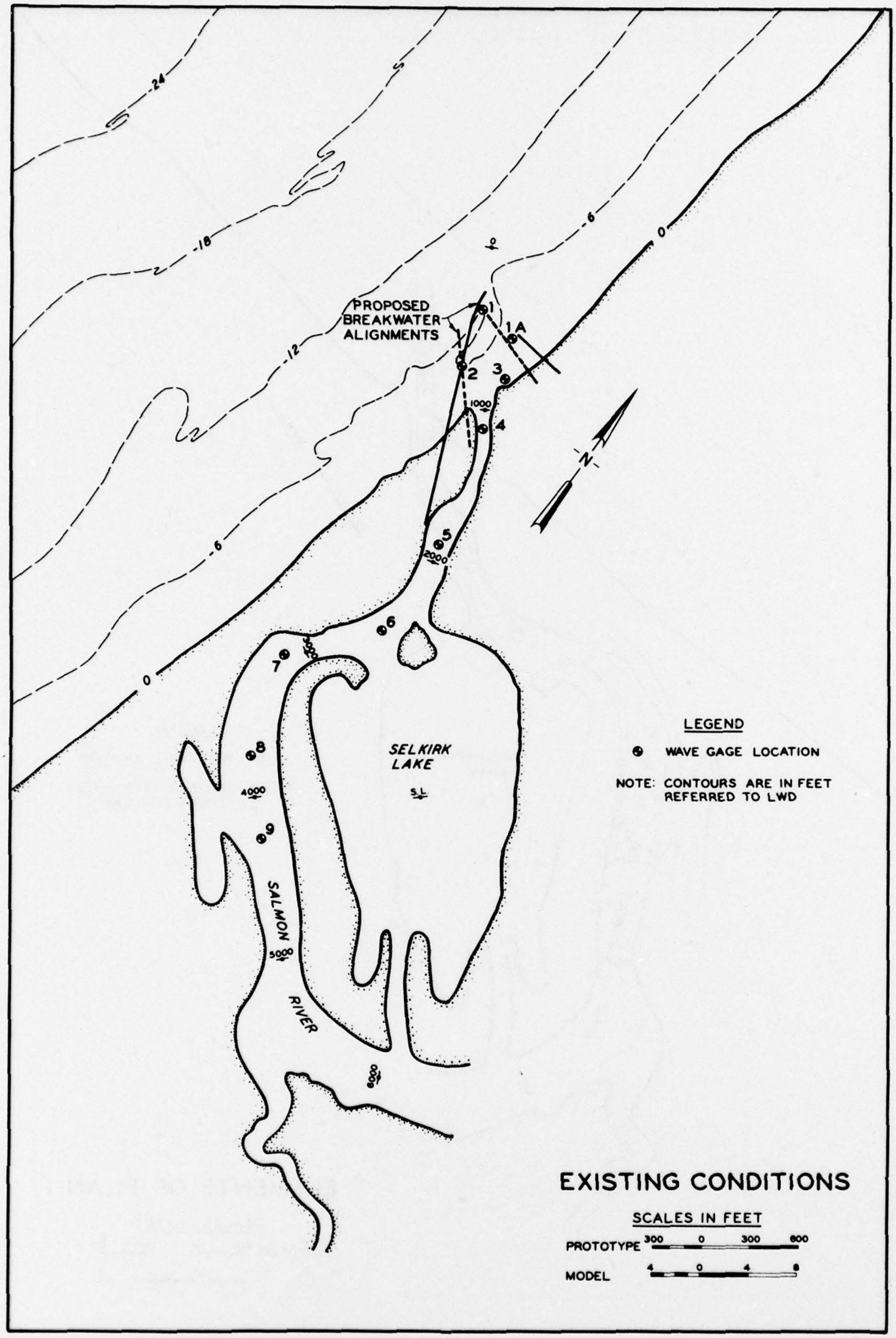
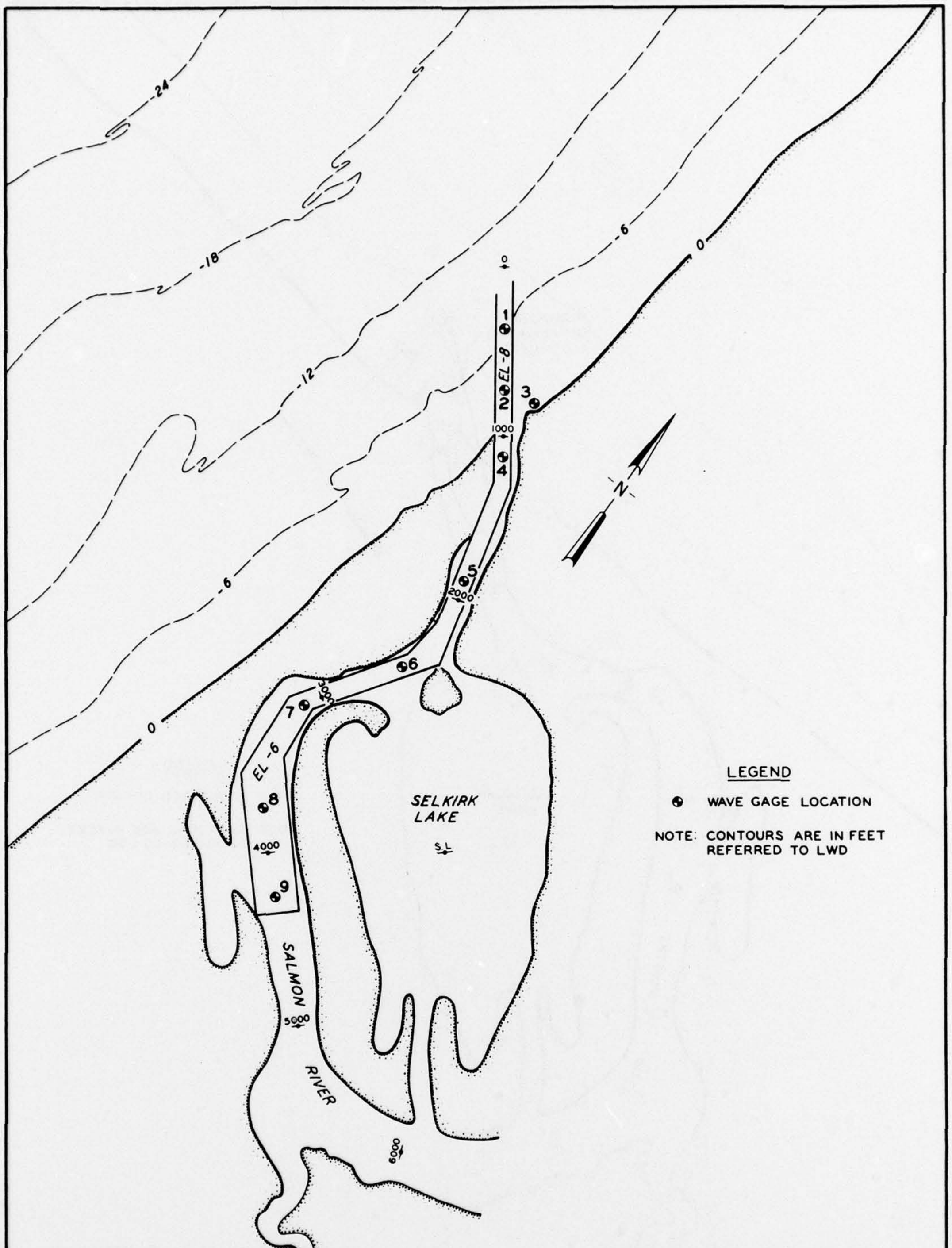
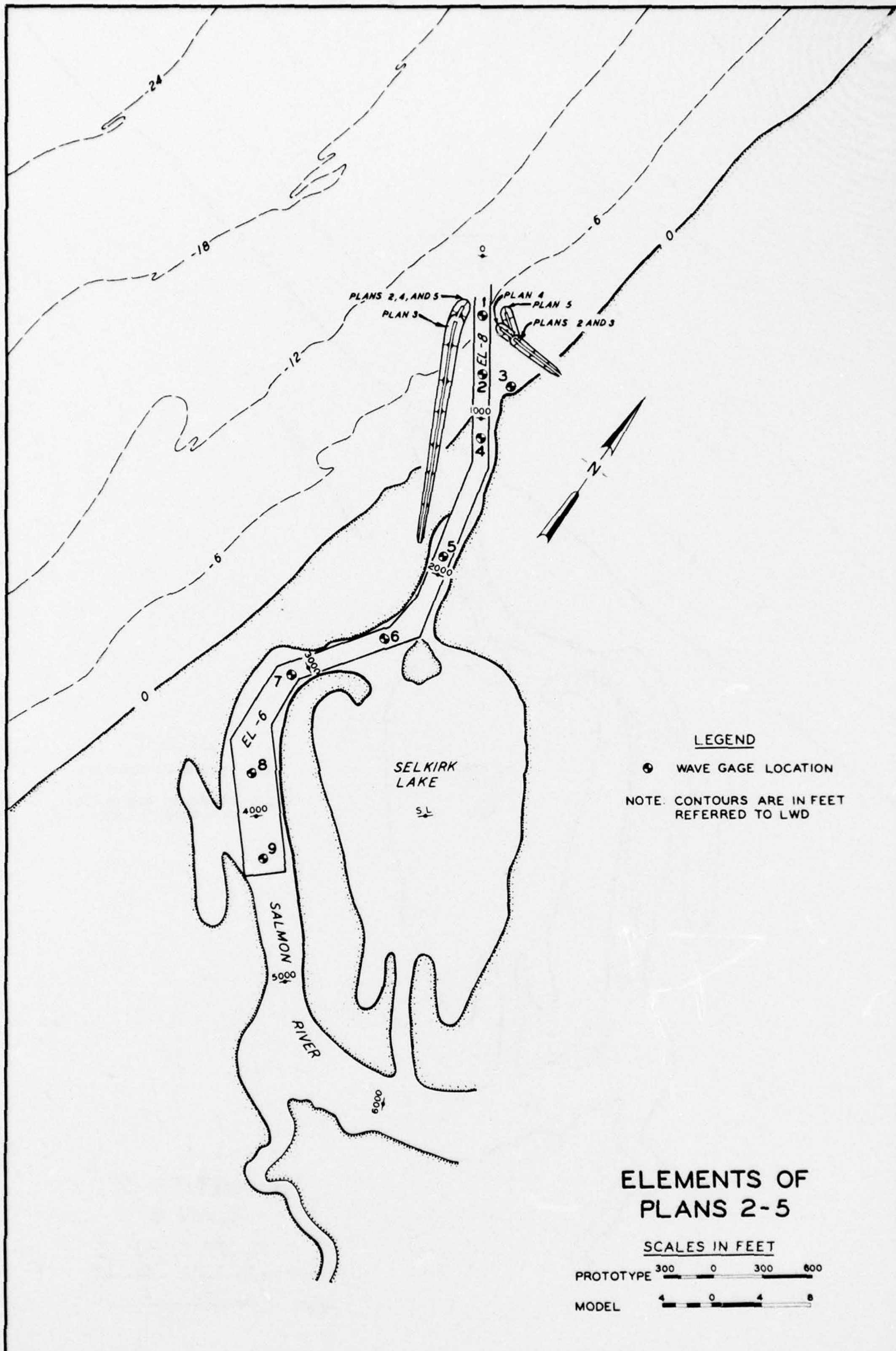

PROPOSED BREAKWATER ALIGNMENTS
1
1A
2
3
4
5
6
7
8
9
1000
2000
4000
5000
6000
0
-6
-12
-18
-24
SELKIRK LAKE
S.L.
SALMON RIVER
N
LEGEND
WAVE GAGE LOCATION
NOTE: CONTOURS ARE IN FEET REFERRED TO LWD
EXISTING CONDITIONS
SCALES IN FEET
PROTOTYPE 300 0 300 600
MODEL 4 0 4 8

PLATE 1

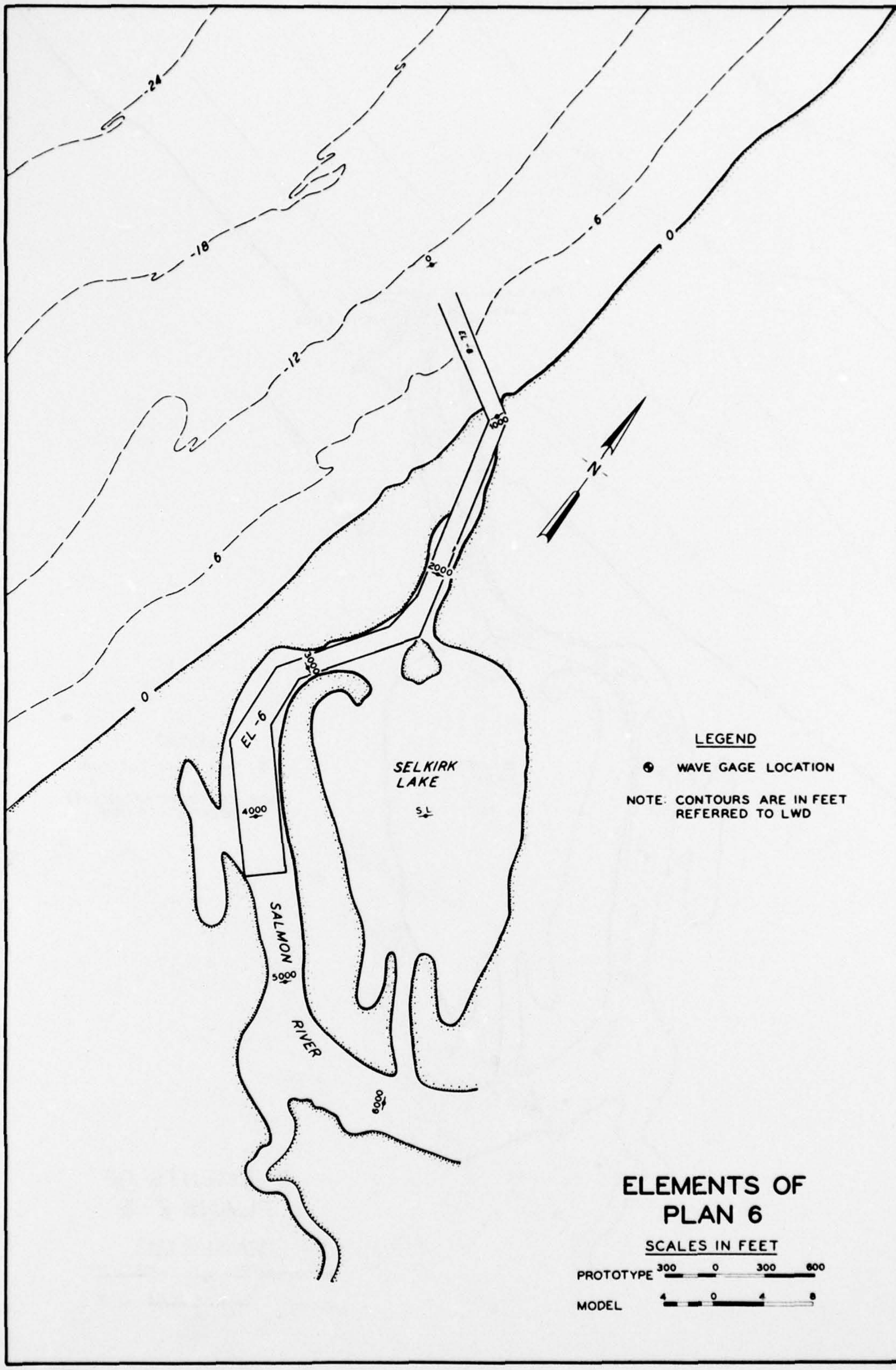


ELEMENTS OF PLAN 1

PLATE 2



ELEMENTS OF
PLANS 2-5

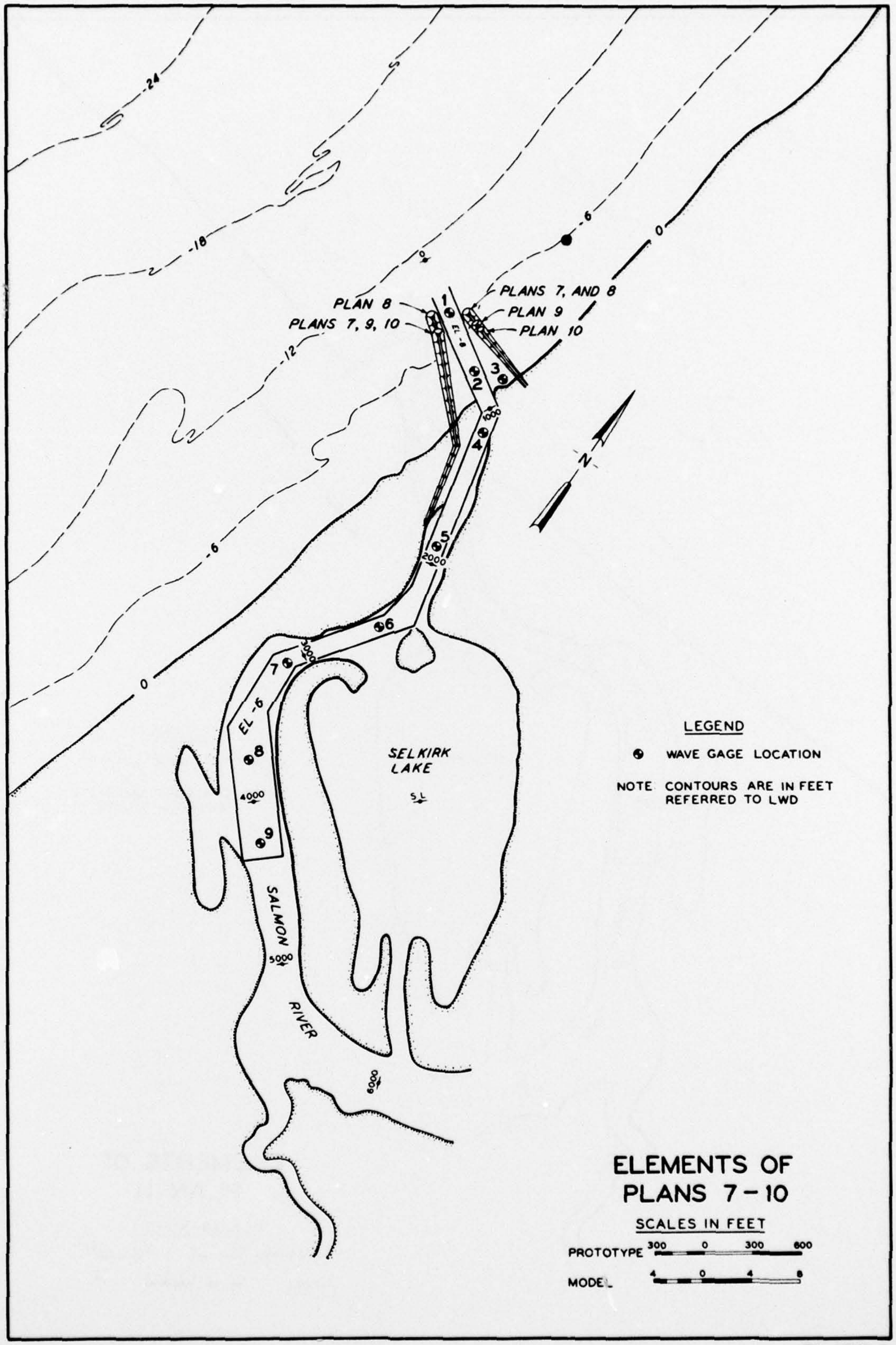


**ELEMENTS OF
PLAN 6**

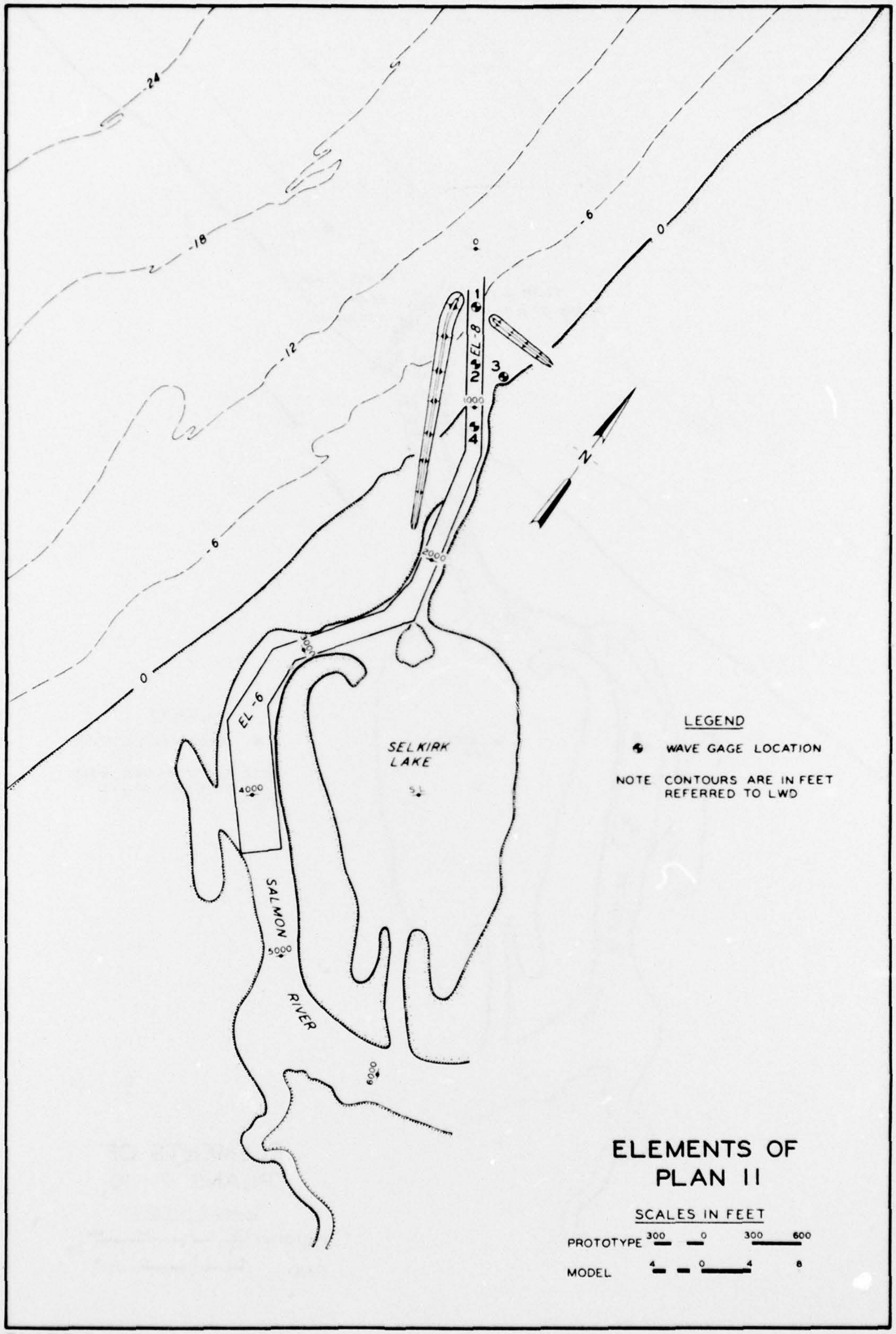
SCALES IN FEET

PROTOTYPE 300 0 300 600

MODEL 4 0 4 8

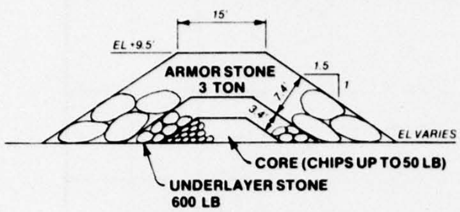


ELEMENTS OF PLANS 7 – 10

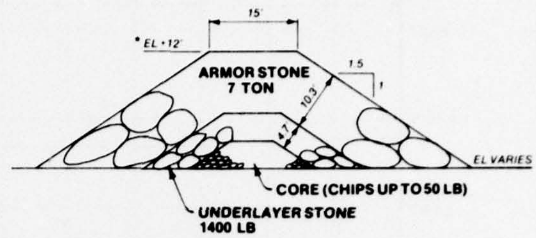


24
18
12
6
0
EL-8
1
2
3
4
1000
2000
3000
4000
5000
6000
EL-6
SELKIRK
LAKE
SALMON
RIVER
N
LEGEND
WAVE GAGE LOCATION
NOTE CONTOURS ARE IN FEET REFERRED TO LWD
ELEMENTS OF PLAN II
SCALES IN FEET
PROTOTYPE 300 0 300 600
MODEL 4 0 4 8

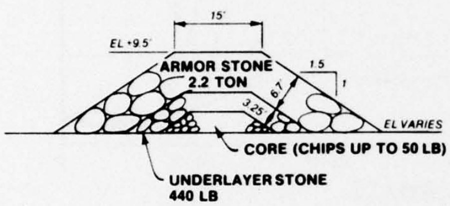
PLATE 6



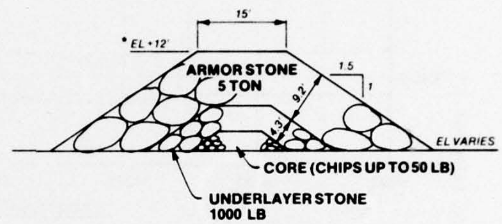
HEAD SECTION



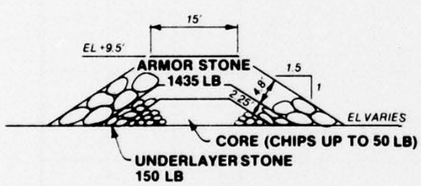
HEAD SECTION



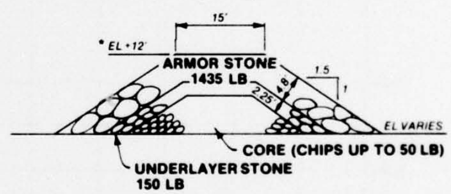
TRUNK SECTION



TRUNK SECTION



SHORE SECTION



SHORE SECTION

NORTH BREAKWATER

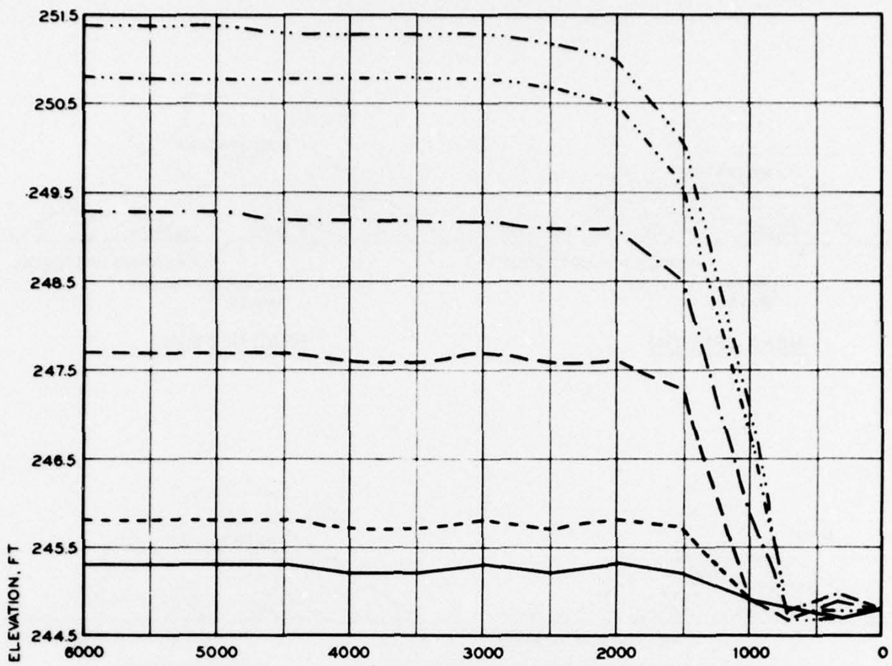
PLAN NO.	LENGTH, FT HEAD SECTION	TRUNK SECTION	SHORE SECTION
1	NA	NA	NA
2	80	100	180
3	80	100	180
4	180	100	180
5	230	100	180
6	NA	NA	NA
7	100	100	375
8	100	100	375
9	50	100	375
10	0	100	375
11	180	100	180

SOUTH BREAKWATER

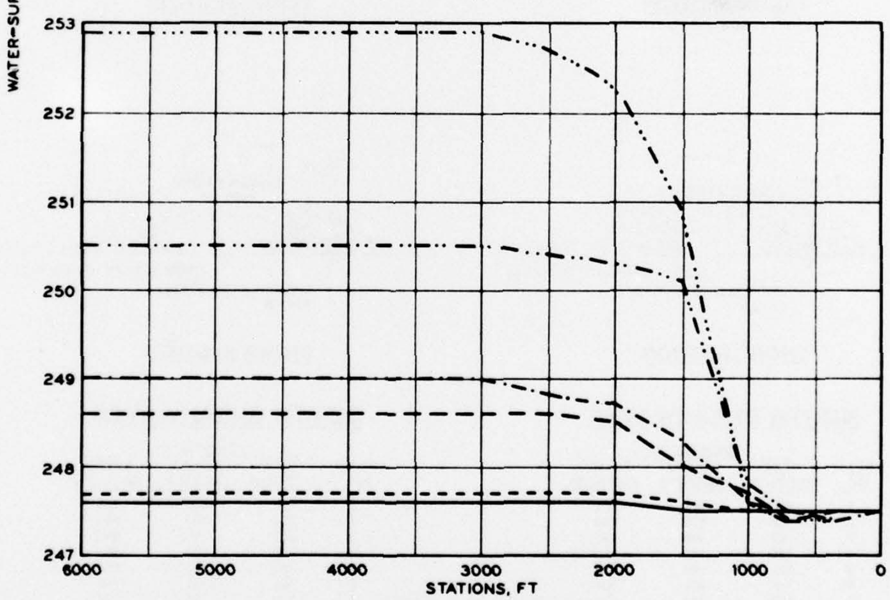
PLAN NO.	LENGTH, FT HEAD SECTION	TRUNK SECTION	SHORE SECTION
1	NA	NA	NA
2	300	300	850
3	200	300	850
4	300	300	850
5	300	300	850
6	NA	NA	NA
7	300	225	225
8	400	225	225
9	300	225	225
10	300	225	225
11	300	300	850

* EL +10 FOR PLAN 11.

NORTH AND SOUTH BREAKWATER SECTIONS



LAKE LEVEL 244.8 FT



LAKE LEVEL 247.5 FT

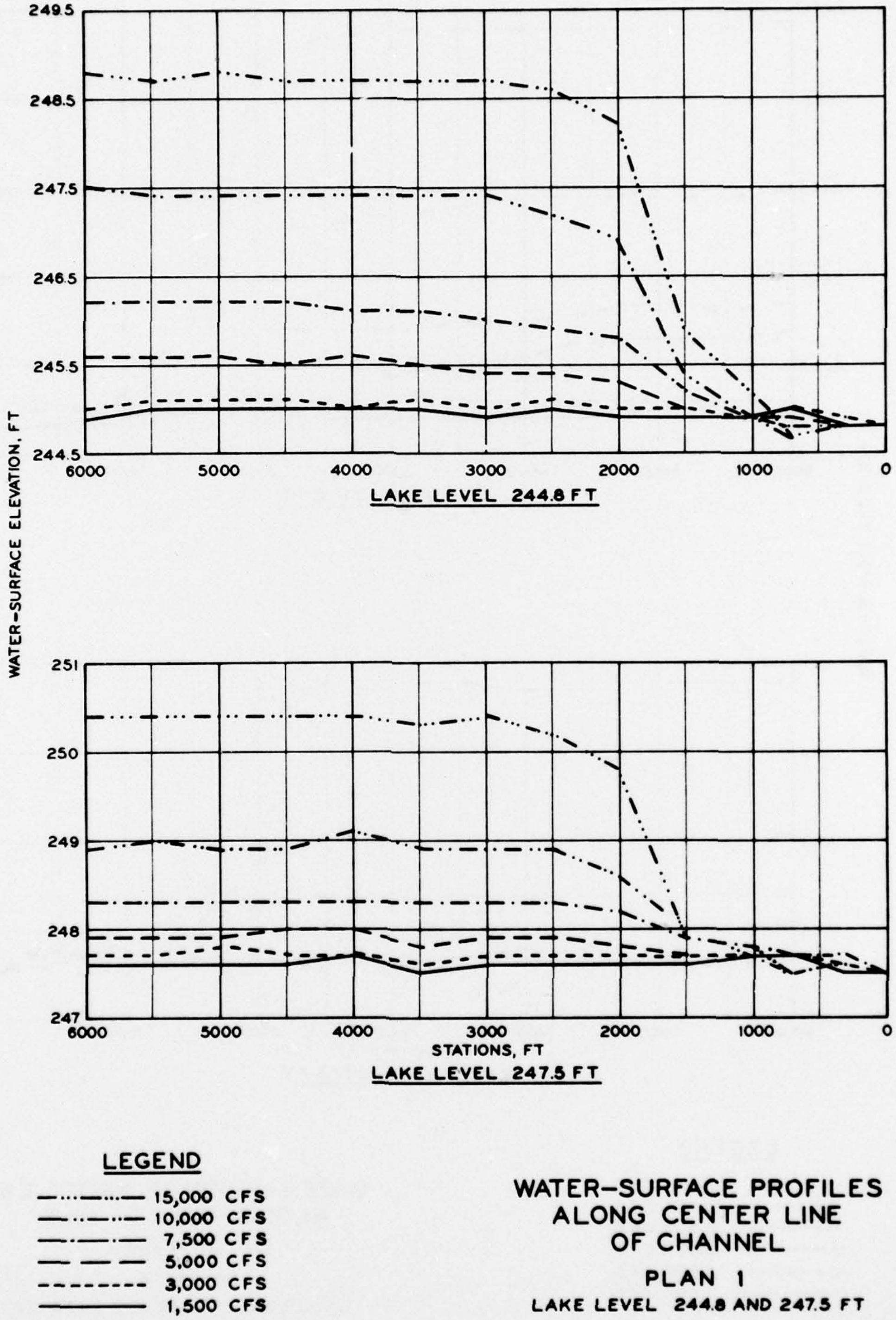
LEGEND

15,000 CFS
10,000 CFS
7,500 CFS
5,000 CFS
3,000 CFS
1,500 CFS

WATER–SURFACE PROFILES ALONG CENTER LINE OF CHANNEL

EXISTING CONDITIONS

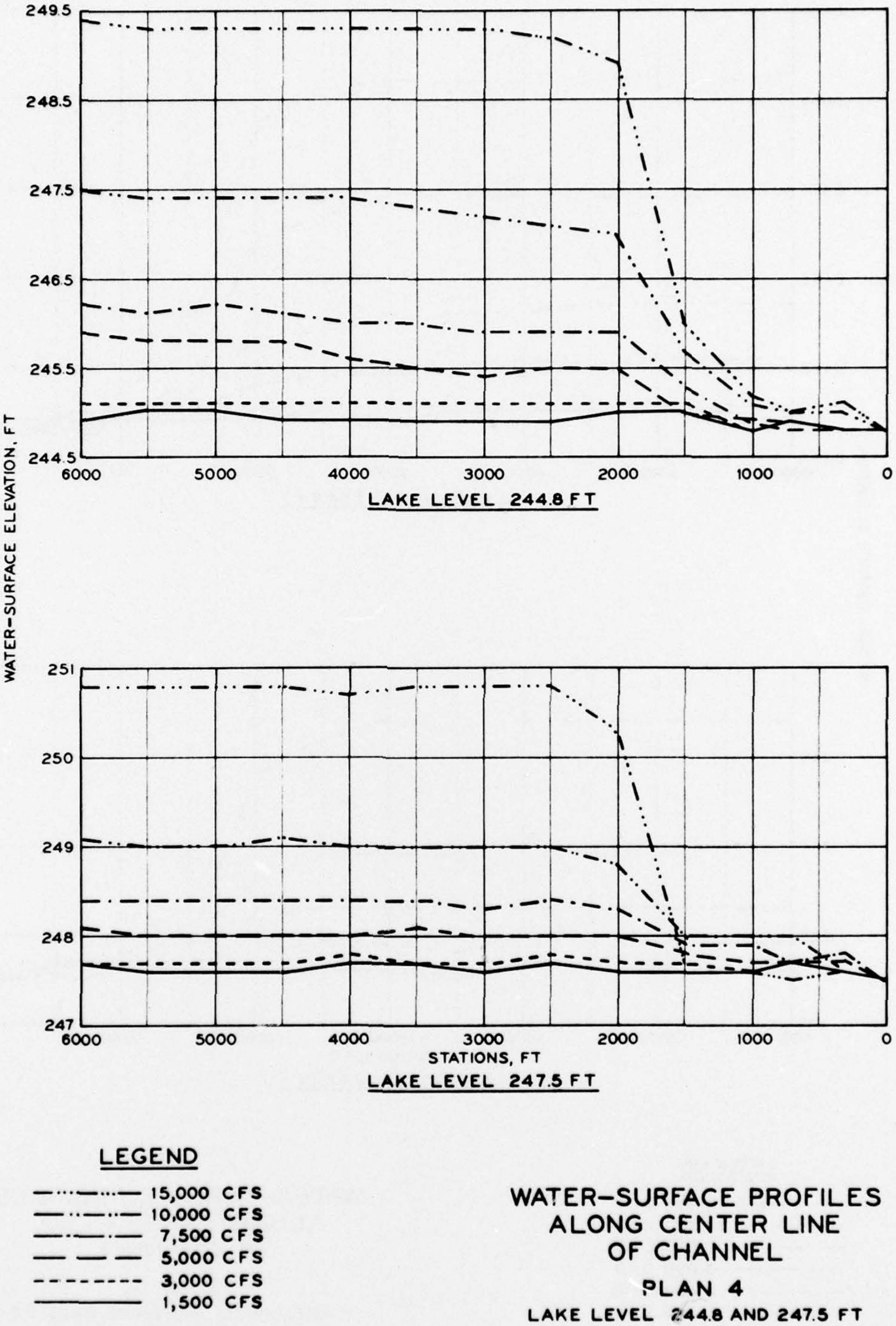
LAKE LEVEL 244.8 AND 247.5 FT



WATER-SURFACE PROFILES
ALONG CENTER LINE
OF CHANNEL

PLAN 1

LAKE LEVEL 244.8 AND 247.5 FT

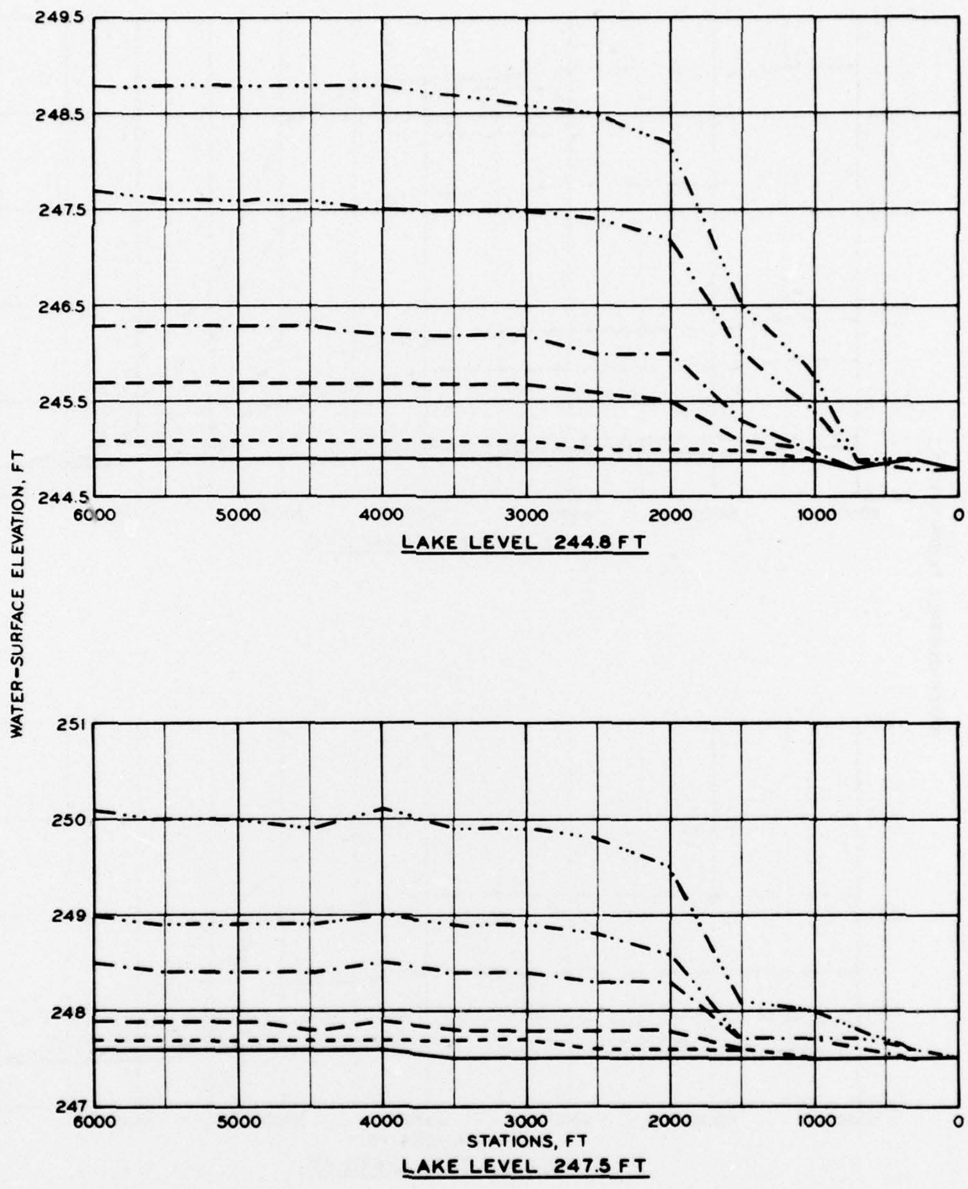


WATER-SURFACE PROFILES ALONG CENTER LINE OF CHANNEL

PLAN 4

LAKE LEVEL 244.8 AND 247.5 FT

PLATE 10



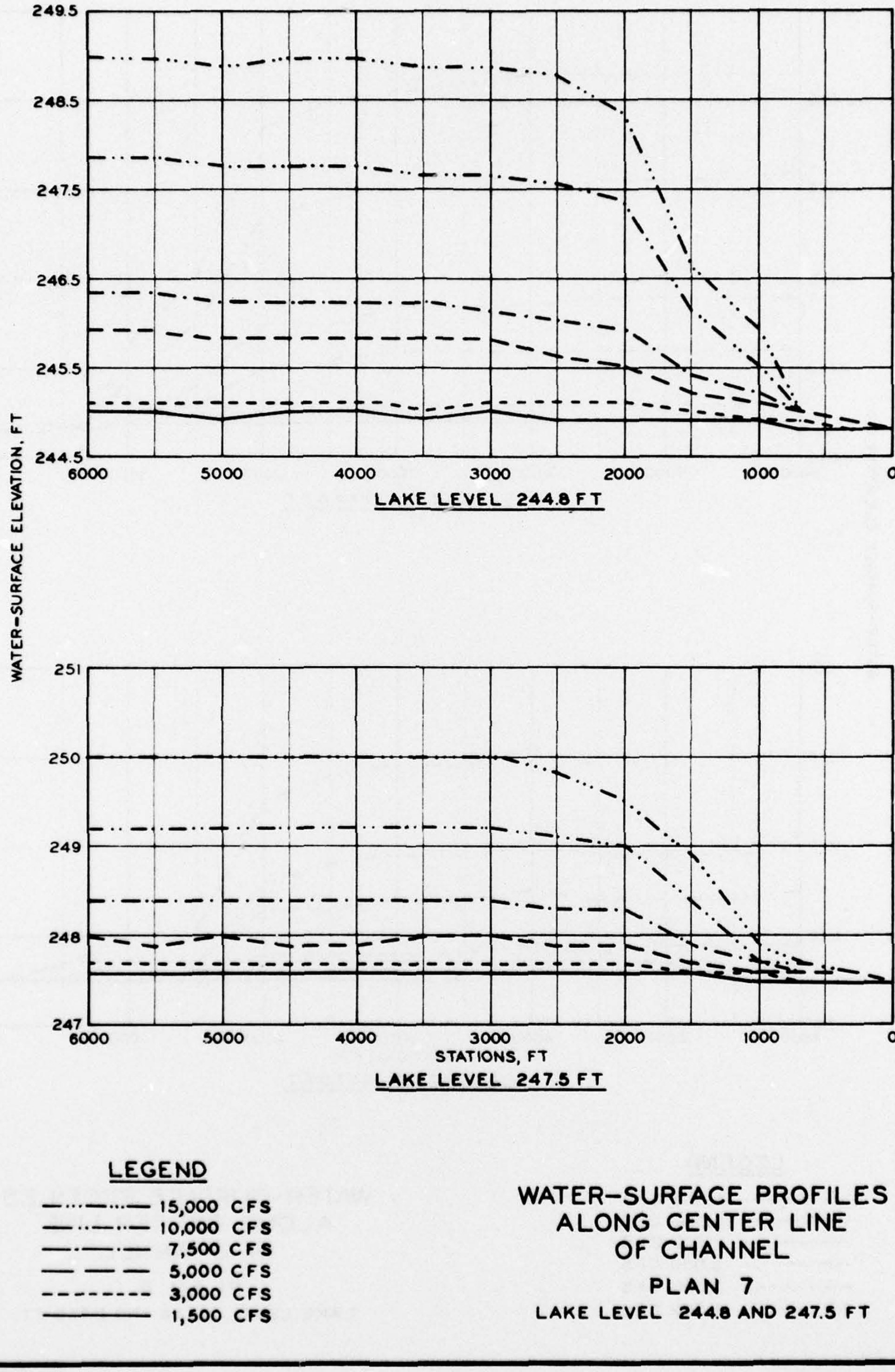
LEGEND

- 15,000 CFS
- 10,000 CFS
- 7,500 CFS
- 5,000 CFS
- 3,000 CFS
- 1,500 CFS

WATER-SURFACE PROFILES
ALONG CENTER LINE
OF CHANNEL

PLAN 6

LAKE LEVEL 244.8 AND 247.5 FT



WATER-SURFACE PROFILES
ALONG CENTER LINE
OF CHANNEL

PLAN 7

LAKE LEVEL 244.8 AND 247.5 FT

APPENDIX A: NOTATION

A	Area
b	Shallow-water orthogonal spacing
b_o	Deepwater orthogonal spacing
$(b_o/b)^{1/2}$	Refraction coefficient, K_r
D_{50}	Median particle diameter
H	Shallow-water wave height
H_o	Deepwater wave height
$H_{1/3}$	Significant wave height
K_r	Refraction coefficient
K_s	Shoaling coefficient
L	Length
n	Manning's roughness coefficient
Q	Discharge
T	Time
V	Velocity
$\mathcal{V}$	Volume
γ	Specific weight
γ'	Apparent specific weight
η_D	Ratio of median diameter particles
η_{γ}'	Ratio of apparent specific weights
λ	Horizontal scale
μ	Vertical scale

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Bottin, Robert R

Port Ontario Harbor, New York, design for wave protection and prevention of shoaling; hydraulic model investigation / by Robert R. Bottin, Jr. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1977.

37, [78], 1 p., 12 leaves of plates : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; H-77-20)

Prepared for U. S. Army Engineer District, Buffalo, Buffalo, New York.

References: p. 36-37.

1. Breakwaters. 2. Harbors. 3. Hydraulic models. 4. Port Ontario Harbor, N. Y. 5. Shoaling. 6. Water wave experiments. I. United States. Army. Corps of Engineers. Buffalo District. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; H-77-20.

TA7.W34 no.H-77-20