REPORT
MODEL STUDY OF PALMAS DEL MAR MARINA
PUERTO RICO

EXTENSION OF THE MODEL STUDY
OF SEPTEMBER 1973

ORKUSTOFNUN STRAUMFRÆÐISTÖÐ
NEA HYDRAULIC LABORATORY
REYKJAVIK ICELAND

NEA Hydraulic Laboratory
Reykjavik Iceland

REPORT
MODEL STUDY OF PALMAS DEL MAR MARINA
PUERTO RICO

EXTENSION OF THE MODEL STUDY
OF SEPTEMBER 1973

The Palmas del Mar Company

San Juan, Puerto Rico

OSSFS 7418 OCTOBER 1974

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I. INTRODUCTION

This report presents the results of tests on an extention of the model study of September 1973 on the Palmas Del Mar Marina, Puerto Rico.

This work was executed under the same contract of January 23rd 1973 as the previous study between the Palmas Del Mar Company and NEA Hydraulic Laboratory in Reykjavík.

The tests were performed jointly by NEA Hydraulic Laboratory and The Icelandic Harbour Authority.

The tests were performed by Mr. BJÖRN ERLENDSSON, Laboratory Engineer, and the NEA's Laboratory Staff.

Dr. P. BRUUN visited the laboratory on March 14th and July 8th to 12th.

The extention of the previous model test of September 1973 is divided in two main test programs.

A. Tests on modification of the outer harbour:

- Comparison between wave action at the earlier location of the pier head and the new location.
- Wave action at the new angular pier head and the new wharf basin.
- The influence of baffles along the pier heads.

These model tests were performed in February 1st to March 20th 1974.

B. Test on modification of the north breakwater.

- The configuration of the breakwater head.
- Wave action in the entrance

  with the new north breakwater design.
- Comparison between uprushes and stability of the three

  suggested rubble mound cross sections.

These model tests were performed on June 28th to July 15th.

NEA HYDRAULIC LABORATORY

Principal Investigator

Gísli Viggósson

Director of Laboratory

Jónas Elíasson

2. SUMMARY AND CONCLUSION

A. Modification of the outer harbour:

Keeping in mind that waves exceeding about 2,0 m with periods equal or larger than 8 sec. only occur for averagely 20-30 days per year the relocation of the pier head seems justified. The pier head should however be closed by as apron possibly provided with baffles to protect the area behind it. The baffles absorb energy and offer thereby some protection for the entrance to the inner harbour.

Baffles along the pier head however increase wave action at the head reducing the number of days for safe berthing. On the other hand the number of safe berthing days inside the pier head increases compared to the situation with open pier head.

The angular pier head at the wharf basin should be closed. By doing the wave condition inside the pier head and in the wharf basin becomes satisfactory under almost all wave conditions.

B. Modification of the north breakwater.

For minimum disturbance of the incoming natural wave trains the breakwater shall turn against the incoming waves as described in the report of September 1973. The head shall be vertical. The best configuration of the breakwater head is the half circle thereby avoiding reflection and washes into the entrance. The length of the vertical head section shall be about 40 ft. The entrance width shall be 150 ft. minimum. With this breakwater head the wave action just outside the harbour and in the entrance is about the same as in the previous layout with a crib breakwater.

The tests on uprushes and stability of the rubble mound show that project 1 is stable and project 4 is stable with modifications. The two cross sections are shown on sheets No. 13 and 14 with the recommended wave screen heights.

3. COMPARISON BETWEEN WAVE ACTION AT THE EARLIER LOCATION OF THE PIER HEAD AND THE NEW LOCATION

3.1 Introduction

The purpose of this model study was to investigate the wave action along the new location of the pier head and compare the results with the comprehensive tests at earlier date.

3.2 Model Scale and Hydraulic Similitude

The model scale used in the present study is the same as the one used previously, i.e. 1:45. According to Froude's modeling law the following scale factors apply:

Geometrical scale			1:45
Velocity scale	$1:(45)^{1/2}$	=	1:6.71
Time scale	$1:(45)^{1/2}$	=	1:6.71
Area scale	$1:45^2$	=	1:2025
Volume scale	$1:45^3$	=	1:91125

3.3 Model Construction and Expermental Techniques

The model layouts are shown on sheets No. 1 and 2. The model was constructed in the same manner as in the previous model study.

The tests were carried out with the crib breakwater as protection, because they were only concerned with wave action inside the harbour entrance. Waves were produced by a pneumatic wave generator located just outside the harbour entrance.

As the boundary condition at the entrance is different from the previous tests some discrepanices can be expected when comparing the two test results.

3.4 Model Tests

All tests were conducted with waves approaching from E 20°S. As in the previous study the wave characteristics investigated were the following:

Prototype			Model (1:45)	
Height m	Height ft	Period sec	Height cm	Period sec
1.8	6.0	6	4.1	0.89
2.4	8.0	8	5.4	1.19
3.0	10.0	10	6.8	1.49

The tests were carried out with the above periods as well as at periods slightly shorter and longer than those listed (± 5 percent) and the average values used.

All test results on the modifications of the outer harbour are given in sheets No. 3, 4 and 5, for 6 sec, 8 sec and 10 sec, respectively.

The legend for each test group is as follows:

: The results of earlier tests. Sheets No. 4, 6 and 8, Report 1973.

( ): The results of tests with the new location of the pier head and the protection of the wharf.

< > : The results of the tests with the baffled walls, installed in the pier head and in the head of angular wall.

> < : The results of the tests with a wall without baffles. These tests were only run with 8 sec. waves.

3.5 Discussion of the Test Results

The wave height indicated the in the earlier tests (sheets No. 4, 6 and 8) shall be compared to those indicated in the latter test (sheets No. 3, 4 and 5).

As it could be expected, comparison of results obtained in the previous model study and the corresponding results obtained in the present study indicate some discrepancies. These discrepancies can be explained by the different outer boundaries of the two models. Perhaps the greatest deviation from previous results is found in measuring area No. 9 where considerably higher waves were encountered in the present study (sheet No. 2). This phenomenon must be caused by wave reflection from the north boundary wall of the present model (sheets No. 1 and 2).

Waves in open sea, 6 sec. and 1.8 m height, Sheet No 3

Very little differences were observed in wave heights along the two locations of the pier head, this is also the case for the entrance to the inner harbour.

Waves in open sea, 8 sec. and 2.4 m height, Sheet No 4

Wave heights are considerably greater along the new pier head than with the earlier location. Waves up to 1.2 m may occur at the inner part of the new pier head, which is unsatisfactory. Very little differences are measured at the entrance to the inner harbour.

<u>Waves in open sea, 10 sec. and 3.0 m height, Sheet No 5</u>

Wave heights are considerably greater along the new pier head when compared to the earlier location. Waves up to 1.4 m may occur there. Waves are also higher in the entrance channel to inner harbour.

4. WAVE ACTION AT THE NEW ANGULAR PIER HEAD AND THE NEW WHARF BASIN

4.1 Introduction

A new wharf is placed on the south side in the outer harbour just east of the synchrolift. In the model the section between the pier head and the shore was constructed as a rubble mound. As shown in sheet No. 2, several measuring points were placed around the pier head and at the two localities outside the synchrolift.

4.2 Discussion of the test results

Waves in the open sea, 6 sec. and 1.8 m height, Sheet No 3

Very little difference was observed in the new basin. In front of the open pier head waves up to 0.4 m may occur.

Waves in the open sea, 8 sec. and 2.4 m height, Sheet No 4

Waves are considerably lower in the new wharf basin. In front of the open pier head waves up to 0.5 m may occur.

Waves in open sea, 10 sec. and 3.0 m height, Sheet No 5

Very little difference was recorded in the new wharf basin. In front of the open pier head waves up to 1.3 m may occur.

5. THE INFLUENCE OF BAFFLES ALONG THE PIER HEADS

5.1 Introduction

The purpose of these tests was to investigate the ability of baffles along the pier heads in order to decrease wave action in the outer harbour and at the entrance to the inner harbour.

Three sets of baffles were tested. The baffle width and spacing between baffles were as follows:

	Model		Prototype	
No.	Spacing s, cm	Width k, cm	Spacing s, cm	Width k, cm
1	10	2.5	450	112
2	20	5.0	900	225
3	11	5.5	495	248

In the first two sets of tests the apron and baffles extended from the pier deck downwards about two thirds of the water depth, but in test No. 3 the apron and baffles extended all the way from the pier deck down to bottom. Details of baffles are shown in sheet No. 6.

5.2 Discussion of the test results

Although the three sets of baffles were tested only the results of the largest ones (No. 3) are presented here as the others were not effective. Photos 2 and 4 demonstrate the influence of the baffles on the wave action. These photos should be compared with photos 1 and 3 which show the original layout.

Waves in open sea, 6 sec. 1.0 m height, Sheet No 3

Wave heights are satisfactory in all areas.

Waves in open sea, 8 sec. 2.4 m height, Sheet No 4

Wave heights along the new pier head are much greater than without baffles. Waves up to 1.45 m may occur. The waves, however, become lower behind the new pier head and at the entrance to the inner harbour, as well as the new wharf basin. The waves along the wharf pier head may reach up to 1.25 m height.

Waves in open sea, 10 sec. 3.0 m height, Sheet No 5

The wave heights are unsatisfactory, except in the new wharf basin. Waves up to 1.70 m may occur at the new pier head and up to 1.25 m at the new wharf pier head.

To evaluate the effect of the baffles one test of 8 sec. waves was run with an apron, but no baffles.

In this case the wave height was about doubled at the entrance to the inner harbour and in the entrance channel. The maximum height increases along the new pier head ranged from 1.4 m with baffles up to about 1.70 m without baffles.

The maximum wave height along the new wharf pier head decrease from about 1.25 m to about 0.70 m with the baffles.

6. THE CONFIGURATION OF THE BREAKWATER HEAD

6.1 Introduction

The aim of this model study was to investigate the influence of the configuration of the breakwater head on the wave action at the breakwater head and in the entrance. It is well know that some types of breakwater heads concentrate the wave energy and reflect waves back in the entrance. The breakwater head shall interfer as little as possible with the incoming natural wave train to make the wave action in to the entrance harbour as smooth as possible.

6.2 Model tests

Based on experience a breakwater head turned seaward gives the minimum wave disturbances in the entrance. Several alternative layouts of such a breakwater head were tested in scale 1:45. Sheet No. 7 shows the layout tested in the model and sheet No. 8 shows the model layout in scale 1:45. The evaluation of the test results were based on visual inspection of the reflection and wash around the breakwater head and the interference of the head with the natural wave train in the entrance.

All tests were run for 8 sec. waves.

The effects of the various breakwater heads are clearly demonstrated on photos 5 to 11.

6.3 Discussion of the test results

Comparison of the tests shows that neither type of rubble mound breakwater heads is satisfactory due to reflection and washes from the breakwater head.

Comparison between the rounded head, type A4, (photos 9, 10 and 11) and the two rubble mound heads shows that the incident waves are least disturbed due to reflection and washes around the rounded head.

Based on these results wave measurements around the rounded breakwater head were undertaken. They are described in following section.

7. WAVE ACTION IN THE ENTRANCE WITH THE NEW NORTH BREAKWATER DESIGN

7.1 Introduction

The purpose of these tests was to compare the wave action in the entrance with the new north breakwater design with the comprehensive tests with the crib described in the Report of September 1973. Several measuring points outside the harbour were added in order to study the wave action in more detail.

7.2 Harbour entrance tests

Sheet No. 8 shows model which was run in scale 1:45 with wave direction E20°S. It should be noted that just a small area in the outer harbour was constructed which makes a smaller model than the model in 1973. For this reason some discrepencies may be expected in results of the two tests.

The tests were run for three wave periods: 6 sec, 8 sec and 10 sec.

7.3 Discussion of the test results

The results of the tests are shown in sheet No. 9, 10 and 11.

These results should be compared with previous tests (Report of September 1973, sheets No. 4, 6 and 8).

The following table gives the average wave heights in the most important areas for the two tests. The measuring areas depicted by A, B, E, F and G are shown on sheet No. 8.

Wave action outside harbour		T=6 sec H=1.8 m		T=8 sec H=2.4 m		T=10 sec H=3.0 m	
Measuring areas		Previous	Present	Previous	Present	Previous	Present
Outside harbour	A	1,65	2,10	2,95	3,00	2,65	3,20
In entrance	B	1,40	1,70	2,95	2,95	2,00	2,40
Inside entrance	E	0,80	1,05	1,35	1,00	1,80	1,30
Further inside entr.	F	0,40	0,50	1,15	1,20	1,20	1,30
At end of pier head	G	0,50	0,70	0,85	0,45	0,65	0,60

The results are not consistent. For 6 sec. waves the present tests show 20 to 30 percent higher waves in all areas. For 8 sec. waves the tests results are more equal. But for 10 sec. waves the present tests show about 20 percent higher waves outside the harbour and in the entrance.

The explanation of these discrepanies however undoubtedly is the different boundary conditions in the outer harbour, sheet No. 8. It is reasonable to estimate the reflection from the boundary to about 20 percent. This wave reflection causes radial stresses in the direction out of the harbour which again increases the wave heights in the entrance and just outside the harbour.

Said reflection will be much smaller in nature and it is estimated that the results of the present tests should be reduced about 20 percent.

According to this the wave heights are about equal or rather a little less than in the previous tests.

8. COMPARISON BETWEEN UPRUSHES AND STABILITY OF THE THREE SUGGESTED RUBBLE MOUND CROSS SECTIONS

The purpose of the model study was to investigate the uprushes and stability of the three suggested rubble mound designs at the seaward side on the north breakwater. These cross sections were designed by Dr. P. Bruun and submitted to the laboratory by letter of June 6th, 1974. They are shown on sheet No. 12 and 13 as well as on project 1, 3 and 4.

8.2 Model scale and hydraulic similitude

The tests were carried out in scale 1:60. According to Froude's modelling law the following scale factors apply:

Geometrical scale		1:60
Velocity and time scale	$1:60^{1/2}$	1:7,75
Area scale	$1:60^{2}$	1:3600
Volume and weight scale	$1:60^{3}$	1:216000

8.3 Model construction and Experimental Techniques

The model layout is shown on sheet No. 14 i.e. the three cross sections were studied simentaneously with a wave attack peripendicular to the sections. Each section was 36 m in prototype.

The core of the mound was replaced by sand and a plastic sheet was placed between the sand and the second armour layer.

The stones used in the two armour layers were river stones with a specific weight of 2,75 t/m^3. In all cases the outermost layer was placed pell-mell although the project 3 shows placed stone layers.

Both the plastic sheet and the pell-mell placement of the armour layers cause less stability of the cross sections and higher uprush so the results of the test is to some extent conservative.

The incident wave height was measured as shown on sheet No. 14 and the wave period was 6 sec. 8 sec and 10 sec.

The uprushes were estimated by visual observation.
The damage to the breakwater was estimated in terms of the percentage of stones moved more than their own dimension. This percentage is estimated on the basis of the total number of stones in the armour layer.

8.4 Discussion of the tests results

Photos 12 to 15 give an impression of the test set-up, the wave action on the rubble mound and the uprush. The test starts with incident wave height of 2,6 m and wave period of 8 sec.

No damage was observed on the three sections. The lowest uprush was at project 3 and the uprush was higherst at project 4.

Then the incident wave height was increased with the same wave period of 8 sec to:

H = 3,3 m at project 1
H = 3,0 m at project 3
H = 2,7 m at project 4

The highest uprush was now observed at project 3 and after about 1 hour duration in the prototype almost total damage occur of project 3 which therefore was eliminated from further testing.

The uprushes for the respective indident wave height was almost the same for projects 1 and 4 or in both cases to the elevation of 9′ and 10′ respectively.

Project 1 was observed stable with damage 3-5 percent. Project 4 however was not stable as damage occurs in the section of the slope 1:2,5 and some distance below that.

After this run following was concluded:

Project 3 is out of the question.

Project 1 stable.

Project 4 may be stable with some modifications.

Modification of project 4.

Project 4 was modified by replacing the 2-4^{ts} stones at the 1:2,5 slope by 5-6^{ts} stones down to about 5′ below M.S.L.

Then project 4 was tested for wave characteristic as follows:

$H = 3.8$ m and $T = 6$ sec
$H = 2.6$ m and $T = 8$ sec
$H = 3.0$ m and $T = 10$ sec

The duration of the test was about 3 hours in the prototype. After about 1 hour wave action it appeared that it would be necessary to replace the 3-4 ts stones at the upper 1:1.5 slope by 5-6 ts stones. After that the damage was about 3-5 per cent.

According to the uprush severe overtopping occur for wave screen elevation of 10′ but no overtopping occur for wave screen elevation of 14.5′.

8.5 Conclusions of the tests

Project 1 and project 4 modified shown in sheets No. 12 and 13 are both stable with acceptable damage up to 5 percent, and the wave screen elevation of 14′ for project 1 and 13′ for project 4 give little overtopping, except under extreme wave condition.

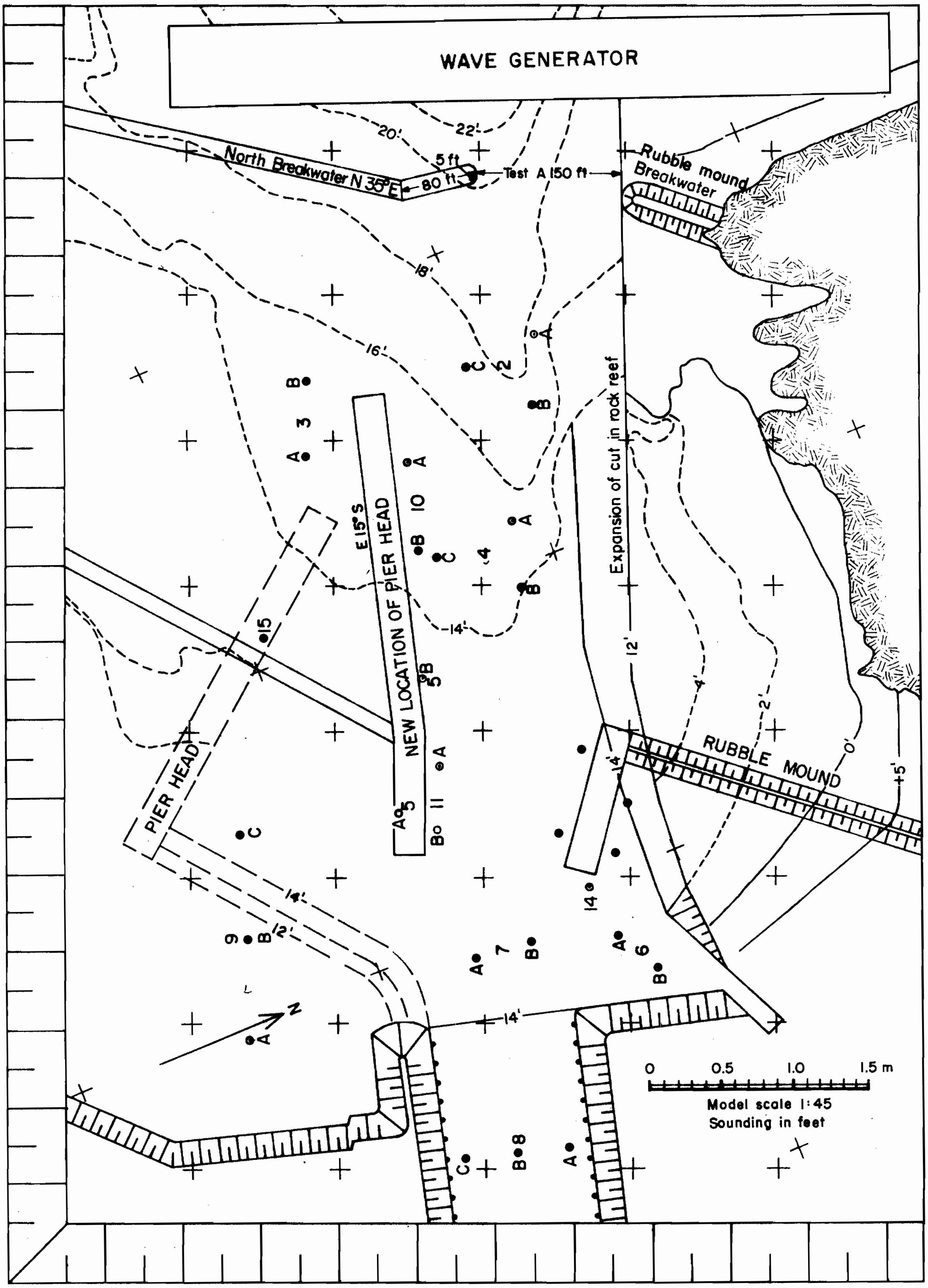
NEA HYDRAULIC LABORATORY	14.11 '74 BE/GV/HO
REYKJAVIK, ICELAND	Tnr. 26
PALMAS DEL MAR MARINA, PUERTO RICO	ORS-4
MODEL STUDY, SCALE 1:45	Fnr. 11784

SHEET NO. 1

MODEL LAYOUT

NEA HYDRAULIC LABORATORY	7.11 '74 BE/GV/HO
REYKJAVIK, ICELAND	Tnr. 26
PALMAS DEL MAR MARINA, PUERTO RICO	ORS-4
MODEL STUDY SCALE 1:45	Fnr. 11749

SHEET NO. 2



LAYOUT OF MODEL

LEGEND:

● MEASURE POINTS.

0 10 20 30 40 50 60 m

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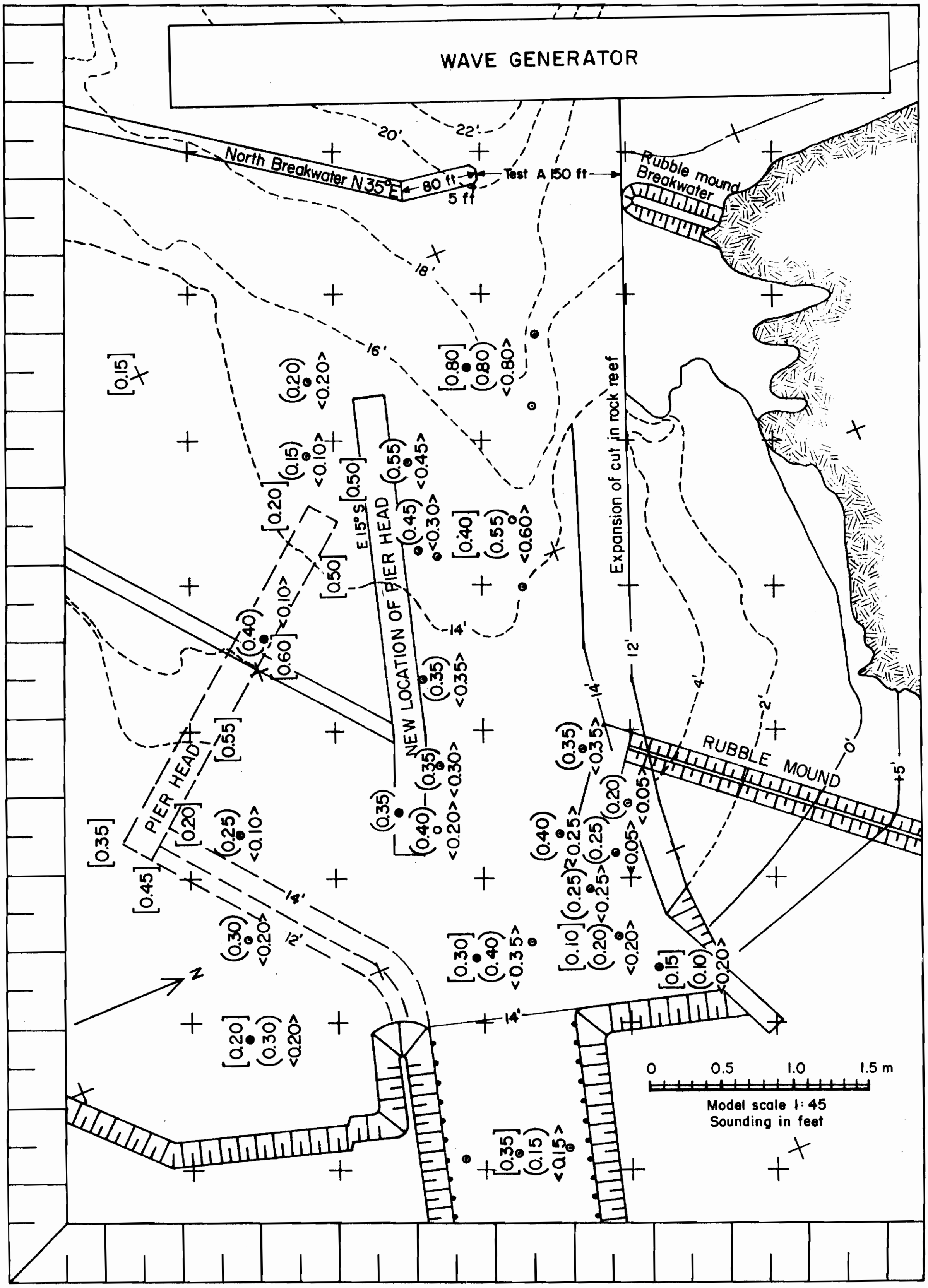
REYKJAVIK, ICELAND	Tnr. 29
PALMAS DEL MAR MARINA, PUERTO RICO	ORS-4
MODEL STUDY SCALE 1:45	Fnr. 12153

SHEET NO. 3



RESULTS OF MODEL STUDY
WAVES IN THE OPEN SEA, 6 SEC, 1,8 M

LEGEND:

[ ]: PREVIOUS TEST (REFS. 1)

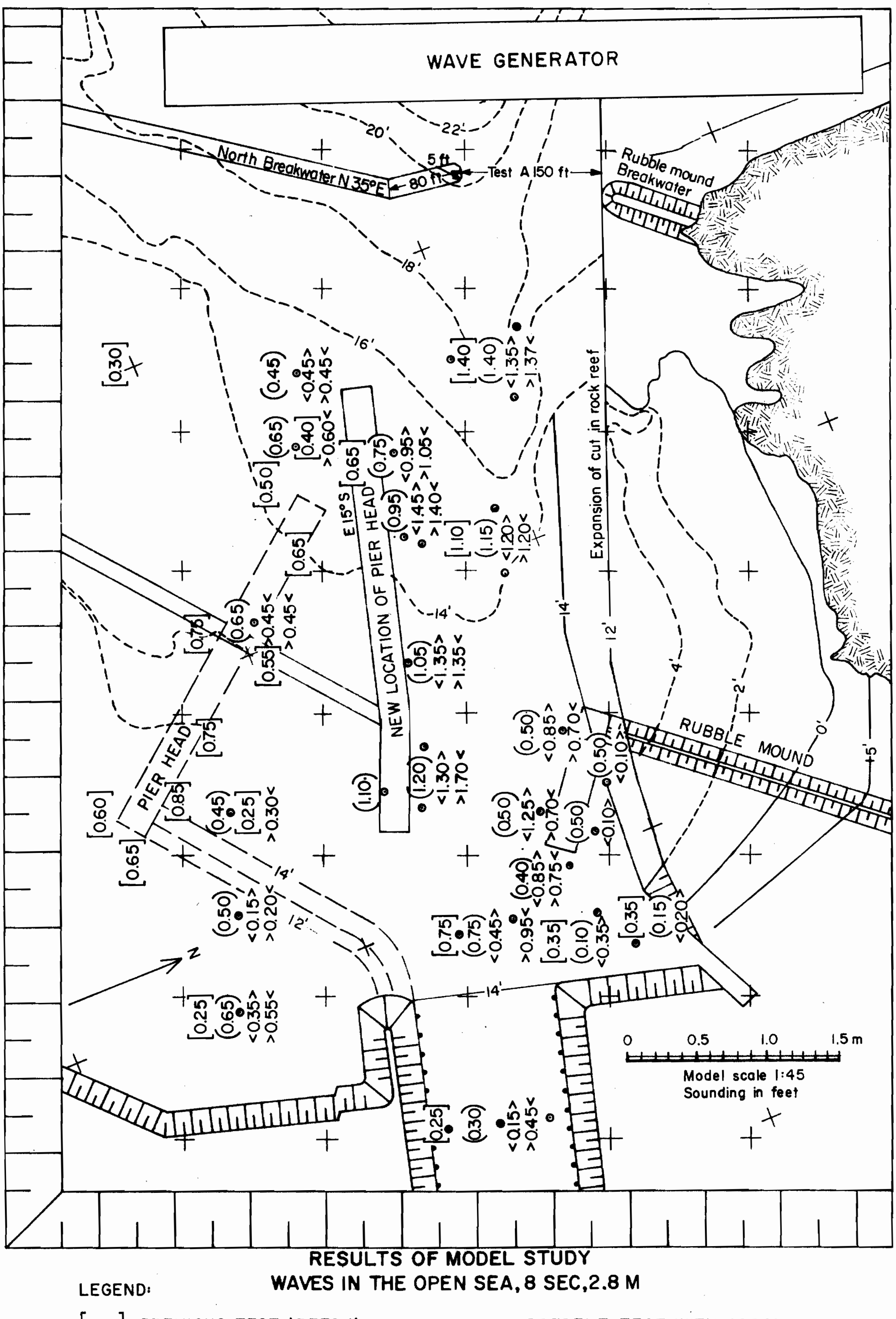
( ): PRESENT TEST OPEN PIER HEAD

< > PRESENT TEST WITH BAFFLES

0 10 20 30 40 50 60 m

NEA HYDRAULIC LABORATORY	7 II '74 BE/GV/HO
REYKJAVIK, ICELAND	Tnr. 30
PALMAS DEL MAR MARINA, PUERTO RICO	ORS-4
MODEL STUDY SCALE 1:45	Fnr. 12154

SHEET NO. 4



RESULTS OF MODEL STUDY
WAVES IN THE OPEN SEA, 8 SEC, 2.8 M

LEGEND:

[ ]: PREVIOUS TEST (REFS. I)

( ): PRESENT TEST OPEN PIER HEAD

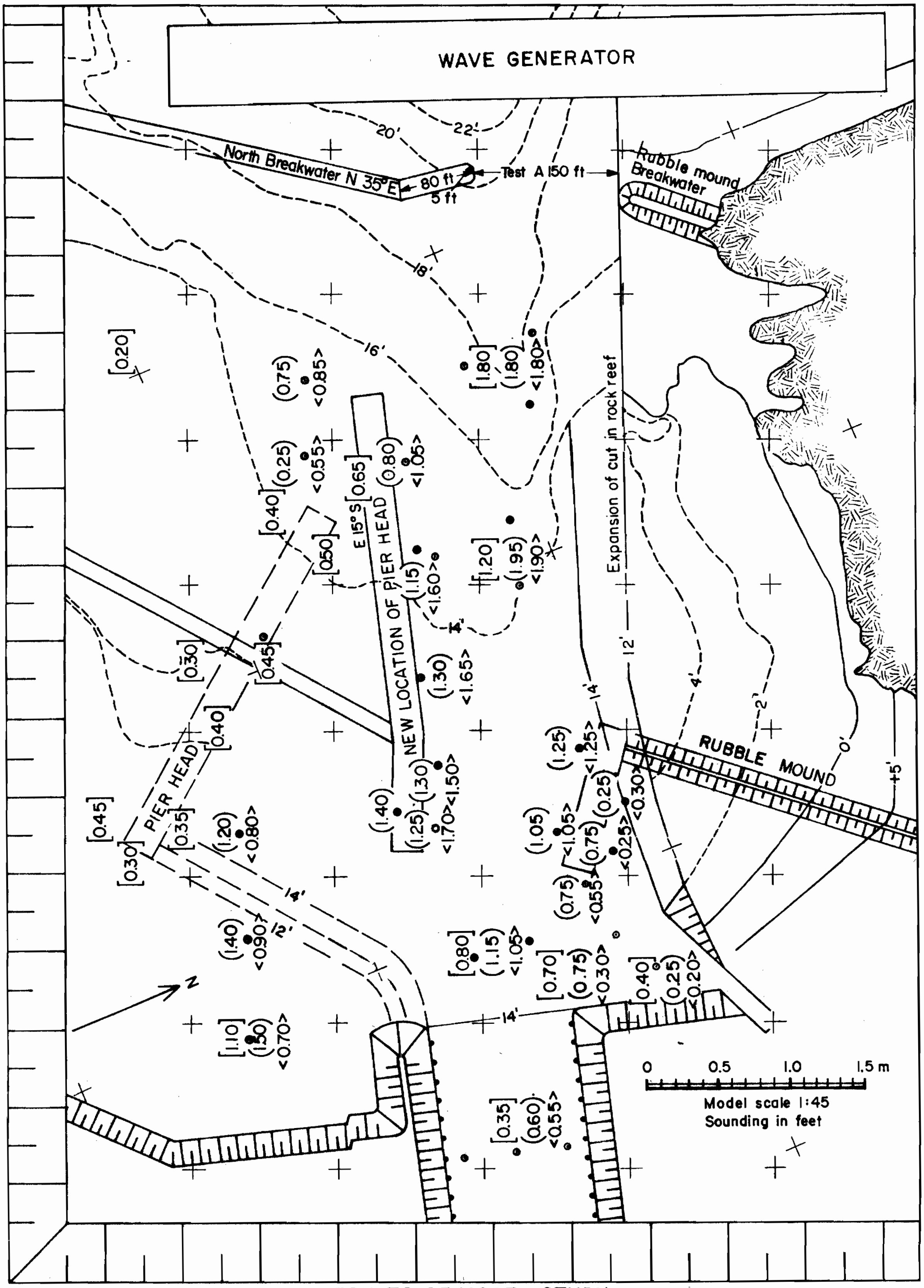
< > PRESENT TEST WITH BAFFLES

> < PRESENT TEST WITH APRON

0 10 20 30 40 50 60 m

NEA HYDRAULIC LABORATORY	7.11 '74 BE/GV/HO
REYKJAVIK, ICELAND	Tnr. 31
PALMAS DEL MAR MARINA, PUERTO RICO	ORS-4
MODEL STUDY SCALE 1:45	Fnr. 12155

SHEET NO. 5



RESULTS OF MODEL STUDY
WAVES IN THE OPEN SEA, 10 SEC, 3,0 M

LEGEND:

[ ]: PREVIOUS TEST (REFS. I)

( ): PRESENT TEST OPEN PIER HEAD

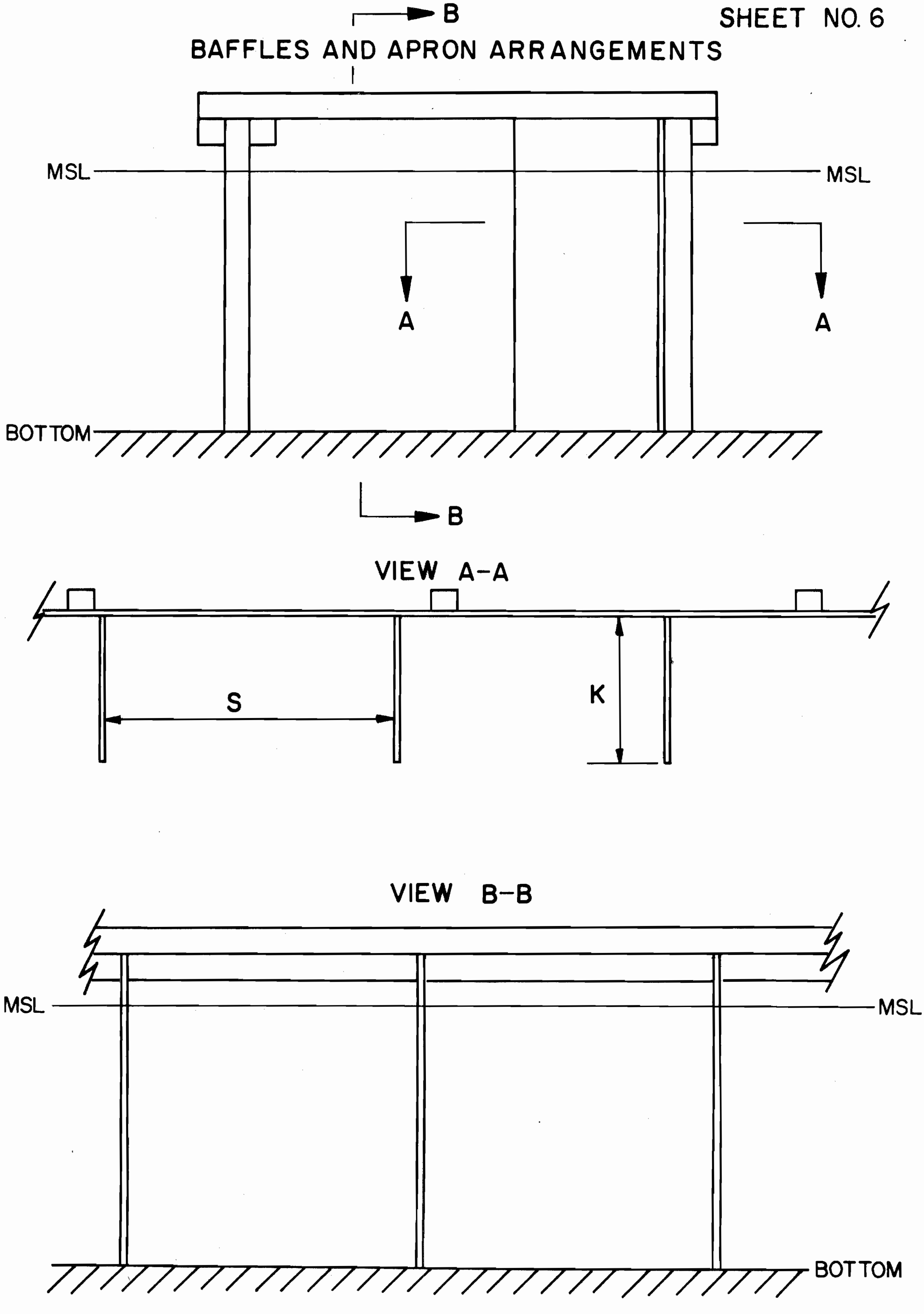
< > PRESENT TEST WITH BAFFLES

0 10 20 30 40 50 60 m

NEA HYDRAULIC LABORATORY	3.5 '74 BE/GV/AV
REYKJAVIK, ICELAND	Tnr. 27
PALMAS DEL MAR MARINA, PUERTO RICO	ORS. 4
MODEL STUDY, SCALE 1:45	Fnr. 11801

SHEET NO. 6

BAFFLES AND APRON ARRANGEMENTS



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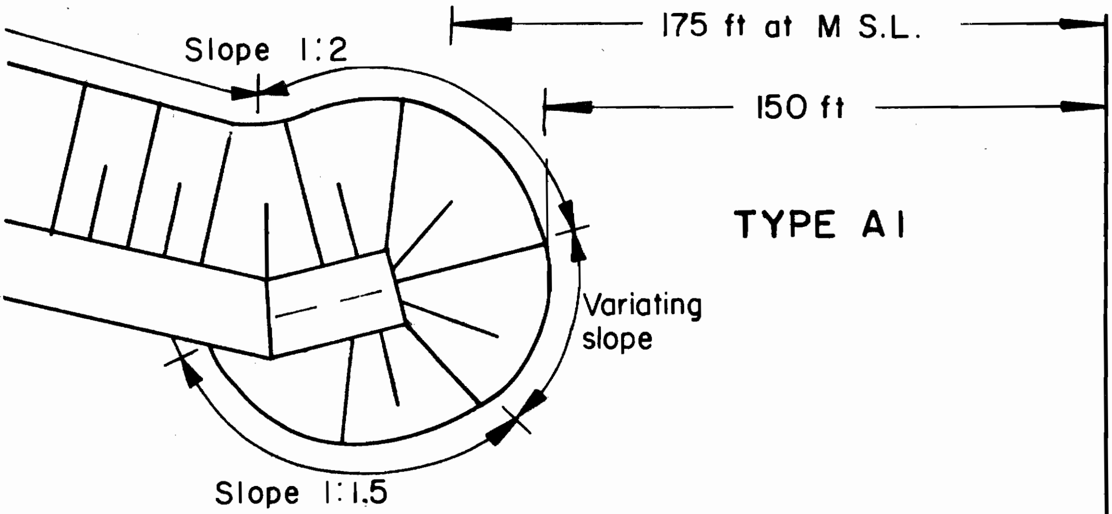
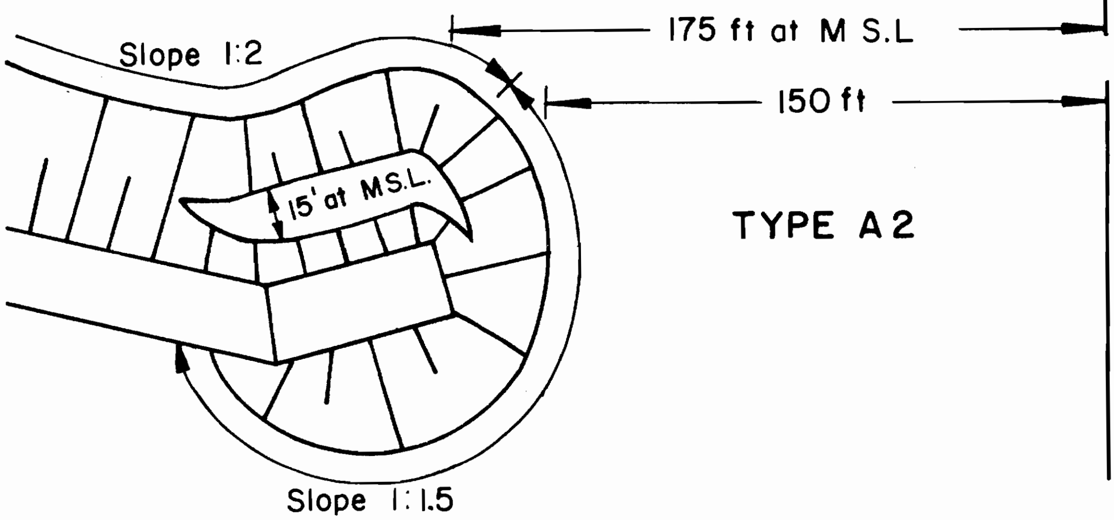
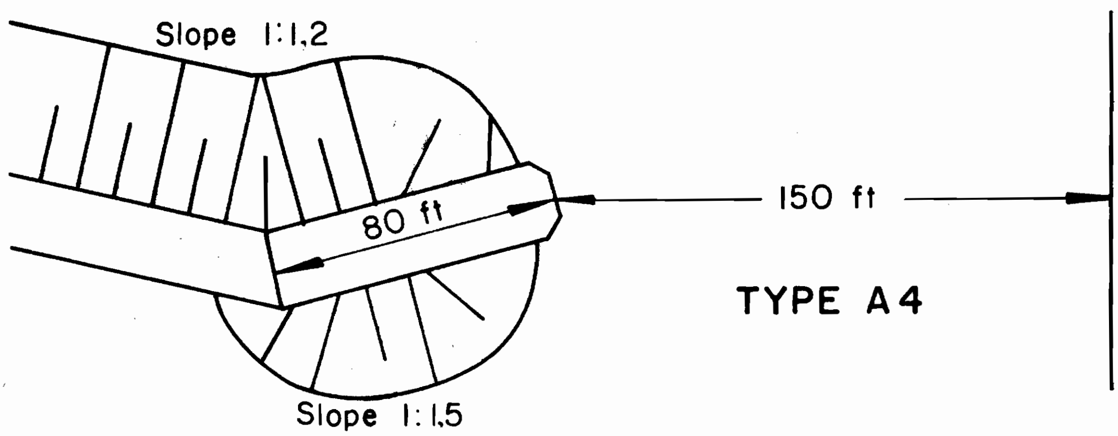
REYKJAVIK, ICELAND	Tnr. 40
PALMAS DEL MAR MARINA, PUERTO RICO	ORS-4
MODEL STUDY, SCALE 1:600	Fnr. 12182

SHEET NO. 7

LAYOUT OF BREAKWATER HEAD TESTED IN THE MODEL.







NEA HYDRAULIC LABORATORY	7.11.'74 BE/GV/SL
REYKJAVIK, ICELAND	Tnr. 33
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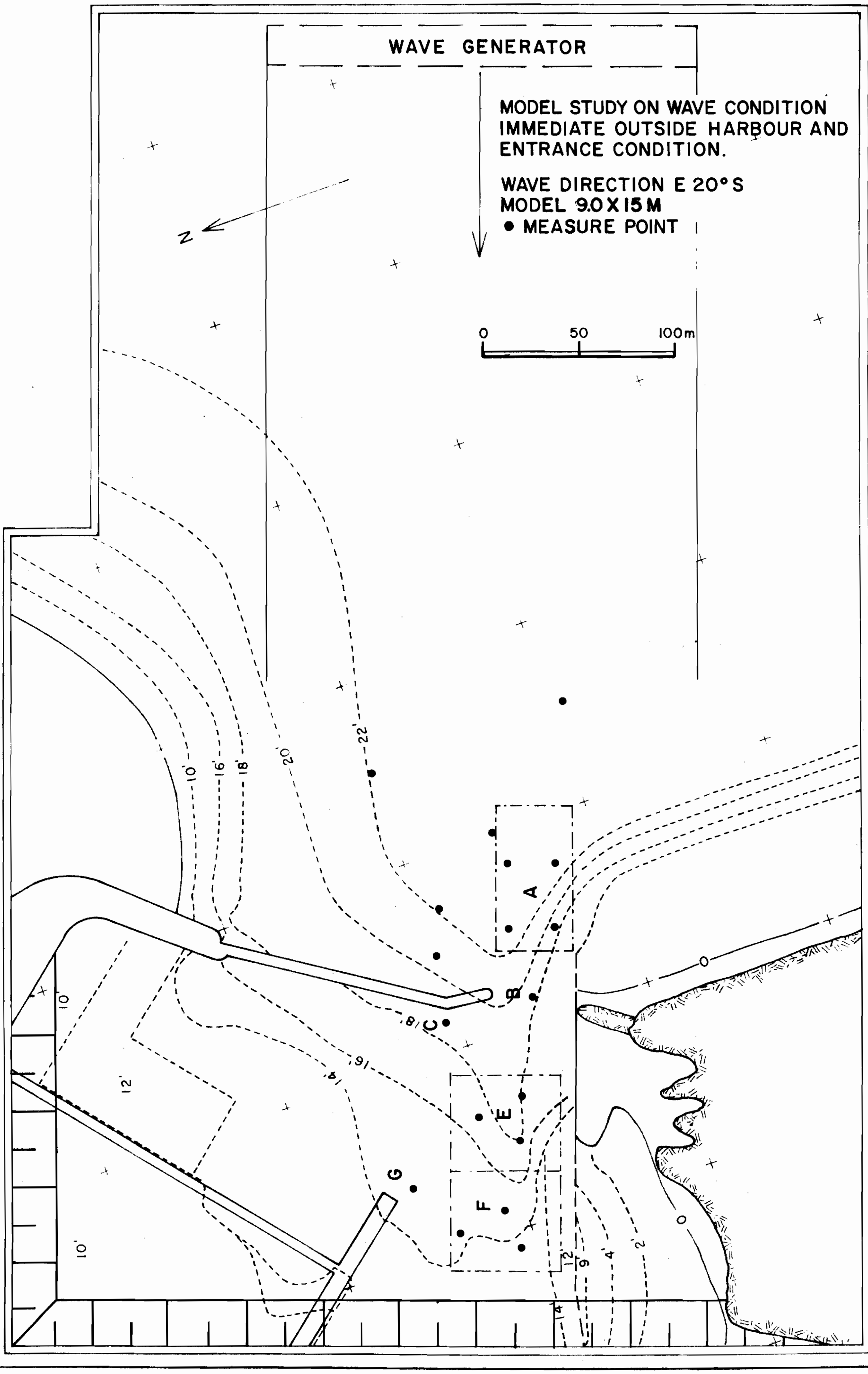
MODEL STUDY, SCALE 1:45	Fnr. 12157

SHEET NO. 8



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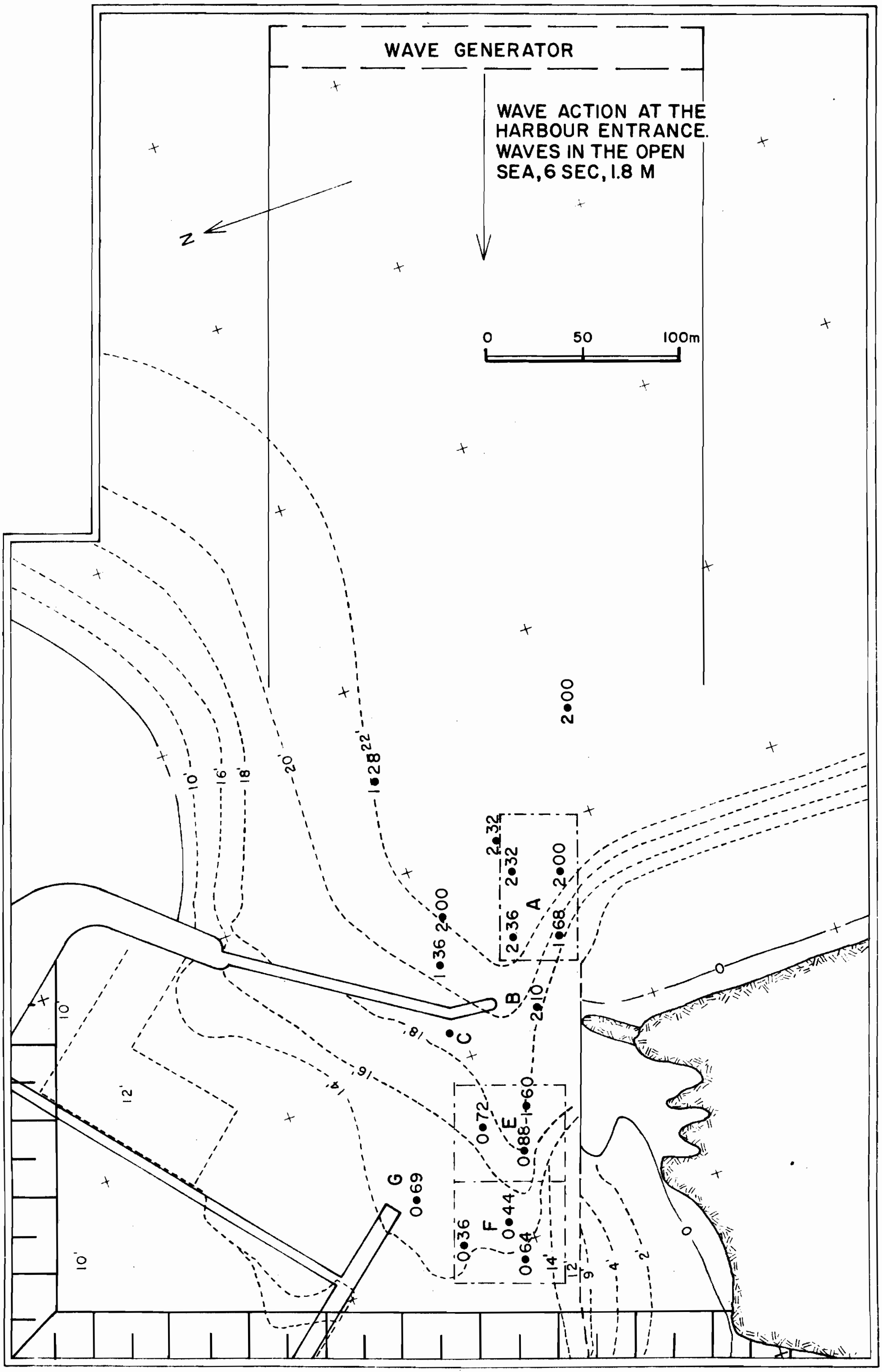
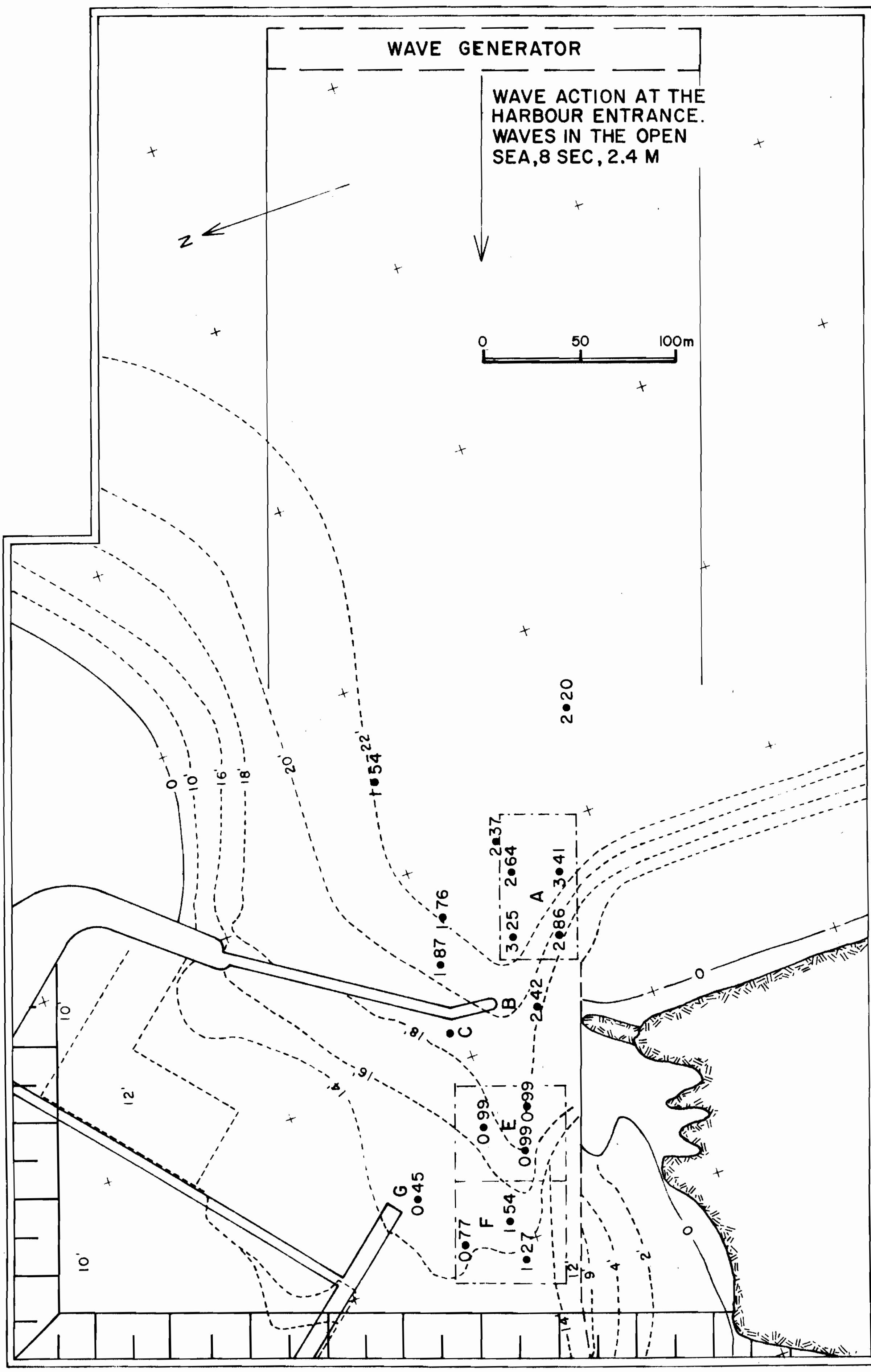
REYKJAVIK, ICELAND	Tnr. 34
PALMAS DEL MAR MARINA, PUERTO RICO	ORS-4
MODEL STUDY SCALE 1:45	Fnr. 12158

SHEET NO. 9



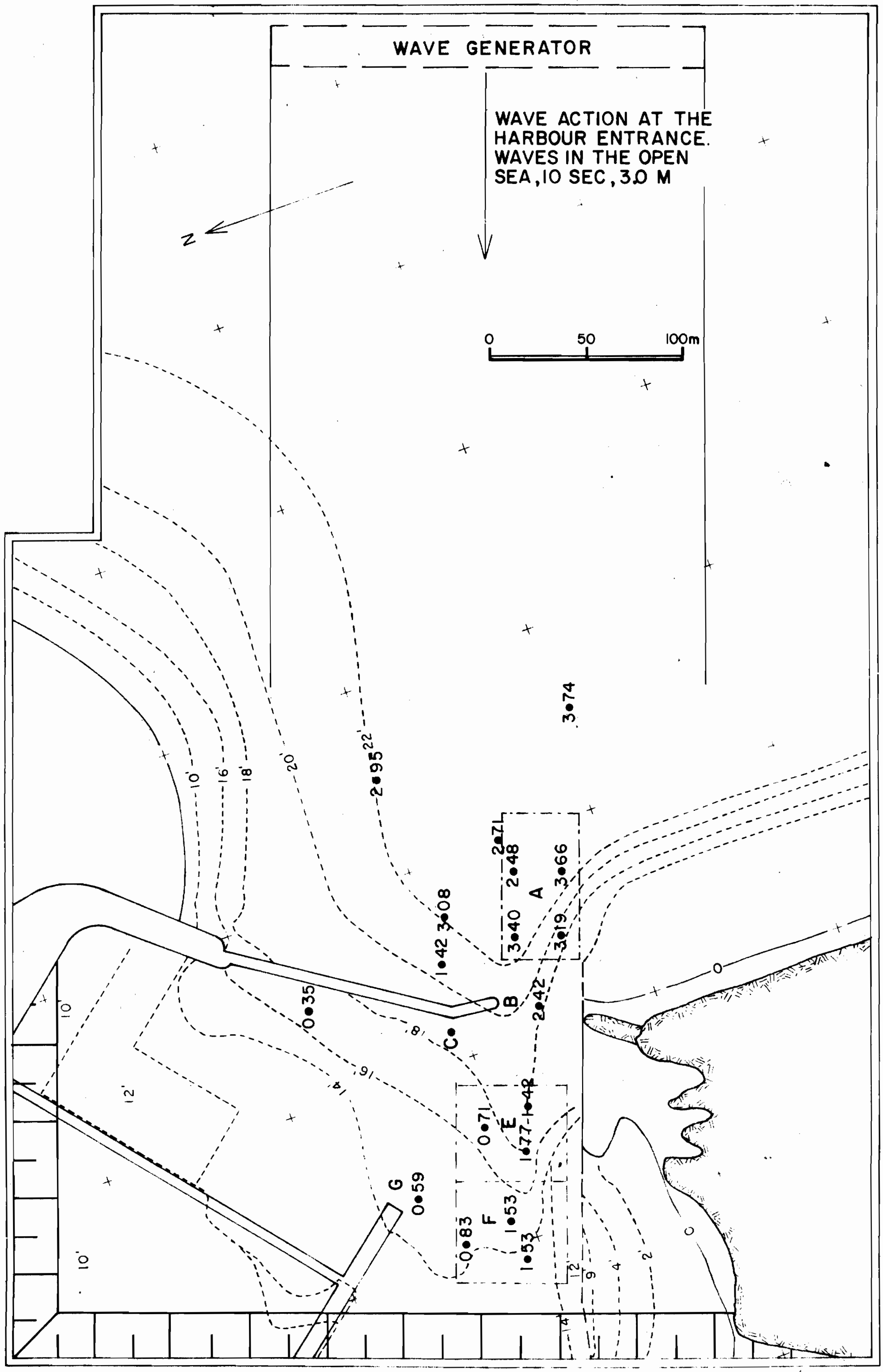
NEA HYDRAULIC LABORATORY	7.11.'74 BE/GV/SL
REYKJAVIK, ICELAND	Tnr. 35
PALMAS DEL MAR MARINA, PUERTO RICO	ORS-4
MODEL STUDY SCALE 1:45	Fnr. 12159

SHEET NO.10



NEA HYDRAULIC LABORATORY | 7.11.'74 BE/GV/SL
REYKJAVIK, ICELAND | Tnr. 36
PALMAS DEL MAR MARINA, PUERTO RICO | ORS-4
MODEL STUDY SCALE 1:45 | Fnr. 12160

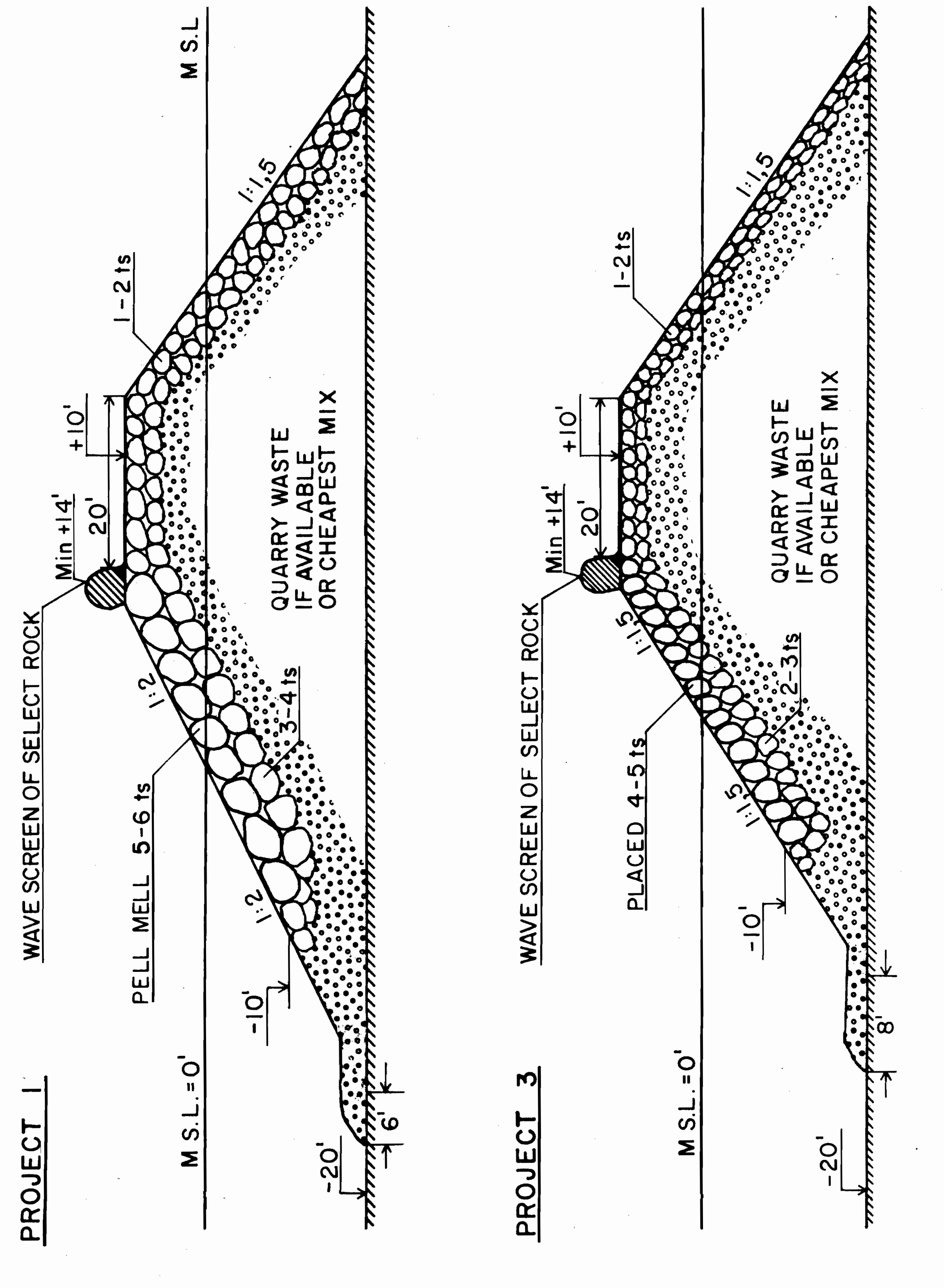
SHEET NO. 11



NEA HYDRAULIC LABORATORY	13.11.'74 BE/GV/SL
REYKJAVIK, ICELAND	Tnr. 37
PALMAS DEL MAR MARINA, PURTO RICO	ORS-4
MODEL STUDY, SCALE 1:60	Fnr. 12161

SHEET NO. 12

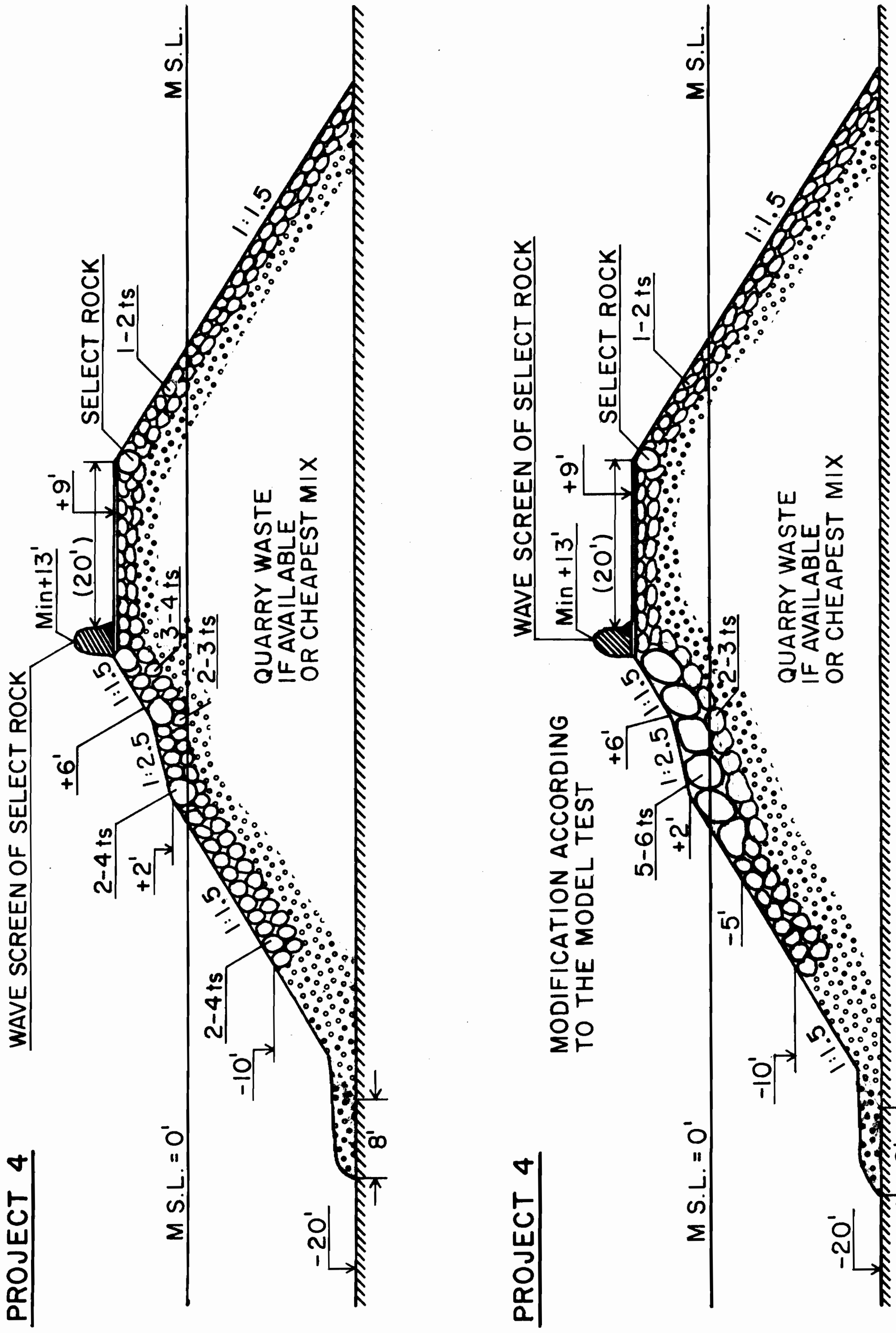
CROSS SECTIONS OF THE NORTH BREAKWATER SUGGESTED BY DR. P. BRUUN



NEA HYDRAULIC LABORATORY	13.11.'74 BE/GV/SL
REYKJAVIK, ICELAND	Tnr. 38
PALMAS DEL MAR MARINA, PURTO RICO	ORS-4
MODEL STUDY, SCALE 1:60	Fnr. 12162

SHEET NO. 13

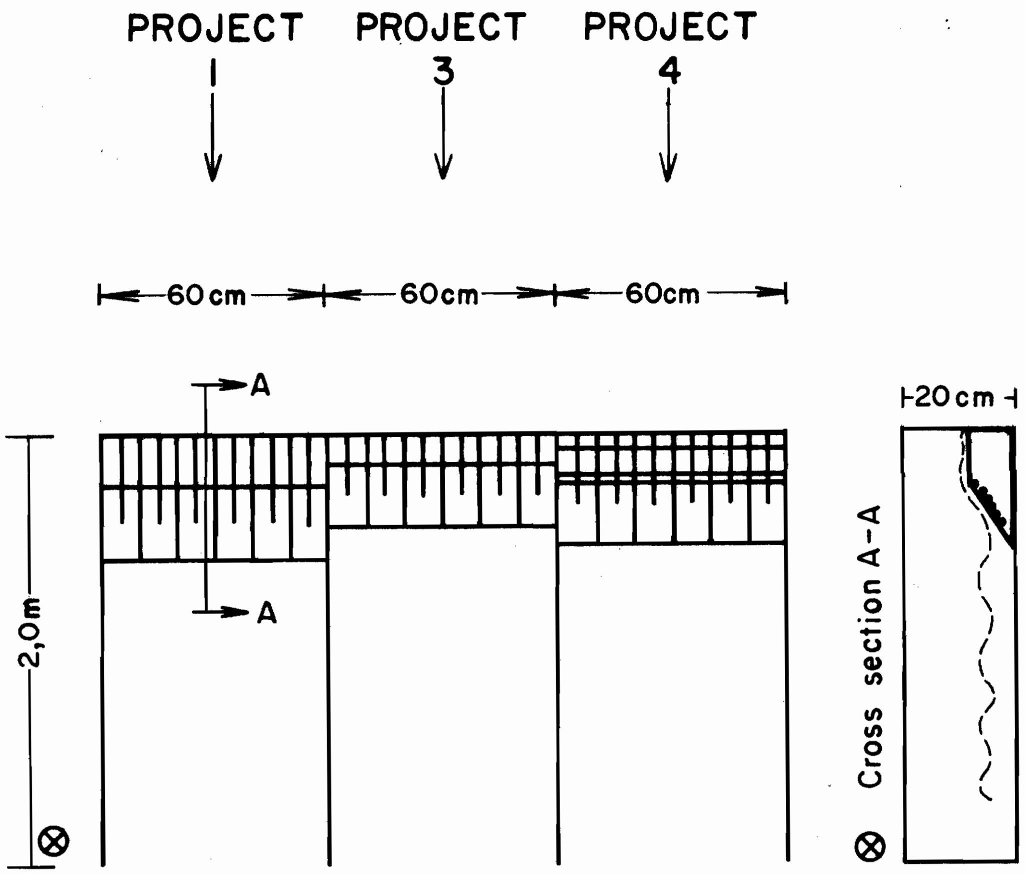
CROSS SECTIONS OF THE NORTH BREAKWATER SUGGESTED BY DR. P. BRUUN



NEA HYDRAULIC LABORATORY	15.11 '74 BE/GV/HO
REYKJAVIK, ICELAND	Tnr. 39
PALMAS DEL MAR MARINA, PUERTO RICO	ORS-4
MODEL STUDY, SCALE 1:60	Fnr. 12163

SHEET NO. 14

LAYOUT OF MODEL TEST ON UPRUSHES AND STABILITY



⊗ Wave recorders

Water depth = 6,0 m in prototype

——11—— = 10 cm in model

WAVE GENERATOR

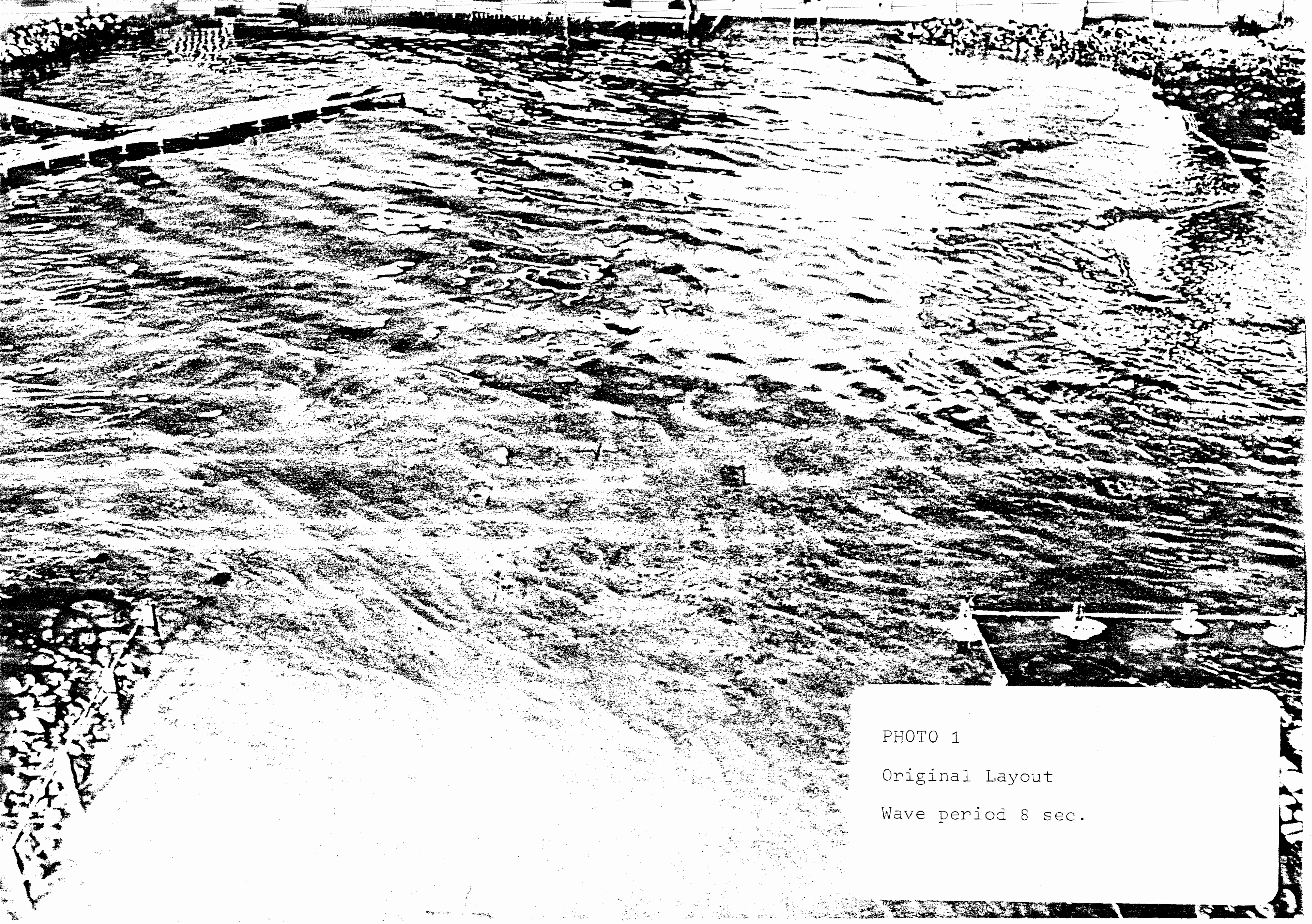


PHOTO 1

Original Layout

Wave period 8 sec.

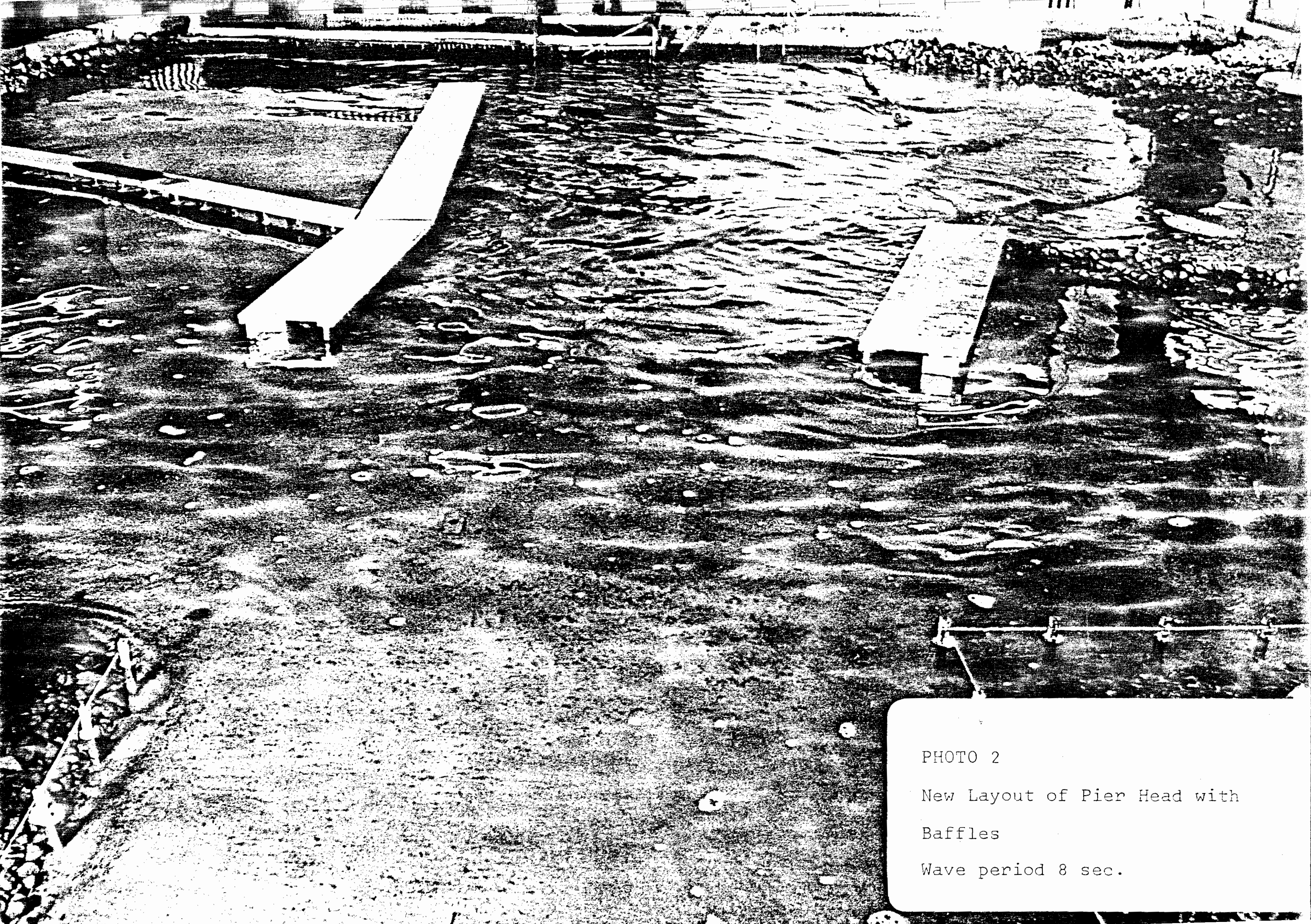


PHOTO 2

New Layout of Pier Head with Baffles

Wave period 8 sec.

PHOTO 3

Original Layout

Wave period 8 sec.

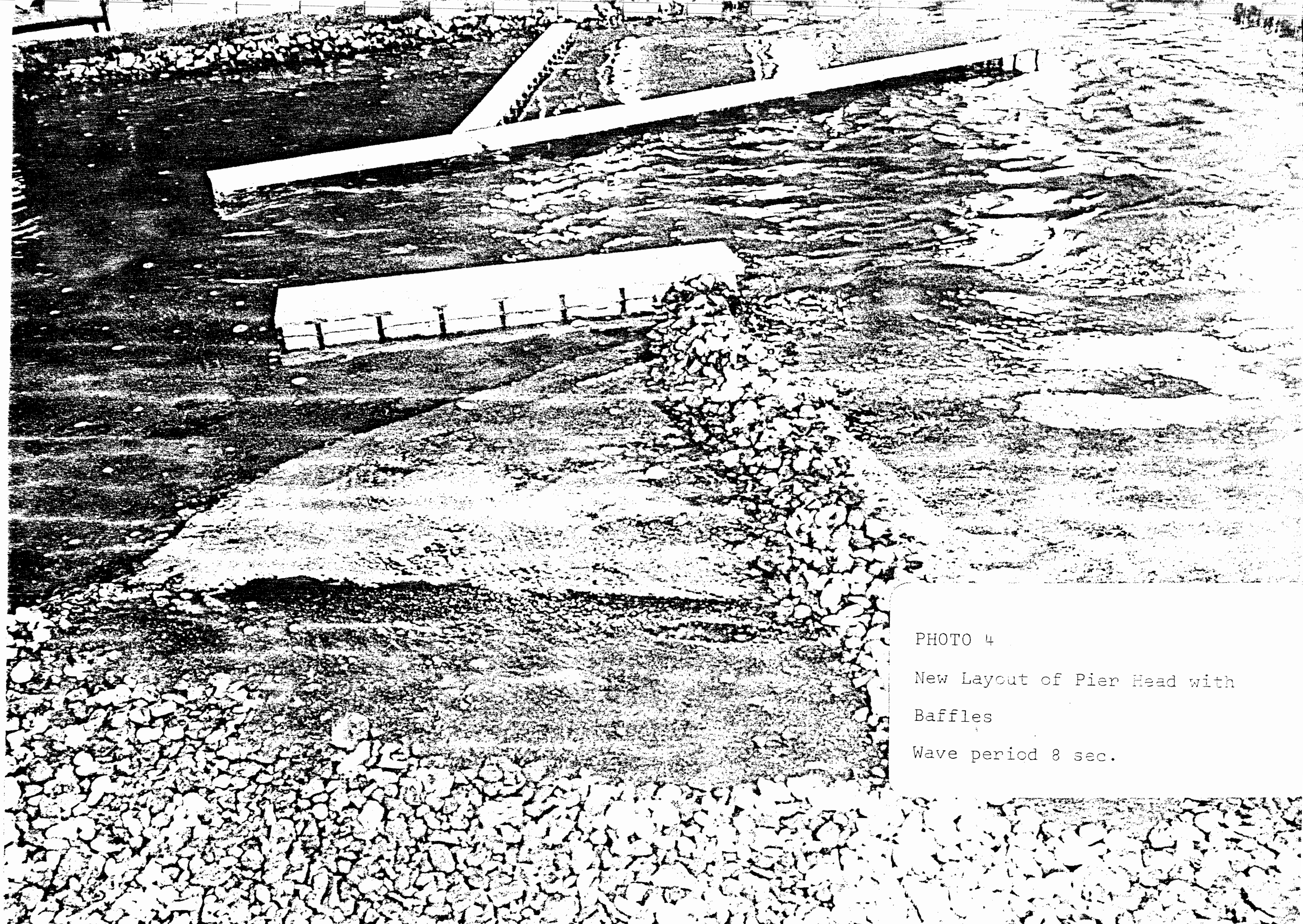


PHOTO 4

New Layout of Pier Head with Baffles

Wave period 8 sec.

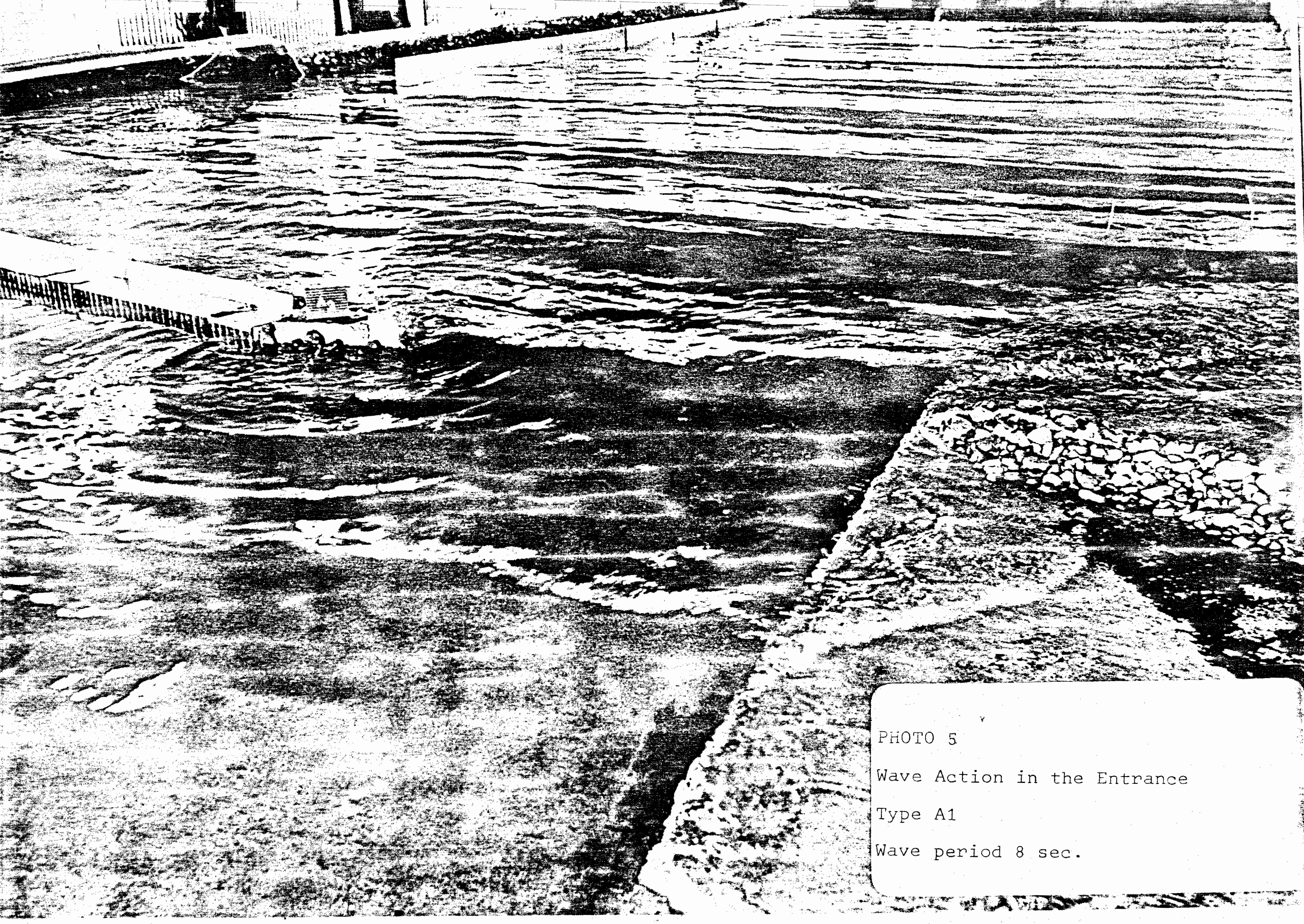


PHOTO 5

Wave Action in the Entrance

Type A1

Wave period 8 sec.

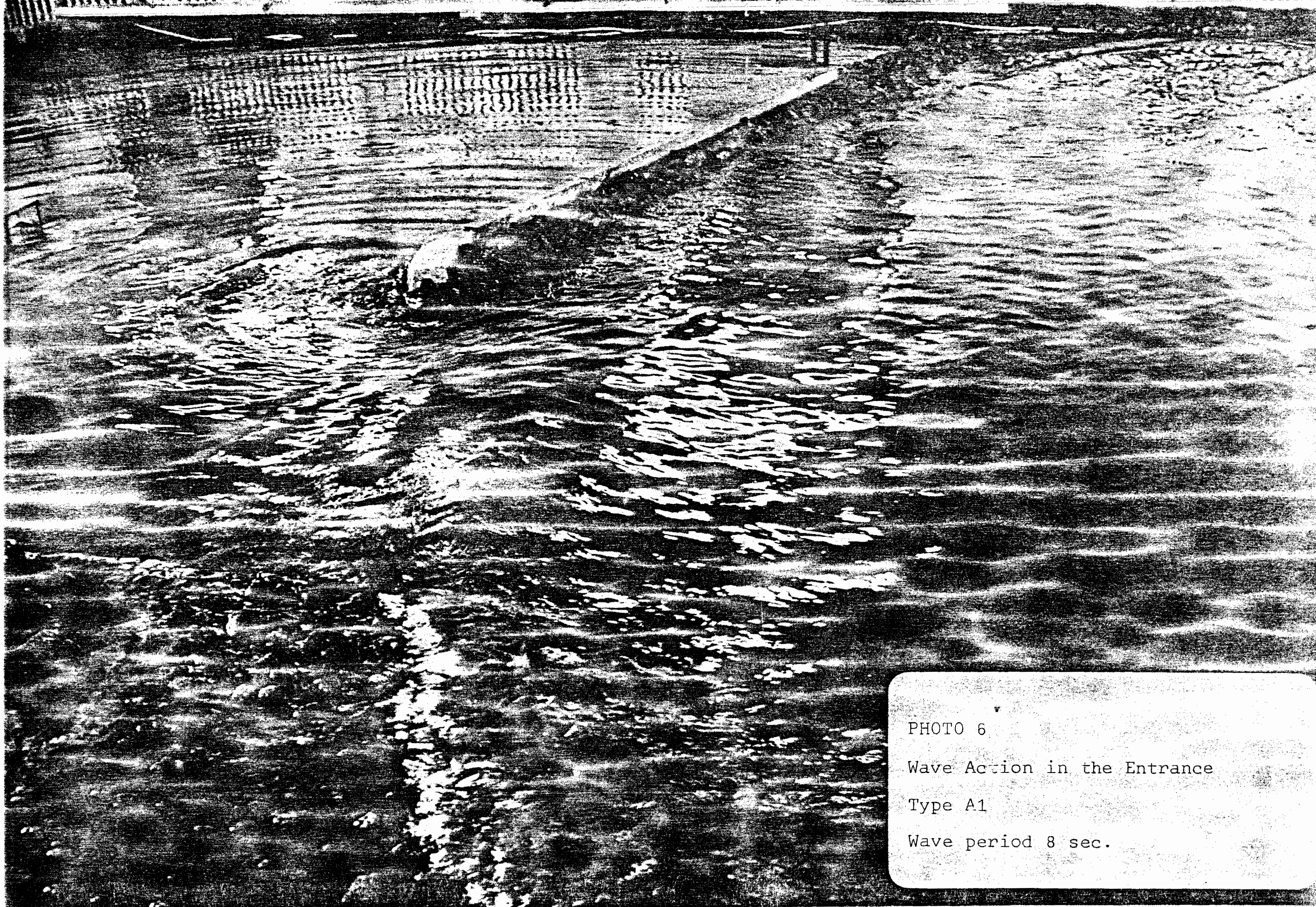


PHOTO 6

Wave Action in the Entrance

Type A1

Wave period 8 sec.

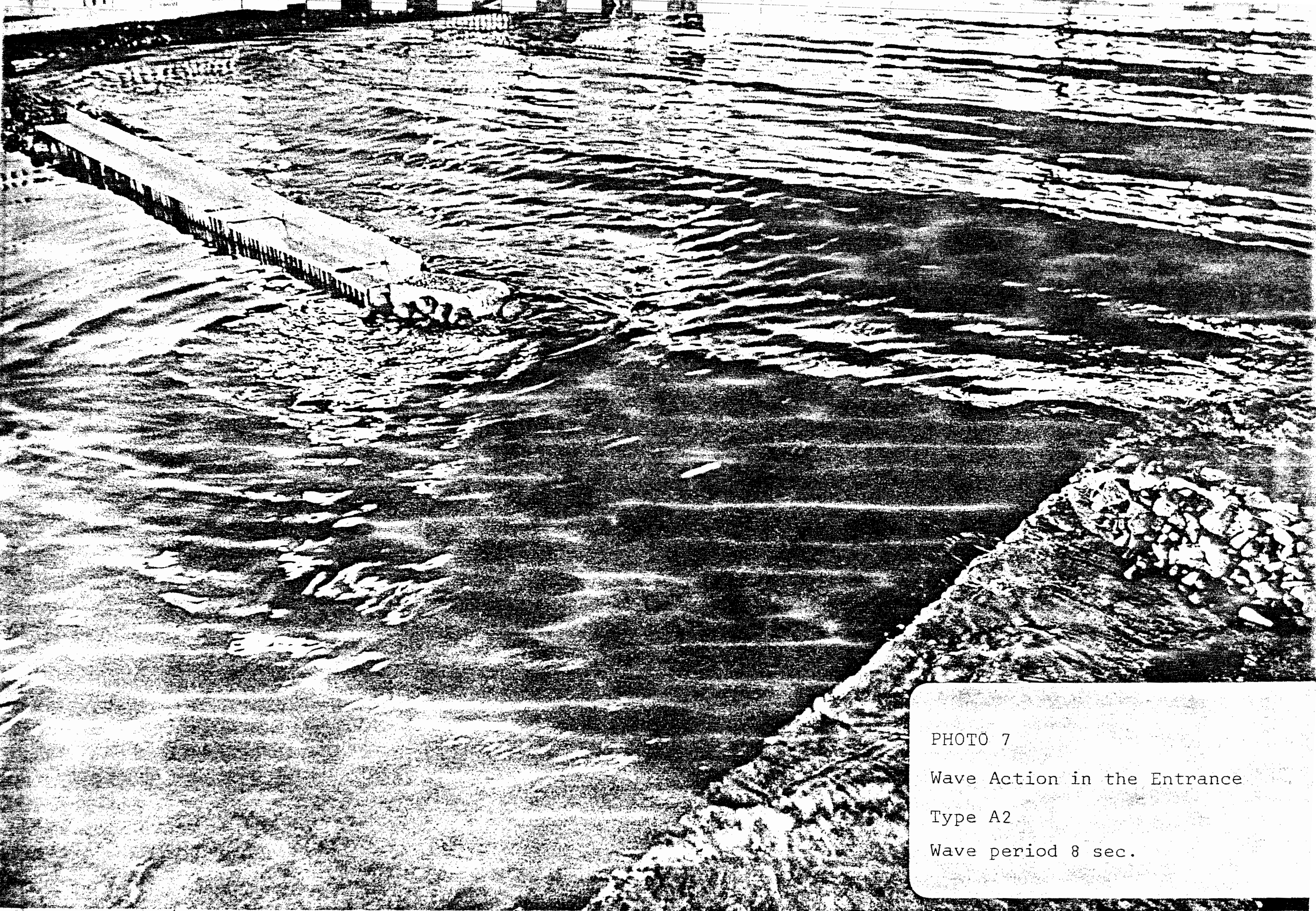


PHOTO 7

Wave Action in the Entrance

Type A2

Wave period 8 sec.

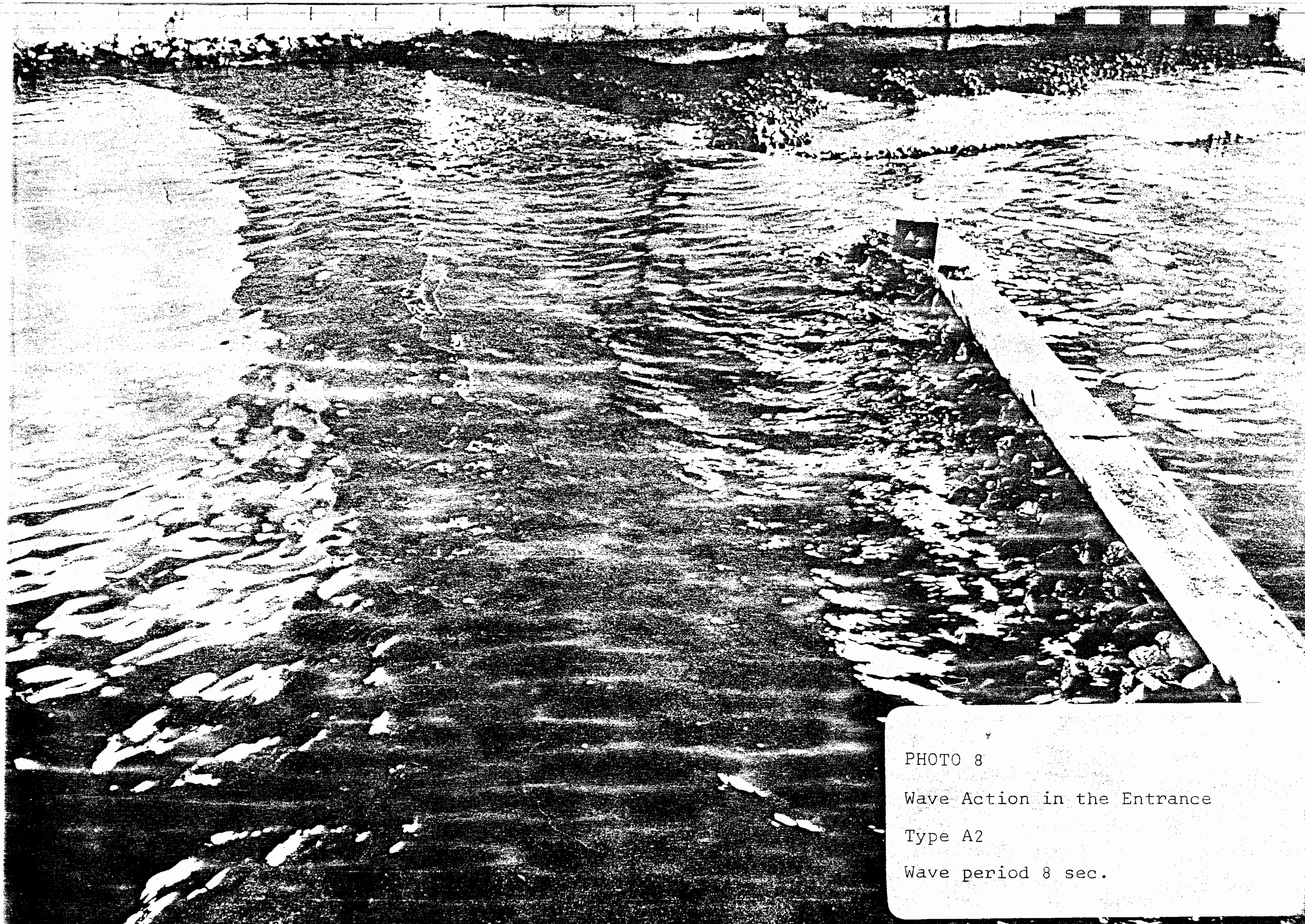


PHOTO 8

Wave Action in the Entrance

Type A2

Wave period 8 sec.

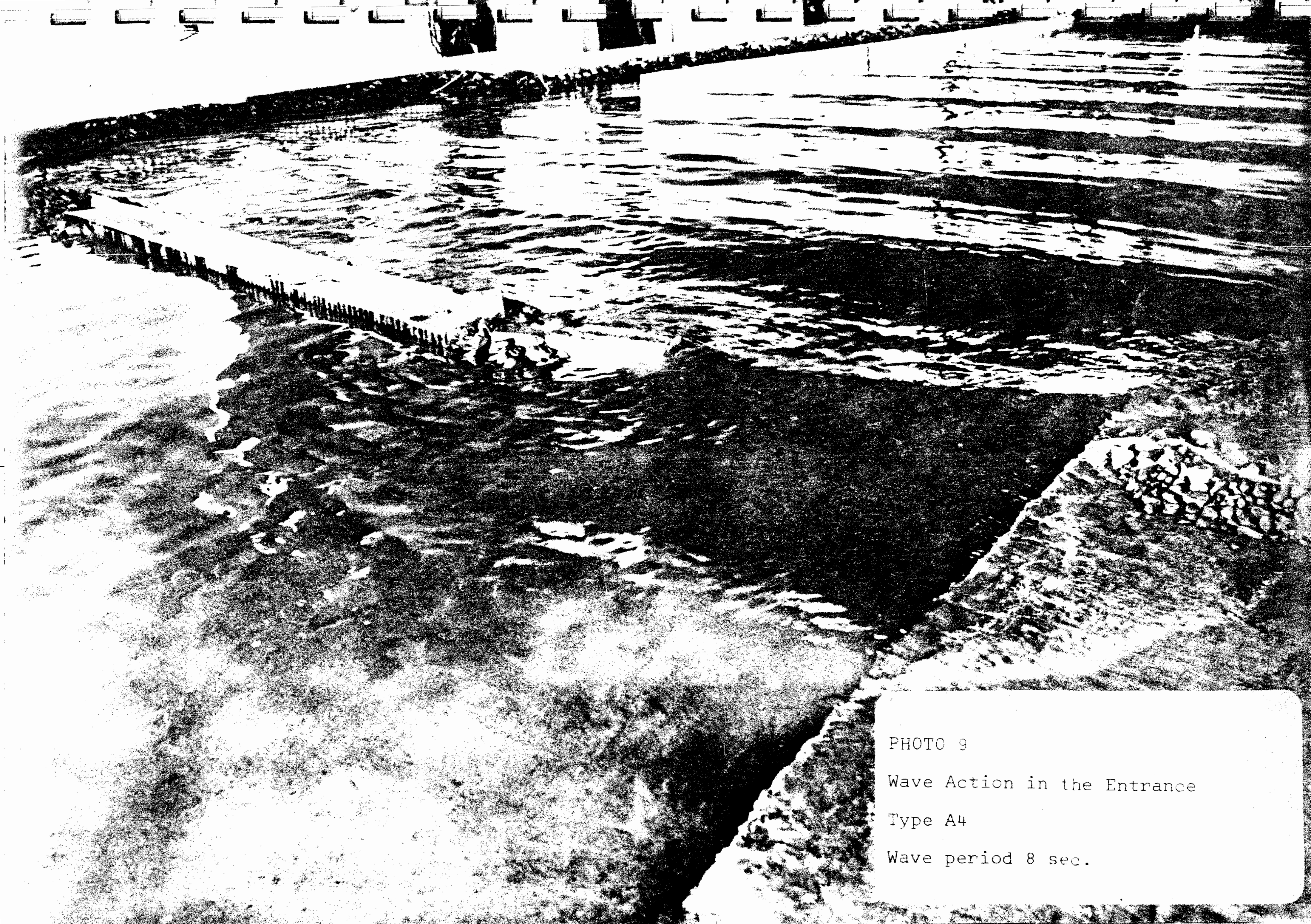


PHOTO 9

Wave Action in the Entrance

Type A4

Wave period 8 sec.

PHOTO 10

Wave Action in the Entrance

Type A4

Wave period 8 sec.

PHOTO 11

Wave Action in the Entrance

Type A4

Wave period 8 sec.

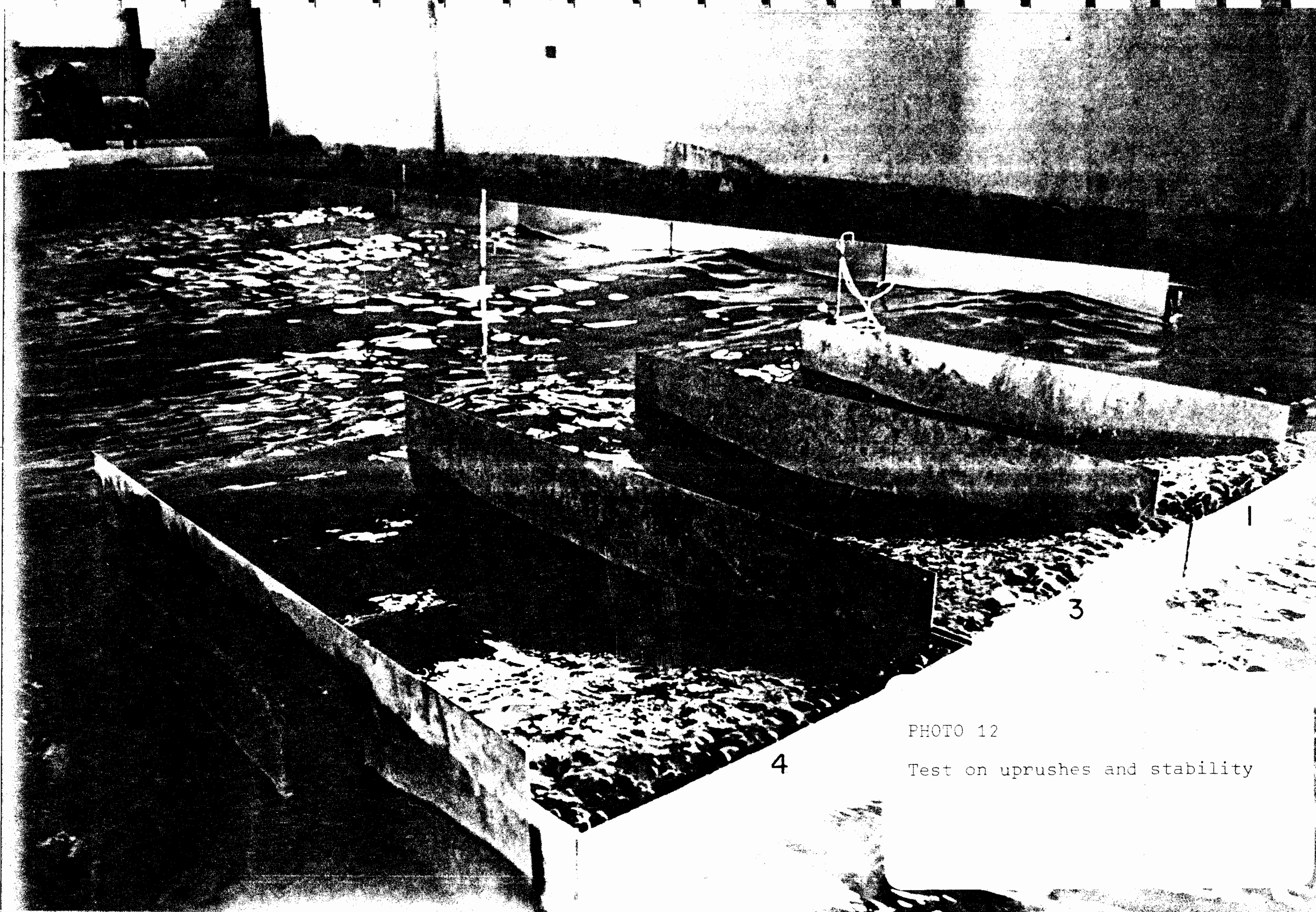


PHOTO 12

Test on uprushes and stability

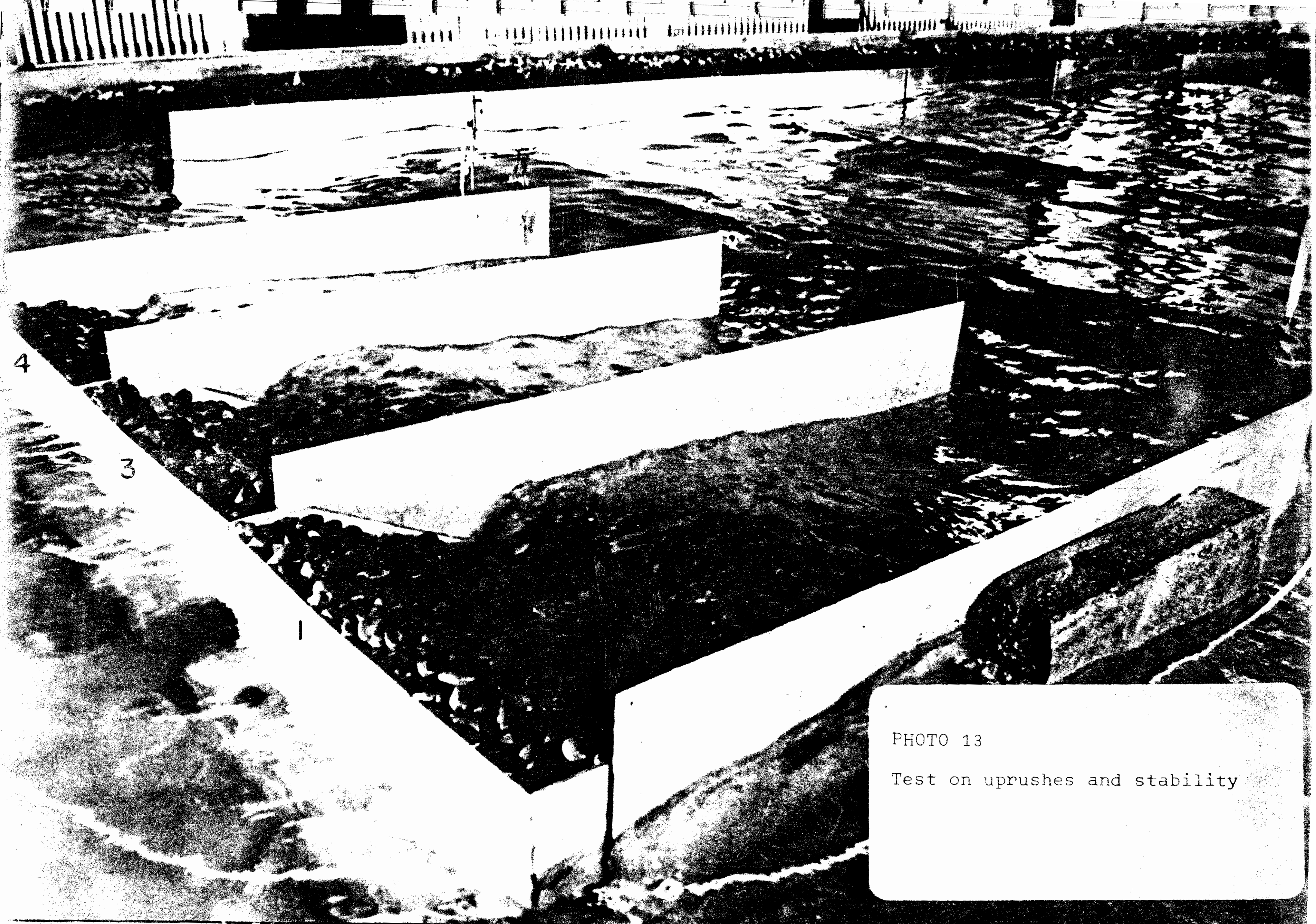


PHOTO 13

Test on uprushes and stability

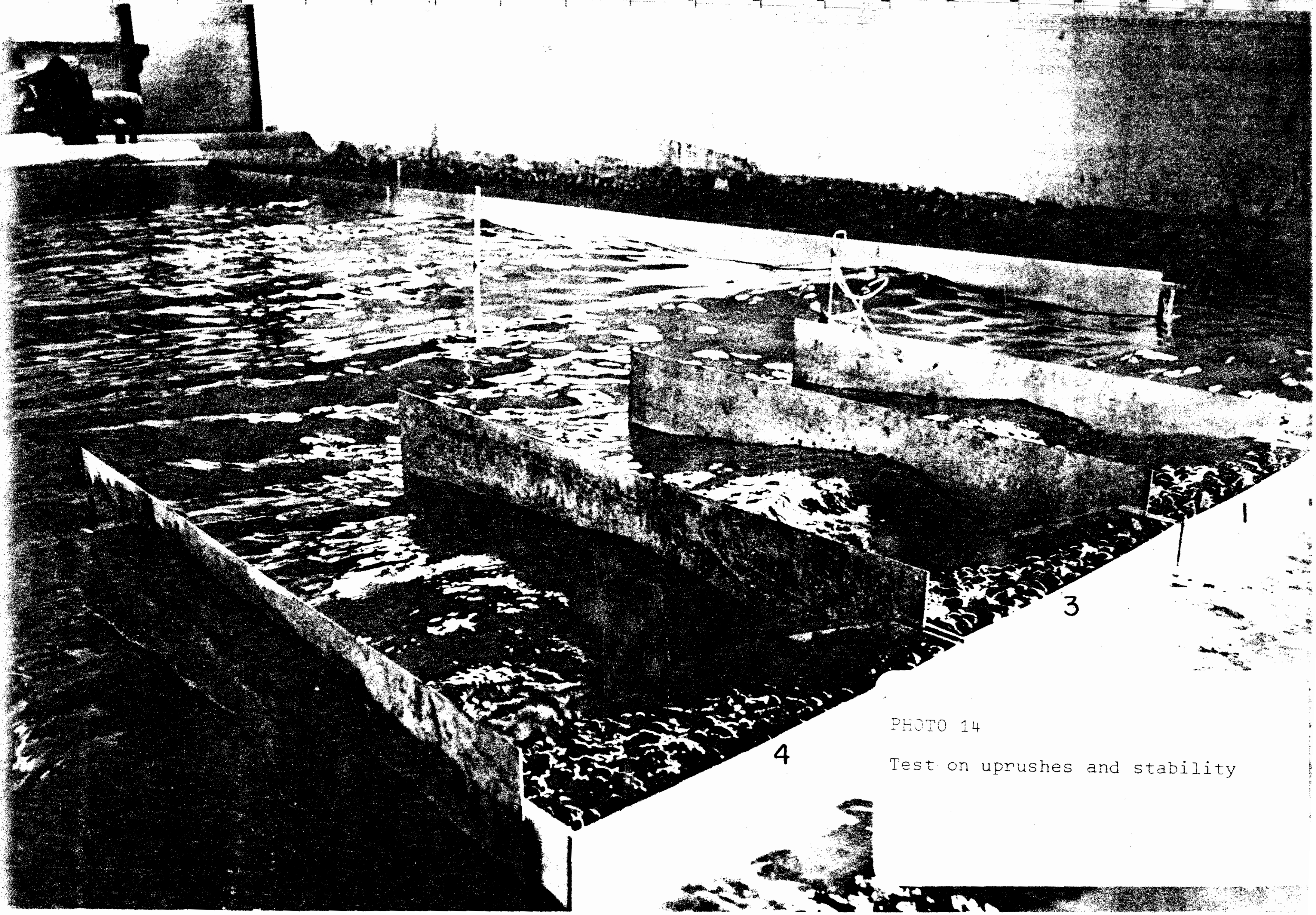


PHOTO 14

Test on uprushes and stability

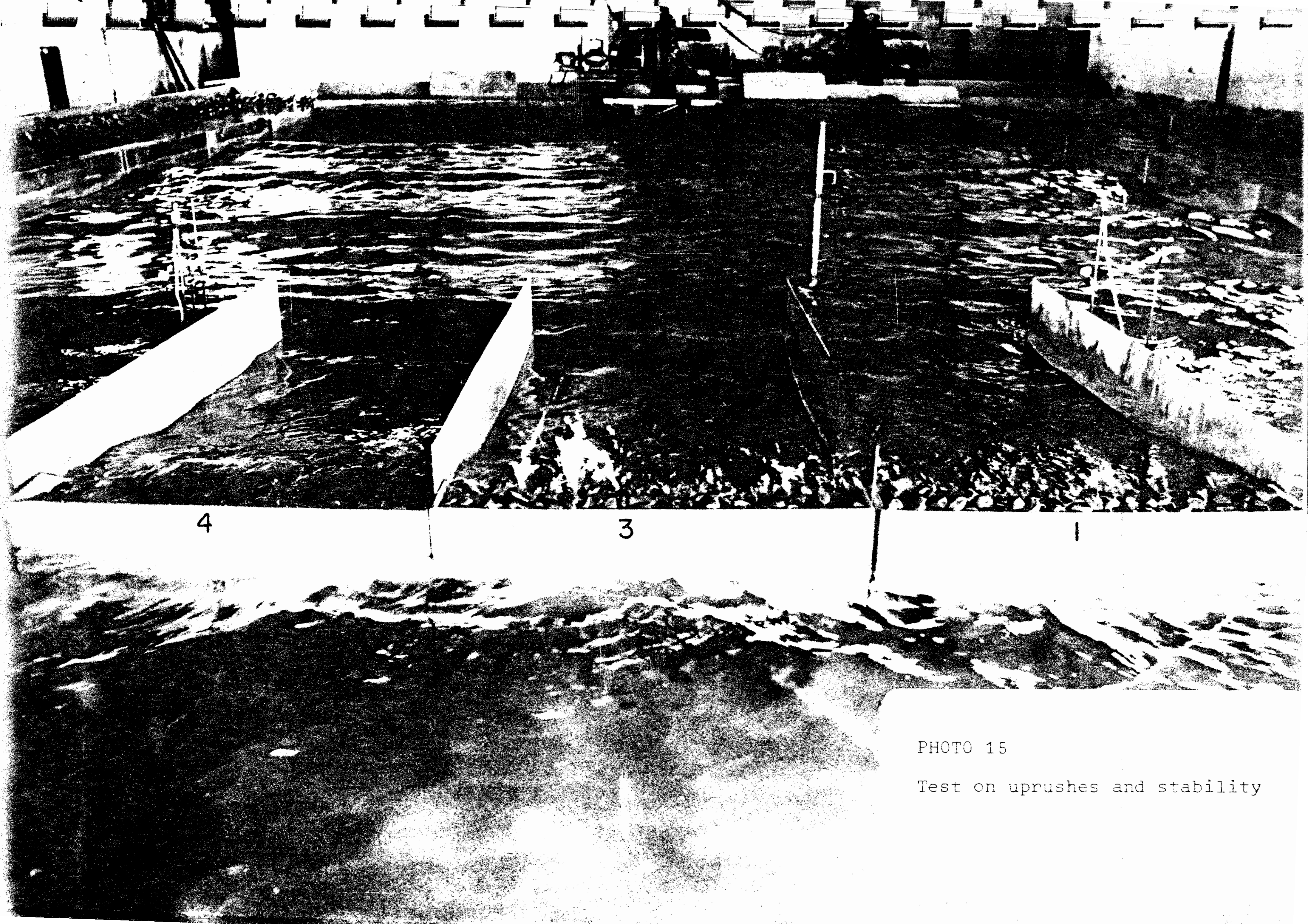


PHOTO 15

Test on uprushes and stability