INLAND WATERS BRANCH

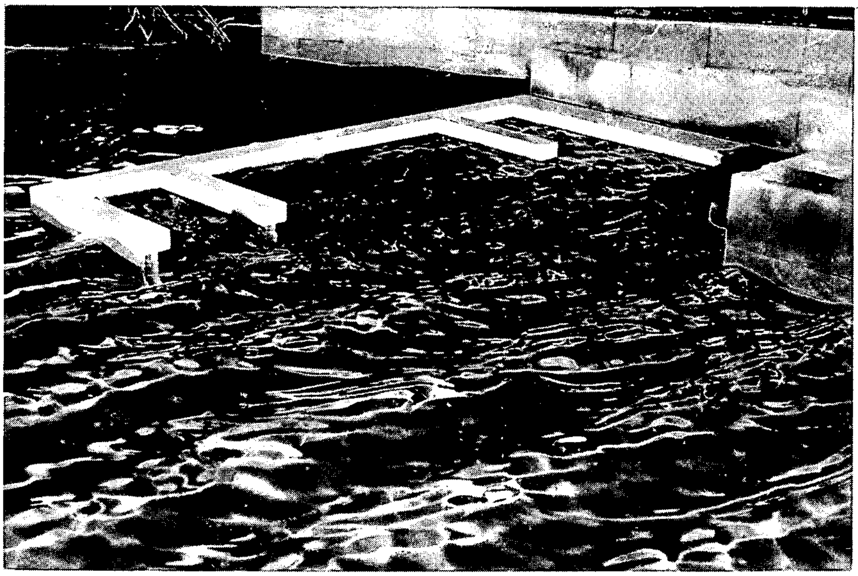
DEPARTMENT OF ENERGY, MINES AND RESOURCES

An Assessment of the Wave Agitation in the Small Boat Basin at the Canada Centre for Inland Waters

(and a Report on a Model Study, by J. Marsalek)

T. M. DICK

TECHNICAL BULLETIN NO.28



Frontispiece - Model with due south waves (Photo by J. Marsalek).

CANADA

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Preface

The model tests were performed at the Coastal Engineering Laboratory of Queen's University by kind permission of Dr. A. Brebner, Head, Department of Civil Engineering. Construction of the model and all of the testing were performed by Mr. J. Marsalek under the general direction of the Hydraulics Subdivision, Department of Energy, Mines and Resources.

Mr. Marsalek's report is included in its entirety as Appendix A to this publication.

Abstract

The geometry of the small boat basin at the CCIW causes multiple reflections of any wave agitation which enters. Consequently, there is a limit to the amount that wave heights can be attenuated by a practical arrangement of additional breakwaters.

The investigation shows that the occurrence of undesirable conditions will probably not exceed four days during the navigation season and that the best scheme will still require special mooring arrangements for the worst storms. It is recommended that a buoy mooring system be installed but additional breakwaters should not be constructed until further experience is gained.

Assessment of the Wave Agitation in the Small Boat Basin at the Canada Centre For Inland Waters

T. M. DICK

1. INTRODUCTION

Small survey boats are moored in a boat basin at the Canada Centre for Inland Waters. The Marine Sciences Branch, the agency responsible for boat operations and maintenance, believes that the level of wave agitation within the boat basin precludes simple moorings for unacceptable periods of time.

The problem, therefore, is to find:

1. The period of time that the boat basin has an unsatisfactory level of wave motion.

2. A method of alleviating wave motion in the boat basin which meets operational and fiscal restraints.

Since the wave motion in the harbour and boat basin is very complex, theoretical models may have large errors. A more accurate method for assessing the wave action for present and proposed structures is probably a scale model study in the laboratory.

Model tests at Queen's University Coastal Engineering Laboratory were conducted by J. Marsalek under the general direction of the Hydraulics Subdivision. Dr. A. Brebner of Queen's University gave permission to use the laboratory for these tests.

2. GENERAL PROCEDURE

Model Tests

Systematic assessment of various schemes was tried out for southerly waves in the laboratory. The results of these tests are given in J. Marsalek's report (Appendix A to this report).

Field Measurements

Direct field measurements of the waves were undertaken by the Tides and Water Levels Section of Marine Sciences Branch. The results of those measurements appear in a separate report (Mackenzie, 1969).

Previous Report

A previous report by the Design Directorate of the Department of Public Works, Donnelly (1969) was consulted and assessed.

Based on available information, the D.P.W. report examined the wind statistics and endeavoured to compute the agitation in the harbour.

3. FREQUENCY OF PROBLEM

Wind Probability

As a beginning it is useful to assess how often the problem is likely to occur. To appreciate the situation, Figure 1 shows the geographical layout of the harbour. It has been observed and can be deduced from Figure 1 that southerly winds will generate waves which will penetrate the harbour.

Waves generated by winds do not follow the wind velocity vector precisely and it is not inappropriate to assume that all winds occurring in the shaded sector may give rise to wave agitation within the harbour. Data were taken from the report of the Department of Public Works (1969), and used to prepare Table 1. Note, however, that the sector should include some proportion, say half, of the wind records for the SW direction and possibly half for the SSE direction.

The latter was omitted from the report of Donnelly (1969) so we have included all of the SW direction to compensate. The error is probably quite small and within the limits of accuracy since we are studying Hamilton Harbour and the records were obtained at Toronto Airport.

It seems from Table 1 that over the navigation season, the wind blows about 18.5% of the time from the southerly sector.

TABLE 1

Hours of Southerly Wind at Toronto Airport (10-Year Average)

Period	Hours	Percent	
Jan., Feb., Mar.	393.6	19	
Apr., May, June	362.7	17	- Navigation Season
July, Aug., Sept.	331.7	20	
Oct., Nov., Dec.	510.0	23	

Thus it follows that since there are one hundred and eighty-three days in the period considered, there will be thirty-five days when the wind comes from the southerly sector as shown on Figure 2.

Another set of data on maximum hourly winds in Hamilton Harbour was supplied by Reid Crowther. These records cover a period of eight years and it was found that *maximum hourly winds* for the period April to September

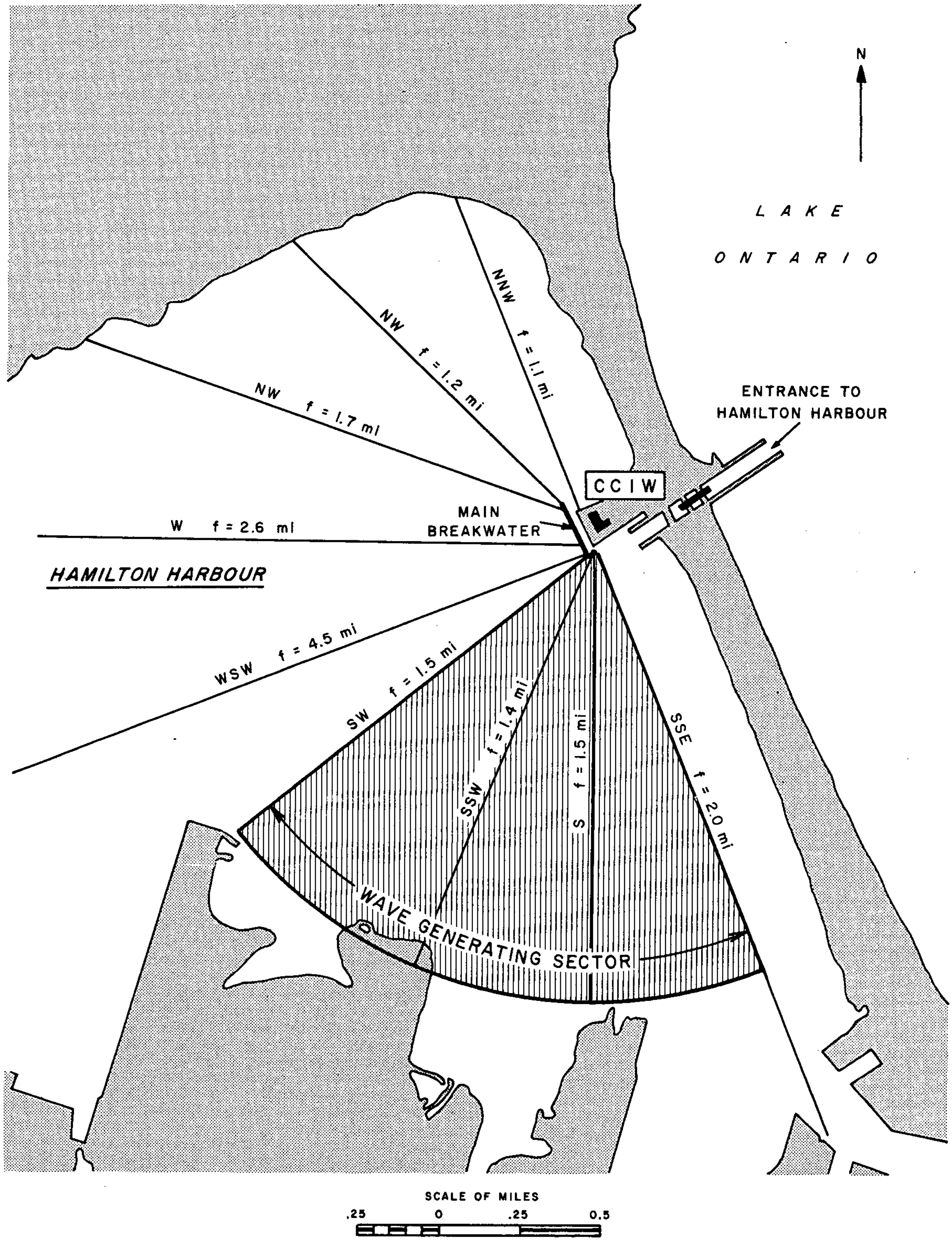


Figure 1. CCIW small boat basin and surrounding area. Shaded area depicts sector producing southerly waves.

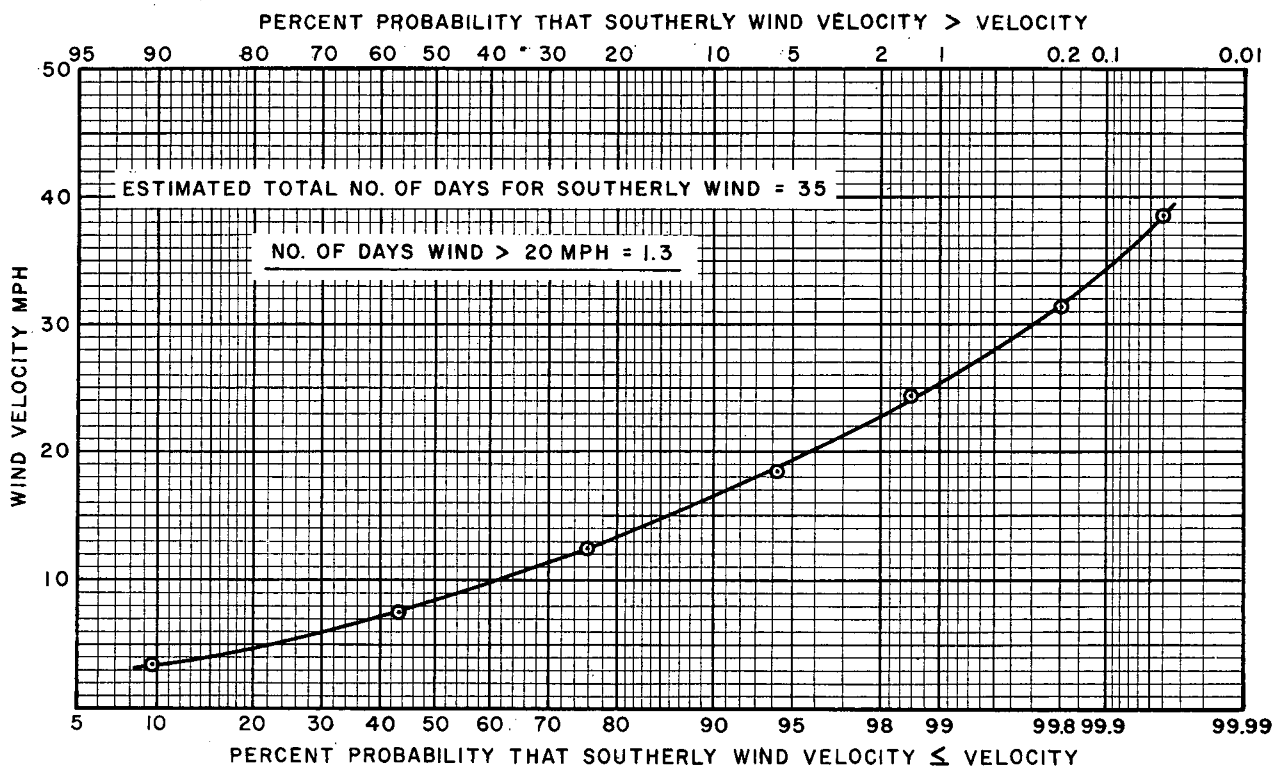


Figure 2. Accumulative probability of southerly winds, April to September inclusive, based on 10 years of record at Toronto Airport.

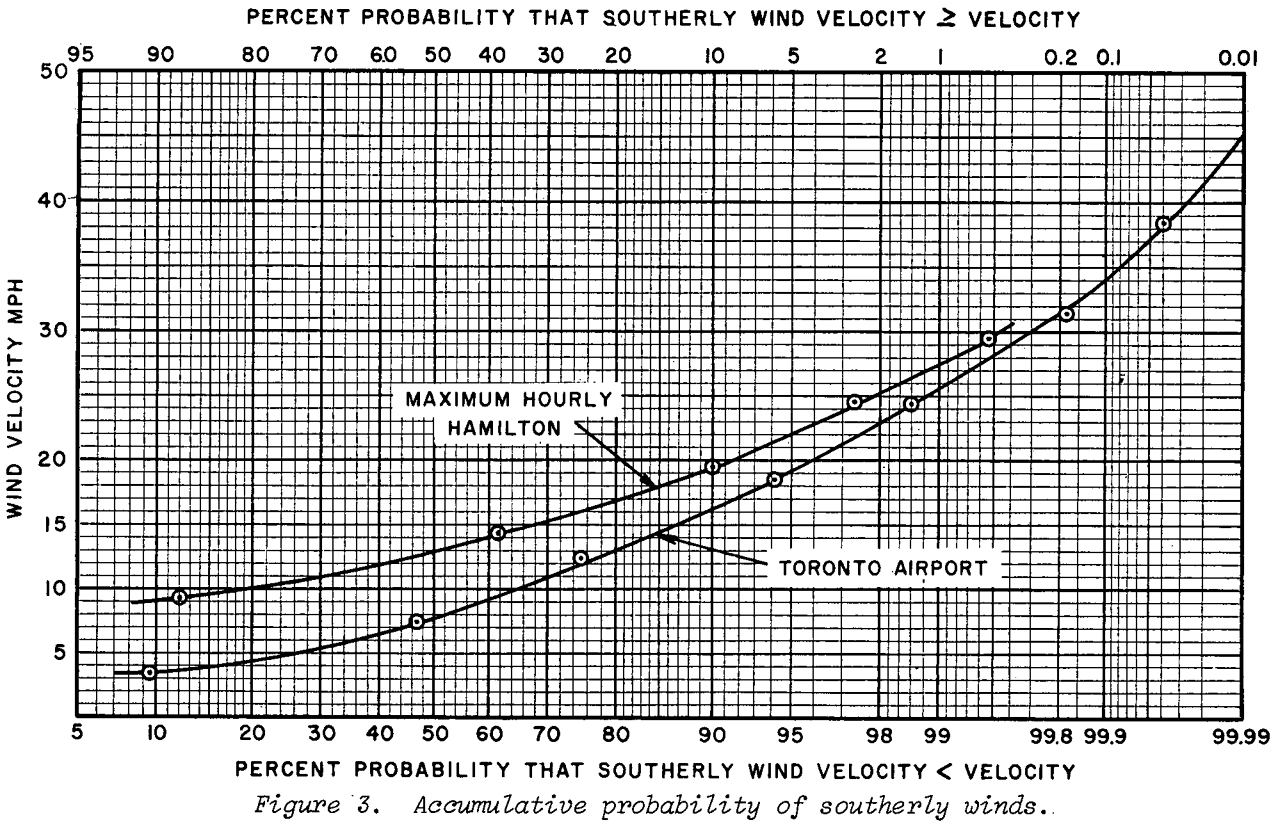


Figure 3. Accumulative probability of southerly winds.

came from the southerly sector an average of thirty-seven days --- or roughly twenty percent of the time.

The distribution functions for both sets of data have been plotted on Figure 3 on normal distribution paper.

Wave Heights

Some of the southerly winds will not be strong enough to generate waves of any consequence. As the next step, it is essential to estimate certain statistical wave parameters for various wind speeds and directions. Here we are fortunate because the wave generating length or fetch is practically identical throughout the southerly sector.

TABLE 2

Wave Forecasting

Wind Directions	Fetch (Miles)	Wave Height H / Wave Period T	Wind Speed (mph) 20	25	30	35	40	45	50
NW	1.2	H_s(ft)	-	1.00	1.40	1.75	2.20	2.60	3.04
		T_s(Sec)	1.70	1.95	2.20	2.40	2.60	2.75	2.85
WNW, SW, SSW, S	1.5	H_s(ft)	-	1.22	1.60	2.00	2.55	3.00	3.30
		T_s(Sec)	1.80	2.15	2.30	2.55	2.77	2.87	3.08
SSE	2.0	H_s(ft)	1.00	1.45	1.80	2.45	3.00	3.25	3.60
		T_s(Sec)	2.00	2.30	2.50	2.75	3.00	3.15	3.30

After Donnelly (1969), Table 2 has been prepared which gives the statistical parameters of Significant Wave Height, H_s, and the Significant Period, T_s, as a function of Wind Velocity. Note that H_s and T_s are statistical averages and both H and T will vary about these averages.

Wave heights in lakes generally have been found to fit the Raleigh Distribution function which shows that once in every two thousand waves, one wave or more will exceed 2 x H_s. Wave periods have a limited range, but their distribution is not so well defined. However, the significant period represents the period of the higher and dominant waves.

4. MODEL AND REAL WAVES

Scale

The model was built to a scale of 1:36 which sets the time and velocity scale at 1:6. Thus, one second in model time represents six seconds in the prototype or real time. This scale was chosen primarily because of space limitations in the laboratory.

Model and Prototype Sea State

Waves in the prototype, or real sea state, are obviously neither constant in height nor in frequency. Unfortunately, the exact duplication of the real sea state in a three-dimensional model is not feasible, and one must undertake the over simplified approach of constant wave height and frequency. Interpretation of model results, especially in harbour agitation studies, is subject to considerable error. In these tests, three frequencies were selected to cover the spectral range of the prototype, and the performance of each model was compared with another. Qualitative predictions, however, are very difficult both because of the simplified wave train and because of scale effects.

Model Wave Attenuation (Scale effect)

Waves lose energy by doing work to overcome internal and boundary viscous friction and to extend the surface films. Breaking, when it occurs, also represents a transfer of energy from the wave motion into turbulence with subsequent decay by internal friction. The rate of energy loss does not follow the Froude scale law, and it is thus necessary to calculate the percentage loss in wave height in the model, add this to the measurements and then bring the results to prototype values. We have chosen to use the model results by comparing models directly with one another and then applying the correction to the residual schemes.

Calculation of a correction can only be approximate since the waves reflect from the wharf and breakwater. Therefore, as a rough estimate we shall consider the attenuation as being equivalent to a wave travelling directly down the harbour area to the boat basin.

The greatest energy loss is caused by work done on a surface film. The theories deal with an ideal film but in practice, owing to dust and other impurities, the properties of the surface film can vary widely. The losses due to surface film effects are greater than those due to the internal friction effects by an order of two magnitudes. For our estimates, we shall calculate losses by using the surface film theory of Philips.

The equations are:

$$H = H_1 \, e^{-Sx} = H_1 \times F \tag{1}$$

$$S = \frac{1}{2\sqrt{2}} \frac{(\upsilon)^{\frac{1}{2}}}{\sigma} \frac{k^2}{\mathrm{TANH}(kh)} \tag{2}$$

Where H = wave height at distance, x, from origin ft.

H_1 = wave height at origin ft.

S = attenuation factor 1/ft.

υ = Kinematic viscosity ft^2/sec.

σ = frequency $2\pi/T$ 1/sec.

k = wave number $2\pi/L$ ft.

h = depth of water ft.

For $\upsilon = 1.1 \times 10^{-5}$, various values of H/H_1 were found for a point halfway along the wharf and for the boat basin. The results are shown in Table 3.

TABLE 3

Wave Attenuation by Surface Film

MODEL

T sec.	S	H/H_1	H/H_1
		(x = 17 ft.)	x = 34 ft.
0.3	.04680	.451	.204
0.4	.01720	.747	.556
0.5	.00860	.866	.747

PROTOTYPE (Assumes no wind energy supplied)

T sec.	S	H/H_1	H/H_1
		x = 600 ft.	x = 1200 ft.
1.8	.0000894	.951	.904
2.4	.0000324	.980	.962
3.0	.0000148	.999	.987

The calculations are not very reassuring. We find that for the higher frequency waves (T = 0.3 sec.) used in the model, the incident wave height at the small boat basin will only be about one-fifth of the height expected. In the prototype, the wave will be about 0.9 of its full height in the absence of added wind energy. Thus, the scale ratio is not constant throughout the model.

From equation 1, the incident wave height is given by

$$(H^1{}_m) = \frac{H^1{}_m}{F_m}$$

where $H^1{}_m$ = observed wave height

F_m = attenuation factor

It follows that

$$(H_1)_p = (H^1{}_m) \times (\text{Scale}) = \frac{H^1{}_m}{F_m} (\text{Scale})$$

Normally in model studies the factor F can be neglected but in these tests any quantitative comparisons must take the factor F into account.

For convenience, values of $^1/F$ have been listed in Table 4 for two locations in the model. It should not be forgotten that these values are not accurate and simply indicate the magnitude of the correction.

TABLE 4

Wave Height Attenuation Factor Ratios for Converting Model Wave Height Measurements to Prototype Heights.

Tm	Tp	$x_m = 17$ $x_p = 600$	$x_m = 34$ $x_p = 1200$
0.3	1.8	2.22	4.90
0.4	2.4	1.34	1.79
0.5	3.0	1.15	1.32

For example if a 0.3 sec. wave in the model was measured as 0.01 ft. high at x = 34 ft. then the prototype is:

$H_p = 0.01 \times 4.90 \times 36 = 1.76$ ft. and not

$H_p = 36 \times 0.01 = 0.36$ ft.

5. MODEL TESTS

General Observations

The results of the model tests can be found in J. Marsalek's report which is appended. Visual observations of the basin and of the model seemed to indicate that wave action was aggravated by reflections from the basin walls. The reflection coefficient for the marina dock sides will depend on the amount of submergence of the curtainwall, and this varies with the lake stage in the prototype. Wave reflections seemed to be a major problem in the boat basin from direct visual observation. It is also possible that some of the basins were capable of resonating and causing standing waves to grow until the rate of energy dissipation balances that received from the incident waves.

Resonant Frequencies

The resonant mode for a basin and any theoretical analysis can be very complex, and for this case is probably not worth the effort. Consequently we shall use the simple approach of calculating the periods of possible resonant modes and compare them with the range of the incident periods.

Possible modes are shown in Figure 4 and the corresponding periods are listed in Table 5. The fundamental period was calculated from

$$T = 2L/\sqrt{(gh)}$$

where h = depth

L = wave length

TABLE 5

Standing Wave Periods for Various Modes

Depth of Water 12 ft.

Mode	Typical Length L Ft.	Fundamental Period T secs	Harmonics T/2	Harmonics T/3
A	40	4.1	2.1	1.3
B	124	12.6	6.3	4.2
C	90	9.2	4.6	3.5
D	130	12.3	6.2	4.1
E	120	12.2	6.1	4.1
F	40	4.1	2.1	1.4
G	120	12.2	6.1	4.1
H	124	12.6	6.3	4.2
J	183	18.6	9.3	6.3

The periods are generally too large for resonance because the incident wave periods tend to be less than three seconds except for bad storms. Resonance, therefore, is not considered a major factor and ordinary wave reflection is likely to be the main contributor to the wave agitation.

Measure of Wave Agitation

Methods of describing the general level of wave agitation in a harbour or other areas are not easily defined. In open water, wind-wave energy has a statistically average wave height which describes the general level of wave agitation. But within the basin there is unequal distribution of wave energy; moreover, the spatial distribution in the model could change quite rapidly as the incoming frequency is varied by small amounts because of slightly uneven rotation at the wave generator.

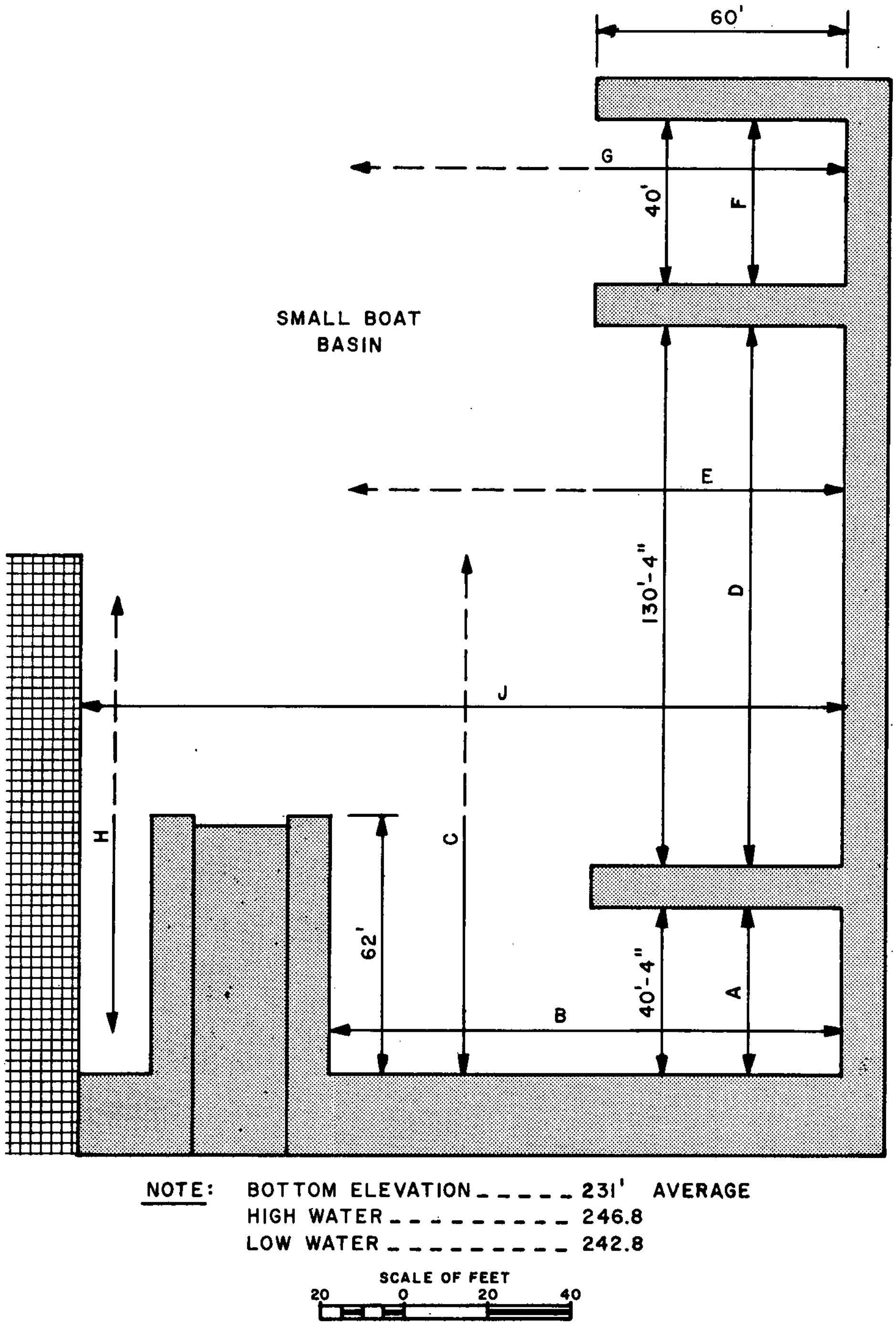


Figure 4. Possible standing wave modes in CCIW small boat basin.

It was decided, therefore, to plot the highest wave occurring at various points in the harbour basin, and make it dimensionless by dividing the measured height by the incident wave emanating from the generator. This ratio was defined by Marsalek as H_{max}/H_{inc}.

The relative effectiveness of the various schemes was obtained by comparing the plot of this ratio for various locations in the basin, and for one line along the main wharf face.

6. COMPARISON OF REMEDIAL SCHEMES

Factors Involved

Comparison of the relative merits of the proposed remedial schemes is complicated by the number of factors which need to be taken into account. In our evaluation, a number of design objectives were kept in mind and then each was given a weight of relative importance.

The objectives are listed below:

Objective 1. To reduce the level of wave agitation in the small boat basin.

Objective 2. To keep the capital outlay low.

Objective 3. To avoid interfering with the navigation of ships as they enter or leave the CCIW wharf.

Objective 4. To avoid increasing the possibility of ice forming in the main dock area.

Objective 5. To avoid the build-up of ice by spray on top of the main wharf.

For convenience these objectives are tabulated, in paraphrased form, in Table 6. The various remedial schemes were then considered as to the extent each met the objectives, and a weighted point score was assigned. These are listed in Table 6.

If the scoring is relatively correct, then the scheme with the highest points comes closest to meeting the stated overall objectives. In this case we find that the three best schemes are,

No. 1 ________ scheme 11

No. 2 ________ scheme 7

No. 3 ________ scheme 1

It is perhaps significant that the present design (scheme 1) scores third indicating that the original layout goes a long way towards meeting the objectives.

Checking back into the various schemes we see that scheme 11, on Marsalek's Figure A-6 is a breakwater 141 feet long inclined at an angle to the main wharf and placed just to the south of the small boat basin. The present situation (scheme 1) and scheme 11 are compared directly in Figure 5, which shows clearly the benefits obtained by the addition of the breakwater.

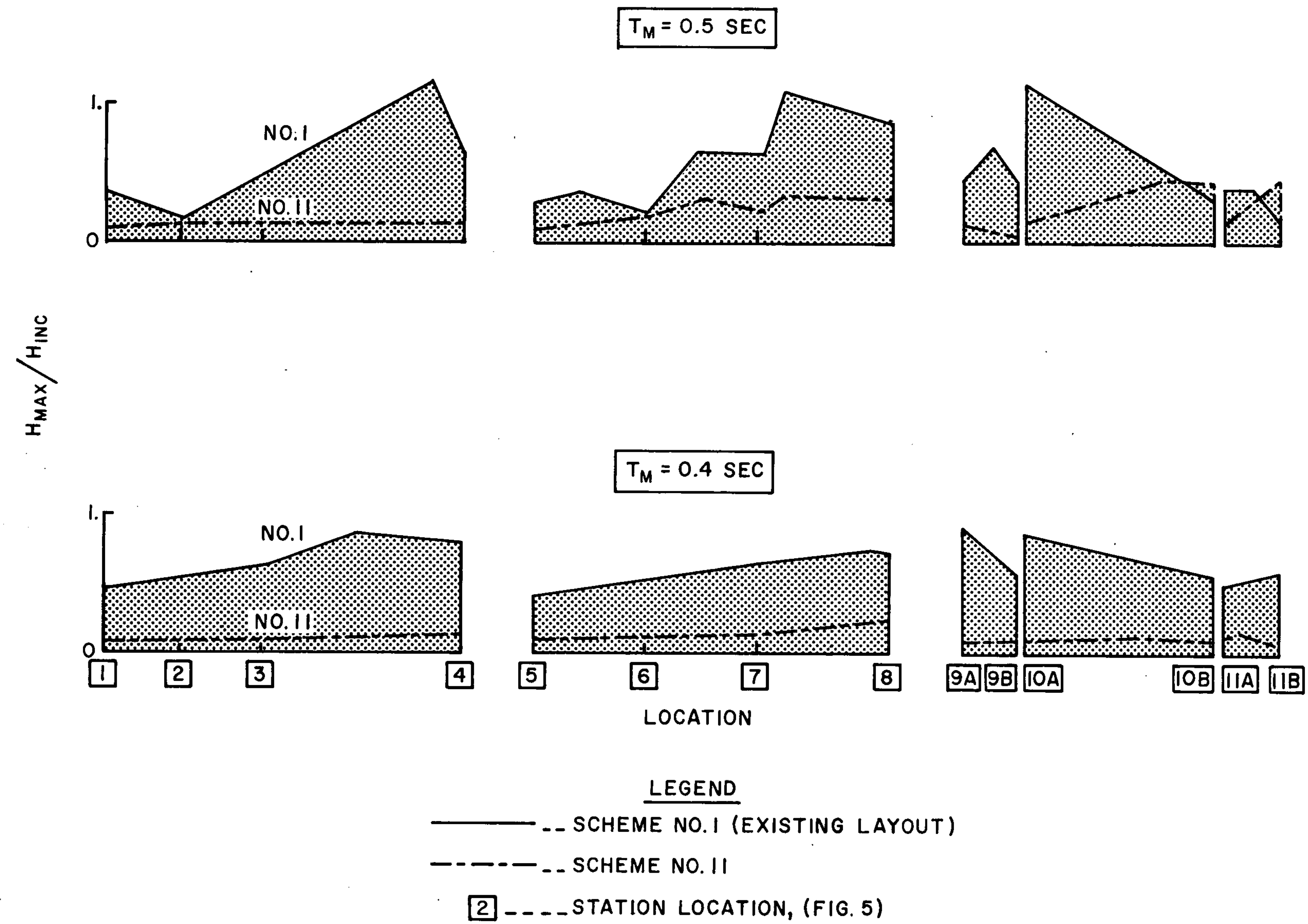


Figure 5. Comparison of wave height ratios inside CCIW small boat basin for Scheme 1 and Scheme 11.

TABLE 6

Comparison of Remedial Schemes by Weighted Points

Objectives	Weight	Scheme Number							
		1	6	7	8	9	10	11	12
To reduce waves in the small boat basin	25	0	75	250	50	100	225	200	225
To keep capital costs down	25	250	150	100	225	150	50	200	25
To avoid interference with navigation	15	150	150	150	120	90	30	150	105
To avoid extra formation of ice	10	100	100	100	80	40	20	100	0
To avoid formation of ice by spray on dock	10	40	20	20	50	70	90	40	100
Total		540	495	620	525	450	415	690	455
Rank		3		2				1	

Total points available 850.

Further Evaluation of Scheme 11

The model tests were run using solid vertical wall breakwaters.

In practice, the additional breakwater could be constructed in several ways. For example:

1. Floating breakwater.
2. Thin wall set on piles as for main breakwater.
3. Timber or concrete crib.
4. Sheet pile caisson.
5. Rubble mound.

It is considered that only a fixed structure with zero transmission coefficient would provide suitable wave reduction in the basin and alternatives 1 and 2 are not considered a good idea. The weakest hydrodynamic aspect of scheme 11 is the reflection of waves from the southerly face, which causes higher waves along the main wharf and the possibility of increased icing in winter. Hence, it is quite desirable to have a structure with a low reflection coefficient. This suggests a rubble mound, or possibly a perforated face breakwater. The design of the latter would require further consideration but is considered to be a better solution than the rubble mound since it would also provide useful.wharf space.

Benefits from Scheme 11

As shown earlier, there are between thirty-five and thirty-seven days when winds will produce waves from the southerly sector. However, not all of these waves warrant concern since some are well within the acceptable level of agitation in the boat basin. The exact point where wave agitation becomes unacceptable is not well defined.

We propose for this situation to adopt the assumption that until the dominant or significant wave period equals or exceeds two seconds, there exists not much more than an acceptable chop. After the two-second period, the wave length is about twenty feet and reaching the same magnitude as the launch dimensions. Furthermore, visual observation confirmed that waves with periods of less than two seconds did not produce serious agitation in the basin.

As shown in Figure 2, a wind velocity of 20 mph produces waves with periods of just under two seconds. Thus, we shall adopt a 20 mph wind as the cut-off point for wave agitation.

From Figure 3 (or Marsalek's Figure A-3) it is evident that on the average a wind equal to or greater than 20 mph occurs during 3.3 days in the navigation season. Rounding this off, it seems that at the most, four days respite would be gained from undesirable wave conditions if scheme 11 were built. During the remainder of the time, wave agitation within the basin should be no more severe than one might expect in a small boat basin.

None of the schemes provides absolute calm in the boat basin. For severe storms with a significant wave period close to three seconds, the wave ratio for scheme 11 is,

$$(H_{max}/H_{inc}) \text{ Model} = 0.3$$

In the prototype, application of the attenuation factor gives

(see Table 4) $(H_{max}/H_{inc}) \text{ PROTO} = 0.3 \times 1.3 = 0.4$

Thus, even with the breakwater in place, the height of the wave agitation, although greatly reduced, will remain at 0.4 of the incident wave height. For wave periods of 3 seconds, waves up to 4.5 feet are likely and thus waves up to 1.8 feet are still possible within the boat basin.

One concludes, therefore, that for severe storms, boats will still need protective mooring systems even if the breakwater was built. It follows, therefore, that if mooring systems will be required in any event, and if properly installed and used, then the moorings will protect the boats even in the absence of a new breakwater.

7. CONCLUSIONS

1. Strong southerly winds will cause problems for four days or so during the navigation season. The maximum hourly wind from the southerly sector was thirty mph over a period of eight years.

2. A breakwater cannot provide "mill pond" conditions in the basin during storms, but substantial reductions in wave action are possible. Wave heights are at present amplified by reflections from the basin wharf walls. These reflections will vary with lake stage, being maximum at higher stages.

3. The "best" remedial scheme which comes closest to meeting design objectives is a breakwater not less than one hundred and forty-one feet long, located as shown by scheme 11 in Marsalek's report.

4. Protective mooring systems will still be required for use in severe storms even with the protection afforded by a new breakwater.

8. RECOMMENDATIONS

The construction of a breakwater one hundred and forty-one feet long will not absolve the need for good mooring practices. Since bad weather can strike without warning, and since the geometry of the harbour encourages multiple reflections, wave energy can approach the boat basin from several directions. Consequently, although the breakwater causes considerable attenuation of wave heights, conditions could arise necessitating proper mooring. Since this study was initiated, further information indicates that long swells passing through the Hamilton Harbour entrance from the west can probably set up disturbances in the harbour. Long waves like these can cause considerable motion in moored boats, and partial breakwaters on the basin piers will give no protection against these longer period waves.

It seems prudent, therefore, to install buoys and develop standard practices for mooring boats to protect them during severe or unusual conditions.

We therefore recommend that,

1. An additional breakwater not be constructed at this time.

2. A good mooring buoy system be installed and used.

3. Records of conditions in the harbour and basin be kept, with emphasis on the amount of time the basin becomes untenable for launches lying alongside the dock. The situation should be reviewed in three years' time.

Model Study of Wave Agitation in the CCIW Small Boat Basin

J. MARSALEK

1. INTRODUCTION

The Department of Energy, Mines and Resources reported excessive wave agitation in the small boat basin at the Canada Centre for Inland Waters (CCIW).

The main offshore breakwater was designed as a vertical rigid barrier extending to about 20 feet below water level; however, it does not protect the boat basin sufficiently against the waves which come from the SW, S and SE directions, Figure A-1, and causes the excessive wave agitation.

An analytical assessment of the extent of wave agitation in the boat basin was given by Donnelly (1969) for the present situation as well as for two new layouts, which would be created by construction of two additional breakwaters.

Due to the complicated wave conditions, the Hydraulics Subdivision of the Department of Energy, Mines and Resources undertook to study the wave agitation problem in a model constructed and operated in the Coastal Engineering Laboratory of Queen's University, Kingston.

2. FIELD DATA

This model study of the boat basin at the CCIW was prepared on the basis of the following field data and material:

a) Set of drawings of the boat basin, the offshore breakwater, and the main wharf and surroundings.

b) Dredging soundings of the area under consideration.

c) Maximum hourly wind abstracts from Hamilton Harbour Station (1959-1967).

Other field data and information were available from Donnelly (1969).

3. WAVE CONDITIONS

Wave conditions were derived from the wind records and fetches by using the method contained in the report of the U.S. Army Coastal Engineering Research Centre (1966).

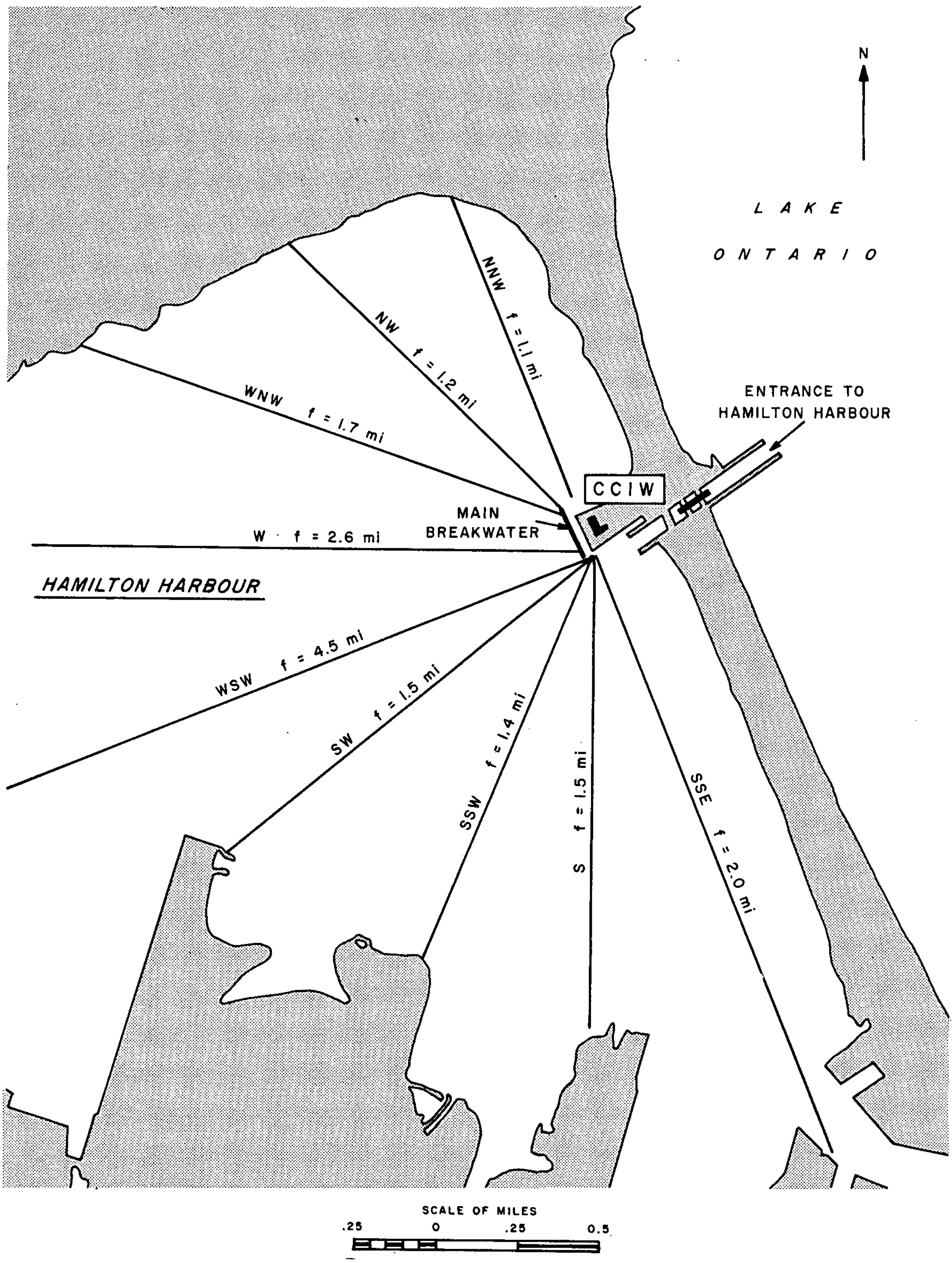


Figure A-1. CCIW small boat basin and surrounding area.

Two types of wind data were available:

a) Wind frequencies expressed as hours per average month (Donnelly 1969). These data were recorded at the Toronto International Airport Station over a period of 10 years.

b) Records of daily maximum hourly winds at Hamilton Harbour Station for the period 1958-1967, submitted by Reid Crowther.

The wind directions which cause trouble were determined. It follows from Figure A-1, that the winds which cause concern come from the SW, S and SE. Wind data for these directions were analyzed and the corresponding probabilities of occurrence were determined.

Toronto Data

Since wind-wave directions are not precisely identical to the corresponding wind directions, and any wind coming from the sector SW-SE could cause trouble, it was decided to sum the frequencies of all winds coming from the sector SW-SE. In this case, only wind frequencies for the directions S, SSW and SW were summed, because only these data are presented in Donnelly (1969). This sum (winds from S, SSW and SW) was called "southerly winds".

The accumulative probability of southerly winds during the navigation season was calculated and plotted in Figure A-2; the numerical results are shown in Table A-1. Probabilities were expressed in per cent as well as in hours of duration, for example, on the average, a wind velocity of 20 mph will be equalled or exceeded by southerly winds for 11.5 hours (4.4%) during the navigation season; the southerly winds' velocity will be less than 20 mph for 253.5 hours (95.6%). Total number of hours of southerly winds in the navigation season was determined as 265 (10-year average).

Hamilton Harbour Station Data

These wind data require careful analysis, because they only consist of daily, maximum hourly, wind records, without any reference to the wind direction and velocity distributions during 24 hours.

A similar procedure as for Toronto Data was used; "southerly winds" were defined as the sum of the winds coming from the SE, S and SW directions (the SE and S directions together contributed only 2% of all the data).

A probability analysis shows that there is a 20% probability (or 37 days) during the navigation season, when the daily maximum hourly wind is southerly.

The accumulative probability of daily, maximum hourly, southerly winds was calculated and plotted in Figure A-3; the numerical results are shown in Table A-1. The probabilities were expressed in per cent as well as in the number of days during which the daily, maximum hourly, wind exceeded a certain speed. Considering the wind velocity of 20 mph, it is shown in Figure A-3 that this velocity will be exceeded by daily, maximum hourly, southerly winds for 3.3 days of the navigation season. It cannot be deduced, however, what the duration of the daily, maximum hourly, wind is; its duration had to be at least 1 hour, but it could have been even longer.

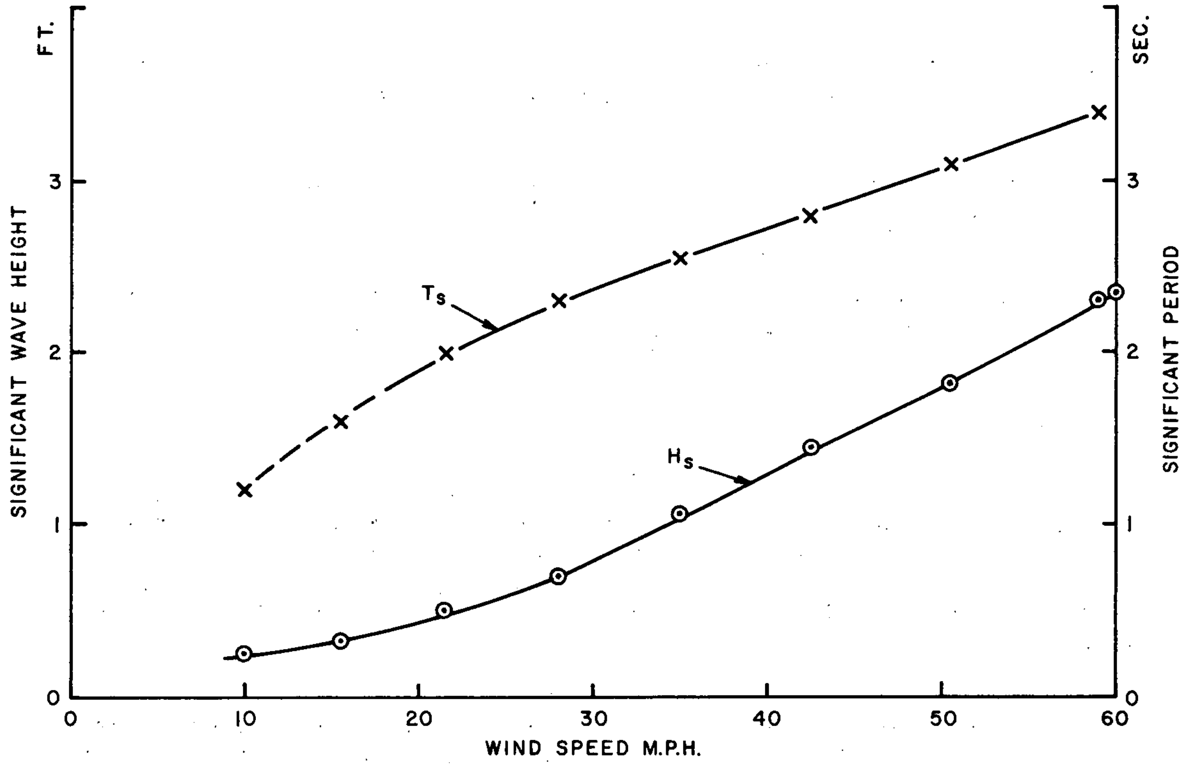


Figure A-2. Significant wave heights and period for southerly winds.

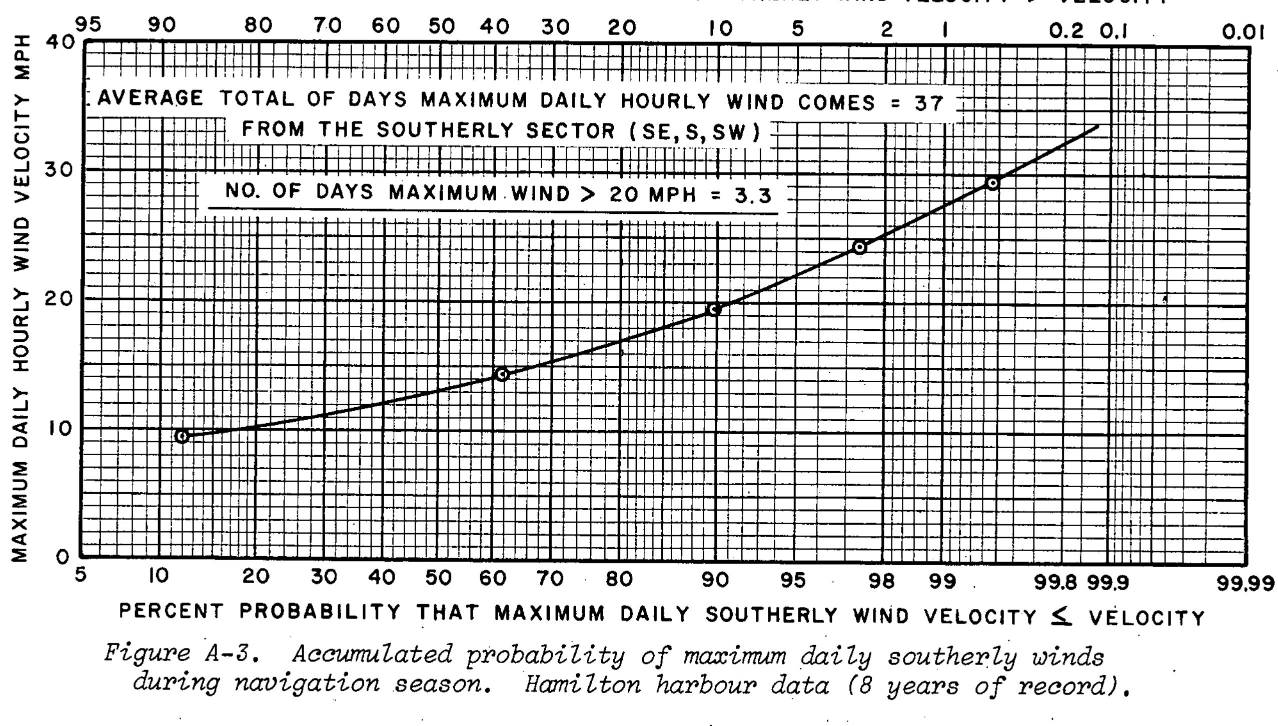


Figure A-3. Accumulated probability of maximum daily southerly winds during navigation season. Hamilton harbour data (8 years of record).

TABLE A-1

A) Probability of Occurrence of Southerly Hourly Wind in Navigation Season - Data from Toronto International Airport. Recalculated after Donnely (1969).

Wind Velocity [mph]	1.3	4-7	8-12	13-18	19-24	25-31	32-38	39-46
Probability of Occurrence in [%]	9.74	33.68	32.48	18.24	4.42	1.25	.15	0.04
Accumulative Probability S. Wind Velocity < Velocity [%]	9.74	43.42	75.90	94.14	98.56	99.81	99.96	100.00

B) Probability of Occurrence of Daily Maximum Hourly Southerly Wind in Navigation Season - Data from Hamilton Harbour Station.

Wind Velocity [mph]	5-9	10-14	15-19	20-24	25-29	30-34
Probability of Occurrence in [%]	11.9857	49.5527	28.2648	7.5134	2.1467	.5367
Accumulative Probability D.M.H.S. Wind Vel. < Velocity [%]	11.9857	61.5384	89.8032	97.3166	99.4633	100.0000

Thus, no direct comparison between the Toronto and Hamilton wind data is possible, and preference is given to the Toronto data. In the next step, significant wave heights and periods were determined by using the wind velocities and fetches in the wave prediction method contained in the report of the U.S. Army Coastal Engineering Centre (1966).

The following conclusions were reached on the basis of the derived wave conditions:

a) The model should be tested for two incident wave directions - SSW and SSE. The NW waves do not contribute significantly to the wave agitation.

b) Considering the velocities of the winds which cause the wave agitation within the range 20 - 45 mph, the model wave periods should correspond to the real life wave periods in the range 1.8 - 3.0 secs.

c) The probability of southerly waves is equal to the probability of southerly winds; the latter, shown in Figure A-3, is based on wind records from the Toronto International Airport Station.

4. DESIGN OF THE MODEL

Model Scale

The main condition for the choice of model scale is usually determined by hydraulic similitude, i.e., the scale should be such that the phenomena which are neglected in the hydraulic similitude chosen (e.g., surface tension, etc.) will have a negligible effect in the model. However, these considerations usually are limited by the laboratory space available, and the final decision has to be a compromise.

Since the problem under consideration was that of wave agitation, it was necessary to use an undistorted three-dimensional model.

The model scale was chosen as 1:36. This scale made it possible to fit the model of the boat basin and its vicinity into the laboratory basin (Figure A-4), and simultaneously ensured a satisfactory reproduction of wave conditions occurring in real life.

Some scale effects were experienced in the case of the shortest wave period (0.3 sec. in the model) and in waves coming from the SSW direction; these effects are discussed in Section 7.

Hydraulic Similitude

Since the predominating force influencing the wave pattern is gravity, it follows that the Froude Number (N_F) should be the same in the model as in the prototype.

$$(N_F)p = (N_F)m$$

$$\frac{U_p^2}{g_pL_p} = \frac{U_m^2}{g_pL_m}$$

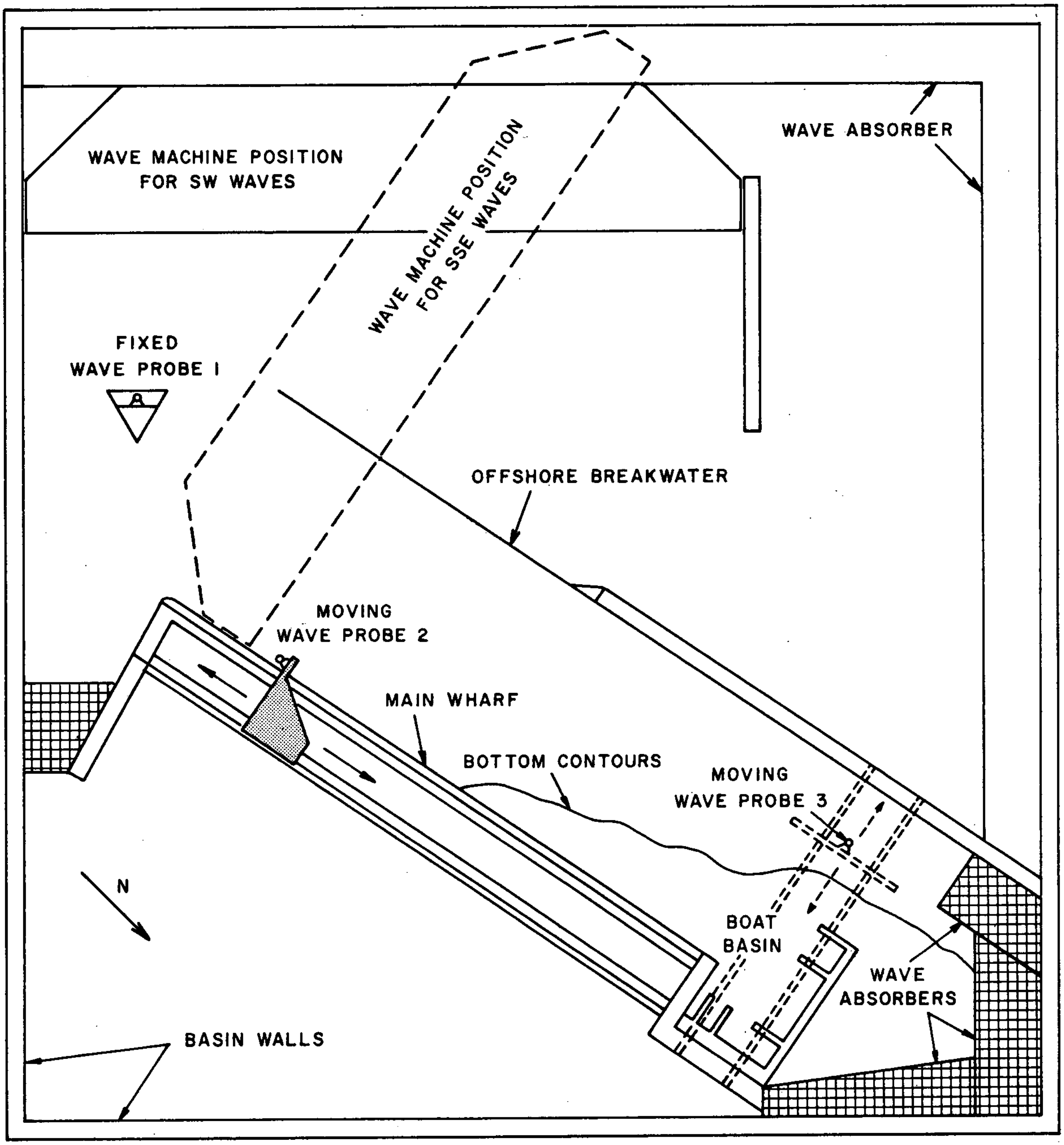


Figure A-4. Model arrangement in laboratory.

where m and p refer to the model and prototype respectively.

Therefore

$$\frac{U_p}{U_m} = \frac{L_p}{L_m}$$ where U = velocity, ft./sec.
L = corresponding lengths, ft.

and

$$\frac{T_p}{T_m} = \frac{L_p}{L_m}$$ T = time interval, secs.

In this study, the numerical values of the scales are as follows:

length scale = 36

velocity scale = 6

time scale = 6

Due to short wave periods in real life (1.8 - 3 secs.), certain difficulties were experienced in the reproduction of these periods in the model. The briefest model wave period giving acceptable results was that of 0.3 sec., although scale effects were present.

Model Construction

Details of the model of the CCIW boat basin are shown in Figure A-5. The model was fitted into the laboratory basin, 46 x 50 ft. The main wharf wall and offshore breakwater were made of concrete blocks, and the boat basin structures were made of plywood. Bottom contours were reproduced only for a small area of shallow water which occurred in the vicinity of the boat basin (see Figure A-4). The bottom relief was modelled using gravel and the surface fixed with a thin layer of cement mortar. Wherever the "deep water" conditions were fulfilled, no contours were modelled on the basin floor.

Two systems of rails were attached to the model. One was a movable wave probe fixed to a carriage and was used for wave measurements along the main wharf, and the other was used for measurements inside the boat basin.

All the layouts, which were derived from the original design by adding different breakwaters, were formed by means of concrete blocks to represent rigid and solid structures.

5. EXPERIMENTAL METHODS

The model layouts of the boat basin (Figure A-6) were tested for waves approaching from two directions (Figure A-4). Incident waves were generated by a wave paddle machine which was set up to generate waves in deep water. The wave period was controlled by adjusting a variable-speed gear connected to the paddle by cranks and connecting rods. Three wave periods were employed:

T_m = 0.3, 0.4 and 0.5 sec. (model)

T_p = 1.8, 2.4 and 3.0 secs. (prototype)

The wave height, which is not a controlling factor in wave agitation studies, was adjusted for the wave period in order to give a good wave pattern.

The water level, which was kept constant in all experiments, corresponded to the typical water level in Lake Ontario, i.e., 244.8 ft. I.G.L.D.

During each of the 60 experiments, the boat basin was exposed to wave action for 10 minutes before wave measurements were taken. Wave heights were measured in three locations as shown in Figure A-4. These were:

1) close to the wave machine by means of fixed wave probe,

2) along the main wharf by means of a movable wave probe,

3) inside the boat basin by means of a movable wave probe.

The fixed wave probe was stationed a minimum of five wave lengths from the wave machine to record the incident waves.

The second wave probe was moved slowly along the main wharf over a distance of 26 feet. This procedure of continuous measurement made it possible to pick up the wave height envelope along the probe path. A similar procedure was used for the third probe for the waves in the basin.

The third wave probe was moved slowly along three lines and a continuous record of wave height measurement was made. At certain points (1 - 11, see Figure A-5), measurements of one-minute's duration were made.

This procedure made it possible to obtain an assessment of wave agitation inside the boat basin, as well as along the main wharf, for different types of layouts and, consequently, to make a comparison and a critical evaluation of these layouts.

The wave probes employed were those of Kemp and Remmers, No. R23, which work on the resistance principle. Measured wave heights and frequencies were recorded with a 4-channel recorder.

The accuracy of wave measurements was ± 0.05 inch for wave heights and ± 0.01 second for wave periods, which were adequate for the purpose of this study.

6. RESULTS

Experimental results of the wave agitation measurements inside the boat basin are presented in Figures A-7 to A-12, and those along the main wharf in Figures A-13 to A-18.

Locations (see Figure A-5) are plotted along the x-axis (points 1, 2, ..., 11a, and 11b in Figures A-7 to A-12 and length readings 0 - 26 ft. in Figures A-13 to A-18). Wave agitation, plotted on the y-axis, is expressed by ratio H_{max}/H_{inc}:

where H_{max} = maximum wave height at a given location

H_{inc} = incident wave height

It is therefore evident that the plotted results represent an envelope of maximum wave agitation occurring in the model.

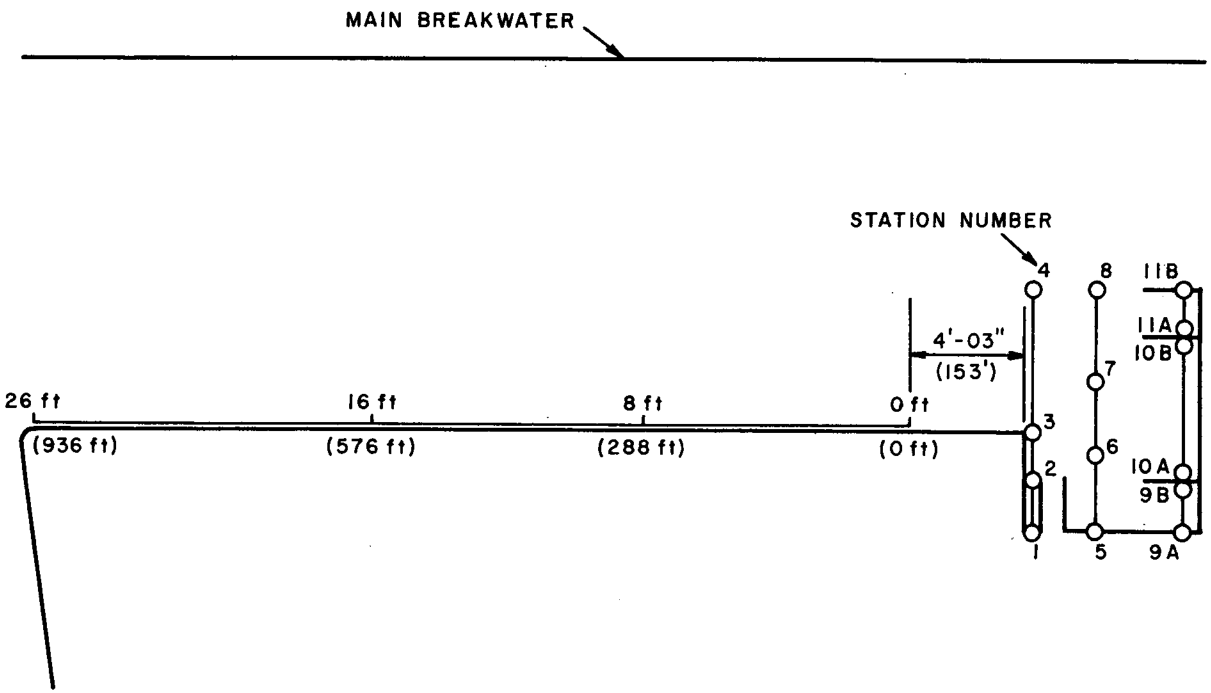


Figure A-5. CCIW boat basin. Location of wave measuring stations in the model.

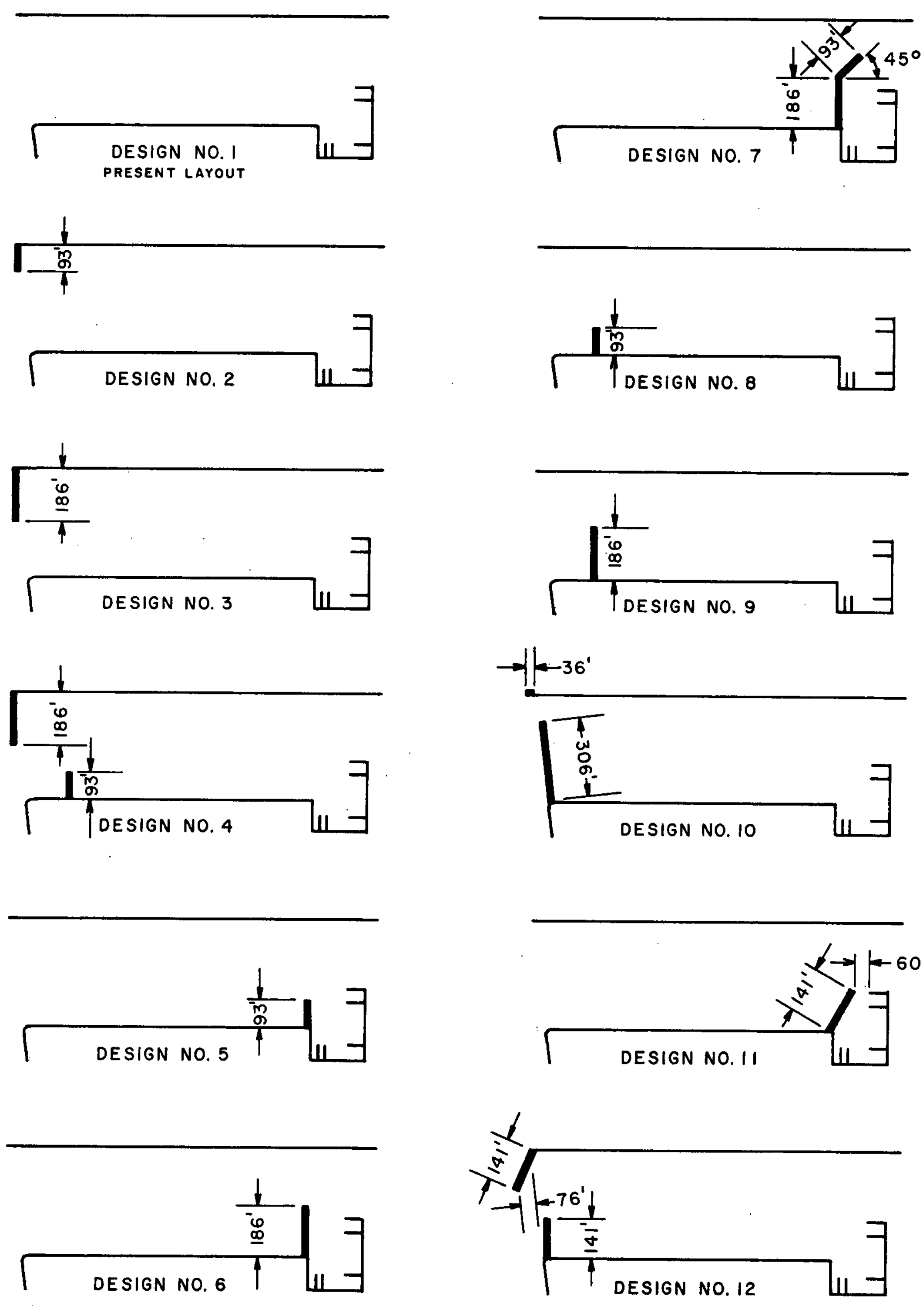


Figure A-6. Schematic plans of breakwater arrangements.

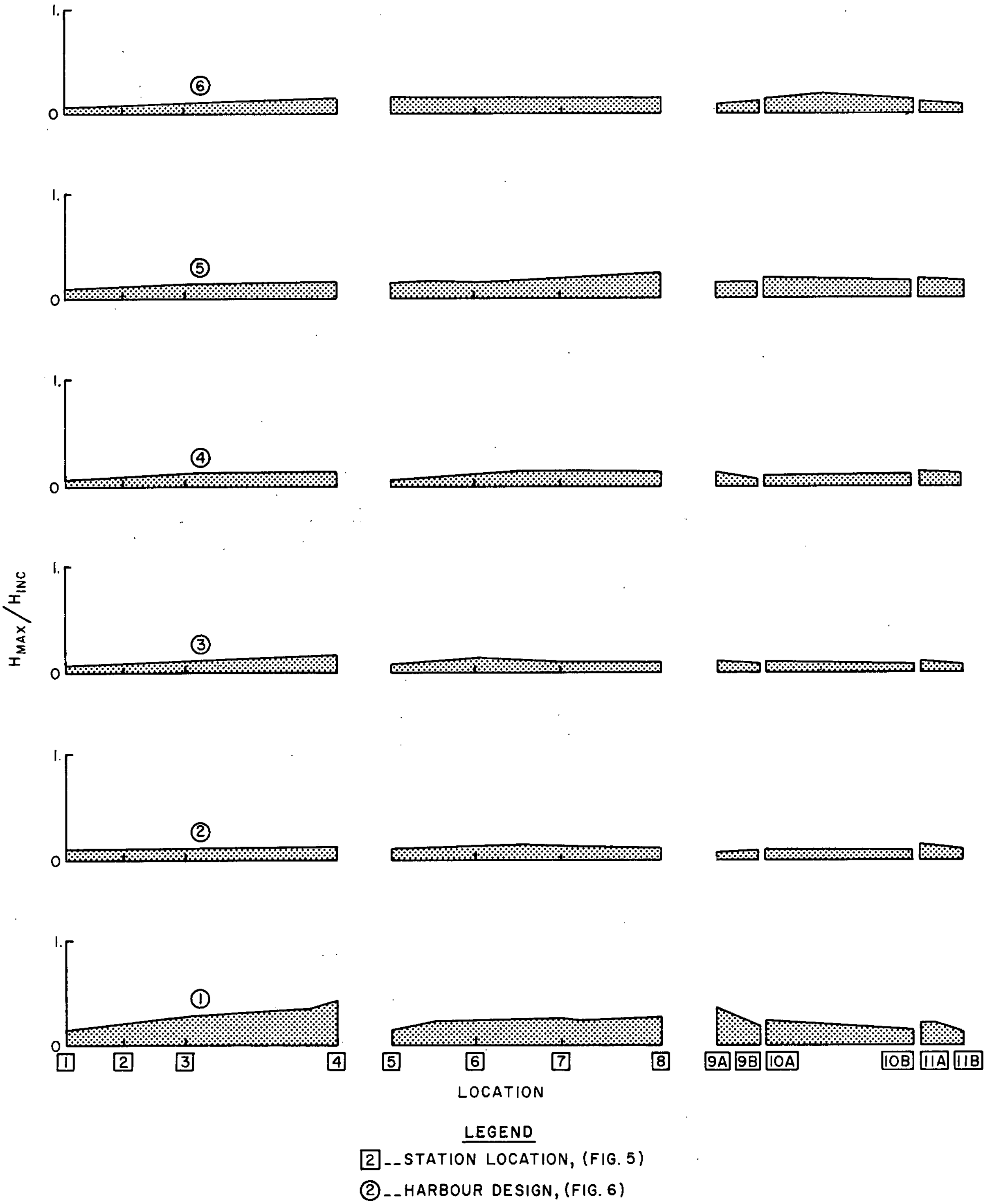


Figure A-7 (Sheet 1). Wave height ratio in boat basin. Waves from SW (T=0.3 sec).

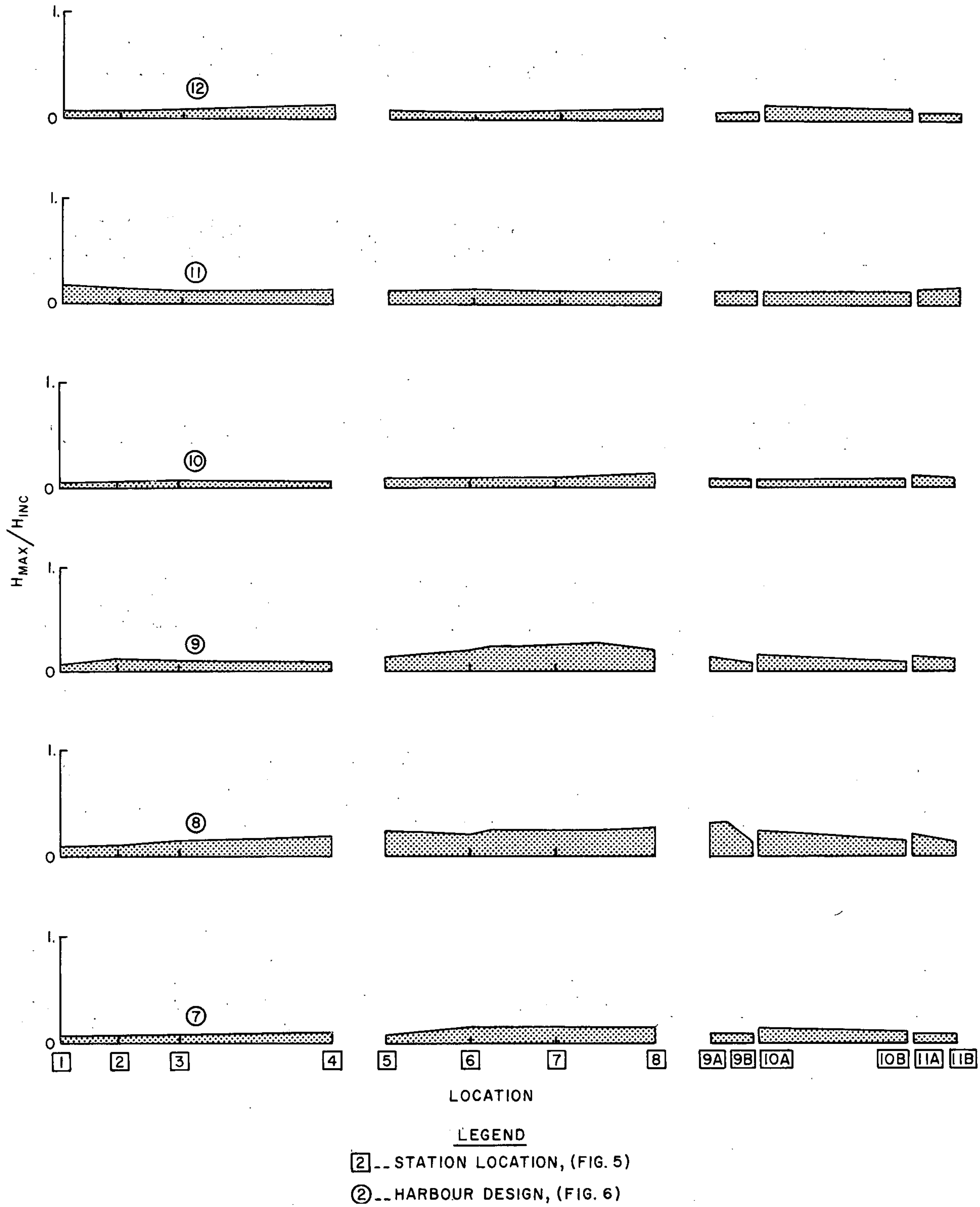


Figure A-7 (Sheet 2). Wave height ratio in boat basin. Waves from SW (T=0.3 sec).

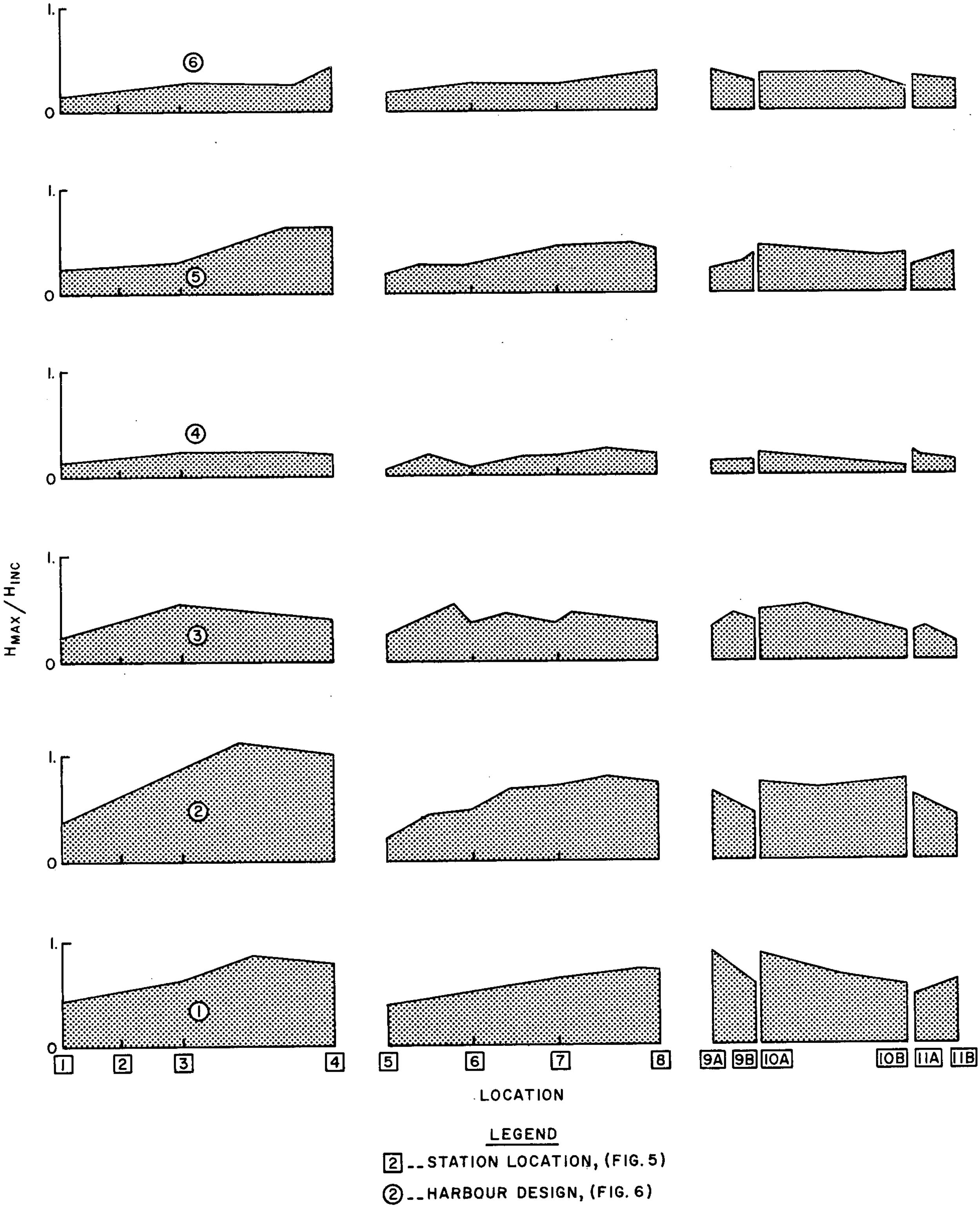


Figure A-8 (Sheet 1). Wave height ratio in boat basin. Waves from SW (T=0.4 sec).

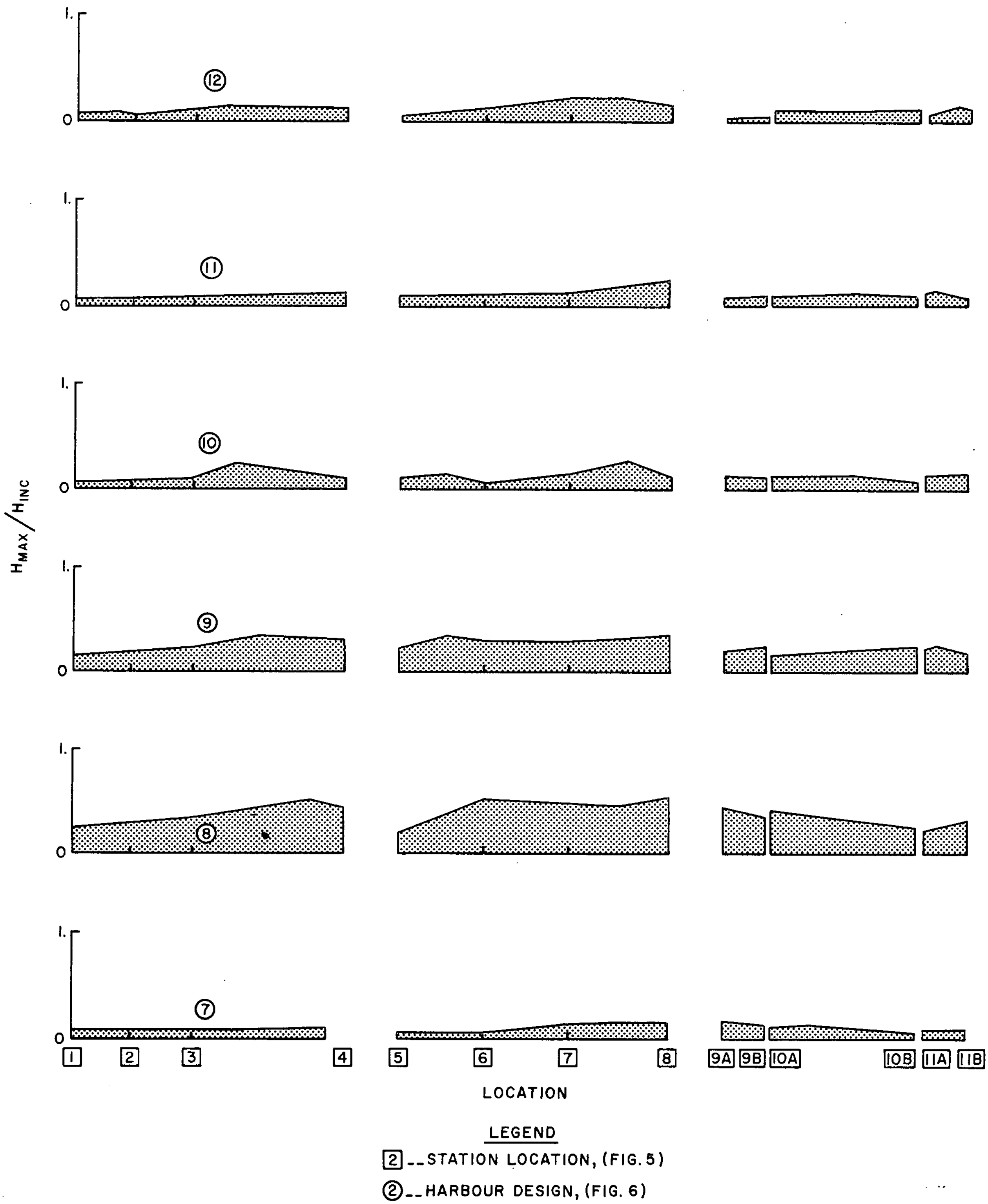


Figure A-8 (Sheet 2). Wave height ratio in boat basin. Waves from SW (T=0.4 sec).

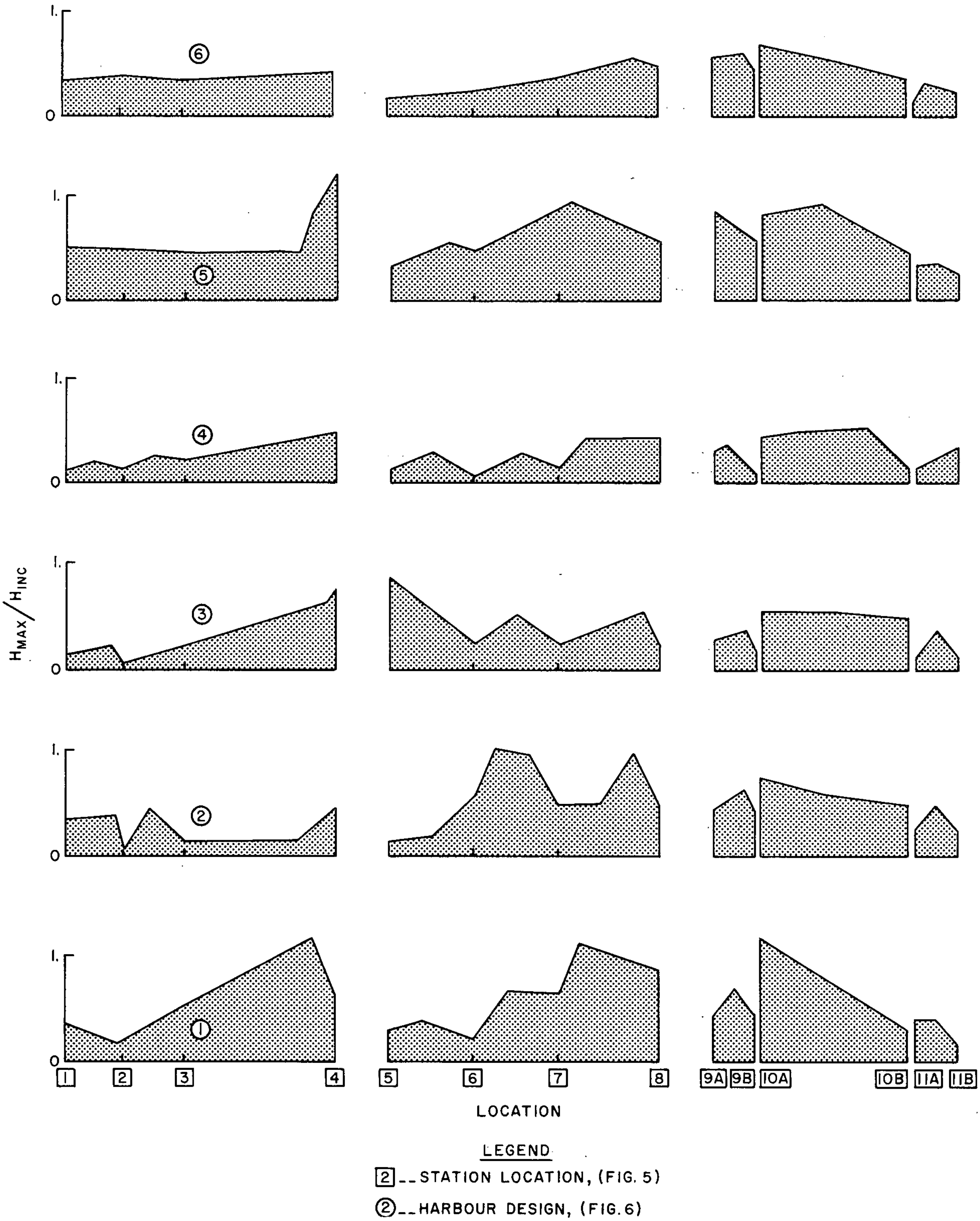


Figure A-9 (Sheet 1). Wave height ratio in boat basin. Waves from SW (T=0.5 sec).

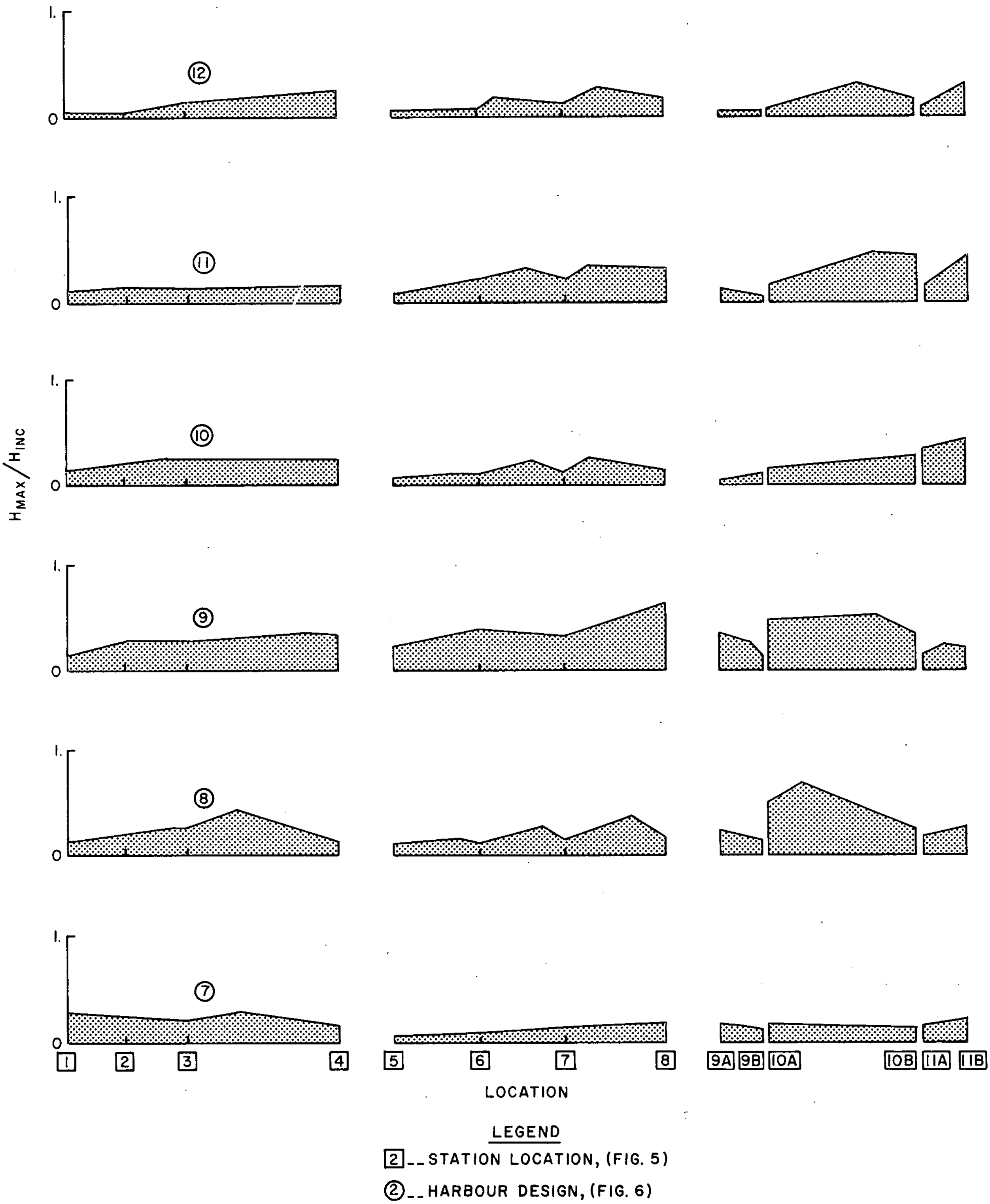


Figure A-9 (Sheet 2). Wave height ratio in boat basin. Waves from SW (T=0.5 sec).

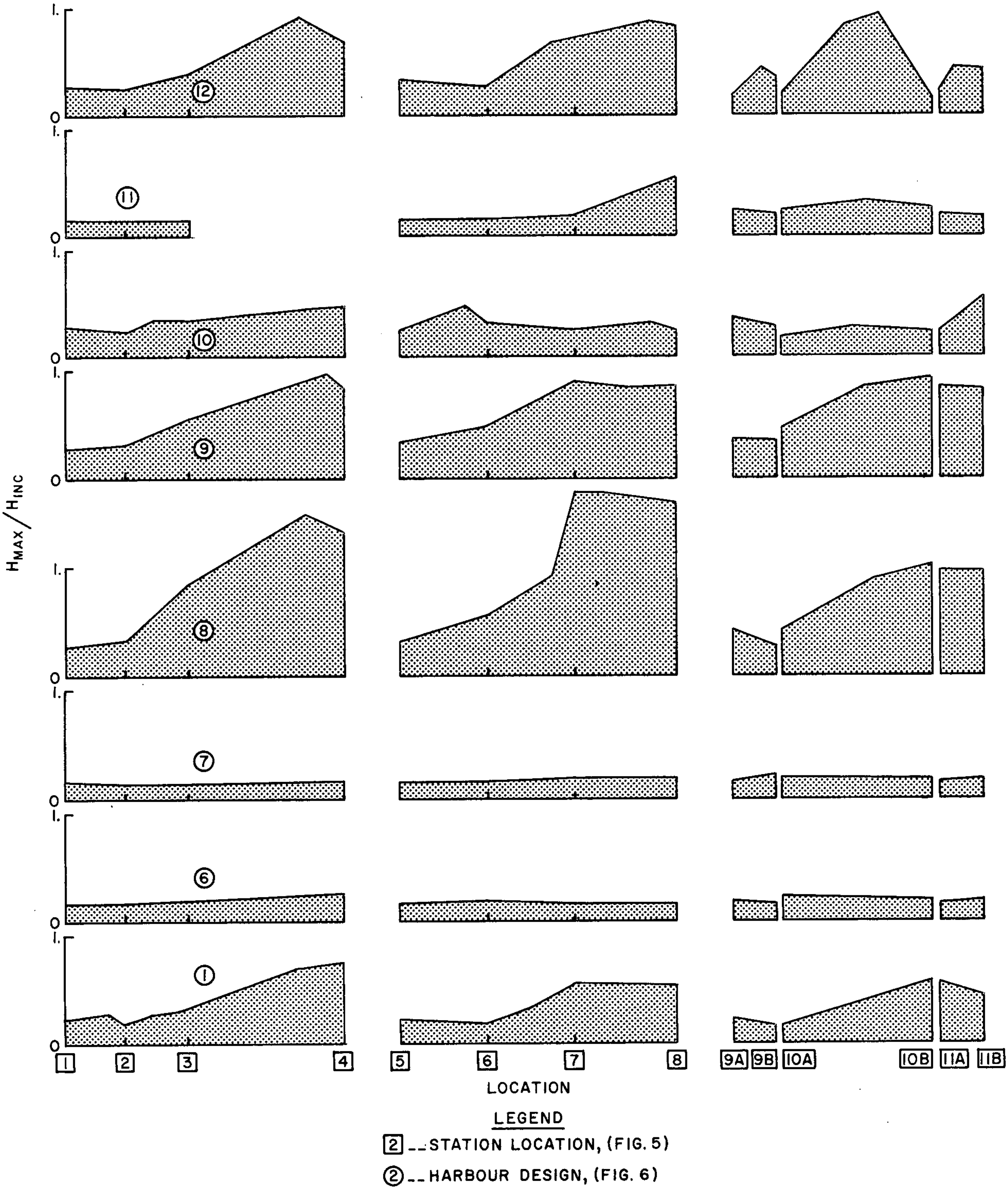


Figure A-10. Wave height ratio in boat basin. Waves from SSE (T=0.3 sec).

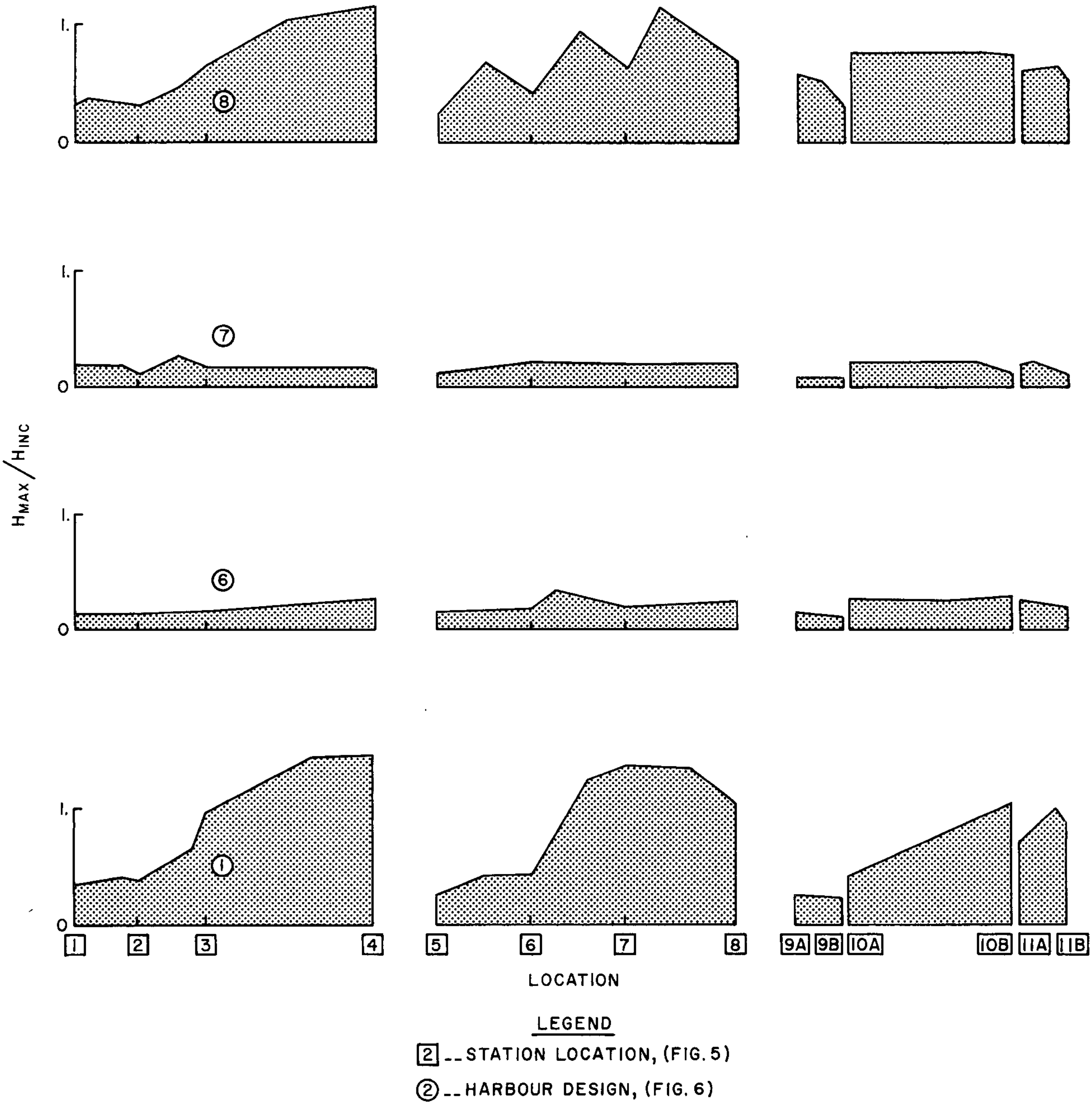


Figure A-11 (Sheet 1). Wave height ratio in boat basin. Waves from SSE (T=0.4 sec).

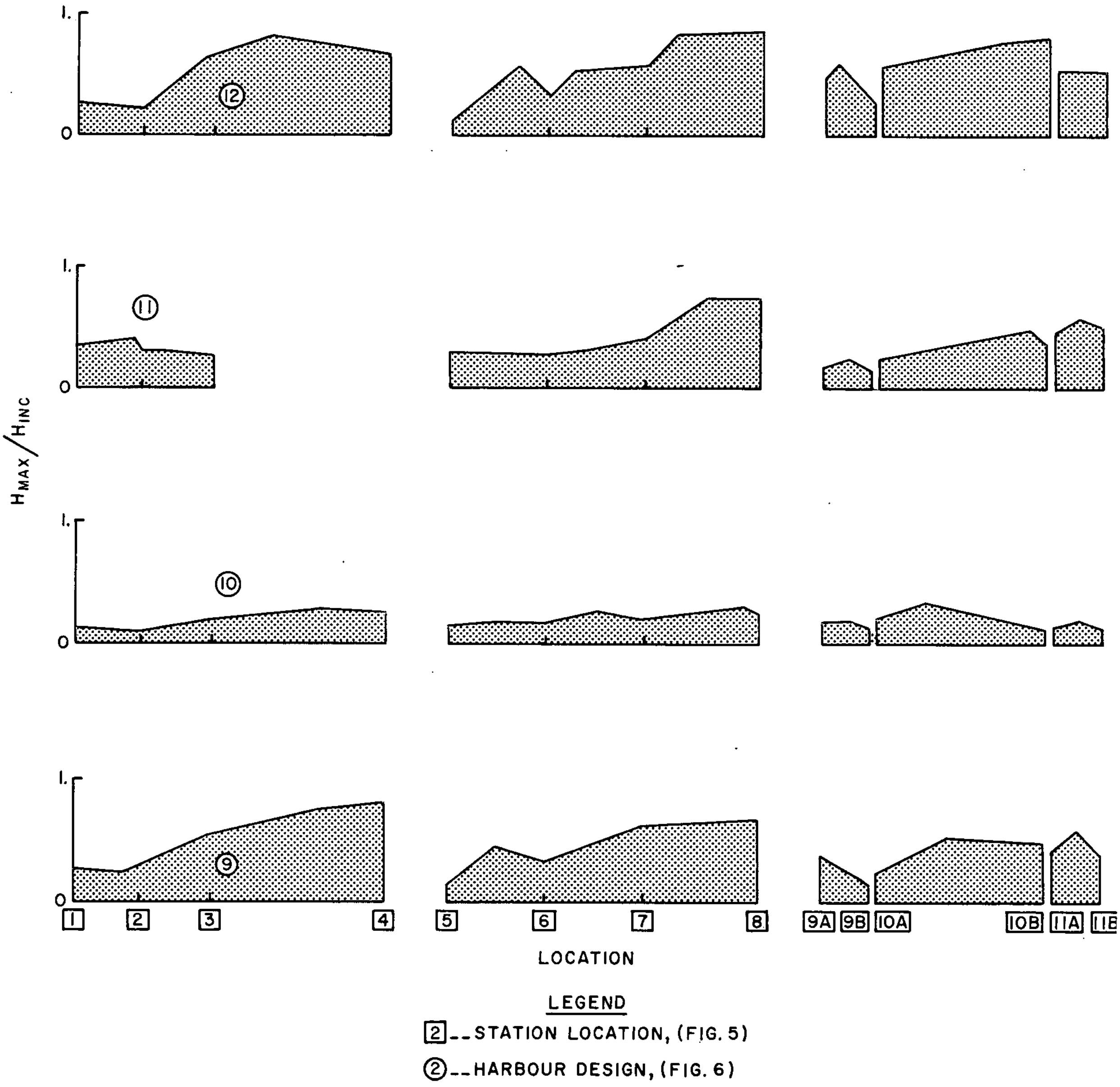


Figure A-11 (Sheet 2). Wave height ratio in boat basin. Waves from SSE (T=0.4 sec).

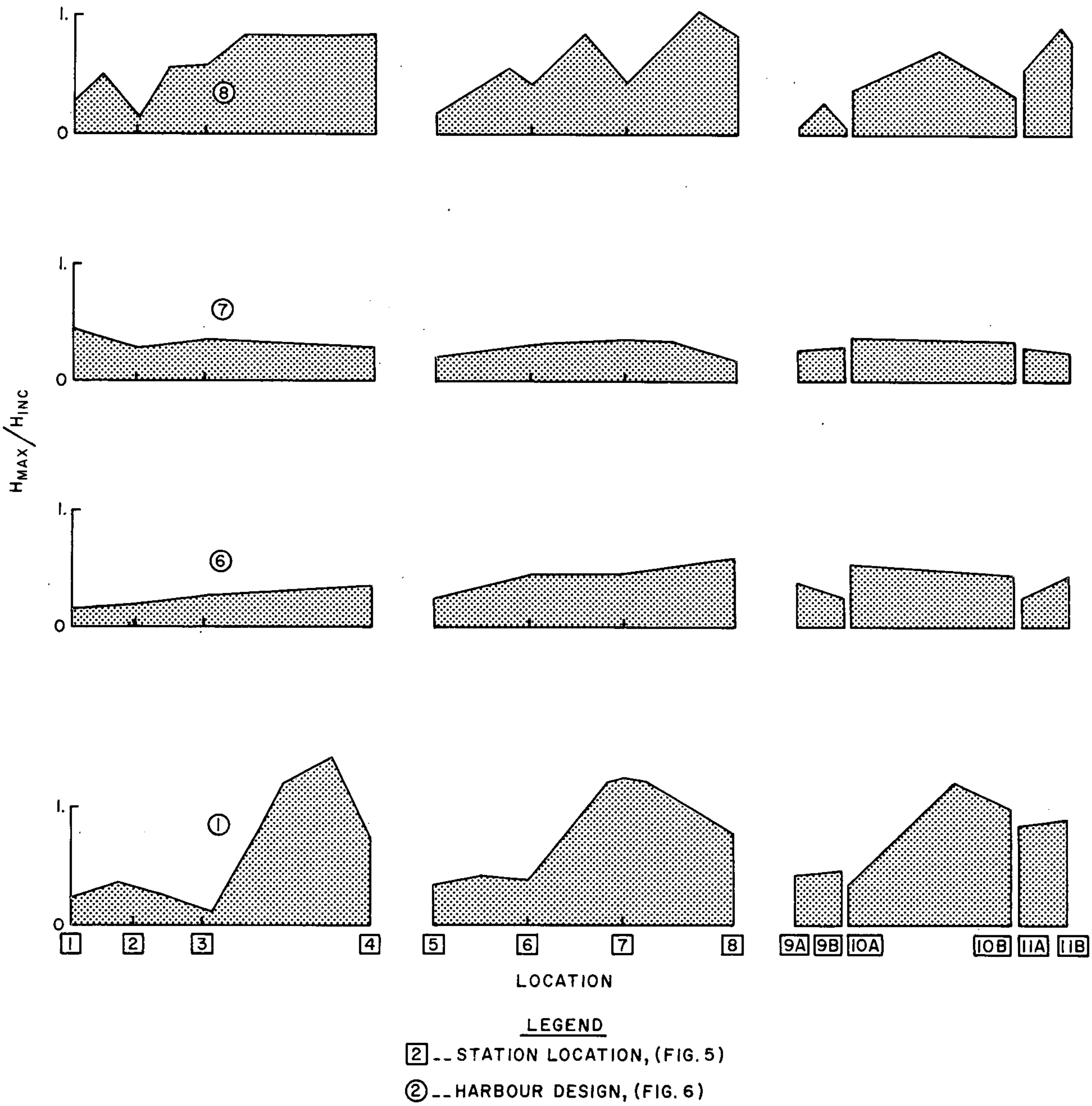


Figure A-12 (Sheet 1). Wave height ratio in boat basin. Waves from SSE (T=0.5 sec).

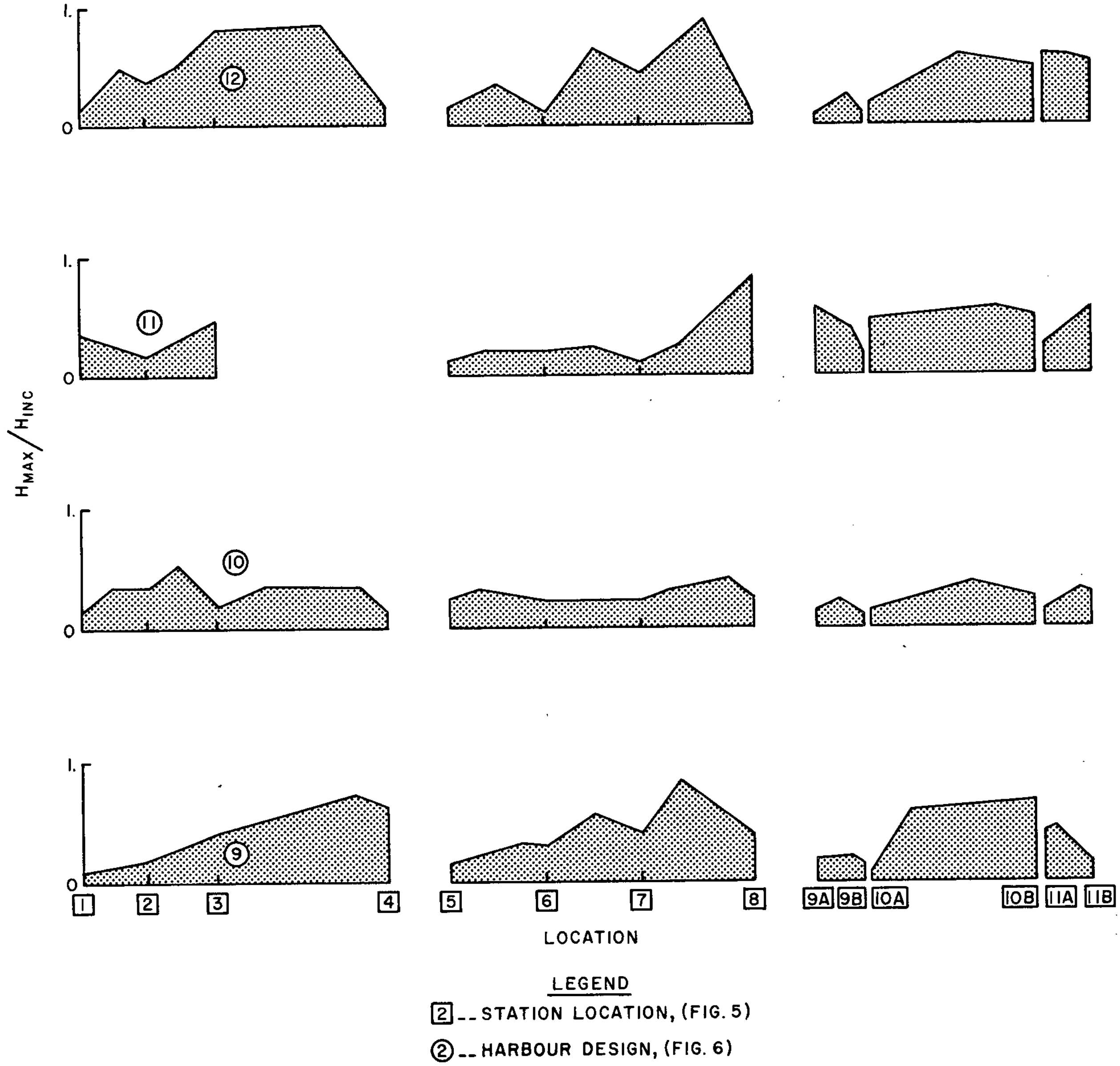


Figure A-12 (Sheet 2). Wave height ratio in boat basin. Waves from SSE (T=0.5 sec).

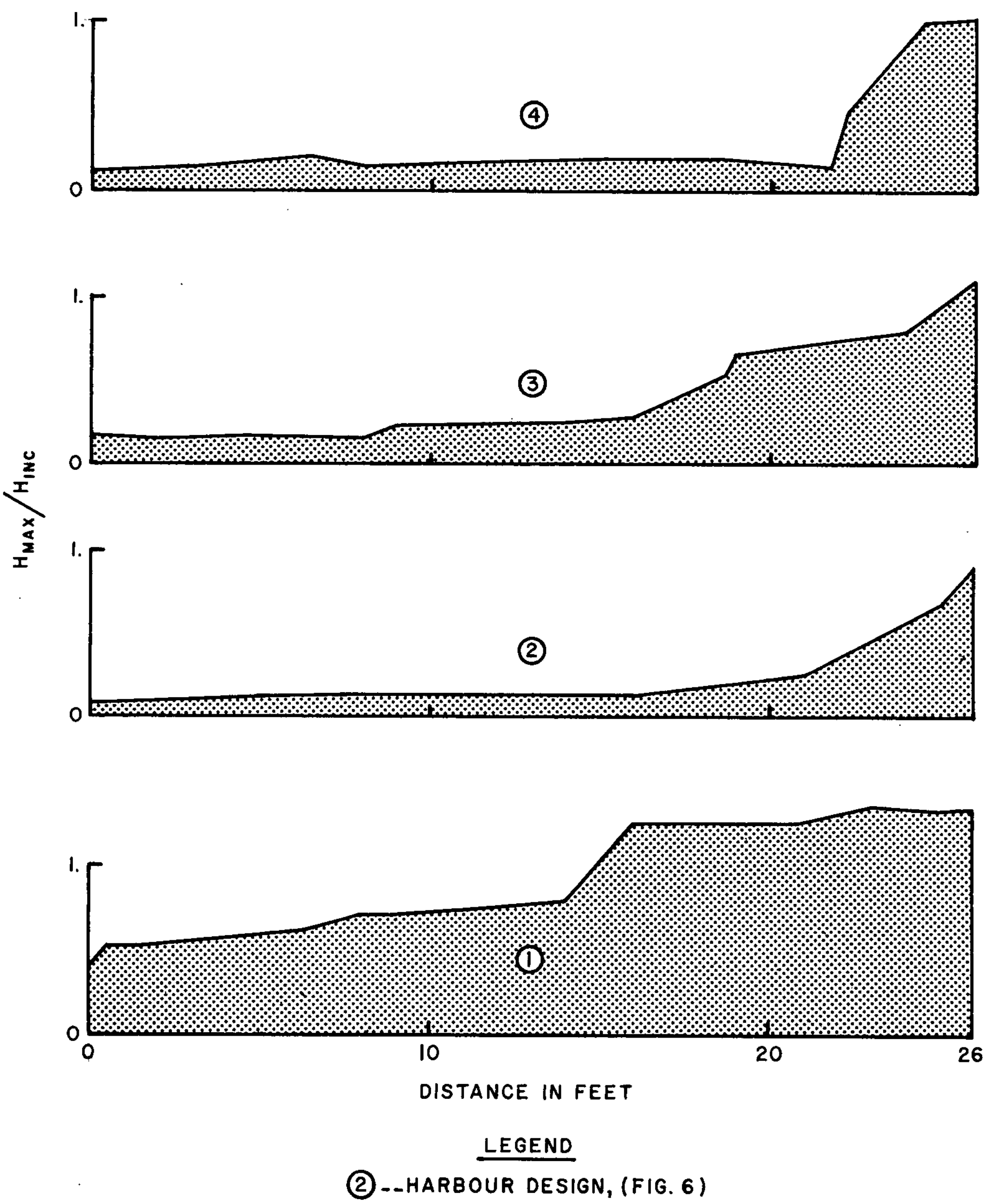


Figure A-13 (Sheet 1). Wave height ratio along wharf wall. Waves from SW (T=0.3 sec).

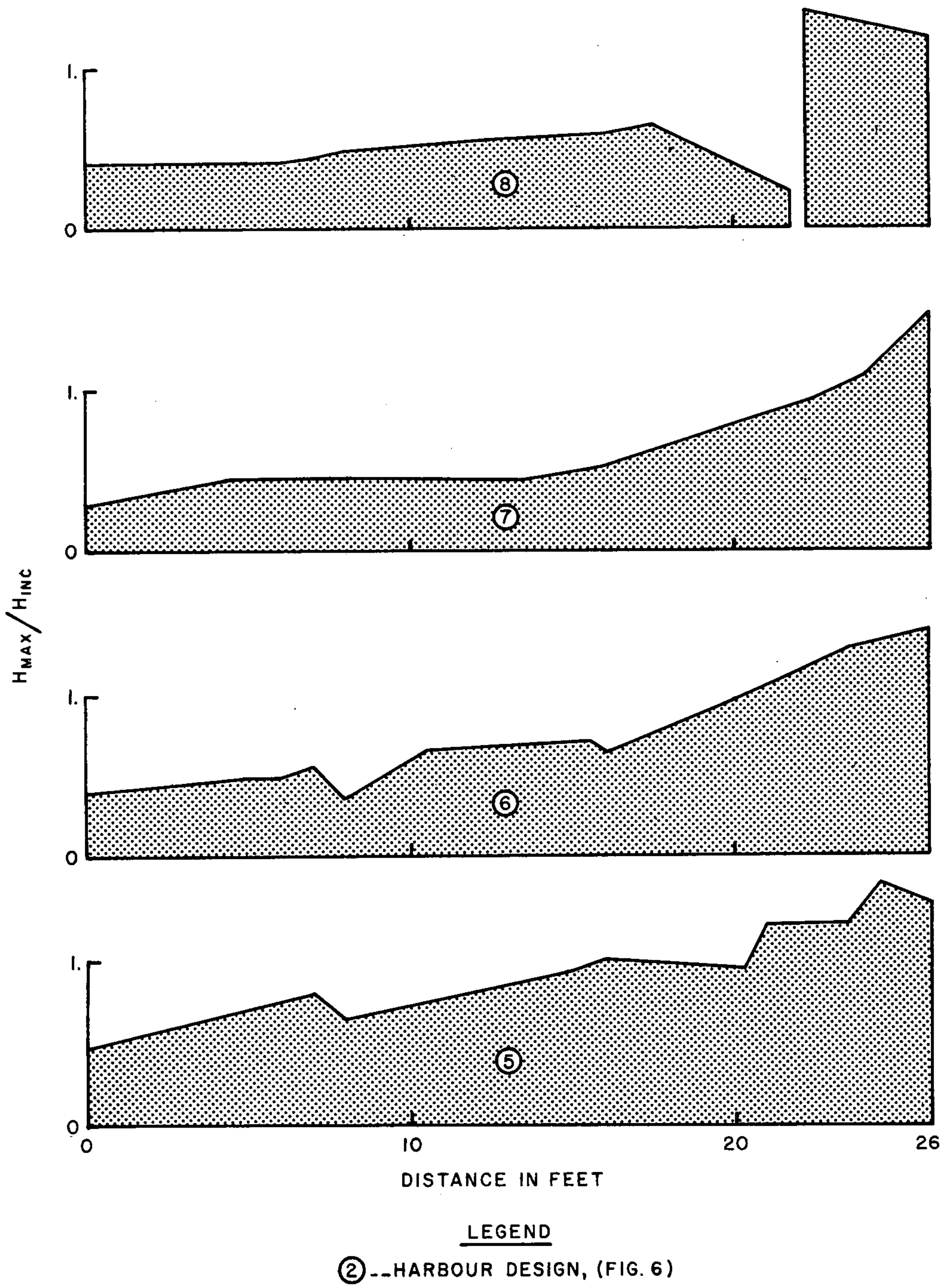


Figure A-13 (Sheet 2). Wave height ratio along wharf wall. Waves from SW (T=0.3 sec).

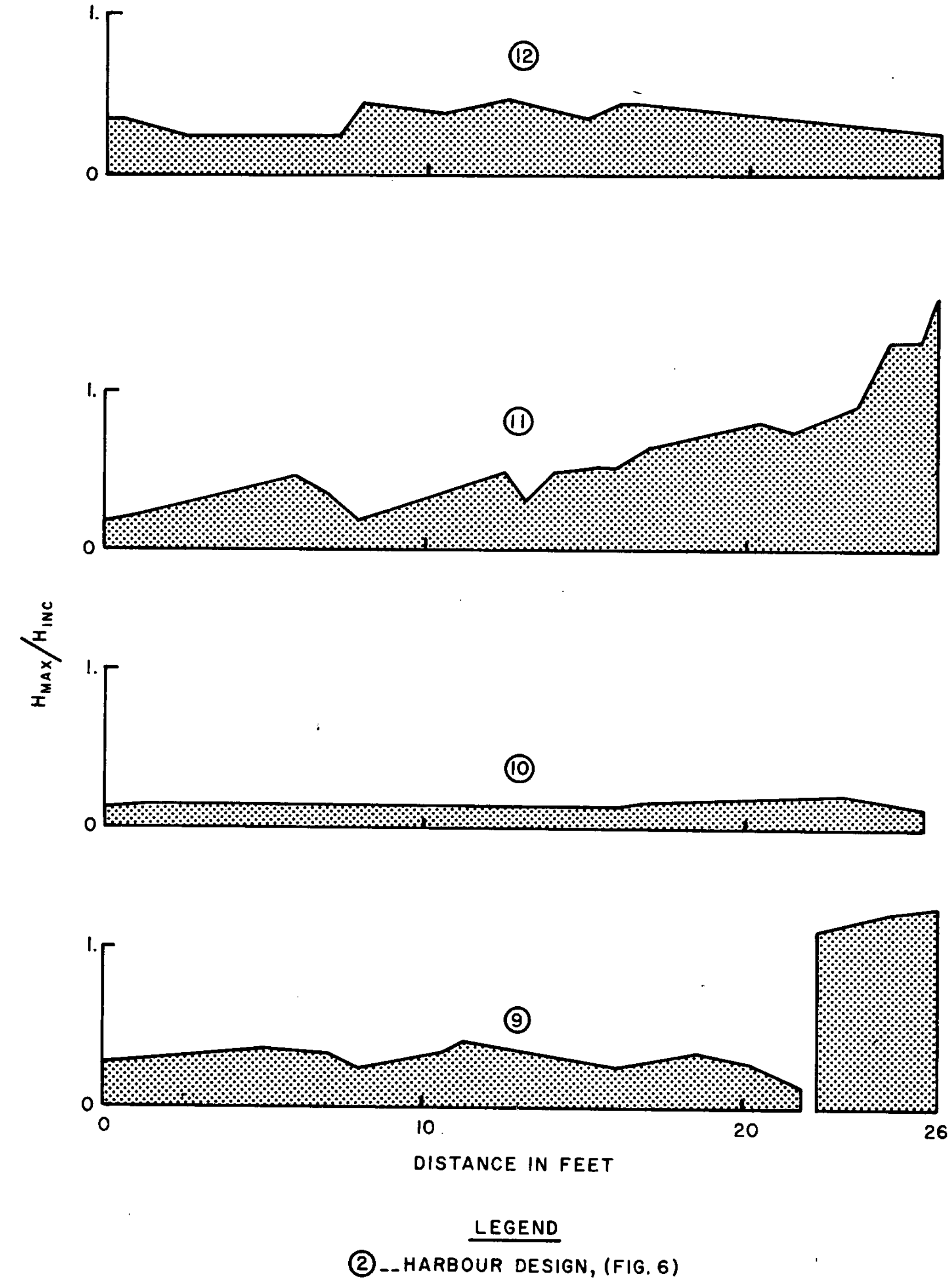


Figure A-13 (Sheet 3). Wave height ratio along wharf wall. Waves from SW (T=0.3 sec).

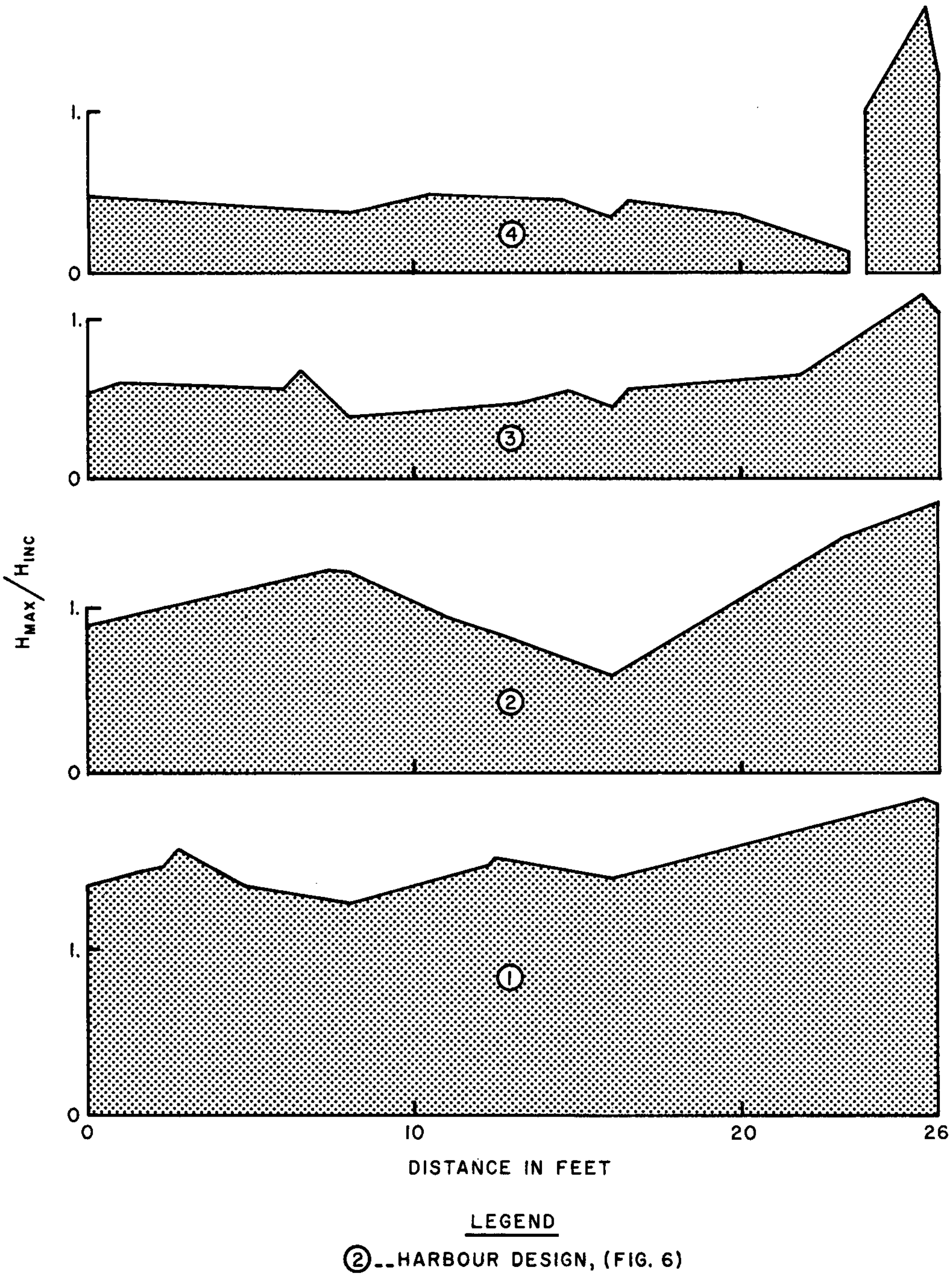


Figure A-14 (Sheet 1). Wave height ratio along wharf wall. Waves from SW (T=0.4 sec).

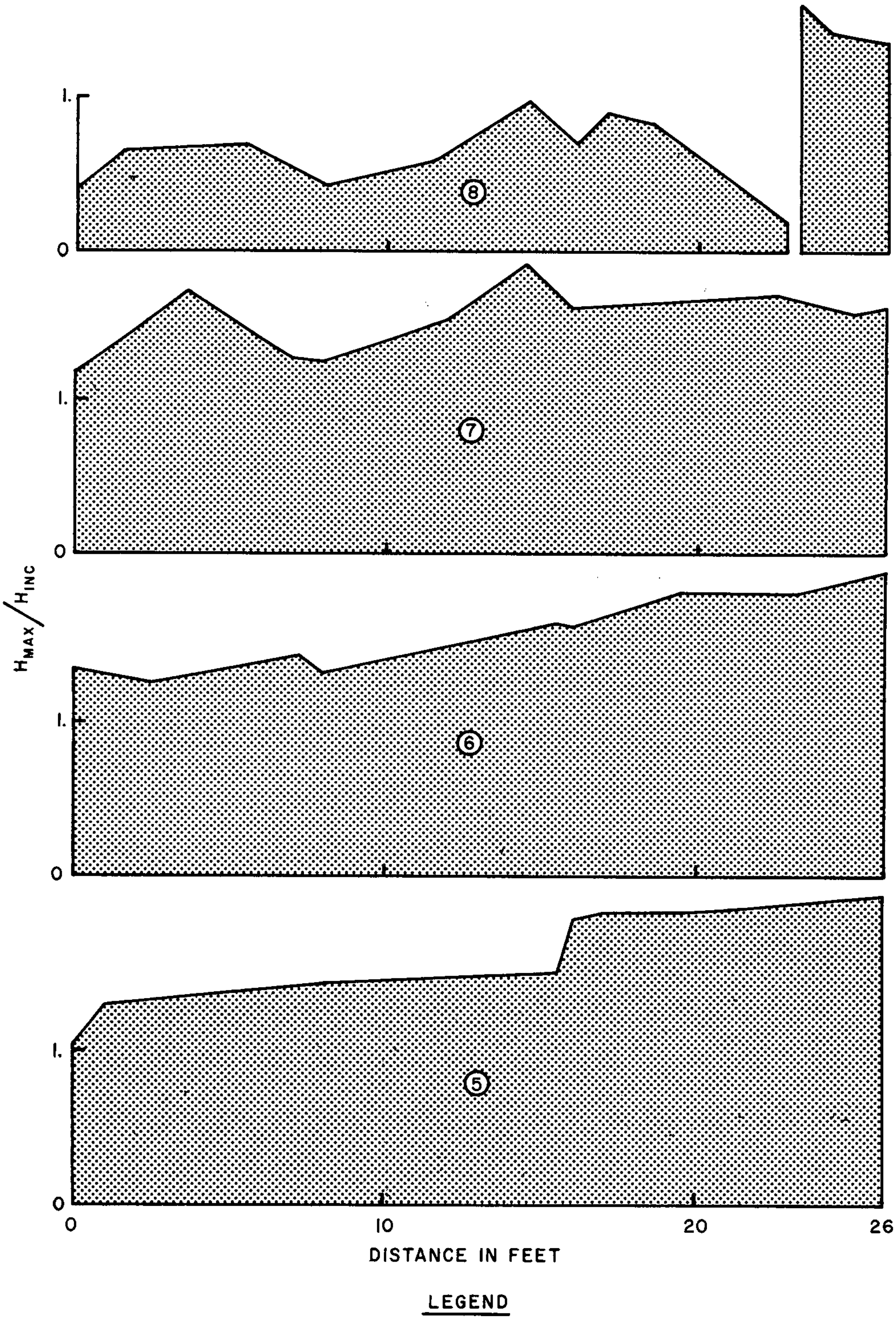


Figure A-14 (Sheet 2). Wave height ratio along wharf wall. Waves from SW (T=0.4 sec).

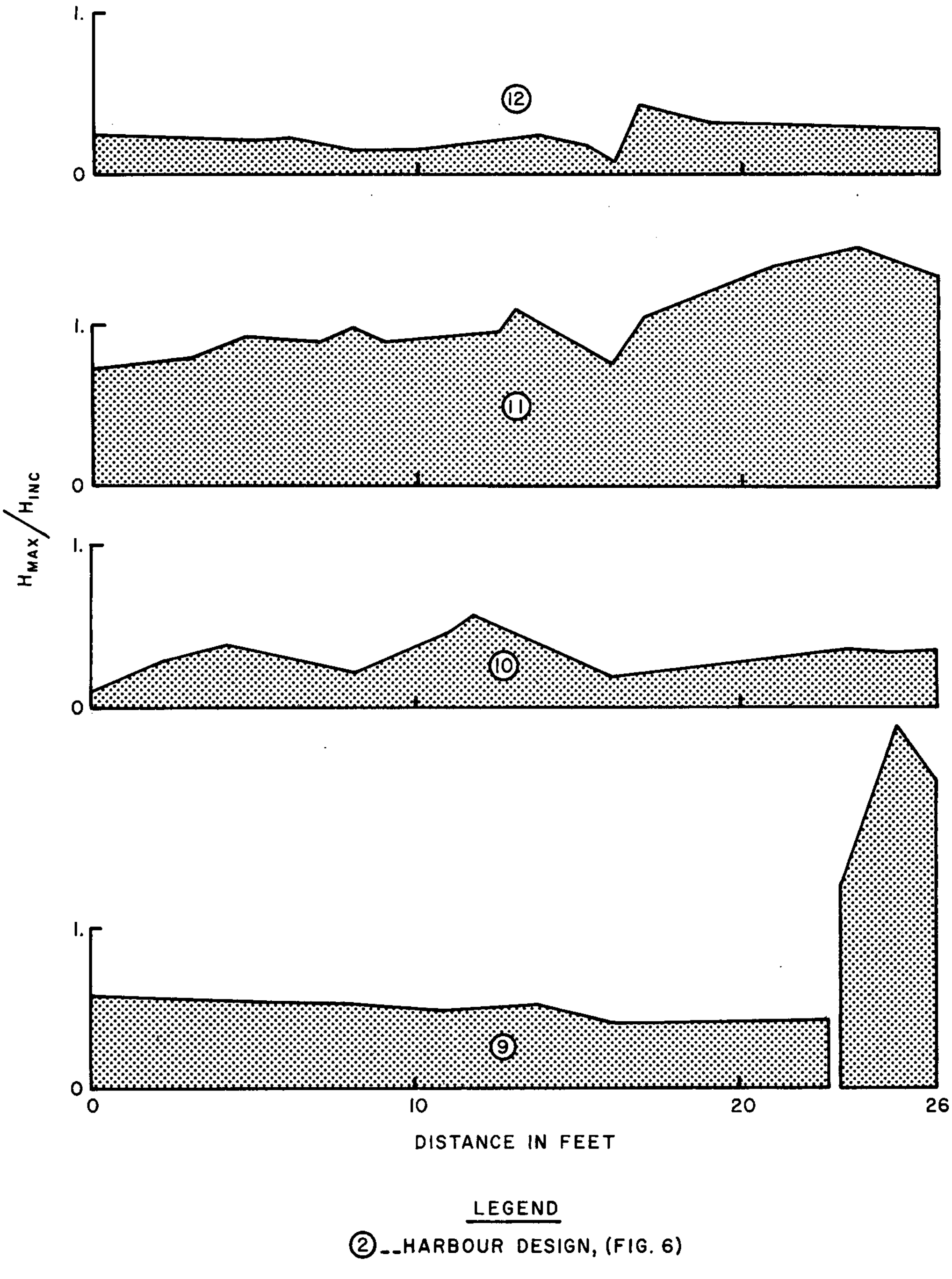
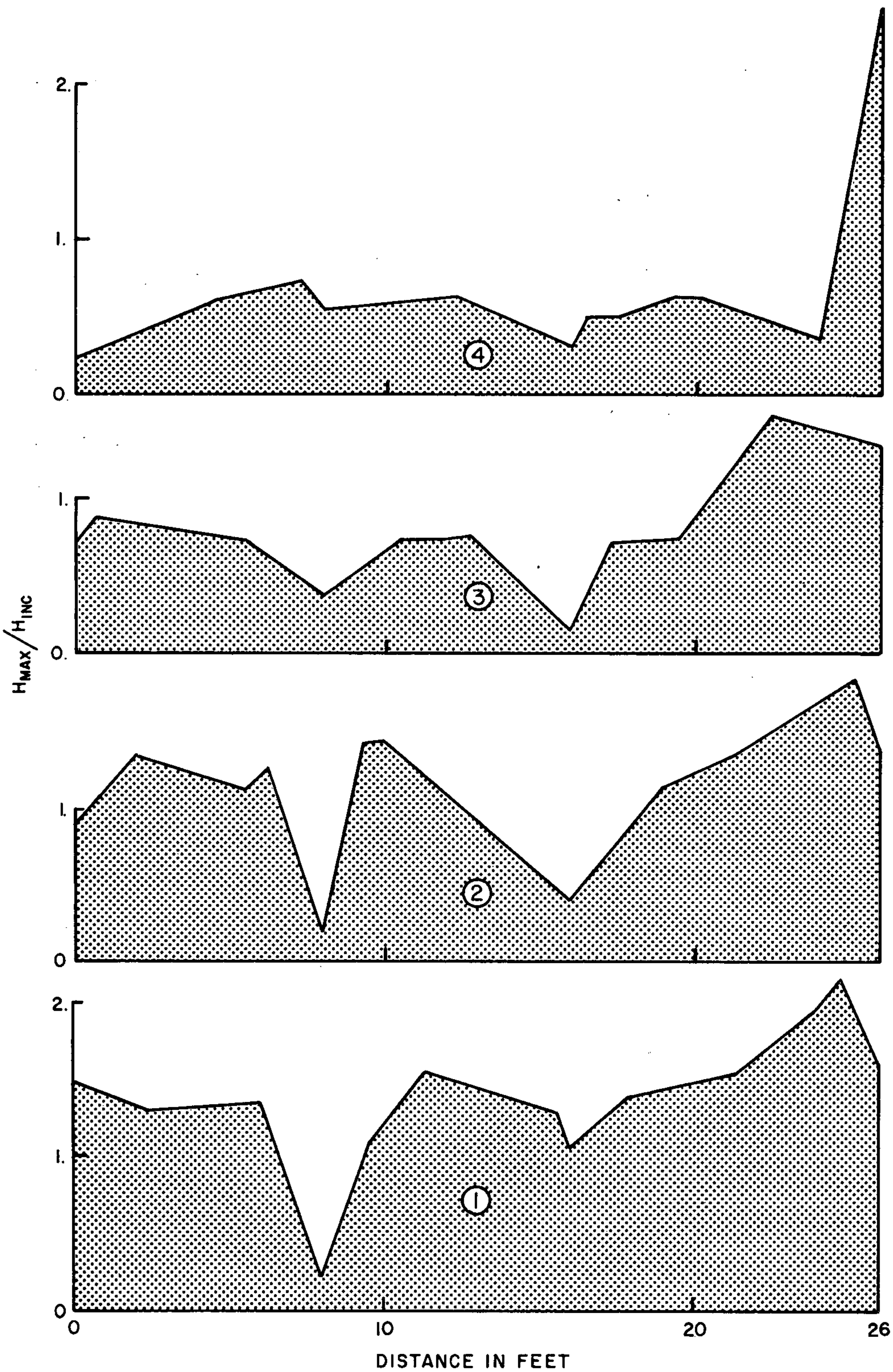


Figure A-14 (Sheet 3). Wave height ratio along wharf wall. Waves from SW (T=0.4 sec).



LEGEND

②__HARBOUR DESIGN, (FIG. 6)

Figure A-15 (Sheet 1). Wave height ratio along wharf wall. Waves from SW (T=0.5 sec).

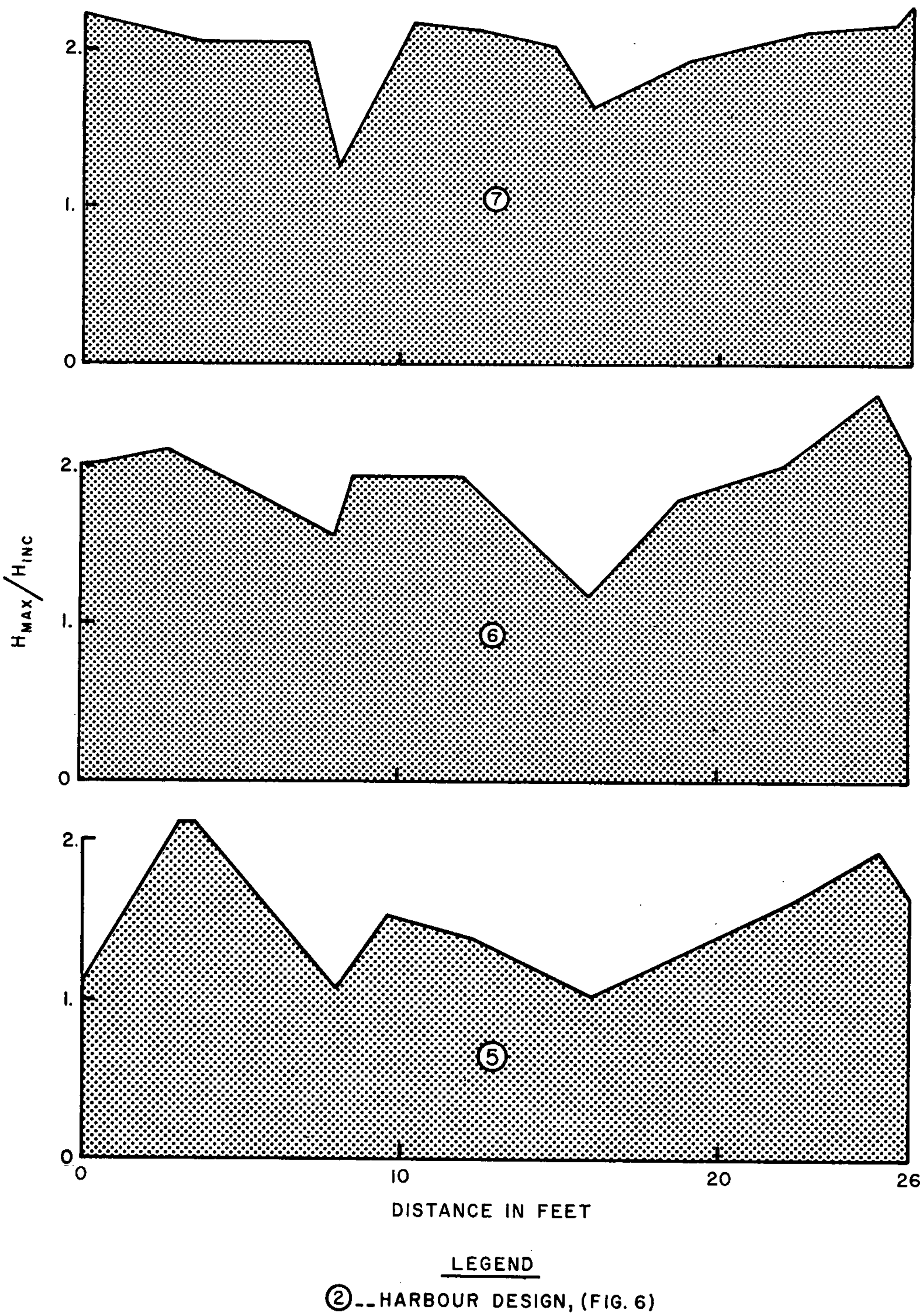


Figure A-15 (Sheet 2). Wave height ratio along wharf wall. Waves from SW (T=0.5 sec).

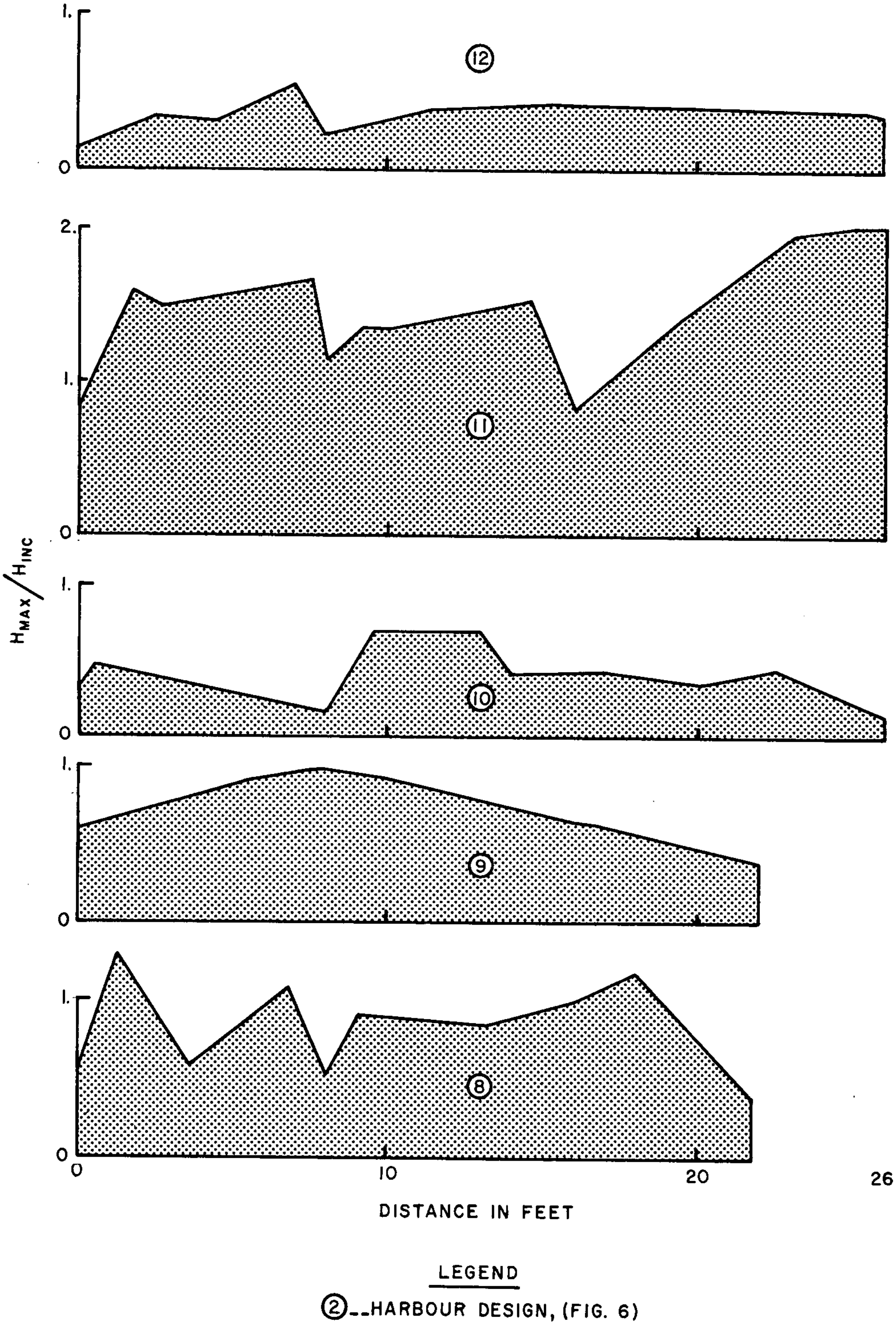


Figure A-15 (Sheet 3). Wave height ratio along wharf wall. Waves from SW (T=0.5 sec).

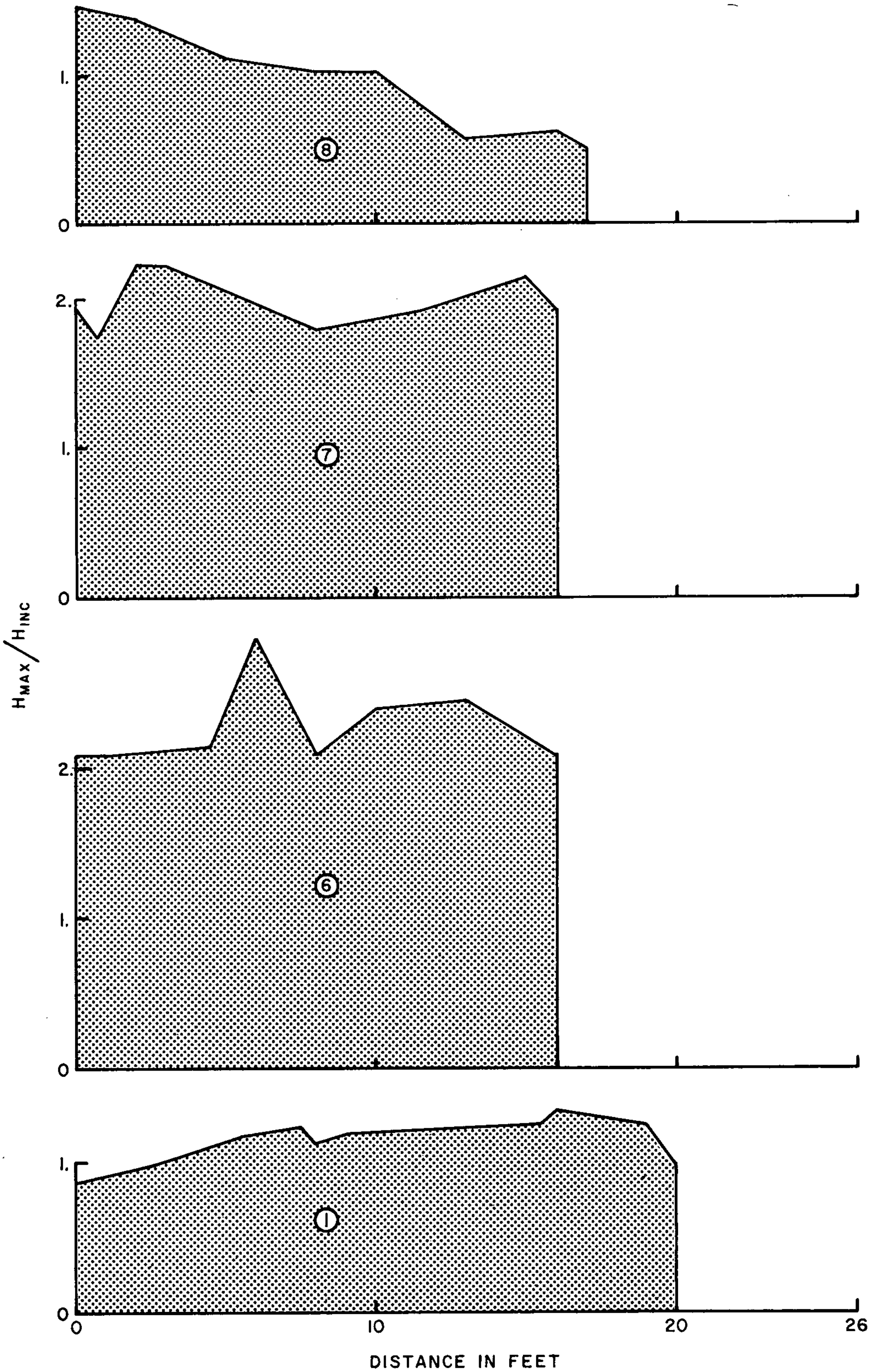


Figure A-16 (Sheet 1). Wave height ratio along wharf wall. Waves from SSE (T=0.3 sec).

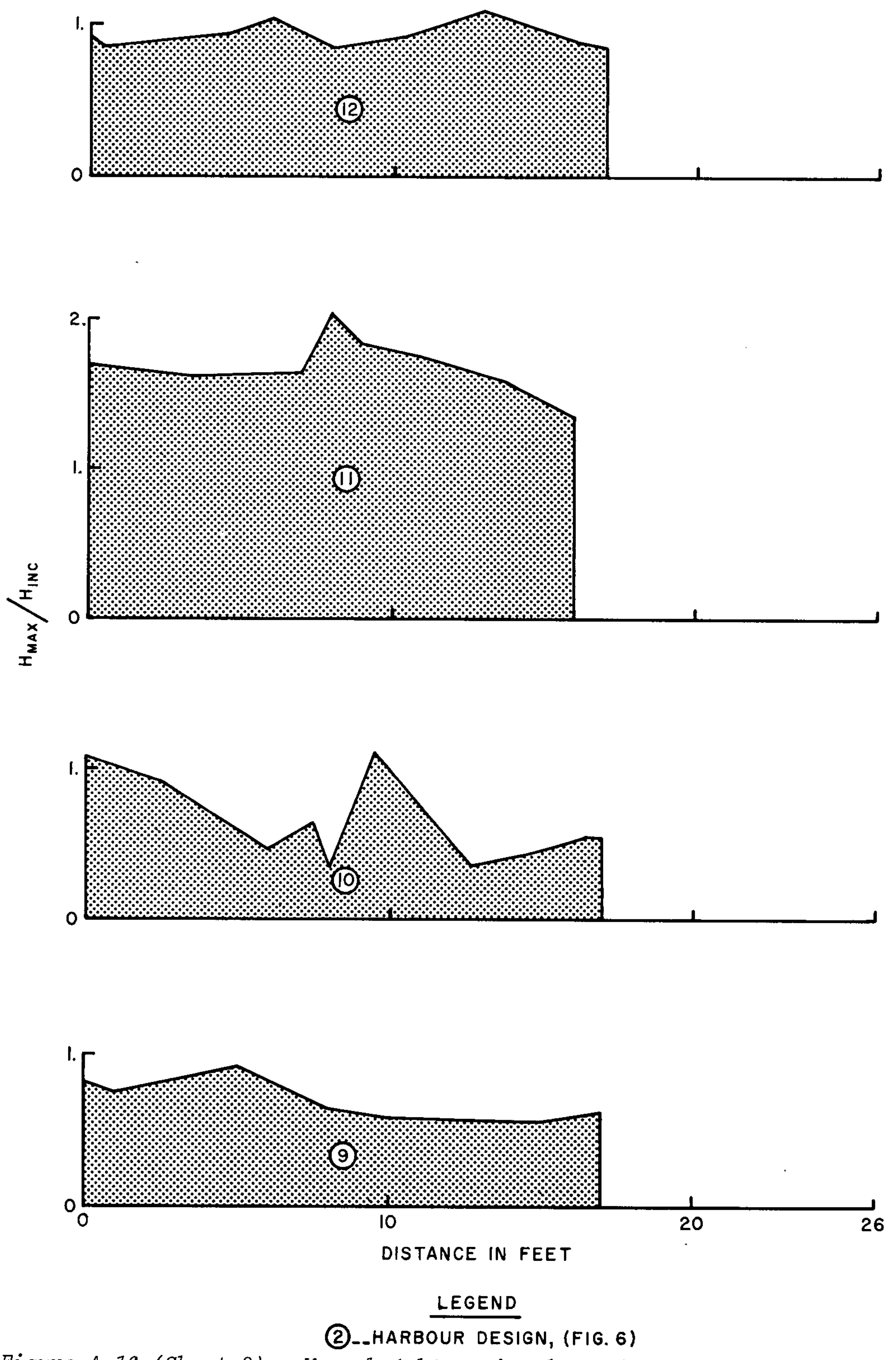


Figure A-16 (Sheet 2). Wave height ratio along wharf wall. Waves from SSE (T=0.3 sec).

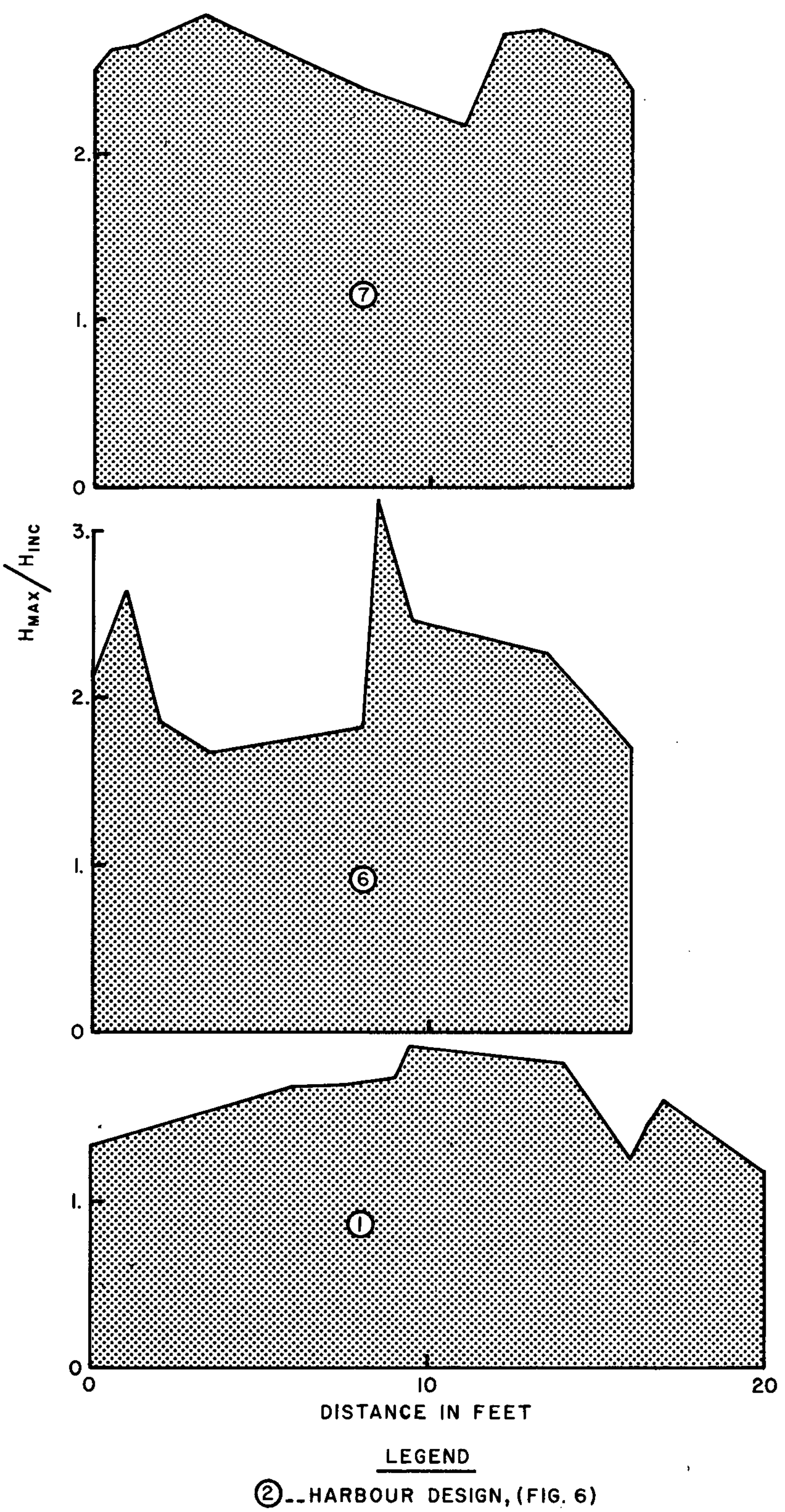


Figure A-17 (Sheet 1). Wave height ratio along wharf wall. Waves from SSE (T=0.4 sec).

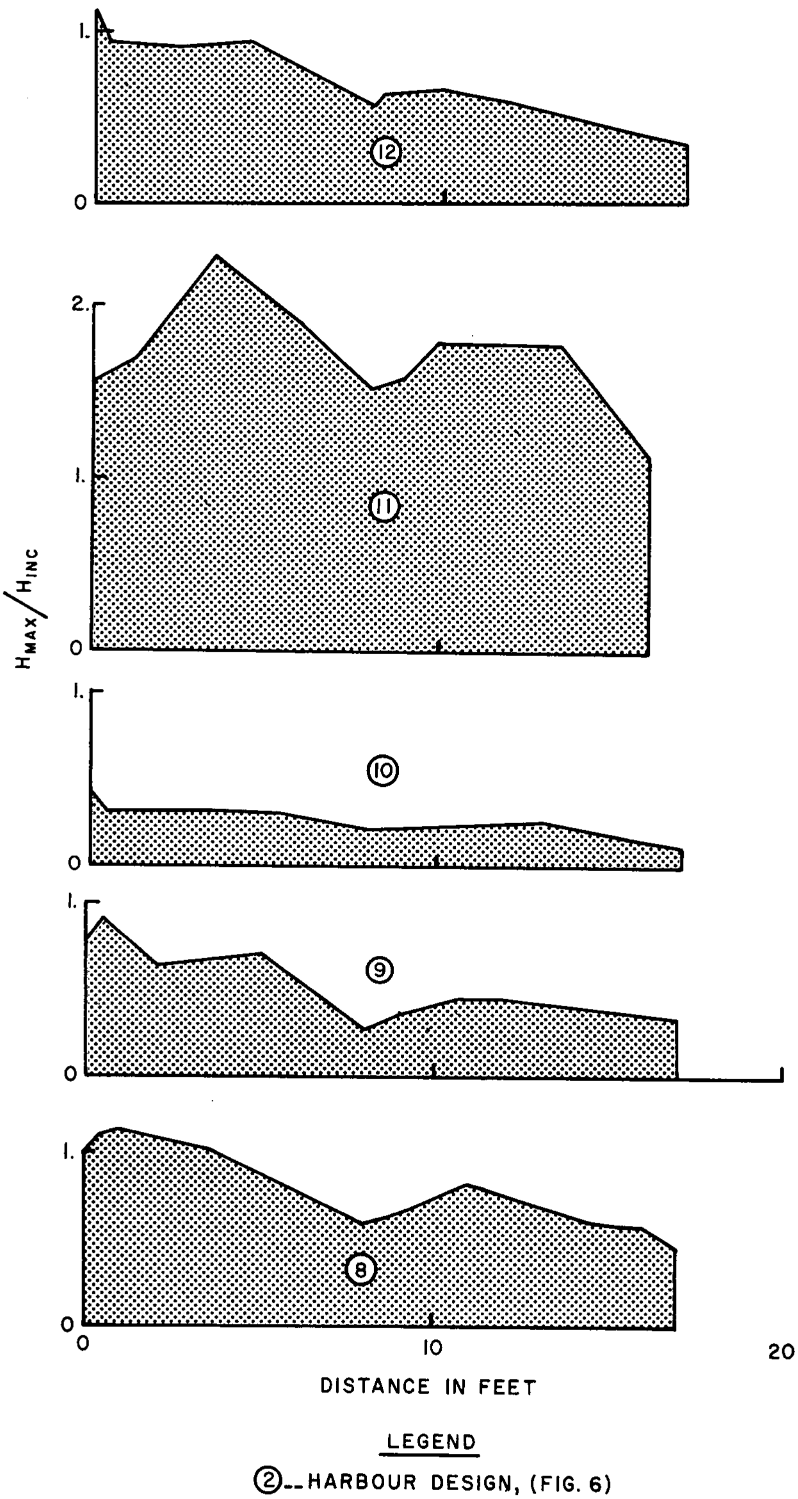


Figure A-17 (Sheet 2). Wave height ratio along wharf wall. Waves from SSE (T=0.4 sec).

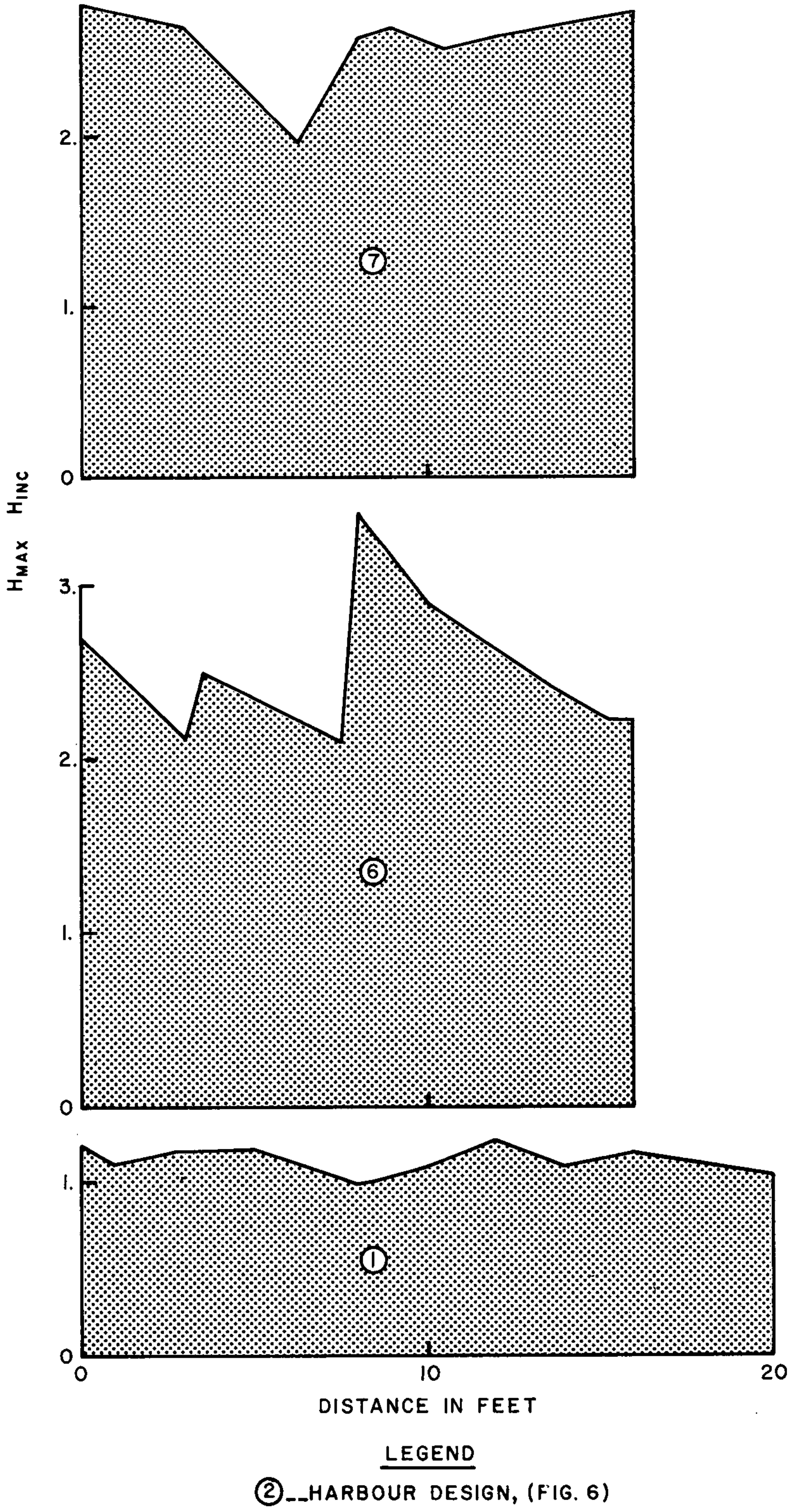


Figure A-18 (Sheet 1). Wave height ratio along wharf wall. Waves from SSE (T=0.5 sec).

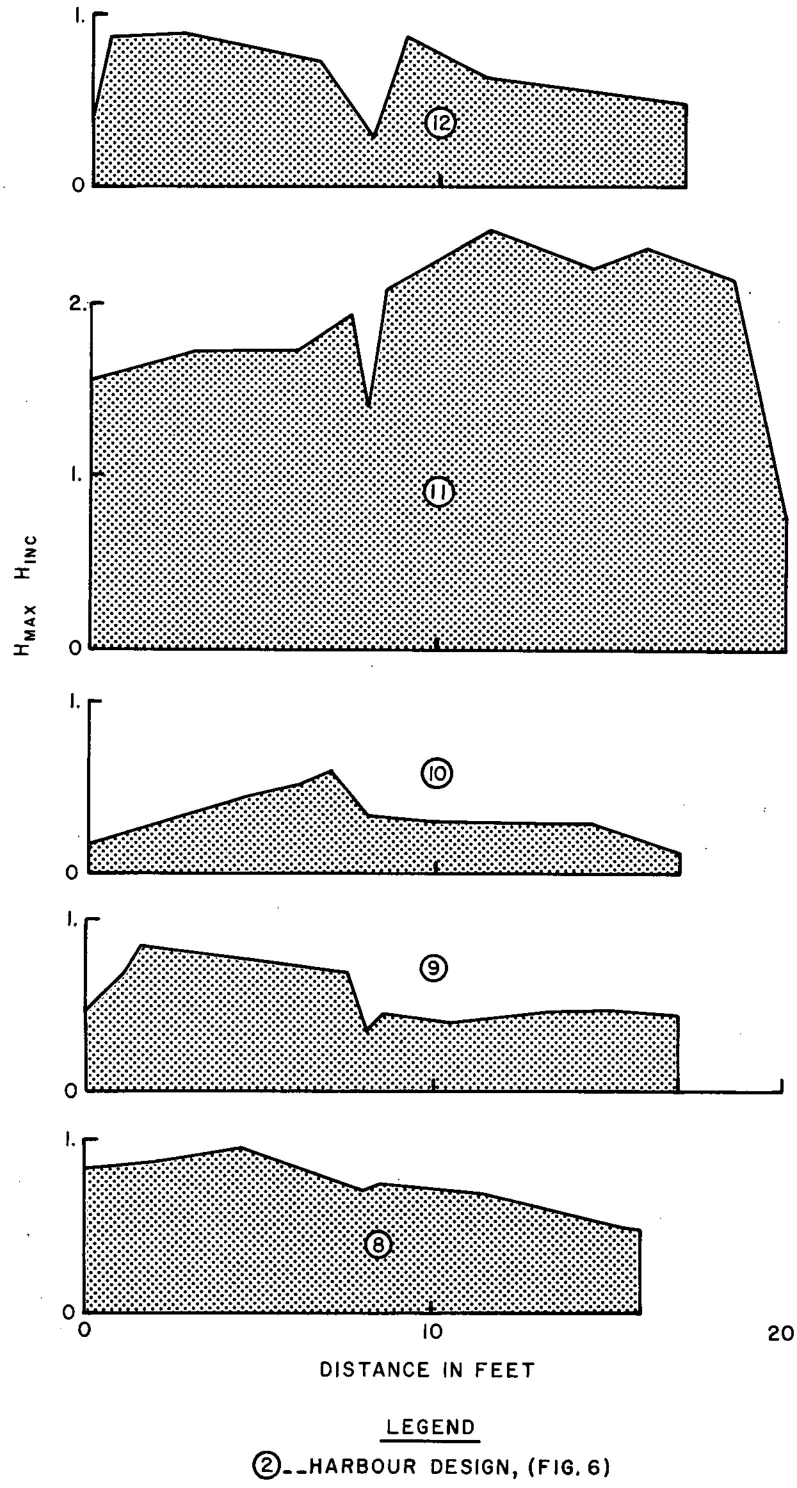


Figure A-18 (Sheet 2). Wave height ratio along wharf wall. Waves from SSE (T=0.5 sec).

7. DISCUSSION OF RESULTS

Layout No. 1

This layout is a model of the present situation and served for an assessment of the present wave agitation in the boat basin.

The results indicated an excessive wave agitation inside the boat basin; wave heights up to *1.45* times the incident wave height were recorded (see Figure A-11, Sheet 1). The highest values of wave agitation were obtained for waves from the SSE direction, which were mainly caused by wave reflection. A high level of wave agitation was therefore experienced in the open section of the boat basin (see Figure A-5), namely at the points 4, 7, 8, 10a, 10b, 11a, and 11b. Also, wave agitation was higher in the case of long wave periods (0.5 sec.) than for short wave periods (0.3 sec.).

Waves approaching the boat basin from the SW direction caused a lower level of wave agitation, due to their diffraction at the south end of the main offshore breakwater and to a multiple reflection between the main wharf and the offshore breakwater. These phenomena caused losses of incident wave energy, which were especially significant in the case of the waves of 0.3 sec. period.

A similar high wave agitation was measured along the main wharf, especially in the case of incident waves approaching from the SSW direction. In this case, measured wave height was *twice as high as the incident wave height* (see Figure 15, Sheet 1). This was caused by a combination of normal reflection and the Mach-Stem effect.

Layout No. 1 served as a standard for evaluating the other layouts.

Layout No. 2

This layout was derived from the present one by extending the south end of the offshore breakwater in the NE direction. The length of this extension was 31 inches in the model, which corresponds to 93 feet in real life.

The main reason for adding this extension was to decrease the width of the channel formed by the offshore breakwater and the main wharf and, consequently, decrease the amount of wave energy entering this channel and causing the excessive wave agitation in the boat basin. However, as the results indicate, this attempt was only a partial success. The SW waves, ratio H_{max}/H_{inc} reached values up to *1.13* (see Figure 8, Sheet 1) inside the boat basin, and *1.85* along the main wharf (see Figure 15, Sheet 1). Both maximum values indicated only a small improvement over the present situation and, therefore, Layout No. 2 was eliminated from further testing and consideration.

Layout No. 3

This layout was derived from Layout No. 2 by a further extension of the additional breakwater, i.e., doubling its length to 62 inches in model, which corresponds to 186 feet in real life (about one-half of the distance between the offshore breakwater and the main wharf).

In this design, a lesser amount of wave energy was able to enter the area under consideration. Consequently, maximum values of ratio H_{max}/H_{inc} dropped to *0.88* inside the boat basin, and *1.55* along the main wharf, in both cases, for the SW waves.

However, the reduction in transmitted wave height would not justify the high construction cost of Layout No. 3 and, therefore, this design was also eliminated from further consideration.

Layout No. 4

This layout was derived from Layout No. 3. The entrance of the channel between the offshore breakwater and the main wharf was further narrowed by erecting an additional breakwater. This second breakwater, with a length of 93 feet, was designed to extend perpendicularly from the main wharf, about 150 feet from the intersection of the main wharf and the Burlington Channel wall. It was expected that this breakwater would decrease wave agitation along the main wharf as well as contribute to a further reduction of wave energy entering the area under consideration. The first breakwater (extending from the offshore breakwater), was kept the same as in Layout No. 3.

The results for SW waves indicate that Layout No. 4 leads to a considerable attenuation of wave agitation inside the boat basin (max. value of H_{max}/H_{inc} equal to 0.53), as well as along the main wharf on the *leeward* side of the second breakwater. This layout was abandoned because it could create navigational problems for incoming ships and because of its high construction cost.

Layout No. 5

This layout was created by designing an additional breakwater which would extend perpendicularly from the main wharf in the immediate vicinity of the boat basin. The length of this breakwater would be 93 feet in real life.

This design of the boat basin protection proved to be ineffective due to the short length of the additional breakwater. Measured wave heights inside the boat basin were up to 1.23 times the incident wave height (SW wave direction). The additional breakwater also caused a worsening of wave conditions along the main wharf due to high wave reflection (values of H_{max}/H_{inc} up to 2.12). These disadvantages led to the elimination of this design.

Layout No. 6

This layout was derived from Layout No. 5 by doubling the length of the additional breakwater (i.e., 62 inches in the model, 186 feet in real life). A breakwater of this length is much more efficient in the protection of the boat basin against high wave agitation, and led to a considerable attenuation of wave agitation inside the boat basin. In case of the SW waves, maximum measured wave heights inside the boat basin were 0.68 of the incident wave height, and the corresponding value for the SSE direction was about 0.60.

However, a very strong wave agitation was formed along the main wharf due to the high reflection from the sea-ward side of the additional breakwater. For SSE waves, the wave heights were slightly greater than three times the incident wave height. For the SW waves, the measured wave heights along the main wharf were somewhat smaller, giving the maximum values of the H_{max}/H_{inc} ratio about 2.44.

Such wave conditions would possibly diminish the utility of the main wharf under certain wave conditions.

Layout No. 7

This layout was derived from Layout No. 6 by extending the additional breakwater farther in the WNW direction. This extension was expected to lead to a still better protection of the boat basin against incident and reflected waves. The total length of this structure was 7'9" in the model, which corresponds to 280 feet in real life.

Regarding the wave agitation inside the boat basin, this layout worked very efficiently for both incident wave directions. Maximum measured wave heights were equal to about one-half of the incident wave height for both wave directions; in most cases the ratio H_{max}/H_{inc} was less than 1/3.

Regarding waves along the main wharf, this layout had the same unfavourable performance as Layout No. 6, and for the same reason, *viz*, high wave reflection on the weather side.

Maximum values of the H_{max}/H_{inc} ratio were about 2.85 for the SSE waves, and 2.23 for the SW waves. Such a wave agitation would possibly disturb operation of the main wharf.

Layout No. 8

This layout was derived from the present situation by an additional breakwater, extending 21 inches perpendicularly from the main wharf in the model, which corresponds to 93 feet in real life. This structure was expected to reduce wave energy entering the boat basin area, as well as reduce wave heights along the main wharf.

For the SW waves, the layout worked efficiently, reducing the wave height inside the boat basin to about 0.5 - 0.7 of incident wave height. Wave height along the main wharf (on the protected side) attained 1.3 times the incident wave height.

However, high wave agitation was recorded in case of the SSE waves. Measured wave height inside the boat basin went up to 1.7 times the incident wave height, and the corresponding value along the main wharf was 1.47.

Neither of these latter values are acceptable.

Layout No. 9

This layout was derived from Layout No. 8. An attempt was made to reduce the wave agitation further, both inside the boat basin and along the main wharf, by doubling the length of the additional breakwater. Thus, the total length was 186 feet, or about one-half of the distance between the main wharf and the offshore breakwater.

The results were only slightly better than those for Layout No. 8.

A fair protection of the boat basin was obtained for the SW waves; maximum values of the H_{max}/H_{inc} ratio were in the range of 0.5 - 0.65. The waves along the main wharf were also attenuated considerably.

Less efficiency was found for the SSE waves. Maximum wave heights inside the boat basin as well as along the main wharf were *equal* to the incident wave height. A very strong wave agitation developed on the weather side of the additional breakwater due to the high wave reflection. (The same phenomenon was observed in case of Layout No. 8.)

Layout No. 10

This layout involved two additional breakwaters. The first was an extension of the offshore breakwater at the south end; the length of this extension was 1 foot in the model, corresponding to 36 feet in real life. The second additional breakwater extended from the main wharf at its intersection with Burlington Channel. The total length of the second breakwater was 306 feet in real life, and the south face of the breakwater paralleled the south face of Pier No. 29.

This design reduced the entrance channel to one-quarter its original width, thus reducing the entry of incident wave energy.

The results for both wave directions indicated that this layout was fairly successful.

Wave heights inside the boat basin were in most cases reduced to about one-third the incident wave height for both wave directions.

Simultaneously, waves along the main wharf were considerably attenuated. In all cases but one (SSE waves, T = 0.3 sec.), the measured wave heights were less than 0.6 - 0.7 of the incident wave height.

So far, this design seems to be one of the most expensive. A small decrease of the cost could be achieved by a modification of the layout (see Layout No. 10A in Figure 19).

Layout No. 11

This layout was created by designing an additional breakwater which would extend from the main wharf at the boat basin in the WNW direction. With a total length of 141 feet such a structure would serve as protection of the boat basin (leaving an entrance width of 60 feet), and should contribute to a lower wave agitation along the main wharf than in the case of similar layouts (Nos. 5, 6 and 7).

However, the results indicated that, while the first objective of the boat basin protection was partially achieved (H_{max}/H_{inc} ratio less than 0.5, exceptionally 0.7), the second objective was not achieved at all. Measured wave heights along the main wharf reached values more than double those of the incident wave height.

Layout No. 12

This layout followed a concept similar to that of Layout No. 10. Its main purpose was to reduce the wave energy entering the space between the offshore breakwater and the main wharf to a minimum, and to create better navigational conditions than Layout No. 10. The offshore breakwater of Layout No. 10 was extended 47 inches (141 feet in real life) in a westerly direction, and another breakwater, of the same length but extending perpendicular to the main wharf, was designed. From the navigational point of view, this arrangement seemed to be somewhat better than Layout No. 10.

Layout No. 12 was tested in six experiments employing two different wave directions (SW and SSE), and three different wave periods.

The results showed that this layout works very efficiently in the case of the SW waves, reducing the wave heights inside the boat basin to 0.30 of the incident wave height. For waves of the same direction, wave heights along the main wharf were less than one-half the height of the incident waves.

However, in the case of the SSE incident waves, wave agitation inside the boat basin as well as along the main wharf was much higher. Ratio H_{max}/H_{inc} reached values up to 0.85 inside the boat basin; corresponding values along the main wharf were about 1.10.

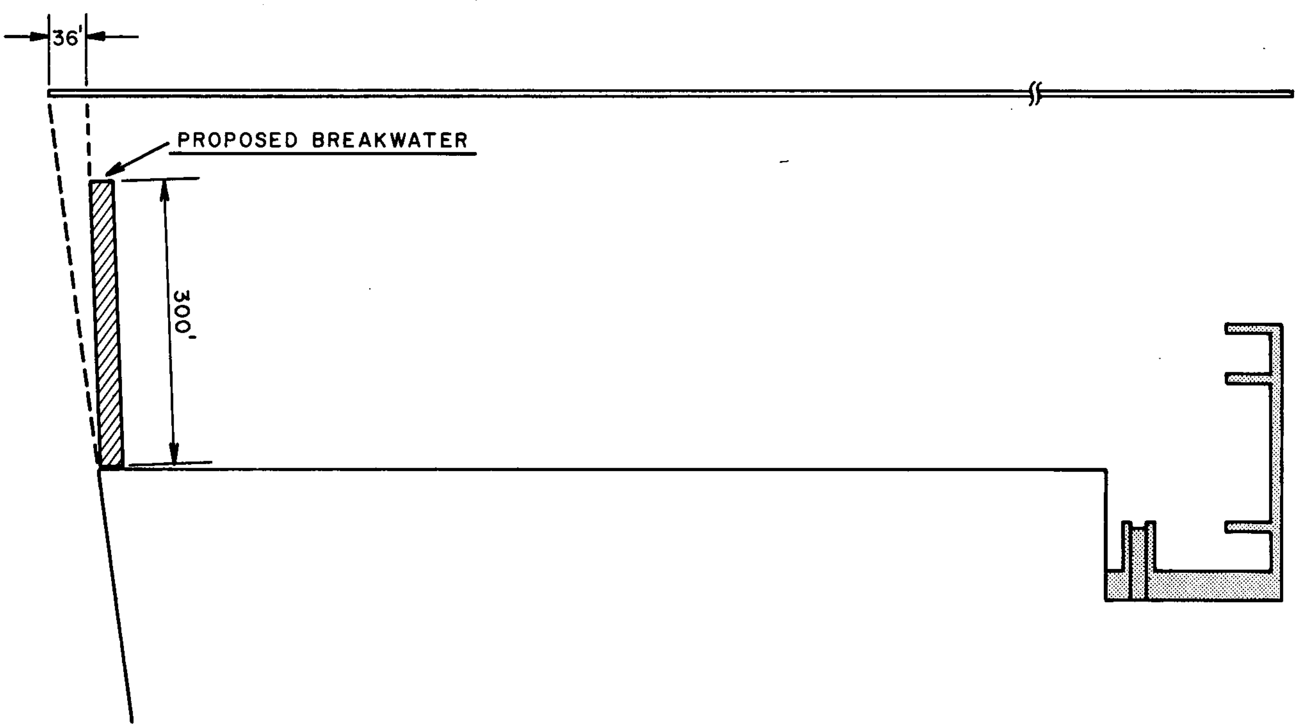


Figure A-19. Recommended layout No. 10A.

8. CONCLUSIONS

On the basis of this model study of the CCIW boat basin, the following conclusions can be drawn:

1) The main sources of excessive wave agitation in the CCIW boat basin area are incident waves approaching from directions inside the segment SW-SE.

   Neither of these winds occurs significantly more often than the other and each will cause the same level of wave motion.

2) Feasible solutions which would reduce the wave agitation inside the boat basin as well as along the main wharf can be divided into two groups:

   Group A - protective breakwaters in close to the boat basin (Layout Nos. 5, 6, 7 and 11).

   Group B - protective breakwaters located at the south entrance of the channel formed by the offshore breakwater and the main wharf (Layout Nos. 2, 3, 4, 8, 9, 10 and 12).

   A comparison of all layouts, as far as their hydraulic efficiency and costs of construction are concerned, is presented in Table A-2.

3) Layouts listed in the A-group effectively protect the boat basin against excessive wave agitation; however, they contribute to a considerable increase in wave agitation along the face of the main wharf.

4) Of the layouts listed in the B-group, only those which markedly reduce the width of the channel entrance (e.g., Layout Nos. 10 and 12), work efficiently and give the lowest wave agitation along the main wharf and also inside the boat basin.

5) Layout No. 10 appears to be the best solution for preventing excessive wave agitation, in most cases reducing wave heights along the main wharf as well as inside the boat basin to one-third of incident wave height.

   This layout was slightly modified to obtain Layout No. 10A (see Figure 19); the latter would possibly cost less and should provide the same protection as Layout No. 10.

   Unfavourable properties of both these layouts are the high costs involved and the formation of ice inside the channel formed by the off-shore breakwater and the main wharf.

TABLE A-2

Performance Order	INSIDE BOAT BASIN						ALONG MAIN WHARF					
	Waves From SSE			Waves From SW			Waves From SSE			Waves From SW		
	T - .3 Sec.	.4 Sec.	.5 Sec.	.3 Sec.	.4 Sec.	.5 Sec.	T - .3 Sec.	.4 Sec.	.5 Sec.	.3 Sec.	.4 Sec.	.5 Sec.
1	7	7	7	7	7	7	9	10	10	10	12	12
2	6	6	10	10	11	12	10	9	9	9	10	10
3	11	10	6	11	12	10	12	12	12	12	9	9
4	10	11	11	12	10	11	8	8	8	8	8	8
5	1	9	9	6	9	8	1	1	1	1	11	1
6	12	12	12	9	6	9	11	11	11	7	1	11
7	9	8	8	8	8	6	7	6	7	11	6	6
8	8	1	1	1	1	1	6	7	6	6	7	7

Estimated Overall Performance Order	Layout No.
1	10
2	12
3	9
4	7
5	11
6	8
7	6
8	1

Cost in Order of Magnitude	Layout No.
1 (lowest)	1
2	8
3	11
4	6
5	9
6	7
7	10
8 (highest)	12

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