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WIARTON MARINA
Investigation of Wave Agitation and Breakwater Protection for Marina Operation

by
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SUMMARY

The Small Craft Harbours Branch, Ontario Region, Department of Fisheries and the Environment is concerned with the development of a safe pleasure craft marina at Wiarton.

This report contains the results of an investigation into wave agitation in the existing marina and the marina with proposed remedial structures that would reduce the agitation to an acceptable level.

SOMMAIRE

La direction des ports pour petites embarcations, de la région de l'Ontario, du ministère des Pêches et de l'Environnement, s'intéresse à l'aménagement d'un port de plaisance sûr à Wiarton.

Le présent rapport comporte les résultats d'une étude sur l'agitation des vagues dans le port actuel ainsi que dans le port doté des structures de protection envisagées, lesquelles réduiraient l'agitation à un niveau acceptable.

1. Study Objectives

The Small Craft Harbours Branch, Ontario Region, Department of Fisheries and the Environment requires an assessment of existing wave agitation problems and a proposal for improved marina protection at Wiarton. An agreement was reached whereby the Hydraulics Research Division, Canada Centre for Inland Waters (DFE) and the Marine Directorate, Department of Public Works, would jointly investigate the problem using a physical model in the laboratory of the Hydraulics Research Division.

Based on the description of the wave agitation problem provided by the marine operator and on the Small Craft Harbours requirement that the significant wave height in a marina should not exceed 0.3 m (1 ft) the problem at Wiarton Marina was defined as:

> To examine the wave conditions in the marina, using a physical model, to determine the extent of the wave agitation caused by storms with northeast winds. To investigate modifications to the present breakwater, whether extensions to the existing structure or additional structures, that will reduce the waves inside the marina to less than 0.3 m for northeast storms. The proposed solution should be cost-effective.

For the purposes of this study, the marina is defined as the area enclosed by the existing Federal breakwater, McNamara's Wharf, and a straight line joining the offshore ends of these structures.

2. Study Outline

The study consists of the following stages:

1. Define the deep water wave climate at Wiarton using a wind-wave hindcast system that presents the data statistically by direction, wave height and period.

2. Study the propagation of the deep water waves into the marina using refraction estimates and a physical hydraulic model.

3. Define the wave climate inside the marina at various locations using the results of 1 and 2.

4. Investigate means of reducing the wave climate inside the marina to the defined criterion by modelling breakwaters in the hydraulic model and redefining the resulting wave climate (as in 2 and 3).

5. Present recommended designs for remedial structures at Wiarton Marina.

3. Wiarton Yacht Basin Marina

Wiarton Yacht Basin Marina is situated at the west end of Colpoy's Bay, on Georgian Bay, approximately 15 miles northwest of Owen Sound (figure 1). At the present time the marina is used by pleasure craft, a representative size being 9 m length and 2 m draft. The marina is protected by a 200 m Federal Government breakwater (built in 1890, with an extension built in 1910, figure 2). This breakwater provides protection from waves out of the northeast, the longest fetch length (figure 1).

Presently, the marina has finger docks along the west side of the breakwater, along the shore between the breakwater and a dock known variously as the Town Wharf or McNamara's Wharf (built in 1901) and on the west side of McNamara's Wharf. In addition, approximately 37 buoys are maintained within the marina between the breakwater and McNamara's Wharf.

There is a requirement to increase the number of mooring facilities, probably in the form of floating docks, out into the area between the breakwater and McNamara's Wharf and along the east side of the wharf. The operator has, however, expressed concern that the wave climate is too severe inside the marina to permit such an expansion. In particular, it has been reported that during storms from the northeast considerable wave energy gets into the marina in two ways. Firstly, waves propagating towards McNamara's Wharf are reflected off it into the otherwise protected area behind the breakwater. Secondly, waves diffract around the end of the breakwater compounding the wave agitation. The result is a confused sea state inside the marina composed of partial standing waves. The operator has also reported that in the past some yachts moored along the shore have broken loose from their moorings. The breakwater appears to provide adequate protection to those berths immediately behind it. Only spray has been reported passing over the breakwater, under the most severe conditions.

4. Wave Climate

Colpoy's Bay is protected from waves generated in Georgian Bay by several islands so that the only waves affecting the marina are generated within Colpoy's Bay. Wiarton Marina is exposed to waves generated from the south, southeast, east and northeast (S, SE, E and NE) with the severest wave conditions produced by the NE winds, the longest fetch direction.

Hourly values of wind speed and direction recorded at the Wiarton airport were obtained for the period 1963 to 1972. The corresponding hourly wave data were hindcast by a computer program developed by Public Works based on the technique developed by Bretschneider* using the recorded wind data. The hindcast system is designed to accept overwater winds recorded 10 m above the water surface. The anemometer is lcoated 10 m above ground at Wiarton airport, which is 42 metres above lake level.

The wave data were hindcast from the ten years of wind data on an hour-by-hour basis. Wave conditions are defined by the significant wave height (the average height of the one-third highest waves) and the peak period (the period of the wave that has the largest amount of energy associated with it).

The results of the hindcast analysis are presented in scatter diagrams, showing hours of occurrence of the hourly significant wave heights and periods, tables 1-5. The data have been selected only for the boating season (May 1st - October 31st) from the ten year period.

Examination of these data show that the most severe wave conditions are caused by northeast winds. The wave climate is moderate, the largest waves hindcasted have heights between 0.9 and 1.2 m (3 to 4 ft) with periods between 3.5 and 4 seconds.

Wave refraction diagrams were prepared for waves from the northeast, the longest fetch, to determine the direction of wave advance off the breakwater. For waves with periods of less than four seconds, there was no

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* Bretschneider, C. L. 1970. Forecasting relations for wave generation. Look Laboratory Quarterly 1 (3), July, 90 pp.

appreciable refraction. For 4½ s period waves there was some refraction, the waves approaching from N50^{o}E at the breakwater. This direction represents the direction of propagation for the most severe condition at the marina, and so it was decided to test the model using incident waves from N50^{o}E.

5. The Model

A review of the existing charts of Colpoy's Bay showed that there was insufficient bathymetric information for constructing the model. Therefore the Shore Properties Section, Central Region, Ocean and Aquatic Sciences was requested to survey the marina and its' approaches. A bathymetric chart was then prepared from their survey.

From this chart, an undistorted model of Wiarton Marina was constructed to a scale of 1 to 35 (Froude scaling). It was constructed as a mirror image of the prototype to fit into the hammerhead beach area of the wind-wave flume. (To avoid confusion, all figures in the report are presented as the prototype, not the mirror image layout.) See figure 2. Details of the model are given in the Appendix.

The model was tested with random waves generated by a piston type mechanical wave machine. The machine was programmed to generate waves with a JONSWAP spectrum (Hasselmann et al, 1973)* with suitably scaled peak period and characteristic wave height. (This parameter is defined as four times the variance of the wave spectrum, and is approximately equal to the significant wave height). Some of the tests were performed with wind generated waves since the wave machine was unable to model 2.5 second (prototype) waves.

Waves from the northeast with heights greater than 0.3 m occur for periods from 2.0 to 4.0 s (table 1). It was desirable to test the model under the range of conditions likely to cause problems, so that the following representative prototype wave conditions were chosen:

1. Wave period of 4.5 s, significant heights of 1.37 m, 1.07 m, 0.76 m;

2. Wave period of 3.5 s, significant heights of 1.07 m, 0.76 m, 0.45 m;

3. Wave period fo 2.5 s, significant height of 0.46 m.

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* Hasselman et al, 1973. Measurement of wind-wave growth and swell decay during the Joint North Sea Wave Project (JONSWAP). Erganzungsheft zur Deutschen Hydrographischen Zeitschrift, Reihe A (8^o), Nr. 12, 95 pp.

Two water levels were used in the experiments. Chart datum plus 1.15 m and chart datum minus 0.3 m. The first corresponds to the highest recorded level in the spring and autumn months (the months when northeast storms are most likely). The second corresponds to the lowest level recorded during the sailing season, since 1916. (Water level data were taken from the April 1977 issue of the Monthly Water Level Bulletin, Great Lakes and Montreal Harbour, prepared by the Department of Fisheries and the Environment. Chart datum is 175.8 m IGLD).

During the course of the study the model was viewed by the marine operator. He noted the similarity between the model and prototype with respect to both the wave conditions and the physical model.

6. Analysis of Wave Agitation

The measurements recorded in the model tests provide transmission coefficients, defined as the ratio of the significant wave height at a probe to the significant wave height at the reference probe, located in deep water. These transmission coefficients are applied to the hindcast wave data for each wave height and period and probe location to determine the wave climate inside the marina.

The hindcast wave data extends over a period of ten years (boating season only). It is assumed that this period of data will serve to define the average wave conditions during the boating season for which the criterion of 0.3 m is being applied. The average number of hours per year of waves of a given type are obtained by dividing by ten the values in tables 1 to 5.

The model could be tested for waves of one direction, northeast. Therefore, the transmission coefficients for the other directions (east, southeast, and south) were assumed to be one.

7. Test Results : Existing Conditions

The existing marina was tested at the two water depths and the range of wave conditions given in section 5. In general, the wave heights inside the marina were reduced to about half of the incident wave heights (tables 6 and 7). This corresponds to approximately five hours per season when the waves will exceed 0.3 m from the northeast). Due to this consistency of results it was possible to utilize one set of conditions to test the various marina improvements. Those conditions were: water level at chart datum plus 1.15 m, waves of period 3.5 s and height 0.76 m.

During the tests of the existing marina, the area near McNamara's Wharf appeared to exhibit the worst agitation of any area in the marina. This appeared to be caused by:

1. waves interacting with the wharf and propagating along the length of the wharf and

2. waves diffracting around the Government breakwater and propagating through the marina towards the shoreline and wharf.

The interaction of 1 and 2 result in complex wave reflection and a noticeable standing wave at the corner where the wharf joins the shoreline.

8. Tests Results : Remedial Structures

The remedial structures for the marina were located primarily to reduce the wave action in the area near McNamara's wharf, based on both test results and visual observations. The 44 tests have been divided into three groups according to their effectiveness in reducing the wave action within the marina.

The very effective remedial structures generally showed transmission coefficients of 25% or less at probes 3, 4 and 5 and less than 20% at probes 6, 7 and 8. These ratios correspond to virtually no waves exceeding the 0.3 m wave height. The moderately effective remedial structures generally showed coefficients of 25-30% at probes 3, 4 and 5, corresponding to four hours per season when $H > 0.3$ m, and 20-25% at probes 6, 7 and 8, corresponding no waves with $H > 0.3$ m. The poor remedial structures generally showed coefficients of 30% or higher at probes 3, 4 and 5, corresponding to five hours per season when $H > 0.3$ m and 25% or higher at probes 6, 7 and 8, corresponding to 0-5 hours when $H > 0.3$ m. It should be noted that some of the improvements were grouped with more emphasis placed on the visual observations. This is due in part to the fact that the probes were not necessarily located at the worst areas of wave action.

<u>Very Effective Structures</u> (See figure 3 and table 8):

1. 60 m offshore rubblemound breakwater plus a 20 m rubblemound extension to the Government breakwater, parallel to the wave crests. Denoted by $D_{60} + A_{20}$.

2. 50 m offshore rubblemound breakwater, parallel to the wave crests plus a 20 m rubblemound extension to the Government breakwater, parallel to the wave crests. Denoted by $E_{50} + A_{20}$.

3. 50 m rubblemound extension to the Government breakwater, parallel to the wave crests with/without a 10 m rubblemound breakwater, perpendicular to McNamara's Wharf at its tip. Denoted by $A_{50} \pm F_{10}$.

4. 60 m rubblemound extension to the Government breakwater, parallel to the wave, crests with/without a 10 m rubblemound breakwater, perpendicular to McNamara's Wharf at its tip. Denoted by $A_{60} \pm F_{10}$.

5. 30 m rubblemound extension to the Government breakwater, parallel to the wave crests, with/without a 10 m rubblemound breakwater perpendicular to McNamara's Wharf at the tip. Denoted by $A_{30} \pm F_{10}$.

6. 24 m stone filled crib+ 6 m rubblemound extension to the Government breakwater, parallel to the wave crests. Denoted by A_{30a}.

<u>Moderately Effective Structures</u> (See figures 4 and 5, tables 9 and 10)

1. 20 m rubblemound extension to the Government breakwater, parallel to the wave crests, with/without a 10 m rubblemound breakwater perpendicular to McNamara's Wharf at the tip. Denoted by $A_{20} \pm F_{10}$.

2. 20 m rubblemound extension to the Government breakwater, parallel to the wave crests plus:
   (a) rubble placed along the outer length of McNamara's Wharf.
   (b) rubble placed along curved section of shoreline and entire length of McNamara's Wharf. Denoted by A_{20} + rubble.

3. 30 m stone filled crib, extending from the Government breakwater, parallel to the wave crests, with/without rubble placed around the end. Denoted by A_{30b} + rubble/no rubble.

4. 30 m sheetpile breakwall, extending from the Government breakwater, parallel to the wave crests. Denoted by A_{30c}.

5. 24 m sheetpile breakwall plus 6 m rubblemound breakwater extending from the Government breakwater, parallel to the wave crests. Denoted by A_{30d}.

6. 30 and 40 m rubblemound breakwaters, parallel to the wave crests, centreline 35.6 m from the corner of the Government breakwater. Denoted by G_{30} and G_{40}.

7. 40 m rubblemound extension to the Government breakwater, parallel to the wave crests, with/without a 10 m rubblemound breakwater perpendicular to McNamara's Wharf at the tip. Denoted by $A_{40} \pm F_{10}$.

8. 40 and 50 m rubblemound extensions to the Government breakwater, perpendicular to the wave crests plus rubble placed along the outer portion of McNamara's Wharf. Denoted by B_{40} and B_{50} + rubble.

9. 40 - 60 m (at 10 m intervals) rubblemound extensions to the Government breakwater at a 45° angle to the wave crests with/without a 10 m rubblemound breakwater perpendicular to McNamara's Wharf at the tip. Denoted by C_{40}, C_{50}, $C_{60} \pm F_{10}$.

10. 50 m rubblemound extension to the Government breakwater, perpendicular to the wave crests plus a 10 m rubblemound breakwater perpendicular to McNamara's Wharf at the tip. Denoted by $B_{50} + F_{10}$.

11. 60 m offshore rubblemound breakwater. Denoted by D_{60}.

12. 40 m offshore rubblemound breakwater plus a 20 m rubblemound extension to the Government breakwater, parallel to the wave crests. Denoted by $D_{40} + A_{20}$.

13. 20 m offshore rubblemound breakwater plus a 20 m rubblemound extension to the Government breakwater, parallel to the wave crests. Denoted by $H_{20} + A_{20}$.

14. 30 m offshore rubblemound breakwater, 10 m on either side of (13). Denoted by H_{30} and H^1_{30}.

Poor Structures (See figure 6 and table 11)

1. 10 m rubblemound extension to the Government breakwater, parallel to the wave crests. Denoted by A_{10}.

2. 30 and 40 m rubblemound extension to the Government breakwater, perpendicular to the wave crests, with/without a 10 m rubblemound breakwater perpendicular to McNamara's Wharf at the tip. Denoted by B_{30}, $B_{40} \pm F_{10}$.

3. Rubberized animal hair mats placed along McNamara's Wharf. *

4. Rubberized animal hair mats placed along the curved section of the shoreline. *

5. Three and four combined. *

6. 20 m rubblemound extension to the Government breakwater, parallel to the wave crests plus rubble placed along the entire length of McNamara's Wharf. Denoted by A_{20} + rubble.

7. 20 m offshore rubblemound breakwater, parallel to the wave crests 40 m off the Government breakwater, with/without a 30 m offshore rubblemound breakwater. Denoted by $J_{20} \pm H^{1}_{30}$.

* These tests were undertaken to provide a better understanding of the causes of the wave agitation problem and were not engineering proposals.

9. Recommended Marina Protection

From the very effective group, the 30 m rubblemound extension to the Government breakwater, parallel to the wave crests (N40°W-S40°E) is recommended as the most suitable solution to the defined problem of wave agitation in the marina. It is shown separately in figure 7, and the results are shown separately in table 12. Although the structures designated $E_{50}+A_{20}$, $D_{60}+A_{20}$, $A_{50}\pm F_{10}$ and $A_{60}\pm F_{10}$ provided better results based on the test results and visual observations, the extra protection provided by these improvements compared to the results for the 30 m extension (A_{30}) were not sufficiently great to justify selecting one of them because of the much larger volume of material that would be required.

The 30 m rubblemound extension allows 4.8 m draft for boats entering the marina whereas the 50 and 60 m extensions allow only 4.0 and 3.6 m draft respectively (referred to IGLD). This solution requires less material than the combined offshore structures plus the 20 m extension.

The 30 m rubblemound extension provides good protection to the marina for incident significant wave heights from the northeast less than or equal to 0.76 m (2.5 ft). The number of hours of exceedance (H 0.3 m) per boating season is reduced from 4½ to 0 hours. For higher wave heights, which occurred for one hour over ten years, the wave heights at some locations in the marina exceed 0.3 m.

The 30 m rubblemound extension will not provide protection for waves approaching from the southeast and south, and will provide only marginal extra protection for waves from the east. From tables 2 to 4, it can be seen that there are, on average, 8.4 hours per year of waves over 0.3 m for all these directions combined, and one hour in ten years of waves greater than 0.6 m.

10. Conclusions

1. Within the existing marina (1977), it is estimated that the significant wave height will exceed 0.3 m for the following number of hours per season, by direction

Northeast	5 hours
East	5 hours
Southeast	1½ hours
South	2 hours

   The marina operator has only expressed concern about wave agitation problems associated with northeast storms.

2. Several breakwater designs tested in the hydraulic model were capable of reducing the agitation due to waves from the northeast to below 0.3 m.

3. The recommended remedial structure, described in this report as A_{30}, consists of the 30 m rubblemound extension to the Federal Government breakwater, parallel to the wave crests (N40°W-S40°E). It will provide suitable protection for northeast waves, limited protection for east waves, and no protection for southeast and south waves.

APPENDIX

To effectively simulate the natural wave conditions and avoid problems associated with just a few centimetres of water in the flume, the model was supported by a platform raised approximately 30 cm above the concrete floor.

The base of the platform was constructed with 5.1 cm x 25.4 cm x 3.65 m (2" x 10" x 12') kiln dried spruce treated with two coats of spar varnish for waterproofing. The base was anchored to the floor at 1.22 m (4 ft) intervals with 0.95 cm (3/8 in) lead concrete anchor bolts. The base was shimmed and planed to level the top surface to within $\pm$1.5 mm using a Wilde automatic level.

The plywood deck consisted of 1.91 cm (3/4 in) plywood treated with one coat of spar varnish and secured to the base at 0.6 m (2 ft) intervals with #10 - 6.3 cm (2½ in) screws. The deck was taken to be the 5.2 m (prototype) reference depth, except for a sunken area which was taken to be 6.2 m (prototype).

A beach at a slope of 1 to 20, constructed of plywood on an anchored framework, was added to provide a smooth transition down from the platform to the flume floor.

The contours (figure 2) were drawn at 0.2 m intervals (prototype) from the hydrographic field chart. Marsawa mahogany plywood, 0.572 cm (0.225 in) thick, were cut according to the bathymetry of the harbour, and placed one on top of the other so that the depth between two contours was level and at the depth of the deeper contour. The transition between levels was smoothed out with a rubber-latex compound applied at a 1:15 slope. The horizontal error involved after sketching, cutting and placing the plywood was estimated to be $\pm$2 cm (model).

Rubberized animal hair mats were placed at the far end of the model to prevent wave reflection, and 1.3 cm (½ in) clear gravel was placed along the inside walls of the model.

The rubblemound breakwaters were constructed with 1.91 cm (3/4") clear crushed limestone surrounding a plywood core, secured to the model. To ensure a completely impermeable structure, plasticine was used to fill in the gaps between the plywood core and the contour intervals of the model.

Single wire capacitance type probes were used to measure the waves in the model. Twenty-four gauge stranded copper wire with Teflon insulation, 1.14 mm OD, was rigidly fastened in a hole countersunk into the floor of the model. Once installed, the holes were filled with plasticine flush to the floor. The locations of the probes are shown in figures 3 to 7.

Calibration of the probes was repeated frequently throughout the tests to ensure the integrity of the data. With suitable filtering and amplification, the signals from the wave probes were digitized and stored on a PDP 11/40 computer. This computer was used for all the data reduction such as computing the significant wave height and peak frequencies.

Two cine films were made, one showing the existing marina, and one showing some of the remedial structures, and the resultant wave agitation.

TABLE I

SCATTER DIAGRAM : NO. OF HOURS OF OCCURRENCE

WAVE HEIGHT VS WAVE PERIOD

WIND DIRECTION = NE

SELECTED FROM 63/04/20 TO 72/10/31 (05/01 TO 10/31), WIARTON AIRPORT

WAVE PERIODS (seconds)

WAVE HEIGHT (feet)	0.0 to 0.5	0.5 to 1.0	1.0 to 1.5	1.5 to 2.0	2.0 to 2.5	2.5 to 3.0	3.0 to 3.5	3.5 to 4.0	A%	B%	C%
0-1	385	770	1630	1703	2884	252	9		85.01	17.29	20.34
1-2					130	982	189		14.49	2.95	3.05
2-3							41	3	0.49	0.10	0.10
3-4								1	0.01	0.00	0.00
A%	4.3	8.6	18.2	19.0	33.6	13.7	2.7	0.0			
B%	0.9	1.7	3.7	3.9	6.8	2.8	0.5	0.0			
C%	20.3	19.5	17.7	14.0	10.2	3.3	0.6	0.0			

NUMBER OF HOURS THIS DIRECTION : 8979

NUMBER OF HOURS ALL DIRECTIONS : 44147

PERCENT IN THIS DIRECTION : 20.34

A = % based on hourly records in this direction

B = % based on total hourly records all directions

C = percentage exceedance derived from "B"

TABLE 2

SCATTER DIAGRAM : NO. OF HOURS OF OCCURRENCE

WAVE HEIGHT VS WAVE PERIOD

WIND DIRECTION = E

SELECTED FROM 63/04/20 TO 72/10/31 (05/01 TO 10/31), WIARTON AIRPORT

WAVE PERIODS (seconds)

WAVE HEIGHT (feet)	0.0 to 0.5	0.5 to 1.0	1.0 to 1.5	1.5 to 2.0	2.0 to 2.5	2.5 to 3.0	3.0 to 3.5	3.5 to 4.0	A%	B%	C%
0-1	92	350	942	882	413				98.24	6.07	6.18
1-2					20	27			1.72	0.11	0.11
2-3						1			0.04	0.00	0.00
3-4											
A%	3.4	12.8	34.5	32.3	15.9	1.0					
B%	0.2	0.8	2.1	2.0	1.0	0.1					
C%	6.2	6.0	5.2	3.0	1.0	0.1					

NUMBER OF HOURS THIS DIRECTION : 2727

NUMBER OF HOURS ALL DIRECTIONS : 44147

PERCENT IN THIS DIRECTION : 6.18

A = % based on hourly records in this direction

B = % based on total hourly records all directions

C = percentage exceedance derived from "B"

TABLE 3

SCATTER DIAGRAM : NO. OF HOURS OF OCCURRENCE

WAVE HEIGHT VS WAVE PERIOD

WIND DIRECTION = SE

SELECTED FROM 63/04/20 TO 72/10/31 (05/01 TO 10/31), WIARTON AIRPORT

WAVE PERIODS (seconds)

WAVE HEIGHT (feet)	0.0 to 0.5	0.5 to 1.0	1.0 to 1.5	1.5 to 2.0	2.0 to 2.5	2.5 to 3.0	3.0 to 3.5	3.5 to 4.0	A%	B%	C%
0-1	6	250	1384	1804	328				99.63	8.55	8.58
1-2					14				0.37	0.03	0.03
2-3											
3-4											
A%	0.2	6.6	36.6	7.6	9.0						
B%	0.0	0.6	3.1	4.1	0.8						
C%	8.6	8.6	8.0	4.9	0.8						

NUMBER OF HOURS THIS DIRECTION : 3786

NUMBER OF HOURS ALL DIRECTIONS : 44147

PERCENT IN THIS DIRECTION : 8.58

A = % based on hourly records in this direction

B = % based on total hourly records all directions

C = percentage exceedance derived from "B"

TABLE 4

SCATTER DIAGRAM : NO. OF HOURS OF OCCURRENCE

WAVE HEIGHT VS WAVE PERIOD

WIND DIRECTION = S

SELECTED FROM 63/04/20 TO 72/10/31 (05/01 TO 10/31), WIARTON AIRPORT

WAVE PERIODS (seconds)

WAVE HEIGHT (feet)	0.0 to 0.5	0.5 to 1.0	1.0 to 1.5	1.5 to 2.0	2.0 to 2.5	2.5 to 3.0	3.0 to 3.5	3.5 to 4.0	A%	B%	C%
0-1	14	702	2982	3956	212				99.72	17.82	17.87
1-2					22				0.28	0.05	0.05
2-3									0.00	0.00	0.00
3-4											
A%	0.2	8.9	37.8	50.2	3.0	0.0	0.0				
B%	0.0	1.6	6.8	9.0	0.5	0.0	0.0				
C%	17.9	17.8	16.3	9.5	0.5	0.0	0.0				

NUMBER OF HOURS THIS DIRECTION : 7888

NUMBER OF HOURS ALL DIRECTIONS : 44147

PERCENT IN THIS DIRECTION : 17.87

A = % based on hourly records in this direction

B = % based on total hourly records all directions

C = percentage exceedance derived from "B"

TABLE 5

SCATTER DIAGRAM : NO. OF HOURS OF OCCURRENCE

WAVE HEIGHT VS WAVE PERIOD

WIND DIRECTION = ALL

SELECTED FROM 63/04/20 TO 72/10/31 (05/01 TO 10/31), WIARTON AIRPORT

WAVE PERIODS (seconds)

WAVE HEIGHT (feet)	0.0 to 0.5	0.5 to 1.0	1.0 to 1.5	1.5 to 2.0	2.0 to 2.5	2.5 to 3.0	3.0 to 3.5	3.5 to 4.0	A%	B%	C%
0-1	488	2382	11207	15761	9018	353	9		88.84	88.84	92.33
1-2					275	1030	189		3.38	3.38	3.49
2-3						1	42	3	0.10	0.10	0.11
3-4								1			
A%	1.1	5.4	25.4	35.7	21.1	3.1	0.5	0.0			
B%	1.1	5.4	25.4	35.7	21.1	3.1	0.5	0.0			
C%	92.3	91.2	85.8	60.4	24.7	3.7	0.6	0.0			

NUMBER OF HOURS THIS DIRECTION : 44147

NUMBER OF HOURS ALL DIRECTIONS : 44147

PERCENT IN THIS DIRECTION : 100.00

A = % based on hourly records in this direction

B = % based on total hourly records all directions

C = percentage exceedance derived from "B"

TABLE 6

TEST RESULTS FOR UNIMPROVED MARINA
WATER LEVEL = CHART DATUM + 1.15 m

Prototype Wave Conditions	$\bar{H}_R$ (cm)	$H_1/\bar{H}_R$	$H_2/\bar{H}_R$	$H_3/\bar{H}_R$	$H_4/\bar{H}_R$	$H_5/\bar{H}_R$	$H_6/\bar{H}_R$	$H_7/\bar{H}_R$	$H_8/\bar{H}_R$	
T = 4.5 s H_c = 1.37 m	3.97	0.91	0.91	0.50	0.52	0.50	0.54	N.O.	0.47	
T = 4.5 s H_c = 1.07 m	3.02	0.88	0.93	0.51	0.51	0.51	0.50	N.O.	0.42	
T = 4.5 s H_c = 0.76 m	2.02	0.96	0.86	0.49	0.57	0.46	0.53	N.O.	0.45	
T = 3.5 s H_c = 1.07 m	3.14	0.80	0.77	0.65	0.67	0.67	0.52	N.O.	0.64	
T = 3.5 s H_c = 0.76 m	2.27	0.87	0.85	0.55	0.62	0.59	0.53	N.O.	0.50	
T = 3.5 s H_c = 0.46 m	1.36	0.81	0.77	0.38	0.40	0.41	0.22	N.O.	0.30	
T = 2.5 s	2.10	0.84	0.86	0.65	0.65	0.63	0.18	N.O.	0.52	

N. O. = not operational

TABLE 7

TEST RESULTS FOR UNIMPROVED MARINA
WATER LEVEL = CHART DATUM - 0.3 m

Prototype Wave Conditions	$\bar{H}_R$ (cm)	$\frac{H_1}{\bar{H}_R}$	$\frac{H_2}{\bar{H}_R}$	$\frac{H_3}{\bar{H}_R}$	$\frac{H_4}{\bar{H}_R}$	$\frac{H_5}{\bar{H}_R}$	$\frac{H_6}{\bar{H}_R}$	$\frac{H_7}{\bar{H}_R}$	$\frac{H_8}{\bar{H}_R}$	
T = 4.5 s H_c = 1.37 m	3.94	0.92	0.96	0.49	0.55	0.51	0.50	0.25	0.50	
T = 4.5 s H_c = 1.07 m	3.31	0.83	0.87	0.47	0.50	0.47	0.52	0.24	0.44	
T = 4.5 s H_c = 0.76 m	2.26	0.76	0.80	0.50	0.51	0.50	0.46	0.20	0.39	
T = 3.5 s H_c = 1.07 m	3.27	0.82	0.74	0.47	0.47	0.51	0.46	0.19	0.41	
T = 3.5 s H_c = 0.76 m	2.37	0.79	0.72	0.48	0.36	0.60	0.27	0.16	0.34	
T = 3.5 s H_c = 0.46 m	1.23	0.91	0.75	0.43	0.51	0.46	0.12	0.13	0.25	
T = 2.5 s H_c = 0.46 m	1.79	0.83	0.82	0.48	0.47	0.38	0.09	0.10	0.29	

TABLE 8

TEST RESULTS FOR VERY EFFECTIVE REMEDIAL STRUCTURES
PROTOTYPE CONDITIONS: T = 3.5s; H_c=0.76 m; WATER LEVEL = CHART DATUM + 1.15 m

Improvement	$\bar{H}_R$ (cm)	$\frac{H_1}{\bar{H}_R}$	$\frac{H_2}{\bar{H}_R}$	$\frac{H_3}{\bar{H}_R}$	$\frac{H_4}{\bar{H}_R}$	$\frac{H_5}{\bar{H}_R}$	$\frac{H_6}{\bar{H}_R}$	$\frac{H_7}{\bar{H}_R}$	$\frac{H_8}{\bar{H}_R}$	
$D_{60} + A_{20}$	2.17	0.73	0.71	0.05	0.22	0.21	0.20	0.10	0.10	
$E_{50} + A_{20}$	2.15	0.83	0.79	0.12	0.18	0.19	0.19	0.09	0.16	
$A_{50} + F_{10}$	2.50	0.82	0.03	0.21	0.20	0.18	0.10	0.08	0.14	
A_{50}	2.44	0.86	0.03	0.21	B.D.	0.20	0.13	0.08	0.14	
$A_{60} + F_{10}$	2.26	0.80	0.18	0.20	0.14	0.19	0.12	0.09	0.17	Avg. of 2 tests
A_{60}	2.21	0.86	0.08	0.19	0.26	0.19	0.09	0.07	0.11	
$A_{30} + F_{10}$	2.19	0.93	0.87	0.27	0.25	0.23	0.14	0.11	0.20	Avg. of 6 tests
A_{30}	2.30	0.86	0.83	0.25	0.26	0.24	0.19	0.08	0.17	
A_{30a}	2.23	0.91	0.30	0.21	0.22	B.D.	0.21	0.08	0.14	

TABLE 9

TEST RESULTS FOR MODERATELY EFFECTIVE REMEDIAL STRUCTURES
PROTOTYPE CONDITIONS: T=3.5 s; H_c=0.76 m; WATER LEVEL = CHART DATUM + 1.15 m

Improvement	$\bar{H}_R$ (cm)	$\frac{H_1}{\bar{H}_R}$	$\frac{H_2}{\bar{H}_R}$	$\frac{H_3}{\bar{H}_R}$	$\frac{H_4}{\bar{H}_R}$	$\frac{H_5}{\bar{H}_R}$	$\frac{H_6}{\bar{H}_R}$	$\frac{H_7}{\bar{H}_R}$	$\frac{H_8}{\bar{H}_R}$	
$A_{20} + F_{10}$	2.33	0.81	0.79	0.32	0.26	0.30	0.23	0.12	0.22	
A_{20}	2.39	0.90	0.78	0.33	0.27	0.29	0.23	0.12	0.19	
A_{20} + rubble (wharf)	2.14	0.90	0.82	0.29	0.35	0.34	0.23	0.13	0.27	
A_{20} + rubble (shoreline & wharf)	2.09	0.89	0.84	0.29	0.40	0.22	0.18	0.10	0.19	
A_{30b} + rubble	2.21	0.96	B.D.	0.27	0.31	0.25	0.21	0.12	0.26	Avg. of 3 tests
A_{30b} no rubble	2.22	0.93	0.85	0.24	0.37	0.32	0.18	0.12	0.28	
A_{30c}	2.41	0.86	0.89	0.28	0.29	0.20	0.25	0.10	0.23	Avg. of 2 tests
A_{30d}	2.32	0.83	0.90	0.27	0.25	B.D.	0.17	0.14	0.20	Avg. of 2 tests
G_{30}	2.33	0.85	0.73	0.28	0.28	0.27	0.25	0.14	0.28	Avg. of 2 tests

TABLE 10

TEST RESULTS FOR MODERATELY EFFECTIVE REMEDIAL STRUCTURES
PROTOTYPE CONDITIONS: T=3.5 s; H_c=0.76 m; WATER LEVEL = CHART DATUM + 1.15 m

Improvement	$\bar{H}_R$ (cm)	$\frac{H_1}{\bar{H}_R}$	$\frac{H_2}{\bar{H}_R}$	$\frac{H_3}{\bar{H}_R}$	$\frac{H_4}{\bar{H}_R}$	$\frac{H_5}{\bar{H}_R}$	$\frac{H_6}{\bar{H}_R}$	$\frac{H_7}{\bar{H}_R}$	$\frac{H_8}{\bar{H}_R}$	
$A_{40} + F_{10}$	2.16	0.45	B.D.	0.29	0.25	0.25	0.17	0.10	0.20	
A_{40}	2.24	0.86	B.D.	0.25	0.29	0.24	0.15	0.09	0.20	Avg. of 4 tests
B_{40} + rubble	2.14	0.88	0.81	0.31	0.32	0.21	0.17	0.07	0.14	
B_{50} + rubble	2.17	0.82	0.78	0.31	0.29	0.19	0.11	0.10	0.17	
C_{40}	2.28	0.92	0.84	0.31	0.31	0.31	0.21	0.11	0.23	
$C_{50} + F_{10}$	2.44	0.93	0.78	0.26	0.19	0.25	0.20	0.10	0.19	Avg. of 2 tests
C_{50}	2.29	0.86	0.83	0.26	0.34	0.30	0.18	0.11	0.21	
$C_{60} + F_{10}$	2.24	0.88	0.81	0.27	0.18	0.24	0.16	0.12	0.22	
C_{60}	2.27	0.87	0.77	0.29	0.30	0.27	0.19	0.11	0.23	
$B_{50} + F_{10}$	2.38	0.88	0.82	0.28	0.22	0.24	0.12	0.09	0.15	
D_{60}	2.49	0.84	0.74	0.15	0.29	0.25	0.29	0.13	0.21	
$D_{40} + A_{20}$	2.29	0.83	0.78	0.16	0.35	0.28	0.20	0.11	0.17	
$H_{20} + A_{20}$	2.19	0.88	0.78	0.31	0.28	0.27	0.26	0.12	0.25	
H_{30}	2.20	0.87	0.84	0.28	0.33	0.27	0.23	0.12	0.25	
H'_{30}	2.17	0.87	0.84	0.32	0.32	0.27	0.24	0.12	0.26	

TABLE 11

TEST RESULTS FOR POOR REMEDIAL STRUCTURES
PROTOTYPE CONDITIONS: T=3.5s; H_c=0.76 m; WATER LEVEL = CHART DATUM + 1.15 m

Improvement	$\bar{H}_R$ (cm)	$\frac{H_1}{\bar{H}_R}$	$\frac{H_2}{\bar{H}_R}$	$\frac{H_3}{\bar{H}_R}$	$\frac{H_4}{\bar{H}_R}$	$\frac{H_5}{\bar{H}_R}$	$\frac{H_6}{\bar{H}_R}$	$\frac{H_7}{\bar{H}_R}$	$\frac{H_8}{\bar{H}_R}$	
A_{10}	2.44	0.83	0.76	0.39	0.43	0.39	0.29	0.15	0.31	Avg. of 2 tests
$B_{30} + F_{10}$	2.33	0.80	0.72	0.35	0.25	0.27	0.21	0.14	0.23	
B_{30}	2.24	0.86	0.75	0.35	0.35	0.30	0.24	0.15	0.27	
$B_{40} + F_{10}$	2.46	0.90	0.77	0.27	0.24	0.23	0.14	0.10	0.16	
B_{40}	2.30	0.86	0.78	0.36	0.37	0.31	0.22	0.12	0.25	
A_{20} + rubble	2.27	0.86	0.82	0.27	0.36	0.27	0.21	0.12	0.24	
$J_{20} + H'_{30}$	2.32	0.88	0.71	0.44	0.37	0.40	0.34	0.15	0.31	
J_{20}	2.29	0.89	0.68	0.48	0.49	0.59	0.34	0.17	0.43	
Rubberized Animal Hair Along Wharf	2.26	0.82	0.78	0.50	0.43	0.35	0.27	0.13	0.30	
Rubberized Animal Hair Along Curved Section of Shoreline	2.34	0.89	0.72	0.46	0.38	0.41	0.23	0.15	0.26	
Above two Combined	2.41	0.86	0.79	0.46	0.47	0.34	0.18	0.10	0.26	

TABLE 12

TEST RESULTS FOR RECOMMENDED STRUCTURE
(30 m RUBBLEMOUND EXTENSION) WITH RANGE OF WAVE CONDITIONS
WATER LEVEL = CHART DATUM + 1.15 m

Prototype Wave Conditions	$\bar{H}_R$ (cm)	$\frac{H_1}{\bar{H}_R}$	$\frac{H_2}{\bar{H}_R}$	$\frac{H_3}{\bar{H}_R}$	$\frac{H_4}{\bar{H}_R}$	$\frac{H_5}{\bar{H}_R}$	$\frac{H_6}{\bar{H}_R}$	$\frac{H_7}{\bar{H}_R}$	$\frac{H_8}{\bar{H}_R}$	
T = 4.5 s H_c = 1.37 m	3.69	0.88	B.D.	0.24	0.37	0.27	0.37	0.20	0.33	
T = 4.5 s H_c = 1.07 m	3.36	0.83	B.D.	0.26	0.36	0.21	0.31	0.14	0.29	
T = 4.5 s H_c = 0.76 m	1.80	B.D.	B.D.	0.23	0.31	0.21	0.30	0.12	0.24	Avg. of 2 tests
T = 3.5 s H_c = 1.07 m	3.03	0.88	B.D.	0.35	0.38	0.37	0.26	0.16	0.30	
T = 3.5 s H_c = 0.76 m	2.27	B.D.	B.D.	0.26	0.26	0.26	0.22	0.07	0.14	Avg of 2 tests
T = 3.5 s H_c = 0.46 m	1.36	B.D.	B.D.	0.15	0.18	0.21	B.D.	0.07	0.14	
T = 2.5 s H_c = 0.46 m	1.95	0.90	0.85	B.D.	B.D.	0.31	B.D.	0.06	0.39	

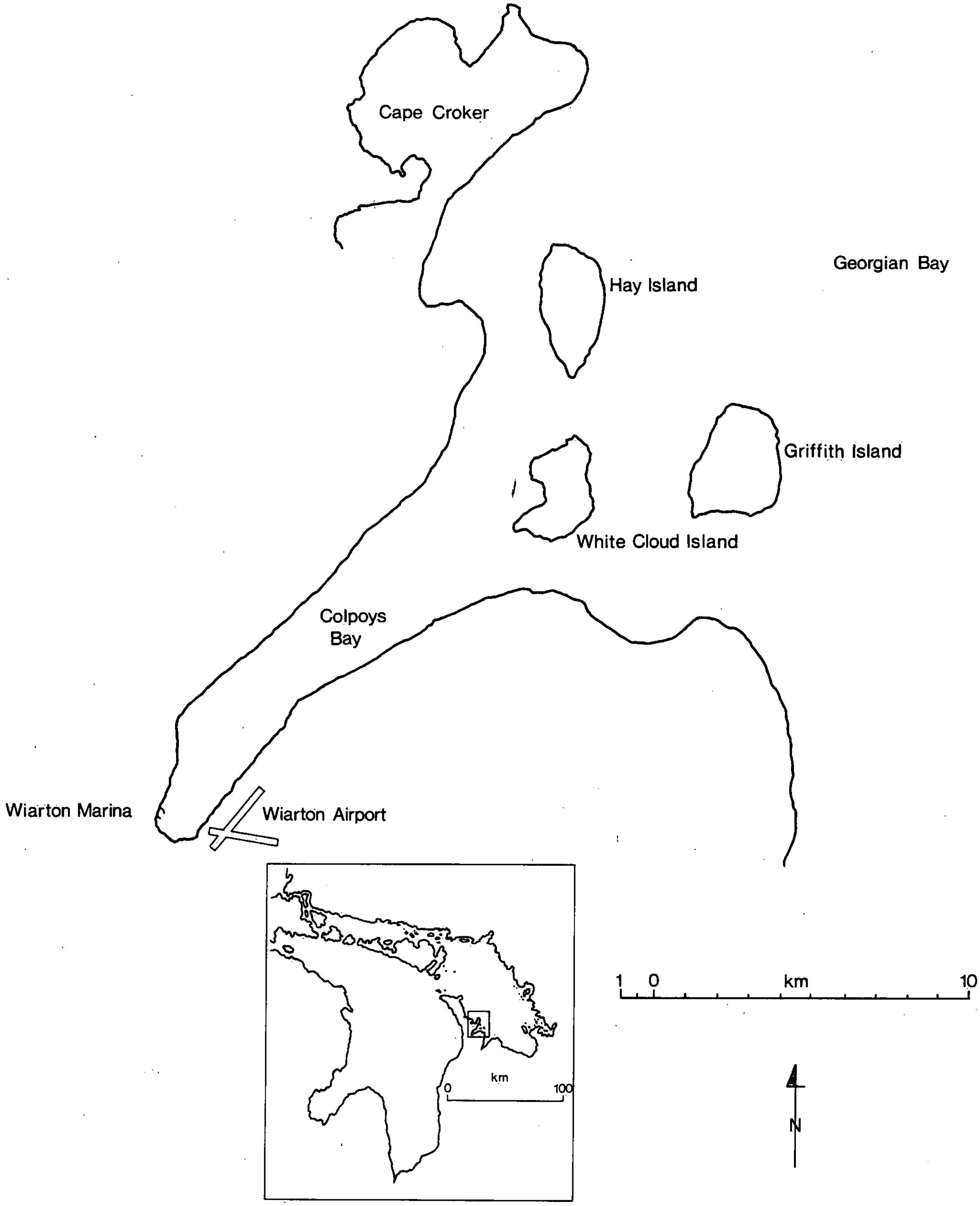


Figure 1 LOCATION OF WIARTON MARINA

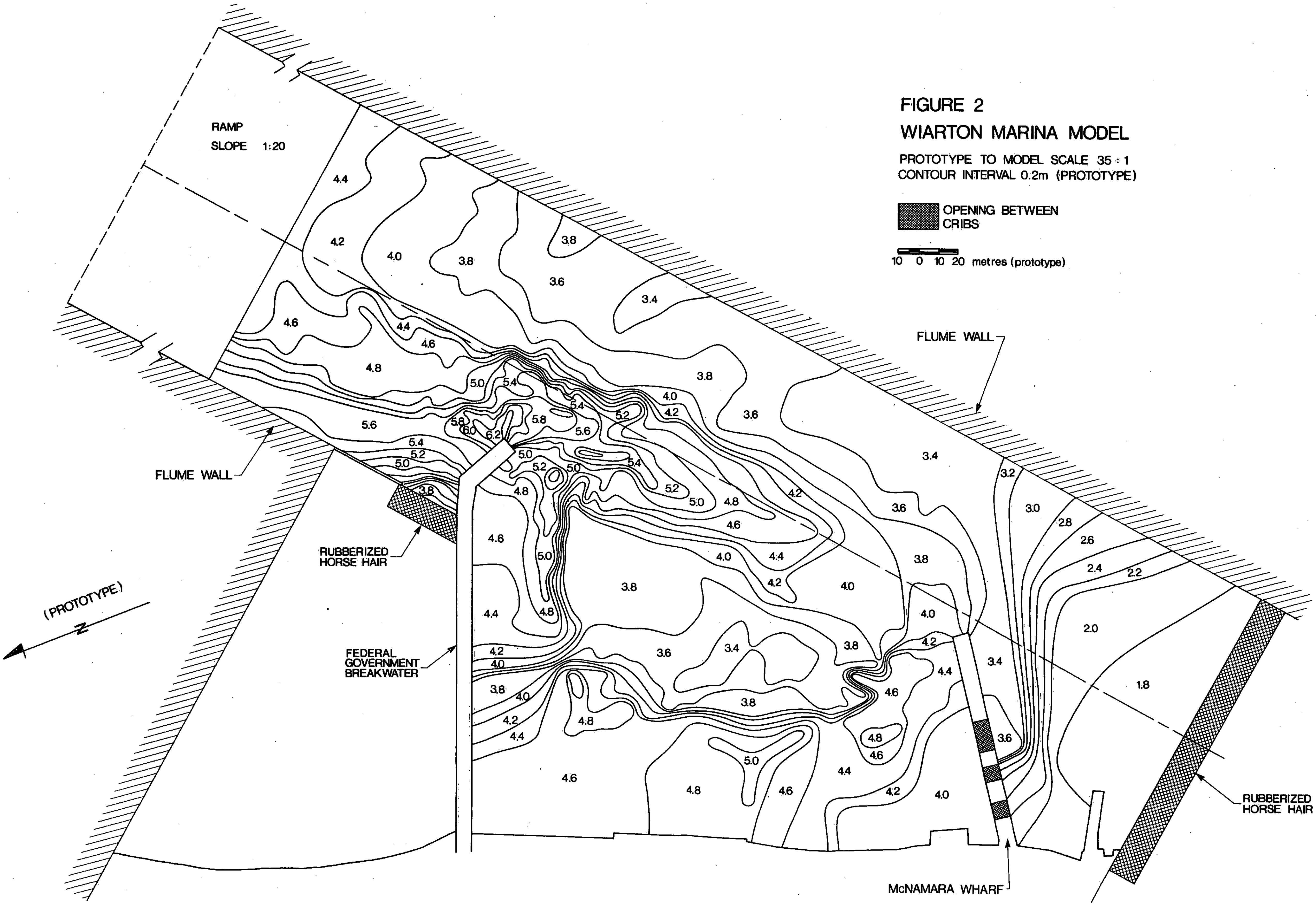


FIGURE 2
WIARTON MARINA MODEL

PROTOTYPE TO MODEL SCALE 35 : 1
CONTOUR INTERVAL 0.2m (PROTOTYPE)

FIGURE 3

VERY EFFECTIVE REMEDIAL STRUCTURES

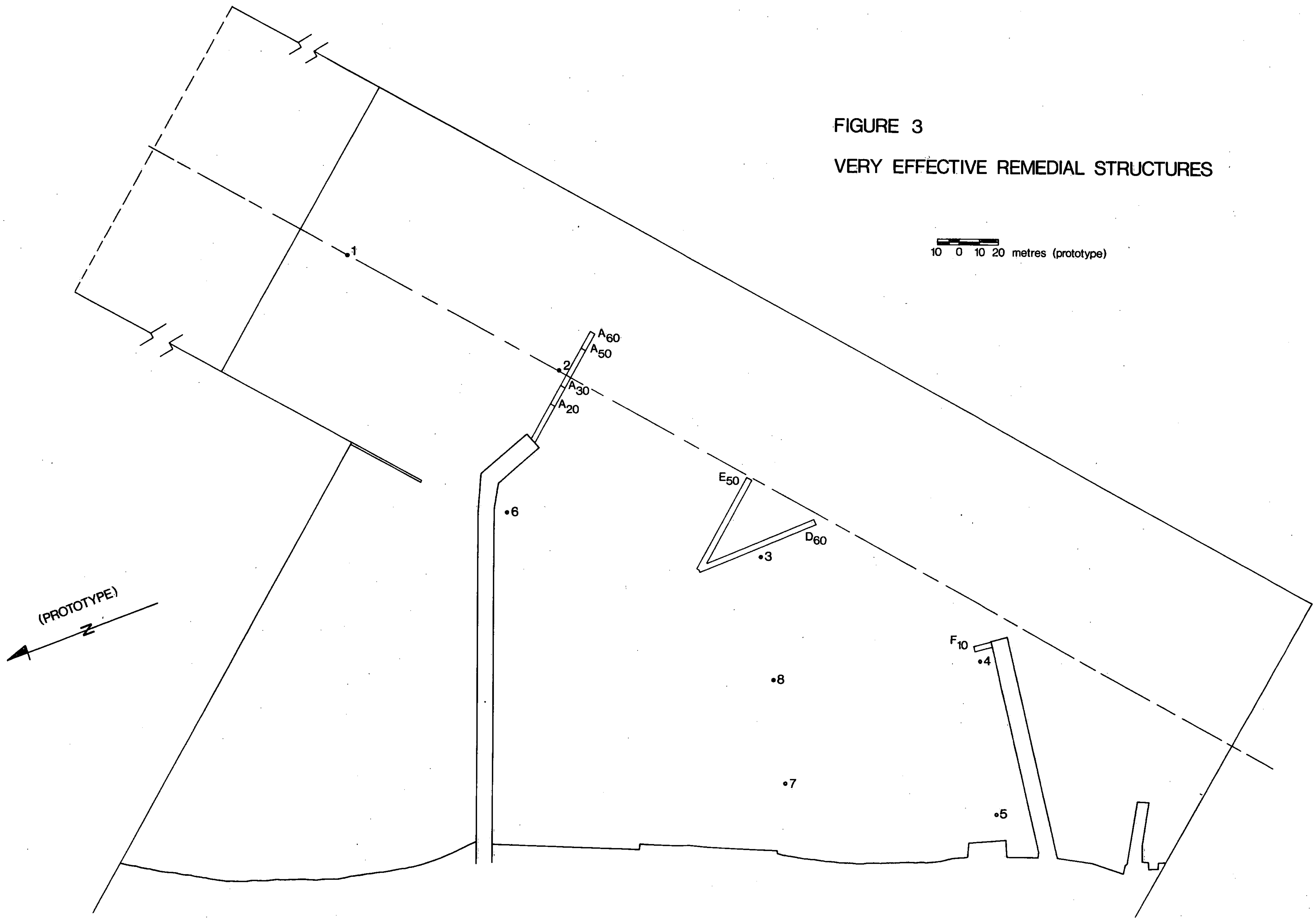
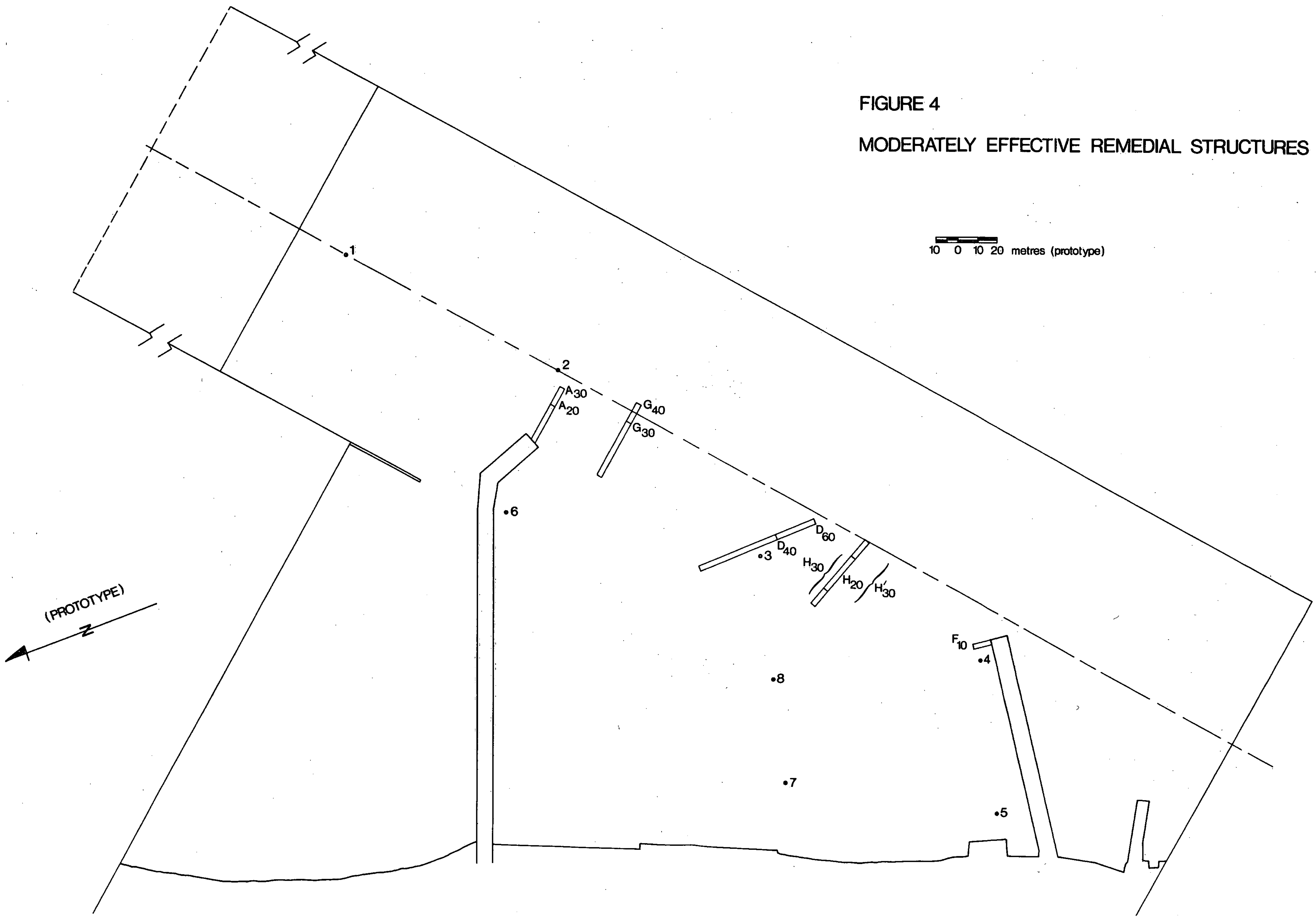


FIGURE 4

MODERATELY EFFECTIVE REMEDIAL STRUCTURES



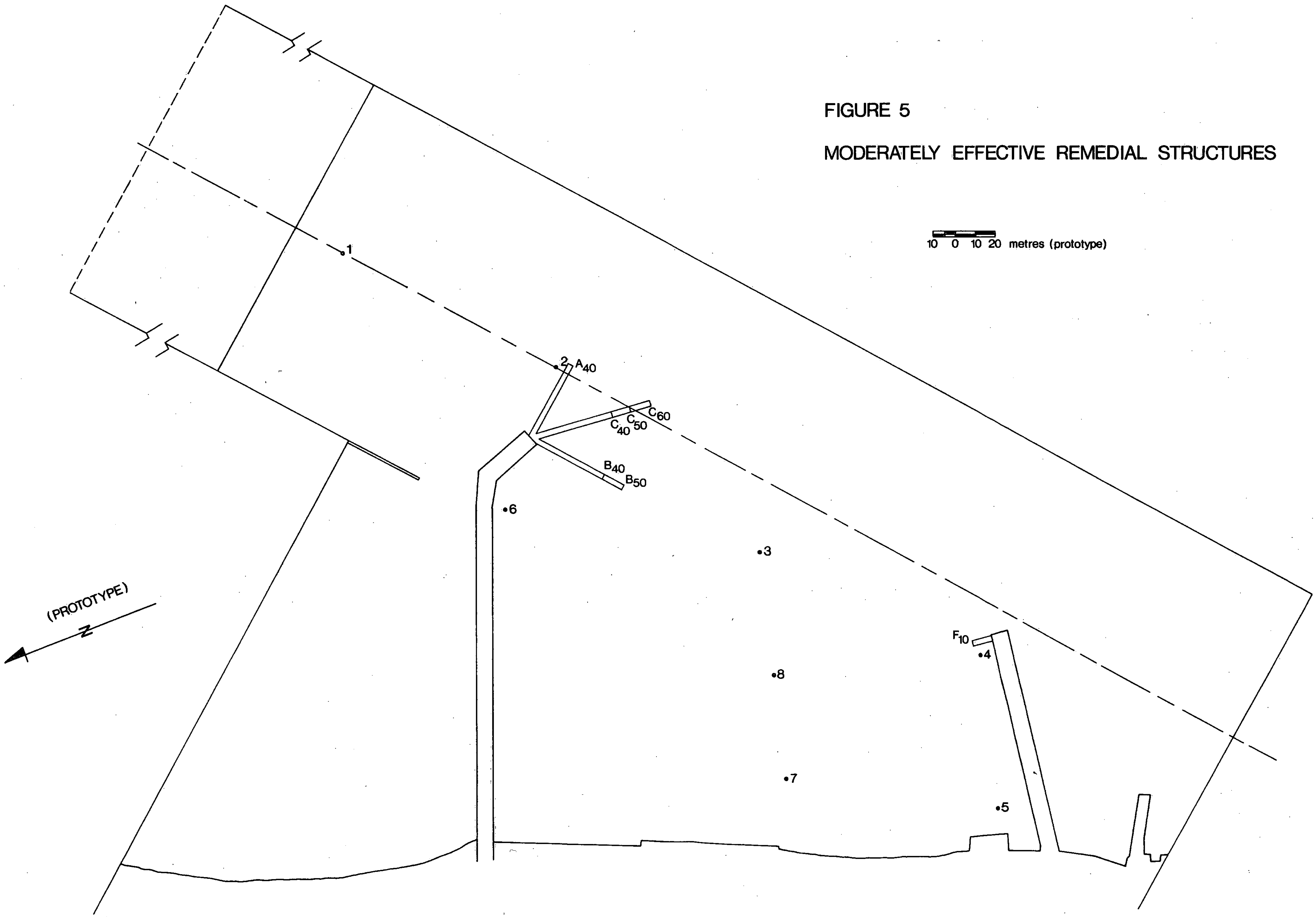


FIGURE 5

MODERATELY EFFECTIVE REMEDIAL STRUCTURES

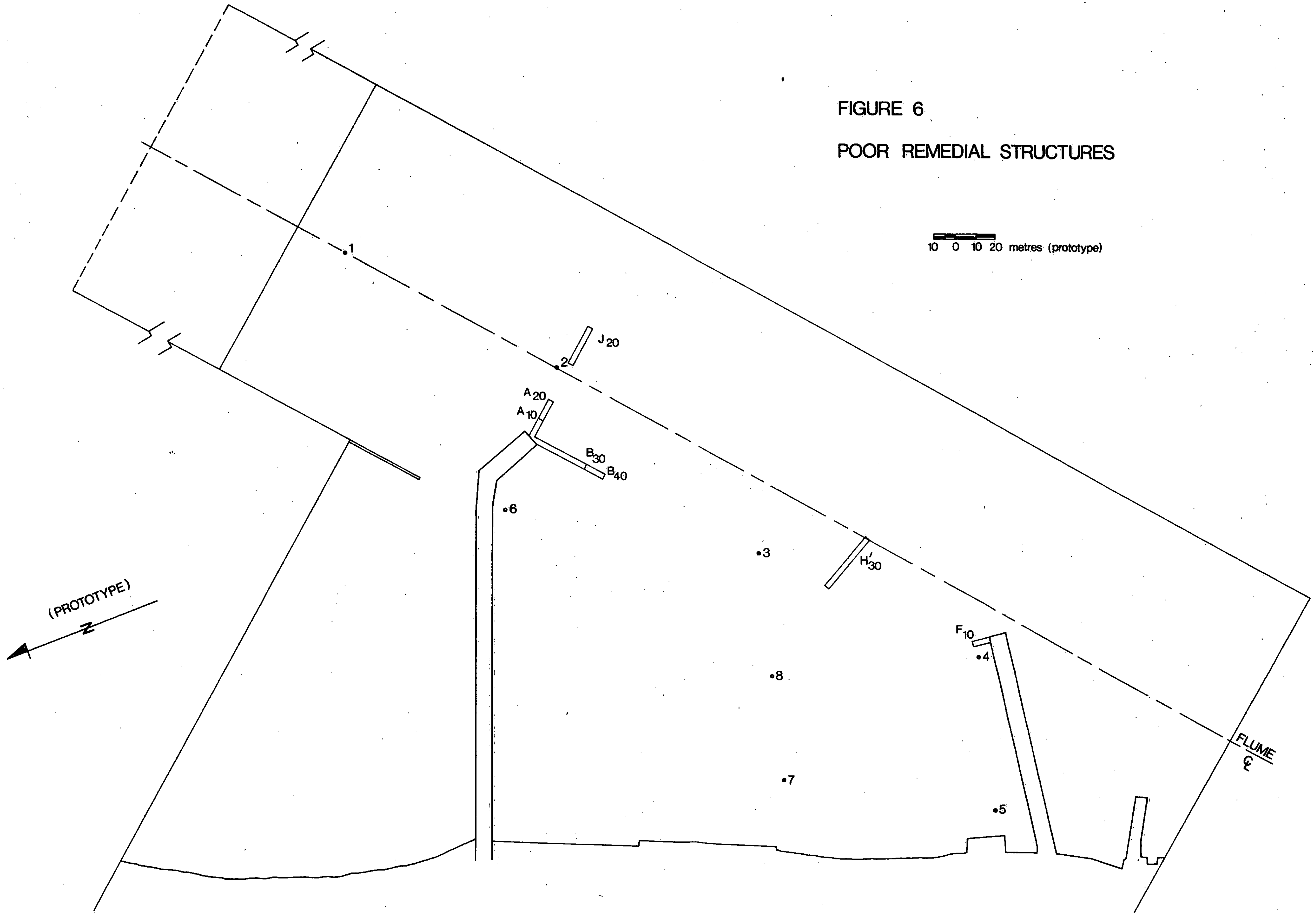


FIGURE 6

POOR REMEDIAL STRUCTURES

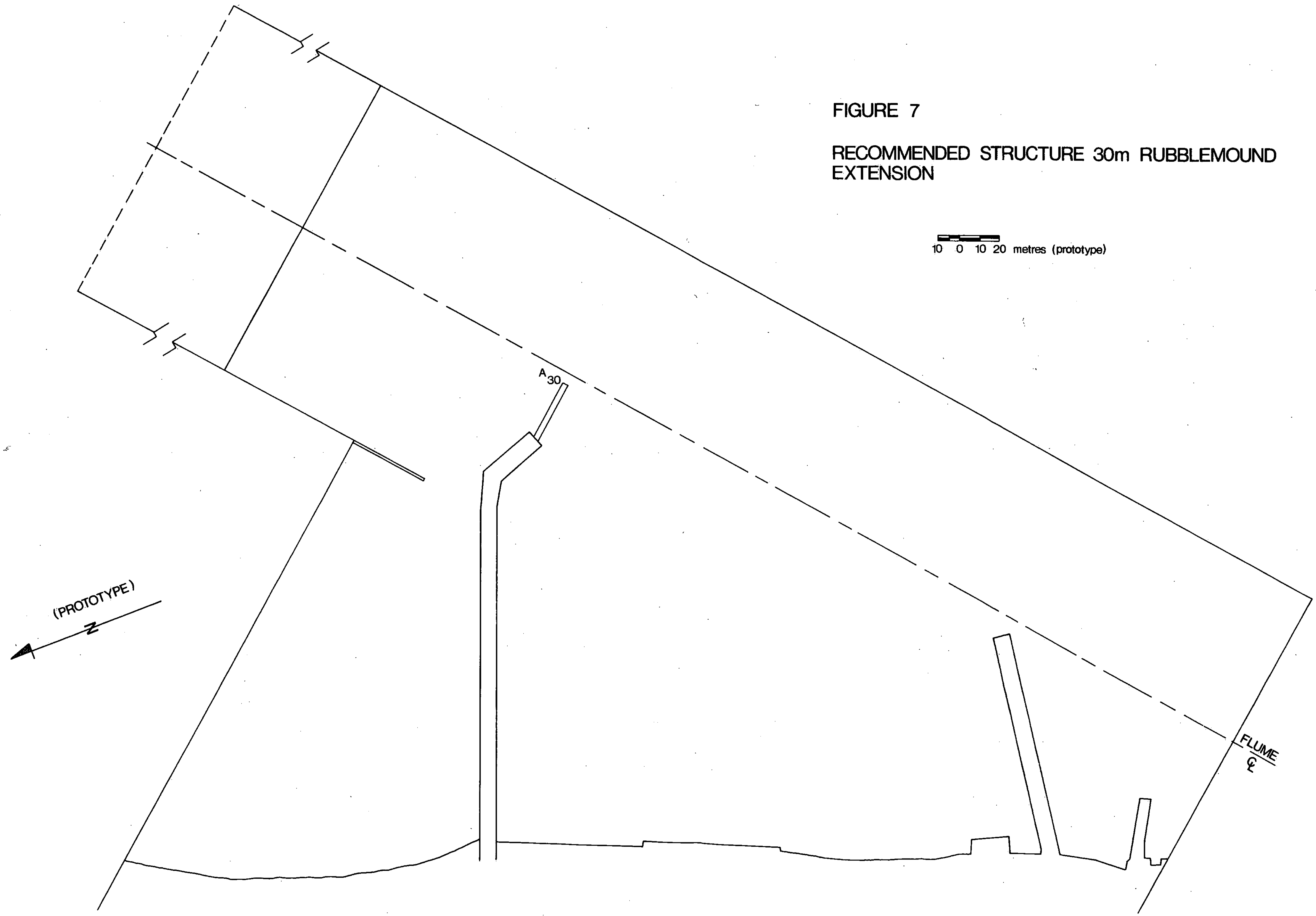


FIGURE 7

RECOMMENDED STRUCTURE 30m RUBBLEMOUND EXTENSION