

CHAPTER ONE HUNDRED ELEVEN

REDESIGN OF ENTRANCE STRUCTURES FOR TWO SMALL CRAFT HARBORS IN OHIO

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1.0 INTRODUCTION

Wildwood Harbor and Chagrin River are two areas on Lake Erie at which intensive boating activities occur throughout the recreational months of April through October. At Wildwood Harbor (Figure 1) an enclosed basin exists with a relatively open mouth which allows excessive wave activity to enter. These waves often focus directly on the launch ramp opposite the entrance. At Chagrin River (Figure 2) a major project was recently completed to stabilize the river entrance which had experienced substantial shoaling, and to help prevent winter ice blockage. The jetties and associated spending beach in the River worked well to prevent continuous shoaling and allow the river to flood and carry the ice lakeward; however, significant reflected waves resulted from this construction affecting navigation through the entrance and boats in the adjacent Yacht Basin.

2.0 HYDRAULIC MODEL CONSTRUCTION

Physical models were constructed of both sites to simulate present conditions, and to evaluate solution alternatives. Since the construction technique developed by Cubit is unique, it is described briefly below. For relatively short (6 months) model life expectancies, models are constructed from 2 ft x 8 ft sheets of styrofoam insulation board. This material is light, and easily cut and assembled, thus minimizing model construction time. The procedure used is as follows:

- (a) First, contour maps of the area to be modelled are developed with contour intervals appropriate to the styrofoam thickness. In this case the 1:24 vertical scale for Wildwood meant that a 1 inch thickness corresponds to 2 ft of depth in the prototype. For Chagrin River, the 1:48 vertical scale meant that 1 inch corresponded to 4 ft of depth in the prototype.
- (b) Based on the horizontal scales selected (1:100 for Wildwood, 1:48 for Chagrin River), equivalent 2 ft x 8 ft rectangles are drawn on the bathymetric maps of the site.
- (c) Overhead projection transparencies of the contour map are prepared.

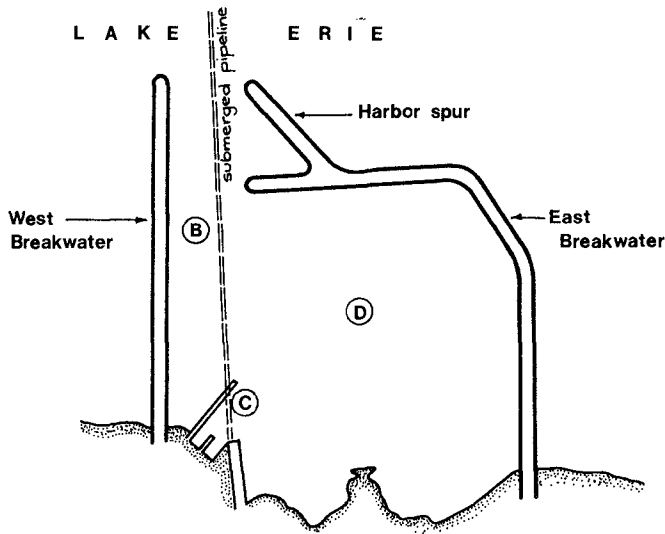


FIGURE 1. MODEL LAYOUT OF EXISTING FACILITIES
WITH DATA COLLECTION POINTS INDICATED

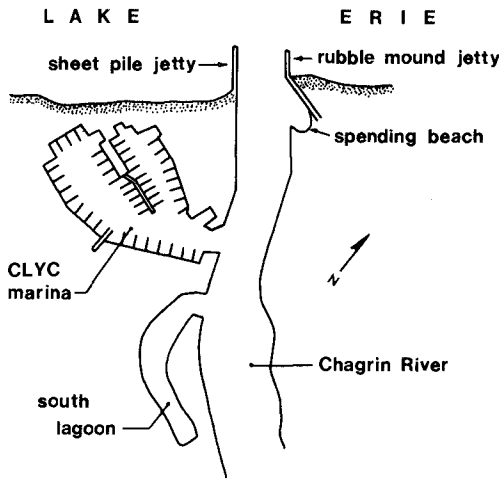


FIGURE 2. MODEL LAYOUT OF EXISTING FEATURES IN
THE CHAGRIN RIVER AREA

- (d) These transparencies are projected onto the styrofoam sheets for each contour interval.
- (e) Each individual sheet is marked and cut.
- (f) Pieces are assembled in the basin using a butyl caulk to secure their location.
- (g) The stairstep edges are smoothed with a hot nichrome wire.
- (h) Finish sanding the edges smooth.
- (i) Coat the entire model surface with a veneer of a mortar surface bonding compound (e.g. Surewall brand).

This completes the construction of the model substrate. With the addition of wave maker and plumbing for riverine flow the model is ready for instrumentation and testing.

3.0 WILDWOOD HARBOR

A physical hydraulic model was constructed for the Wildwood Park vicinity located within the Cleveland Lakefront State Park and on the shoreline of Lake Erie. The harbor was constructed in the 1950's to protect construction equipment laying a 12 ft diameter pipeline into Lake Erie for the City of Cleveland's raw water supply. Since that time the area has become a major recreational area for swimming, boating and fishing. The pipeline however still poses a problem for the final design since it runs through the mouth of the harbor.

A. The Model

Data obtained from the physical model were analyzed with the purpose of reducing the wave problem within the harbor, and in particular, near the location of the boat launching ramp. This study examined the current configuration of the harbor and tested design alternatives designed to reduce the wave action within the marina. The designs which showed the most promise were examined in further detail. Following these analyses, conclusions as to the best alternatives were evaluated based on their impact on the wave conditions in the harbor, constructability, and navigation.

A horizontal scale of 1:100 and a vertical scale of 1:24 were used in constructing the model of the harbor and the immediate offshore area to a depth of 14 ft. Distorted model scaling was utilized to insure adequate depths in the model. The model was 24 ft long and 14 ft wide. To produce the desired wave effects, a wave generator consisting of a rotating offset cylinder was employed. Wave periods from 6 to 8 seconds were used with approach directions and wave heights taken from Resio and Vincent [1976]. This resulted in a prototype wave height of approximately 12 ft and directions varying between normal to the entrance to N 25 degrees W.

The testing program proceeded through several different phases. The initial phase consisted of producing waves with prototype periods of 5 and 8 seconds. Many different design alternatives were tested and both subjective and quantitative data were collected. Visual observations were made of the waves as they traveled from the deeper offshore water between the present breakwaters and into the marina. Observations of the wave activity within the marina itself were also made. The quantitative phase of the data collection process consisted of obtaining wave heights at three separate locations within the marina. These collection points were designated as B, C, and D and are shown in Figure 3. Data were collected through the use of capacitance wave gauges and displayed on a strip chart recorder.

Following the analysis of the results from the preliminary phase, several alternatives were chosen for more in-depth study. Also, wave periods of 6 and 8 seconds were used during this phase. During this stage, varying structure lengths of the selected design alternatives were tested in an attempt to determine the optimum length.

Finally, to observe the effect of the wave approach angle on the selected design alternatives, the wave generator was placed such that it produced waves which approached the marina from a direction of N 25 degrees W. During this third stage, wave periods of 6 and 8 seconds were again tested.

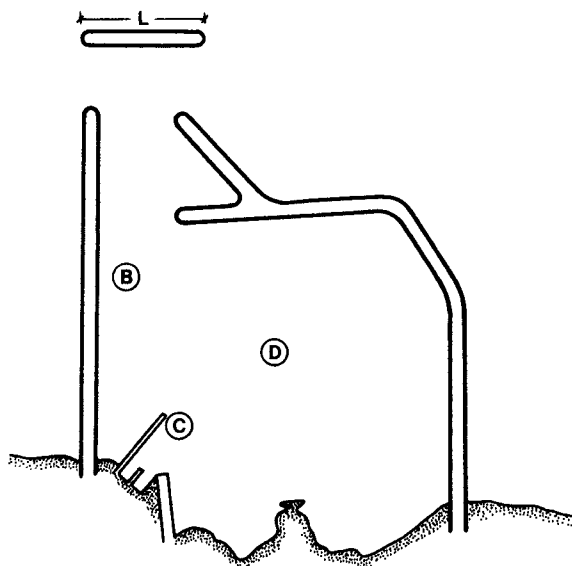


FIGURE 3. MODEL LAYOUT OF ALTERNATIVE 1: OFFSHORE BREAKWATER

B. Results

The initial testing phase involved thirteen different layouts of the breakwaters and possible additions surrounding Wildwood Park. The first test condition was that of the facilities as they presently exist and this test will be referred to as the "existing conditions" throughout the remainder of this paper.

In distorted physical models of harbors, such as this, it is impossible to effectively scale the wave heights. Because of this fact and also for ease of comparison, the wave data were non-dimensionalized with the existing wave height at each station.

Twelve alternatives were tested, however only three geometries are shown here:

- 1) offshore breakwater,
- 2) spur on west breakwater, and
- 3) west breakwater spur in combination with an east breakwater extension.

In Figures 3, 4, and 5, the geometries are self explanatory without need for additional comment. These conceptual designs exhibited significant wave height reductions, and were studied in some detail.

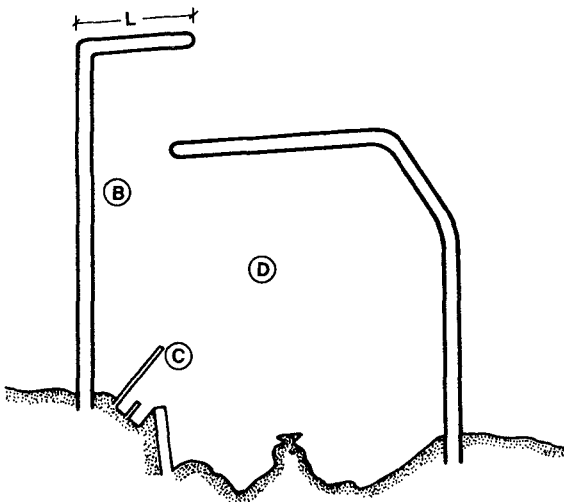


FIGURE 4. MODEL LAYOUT OF ALTERNATIVE 10: L-SHAPED WEST BREAKWATER

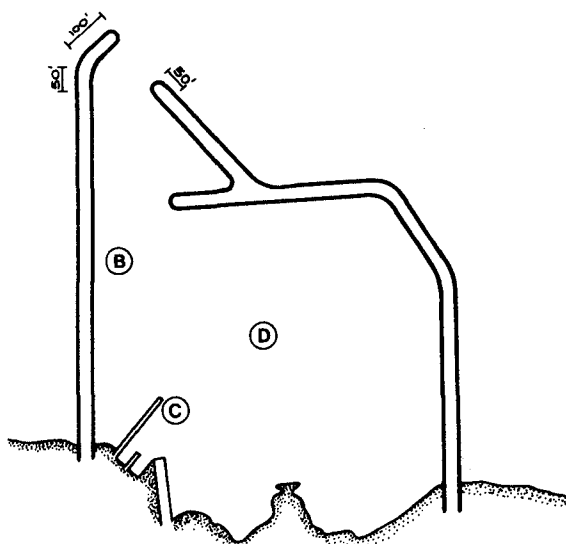


FIGURE 5. MODEL LAYOUT OF ALTERNATIVE 14: ANGLED EXTENSION OF WEST BREAKWATER

Following the initial testing of all alternatives, the three previously mentioned designs were selected for additional testing. One of the original restrictions placed on the project was the requirement that no construction take place over the water supply pipeline. The alternatives that met this requirement were generally less effective in reducing the wave activity within the harbor when compared to some of those that extended over the pipeline. As would be expected, in order to most effectively reduce the wave action, the best solution was found to be the prevention of the majority of the waves from directly entering the marina with a detached breakwater. Once the waves were allowed in the harbor, they were difficult to dissipate without taking up substantial harbor space. On the other hand, the alternatives that were laid over the pipeline and more or less shadowed the harbor entrance prevented waves from entering the harbor directly. Thus, the waves were dissipated before they entered the harbor. Because of the success obtained during the initial phase of testing, it was decided to further examine two alternatives which prevented the waves from entering the marina directly even though they called for construction to take place over the in-place pipeline.

C. The Second Tasting Phase

The next phase of the testing program consisted of the use of 6 and 8 second wave periods. From the initial testing phase, the arrangements shown in Figure 3 and 4 were determined to be the most feasible solutions to the wave problem. In an attempt to determine the optimum length of the two alternatives, the two structures were tested and examined under the above wave conditions. The various lengths tested for the offshore breakwater and for the L-shaped west breakwater are given below.

<u>Offshore Breakwater</u>	<u>L-Shaped Extension</u>
150'	50'
200'	100'
250'	150'
300'	200'
350'	250'
	300'

The offshore breakwater was positioned such that the water supply pipeline was located in the center of the new breakwater. Consideration was given in the design to a removable center section to give access to the pipeline.

Non-dimensionalized wave heights are plotted as a function of structural length in Figures 6 and 7. The results indicated that for both the alternatives the best reductions in wave activity took place when the length of the added structure equalled or exceeded the width of the harbor entrance channel. Additional lengths of the structure provided diminishing benefits once this length was achieved relative to costs.

The tests also showed that the offshore breakwater had to be slightly longer than the L-shaped west breakwater to achieve the same level of effectiveness within the harbor. Thus, for the same structure length, the west breakwater modification appeared to do a slightly better job of reducing the wave action than the offshore breakwater alternative.

This may be due to the fact that the L-shaped west breakwater allows the waves to enter from only one end whereas the offshore breakwater permits waves to diffract around both ends. Also, with the removal of the harbor spur in the west breakwater modification, this stone could be used in the construction of the new L-shaped west breakwater. Therefore, at a first glance, the volume of new stone required for alternative ten appeared to be less than that for alternative one.

D. Selection of Concept For Design

The conceptual designs developed from the model study were evaluated by the Ohio Department of Natural Resources. Their evaluation procedure considered both engineering and non-engineering considerations. They have authorized final design for a minor variation

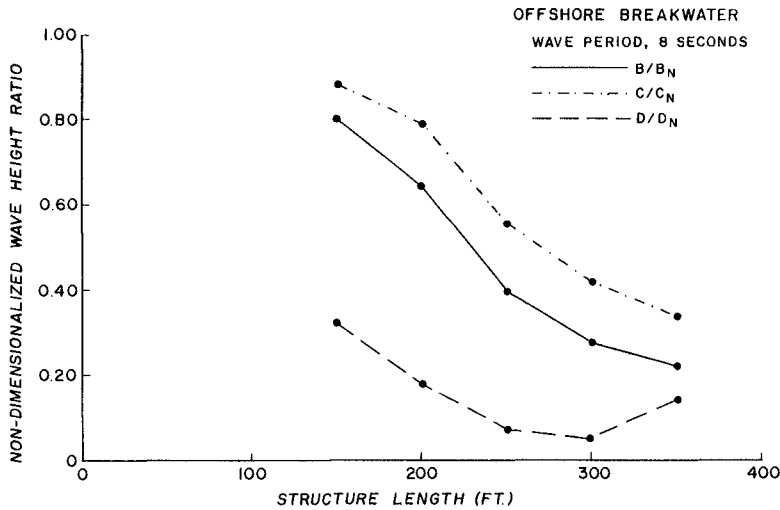


FIGURE 6. SECOND PHASE NON-DIMENSIONALIZED WAVE HEIGHTS vs. STRUCTURE LENGTH FOR OFFSHORE BREAKWATER

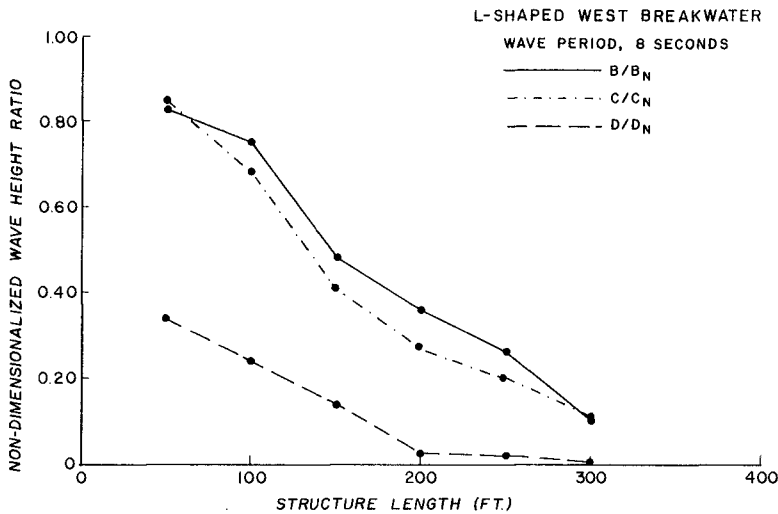


FIGURE 7. SECOND PHASE NON-DIMENSIONALIZED WAVE HEIGHTS vs. STRUCTURE LENGTH FOR L-SHAPED WEST BREAKWATER

of the breakwater configuration shown in Figure 5. This design is a necessary compromise between the most effective concept and the need to maintain access to the 12 ft diameter water supply pipe.

4.0 CHAGRIN HARBOR

A. Introduction

The Chagrin River (see Figure 2) which discharges approximately 16 miles east of Cleveland, Ohio, is stabilized on both banks with revetments, sheet piling and a spending beach. Lakeward of the spending beach are parallel rubble mound and sheet pile jetties on the east and west, respectively, which have been constructed to stabilize the shoreline and river channel. The entrance to the Chagrin River is exposed to wind and storm generated waves originating mainly from the north and northwest. These waves approach and enter the Chagrin River between the two jetties and proceed to travel upstream whereupon they enter two inland lagoons where small craft are moored. The waves entering lagoons produce a situation that is unfavorable for the berthing of boats. In fact, numerous mooring lines have broken with the subsequent damage of several boats. The sinking of one vessel has resulted from excessive wave activity within the basin.

As a part of the design to alleviate the wave problem a hydraulic model study was to examine the existing conditions in the Chagrin River and the two lagoons, to confirm model performance, and to evaluate various design configurations and modifications to determine possible solutions for protection of the two lagoons from wave action damage. The possible solutions as well as the existing condition were tested using waves of varying direction and period in an attempt to arrive at a solution that performed well under various conditions.

The Chagrin River model was constructed to an undistorted linear scale of 1:4R, model to prototype. Selection of this scale was based on reducing the effects of bottom friction, the size of the indoor laboratory facility to house the model, flow from the Chagrin River, and the need to properly simulate wave reflection in the two lagoons. To ensure accurate reproduction of reflected wave patterns, a geometrically undistorted model was deemed necessary using Froude's model law.

Available bathymetric data indicate that the harbor is uniformly six (6) feet in depth at the low water design level. In order to best simulate the actual prototype conditions with storms, the water level in the model was raised a scaled amount to represent an increase in depth of one (1) foot so that the depth of water in the main lagoon used for testing procedures was approximately seven (7) feet.

The selection of wave conditions was based on three sources of information. One was the U.S. Army Engineers report on Vermilion Harbor (1970). This report indicated that the storm waves had periods from 4 to 8 seconds with most wave action occurring with the 4 to 5 second waves. These waves generally originate from the north or northeast. A report obtained from the Chagrin Lagoons Yacht Club (3) compiled the wave conditions on the Lake, the river, and the lagoons at various times from December, 1982 through May, 1983. This was used for obtaining

predominant wave directions as well as a comparative analysis of wave heights at the various locations. A report from Stanley Consultants [1979] showed the worst wave conditions occurring in the Chagrin River area had wave periods of four to five seconds.

R. Test Program

With this available data, waves with periods of five (5) to ten (10) seconds were chosen to start the testing procedure. After examining the quantitative data and the qualitative comparisons, waves with a 5-second period were selected to best represent the conditions that create a major problem in the two lagoons. Further testing was primarily based on this wave period.

The test data obtained during the testing program included the measurement of wave heights in Lake Erie, the river channel and the lagoons. A schematic of the selected locations where wave heights were recorded is shown in Figure 8. The encircled letters indicate points where wave data were collected throughout the testing program. The letters not encircled are additional data points that were added during the testing program to give a more complete picture of the wave activity in the two lagoons.

The base tests were performed with existing prototype conditions simulated in the model and are termed "present conditions" hereafter. Preliminary tests were conducted with waves varying in period from five (5) to ten (10) seconds to test the model's response and to aid in producing worst case conditions. These results indicated that a wave period of five seconds created substantial wave activity in the lagoons. The wave attenuation in the model as the waves traveled from the lake into the lagoons was found to compare favorably with the observed wave data (3). Additionally, the wave generator was positioned directly in front of the river channel to produce waves that would travel directly up the river. It was reasoned that if a given set of test conditions can be determined to be the worst or more critical in the model, a solution that alleviates this problem should, with minor revisions, solve the problem for the lesser conditions.

The alternatives tested in the model ranged from offshore breakwaters to various spending beaches to the partial and complete closure of the lagoon entrances. For the most part, the alternatives were designed to either prevent the excessive wave energy from entering the river and lagoons or to dissipate the wave energy along the river channel. Brief descriptions of the various alternatives are given in the following subparagraphs. Alternative 1 was that of leaving the existing facilities in their present condition.

Alternative 2 consisted of the placement of a wave absorbing material along the east bank. This represented the removal of the rubble and gabion revetments in that location and the establishment of a smooth sloping beach in its place.

Alternative 3 considered the installation of a rubble mound offshore breakwater. The breakwater spanned the distance of the river channel between the two existing jetties.

Alternatives 4 and 5 involved additions to the present jetties. The two reduced the entrance channel width to reduce the amount of wave energy entering the river channel.

Alternatives 6, 12, 13, 17, and 18 considered the placement and combination of various spending beaches. The spending beaches were located on both sides of the river channel and functioned to dissipate the wave energy at those locations.

Alternatives 7 and 10 consisted of the placement of wave tripping structures along the vertical bulkhead located on the west bank. These structures served to disrupt the waves traveling along the bulkhead and in turn dissipate the wave energy.

Alternatives 8, 9, and 11 involved reductions in the entrance widths to the two lagoons. A reduction in the entrance width should lead to a corresponding reduction in the wave energy that is allowed to enter the lagoons.

Alternative 15 consisted of the placement of wave absorbing material along the entire length of the vertical bulkhead lining the west bank. This simulated the installation of a continuous wave dissipating structure.

Alternative 16 combined two of the previously mentioned alternatives. It utilized the reduction of the lagoon entrances as well as a new spending beach located on the eastern shoreline.

Alternative 19 involved the complete closure of the entrance to the main lagoon. To achieve access to the main lagoon, a channel connecting the main and south lagoons was constructed.

Alternatives 20 and 21 consisted of the placement of riprap along the vertical bulkhead on the western shore. This was intended to disrupt wave travel along the bulkhead and into the lagoons. Additionally, alternative 21 included the installation of a spending beach on the east bank.

Alternatives 24 and 25 involved the cutting of "tooth-like" indentions along the vertical bulkhead as shown in Figure 9. To reduce the reflection and increase the energy dissipation, the cuts were filled with riprap. Also, a spending beach on the east bank was included in alternative 24.

Tests involving the present conditions along with all of the alternatives were conducted during the testing program. In order to appropriately derive the effectiveness of each of the alternatives, the collected wave data were non-dimensionalized by the existing wave heights at each station. The values recorded during the existing condition were used as the denominator in the non-dimensionalization. Therefore, the non-dimensionalized value is an indication of how much wave activity will occur at a particular location as compared to that of the present condition.

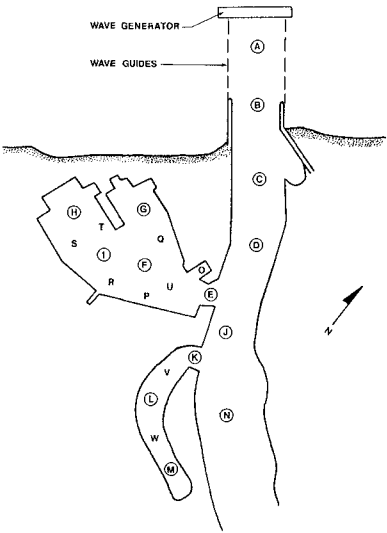


FIGURE 8. SCHEMATIC OF THE WAVE DATA COLLECTION POINTS

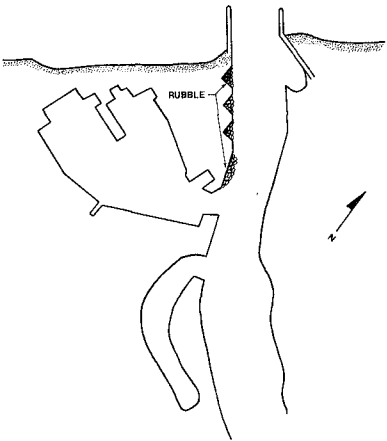


FIGURE 9. MODEL LAYOUT OF ALTERNATIVE 24: TOOTH-LIKE CUTS ALONG THE BULKHEAD WITH RUBBLE FILL

C. Results

The most striking observation made during the testing phases was the progression of waves along the vertical bulkhead lining the west bank (the deep side of the river channel) and leading into the two lagoon entrances. As the waves entered the Chagrin River between the jetties, a portion of the wave was dissipated very effectively by the existing spending beach. On the other hand, that portion of the wave traveling down the deeper western side encountered no such interference. Thus, a large amount of the wave energy was able to reach and enter the two lagoons.

Once the waves and their accompanying energy entered the lagoons, various oscillations and formations resembling standing waves were established at numerous locations. The basic reason for such activity is that once inside the lagoons the waves had no place to effectively dissipate their energy. This is due to the fact that the lagoons are lined by a vertical bulkhead which serves to reflect the waves with little energy dissipation.

Tests were also conducted in which the flow in the Chagrin River was simulated. The effects of the river flow were tested in most of the previously mentioned scenarios and for various wave periods. The results indicated that the waves in the river were steepened, and in general, the wave heights measured in the lagoons during the flow tests were the same or less when compared to the tests without flow. The wave activity in the south lagoon was significantly reduced because the river flow assisted in dissipating the wave energy in the river. Therefore, in order to create worst case conditions as far as the wave activity in the lagoons was concerned, particular attention was paid to the results of the tests with no flows.

The majority of the alternatives in varying degrees functioned to produce a reduction in wave activity in the two lagoons. Results for non-dimensional wave heights are given in Table 1 for the 5 sec condition in the main lagoon. Following the tests, it was possible to eliminate most of the alternatives based on several criteria:

1. poor overall performance or poor performance in either lagoon;
2. economically unattractive;
3. structures exposed to winter ice flows in river; and
4. potential navigation problems.

Following performance evaluations and utilizing the above criteria, Alternative 24 was considered the best overall for the Chagrin Harbor system. Since the saw tooth cuts do not extend into the river, they will not be subject to ice damage. Further, the size of the harbor entrances do not have to be reduced and thus the alternative poses no additional navigation difficulties. Finally, the solution is economically more attractive than the offshore breakwater, Alternative 3 and most important tests indicated that wave heights were reduced substantially.

TABLE 1

PRELIMINARY TEST RESULTS FOR THE MAIN LAGOON, 5-SECOND PERIOD

Non-Dimensionalized Wave Height					
Alternative	E/E_n	F/F_n	G/G_n	H/H_n	I/I_n
2	0.97	1.00	0.80	1.30	1.13
3	0.14	0.08	0.11	0.02	0.38
4	1.18	1.08	1.07	0.80	0.50
5	0.21	0.31	0.13	0.30	0.13
6	0.35	0.23	0.53	0.40	0.63
7	0.17	0.18	0.07	0.06	0.25
8	1.04	1.08	0.24	0.40	0.25
9	0.90	0.77	0.07	0.40	0.50
10	0.75	0.20	0.20	0.35	0.08
11	0.43	0.00	0.36	0.10	0.14
12	0.23	0.50	0.18	0.25	0.29
13	0.14	2.25	0.18	1.00	0.09
15	0.06	0.14	0.27	0.22	0.33
16	0.87	0.07	0.27	0.11	0.17
17	0.25	0.17	0.07	0.67	0.17
18	0.62	0.21	0.14	0.55	0.33
19	0.19	0.00	0.00	0.00	0.03
20	1.11	0.29	0.16	0.14	0.31
21	0.27	0.01	0.07	0.08	0.11
24	0.11	0.04	0.03	0.04	0.01
25	0.18	0.03	0.02	0.01	0.06

5.0 CONCLUSION

In both cases the models produced several alternatives which were presented to the Ohio Department of Natural Resources as potential solutions to the wave problems. Representatives of the local yacht clubs were invited to work with the model in confirming the natural conditions and identifying acceptable solutions. Based on various design constraints presented by the local officials and State government a single solution was selected for each site. Both projects are now in final design stage and construction should be completed for the 1985 boating season. At that time, prototype results will be available to compare with the results of the model studies.

REFERENCES

1. Resio, Donald T. and Charles L. Vincent, "Design Wave Information for the Great Lakes," Technical Report H-76-1, 1976, U.S. Army Engineer Waterways Experimental Station, Vicksburg, Mississippi.
2. Stanley Consultants, "Chagrin River Sedimentation and Stabilization Analysis," A report prepared for the Ohio Department of Natural Resources, July 1979.
3. Unpublished data from the Chagrin Lagoons Yacht Club addressing the existing surge problem. Data delivered to ODNR on May 18, 1983. A copy of this sent to Cubit Engineering Limited on May 19, 1983.
4. Brasfield, C. W., "Wave Action and Breakwater Location-Vermilion Harbor, Ohio," Technical Report H-70-5, 1970, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.