FINAL PROJECT REPORT

HYDRAULIC MODEL STUDY

WAVES AND CIRCULATION WITHIN SISTER BAY MUNICIPAL HARBOR EXPANSION

SISTER BAY, WISCONSIN

Report UMCEE 91-19

By

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For

Village of Sister Bay
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INTRODUCTION

This report describes the details of a physical hydraulic model study performed in the Civil and Environmental Engineering Hydraulics Laboratory of The University of Michigan. The model study simulated wave motion and water circulation within the municipal marina at Sister Bay, Wisconsin. Specifically, the investigation examined the effects of a proposed breakwater extension and construction of additional breakwater structures. The model testing considered water circulation as well as the relative changes in wave amplitudes within the marina due to the new construction.

GENERAL SYSTEM DETAIL

The Village of Sister Bay Wisconsin currently has a municipal marina at Sister Bay Harbor which is a part of Green Bay in Lake Michigan. The physical location of Sister Bay generally allows for waves between the directions of west-northwest to north to enter the harbor. The marina opening permits waves from the west-northwest to directly enter the marina and create excessive wave activity, interfering with the current use of the marina as a small craft harbor. Waves from other directions can reflect off sheet pile structures within the harbor and enter the marina. The existing harbor configuration is indicated in Fig. 1. As part of a marina expansion project, it has been proposed to extend the breakwater protecting the marina as well as to extend a breakwater wall from the shore to reduce the marina entrance opening with the intention of mitigating wave activity within the marina. Simultaneous concern regarding water circulation within the marina (and associated implications with respect to water quality) due to the reduction of the opening has resulted in the conduct of this physical model study to investigate the effects of the proposed design. This proposed design was supplied by the Green Bay Office of STS Consultants, Ltd. and is indicated in Fig. 2. The purpose of the physical model study was to evaluate the effectiveness of this design from the standpoint of both wave mitigation and water circulation and to investigate possible modifications to this design which might be cost effective in improving the marina response related to one or both of these criteria.

The Sister Bay harbor lies in a relatively protected section of the coastline with a straight line fetch across Green Bay existing only for waves propagating through roughly a 90 degree sector ranging from slightly north of west to just east of north. The fetch length is limited by the width of Green Bay and ranges from around 17 miles from the west-northwest to about 40 miles from the north; offshore islands and shoals may also limit the size of waves that are incident upon the harbor. Although only waves arriving from the west-northwest can propagate directly through the existing marina entrance, the considerable expanse of reflective sheet pile walls within the general harbor area allows for the possibility of significant wave energy entering the marina from other directions as well. This may be important for waves from the north since the longer fetch allows for the possibility of a larger incident wave height from that direction.

The proposed marina expansion involves an extension of the existing breakwater as well as the construction of an additional breakwater arm extending from the shoreline as indicated in Fig. 2 which indicates the modification originally provided at the initiation of this study. The intended purpose of these extension is to prevent direct passage of waves through the breakwater opening as well as to interfere with waves that might be reflected off sheet pile walls elsewhere in the harbor. An important component of this construction is the placement of riprap or armor stone along much of the marina perimeter including some existing structures near the marina entrance; this serves to dissipate wave energy and should reduce the wave climate within the marina independent of the other changes.

Water circulation within the harbor and marina will be driven by both wind and wave generated currents. Unfortunately, extensive measurements have not been made to determine their magnitude and direction. Since the harbor is relatively protected, it is reasonable to assume that the majority of the local circulation is driven by local winds and by the shoaling of waves in the shallower water of the harbor. STS (*Current Measurement, Sister Bay Area* October 9, 1990 by Water Resources Group, STS Consultants, Ltd.) performed a very limited survey on October 1, 1990 during which current magnitudes were measured with a Price current meter during a condition in which the local wind speed was about 15 mph and wave heights were estimated to be on the order of 2-5 ft. Current magnitudes were measured in the range of 1.1 to 1.7 ft/s but it appears that this may include some portion of the oscillatory wave-induced velocities as well as the longshore circulation currents. The results of a more extensive drogue study at the nearby Horseshoe Reef performed by the University of Wisconsin Sea Grant Institute (reported in a letter dated 23 July, 1990 from Phillip Keillor of Sea Grant to Terry Peterson of STS) indicated current velocities up to a maximum of 0.5 ft/s even though the maximum wind speeds were generally similar to those during the STS measurement and Horseshoe Reef is a more exposed site. Current speeds of up to 0.8 ft/s are reported for an additional fixed current meter study at Horseshoe Reef, but no information is provided as to the meter type.

The longshore currents in the vicinity of the harbor appear to create a littoral drift from the west to the east as judged by deposition near the public beach dock and other locations within the harbor. This would be consistent with waves generated by prevailing winds from the southwest and west which are common throughout the region. These would generate a clockwise circulation within the general harbor area with flow entering through the marina entrance and exiting through a circulation opening in the north end of the breakwater. At the present time, armor stone placed outside this circulation opening does not permit any significant flow and thus flow into the marina must also exit through the entrance, presumably along the north side under nearly all flow conditions.

The water quality within the marina is generally good but not well quantified. One concern regarding the proposed marina expansion is that water motion will be significantly reduced within the marina and this stagnant water condition will generally lead to a deterioration in water quality. Circulation openings in the breakwater extensions have been proposed to alleviate this concern. Since it is generally not possible to directly quantify water quality changes within the marina by means of a physical model study, the rate of water circulation within the marina is used as a surrogate. The objective of the physical model study in this regard was to provide roughly the same rate of water exchange between the marina and the rest of the harbor as existed prior to the marina expansion project.

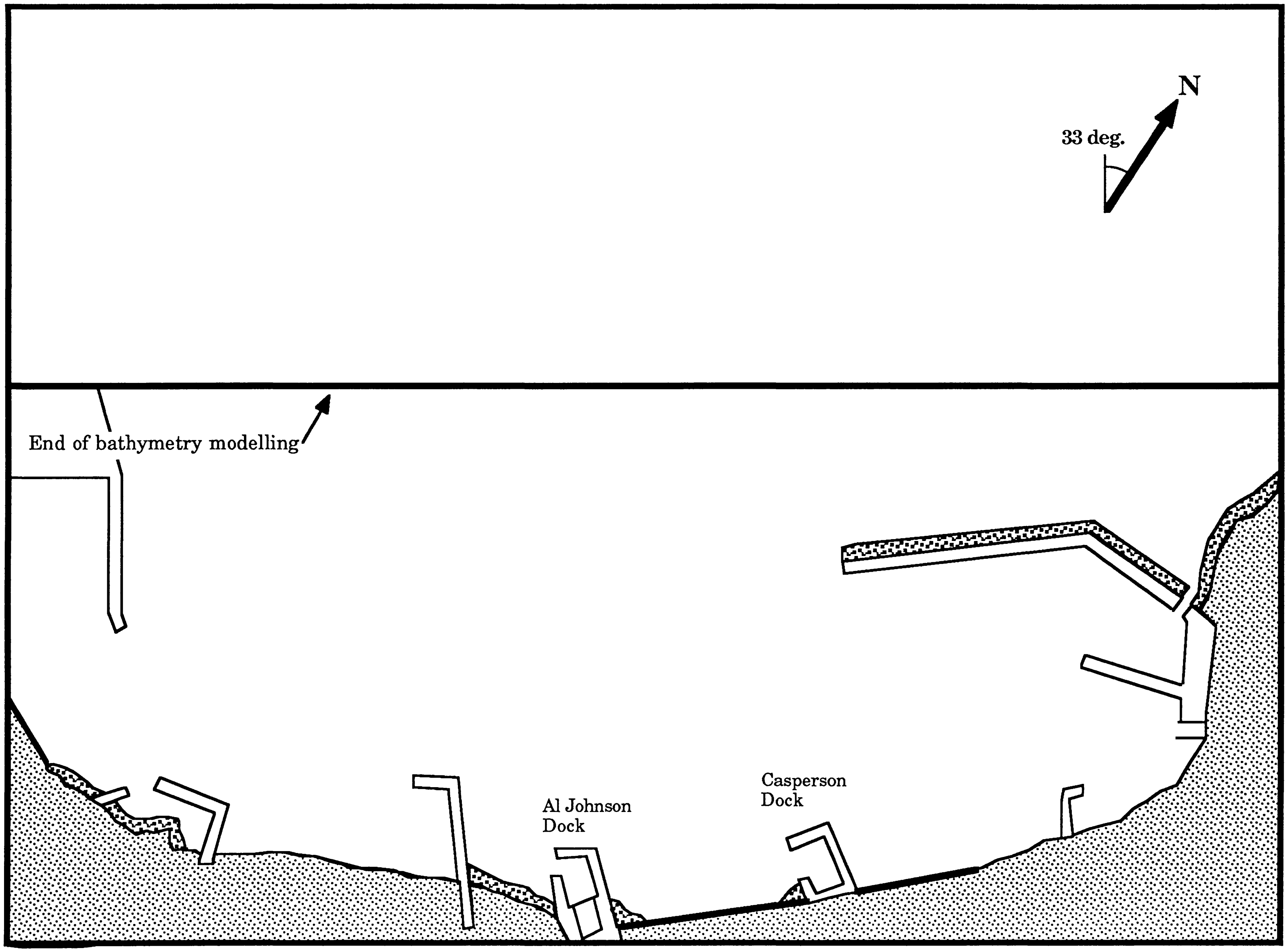


Figure 1. Overview of Existing Sister Bay Harbor

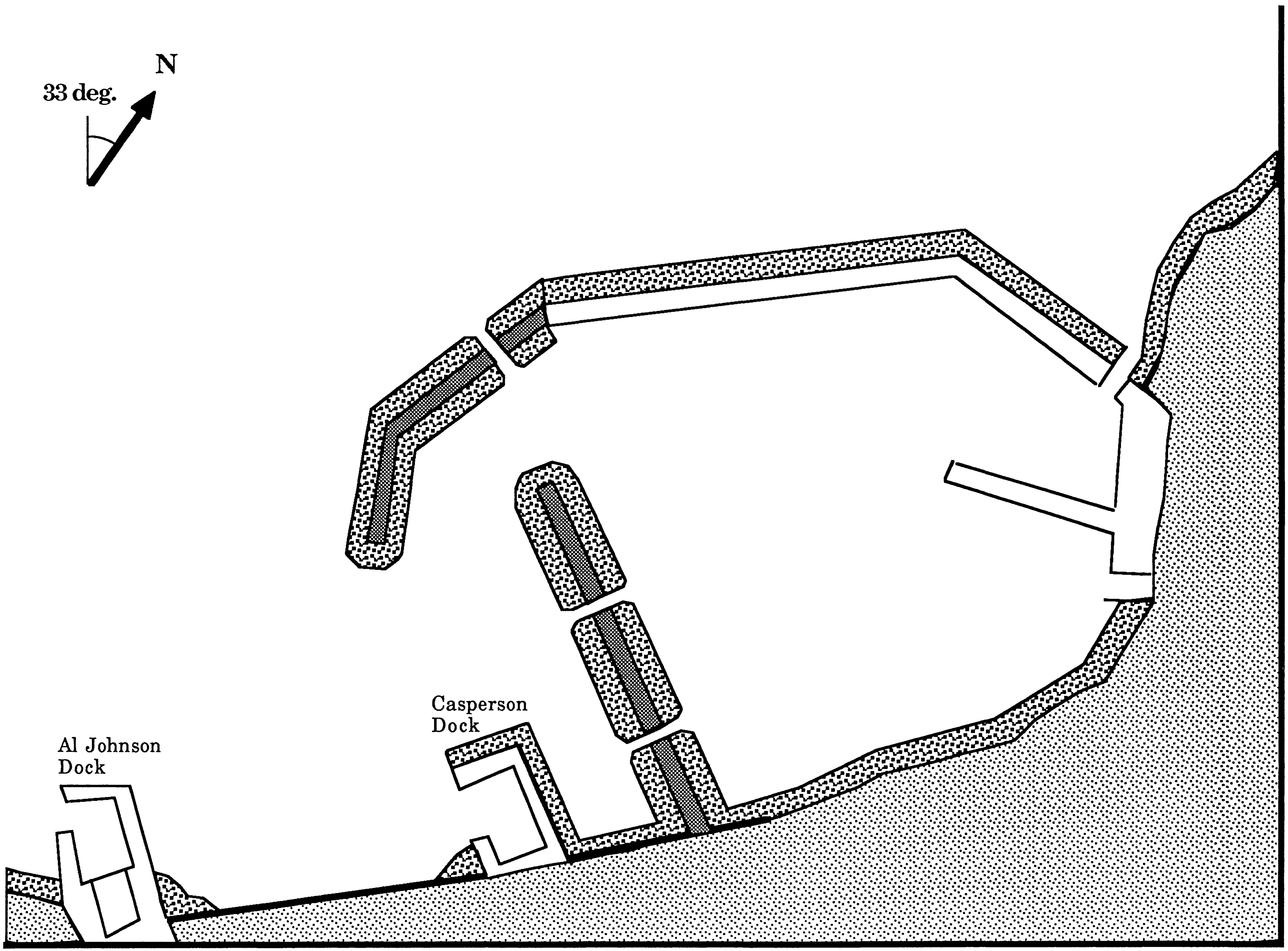


Figure 2. Proposed Marina Project Modification

CONCLUSIONS AND RECOMMENDATIONS

- Based on the wave conditions investigated in this physical model study, the proposed marina modifications can reduce the wave heights within the marina to no more than 1/5 of what they are in the existing marina. In order to accomplish this, the required marina modifications include an extension of the existing breakwater and a shore attached breakwater arm, flap gates on the proposed circulation openings, and riprap placed along the Al Johnson property. The exact configuration tested is described in more detail within the text of this report. Construction costs can be reduced by the elimination of some of the system components, but these will result in an increase of marina wave heights as described in the Results section of this report. There is no significant effect on water circulation due to the presence or absence of any of these modifications except for the breakwater extension and the circulation openings placed in it.

- The configuration tested involved breakwater extensions with a rubble mound cross-section; other configurations that have exposed vertical walls will result in more wave activity; the influence of these were not examined in this investigation.

- The height of the breakwater extensions was considered to be limited to a maximum elevation of 584 ft USGS datum. Larger waves at higher lake levels can result in significant overtopping of the breakwater and presumably larger waves within the marina; this effect was not considered in the model study.

- The existing circulation opening in the outer breakwater must be opened up to permit circulation through the marina; alternatives such as pumping water from the marina are not considered to be economically feasible at the rates of flow through the circulation opening observed in the model.

- The proposed design changes appear to somewhat adversely affect the water circulation within the marina. However, compared to the existing condition with the circulation opening blocked, decreases in marina flushing time were observed for some wave and current directions while increases were seen for other directions. The net effect depends upon the frequency of occurrence of the different wave and current directions and field observations are not available to quantify this. Given this uncertainty, it appears that water quality within the marina can be maintained with the proposed marina expansion.

• The flow conditions within the marina near the existing launch ramp are fairly stagnant under both existing and proposed conditions. The construction of a circulation opening in the internal pier is not recommended as it appears to have no significant effect on water circulation within this area. The relocation of all current water discharges into this area of the harbor (there is an intermittent stormwater discharge and a continuous dewatering discharge during the summer months) is strongly recommended unless water quality sampling within these discharges does not indicate the potential for a water quality problem.

• Although the current proposed marina modification involves the placement of five circulation openings, observations in the physical model study indicated that only two of them passed a significant flow of water. These are the two closest to the existing breakwater on the proposed breakwater extension. Although the effect of removing the others was not investigated, it is felt that they could be eliminated with little if any deterioration in marina response.

PHYSICAL MODEL DESCRIPTION

The physical model of the Sister Bay marina was constructed in the wave model test basin in the Civil and Environmental Engineering Hydraulics Laboratory at The University of Michigan. This wave basin is 35 ft by 45 ft in plan dimension. The model was laid out in such a way that the back of the harbor was oriented along the 45 ft length of the test basin. This permitted the entire harbor area to be modeled at a 1:35 scale. All relevant detail including bathymetry, sheet pile walls, armor stone, etc. were reproduced at this scale in the model. The one exception is that floating docks were not modeled to facilitate access in the model and these should have an insignificant influence on water and wave motion.

Modeling Criteria

Physical hydraulic models of wave phenomena and lake circulation are modeled using a Froude scale model, which fixes the relations between model and prototype flows once the physical model scale has been selected. Dynamic similarity requires keeping all Froude numbers defined by $V/(gL)^{1/2}$ equal in the model and prototype, where V refers to any representative fluid velocity, g the acceleration due to gravity, and L is any system length (e.g. water depth or wavelength). The relations between prototype and model parameters are related to the scale ratio L_r which is the geometric ratio between any length in the prototype and the corresponding one in the model (L_r = $\text{Length}_{\text{prototype}}$ / $\text{Length}_{\text{model}}$). For a Froude scaled model, assuming the same gravity in model and prototype, the following relations must hold:

PARAMETER		RATIO	
Length	L_r	L_r	= 35
Velocity	V_r	$L_r^{1/2}$	= 5.92
Discharge	Q_r	$L_r^{5/2}$	= 7247
Time (including wave period)	T_r	$L_r^{1/2}$	= 5.92

A critical factor with respect to physical model tests is the model size. The effect of fluid viscosity through wall friction and other effects is more pronounced in the (smaller) model than in the full scale situation. If the model is too small, viscous effects may become too great in the model and the model results will not accurately reflect the prototype condition. This consideration generally fixes the minimum model size to as large as feasible with the available facilities. In the context of the present study, critical viscous effects will be associated with the modeling of water circulation velocities and in the wave motion through the riprap. At a scale of 1:35 and with prototype water depths on the order of 5-10 feet and circulation velocities on the order of 0.5 ft/s, the circulation flows will be barely turbulent and wall and bottom friction may be overemphasized relative to the prototype condition. Within the marina where velocities are even lower, the flows will not be turbulent and this was observed in the model study. The implication is that prototype velocities within the marina are likely to be higher than what would be scaled from model observations. Furthermore, turbulent mixing in the prototype will tend to keep the water within the harbor better mixed than observed in the model. Both of these factors imply that the marina flushing time will actually be less than observed in the model. However, since the model study was a comparative one, comparing design modifications to each other and to the existing marina conditions, the general conclusions of the model study are still valid.

The exact size of the proposed riprap and armor stone was also unknown, but estimates were made based upon conventional design criteria and estimated design wave heights and model stone was approximately scaled from these estimates. Previous model tests (*Scale Effect Tests for Rubble Mound Breakwaters,* Y.B. Dai and A.M. Kamel, Research Report H-69-2, Waterways Experiment Station, Vicksburg Mississippi) that wave runup and reflection may be higher for a smaller model. This is apparently due to the fact that viscous effects retard the flow of water into the pore spaces. Approaches to alleviate this problem have included an increase in the size of the model riprap above that required for a geometrically scaled model (see e.g. *Prevention of Shoaling at Port Orford, Oregon,* C.E. Chatham, Journal of the Hydraulics Division, ASCE, Nov. 1981, pp. 1303-1316) or a reduction in the surface roughness of the model riprap which also tends to have the desired effect. Again, since the model was tested in a before/after mode (i.e. with the existing conditions as compared to any proposed changes) a proper assessment of relative performance was made. It is likely that the amount of wave activity within the marina was somewhat over-estimated in the model, but this effect is considered to be minimal.

Model Construction

The bottom bathymetry in the harbor region was reproduced in the model from a chart provided in the STS report (*Feasibility Study, Harbor/Marina Development, Sister Bay Wisconsin*, Revised June 1989, by STS Consultants, Ltd.) provided at the initiation of the project. Drawing 16092-1 in that report depicts the existing harbor conditions and this provided the detail for model construction with respect to bathymetry, location of sheet pile walls, riprap, etc. Bottom contour elevations on that chart are given to USGS datum and were reproduced to a distance of approximately 700 ft offshore (For convenience in this presentation, all dimensions are expressed as prototype dimensions rather than the scaled down model dimension). This included the entire harbor area in excess of 200 ft offshore of the breakwater in the existing marina and included water depths out to about 20 ft. Fig. 3 indicates the general area of the harbor constructed in the model along with other relevant details

The bathymetry was reproduced by first determining the bottom profiles along a series of parallel lines spaced about 75 ft apart. These bottom profiles were cut from plywood sheets such that the maximum depth in the model corresponded to the floor of the model basin. These profiles were then joined into a framework, leveled to a common base elevation and filled with sand as indicated in the photographs in Fig. 4. For those areas of the harbor in which no changes in bottom topography were required during the testing, dry cement was mixed with the top layer of sand and this was allowed to harden into a rigid surface. Within the marina and just outside, no cement was used in the sand since this was reworked during the investigation of the proposed expansion modifications. For these modifications, the marina interior was assumed to be dredged to a constant elevation of 570 ft as provided in discussions with Terry Peterson of STS. Paving bricks were placed on top of leveled sand to provide the correct bottom elevation and gaps around the edges were filled with loose sand. Some movement of the loose sand was observed during model operation, but this was relatively minimal and involved only the finest fraction of the model sand; no significant volumes were transported.

All sheet pile walls indicated in the drawings of the existing marina were constructed in the model to provide appropriate surfaces for reflection of wave energy. Wall dimensions were scaled off the provided drawings for the existing municipal marina, the Sister Bay Yacht Club, various private and public docks, and for walls constructed along the shoreline. These were generally constructed to scale from painted plywood and installed in appropriate locations within the model prior to the placement of the bottom sand. Some of the shore walls were constructed from concrete blocks. During the test phase for the proposed marina expansion, sheet pile extensions in the existing marina were generally formed from concrete blocks to provide flexibility for the convenient insertion and removal of walls as the testing program proceeded. Since most of these walls were covered on both sides by armor stone or riprap, the exact details of the walls such as thickness were not felt to be critical.

Once the existing sheet pile structures and bottom topography had been reproduced in the model, armor stone was added to the model in those locations that were indicated to be protected. These included the outside of the existing marina breakwater and various locations along the shoreline in the back of the harbor. Two different sizes of armor stone were used in the model; one had a maximum dimension of approximately 3.5 ft while the other was somewhat smaller with a maximum dimension of 2 - 2.5 ft (these are the long dimension of the largest stones; some others may only be half this size). At the time of the site visit to Sister Bay in May, 1991, it was observed that the armor stone on the

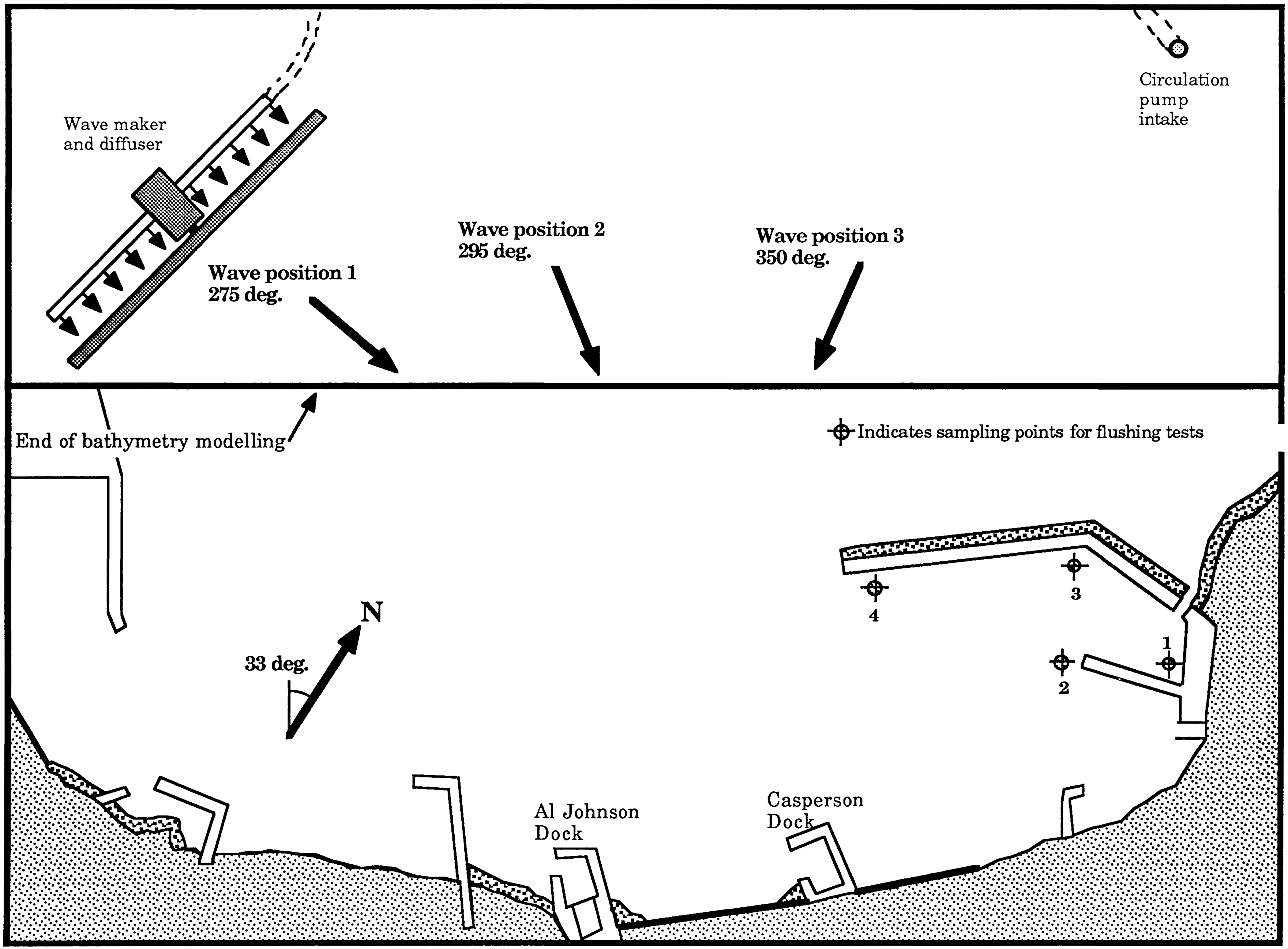


Figure 3. Layout of Physical Model in Test Basin

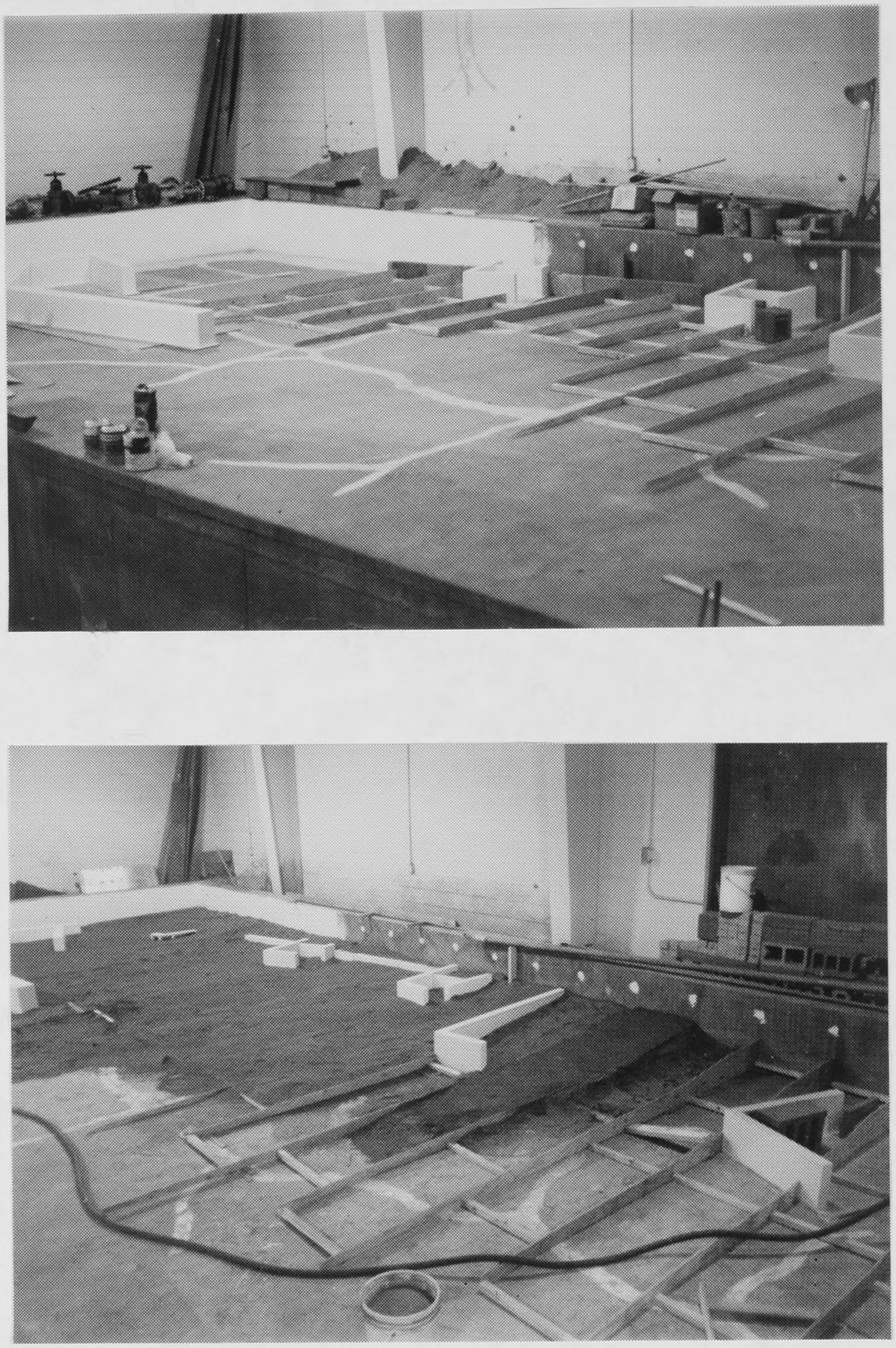


Figure 4. Reproduction of Bathymetry in the Physical Model.

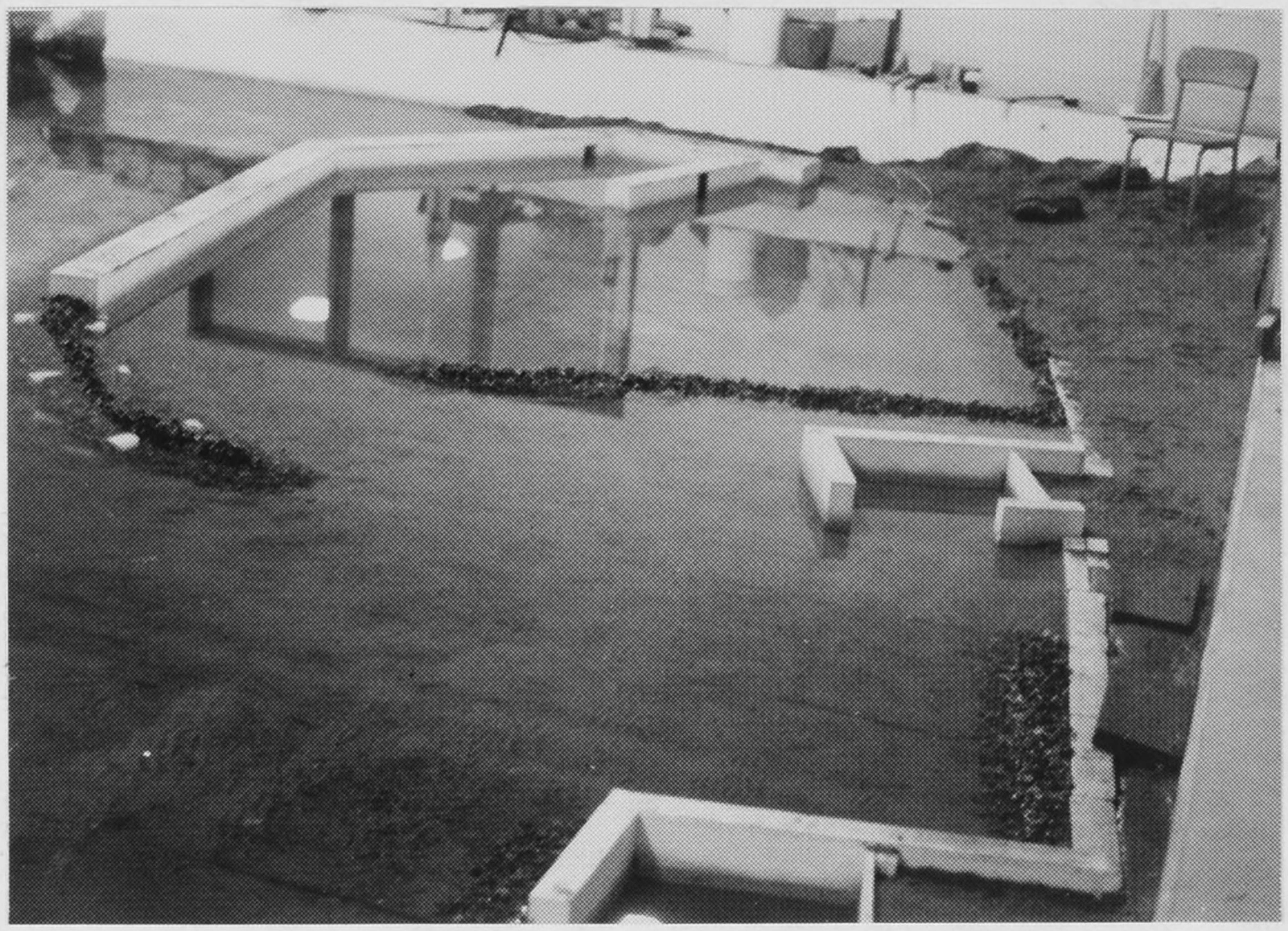


Figure 5. Photograph of Physical Model with Marina Modifications.

outside of the marina breakwater appeared to be somewhat larger than along most of the shoreline. The larger stone size was used along the breakwater and the shoreline just to the north of the marina while the smaller stone was used elsewhere. The actual armor stone appeared to be produced from the local Silurian dolomite and tended to be relatively flat with a maximum thickness in one dimension of 1-1.5 ft. These stones tended to stack on top of each other with the long dimensions oriented parallel to each other. The model stones were more equi-dimensional and angular, resulting in a more open packing formation than the prototype. Given this consideration, it was decided to model the stone at the correct physical scale with the expectation that the difference in stone configuration would tend to offset the increased viscous effects in the model.

During the phases of the model tests which involved a breakwater modification, armor stone was designed to be placed at a number of locations near the marina entrance. For these testing sequences the larger model stone size was used to simulate all armor stone. Figure 5 is a photograph of one of the modified marina configurations with the water at the correct model elevation showing the riprap, existing piers, etc.

Wave and Current Generation

The purpose of the model was to determine the response of the harbor under various conditions of waves and current. The generation of waves will produce currents, especially in the shallow water environment of Sister Bay. However, that current may not be representative of the actual current that exists in the prototype for several reasons. One limitation is that the physical boundaries of the wave basin provides limits on the directions that the overall currents can propagate as the model circulation must be confined to the extent of the basin. A second factor is that at least a portion of the current in the prototype will be associated directly with the wind stress at the water surface and the current induced by the mechanically generated waves in the laboratory will not include this component and will therefor be too small. A laboratory setup that could independently generate waves and/or current was therefore employed.

The waves were generated by a plunger wave maker for which the wave height and period could be continuously adjusted. The length of the plunger was 12 ft. Generally this was long enough to form a uniform train of waves at the marina entrance but the diffraction of wave energy near the end of the plunger reduced the wave heights along the breakwater itself. This should have little effect on the test results however, since the armor stone absorbed most of the incident wave energy and these would not be reflected directly into the marina in any case. The wave generator was positioned offshore from the model itself so that the waves were formed in deeper water and propagated onto the model bathymetry. The plunger generated a constant period - constant wave height train of waves. Wave energy adsorption material was placed along the basin walls behind the wave generator to eliminate the wave reflections off the back wall of the tank.

The current was generated under the assumption that the wind induced current and waves would be propagating in the same direction. The flow was established by means of a recirculating pump that discharged through a manifold approximately the same length as the wave plunger and directly below and behind it. The pump discharge rate was varied to provide currents of the desired magnitude as described below in the testing program. The suction pipe for the pump was in the opposite corner of the wave basin as indicated schematically in Fig. 3. This withdrawal location was selected under the reasoning that the overall current in the vicinity of the harbor (at least under northwest wind conditions) would be counterclockwise with the flow leaving the harbor to the north. Tests were performed to observe the differences in the circulation patterns within the test basin with only the wave generator in operation and with both the wave generator and the pump in operation. Although the circulation patterns were generally similar, there were some differences that could be expected based upon the local flow processes. First of all, the current speeds were generally faster with the pump in operation and this is expected. Secondly more of the overall circulation would bypass the marina with the pump in operation. This is also to be expected and realistic since flow entering the marina must either exit the marina through the circulation opening in the breakwater wall or else return through the entrance. Both of these would be more difficult than for the flow to simply turn and bypass the marina directly. Without extensive field studies, it is not clear that the flow conditions within the marina were precisely modeled. However, the procedure of looking at the effect of waves alone, waves plus current, and current alone on the circulation within the marina gives assurance that conditions that were generally representative of the prototype were investigated. Together with the comparison of flow under existing conditions as well as any proposed design changes provides confidence that a good understanding of the water circulation within the marina has been developed.

Model Testing Methodology and Procedures

Two basic types of model tests were performed; wave height measurements and water circulation tests. A number of different variables that could be adjusted in the model could influence the results of one or both of these types of tests. Since these variables could be varied over a continuous range that are to be expected in the prototype, it was necessary to make decisions as to specific testing conditions in order to limit the number of individual experiments that were to be performed. Following is a short discussion of the rationale that was used to select the specific experimental conditions that were adopted for this study.

Water level - The water level used in the model could potentially influence a number of different phenomena. It is reasonable that varying the water level would have some effect on the magnitude and direction of the circulation velocities in the harbor area. More important and direct influences would be observed in the wave height studies. Since the wavelength depends upon both depth and period, the potential for wave resonance conditions within the harbor for a particular wave period would be dependent upon the depth. Another important factor is that since the breakwater on both the existing marina and the proposed extensions are not to exceed a certain elevation (584 ft USGS datum), the possibility of wave overtopping would increase with an increase in the water surface elevation utilized in the model. All of these potential factors would complicate the interpretation of results. In order to avoid this difficulty, it was decided to hold the water surface elevation in the model at one fixed elevation throughout the model testing. The water surface elevation selected for the model tests was the approximate average elevation for Lake Michigan. This is given as 578.23 ft IGLD (International Great Lakes Datum) in the Shore Protection Manual (1984). All information on the harbor bathymetry, shoreline, etc. were given relative to the USGS datum which is approximately 1.3 ft higher than the IGLD datum. The water surface elevation tested in the model was set at 579.6 ft (USGS) for the duration of the experiments and was determined initially by surveying the model elevation at the offshore end of the existing breakwater once the model had been constructed but prior to any testing. The appropriately scaled depth was established by filling the tank and setting a fixed point gage to the surface level. The water levels would diminish over time due to evaporation and slight tank leakage. Every day, the water level within the model basin was returned to the same point gage level so that a constant depth was maintained in the model for all experiments.

The issue of wave overtopping of the breakwater was also a potential difficulty in the interpretation of results. It was eventually decided to conduct tests so that no wave overtopping of the breakwater section occurred in order that a common basis for comparison of results could be developed. This was accomplished by increasing the height of the existing breakwater and all other pier and dock structures in the model above the actual 584 ft elevation. It should be kept in mind therefore that wave activity within the marina for higher lake levels will most likely be greater than presented in this model study because of the overtopping that would be expected for large wave heights and high water levels. At the water surface elevation studied in the model, overtopping of the breakwater was indicated for an incident wave height of approximately 5-6 ft, which would be expected to occur relatively infrequently, but if Lake Michigan were to approach its 1986/87 levels, overtopping of the breakwater could be expected on a relatively frequent basis.

Wave Climate - The selection of appropriate wave heights and periods poses even more potential difficulties in the interpretation of results than does the choice of water surface elevation. Generally, wave amplification within a confined area such as the municipal marina is very sensitive to wavelength which is in turn related to wave period and water depth. In confined harbors, it is often observed that certain wave periods will result in large wave amplification. This is due to the resonant interaction of the harbor geometry with that particular wave length. Changing the wave period or depth a little may reduce the wave amplitudes significantly. Changing the harbor geometry will also result in a change in those wave periods which result in significant wave amplification. Therefore, testing an existing and modified harbor configuration at the same wave period may not be indicative of the general status of the harbor since the wave condition may change from a non-resonant to a resonant condition or vice versa. For this reason, it is preferable to test at several wave periods so that a better understanding of the overall performance of the harbor can be developed.

Harbor resonance is generally more important for harbors with small openings and with a high percentage of the harbor perimeter covered with sheet pile walls since this allows for the multiple reflection of waves and the development of a resonant condition. Neither of these conditions apply to the municipal marina in either the existing or proposed configurations as there is a considerable amount of riprap to adsorb wave energy in either case. Therefore, it is not expected that the choice of wave period will have a critical influence on the wave climate within the marina.

It was originally intended to attempt to estimate wave heights and periods based upon wave records for other locations within Green Bay. However, a number of difficulties arose to make this practically infeasible. The National Climatic Data Center in Asheville, North Carolina keeps records of wind speeds and wave heights at various locations around the Great Lakes. It was intended to correlate the two at some location for which information was generally available with an appropriate wave forecasting model and then to extrapolate the results to the Sister Bay Harbor Area. However, there are apparently no locations within Green Bay for which continuous records of wave heights are recorded. Only shipboard measurements of wave heights were available from the NCDC. It would have been extremely tedious to go back through the meteorological data and correlate them against the infrequent observations of wave heights. Coupled with the fact that Sister Bay is in such a sheltered location and that offshore islands and shoals may limit the effective fetch, it is not clear that the effort involved would have been justified for the determination of model test conditions. Consequently, a decision was made to directly use the deep water wave forecasting model in the Shore Protection Manual (1984) to determine several test conditions that would be reproduced in the model. STS (summarized in *Feasibility Study, Harbor/Marina Development, Sister Bay Wisconsin,* STS Consultants, Ltd. June 1989) estimated a maximum wind speed of 56 mph for the Sister Bay area with a maximum wind speed of 40 mph during that portion of the year when boats occupy the harbor (late spring to early fall). The following gives the wind speed used to estimate the wave height and period for the three test conditions:

Wave Condition	Wind Speed (mph)	Wave Height (ft)	Wave Period (s)
1	15	2.0	3.5
2	25	3.6	4.3
3	40	6.4	5.3

These wave conditions were taken from the deep water forecasting relation in the Shore Protection Manual (1984) for the fetch that would be appropriate for waves from the west-northwest (approximately 15 miles). The Shore Protection Manual suggests not to use an effective fetch correction although there is some indication from a comparison of data from other sites that this may be necessary. Water circulation tests were performed with only the first two wave conditions while wave height studies were done with all three. During the course of the study, concern was expressed regarding the possibility of larger waves (on the order of 8-10 ft) from the north and their impact on the harbor. For tests which considered this situation, the wave height and period actually used was the same as the third condition and the observed wave heights were scaled up in proportion to the incident wave heights. Since water waves are basically linear phenomena, this is a reasonable approach and allows for a better comparison from one test to the other since the same wave periods are used in those particular tests. The above wave conditions will be referred to as low, medium, and high waves, respectively, at relevant points in the presentation of results.

Three wave directions were also investigated in the study. They were intended to range over the directions that waves can actually enter into the Sister Bay Harbor over a reasonable fetch. No refraction computations were made but the consideration of refraction would only reduce the range of possible angles as the incident waves would turn so that their crests would tend towards alignment with the shoreline. The three wave directions studied have the following approximate compass bearings (wave rays propagating from that bearing):

Position	Bearing (degrees)
1	280
2	295
3	350

Positions 1 and 2 are considered to be most critical with respect to waves actually entering the marina. Position 1 involves the most direct approach into the existing marina and is also the most westerly bearing from which waves can propagate. Since prevailing winds in this area are from the southwest to northwest, this also represents the one of the most likely directions for incident waves and thus was used for much of the tests that screened potential alternatives to the harbor modification. Position 2 was selected as an angle for which reflection of wave energy off of existing dock structures outside the marina might result in significant wave propagation into the marina. Finally, Position 3 represents the most northerly bearing from which waves can propagate into the harbor area. Although these waves cannot enter directly into the existing marina, the longer fetch from that direction allows for potentially larger incident wave heights.

<u>Current Speed and Direction</u> - Currents in the Sister Bay harbor will be important for the consideration of water circulation within the marina and were considered during the performance of the flushing tests. There is very little information on currents within the vicinity of Sister Bay harbor as discussed previously. Local currents may have various origins including wave generated currents, currents generated by local wind (in addition to waves), and currents that are part of the overall Lake Michigan/Green Bay circulation. Due to the relatively sheltered position of Sister Bay, it is felt that the first two sources will be the most important. Waves are generated in the physical model with a wave generator and thus the applied wind stress that occurs naturally in the wave generation process is not present. This led to the decision to add the recirculation pump as discussed previously. The pumping rate was set so that the currents in the outer harbor area (near the outer extent of the model) would be approximately 0.5 ft/s in the absence of wave generation. In the absence of more specific field data, this was felt to be a reasonable estimate of local currents based upon the two field studies previously mentioned. Water circulation experiments could then be performed in any of three different basic configurations; waves only, current only, and waves plus current. By investigating all three cases, at least for some test conditions, these should give a reasonable view of the marina circulation and perhaps bracket what actually occurs.

Testing Procedure

There were essentially two different types of experiments that were performed. These were the wave height tests in which the reduction in wave heights within the marina was investigated and the water circulation or flushing tests to examine the movement of water through the marina. Initial testing was performed on a model of the existing marina in order to develop a baseline data set against which all marina modifications could be assessed. Further testing was performed on a variety of different marina modifications to determine which ones would most successfully meet the objectives of wave height reduction and continued water circulation.

Wave Height Measurements

Prior to any measurement, the wave height five to ten feet directly in front of the wave generator was measured and adjustments were made to obtain the desired wave height. Once the correct wave height was established, the wave height at selected locations within the marina were measured. In general, measurements were not made at preselected locations, but visual observations were made to determine the locations of maximum wave activity and wave height measurements were made at those locations. For the initial tests with the existing marina configuration, the wave heights within the marina were roughly the same magnitude as the generated wave height and it was not particularly difficult to determine the locations of maximum wave activity. The wave activity in several of the tests for the marina modifications was extremely low (with wave heights on the order of 1/16 - 1/8 inch in some cases) and it was difficult to determine where the waves were the largest. Consequently, measurements were always made in at least three different locations; the values reported in the Results section are always the largest of these in the back area of the marina where vessels are currently berthed. At these very low wave conditions, measurement error may have some influence on results and unexpected and small differences between different configurations may be attributed to this cause.

Water Circulation Measurements.

The rationale for these tests were discussed previously; and a testing procedure was selected to maintain consistency in the way that tests were performed from one marina configuration to another. The initial step was to dam up the marina entrance and all circulation openings with temporary dams. The placement of the dam across the marina entrance was selected to maintain a constant surface area within the marina for all configurations. Fluorescein dye was mixed with the water in the marina until the dye concentration was uniform; all measurements of dye concentration were made with a fluorometer and dye concentrations were adjusted to obtain an arbitrary but large reading on the fluorometer scale. Preliminary measurements indicated that sediment entrained from the marina bottom during the mixing process would give a falsely high fluorescence indication so the mixing was followed by a settling period in which dye concentrations were monitored until they became stabilized due to the settling of this sediment. At this time, the dams were removed, the wave generator and/or current diffuser were turned on and the flushing experiment begun. Measurements were made every five to ten minutes until the dye concentration within the marina dropped to below one-half its initial value. The time for the dye concentration to drop to half its initial value is reported below as the flushing time. Although this is a somewhat arbitrary definition, it is consistent among all experiments and as such forms a reasonable basis for analyzing water circulation within the marina.

Dye concentrations were determined by taking the arithmetic average of four samples taken at the locations indicated in Fig. 3. These four locations were selected during the preliminary testing on the existing marina configuration and maintained for all subsequent tests. The rationale for the selection of the locations is that water circulation within the marina was generally in a counterclockwise sense in the existing configuration (at least for waves from direction 1 at 280°) and the inside of the breakwater wall would be one of the last locations to be flushed. In the testing that followed, the sense of the circulation in the marina depended upon the specific geometry studied, and it became clear that a different selection of sampling locations would probably result in different estimates of flushing times. However, it was considered preferable to be consistent in sampling location to avoid even more arbitrary decisions on where to collect samples.

At the model scale, especially with the modified marina configurations, the flow within the marina was not turbulent and the dye concentration was not uniform within the marina area. In fact, the dye concentration might be fairly low at one sampling location while remaining near the initial concentration at another. Fig. 6 indicates the time variations of concentration versus sampling location for a typical run along with the average concentration and an indication of how the flushing time was actually determined.

During the preliminary phases of the testing, repeated tests were made for two different test conditions to obtain some idea of the repeatability of the results. For the same test conditions, the flushing times were within five minutes (at the model scale which translates into approximately one-half hour at the prototype scale) in the repeated tests. These variations are most likely due to fluctuations the current and wave field and slight differences in the removal of the dams, initiating the wave generator and current, etc. In the results presented below, all flushing times are reported to the nearest half hour to be consistent with this basic uncertainty.

During various of the experiments, additional dye studies were performed to get a better idea of the actual sense of water movement internally within the marina. In order to accomplish this a second (black) dye was added at discrete locations within the marina and its displacement with time was noted in the sense of general circulation patterns. For example, the photograph in Fig. 7 indicates a displacement of dye from left to right in the existing marina under a condition of no waves. Because of the location of the dye cloud, this also indicates a counterclockwise sense of rotation of the flow within the marina. These dye injections were performed infrequently so that a clear idea of the sense of the flow within the marina could be developed and this information is contained in notes and sketches generated during the data acquisition and some of these observations are discussed below.

MODEL TEST RESULTS

A summary of all experimental data collected is presented in Appendix A. This data includes results for marina configurations that were ultimately rejected and for some tests that were performed in order to obtain a clearer idea of whether a specific change was significant or not. Only those results that are relevant to the conclusions of this study are summarized in this section. In addition to the experimental data recorded, a videotape recording of various phases of the model construction and testing was made. The entire video record is on the order of one hour so an edited version of much shorter length was also made in which only the major components of the model study are presented. A copy of that videotape is being supplied to the Village of Sister Bay along with the final report and the original will be retained at the University of Michigan.

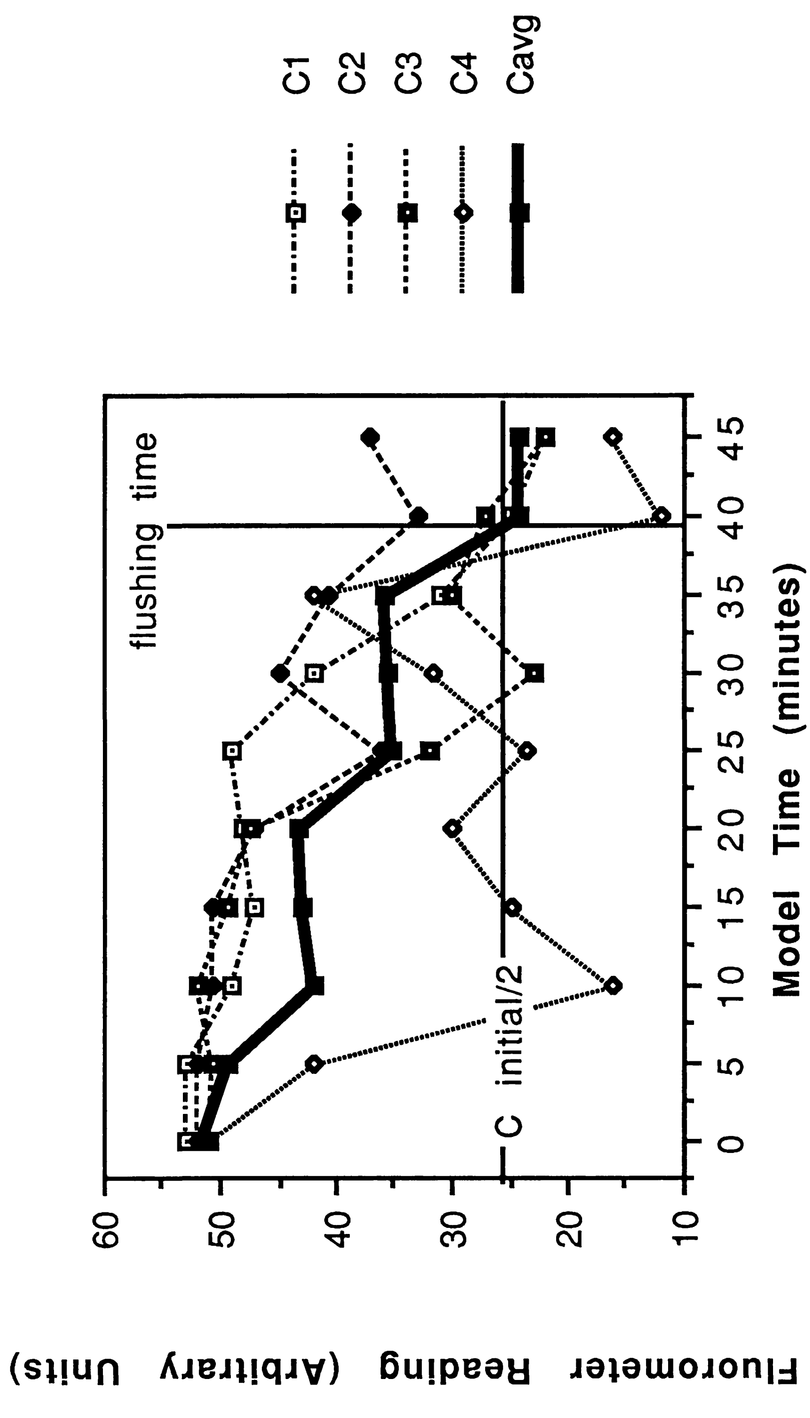


Figure 6. Time Histories of Fluorescent Dye Concentration During Typical Flushing Test.

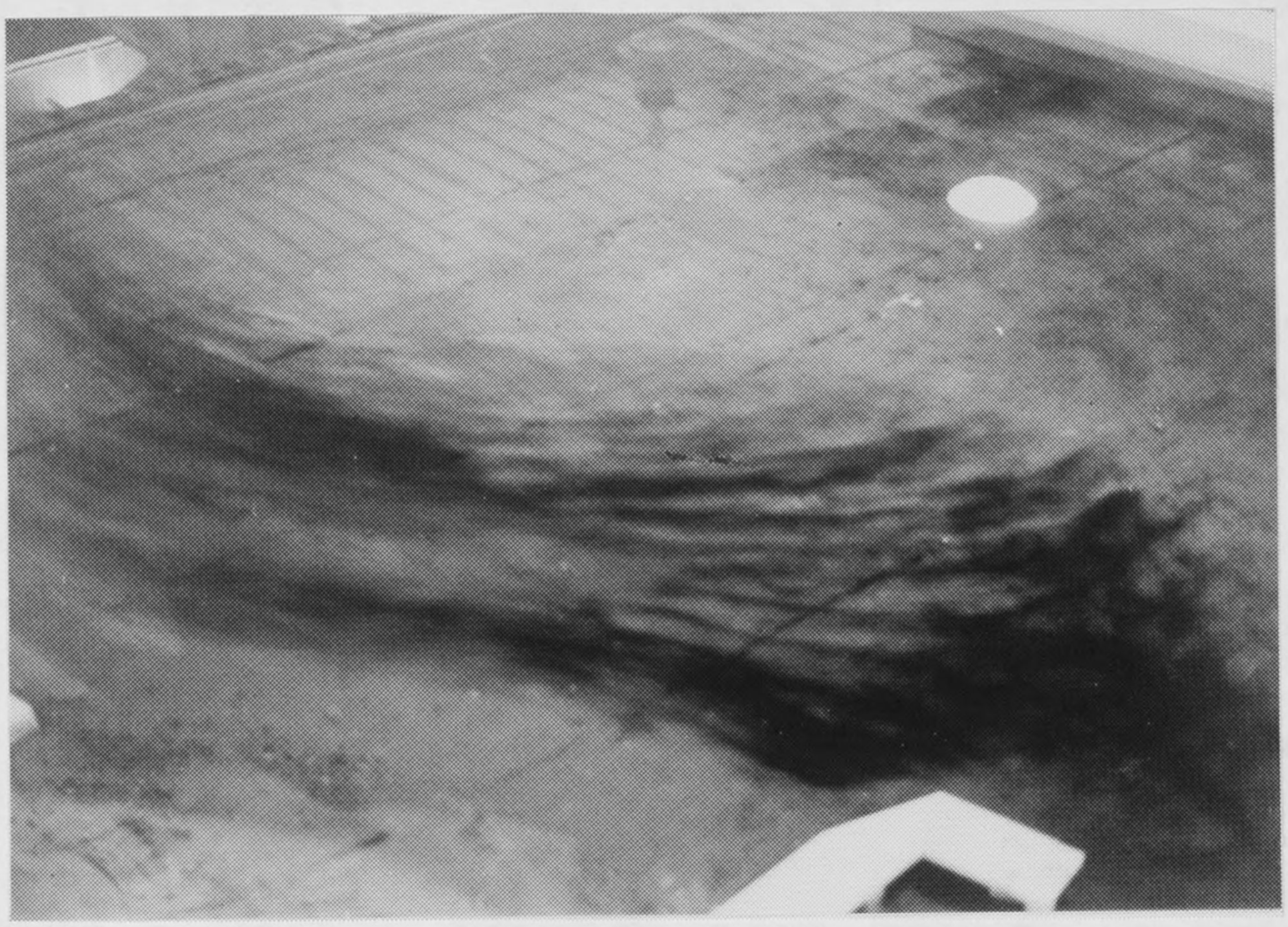


Figure 7. Displacement of Tracer Dye During Flushing Test.

A code for identifying the individual experiments has been developed: this four digit and letter code attempts to provide an easier way for presenting the experimental results. The general format of the code is $nXmY$ where n and m are digits and X and Y letters. The first digit n refers to the wave height and period condition while the digit m refers to the wave direction. Both the wave heights and directions have already been identified by digit and this previous designation is followed in this code. The letter X is used only to refer to conditions associated with the flushing experiment; A means the test was performed with waves only, B that the test was performed with current only and C that both waves and currents were present. Finally the letter Y is replaced in the code with an E referring to existing marina conditions, I to the intermediate set of experiments that represent a variety of different geometrical configurations, and M refers to the final modified marina configuration. Any further description that is necessary to classify a particular experiment is simply included as a verbal description in Appendix A.

Existing Conditions

The measurements of wave height within the marina generally followed an expected trend with some minor scatter in the data that are probably associated with the precision of the measurements. In general, one would intuitively expect a larger wave height within the marina for waves arriving from Position 1 since those waves have a fairly direct path into the marina. Even though Position 2 involves only a 15° rotation from Position 1, waves can no longer enter directly into the marina from this direction but instead must reflect off the sheet pile walls in the vicinity of the Casperson property. The trend in the data indicate slightly lower wave heights (for a given incident wave height) from Position 2 as compared with Position 1, confirming this expectation. Individual experiments have a range in wave height ratios (maximum within the marina divided by incident wave height) of 0.63 to 1.33 for waves from direction 1, while the range is 0.49 to 1.16 for waves from direction 2. In any case, the maximum waves within the marina are approximately the same height as the incident waves. Figure 8 indicates the general wave condition within the marina and this result is also apparent visually.

The water circulation tests performed indicated fairly consistent flushing time for the marina with only a few obvious factors that were important. In general, all individual tests indicated flushing times that were on the order of

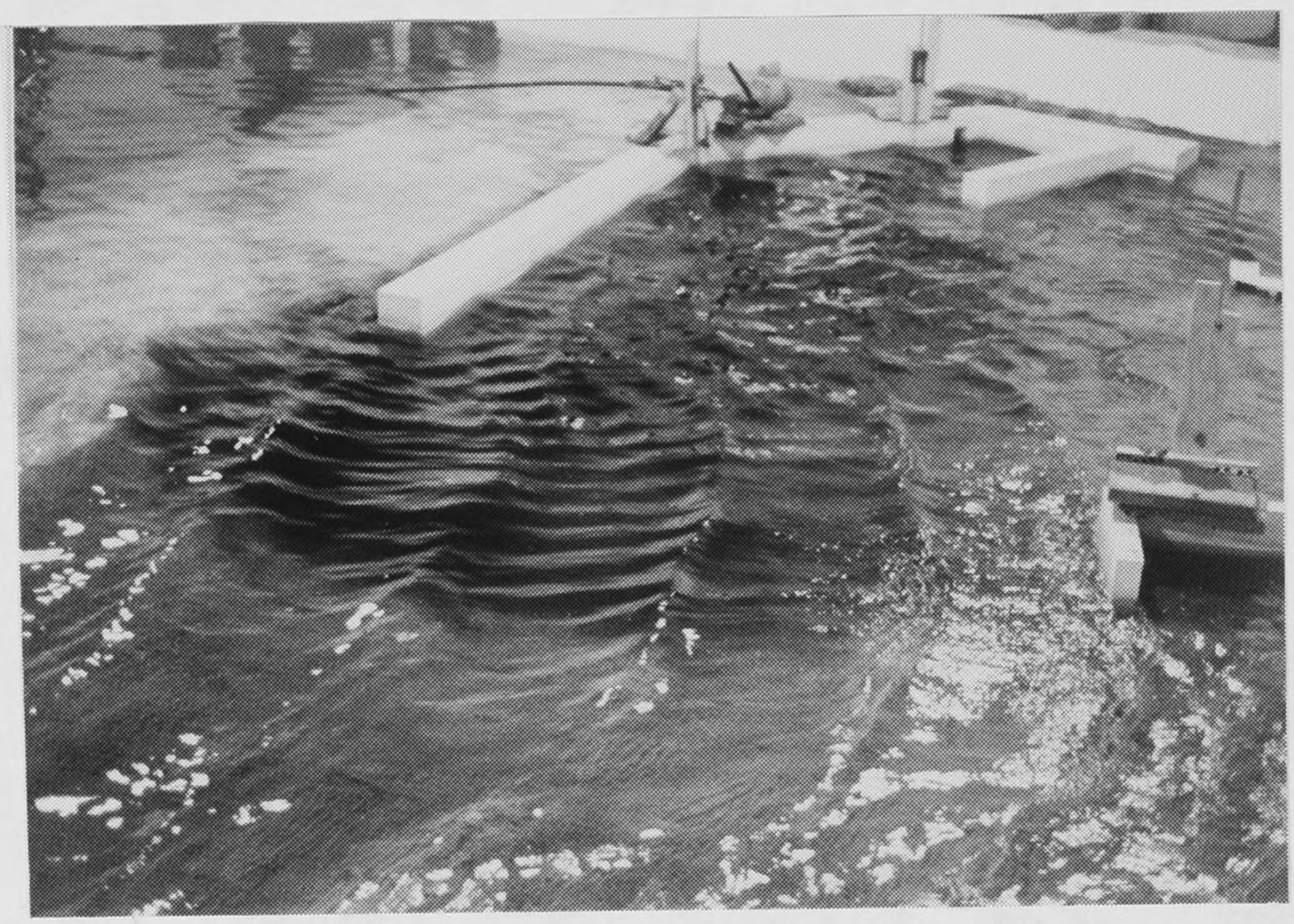
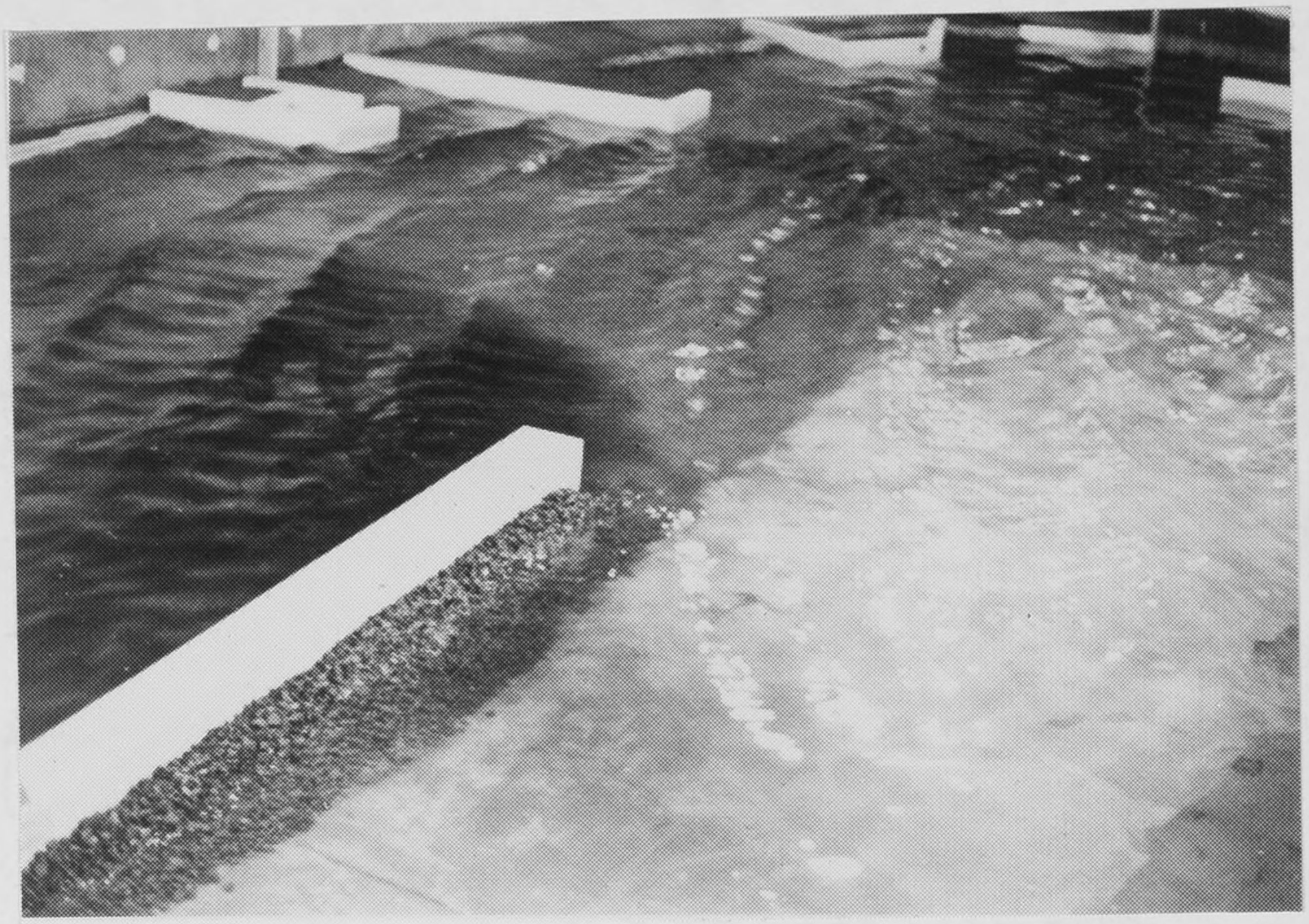
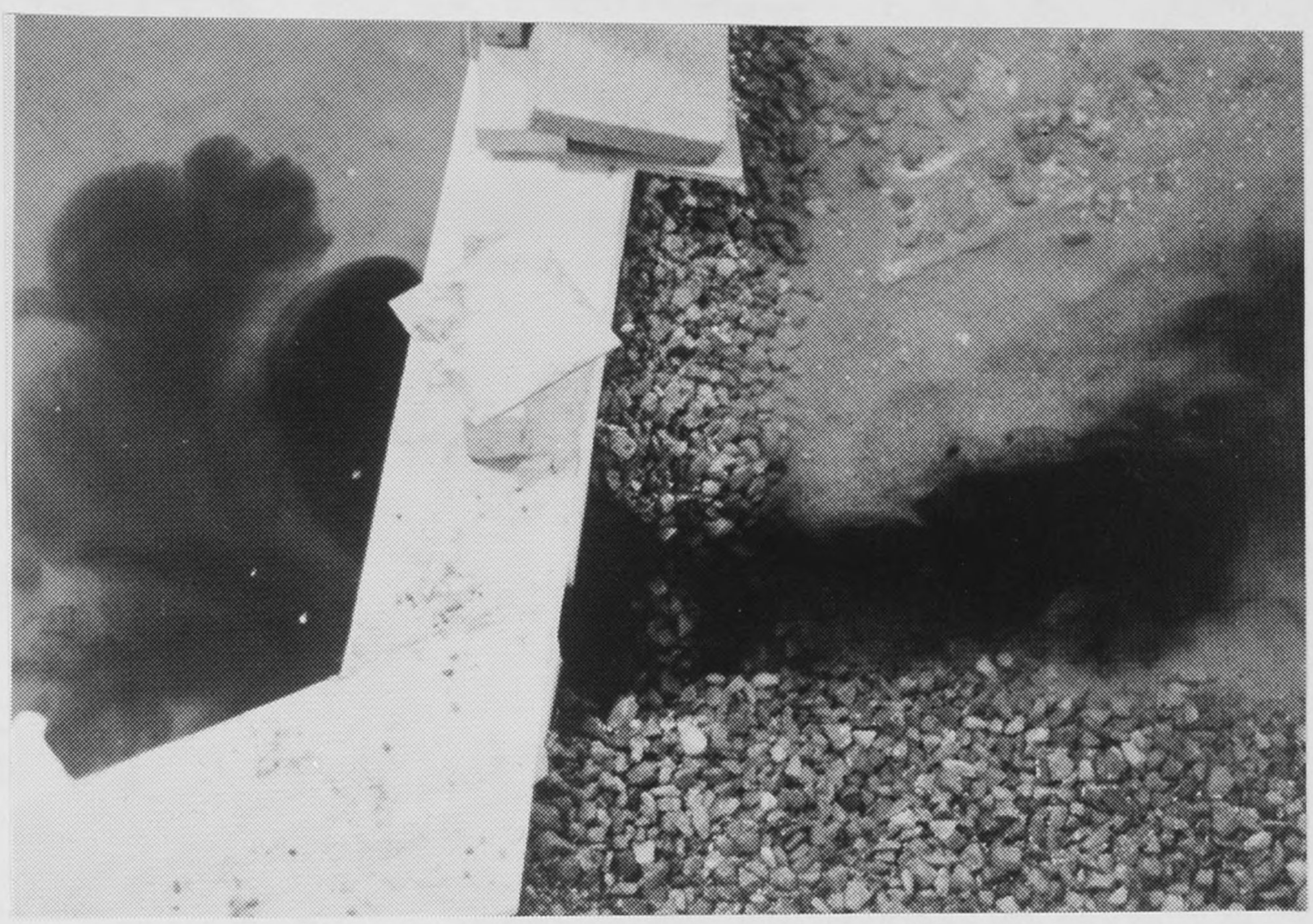


Figure 8. Wave Conditions within Existing Marina.



a.) Flow around end of Breakwater



b.) Flow through circulation opening.

Figure 9. Displacement of Tracer Dye During Flushing Test, Existing Marina.

one to two hours. The general nature of the circulation within the marina did change however, with different wave directions. With the waves from direction 1, the circulation within the marina was generally counterclockwise and most of the dyed fluid escaped through the marina entrance along the outer breakwater (see Figs. 8 and 9 for photographs of dye motion). Since the existing harbor currently has the circulation opening blocked, tests were performed both with it open and with it blocked to see if any significant differences in flushing time resulted. For waves from direction 1, there was no significant difference in the flushing times and this was consistent with most of the dye leaving through the marina entrance. With the wave generator shifted to direction 2, the waves entering the marina were reflected off other sheet pile walls in the harbor and thus entered the marina more from a southerly direction. This tended to result in a more complicated flow pattern within the marina, but much more dyed fluid exited through the circulation opening than in the previous wave direction. Consequently, the effect of closing the circulation opening was much more pronounced for two sets of experiments in which equivalent sets of conditions (with the exception of the circulation opening closure) were studied. For Runs 1C2E and 2C2E, it took about 65 percent longer to flush the marina with the circulation opening closed than with it open. The longest flushing time for all of the existing model tests, which was 3.75 hours, was observed for one of these cases with the circulation opening closed. Thus, it is obvious that the strength of the counterclockwise circulation is reduced as the incident wave direction moves towards the north and the flushing time is subsequently increased. This leads to the expectation that the flushing within the existing harbor could be improved for many conditions by opening up the existing circulation opening. An even more important consideration is with respect to modifications that tend to reduce the marina entrance opening. This effect would appear to be more critical for waves from the westernmost directions since the decrease in marina entrance opening would tend to decrease flow in through or out of the entrance. This led to a decision to focus most of the initial experimentation for harbor modifications on direction 1 waves since these should be most affected by entrance geometry.

Additional testing was performed to examine the possible influence of the present pumped water discharge (due to a dewatering operation) into the back of the marina near the launching ramp. It is understood that this discharge into the marina is continuous during the summer months and was estimated at around 500-1000 gpm during a visit to Sister Bay in May. Model tests with this range of flow rates and the requisite flow velocities did not appear to have an appreciable influence on the measured flushing times or in the nature of the flow within the marina.

Marina Modifications

Riprap on Casperson Property

All initial experimentation that was conducted with respect to the marina modifications was guided by the expectation that it would be possible to place riprap along the Casperson dock as indicated in Fig. 2. After this was eventually determined to not be possible, the testing program was abandoned although at a fairly advanced state. Although the experimental observations may not be directly applicable to any final design configuration, there were a number of observations that may be generalized to somewhat similar configurations. Therefore a brief discussion of the experimental findings are presented herein with a full presentation of the experimental results in Appendix A.

The initial configuration involved an extension of the existing breakwater and a shore wall that extended out from the shoreline near the Casperson dock. Initial testing indicated a large reduction in wave activity within the marina. The reduction was so significant that it was felt to be feasible to suggest a modification in which the shore wall was removed and most of the testing was performed with that configuration. Whereas the existing marina configuration indicated wave height ratios for waves from direction 1 to be 0.6 - 1.3, the modifications reduced the magnitude of this ratio down to 0.18 to 0.46 or about a threefold reduction in wave amplitudes depending upon the specific interpretation of the data. Retention of the shore attached breakwater would have resulted in even lower wave heights. The wave height ratios were correspondingly lower for directions 2 and 3. The visual sense was of an even more significant reduction in wave activity.

Attendant with this reduction in wave activity, however, was an increase in the marina flushing times. With the initial configuration with one circulation opening in the breakwater extension, flushing times varied over a range of about 4 to 8 hours with most measurements near five hours. These were all for waves and/or current from direction 1. One test performed after removing the breakwater extension reduced the flushing time down to about 1.5 hours so it was clear that the problem was with the reduced flow through the breakwater entrance. There was significant outflow through the circulation opening in the breakwater extension as indicated in Fig. 10. Spacing three circulation openings along the breakwater extension brought the flushing time down to 2-3 hours for most of the tests conducted and outflow through the two circulation openings closest to the existing breakwater was substantial (see Fig. 4 for the locations of these circulation openings). Most of the flow was through the inner two openings. Changing the wave direction to position 2 reduced the flushing time further down to about 1.5 hours. All of these tests were with the existing circulation opening open indicating that waves from direction 2 are less sensitive to conditions at the marina entrance.

Additional trials considering rubble mound breakwater or timber crib cross-sections in the breakwater extension (but with no circulation openings, under the assumption that sufficient flow through the pore spaces in these sections may improve the water circulation) were unsuccessful with some of the longest flushing times recorded during the entire testing procedure. Therefore, the presence of at least two circulation openings in the breakwater extension appears necessary in order to obtain adequate water exchange between the marina and the rest of the harbor.

<u>Final Configuration</u>

With the results of the above testing and new information regarding acceptable locations for armor stone, a final design configuration was developed by the Village of Sister Bay and STS personnel. A single design configuration was not specified, but instead a general design with several options were to be tested. These included the following potential elements (see Fig. 12 for the general layout of this modification):

- The breakwater extension and shore walls were to be rubble mound sections with three circulation openings in the extension and 2 circulation openings in the shore wall.
- The circulation openings were to be constructed from circular culverts so that flap gates could be installed to block wave action during storm events; these openings were modeled as 8-ft diameter culverts with PVC pipe of the appropriate diameter in the model.
- The inner pier within the existing marina could have a circulation opening installed in it if doing so would improve the flushing time.

Experiments with this opening in place in the model failed to indicate any evidence of water flow through it and therefore this modification is not recommended due to its lack of any observable effect in the model.

- The possible installation of a pump in the existing circulation opening was suggested. The range of pumping rates was suggested at 3000-5000 gpm. In the model tests, the flow rate through the circulation opening without any pumping and with the low wave heights tested (wave condition 1) was determined to be about 30,000 gpm. Therefore, it appears that the pumping would only be effective at very low wave and current conditions. There is no procedure by which the frequency of occurrence of a condition for which a natural flow through the circulation opening of, say 5000 gpm, could be estimated based on the model study itself. However, this would be expected to be fairly infrequent based upon the available meteorological and other field information and it is suggested that this supplemental pumping will be relatively ineffective in flushing the marina and therefore not viable from an economic perspective. It should also be emphasized that pumping as an alternative to opening up the existing circulation opening is not considered to be a viable alternative as there is a much higher natural flow through the opening under the conditions studied in the model.
- Riprap may be placed along the Al Johnson property but not the Casperson property. The results are discussed below, but generally, the placement of riprap along the Al Johnson property will reduce wave heights within the marina but not as effectively as riprap along the Casperson dock.
- In order to reduce construction costs, a reduction in the length of the shore wall would be desirable. Because of the reflection of wave energy off the Casperson dock, the shore wall will generally be necessary to minimize wave activity within the marina. However, there is apparently little influence of the shore wall on the flushing time. The connections between wave amplitudes within the marina and the shore wall is fairly intuitive and the model test results are discussed below.

The influence of various of the options listed above were extensively tested to determine the effect on marina wave heights. If the base configuration is considered to be the maximum shore wall length, flap gates closed on the circulation openings and riprap on the Al Johnson property, the influence of various alternatives to this are discussed below. This base condition resulted in wave height ratios of about 0.10 - 0.20 which are about a factor of six less than the

Figure 10. Flow Through Circulation Opening in Breakwater Opening, Intermediate Tests.

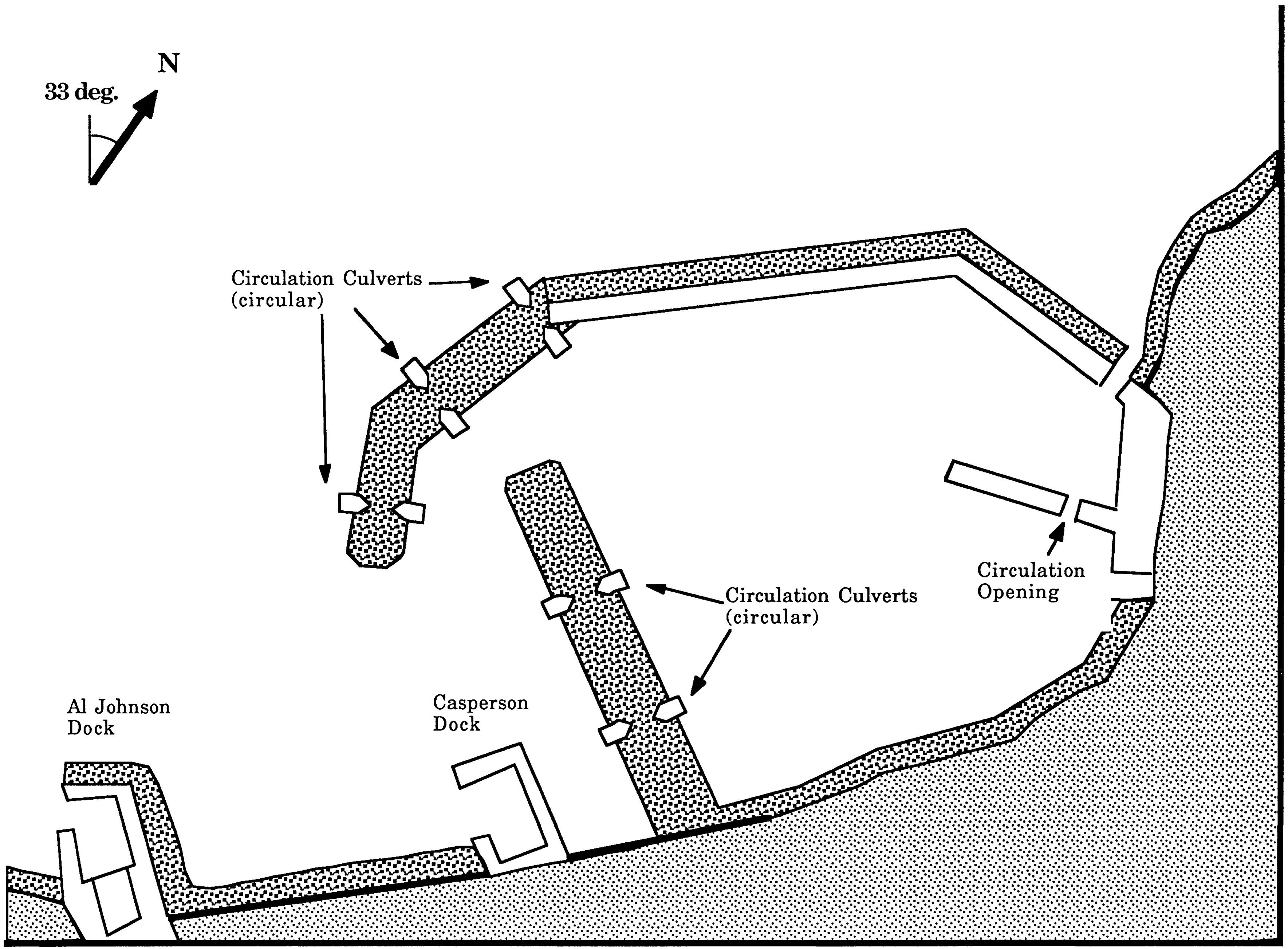


Figure 11. Final Proposed Marina Modification

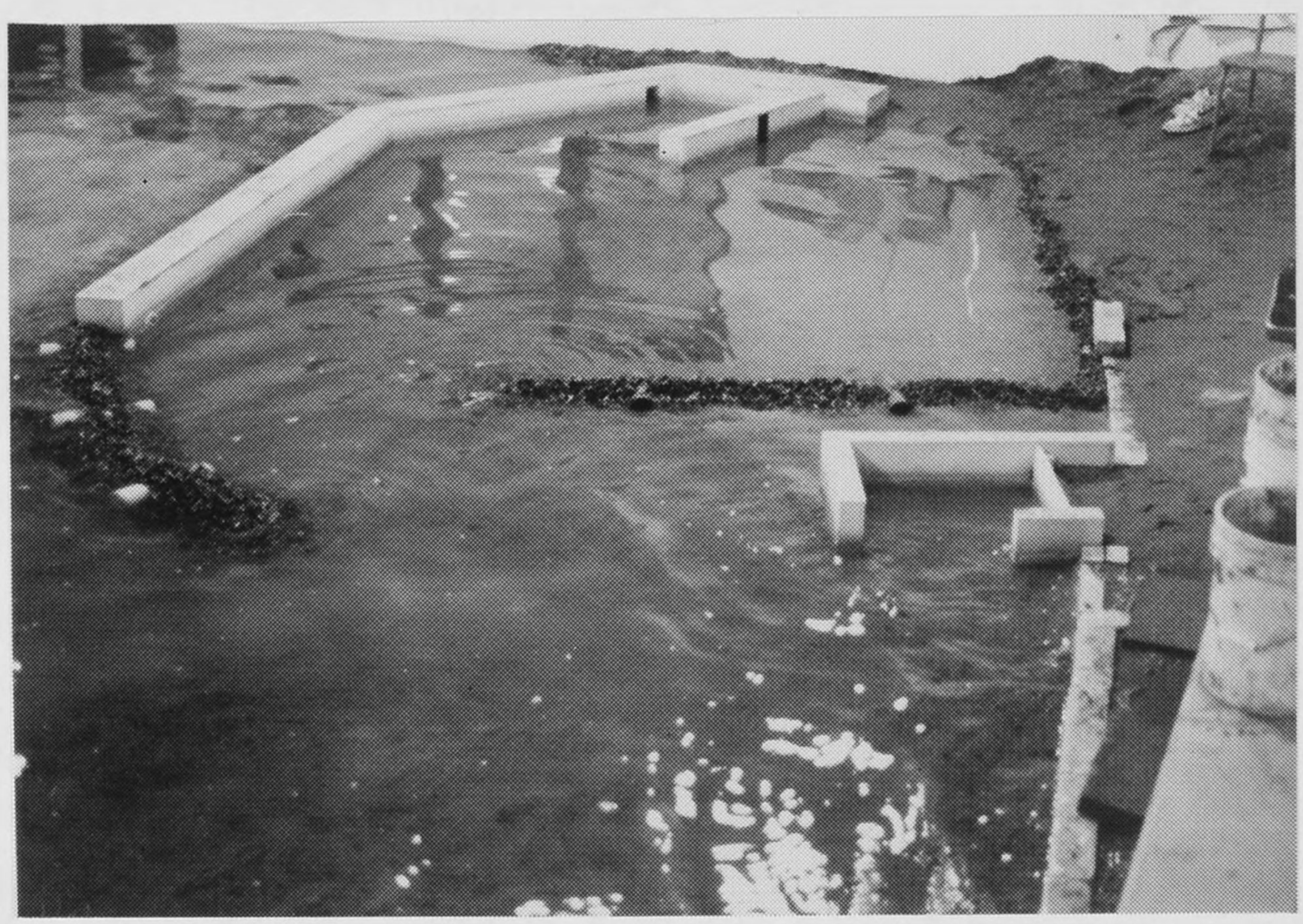
existing conditions. This improvement over the intermediate test results is apparently to two different effects: 1.) most of the intermediate tests were performed with no shore wall and the presence of this wall offsets the advantage of the armor stone on the Casperson dock; and 2.) The shore wall was of a rubble mound construction in these latter tests as opposed to a sheet pile wall with armor stone only on the outside of the harbor; the inner wall thus reflected rather than adsorbed wave energy.

Increases in wave heights within the marina were observed as deviations from the base condition were tested. Following is a general summary of the test results:

- If the circulation openings were opened, a general increase in wave activity was observed. While the increase was near zero for some conditions tested, it ranged up to 30 percent for some tests. Because of the low wave heights, the range may be partially associated with experimental precision, but it appears as though opening the flap gates increases the wave heights by about 15-20 percent compared to if they are closed.
- The placement of riprap on the Al Johnson property was found to have a more consistent influence on wave height reduction. The absence of the riprap increases the wave heights within the marina by around 30 percent. For waves from direction 3, the increase in wave height was more substantial (around 55 percent); this is reasonable since wave direction 3 was selected to examine the influence of wave reflection off the harbor walls in the vicinity of the Al Johnson property.
- A reduction in length of the shore wall also serves to increase the wave heights within the marina. Most of these tests were performed for the maximum wave condition and there is a fair amount of scatter in the results. Reducing the shore wall to two-thirds of its original length increases the maximum wave height by about 20 percent and removing the wall entirely makes the increase about 40 percent.

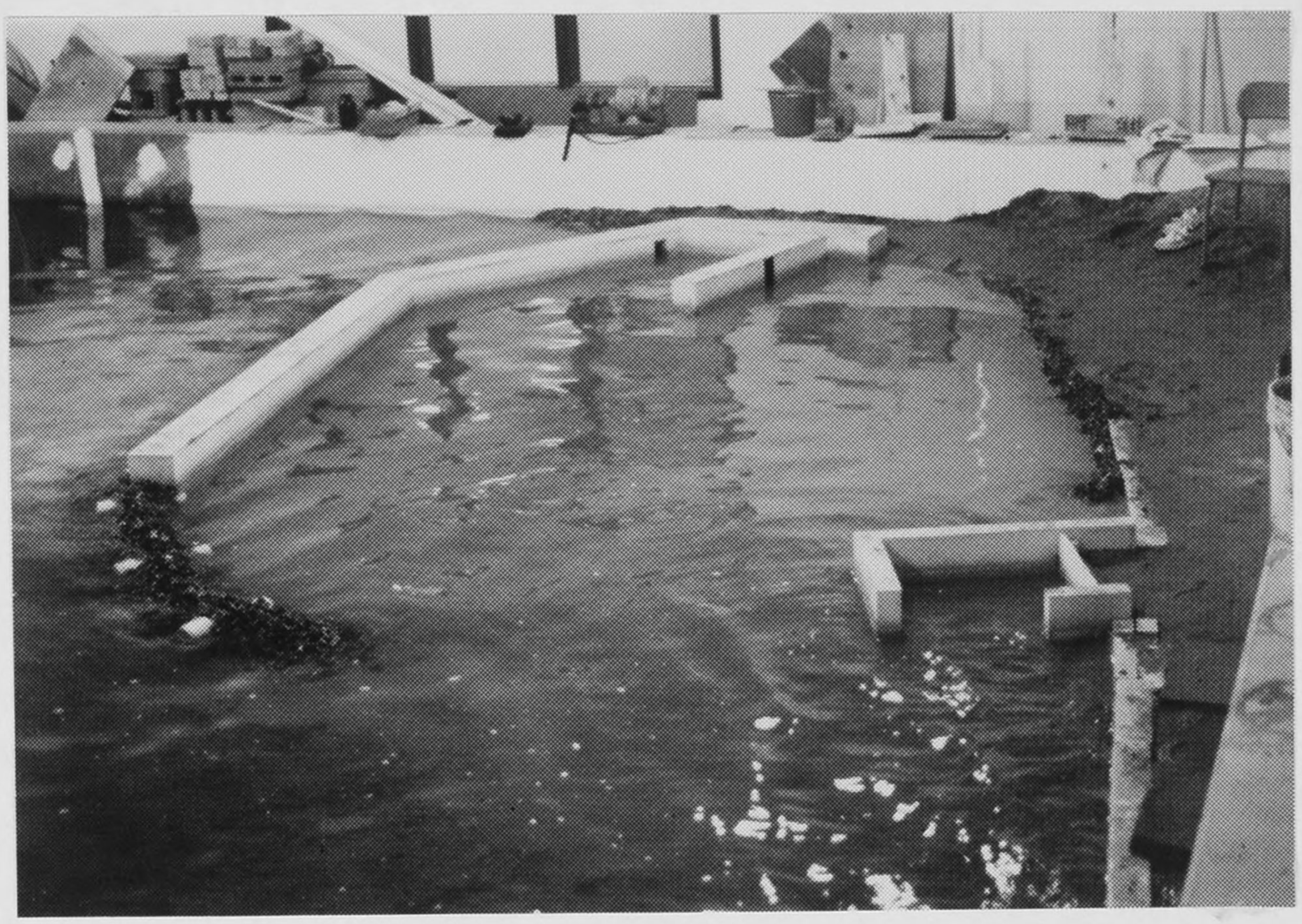
A series of photographs was made of the waves conditions within the marina; these should be also compared to the photograph of the existing conditions in Fig. 8 to observe how much reduction in wave activity has been accomplished by the marina modifications. Fig. 12a refers to a layout that is the same as the base condition above, i.e. with the full shore wall length and the riprap on the Al Johnson property. Fig. 12b shows the wave conditions with the riprap removed while Fig. 12c has also included the removal of the shore wall.

a.) Basic configuration.



b.) Riprap from Al Johnson property removed.

Figure 12. Wave Activity in Marina for Various Tested Marina Modifications.



c.) Riprap from Al Johnson property and shorewall removed.

Figure 12 (continued).

The increase in wave activity in the three photographs is readily apparent, although quantification is not possible.

Because all the possible permutations of marina geometries and wave conditions were not tested, it is not possible to state with certainty that the above influences are cumulative in their influence on the wave height, but it appears reasonable to take this approach in the decision making process.

Flushing times within the marina varied more significantly with wave direction than in the previous sets of tests, but generally the same results as observed for the intermediate tests were found for this configuration. For waves from direction 2, the flushing was fairly rapid, about the same as the times found for the existing marina with the existing circulation opening unblocked. This was generally much better than any of the intermediate tests. However for waves from direction 1, the flushing times were slower (and varied more erratically) than for the existing marina or those intermediate tests that considered three circulation openings. Flushing times ranged from 3 to 7 hours with most about 4 hours. This increase in time relative to the intermediate test results is probably associated with the presence of the shore wall; however, one test with the shore wall removed did not indicate a decrease in flushing time. The large range in flushing times is due to the fact that the average dye concentration decreased down to about the 50 percent level and then tended to remain there for an extended period in time and so the flushing time might be significantly influenced by very small variations in the circulation within the marina. Again, the flushing through the existing circulation opening was reduced for waves from direction 1 as compared to direction 2 for the reasons discussed previously and this is indicated in the results by a more than doubling in the flushing time. The current alone experiments also indicated faster flushing than with the waves present. While this may be associated with the variability in the flushing time determination discussed above, it does indicate that the imposed current is the primary factor in determining the circulation within the harbor.

Although these results indicate that circulation in the modified marina will be sensitive to wave direction, they do not directly address the issue of the relative frequency of occurrence of different wave and current conditions. Consideration of the wind rose developed for the Sturgeon Bay airport (contained within a calculation sheet by Terry Peterson of STS dated 7/28/88) indicates that winds from direction 2 would occur more frequently than from direction 1. This would imply that the direction 2 results should be weighed more heavily in any decisions on marina design than the direction 1 results. However, there would be even more times when wind would be from the southwest and this would presumably be associated with a low wave condition (there is no fetch to the southwest) and a current pattern that would be determined by the overall shoreline geometry in Green Bay and thus not subject to investigation in this hydraulic model unless field data on current directions within the Sister Bay harbor as a function of wind speed and direction were available. Since this is not the case, no absolute statement regarding changes in marina circulation is possible other than to say that it will improve compared to the existing marina (with the circulation opening blocked as is presently the case) for certain current directions and flushing timed will be somewhat increased for other current directions.

ACKNOWLEDGEMENTS

Several undergraduate students at The University of Michigan were involved in different phases of this project. Mike DeFinis helped in the early phases of the model construction, while Robert Gagnon and Elizabeth Hilbert were involved with both the model construction and testing.

Mr. Terry Peterson from STS Consultants, Ltd. in Green Bay was the liason person between Sister Bay and The University of Michigan and greatly facilitated the transfer of information between the two parties.

APPENDIX A
SUMMARY OF TEST RESULTS

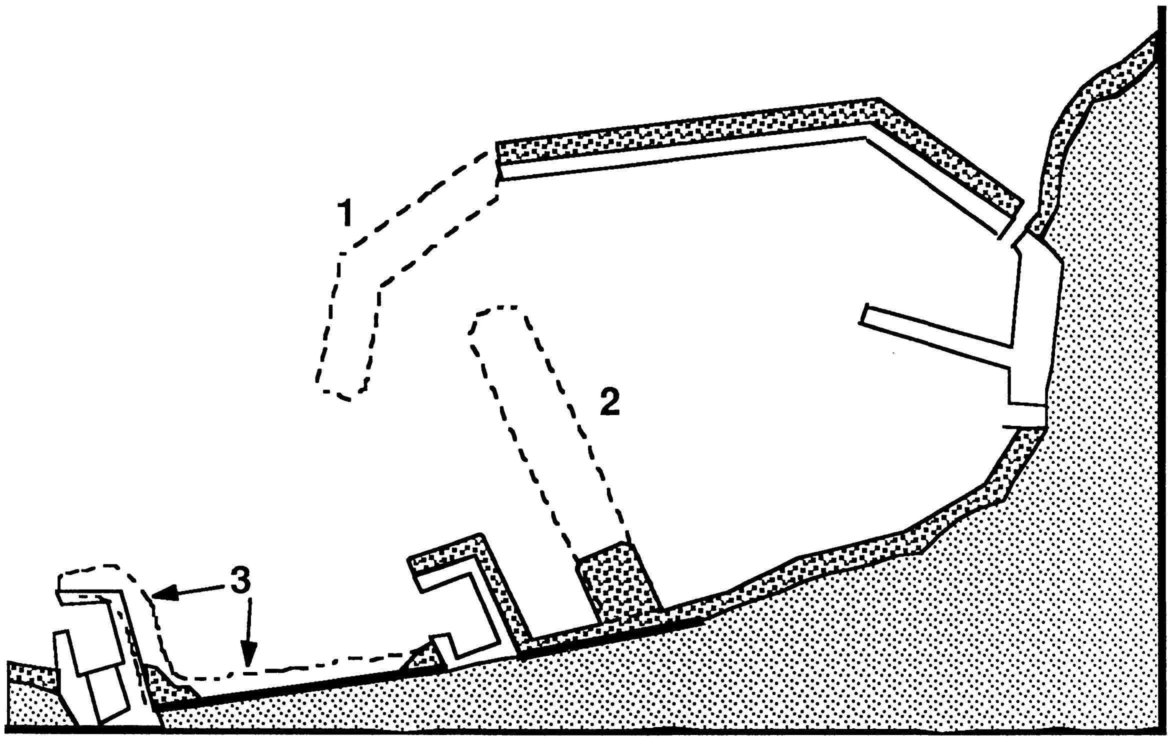
EXPLANATION OF RUN CODE ID - nXmY

Indicator	Value	Explanation
n	1	Wave Height - 2.0 ft ; Wave Period - 3.5 s
n	2	Wave Height - 3.6 ft ; Wave Period - 4.3 s
n	3	Wave Height - 6.4 ft ; Wave Period - 5.3 s
X	A	Flushing tests with waves only
X	B	Flushing tests with current only
X	C	Flushing tests with waves and current
m	1	Wave Direction - 280°
m	2	Wave Direction - 295°
m	3	Wave Direction - 350°
Y	E	Existing marina configuration
Y	I	Intermediate tests with modified marina
Y	M	Final marina modification tests

Sister Bay Harbor Model Study
Existing Configuration Tests

Run Code	Circulation Openings		HR	T50 (min)	Comments:
1C2E	open		0.491	12	
1A2E	open		-	23	
1A2E	closed		0.772	38	
1C2E	closed		0.772	12	
1A1E	open		0.632	26	
1A1E	open		0.877	16.2	
1A1E	open		-	18.9	
1C1E	open		1.333	17.2	
1C1E	open		-	13.4	
1C1E	closed		1.228	12.3	
1A1E	closed		-	11.2	
1B1E	open		-	19.8	
2C2E	open		0.777	12	
2C2E	closed		1.165	19.5	
2A1E	open		0.738	18.5	
3A1E	open		0.500	-	

HR = Relative Wave Height (Incident wave height/Marina wave height)
T50 = Flushing time



Numbering System for Break water Extensions

Sister Bay Harbor Model Study
Intermediate Tests w/ Riprap on Casperson Property

Key:	Break water extension #1
1a	Proposed extension
1b	Armor stone w/o circulation opening
1b	Section of wall removed
1d	Section of B.W. w/o opening removed
1e	Three circulation openings in wall
2	Proposed extension

Run Code	Circulation Openings	B.W. Extension(s) in place	HR	T50 (min)	Comments:
1C1I	open	1a,2	0.421	42	
1B1I	open	1a,2	-	51.2	
1A1I	open	1a,2	-	46.3	
1C1I	open	1a	0.175	55	
1A1I	open	1a	0.456	80	
1C1I	open	2	0.386	15.4	
1C1I	open	1b	0.456	50	
1C1I	open	1c	0.281	-	
1B1I	open	1b	-	80+	
1C1I	open	1d	0.281	63.5	
1C1I	open	1e	0.175	21.7	
1B1I	open	1e	-	24.1	
1C1I	open	1e*	0.246	75+	* 2 of 3 gates were closed
1A1I	open	1e	0.632	27.4	
2A1I	open	1e	0.408	27.8	Closing gates had little effect on HM
2C1I	open	1e	0.388	42	
2C1I	open	1e,2	0.466	39.3	HM varied with closing gates
2A3I	open*	1e,2	0.155	-	*Only gate at back of harbor closed
2A3I	closed*	1e,2	0.078	-	*Gates on 1e closed
2A3I	closed*	1e,2	0.097	-	*All gates closed
2A2I	open	1e	0.291	-	
2A2I	closed	1e	0.330	-	
2A2I	open	1e,2	0.369	-	
2A2I	closed*	1e,2	0.194	-	*Gates on 1e closed
2A2I	closed*	1e,2	0.175	-	*All gates closed
2C2I	open	1e	-	13.6	
2C2I	open	1e	-	17.9	
3C1I	open	1e	0.277	55	Pool filter was left on
3A3I	closed*	1e	0.138	-	*Gate at back of harbor closed
3A3I	closed*	1e	0.092	-	*All gates closed

HR = Relative Wave Height (Incident wave height/Marina wave height)
T50 = Flushing time

Sister Bay Harbor Model Study
Final Configuration w/ Culverts, no Riprap on Casperson Property

Key:	Break water extensions
1	Proposed extension
2a	Armor stone with circular culverts
2b	Wall removed to first culvert
3	Riprap along Johnson property

Run Code	Circulation Openings	B.W. Extension(s) /Modifications	HR	T50 (min)	Comments:
1C2F	open	1,2,3	0.070	10	
1C2F	open	1,2	0.105	-	
1A1F	open	1,2a,3	0.175	57	
1B1F	open	1,2a,3	-	31	
1C1F	open	1,2a,3	-	46.5	
2C2F	open	1,2a	0.427	15	
2B2F	open	1,2a	-	13	
2B2F	open	1,2a	-	13.9	
2A2F	open	1,2a	0.252	13.8	
2C1F	open	1,2a,3	0.194	36.4	
2C1F	closed	1,2a,3	0.136	-	
2A1F	open	1,2a,3	0.155	75	
2A1F	closed	1,2a,3	0.117	-	
3C2F	open	1,2a	0.164	55	
3C2F	open	1,2b	0.194	-	
3A1F	open	1,2b	0.254	-	
3A1F	closed	1,2b	0.270	-	
3A2F	open	1	0.224	-	
3A2F	closed	1	0.194	-	
3A1F	open	1	0.206	-	
3A1F	closed	1	0.317	-	
3A1F	open	1,2a	0.254	-	
3A1F	closed	1,2a	0.190	-	
3A1F	open	1,2a,3	0.190	-	
3A3F	open	1,2a	0.311	-	
3A3F	open	1,2a,3	0.197		

HR = Relative Wave Height (Incident wave height/Marina wave height)
T50 = Flushing time

Addendum To
Final Project Report
SISTER BAY MUNICIPAL HARBOR/
MARINA EXPANSION
(September, 1991)

March 21, 1992

By

Steven J. Wright

For

STS Consultants, Ltd
and
Village of Sister Bay, Wisconsin

Purpose of Additional Model Tests

A physical hydraulic model study of wave action and water circulation within the Sister Bay municipal marina was conducted at The University of Michigan during the period of June-September, 1991. The results of preliminary testing indicated that changes in the preliminary design developed by STS Consultants, Ltd. would be helpful in meeting the combined objectives of reducing wave activity while maintaining water circulation within the marina. Following presentation of the preliminary results, a modified marina configuration was developed in mid-August of 1991; this modification involved several issues to be addressed in the model testing. Following submission of the project report, additional issues have arisen during the finalization of the design for the marina expansion. Certain of these issues were not anticipated during the original model testing and therefore could not be addressed by the results presented in the report. This led to a decision to perform a limited number of tests with the existing physical model. Specifically, these tests were intended to address:

- Whether or not either the proposed breakwater extension or the proposed shore-attached breakwater could be replaced with a timber crib section as opposed to the rubble mound cross sections suggested in the modified marina design.
- If a timber crib section was to be utilized, whether or not riprap would be required along the outside of these structures.

These questions are raised in regards to the potential for an increase in wave activity within the marina. Since a basic timber crib structure involves vertical walls, and these are known to reflect more wave energy than the sloping sides of a rubble mound structure, it could be anticipated that an increase in wave activity within the marina would occur. The questions intended to be resolved by the additional testing presented in this addendum revolved around the determination of the degree of increase in wave activity with proposed timber crib structures and which, if either, of the two proposed breakwaters most influenced any observed changes in wave action.

Details of Additional Tests

The basic physical model described in the project report of September, 1991 was utilized in this additional testing and the original report may be referred to for more details regarding the model construction, test conditions, etc. The relevant issue here is that timber crib construction was to be considered for either the proposed breakwater extension or the proposed shore-attached breakwater or both. Three specific cases were considered:

- The shore attached breakwater as a timber crib structure with the breakwater extension as a rubble mound structure;
- Both proposed breakwaters as timber crib structures;
- Both proposed breakwaters as timber crib structures with riprap protection along the side outside of the marina.

The issue of water circulation within the marina was not addressed in this additional testing. The results of the original testing indicated that details with regards to the breakwater cross sections would have little, if any, influence on the flushing of water through the marina so long as the circulation openings recommended in the project report are retained. There may be a minor decrease in flushing time associated with an increase in wave activity, but it is doubtful that this could even be accurately quantified.

Since the submission of the project report, a tentative decision has been made to install riprap along the Al Johnson property. Therefore, all results presented herein were obtained with riprap placed in the model in those locations.

All rubble mound cross sections were constructed following the procedures utilized in the original testing. The same gravel materials were used and installed in the same fashion as described in the project report. Furthermore, the riprap facing used for the timber crib structures in some of the tests was prepared from the same gravel since riprap on the outside of the breakwaters would be subject to the same stability requirements as armor stone on a rubble mound breakwater.

The timber crib structures studied in the model were constructed from 3/8 inch dowels with 3/8 inch spacing between the edges of the dowels. The intention was to create an open face to permit penetration of as much wave energy as possible into the stone matrix contained within the timber cribs. While it is understood that the final design for the proposed timber cribs has not been developed, preliminary discussion led to the decision to construct the model cribs in this fashion. Gravel contained within the timber cribs was the same as used for the rubble mound sections. Again, a final decision with regards to timber crib materials has not been made and this selection of gravel was largely a matter of convenience. However, based on previous tests performed by others, there is no reason to anticipate that this will have a significant influence on the test results, specifically with regard to reflection of wave energy.

The modeled width of the timber crib structures was 20 ft for the shore attached breakwater and 15 ft for the breakwater extension. These were selected after consultation with Terry Peterson of STS regarding tentative design of the timber crib structures.

Blake Tullis performed all of the additional tests that are described in this addendum; he also performed the majority of the tests presented in the project report. All methods for the determination of wave heights thus follow the same protocols developed in the original study and all results presented in this addendum can therefore be directly compared to the previous test results. The results presented herein were obtained in March, 1992.

Results

The test conditions in the current study simply continued the rationale outlined in the project report with regard to selection of wave direction, height, and period; the project report can be referred to for these details. All tests presented herein were with waves from direction 1 as defined in the project report (wave approach from an angle of 280° or from just north of west). This wave direction produced the largest waves within the harbor under existing conditions and was more extensively studied in the original study. All three wave conditions, corresponding to periods of 3.5, 4.3 and 5.3 s, were examined in the current round of testing. The results of all of these tests are presented in Table A.1. There were no tests in the previous testing sequence that would directly compare with these, but the results reported as runs 3A1F are generally most comparable with regards to the proposed breakwater geometries. These two tests (one with the circulation openings blocked and the other with them open) report relative wave heights of 0.25 and 0.27. These tests were performed without the riprap in place in front of the Al Johnson property and a similar test with the riprap in place would probably resulted in a wave height of about 20 percent less or with relative wave heights on the order of 0.2. This may be taken as the result with both proposed breakwaters constructed with rubble mound sections. This result compares to the current round of testing with relative wave heights of 0.19 if the shore attached breakwater is of timber crib construction and the breakwater extension is rubble mound while the relative wave height of 0.34 was observed with both breakwaters of timber crib construction. This leads to a tentative conclusion that the shore attached breakwater construction is not particularly important to the wave amplitudes within the marina while the breakwater extension is. This is a fairly intuitive result in the sense that, in order for waves to enter into the marina, they must generally reflect off the inside of the breakwater extension while reflections from the inside of the shore attached breakwater can occur only once waves have propagated into the marina.

The results at other wave periods basically confirm this finding. In all cases tested, changing the breakwater extension to a timber crib section nearly doubled the wave height within the marina and this is consistent with a larger reflection coefficient off the inside of the breakwater extension. The results from the previous round of testing, although not directly comparable, tend to follow the same patterns found with the offshore breakwater of rubble mound cross section and the shore attached breakwater of timber crib section.

It is not immediately obvious what the effect of placement of riprap on the outside of the proposed breakwater sections should be. It is extremely unlikely that riprap on the outside of the breakwater extension would influence waves within the marina since this wave energy would manly be reflected back offshore. In the case of the breakwater extension, it would be anticipated that at least some of this energy would be able to enter into the marina and thus might be indicated as an increase in wave activity. The limited test results are ambiguous and only indicate that such an effect is relatively small. A consideration of perhaps more concern is with respect to conditions in the vicinity of the marina entrance where no detailed measurements were made. In the previous study, significant increases in wave amplitudes between the shore attached breakwater and the Casperson dock were observed in preliminary tests with vertical walls for the shore attached breakwater. This increased wave activity would be likely to extend out to the marina entrance. It is to be expected that the placement of riprap along the outside of the shore attached breakwater would serve to reduce this sort of wave activity.

Conclusions

Even though the test results presented in this addendum are somewhat limited, they clearly support the following conclusions:

- Since the geometry of the proposed design was intended to satisfy a general limit on wave activity within the marina, it is not recommended to alter the breakwater extension to a timber crib type of section as this results in a nearly doubling of wave activity within the marina. One alternative would be to place riprap on the inside of the breakwater extension, but unless economic or other considerations indicated that this is an attractive alternative, it is not clear why this would be chosen over the rubble mound section proposed in the original testing.
- The shore attached breakwater can be constructed with whatever cross-section is desirable based on aesthetic, economic, or other considerations and it will not have a major influence on wave activity within the marina. However, if the timber crib section is selected as the preferred section, several factors should be considered in the final design: 1.) The timber cribbing should be made as porous as possible to minimize reflection and promote wave propagation into the stone fill; 2.) The stone size within the cribbing should be sized to maximize energy dissipation within the pore spaces within the stone matrix, but still prohibiting significant wave transmission through the structure. From previous experimental and theoretical studies, it appears that the width of the timber crib structure should be on the order of 10-20 times the average stone width; 3.) The placement of riprap on the outside of the shore attached breakwater will definitely improve the wave conditions near the Casperson property and this improvement will likely extend out to the marina entrance.

TABLE 1. Relative Wave Heights Observed in Model for Various Proposed Breakwater Configurations.

Breakwater Configuration	Wave Period (s) 3.5	4.3	5.3
Shore Attached B.W. timber crib B.W extension rubble mound	0.14	0.12	0.19
Both Breakwaters timber crib No external riprap	0.29	0.24	0.34
Both Breakwaters timber crib With external riprap	0.41**	*	*

* - Not tested

** - Intermittent maximum occurring infrequently

Reported are maximum relative wave heights ($H_{inside\ marina}/H_{incident}$) recorded at any measurement location.

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AIIM SCANNER TEST CHART #2

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MESH HALFTONE WEDGES

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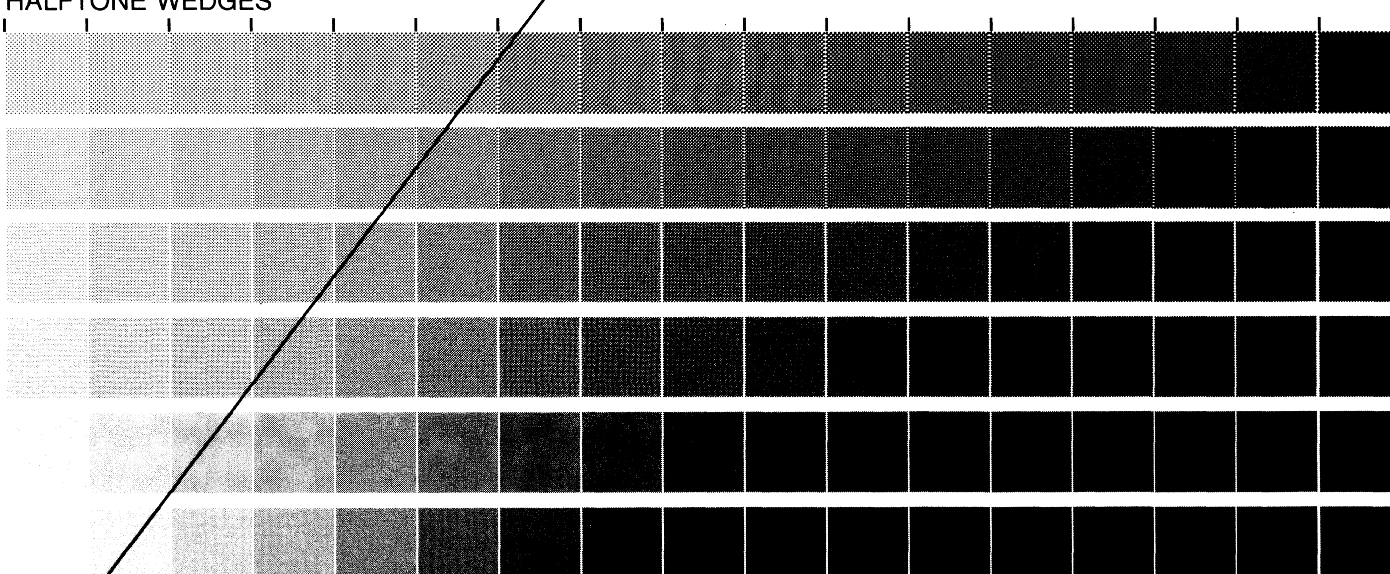

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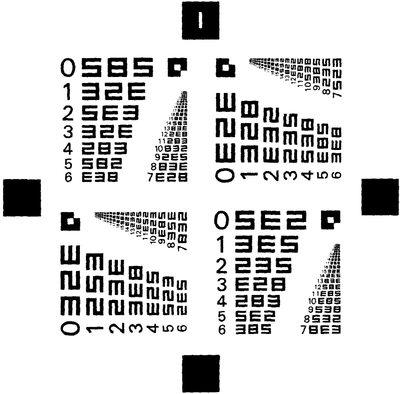
RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-1-71



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