CHAPTER 88

LABORATORY INVESTIGATION OF SHORE EROSION PROCESSES

by

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The laboratory investigation was undertaken as part of a shore protection demonstration program sponsored by the Michigan Department of Natural Resources. Subsequently funding was also provided by the Sea Grant Program. The laboratory work is being done in the Lake Hydraulics Laboratory, a facility of the Department of Civil Engineering of the University of Michigan. The field demonstration program consists of 19 field installations at locations on Lakes Michigan, Huron and Superior. The laboratory program was planned to supplement information from the field installations by testing over a wider range of variables and to test procedures not included in the field program. This program has also proven to be useful in the demonstration of shore erosion processes to groups concerned with shore problems. Although erosion rates determined in a model cannot be converted quantitatively to nature it was reasoned that if natural shore erosion processes could be simulated and if repeatable erosion rates could be produced in the model the results could help to evaluate the relative effectiveness of many protective methods. The advantages of using a model are the much lower cost compared with field installations, the control over such variables as wave height and water level and the speed with which results can be obtained.

The Testing Arrangement

The tests were conducted in a wave tank located in the University of Michigan Lake Hydraulics Laboratory. The tank is about 40 feet (12 m) square. The testing arrangement is shown in Fig. 1. The movable bed area indicated as "sand" in Fig. 1 was about 11 feet (3.3 m) by 16 feet (5 m). The test section consisted of six feet (1.8 m) of bluff. The area of the model outside of the movable bed portion was surfaced with concrete. At the beginning of each test the sand bottom was formed to a slope of 1 to 20 which conformed with the surrounding concrete

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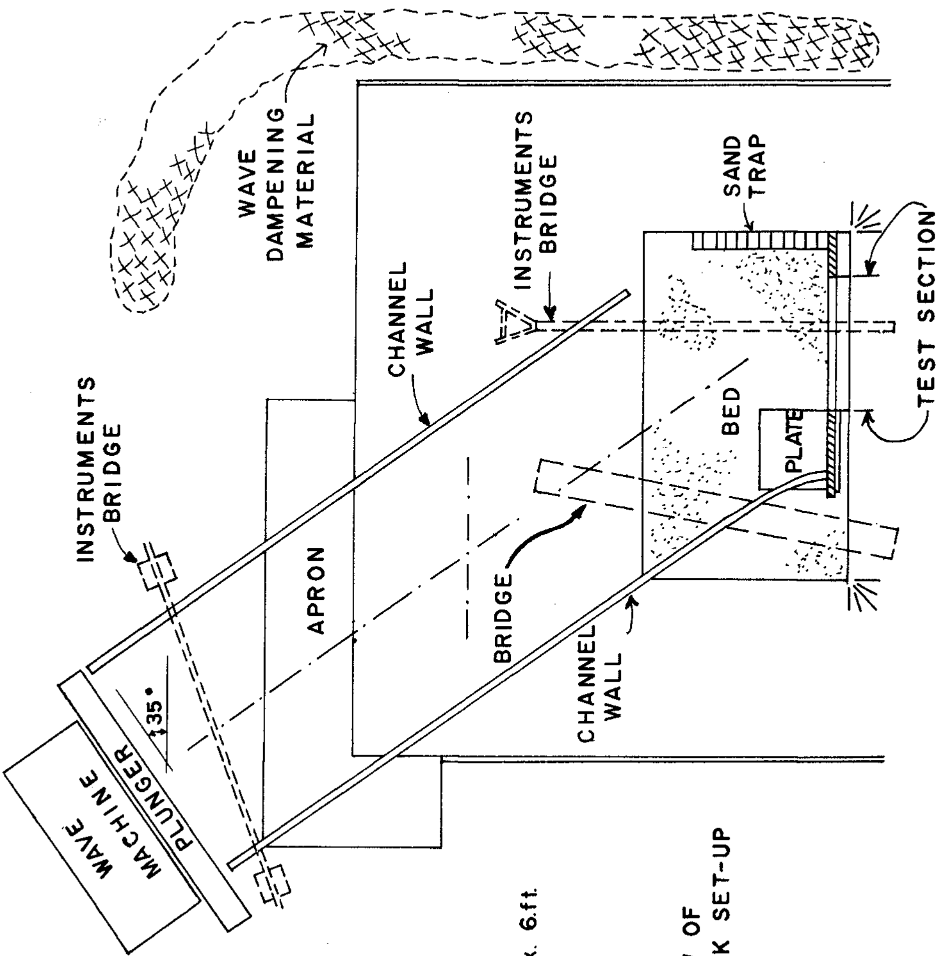


SCALE
1in. approx. 6 ft.

FIG. 1
PLAN VIEW OF
WAVE TANK SET-UP

bottom. The sand bluff test section was formed with a smooth surface having a slope of 60° with the horizontal. The waves were created by a plunger type wave machine and projected at an angle of 35° with the shoreline in order to produce a natural littoral current. In order to prevent lateral dispersion of the wave energy the portion of the waves which attacked the test section were confined in a channel until near the testing zone. Waves outside of the test section were destroyed by dampening material placed along the walls of the tank.

The Testing Procedure

The wave selected as a standard for most of the tests had a deepwater height of 0.20 feet (6.1 cm) and a period of 1.0 seconds. At a scale ratio of 1:50, which was the basis for constructing the model protective devices, this model wave would correspond to a prototype wave having a height of 10 feet (3 m) and a period of 7 seconds. The standard water surface elevation which was used in most of the tests was the elevation of the junction between the bluff face and the bottom which is designated as a_1 in Fig. 2. The standard test durations were 30 and 60 minutes. At a scale ratio of 1:50 these durations correspond to storms lasting 3.5 and 7 hours, respectively. The preliminary tests showed that 60 minute tests provided sufficient data to meet the objectives of the program. At the end of 30 minutes of testing and again at 60 minutes the bluff and bottom profiles were determined and the littoral drift was measured. The distance along the test section starting at the updrift end was designated as x (See Fig. 1) and six profiles were taken at one foot (0.3 m) intervals with the first one at x = 0.5 feet (0.15 m). The rate of littoral drift was measured in 12 sand traps extending 6 feet (1.8 m) from the shore line at the down-drift end of the test section as shown in Fig. 1.

The first tests were made with a well graded sand which was too coarse to produce a typical continuous erosion pattern. The bluff erosion started in a typical manner but the coarser eroded material could not be readily moved by the littoral currents and therefore created a protective revetment which prevented further erosion. This problem was overcome by using a finer sand. This sand was quite uniform in size with about 80 percent less than 0.4 mm and about 20 percent less than 0.3 mm. No noticeable portion of this sand went into suspension during the tests. Thereafter, the most difficult modelling problem was the development of a procedure for re-forming the bluff and bottom which was not too time consuming and would produce repeatable results of a given set of conditions. The procedure which gave satisfactory results consisted of forming a rectangular volume of sand after which the front form was removed and the face of the bluff was cut at the desired angle. Uniform compaction of the sand was achieved by simultaneous vibration and saturation.

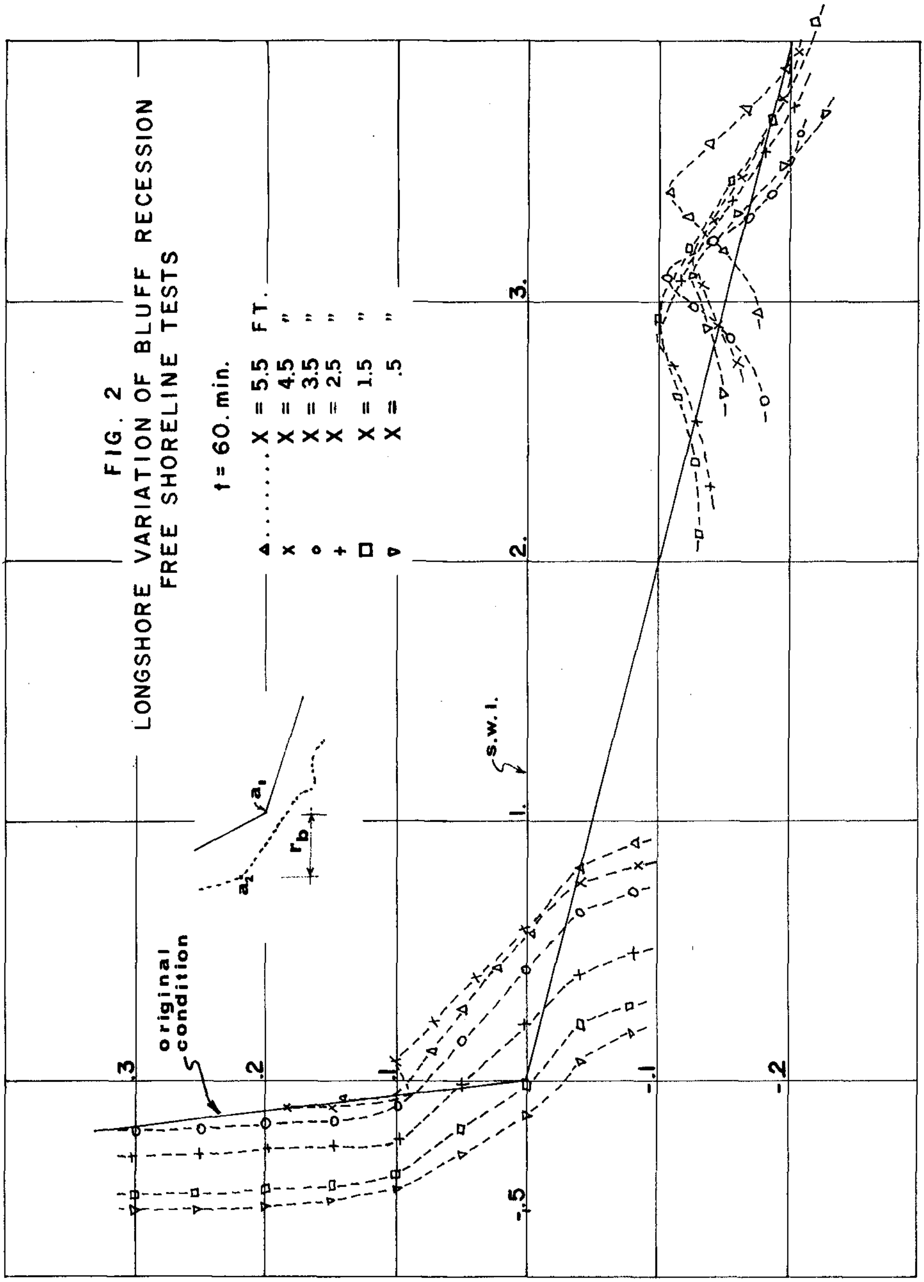


FIG. 2
LONGSHORE VARIATION OF BLUFF RECESSION
FREE SHORELINE TESTS

Creating a Typical Repeatable Natural Shore Process

As previously stated, the experimental arrangement produced typical bluff erosion. Similarity between the model processes and natural processes is illustrated in Fig. 2 by six profiles which show the decreasing bluff recession as one procedes downdrift along the bluff. This is due to the protection provided by the increasing amount of littoral drift in the downdrift direction. As shown in Fig. 1 a section of the bottom just updrift from the test section was a plate rather than a sand bed. Therefore the only source of littoral drift along the bluff test section was the erosion and slumping of the bluff itself. Consequently at the first test section (x = 0.5) there was virtually no littoral drift and at each successive section the amount of littoral drift was greater thus providing increasing natural protection by the presence of this increasing amount of beach material. It may be seen in Fig. 2 that the recession was a maximum at x = 0.5 and decreased at successive sections to nearly zero at x = 5.5 feet. Checks on wave height along the breaker zone indicated no significant orderly change in wave height along the test section. Other indications that the model was creating typical natural shore processes were the creation of typical sand bars in the breaking area, as shown in Figs. 2 and 3 and the increase in recession rates with an increase in water surface elevation as illustrated in Fig. 3. The profiles in Fig. 3 show bluff recession at the same section (x = 1.5) for the normal water level, and for levels 0.02 feet(0.6 cm) above and below normal respectively. This change in level in the model corresponds to an increase in level in nature of 1.0 feet (0.3 m). In studying the profiles shown in Figs. 2 and 3 it should be noted that the vertical and horizontal scales differ in a ratio of 1 to 5.

The investigation of the repeatability of the tests required that consideration be given to the fact that it was impossible to reproduce identical test conditions. This was because there was no control over the water temperature in the wave tank and because the actual average wave height was not known until the end of the test when it was determined from the oscillograph charts. In order to take into account these variables an application of dimensional analysis was made to develop a parameter which would include the effect of small variations in wave height and viscosity as well as the duration of the tests. The following parameter was found to serve the purpose of coordinating the test results.

$$N = k \left(\frac{gTH}{\nu}\right)^2 \qquad (1)$$

In this parameter g is the gravitational acceleration, T is the wave period, H is the wave height, ν is the kinematic viscosity and k includes various constants and the duration of the tests. Since the only variable in k was the duration of the tests k was taken as unity for 30 minute tests and as two for 60 minute tests. The presence of H in the numerator is a measure of the original wave energy and ν in the denominator is related to the energy dissipation. The usefulness of this parameter will be demonstrated

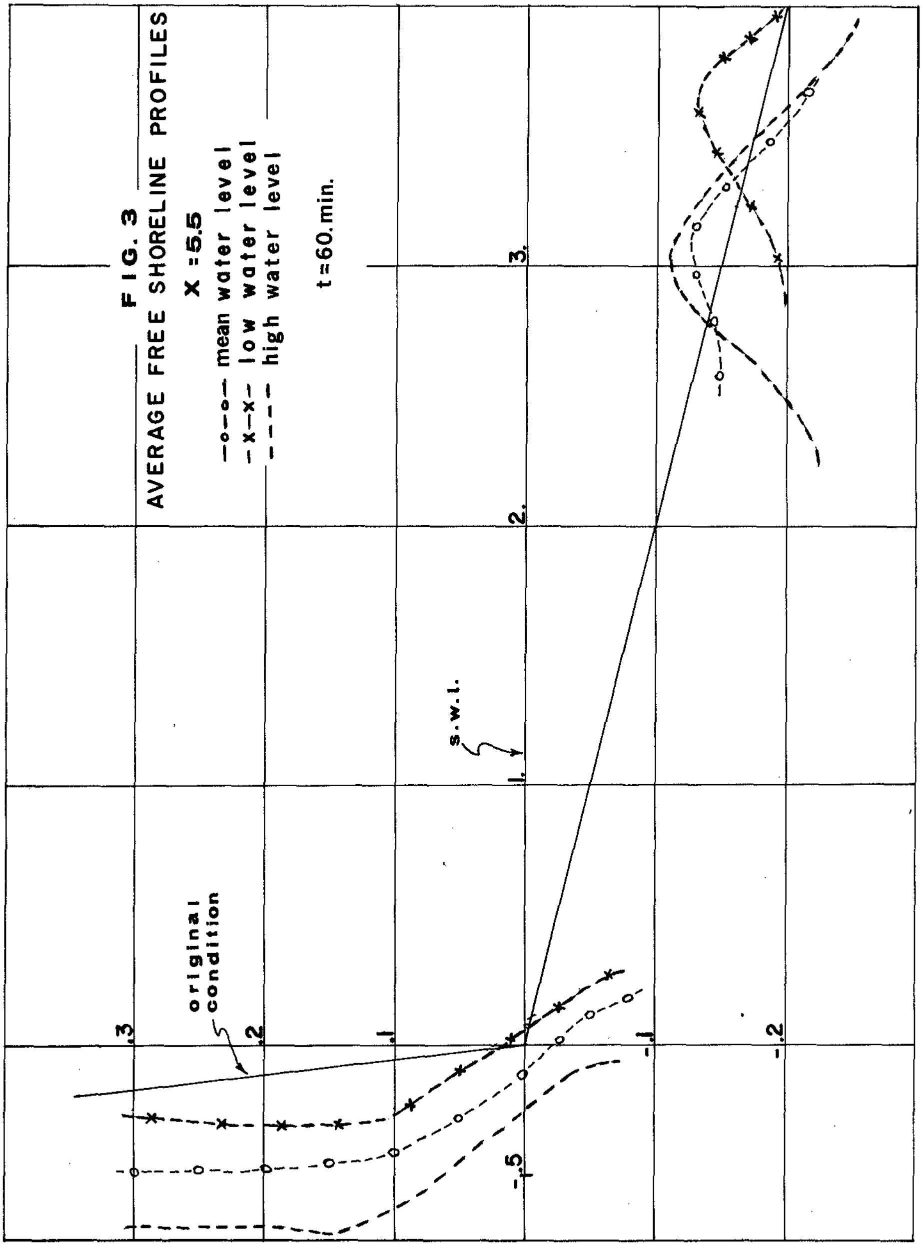


FIG. 3
AVERAGE FREE SHORELINE PROFILES
X = 5.5
—o—o— mean water level
—x—x— low water level
- - - - high water level
t = 60. min.
s.w.l.
original condition
.3
.2
.1
-.1
-.2
-.5
1.
2.
3.

but it should be noted that additional work is in progress which indicates that a more rational parameter can be developed which will also include the distance from the breaker zone to the bluff.

In order to relate bluff recession to the parameter N, the recessions at any section was represented by r_b the recession of the toe of the bluff. As illustrated in Fig. 2, r_b is the recession of a_1 to the new position a_2. The parameter r_b was considered to be the best single measure of the recession of the bluff. The use of this procedure is illustrated in Figs. 4 and 5 where values of r_b are plotted against N for three test conditions. In Fig. 4 the results are for section x = 1.5 and Fig. 5 shows the results at section x = 2.5. Consider first the center group of nine points in each figure. These show the results of tests made with a standard wave height on the free shoreline. The five points with the smaller values of N show the recessions produced in 30 minutes and the four with the greater values of N give recessions produced by 60 minute tests. The lines drawn through the values plotted in Figs. 4 and 5 were determined by a least squares analysis. The correlation coefficients shown are much higher than the values required for a one percent confidence level, thus indicating that these lines represent a logical relation between recession and important variables. Considering the random nature of the sand slumping process it was concluded that these correlations provided satisfactory evidence that repeatable recession rates were being obtained. The upper and lower graphs in Figs. 4 and 5 give two additional examples of groups of test points, the upper sets being for a smaller wave height with a free shoreline and the lower sets show the effect of a groin system using the standard wave height. Again, the high correlation coefficients indicate that the linear relationship is a reasonable interpretation of the trends and that a good repeatability was being obtained.

It is of interest to note in Figs. 4 and 5 that the smaller wave height of 0.12 feet (3.7 cm) produced greater bluff recession than the normal wave height which was 0.20 feet (6.1 cm). This is believed to be because the larger waves broke nearly twice as far from the bluff as the smaller waves and therefore had nearly twice as much energy dissipation. This research did not include sufficient tests on variable locations of the breaker zone to permit drawing conclusions. However, as previously mentioned, it is expected that future work may provide more information on this important variable.

The relations between bluff recession, r_b, and the parameter N provide a better way of presenting test results than the comparison of beach profiles because the effect of small variations in wave height and viscosity do not obscure the trends and because the two different test durations can be included in one graph. For example, the difference in bluff recession between the updrift and downdrift portion of the model which was demonstrated

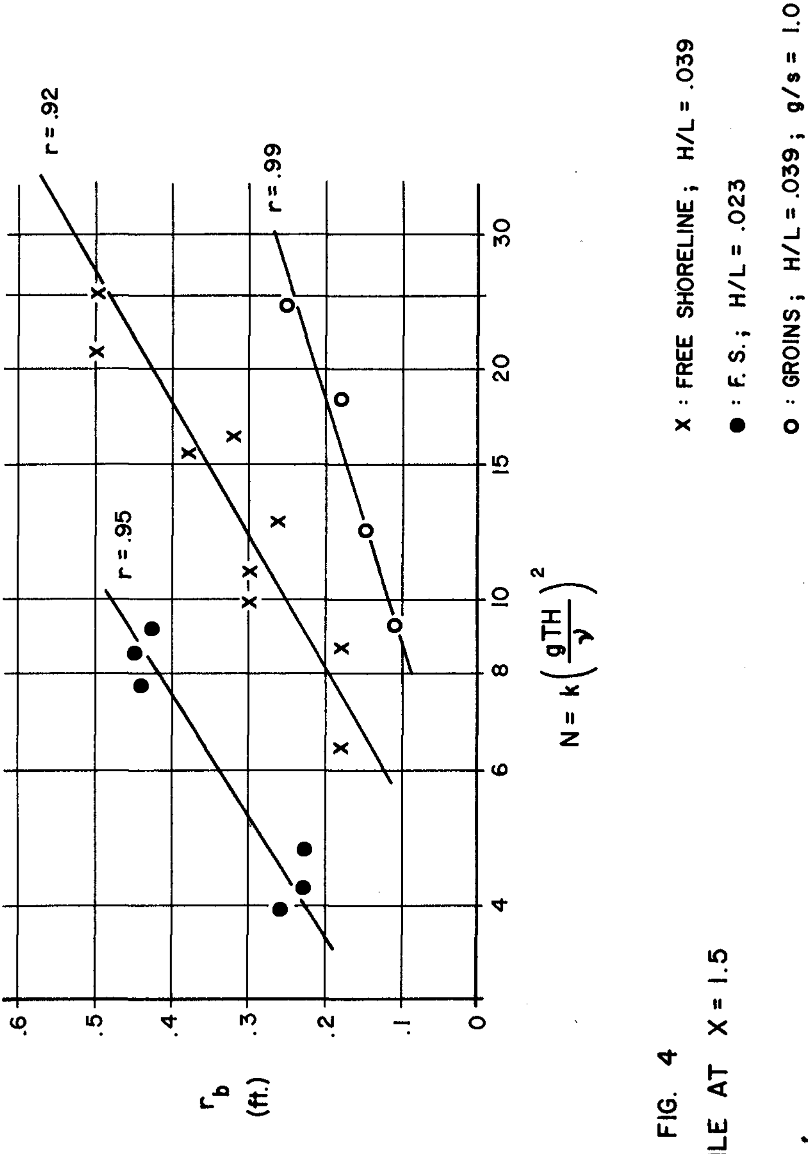


FIG. 4

PROFILE AT X = 1.5

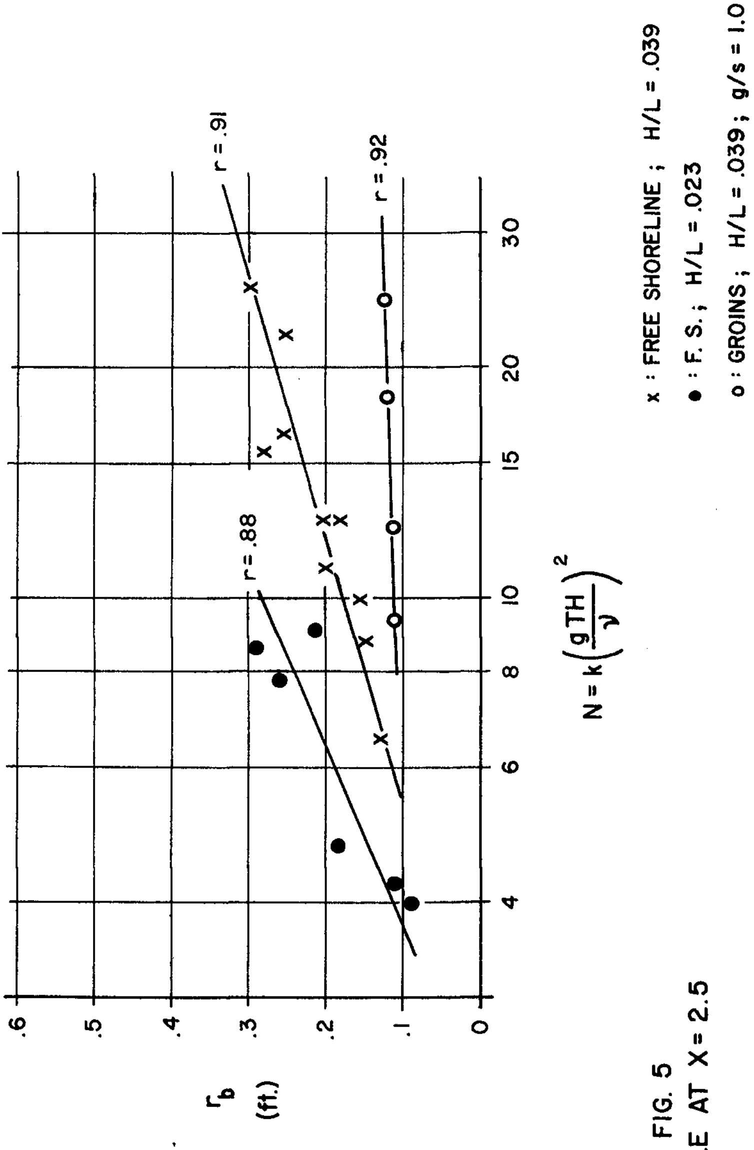


FIG. 5
PROFILE AT X = 2.5

with individual profiles from a single test in Fig. 2 is shown in Fig. 6 by lines relating r_b to N. Each of these lines is the best fit line for all the values of r_b determined at a given section from a series of test runs. The individual test points are not shown in order to simplify the presentation. The information in Fig. 6a is for a free shoreline. The lines are for successive test sections from x = 0.5 to x = 5.5. Again, the protection provided by increasing littoral drift is demonstrated. Figure 6b shows a similar set of curves for the same wave conditions but with a groin system which was initially nourished. Again, the maximum erosion occurred at the updrift sections, x = 0.5 and x = 1.5. However, section 5.5 experienced slightly more erosion than sections x = 2.5, 3.5, and 4.5. The reason for this is believed to be that the groin system delayed some of the littoral drift near the center of the groin system and there was slightly less sand present at x = 5.5.

Another indication of the reproduceability of the tests is obtained by plotting total sand transport against the parameter N as shown in Fig. 7. Here again, there is some random scatter as one would expect in a process so susceptable to variations in behavior but points fell well within the one percent confidence level based on the statistical test provided by the linear correlation coefficient. It should be reiterated that except for the difference in the test duration (30 to 60 minutes) the variations in N within each group of points are due to variations in the viscosity of the water and to minor variations in wave height.

The Effect of Shore Protection Procedures

Various procedures were tested and compared with recession rates for free shoreline conditions. Some of the procedures were selected to supplement data from the demonstration projects, others because of ideas advocated by public groups and some to check procedures observed in the field.

Some results of tests on groin systems are shown in Fig. 8. The curves of recession versus N are shown for two locations x = 1.5 (Fig. 8a) and x = 2.5 (Fig. 8b). The results are shown for three groin systems having different lengths and spacing and for one of these systems combined with a permeable wall. In all cases the freeboard of the groins was 0.02 feet (0.6 cm) in the model which corresponds to one foot (0.3 m) in nature. For each set of tests the length of the groins (g) as well as the ratio of length to spacing (g/s) is shown. Comparison with the free shoreline results which are also shown in Fig. 8 shows that the erosion rate was reduced by means of all the groin systems. The two sets of tests for the same length to spacing ratio (g/s = 0.8) give approximately the same results and when g/s was increased to 1.0 the protection was much greater. While this is as would be expected the range of variables used in the tests may be too limited to warrant reaching any general conclusion regarding groin spacing. It should be noted that the two groins

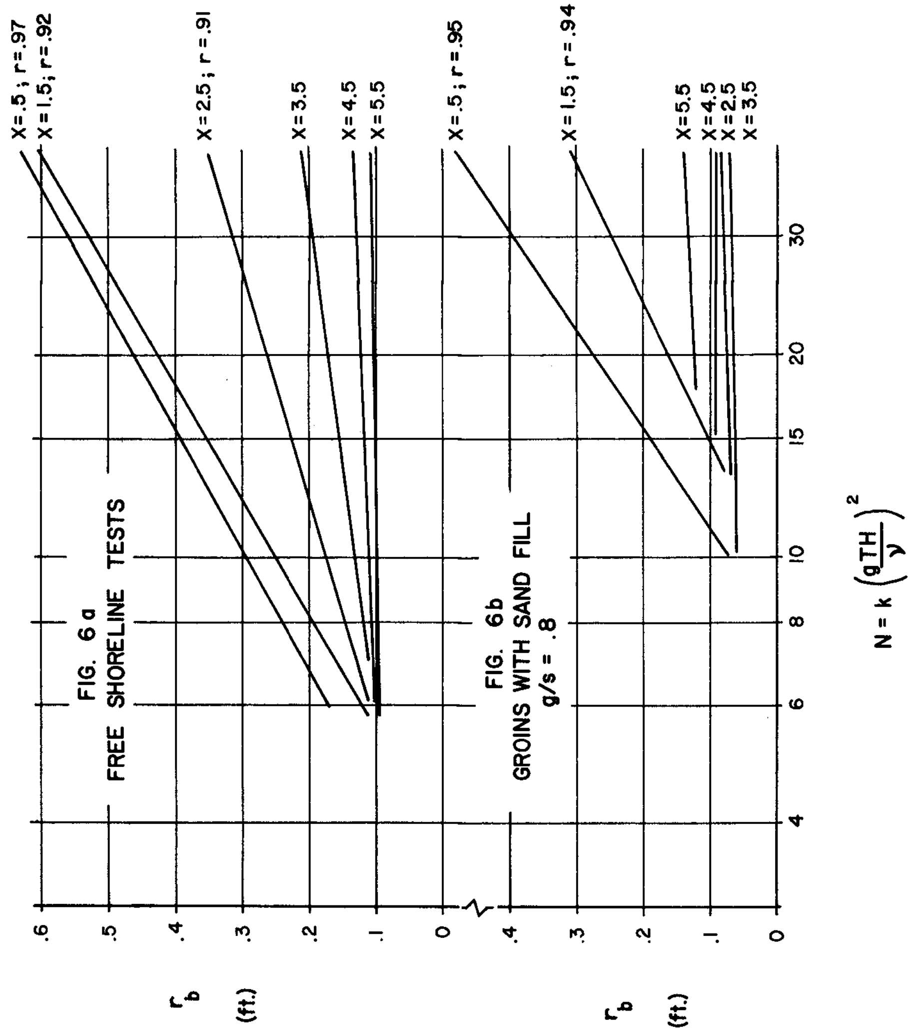


FIG. 6a
FREE SHORELINE TESTS
FIG. 6b
GROINS WITH SAND FILL
g/s = .8
X = .5 ; r = .97
X = 1.5 ; r = .92
X = 2.5 ; r = .91
X = 3.5
X = 4.5
X = 5.5
X = .5 ; r = .95
X = 1.5 ; r = .94
X = 5.5
X = 4.5
X = 2.5
X = 3.5
r_b (ft.)
$N = k\left(\frac{gTH}{\nu}\right)^2$

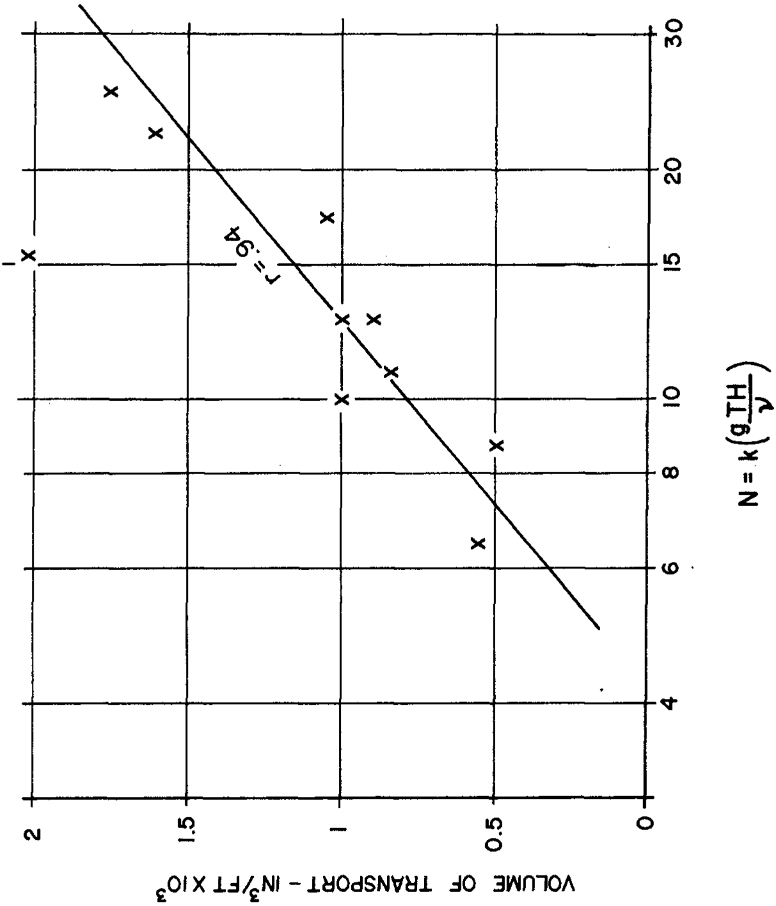

VOLUME OF TRANSPORT - $IN^3/FT \times 10^3$
2
1.5
1
0.5
0
$N = k\left(\frac{gTH}{\nu}\right)$
4
6
8
10
15
20
30
r = .94

FIG. 7

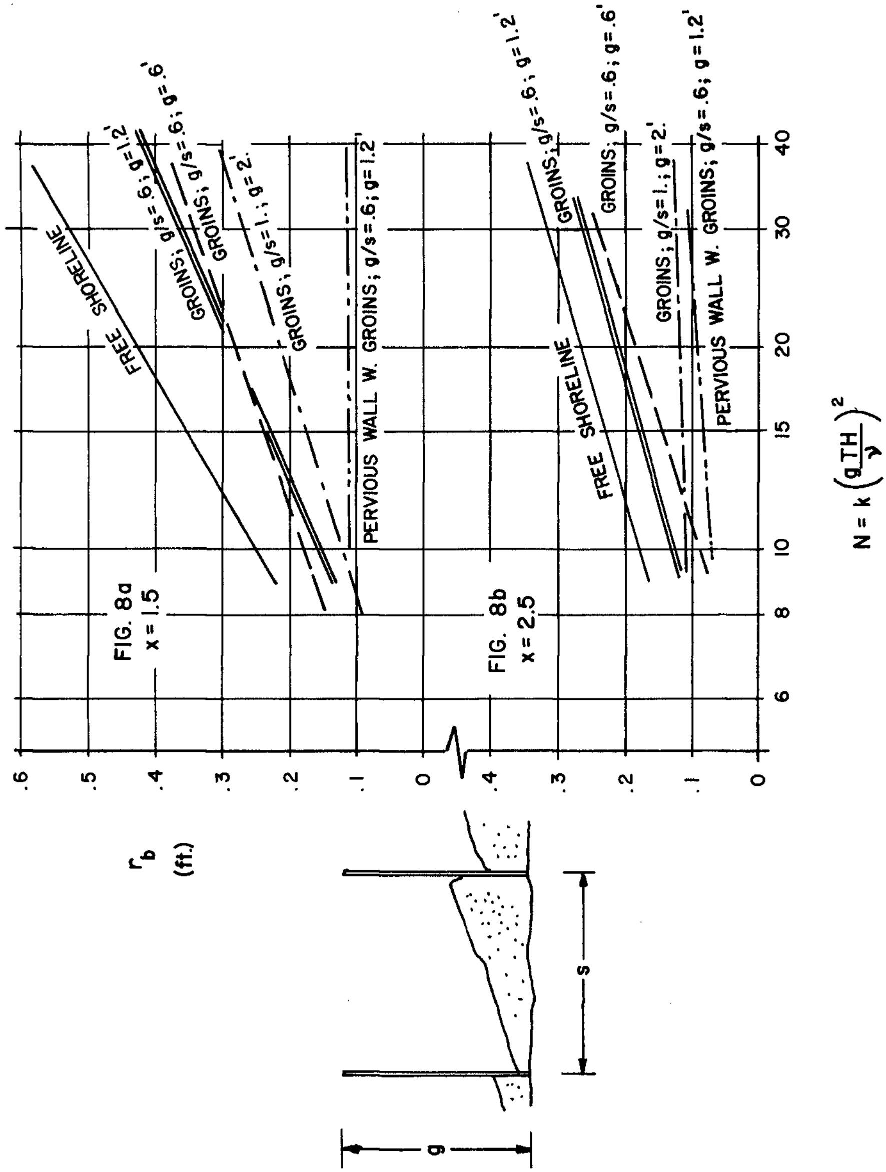


FIG. 8a
x = 1.5
FIG. 8b
x = 2.5
FREE SHORELINE
GROINS; g/s = .6; g = 1.2'
GROINS; g/s = .6; g = .6'
GROINS; g/s = 1.; g = 2'
PERVIOUS WALL W. GROINS; g/s = .6; g = 1.2
PERVIOUS WALL W. GROINS; g/s = .6; g = 1.2'
r_b (ft.)
N = k(gTH/ν)^2
s
g

lengths used in the tests correspond to lengths of 40, 80, and 100 feet (12, 24 and 30 m) in nature. The greatest reduction in erosion occurred when a permeable wall was combined with the groins. The system was tested because it has been extensively used with considerable success on the Michigan coast. The location of the pervious wall is shown in Fig. 8c. The wall had a porosity of 30 percent which simulated a wall made with 2 inch x 8 inch (5 cm x 20 cm) lumber placed vertically with 3 inch (7.5 cm) gaps. The addition of the pervious wall to the groin system brings the cost up to the upper range of "low cost" shore protection.

Some of the graphs from Fig. 8 are repeated in Fig. 9 where results are also shown for groins with sand fill, pervious walls without groins and for a submerged breakwater. The sand fill consisted of the same sand used in constructing the model bluff and bottom. It was placed in the space between the groins, with the top of the sand at the elevation as the tops of the groins. At the outer ends of the groins the sand was allowed to assume its natural slope. It will be seen in Fig. 9 that this provided very effective protection through the full 60 minutes of testing. Note that at x = 1.5 the pervious wall with groins is more effective than the nourished groins, whereas at x = 2.5 the reverse is true. The pervious wall for which results are shown in Fig. 9 had a porosity of 40 percent and was located a distance from the shore line which corresponds to 20 feet (6.1 m) in nature. It may be seen in Fig. 9 that this wall provided considerable protection.

The submerged breakwater was tested because of considerable public interest in this type of protection. It was installed in the breaker zone. When originally installed the ratio of its height above the bottom to the depth was 1:3.4. After the model was in operation for a few minutes this ratio became about 1:2.5. The use of such a low barrier produced no noticeable reduction in bluff recession.

The effect of groin systems on littoral drift is shown in Fig. 10. The ordinates of the graphs are rate of transport and the abscissas are distance from the shore line, zero being point a_1 in Fig. 2. The twelve plotted points give the rate at which sand was collected in the twelve individual pans and each point is plotted at the center of the pan. The values of total transport plotted in Fig. 7 were obtained by integrating the area under one of the curves. Each of the curves in Fig. 10 represents an average of the curves for that series of repeated tests. The free shoreline curve shows that the littoral drift is large in the uprush zone near shore and in the breaker zone. The curve for a groin system shows that the groins stopped most of the littoral drift in the near shore area and reduced the drift in the breaking zone. However, the curve for groins with sand fill shows that when nourishment was supplied to the groin system the littoral drift is even greater than with the free shoreline. It should be recognized that the transport during tests with the

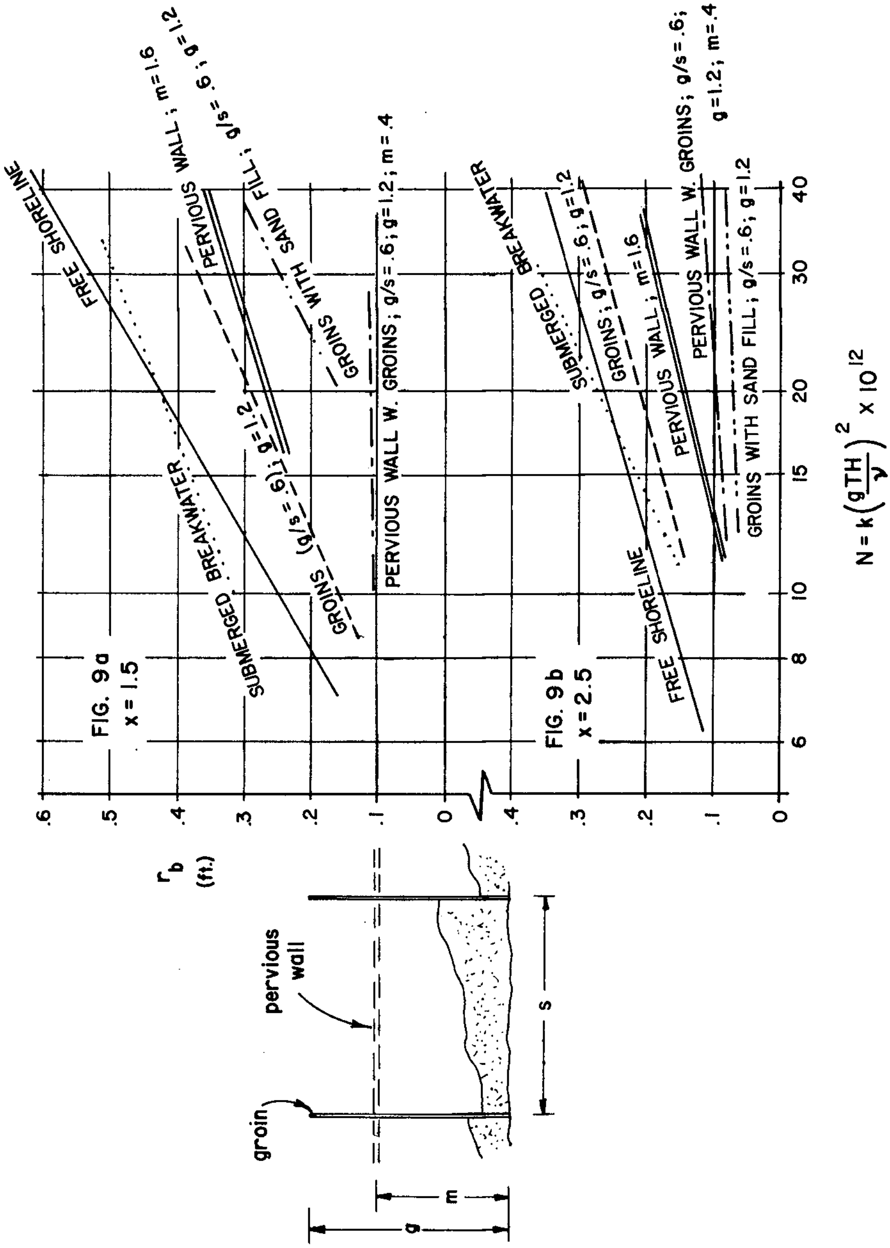
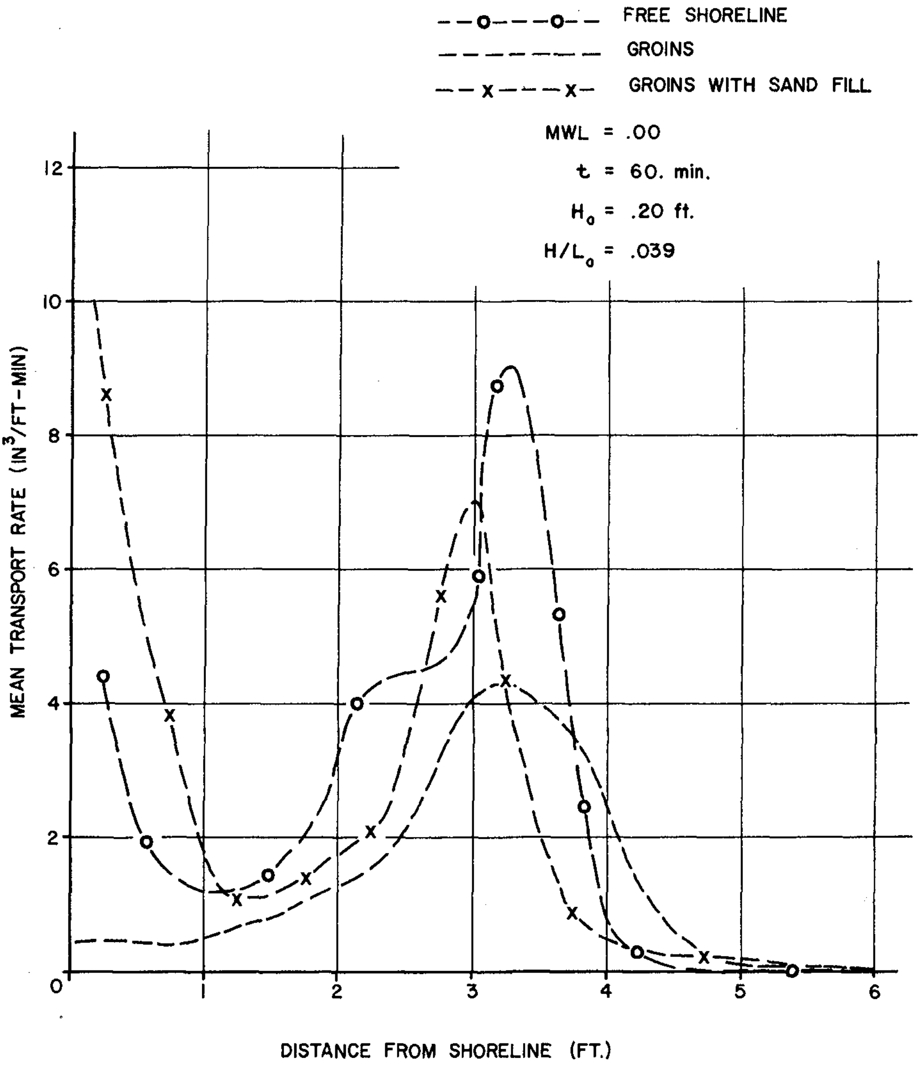


FIG. 9a
x = 1.5
FIG. 9b
x = 2.5
FREE SHORELINE
SUBMERGED BREAKWATER
PERVIOUS WALL ; m=1.6
GROINS WITH SAND FILL ; g/s = .6 ; g = 1.2
GROINS (g/s = .6) ; g = 1.2
PERVIOUS WALL W. GROINS ; g/s = .6 ; g = 1.2 ; m = .4
GROINS ; g/s = .6 ; g = 1.2
PERVIOUS WALL W. GROINS ; g/s = .6 ; g = 1.2 ; m = .4
GROINS WITH SAND FILL ; g/s = .6 ; g = 1.2
r_b (ft.)
$N = k\left(\frac{gTH}{\nu}\right)^2 \times 10^{12}$
groin
pervious wall
s
m
g

FIG. 10
SAND TRANSPORT



groins, without and with nourishment, would probably change if the tests were continued longer. The un-nourished groins would be expected to fill with sand naturally and then littoral drift could be expected to resume in the near shore area. On the other hand, the nourished groins would eventually lose enough of the fill material so that the littoral drift could be expected to drop back to a more normal rate. Some tests were made with higher groins. These created wave reflections which directed the wave energy toward shore on the updrift sides of the groins. This is illustrated by the shorelines plotted for high and low groins in Fig. 11.

Tests were also made to gain more information on one of the field demonstration projects. This project consisted of an offshore breakwater constructed of zig-zag concrete walls. The structure provided considerable protection for a number of years but during a large storm severe erosion occurred behing the structure. The model studies also showed that some protection was being provided by this structure when compared with a free shoreline. However, when the water level in the model was raised one foot (0.3 m) to the top of the structure the bluff erosion was more than doubled. This information showed that the wind tide which occurred during the destructive storm was an important factor in reducing the effectiveness of the structure.

Summary

A laboratory investigation of shore erosion processes was undertaken to supplement a shore protection demonstration project in Michigan. The purpose of both the field project and the laboratory studies was to familiarize individuals and public agencies with shore protection methods and to provide information on the selection and construction of protective procedures. Although it was recognized that erosion rates could not be converted quantitatively from the model to nature it was hoped that the relative effectiveness of various protective procedures could be evaluated. If this could be done, the advantage of the model over field projects would be lower cost, the control over variables such as wave height and water surface elevation and the shorter time required. One of the chief difficulties with field demonstration projects is the random nature of the occurrences of onshore storms of a size sufficient to test but not destroy the structures.

The preliminary tests showed that natural processes were simulated in the model and after considerable experimentation in constructing the model it was found that repeatable results could be obtained. Because, even in the model, it was impossible to control wave height exactly and because there was no control over the water temperature it was necessary to test for repeatability by plotting bluff recession against a parameter which was related to the ratio of original energy and energy dissipation.

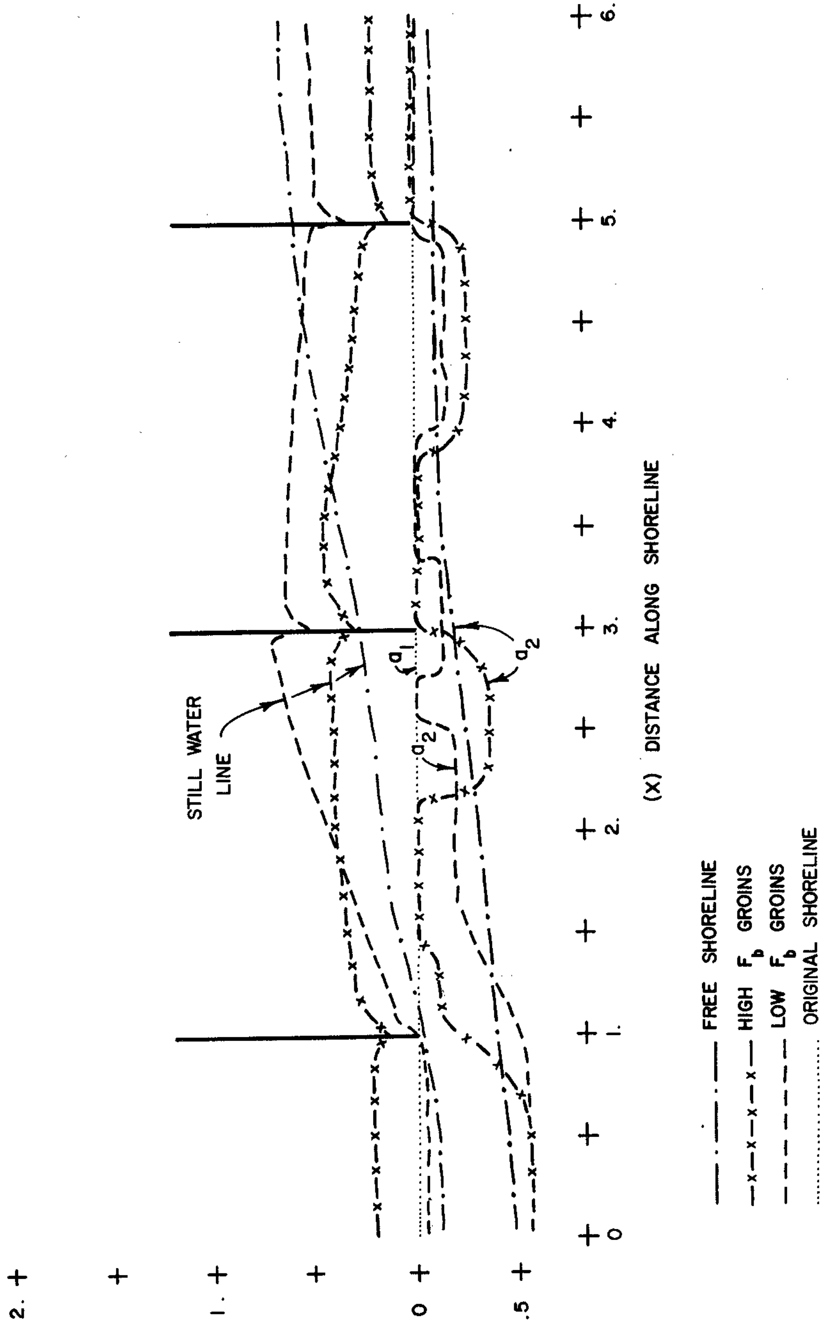


FIG. 11: PLAN VIEW OF EROSION PATTERNS

This parameter also included the duration of the model storms. It was found that comparison of results of tests on various methods of shore protection with the erosion of a free shoreline could also be made by means of plotting bluff recession against this parameter. This parameter also served to coordinate rates of littoral drift. The model was used to evaluate some well established procedures such as groin systems and beach nourishment. Tests were also made to provide additional information on one of the demonstration projects and to determine the effectiveness of some procedures which had considerable public interest but were not included among the field projects.