FINAL PROJECT REPORT

HYDRAULIC MODEL STUDY
TAWAS BAY MARINA
HARBOR MODIFICATION EVALUATIONS

Report UMCEE 00-08

By

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ANN ARBOR, MICHIGAN

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For

Tawas Bay Marina Board of Directors

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1.0 INTRODUCTION

In the past, resonant interactions during periods of significant wave activity have caused an increase in wave heights within Tawas Bay Marina. The increased wave heights produced significant problems. First was the excessive motion of the finger piers, which made it difficult to use portions of the marina, and second was structural damage to the piers themselves. These problems led to the Tawas Bay Marina Board of Directors to commission a study of various internal modifications to reduce internal wave heights. The study was completed in 1994 and conducted by the University of Michigan (1). The results led to the construction of dock and side gabions within the harbor. These changes reduced finger pier motions sufficiently enough to facilitate use of the marina, but structural damage has continued to this date. Another important issue is lake-level. When the gabions were initially installed they were approximately two-thirds submerged (2 ft above the water line, 4 ft below). As of last summer, they were only about one-third submerged. If lake-levels continue to fall, their effectiveness will diminish. Therefore, a decision was made to examine the possibility of external options to further alleviate wave activity within the harbor.

The purpose of this report is to summarize the results of the second hydraulic model study completed by the University of Michigan on Tawas Bay Marina. This study considered four potential breakwater configurations to determine which is most effective at dissipating the problematic wave activity within the marina.

Tawas Bay Marina is located on Lake Huron near Tawas City, Michigan (Figure 1). The marina has dimensions of approximately 600 feet by 600 feet, and is protected by seawalls and beaches. The average water depth within the marina is about 11 feet, depending on Lake Huron water levels. Significant fetches exist from the east around to the south across Lake Huron and Tawas Bay respectively.

Figure 2 is a wave rose that depicts the offshore wave climate for Tawas Bay. The figure was obtained from the Lake Huron Wave Information Study produced by the U.S. Army Corps of Engineers (2). The percent of wave events for each direction is represented on the outside of the circle in the small "pie" pieces. For example, 15 percent of all wave events are from the South and 8 percent are from the East. The magnitude of the waves is then depicted inside the circle also as a percentage. For example, when more closely examining the 15 percent of wave events from the South, about 42 percent are less than 0.4 meters and about 85 percent are less than 0.9 meters. So, when considering the wave climate for the Marina, a majority of the waves under one meter are from the South, while the larger wave events, greater than 1.5 meters, are from the East. In addition, wave heights on the order of one meter, which are thought to cause the problems within the marina, are common from all directions.

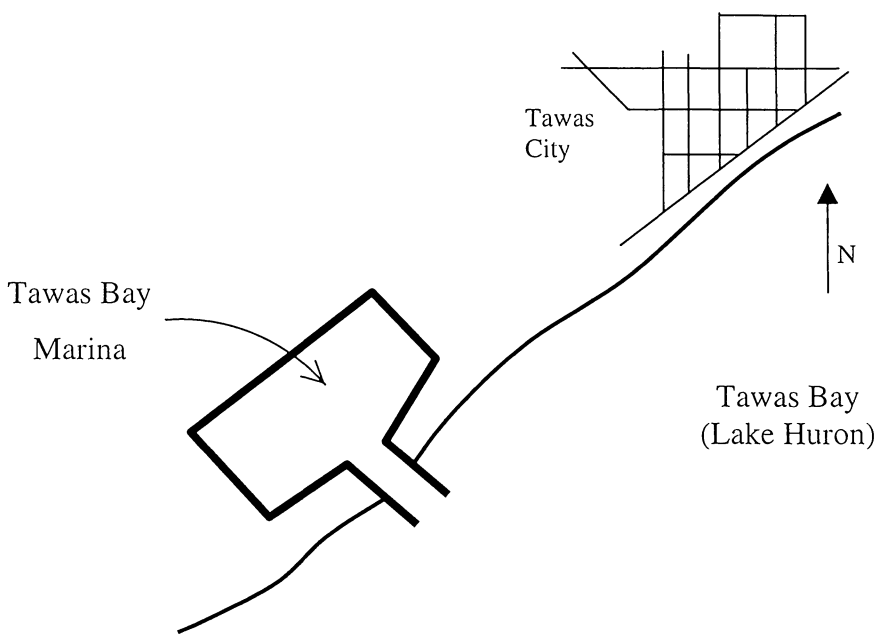

Figure 1: Site Location (not to scale).

2.0 ANALYSIS PROCEDURE

2.1 Physical Model

A physical model of the Tawas Bay Marina was constructed in the University of Michigan Civil and Environmental Engineering Structures Laboratory. The model was constructed to a nominal linear scale of 1 to 30 (model to prototype). The selection of model scale ratio was based primarily on the available space and the area required for construction of the model. The basic model layout consists of a general outline of the marina with approximate overall dimensions of twenty feet by twenty feet. Wire mesh baskets filled with gravel were constructed at the correct scaled size to represent the existing gabion systems installed within the marina. These were installed such that at the tested water depth, approximately one-half the vertical extent of the baskets were submerged in a still water condition. This corresponds to what was described as the existing state in the marina in the fall of 1999. A constant water depth of 4.5 inches was maintained throughout the tests.

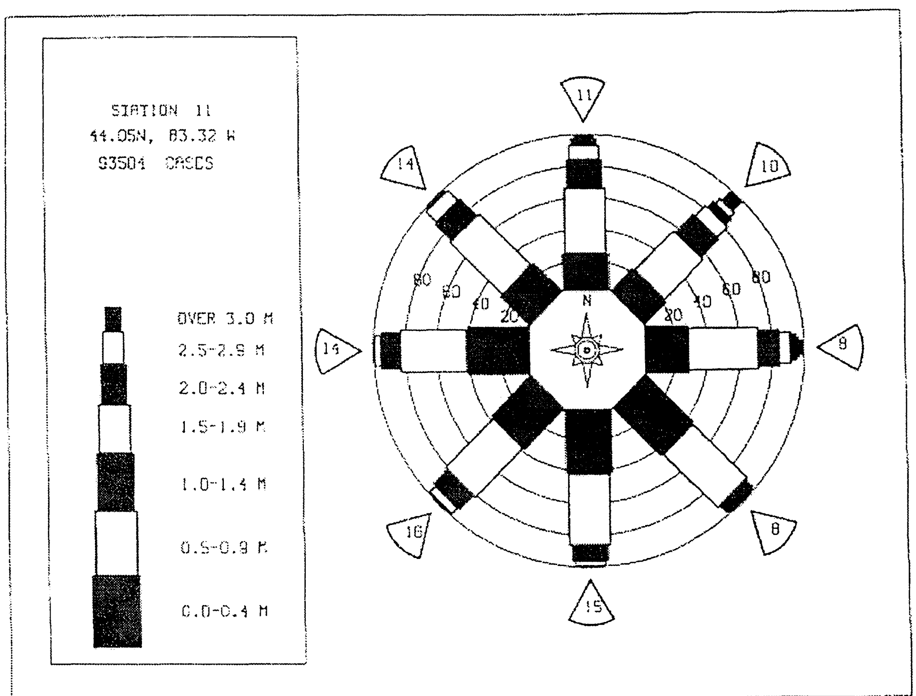

Figure 2: Wave Rose for Tawas Bay.

To determine which wave conditions should be evaluated, visual observations of internal wave heights were made utilizing the existing harbor conditions. Wave periods were varied from 3.9 to 5.5 seconds in 0.1-second increments and areas of increased wave heights recorded. The periods that appeared to result in the worst resonant conditions within the marina were 3.9, 4.2, and 4.6 seconds. These periods were used for the remainder of the study. The three locations within the marina that exhibited the largest wave motion were at the rear opposite the entrance, the NW corner, and the NE corner. Wave measurement gauges were placed at these three locations and at the entrance (Figure 3).

After measurements with the existing conditions were taken, four variations of a breakwater design were built in front of the entrance. Option 1 consisted of extending the jetty from the south side of the harbor entrance coupled with a straight detached breakwater towards the east (Figures 4 and 5). Option 2 was a detached breakwater with wings placed symmetrically in front of the harbor entrance (Figures 6 and 7). Option 3 connected the detached breakwater from Option 2 to the existing south jetty (Figures 8 and 9). Option 4 was similar to Option 3 except the breakwater is detached from the jetty (Figures 10 and 11). In the design figures, bold lines represent the crest of the breakwaters and prototype dimensions are labeled accordingly.

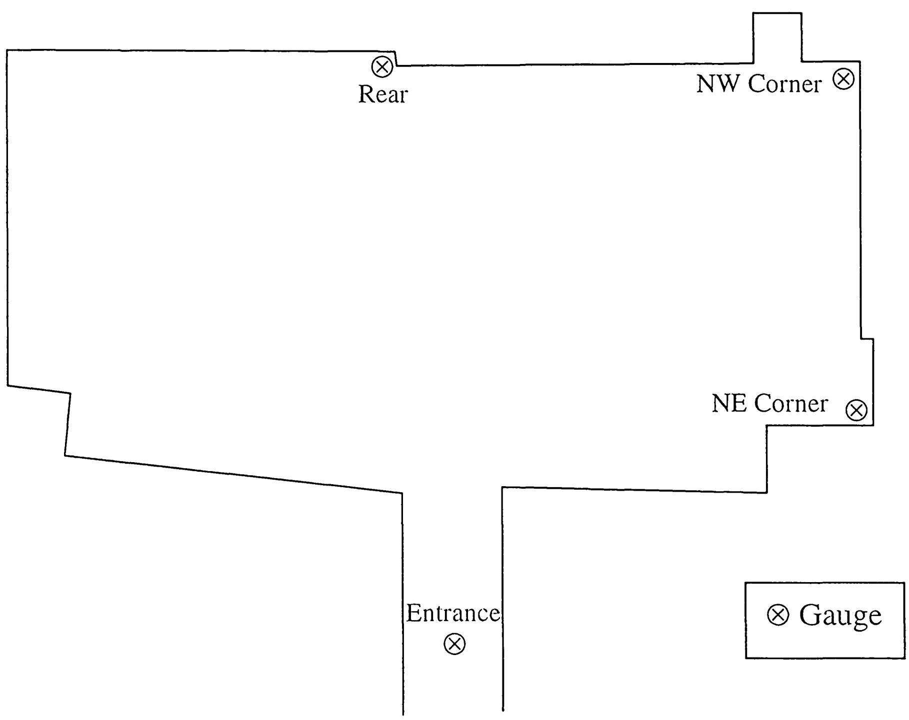

Figure 3: Gauge Locations.

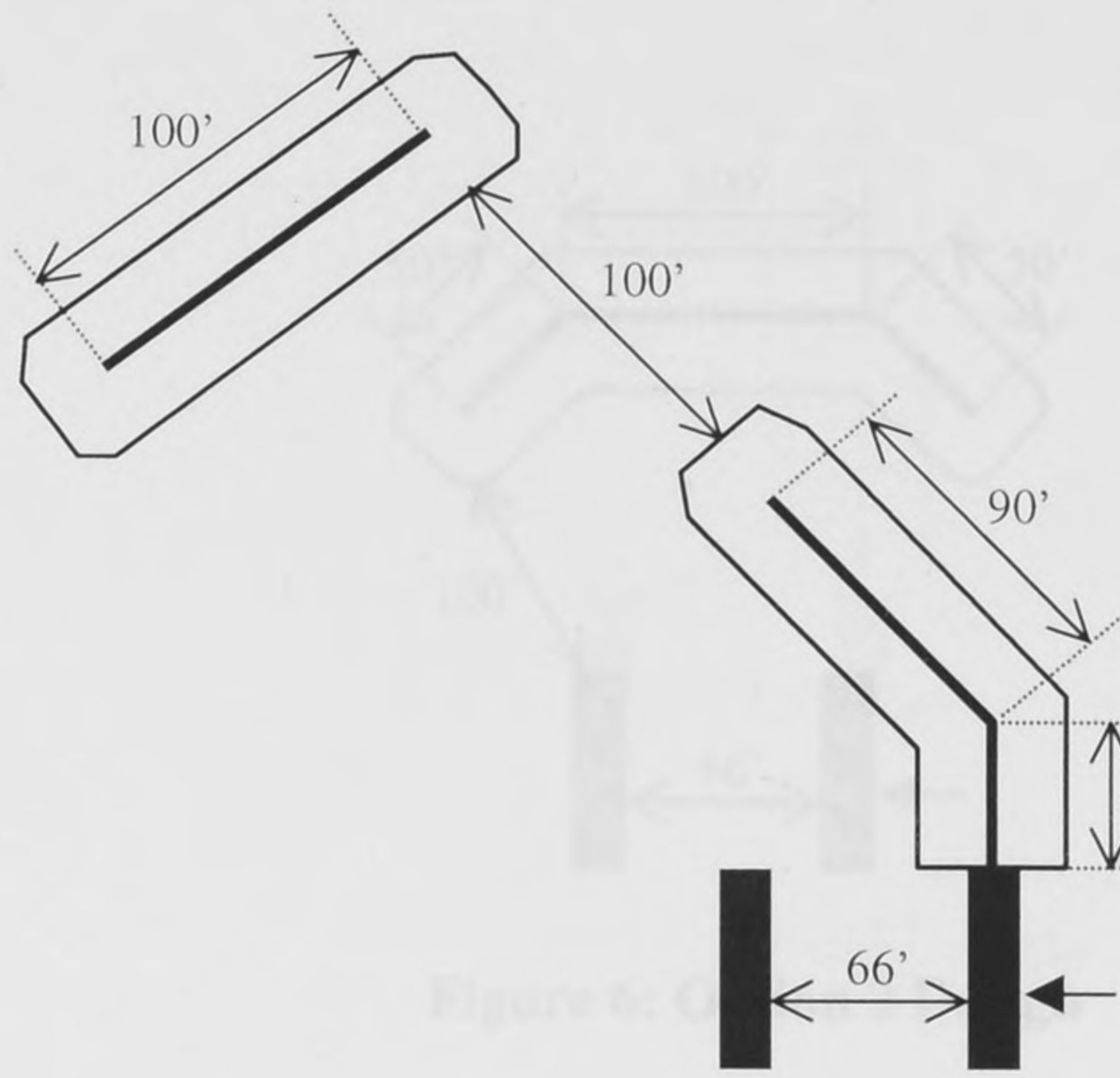

Figure 4: Option 1 Design.

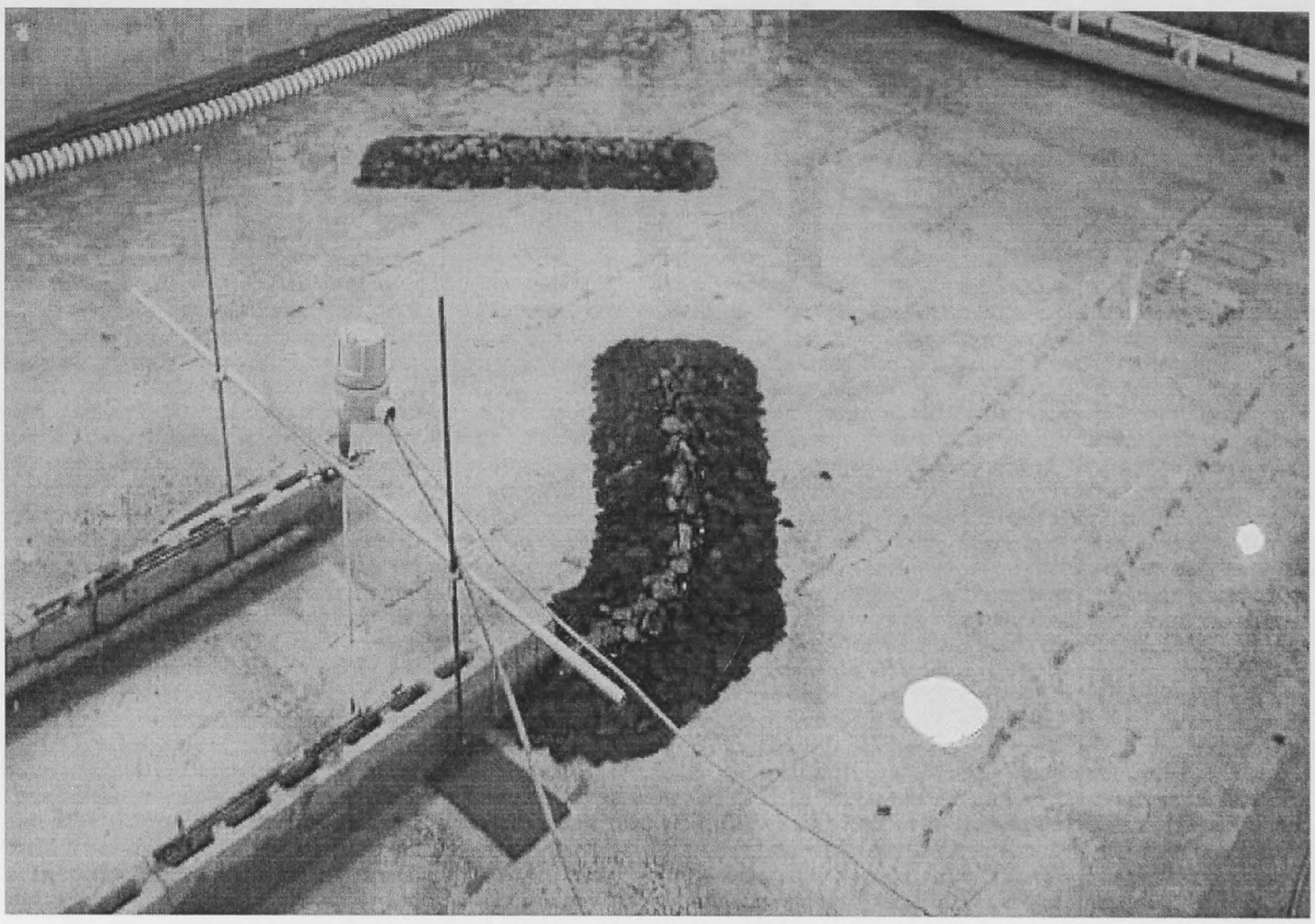

Figure 5: Photograph of Option 1.

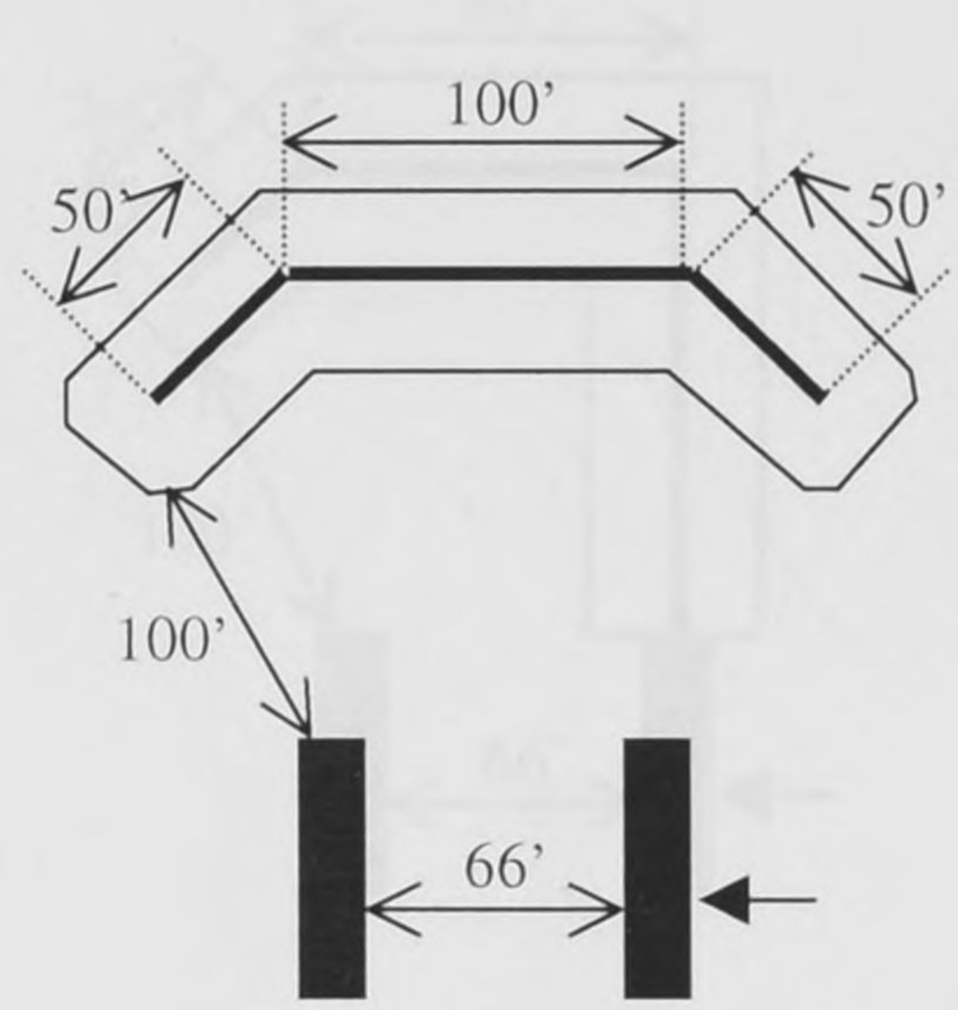

Figure 6: Option 2 Design

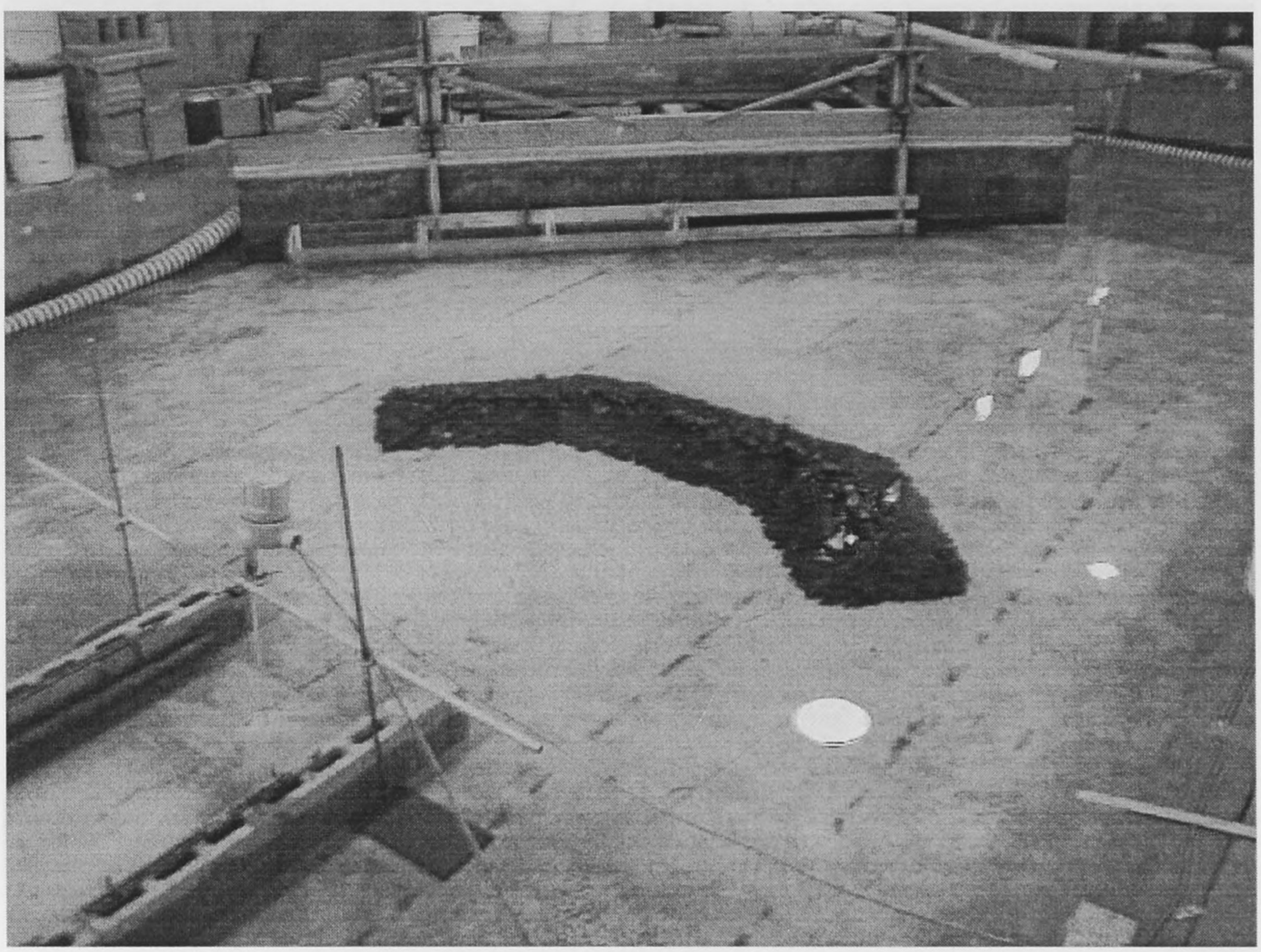

Figure 7: Photograph of Option 2.

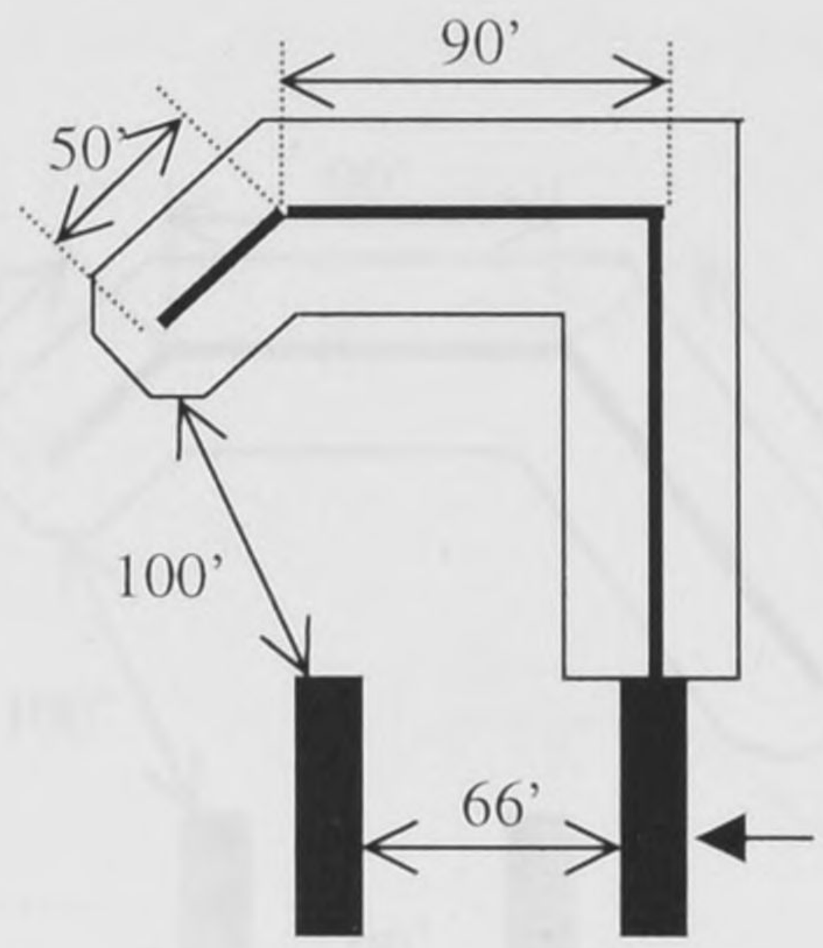

Figure 8: Option 3 Design.

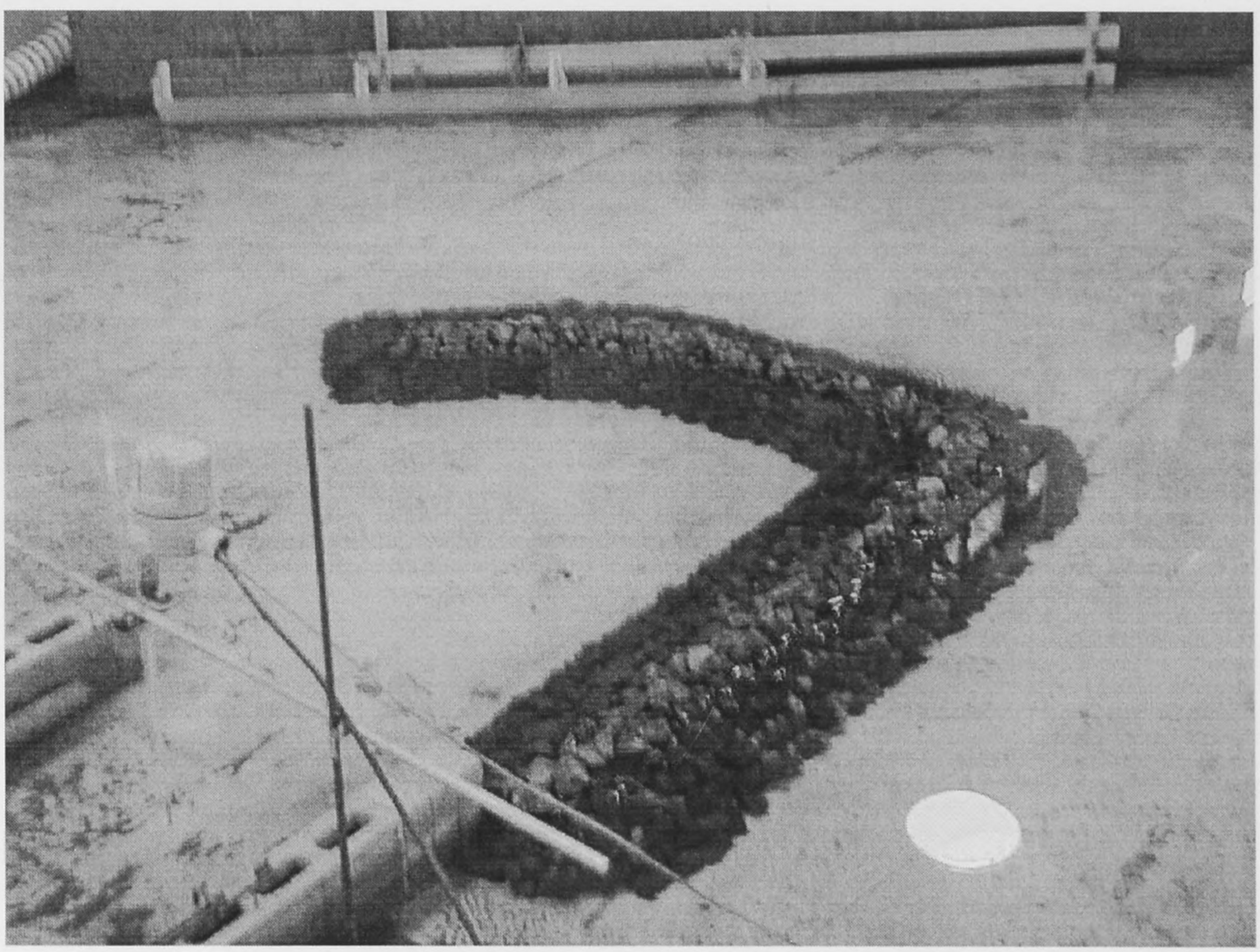

Figure 9: Photograph of Option 3.

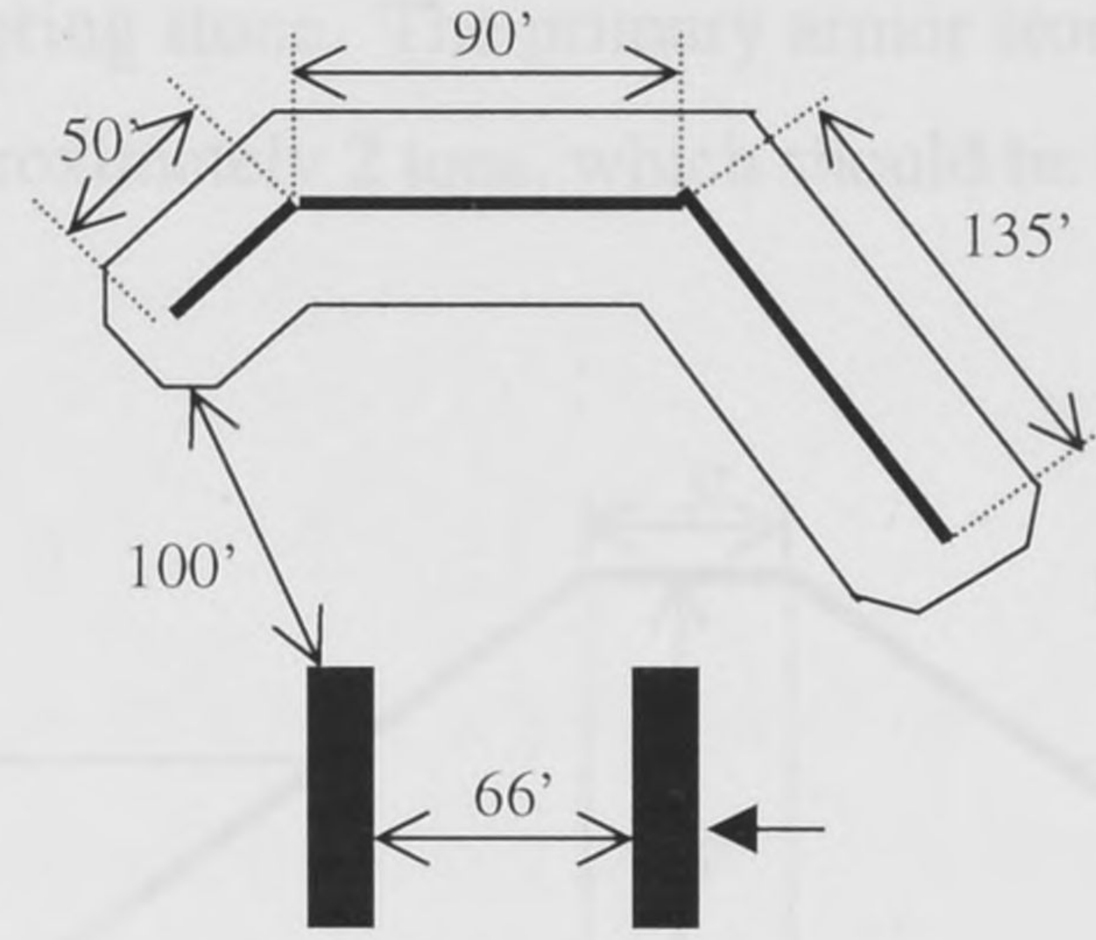

Figure 10: Option 4 Design.

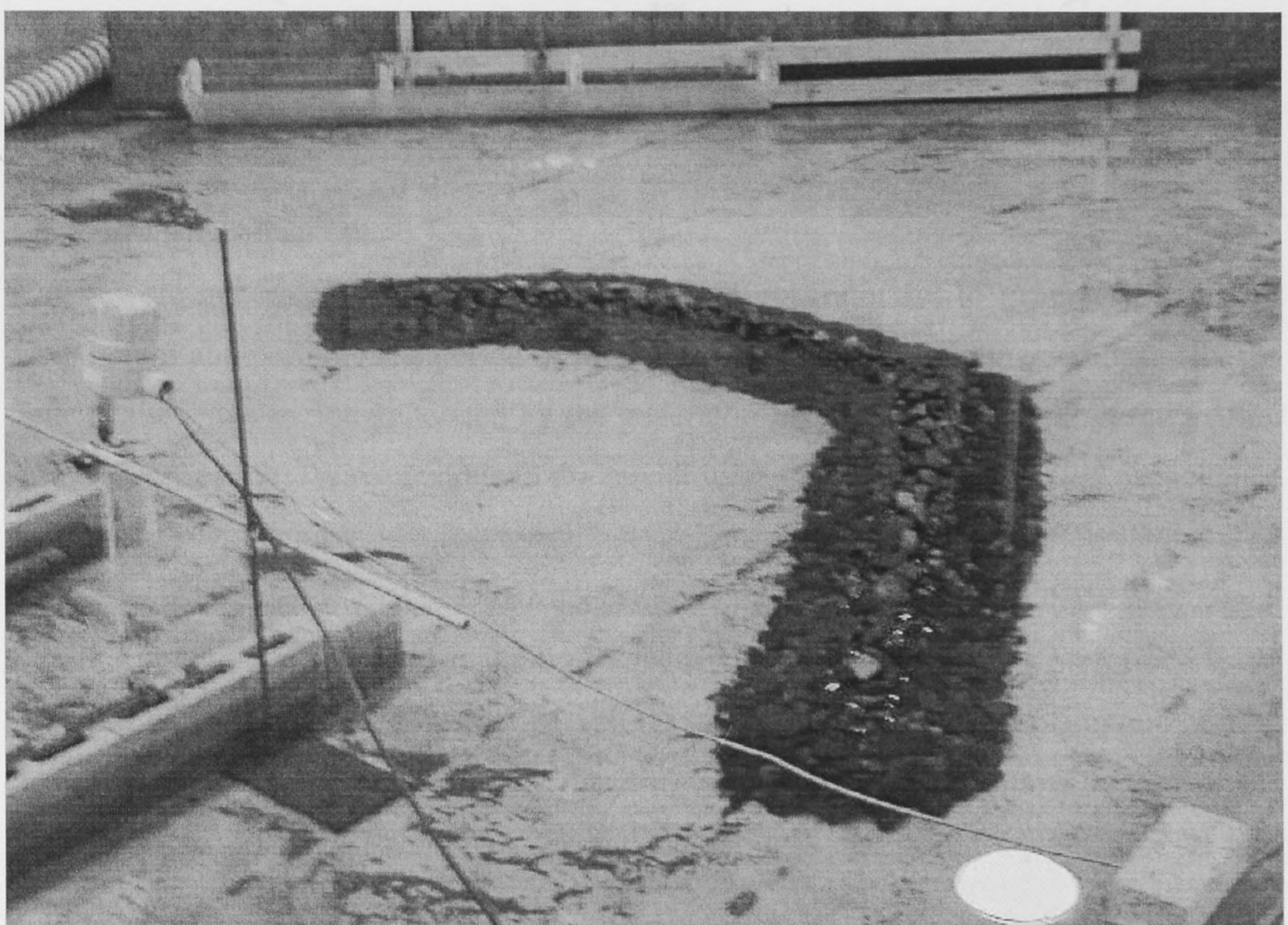

Figure 11: Photograph of Option 4.

The stone for the breakwaters was placed on a 1:1.5 (vertical to horizontal) slope (Figure 9). Two relatively uniform stone sizes were used in building the breakwaters: a finer core stone and a coarser armoring stone. The primary armor stone corresponded to a prototype weight of approximately 2 tons, which should be in the range of the size required at this site.

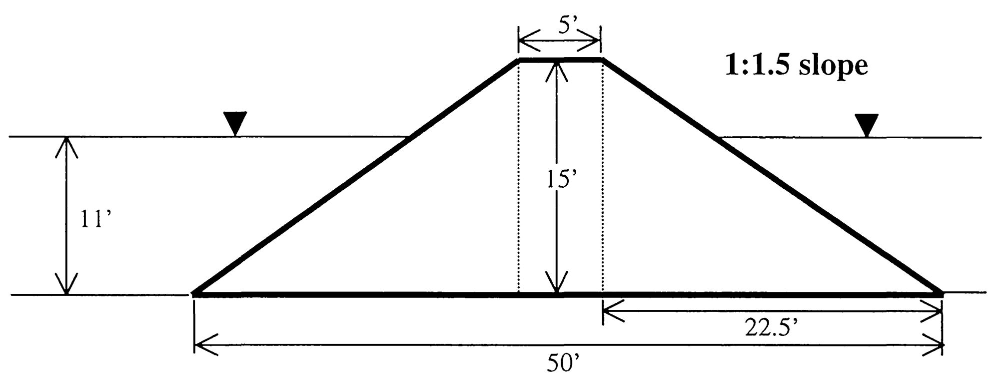

Figure 12: Typical Cross Section

2.2 Wave Measurement

Waves were generated using a plunger-type wave generator set to generate a constant wave height and period. The period of motions within the marina remained basically the same as the wave period generated by the generator, but wave heights within the marina varied significantly. Wave heights in the model were measured using analog capacitance wave gauges which provide a voltage output proportional to water surface elevation. The gauges send a voltage signal to a data acquisition board, which converts the analog signal to digital output at a specified sampling frequency. The data analysis is performed using LabView software, which converts the voltage signal into a water level from calibration curves supplied by the user. Finally, the software supplies a continuous record of the water surface, the mean water level, the wave amplitude, and peak frequency for all four gauges.

The mean water level is defined as,

$$\mu = \frac{1}{n}\sum_{i=0}^{n-1} x_i \quad .$$

where x is the water displacement in feet. The wave amplitude is defined as,

$$\sigma = \sqrt{\sigma^2} = \sqrt{\frac{1}{n}\sum_{i=0}^{n-1}(x_i - \mu)^2} \quad ,$$

which is the standard deviation of the wave record. The variance can also be related to the continuous energy spectrum by,

$$\sigma^2 = \int_0^{\infty} E(\omega)\, d\omega \quad ,$$

where *E(ω)* is the energy density as a function of the frequency, *ω*. This relationship associates the wave record to a frequency spectrum. The data analysis software provides the peak frequency (which is equivalent to the frequency of the wave phase), which is associated with the maximum magnitude of *E (ω)*.

In a true sinusoidal wave the average energy of a wave train is proportional to the average value of the square of the water surface (η^2), which is analogous to the variation of the wave train, σ^2. Therefore, for a sinusoidal wave, the wave amplitude, σ, can be directly related to a maximum wave height ($\sigma = .707\ H_{max}$). Wave amplitude can also be approximately related to typical measurements of wave heights by assuming the Rayleigh distribution is an appropriate description of the wave distribution. Some common wave height indicators are significant wave height (H_s), root mean square wave height (H_{rms}), and average wave height (H).

$$H_s \approx 4\ \sigma \qquad H_{rms} \approx 2.828\sigma \qquad H \approx 2.506\ \sigma$$

Since wave amplitude is directly related to wave energy and approximately related to wave height, all results are displayed in wave amplitude. In addition to wave amplitude, test results are reported in terms of attenuation of the wave amplitude relative to the wave height measured in the preliminary testing with no breakwater in front of the marina entrance. This result is referred to as *% dissipated* and is defined as,

$$\%\ dissipated = \frac{(incident\ wave\ amplitude - dissipated\ wave\ amplitude)}{incident\ wave\ amplitude} * 100 \quad .$$

2.3 Error Analysis

All experimental measurements are subject to some uncertainty. The following is a list of potential error for the project:

- Water depth was kept primarily constant but slight variations in water depth over the course of the investigation can have an effect on the results.
- The individual trials represent the average of between 4 and 10 separate runs. The number of repetitions was dictated by repeatability. For example, if the amplitudes of the first four runs differed by no more than 0.001 ft, then the trial was halted. If greater variability existed, than additional runs were made.
- To make comparisons easier and results more meaningful, an attempt was made to keep the incident wave amplitudes (i.e. the heights of waves generated by the plunger) equal for the duration of the experiment. This was not always possible due to frequency changes, water depth variation, and tank reflections.

3.0 RESULTS

3.1 Data Analysis

When evaluating the effectiveness of the different options, two wave directions were considered. Waves were generated at a 0-degree angle (incident) from the entrance and at a 45-degree angle (angled) from the channel entrance axis, which correspond to waves propagating approximately from the southeast and east accordingly. The current harbor conditions (initial), Option 1, and Option 2 were evaluated under both wave directions, while Option 3 and Option 4 were evaluated for only the angled case. Wave heights within the harbor entrance channel were relatively large in the initial tests with incident waves, but were much smaller in magnitude for the angled wave cases. With all breakwater configurations tested, diffraction around the ends of the breakwater causes the initial waves to enter the entrance channel at an angle. Therefore, much smaller wave heights were observed in the entrance channel for all configurations tested. Since there are no piers in the entrance channel, this region is not important for structural damage considerations and these measurement results are not included on the graphs. Also, when using the graphs to evaluate the different options, one should consider both the amount of dissipated wave amplitude (percent dissipated) and the actual magnitudes of the waves. In some cases the percent dissipated for a case may not have been very high, but the actual wave magnitudes were small anyway.

Figure 13 shows the results for the 3.9-second period incident wave case. It can be seen that the NW corner has the most severe amplification and both options dissipate the wave amplitudes by more than 75 percent with Option 1 being slightly more effective. The NE corner also exhibited some resonant interactions, which Option 2 dissipated by approximately 75 percent. No data was available for Option 1 due to equipment failure, but visual observations indicate that Option 1 was similarly effective. Both options dissipate the rear wave amplitudes by more than 50 percent, but wave heights from the existing configuration were not very large to begin with.

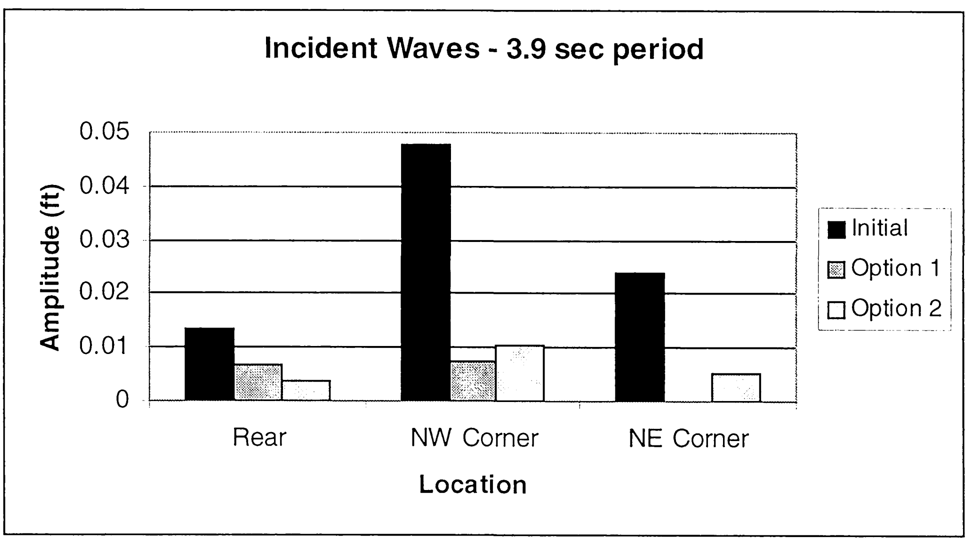

Figure 13: Wave amplitudes for incident wave with a 3.9-second period.

Figure 14 shows the results for the 4.2-second period incident wave case. All three locations exhibited some initial resonant interactions with the NW corner again being the most severe. Option 2 performs slightly better overall dissipating initial wave amplitudes by between 50 and 75 percent for all three cases and outperforming Option 1 in the two corner locations.

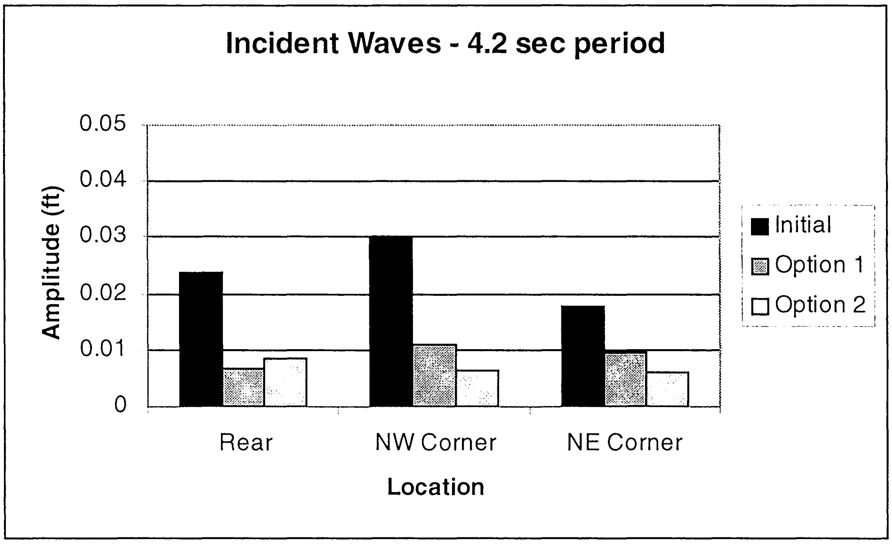

Figure 14: Wave amplitudes for incident wave with a 4.2-second period.

Figure 15 shows the results for the 4.6-second period incident wave case. Again, all three locations exhibited some initial resonant interactions, but the rear is the most severe location. Option 2 performs better than Option 1 on all three locations and dissipates initial wave amplitudes between 75 and 80 percent. Option 1 dissipated the wave amplitudes at the rear location relatively well, but dissipated less than 50 percent of wave amplitudes for both the NW and NE corners.

Figure 16 shows the results for the 3.9-second period angled wave case. The graphs now include the wave amplitudes for all four options considered. For this case, Options 2, 3, and 4 were all relatively effective at dissipating wave amplitudes. All three dissipated the wave amplitudes by more than 70 percent in the NW and NE corners with Option 2 performing worse at the rear location. Option 1 was not very effective only dissipating wave amplitudes on the order of 50 percent throughout the marina.

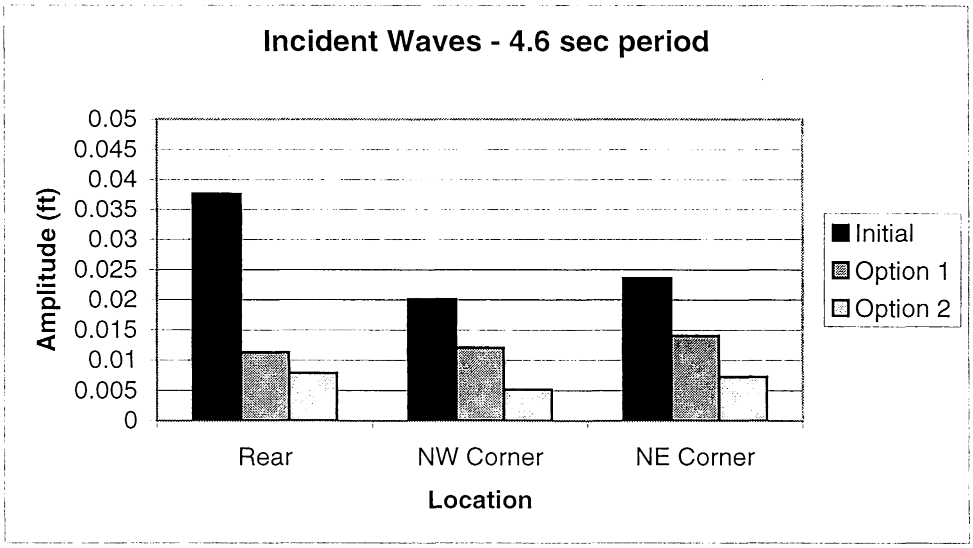

Figure 15: Wave amplitudes for incident wave with a 4.6-second period.

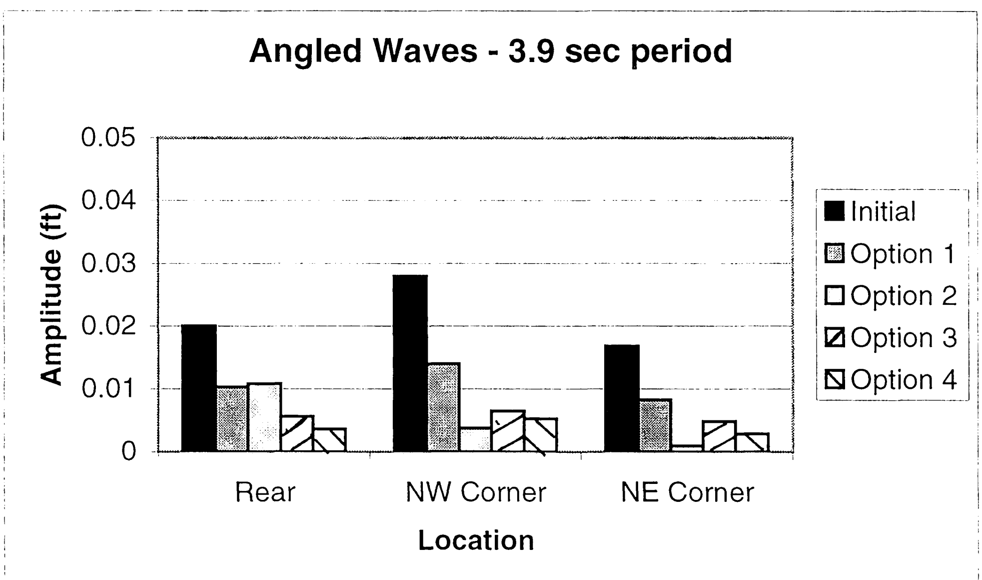

Figure 16: Wave amplitudes for an angled wave with a 3.9-second period.

Figure 17 shows the results for the 4.2-second period angled wave case. For this case, more limited resonant interactions were exhibited with the NW corner being the worst location. Options 2 and 4 both dissipated wave amplitudes on the order of 50 percent. Option 1 did not effect the wave amplitudes significantly and Option 3 only dissipated wave heights by more than 50 percent in one location, the NE corner.

Figure 18 shows the results for the 4.6-second period angled wave case. For this case only the rear and NE corner exhibited any significant initial resonant interactions. In the NE corner none of the four options dissipated the wave amplitudes by more than 50 percent. In the rear location, Options 1 and 3 dissipate wave amplitudes by about 50 percent while Options 2 and 4 dissipate wave amplitudes by around 75 percent. In the NW corner, only Option 4 showed any significant dissipation and Option 1 actually made the wave amplitudes higher, the only case and location where that occurred.

In addition to the percentage that the various options attenuated waves at the specific locations, it is important to consider the absolute magnitudes of the wave amplitudes as well. For example, considering all periods tested in the incident wave conditions, the maximum wave amplitude for either option 1 or 2 was 0.14 ft. For angled waves, the maximum wave amplitude for all options is 0.17 ft or approximately the same magnitude. Thus, although the angled waves do not appear to have as large a percentage dissipated compared to the existing conditions, the actual wave activity within the harbor appears to be generally similar for the breakwater configurations tested.

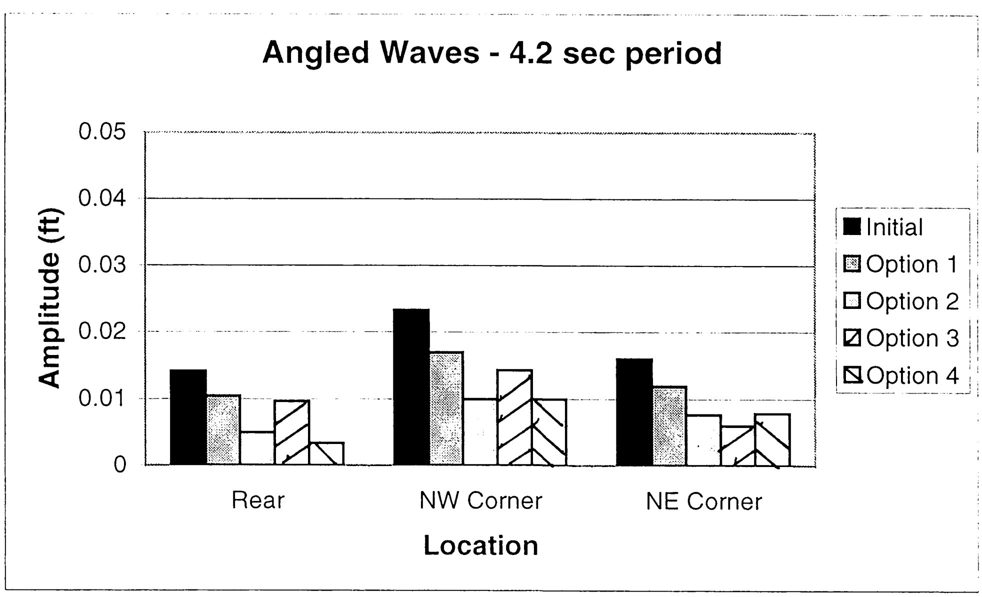

Figure 17: Wave amplitudes for an angled wave with a 4.2-second period.

3.2 Discussion

The four breakwater designs can be grouped into two general categories, attached or not attached to the south jetty. By including an opening on the south side of the jetty, some of the wave energy diffracting around the north side of the breakwater escapes, whereas the connection of the breakwater to the south jetty tends to reflect more wave energy down the channel entrance. This observation was also made in the 1994 study. It is reasonable to conclude that a better solution will be to design the breakwater so that it is detached from the channel entrance jetties.

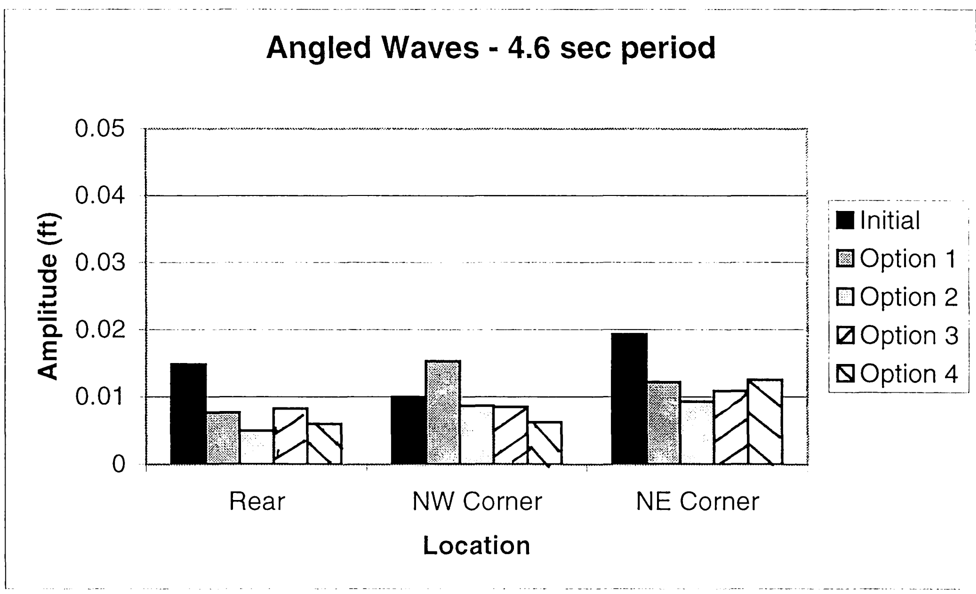

Figure 18: Wave amplitudes for an angled wave with a 4.6-second period.

It is obvious that the extension of the arms of the detached breakwater back towards the shoreline, such as was done with Option 4, will reduce the magnitudes of waves within the marina. However, there was no room within the wave basin to place the wave generator so that it could generate waves incident on the end of the Option 4 breakwater. Incident waves from the south or the east with the same height and period will give roughly the same heights within the marina for Option 2. The southern extension in Option 4 will reduce wave heights within the marina coming from the south as compared to from the east, but the magnitude of this additional reduction could not be assessed due to the space constraints in the model. It is also logical to expect that a similar extension of the north end of the breakwater would reduce wave heights for waves arriving from the east. Also, the net effect of extending either of the two ends of the breakwater will be to reduce wave height when the waves arrive from the southeast. The extension of one or both ends of the breakwater obviously adds to total construction costs as well as difficulty of access to the marina. Not enough is currently known regarding the magnitude of wave heights within the marina as they relate to structural damage to make definitive statements as to whether these breakwater extensions are required. However, with regard to structural damage, it is reasonable to suggest that wave energy (which is proportional to the square of the wave height) should be somehow related to the degree of damage. The Option 2 or 4 configurations appear to reduce the maximum wave heights to about one-fourth of those observed in the existing configuration. Therefore, the wave energy will be substantially reduced below that currently being experienced.

Also of interest to the Marina Board was the potential removal of the south gabion after the construction of the external breakwater. Lack of gauge locations in the vicinity of the south gabion prevented a detailed study of this option without changing the experimental layout, however some experiments were run for the final breakwater configuration (Option 4). Results indicate that the removal of the south gabion had little effect on the wave amplitudes at the gauge locations. However, when visual observations of the south side of the harbor were made during the experiments, there appeared to be a slight increase in wave heights. Overall, the removal of the south gabion did not have a pronounced effect on wave activity within the harbor once the breakwater is in place.

4.0 SEDIMENT TRANSPORT ISSUES

Any coastal environment will experience some degree of littoral transport along the shoreline, depending on the magnitude and prevailing directions of incident waves. The bulk of this transport occurs in the vicinity of the breaker zone and storm wave conditions are capable of transporting much more littoral material. Any shore normal structure placed from the shoreline offshore (a groin or jetty for example) will intercept sediment moving in the littoral zone and trap it on the updrift side of the structure. The resulting loss of sediment to the littoral zone will create some scour on the downdrift side of the structure as the sediment transport is re-established. Normally, one also observes some buildup of sediment on the immediate downdrift side of such a structure. This is due to the sheltering effect of the structure. Changes in mean lake elevation may tend to obscure this general pattern as declines in lake level will provide the appearance of deposition along the entire shoreline and vice versa. The existing jetties serve the function described above. We have not visited the site for several years, but from general knowledge of littoral transport along the Lake Huron shoreline, it is understood that the net transport of sediment in the vicinity of Tawas Bay marina is from the southwest to the northeast. Presumably sand has been accumulating along the south jetty although, again, recent declines in lake-levels may create the appearance of accumulation along the entire shoreline. Once the accumulated sand builds a fillet that extends the majority of the length of the south jetty, it will begin to bypass the end of the jetty towards the north. There will be some tendency for sand accumulation at the harbor entrance in this condition regardless of the presence of the proposed breakwater. However, the reduced wave environment in the lee of the breakwater will tend to exacerbate the deposition of sand at the marina entrance and more maintenance dredging will be required. There will also most likely be a tendency of a bar to form along the southern side of the opening between the jetty and the breakwater if Options 2 or 4 are selected. If this area is no longer used for marina access, there may be a tendency to leave this bar in place. However, this bar may reduce the effectiveness of the intended opening in scattering wave energy out that opening, causing an increase in wave activity within the marina. If the bar does form, it would be prudent to observe its impact on waves within the marina and to make dredging decisions accordingly.

5.0 CONCLUSIONS

In order to make a specific recommendation as to which alternative to select, more information would be required than was available for this study. Specifically, information on the wave conditions leading to the damage experienced within the marina would be necessary. It would be important to know whether a wave condition above a certain threshold is required to initiate damage. If this were the case, then it may be more important to protect against the larger waves with lower frequency of occurrence, which primarily originate from the east and southeast. On the other hand, if damage were simply an accumulation of effects from waves of all sizes and from all directions, it would be more important to protect against the more common waves from a southerly direction. Lack of this information makes it difficult to make a specific recommendation. The same information may also provide some guidance as to how much reduction in wave activity is actually required before damage is largely eliminated. Without these two issues being resolved, all that becomes possible is to quantitatively compare the various options to each other.

When considering the results of the angled wave cases, frequently Options 1 and 3 yielded similar results, as did Options 2 and 4. Overall, Options 1 and 3 tended to have higher wave heights within the harbor. This should be somewhat expected because both Options 1 and 3 included an extension of the south jetty while the breakwaters in Options 2 and 4 were completely detached. The extended portion of the jetty traps some of the wave energy that passes across the harbor entrance in Options 2 and 4. While it is easy to conclude that Options 2 and 4 generally perform better than Options 1 and 3 for the angled wave case, it is not as clear which of the former options is preferable. An important consideration is that Option 4 requires an additional volume of stone compared to Option 2 and this in turn provides more wave protection.

Option 4 was not tested for the incident wave cases, but the south extension would reduce the amount of wave energy diffracting around the end of the breakwater and into the entrance channel. The wave amplification within the marina is a function of both the amount of wave energy entering the marina as well as the internal interactions. Therefore, the improvement may not be proportional to the reduction in wave energy incident upon the channel entrance, but overall, the waves within the marina will be reduced. Also, if waves from a southerly direction are important in terms of producing damage within the marina, it is clear that Option 4 will serve to reduce the damage since the extension to the southern breakwater arm will again reduce the wave energy incident upon the marina opening. For waves from an easterly direction, Options 2 and 4 behaved in a fairly similar fashion indicating that the breakwater extension on the south side did not materially affect waves passing the north end of the breakwater. Given the above considerations, Option 4 will result in less total wave energy within the marina; again, how this translates into reduction of damage due to waves is not as clear.

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(1) **Wright, Steven J. and Lee G. Glascoe.** 1994. "Tawas Bay Marina. Scale Model Tests" Hydraulic Model Study UMCEE 94-39.

(2) **Hubertz, Jon M., David Driver, and Robin Reinhard**. 1991. "Hindcast Wave Information for the Great Lakes: Lake Michigan" WIS Report #24.

APPENDIX A

PERIOD = 3.9 Seconds

	Amplitudes (ft)			
Initial Incident	1	3	4	5
	0.041	0.014	0.049	0.024
	0.041	0.014	0.051	0.024
	0.043	0.015	0.047	0.024
	0.041	0.011	0.052	
	0.039	0.012	0.045	
	0.042	0.016	0.050	
	0.043	0.011	0.040	
Average	0.0414	0.0133	0.0477	0.0240
Std Dev	0.0013	0.0018	0.0038	0.0000

Option 2 Incident	1	3	4	5
	0.006	0.003	0.010	0.006
	0.007	0.004	0.011	0.005
	0.009	0.004	0.011	0.005
	0.008	0.003	0.010	0.005
	0.008	0.003	0.011	0.005
	0.009	0.004	0.011	0.005
	0.007	0.004	0.010	0.005
	0.007	0.004	0.011	0.005
Average	0.0076	0.0036	0.0106	0.0051
Std Dev	0.0010	0.0005	0.0005	0.0003

Option 1 Incident	1	3	4	5
	0.003	0.006	0.006 NA	
	0.004	0.007	0.009	
	0.003	0.008	0.008	
	0.004	0.007	0.008	
	0.004	0.008	0.008	
	0.004	0.006	0.007	
	0.002	0.005	0.007	
Average	0.0034	0.0067	0.0076	
Std Dev	0.0007	0.0010	0.0009	

Option 2 Angled	1	3	4	5
	0.001	0.005	0.004	0.001
	0.001	0.04	0.005	0.001
	0.001	0.005	0.004	0.001
	0.001	0.005	0.004	0.001
	0.001	0.005	0.003	0.001
	0.001	0.005	0.003	0.001
Average	0.0010	0.0108	0.0038	0.0010
Std Dev	0.0000	0.0130	0.0007	0.0000

Option 1 Angled	1	3	4	5
	0.006	0.007		0.008
	0.005	0.01	0.016	0.009
	0.006	0.009	0.013	0.008
	0.006	0.015	0.015	0.01
	0.005	0.011	0.013	0.008
	0.005	0.01	0.013	0.007
Average	0.0055	0.0103	0.0140	0.0083
Std Dev	0.0005	0.0024	0.0013	0.0009

Initial Angled	1	3	4	5
	0.01	0.017	0.029	0.018
	0.006	0.019	0.027	0.018
	0.007	0.019		0.016
	0.005	0.023		0.015
	0.004	0.022		
Average	0.0064	0.0200	0.0280	0.0168
Std Dev	0.0021	0.0022	0.0010	0.0013

Option 3 Angled	1	3	4	5
	0.002	0.008	0.01	0.007
	0.002	0.009	0.008	0.007
	0.001	0.005	0.005	0.005
	0.002	0.003	0.005	0.004
	0.002	0.004	0.005	0.003
	0.002	0.005	0.005	0.003
	0.001	0.006	0.007	0.005
	0.003	0.005	0.007	0.005
Average	0.0019	0.0056	0.0065	0.0049
Std Dev	0.0006	0.0019	0.0017	0.0015

Option 4 Angled	1	3	4	5
	0.002	0.003	0.003	0.002
	0.002	0.004	0.006	0.003
	0.002	0.003	0.004	0.002
	0.002	0.004	0.004	0.003
	0.002	0.003	0.003	0.002
	0.002	0.004	0.008	0.003
	0.002	0.004	0.007	0.004
	0.002	0.004	0.007	0.004
Average	0.0020	0.0036	0.0053	0.0029
Std Dev	0.0000	0.0005	0.0019	0.0008

PERIOD = 4.2 Seconds

Amplitudes (ft)

	1	3	4	5
Initial Incident	0.023	0.022	0.028	0.019
	0.022	0.024	0.03	0.019
	0.02	0.024	0.03	0.016
	0.019	0.025	0.031	0.018
	0.021	0.023	0.032	0.016
Average	0.0210	0.0236	0.0302	0.0176
Std Dev	0.0014	0.0010	0.0013	0.0014

Option 2 Incident	1	3	4	5
	0.002	0.008	0.006	0.007
	0.002	0.009	0.007	0.007
	0.002	0.008	0.007	0.007
	0.002	0.01	0.006	0.006
	0.002	0.009	0.008	0.006
	0.001	0.008	0.006	0.004
	0.002	0.008	0.005	0.005
Average	0.0019	0.0086	0.0064	0.0060
Std Dev	0.0003	0.0007	0.0009	0.0011

Option 1 Incident	1	3	4	5
	0.002	0.006	0.007	
	0.003	0.005	0.007	
	0.005	0.01	0.014	0.009
	0.005	0.007	0.013	0.01
	0.005	0.007	0.011	0.01
	0.007	0.006	0.011	0.01
	0.006	0.007	0.012	0.01
	0.005	0.007	0.012	0.009
Average	0.0048	0.0069	0.0109	0.0097
Std Dev	0.0015	0.0014	0.0024	0.0005

Option 2 Angled	1	3	4	5
	0.001	0.005	0.011	0.007
	0.001	0.005	0.01	0.007
	0.002	0.005	0.01	0.008
	0.001	0.005	0.009	0.008
	0.001	0.005	0.01	0.008
	0.002	0.005	0.01	0.008
Average	0.0013	0.0050	0.0100	0.0077
Std Dev	0.0005	0.0000	0.0006	0.0005

Option 1 Angled	1	3	4	5
	0.004	0.012	0.017	0.012
	0.004	0.01	0.017	0.012
	0.003	0.01	0.017	0.012
	0.003	0.01	0.017	0.012
Average	0.0035	0.0105	0.0170	0.0120
Std Dev	0.0005	0.0009	0.0000	0.0000

Initial Angled	1	3	4	5
	0.007	0.017	0.026	0.017
	0.006	0.014	0.022	0.016
	0.007	0.012	0.019	0.015
	0.006	0.015	0.026	0.017
	0.007	0.014		0.016
	0.006	0.013		0.015
Average	0.0065	0.0142	0.0233	0.0160
Std Dev	0.0005	0.0016	0.0029	0.0008

Option 3 Angled	1	3	4	5
	0.002	0.01	0.014	0.006
	0.002	0.009	0.014	0.006
	0.002	0.01	0.015	0.006
	0.002	0.009	0.014	0.006
	0.002	0.011	0.015	0.006
	0.002	0.009	0.014	0.006
Average	0.0020	0.0097	0.0143	0.0060
Std Dev	0.0000	0.0007	0.0005	0.0000

Option 4 Angled	1	3	4	5
	0.002	0.007	0.013	0.009
	0.002	0.004	0.012	0.008
	0.002	0.003	0.005	0.011
	0.002	0.002	0.008	0.007
	0.002	0.002	0.008	0.006
	0.002	0.002	0.011	0.006
	0.002	0.003	0.013	0.008
	0.002	0.004		0.008
Average	0.0020	0.0034	0.0100	0.0079
Std Dev	0.0000	0.0016	0.0028	0.0015

PERIOD = 4.6 Seconds

		Amplitudes (ft)			
		1	3	4	5
Initial Incident		0.028	0.04	0.028	0.023
		0.028	0.04	0.025	0.024
		0.031	0.036	0.026	0.024
		0.028	0.037	0.0023	0.024
		0.029	0.035	0.019	0.023
	Average	0.0288	0.0376	0.0201	0.0236
	Std Dev	0.0012	0.0021	0.0094	0.0005
Option 2 Incident		1	3	4	5
		0.002	0.01	0.005	0.008
		0.001	0.007	0.006	0.007
		0.002	0.007	0.006	0.007
		0.002	0.008	0.007	0.007
		0.002	0.008	0.005	0.007
		0.002	0.007	0.005	0.008
		0.001	0.007	0.006	0.006
		0.0020	0.0070	0.0050	0.0070
		0.0020	0.0090	0.0040	0.0070
		0.003	0.008	0.003	0.008
		0.003	0.009	0.005	0.007
		0.002	0.008	0.005	0.008
	average	0.0020	0.0079	0.0052	0.0073
	std dev	0.0006	0.0010	0.0010	0.0006
Option 1 Incident		1	3	4	5
		0.004	0.008	0.012	0.014
		0.004	0.009	0.011	0.014
		0.003	0.009	0.011	0.014
		0.003	0.015	0.013	0.015
		0.003	0.012	0.012	0.014
		0.003	0.013	0.014	0.014
		0.003	0.013	0.012	0.014
	average	0.0033	0.0113	0.0121	0.0141
	std dev	0.0005	0.0024	0.0010	0.0003
Option 2 Angled		1	3	4	5
		0.003	0.005	0.009	0.009
		0.002	0.005	0.009	0.009
		0.002	0.005	0.008	0.01
		0.003	0.005	0.009	0.009
		0.002	0.005	0.009	0.009
		0.002	0.005	0.008	0.01
	Average	0.0023	0.0050	0.0087	0.0093
	Std Dev	0.0005	0.0000	0.0005	0.0005
Option 1 Angled		1	3	4	5
		0.002	0.007	0.018	0.012
		0.002	0.008	0.016	0.012
		0.002	0.008	0.014	0.012
		0.001	0.008	0.014	0.013
		0.002	0.008	0.016	0.012
		0.002	0.007	0.014	0.012
	Average	0.0018	0.0077	0.0153	0.0122
	Std Dev	0.0004	0.0005	0.0015	0.0004
Initial Angled		1	3	4	5
		0.003	0.013	0.009	0.019
		0.004	0.016	0.011	0.02
		0.004	0.015	0.01	0.019
			0.015	0.01	0.019
	Average	0.0037	0.0148	0.0100	0.0193
	Std Dev	0.0005	0.0011	0.0007	0.0004
Option 3 Angled		1	3	4	5
		0.001	0.008	0.01	0.01
		0.002	0.01	0.009	0.011
		0.001	0.009	0.007	0.011
		0.004	0.007	0.007	0.011
		0.002	0.007	0.008	0.011
		0.002	0.009	0.009	0.011
		0.001	0.008	0.009	0.011
		0.001	0.008	0.009	0.011
	Average	0.0018	0.0083	0.0085	0.0109
	Std Dev	0.0010	0.0010	0.0010	0.0003
Option 4 Angled		1	3	4	5
		0.002	0.005	0.006	0.015
		0.002	0.006	0.007	0.014
		0.002	0.004	0.007	0.013
		0.002	0.006	0.006	0.012
		0.002	0.003	0.007	0.012
		0.002	0.009	0.005	0.011
		0.002	0.006	0.006	0.012
		0.002	0.009	0.006	0.011
	Average	0.0020	0.0060	0.0063	0.0125
	Std Dev	0.0000	0.0020	0.0007	0.0013

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AIIM SCANNER TEST CHART #2

Spectra

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Times Roman

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Century Schoolbook Bold

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News Gothic Bold Reversed

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Bodoni Italic

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Greek and Math Symbols

4 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⊁⊀⋗⋖≡

6 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⊁⊀⋗⋖≡

8 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⊁⊀⋗⋖≡

10 PT ΑΒΓΔΕΞΘΗΙΚΛΜΝΟΠΦΡΣΤΥΩΧΨΖαβγδεξθηικλμνοπφρστυωχψζ≧∓",./≦±=≠°><⊁⊀⋗⋖≡

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Black

Isolated Characters

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MESH	HALFTONE WEDGES
65	
85	
100	
110	
133	
150	

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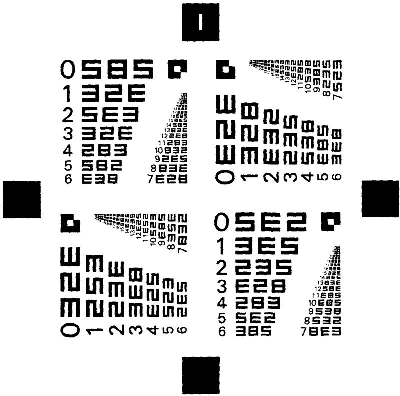
RIT ALPHANUMERIC RESOLUTION TEST OBJECT, RT-1-71



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