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INVESTIGATION OF LONG WAVES AND THEIR EFFECTS  
ON THE COASTAL AND HARBOR ENVIRONMENT  
OF THE LOWER CHESAPEAKE BAY (U)

VOLUME I

CONTRACT NO. DA-49-146-XZ-151



APRIL, 1963

NATIONAL ENGINEERING SCIENCE CO.

PREPARED FOR

HEADQUARTERS, DEFENSE ATOMIC SUPPORT AGENCY

WASHINGTON, D.C.

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INVESTIGATION OF LONG WAVES AND THEIR EFFECTS  
ON THE COASTAL AND HARBOR ENVIRONMENT  
OF THE LOWER CHESAPEAKE BAY (U)

VOLUME I

Prepared by NESCO Staff

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PREFACE

This technical report covers the work performed by NESCO during the period May 1, 1962 to March 31, 1963, pertaining to the determination of damage in the lower Chesapeake Bay due to long waves generated by underwater nuclear explosion.

It is emphasized that this work has required the development of new theories relevant to the hydrodynamics of long waves. These theories may find many other applications than those which have been carried out for the particular problem under study. On the other hand, because of the timing of the project, some of these new theoretical tools have not been fully exploited as they could have been for obtaining more accurate and reliable quantitative results in the lower Chesapeake Bay areas.

Despite these new theoretical developments, and due to the complexity of the problem under study, a number of gaps remain. Hence some practical results rely more on engineering judgment and approximate methods than on scientific calculations. New theoretical or experimental research will certainly permit a better quantitative investigation. However, it is believed that the general conclusions presented in this report will not be greatly modified by increasing the scope of this study; but they can be given with more accuracy and details.

The work on this project was carried out in the NESCO offices at Pasadena, California, Washington, D. C., and Houston, Texas.

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The work in the Pasadena office was under the supervision of Dr. Basil W. Wilson, with added contributions from Dr. James A. Hendrickson and Miss Lois Webb. The work in the Houston office was under the supervision of Dr. John C. Freeman, Jr., with added contributions from Dr. Larry Armijo and Miss Mary Ann Noser. The work in the Washington office was under the supervision of Dr. Bernard Le Méhauté, who was also project principle investigator and coordinator, with added contributions from Dr. J. Ian Collins, Mr. Raminder S. Grewal and Mr. Carl Fast.

Dr. Le Méhauté is credited with the content of Volume I of the report dealing with the engineering aspects of the study. Various members of the NESCO team have made original contributions to the advancement of hydrodynamic theory. Their scientific contributions are reflected in the various appendices of this report presented in Volume II:

- Appendix I: Long Waves Generated by Nuclear Explosions;  
Type of Waves and Initial Decay in Deep Water,  
by Dr. Basil W. Wilson.
- Appendix II: Theoretical Considerations and Computations for  
Water Waves Produced by Explosion, by Dr.  
Larry Armijo and Miss Mary Ann Noser.
- Appendix III: Surface Waves Generated by a Disturbance on  
Sea Bed in Constant Depth Open Sea, by Dr.  
James A. Hendrickson.

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- Appendix IV: The Principle of Superposition and the Theory of Cauchy-Poisson by Dr. Bernard Le Méhauté.
- Appendix V: The Shoaling, Damping, Breaking and Run-up of Long Waves Over the Continental Shelf; On Saturated and Nonsaturated Breakers, by Dr. Bernard Le Méhauté.
- Appendix VI: The Wave Run-up by the Method of Characteristics, by Dr. Bernard Le Méhauté. Dr. John Freeman was scientific advisor and Mr. Raminder Grewal did the numerical computations.
- Appendix VII: Two-Dimensional Nonlinear Wave Motion in an Estuary, by Drs. John C. Freeman, Jr., and Larry Armijo, and Miss Mary Ann Noser.

The undersigned would like to express his appreciation for the assistance rendered to the project by Dr. Lewis McCammon, Associate Director of Engineering at the Pasadena office of NESCO, and to Mr. Zemgals for supervising the drafting of figures.

Acknowledgement is given to Mr. Kenneth Kaplan of United Research Service, Inc. of Burlingame, California, who furnished the input data for the project and acted as a consultant to NESCO.

Charles L. Bretschneider  
Director, Washington Office

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ABSTRACT

This report deals with the effect of tsunami-type gravity waves caused by huge underwater disturbance, on the shoreline environment.

In particular, the cases of three different power disturbances off the United States Atlantic Coast and their effects in the lower Chesapeake Bay are analyzed. It is shown that because of the natural protective action of the continental shelf, a maximum possible amount of damage is reached for a given power. If a disturbance is due to a larger power, the difference of energy is dissipated over the continental shelf by spilling breakers.

The flooding areas, dynamic destruction and harbor resonance due to this optimum power disturbance are calculated and/or evaluated.

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PART A

PURPOSE OF THE STUDY  
AND  
GENERAL CONCLUSIONS

by

B. Le Méhauté

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CHAPTER I  
PURPOSE OF THE STUDY

From time to time disturbances at sea produce long waves which cause considerable damage to populated areas. These waves, so-called tsunami waves, are most often due to a disturbance of seismic origin. The time has now arrived when such destructive waves can also be generated by explosions of large nuclear devices. Therefore, in view of the range of wave heights to be expected from such explosions, impetus has been added to the immediate need to study this problem.

The first purpose of this study is to determine whether or not there is a probability of damage along coastal areas due to large yield nuclear explosions at sea. Then, if this probability does exist (and it has been found that it does exist), the study would include an estimation of the damage to coastal and harbor facilities due to these long waves in the lower Chesapeake Bay, and possible engineering solutions.

The size of the assumed initial disturbance at the free surface and the wave trains due to an explosion for a point twenty miles from the disturbance were provided to NESCO by United Research Services\* for three cases corresponding to three different bomb yields. For purposes of this report the smallest yield is defined as Case I

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\* Report No. URS-B192-2

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and the largest as Case III (see Fig. 1). The origin of the disturbance is assumed to be some 370 nautical miles from the Atlantic coast (Fig. 2). The evolution of these wave trains is then calculated from 20 miles from the explosion to the extreme end of the estuaries and harbor basins until their effects become negligible.

A practical analysis of the travel of the wave trains involves a succession of various hydrodynamic problems to be solved by completely differing methods.

The first problem consists of the calculation of the evolution of the wave train in deep water up to the toe of the continental slope. Then shoaling and reflection effects due to the continental slope have to be evaluated.

Waves travelling over a continental shelf are subjected to shoaling, refraction and relative damping by bottom friction. Ultimately they break up into a succession of spilling breakers, then bores, and finally run-up over the coastal dunes causing flooding and destruction along great stretches of the Atlantic coast. Also, when the waves penetrate river estuaries and bays, they diffract and give rise to multiple reflections causing severe local agitation combined with wave run-up and flooding. Finally, upon reaching harbor basins they are the cause of great agitation induced by resonance with the natural oscillatory characteristics of these basins.

Qualitative details of all these different aspects are described

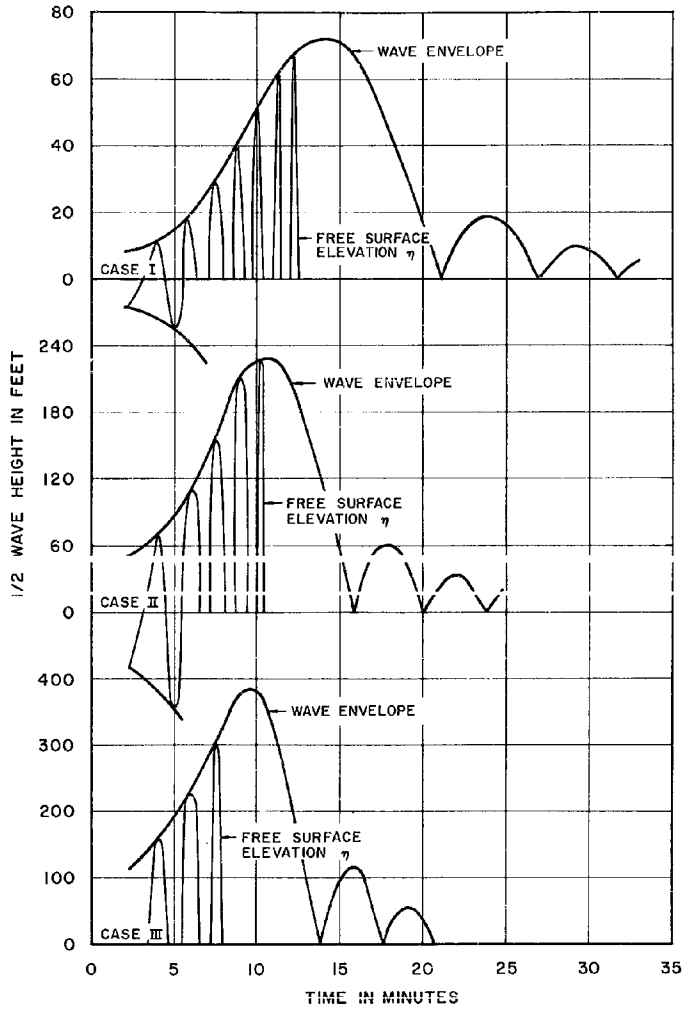


FIGURE 1  
WAVE TRAINS AT 20 MILES FROM THE EXPLOSION



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in Part B. Each of them has been subjected to theoretical studies and applications. Great use has been made of existing theoretical and experimental studies. However, it was found almost impossible to provide a satisfactory answer to these complex problems without an extension of existing knowledge. Hence, new research on these problems became a major part of this study.

Some of these new theories can be further refined and improved. Others which will be directly related to the problem under study have not been investigated in this report because of the extreme complexity of the phenomena. These gaps have been tentatively filled by using approximate methods of calculation, common sense and engineering judgment. For example, a problem such as the penetration and propagation of a three-dimensional bore in deep estuaries needs extensive theoretical research and development which is beyond the scope of this study and report.

It is pointed out that the general conclusions of this report would certainly not need to be changed as a result of further investigations, but these conclusions could be refined, improved and presented with more accuracy subsequent to further theoretical and experimental investigations.

It is recalled that at the beginning of the study it was decided that this work would be accomplished in two phases: Phase I would define the problem, determine whether a problem actually existed, and give, by use of available techniques and formulas, a rough quantitative estimate of the amount of damage probable in the

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Chesapeake Bay area.

Phase II of the study was designed for improving the state of the art by constructing new theories. These new theories have been applied to improve the accuracy of the final results presented in Parts A and B. Some of the new theories have been formulated only. Their application would require the use of high speed digital computers with programming based on the equations which have been established.

This report is composed of two main parts and seven appendices. Part A is essentially nontechnical and explains the purpose of the study and presents broad conclusions for the lower Chesapeake Bay. Part B is more technical and gives all the essential aspects of the problem, a brief presentation of the methods of calculation, and their application to the Norfolk area. The methods of calculation are justified in indicated references or in the appendices which follow. Some considerations on the accuracy of these different methods are also developed and suggestions given for further investigations. The various appendices are essentially of a theoretical nature. Most of them are original contributions to the science of hydrodynamics.

Appendix I is a synthesis on a critical literature survey including theoretical and experimental information on wave motion due to a local disturbance. The variation of wave heights and wave periods with distance is also investigated quantitatively for Case III.

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Appendix II reviews the now classical theory of Kranzer and Keller for waves caused by explosions and its application to the problem under study. This theory has been applied with the input data at 20 miles from the explosion as provided to NESCO by United Research Services for the three cases.

Appendix III presents the establishment of the surface waves generated by a disturbance on the sea bed in constant water depth. This disturbance is defined as a circular upthrust varying exponentially with time with an extremely rapid decay time. It is an attempt at refinement of the Kranzer and Keller theory for waves generated by underwater explosions. Similar results for the wave height decay with distance are presented.

Appendix IV is also a new approach for calculating the wave motion generated by a surface disturbance of finite magnitude by application of the principle of superposition to the classical Cauchy-Poisson theory for an infinitely small disturbance.

Appendix V deals with the propagation of the wave on the continental shelf. The shoaling and bottom friction effects are considered. Then a new theory on "saturated" and "nonsaturated" breakers which follow the breaking index curve is established. A literature survey on the wave run-up of long waves on an open coast has been summarized.

Appendix VI presents a new method of computation of the wave run-up based on the method of characteristics. Various related phenomena have been investigated where required for carrying out successive steps in the calculations. In particular the problem of

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the wave run-up on a dry bed has been analyzed. This method has been applied to one particular case. The systematic application of this method for various bottom slopes would require the use of a high speed digital computer.

Appendix VII presents a new method of numerical computation for the penetration of nonlinear two-dimensional waves into bays and estuaries. This method would also require use of a computer for its practical application to the problem under study.

The theories and methods developed in these various appendices, and in particular in Appendices VI and VII, may have many other applications than this immediate problem; for example: tsunami run-up, waves due to the breaking of a dam and tidal motion, among others.

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where the bottom slope is 0.01 near the shoreline. This corresponds roughly to a twenty-foot high breaker. It is also the maximum possible average wave height for the wave penetrating within Chesapeake Bay. This maximum effect will occur from Cape Cod to Cape Hatteras. However, the larger yield (Case III) located further from the coast will cause more damage because its effects will be spread on a longer shoreline, which could even extend from Canada to Florida.

Considering the present assumed location for the initial disturbance, the following effects can be feared.

Case I: The amount of damage due to Case I is very limited and is similar to the damage due to a very strong storm. The open coast will be subjected to a strong attack and the protecting dunes, which are over 15 feet high above the sea level, will be submerged. Inside Chesapeake Bay the damage will be limited to unprotected areas such as Old Point Comfort (see enclosed map: Fig. 3).

However, considering the range of wave periods generated by the disturbance and the range of natural oscillation of the harbor basins, resonant motion will certainly occur. This motion will have a large amplitude and could cause considerable damage to moored ships. Hence this problem of resonance is the most important factor in the case of a disturbance of relatively small yield as in Case I. This problem can be partly solved in some locations. A breakwater between Tanner Point and Craney Island can be

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considered as a possible solution for the harbor basins in the Elizabeth River. An increase in the length and height of the causeway between Old Point Comfort and Willoughby Spit will have a greater effect but would probably be a more expensive solution. However, this solution may be worth consideration since both sets of harbor basins at Newport News and Norfolk would be protected. It is evident that the choice of such a solution would require further studies than those presented in this report. Scale model studies would be highly desirable.

Another solution might be more economical. It would consist of adapting the moorings of ships in such a way that the period of oscillation of the mooring system does not match the natural period of oscillation of the harbor basin. This solution will also require a special study, despite the fact that considerable work has been done on this problem in the last decade.

Finally, since the waves would take about one hour to travel from the entrance of Chesapeake Bay to the harbor basin located in the Norfolk area, an alarm system might be set up in such a manner that certain precautionary measures could be taken.

Cases II and III: It has already been pointed out that the damage due to these two different yields located at the same distance from the coast will be very similar in the Norfolk area. Hence the results can be presented together. This result can be considered as the maximum possible damage.

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The enclosed map (Fig. 3) presents an overall picture of the areas subject to destruction, flooding, and harbor resonance. First it can be anticipated that the open coast will be subjected to such a strong attack that the protecting dunes will be submerged and destroyed. The shoreline installations will be destroyed by the dynamic effects of the successive bores generated by the waves. The flooding area will extend deeply within the coastal area because the elevation is generally low. Cape Charles will be half covered by water; but the most important parts (roads) which are above the 20-foot contours will not be submerged.

The waves penetrating within Chesapeake Bay will first cause a slight rise in the mid water level of about one foot. But this rise is too small to be a real cause of concern. (It is even smaller than the rise due to storm surge which sometimes causes flooding in downtown Norfolk.) Hence the real danger would result from the dynamic effects of long waves. These effects will be particularly important from Cape Henry to Willoughby Spit. This is the first area attacked by the waves. In this area waves will also pass over the dunes and through Lynnhaven Inlet and Little Creek. The harbor installations near Little Creek will be submerged. The installations near Ocean View will not only be submerged but destroyed because of the dynamic effects of the waves.

On the other side of Hampton Roads, i. e. north and west of Old Point Comfort, the dynamic effects of the waves arriving directly from the entrance of Chesapeake Bay are superimposed on

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the waves reflected along the coast at the south of Chesapeake Bay. Hence a great amount of destruction can be anticipated along Buckroe Beach and between Newport News and Hampton.

Flooding can be anticipated in local places in the downtown area of Newport News and Norfolk because of resonance of harbor basins.

Finally the resonance problem of the harbor basin is similar to the problem previously described for Case I. It will be greater in amplitude because the incident wave height itself is greater. The destructive effect of the waves decreases very quickly north from a line from Back River to Cape Charles. They can cause some flooding in the lower part of the Bay.

The bridge-tunnel between Chesapeake Beach and Fisherman's Island will not change the overall results presented in this report but this bridge-tunnel can be undermined by erosion and subjected to destruction because of wave forces.

Now the military installations in the Norfolk area will be considered more specifically. Most of the military installations are subjected to wave effect because they are located in the lowest areas. The probably maximum damage can be summed up as follows:

1. Langley Air Force Base

Local flooding around shoreline. No dynamic destruction.

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2. Fort Monroe  
Dynamic destruction along shoreline accompanied by extensive flooding throughout.
3. U. S. Military Reservation at Newport News  
Some flooding near the waterline.
4. U. S. Naval Reservation at Willoughby  
Some dynamic damage at Sewell's Point. Extensive flooding. Seiche motion of high amplitude in harbor basins.
5. U. S. Naval Reservation at Craney Island  
Extensive flooding.
6. Naval Hospital at Hospital Point  
Little or no damage.
7. Norfolk Naval Shipyard  
Resonance of harbor basins causing seiche motion and possible local flooding due to the resonance.
8. U. S. Military Reservation near Paradise Creek  
Resonance of harbor basins causing seiche motion and possible local flooding due to the resonance.
9. U. S. Naval Reservation at Little Creek  
Dynamic destruction along the shoreline. Extensive

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2. Fort Monroe  
Dynamic destruction along shoreline accompanied by extensive flooding throughout.
3. U. S. Military Reservation at Newport News  
Some flooding near the waterline.
4. U. S. Naval Reservation at Willoughby  
Some dynamic damage at Sewell's Point. Extensive flooding. Seiche motion of high amplitude in harbor basins.
5. U. S. Naval Reservation at Craney Island  
Extensive flooding.
6. Naval Hospital at Hospital Point  
Little or no damage.
7. Norfolk Naval Shipyard  
Resonance of harbor basins causing seiche motion and possible local flooding due to the resonance.
8. U. S. Military Reservation near Paradise Creek  
Resonance of harbor basins causing seiche motion and possible local flooding due to the resonance
9. U. S. Naval Reservation at Little Creek  
Dynamic destruction along the shoreline. Extensive

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flooding around Little Creek, resonance of harbor.  
basin, the navigation channel filled by sand.

10. U. S. Military Reservation at Camp Pendleton  
Little or no local flooding. No damage.
11. U. S. Naval Reservation South of Virginia Beach  
Complete erosion of the dunes. Complete dynamic  
destruction along the shoreline. Complete flooding.
12. U. S. Military Reservation at Fort Story  
Dynamic destruction along the shoreline. Partial  
flooding.
13. Cape Charles Air Force Base  
Some dynamic destruction and extensive flooding.

Any possible solution to the problem on the open coast would be unrealistic. Engineering solutions could only be considered for local protection. Some solutions have already been outlined in Case I. The solutions for protection against conditions arising from Cases II and III will be similar and also would require further investigations, involving scale model studies.

Finally, it may be pointed out that the results presented here have been obtained first by application of existing wave theories. The difficulty of the problem is such that a number of new theories were necessary in order to improve the state of the art. Despite these new theories, many points rely on engineering judgment rather

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than scientific calculation. The results obtained have indicated the extent of the problem. They can be subjected to slight change after further analysis. There is no doubt that more theoretical and experimental studies in the field or in a laboratory would give considerable complementary information of great value.

The conclusions can be summarized in a few words. The destructive action of long waves generated by a nuclear explosion off the Atlantic coast would be less serious than may be expected, a priori, because of the natural protection afforded by the continental shelf. On the other hand, the possible destruction would be much more extensive than that due to the strongest observed storm. Hence the destruction may be sufficient to be a cause of concern.

Finally, it is pointed out that the protective action of the continental shelf arises as a consequence of its length and slope. Hence the conclusions presented in this report cannot be extrapolated to the Pacific coast because there the continental shelf is much shorter and steeper.

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PART B

DESCRIPTION OF PHYSICAL PHENOMENA, METHODS  
OF COMPUTATION, RESULTS AND RECOMMENDATIONS  
FOR FURTHER INVESTIGATIONS

By

B. Le Méhauté

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### CHAPTER III

#### A DESCRIPTION OF THE PHYSICAL PHENOMENA

The detailed description of the wave motion at the location of the disturbance is not within the scope of this study. However, for a better understanding of the subsequent phenomena which concern this report, a brief description is presented.

Consider the case of a sudden disturbance, such as that due to the explosion of a nuclear device below the sea surface in deep water. From a hydrodynamic point of view this disturbance could be considered as due to successive expansions and contractions of a huge cavity. A hot gas and steam bubble is quickly created which initiates shock waves. Upon arrival at the free surface these shock waves cause the formation of a spray dome which, when falling, causes a "base surge." Then the hot gas and steam bubble bursts through the dome causing considerable water movement.

Both theory and experiment yield methods for the determination of the wave motion far from the location of the disturbance. All the theories are based on a relatively simple mathematical model assuming the depth to be either constant or infinite. This assumption of infinite depth is reasonably justified for the wave somewhere near the disturbance.

In spite of the complexity of the real motion and great simplification of the mathematical models, the essential features of the waves given by theory are relatively well verified by tests.

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These waves are characterized by relatively long periods, usually ranging from 30 seconds to a few minutes. They are slightly shorter but much steeper than tsunami waves due to seismic disturbances. However, like tsunami waves, the length between crests is so great that their steepness in deep water is often insignificant and they are seldom visible some distance from their source. The waves are circular and travel radially from the origin in a manner similar to waves caused by a stone dropped in a pond. This results in the wave energy being spread over a circle of increasing radius, and the wave height decreases rapidly.

Such waves are not periodic in the mathematical sense: their "pseudo period" varies with time and distance. Due to the increase in wave period with distance, the "wave length" (or more exactly the distance between wave crests) and the group velocity (or velocity of propagation of energy) also increases with distance. In other words, the "wave lengths" are "stretched" radially. This stretching effect is particularly important in deep water. Stretching and spreading effects cause the average wave height to decrease very quickly with the distance from the origin of the disturbance. The stretching effect becomes less important in shallow water and becomes negligible in very shallow water, such as on the continental shelf. In very shallow water the wave decay is due only to the "spreading" effect and the wave height decreases in a manner similar to an ordinary periodic gravity wave in constant water depth.

The wave height and wave period also vary from one wave to

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another. The wave motion appears as a succession of wave train presenting a "beating effect." The number of waves in a wave train also increases with time and distance. The number of maximum waves also increases with distance from the origin. In practice, the wave which is the highest in the wave train at any given instant retains the same period.

The first wave train is the most important because it contains the most energy and the highest waves. The general tendency at a given point is that the wave period, height of the envelope of wave trains, number of waves within a wave train, and length of the wave train decrease with time. Also, the wave height at a given point and a given time depends upon the yield of the disturbance, while the wave period depends upon time and distance only.

Because the sea bottom is not horizontal, these wave trains are theoretically influenced by a "shoaling effect." The shoaling effect, due to changes in depth, is added to this spreading and stretching effect on a bottom of varying depth. The decrease in depth causes an increase in wave height which partly compensates for the wave decay due to spreading and stretching and becomes particularly important upon arrival on the continental slope. It is interesting to note that a shoaling effect also exists, in a sense, for a constant depth. Since the "wave length" increases with distance from the origin, a wave motion near the source in deep water travels on a continually smaller "relative depth" and will even tend to travel in relatively shallow water at an infinite

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slope. However, in practice the values found from both assumptions are smaller than double the deep water wave height for this particular problem. The truth lies between the two extremes of reflection on a vertical wall and total energy transmission. No reliable theories are available to give the exact feature of the change in wave height. However, an attempt is made to give a reasonable answer to this problem. It has been found that the two extremes corresponding to either of the two assumptions above give results close enough to permit a very good estimation of the effect of the continental slope.

Over the continental shelf the waves can first be considered as periodic waves. The wave height tends to increase due to shoaling. At the same time, the bottom friction becomes more and more important and causes damping, partially compensating for the increase of wave height due to shoaling. For the highest waves the bottom friction is so important that the wave height can even decrease despite the shoaling effect. At the same time that the wave height tends to increase, the "wave length" or distance between two successive wave crests is decreasing. As a consequence, the wave becomes steeper and steeper.

These so-called pseudo-periodic waves, which have an almost sinusoidal profile in deep water, then appear as a succession of solitary waves which are characterized by high peaks of water between very long, flat troughs. These solitary waves are not oscillatory waves, for which the flow is periodic and the total discharge

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over the period of the wave is theoretically zero. They are actually translatory waves, characterized by a transportation of water in the wave direction with no return flow. Actually the transport of water with the crest of a solitary wave causes a rise in the mean water level near the shoreline resulting in a slow return flow in the succeeding long, flat trough. Despite the return flow the wave profile is actually very close to that of a solitary wave when the ratio of the depth and the distance between crests becomes small and the bottom slope is very gentle, conditions encountered on the continental shelf of the Atlantic coast.

The wave height in a solitary wave also increases when the depth decreases but theoretically this change is much greater than for a periodic wave. The increase in wave height depends upon the slope and the relative depth. However, a reliable mathematical law to predict the increase accurately is unknown. The theory based on the conservation of energy in a solitary wave is not verified by experiment. Long waves are partially reflected on a very gentle slope. At present the best method available, as a preliminary approximation, is to extrapolate available experimental data. However, it must be borne in mind that such extrapolation is always subject to doubt or at least to qualitative interpretation.

As the solitary wave becomes steeper, its celerity becomes strongly influenced by its amplitude. As the depth decreases the particle velocity at the crest tends to become greater than the wave

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energy is dissipated by a spilling breaker before reaching the shoreline.

The greater the wave height in deep water, the sooner the wave will break. Hence, the amount of damage caused by this wave is almost independent of the initial height of the wave in deep water once this wave height exceeds a critical value. This critical value of wave height is characterized by a slope steep enough for the breaker to become saturated or fully developed bore. Such a slope exists only near the shoreline in relatively very shallow water. (It can also exist on the continental slope but either the water is too deep to cause any type of breaking or, even if breaking occurs for huge waves over the continental slope, the saturated breaker again becomes a nonsaturated breaker upon reaching the gentle slope of the continental shelf. Nonsaturated breakers and even nonbreaking waves can again reform from this breaker.) Hence the continental shelf off the Atlantic coast acts as a natural protection. This natural protection would be much less efficient on the Pacific coast because of the steeper slope of the continental shelf.

The maximum damage can occur either from a breaker initially spilling all along the continental shelf or from a wave of initially smaller amplitude in deep water which reaches its breaking condition at a critical slope where the breakers are saturated. Therefore, despite the natural protection of the continental shelf, the slope near the shoreline becomes steep and the breaker is saturated at a

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depth where the wave is high enough (say 20 feet) to be a cause of great concern, in which case the wave is transformed into a translatory wave such as a tidal bore.

No simple relationship exists to relate the height of a breaking solitary wave to the height at the front of the tidal bore issued from this wave. However, they are of the same order of magnitude near the point where the breaker becomes saturated. The tidal bore moves at a speed which depends upon the height of the wave front and the depth.

As in any shock wave, a great amount of energy will be dissipated near the front of the bore by sudden changes in depth. The bottom friction becomes relatively less important than the internal friction within the bore. The front of the tidal bore could be considered as a moving hydraulic jump with the velocity of the upstream flow equal to zero. The relative amount of wave energy dissipated increases as the depth decreases. The bottom friction becomes more and more important as the wave nears the shoreline.

From a hydrodynamic point of view the bore considered as a shock wave does not exist any more when climbing on a dry bed. It is a "rarefaction wave" or edge of water which moves upwards. But the bottom friction forces tend to be infinite at the extreme edge of the wave. Hence, in practice this edge is cut short and has the physical appearance of a bore.

However, the rise of the wave on a dry bed cannot be treated

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happens to be near the "period" of the incident wave, resonance would be possible. In fact, considering the range of wave periods and the periods of the wave trains, resonance of the continental shelf at the period of the wave trains is most probable. This resonance is due to the mass transportation accompanying long waves which has a component at the period of the wave trains. It is a phenomenon similar to what is called "surf beat" phenomenon for wind waves.

Inasmuch as the most crowded areas in question are located around an estuary (particularly in the Norfolk area), an investigation of the penetration of long waves into the estuary must also be considered. When a relatively short periodic wave reaches a breach (say the entrance to Chesapeake Bay or Delaware Bay), the wave diffracts. Diffraction theories are based on periodic short waves in water of constant depth. These are linear theories. Since in this case: (1) consideration is being given to long waves in shallow water and even breakers (which are essentially nonlinear phenomena), (2) the motion is not strictly periodic, and (3) the long waves are partly reflected by the limits of the estuary, the diffraction theories give only a qualitative idea of the directional effect. However, it must be expected that the wave motion does have a directional effect: the wave agitation remains high in exposed areas while it decreases very rapidly in "shaded" areas. (The work "shaded" is used here with the same meaning as applied in the theory of light, i. e. the shaded areas are those which are protected

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from incident waves.) According to the linear theory, along the separation line the wave amplitude has half of the value at the entrance. In the shaded areas the breaker can generate a non-breaking wave of much smaller wave height. These new waves will break when they arrive on the shoreline generating a bore. This bore, while being much smaller than on the open coast and in unprotected areas, would still be large enough in many places to cause considerable flooding and damage on a flat shoreline.

Finally, the harbor basins could be subjected to resonance effects. Available theories permit the calculation of the periods of resonance, and even amplification factors, with relatively good accuracy. Most harbor basins are similar to an open acoustic pipe. They can be subjected to a resonance phenomenon when the period of the waves at the entrance matches one of the natural periods of free oscillation of the basin. This occurs when the length of the open basin is close to a quarter of the wave length (or  $3/4$ ,  $5/4$ , . . . for harmonics). Considering the usual size and depth of the harbor basins, it has been found that most of them will be strongly excited by the range of periods of the expected incident waves.

Since the incident waves are not periodic in the mathematical sense, the problem of resonance in a harbor must be considered from a spectrum point of view. The incident waves can be considered as defined by a wave spectrum. The time of response of an open basin to this incident spectrum is very short (three or four wave

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periods) and at resonance the amplitude is great (at least four times the incident wave height). This can be explained from the fact that a great amount of wave energy penetrates through the entrance, establishing the resonance condition very quickly.

Hence, even in the areas protected from destruction by the dynamics of the waves and from gradual flooding, this problem must also be a cause of concern. This is due to the fact that moored ships are then subjected to a seiche motion of large amplitude. They may act in a similar manner to a body between two springs. In these cases also the period of resonance of the basin is close to the period of resonance of large boats moored along the quays. Consequently the moorings can be broken and the ships thrown bodily against the quays or each other.

Now that the main physical phenomena have been qualitatively described, the methods of calculation will be briefly summarized in the following chapter. Their justification will be found in various appendices or listed references.

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convective inertia term, which is usually neglected in theories, becomes important for steep waves. Hence, it can be expected that the wave decay near the origin is larger than the decay given by theories, namely  $H \propto R^{-1}$  for the maximum of the wave envelope ( $H$  is the wave height and  $R$  the distance from the origin of the disturbance). (See Appendices I, II and III.)

The shoaling effect due to the change of depth is added to the shoaling effect due to the increase of wave length.

It may be seen then that as the waves spread out and their periods increase with distance, the leading waves which are the longest begin to feel the bottom first and could be subject to shoaling effects before even reaching the continental slope. This shoaling effect due to a change of relative depth can be estimated at least in a first approximation.

Before studying the shoaling effect for the first waves before the continental slope, some considerations must be given to the variation of the "wave period" with distance. It has been seen that this kind of motion is nonperiodical in the mathematical sense. However, a "pseudo" wave period can be defined. This wave period is defined as the period of a periodic wave having the same distance between wave crests. Then it is found that this wave period varies with distance. The law of variation is actually very complex (see Appendices I and II). It varies both from the first wave to the last wave and with distance. As a first approximation the wave period for a given wave may be considered as increasing with the cube root

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of the distance from the origin in deep water ( $R^{1/3}$ ). However this law does not hold true for the leading waves.

It must be noted that a relatively large error is permissible in the wave period because a large variation of wave period involves only a small variation in the wave amplitude near the shore and the wave run-up. A more accurate result is desirable only for studying special cases of resonance.

Since theories for circular waves on a nonuniform depth do not exist, we must rely on engineering judgment as guided by available theory in order to obtain reasonable estimates of the effect of shoaling on wave height.

It is known that for a periodic wave the change of wave height  $H$  due to the change of depth is calculated by assuming that  $H^2V = \text{constant}$  where  $V$  is the group velocity. This formula is based on the assumption of the conservation of transmitted energy from one plane to another. Hence, the correcting factor within an interval  $\Delta R = R_2 - R_1$  where the wave period is assumed to be the constant is  $\Delta \Gamma = \left(\frac{V_1}{V_2}\right)^{\frac{1}{2}}$  where  $V_1$  is the group velocity of a wave of period  $T$  at depth  $d_1$  at distance  $R_1$  and  $V_2$  is the group velocity of a wave of the same period at depth  $d_2$  at distance  $R_2$ .

Such a process of calculation has to be applied step by step for a succession of small intervals  $\Delta R$  where the wave period  $T$  does not vary too much. Then  $T$  is taken as  $\frac{T_1 + T_2}{2}$  for example.

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It is recalled that the group velocity  $V$  is given by the formula

$$V = \frac{C}{2} \left[ 1 + \frac{\frac{4\pi d}{L}}{\sinh \frac{4\pi d}{L}} \right]$$

where  $L$  is the wave length ( $L = \frac{g T^2}{2\pi} \tanh \frac{2\pi d}{L}$ ),  $d$  is the depth and  $C$  the wave celerity ( $C = \frac{L}{T}$ ). Hence,

$$\Delta \Gamma = \left[ \frac{\tanh m_1 d_1 \left[ 1 + \frac{2m_1 d_1}{\sinh 2m_1 d_1} \right]}{\tanh m_2 d_2 \left[ 1 + \frac{2m_2 d_2}{\sinh 2m_2 d_2} \right]} \right]^{\frac{1}{2}}$$

$$(m = \frac{2\pi}{L})$$

The cumulative correcting factor to be applied to the results obtained from the Kranzer and Keller theory (Appendix II) is given by their product:  $\Gamma = \prod_1^n \Delta \Gamma$

It can be seen that due to the change of depth, the wave height of the first waves must be increased by a small coefficient. This correction is negligible for the highest waves which are always in deep water (in the mathematical sense) before reaching the continental slope. The same method may be applied to the continental slope and will give a much higher coefficient for all waves. This is the subject of the following paragraph.

## 2. The Evolution of the Waves on the Continental Slope

All the wave energy is not transmitted over the continental shelf. A part of it is reflected towards the sea by the continental

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slope. This problem is considered in this section.

First, the variation of the wave period over the continental slope is negligible and  $T$  can be considered as a constant for a given wave. The continental slope on the Atlantic coast extends from a mean depth of 100 fathoms to a depth of 1000 fathoms over an average horizontal distance of 10 miles, giving a slope of 1/100. This slope may be steeper in local areas but it is never steeper than 1/10.

It has been demonstrated that when  $L \gg 2\pi a$ , where  $a$  is the horizontal distance (presently about 10 miles), the continental slope may be considered as a vertical wall. (See Yoshida (1950)) This condition is not satisfied for this case but considering the continental slope as a vertical wall presents the advantage of having a limit. The true fact lies between this limit obtained by considering that all energy is transmitted over the continental shelf following the method previously presented. It will be seen that (1) these two limits yield relatively close solutions, and (2) any error in the deep water wave problem does not change the final conclusions because of the regulating effect of the continental shelf.

Considering the continental slope as a vertical wall, the coefficient of transmission  $\alpha$  is: (See Le Méhauté, (1961))

$$\alpha = \frac{2}{1 + A \frac{L_2}{L_1}} \quad (1)$$

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$\alpha$  is the ratio of the wave height over the continental shelf and the wave height in deep water.

$$A = \frac{1 + \frac{2m_2 d_2}{\sinh 2m_2 d_2}}{1 + \frac{2m_1 d_1}{\sinh 2m_1 d_1}} \quad (2)$$

$$(m = \frac{2\pi}{L})$$

(Index 1 refers to deep water; index 2 refers to the continental shelf.) It may be noted that in the case of relatively shallow water,  $A = 1$  and

$$\alpha = \frac{2}{1 + \frac{L_2}{L_1}} = \frac{2}{1 + \frac{C_2}{C_1}} = \frac{2}{1 + \left[\frac{d_2}{d_1}\right]^{\frac{1}{2}}} \quad (3)$$

a formula also proposed by Poincare (Ed. 1931), Lamb (1932), Cochrane and Arthur (1948) and others for tidal waves or very long waves.

Considering the usual approximations, the coefficient of transmission for the first waves of the wave train is:

$$\alpha = 2 \left[ 1 + \frac{2 [gd_2]^{1/2}}{\frac{gT}{2\pi} \left\{ 1 + \frac{2m_1 d_1}{\sinh 2m_1 d_1} \right\} \tanh m_1 d_1} \right]^{-1} \quad (4)$$

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For the highest wave which has a shorter period, this simplifies to:

$$\alpha = 2 \left[ 1 + \left\{ 1 + \frac{2m_2 d_2}{\sinh 2m_2 d_2} \right\} \tanh m_2 d_2 \right]^{-1} \quad (5)$$

In any case, the general formula (1) always applies.

The spreading effect will be taken into account by multiplying the above values for  $\alpha$  by  $\left(\frac{R_2}{R_1}\right)^{1/2}$  as for a periodic wave,  $R_1$  and  $R_2$  being the distance from the origin at the toe of the continental slope and on the edge of the continental shelf, respectively. The problem remains to choose values of  $d_1$  and  $d_2$  because the variation of bottom slope is progressive. This point will be fully discussed in the following chapter.

The continental slope is connected to the continental shelf by a gentle bend from a depth of 100 fathoms to a depth of 20 fathoms. Over this part the Green Law applies (see Appendix V):

$$\frac{H_2}{H_1} = \left( \frac{d_1}{d_2} \right)^{1/4}$$

which can be refined by application of

$$\frac{H_2}{H_1} = \left( \frac{d_1 + H_1}{d_2 + H_2} \right)^{1/4}$$

which itself can also be corrected for friction forces.

Passing over the continental slope, the waves arrive on the continental shelf.

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3. The Evolution of the Waves on the Continental Shelf Before Breaking (Appendix V)

The continental shelf of the Atlantic coast of the United States is characterized by a slope so gentle that it can almost be considered horizontal. The slope above the 20 fathom contour varies from 2/10,000 to 5/10,000, becoming steeper near the shoreline, and ending with a beach slope between 1/30 and 1/10. The continental shelf extends over a length ranging from 50 to 100 miles or more.

The periodic wave theories and solitary wave theory can be used as sound guides for studying wave motion over the continental shelf. At first the most convenient theory for treating the problem at hand must be chosen. The chart shown in Fig. I-4 of Appendix I indicates the range of validity of the various theories used in practice. However, according to some experimental results presented in Fig. V-2 of Appendix V, it seems that the Airy theory extends up to a point where  $\frac{d}{d_b} = 1.4$  where  $d_b$  is the breaking depth. It is this last criterion which will be adopted. (See Munk (1949) )

Now the Airy law has been modified to take nonlinear convective inertia and bottom friction into account. Then it is demonstrated (see Appendix V) that the formulas to be used are:

$$H_2 = H_1 \left( \frac{d_1 + H_1}{d_2 + H_2} \right)^{1/4} \left[ 1 - \frac{2}{3\pi} \frac{f H_1 \Delta R}{d_1^2} \right] \left( \frac{R_1}{R_2} \right)^{1/2}$$

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where  $\Delta R$  is the horizontal distance between two cross sections defined by the distance  $R_1$  and  $R_2$  from the origin ( $\Delta R = R_2 - R_1$ ) depth  $d_1$  and  $d_2$ , and  $f$  is the friction factor dimensionless.

If the wave period is long enough, the wave motion can be considered as a succession of steady motions as far as the vertical velocity distribution, shearing stress and friction forces are concerned. In that case,  $f = \frac{8}{C_h^2}$  where  $C_h$  is the Chezy coefficient and  $C_h = 1.486 \frac{d^{1/6}}{n}$  where  $n$  is the Manning coefficient, and  $d$  is measured in feet. With  $n = 0.02$ , this yields:

$$\frac{2}{3\pi} \frac{f H_1 \Delta R}{d_1^2} = 0.00124 \frac{H \Delta R}{d^{7/3}}$$

$\Delta R = 5$  miles  $\cong 30,000$  feet is a convenient interval for calculating the wave height step by step over the continental shelf. For shallower water  $\Delta R$  must be chosen as a smaller value since the formula for the damping of the wave assumes that  $\Delta R \ll \frac{d^{7/3}}{.00124 H}$  feet.

Now, if  $\frac{d}{d_b} < 1.4$ , the following formula based on the solitary wave theory can be applied: (See Appendix V and Ippen and Kulin (1954) )

$$H_2 = H_1 \left( \frac{d_1}{d_2} \right) \left[ 1 - \frac{8}{15} \frac{f H \Delta R}{d^2} \right] \left( \frac{R_1}{R_2} \right)^{2/3}$$

up to the point where  $H$  reaches a value  $\geq 0.78 d$ . Then the wave breaks.

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4. Breakers and Wave Run-up  
(Appendices V and VI)

According to Miche (1944), a wave breaks when  $\frac{H}{L} = 0.14 \tanh \frac{2\pi d}{L}$ . However, in the case of long waves, the McGowan theory (1894) for a solitary wave is better verified:  $\frac{H}{d} = 0.78$ . The work of Bretschneider (1960) essentially combined the above two relations.

In actuality the slope  $S$  has a strong influence on the inception of breaking but the slope over the continental shelf is so gentle that this criterion can be retained.

When  $S < 0.01 f$ , the wave is damped due to bottom friction without breaking, i.e.  $S < \frac{58.10^{-6}}{d^{1/3}}$ . It can easily be verified that this will not happen except very locally (after passing over a shoal, for example, such that  $S$  becomes negative). Hence, the wave height evolution curve calculated according to the formulas presented in the previous sections will eventually meet the breaking index curve. The breaking index curve is simply defined by  $H = 0.78 d$ .

When  $0.01 f < S < 0.01$ , the breaker is not saturated\*, hence the wave height follows the breaking index curve\*. The wave maintains an almost symmetrical profile with relatively small spilling breakers at the crest. If the slope never exceeds 0.01, the wave energy is completely destroyed over the continental shelf and arrives at the shoreline with a negligible amplitude. No run-up will occur.

\* Explanation of these terms is given in Appendix V.

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Such is rarely the case. The wave arrives at the entrance to the bay and estuary with a wave height equal to  $0.78 d$ . This result holds true whatever the deep water wave height, provided it is high enough to reach the breaking index curve at the considered depth.

The maximum run-up can be calculated by considering the breaking wave height ( $H_b = 0.78 d$ ) at the location where the breaker becomes saturated, i. e. where the slope becomes steeper than  $0.01$ . This holds true provided the slope maintains this constant value or becomes even steeper. It cannot be true if the slope becomes less than  $0.01$ . The breakers reform into nonsaturated breakers over this slope. A saturated breaker on the continental slope, if any, will become nonsaturated over the continental shelf and finally reach the breaking index curve  $H = 0.78 d$  as a smaller wave.

It has been found that the run-up  $R$  of a long wave over a  $1/10$  slope is given by  $\frac{R}{H} = 2.82$ ,  $H$  being the wave height where the bottom slope is  $0.01$ , i. e.  $0.78 d_b$  (see Appendices V and VI). Other formulas based on experimental observations for slopes of  $1/30$  and  $1/60$  are also reviewed in Appendix V and indicate similar results.

5. Penetration of the Waves Into Estuaries

(Appendix VII)

The penetration of waves into the lower Chesapeake Bay may be considered as the penetration of a wave into a basin, as shown

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by Fig. 4. The wave diffracts northward and quickly becomes negligible. According to the linear theory of wave diffraction, the wave height along AB is 1/2 of the wave height at the entrance and decreases very quickly northwards.

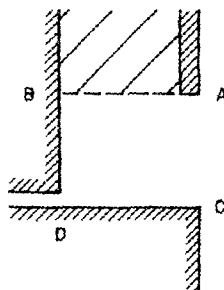


FIGURE 4  
SCHEME OF LOWER CHESAPEAKE BAY

Although the wave motion under consideration is highly nonlinear, such results remain qualitatively true, but nonlinear long waves and bores will be expected to give more wave agitation than is indicated by the linear theory.

As a first approximation, the side CD can be considered as a vertical wall. But the usual variations of wave height given by the theory on the side BD, which theoretically range from 1.175 to 0.88, cannot be expected to be observed in this case because of the complexity of the sea bottom.

The wave calculation from AC to BD should instead take

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Then the maximum wave height (and limit steepness) of short crested waves is much higher than in the case of long crested waves. A formula has been proposed for the limit steepness of superimposed waves, whatever their relative direction. It applies for irregular waves, clapotis, or short crested waves. (See Le Méhauté (1961)). For N waves this formula is:

$$\sum^N \left( \frac{H}{L} \right)^2 = \frac{0.02}{N} \sum^n \tanh^2 \frac{2\pi d}{L}$$

In the case of long waves this formula gives:

$$H_{\max} = 2 H_b = 1.256 d$$

In the second case, where the energy diverges, the wave celerity is  $\sqrt{g(d+H)}$  where H can be calculated by applying the principle of conservation of energy between orthogonals from the considered location and the previous location where H departs from the breaking index condition, i. e.

$$H_1^2 b_1 \sqrt{g(d_1 + H_1)} = H_b^2 b_b \sqrt{g(d + H_b)}$$

In this formula  $b_1$  and  $b_b$  are the distances between orthogonals.

In fact the principle of conservation of energy between orthogonals must also be corrected empirically to take into account diffraction effects. Hence in "shaded" areas the wave breaks only near the shoreline.

Now consider more carefully the side DC, which in fact is

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curved. A part of the wave energy will be dissipated by lateral breaking and penetration over land (see Chen (1945) ). A part will be reflected towards side BD. This reflected part will be superimposed on the wave arriving directly from AC. But also, along the side DC the so-called phenomenon of a stem or edge wave may be expected because the angle of incidence is small. This phenomenon of stem wave partly reduces the loss of energy from C to D.

The problem of wave run-up along the side BD is theoretically very similar to the problem of the wave run-up on the open coast. In practice the problem is quantitatively different because the depth near the shoreline is smaller. Inside the smaller estuaries, near Norfolk, the problem is that of successive diffraction with multiple reflection and damping due to bottom friction.

The ratio of width of the channel to the wave length is always too large to consider the motion as similar to that of a tidal wave. Hence the diffraction phenomenon assumes great importance. Unfortunately very little is known about the diffraction of long waves. The method proposed in Appendix VII can be applied to solve this problem.

Another problem which can be a cause of concern is the elevation of the mean water level by mass transportation effect. This average elevation of the water level can be roughly estimated by computing the discharge due to a succession of limit solitary waves at the entrance. Then, by assuming that this volume of water

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is spread northward in the bay at a speed  $\sqrt{gd}$ , this permits calculation of the total horizontal area on which an elevation of the mid-water level exists by mass transportation. It is known that the mass transportation due to a solitary wave is:

$$\Omega = \frac{4}{\sqrt{3}} d^{3/2} H^{1/2} \quad (\text{ft. -sec. units})$$

where  $d$  is the depth and  $H$  is the height. Since in the case of a limit solitary wave  $H = 0.78 d$ , the total volume of water entering the Chesapeake Bay over a length  $\ell$  is:

$$V = 1.7 N \int_0^{\ell} d^2 dx$$

where  $N$  is the number of waves.

6. Resonance of Harbor Basins

The penetration of comparatively small waves into local harbor basins could cause severe agitation if the wave period is close to a resonant frequency of the harbor basin. The majority of harbor basins in the Newport/Norfolk area consist of two parallel piers open to the main channel. The natural resonant frequencies in the longitudinal direction are phenomena very similar to the resonance of sound in organ pipes (see Defant (1961), Le Méhauté (1954, 1961, 1962), Miles and Munk (1961)). The amplitude of agitation in the harbor basin at resonance theoretically tends to infinity if friction is neglected. In practice also, the agitation is

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limited physically by a maximum wave steepness. In some basins it is also possible that resonance could occur in a lateral direction. A standing wave of length equal to twice the width of the basin would give such conditions.

The resonant wave frequency in the lateral direction is given by  $T = \frac{2b}{\sqrt{gd}}$  where  $b$  is the width of the basin and  $\sqrt{gd}$  is the velocity of propagation of a long wave in shallow water. In fact there could also exist harmonics but it will be seen that only the fundamental lateral resonant frequency is of interest here.

The longitudinal resonance phenomenon is a little more complicated. When the harbor basin is long and narrow, the resonant wave lengths are given by  $L_n = \frac{4\ell}{2n+1}$  where  $\ell$  is the length of the basin and  $n$  is an integer (0, 1, 2, 3...).  $L_n = T_n \sqrt{gd}$  gives  $T_n = \frac{4\ell}{\sqrt{gd}(2n+1)}$  as the resonant wave periods. In practice when the width of the harbor basin is not negligible with respect to the length of the basin, a correction factor has to be applied such that:

$$T_n' = T_n (1 + \xi)$$

where  $1 + \xi$  is a function of  $b/\ell$  and  $L/\ell$  (i.e. with  $L/\ell = \beta$ ).

The resonance phenomenon can be closely evaluated from:

$$\frac{\beta}{4 \left(1 - \frac{n\beta}{2}\right)} = 1 + \frac{b}{\pi\ell \left(1 - \frac{n\beta}{2}\right)} \left[ \frac{3}{2} - 1n \frac{\pi b}{\ell \left(1 - \frac{n\beta}{2}\right)} - \gamma \right]$$

where  $\gamma$  is Euler's constant ( $\approx 0.577215$ ). This equation has to

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be solved by trial and error for  $\beta$ , when  $b/L$  is given for various values of  $n$ . Two graphs which greatly facilitate this procedure are presented as Figs. 5 and 6. The general method of obtaining the resonant periods can be described as follows.

(1) For a given value of  $n$  corresponding to the harmonic to be investigated a value of  $\beta$  has to be assumed. This value can be estimated quite closely with practice. (2) Fig. 5 is used to derive  $\frac{\beta}{4 \left(1 - \frac{n\beta}{2}\right)}$  (3)  $\frac{b}{L \left(1 - \frac{n\beta}{2}\right)}$  can be computed from the basin characteristics. (4) Using this value of  $\frac{b}{L \left(1 - \frac{n\beta}{2}\right)}$  the value of the right hand term above can be read off Fig. 6.

(5) This value should check with  $\frac{\beta}{4 \left(1 - \frac{n\beta}{2}\right)}$  calculated in step 2.

In practice it is found that the correction term  $\xi$  is almost negligible for any resonant frequency but the fundamental ( $n = 0$ ). For harmonics ( $n \neq 0$ ),  $\beta$  may be considered as a constant within the usual range of  $\frac{b}{L}$  as is seen in Fig. 6.

The amplitude of agitation at resonance has been estimated from an adaptation of a theory and some experimental results by Le Méhauté (1959, 1961) for the agitation in a harbor caused by waves at least four or more times the wave amplitude in the main river channel.

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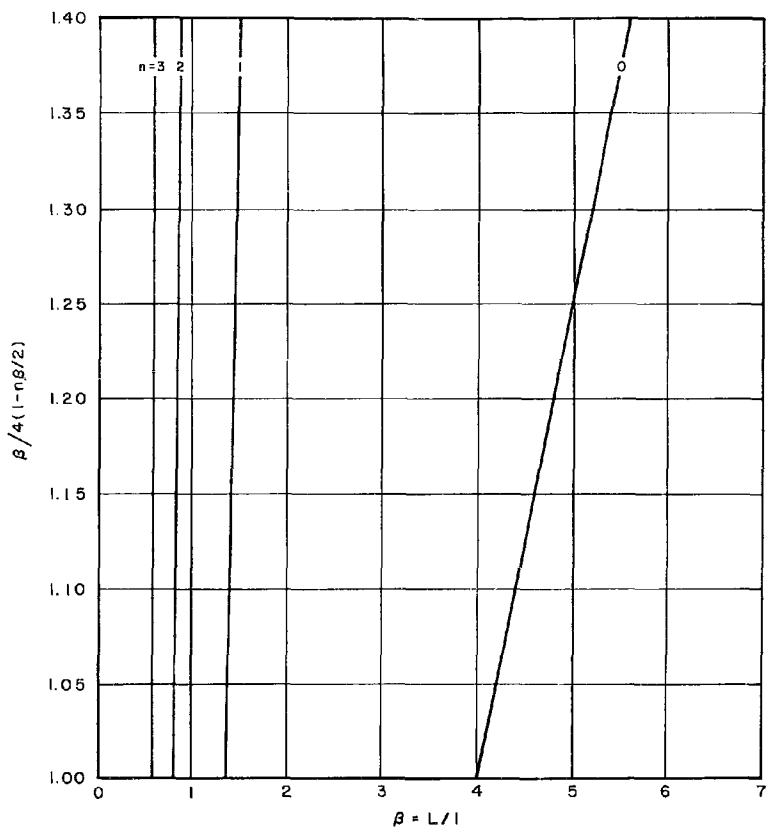


FIGURE 5  
NOMOGRAPH FOR HARBOR RESONANCE

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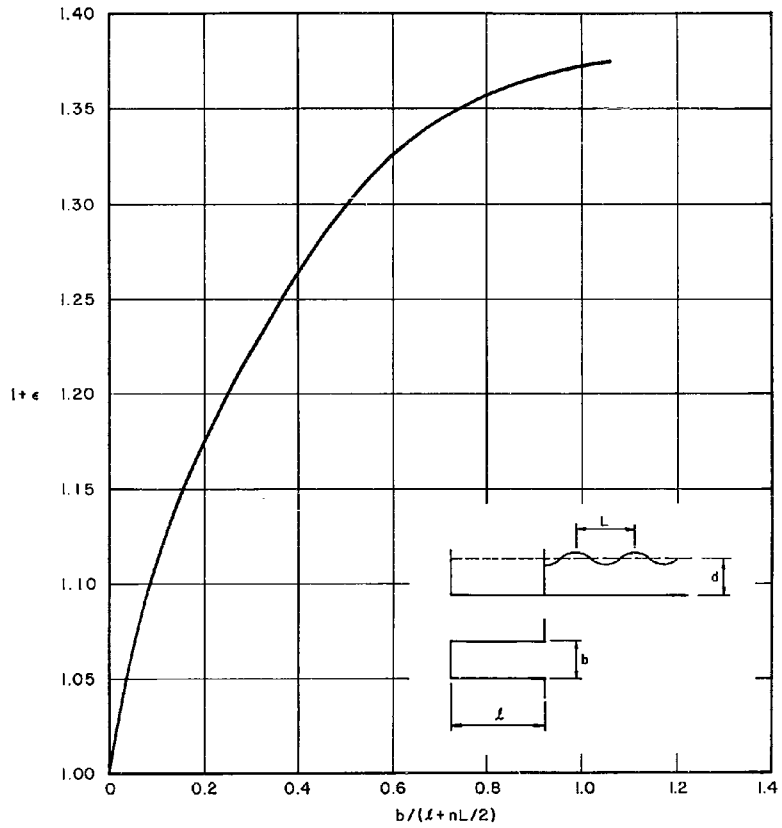


FIGURE 6  
NOMOGRAPH FOR HARBOR RESONANCE

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CHAPTER V

MAIN QUANTITATIVE RESULTS

The most important results have already been summarized in Chapter II, Part A of this report. In particular, Fig. 3 representing a map of the maximum possible flooded areas, gives the essential results of this study.

The purpose of this chapter is to complement the results already given by presenting more information and results given by the intermediate steps of the calculations without entering into all the details presented in the various appendices. These quantitative results have been obtained by applying the methods of calculation described in Chapter IV to the Atlantic coast, with special considerations for the Chesapeake Bay area.

The original disturbance was assumed to be located at a point defined by:

Longitude	$35^{\circ} 57-1/2' N$
Latitude	$68^{\circ} 25' W$

where the depth is 3,000 fathoms, but surrounded by a depth of 2,600 fathoms. (Fig. 2)

The wave characteristics were provided for three values of the magnitude of the disturbance at 20 miles from their origin. They are defined by using an asymptotic solution developed by Kranzer and Keller, which is assumed to be valid at a sufficient distance from the disturbance (See Appendices I and II). The wave input is characterized by wave trains,

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the distance between crests varying with time as shown by Fig. 1.

Considering the particular location of the disturbance and the bottom contours, it appears that refraction effects are absolutely negligible except at some locations near the shoreline and, in particular, at the entrance to Chesapeake Bay. The bottom contours are practically concentric circles around the location of the disturbance as shown by Fig. 7. Hence, up to the continental slope, the plan of the wave crests is represented by a series of concentric circles, despite any shoaling effect. The refraction effects over the continental shelf appear only locally and can be neglected. It is for this reason that the distance between orthogonals given in Appendix V can be replaced by a linear function of the distance from the origin  $R$ . It is for this reason, also, that a wave refraction diagram has not been included since it would simply give the radii as orthogonals even for the first longest waves which "feel" the bottom at an early stage.

The results of all calculations are presented in the enclosed tables and graphs. Fig. 7 shows the bottom profile from the original disturbance up to the continental slope along four significant radii shown on Fig. 2. This figure demonstrates the relative degree of accuracy obtained by assuming that the bottom contours are concentric circles.

Fig. 8 summarizes the main results which have been obtained by applying the methods of calculation presented in Appendix II for the deep water wave problems. A correction factor for the first longest wave has also been calculated to take into account the shoaling effect. The results have been calculated for a radius of 300 nautical miles, i.e. the toe of the continental slope.

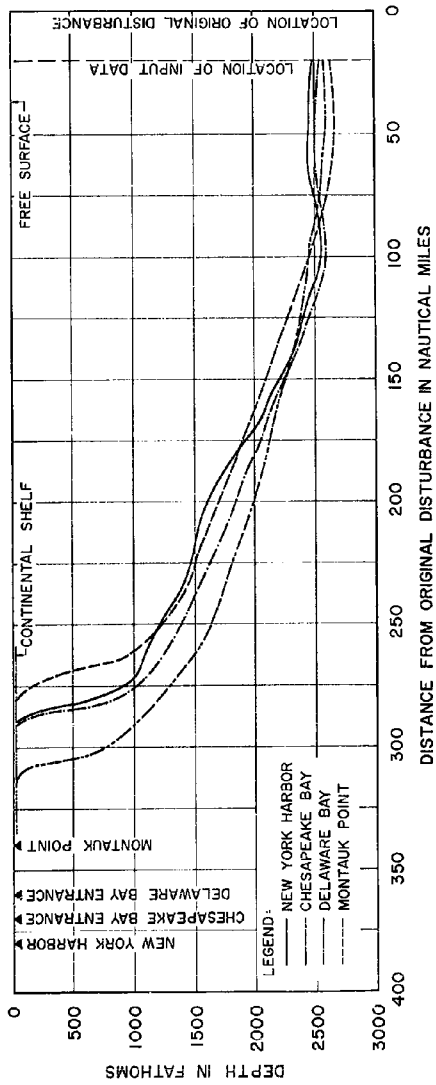


FIGURE 7  
DEEP WATER BOTTOM PROFILES ALONG FOUR SIGNIFICANT RADII

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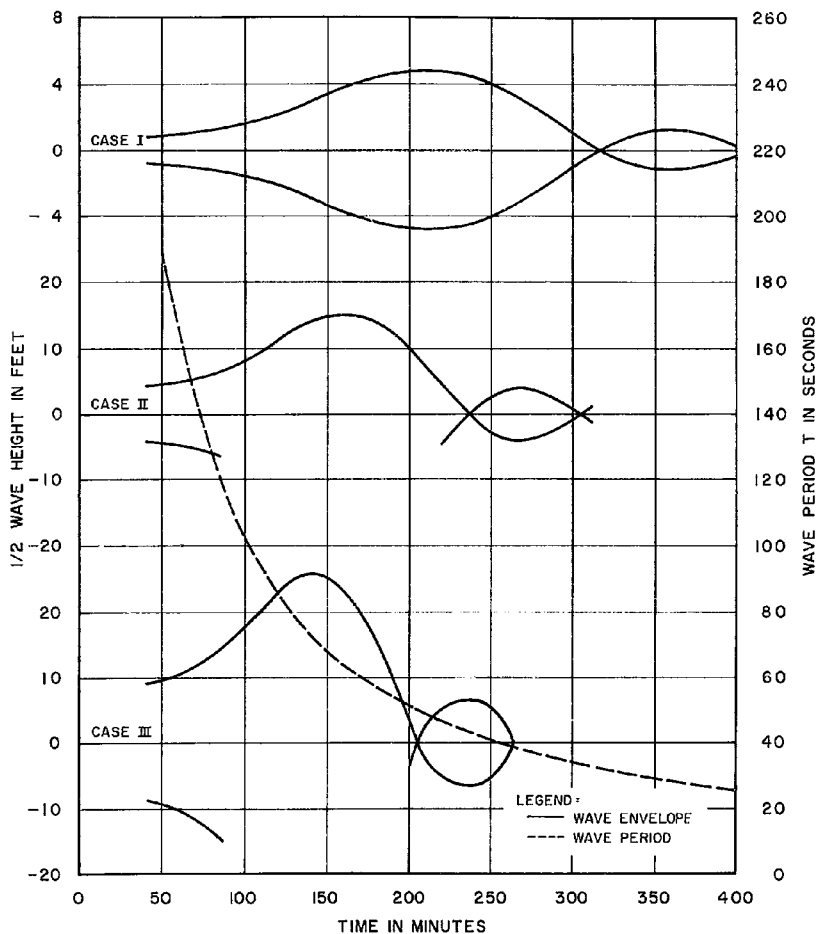


FIGURE 8  
WAVE PERIOD AND WAVE ENVELOPES VERSUS TIMES AT  
300 NAUTICAL MILES (TOE OF THE CONTINENTAL SLOPE-  
DEPTH 6000 FEET)

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The coefficient of transmission over the continental slope has been calculated as shown in Chapter IV. The values of this coefficient of transmission depend upon the choice of depths  $d_1$  and  $d_2$  and the wave period. The period is well known and varies with time as shown in the previous graphs (Fig. 8). But the choice of depths is quite arbitrary because the bottom does not present any sharp angle which would permit a clear interpretation of  $d_1$  and  $d_2$ .

Considering the bottom profile presented in Fig. 7, it seems that the most reasonable value for  $d_1$  is 1000 fathoms, which for most of these wave periods can be considered mathematically as infinity.

The continental slope ends at a depth of  $d_2 = 20$  fathoms, but in practice the slope between 120 fathoms and 20 fathoms is very gentle. Hence it can be assumed that all the wave energy is transmitted from a depth of 120 fathoms to a depth of 20 fathoms, and the wave reflection is due to that part of the slope below a depth of 120 fathoms only.

However, in order to be able to estimate the error which is introduced by choosing other combinations of depths (and more advanced theory), the coefficient of transmission has also been calculated for two extreme cases, namely: (1) all wave energy is transmitted over the continental shelf (there is no reflection), and (2) the continental shelf acts as a vertical wall from  $d_1 = 1,000$  fathoms to  $d_2 = 20$  fathoms. The values obtained in the intermediate case considered as the best case, and these two extreme cases, are presented in Table 1 as a function of the wave period.

It is seen that except for the first longest waves (which are not the

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TABLE 1  
 COEFFICIENT OF TRANSMISSION OVER THE CONTINENTAL SHELF  
 VERSUS WAVE PERIOD

Wave Period T = sec.	No Reflection	Reflection from 1000 fath. to 120 fath. Transmission from 120 fath. to 20 fath.	Reflection from 1000 fath. to 20 fath.
343	3.75	2.55	1.98
257	3.25	2.40	1.95
188	2.80	2.31	1.87
171	2.65	2.25	1.75
147	2.45	2.16	1.72
128	2.31	2.09	1.69
114	2.17	2.02	1.65
103	2.07	1.96	1.62
85.8	1.88	1.83	1.56
73.5	1.74	1.71	1.50
64.4	1.64	1.61	1.46
57.2	1.55	1.53	1.41
51.5	1.47	1.46	1.37
46.8	1.40	1.39	1.32
42.9	1.37	1.36	1.29
39.6	1.35	1.35	1.26
38.8	1.21	1.31	1.23
34.3	1.27	1.27	1.20
32.2	1.22	1.22	1.17
30.3	1.19	1.19	1.14
28.6	1.16	1.16	1.10
27.1	1.11	1.11	1.09
25.7	1.09	1.09	1.07
24.5	1.07	1.06	1.06
23.4	1.05	1.05	1.05

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highest waves) the difference is very small and consequently the error due to the proposed approximations is quite negligible. It is also seen that the waves of the second wave train are relatively so short that they pass over the shelf without being significantly influenced by the continental slope. (A more exact theory will be necessary for tsunami waves due to seismic disturbances because being of longer wave period, these waves are more influenced by the continental slope.)

Now the wave trains over the continental shelf can be calculated by a simple multiplication of the coefficient of transmission with the wave heights given at 300 miles by taking into account the variation of wave period with time and the spreading effect. Hence the wave trains at 320 miles from the explosion over a depth of 20 fathoms are given in Fig. 9.

The wave motion over the continental shelf no longer depends upon the period and shallow water wave theories can be applied. The bottom friction is of great importance. Then the calculation has to be continued step by step as indicated in Chapter IV. The results of such calculations are presented in Table 2 and in Fig. 10. The interval  $\Delta R$  has been chosen to be five miles. The maximum wave heights corresponding to the three cases under study are also indicated on the graphs.

It is seen that in Case II and Case III most of the waves reach the breaking index curve prior to reaching a bottom slope smaller than 1/100 where the breakers become saturated. Hence the waves follow the breaking index curve whatever their original wave height in deep water. It is seen that only in Case I does the wave reach the breaking index curve over a slope which is steep enough for the breaker to be saturated. Hence there is less wave run-up than in Cases II and III.

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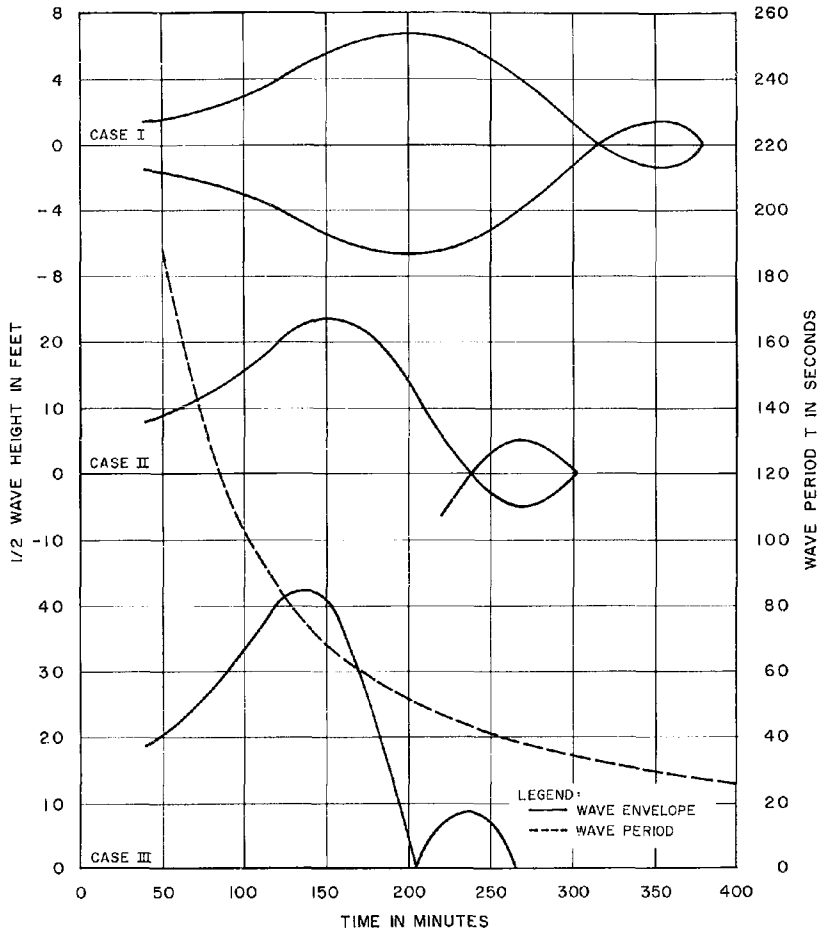


FIGURE 9  
WAVE PERIOD AND WAVE ENVELOPES VERSUS TIMES AT  
320 NAUTICAL MILES (EDGE OF CONTINENTAL SHELF)

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TABLE 2  
WAVE HEIGHTS VERSUS DISTANCE OVER THE CONTINENTAL SHELF  
(in feet)

Distance R in Naut. Mi.	320	325	330	335	340	345	350	355	360	365	370
	1	1.006	1.010	1.017	1.020	1.034	1.062	1.108	1.158	1.304	2.707
	2	2.014	2.021	2.034	2.038	2.064	2.118	2.207	2.295	2.559	2.593
	5	5.025	5.029	5.044	5.043	5.088	5.192	5.370	5.524	6.002	5.658
	10	10.015	9.989	9.986	9.936	9.967	10.091	10.309	10.404	10.111	8.707
	15	14.967	14.869	14.808	14.677	14.641	14.727	14.884	14.780	14.957	11.492
	20	19.902	19.710	19.526	19.264	19.113	19.082	19.101	18.676	18.403	12.948
	30	29.661	29.159	28.660	28.066	27.555	27.177	26.715	25.427	---	14.390
	40	39.296	38.392	37.443	36.379	35.389	34.500	33.372	30.999	---	14.767
	50	48.800	47.297	45.829	44.183	42.609	41.087	38.934	35.470	---	14.658
	60	58.182	56.046	53.899	51.624	49.368	47.156	44.402	--	---	14.299
	70	67.501	64.564	61.639	58.606	55.646	52.680	--	--	---	13.842
	80	76.640	72.869	69.130	65.327	61.551	--	--	--	---	13.359
H <sub>max</sub>	93.6	87.3	82.7	77.0	73.0	66.4	57.1	45.9	36.5	21.0	

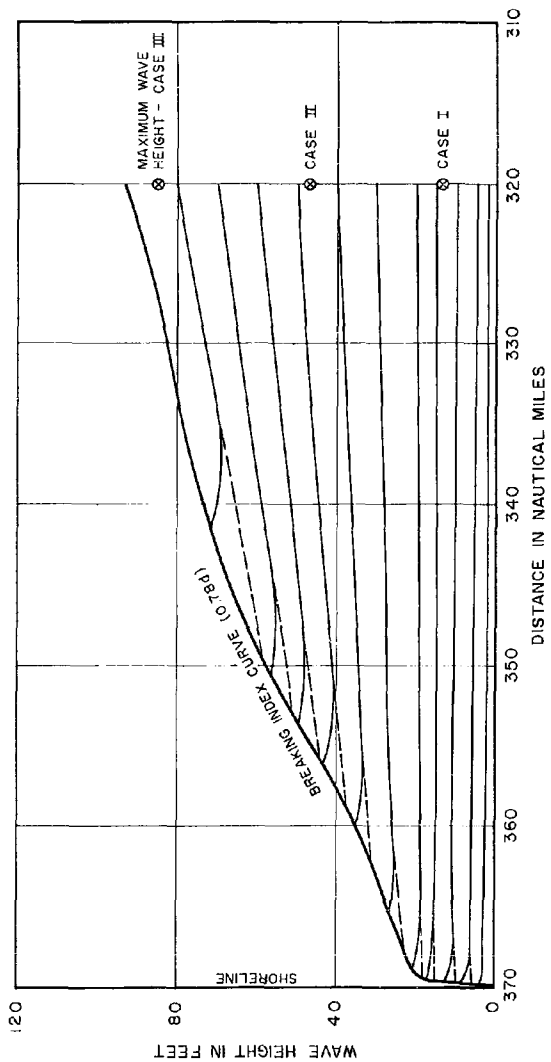


FIGURE 10  
WAVE HEIGHT EVOLUTION OVER THE CONTINENTAL SHELF

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A very simple conclusion can be deduced from these calculations: the maximum possible wave run-up is the same in Cases II and III for practically all the waves. This maximum wave run-up is about twice the value of the breaking wave height where the slope is 0.01. Such a slope is encountered on the lower part of the entrance of Chesapeake Bay over a depth varying from 20 feet to 30 feet. Hence the maximum vertical run-up over an average slope of 1/15 is  $2 \times 0.78 \times \frac{20}{30}$  ft. =  $\frac{31}{47}$  feet. It can therefore be concluded that the dunes which are only 15 to 20 feet above the sea level will certainly be overtopped and breached. These dunes, once overtopped, are very rapidly destroyed by erosion. This destruction occurs with the first few waves. Then the following waves penetrate deeper and deeper inshore destroying all existing coastline structures by dynamic action.

However, it can be expected that this dynamic effect will be limited to the neighborhood of the shoreline because the waves are rapidly damped as they penetrate further over land. Since the coastline is very flat, a large area can be flooded by a rise in water level caused by the mass transportation effect due to the series of "solitary" waves.

On the northern part of the entrance of Chesapeake Bay the bottom slope has a small value before reaching the shoreline. Hence the natural protection of the shelf is extended almost up to the shoreline. The waves lose their energy over the existing swamps and give a relatively smaller run-up on dry land.

At the entrance of the Bay itself the wave heights in Cases II and III are directly related to the depth by the relationship  $H = 0.78 d$ . This

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very simple law results from the fact that the waves follow the breaking index curve and appear at the entrance of Chesapeake Bay as a succession of almost symmetrical spilling breakers which can be considered as a succession of limit solitary waves. Since the average depth is 26 feet (including 2.8 feet of tide), they will appear as a succession of solitary waves of about 20 feet in height.

Further quantitative information has been obtained in Appendix VI for a 1/10 bottom slope. Despite the fact that the slope is usually smaller, these values give a rough estimate of what can be expected in the present case.

First, it has been seen that the height of the leading front when reaching the shoreline on the open coast is about seven feet ( $\eta/d_1 = 0.22$  and  $d_1 \approx 30$  feet). The thickness of the water which follows the leading front will increase up to a value of about 13 feet (see Fig. VI-21). The speed of the water of the leading front at the shoreline is 48.6 ft/sec ( $u_S / \sqrt{g d_1} = 1.568$ ).

The maximum horizontal run-up for the leading front (assuming this leading front is not caught up by the following wave elements) gives the minimum horizontal distance where the waves may have a dynamic effect. This value  $x_{\max} - x_s$  is found by giving a zero value to  $u$  in the formula (VI-56). Then it is found that  $x_{\max} - x_s$  could be as high as 1000 feet.

In fact the bottom slope is so small after the dune has been overtopped that the obtained formula gives only the very minimum. Because of the limitation of validity of the obtained formula, it must be expected that the dynamic effect of the succeeding wave will be spread over a 3000 foot area from the shoreline. Also, since the water depth will increase

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because of the mass transportation over the dune, the wave effect will be felt deeper and deeper.

The present state of the art is such that it is not possible to give presently more information about this problem, which would require taking account of the dune evolution by erosion. Fig. 3 gives what can be considered as the maximum possible flooding areas which will certainly not be exceeded.

Fig. 11 presents the wave refraction diagram for the spilling breakers entering the Chesapeake Bay. It is seen that the wave motion rapidly becomes very complex. A number of orthogonal cross each other limiting the value of the method, in spite of the improvements for nonlinear effects proposed in Chapter IV. It is interesting to note the differences in wave refraction between a spilling breaker as presented in Fig. 11 and a bore as presented in Appendix VII.

Also, it is known that when a long wave passes over a shoal, a double crest appears. This fact has been observed in practice at sea, as well as in a laboratory. The long waves are transformed into a succession of shorter waves. Such a phenomenon can be analyzed only in the case of a simply shaped two-dimensional shoal. It is impossible to analyze it for a two-dimensional case with the present state of the art.

Whatever the probable quantitative accuracy of the motion calculated in Chesapeake Bay, the wave run-up has been evaluated from the wave refraction diagram presented in Fig. 11 and the methods presented in Chapter IV. The results of these computations are also given by Fig. 3.

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\*Because of its size, Fig. 11 is included at the back of this report

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Because of the complex wave patterns just inside the entrance to Chesapeake Bay, it is difficult to assess the damage to the Chesapeake Bay bridge-tunnel which is currently under construction. Although the mean wave height is about 20 feet, because of the formation of short-crested waves, the local wave heights in parts of the structure could greatly exceed this value and be as high as 40 feet locally. Also there could be serious erosion effects around the foundations. Knowledge of the movement of sand and silt by long waves is very limited. These same considerations apply to the Newport-Norfolk bridge-tunnel from Willoughby Spit to Old Point Comfort.

Even though the structures may be saved from destruction by dynamic effects or undercutting of their foundations, any wave overtopping near the tunnel entrances would flood out the tunnel sections and render the structures inoperative for traffic.

The mid-elevation in the lower Chesapeake Bay due to the mass transportation by the waves at the entrance has been found to be about one foot. In any case, it cannot exceed two feet. However, this average elevation generates a very long wave which propagates northward at a velocity  $\sqrt{gd}$  similar to a tidal wave. Hence the elevation in the upper Chesapeake Bay and Potomac River can be as high. For example, the tide is slightly higher at Washington than at the entrance to Chesapeake Bay. However, the shorter waves due to explosion are quickly damped by successive diffraction and do not penetrate so far.

Now the results of the problem of harbor resonance are presented.

About one hour after the waves arrive at the entrance to Chesapeake

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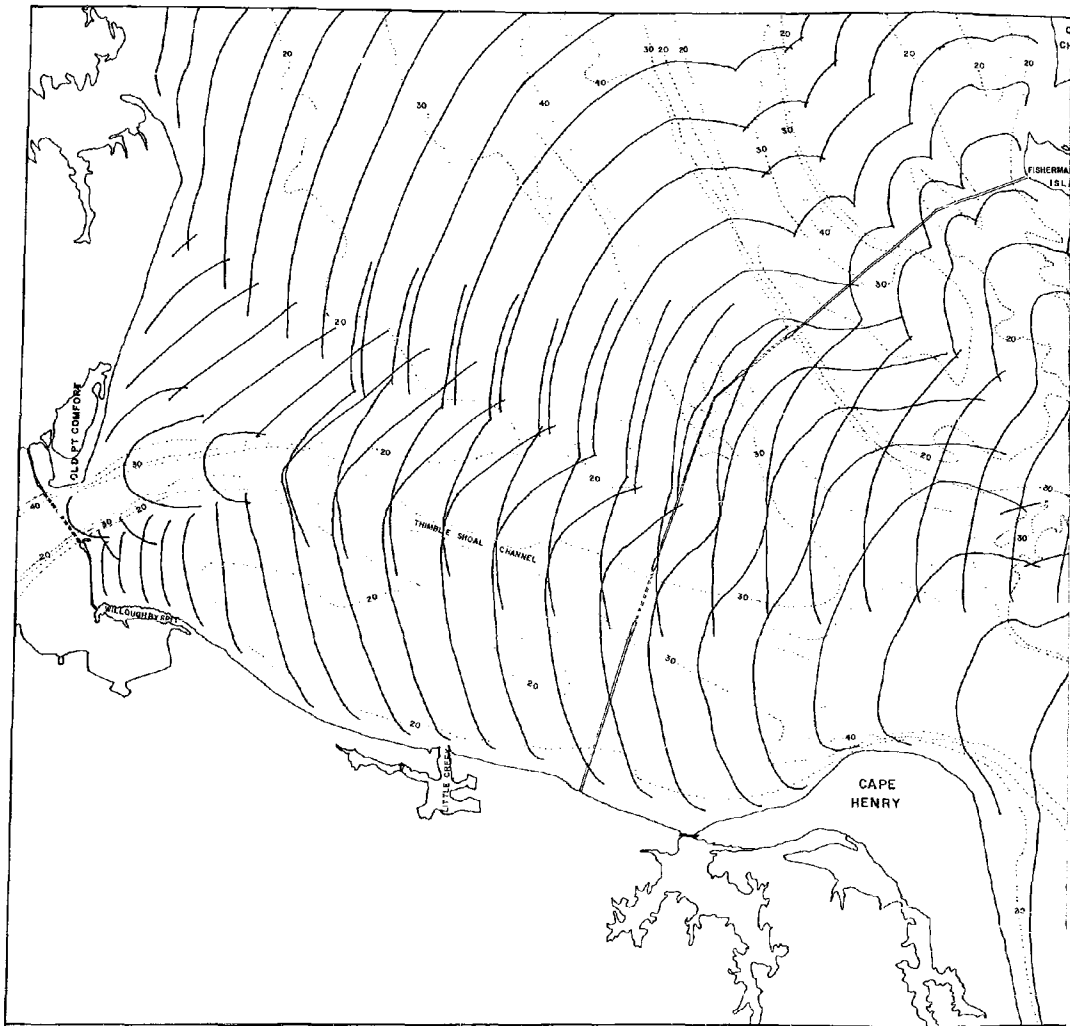
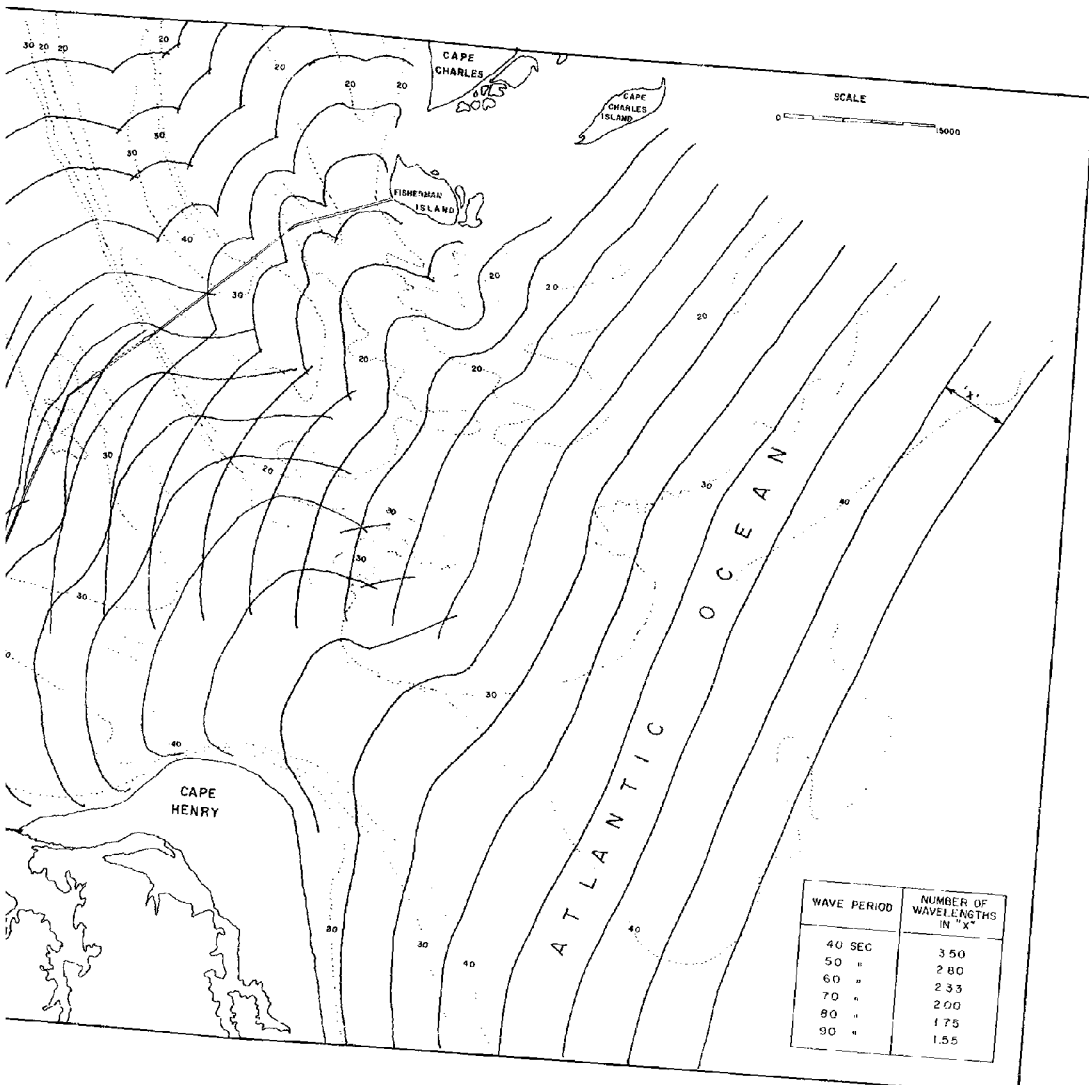


FIGURE 11  
WAVE REFRACTION DIAGRAM FOR A SPILLING BREAKER



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Bay, they will arrive at Newport News and a short while later their effects will be apparent in the Elizabeth River at Norfolk and Portsmouth. The fundamental natural period of oscillation in the longitudinal direction, the first two harmonics, and the natural period of lateral oscillation for various harbor basins have been computed and are presented in Tables 3 through 6. The locations of these basins are shown on Fig. 12 through 14. It is seen that the range of wave periods at which resonance will occur compares very closely with the range of wave periods in the wave trains generated by the disturbance as they are shown by Fig. 15.

Also, in some of the harbor basins and navigation channels serious siltation and/or erosion could occur very rapidly because of the high velocities and severe agitation. In particular, the navigation channel of Little Creek would certainly be filled by sand. This will be especially true if resonance occurs as the motion is then amplified. Undermining by scouring would occur at pier foundations and the material deposited in relatively calm water in the main shipping channels. Such phenomena have already been observed by the action of tsunami waves. (Iwasaki and Horikawa (1960) ).

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TABLE 3

NATURAL OSCILLATORY PERIODS OF HARBOR BASINS IN THE  
NEWPORT NEWS AREA

Harbor Basin No.	Fundamental Period of Oscillation	First Harmonic	Second Harmonic	Lateral Oscillation Period
	Sec.	Sec.	Sec.	Sec.
1	816	261	142	32
2	125	34	19	46
3	123	38	22	46
4	155	45	25	18
5	151	40	23	49
6	104	29	17	25
7	95	28	16	9
8	97	29	16	9
9	96	29	16	9
10	86	23	13	17
11	159	46	26	30
12	146	42	24	30
13	153	44	25	30
14	129	38	22	20
15	129	38	22	20
16	101	27	15	19
17	117	32	18	16
18	96	27	15	13

Approximate arrival time after entering Chesapeake Bay;  
Slightly greater than one hour.

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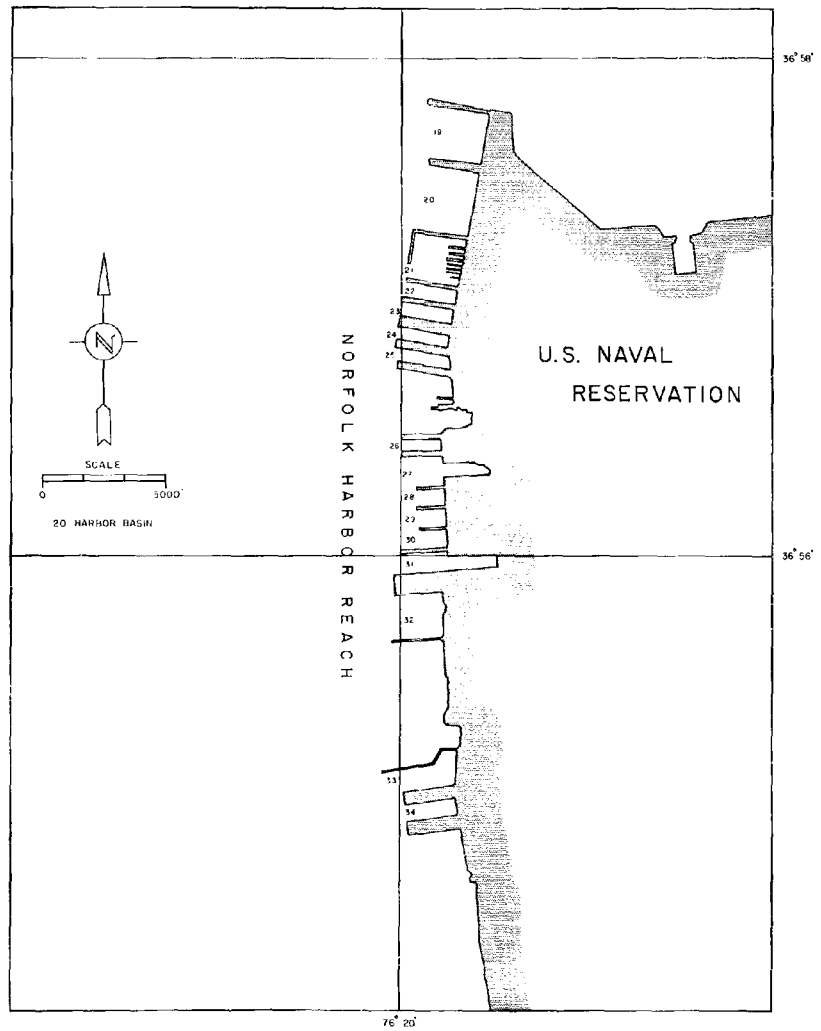


FIGURE 12  
HARBOR BASINS AT NEWPORT NEWS

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TABLE 4  
 NATURAL OSCILLATORY PERIODS OF HARBOR BASINS  
 IN NORFOLK HARBOR REACH

Harbor Basin No.	Fundamental Period of Oscillation	First Harmonic	Second Harmonic	Lateral Oscillation Period
	Sec.	Sec.	Sec.	Sec.
19	321	96	55	119
20	400	102	58	190
21	203	59	34	21
22	191	54	31	24
23	191	54	31	24
24	196	58	33	19
25	196	58	33	19
26	156	45	26	18
27	379	105	60	60
28	122	33	19	27
29	121	32	18	32
30	173	47	27	33
31	378	113	64	32
32	230	59	34	78
33	261	74	42	35
34	188	54	32	24

Approximate arrival time after entering Chesapeake Bay;  
 Slightly less than one hour.

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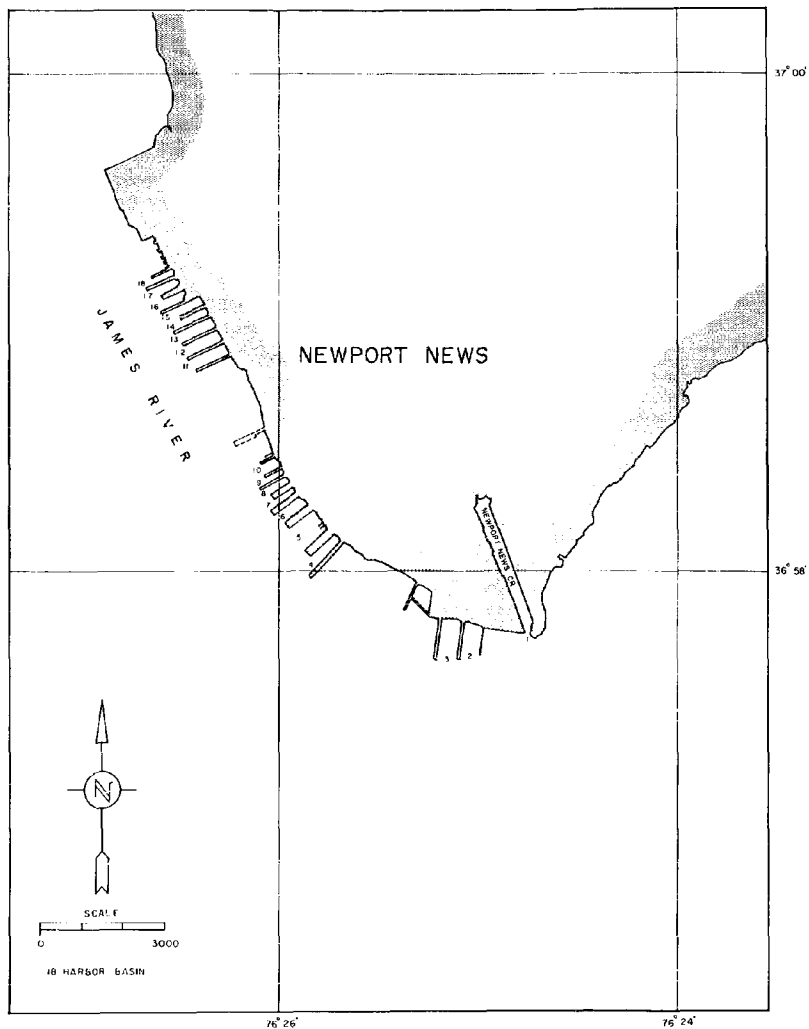


FIGURE 13  
HARBOR BASINS IN NORFOLK HARBOR REACH

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TABLE 5

NATURAL OSCILLATORY PERIODS OF HARBOR BASINS IN PORT  
 NORFOLK AND TOWN POINT REACH ON THE EAST SIDE OF  
 ELIZABETH RIVER

Harbor Basin No.	Fundamental Period of Oscillation	First Harmonic	Second Harmonic	Lateral Oscillation Period
	Sec.	Sec.	Sec.	Sec.
35	165	46	26	24
36	191	54	31	24
37	167	47	27	23
38	172	50	29	20
39	165	48	27	18
40	186	54	31	19
41	183	54	31	16
42	68	19	11	9
43	68	19	11	9
44	68	19	11	9
45	104	32	18	7
46	113	33	19	12
47	159	50	28	8
48	110	33	19	8
49	93	28	16	8
50	92	26	15	14
51	126	36	20	15
52	65	18	10	10
53	113	33	19	12

Approximate arrival time after entering Chesapeake Bay:

1-1/4 hours.

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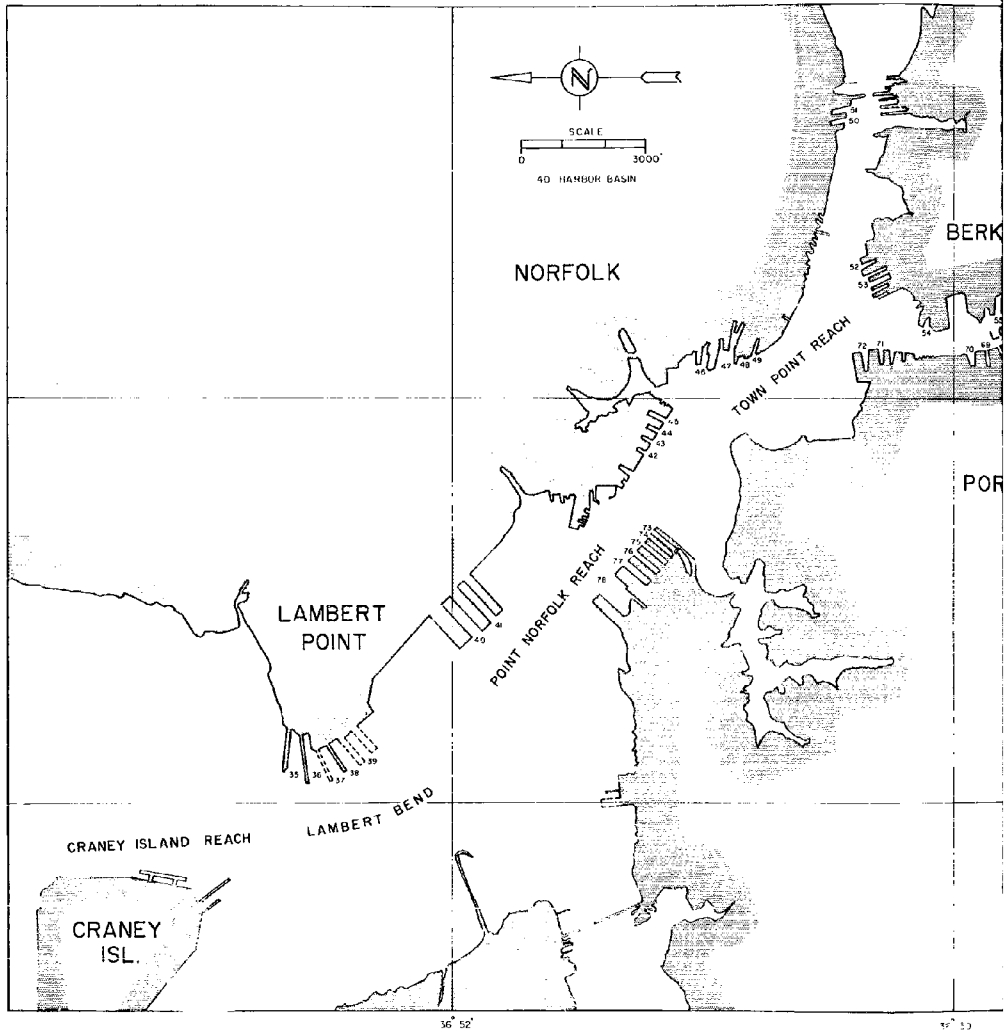
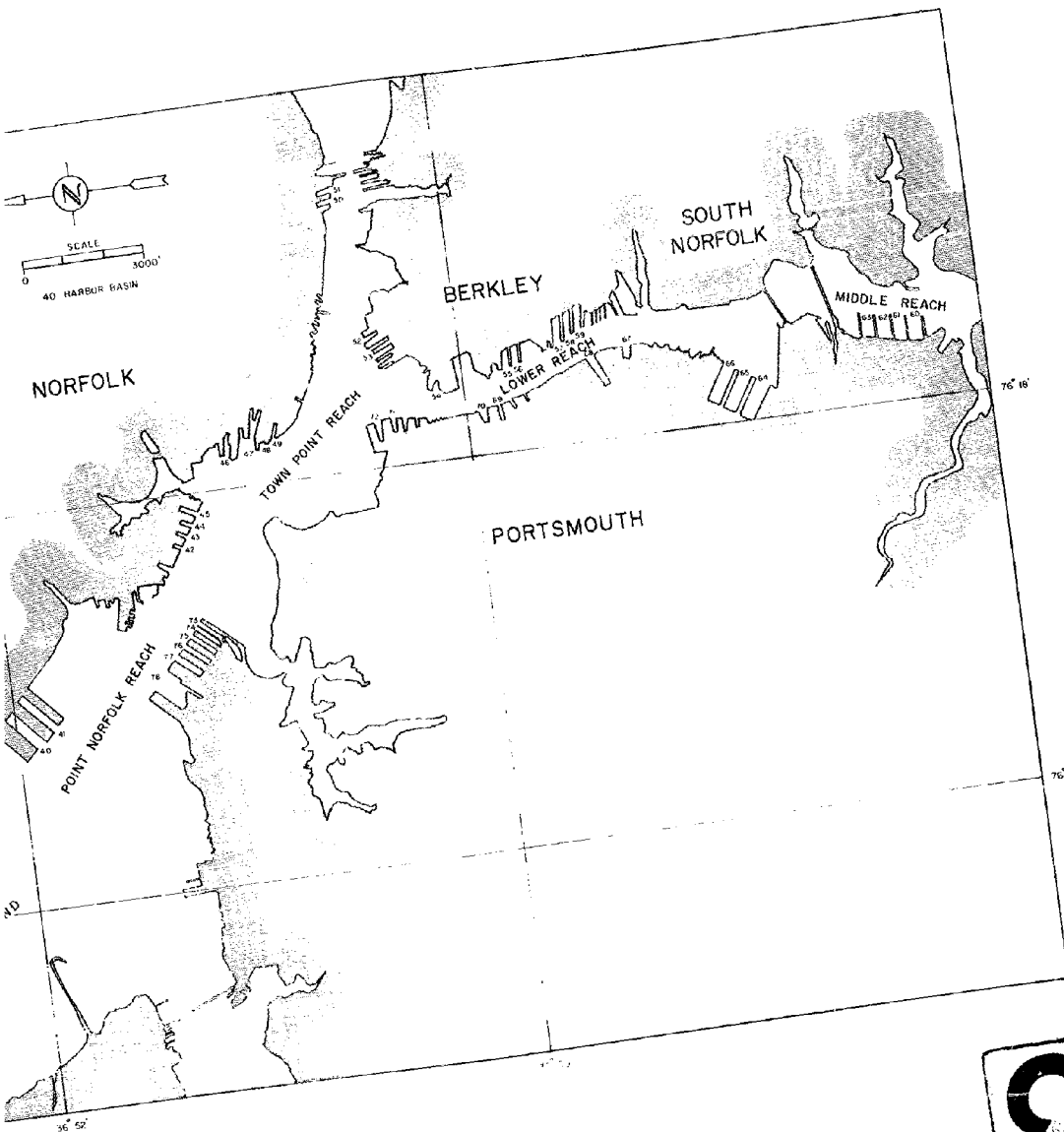


FIGURE 14  
HARBOR BASINS IN ELIZABETH RIVER

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TABLE 6

NATURAL OSCILLATORY PERIODS OF HARBOR BASINS IN THE  
LOWER REACH AND WEST SIDE OF PORT NORFOLK REACH IN  
ELIZABETH RIVER

Harbor Basin No.	Fundamental Period of Oscillation	First Harmonic	Second Harmonic	Lateral Oscillation Period
	Sec.	Sec.	Sec.	Sec.
54	59	17	10	8
55	89	25	14	14
56	115	32	18	18
57	114	33	18	13
58	102	29	17	12
59	112	32	18	13
60	121	32	19	27
61	118	32	18	26
62	118	32	18	26
63	118	32	18	26
64	142	40	23	17
65	142	40	23	17
66	142	40	23	17
67	62	18	10	7
68	129	40	22	10
69	72	21	12	8
70	79	21	12	15
71	88	26	15	9
72	104	31	17	9
73	142	44	25	8
74	142	44	25	8
75	142	44	25	8
76	142	44	25	8
77	142	44	25	12
78	267	70	40	67

Approximate arrival time after entering Chesapeake Bay:

1-1/4 hours.

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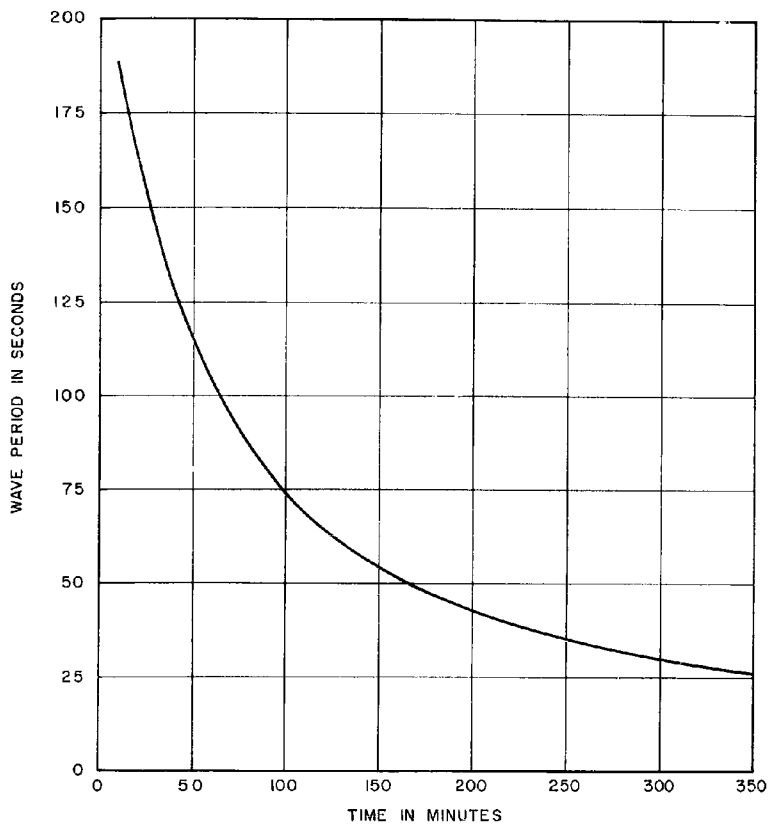


FIGURE 15  
WAVE PERIOD VERSUS TIME AT THE ENTRANCE OF  
HARBOR BASINS

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CHAPTER VI

ON THE ACCURACY OF THE OBTAINED RESULTS AND  
RECOMMENDATIONS FOR FURTHER INVESTIGATIONS

1. The theory of Kranzer and Keller and its application to the problem of the disturbance caused by a nuclear explosion is probably the best approach presently available to analyze the phenomenon. However, because of the simplifying assumptions necessary to relate the factual disturbance to the mathematical model some inaccuracy will be inherent in the input data as shown in Appendix I. It is believed that the presented results of this report, whatever the difficulty encountered in their calculation, are no less accurate -- and probably more accurate, than the input definition. It must be understood that this statement is not a criticism of the application of the Kranzer and Keller theory for defining the input because no better method presently exists.

Hence, if further investigations are considered, the particular problem of wave agitation due to explosion should be one of the problems of primary importance. This particular problem can be subjected to field experimentation. It can also be handled by further theoretical analysis. First a mathematical model for the impulse needs to be related to the yield of the nuclear explosion. Then the wave motion due to this impulse definition (which can be different from that chosen by Kranzer and Keller) can be calculated.

2. The Kranzer and Keller theory has been established for a horizontal bottom. The effect of a change of depth is also an important aspect

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which should require further analysis with the aim of determining the effect of the continental slope. This problem can be solved by modification of the Kranzer and Keller theory. In the case of a bottom slope, the Bessel functions should be replaced by the Mathieu functions. The problem can also be approached by analyzing the problem of long waves over a bottom slope. This work then should begin with a literature survey and synthesis of the existing theories on this subject. This problem can also be studied quite easily experimentally in a two-dimensional wave tank, specially adapted for the problem.

3. Now considering the problem of the wave deformation over the continental shelf, it can be said that there is presently no satisfactory theory which takes into account all the important factors such as the bottom friction and the vertical acceleration component (see Morrison and Crook (1954)). There is a need for a theory for long waves over a gentle slope where the vertical acceleration term and path curvature effects are taken into account. Then the determination of the friction coefficient must rely on careful field experiments. If one neglects the bottom friction term, a possible approach to the problem exists in neglecting the quadratic term  $w \frac{\partial u}{\partial z}$  in the Euler equation, which gives in the usual symbolism:

$$\frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} = -\frac{1}{\rho} \frac{\partial p}{\partial x}$$

along a horizontal axis OX. All quadratic terms along the vertical axis OZ could be neglected, namely  $u \frac{\partial w}{\partial x}$ ,  $w \frac{\partial w}{\partial z}$ , such that:

$$\frac{\partial w}{\partial t} = -\frac{1}{\rho} \frac{\partial (p + \rho g z)}{\partial z}$$

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In a word, the motion is nonlinear horizontally, but linearized vertically. However, it is pointed out that in the usual theory of long waves,  $\frac{\partial w}{\partial t}$  is neglected and the pressure is assumed to be hydrostatic. Such an assumption gives a wave deformation even on a horizontal bottom. This deformation is never observed in practice on a gentle slope such as that of the continental shelf. The neglect of  $\frac{\partial w}{\partial t}$  leads to the Earnshaw paradox discussed in Appendix VI.

Combining these two equations with the continuity and boundary conditions ( $w_b = S u_b$  at the bottom of slope  $S$ ), a solution can be found by assuming:

a.  $w$  is linearly distributed from the bottom (at depth  $d$ ) to the free surface (at elevation  $\eta$  above the mean water level):

$$w = w_b + \frac{d+z}{d+\eta} \frac{\partial \eta}{\partial t}$$

b. In first approximation  $u$  can be assumed to be a constant over a vertical:  $u = u(x, t)$ . Then, to second and third approximation:

$$u = u(x, t) \left[ 1 + A(d+z) + B(d+z)^2 \right]$$

The problem then consists of determining the coefficients  $A$  and  $B$ . A bottom friction term can also be added, but a good determination of the friction factor would require analysis of field results.

4. A theoretical approach to wave run-up has been developed in this report (Appendix VI). It can be said that the run-up can now be calculated theoretically. However, in order to obtain practical results from the theory, further studies remain to be done, namely the calculation of the wave run-up for various slopes and various friction coefficients. This

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can be done after development of a computing program which would be necessary because of the length of the calculations. Some experiments in a specially built wave flume for long waves will also be of great value in order to verify the theory presented and give some information on the friction coefficient. Analysis of field tests will also yield a major contribution which will complement the theoretical research.

5. A new numerical method for studying the problem of two-dimensional waves in bays and estuaries has been developed in this report (Appendix VII). It has also been applied to the lower Chesapeake Bay. However, this method can render more information after an extension of the calculations over a larger area. The accuracy can be improved by using smaller intervals.

One theoretical problem which must also be investigated in connection with the propagation of two-dimensional waves in estuaries is the problem of successive diffraction of nonlinear long waves. Practically nothing is known in hydrodynamics on nonlinear diffraction. This study can be approached by the "method of wave derivatives" as presented in Appendix VII. However, it would be desirable to improve the presented method by taking into account the effects due to vertical acceleration and path curvatures as it is prepared in Appendix VI. Otherwise, an unrealistic wave profile deformation will be obtained in view of the Earnshaw paradox.

6. The problem of resonance in a harbor basin due to periodic waves has already been subjected to extensive theoretical studies (McNown (1951) ),

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Le Méhauté (1954, 1961), Miles and Munk (1961) ). However, the waves due to explosion are not periodic. Hence the problem of harbor response to a nonperiodic wave must also be analyzed. It is a problem similar to the problem of harbor response to a wave spectrum.

The determination of the duration for establishing a given state of agitation under a periodic wave, or the damping of this agitation when the outside agitation ends, are similar problems. They all must be solved by equations of energy. Systematic experiments in a laboratory will be of great help in solving these problems.

The problem of the response of moored ships to seiche motion is also one of the topics which can be considered for further analysis (Wilson (1951, 1959) ). Then it can be reasonably thought that a solution of moorings could be found which would avoid the danger of wrecking the ships by collision against the quays or against each other.

7. Finally, full use can be made of scale model technology and the analog computer. An analog computer is particularly interesting for studying the propagation of linear periodic waves. However, one cannot expect too much accuracy by this method because long waves are nonlinear and the problem of bore propagation -- which is so important for this problem -- is essentially a nonlinear phenomenon. A scale model automatically takes care of nonlinear effects and faithfully reproduces the bore phenomenon. The bottom friction can even be adjusted experimentally.

Considering all the problems involved in this study and the size of the area to be represented, it would be particularly convenient to have several different models at various scales. A first model at a relatively

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small horizontal scale (say  $\lambda = 1/2,000$ ) will represent the lower Chesapeake Bay. A wave paddle could be set up at the entrance of Chesapeake Bay to reproduce waves of limit steepness with a wave period which is slowly decreased with time according to the theory of waves due to an explosion as shown by Fig. 15. This could be achieved by a piston wave paddle in front of a small gentle slope. Such a model would need to be distorted in order that the depth would not be smaller than 1" and averaging 2". Hence the scale model would have a vertical scale of about  $\mu = 1/200$ , i.e. a distortion of 10. The time scale  $\tau = \frac{\lambda}{\sqrt{\mu}}$  is 0.007. Hence an 80 second wave period would be represented by a 0.48 second wave, which is above the limit of capillarity waves. Because of this distortion, this model would have a steeper slope than encountered in the field. Hence the reflection coefficient may have to be adjusted by use of screen and roughness on the concrete of the model.

The area to be represented would be roughly 150,000 x 100,000 ft. Efficient wave absorbers would have to be located at the limit of the model because the northern part of Chesapeake Bay would not be represented. Hence, this indicates a 75 ft. x 50 ft. model.

It is evident that all the numbers presented in this discussion of model studies are only for indicating the order of magnitude of the model size. Such a model is possible even though unusual, but so is the problem under study. It can reasonably be considered as the best tool for solving the problem within the estuary. A model study as proposed would yield the following results:

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a. The complete wave pattern in the Chesapeake Bay entrance would be faithfully reproduced, indicating bore phenomena and local areas of high amplitude, short-crested waves of particular interest for the Chesapeake Bay bridge-tunnel structure.

b. It is comparatively easy to make modifications on the model to determine the most efficient and economical solutions to reduce damage. The best solutions can be found by relative comparison of the results, whatever the exactness of their absolute value.

c. Areas near the shoreline or off-shore which have their own special problems will be indicated and more detailed investigations can be concentrated on these places.

d. Use of gilsonite or other movable material will indicate areas where serious deposition of silt will be likely to occur and also possibly indicate areas subject to erosion. (Le Méhauté and Collins (1961))

The problem of resonance in a harbor requires a larger scale model, but a smaller area needs to be represented. Such a model would follow after the previous one once the wave amplitude in the main navigation channels is known. This model would indicate the time of response of the harbor under consideration and the amplitude of the resonance motion.

These two models would be built after a small preliminary theoretical study on distortion, determination of the scales developed from an adaptation of the ideas already expressed by Biéssel and Le Méhauté (1954, 1955). Since many models for long waves have been built following the presented rules: Alger, Table Bay, Funchal, and so on, it seems that they have been successfully exploited.

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8. Finally, this problem cannot be handled with complete satisfaction until some field tests are performed to substantiate theoretical research and laboratory experiments. Then a careful analysis of field records should give a complement of information of great value. For this purpose it would be desirable to record wave trains not only near the shoreline, but also at the edge of the continental shelf and even offshore at the continental slope. For this purpose a wave recorder needs to be developed because the present long wave recorders do not operate satisfactorily in deep water. Such a problem is difficult and is a challenge requiring some new ideas.

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LIST OF SYMBOLS

a	Horizontal length of the continental slope
A	Function of $md$ (Eq. 2); also numerical coefficient
b	Width of harbor basin; also distance between wave rays or orthogonals
$b_1$	Distance between orthogonals in nearshore zone
$b_b$	Distance between same orthogonals at breaking inception
B	Numerical coefficient
C	Wave (phase) velocity
$C_1$	Value of C in deep water off the continental shelf
$C_2$	Value of C over the continental shelf
$C_h$	Chezy coefficient
d	Water depth
$d_1$	Value of d off the continental shelf; also at any one position; also depth at which bottom slope is 0.01
$d_2$	Value of d over the continental shelf; also at any second position
$d_b$	Value of d at breaking inception
f	Friction coefficient ( $= g/C_h^2$ ).
g	Acceleration due to gravity
H	Wave height from crest to trough
$H_b$	Value of H at breaking inception
l	Length of a harbor basin
ln	Natural logarithm (to base e)
L	Wave length between crests

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$L_1$	Value of $L$ off the continental shelf
$L_2$	Value of $L$ over the continental shelf
$L_n$	Resonant value of $L$ to excite basin oscillation of n-th mode
$m$	Wave number ( $= 2 \pi / L$ )
$m_1$	Value of $m$ off the continental shelf
$m_2$	Value of $m$ over the continental shelf
$n$	Manning's friction coefficient; also positive integer (0, 1, 2, 3...) representing modes of oscillation; also numerical exponent
$N$	Number of waves
$p$	Fluid pressure
$R$	Radial distance from origin of disturbance; also run-up elevation at shoreline
$R$	Increment of radial distance
$S$	Bottom slope of sea bed
$T$	Wave period
$T_n$	Period of resonance of harbor basin in n-th mode.
$u$	Component of water velocity in horizontal direction
$u_s$	Value of $u$ at the shoreline
$V$	Group velocity of waves ( $= C$ for shallow water waves)
$w$	Component of water velocity in vertical direction
$x$	Variable horizontal distance in a coordinate system
$z$	Variable vertical distance
$\alpha$	Coefficient of transmission (of wave height) over the continental slope
$\beta$	Ratio ( $= L / \lambda$ )

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$\gamma$	Euler's constant (= 0.577215...)
$\Delta \Gamma$	Shoaling coefficient
$\eta$	Elevation of free surface above still water level
$\lambda$	Horizontal linear scale for a model
$\mu$	Vertical linear scale for a model
$\xi$	Correction factor for computing oscillation periods
$\pi$	Universal constant (3.14159...)
$\rho$	Mass density of fluid
$\Sigma$	Summation symbol
$\Omega$	Mass transportation due to a solitary wave

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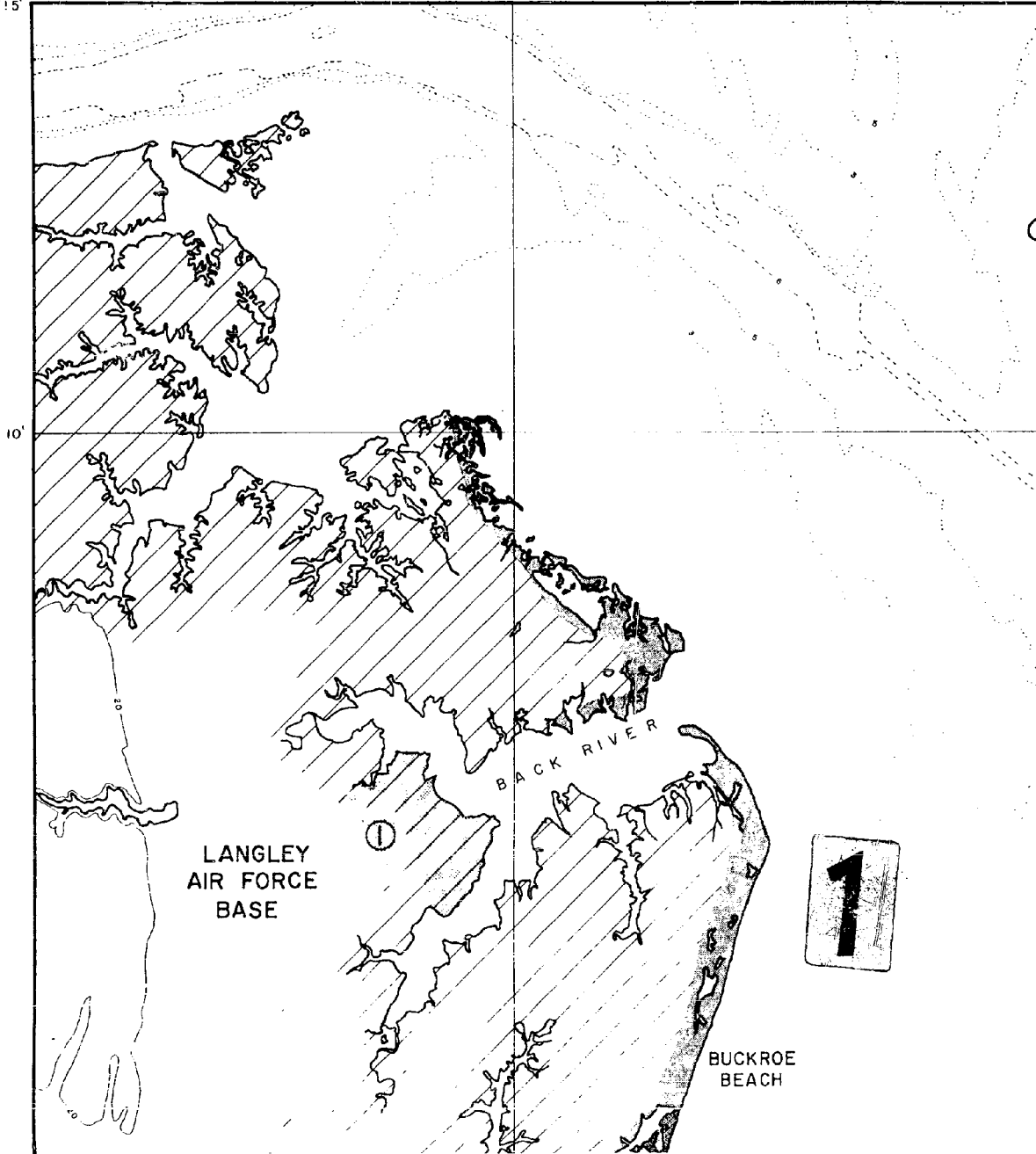
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<p>National Engineering Science Co., Pasadena, California. INVESTIGATION OF LONG WAVES AND THEIR EFFECTS ON THE COASTAL AND HARBOR ENVIRONMENT OF THE LOWER CHESAPEAKE BAY (U), by NESCO Staff, Vol. I CONFIDENTIAL, 122 p. incl. figs., tables and refs., NESCO Tech. Rept. No. SN-93, Contract No. DA-49-146-XZ-151.</p> <p>This report deals with the effect of tsunami-type gravity waves caused by huge underwater disturbance, on the shoreline environment. In particular, the cases of three different power disturbances off the United States Atlantic Coast and their effects in the lower Chesapeake Bay are analyzed. It is shown that because of the natural protective action of the continental shelf, a maximum possible amount of damage is reached for a given power. If a disturbance is due to a larger power, the difference of energy is dissipated over the continental shelf by spilling breakers. The flooding areas, dynamic destruction and harbor resonance due to this optimum power disturbance are calculated and/or evaluated.</p>	<p style="text-align: center;">CONFIDENTIAL</p> <p>Hydrodynamics of Waves</p> <ol style="list-style-type: none"> <li>1. Tsunami Wave</li> <li>2. Explosion</li> <li>3. Harbor Resonance</li> </ol> <p>I. Le Mehaute, B. II. Breitschneider, C.</p>	<p style="text-align: center;">CONFIDENTIAL</p> <p>Hydrodynamics of Waves</p> <ol style="list-style-type: none"> <li>1. Tsunami Wave</li> <li>2. Explosion</li> <li>3. Harbor Resonance</li> </ol> <p>I. Le Mehaute, B. II. Breitschneider, C.</p>
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76° 25'

20'

37° 15'



LANGLEY  
AIR FORCE  
BASE

①

BACK RIVER

1

BUCKROE  
BEACH

CONFIDENTIAL

10'

76°

C  
H  
E  
S

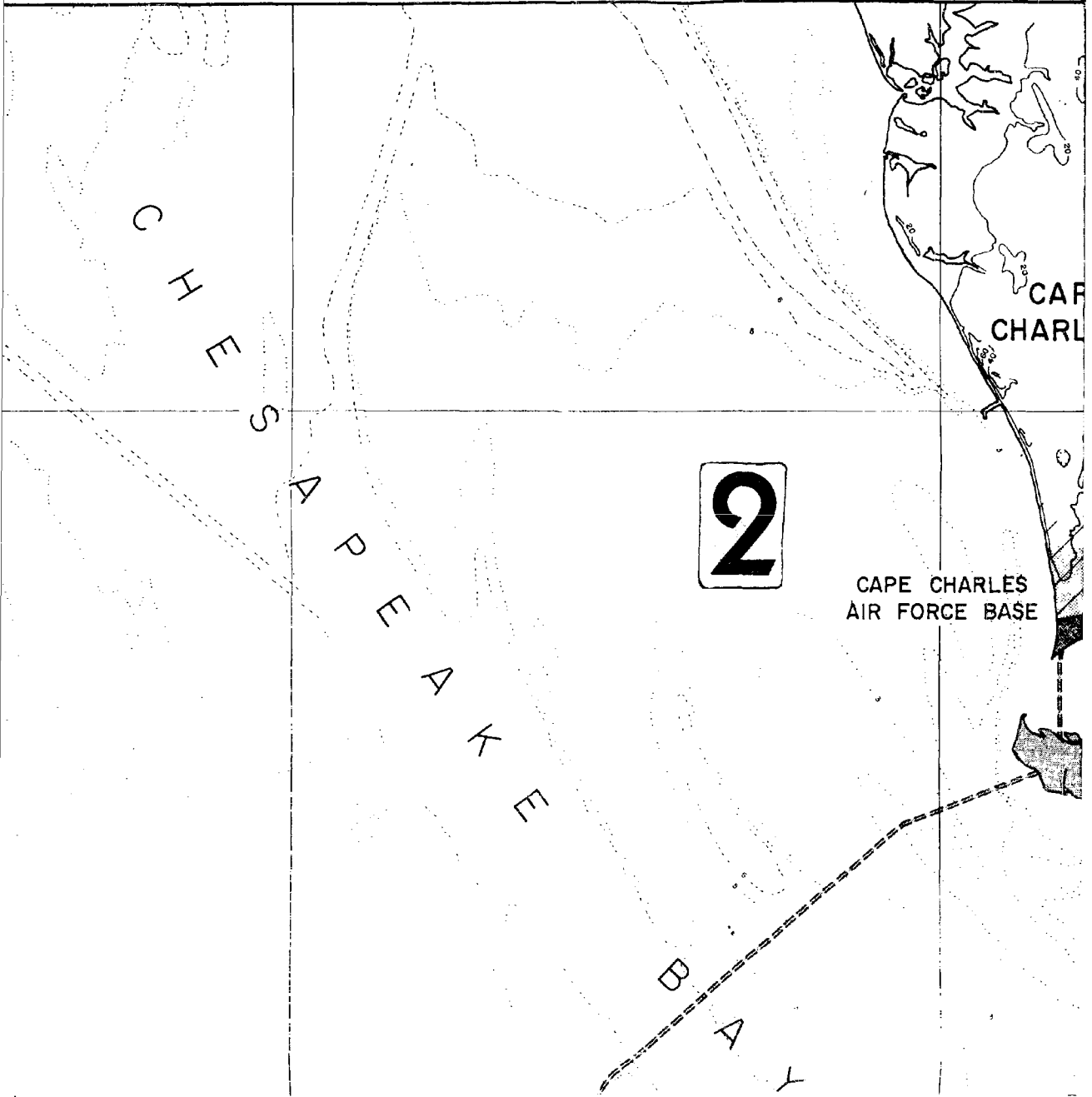
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P  
E  
A  
K  
E

2

CAPE  
CHARL

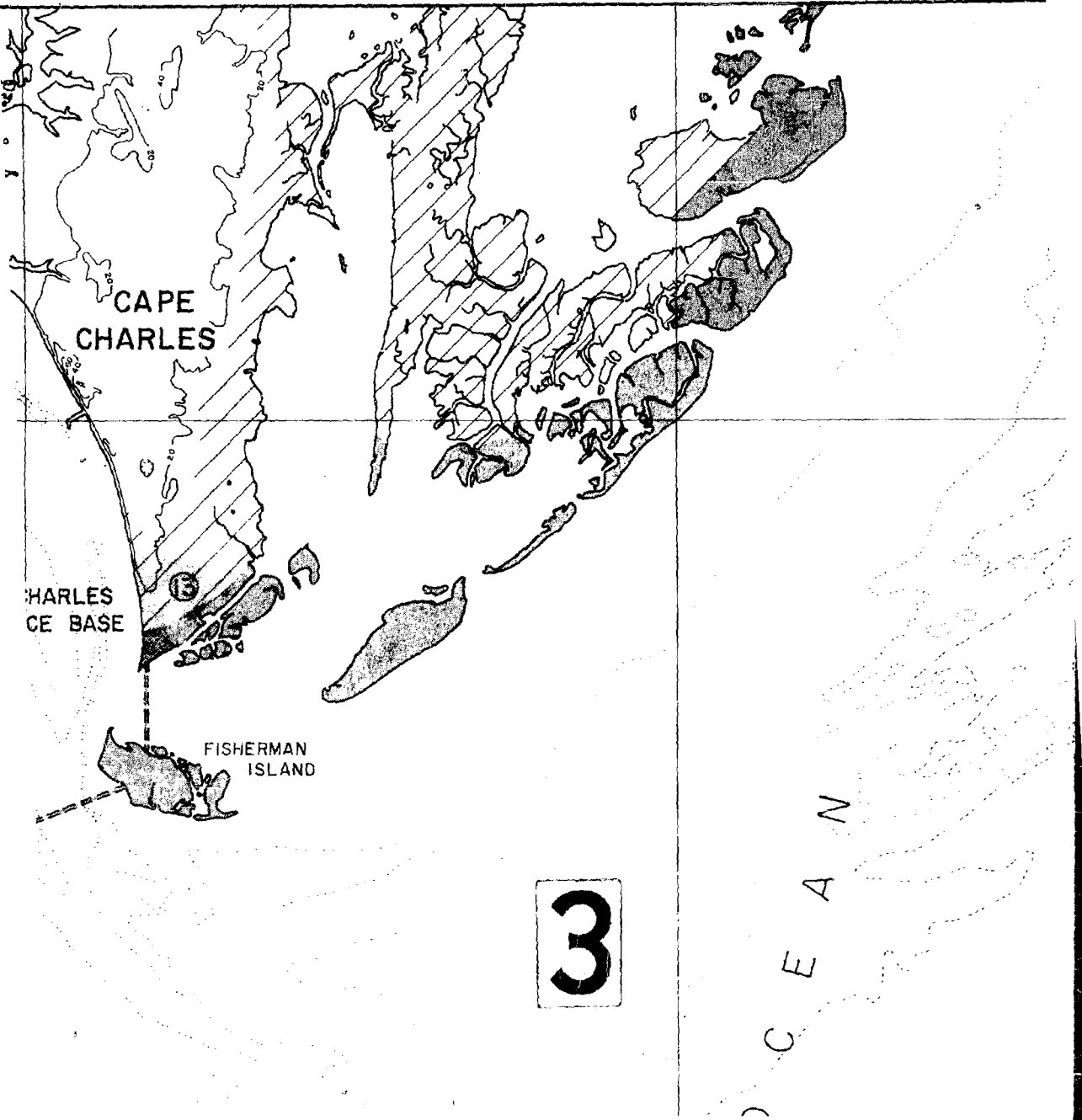
CAPE CHARLES  
AIR FORCE BASE

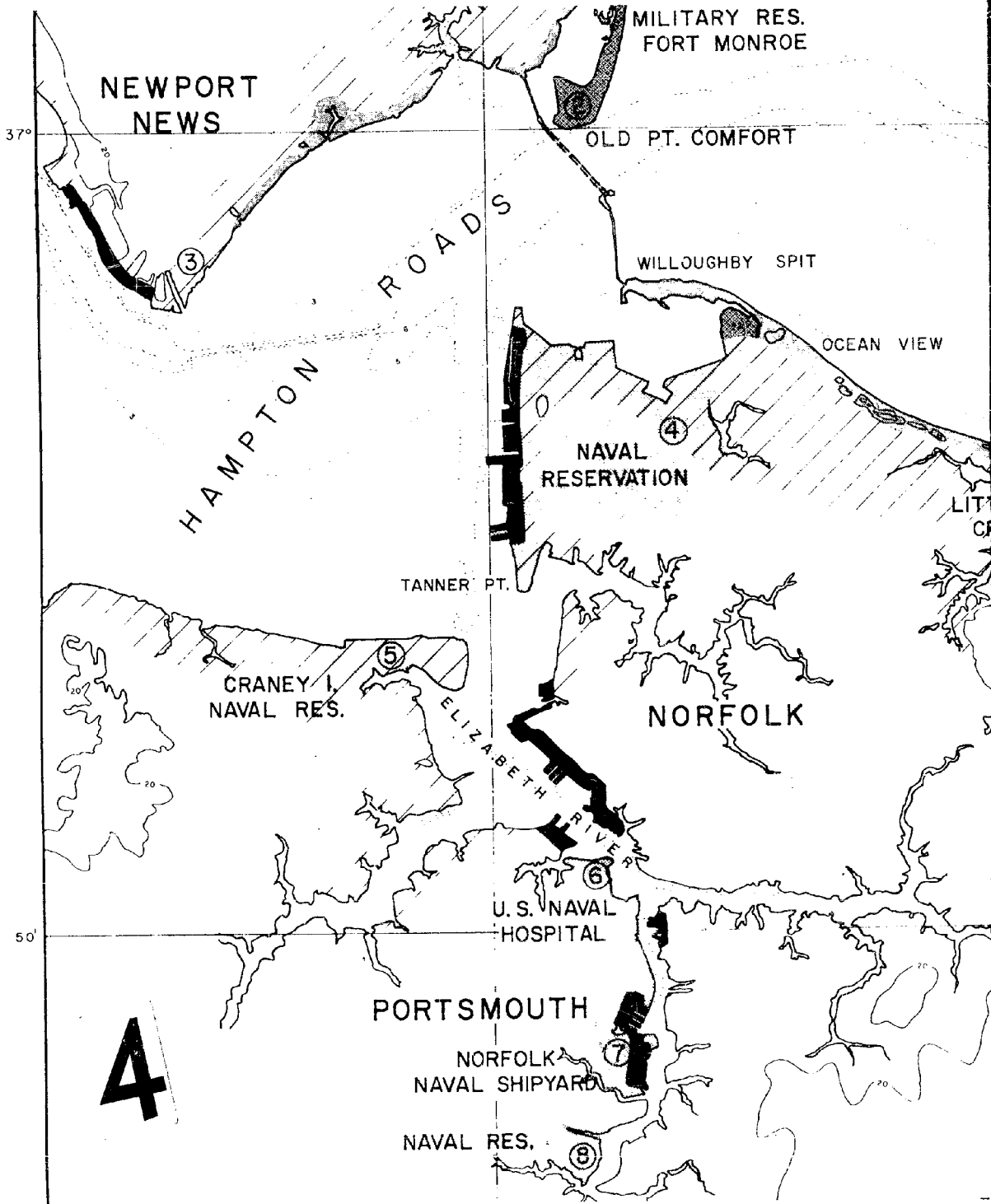
B  
A  
Y



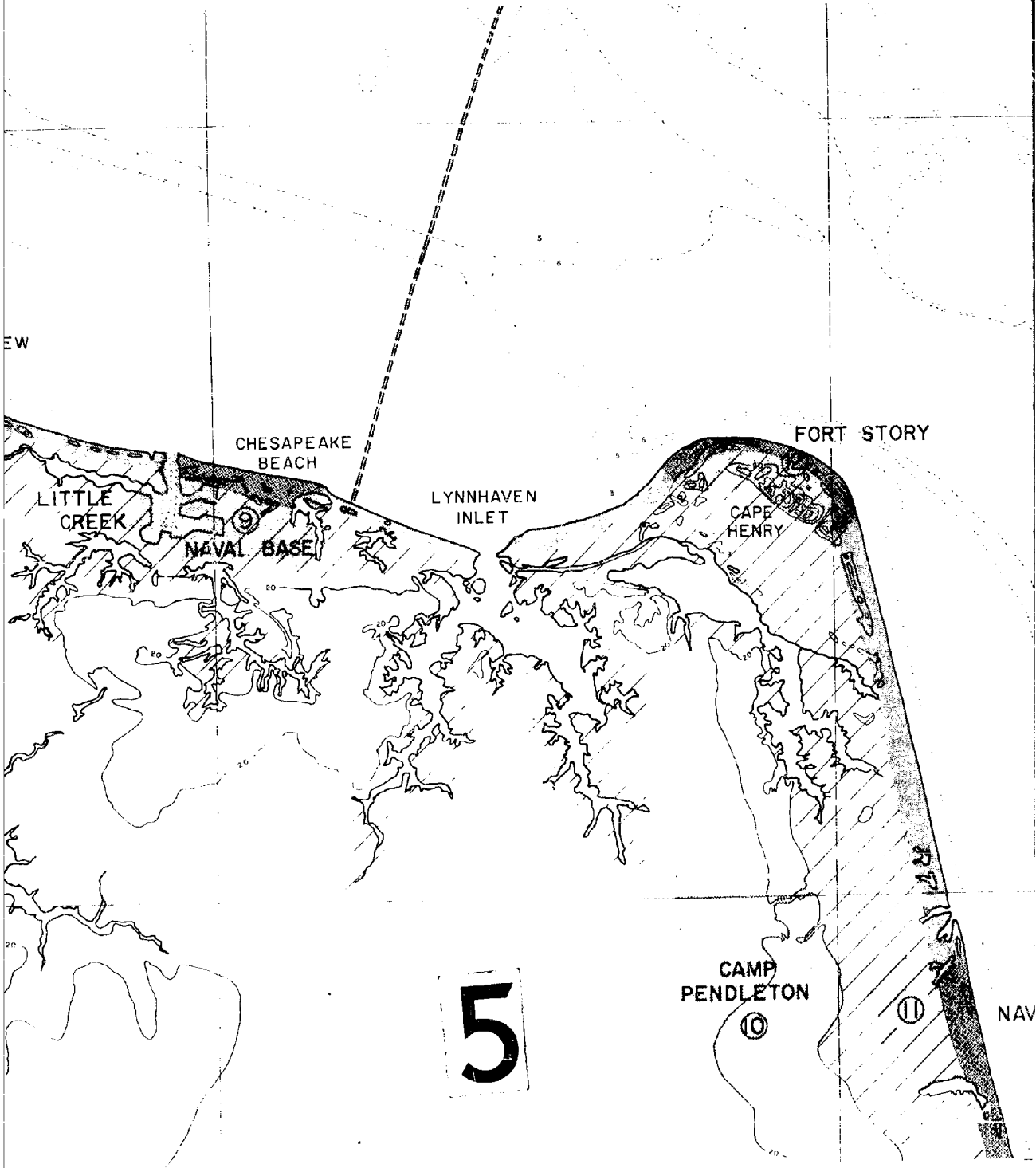
76°

50'





4



EW

CHESAPEAKE  
BEACH

LITTLE  
CREEK

NAVAL BASE

LYNNHAVEN  
INLET

FORT STORY

CAPE  
HENRY

CAMP  
PENDLETON

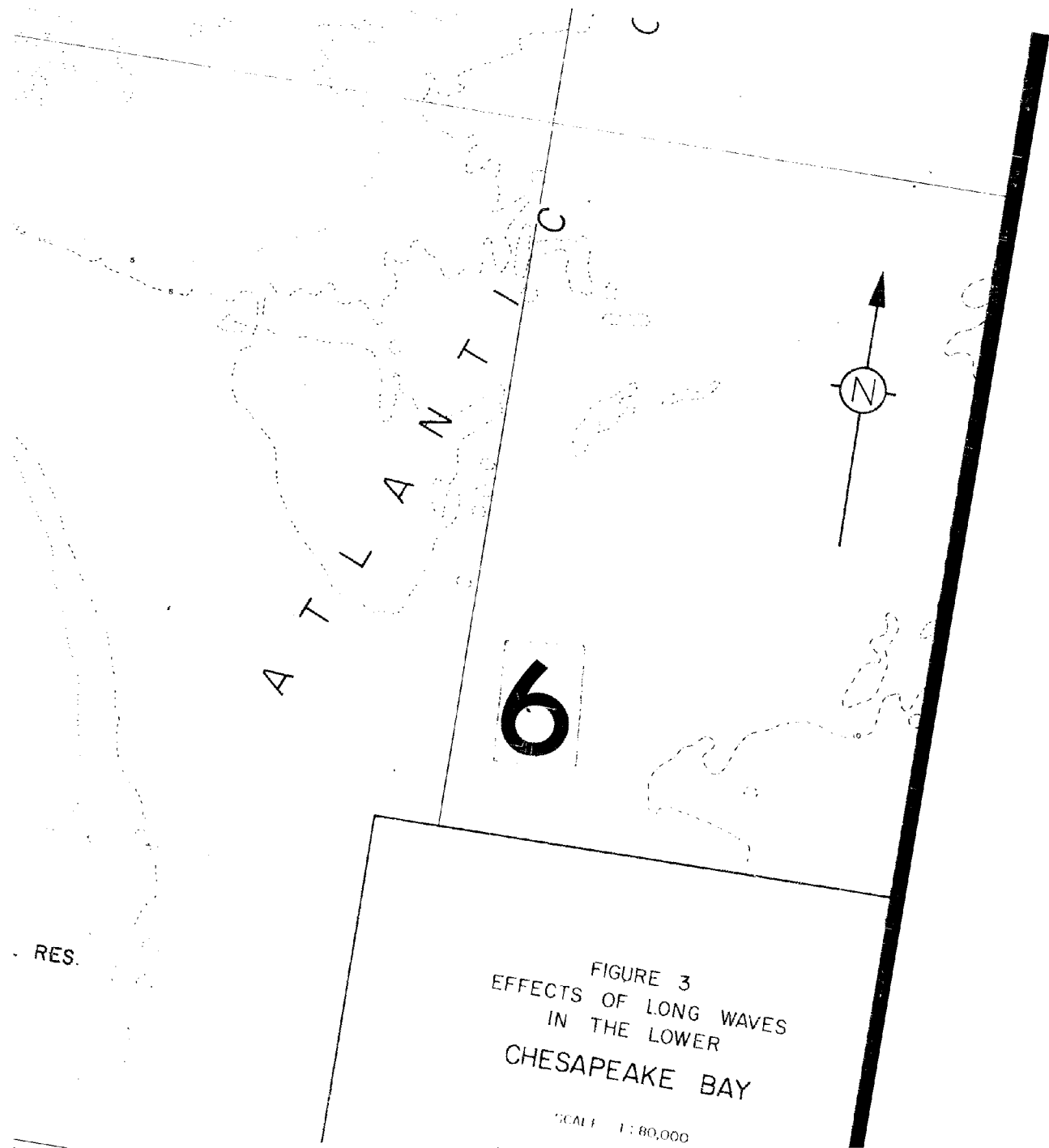
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NAV

NAV

10

11



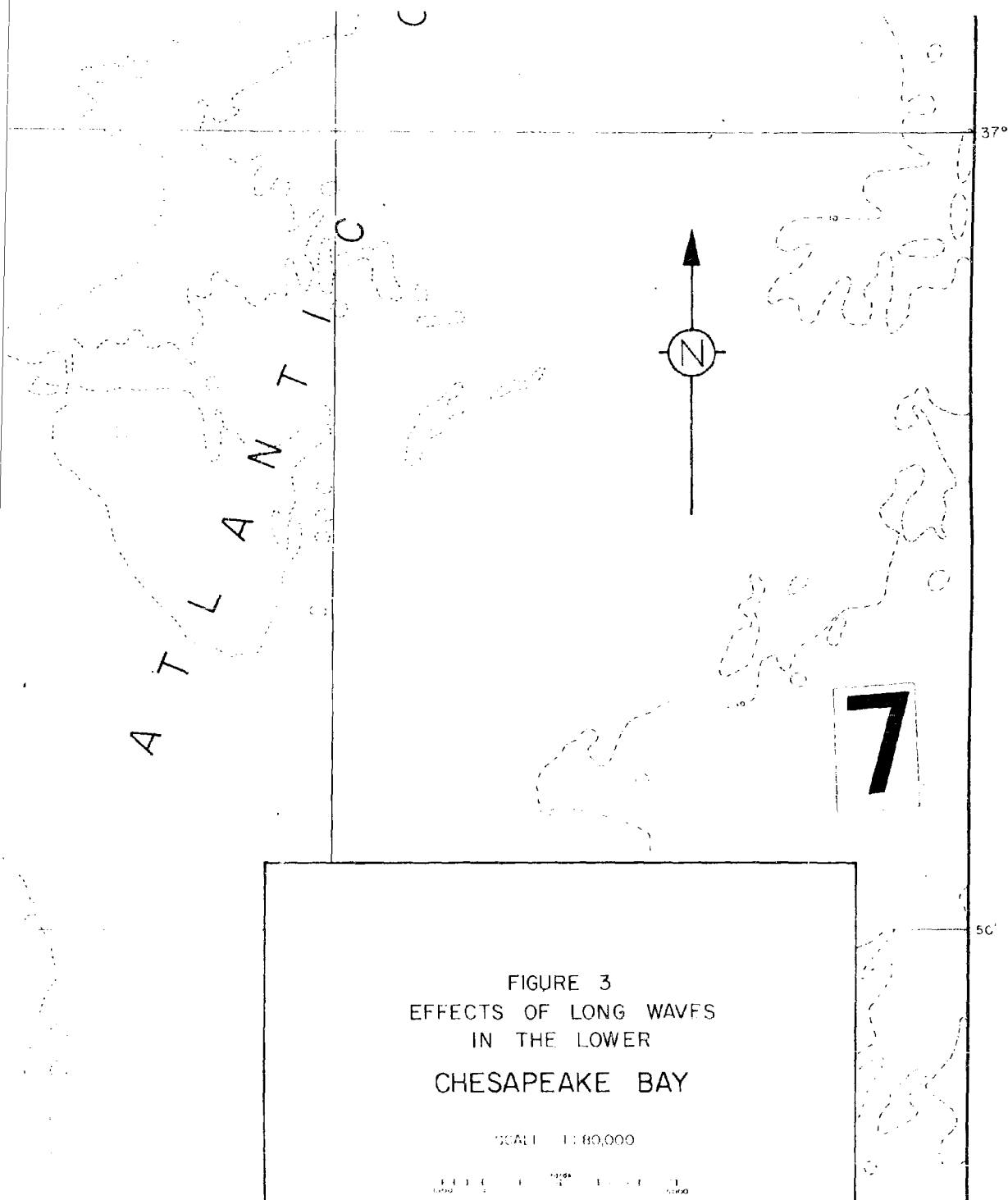
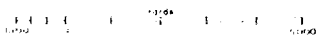
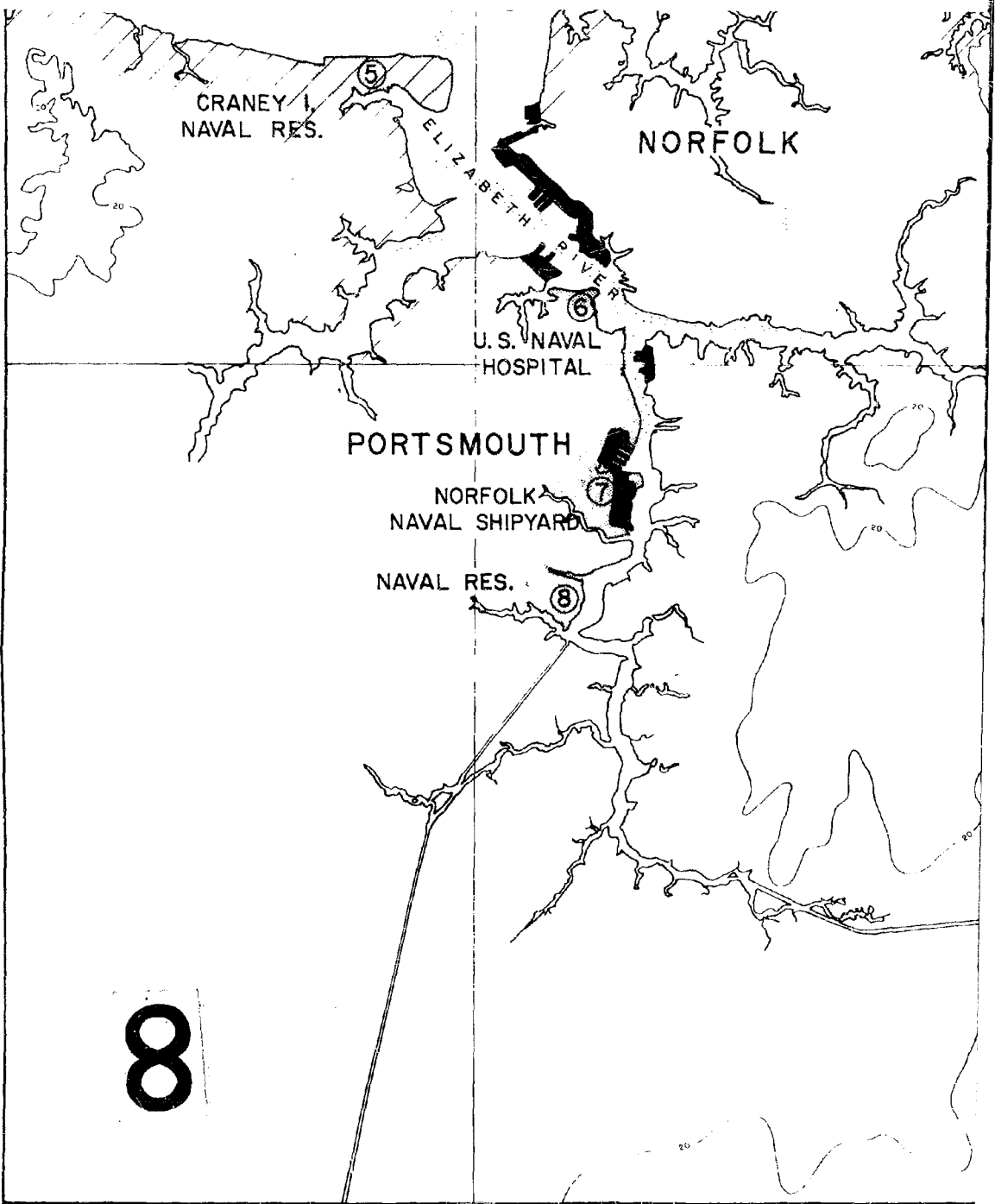


FIGURE 3  
EFFECTS OF LONG WAVES  
IN THE LOWER  
CHESAPEAKE BAY

SCALE 1:80,000





CRANEY I.  
NAVAL RES.

NORFOLK

PORTSMOUTH

U. S. NAVAL  
HOSPITAL

NORFOLK  
NAVAL SHIPYARD

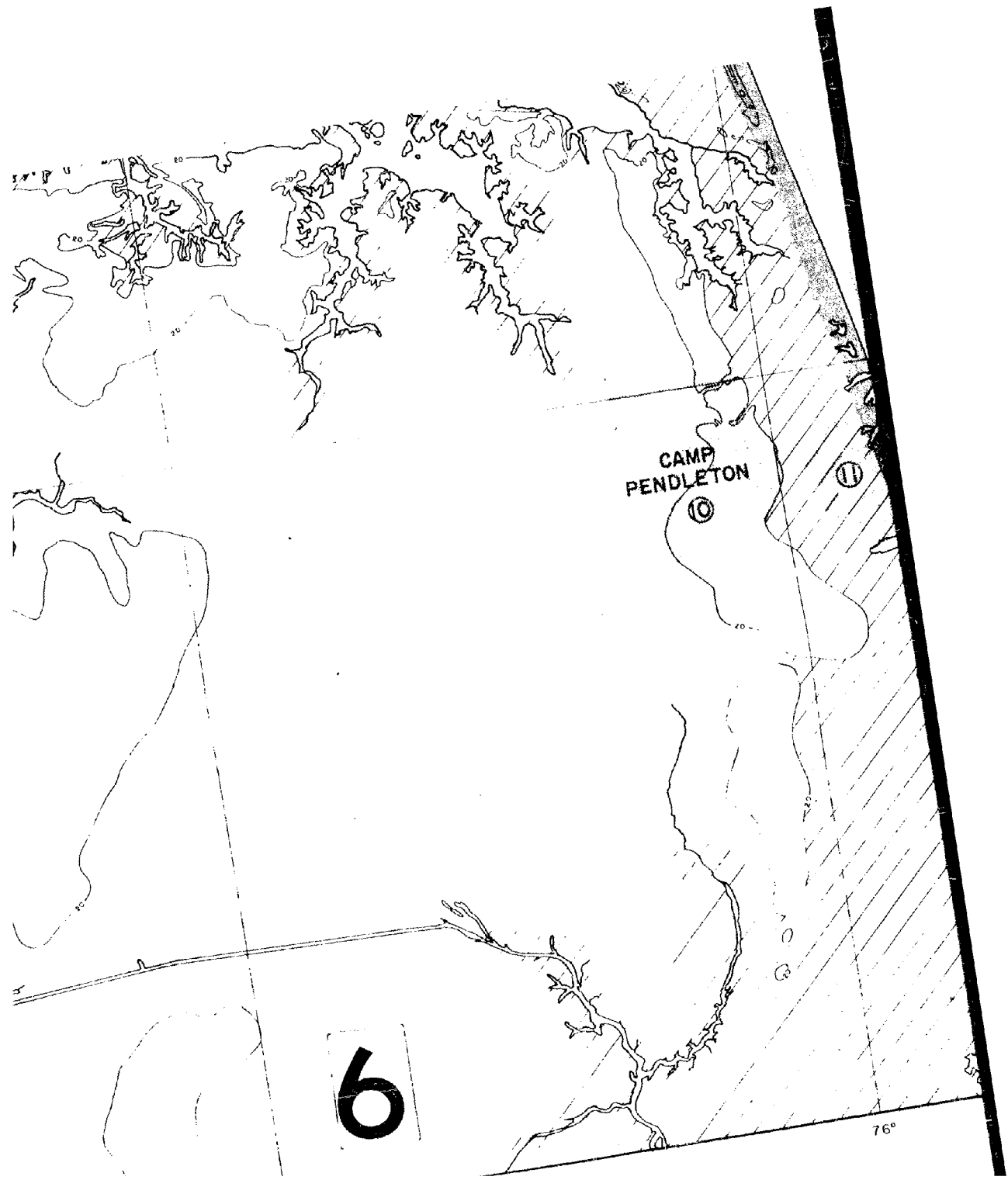
NAVAL RES.

8

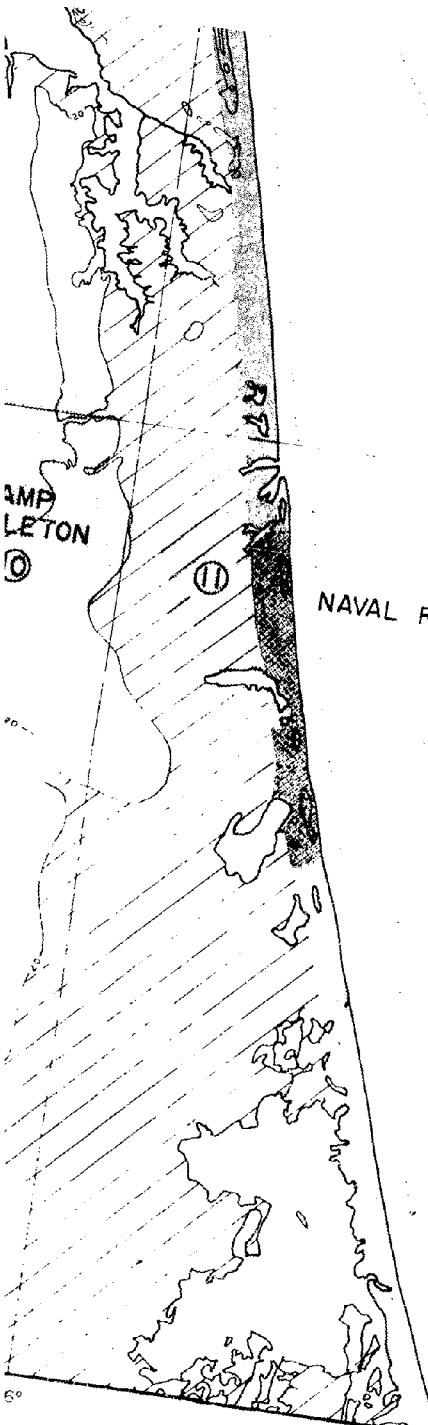
36° 40'

76° 25'

20'



A T L



NAVAL RES.

FIGURE  
EFFECTS OF L  
IN THE  
CHESAPEA

SCALE 1:

1:1000

Ocean Contours

LEGEND

- COAST LINE
- CONTOURS - 20
- █ MILITARY INS
- RESONANCE C
- ESTIMATED D
- PROBABLE F

NATIONAL ENGINEERING

**NESCO CONTROL**

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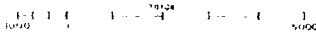
7

A T L

FIGURE 3  
EFFECTS OF LONG WAVES  
IN THE LOWER  
CHESAPEAKE BAY

8

SCALE 1:80,000



Ocean Contours in Fathoms

LEGEND

- COAST LINE
- - - CONTOURS - 20 FT. INTERVALS
- MILITARY INSTALLATIONS
- RESONANCE OF HARBOR BASINS
- ESTIMATED DYNAMIC DESTRUCTION
- ▨ PROBABLE FLOODED AREA

NATIONAL ENGINEERING SCIENCE COMPANY

**NESCO CONTROL**  
No 1500 Copy 94

JANUARY 1963

50'

50'

36° 40'  
75° 40'

CONFIDENTIAL

CONFIDENTIAL



## Defense Threat Reduction Agency

45045 Aviation Drive  
Dulles, VA 20166-7517

CPWC/TRC

May 6, 1999

MEMORANDUM FOR DEFENSE TECHNICAL INFORMATION CENTER  
ATTN: OCQ/MR WILLIAM BUSH

SUBJECT: DOCUMENT REVIEW

The Defense Threat Reduction Agency's Security Office has reviewed and declassified or assigned a new distribution statement:

- AFSWP-1069, AD-341090, STATEMENT A
- ✓ DASA-1151, AD-227900, STATEMENT A
- DASA-1355-1, AD-336443, STATEMENT A
- DASA-1298, AD-285252, STATEMENT A ✓
- DASA-1290, AD-444208, STATEMENT A ✓
- DASA-1271, AD-276892, STATEMENT A ✓
- DASA-1279, AD-281597, STATEMENT A
- DASA-1237, AD-272653, STATEMENT A
- DASA-1246, AD-279670, STATEMENT A
- DASA-1245, AD-419911, STATEMENT A
- DASA-1242, AD-279671, STATEMENT A
- DASA-1256, AD-280809, STATEMENT A
- ✓ DASA-1221, AD-243886, STATEMENT A
- DASA-1390, AD-340311, STATEMENT A
- DASA-1283, AD-717097, STATEMENT A
- DASA-1285-5, AD-443589, STATEMENT A
- DASA-1714, AD-473132, STATEMENT A
- DASA-2214, AD-854912, STATEMENT A
- DASA-2627, AD-514934, STATEMENT A
- DASA-2651, AD-514615, STATEMENT A
- ~~DASA-2536, AD-876697, STATEMENT A~~
- DASA-2722T-V3, AD-518506, STATEMENT A
- DNA-3042F, AD-525631, STATEMENT A
- DNA-2821Z-1, AD-522555, STATEMENT A

*RD waiting for reply*

*FRD*

*OK*

If you have any questions, please call me at 703-325-1034.

*Arduith Jarrett*

ARDITH JARRETT  
Chief, Technical Resource Center