GUIDELINES FOR THE REVIEW AND EVALUATION OF BREAKWATERS: DESIGNED A PROTECTIVE STRUCTURE FOR OFFSHORE NUCLEAR POWER PLANTS

by

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PREFACE

The present document, prepared under the sponship of the U.S. Nuclear Regulatory Commission, Contract No. E(11-1)-2707 provides guidelines for the review and evaluation of breakwaters designed as protective structures for offshore nuclear power plants. To serve the intended purposes, as a regulatory guideline, the content is confined mainly to safety considerations rather than economic criteria generally associated with engineering projects of this kind.

The main text consists of two parts--performance specifications and procedural specifications; both of which are organized in codified form in the hope to aid in clarity and quick reference. Expanded presentations concerning "state-of-the-art" design and testing procedures are contained in the appendices.

Currently, there exists no code for breakwater design within and outside the United States and the present work does not purport to be the first of its kind either. The usage of this report should be restricted to aid in staff review by the NRC personnel of breakwater designed specifically for nuclear power plant protection. Many of the proposed criteria are untested and the document should be used with discretion.

Finally, it is hoped that this work can serve as a baseline for the eventual development of a formal design code. Such a task should be undertaken by a code committee and the final document should be radified by both the regulatory bodies and the professions.

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PART I

PERFORMANCE SPECIFICATION

CHAPTER 1 DEFINITION

1.1 Design Criteria

1.1.1 General--As stipulated by the Nuclear Regulatory Commission, the breakwaters designed as the main protective structure for offshore nuclear power plants should be treated as Category I structures. Therefore, they should be designed to withstand the effects of natural phenomena such as storms, earthquakes and tsunamis without loss capability to perform their safety functions. The design bases should conform with, or be compatible with, the Regulatory Guides issued by the Nuclear Regulatory Agency, in particular, those related to Environmental and Siting.

1.1.2 Category--Design Criteria should be defined for the following conditions:

A. Construction condition--Under this condition, the construction could proceed with proper quality control and low probability of causing damage to the surroundings.

B. Operating Basis--Under this condition, the plant can operate without interruption.

C. Design Basis--Under this condition, the plant can be placed into and maintained in a safety shutdown condition.

D. Accident--Under this condition, the plant shall not be directly affected and can operate without interruption.

1.2 Design Environmental Factors

Two kinds of descriptions--design events and design parameters--should be used to define design environmental factors. The combinations of events with events, events with parameters, or parameters with parameters define the design conditions. The following requirement is recommended.

1.2.1 Design Events:

A. Operating Basis Storm--(OBS)--synthetic histogram of wind condition including strength, duration and direction, hypothetical histogram of combined wave and water level changes.

B. Design Basis Storm--(DBS)--synthetic histogram of wind condition including strength, duration and condition, hypothetical histogram of combined wave and water level changes. The waves can be described in terms of time history, deconvolutes from wave spectrum (random) or in terms of maximum possible waves (MPW) at the corresponding water levels (regular).

C. Safe Shutdown Earthquake--(SSE)--design time history in conformance with NRC Regulatory Guide.

D. Operating Basis Earthquake--(OBE)--same as (SSE).

E. Limiting Case Accident--(LCA)--limiting case ship collision as dictated either by the maximum possible water depth or the expected largest ship. Cases of military class and commercial class should be considered separately.

F. Operating Basis Accident--(OBA)--hypothetical ship collision event compatible with OBS both in environmental condition and degree of risk.

G. Tornado--synthetic histogram including strength, radius of inference, and advancing speed.

H. Tsunami--direction and magnitude.

1.2.2 Design Parameters:

A. Wind--speed (fastest mile or fixed duration), direction and duration

W_1 (normal condition, annual mean)

W_2 (OBS)

W_3 (DBS)

B. Waves--height, period and direction

V_1 (normal condition, annual mean)

V_2 (OBS)

V_3 (DBS)

V_4 (DLW) - depth limited wave

C. Water Level

L_1 mean sea level

L_2 highest level during OBS

L_3 lowest level during OBS

L_4 highest level during DBS

L_5 lowest level during DBS

D. Current

C_1 Normal Current

C_2 Maximum Current During OBS

C_3 Maximum Current During DBS

E. Swell

F. Ice

1.3 Definition of Damages

At present, there is no standard practice on the description of damage for rubble mound breakwaters. It is the responsibility of the applicant to clearly define, in a quantitative manner, the damage criteria and related terminologies that have been used for the specific design.

For breakwaters designed as the protective structures for nuclear power plants, the current practices of defining only armor layer stability should not be considered sufficient. The damage criteria should contain, at minimum, the information on the mode of damage, the area of damage, the extent of damage and the safety margin.

1.3.1 Damage Modes--the following terminologies are commonly used to describe the mode of damage:

A. Armor Unit Rocking--the unit moves a distance less than the overall length of the unit.

B. Armor Unit Damaged--the unit loses material of not more the 1/3 of its original weight but maintains roughly the original form.

C. Armor Unit Breakage--the unit loses more than 1/3 of its original weight or loses its original form.

D. Armor Unit Displaced--the unit moves a distance greater than the overall length of the unit.

E. Armor Layer Rifting--armor units become completely disassociated with each other thus losing the interlocking ability.

F. Sublayer Exposure--Ten or more units in a cluster in the sublayer is completely exposed without primary armor unit protection.

G. Core Exposure--the unit area of core exposure exceeds the projected area of five secondary armor units clustered together.

H. Core Leaching--core material leaches out without apparent damage at the secondary or primary layers.

I. Undermining--core or foundation material leaches out through a passage (or passages) extending below foundation level.

J. Scouring--erosion of material occurs at the toe of the breakwater.

K. Settlement--breakwater loses its total height without appreciable change of its shape.

L. Slope Deformation--breakwater changes its shape due to causes other than shear failure.

M. Slope Instability--sudden loss of shape material occurs due to shear failure.

N. Section Breaching--a section of the breakwater is completely washed out with an opening exceeding two armor units wide and one armor unit deep.

1.3.2 Damage Zones--The loadings on breakwaters are known to be nonuniform; certain areas are subject to severer loading conditions than the others and, thus, are more vulnerable to sustain damage. On the other hand, certain areas, if damaged, are more critical to structure safety than the others. The areas most susceptable to damage do not necessarily coincide with the areas most critical to structure safety. In describing and defining damages, it is important that the places of damage are clearly identified. The following definitions of various structural zones are proposed here:

A. Longitudinal

Structural Head

Structural Tank

B. Vertical

Water Level Zone--from MLW + $\frac{1}{2}$ (design wave height) to MHW + design wave height.

High Water Zone--above water level zone and below structure crest.

Low Water Level Zone--below water level zone and above berm.

Crest Zone--the horizontal section of the breakwater crest.

High Leeward Zone--on the Lee side, below crest and above MWL.

Low Leeward Zone--on the Lee side, below MWL.

Berm and Toe Zone--below low water zone to the structure toe.

Figure 1.1 delineates the various zones as proposed.

1.3.3 Damage Degrees-The degree of damage should be expressed in terms of percentage per unit area coupled with description. The following code is recommended:

A. Degree 0 (none)--less than 1% armor unit displaced and less than 1% armor unit damaged; unit armor layer rifting no more than 1 unit; no other mode of damage.

B. Degree 1 (slight)--less than 2% armor unit displaced; less than 2% armor unit breakage and damaged; unit armor layer rifting less than 3 units; no other mode of damage.

C. Degree 2 (little)--less than 3% armor unit displaced; less than 3% armor unit breakge; unit armor layer rifting less than 5 units; unit sublayer exposure occurred but no more than 15 units; no other mode of damage.

D. Degree 3 (moderate)--less than 5% armor unit displaced; less than 5% armor unit breakage; unit armor layer rifting less than 8 units; unit sublayer exposure occurred but no more than 25 units; settlement occurred (excluding initial settlement) but less than 2% of the original height; no other mode of damage.

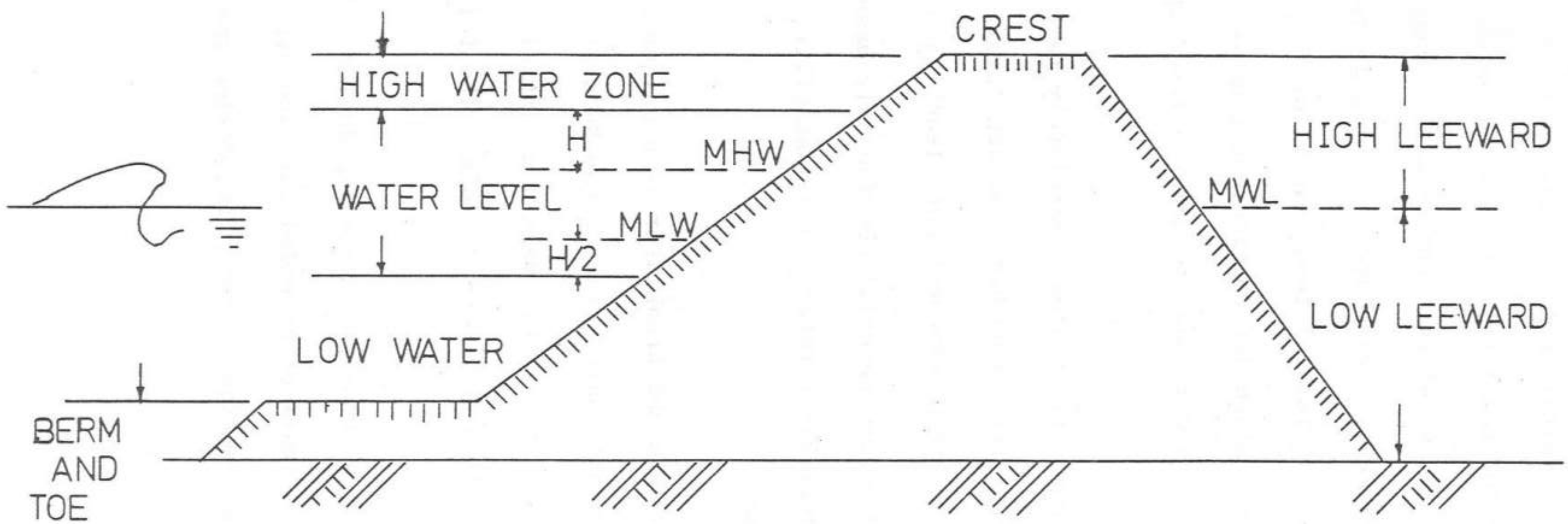

Fig. 1.1 Breakwater Damage Zones

E. Degree 4 (considerable)--less than 10% armor unit displaced; less than 10% armor unit breakage; unit armor layer rifting less than 15 units; unit sublayer exposure no more than 50 units; unit core exposure less than 10 sublayer units; settlement (excluding initial settlement) less than 4% of the original height coupled with moderate slope deformation; no other mode of damage.

F. Degree 5 (severe)--unit core exposure less than 25 sublayer units; settlement (excluding initial settlement) less than 8% of the original height coupled with significant slope deformation; section breaching occurred but no more than four armor units wide and two armor units deep; no other mode of damage.

G. Degree 6 (unsafe)--armor layer almost completely removed; unit core exposure more than 25 sublayer units; breaching section larger than four armor units wide and two armor units deep; permanent core leaching or undermining.

H. Degree 7 (destroyed)--core material continuously washed out; core leaching or undermining at steadily increasing rate; slope instability occurred; damage progressing rapidly with time.

1.3.4 Safety Margin--Rubble mound breakwater is a unique structure that the conventionally accepted design safety factor may be difficult to establish. Therefore, it is suggested here that safety margin be established in terms of events and parameters exceeding design conditions. The following conditions are considered important:

A. Safety margin against wave height exceeding design value.

B. Safety margin against storm of extended duration over DBS.

C. Safety margin against earthquake exceeding design condition.

CHAPTER 2 RECOMMENDED REGULATORY POSITIONS

2.1. Design Specification Requirement

2.1.1 Design Basis Conditions--Under design basis conditions, the protective breakwater should maintain its functional integrity and insure safe shutdown of the plant(s). The DBC should, at minimum, meet the general specifications stipulated by NRC and with due consideration of environmental factors unique to the site. In addition, other combinations of extreme environmental parameters that have not been explicitly stipulated, but will have the same degree of likelihood of encountering, should also be examined and included. A list of design basis conditions and their associated allowable damage levels are suggested herewith.

A. DBC 1 - Design Basis storm (DBS) + Maximum Current during DBS (C_3) + Annual Extreme Swell (S) + Ice Effect* Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 3 (moderate) damage.
- No more than 8' Green water overtopping should occur.

B. DBC 2 - (This condition is only applicable to shallow water cases.) Water Depth Limited Wave (V_4) + Highest Water Level (L_4) + Maximum Current (C_3) + Ice Effect*. Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 3 (moderate) damage after a period of 36 hours and it shall not deteriorate beyond

*This effect could be ignored if DBS (or OBS) and Ice Conditions are incompatible.

Degree 4 (considerable) damage after 72 hours.

- For a ten percent increase in L_4 and a corresponding increase in wave height, the damage shall not progress beyond Degree 4 after a period of 36 hours.
- No more than 8' green water overtopping should be allowed under the DBC 2.

C. DBC 3 - Breakwater deteriorated to the state requiring safe plant shut down + Operation basis storm + Maximum current during OBS (C_2). Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 5 (severe) damage for an extended period sufficient for shut down operation to complete.

D. DBC 4 - Safe shutdown earthquake (DBE) + Lowest water level for astronomical tide. Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 4 (moderate) damage.

E. DBC 5 - Safe shutdown earthquake (DBE) + Highest water level for astronomical tide. Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 4 damage.

F. DBC 6 - Design basis tsunami + Highest astronomical tide + Ice Effect. Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 4 damage.
- Major structural components, in particular, cassion type, if any, shall sustain no dislocation, overturning or rotation.

G. DBC 7 - Design basis tsunami + Lowest astronomical tidal + Ice Effect. Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 4 damage.
- Major structural component, in particular, cassion type, if any, shall sustain no dislocation, overturning or rotation.
- No more than 12' green water overtopping shall result.

H. DBC 7 - Limiting case accident + Design basis storm. Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 4 damage immediately after accident.
- The damage will not progress beyond Degree 5 for an extended period of DBS after the accident to allow plant safe shutdown.

I. DBC 8 - Tornado. Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 3 damage.

2.1.2 Operating Basis Conditions--Under this condition, the protective breakwater should maintain complete functional as well as structural integrity. The plant can remain in operation and the breakwater, in general, requires no immediate repairing.

A. OBC 1 - Operating Basis Storm (OBS) + Maximum Current During OBS (C_2) + Annual Extreme Swell + Ice Effect.* Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 3 damage in each damage zone.
- No more than 4' white water overtopping shall occur.

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*See footnote on p. 9.

B. OBC 2 - Operating Basis Earthquake + Lowest Water Level for Astronomical Tide. Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 3 damage in each damage zone.
- No more than 2% total settlement shall occur as a direct result of the earthquake.
- No large structural components, such as caissons, shall sustain dislocation, overturning, rotation or other structural damage.

C. OBC 3 - Operating Basis Earthquake + Highest Water Level for Astronomical Tide. Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 3 damage in each damage zone.
- No more than 2% total settlement shall occur as a direct result of the earthquake.
- No large structural components, such as caissons, shall sustain dislocation, overturning, rotation or other structural damage.

D. OBC 4 - Operating Basis Accident (OBA) + Operating Basis Storm (OBS). Under this condition, the design should show that:

- The breakwater shall sustain no more than Degree 3 damage immediately after the accident.
- The damage shall stabilize after accident for an extended period under OBA. Under no circumstance, should the damage be allowed to progress beyond Degree 4.

2.2 Material Construction and Quality Control Specifications Requirements

2.2.1 General

The specifications should include the following information:

A. Material specifications should include detailed descriptions of physical properties and chemical and biological stabilities in marine environment.

B. Whenever applicable, material property testings should be in conformance with the standards and procedures recommended by the American Society of Testing Materials (ASTM), U.S. Bureau of Mines (USBM), U.S. Bureau of Reclamation (USBR), American Concrete Institute (ACI).

C. For materials of heterogeneous nature such as sand and gravel or quarry stone, the source of origin and the ranges and gradations of size and weight should be specified. Information on physical and mechanical properties should be specific and should include not only the mean value but also range of expected variations.

D. Quality control standards, procedures of implementation and expected results should be specified.

E. Methods and procedures of material handling including extraction and excavation, fabrication and manufacturing, transportation and placement should be specified.

F. Construction programs including installation techniques and procedures and inspection standards and practices should be specified.

2.2.2 Recommended Requirements

A. Fill material

- The material, whether sand, gravel or shell, should be clean, hard and durable, inert to seawater and weathering. No more than 10% fines (passing the #200 sieve) should be allowed. Specific gravity larger than 2.5 is desirable.

B. Foundation fill, filter layer and scouring blanket material:

- 90% or more of the stones should fall within the size range specified. Within the range, the aggregate should be well blended.
- Individual stone should be equidimensional. Flat or elongated aggregates should be rejected. Stones with the greatest dimension, greater than three times the least dimension, should not constitute more than 10% of the total aggregates.
- The material, either natural fragments, crushed stones or artificial aggregates, should be inert to chemical and biological degradations in sea water. Rocks of sedimentary origin with high calcite content (more than 50%) should be avoided unless their chemical stability throughout the structure life can be positively established.
- The average specific gravity should be greater than 2.6.
- The following standard tests should be performed to establish the durability of the material:

  a. abrasion test--Los Angeles abrasion test (ASTM C-535) or equivalent.

  b. toughness test--ASTM C-170 or equivalent.

  c. hardness test--ASTM C-235 or equivalent.

  d. apparent specific gravity and absorption test--ASTM C-127 or equivalent.

f. the soundness tests by magnesium and sodium sulphate-ASTM C-88 or equivalent.

C. Core material:

- The requirements are similar to that of foundation stone. In addition:
- The aggregates may be uniformly graded within the size range.
- More than 50% of the stones (by weight) should be of sizes greater than the median weight.
- The stones should be free from laminations, weak cleavages and foreign materials which might contribute to breakage.
- The percentage abbrasion shall not exceed 3.5.
- Testing should be performed to establish the compressive and shear strengths of the material.

D. Secondary armor material (natural stones)

- The requirements are similar to that of core material. In addition:
- Stones with their greatest dimension greater than three times the least dimension should be rejected.
- Testing should be performed to establish compressive, shear strengths, toughness and abrasion resistance.
- The material should prove to possess adequate freezing and thawing resistance commensurate to the design weather condition.

E. Primary armor material (natural)

- Primary armor units are the most important structural elements in rubble-mound breakwaters. The structural safety as a whole depends heavily upon the ability of these units to withstand the ever present dynamic loadings rendered by the hostile oceanic environment. Man-made artificial units are commonly employed in favor of natural stones. However, if natural stones are to be used as primary armor material, they must meet, in addition to all the material requirements for the secondary armor material, the following conditions:

- Sedimentary rocks such as limestones that are vulnerable to long-term chemical dissolution in sea water should be excluded.
- The stones should have high specific gravity and low absorption. Stones with specific gravity less than 2.6, or with absorption more than 2% should be rejected.
- The stones should prove to be impact resistant and impact fatigue resistant commensurate to design impact loading conditions. Stones susceptible to violent shattering under impact should be rejected.

F. Primary armor material (precasted concrete units)

- The concrete should have a minimum 28-day compressive strength of 5,000 psi. Higher compressive strength concrete should be considered for larger armor units.
- The aggregate should be inert to chemical and biological degradations in sea water.
- Whenever applicable, material and fabrication should conform to the current editions of ASTM, ACI and ACRI specifications, codes and manuals.
- The mixture should have satisfactory freezing and thawing resistances commensurate to the design environment condition.
- The individual units should prove to be impact resistant and impact fatigue resistant commensurate to design impact loading conditions.

G. Reinforcement

Wherever applicable steel reinforcing material should conform to ASTM and ACRI specifications and manuals.

Fiber reinforcing material should be inert to chemical and biological degradations in sea water.

2.3 Inservice Surveillance Requirements

2.3.1 Criteria

The applicant is required to clearly establish the criteria for safe shutdown and for repair. When the breakwater sustains damages exceeding the established criteria, appropriate action should be taken within the specified time span. Some basic recommendations are provided here.

A. Safe Shutdown Criteria--when the breakwater sustains damages exceeding any of the following conditions, the plant requires immediate shutdown.

- When the damage exceeds Degree 5 and progresses to Degree 6.
- When a sector of the breakwater crown , equivalent or exceeding half the major dimensions of a single floating power plant , has been breached to the extent that green water overtopping is expected to exceed half the design wave height under DBS.
- When continuous and a large quantity of core leaching is evident and threatens the collapsing of the breakwater.
- When slope instability is evident.

B. Repair criteria--When the breakwater sustains damages exceeding any of the following conditions, repair should be undertaken:

- When the damage exceeds Degree 4 and progresses toward Degree 5.

- When a sector of the breakwater crown equivalent or exceeding one-fourth the major dimension of a single floating power plant has been breached to the extent that green water overtopping is expected to exceed one-third the design wave height under DBS.
- When more than a sector of 100 feet of the toe protection experiences scouring to the extent that the critical failure zone of the breakwater slope is no longer protected.
- When the pressure gradient within the breakwater consistantly exceeds the accepted limits for either slope stability or piping resistance.
- When the pore pressure in the foundation soil consistantly exceeds the accepted limits for either slope stability or piping resistance.
- When pore pressure in the foundation soil consistantly exceeds the accepted limits to threaten foundation failure or potential liquefication.
- When lateral cracking extends across the breakwater.
- When piping and/or core leaching is evident.

2.3.2 Requirements

The surveillance program should include the following schedule:

A. Post construction survey--post construction survey is necessary to establish baseline conditions and to insure that design stress and strain requirements are not exceeded.

- Profile and cross-section of breakwater.
- Short-term settlement at foundation level.
- Change of pore pressure in the foundation soil.
- Stress in the foundation soil.
- Change of foreshore geometry.
- Stress and strain of large concrete elements, if any, such as caisson.

B. Regular surveillance--regular surveillance is required for the following items to insure structural safety:

- Breakwater crown elevation at intervals not exceeding 300 feet.
- Pore pressure in the foundation soil at critical loading zones such as under large caisson structures and at areas when wave loadings are significant.
- Core leaching rate or bulk core density at critical loading zones.
- Foreshore geometry change and toe scouring.

C. Post-storm, Post-Earthquake or Post-Accident Survey--After every major storm or accident, a thorough survey should be conducted to include the following:

- Change of breakwater profiles and cross-sections.
- Damages of armor units including dislocation and breakage.
- Breakwater displacement including large structural elements such as caisson.
- Scouring and foreshore geometry changes.
- Foundation settlement.
- Core leaching and piping.
- Longitudinal or lateral cracking.

D. Long-term surveillance--long-term surveillance should be conducted to assess the structural performance and to readjust the expected structural life:

- Total or cumulative foundation settlement.
- Armor material deterioration, including abrasion, weathering and strength reduction.
- Structure subsidence and long-term core leaching.

PART II

PROCEDURAL SPECIFICATION

This section was prepared in an attempt to cover the following information to aid in the review of design methods and procedures:

- The main elements that require engineering decision and design.
- The current practice, experience and opinions.
- The recommended position, whenever applicable, from the standpoint of structural safety.

CHAPTER 3

DESIGN ENVIRONMENTAL FACTORS DETERMINATION

To a large extent, the environmental factors for breakwater design coincide with those for power plant siting. The review responsibility and decision making for those factors actually fall outside the structural branch. Therefore, the material covered in this section is mainly for the purposes of information transformation and of providing continuity from environmental factors to design factors. The position of site branch should supercede any opinion expressed here in case of conflict or contradiction. Explicit recommended review positions are provided only for those factors exclusively related to breakwater design.

3.1. Water Level Computation

In breakwater design, both water level and water level variation are the fundamental design parameters. The extreme water level dictates the breakwater height and affects the wave climate, wave run up and overtopping, scouring and the characteristics of wave impinging on the structure. The water surface variation determines the zone of damage, scouring and critical loading conditions. The major factors that govern the water depth and water depth variations are the bottom contour, tidal variation and storm surge. Other minor influences include river discharge, rain fall and other factors unique to the site. The computations of bottom contour needs not be elaborated here other than the fact that it must be positively established. The other two contributing factors are discussed below.

3.1.1 Astronomical tide--Astronomical tide is a cyclic phenomenon. Thus, prediction presents little problem provided tidal records are available at or near the location. Statistical analysis can be easily performed to determine the extreme values, their recurrance intervals and confidence levels. Depending on the method used, variations in predicted values may result.

A. Analysis based upon exceedance in real time (Ref. 3.133)

The probability of water level exceeding level z_o is computed according to

$$P(z \geq z_o) = \frac{\Sigma\, t_i}{T_t}$$

where T_t is the length of the record analyzed and $\Sigma\, t_i$ is the amount of time $z \geq z_o$.

B. Analysis based upon exceedance in tidal cycle

The probability of exceedance is computed according to,

$$p(z \geq z_o) = \frac{\Sigma\, n_i}{N}$$

where N is the total number of tidal cycles analyzed and Σn_i is the number of tidal cycles $z \geq z_o$.

C. Analysis based upon annual extreme values (Ref. 3.6.1)

Based upon the annual extreme values of astronomical tides, predictions can be made utilizing extreme-value statistical methods similar to those presented in the storm surge computation (see 3.1.2).

Among these three variations, the third one is considered as the most rational method and will yield reasonable but somewhat conservative estimates. However, it requires long-term data before such analysis can be performed. The first method tends to underestimate the probability of encountering even if it faithfully represents the probability of exceedance in real time.

At any case, since the astronomical tide is such a regular phenomenon and its range of variation is usually much less than the storm surge along the open coast, therefore, slight variations in prediction are not detrimental.

In the case of bay location, the situation might be quite different. The tides may be significantly amplified due to the geometrical boundary condition. There are a number of computational methods available (Refs. 3.6.1).

In summary:

- The prediction of extreme values of astronomical tide along the coast of the U.S., with the required accuracy for breakwater design, is within the state-of-the-art and does not present a serious problem that requires additional exhaustive investigation.

- Along the open coast, statistical analysis, interpretation and extrapolation of historical data, both in temperal and spacial variations, should be considered adequate. Extreme value analysis is preferred over other methods, if historical data is available.
- For in-bay locations, computations should be coupled with field verification if historical data does not exist. At present, the basic computational techniques, mainly based on long-wave theory, are adequate (Refs. 3.11, 3.23, etc.) to predict the tidal variations. Slight modification may be required to apply to the specific site.
- Ample detailed data concerning tidal ranges along the U.S. coast has been collected and compiled by the U.S. Department of Commerce, the National Ocean Survey, the U.S. Army Corps of Engineers and the U.S. Navy. This data shall be considered adequate.

3.1.2 Storm Surge--Storm surge is a highly irregular phenomenon both in its magnitude and in its frequency of occurrence. Along the U.S. Coast, its magnitude is usually significantly higher than the astronomical tide. Therefore, design storm surge determination represents an important task in breakwater design.

Since it is also one of the most important factors for power plant siting, the material presented here is mainly for reference. No explicit recommendation is required.

A. General--Storm surge computation consists basically of the determination of extreme value and historiogram. In the context of breakwater design, the former is essential in calculating maximum water level whereas the latter is important to formulate design storm condition. The contributions on this subject are voluminous.

The computation should at least include the following contributing elements: (a) direct wind stress tide, (b) coriolis tide, (c) atmospheric setup and (d) wave setup. Usually the storm surge computation includes only the first three elements, the wave setup is computed separately. It is also true that the presence of breakwater has a more pronounced effect on wave setup than the rest of the three elements. Therefore, the wave setup is discussed separately.

B. Methods of Computation--Based on geographical situations, the surge computations can be grouped into four categories: namely, open coast case, open bays and estuaries, enclosed lakes and reservoirs and behind coastal barrier. Among them, open coast storm surge is by far the most important one for offshore nuclear power plant. Various methods presently available for this case are summarized here.

a. Extreme-Value Statistical Method: This is one of the most common ways of analyzing maximum storm tide level when sufficient amounts of actual data are available. There are a number of different schemes for data manipulation. The more popular ones are Gumbel's first asymptotic distribution (sometimes known as the Fisher-Tippett Type I distribution), log-normal distribution, the Poisson distribution, the Frechét extreme value distribution (the Fisher-Tippett Type II distribution), Weibull distribution and chi-squared distribution. It is generally recognized that if the data is a set of maximum values in terms of events within a set of time spans, the first three types of distribution represent a better fit. On the other hand, if data is equally time-spaced, the latter three types of distributions are more applicable. For storm tide analysis, the first three types are recommended. Table 3.1 summarizes the various distribution functions now being used for extreme value analysis.

If this method is to be used, the following information should be established:

- The expected extreme surge level versus return period.
- The most probable surge level versus return period.
- The confidence level of non-exceedance (or its equivalent).
- The risk of encountering.

Some of the drawbacks of this method are:

- The method is data dependant. Therefore, it requires large amounts of historical data at or near the site location to facilitate adequate prediction.
- Effects of local hydrograph and/or effects of site modification are not included in the results.
- It is not suitable for very long-term prediction commensurate to the design risk and return period stipulated for the offshore nuclear power plant.
- It provides no information on storm surge histogram.

b. Hydrodynamical-Numerical Method: This method computes the storm surge level as well as histogram on the basis of hydrodynamic principles. The actual computations are usually accomplished with the aid of digital computers. The governing equations in the problem formulation are commonly referred to as bathystrophic storm tide equations which are equations of motions integrated in the vertical direction. For open coasts, a number of numerical schemes are available. Among the contributions, Jelesnianski's SPLASH I and II computer programs (Refs. 3.24, 3.25), Leendertse's model for long wave propagation, (Ref. 3.22), Miyazaki's model for the Gulf of Mexico (Ref. 3.29) and Pearce's computation for Hurricane Camille (Ref. 3.32) are some of the representative ones. Some of the problems encountered using this method are:

Distribution Type	Function	Remarks
Gumbel	$F(S) = e^{-e^{-\alpha(S-u)}}$	Two parameters α,u - data related constants
Log-normal	$F(S) = \frac{1}{\sigma} e^{-(S-\beta)^2}$	Two parameters σ,β - data related constants
Poisson	$F(S) = \frac{\nu^S e^{-\nu}}{S!}$	One parameter ν data related
Frichet	$F(S) = e^{-(as)^n}$	Two parameters a,n - data related constants
Weibull	$F(S) = 1 - e^{-(bs)^k}$	Two parameters b - data related constant k - positive integer
Chi-Square	$F(S) = e^{-(ps)^2}$	One parameter p - data related constant

F(S) is the probability that the storm surge exceeds the value S.

TABLE 3.1 Formulas for Extreme-Value Statistics

- Extensive program revision is usually required when applied to specific cases.
- The application of vertically integrated equations of motion is not completely justified.
- There often lacks rational ways of defining boundary conditions.
- A number of important coefficients related to the determination of bottom and surface stresses remain largely unknown at present and, hence, must be estimated or guessed.
- Consequently, the computer model often requires extensive calibration and adjustment against real events before one can claim certain degrees of confidence on the results.

c. Hybrid Method: This method refers to any semi-empirical techniques that are derived from the combinations of the hydrodynamic model, statistical techniques and/or analysis of actual records. At present, the contribution in this category is scant. However, it apparently has drawn more attention recently and may offer a promising approach to the problem. The monograph method referred to in Ref. 3.133 represents one of the initial attempts in this category.

3.1.3 Wave Setup

Wave setup represents another contributing factor to the total water depth. Since this phenomenon is associated with the presence of a specific structure with a sloped or vertical wall, it is an important factor for breakwater design.

A large number of wave setup formulas have been proposed based upon theoretical, laboratory and field investigations. They are summarized in Table 3.2 These formulas predict maximum wave setup ranging from 0.1 to 0.3, the deepwater wave height. The most recent field measurements in the

TABLE 3.2 Summary of Wave Set-Up Formulas (From Ref. 3.16)

Authors/Years	$\overline{\eta}(x)$	$\overline{\eta}_{max}$
Battjes, 1974	--	$\frac{5}{16} \cdot \gamma \cdot H_B$
Bowen, et al., 1968	$\frac{\Delta\overline{\eta}}{\Delta x} = - K \cdot \frac{\Delta h}{\Delta x}$ $K = \frac{1}{1 + (813\ \gamma^2)}$	--
Dorrestein, 1962	--	$\overline{\eta}_{max} \cong 0.1 \cdot H_o$
Goda, 1975	$\eta_{i+1} = \eta_i - \left(\frac{K_E}{h+\overline{\eta}}\right)_{i+1/2} \cdot (\overline{H}^2_{i+1} - \overline{H}^2_i)$ $K_E = \frac{1}{8}\ (\frac{1}{2} + \frac{2kh}{\sinh\ 2kh})$	--
Hansen, 1976	--	$0.3\ H_o$
Hwang, Divoky, 1971	--	$0.2\ H_B$
Iwata, 1970	$\frac{\Delta\overline{\eta}}{\Delta x} = \frac{- \frac{\Delta h}{\Delta x} + \frac{\gamma^2}{8} K_f \frac{\cos^2 \theta_{St}}{2+\cos\ 2\theta_{St}}}{1 + \frac{8}{\gamma^2(2+\cos\ 2\theta_{St})}}$	--
Jonsson, Jacobsen, 1973	$[1 + 8(3\gamma^2) - 2hc^*]\overline{\eta} =$ $-h + 5d_B/6 + c^*(h^2 + \overline{\eta}^2 - d_B^2)$	$\overline{\eta}\ \frac{5}{16} \cdot \gamma^2 \cdot d_B \cdot (1 - \frac{6}{5} \cdot c^* \cdot d_B)$ $c^* = \frac{g \cdot \sin^2 \theta_{St}}{2 \cdot c^2_{St}}$

Longuet-Higgins, Stewart, 1963	$\frac{\Delta\bar{\eta}}{\Delta x} - = Q \cdot \frac{\Delta h}{\Delta x}$ $Q \stackrel{\sim}{=} 0.15$	--
Longuet-Higgins, Stewart, 1962	$\bar{\eta}(x) = -\frac{H_o^2 \quad g^{1/2}}{32 \cdot \sigma \cdot h^{3/2}}$ $\sigma = \frac{2 \cdot \pi}{T}$	--
Munk, 1949(2)	--	$\frac{13.7}{g} \cdot (\frac{H_B}{T})^2$
Saville, 1961	--	$(0.1 \text{ bis } 0.2) \cdot H_o$

TABLE 3.2 Summary of Wave Set-Up Formulas (From Ref. 3.16)

North Sea (Ref. 3.16) suggested the actual setup to be on the high side. Therefore, it is recommended here that the applicant should conform to the higher value unless proved otherwise.

3.2 Storm Conditions

The storm condition should include the following information:

- The peak hourly wind.
- The forward speed and direction.
- The mean radius.
- The center pressure depression.
- The histogram.
- The gust factor (the peak 3-s sec. wind).

Both operating basis storm and design basis storm should conform to the standard stipulated by the Nuclear Regulatory Commission. The design basis storm is a hypothetical hurricane having that combination of characteristics which will make the most severe storm that is reasonably possible in the region involved, if the hurricane should approach the point under study along a critical path and at an optimum rate of movement. Procedures for developing a DBS can also be found in Ref. 3.133 . If a DBS had not been specified by NRC, the one developed on the basis of Ref. 3.133. should be considered acceptable.

The 100-year storm is usually accepted as the operating basis storm. If the OBS condition had not been specified, methods based on extreme-value statistics similar to that described in Table 3.1 could be used to determine the peak wind speed. The value with 95% confidence of non-exceedance is recommended here instead of the expected value or the most probable value. The present state-of-the-art does not permit accurate prediction of wind direction associated with the storm. Therefore, direction or directions that are likely to result in the most destructive ones should be used. The predicted condition should be cross-checked and combined with actual case histories to produce the design histogram.

3.3 Wave Conditions

Waves are one of the most important factors in breakwater design and it is well known that the breakwater stability is effected by the wave height to the cubic. Design conditions for both irregular and regular waves should be carefully established.

3.3.1 Storm Waves (Irregular Waves)

Based upon the availability of field data and the nature of the data, storm wave conditions can be established by a number of different methods. The four commonly used ones are listed here. All these methods have their drawbacks and should be judiciously applied.

A. Storm wave computation based upon wind and pressure field (either hypothetical model or real cases)(Ref. 3.63).

B. One-dimensional wave energy spectra based upon stationary and uniform wind field (Refs. 3.132, 3.76, etc.).

C. Wave spectra based upon actual field measurements.

D. Wave Spectra estimation based upon historical data of wave climate or wave height and period information (Ref. 3.67).

For DBS wave conditions, Method B is recommended. The selection of spectral shape should be based upon local conditions. For OBS wave condition, Method D is recommended. The wave conditions, both DBS and OBS, so established should be verified, whenever possible, by field data.

3.2 Depth Limited Waves

When a deepwater wave propagates into shallow region, the wave eventually becomes depth limited known as the breaking wave which is one of the most destructive wave forms. A number of empirical formulas exist to compute the breaker height. They yield values of breaker height ranging from 0.6 to 1.2 of the local water depth.

For the present case, the breaking wave condition should be established through laboratory experiments. Various combinations of height and period should be tested with the structure in place. The most destructive combination or combinations should be selected as the design limiting waves.

3.4. Current Computation

3.4.1 Normal Current Condition at the Site

For design purposes, the normal current condition could be taken as the expected annual maximum current. Two different conditions should be defined: (a) current condition without the structure and (b) current condition with the proposed structure. For the condition with the structure, the longshore as well as the onshore-offshore current components with respect to the structure should be established.

At present, there is no better way to establish current condition than field measurement supplemented by, possibly, laboratory experiment.

3. 4.2 Current Under Storm Conditions

Under storm conditions, the currents induced by wind and wave should also be included in addition to the normal current condition at the site.

The wind induced current computation in offshore area is commonly computed in accordance with the method outlined in Ref. 3.104. The wind induced current near coast is a complicated problem. There is very little information available. In principal, the bathystrophic tide equation can be used to estimate the longshore as well as onshore-offshore current components if one can assume reasonable current profiles across the depth.

In general, wave-induced current is important only in shallow water. For the case of constant water depth, there are numerous formulas available to compute the induced current. Unfortunately, they do not usually yield compatible results. Sometimes, the results could even differ in direction. At present, the method proposed by Longuet-Higgins (Ref. 3.101 and 3.102) is considered to represent the state-of-the-art (a number of modifications have been suggested by a number of investigators; for instance, Huang, Ref. 3.98, Wang and Liang, Ref. 3.110). For the case of shoaling water, the problem remains largely unsolved. Perhaps one of the most pronounced wave-induced phenomenon is the longshore current along a beach or a long structure. The longshore current is almost entirely attributed to the incident waves oblique to the structure. For a massive rubble-mound structure the magnitude of this longshore component should be established. Because of the importance of longshore current tó shore erosion problems, a significant amount of research information has been accumulated in the past. More than a dozen formulas have been suggested (Table 3.3 provides a survey of these formulas). Unfortunately, almost all of them are for straight shoreline and they will yield a wide range of predicted value for the same input condition. At present, formulas 11 and 12 as listed in Table 3.3 have received wider support than the rest. The applicants should be encouraged to develop their own method to estimate the current strength using the above suggested formulas as guidance.

TABLE 3.3 Longshore Current Formulas

No.	Authors	Mean Longshore Current, V
1	Putnam-Munk-Traylor (1949)	$[6.97g \frac{s}{f} \tan\beta \; H_b^{\,2} \frac{\sin 2\alpha_b}{T}]^{1/3}$
2	Eagleson (1964)	$[\frac{3}{8} g\kappa H_b \frac{\sin\beta \; \sin \alpha_b \sin 2\alpha_b}{f}]^{1/2}$
3	Putnam-Munk-Traylor (1949)	$\frac{A}{2} \; [(1 + \frac{4}{A} \; 2.28g \; H_b \; \sin\alpha_b)^{1/2} - 1]$ $A = 20.88 \frac{\tan\beta}{fT} \; \cos\alpha_b \; H_b$
4	Galvin-Eagleson (1965)	$gT \tan\beta \; \sin 2\alpha_b$
5	Inman-Bagnold (1963)	$2.31 \frac{\kappa \ell \tan\beta}{T} \cos \alpha_b \; \sin\alpha_b$
6	Bruun (1963)	$C_f \; [\frac{0.95}{\sqrt{g\kappa}} H_b^{\,3/2} \frac{\tan\beta \sin 2\alpha_b}{T}]^{1/2}$
7	Inman-Quinn (1951)	$[(\frac{1}{4A^2} + 2.28gH_b \; \sin\alpha_b)^{1/2} - \frac{1}{2A}]^2$ $A = 108.3 \; \frac{\tan\beta H_b \cos\alpha_b}{T}$
8	Brebner-Kamphuis (1963)	$8.0 \; \sin^{1/3}\beta \; \frac{H_o^{\,2/3}}{T^{1/3}} \; [\sin 1.65\alpha_o$ $+ \; 0.1 \; \sin 3.30\alpha_o]$
9	Brebner-Kamphuis (1963)	$14.0 \; \sin^{1/2}\beta \; \frac{H_o^{\,3/4}}{T^{1/2}} \; [\sin 1.65\alpha_o$ $+ \; 0.1 \; \sin 3.30 \; \alpha_o]$

10	Harrison (1968)	$0.241\ H_b + 0.0318\ T + 0.0374\ \alpha_b$
		$+ 0.0309 \tan \quad - 0.170$
11	Longuet-Higgins (1970)	$\frac{5\pi}{8} \frac{\kappa}{C_f} (g\ H_b)^{1/2} \operatorname{Tan}\beta \sin 2\alpha_b$
12	CERC (1973)	$20.7\ (g\ H_b)^{1/2} \operatorname{Tan}\beta \sin^2\alpha_b$

Symbols: α_o = deepwater wave angle

α_b = breaking angle

H_b = breaking wave height

T = wave period

f = bottom friction

g = gravitational acceleration

$\operatorname{Tan}\beta$ = bottom slope

s = fraction of incoming energy available to the longshore current

κ = ratio of wave height to breaking

ℓ = rip current spacing

Since under storm conditions it is known that the current interacts strongly with the wave field, such effects should also be established. Methods proposed in Reference 3.108 could be used as a first-order approximation. The applicants are encouraged to develop their own method to treat this problem.

3.5 Earthquake and Tsunami

The earthquake conditions should be in conformance with the NRC specifications.

The estimation of tsunami should be accomplished with the following requirements:

- Review of historical data and perform statistical extrapolation.
- Compute tsunami generation, propagation and amplification based upon hypothetical distant earthquakes.

At present, a number of numerical methods (Refs. 3.112, 3.129) are considered adequate for these purposes provided the earthquake ground motion is given.

3.6 Ice

For areas where ice hazard may exist, the following information should be established:

A. Type of ice--sheet, floes, ridges, etc.

B. Mechanical properties of ice.

C. Thickness of ice floes-mean and design values.

D. Drifting speed and direction-mean and design values.

E. Probability of encountering.

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CHAPTER 4 FORCE COMPUTATIONS

The types of forces commonly considered in breakwater design is delineated in Table 4.1. For rubble-mound breakwater design, not all these forces are to be explicitly determined. However, if all or any of these forces are required for the design of structural components the following specifications are recommended. Forces on armor units will be treated separately in Chapter 8.

4.1 Wave Forces

Wave forces exerted on structures can be distinguished as being due to non-breaking waves, breaking waves and broken waves. Whether a structure is subject to either or a combination of these forces depends on the wave characteristics, the water depth at the toe of the structure and the foreshore slope and configuration.

4.1.1 Non-Breaking Wave Forces

The force due to non-breaking waves is essentially hydrostatic. The Sainflou method (Ref.4.37) or the modified Sainflou method (also referred to as Miche-Rundgren method in Ref. 3.133) are generally considered adequate for the case of vertical wall. Design curves have been developed based upon the Miche-Rundgren method in Ref.3.133. Figure 4.1 shows the wave pressure distribution according to Sainflou method. ABED is the pressure diagram of the sur-pressure due to wave action, DEC is the still-water pressure diagram, p, is the value of the pressure due to wave action at the seabed, h_o is the rise of the mean level of the Clapotis formed due to

TABLE 4.1 Load Types on Breakwater

(1) Loading on Vertical or Sloped Wall Static and Stoutly-Varying Loads

- Dead Weight
- Buoyancy
- Hydrostatic Load Due to Differential Water Level
- Wind Load (Excluding Gust)
- Current Laod
- Forced Due to Non-Breaking Waves

Dynamic and Rapidly Varying Loads

- Forces Due to Breaking Waves and Wave Impact
- Earthquake Load
- Wind Gust
- Ship Collision

(2) Loadings on Caisson-Type Structures

Element	Loading Types	Loading Stages
1. Outer Wall • Seaside • Leeside • Side Wall	• Hydrostatic Pressure • Dead Weight • Buoyancy • Wave Force • Surcharge • Foundation Reaction	• Fabrication • Launching • Installation • In-Service
2. Inner Wall • Long End • Short End		
3. Bottom Slab • Seaside • Leeside • Central		

(3) Loading on Individual Armor Unit

- Wave Loading
- Current Loading
- Impact Loading
- Load Due to Adjacent Units

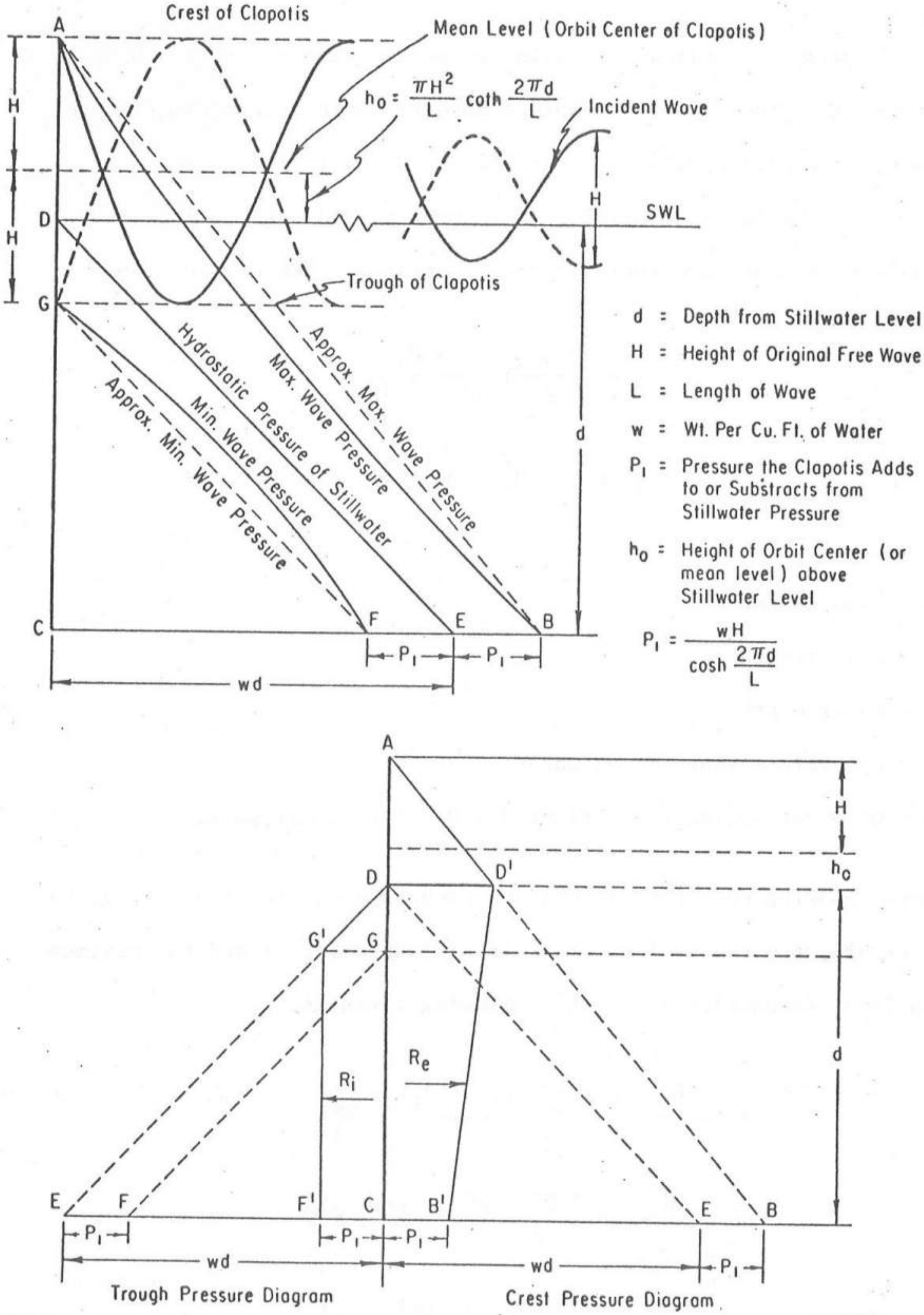

Figure 4.1 Loading on Vertical Wall Due to Non-Breaking Waves

reflecting wave. Sainflou's formula for peak pressure involves hyperbolic trigonometrical functions. The Miche-Rundgren method approximates the pressure distribution by a straight line as shown in Figure 4.1. In this case, the only quantities which must be evaluated before the diagram can be drawn are the values of p_1 and h_o. These values are:

$$p_1 = \left(\frac{1 + x}{2}\right) \frac{w\ H_i}{\cosh(2\pi d/L)}$$

$$h_o = \frac{\pi H^2}{L} \coth \frac{2\pi d}{L}$$

where

H = wave height

L = wave length

d = water depth

w = specific weight of sea water

x = wave reflection coefficient $\approx$ 1.0 for vertical wall.

The corresponding resultant forces and moments about the base are given, respectively, for the maximum crest level (subscript e) and the maximum trough level (subscript i) by the following formulas

$$R_e = \frac{(d + H + h_o)\ (wd + P_1)}{2} - \frac{wd^2}{2}$$

$$M_e = \frac{(d + h_o + H)^2\ (wd + P_1)}{6} - \frac{wd^3}{6}$$

$$R_i = \frac{wd^2}{2} - \frac{(d + h_o - H)\ (wd - P_1)}{2}$$

$$M_i = \frac{wd^3}{6} - \frac{(d + h_o - H)^2\ (wd - P_1)}{6}$$

4.1.2 Breaking Wave Forces

Waves breaking directly against the structure face sometimes exert high, short duration, dynamic pressure that acts near the region where the crests hit the structure. At present, the Minikin's formula (Refs. 3.133 and 4.26) are widely used in the United States. In Japan, Hiroi's formula is generally accepted. The Minikin's formula gives a pressure distribution shown in Figure 4.2 with peak pressure at or near the SWL. The Hiroi's formula, on the other hand, assumes a uniform pressure distribution as shown in 4.3. The Minikin's formula yields considerably higher peak pressure than Hiroi's, although the resulting total forces given by these two formulas are similar for shallow water cases. Both formulas grossly over-estimate the total force and overturning moment when the water depth gets deeper. Nagai (Ref. 4.32) and Goda (Ref. 4.9) have proposed alternative formulas for computing the wave loading. Based on these works, the following practice is recommended.

A. Vertical Structural Elements on Rubble Mound ($\frac{B}{D} \geq 0.2$, see definition sketch in Figure 4.4)

a. full breaking wave:

- peak impact pressure $p_m = 25\gamma\ H_b$ Ton/m^2
- total force $p_T = 8\ H_b$ (Ton/m) + force computed according to Sainflou's formula for $H/L_o < 0.045$

  $p_T = 13\ H_b$ Ton/m for $H/L_o \geq 0.045$
- moment $M = 8\ H_b{}^2 h_1$ Ton/m-m/m for $H/L_o < 0.045$

  $M = 12.5\ H_b{}^3$ Ton-m/m for $H/L_o > 0.045$

b. partial-breaking wave (see Figure 4.5)

- peak impact pressure $p_m = 18\gamma\ H_b$ Ton/m^2
- total force (Goda's formula)

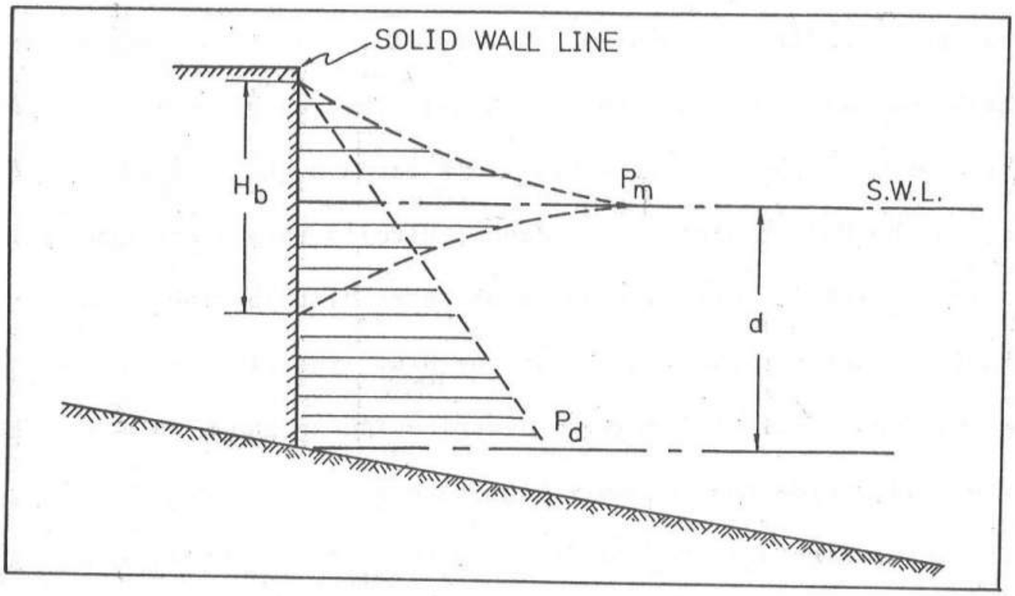

Fig. 4.2 Pressure Distribution due to Breaking Wave
(Minikin's Formula)

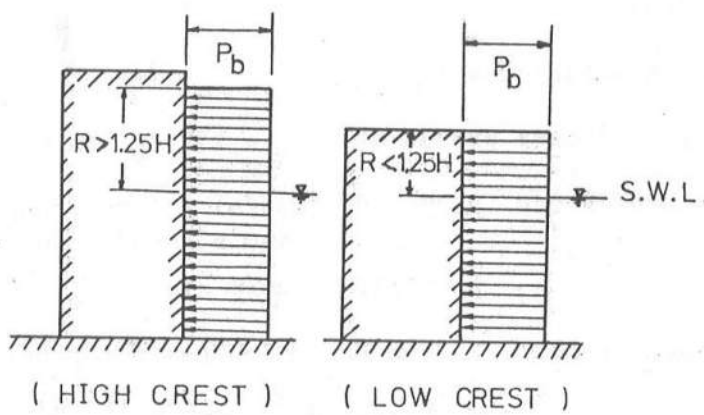

Fig. 4.3 Pressure Distribution due to Breaking Wave
(Hitoi' Formula)

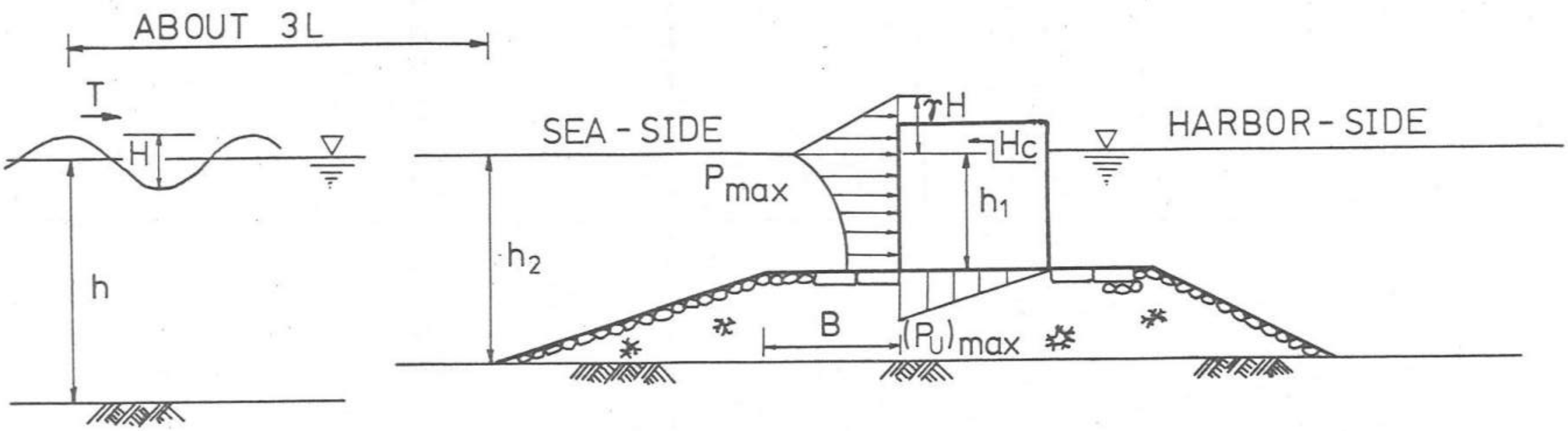

Fig. 4.4 Definition Sketch of Pressure Distribution on Breakwater

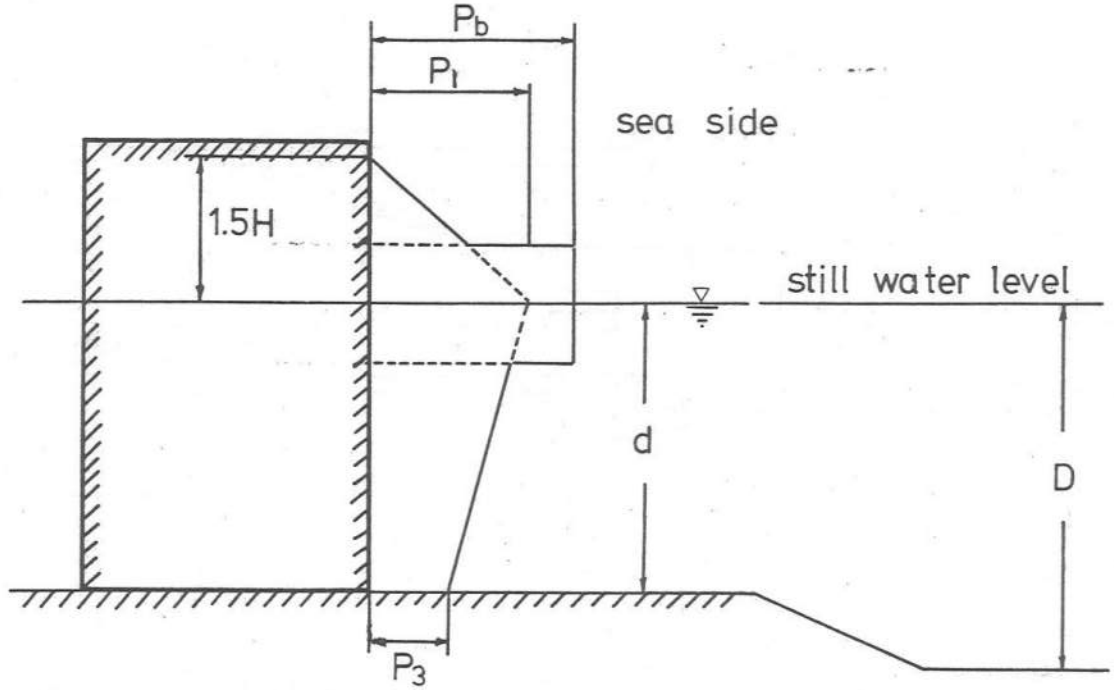

Fig. 4.5 Definition Sketch of Pressure Distribution Due to Partial-Breaking Waves

$$p_T = 0.75\ p_1\ H_b + \frac{1}{2}\ (p_1 + p_3)\ d_s \quad \text{Ton/m}$$

where $p_1 = \gamma\alpha_1\ H_b$

$p_3 = \alpha_3\ p_3$

$$\alpha_1 = 0.6 + \frac{1}{2}\ [\frac{4\ D/L}{\sinh\ 4\pi\ D/L}]$$

$$\alpha_3 = 1 - \frac{d_s}{D}\ [1 - \frac{1}{\cosh\ 2\pi\ D/L}]$$

- moment

$$M = \int_{-d_s}^{1.5\ H_b} (p_1 + p_3)\ dy$$

c. Non-breaking wave

- peak impact pressure = 0
- forces and moments according to Sainflou formulas

A. Broken Wave

Wave forces due to broken waves can be computed by momentum balance much the same as the hydraulic bore running against a sloping beach.

The regions of wave conditions are delineated in Figure 4.6.

B. Vertical structural elements on sloped seafloor. The formulas presented above can equally be applied to this case. The criteria of wave conditions can be found in Refs. 3.133.

C. On sloped wall or structures.

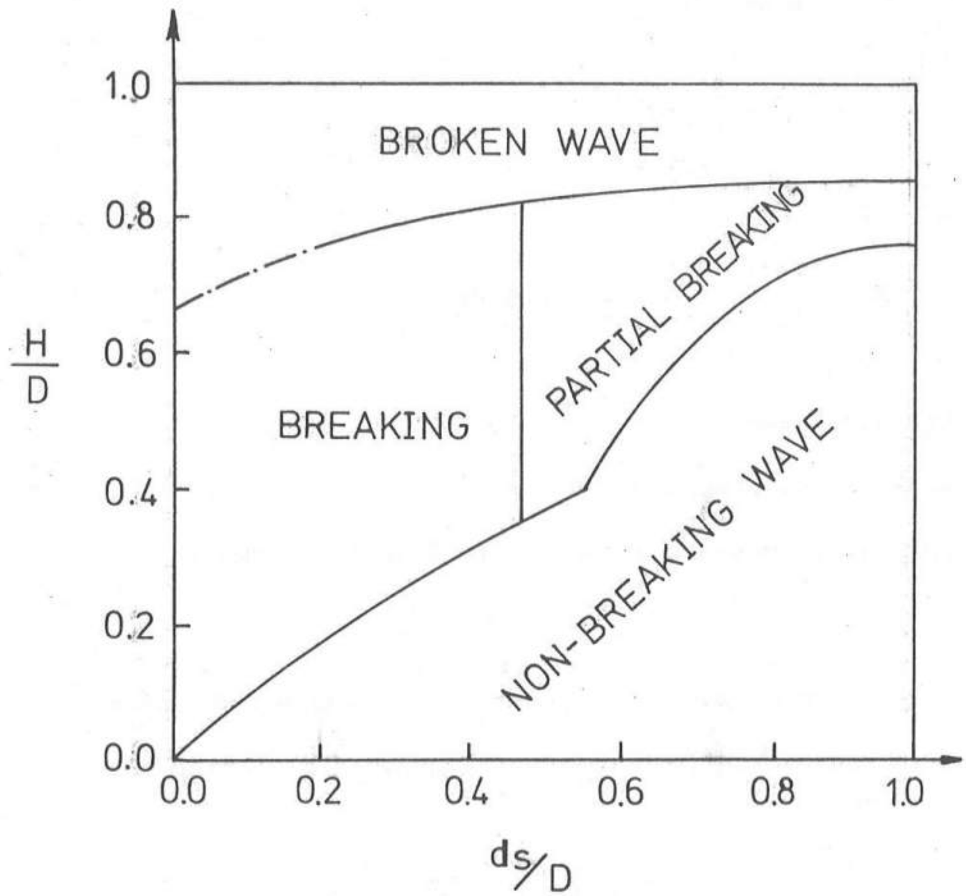

Fig. 4.6 Zones of Wave Conditions

The formulas presented on the preceding pages are all for vertical walls. For sloped walls, designers are usually tempted to use lower impact loadings. However, some recent field measurements on sloped sea dikes in the North Sea (Ref. 4.1) registered impact loading as high as 10 to 15 times the incoming wave heights for 1 on 4 slope. This is in the same order of magnitude as one would obtain for vertical wall using the Minikin formula. For 1 on 6 slopes, the impact loading is considerably reduced to about 3 times as large as the incoming wave height. The authors also observed that the impact zone extends from $\Delta h = 0.6H_s$ to $\Delta h = +0.3H_s$ below and above the still water level. The maximum impact frequency occurs about $\Delta h = -0.5H_s$. The duration of the impact has been estimated to be of the order of 0.01 - 0.05 seconds. Because of this relatively short duration and the massiveness of the breakwater, one does not expect the impact loading to be critical insofar as the structural stability is concerned. On the other hand, this high impact should definitely be considered in design structural elements that are located in the impact zone. For design purposes, the upper bound values both in magnitude and duration are recommended.

4.2 Hydrostatic Force

In rubble mound breakwater design, two types of hydrostatic forces should be considered: (1) the hydrostatic force due to the water level difference within and outside the breakwater induced by tsunami and storm surge (2) the pore pressure within the rubble mound induced by wave action. Both forces are strongly dependent on the material properties and composition of the structure such as porosity, grain size, etc. At present, the best method of determining these forces are still through model testing .

4.3 Current Forces

Current forces can be considered as steady load. It is perhaps insignificant to the structure as a whole. For individual structural elements, the current loading should be considered. The formula of current loading computation is quite standard:

$$F_D = \frac{\rho}{2} C_D A V^2$$

where

A is the projected area

V is the current velocity

ρ is the density of seawater

The problem remains in the determination of proper drag and lift coefficients and the magnitude of the current velocity. At present, the best and perhaps the only way of establishing C_D is through experiment. Values of C_D and C_L for common structural shapes can be found in many literatures. Most of these values were obtained through wind tunnel testings and were for Reynolds numbers smaller than one would expect in oceanic application. They should be judiciously adopted for the present application. For structural elements with unconventional shapes, laboratory experiments should be conducted to determine the drag force.

4.3.1 Scouring forces due to current--This is one of the most important effects of currents on breakwaters. Scouring occurs due to vertical jetting and horizontal shearing resulting from, respectively, the vertical and horizontal velocity components. The transport due to these two mechanisms is usually referred to as suspended load and bed load, respectively. The magnitudes of these forces can be estimated if the current profile and structural geometry are known. The problem becomes considerably complicated when waves are also present. Experiments should be conducted.

4.4 Wind Forces

The wind force computation for structural elements is quite similar to that of current force. For structural elements to be located at critical locations such as the armor unit at or near the structural crest, the effect of gusts should be included. Further details can be found in Chapter 8.

4.5 Spray Forces

The spray force is a complicated phenomenon involving the interactions of wind, wave and water level. The magnitude of this force largely remains unknown. For structural elements to be located near the spray zone this force should be considered. No guidelines can be provided.

4.6 Earth Forces

Earth forces need to be computed to establish:

- The slope stability of the breakwater.
- The dead weight of the structure and thus the loading on foundation.

Various standard texts (Refs. 4.43, 4.44) provide adequate methods for such computations. The design manual, Soil Mechanics, Foundations and Earth Structure, by the U.S. Navy (Ref. 4.45) is a standard one widely adopted for marine construction.

4.7 Ice and Snow Loads

The following loadings should be established for rubble mound breakwater design:

- Static vertical loadings due to the weight of accumulated ice and snow.
- The horizontal crushing force on structural elements.
- The uplift force on structural elements.
- The impact loading on structural elements due to large ice floes.

The knowledge of ice loading on breakwater is rather limited. No guidelines can be provided at present.

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CHAPTER 5 HYDRAULIC STABILITY COMPUTATIONS

The computation guidelines discussed here should be considered as providing only the necessary design considerations but not sufficient ones to insure structural safety. The final design has to rely heavily on the results of model testing. However, in any case, the final design bases should not be considered acceptable if they are less than computed.

5.1 Armor Layer Stability

5.1.1 Weight Determination

A. Stability Formulas

The individual armor unit weight is usually determined by stability formulas developed on the basis of small-scale model testing with limited field verification.

At present, there are more than a dozen stability formulas being proposed. Among them, the Hudson's formulas is the most popular and it takes the following form:

$$W = \frac{W_r \; H^3}{K_D (S_r - 1) \cot\theta}$$

where W = weight in pounds of individual armor units.

W_r = unit weight (saturated surface dry) of armor unit lbs/ft^3

H = design wave height

S_r = specific gravity of armor unit

θ = angle of structure slope measured from horizontal in degrees

K_D = stability coefficient.

The following restrictions should be observed in applying this formula:

1. The values of K_D should not exceed those recommended in Ref. 5.9 for no-damage condition (Also see Tables 5.1 and 5.2)
2. The formula is valid only for armor unit of nearly uniform size. For quarry stones, the size range should be restricted within 0.75 to 1.25 W with 75% of the individual stones weighing more than W.
3. The formula is for structures with uniform slope varying from 1.5 to 3.0.
4. The specific weight of armor unit should be within the range of 120 lbs/ft^3 to 180 lbs/ft^3 (corresponding to specific gravity of 1.9 to 2.8).
5. The formula is for regular wave impinging on the structure at right angles.

In the Hudson formula many contributing factors are lumped into the stability coefficient K_D, among them are:

a. the shape of armor unit

b. the manner of placing

c. the portion of breakwater

d. the wave shape

e. the friction among units

f. the porosity and voids

g. the number of layers

TABLE 5.1 Recommended Values of K_D (From Ref. 5.9)

Recommended* Values of K_D for Design of Structure Trunk

Breaking and Nonbreaking Waves, No-Damage and No-Overtopping Criteria

Unit	n	Placing Technique	K_D Breaking Waves	K_D Nonbreaking Waves
Smooth quarrystone	2	Random	2.1	2.4
Rough quarrystone	2	Random	3.5	4.0
Tetrapod	2	Random	7.2	8.3
Quadripod	2	Random	7.2	8.3
Tribar	2	Random	9.0	10.4
Tribar	1	Uniform	12.0	15.0
Dolos	2	Random	22.0**	25.0**

* Breaking-wave data are tentative and subject to change after more comprehensive ES 815 tests are completed.

** Tentative and subject to change after comprehensive ES 815 tests are completed. A few preliminary ES 815 tests, conducted in 1971, indicated that K_D for dolosse on steep slopes may be limited by slope failure rather than damage to the armor-unit cover layer. Therefore, a sea-side slope steeper than $\cot \alpha = 2.0$ is not recommended at this time.

TABLE 5.2

Recommended* Values of K_D for Design of Structure Head

n = 2, Random Placing Technique, No-Damage and No-Overtopping Criteria

Unit**	$\cot \alpha$	K_D Breaking Waves	K_D Nonbreaking Waves
Smooth quarrystone	1.5-3.0	1.7	1.9
Rough quarrystone	1.5	2.9	3.2
Rough quarrystone	2.0	2.5	2.8
Rough quarrystone	3.0	2.0	2.3
Tetrapod and quadripod	1.5	5.9	6.6
Tetrapod and quadripod	2.0	5.5	6.1
Tetrapod and quadripod	3.0	4.0	4.4
Tribar	1.5	8.3	9.0
Tribar	2.0	7.8	8.5
Tribar	3.0	7.0	7.7
Dolos	2.0	15.0	16.5
Dolos	3.0	13.5	15.0

* Tentative and subject to change after comprehensive ES 815 tests are completed.

** No data presently available for other armor units.

There are other factors that are known to affect the stability but are not adequately covered in the equation:

a. the effect of wave length variations

b. the duration of the storm

c. the randomness of impinging waves

d. the degree of overtopping

e. the variations of the water depth

f. the oblique wave incidence.

All of these factors should be studied through model testings. The stability formula considers waves as the sole external destructive force. This should not be considered sufficient. The other loadings such as current, wind and ice loadings should also be included.

B. Force Balance Method

Although stability formulas are, at present, exclusively used in breakwater design, this practice has not been completely satisfactory. A method based on force balance has been proposed (Ref. 5.5). Such methods, although neither widely accepted nor extensively tested, offers some promising aspects as it is based upon a more rational analysis and is more flexible to be able to include loadings other than just wave forces. Only brief summary is given here; further details can be found in Ref. 5.5. This method is based on the argument that fluctuating forces due to wave up-rush, down-rush and uplift are the major causes for armor unit movement. Therefore, the armor stability should be analyzed on the basis of force balance on individual units. If the resultant up-rush or down-rush force is greater than the interlocking and frictional forces on the armor unit, the layer becomes unstable. Similarly, if the uplift force becomes larger than the net weight of the armor unit, the layer is unstable. The difficulty of applying this method is the lack of experience and adequate means to estimate the individual force components and the interlocking and frictional forces of armor units.

C. Recommendation

- The Hudson's formula could be used as a preliminary determination of armor unit weight.
- The other force elements not included in the Hudson's formula should be properly accounted for either by method similar to the force balance experiment.
- The following combinations of forces should be considered:

  1. DBS wave perpendicular to structure + DBS current parallel to structure - both structured trunk and head.
  2. DBS wave 45° to structure - DBS current 45° to structure, both structure trunk and head.
  3. DBS wave 45° to structure + DBS current 135° to structure - both structure trunk and head.
  4. OBS wave perpendicular to structure + design ice loading.

5.1.2 Thickness and Density

The thickness of the armor layer should be determined in conjunction with the seismic design (See Chapter 6) such that:

- No sheet slide will occur.
- The total thickness consists of no less than 2 layers.

The density is computed in terms of units per unit area and is computed according to the following equation:

$$Nr = A\ n\ K_{\Delta}(1 - \frac{1}{100})(\frac{W_r}{W})^{2/3}$$

where Nr is the required number of individual armor units for a given surface area, A is surface area in square feet, K_{Δ} is the layer coefficient, and

P is the average porosity of the cover layer in percent. The recommended layer coefficient and porosity for various armor units are shown in Table 5.3 (from Ref. 3.133). The layer coefficient should not be less than the recommended values.

5.2. Cross-Section Design

The cross-section design consists of determining the following structure geometry:

a. crest elevation and width

b. concrete cap, if any

c. underlayers

d. structure core

e. bedding and filter layers

f. toe protection.

The cross-section design should at least equal or exceed the recommended specifications in Ref. 3.133. The typical breakwater sections recommended by the U.S. Corps of Engineers are shown in Figures 5.1 and 5.2 for non-breaking and breaking wave conditions respectively. Depending upon actual site situations the cross-section might differ considerably from the typical cross-sections shown. The following fundamental principles should be observed:

- The crest elevation should be computed in conformance with the performance specification for the maximum allowable overtopping. The maximum water level should include the effects of storm surge and wave runup. The wave runup should be based upon model testing with due consideration of ice effect, siltation and marine growth that might enhance the runup. The results should be double checked with existing runup information.

- The crest width should be wide enough to insure minimum water downrush to the leeside of the structure. Under no circumstance should the crest width be less than the combined widths of four armor units if greenwater overtopping is allowed (see Ch. 2). The width should not be less than the combined widths of three units if no more than white water overtopping is allowed.

TABLE 5.3 Layer Coefficient and Porosity for Various Armor Units

Armor Unit	n	Placement	Layer Coefficient, k_Δ	Porosity(P) percent
Quarrystone (smooth)	2	random	1.02	38
Quarrystone (rough)	2	random	1.15	37
Quarrystone (rough)	3	random	1.10	40
Cube (modified)	2	random	1.10	47
Tetrapod	2	random	1.04	50
Quadripod	2	random	0.95	49
Hexapod	2	random	1.15	47
Tribar	2	random	1.02	54
Dolos	2	random	1.00	63
Tribar	1	uniform	1.13	47
Quarrystone	graded	random	--	37

- In principle, the breakwater slope and berm geometry should be selected and designed to avoid wave breaking and to minimize wave impact occurrence.

- The armor units in the cover layer should be extended downslope to the structure berm in the seaward slope and at least to the mean water level in the Leeward slope.

- The weight of armor units in the secondary layer should be greater than above one-half of the weight of armor unit in the primary armor layer on the seaward slope. The thickness of the secondary layer should be as thick or thicker than the primary armor layer.

- The first underlayer should weigh about one-fifth the weight of the primary armor units and have a minimum thickness of two stones size.

- The secondary underlayer should have a minimum thickness of two stones size; these stones should weigh about one-twohundredth of the primary armor units.

- The care stones should weigh about one-fourthousandth of the primary armor units.

- The filter layer should be drainage type. The thickness of the layer should be adequate for complete coverage of subgrade and base material and should be no less than two complete layers. The grading should conform to that recommended by the U.S. Army Engineers Waterways Experiment Station and the U.S. Bureau of Standards. It stipulates that:

$$\frac{D_{15\ \text{filter}}}{D_{85\ \text{base}}} < 5$$

$$4 < \frac{D_{15\ \text{filter}}}{D_{15\ \text{base}}} < 20$$

$$\frac{D_{50\ \text{filter}}}{D_{50\ \text{base}}} < 25$$

$$\frac{D_{85\ \text{filter}}}{D_{\text{void, stone}}} < 2$$

where D = nominal diameter of grain size.

- The toe protection could be an integrated part of the filter blanket or could be a separate structured element. The material should be so graded as to have sufficient resistance against being washed away by scouring forces. The toe protection should extend beyond the scouring zone or the foundation shear failure circle whichever is greater.

5.3. Foundation Stability

Detailed foundation design specifications and design review fall beyond the scope of this manual. The purpose of this section is to provide a checklist for items specifically related to superstructure safety.

In breakwater design, the method of analysis follows quite closely to the earth structure design on land, in particular, in the category of earth and earth rock dams. Standard texts in the treatise of foundation design are abundant (Refs. 5.18, 5.19, etc.). For marine structures, the Design Manual 7 - Soil Mechanics, Foundations and Earth Structures, by the U.S. Naval Facility Engineering Command (Ref.5.19) and Design Standard for Port and Harbor Structures published by the Japan Port and Harbor Association (Ref.5.16) are two standard manuals frequently consulted. The following items related to breakwater foundation safety should be thoroughly analyzed:

- Static analysis should be performed to establish the bearing strength of the foundation against rotational and translational failures under the combined conceivable vertical and horizontal loadings as specified in Chapter 2 . The safety factor should be no less than 1.5 for permanent or sustained loading conditions and no less than 2.0 to limit movements necessary for strength mobilization or local plastic strain at the foundation edge. Detailed requirements for safety factors in bearing capacity analysis can be found in Ref. .
- Post construction maximum probably total foundation settlement and differential settlement (both longitudinal and transverse) should be clearly established. These values should be used as inputs to compute crest settlement, stress and stability analysis of structural elements sensitive to the movement such as the prefabricated armor unit and caisson structures.

- Analysis or laboratory experiment should be conducted to determine piping resistance along the least resistance passages against differential hydrostatic pressure and induced by waves. The potential reduction of soil strength due to seepage forces should also be thoroughly investigated.

- Analysis should be performed to insure that the foundation is free from the potential liquefaction against dynamic wave loading and earthquake loading.

- The scouring zone and the shear failure circle should be clearly established to determine the extent of toe protection.

5.4 References

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CHAPTER 6 SEISMIC DESIGN

6.1. Scope

The seismic design should encompass performing dynamic analysis of the breakwater-foundation-fluid system supplemented by model testing to establish the following safety-related design information:

Pressure distribution and total loading on the structure.

Stress and strain sustained by structural elements.

Structural cracking, deformation and settlement.

Slope stability.

Armor layer stability.

Foundation liquefaction potential.

6.2. Procedures

At present, seismic design of rubble-mound breakwaters is still a new engineering endeavor. There exists no well-developed method or procedure, either analytically or experimentally. The safety-related design problems should be investigated through a combination of different methods.

- Semi-static methods have been developed in Japan and in the United States to compute the total loading on earth structures due to earthquake-induced horizontal ground motion by simply adding a surcharge to the structure. This method can be used as a preliminary assessment to the slope stability.

- Semi-dynamic methods have been developed to compute the pressure distribution on the surface of dam structure due to earthquake-induced horizontal motion. This analysis has been partially verified by experiments. This method assumes the surface to be impermeable which may result in over estimating the pressure when applied to rubble-mound structures. On the other hand, the method neglects the free surface affect which may grossly underestimate the pressure near the water line.

Such a method can be modified by properly taking into account the gravity wave effect and structure porosity for breakwater application. The resulting pressure distribution can be used as the boundary input condition for stress and strain analysis.

- Rubble-mound breakwaters are a heterogeneous assemblage of discrete material and is not amendable to analytical treatment. To determine the stress and strain level under earthquake loading, two different approaches can be employed. One of them is finite element method and the other is model study.

A. When using the finite element method the applicant should establish the following parameters as rigorously as possible:

   a. The stiffness and damping coefficients of the material including foundation, core material, sublayers and armor layers. At present, limited information is available for the simulation of soil characteristics. New experiments must be devised in determining these coefficients for the armor layers and the underlayers.

   b. The effect of surrounding water must be properly incorporated including the gravity waves generated by the earthquake-induced structure motion.

   c. The boundary conditions should be clearly defined and appropriately justified.

B. When using the experimental method the following conditions must be fulfilled:

   a. The internal friction characteristics of the under layer and core should be properly modelled.

   b. The interlocking characteristics of armor layer should be properly modelled.

   c. The inertial and gravitational effects should be properly modelled.

   d. The earthquake response spectrum at the ground level should be properly simulated in accordance with the NRC specifications.

   e. The foundation effect if not properly modelled should be adequately simulated.

   f. The tests should prove to be repeatable within the range of established variability.

   g. The model should be adequately instrumented to cover the entire structure.

CHAPTER 7 ACCIDENT ANALYSIS

7.1 Scope

Accident analysis concerns mainly with ship collision against the breakwater. The following cases should be covered, at minimum, to establish the structure safety:

A. Damage to the structural trunk due to:

(a) Ship collision of military classes head-on with ship cruising.

(b) Ship collision of commercial classes head-on with ship adrift.

(c) Ship collision of commercial classes broad side with ship adrift.

(d) Repeated ship collision of commercial classes broad side with ship adrift.

B. Damage to structural head due to:

(a) Ship collision of military classea head-on with ship cruising.

(b) Ship collision of commercial classes head-on with ship adrift.

(c) Ship collision of commercial classes broad side with ship adrift.

(d) Repeated ship collisions of commercial classes broad side with ship adrift.

(e) Repeated stammings of commercial classes head on with ship adrift.

C. The maximum impact energy imparted to individual armor units under the above cases.

7.2 Procedures

Analysis based on momentum principal has been widely used in ship impact problems. Methods based upon the same principal could be used to estimate the total impact loading on the breakwater. This impact loading should then be translated into damage. The following damage situations should be established:

A. The extent and size of penetration. Here, the energy absorption characteristics of the rubble-mound must be realistically established through experiment.

B. The breakage of armor units. The extent of breakage, the estimated sizes and numbers of breakage.

C. The possible structural deformation, slope change or sliding, section breaching.

Laboratory experiments should be conducted to substantiate the analysis. The experiment should consist of two parts:

(a) The effects on the structure as a whole.

(b) The effects on individual structural elements that are susceptable to impact loadings. Such elements should include prefabricated armor units and caisson structures (see Chapter 8).

In the first part of the experiment, the geometrical similarity, at both macro and micro scales, should be preserved. The impact force histogram and energy absorbing characteristics of rubble mound material should be simulated.

In the second part of the experiment the impact force histogram and the material properties should be simulated.

CHAPTER 8 STRUCTURAL COMPONENTS SAFETY FACTORS

8.1. Prefabricated Armor Units

The prefabricated armor units usually assume unconventional shapes and are subject to irregular load pattern. Rigorous analysis may not be realizable. However, concrete design is a highly developed subject. Experience of using concrete work in marine environment is also considerable. Therefore, there exists a vast amount of information that can be consulted upon. The general design practice including standard stress analysis, concrete forming, and handling, strength of material and standard testing are considered acceptable if recognized design codes are followed, such as the manuals by ACI, ASTM and Portalnd Cement Association. The most updated editions should be used whenever possible. The guidelines provided here emphasize only those unique to armor units.

8.1.1 Analysis

The design should consist of performing stress analyses:

A. Analysis should be performed to determine the maximum preplacement stress level in the unit and the critical areas or surfaces of possible fracture, shear or crack failures. The preplacement stress condition should include at least shrinkage and temperature.

B. Possible strength degradation due to transportation should be estimated.

C. Analysis or experiment should be performed to determine the strength degradation due to sea water, marine biological effect and thawing and freezing.

D. Analysis should be performed to determine the stress condition and stability under design static loadings. The loading combinations should include, at least, the following:

(a) Dead load and DBS wave, current and gust loads. The dead load should include not only the compressive loadings due to superstructures or other armor layers, but also tensil loadings due to adjacent interlocking units.

(b) Dead load, DBS wave current and gust loads, ice and snow loads.

These loads should be applied at locations that will result in maximum stresses at the critical stress planes as determined earlier.

E. Analysis should be performed to determine the stress condition under dynamic loadings for the following cases:

(a) Wave impact plus rocking.

(b) Ship impact.

(c) Seismic loading.

(d) Diagonal and axial rollover.

8.1.2 Testing

Drop tests should be performed to determine the impact resistance and impact fatigue resistance due to the following loading conditions:

A. Impact Resistance Test: The purpose of this type of test is to determine the impact resistance of the armor units under extreme design impact loadings. Such impact loadings include:

(a) Accidental placement--vertical and swing drops should be properly simulated.

(b) Accident due to ship collision--the expected impact energy to be absorbed by an individual unit and the expected peak impact force should be preserved or properly simulated.

B. Impact Fatigue Resistance Test--The purpose of this type of test is to determine the impact resistance of armor units under repetative impact loadings. Such impact loadings include:

(a) DBS wave impact.

(b) DBS wind loading and induced rocking.

In both cases, the expected impact energy and peak impact forces should be carried out to the extent that a reasonable safety margin can be established with a designated confidence level.

8.2 Caisson Structure

The caisson type structure has been used quite extensively as a breakwater element. In Japan, for instance, most of the breakwaters employ the caisson one way or the other. Extensive research work has been carried out in Japan. Design manuals and guidelines are also available--most of them in Japanese. Some of the fundamental procedure guidelines are provided here.

8.2.1 Scope:

The following damage modes should be considered:

A. The stability of the caisson including sliding, overturning and collapsing. A safety factor of no less than 1.5 should be employed.

B. Structure cracking, ruptures and sections being sheared off. A safety factor of no less than 1.5 should be used.

C. Local damage and rate of deterioration (This is particularly important if waves are expected to break on structures).

D. The stability of toe protection on rubble mound foundation (if any) including disintegrating and sliding.

E. The stability of the structure as a whole including settlement, foundation failure and scouring.

The following load combinations should be analyzed:

A. DBS wave + DBS wind + DBS current + extreme high and extreme low water.

B. OBS wave + OBS wind + OBS current + high and low water.

C. Tornado + low water.

D. Differential pressure and possible overtopping due to tsunami + normal wave + normal wind.

E. DBE earthquake + low water + normal wind and wave.

F. Ship collision + DBS waves and water level.

8.2.2 Procedures:

The hydraulic design should follow, whenever applicable, the following manuals.

A. Design Standard of Port and Harbor Structures, Japan Association of Port and Harbors, 1969 (in Japanese).

B. Design Handbook of Shore Structures, Japan Association of Civil Engineering.

C. Shore Protection Manual, U.S. Army Coastal Engineering Research Center, 1973.

For structural design, the latest editions of ACI building codes for reinforced concrete, of other manuals by ASTM and Portland Cement Association should be closely followed.

For seismic design the guidelines published by NRC should be conformed to.

CHAPTER 9 HYDRAULIC MODEL TESTING

Model testings are essential in breakwater design. Experiments are usually designed to serve three different purposes: (1) to determine the value of coefficients that are required in analysis, (2) to perform material testings for the design of individual structural elements that are not analytically tractable and (3) to test the structural stability and suitability as a whole against design conditions. Tests of the third kind only are discussed here.

9.1 Scope

The hydraulic model test should be so designed as to provide the necessary information required to fulfill the Design Specification Requirements stipulated in Chapter II. The following safety-related items which are usually untractable analytically should be obtained through model testing.

A. Breakwater armor stability under DBS and degree of damage.

B. Breakwater armor stability under OBS and degree of damage.

C. Breakwater armor stability under water-depth-limited waves and degree of damage.

D. Breakwater stability safety margin against waves exceeding design value.

E. Breakwater stability safety margin against storm of extended period.

F. Breakwater stability and degree of damage under OBA.

G. Stability of toe protection and degree of scouring.

9.2 Procedures

The hydraulic modeling theory is a widely studied topic. A large number of pertinent references can be found. For breakwater testing, in general, the following rules should be observed.

A. The micro and macro geometrical scales should be properly modeled.

B. The material density should be properly modeled.

C. The frictional characteristics should be simulated as close as possible.

D. For testing concerning mainly with wave loadings the inertial and gravity forces should be properly preserved.

E. For testing concerning mainly with current effect the velocity-induced forces should be properly simulated.

It is recommended here that the following conditions be met in testing breakwaters used as nuclear power plant protections.

A. The storm condition should be properly simulated.

B. The breakwater should be tested against both regular waves and irregular waves.

C. The scale should be sufficiently large to minimize scale effects. The probable scale effects should be clearly identified and appropriate corrections should be made.

D. For wave force experiments, both two-dimensional and three-dimensional tests should be conducted to insure that the test conditions encompass the most destructive combinations of wave heights, wave periods and wave angles of approach.

E. For current force experiments, three-dimensional tests should be conducted.

F. The method of damage measurements and description should be consistant and objective.

G. The model construction procedure should closely simulate the prototype construction.

CHAPTER 10 DESIGN REVIEW INFORMATION REQUIREMENTS

To facilitate review, design information should be clearly documented accompanied with necessary illustrations, drawings and tables. When design codes are used, sources of information should be identified. The information commonly required for review is discussed here.

10.1 General Description

Narrative description of the function of the breakwater and its geometry, cross-section, major structural feature should be given with proper illustrations.

10.2 Design Criteria

The limiting performance criteria under various design loading conditions should be described. The performance criteria should include allowable structure damage, degree of wave overtopping and basin agitation.

10.3 Design Factors

A. Environmental Factors--Information on design environmental parameters which, in general, include those listed in Table 10.1 should be provided. Any additional parameter pertinent to the specific plant location should also be included. Methods and means employed in determining these parameters and risk analysis should be presented. Data and their source or origin should be clearly presented.

B. Geological and Soil Information--Material to support (1) foundation design, (2) near-field wave interaction analysis, (3) erosion assessment and (4) liquefaction analysis should be presented. The following information and analysis are generally required.

   (a) Hydrographic survey--(1) One to five foot contour maps required for breakwater design and construction purposes, (2) Ten foot contour maps required for wave refraction computation.

(b) Soil information--(1) documentation of field data, (2) documentation of seismic profile survey, (3) documentation of laboratory experiments and analyses including instrumentation, test procedures and results.

10.4 Design and Analysis

The information should include:

A. Method of analyses and assumptions.

B. Computational schemes and procedures.

C. Results and margin of safety.

D. Drawings and plans with dimensions.

10.5 Laboratory Experiments

The following information should be provided:

A. Purpose of experiments.

B. Experimental set-up and test procedures.

C. Instrumentation and measurement devices.

D. Data analysis.

E. Results and assessment.

10.6 Material, Quality Control and Construction Program

The following information should be included:

A. Material specifications including physical properties, engineering properties and chemical and biological stabilities.

B. Source of origins of material.

C. Quality control standards, procedures and results.

D. Material handling procedures including manufacturing, transportation and installation.

E. Construction program, inspection schedule and acceptance standards.

10.7 Inservice Surveillance Program

The following information should be provided:

A. Surveillance program.

B. Instrumentation and recording.

C. Warning system.

D. Maintenance and repair schedule.