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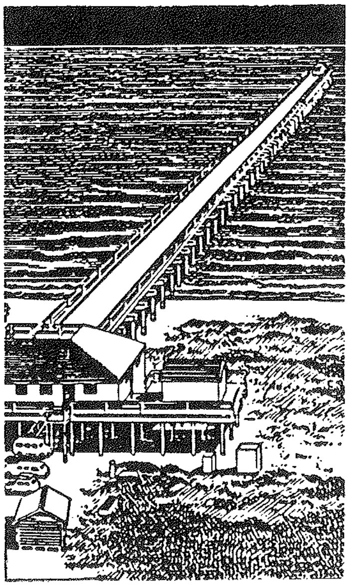

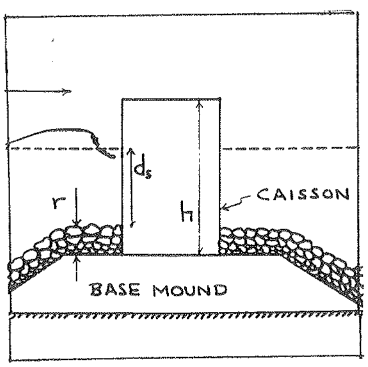

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WAVE FORCES ON VERTICAL WALLS

AN OVERVIEW OF RECENT WORK WITH ANNOTATED BIBLIOGRAPHY

by

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The recent literature dealing with wave forces on a vertical wall has been reviewed. An annotated bibliography for literature since 1958 is presented. The method of Goda for calculating such forces (with or without a rubble-mound base) is outlined with some minor changes. The associated flow charts are also given, along with some recommendations for future work.

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PREFACE

The investigations presented in this report were authorized by the Office, Chief of Engineers (OCE), US Army Corps of Engineers, and performed as part of Civil Works Research Work Unit 32533, "Coastal Engineering Technical Notes," formerly, Work Unit 31234, "Developing Functional and Structural Design Criteria." Funds were provided through the Coastal Structures Evaluation and Design Research and Development Program administered by the Coastal Design Branch of the Coastal Engineering Research Center (CERC) at the US Army Engineer Waterways Experiment Station (WES).

This report was prepared based on work performed during 1 May 1985 to 30 September 1986 when the author was employed by the Coastal Design Branch, CERC, under Assignment Agreement, Title IV of the Intergovernmental Personnel Act of 1970. Dr. Yen-hsi Chu, Coastal Design Branch, was the Technical Monitor, and Mrs. Karen R. Wood, Coastal Design Branch, was the typist. Comments by Dr. Y. Goda, Deputy Director General of the Port and Harbor Research Institute of the Ministry of Transport, Japan, are gratefully acknowledged.

The work was conducted under direct supervision of Dr. Frederick E. Camfield, Chief, Coastal Design Branch, and Mr. C. E. Chatham, Jr., Chief, Wave Dynamics Division; and under general supervision of Mr. Charles C. Calhoun, Jr., Assitant Chief, CERC, and Dr. James R. Houston, Chief, CERC.

Acting Commander and Director of WES during publication of this report was LTC Jack R. Stephens, EN. Dr. Robert W. Whalin was Technical Director.

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WAVE FORCES ON VERTICAL WALLS:

An Overview of Recent Work with Annotated Bibliography

PART I: INTRODUCTION

The Aim of this Report

1. The goal of this work is to provide an annotated review of the recent literature on wave forces on a plane, vertical wall. This goal was changed slightly early in the work. Many of the pertinent studies have considered a vertical wall (usually a concrete caisson) which rests on a rubble-mound base. This base alters the wave characteristics near the wall, and hence the induced pressures and total force on the wall. Since so many investigators have included such a base and many actual structures also have a base, it was included in the overview.

2. Some important quantities are not reviewed herein. The uplift pressures on the base of the wall or caisson are not discussed. Also, studies of the slamming of ships, the hydrodynamics of which is related to shock pressures due to breaking waves, are not reviewed.

Early Work

3. Some point in time must be picked in order to divide the work on wave forces into "early" and "recent". In this report, the paper by Rundgren (1958) was chosen to be the last paper describing "early" work. This paper has a very thorough and clearly written review of the literature which appeared before that time, and a corresponding complete bibliography. Moreover, the design formulas proposed in this paper appear to be the last ones considered for incorporation into the Shore Protection Manual (SPM) (1984). Also, this paper describes one of the first extensive series of laboratory experiments that were oriented towards obtaining working design formulas. This mode of attack on the wave-force problem has been continued by many others, as will become clear below.

4. Some general comments on the early work are in order, to set the stage for the recent work discussed below. Good reviews of this work are found in

the papers of Rundgren (1958) and Hudson (1953), and the books by Minikin (1963) and Wiegel (1964). With few exceptions, the following statements characterize the work. First, the "theory" is often rather incomplete, and is based on a Lagrangian treatment of the hydrodynamics. The water motions are rotational throughout the fluid, and thus the results are generally at odds with recent hydrodynamic treatments of wave motions. Second, the waves are taken to be periodic, and the intrinsic randomness associated with the sea state is neglected. Third, little experimental work was done. Most of the experiments that were carried out were done to study the nature of the "shock pressures" associated with waves breaking directly on the wall as shown by Figure 1. This is an exceedingly complex situation, and has not yet yielded, based on rational analysis, an agreed upon design formula. (The experiments were, of course, also hampered by the lack of sophisticated pressure sensors, and recording equipment). The design formulas that were used were based on a small number of field measurements, some hydrodynamic intuition, and observations of failures of actual breakwaters. That is, these formulas are based more on the creativity of the individual (and the dire need for <u>some</u> method of "rational" design) than on data, or on logical, hydrodynamic reasoning. The more recent work described below has sought to rectify this situation, while keeping in mind the ultimate need for a relatively simple, working design formula.

The Physical Situation

5. The situation to which the remainder of this papers refers, together with some of the notation used, is shown in Figure 2. Note that a base mound has been added to the simple vertical wall, as commented earlier. The stability of such a base mound is discussed by Tanimoto et al, 1982. The notation shown is chosen to be in accord with that in the SPM (below) when possible.

6. The forces or pressures discussed below will be the <u>additional</u> forces or pressures, due to the presence of the waves. Whether or not the no-wave hydrostatic pressure must also be taken into account in the design depends on the situation on the harbor side. The presence or absence of a base mound will be noted in each review given below. Note that d_s is water depth to the top of the base-mound armor layer. Also D, H_i, and L are the water depth,

wave height, and wave length just in front of the structure. Finally, the small offshore slope m shown in Figure 1 is often zero in laboratory work.

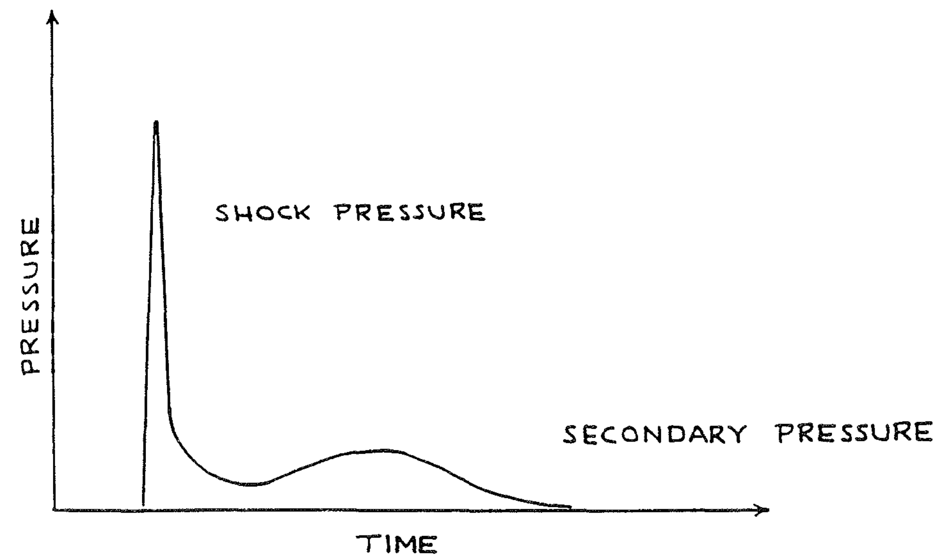

Figure 1. A typical example of the pressure exerted by a breaking wave on a vertical wall

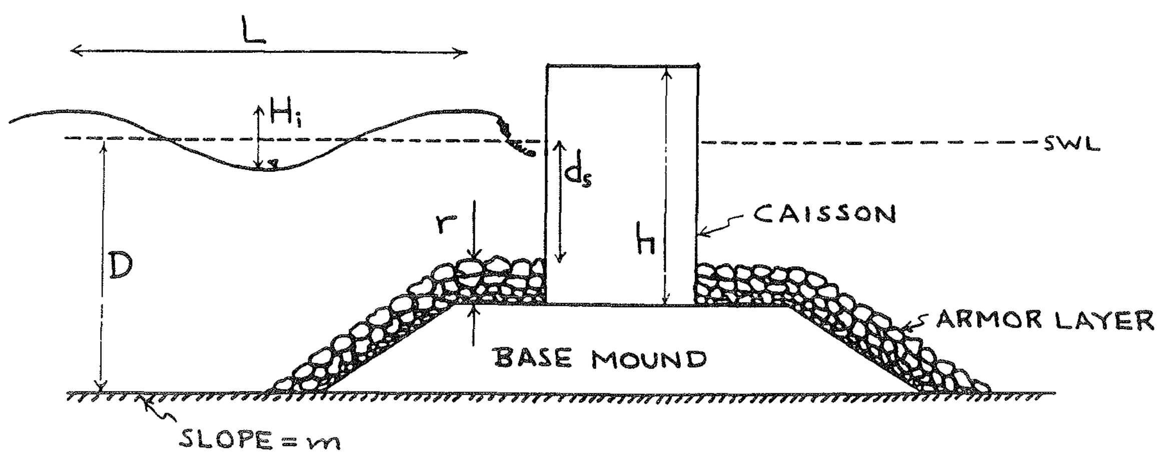

Figure 2. The situation considered, and the notation used in this report

PART II: OVERVIEWS OF RECENT WORK

7. The period around the late 1950's roughly marks the time when investigators began serious efforts to cope with the complexity of the problem. At the same time, the urgency for a solution increased, as more breakwaters were built in rather deep water. Research began on smaller pieces of the problem, and most significant progress usually was achieved by a rather prolonged effort of a group of people. Also, more emphasis was placed on finding a rational basis for design formulas. On the other hand, the need for a solution dictated that design formulas be made available relatively quickly, whether completely rational or not.

8. Compared to earlier work, and at the risk of over generalizing, recent research has been characterized by:

- a. A shift from Lagrangian to Eulerian theories for standing waves, and to higher-order Eulerian theories.
- b. Some efforts to do strictly numerical work (although these efforts have not been pursued very far yet).
- c. A shift from field measurements to controlled laboratory work, and consequent concern with the appropriate scaling laws.
- d. Detailed experimental studies of shock pressures.
- e. Exploratory studies of the response of the breakwater to wave forces.
- f. Exploratory work on the forces exerted by random waves.
- g. The formulation of "complete" models which, though useful, do not rest on a rational argument.
- h. The inclusion of more adjustable "constants" in design equations, to be found using models.

9. The result of the recent work is a number of more complex formulas, which apparently have yet to be fully compared. Although the effects of wave randomness have been included in some design techniques, there is still room for improvement. Also, most work has concentrated on the force on a wall when the

crest is present at the wall. It is not clear how to calculate the force when a trough is at the wall, especially when the waves are breaking.

10. Some groups stand out when discussing recent work. The Japanese work on vertical-wall breakwaters is distinguished by the intensity and persistence of the effort, and the volume of publication in English. The groups led by Goda and Nagai stand out in this respect. Unfortunately, the English publications are (understandably) compressed, so that it is sometimes difficult to assess the experimental techniques, or the reasoning leading to various formulas. Moreover, there is little cross reference, so that it is hard to compare the work of various groups. The Scandinavian work along these lines is also distinguished, most notably that at the Hydraulics Laboratory at the Swedish Royal Institute of Technology, and the Danish Hydraulic Institute.

11. An overview of recent work is given below, consisting of tables and brief comments. A more detailed, annotated bibliography of all of this work is found in Appendix A.

Experiments Using Monochromatic Waves

12. This work is shown in Table 1. Most of the recent work has been of this type. It seems that by now, essentially all of the straight forward work of first-order importance has been carried out, and that further refinements may be mainly of scientific interest. However, few of the results have been carefully compared with those of other, similar investigations. The same could be said about resulting design formulas. Such comparisons would be a worthwhile project.

13. Further improvements will most likely deal with the statistical distributions of maximum shock pressures, forces, and perhaps moments, and random waves. The random aspect seems to be the major ingredient lacking in this work.

Experiments Using Random Waves

14. The list of laboratory work done using random waves is quite short. However, this seems to be the most promising method of attack, especially if it can be shown that two-dimensional random waves represent a worst case (as

compared with short-crested waves, or two-dimensional waves striking a wall at an angle). There are undoubtedly more random-wave results by now; one can only hope that they will appear in the open literature. It seems that only when one can estimate the "n-year shock pressure" (and the associated breakwater response) can a rational design procedure be formulated. Much more laboratory work on simple, generic situations (as opposed to specific cases) on a wide variety of scales would be very useful in making progress towards this goal. The reported work is on two-dimensional waves. The papers are:

- a. Iwagaki et al (1980) (nonbreaking waves)
- b. Gravesen and Lundgren (1977) (breaking waves) See also Goda (1985)

Field Measurements

15. This work is shown in Table 2. ("Casual" or exploratory measurements are not listed.) The number of efforts to measure wave pressures on vertical walls seems to have diminished in the last few decades, save for very recently. This is probably due to a fuller appreciation for the complexity of the problem, and the consequent shift to laboratory work. As usual, such field work is distinguished by its scarcity, and also by the weakness of the conclusions that can be drawn from the results. The Russians seem to have done some work in this area (see the papers by Lappo and Zagryadskaya (1977), and by Huang and Zhao (1984); see also the article by Plakida in the 1965 PIANC report), which would be worth translating.

16. It would seem that large scale laboratory work under controlled, random-wave conditions would be more productive for the price than field work. However, this may be impossible for many directional wave spectra.

Standing Wave Theory

17. This work is shown in Table 3. As would be expected, theoretical work on nonbreaking waves has progressed much further than other kinds of wave force research. The pressures due to two-dimensional standing waves have been calculated to fourth order, and those due to waves approaching the wall at an angle other than 90 degrees to third order (with some rather surprising results). In

view of the better agreement at fourth order found by Goda, it may be worthwhile to extend this latter work to fourth order. Any more accurate calculations may need to consider the various viscous effects.

Table 1

Recent Experiments Using Monochromatic Waves

Authors	Standing/ Breaking?	Measured Force/Pressure?	Parameter Ranges: Mound	Wave Height (cm)	Wave Period(s)	Flume Length (m)
Garcia (1968)	B	P	No	2.5-9	1.5-2	20
Goda (1972)	S-B	P	No	7-42	1-2	30
Hayashi & Hattori (1958)	B	P	No	10-25	2	14
Homma & Horikawa (1965)	B	F	Both	10-13	0.8-2.5	17,36
Kirkgoz (1982)	B	B	No	4-8	0.8-1.6	13
Leedertse (1961)	B	F	No	6-18	1.9-3.3	33
Massel, Oleszkiewicz, & Trapp (1978)	B	F	Yes	?	1	10
Mitsuyasu (1962)	B	F	No	10	1-2.5	22
Mitsuyasu (1966)	B	P	No	10	1.9	25
Nagai (1960)	B	P	Yes	6-22	1-2	25
Nagai & Otsubo (1968)	B	P	Yes	5-38	1.2-3.0	50
Nagai (1969)	S	P	No	3.5-38	1-3.5	25,60
Richert (1968)	B	P	Yes	1-20	1.4-1.6	15
Rundgren (1958)	S-B	P	No	<4	1-2.5	24
Takezawa (1979)	S-B	F	No	2-23	1-2 8-30	25
Weigel & Maxwell Maxwell (1970)	B	P	No	?	?	33

Table 2

Recent Field Measurements

Author	Wall Type	Approx. Water Depth (m)	Pressure or Force?	Breakwater Response?	Waves Measured?
Blackmore & Hewson (1984)	Curved Reentrant Face	1-4	P	No	Yes
Huang & Zhao (1984)	Vertical	2-6	P	No	Yes
Kuribayashi, Muraki & Udai (1959)	Vertical	2-6	P	Yes	Yes
Marchi, Raiteri, Scarsi & Stura (1975)	Vertical	17	P	No	Yes

Table 3

Recent Theoretical Work on Standing Waves

Name	Eulerian/ Langrangian	Order of Accuracy in Slope	Waves Normal to Wall?	Numerical/ Analytic
Fenton (1985	E	3	No	A
Goda (1967)	E	4	Yes	A
Goda (1960)	E	2	Yes	A
Hsu, Tsuchiya & Silvester (1979)	E	3	--	A
Lappa & Zagryadskaya (1977)	L	3	Yes	A
Nasser & McCorquodale (1976)	E	(long, nonlinear waves)	Yes	N
Nichols & Hirt (1976)	L (marker and cell method)	--	Yes	N
Tadjbakhsh & Keller (1960)	E	3	Yes	A

18. It appears that there is a significant amount of Russian work in this area (see the paper by Lappo and Zagryadskaya). Again it would be worthwhile to translate this work.

19. It is a bit surprising that the presence of the base mound has yet to be taken into account. This may be because numerical treatments of the problem (almost certainly needed here) seem rather tentative to date. This would seem to be the next step.

20. Finally, little work has been directed to determining the response of a vertical wall to standing-wave pressures. This is almost certainly due to the fact that design pressures are usually associated with breaking waves. However, the problem is far more tractable with standing waves, and probably worth considering.

Breaking Wave Theory

21. This work is shown in Table 4. Unfortunately, the theoretical calculations of shock pressures are still somewhat sterile, despite much work. The models are closer to reality and usefulness than the pioneering work of Bagnold, but are not yet to the point of being used in design. It may well be that the most that can reasonably be expected is a pressure scaling law to use in conjunction with laboratory experiments. The work of Richert (1974) seem closest to being useful in design.

Table 4

Recent Work on Breaking Wave Pressures

Author	Model Description
Kamel (1968)	Algebraic tracking of impact-produced pressure shock waves through water (air cushion neglected)
Mitsuyasu (1966)	Numerical solution of Bagnold model for finite compression of air cushion, and air cushion with leakage
Richert (1974)	Numerical tracking of impact-produced pressure shock waves through water
Weigel & Maxwell (1970)	Numerical Model assuming that air and water are uniformly mixed

Breakwater Response

22. This work is shown in Table 5. The response of a breakwater to wave forces does not seem to have been studied before 1958. This response is, of course, a crucial step for the designer. The main point is usually whether the breakwater will slide on its rubble foundation, and perhaps how far. Finding the answer entails considering the interaction of sliding and rocking motions of the breakwater.

Table 5

Recent Work on the Response of Vertical-Walled Breakwaters to Wave Forces

Name	Rubble Mound?	Breaking/ Standing Wave?	Response Type	Laboratory/ Theoretical/Field
Goda (1974)	Yes	Both	Sliding	F
Hayashi & Hattori (1961)	No	B	Rocking	T
Hayashi (1963)	Yes	B	Sliding	T
Hayashi & Hattori (1964	Yes and No	B	Rocking, Sliding	T
Ito (1971)	Yes	Both (Random)	Sliding	T,L
Kirkgoz & Mengi (1985)	Yes	B	Plate Deformation	T
Lundgren (1969)	Yes	B	Rocking	T
Muraki (1966)	Yes	Both	Rocking	F
Nagai (1963)	Yes	B	Sliding	L
Nagai & Kurata (1974)	Yes	Both	Sliding	F,L

23. This problem has been considered analytically but not experimentally, at least in any controlled sense. However, although there are a number of weaknesses in the assumptions behind the theories, the theories are a reasonable first attempt at putting the design on a rational basis. As an example, it is not clear that sliding on an underwater rubble base is well understood, especially when the base pressure is uneven because of an associated rocking. It would be interesting to apply a known force to a laboratory breakwater, and compare the response with theoretical predictions. A good discussion of some other difficulties is found in Lundgren (1969), and the comments following that presentation.

PART III: GODA'S MODEL FOR PREDICTING WAVE FORCES

Introduction

24. There is a clear need to be able to predict forces on vertical walls, either with or without a rubble base, with reasonable accuracy. Any generally useful method must satisfy several criteria. It must be relatively simple to use, have some rational basis (without having to be entirely derived from first principles), and must cover many common situations. It would be nice to have a method which has been used fairly often (with apparent success), which has survived critical comment in the engineering literature, and which also has some "official approval." Finally, the model should cope with the fact that a natural sea state is random in nature, and should, at least in the surf zone, include breaking waves.

25. Goda's model is for random waves. No distinctions are made among nonbreaking, breaking, or broken waves. The formulas apply to vertical walls with or without rubble base mounds, but only when the wave crest is at the wall. The formulas give the additional force due to the waves, so that the still-water hydrostatic force must be added to give the total force on the wall. The effects of diffraction and refraction are not discussed below, but are in Goda (1985).

The Geometry

26. The geometry is shown in Figure 3. All depths are referred to still-water level. The symbols are not those of Goda, and are chosen to correspond to the SPM when possible. At the wall, the water height above still-water level when a crest is present is R. Note that with no base mound or toe protection, $D = d_s$. When there is toe protection but no base mound, $D = D_s + r$. The (small) bottom slope is m. Just before encountering the wall, the design waves have incident height H_i and move at an angle α, with respect to a horizontal normal to the wall.

Pressure Distribution

27. For now, assume that the design wave height H_i and wave direction α are known. The pressure distribution on the wall is described by straight lines connecting the points A, B., and C in Figures 3 and 4. The pressure vanishes at A, the point of maximum runup. It is a maximum p_H at the level of B (i.e., at still water level). It then decreases to p_L at the level of C, the top of the base mound (not at the top of the armor stone protecting the mound).

The maximum runup is

$$R = \frac{3H_i}{4} (1 + \cos \alpha) \qquad (1)$$

The pressure p_H and p_L shown in Figure 4 are

$$p_H = 1/2\ w\ H_i(1 + \cos \alpha)(\gamma_1 + \gamma_2 \cos^2 \alpha) \qquad (2)$$

$$p_L = \gamma_3\ p_H \qquad (3)$$

Here,

$$\gamma_1 = 6/10 + 1/2 \left[\frac{4\pi D/L}{\sinh(4\pi D/L)}\right]^2 \qquad (4)$$

γ_2 is the smaller of expression (a) and (b) below:

$$\text{(a)}\quad \frac{D_b - d_s}{3D_b} \left(\frac{H_i}{d_s}\right)^2 \qquad \qquad \text{(b)}\quad \frac{2d_s}{H_i} \qquad (5)$$

and

$$\gamma_3 = 1 - \frac{(r+d_s)}{D} \left[1 - \frac{1}{\cosh(2\pi D/L)}\right] \qquad (6)$$

In these definitions, L is the design wavelength at depth D, and is calculated from linear theory (e.g., SPM Appendix C-1). Finally, the new water depth D_b is the depth 5 $H_{1/3}$ seaward of the wall, where $H_{1/3}$ is the significant wave height at depth D. That is $D_b = D + 5\ m\ H_{1/3}$. When the breakwater is in the surf zone, the choice of height to be used for H_i will be based in part on H_{max} at the depth d_b. We now turn to estimating H_i and $H_{1/3}$.

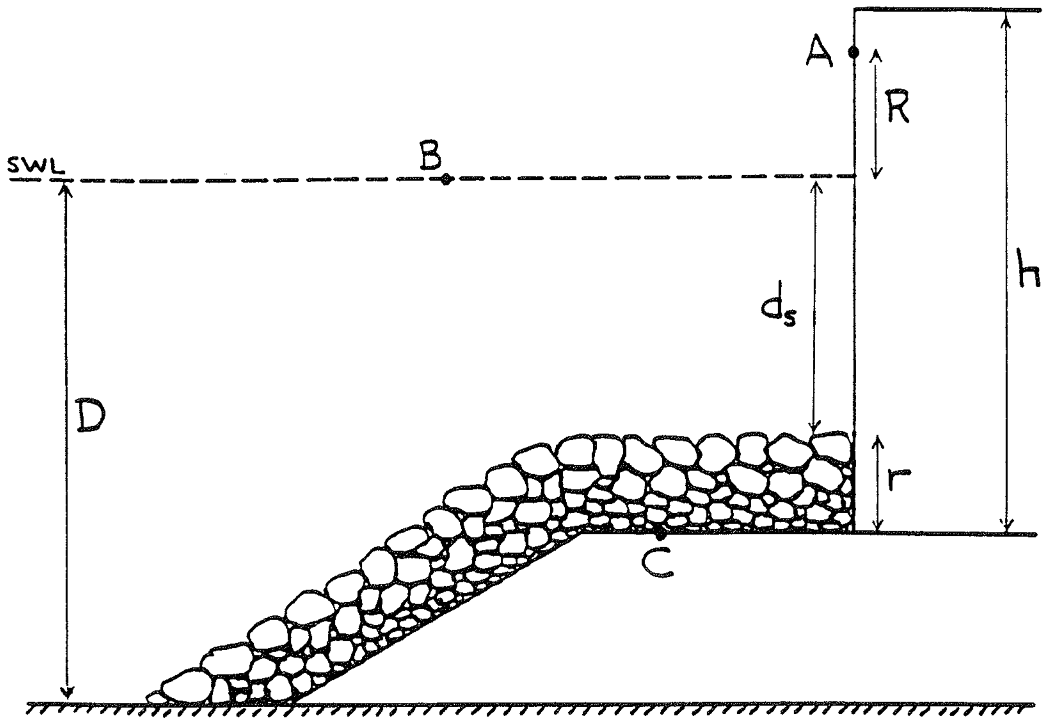

Figure 3. Geometry and notation for use with the Goda Method for estimating wave forces on a vertical wall. See also Figures 1 and 4.

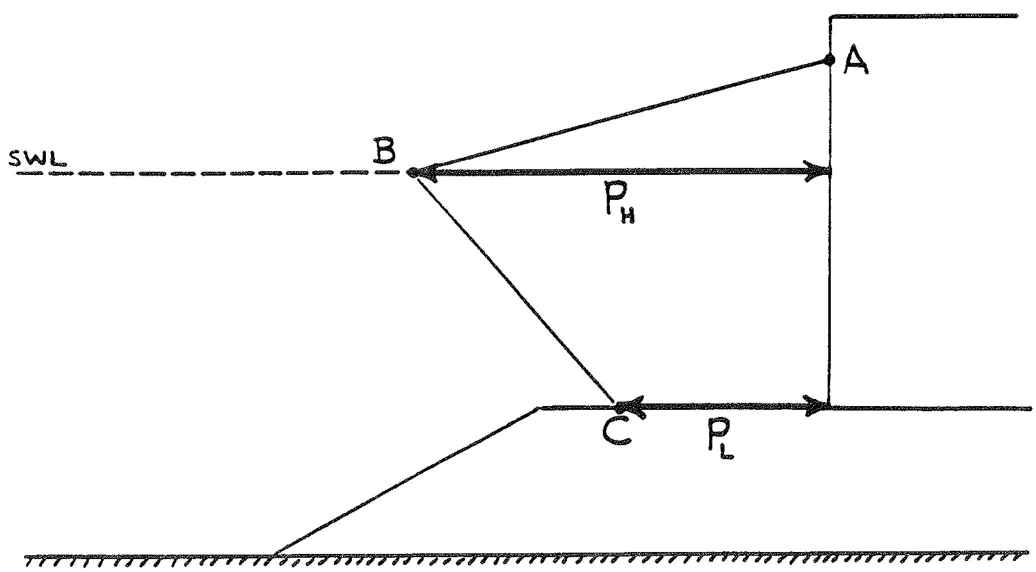

Figure 4. The pressure distribution assumed in the Method of Goda. See also Figures 1 and 3.

Design Wave Height

28. The design wave height is the maximum wave height at the wall. There are many ways to estimate nearshore wave heights. In order to be consistent with the pressure model of Goda given above, wave heights will be estimated using the Goda model for random, breaking waves in the surf zone, which has been found (again by Goda) to give satisfactory results both in the laboratory and in nature.

29. Goda's shoaling model gives estimates for the significant wave height $H_{1/3}$ and the "maximum" wave height $H_{max} = H_{1/250}$. The user must supply the significant offshore wave height corrected for refraction (H_o'), the corresponding wave steepness ($s = H_o'/L_o$), and the bottom slope (m). Following Goda, the offshore wavelength is found from $2 \pi L_o = gT_{1/3}{}^2$, where $T_{1/3}$ is the average period of the highest third of the offshore waves, and g is gravity. The quantities H_o' and $T_{1/3}$ are usually determined from a wave climatology, and refraction theory (see Goda (1985) and the SPM).

30. The Goda shoaling model specifies a Rayleigh distribution of offshore wave heights and a linear probability of wave breaking at the waves approach shore. It adds a fraction of the broken-wave energy to the smaller, unbroken waves. The effects of radiation stress and surf beat on still water level are taken into account. A better (but still incomplete) discussion of the complete model is found in Goda (1985), along with graphical results.

31. Outside of the surf zone, wave breaking is not important. Here, the wave height can be found by using a shoaling coefficient K, defined by $H_{1/3} = KH_o'$, and found with the method given in Appendix B. Following the Goda model, this is always true for water depths greater than $L_o/5$. The design wave height H_i is then taken to be $H_i = H_{max} = 1.8\ H_{1/3}$ at depth D.

32. When the wall depth $D < L_o/5$, the wall may be in the surf zone, and the equations for $H_{1/3}$ and H_{max} are those given in Appendix C. Now we must usually determine $H_{1/3}$ at $d = D$, and then find the depth

$$D_b = D + 5\ m\ H_{1/3} \tag{7}$$

We then find the largest value of H_{max}, for water depths between D and D_b. This largest value is taken to be the design wave height H_i. (This method differs from that of Goda, in which H_{max} at $d = D_b$ is always used as H_i. The new procedure seems slightly more conservative, especially when the wall is near the outer edge of the surf zone. However, it is somewhat tedious when the calculations are done manually).

33. In either case, we must find the quantity D_b, in order to find the pressure-distribution parameter γ_2.

Finding α

34. The estimated value of α is decreased by 15 degrees to account for our inability to measure wave directions accurately. (However, it is not decreased below zero).

Total Horizontal Force - No Overtopping

35. The increase in force per unit distance along the wall due to the presence of the waves is found from the pressure distribution shown in Figure 4:

$$F_w = 1/2\ (p_H + p_L)(r + d_s) + 1/2 p_H\ R \tag{8}$$

To this must be added the still-water hydrostatic force $F_s = w(r + d_s)^2/2$. Then the total force per unit distance along the wall is

$$F = F_s + F_w \tag{9}$$

Total Horizontal Force - Overtopping

36. If R is large enough that water runs over the top of the wall, the pressure distribution is simply truncated at this point. That is, p_H remains the same, but the pressure at the wall top is

$$p_T = p_H \frac{(R - h + r + d_s)}{R} \tag{10}$$

and the additional force due to the wave is

$$F_w = 1/2\ (p_H + p_L)(r + d_s) + 1/2(p_H + p_T)[h - d_s - r] \tag{11}$$

37. A schematic version of one way to proceed is shown in Figure 5. As above, neglect any effects of refraction or diffraction, and assume that you know H_o', $T_{1/3}$, and m. (Of course, you also know d_s, r, h, and D). You want to estimate F_w. Assume no overtopping. The changes to account for overtopping are straight forward (see above).

38. A safety factor of at least 1.2 has been used by Japanese engineers with the Goda approach. The coefficient of friction between concrete and a rubble base mound is usually taken to be 0.6.

Comments

39. Because of the arguments given in the introduction to this Section, it is hard not to regard the combined Goda shoaling and wave-force model as the best available (at least, for random waves). However, the Draconian methods used to obtain it leave room for improvement in the future. For example, the definition $H_{max} = H_{1/250}$ and the formula used to estimate surf beat seem amenable to improvement, at least in terms of statistical reliability. Also, the problem of whether or not (or when) shock pressures are important to design is not yet well answered.

40. It is also frustrating that one does not have easy access to the large amount of data that went into formulas such as those of Goda and Nagai. Moreover, the available literature does not contain comparisons of data to formulas. One has difficulty following, and judging, the reasoning process in going from data to the resulting design formulas, especially when the reasoning is somewhat intuitive. Thus, while it seems fairly certain that the Goda model given here is appropriate for design use, it also seems that perhaps more use could be made of the same data, and that the model could be improved with more data, at least on the grounds of its statistical certainty, and its treatment of the very high shock pressures associated with a small fraction of the breaking waves.

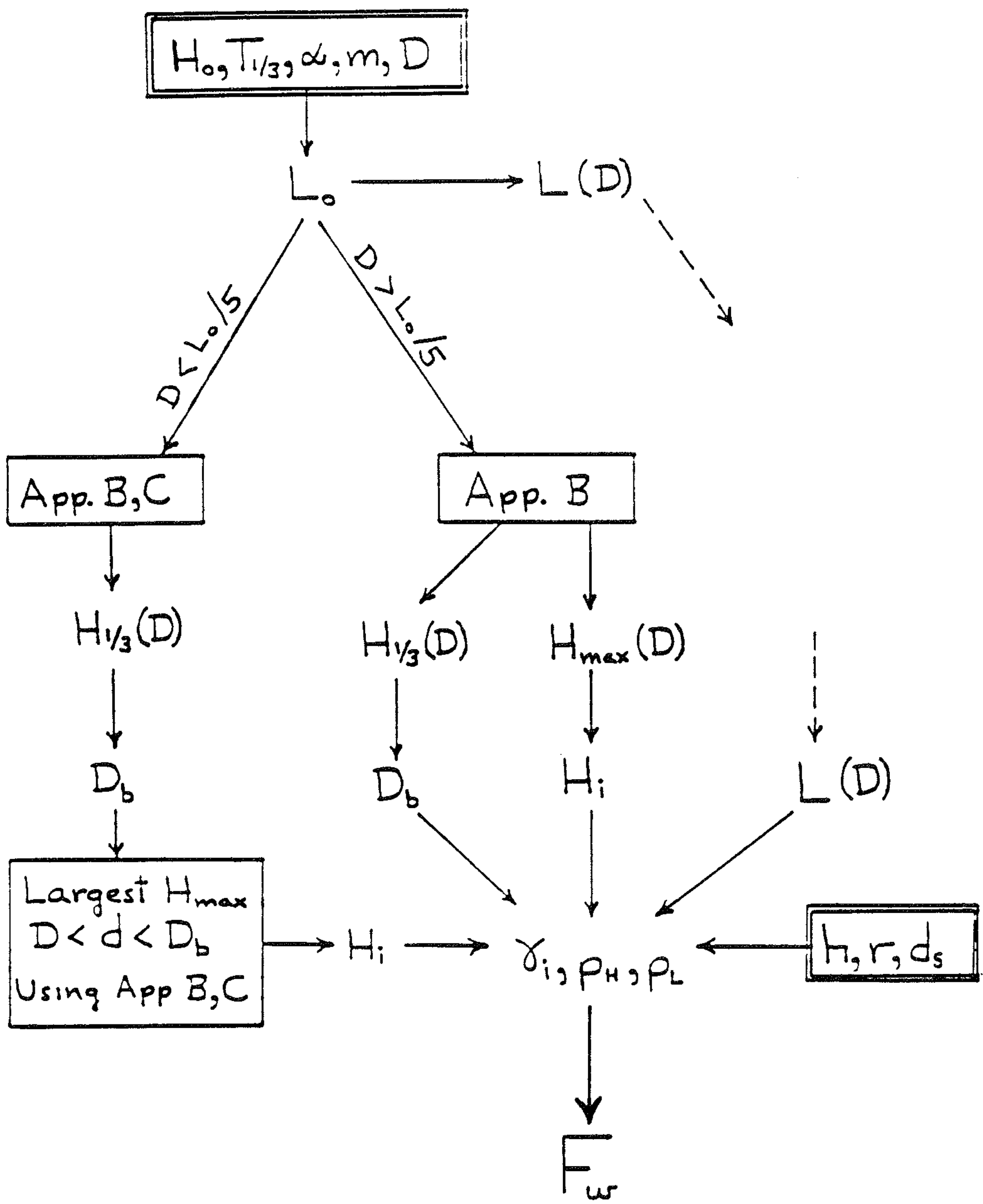

Figure 5. A flowchart for using the method of Goda for estimating wave forces on a vertical wall

PART IV: CONCLUSIONS AND RECOMMENDATIONS

41. A review of the recent literature on wave forces on a vertical wall suggests that the available theory for nonbreaking standing waves is probably quite adequate for choosing a design wave (and wave forces), while that for waves breaking on walls is still far from yielding reliable design formulas. Experiments with monochromatic waves have been pushed to the limit of their usefulness. However, experimental work using random waves is only just beginning, but shows great promise. It is still unclear what characteristics of the wave spectrum need to be modelled in order to give a reasonable simulation. The extent to which directional aspects of the spectrum are important is also unclear. It would seem that much more experimental work using random waves is needed, especially with rather simple, often-encountered breakwater configurations, and probably at a fairly large scale. A first step may well be a careful comparison of the existing data. (Such a comparison was beyond the scope of the present review).

42. It seems that we are at or near the end of productive activity in certain areas. Work with monochromatic waves in the laboratory has probably been pressed as far as possible, with respect to reflecting the natural situation. Standing-wave theory also is essentially complete, same for the inclusion of a partially absorbing rubble mound, and perhaps the inclusion of the ciasson response. (However, nonbreaking waves rarely give the design criteria used for breakwater design). Theoretical work on breaking waves and shock pressures will, it seems, remain somewhat sterile until a better understanding of turbulence in a two-phase flow is achieved, and incorporated into models of breaking waves.

43. Large-scale field tests are still extremely difficult. Unless a major, prolonged effort is undertaken, perhaps by several groups, it is unlikely that solid results will be obtained.

44. This brings us to laboratory tests using random waves. This work has been underway for a decade, but much remains to be done (although many results are proprietary, and thus not available for perusal). Most tests use two-dimensional random waves; the extension to three dimensions would seem to be the final step to achieving a realistic physical model. However, there are very few basins where such work could be accomplished.

45. It may be that small-scale field testing (in a small lake) would be a viable alternative. Directional spectra would be present, but the scale would be small enough that the large number of measurements needed for statistical reliability would not be prohibitively expensive.

46. The problem of wave forces on walls is both common and important. Many groups around the work make such measurements. It seems that a first step would be to collect all available data on unidirectional random tests, especially in fairly generic (i.e., not very site specific) situations. Such a "catalog" of results could then be updated on a regular basis, and form a reference for design engineers.

47. Despite eighty years of thought and work on this common and important problem, it seems that there is still far to go to produce a statistically reliable model. The first step in further major progress should be a thorough comparison of previously obtained data. The next step may well be so large that it will either entail cooperation among a number of laboratories, or never be carried out because of the effort entailed.

48. A slightly modified version of Goda method is presented, along with ancillary procedures for determining nearshore wave heights. (The difference lies in the manner in which the design wave height is chosen. The method given here seems slightly more conservative in the case where the wall is near the outer edge of the surf zone). The Goda method skirts the problem of estimating shock pressures (or forces) (Goda, 1985) by designing so that such pressures will not occur.

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APPENDIX A: AN ANNOTATED BIBLIOGRAPHY OF THE RECENT LITERATURE

ANNOTATED BIBLIOGRAPHY OF RECENT WORK ON WAVE FORCES ON VERTICAL WALLS

Introduction

A list of the articles in English on the topic of wave forces on vertical walls is given below, together with a short review of their content. Only articles which have appeared since 1958 are listed. The referred literature given below is probably almost complete. However, some technical reports may have been overlooked. In addition, some articles have been cited in the articles reviewed, but could not be obtained and reviewed. These are listed at the end, together with the paper in which they were cited.

0001 **AARTSEN, M. A.** 1957. "Model Study on the Impact of Waves," Proceedings of the 6th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 45.

This is a brief interim report on an investigation of wave forces on vertical and slanting sluice gates. It is included primarily as an example of early experimental work of this type, and as an example of Dutch work in the field (much of which does not appear in the refereed English literature).

0002 **AARTSEN, M. A. and BENIS, W. A.** 1959. "Model Investigations on Wave Attack on Structures," 8th Congress of the International Association for Hydraulic Research, Vol. 1, pp 22-a-1 to 22-A-13.

Although this report is concerned with wave forces on curved walls, it is mentioned as an example of early, systematic work on pressures due to breaking waves. It also describes work conducted at the Delft Hydraulics Laboratory, much of which does not seem to have made its way into the easily accessible English literature.

Waves are generated, by both wind and a wavemaker, in a large flume. Various wave spectra are considered, and typical pressure data shown. The complete results are given in a separate report.

0003 **ACKERMANN, N. L. and CHEN, P.** 1974. "Impact Pressures Produced by Breaking Waves," Proceedings of the 14th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 104.

A flat plate was dropped onto a still water surface in a vacuum tank, to test the effect on an air cushion on the resulting impact pressure. The impact pressure was found to diminish with decreasing air pressure. This finding seems to have important implications for the models currently used to establish shock pressures.

0004 **BATTJES, J. A.** 1982. "Effects of Short-Crestedness on Wave Loads on Long Structures," Applied Ocean Research, Vol. 4(3), pp. 165-172.

The author calculates the reduction of the wave force on a vertical wall due to the fact that the waves are short-crested, using linear spectral-transfer theory. Specific results are obtained for a directional spectrum in which energy is distributed according to the cosine (squared) of direction from the wind, and for various structure lengths. The use of linear transfer theory is a detriment to the use of the theory for the case of breaking waves. However, the work seems seminal, and will eventually have to be refined to the point of being useful to design.

0005 **BLACKMORE, P. A. and HEWSON, P. J.** 1984. "Experiments on Full-Scale Wave Impact Pressures," Coastal Engineering Vol. 8, pp. 331-346.

Field measurements of the shock pressures due to waves breaking on a slightly curved, vertical breakwater are reported. The Minikin formula is found to over estimate such pressures by an order of magnitude. A model is formulated, starting from the impulse-momentum equation applied to the water near the wave crest, which relates the maximum shock pressure to the rise time of the pressure-time curve. However, the result contains an unknown coefficient. A bound on this coefficient is found from the pressure data. A model for the pressure distribution over the entire wall is then proposed. This model seems at present to be based on a rather small amount of data.

0006 **CHAN E. S., and MELVILLE, W. K.** 1984. "Deepwater Breaking Wave Forces on Surface Piercing Structures," Oceans '84, Marine Technology Society, pp. 565-570.

Some preliminary measurements of shock pressures associated with deep-water waves breaking on both vertical and inclined plates are reported. Although the maximum pressure varies greatly from one experiment to another, the pressure impulse is repeatable. There is no discussion of the relation of these data to design criteria.

0007 **FENTON, J. D.** 1985. "Water Forces on Vertical Walls," Journal of the ASCE Waterway, Port, Coastal and Ocean Engineering Division, Vol. 111, pp. 693-718.

The force and moment on a wall due to waves approaching the wall from any angle are calculated, using an expansion to the third order in wave height. Breaking is not considered: the waves are completely reflected. The resulting formulas are compared to the experimental values of Goda and Nagai. The formulas seem reasonably accurate, although cumbersome. The maximum force per length of wall is shown to be associated with waves striking the wall at an angle, rather than with normally incident waves. Also, the greatest onshore force sometimes does not occur precisely under a wave crest, and the largest force can be associated with a wave trough, and be directed offshore. It would seem important to test these predications in the laboratory.

0008 **FUHRBOTER, A.** 1969. "Laboratory Investigation of Impact Forces," Symposium on Research on Wave Action, Vol. 2, Delft, pp. 1-26.

The water impact generated by a jet suddenly diverted to impinge on an instrumented striking area was measured, in an attempt to simulate the shock pressures associated with breaking waves with a model more amenable to theoretical treatment, and with higher pressures. Histograms of maximum pressures from many similar experiments are presented. The damping effect of a thin sheet of water on the striking area is also illustrated. The experimental results are compared with theory presented by the same author in a previous paper. Shock pressures do not seem to scale according to the Froude law.

This paper is typical of the research that seems to be needed in order to understand wave-induced shock pressures. It also suggest that much has yet to be done to achieve such an understanding, and to be able to use this "rational" approach for design (or even to deduce the appropriate scaling laws).

0009 **GARCIA, W. J.** Sept. 1968. "An Experimental Study of Breaking-Wave Pressures," RR H-68-1, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

This is a nicely written account of some experiments done at the Waterways Experiment Station to measure breaking-wave pressures on a vertical wall. The experimental procedure is described carefully, and the raw data presented. The author proposes a simple relation between shock pressure and offshore wave energy, and a vertical distribution of maximum pressures similar to that of Minikin. He also shows that the secondary pressures (and thus, force) can be described quite well with the Sainflou method. It would be interesting and useful to test these conclusions using the data of other shock-pressure investigators (and vice versa). The bibliography is very complete.

0010 **GODA, Y.** 1967. "The Fourth Order Approximation to the Pressure of Standing Waves," Coastal Engineering in Japan, Vol. 10, pp. 1-11. (See also a later "Errata" sheet for Vol. 10).

The theory of standing, monochromatic, irrational waves is developed to the fourth order in wave slope, and then compared to experiments performed in a long wave tank. (There is little information given regarding the experiments). Special attention is paid to the two peaks (per wave) in the pressure-time history, and the relation of this phenomenon to wave steepness.

Design diagrams for maximum offshore and onshore wave forces are presented. The effect of randomness of the wave train is also considered in a preliminary fashion. The monochromatic theory is shown to give results accurate to within 25 percent in this case.

0011 **GODA, Y.** 1972. "Experiments on the Transition from Nonbreaking to Postbreaking Wave Pressures," Coastal Engineering in Japan, Vol. 15, pp. 81-90.

Wave pressures on vertical walls both with and without a rubble base mound were measured in the laboratory, for several (gradually increasing) incident wave heights. The monochromatic waves were first (i.e., for small heights) standing waves, then broke at the wall, and finally (for sufficiently large heights) broke before reaching the wall. The pressures exerted on the wall were found to increase gradually and continously during this process. Also, for the gentle bottom slope of 1/100 used here, no dramatic shock pressures were observed. The presence of the base mound was found to increase the breaking-wave pressures on the wall. This paper seem to be one of the principal foundations for Goda's (1974) method of estimating wave pressures on a wall (in which method no distinction is made between breaking and non-breaking waves).

0012 **GODA, Y.** 1974. "New Wave Pressure Formulae for Composit Breakwaters," Proceedings of the 14th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 100.

This is an English rendition of the paper in which the pressure formulas now in use in Japan for breakwater design were first introduced. In these formulas, no distinction is made between breaking and nonbreaking (random) waves. Also, the very high shock pressures encountered in nature are ignored. This is done on the belief that one can design so that such forces will not occur (and that even if they do, much of the impulse will be absorbed by the elasticity of the foundation). However, Goda does present methods for deciding whether or not these shock pressures will occur, in his 1985 book.

The pressure formulas contain factors which have been evaluated experimentally. The resulting predictions are compared with a few experiments. However, much of the confidence in the formulas stems from their success in predicting the sliding failure (or the lack thereof) of full-scale breakwaters along the Japanese coast. The degree of success is significantly greater than that found using the Minikin, Sainflou, or Hirio methods.

0013 **GODA, Y.** 1985. "Random Seas and Design of Maritime Structures," University of Tokyo Press, Tokyo, pp. 323.

This is a well-written distillation of all of the author's work on wave forces on vertical walls (some of which is only available in Japanese). It presents formulas now used in Japan to design breakwaters with vertical walls. These formulas are also given in the design manual "Technical Standards for Port and Harbour Facilities in Japan, 1980" (available through the Overseas Coastal Area Development Institute of Japan), and are described in Section III of this report. The 1985 Goda book is a translation and revision of the first edition, which appeared (in Japanese) in 1977.

It is a bit surprising that so little of the other extensive Japanese work and proposed design formulas for wave forces is discussed in this book (e.g., the work of Nagai). However, the Goda approach is intimately related to his calculations of the heights of random waves in and near the surf zone. This may make such comparisons difficult.

The serious coastal engineer would do well to read Chapters 3 and 4 of this book (referring to Chapter 2 when necessary), before reading the original papers which are reviewed in this report.

0014 **GOUDA, M. A.**, "Hydrodynamic Wave Pressure on Breakwaters," Journal of the ASCE Waterways and Harbors Division, Vol. 86, 1960, pp. 13-25.

Second-order potential theory is used to calculate the pressure on a vertical wall due to standing waves. The results are compared with the pressure distributions of Luiggi and Cagli (see the book of Minikin for a description of these distributions). This paper has been superseded by several others. For example, see the reviews herein of Goda (1967) and Fenton (1985).

0015 **GRAVESEN, H., AND LUNDGREN, H.**, "Forces on Vertical and Sloping Face Breakwaters", 17th Congress of the International Association for Hydraulic Research, 1977, pp. 326-333.

This brief paper analyzes the forces exerted by random, breaking waves on a laboratory breakwater, using extreme-value statistics. The technique seems well-suited to the wave-breaking situation, and potentially quite useful. Several examples are shown, and the effects of various parameters such as mean water depth illustrated. However, the sensitivity of the design values (which are the ultimate products of the analysis) to the assumed extreme-value distribution is not discussed.

This method seems quite promising, but also seems to entail quite a bit of laboratory testing for each design problem. Perhaps a "catalog" of such results could eventually be built up. Also, systematic work with "typical" cases would be very useful, if only to determine the most important parameters. (It would appear that some measure of spectral width would be important, in addition to the significant wave height considered in the paper.) Finally, it seems that many of the sophisticated tools so useful in statistical hydrology could also be used in this approach. Moreover, hypothesis testing would be simpler, in that "long" time series are relatively easy to obtain.

0016 **Hashimoto, H.**, "An Experimental Study of Irregular Wave Forces on a Coastal Dike", Coastal Engineering in Japan, Vol. 17, 1974, pp. 71-79.

Forces and pressures due to irregular waves striking a vertical wall with no base mound were measured in the laboratory. Both breaking and standing waves were used. The spectrum and the wave-height distribution were found to narrow markedly as the waves approached the wall, so that in the breaker zone there was only a small difference between regular and irregular waves. A formula similar to that of Hironi is found to predict the force quite well, when used with the significant wave height, and the corresponding significant wave force.

0017 **Hayashi, T., and Hattori, M.**, "Pressure of the Breaker Against a Vertical Wall", Coastal Engineering in Japan, Vol. 1, 1958, pp. 25-37.

The shock pressures associated with waves breaking against a vertical wall with no rubble mound are measured, and correlated with $U^2/2g$, where U is the water velocity at impact. Solitary waves were used in the experiments.

The results showed a rather poor correlation between the two quantities, even though most formulas available at that time could be recast in such a form that one would expect a good correlation.

0018 **Hayashi, T., and Hattori, M.,** "Stability of the Breakwater Against Sliding due to Pressure of Breaking Waves", Coastal Engineering in Japan, Vol. 4, 1961, pp. 23-33.

The focus in this this is on the breakwater response to the force due to a breaking wave, rather than the force itself. However, such considerations must eventually be taken into consideration in design (although this does not yet seem to have been done in any systematic manner). The paper is a purely theoretical analysis of the rocking motion of a solid breakwater on rubble foundation, which is assumed to be elastic. The shearing resistance of the breakwater to sliding is shown to depend on the duration of the impact force, and on the natural period with which the breakwater rocks on the foundation. How to use the results in an actual situation is not made clear. It appears that this work has been followed up by Goda. (See Reference 4.15 in his book).

0019 **Hayashi, T., and Hattori, M.,** "Thrusts Exerted Upon Composite-Type Breakwaters by the Action of Breaking Waves", Coastal Engineering in Japan, Vol. 7, 1964, pp. 65-84.

This is an extension of the authors' 1961 paper (see above review). In this paper, the analysis is extended to consider both the shock and the secondary pressures associated with waves breaking on a vertical wall. Also, the analysis is applied to several actual breakwaters in Japan. Breakwaters constructed of horizontal blocks laid atop each other are also considered. The time history and maximum value of the shearing resistance (to sliding) are calculated. It is not clear why the minimum value of shearing resistance is not also of interest.

0020 **Hayashi, T.,** "Virtual Mass and the Damping Factor of the Breakwater During Rocking", Coastal Engineering in Japan, Vol. 8, 1965, pp. 105-117.

The effect of the virtual mass of a rigid wall on the rocking caused by waves breaking on the wall is studied, assuming a triangular shock-pressure distribution. Expressions are derived for the virtual mass and the virtual moment of inertia of the wall, by considering the wall rocking in still water. However, the calculated decay rate of rocking is much smaller than that actually observed.

0021 **Hom-ma, M., and Horikawa, K.,** "Wave Forces Against Sea Wall", Proceedings of the 9th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 31, 1964.

The pressure distribution due to broken waves striking a vertical wall are studied in the laboratory. Formulas governing this distribution are proposed, and then compared with field data. See also the 1965 paper by the same authors.

0022 **Hom-ma, M., and Horikawa, K.**, "Experimental Study on Total Wave Force Against Sea Wall", Coastal Engineering in Japan, Vol. 8, 1965, pp. 119-129.

A simple design formula proposed in a previous paper (1964; same authors) is tested against laboratory measurements of the total force due to waves breaking on a vertical wall. Bottom slopes of 1/15 and 1/30 were used; there was no base mound. The average total horizontal force (over a fairly wide scatter from wave to wave) is predicted well for the large slope, but only fairly for the small slope. The statistical distribution of the total force is also discussed briefly, as is the absorption of wave energy in the case where a protective rubble mound in front of the wall rises above still water level.

0023 **Huang, P., and Zhao, B.**, "The Probability Characteristics of Waves and Wave Pressures at a Vertical Breakwater:, Proceedings of the 19th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 50, 1984.

The probability distributions of wave heights, periods, and pressures (at various depths) are determined from field data taken near and along a vertical-wall breakwater. In shallow water, the wave-height distribution is no longer the Rayleigh distribution. The attenuation with depth of the spectral components of the pressure is also calculated (and found to behave according to linear theory). A new expression governing the equilibrium range of the wave pressure spectrum is proposed.

0024 **Ito, Y.**, "Stability of Mixed-type Breakwater", Coastal Engineering in Japan, Vol. 14, 1971, pp. 53-61.

The idea of a "probable sliding distance" is advanced as a means of evaluating the sliding stability of a composite breakwater under wave attack. The method seems potentially very fruitful. However, the calculations given in this paper make rather far-reaching assumptions about the sliding and static friction of concrete on rubble, and contain an adjustable constant (in the relation between wave-pressure duration and wave height). Despite this drawback, the paper is seminal, and the method seems to be capable of refinement (with more and better wave force and resistance data) to the point of being quite useful.

0025 **Iwagaki, Y., Sakai, T., Asano, T., Mase, H., and Koseki, M.**, "Experimental Study on Pressures Due to Irregular Standing Waves", Coastal Engineering in Japan, Vol. 23, 1980, 121-129.

The pressures on a vertical wall due to random, nonbreaking waves were measured at two water depths, and compared to those predicted by a third-order irrotational theory. Good agreement was found when individual waves were treated as one of an infinitely long train of periodic waves, especially in the absence of double peaks in the pressure trace. A linear filter was also used to estimate the pressure variation (over time at a fixed point) from water-level variations, and found to work when finite-amplitude effects are not too strong. No comparisons are made with design formulas.

0026 **Jensen, O. J.**, "Breakwater Structures", Coastal Structures '83, American Society of Civil Engineers, 1983, pp. 272-285.

This is a rather qualitative overview of the experiences of the Danish Hydraulic Institute with the hydraulic performance of the superstructures built atop breakwaters. While most of the material presented is not entirely pertinent to wave forces, such forces on vertical faces and their exceedance probabilities are discussed briefly. The paper is interesting in the present context primarily for its bibliography, which contains work not referenced elsewhere (and also not readily available).

0027 **Kamel, A. M.**, "Water Wave Pressures on Seawalls and Breakwaters", RR 2-10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., Feb. 1968.

Pressures due to both standing and breaking waves are considered, along with design wave heights in shallow water. However, the main part of the paper is concerned with advancing and testing a new and quite different formula for estimating the shock pressure associated with a wave breaking on a vertical wall. The formula is based on analytical considerations similar to those describing the water-hammer effect. The measured shock pressures are never more than half the theoretical values, and usually far less than that. A frequency diagram is introduced, in order to take this discrepancy into account in design techniques. However, the author finally considers it unnecessary to design against such shock pressures (or the associated forces), because of their short duration. Minikin's method for estimating the total shock force is evaluated, and is considered inadequate for use in design.

See also the 1970 paper by the same author, and his other 1968 report (RR EH-68-2).

0028 **Kamel, A. M.**, "Shock Pressures Caused by Waves Breaking Against Coastal Structures", RR H-68-2, U.S. Army Engineers Waterways Experiment Station, Vicksburg, Miss., Sept. 1968.

Shock pressures are measured by dropping a hinged plate onto a water surface. The results are compared with the "water-hammer" theory derived both here in in his Waterways Experiment Station Report 2-10 (1968), which is also reviewed herein. The ratios between the measured and theoretical shock pressures are found to fit a Poisson distribution. Here, too, the reader is cautioned against considering such pressures in estimating the overall stability of a massive structure such as a breakwater.

0029 **Kamel, A. M.**, "Shock Pressure on Coastal Structures", Journal of the ASCE Waterways, Harbors and Coastal Engineering Division, Vol. 96, 1970, pp. 689-699.

This paper is a condensed version of the two 1968 Waterways Experiment Station Research Reports by Kamel (RR 2-10, and H-68-2). See the reviews of those reports.

0030 **Kirkgoz, M. S.**, "Shock Pressure of Breaking Waves on Vertical Walls", Journal of the ASCE Waterway, Port, Coastal and Ocean Division, Vol. 108, 1982, pp. 81-95.

Monochromatic waves breaking on a vertical wall with no base mound are studied experimentally. The wall position is adjusted so that the waves exert the maximum pressure. Minikin's formula is found to underestimate the maximum pressure by up to a factor of two. The vertical pressure distribution is linear above the point where the pressure is a maximum, and parabolic below. The bottom slope and the breaking-wave height are found to govern the pressure distribution. To this reviewer, the conclusions of this quite recent study suggest that our understanding of the shock pressures associated with breaking waves remains at a very rudimentary level. See also Kirkgoz, 1983.

0031 **Kirkgoz, M. S.**, "Secondary Pressures of Waves Breaking on Seawall", Journal of ASCE Waterway, Port, Coastal and Ocean Engineering Division, Vol. 109, 1983, 487-490.

The secondary pressures associated with waves breaking on a vertical wall with no base mound are measured experimentally, and found to be on the order of the hydrostatic pressure (computed from the crest position). See also Kirkgoz, 1982.

0032 **Kirkgoz, M. S., and Mengi, Y.**, "Dynamic Response of Caisson Plate to Wave Impact", Journal of the ASCE Waterway, Port, Coastal and Ocean Engineering Division, Vol. 112, 1986, pp. 284-295.

The breaking-wave shock pressures measured and reported in earlier papers by Kirkgoz are used, together with elastic plate theory and the standard method of finite elements, to calculate the deformation of a caisson wall due to a wave breaking on the wall. The resulting deflections behave more or less as expected. A design procedure based on such calculations is proposed.

0033 **Kjeldsen, S. P., and Myrhaug, D.**, "Breaking Waves in Deep Water and Resulting Wave Forces", Proceedings of the 11th Conference on Offshore Technology, 1979, pp. 2515-2522.

Shock pressures due to waves breaking on a vertical wall in deep water are measured in the laboratory, in an exploratory program. The situation is not two-dimensional: the wall does not extend across the entire width of the wave tank. The measured shock pressure is found to vary with the type of breaking wave. The relation of such pressures to those occurring in shallow water is not discussed. The authors caution against scaling up laboratory data for use in nature.

0034 **Kjeldsen, S. P.**, "Shock Pressures From Deep Water Breaking Waves", Proceedings of the International Symposium on Hydrodynamics in Ocean Engineering (Trondheim), 1981, pp. 567-584.

Careful measurements of impact pressures, impact durations, and resultant shock forces due to deep-water waves breaking on a narrow, vertical plate in a wave flume are reported. Spilling breakers and steep, nonbreaking waves were used. The frequency of occurrence of shock forces was found to be very high. The model laws for "ventilated-shock" impact pressures due to breaking waves are discussed, and found wanting; such pressures may well be significantly different in salt water than in fresh water.

0035 **Kuribayashi, T., Muraki, Y., and Udai, G.**, "Field Investigation of Wave Forces on Breakwater", Coastal Engineering in Japan, Vol. 2, 1959, pp. 17-27.

Nearshore waves and the resulting pressures on a breakwater are measured at Harbour Harbor, Japan. The response of the breakwater is also measured, using an accelerometer. Some typical data are shown, categorized, and discussed briefly. The maximum shock pressures are about 14-15 tons/meter^2, and are associated with wave periods in the range 7-9 seconds. The period of breakwater rocking is usually about 0.2 second. This paper seems to be more of an interim report than a final, summary document.

0036 **Lappo, D. D., and Zagryadskaya, N. N.**, "Studies of Pressure and Energy of Standing Waves", Journal of the ASCE Waterway, Port, Coastal and Ocean Engineering Division, Vol. 103, 1977, pp. 335-347.

A rather involved Lagrangian calculation gives the pressures exerted by a standing wave on a vertical wall. It is found that the third-order results are closer to experimental values than are those of the fourth order. However, no details are given regarding the experiments. The paper gives us a somewhat narrow window on design theory and practice in the USSR. One gets the impression that theoretical work is relied on much more heavily in the USSR than in most other countries.

0037 **Leendertse, J. J.**, "Forces Induced by Breaking Waves on a Vertical Wall", Technical Report 092, U.S. Naval Civil Engineering Laboratory, 1961.

The total forces due to waves breaking on a vertical wall are measured, and discussed from the standpoint of impulse-momentum theory. This theory is then used to construct a design procedure which takes the net response of the structure into account. No suggestions are made on how to scale the experimental results up for use in actual situations. Although the parameter ranges considered are rather narrow (for example, the bottom slope was always one on ten), the results are certainly stimulating, and deserve extending and checking against the more recent work of others (e.g., Nagai).

0038 **Li, Y., Yu, Y., and Xoy, M.**, "Investigation of Wave Pressure on Vertical Wall", International Conference on Coastal and Port Engineering in Developing Countries (Colombo, Sri Lanka, March 20-26, 1983), pp. 755-776.

This paper is a summary of twenty years of research in the Peoples Republic of China on wave forces on vertical walls. The design formulas used for standing, breaking, and broken waves are presented and discussed. Some comparisons with the formulas of Goda are given. The bibliography is a useful introduction to work in the PRC and the USSR on wave forces on walls (most of which, of course, are not in English).

0039 **Lundgren, H.**, "Wave Shock Forces: An Analysis of Deformations and Forces in the Wave and in the Foundation", Symposium on Research on Wave Action, Vol. II, Delft, 1969, pp. 1-20.

This paper is composed of a series of short, incisive articles on various aspects of wave breaking. It discusses the physics of ventilated, compression, and hammer shocks, and the forces important to each process. The relevant scaling laws are then discussed, but not always decided upon. The physics of breakwater rocking (in response to the forces due to breaking waves) is also discussed at some length. The emphasis of the entire paper is on using laboratory results for design. No design formulas are proposed. See also the discussion following the paper.

0040 **Lundgren, H., and Gravesen, M.**, "Vertical Face Breakwaters", Proceedings of the Sixth International Harbour Conference (Antwerp, 1974, paper 2.11.

This is a nice, although very abbreviated, description of the Danish technique for designing vertical-face breakwaters. This technique allows design in terms of the "N-year" wave pressure. (Of course, the N-year breakwater response is probably more interesting: this may be the next logical step in any improvements made.) As is said elsewhere in the present report, the statistical approach used in the Danish method seems both logical and promising. However, it does necessitate laboratory modelling, at least until the results for a wide variety of design (and wave conditions) are available.

0041 **Marchi, E., Raiteri, E., Scarsi, G., and sturo, S.**, "Storm Wave Pressures on the Breakwater of Genoa Harbor Measurement Station", Proceedings of the 16th Congress of the International Association for Hydraulic Research A31, 1975, pp. 246-253.

A station for measuring wave pressures against the vertical seawall at Genoa Harbor is described, and some preliminary data are shown and compared to the Sainflou formula.

0042 **Marchi, E.**, "Problems of Vertical Wall Breakwater Design", Proceedings of the 17th Congress of the International Association of Hydraulic Research, 1977, pp. 337-349.

This brief overview deals mainly with forces on vertical walls due to breaking waves. Maximum shock pressures and various proposed scaling laws are discussed, along with uplift forces and risk criteria.

0043 **Massel, S. R., Oleszkiewicz, M., and Trapp, W.**, "Impact Wave Forces on Vertical and Horizontal Plates", Proceedings of the 16th Coastal Engineering Conference, American Society of Civil Engineers, 1978, pp. 2340-2359.

The shock forces due to waves breaking on a vertical wall atop a mound are measured in the laboratory. The forces are determined from the response of the wall, and are found to follow a Weibull distribution. Scale effects are investigated; it seems that Froude scaling should not be used to estimate design impact forces. The studies reported on seem to be of a preliminary nature; the final results and reports should be of some interest.

0044 **Mitsuyasu, H.**, "Experimental Study on Wave Force Against a Wall", Coastal Engineering in Japan, Vol. 5, 1962, pp. 23-47.

The net force due to monochromatic waves breaking against a vertical wall in the laboratory is measured, in the situation where there is no base mound.
The standard deviation of this force was also determined. The results are presented in terms of nondimensional force coefficients, which vary with water depth at the wall, deep-water wave steepness, a nd bottom slope. Graphs of these coefficients are given for slopes of 1/15, 1/30, and 1/50. There is a critical depth at which the force is a maximum, for fixed slope and offshore wave steepness. The measured forces were found to vary markedly, under similar conditions.

This work does not seem to have been continued by other investigators. Most subsequent work is couched in formulas, rather than in nondimensional coefficients like those proposed here.

0045 **Mitsuyasu, H.**, "Shock Pressure of Breaking Wave", Proceedings of the 10th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 18, 1966.

The shock pressures measured in laboratory experiments are discussed in some detail. An extended version of the Bagnold air-cushion model is considered. Numerical solutions for the pressure time history are obtained. The leaking of air from the air pocket is considered analytically. While these theoretical attempts to model breaking-wave pressures are quite interesting in their own right, they do not lead to any design-oriented formulas. (Nor have they in the twenty years following this paper.)

0046 **Mitsuyasu, H.**, "Shock Pressure of Breaking Waves", Coastal Engineering in Japan, Vol. 9, 1966, pp. 83-96.

This is a very careful experimental study of the detailed characteristics of shock pressures associated with waves breaking against a vertical wall. No theoretical analysis is offered, and no formulas or expressions of immediate practical value are presented.

0047 **Mogridge, G. R., and Jamieson, W. W.**, "Wave Impact Pressures on Composite Breakwaters", Proceedings of the 17th Coastal Engineering Conference, American Society of Civil Engineers, 1980.

Many experiments were conducted with very high frequency response data recording equipment to measure wave pressures due to waves breaking on the vertical faces of various designs of composite breakwaters. The shock pressures recorded are among the highest reported in the literature. No design formulas are proposed.

0048 **Mouzouris, C.**, "Influence of Ambient Air Pressure on Impact Pressure Caused by Breaking Waves", Internal Report No. 10-79, Delft University of Technology, Department of Civil Engineering.

This is a detailed experimental and theoretical study of wave breaking on a sloping beach. (The slope is neither large nor small - one on six). Special attention is paid to the role of entrapped air. The method could, perhaps, be applied to a vertical wall, although it is unclear whether some of the approximate models used would then apply.

0049 **Muraki, Y.**, "Field Investigation on the Oscillation of Breakwater Caused by Wave Action", Coastal Engineering in Japan, Vol. 9, 1966, pp. 97-106.

Displacement vibrographs are used to study the rocking of the breakwater at Harboro Harbor. The coefficient of bearing resistance is then calculated from the observed rocking period, following the approach of Hayashi and Hattori.

0050 **Nagai, S.**, "The Shock Pressures Exerted by Breaking Waves on Breakwaters", Coastal Engineering in Japan, Vol. 2, 1959, pp. 100-101.

This is a very short overview of the early work of Nagai on shock pressures. No experimental results are given; no formulas are proposed.

0051 **Nagai, S.**, "Shock Pressures Exerted by Breaking Waves on Breakwaters", Journal of the ASCE Waterways and Harbors Division, Vol. 86, 1960, pp. 1-38.

This is Nagai's basic paper: most of his later papers represent, to some extent, refinements of this work. Many experiments with solitary and monochromatic (but not with random) waves lead to empirical formulas for the maximum pressure, the vertical pressure distribution, and the maximum force due to waves breaking on a vertical wall, with and without a base mound.

Three different pressure distributions are found. Which distribution is to be used depends on the wave steepness and mound slope. Two maximum shock-pressure formulas are given, according to whether the breaking wave is "ordinary" or "extraordinary" (in which case the pressure is twice as high). According to Nagai's experiments, both formulas are superior to those of Minikin and Hiroi. The (Froude) scaling law is discussed, and used to find formulas applicable to prototype breakwaters. These compare favorably with French field measurements made in the 1930's. The "extraordinary" breaking waves seems most likely to occur with a small base-mound slope, although no general rule is given. The top width of the mound also seems important.

0052 **Nagai, S.**, "Shock Pressures Exerted by Breaking Waves on Breakwaters", Transactions of the American Society of Civil Engineers, Vol. 126, 1961, pp. 772-809.

This is the same paper as Nagai (1960). See that review.

0053 **Nagai, S.**, "Sliding of Composite-Type Breakwaters by Breaking Waves", Journal of the ASCE Waterways and Harbors Division, Vol. 89, 1963, pp. 1-20.

The equations proposed by Nagai (1960) are tested by determining the conditions under which a vertical-walled breakwater slides on its rubble-mound base, in the laboratory. Good agreement is found with these equations. Also, the use of Froude scaling is tested by comparing experimental results obtained with 1/10 scale and 1/20 scale models, and found to give good results. See also the discussion by Hayashi (November, 1963, pp. 91-97).

0054 **Nagai, S.**, "Pressures of Partial Standing Waves", Journal of the ASCE Waterways and Harbors Division, Vol. 94, 1968, pp. 273-286.

The effects of bottom slope, strong wind, a rubble-mound base, and wave overtopping on the pressures on vertical walls due to waves are studied. There is very little wind effect, and almost no effect of the bottom slope, at least for small slopes. The overtopping effect is estimated by simply ignoring the pressures above the top of the wall. Low rubble mounds have little effect on the pressure, if the wall is in deep water. In general, scale effects on wave-induced pressures can be neglected when scaling up to a prototype, when the model scale is greater than 1/25.

0055 **Nagai, S.**, "Pressures of Standing Waves on Vertical Wall", Journal of the ASCE Waterways and Harbors Division, Vol. 95, 1969, pp. 53-76.

The measured pressures on a (laboratory) vertical wall due to standing waves are compared to several existing theories. (The channel was level; there was no base mound.) Various formulas are found to describe the observations best, depending on the values of H/L and D/L (see Figure 1). The pressure is assumed to vary linearly between mean sea level and the wave crest, when using Eulerian theories.

See also the discussion of this paper by Goda (same journal, 1970, pp. 155-158).

0056 **Nagai, S., and Otsubo, T.**, "Pressures by Breaking Waves on Composite-Type Breakwaters", Proceedings of the 11th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 59, 1968.

Many experiments are performed in which monochromatic breaking-wave pressures on composite breakwaters are measured. Standing waves are found for $d_s/D > 0.75$ (see Figure 1) and are governed by the vertical pressure distribution (B) given in Nagai (1960). Most experiments are in the range where the ratio d_s/D has values from 0.4 to 0.75. In this range, a rather complicated dependence of the maximum pressure on D/H, B/D, and d_s/D is determined, and presented graphically. (Here, B is the horizontal distance along the top of the rubble mound, from the front of the caisson to where the mound becomes inclined.) A new type of vertical pressure distribution is also found. Ranges of the parameters d_s/D, B/D, D/H, and D/L in which breaking and standing waves occur (in the same d_s/D range as above) are also found, and shown in graphs.

0057 **Nagai, S.**, "Wave Forces on Structures", Advances in Hydroscience, Vol. 9, 1973, pp. 253-324.

This is an overview of wave forces on both circular cylinders and long, usually vertical walls with or without a base mound. It serves as a good introduction to the author's research on wave forces on vertical breakwaters, but does not seem to cover other work in the same area quite as thoroughly. There seems to be no new work reported. However, someone interested in using Nagai's results would do well to read this paper (and the appendix in Bruun's book: see Nagai 1976) before reading the original papers.

0058 **Nagai, S., and Kurata, K.**, "Investigations of Wave-Pressure Formulas due to Damages of Breakwaters", Proceedings of the 14th Coastal Engineering Conference, Ch. 101, 1974.

The observed slidings of actual breakwaters in Japan in the period 1959-1974 are investigated, using the formulas previously proposed by Nagai (see above reviews). Four specific cases are discussed in detail. The proposed formulas are found to be satisfactory in explaining the observations.

0059 **Nagai, S.**, "Wave Forces on Vertical-Wall Breakwaters in Deep Water", in P. Bruun, Port Engineering, Second Edition, Gulf Publishing Company, 1976 (pp. 554-561).

There is no new work reported here. Rather, it is a distillation and an apparent final summary of all of Nagai's research on wave forces on composite breakwaters. It is the counterpart of the breakwater section of the book of Goda (1985), and is probably the first reading to do in understanding the more practical contributions of Nagai.

0060 **Nasser, M. S., and McCorquodale, J. A.**, "Action of Nonlinear Waves at a Solid Wall", Proceedings of the 15th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 47, 1976.

A one-dimensional finite-difference model is developed to simulate the action of nonlinear, shallow-water waves striking a vertical wall. The numerical predictions of wave reflection and run-up compare reasonably well with experimental results. However, no forces are calculated.

0061 **Nichols, B. D., and Hirt, C. W.**, "Numerical Calculation of Wave Forces on Structures", Proceedings of the 15th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 132, 1976.

0062 **Plakida, M. E.**, "Pressure of Waves Against Vertical Walls", Proceedings of the 12th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 89, 1970.

Laboratory tests coupled with formulas coming from standard wave theories lead to design formulas for pressure and forces due to both breaking and broken waves acting on a vertical wall. The paper is highly condensed, but seems to portray standard (at the time) USSR design procedures. The references are only to work done in the USSR. There are no comparisons with other design formulas.

0063 **Ramkema, C.**, "A Model Law for Wave Impacts on Coastal Structures", Proceedings of the 17th Coastal Engineering Conferences, Ch. 139, 1978.

This is a nice review of previous work on shock pressures, although it does not deal directly with waves breaking on vertical walls. Also, Bagnold's piston model is generalized to include both adiabatic and isothermal compression of the air cushion, and to allow for compression of the water. A scaling law to be used with laboratory experiments is also suggested. This is an important paper for people interested in the theory of shock forces, and has a very complete bibliography.

0064 **Richert, G.**, "Experimental Investigation of Shock Pressures Against Breakwaters", Proceedings of the 11th Coastal Engineering Conference, American Society of Civil Engineers, Ch. 62, 1968.

The shock pressures of waves breaking on a vertical wall (and preceded by smaller, nonbreaking waves) were measured. Many experiments were conducted for each configuration (which usually included a base mound). The main results are shock-occurrence diagrams (shock pressures plotted against both wave height and wave period) and instantaneous pressure-distribution diagrams. Large scatter is encountered in the shock-occurrence diagrams. Some sloped walls were also studied. No design formulas are suggested.

0065 **Richert, G.**, "Shock Pressures of Breaking Waves", Bulletin No. 84, Hydraulics Laboratory, Royal Institute of Technology, Stockholm, 1974.

This paper describes an experimental and theoretical study of compression-shock pressures due to waves breaking on a vertical wall. The experimental part is a more elaborate version of the 1968 paper by the same author. However, the theoretical part is new. To begin with, it contains a nicely presented account of previous theories - a good first introduction to the topic.

A theory is then developed to relate breaking-wave characteristics to the resulting wave pressure and impulse on a wall. The wall is assumed to be perfectly rigid. The theory is designed to circumvent the need for assuming an equivalent hydrodynamic mass of water which is brought to rest. However, the assumed geometry of the pressure waves is complex, and necessitates a numerical integration over time to find the pressure maxima. The principal result is a scaling law to relate experiments to design values. The pressure scalings predicted differ considerably from those proposed by others, and from Froude scaling. Consideration is given to how to use this method in design. Although the method seems promising, the results are not yet especially simple to use.

0066 **Sellers, F. H.**, "Water Impact Loads", Marine Technology, Vol. 13, Jan. 1976, pp. 46-58.

This is a rather elementary introduction to shock pressures associated with the slamming of ships. It is only mentioned here because some authors (e.g., Kamel) have used similar approaches in studying pressures due to breaking waves.

0067 **Takezawa, M.**, "Experimental Study of Wave Forces on a Vertical Wall", Coastal Structures '79, American Society of Civil Engineers, Vol. 1, 1979, pp. 48-66.

Wave forces on a vertical wall with no mound base are measured in the laboratory. The wave type ranges from a standing wave to one which breaks before hitting the wall. The measured forces are compared to those predicted by the formulas of Sainflou, Hiroi, Minikin, and Hom-ma and Horikawa. The formula of Hom-ma and Horikawa gives fair agreement, but only for broken waves. The other formulas agree less well. A hydrostatic-type formula based on wave run-up is proposed, presumably to give an upper bound on the forces encountered.

0068 **Vinje, T., and Brevig, P.**, "Numerical Calculations of Forces from Breaking Waves," Proceedings of the International Symposium on Hydrodynamics in Ocean Engineering (Trondheim), 1981, pp. 547-565.

Forces on a vertical wall due to (plunging) breaking waves are estimated using potential theory, Cauchy's theorem, and a numerical solution of the resulting integral equation. The time variation of the calculated impulse gives the force. Only a few results are given for vertical walls, for times shortly after the breaker hits the wall. The emphasis of this paper is on the numerical procedure, rather than on calculated results or design formulas.

0069 **Weggel, J. R., and Maxwell, W. H. C.**, "Experimental Study of Breaking Wave Pressures", Proceedings of the Second Offshore Technology Conference (Houston, 1970), OTC 1244, Vol. 2, pp. 175-188.

Breaking waves and their associated shock pressures are studied experimentally. The breaking-wave geometry and wave-crest velocity are determined photographically. The shock pressures on a wall about one quarter the width of the experimental flumes are measured using piezoelectric transducers, and interpreted using a simplified momentum analysis. The relation of such pressures to those on a wall extending across the entire flume is not discussed, nor are any design formulas proposed. See also the paper by the same authors in the Journal of the Waterways and Harbors Division (ASCE).

0070 **Weggel, J.R., and Maxwell, W. H. C.**, "Numerical Model for Wave Pressure Distributions", Journal of the ASCE Waterways, Harbors and Coastal Engineering Division, Vol. 96, 1970, pp. 623-642.

A numerical model of the wave equation for the pressure of an air-water mixture is used to find the shock pressure due to a wave breaking on a wall. The air and water are assumed to be uniformly mixed. Comparisons with a limited amount of data seem favorable. The use of Froude scaling to relate model to natural shock pressures is questioned. No simple predictive equations for pressure or force are put forward. This work is somewhat related to that of Kamel (1968a,b; 1970), which is also reviewed herein. Some subsequent discussion between these authors is also given in the same journal (e.g., Feb. 1972, pp. 85-86).

0071 **Weggel, J. R.**, "Wave Loading on Vertical Sheet-Pile Groins and Jetties", U.S. Army Coastal Engineering Research Center Report CETA 81-1, Jan., 1981.

An approximate method is given for finding the force and moment on a vertical wall, in the case where waves strike the wall at an angle so that the crest moves along the wall. Experiments on the Mach-stem effect (see, e.g., Wiegel, 1964) are used to find the height of the wave at the wall.

0072 **Whillock A. F.**, "Forces on Sea Walls Under Oblique Wave Attack", Report no. It 225, Hydraulics Research Station, Wallingford, England, July 1982.

Shock pressures and total forces associated with monochromatic waves striking a wall at angles up to 25 deg from normal are measured. The pressures and forces decrease considerably with increasing angle of incidence. No comparisons with design formulas are made.

0073 "Breakwaters: Design and Construction", Proceedings of the Conference on this topic, organized by the Institution of Civil Engineers, and held in London, Ma, 1983 (Thomas Telford Ltd, publisher).

This collection of thirteen papers on breakwaters is mentioned here only to note that the papers contain virtually no mention of vertical-wall breakwaters. They deal almost entirely with rubble-mound structures. Of some general interest are the overview articles by I.W. Strickland (pp. 1-8), J. E. Clifford (pp. 53-64), and H. F. Burcharth (pp. 177-187).

0074 The Permanent International Association of Navigation Congresses (Brussels, Belgium).

The reports given at these Congresses offer a glimpse at the actual design procedures used in several of the major maritime nations. Although these reports are of uneven quality, often not in English, and are often lacking important references, they do emphasize practical issues rather the fine points sometimes stressed in research papers. Probably because of this, the papers tend to be rather conservative, and contain almost no original material. Of special interest are the reports from countries not well represented in the English literature (e.g., Russia and Italy). This reviewer found little in these articles to help in a search for an up-to-date method for predicting wave forces on walls.

The following articles are of some interest:

(a) Proceedings of the 21st International Navigation Congress, Stock holm, 1965 (Sjubject 1: Breakwaters with vertical and sloping faces. Measurement of waves. Study of wave forces. Methods of Calculation).

(i) General Report (A. Brandtzaeg), pp. 1-20.

(ii) Nagai, S., "Wave Pressures on Various Types of Vertical-Wall Breakwaters", pp. 111-132.

(iii) Saville, T., Garcia, W. J., and Lee, C. E., "Development of Breakwater Design", pp. 229-253.

(iv) Djounkowski, N. N., Kouznetsor, M. A. I., and Smirnov, M. G. N., "Recherche et Calcul des Ouvrages de Protection Verticaux et a Profil en Talus en URSS", pp. 225-267.

(b) Proceedings of the 24th International Navigation Congress, Leningrad, 1977 (Subject 1: Improvement in the design and building of major port structures).

(i) Paper by V. Panunzio and F. Grimaldi, pp. 99-130 (There is no title. The paper compares wave forces predicted by the methods of Sainflou, Miche, and Kouznetsov. It also discusses, with few details, model tests used to estimate shock-pressure distributions.)

(iii) RECENT LITERATURE WHICH WAS NOT REVIEWED

A few articles proved difficult to obtain. It is very unlikely that these will have much bearing on the design procedures advocated in this report. Thus, in the interests of time, these articles are simply listed below, along with the citation source.

Maravas, T., 1977, "Shock Forces on Vertical-Face Breakwaters", M.S. Thesis, Institute of Hydrodynamics and Hydraulic Engineering, Techn. univ. Denmark. (cited in Gravesen and Lundgren, 1977)

Mejlhede, N., 1975, "Standing Waves of First and Second Order and Wave Pressure", Techn. Univ. Denmark, Institute of Hydrodynamics and Hydraulic Engineering, Progress Report No. 35. (Fluidex BHRA Index)

Van de Kreeke, J., and Paape, A., 1964, "On Optimum Breakwater Design", Delft Hydraulics Laboratory, Publication No. 31. (cited in Richert, 1974)

APPENDIX B: FINDING H_i WHEN THE WATER DEPTH IS GREATER THAN $L_o/5$

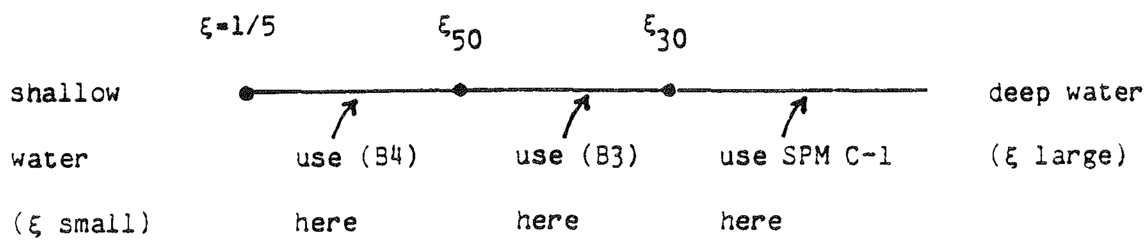
When the general water depth d $\geq L_o/5$, one is outside of the surf zone, there is little wave breaking, and the wave height is found using a shoaling coefficient, K. The method described by Goda (1985) following the work of Shuto (1974) is given here. The equations of Goda are nonlinear and somewhat awkward, and are modified below. Also, it helps to introduce

(i) the nondimensional depth $\xi = d/L_o$
(d will be between D and D_b for us)

(ii) the deep-water wave steepness $S = H_o'/L_o$
(disregarding refraction)

Note that we know both ξ and S.

The shoaling coefficient K is defined by $H_{1/3} = K \cdot H_o'$. There are three methods used to find K. These are equations (B3) and (B4) below, together with SPM Appendix C-1. Each only holds for a certain range of ξ. Then do decide which formula to use, we must first find the boundaries of these ranges. Call these boundaries ξ_{30} and ξ_{50}, following Goda, and consider the schematic below:

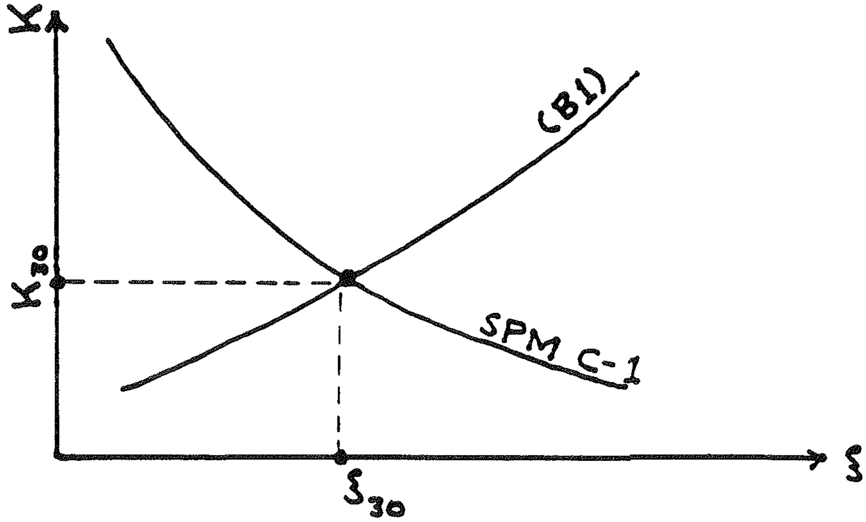


For the large water depths far from shore, the standard small-amplitude wave theory can be used to find K. The results are given in SPM Appendix C-1, where K is called H/H_o', and is listed as a function of ξ. The <u>smallest</u> ξ for which we can do this is ξ_{30}, which is given by

$$\xi_{30}^2 = \frac{2\pi S}{30} K_{30} \tag{B1}$$

Here, K_{30} is found using Appendix C-1. That is, we guess ξ_{30}, then find the associated K_{30} from Appendix C-1, and finally see if eqn (B1) is satisfied. When it is, we know ξ_{30} and K_{30}. The change of K with a general value of ξ

using both (B1) and SPM (C-1) is shown below, in order to facilitate the iterations.



Then if $\xi \geq \xi_{30}$, we calculate K using d = D and Appendix C-1. However, suppose that $\xi < \xi_{30}$. Now the water is so shallow that nonlinear effects are important in fixing K. We first find the second boundary, ξ_{50}. This is given by

$$\xi_{50} = 0.7997\ \xi_{30} \tag{B2}^*$$

This boundary is now compared to ξ. If $\xi_{50} \leq \xi < \xi_{30}$, K is given by

$$K = K_{30}\ (\xi_{30}/\xi)^{2/7} \tag{B3}$$

Here, we know ξ_{30} and K_{30} from our work with eqn (B1) above.

However, if $\xi < \xi_{50}$, K is given by

$$K\ (\sqrt{K} - B) = C \tag{B4}$$

* Note that equation (B2) is different from the equations given in Goda (1985). It comes from combining equations 3.17, 3.18, and 3.19 of that reference.

where the constants B and C are given by

$$B = 2\xi\sqrt{\frac{3}{2\pi S}}$$

$$C = K_{50}\left(\frac{\xi_{50}}{\xi}\right)^{3/2}\left(\sqrt{K_{50}} - \sqrt{\frac{6}{\pi S}}\,\xi_{50}\right)$$

and K_{50} is calculated from eqn (B3), using $\xi = \xi_{50}$.

Eqn (B4) is solved by iteration, noting that $K > B^2$, and that the left side increases with K.

Knowing K, the maximum wave height is calculated from

$$H_{max} = 1.8H_{1/3} = 1.8\ KH_o' \tag{B5}$$

APPENDIX C: FINDING H_i WHEN THE WATER DEPTH IS LESS THAN $L_o/5$

The wall is now in shallow water, and wave breaking may be important. The model of Goda leads to the following procedure.

(i) Find H_{max}/H_o'

The ratio H_{max}/H_o' is given by the smallest of the following three quantities

$$\begin{array}{cccc} \text{(a)} & \text{(b)} & \text{(c)} & \\ \beta_o^* + \dfrac{\beta_1^* d}{H_o'} \; ; & \beta_m^* & 1.8\ K & \text{(C1)} \end{array}$$

Here,

$$\beta_o^* = 0.252\ S^{-0.38} \exp\ (20\ m^{3/2})$$

$$\beta_1^* = 0.63 \exp(3.8\ m)$$

β_m^* is the larger of the two quantities

$$\begin{array}{cc} \text{(a)} & \text{(b)} \\ 1.65 \; ; & 0.53\ S^{0.29} \exp\ (2.4\ m) \end{array}$$

In these expressions S is the deep-water steepness H_o'/L_o, and m the bottom slope. The shoaling coefficient K is botained using the method given in Appendix B.

If 1.8K is the smallest of the three quantities in (C1), wave breaking is not important, the wall is outside the surf zone, and the design wave height is

$$H_i = 1.8K \cdot H_o' = H_{max} \qquad \text{(C2)}$$

If 1.8K is <u>not</u> the smallest, the wall is within the surf zone, and the quantity $H_{1/3}$ must first be obtained as shown below.

(ii) Find $H_{1/3}/H_o'$

The ratio $H_{1/3}/H_o'$ is given by the <u>smallest</u> of the following three quantities

$$\begin{array}{ccc} \text{(a)} & \text{(b)} & \text{(c)} \\ \beta_o + \beta_1 d/H_o' \; ; & \beta_m \; ; & K \end{array} \qquad \text{(C3)}$$

Here,

$$\beta_o = 0.028\ S^{-0.38} \exp(20\ m^{3/2})$$

$$\beta_1 = 0.52 \exp(4.2\ m)$$

β_m is the <u>larger</u> of the two quantities

$$\begin{array}{cc} \text{(a)} & \text{(b)} \\ 0.92 \; ; & 0.32\ S^{-0.29} \exp(2.4\ m) \end{array}$$

The shoaling coefficient K again is obtained from Appendix B, and S,m are defined above.

(iii) To find the Design Wave Height H_i

If 1.8K is the smallest of the three quantities in (C1), set the design wave height $H_i = H_{max}$. Otherwise, first find $H_{1/3}$ at d = D, using eqn (C2). Then use $H_{1/3}$ to calculate a new water depth D_b from

$$D_b = D + 5\ m\ H_{1/3} \qquad \text{(C4)}$$

Now calculate H_{max} at $d = D_b$, using eqn (C1). It seems that D_b is used because waves breaking at a point are expected to exert their maximum force for some distance shoreward of that point. Then it is prudent to estimate H_{max} at several intermediate depths, and use the largest value obtained for H_i. That is

$$H_i = \max (H_{max}, \text{ for } D_b \geq d \geq D) \qquad \text{(C5)}$$

It should be noted that some of these intermediate depths may be outside the surf zone.

Thus, either (C2) or (C5) will be used to find the design wave height, according to whether the wall is outside or within the surf zone. (However, the depth D_b must be calculated in either case, to find γ_2, which is needed for the pressure distribution on the wall.)