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JOURNAL OF
THE WATERWAYS
HARBORS AND COASTAL
ENGINEERING DIVISION

DISCUSSION

Proc. Paper 9897

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Ultimate Energy Design of Prestressed Concrete Fender Piling[a]

Closure by Shu-t'ien Li,[7] F. ASCE and V. Ramakrishnan,[8] M. ASCE

The authors of the paper are appreciative of the discussions contributed by Gerwick, Pandit, Yang, and Lee.

Gerwick indicates his experience with, and observations on, prestressed concrete fender piling installed at ship berths in three countries and bridge-pier protective fenders in California. He has authoritatively testified to the practicality and advantages of such fender piling in his pioneering applications.

To Gerwick's statement that resistance to fire is particularly important in the protective fenders for bridge piers where the superstructure is of steel, the writers add that even more importance should be attached to fire resistance with fender piling at ship berths where transit sheds and warehouses for the storage of high-priced commodities are located within a short distance behind the waterfront.

His indication of the ability to vary the cross-section of prestressed concrete piling for maximizing the benefit, his plea for using mild steel reinforcement to increase ultimate bending resistance at critical points, and his suggestion for incorporation of unstressed strand, are important to designers. These three measures, combined with the optimum partial prestressing that is to be used in most cases with prestressed concrete fender piling, can give such piles the most versatile design.

Gerwick's further suggestions for potential developments of prestressed polymer concrete to combine a moderate E with high strength, and prestressed wire reinforced concrete to secure greater bending resistance and ductility with no increase in rigidity, will certainly constitute feasible imnprovements over the present conventional prestressed concrete.

Pandit's unification of USD, WSD, or UED as categories of limit state design (LSD) respectively, for strength, stiffness, or energy, is appropriate. The UED method is especially adapted to fender piling in front of wharves, quays, and piers at river and sea ports, protection fenders around bridge piers in navigable rivers, and bumpers and guardrailing in highway and railroad construction.

Naturally in long piles, the condition of buckling load must be investigated. And before this investigation, the effective length of the pile whose lower end is driven into the harbor or river bed must be determined by first finding the lower point of fixity. For such determinations, the writer's refer to Ref. 22 on prestressed concrete piling design practice and recommendations. By applying a factor of safety to the buckling load, the stress limitations imposed by the writers on both sides of the pile as originally given in their paper still hold

[a]November, 1971, by Shu-t'ien Li and Venkataswamy Ramakrishnan (Proc. Paper 8527).

[7]Chmn. and Pres., Li Instn. of Sci. and Tech., Rapid City, S. Dak.
[8]Prof. of Civ. Engrg., South Dakota School of Mines and Tech., Rapid City, S.Dak.

good, the tensile stress wave during recoil having been taken care of in arriving at the optimum prestress.

Pandit's question about the durability of structural lightweight concrete under marine environments would generate no problem, because not only has test installation at the Port of San Frncisco (see Gerwick's discussion) shown good results, but also prestressed concrete, whether normal weight or lightweight, is always more durable than unprestressed. This is because the possibility of horizontal cracks, adverse to steel corrosion, is positively eliminated by the built-in residual compression.

Furthermore, with polymer concrete, containing 6% polymer by weight in the samples, in addition to improvements in compressive, tensile, and flexural strengths up to about four times those of conventional concrete, significant improvement in durability was also noted by Kukacka, et al. (13). Methyl methacrylate-impregnated specimens have not failed after exposures of >1 yr to sulfate brines and 15% hydrochloric acid. Exposure to 5,120 cycles of freezing and thawing has resulted in essentially no attack.

To keep the original paper short and confine its treatment to ultimate energy design of prestressed concrete fender piling, the writers could not find space to cover the latest applicable concrete technology. Now, in answering Pandit's comment, ti may briefly be stated that in the near future, prestressed and polymerized lightweight aggregate concrete will show better performance than prestressed lightweight concrete, while the latter is undoubtedly better than unprestressed. In marine environments, in view of steel corrosion even with copper-bearing steels and wood deterioration, even though creosote-treated, today's prestressed concrete piling and tomorrow's prestressed polymer concrete piling should find more and better applications. It is envisioned that concrete under prestress can even better resist a kind of marine borer that attacks concrete which was found in the Los Angeles Harbor, but not elsewhere, to the first writer's knowledge (14,15).

Yang's comment on tensile stress wave upon recoil prompts the answer that it was just because it varies widely that the maximum tensile stress amplitude was represented by a symbol in the criteria equations given for arriving at the optimum prestress. The importance of the tensile stress wave merits treatment in a separate paper.

Where more eccentricity is desired, a rectangular section with the long axis perpendiculr to the waterfront is preferable. Where the torsion is high, a hollow square section should be chosen. And where the flexural duty is higher than the torsional duty, the bending flange walls can be made thicker than the shearing walls of the closed section. These and other configurations are always a matter of the designers' optimization.

In addition to Yang's fine discussion of maximum moment occurrences and their effects, they can be minimized by appropriately choosing support points or introducing intermediate supports for long fender piles. Supports can be further varied to equalize the maximum negative and positive moments so that symmetrical prestressing can be applied to the best advantage. Similar practical treatment was advanced by the writers in their joint paper on prestressed concrete sheet piling (23).

Not only is Yang's concern about picking-up stresses in long piles appropriate, but there may also be other critical stress conditions to be carefully considered,

such as creep under sustained dead weight during storage, dynamic effect due to vibration during tansportation, and pitching at the site. All these effects can be easily controlled to within the equivalent static working limit by carefully choosing suitable support points or increasing support points, not by guessing, but through continuity calculations as advanced in a previous paper (21). In addition to spacing multiple points of support, the face of support can be reversed so that the negative moments will occur on the prestressed compression face during handling. Since these support points can number one, two, three, four, six, eight, etc., there is never a need to strengthen the pile for picking up, storing, transporting, and pitching.

Lee's discussion begins with energy absorbing capacity of a fender pile being proportional to f^2/E. This applies when bending is predominent. When the lateral load (P) shifts to near the top support at times of high tide, an additional shearing energy absorbing capacity proportional to P^2/G cannot be dismissed. Likewise, as Yang brought out, when torsion due to torque (T) is large, another torsional energy absorbing capacity proportional to T^2/G must be taken into account.

Note that Lee's 5,000-psi ultimate strength for concrete is the present minimum requirement for prestressed concrete. Much higher ultimate strength up to nearly 11,000-psi cube strength has been used in prestressed concrete piling in a harbor in Israel. The tendency to use higher strength concrete and lightweight aggregate to keep a lower E is favorably meeting the required properties for fender-piling concrete.

As to Lee's worry about prestress losses, advanced concrete technology permits great reductions in losses due to shrinkage, creep, and compression shortening, as shown in Ref. 24, by using gap-grading, which has already been introduced into the German national Code of Concrete Practice. The reduction can even more than compensate for the difference between 35,000 psi for normal weight concrete and 54,300 psi for lightweight concrete of 100 lb per cu ft. In fact, the manufactured lightweight aggregate of equal size conforms perfectly to ideal gap-grading.

In answer to Lee's favorable comparison of Greenheart with prestressed concrete in energy absorbing capacity, the first writer is fully aware of the superior qualities of Greanheart timber (Nectandra Rodioei) as a fender piling material (16,17,18). But this timber is only produced in Demerrara, Guyana, South America. Today, long lengths are scarce. But the latest tankers require ever deeper berths for their ever larger capacity; the largest tanker, with a 477,000-ton capacity, is already in service. For the deeper berths on shore and for even deeper berths at offshore superports, it is the prestressed lightweight aggregate polymer concrete that will be the composite material of tomorrow.

With reference to Lee's consideration about biological deterioration, the authors favor prestressed concrete rather than timber whose depredation can be seen from Refs. 19 and 20.

Lee's contribution of Tables 3 and 4 in his confirmation of prestressed lightweight concrete as the most promising composite material for fender piling, is particularly appreciated. His suggestions for further research and in-service tests should receive the attention of government research and development funding agencies.

From studying these illuminating discussions, it is clear that the discussers

have all agreed: (1) That ultimate energy design is the only correct way to design fender piling; (2) that prestressed lightweight aggregate concrete should become the most promising composite material for fender piling; and with this foundation, the authors now further propose (3) that prestressed lightweight polymer concrete proportioned according to gap-grading by the void-filling method should prove to be the future fender piling material (25).

Appendix.—References

13. Kukacka, Lawrence E., et al., "Concrete-Polymer Composites," *Journal of the Structural Division*, ASCE, Vol. 97, No. ST9, Proc. Paper 8363, Sept., 1971, pp. 2217-2227.
14. Li, Shu-t'ien, "Service Performance of Construction Materials Used Completely or Partially under Water in Waterfront Facilities in Continental United States," *AREA-Bulletin*, Vol. 59, No. 539, Nov., 1957, pp. 519, 523-546; *Proceedings*, American Railway Engineering Association, Vol. 59, 1958, pp. 519, 523-546, discussion, pp. 1159-1160.
15. Li, Shu-t'ien, discussion of "Use of Concrete in Marine Environment," by C. M. Wakeman, et al., *ACI Journal*, American Concrete Institute, Vol. 30, No. 6, Dec., 1958, Part II, Disc. 54-46, pp. 1327-1336; *Proceedings*, American Concrete Institute, Vol. 54, 1957-1958, Disc. 54-46, pp. 1327-1336.
16. Li, Shu-t'ien, "Greenheart (Nectandra Rodioei)—Its Outstanding Merits, Properties, and Economics in Waterfront Facilities," *AREA-Bulletin*, Vol. 61, No. 553, Nov., 1959, pp. 311-330: *Proceedings*, American Railway Engineering Association, Vol. 61, 1960, pp. 311-330.
17. Li, Shu-t'ien, "Energy Design Criteria for Fender Piling and Relative Merits of Different Materials," *AREA-Bulletin*, Vol. 62, No. 560, Nov., 1960, pp. 394-406; *Proceedings*, American Railway Engineering Association, Vol. 62, 1961, pp. 394-406.
18. Li, Shu-t'ien, "Mechanical Properties of Greenheart and Determination of Recommended Allowable Values for Design of Waterfront Facilities," *AREA-Bulletin*, Vol. 62, No. 560, Nov., 1960, pp. 386-393; *Proceedings*, American Railway Engineering Association, Vol. 62, 1961, pp. 386-393.
19. Li, Shu-t'ien, "Depredation of Timber in Marine Construction I—Marine Borers," *The Dock & Harbour Authority*, London, England, Vol. XLII, No. 489, July, 1961, pp. 85-89.
20. Li, Shu-t'ien, "Depredation of Timber in Marine Construction II—Physical and Biological Damage," *The Dock & Harbour Authority*, London, England, Vol. XLII, No. 490, Aug., 1961, pp. 127-130.
21. Li, Shu-t'ien, and Huang, Michael W., "Optimum Handling, Supporting, and Hauling Provisions for Prestressed Concrete Piles," *ACI Publication SP-26*, presented at the April 2-3, 1969, Second International Symposium on Concrete Bridge Design, held at Chicago, Ill., published 1971, *Paper SP 26-44*, pp. 1239-1279.
22. Li, Shu-t'ien, and Liu, Tony Chen-Yeh, "Prestressed Concrete Piling—Contemporary Design Practice and Recommendations," *ACI Journal*, Vol. 67, No. 3, Title No. 67-10, Mar., 1970, pp. 201-220.
23. Li, Shu-t'ien, and Ramakrishnan, V., "Optimum Prestress, Analysis, and Ultimate Strength Design of Prestressed Concrete Sheet Piles," *PCI Journal*, Vol. 16, No. 3, May-June 1971, pp., 60-74.
24. Li, Shu-t'ien, and Ramakrishnan, V., "Gap-Graded Aggregates for High-Strength Quality Concrete," presented at the January 25, 1973, HRB International Symposium on the Grading of Concrete Aggregates, held at Washington, D.C., HRB *Highway Research Record*, 1973.
25. Li, Shu-t'ien, and Stewart, D. A., "Compatible Gradation of Aggregates and Optimum Void-Filling Concrete Proportioning for Full Consolidation," presented at the January 25, 1973, HRB International Symposium on the Grading of Concrete Aggregates, held at Washington, D.C., HRB *Highway Research Record*, 1973.

Effects of Irregular Wave Trains on Rubble-Mound Breakwaters[a]

Closure by Yvon Ouellet[4]

The writer is grateful to Bruun and Dattatri for their appropriate comments, which will permit elucidation of some of the points raised in the paper.

Regarding Bruun's discussion on the effect of using different frequency bands, the author's intention was to represent on the model some typicl wave spectra, and for that reason he chose those formulated by Scott and Newmann. Since the irregular wave pattern is obtained by superimposing 20 different frequencies, the continuous spectra are then simulated by discrete spectra over the frequency band obtainable on the programming device. Reducing the frequency band would then allow the use of only a few discrete frequencies, which would in the limit result in a regular wave train. The author was aware of that, as mentioned in the last paragraph of the conclusion, but the effect of using a discrete spectrum with very few frequencies must be taken into consideration.

However, it is the author's opinion (20) that the wave period is of less considerable importance than the wave height, as demonstrated by some tests in Wallingford (19). In fact there exists a range of wave periods around which damage is of the same order of magnitude. This range is relatively large, e.g., 8 sec to 12 sec in a specific case, outside of which damage becomes less and less severe. As mentioned, this range depends on the structure characteristics, mainly on the weights of the units, so that resonance occurs between impinging forces and the resulting displacement of the units.

The author agrees with Bruun's remark on the fact that wide spectra should not be ignored. The Scott and Newmann spectra have been characterized, respectively, as narrow and wide band in the sense that in one case the energy is more concentrated in the middle frequencies than in the other case. It is, then, not the width of the frequency band that is involved in the present study, but in future tests these two notions should be distinguished.

The writer shares the discusser's preference for a replacement of the terminology of design wave by design storms. But when it comes to the practice of designing a rubble-mound breakwater, someone has to calculate it for a particular value of the wave parameters which are thought to result in a structure that meets the requirements at minimal total cost. And now most rubble-mount breakwaters are designed on the basis of Hudson's formula, in which the only wave parameter used is the wave height.

Concerning Dattatri's comment on the Important Parameter, the ratio of the maximum wave height to the significant wave height, it was not the intention of the paper to explain by this parameter the difference in action on a rubble-mound breakwater between a narrow and a wide band spectrum having approximately

[a]February, 1972, by Yvon Ouellet (Proc. Paper 8693).

[4]Asst. Prof., Dept. of Civ. Engrg., Laval Univ., Québec, Canada.

the same height distribution, but the difference in results between two of the same types of spectra. Other explanations, such as the one given by Cartsen, et al., should be found to explain the difference in action between different types of spectra that have approximately the same wave height distribution. The author wanted to stress that one could find an explanation in the difference in damage produced by two spectra of the same kind by looking at the wave height distribution from which he deduced the parameter mentioned which was found to be a simple one. The subject is not closed for those who may find a more representative parameter, but it is most probable that this would come from the wave height distribution.

As mentioned by Dattatri, it is possible that the ocean waves belonging to narrow and wide band spectra may be well described by the Rayleigh distribution, but the ratio of the maximum wave height to the significant wave height could vary, e.g., with the number of waves. Then it is not always possible nor sometimes desirable to follow in laboratory experiments the criteria met in practice but, if this is so, it should be taken into account in the test results. For that reason, most laboratory tests with irregular waves made by different investigators do not correspond to the ratio of the most probable wave height to the significant wave height of 1.77 as obtained from Longuett-Higgins. In fact, in many flume tests the most probable wave height can be replaced by a maximum wave height. It was then natural to look at the difference in the wave height distribution to explain part of the difference in test results.

The writer still believes that the effect of an irregular wave train can be made equivalent to the action of a regular wave train by choosing the appropriate parameters. The subject is still in an early stage of development and it will be made clearer as more tests are carried out.

Appendix.—References

19. "High Island Water Scheme Hong Kong: A Study on the Use of Dolos Armor Units for Wave Protection on the Seaward Face of the Easters Dam," *Report No. EX. 532*, Hydraulics Research Station Wallingford, Oct., 1970.
20. Ouellet, Y., "Considerations on Factors in Breakwater Model Tests," *Proceedings*, 12th Conference on Coastal Engineering, Vancouver, Canada, 1972.

Wave Forces on Submerged Bodies[a]

Closure by C. J. Garrison[3] and Philip Y. Chow,[4] F. ASCE

The purpose of this closure is to call attention to Fig. 2, in which MacCamy and Fuchs' (7) analytical results for a circular cylinder are compared with the results of the more general theory presented in the paper. In generating the numerical results by use of MacCamy and Fuchs' equation, an error was made so that the curve presented in Fig. 2 is slightly in error, making the comparison appear poorer than it, in fact, was. The corrected result of MacCamy and Fuchs was plotted in the original figure as a dashed line and is presented as Fig. 12. The general agreement appears to be excellent with the results based on 252 nodal points showing slightly better agreement with the closed form

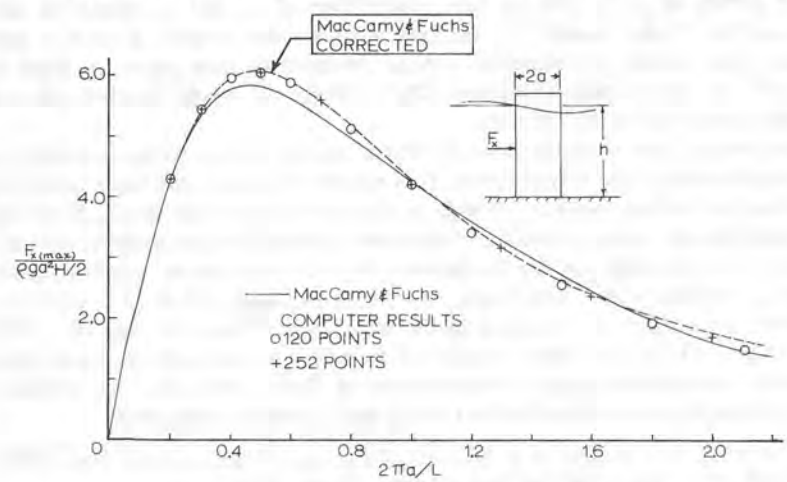

FIG. 12.—Horizontal Force Coefficient for Circular Cylinder, $h/a = 4.0$

result at large a than the results based on 120 nodal points.

Also, the scale on the ordinate in Fig. 2 is in error by a factor of 10. The correction is made in Fig. 12.

[a] August, 1972, by C. J. Garrison and Philip Y. Chow (Proc. Paper 9098).

[3] Assoc. Prof. of Mech. Engrg., Naval Postgraduate School, Monterey, Calif.

[4] Project Engr., Santa Fe-Pomerey, Inc., Orange, Calif.

Spectral Resolution of Breakwater Reflected Waves[a]

Discussion by John J. Earattupuzha[3] and Manas Rakson[4]

It is assumed in the calculations, as stated by the authors, that sensor No. 2 is located at −130 ft (−39.6 m) from the reflecting surface. This means that the assumed location of the reflecting surface is 5 ft (1.5 m) offshore of the breakwater face at MLLW. Since this is significant in the calculation of a_R and a_I it will be of interest to know the basis of this assumption. In fact, all reflection evaluations in the case of waves acting on a sloping surface involve this problem. In the laboratory, by moving a wave probe along the length of the wave flume, nodes and antinodes are easily fixed and calculation of a_R and a_I becomes simple. In the case of natural beaches, even with two fixed wave probes as in the present case, calculation of a_R and a_I cannot be done without first fixing a point on the beach-breakwater profile, if such a point exists, from which all reflection can be assumed to take place; at least for the sake of wave profile prediction. Theoretically the whole beach-breakwater profile should act as the reflector.

The writers' observations indicate that if one point has to be selected it is the point at which the wave breaks. This has been noted as the most frequently observed reflecting surface, though it was not always the case. From this consideration it appears that the point now selected by the authors, which is 3.33 ft (1.0 m) deep on the breakwater face is very nearly correct since a 4-ft (1.2-m) high wave may break at a depth of about 5.5 ft (1.7 m) to 6 ft (1.8 m). The difference involved in the horizontal distance is only $(6 - 3.33) \times 1\text{-}1/2 = 4$ ft (1.2 m) which works out to 1.54% of one half the wave length (distnce between two nodes or antinodes) of 260 ft (79.2 m). The effect of this change on the computed values of a_R and a_I will be negligible.

[a]November, 1972, by Edward B. Thornton and Ronald J. Calhoun (Proc. Paper 9318).

[3]Deputy Dir., Kerala Highway Research Inst., Kerala, India.
[4]Research Officer, Kerala Engrg. Research Inst., Kerala, India.

Equilibrium Beach Profile Scale-Model Relationship[a]

Discussion by John J. Earattupuzha[2] and Harihar Raman[3]

In this paper a rational approach is taken toward a solution of the problem of scale ratios of two-dimensional, movable bed models of the coastal zone. The information regarding distortion of models and limitations in the choice of the four basic parameters controlling model characteristics throws light on a long standing puzzle involving the behavior of shore models. The writers, however, wish to point out one possible limitation that may affect the results if the right choice of scales using Fig. 5 is not made.

The writers tried to verify whether the model and prototype profiles will be qualitatively similar with regard to type of profile (storm, normal, or transition). For this purpose Fig. 5 of the paper was used for fixing the model scales and the criteria developed by Iwagaki and Noda (5) and Nayak (10) were used to verify the type of profile. The plotting of H_0/L_0 versus H_0/D_{50} on a log-log sheet gives a falling curve as the critical combination of these ratios for generation of longshore bar in Ref. 5. The critical value of H_0/L_0 decreases as H_0/D_{50} increases. Therefore in the neighborhood of this curve, if H_0/D_{50} is changed without a corresponding change in H_0/L_0, there exists the possibility that the type of profile also changes. In the present case, the H_0/L_0 ratio of prototype is kept unchanged for the model while H_0 and D_{50} change according to Eq. 29. Taking sand itself as the sediment for the model and prototype, if $n_D = 0.6$ is chosen, from Fig. 5, $\mu = 1/2.5$, i.e., while H_0 of the model $= H_{0p}/2.5$, D_{50} of the model $= (D_{50p}) \times 0.6$, in which suffix p stands for prototype. Therefore, as a hypothetical case, if $H_{0p} = 100$ mm, $L_{0p} = 5{,}000$ mm, and $D_{50p} = 0.2$ mm are chosen, $(H_0/L_0)_z p = 0.02$ and $(H_0/D_{50})_p = 500$. This indicates a storm profile for the prototype as in Ref. 5. With the already selected values of $\mu = 1/2.5$ and $n_D = 0.6$, $H_{0m} = 40$ mm and $D_{50m} = 0.12$ mm. This gives $(H_0/D_{50})_m = 333$. The $(H_0/L_0)_m$ term is kept at 0.02 itself. As in Ref. 5, this combination gives a normal type profile. For storm type profiles in prototype with $(H_0/L_0)p$ versus $(H_0/D_{50})p$ combinations in the neighborhood of the critical values, the model profiles may become normal type. The reverse will be the case when model results are extrapolated to prototype.

With reference to the criterion for bar formation developed in Ref. 10, it is seen that the previously mentioned limitation is again applicable except in the range of H_0/SD_{50} varying from 10 to 100 in which region the critical deepwater wave steepness line is seen horizontal (S = the specific gravity of sediment in water).

The foregoing limitation will be brought out in the model design stage itself if prototype and model values of H_0/L_0 and H_0/D_{50} (or H_0/SD_{50}) are checked

[a]November, 1972, by Edward K. Noda (Proc. Paper 9367).

[2]Deputy Dir., Kerala Highway Research Inst., Kerala, India.
[3]Asst. Prof., Hydr. Engrg. Lab., I.I.T., Madras, India.

for discrepancy in type of profile with the criteria developed in Refs. 5 or 10, as the case may be. This will make possible suitable precautionery changes in design where necessary.

Maximum Breaker Height[a]

Discussion by Frederick E. Camfield,[2] M. ASCE

While the author has summarized and reevaluated a considerable amount of previous work on breaking waves, he has not considered the complete effect of wave asymmetry on the breaking process. The asymmetry of the wave is affected by both the beach slope, m, directly in front of the structure, and the offshore bottom slope over which the wave travels before reaching the nearshore area. As these two slopes may not be the same, it is necessary to account for the offshore bottom slope in the analysis.

The writer carried out extensive laboratory experiments on solitary waves shoaling for long distances over a constant bottom slope (21). It might be noted that these experiments gave values for H_b/d_b somewhat higher than those shown in Fig. 5. Additional experiments were carried out (20) with a transition from an initial bottom slope to a second, beach slope. While these additional experiments were limited, they showed that the initial bottom slope had a marked effect on the breaking characteristics of a wave breaking on the final beach slope.

Appendix.—References

20. Camfield, F. E., and Street, R. L., "The Effects of Bottom Configuration on the deformation, Breaking, and Run-Up of Solitary Waves," *Proceedings of the Eleventh Conference on Coastal Engineering*, ASCE, Chp. 11, Vol. 1, 1968, pp. 173-189.
21. Camfield, F. E., and Street, R. L., "Shoaling of Solitary Waves on Small Slopes," *Journal of the Waterways, Harbors and Coastal Engineering Division*, ASCE, Vol. 95, No. WW1, Proc. Paper 6380, Feb., 1969, pp. 1-22.

Discussion by John J. Earattupuzha[3] and Harihar Raman[4]

In relatively deep water, as the author has correctly pointed out, breaking condition is created by the continuous increase in the steepness of the wave

[a]November, 1972, by J. Richard Weggel (Proc. Paper 9384).

[2]Asst. Prof. of Civ. Engrg., Univ. of Delaware, Newark, Del.
[3]Deputy Dir., Kerala Highway Research Inst., Kerala, India; formerly, Research Scholar Hydr. Engrg. Lab., I.I.T., Madras, India.
[4]Asst. Prof., Hydr. Engrg. Lab., I.I.T., Madras, India.

tending to the critical steepness ratio. Usually this is the result of increasing accumulation of energy from the prolonged action of wind. The type of breaker in this case is generally "spilling."

Outside the fetch area, as the wave moves into transitional and shallow water regions of the coast, breaking condition is induced as a result of the higher velocity with which the crest travels forward compared to the trough which precedes it. The crest overtakes the trough and breaking occurs. This is evident from the steadily increasing steepness of the onshore part of the crest and the offshore part of the trough. In order for the crest to overtake the trough with a relative velocity which continuously increases with decreasing depth onshore, a definite traveling distance dependent on the changing relative velocity is essential. This indicates that, apart from considerations of deep water wave height, period, and slope of beach, the distance available for the wave to travel from the onshore limit of deepwater condition, $d = L_o/2$, to the shore, is also an essential factor governing the location of the breaker on a beach. This is not considered so far in any examination of breaking condition.

For a wave shoaling on a relatively flat sloped bottom, the crest overtakes the trough earlier in terms of depth of water compared to an identical wave shoaling on a steeper bottom because of the longer distance traveled on a flatter slope, assuming the same starting depths in both cases. The phase lag between the trough and its following crest is shortened more in the case of the former than in the case of the latter. For example, when the wave travels from 20 m to 10 m in depth, on a 1:50 slope it covers 500 m, and on a 1:10 slope it covers 100 m. The velocities of the crest and trough in one case are the same as in the other when still water depths are identical. Let the velocities of the trough and crest = C_t and C_c at the mean depth of 15 m:

Time taken by the trough to travel from 20 m to 10 m of depth on 1:50 slope = $500/C_t$

Time taken by crest to travel from 20 m to 10 m of depth on the same slope = $500/C_c$

Shortening of time lag between trough and crest = $t_{500} = 500/C_t - 500/C_c$

Similarly, shortening of time lag in traveling from 20 m to 10 m of depth on 1:10 slope = $t_{100} = 100/C_t - 100/C_c$. If C_t and C_c remain the same in both cases $t_{500} > t_{100}$. Obviously summation of t's over such discrete intervals of depth will equal $T/2$ earlier in terms of depth in the former case than in the latter (when the sum of t's = $T/2$, the crest just overtakes the trough).

Therefore waves having identical deep water characteristics will break at a greater depth on a flat slope and at a lesser depth on a less flat or steeper slope. This results in a smaller breaker height in the case of the flat slope since wave height increases with decreasing depth onshore of the point where $d/L_0 = 0.16$. This is clear from Fig. 2 of the paper, which shows consistently higher breaker height indices as the slope of the beach becomes steeper while the deep water wave characteristics remain constant.

For the world's coastlines confronted by identical deep water wave conditions, this should be manifested by comparatively smaller values of d_b, larger H_b, and higher breaker height indices for steep or narrow continental shelves and larger values of d_b, smaller H_b and lower breaker height indices for flat or wide continental shelves.

In the laboratory when shallow water waves are tested, the distance between the wave generator and the beach itself becomes a factor to be considered from this point of view.

Even though the intention of this discussion is to draw attention to this important factor of traveling distance available for the wave, it is admitted that reliable formulas for the evaluation of crest and trough velocities separately do not exist at present. However, it is to be mentioned that it may not be possible to correctly establish the conditions of breaking and evaluate breaker height indices without due consideration of this factor.

Discussion by Henry T. Falvey[5]

The author has been able to develop some useful design recommendations for determining maximum breaker height. The purpose of this discussion is to augment his evaluation with some additional available breaker data. The two primary areas of interest are the breaker classification curves, Fig. 1, and the breaker steepness curves, Fig. 5.

The limiting condition of the deepwater steepness in Fig. 1 is a value of $H_0'L_0 = 0$. This condition corresponds with the shoaling of a solitary wave. Investigations of this problem by Camfield and Street (21) and by Ippen and Kulin (6) indicate that for small values of the slope the waves break as spilling breakers, while for large values the waves break as plunging breakers. The other limiting condition to be considered is the slope. On zero slopes, periodic and solitary waves break as spilling breakers. An apparent contradiction to this statement was found by Galvin (4) who made waves complete their plunging on zero slopes. However, he was testing compound slopes. In addition, he could not initiate the plunging breakers on zero slopes. Therefore, one is led to conclude that the equations of the transition curves in Fig. 1 are not applicable for beach slopes less than 0.02 and deepwater steepness ratios less than about 0.005.

Once again, considering the limiting condition of the breaker steepness of Fig. 5, one finds that for a solitary wave, $H_b/T^2 = 0$. The investigations of Camfield and Street (21) indicate two significant findings. One, there is no upper limit to the ratio, H_b/d_b, with solitary waves. The extreme case is comparable to a finite height wave breaking over zero depth. Their maximum measured value of H_b/d_b was 2.0 for a slope of about 1:22. Thus, it appears that shoaling solitary waves may produce much higher breaker heights than periodic waves. The second significant finding was that for slopes greater than 1:5, there was no evidence of breaking waves. This might explain some of the trends found by Jackson (9) which were presented in Fig. 7. In this case, the structure would have to be considered as a steeper extension of the bottom slope.

Admittedly, the wave types discussed herein represent the extreme values of the range of conditions considered by the author. However, consideration of these extremes may help to bracket the ranges of variations in the data abstracted from the various experimenters.

[5]Sr. Research Engr., Laboratoire d'hydraulique, Ecole Polytechnique Fédérale de Lausanne, Lausanne, Switzerland.

In abstracting the data of others, extreme care must be exercised if a generalized theory is to be developed. Part of the variance comes from the equipment used and part from the definitions of terms. For instance, the breaking depths of Galvin (4) presented in the paper are referenced to the mean water level. The depths of Camfield and Street (21) are referenced to the still water level. Significant differences in the comparisons can result from these different definitions when the breaker depth is large and the water depth is small. The writer assumes that all of the data presented in Table 3 have been corrected to a common base definition.