

LOADINGS ON RUBBLE-MOUND BREAKWATERS DUE TO EARTHQUAKES

by

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It might reasonably be asked, why study the reaction of a rubble-mound breakwater to earthquake loadings? After all, it is essentially a pile of rubble before an earthquake and is probably only a lower or otherwise deformed pile of rubble afterwards. Repairs are simply a matter of adding more stones. Even if damage occurred, the breakwater might still offer partial protection. This is precisely why, to date, breakwaters are mainly designed for wave loadings. There is practically no documented literature⁽¹⁾ concerning breakwater design for earthquake loading!

However, the oceans are now being tapped as possible locations for industrial installations such as offshore deep water ports, refineries, and power plants. Such facilities must be adequately protected since failures might result in heavy financial losses and cause severe environmental repercussions. Therefore, breakwaters which might be used for this protection can no longer be treated as structures whose failure would only be of secondary consequence. It is thus reasonable to ask whether breakwaters to serve these purposes should also be designed against earthquake loading, since even partial failures might not be acceptable. If they should, then what type of design problems can one expect to encounter and how should one handle them? The present work is aimed at exploring these problems through laboratory experiments.

At present, there is no existing breakwater⁽¹⁾ that has been designed on the basis of earthquake loading. Likewise, observations of breakwaters after continuous earthquakes are also scarce. Okamoto⁽²⁾ reports crumbling of riprap, uneven settlement and loosening or tilting of the upper parts of Japanese breakwaters that have undergone earthquakes. After the large Kanto earthquake of 1927, for instance, breakwaters at the ports of Yokosuka and Yokohama approximately 50 kilometers from the epicenter developed irregularities over their entire lengths. In no cases, however, did any of the breakwaters topple or overturn. During the great Alaska earthquake of 1964, breakwaters at Kodiak City and Seldovia, Kenai Peninsula were badly damaged. The damage to the Kodiak City breakwaters was well documented⁽³⁾ with photographs and surveys taken after the earthquake. The damage was attributed to both earthquake motion and the tsunamis which swept the area. Unfortunately, the

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proportion of damage due to each cause could not be determined.

The present work represents an initial attempt to model the breakwater under earthquake loading, to identify failure modes, to determine the trends and degrees of relative importance of various factors and to estimate pressure distributions on the surface. It is an acknowledged fact that the armor layers are the main protection for a breakwater and the current practice in breakwater design is almost exclusively concentrated in the design of armor layer stability against wave force. One of the primary interests of the present tests is, thus, to examine the stability of these armor units against earthquake loading. The dolos was selected for the test program because it is gaining popularity in breakwater construction due to its superior stability against wave attack(4).

Scaling Laws and Model Design

Breakwater Simulation. It is a well-known fact that proper modeling requires preservation of both geometrical and dynamical similarities. Since a rubble-mound breakwater is not monolithic, but an ensemble of discrete elements, the geometrical similitude requires that both the structural shape and individual elements maintain the same scale ratio.

The model scale is dictated by the model dolosse available for testing, which have dimensions shown in Figure 1. They weigh 210 g. each with specific weight equal to 2.34 g/cm^3 . The overall structure is modeled in accordance with a typical "three-layer breakwater in breaking wave environments" as recommended in the Shore Protection Manual(5). The cross section of the model is shown in Figure 2. The crushed stones available for sublayer and core constructions have an average specific weight of 1.39 g/cm^3 as measured in air, while in actual breakwaters, heavier stones of approximate unit weight 1.68 g/cm^3 , are usually used. Table 1 provides the weight composition of crushed stones for the sublayer and the core in the model.

TABLE 1. WEIGHT DISTRIBUTIONS OF CRUSHED STONE FOR SUBLAYER AND CORE

<u>Sublayer</u>		
<u>Average Weight (g)</u>	<u>Percent</u>	<u>Retained on Sieve Size (cm)</u>
108/stone	22	3.80
42.5/stone	59	2.67
18.4/stone	17	1.59
4.8/stone	2	PAN
<u>Core</u>		
<u>Average Weight (g)</u>	<u>Percent</u>	<u>Retained on Sieve Size (cm)</u>
0.187/stone	27	0.942
0.074/stone	49	0.635
0.028/stone	18	0.475
0.012/stone	6	PAN

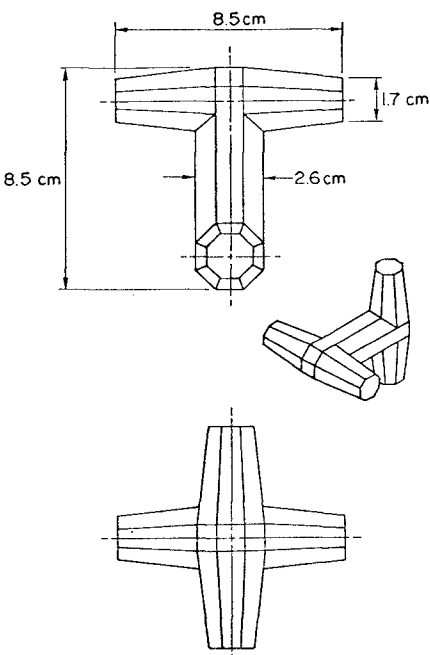


Figure 1. Dimensions of Armor Unit Used in the Experiment.

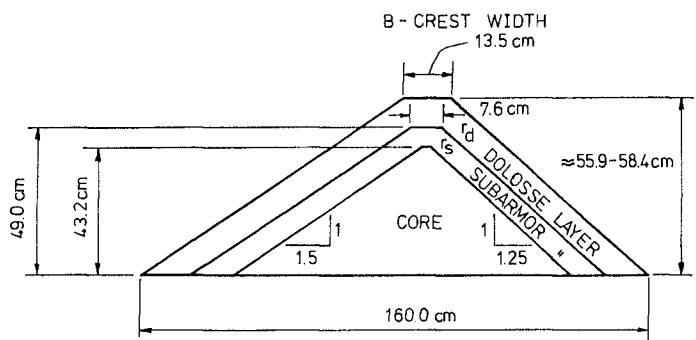


Figure 2. Model Breakwater Cross-Section.

The dynamic effect of an earthquake is mainly an inertia one. The similarity is preserved by the following criterion:

$$\left(\frac{U}{\sqrt{L(1 + C_a \Omega) a_e}} \right)_m = \left(\frac{U}{\sqrt{L(1 + C_a \Omega) a_e}} \right)_p \quad (1)$$

where U = characteristic velocity; L = characteristic length; C_a = added mass coefficient; Ω = density ratio of the fluid to that of the breakwater material and a_e = earthquake acceleration. The subscripts m and p refer to model and prototype, respectively.

If the model and the prototype are geometrically similar and have the same Ω ratio as the present case, the above equation reduces to a modified Froude number provided

$$(a_e)_m = (a_e)_p \quad (2)$$

To preserve the similarity of the body force, the Froude criterion should be observed, that is:

$$\left(\frac{U}{\sqrt{gL}} \right)_m = \left(\frac{U}{\sqrt{gL}} \right)_p \quad (3)$$

where g = acceleration of gravity. It is evident that this criterion is automatically satisfied if (1) and (2) are satisfied.

In order to simulate the frictional force of the system, one should first examine the contributing factors. The breakwater is now being looked upon as constructed of solid-fluid structural elements. The frictional force arises from both solid-to-solid contact and fluid-to-solid contact. These two types of frictional forces play opposite roles in breakwater stability. The frictional force due to the relative motion of fluid to solid is an upsetting force, whereas the force due to friction among solids offers the resistance. To model the former force properly, one requires the Reynolds criterion to be preserved:

$$\left(\frac{\rho UL}{\mu} \right)_m = \left(\frac{\rho UL}{\mu} \right)_p \quad (4)$$

where ρ = the density of the fluids and μ = the dynamic viscosity of the fluids.

This criterion is not compatible with the modeling conditions stated above. However, for a structure as bulky as a breakwater, appreciable error will not be introduced by not exactly modeling this force.

The frictional force developed among solid elements, if properly modeled, should follow the criterion below:

$$\left(\frac{ma}{\mu_s W} \right)_m = \left(\frac{ma}{\mu_s W} \right)_p \quad (5)$$

where μ_s = frictional coefficient between solids; m = element mass; and w = element weight. Based on Equation (1), this criterion reduces to

$$(\mu_s)_m = (\mu_s)_p \quad (6)$$

This means that the coefficients of friction in both model and prototype should be the same. The simplest and probably most accurate choice is to build the model with the same materials that one expects to be used in the prototype. In the armor layer, the interlocking behavior may play a dominant role in resisting deformation. Since interlocking among armor units is mainly a function of geometrical shape, there remains little choice but to preserve the shape of the armor units.

As to the elastic restoring force when the structure is subjected to earthquake loading, the apparent shear modulus of elasticity is probably the most pertinent parameter that should be properly modeled. To properly preserve this elastic behavior, the model has to be built with materials of considerably smaller modulus of elasticity than that of the prototype (of the order of the length ratio). Such materials, which at the same time have to satisfy the other modeling criteria as outlined above, are difficult, if not impossible, to find.

In general, the apparent modulus of elasticity affects the amount of energy that can be absorbed before failure. It also affects the natural period of the structure which in turn dictates the frequency at which failure is most likely to occur under vibrating loading. Since a breakwater, being a porous structure of no less than 30% porosity and constructed of discrete particles interacting through friction and interlocking, is a highly damped system the aspect of not being able to match the natural period between model and prototype is not a critical one. In the model the corresponding scaled shear modulus is considerably higher than required. This higher than required model shear modulus was deemed conservative from the point of view of energy absorption.

Earthquake Scaling and Selection. An earthquake can be considered as a vibratory ground motion with components in three orthogonal directions and the random characteristics of "white" noise. Only the horizontal motion was modeled. The principal direction of motion was chosen to be perpendicular to the face of the breakwater. A one-pulse earthquake was selected which corresponds to a prototype frequency of 1.61 Hz with varying amplitude from 0.2 g to greater than 1 g. The frequency was so selected as to provide a time history of motion that envelopes a typical earthquake spectrum similar to that of the El Centro earthquake as given by Newmark and Rosenblueth⁽⁶⁾.

Experiments

Setup. Testing of the model breakwater was done in a tank measuring 1.5 m (depth) by 2.3 m (width) by 37 m (length). A shake table was situated in the center of the tank at an elevation which both minimized the eccentricity of the applied load and allowed a wide range of water levels to be tested. The shake table was designed to support models up to one

ton and to impart horizontal acceleration motions up to 1.6 g. It was actuated by a 20 kip Gilmore hydraulic actuator with a Moog 72-102, 40 gallon per minute Servo valve, controlled by a Gilmore, Model 431B Servo Controller and a Model 112 Wavetek. A variety of wave forms can be generated with capability of all frequencies ranging from 1 to 100 Hz and double amplitudes of from 0.25 to 25 cm. The electronic hookup is shown schematically in Figure 3.

Stability Tests. A total of eight complete breakwaters was built and tested along with one case consisting of the breakwater core only. In general, each breakwater underwent at least four successive shakes. It was intended that each pulse be larger than the one preceding it and this was most often the case. Table 2 summarizes the cases tested.

TABLE 2. SUMMARY OF CASES TESTED

<u>Case</u>	<u>Breakwater Designation</u>	<u>Water Level As a Fraction of Breakwater Height</u>
Core*	Core	0**
455 Colosse	I	0**
455 Dolosse	II	0**
455 Dolosse	III	0**
455 Dolosse	IV	1/3
455 Dolosse	V	2/3
455 Dolosse	VI	1/2
505 Dolosse	VII	0**
505 Dolosse	VIII	1/3

*No Armor or Dolosse

**No Water

In each test case, the dolosse were placed randomly to an even calculated density as in a real breakwater. An exception was made at the toe of each face where the dolosse were placed in a regular fashion for maximum stability against the flat bottom. Detailed test procedures for this part can be found in Ref. (7).

Pressure Distribution Tests. The experiments in this test were divided into two groups. In the first group, a model breakwater with a vertical front face was tested to measure the pressure change on the front face at different elevations. In the second group, a model with a 30° inclined front face was tested. The model with the inclined faces was basically the same as the stability tests with slight front face angle adjustment.

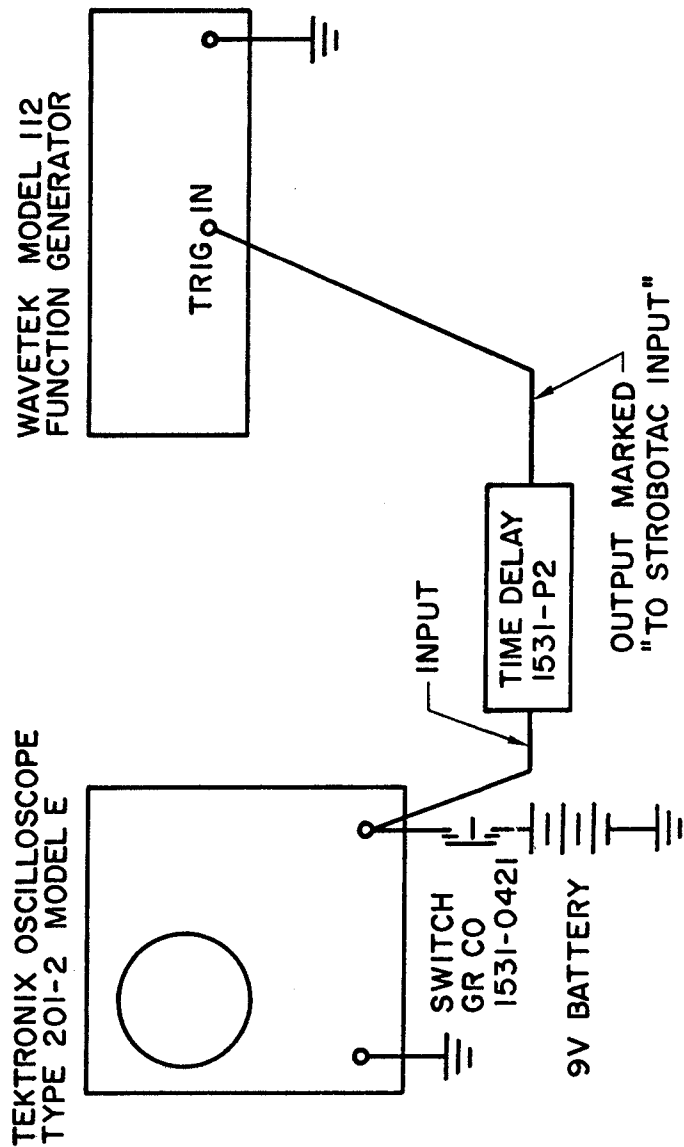


FIGURE 3. Electronic Hookups

For both models, pressure change on the surface of the breakwater at four different elevations were measured by Statham Model P131 and Viatran Model 103 pressure transducers. The water level in all tests was kept at 45 cm. The detailed test program is documented in Ref. (8).

Test Results

Stability. Post test surveys clearly indicated that the dominant mode of damage was settlement. None of the breakwaters tested showed any indications of a catastrophic sliding type failure. It was also apparent that the breakwater crest as opposed to the sides, was most sensitive to the shaking. The crest rounded and settled in much the same way as that of a rock-fill dam under similar conditions⁽⁹⁾. The observed effect of adding more dolosse (Cases VII and VIII) was an increased tendency for some to roll down the face of the breakwater. Possible cracking might have occurred at the crest during a test but none were observed afterwards. Bulging was also noted on the lower and middle parts of some breakwaters. Figure 4 illustrates the breakwater profile changes due to successive shaking.

As one would expect, the steeper slopes were slightly more sensitive to earthquake loading than the milder slopes. In the case tested, more dolos movement was observed on the rear slope (1.25:1) than the front slope (1.5:1) which resulted in, generally, greater outward displacement of the rear toe than the front toe (Figure 4). However, the dolos mat remained intact on both faces under all conditions. Some densification of the dolos layer occurred on the sloping parts of the breakwater at the expense of thinning the crest (less dolosse per unit area than originally).

The thinning in the dolos armor layer would develop into a rift as testing proceeded. On either side of the rift a single sparse layer of dolosse would remain, but this rapidly densified on the breakwater faces. In some cases, the crest became slanted toward the rear. While the crest sometimes remained level it never slanted appreciably towards the front or milder slope.

After examining the physical damages of the breakwater, it became clear that the change in crest elevation is the primary damage indicator. This change in elevation, expressed in terms of the percentage of height prior to the shake, has been plotted in Figure 5 as a function of horizontal acceleration.

As expected, the settlement increases with the acceleration as a trend, but a wide data scattering exists. This wide data scattering is at least partially attributed to the accuracy of the measurement which is about 1% of the total height. If an envelope is drawn around the points on this graph, the intersection of the acceleration lower bound and the zero damage line occurs around 0.4 g. This would indicate that a breakwater of the rubble-mound type is highly resistant to earthquake damage. Clough and Pirtz⁽⁹⁾ reached similar conclusions for rock-fill dams as they found no significant changes in profile until earthquakes produced accelerations of about .4 g. They attribute the relatively high degree of resistance to the flexibility resulting from the discrete nature of a rock-fill dam. Under cyclic loading at

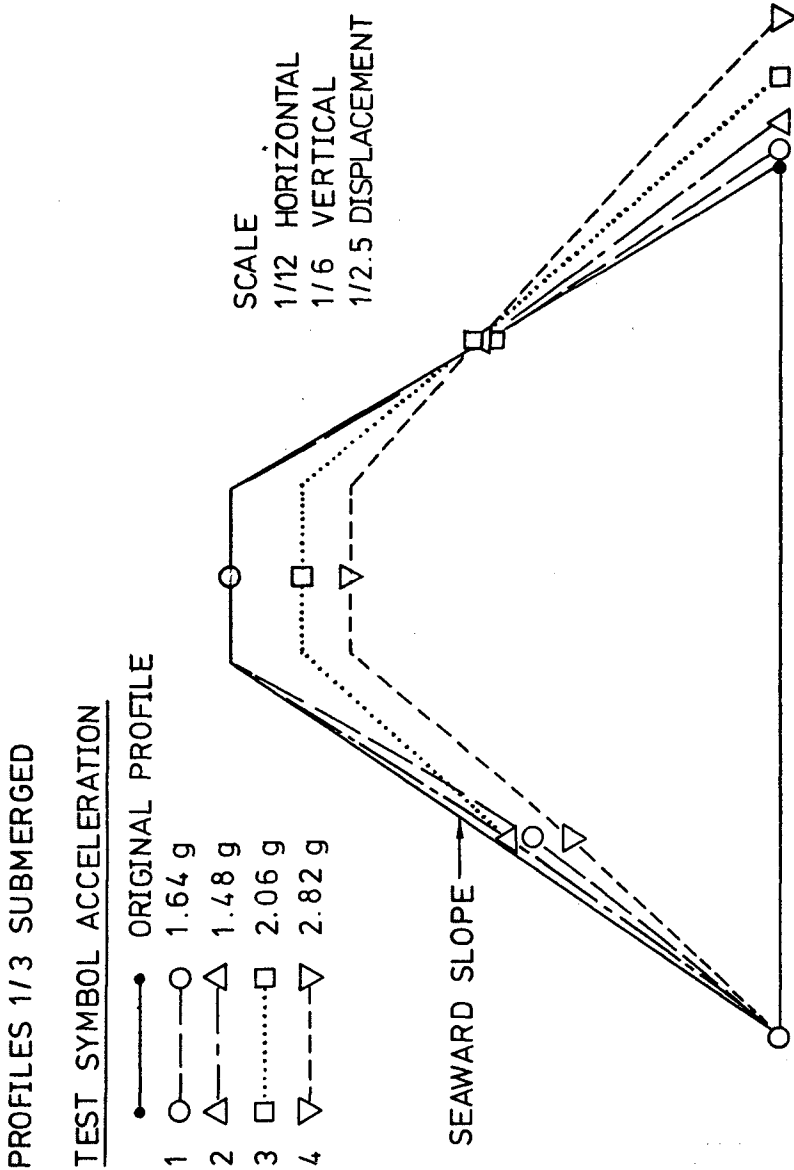
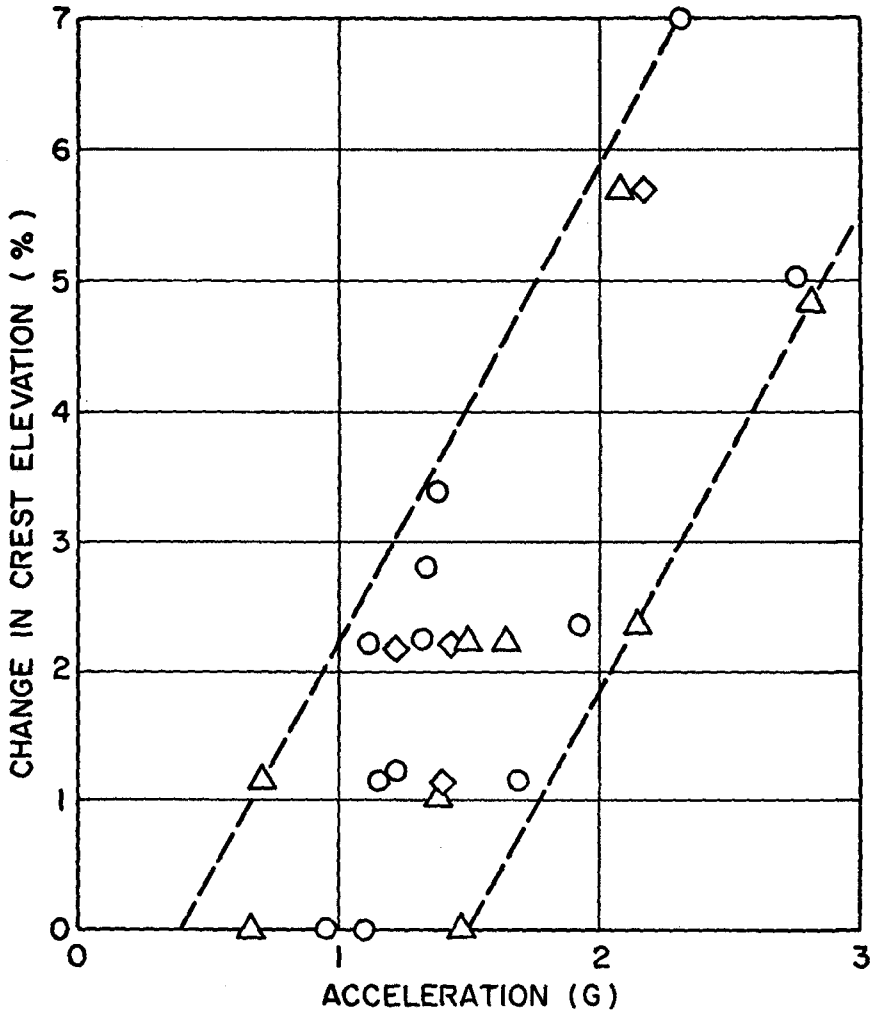


Figure 4. Breakwater Profile Changes Due to Earthquake



○ BREAKWATER DRY
△ BREAKWATER 1/3 SUBMERGED
◇ BREAKWATER 1/2-2/3 SUBMERGED

FIGURE 5. Crest Elevation Change Versus Earthquake Acceleration

earthquake frequencies, the dam would tend to commence shearing in one direction only to find the force reversed and the next shearing displacement cancelling the first. Permanent deformation was mainly produced by shake down; a secondary effect. Further indications of this strength can also be seen in that, for all the earthquakes under about 1.5 g, the settlement is of the order that might be expected from normal shake down over a long period of time. Since settlement is the major mode of damage, one should expect that presettlement prior to the earthquake should also play an important role in addition to the earthquake magnitude. In order words,

$$\text{Settlement} = f(\text{presettlement, acceleration})$$

In the experiments, the best indicator of presettlement is probably the cumulative crest settlement prior to any specific test (defined as the total settlement sustained due to previous shakes, measured from a freshly-built model, prior to the specific test run). If this factor is taken into consideration, the data in Figure 5 can be replotted as shown in Figure 6. It is evident from this plot that a new breakwater is more susceptible to damage than an older one. The presettlement becomes less important for larger ground accelerations. In other words, for a large earthquake, the breakwater will settle the same amount irrespective of the degree of presettlement. This is because, for moderate shocks, the settlement is mainly due to internal densification which is certainly closely related to the initial density, whereas for strong shocks, the crest settlement is caused by change of structural shape and modification of side slopes as were observed during tests. For new breakwaters with little or no initial shake down, Line A-A provides a conservative design criterion; for cases of older breakwaters with more than 4% presettlement, Line B-B is a more reasonable criterion.

The small effect of water level on breakwater response was unexpected. An increased water level would reduce the intergranular effective stresses, here directly proportional to the frictional resistance of the breakwater to deformation. Thus, increased displacement might be expected at high water levels. While the very high water levels did display a tendency in this direction they were interspersed with data from all the lower water levels as well. The tests showing the least settlement for the applied loads were about equally divided between the 0 and 1/3 submerged breakwaters.

It is possible that the reduction in frictional resistance is too small to be apparent. It is also possible that the very nature of the settlement may be fairly independent of the intergranular friction as compared with the resistance due to the armor unit interlocking, which is clearly independent of submergence.

Pressure Distributions

(a) Theoretical Considerations. Westergaard⁽¹⁰⁾ was the first one who in 1933 solved the two-dimensional case of horizontal vibrations of a rigid dam with vertical upstream face placed at one end of an infinitely long reservoir of uniform depth. In the same year, Von Kármán⁽¹¹⁾ developed an approximate but very simple method for the same case. Wang et al.⁽¹²⁾ extended the Von Kármán theory to an inclined surface. The essential results of the latter are summarized here.

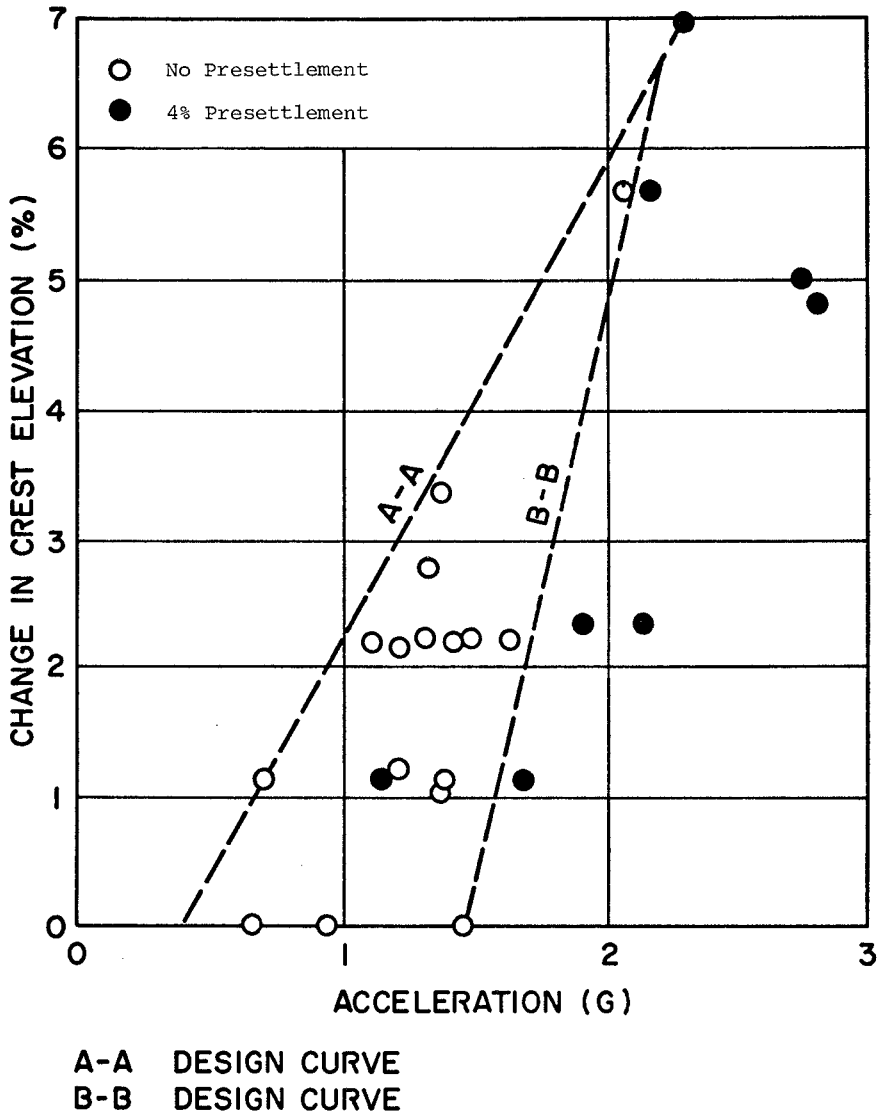


Figure 6. Effect of Presettlement on Breakwater Damage Due To Earthquakes.

Consider a unit slice of a breakwater with an inclined front face as shown in Figure 7. The following sets of equations can be established:

Continuity Equation

$$y a_x = b a_y \quad (7)$$

Equations of Motion in the x- and y-Directions

$$p = \rho b a_{xf} \quad (8)$$

$$p \cot \theta - p \frac{db}{dy} - b \frac{dp}{dy} = \rho b a_y \quad (9)$$

with the symbols as identified in Figure 8, and the subscript f refers to fluid.

The required boundary conditions are, at the water surface:

$$p = 0 \text{ at } y = h \quad (10)$$

and at the inclined front surface:

$$a_{xf} = a_x \sin^2 \theta \text{ for ideal fluid} \quad (11)$$

or

$$a_{xf} = a_x - a_y \cot \theta \text{ for non-slip boundary} \quad (12)$$

The close form solution for the ideal fluid case is

$$\ln \frac{k}{b} = \frac{1}{2} \ln \left| 2 \left(\frac{y}{b} \right)^2 - \frac{y}{b} \sin 2\theta + \sin^2 \theta \right| - \frac{\cos \theta}{\sqrt{8 - \cos^2 \theta}} \tan^{-1} \frac{4 \left(\frac{y}{b} \right) - \sin 2\theta}{2 \sin \theta \sqrt{8 - \cos^2 \theta}} \quad (13)$$

where k, the integration constant, is:

$$k = \sqrt{2} h \exp \left(- \frac{\pi \cos \theta}{2 \sqrt{8 - \cos^2 \theta}} \right) \quad (14)$$

Once the value of b is determined the pressure can be obtained from Equation (8).

(b) Comparison of Experiment With Theory. In Figure 9, the experimental results of the vertical wall case were compared with the extended Von Kármán theory and the experimental results of Zangar⁽¹³⁾.

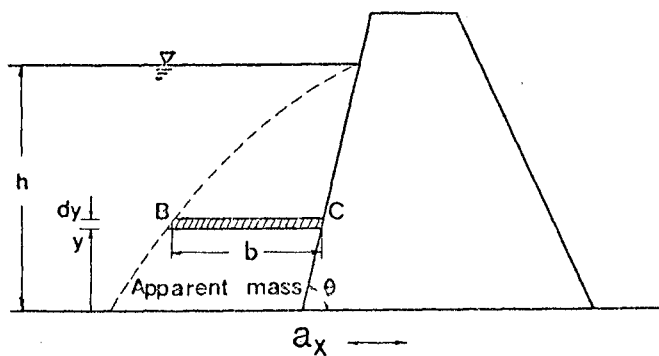


Figure 7. A Breakwater With An Inclined Front Face.

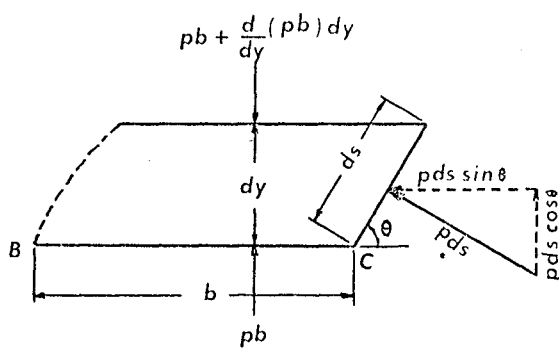


Figure 8. Equilibrium of Forces on a Fluid Element.

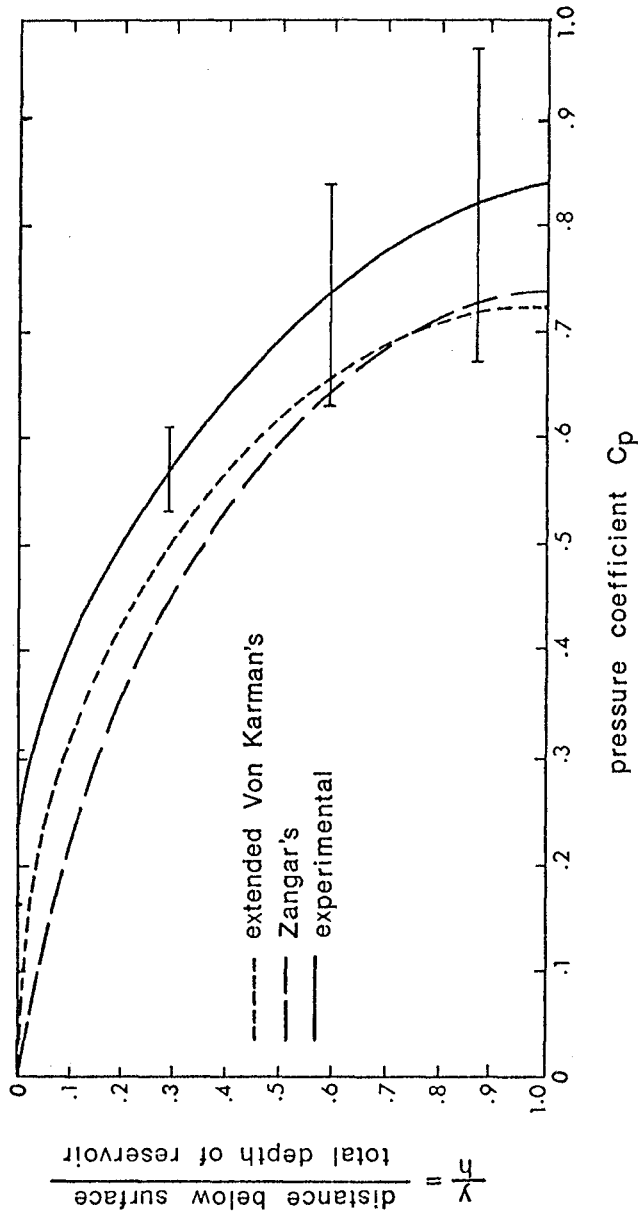


FIGURE 9. Comparison of Pressure Coefficients at Different Elevations on the Breakwater with a Vertical Front Face

The pressure coefficient, C_p , which appeared in the abscissa, is defined as

$$C_p = \frac{p}{\alpha \gamma h} \quad (15)$$

where γ is the specific weight of water and α is the ratio of horizontal earthquake acceleration to the gravitational acceleration, i.e.

$\alpha = \frac{a_x}{g}$. The experimental results for the inclined-surface case were shown in Figure 10, together with that of Sangar's and the theoretical curve.

It can be seen that for the vertical wall case, the comparisons were reasonably good with the exception that the pressure obtained in the experiment was non-zero at the free surface. This was caused by the surface waves induced by the breakwater motion. This effect was neglected in the theory. The surface-wave induced pressure became larger as the horizontal acceleration increased.

For the breakwater with a 30° inclined surface, the three curves were separated apart with the actually measured pressure significantly larger than that predicted by theory. The effect of surface wave, on the other hand, was not as pronounced as the vertical wall case.

Conclusions

Based on experimental evidence, rubble-mound breakwaters on a rigid foundation are found to be highly earthquake resistant. Extrapolating an envelope on the test result predicts that earthquakes of less than $1/2 g$ would not affect a breakwater to any significant extent. Possible failure due to foundation is not included in the consideration.

The fundamental damage mode is clearly the settlement of the crest, coupled with slight slope deformation. Slope steepness has much the influence that might be expected; the crest settled more on the side adjoining the steeper slope, and the horizontal displacement at the toe was also larger on the steeper side. The armor layers, two layers of dolosse in this case, remained largely intact. No hole greater than three clustered units (a commonly-used measure to indicate the extent of core exposure) was observed for all the tests performed, which includes tests with a horizontal acceleration as high as $2.8 g$. Under severe or repeated shocks, the dolosse mat tended to settle as a whole down the face of each side, causing thinning or rifting at the breakwater crest. Since settlement is the major mode of damage, presettlement becomes an important factor, which strengthens the resistance against earthquake loading.

Water level does not appear to be an important factor affecting the damage. One may tentatively conclude that the interlocking behavior of armor units plays an important role in earthquake resistance in addition to the internal friction of core material.

The approximate solution of Von Kármán agrees with experiments for the breakwater with a vertical face. For the breakwater having a face

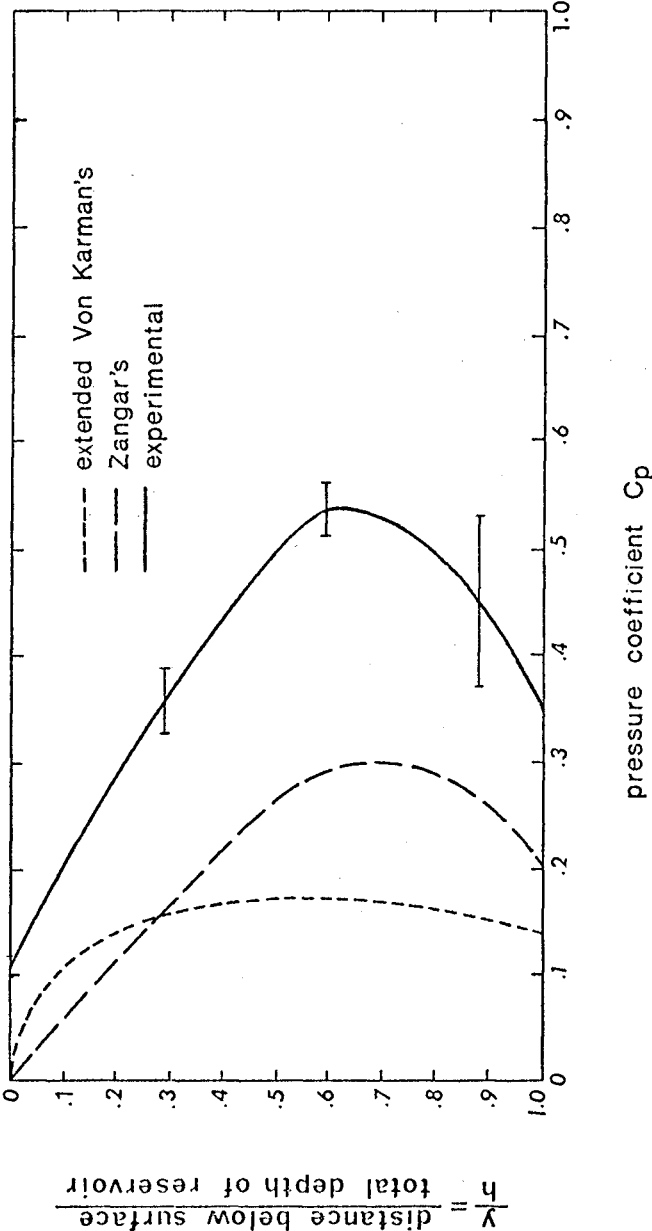


FIGURE 10. Comparison of Pressure Coefficients at Different Elevations on the Breakwater with a 30° Inclined Face

of large inclined angle, an approximate solution from the extended Von Kármán's theory deviates considerably from the experimental result. The maximum pressure is found to be at some distance above the bottom of the breakwater with 30° inclined face but at the base of the breakwater with a vertical face. The magnitude of maximum pressure as induced by earthquakes is not excessive as would result in structural damage. The total force, however, might contribute to the general instability.

In conclusion, rubble-mound breakwaters alone (without foundation consideration) are earthquake resistant in the structural sense. However, this conclusion should not be interpreted as suggesting that earthquake effects can be neglected in design. On the contrary, since crest elevation is an important factor in the functional and structural integrity of breakwaters against wave attack, the degree of possible settlement due to the design earthquake must be established and properly allowed for.

Acknowledgments

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