

Wave Stresses on Rubble-Mound Armour

Andrew Cornett* and Etienne Mansard*

Abstract

In this study, physical experiments are used to investigate the relationship between fluid velocities on the surface of two rubble-mounds and the shear stresses τ and normal stresses σ acting on the surface layer of rock armour. Results presented herein indicate that the peak magnitude of slope-parallel hydrodynamic forces on rubble-mound armour located below the still waterline can be reasonably well estimated using wave friction factors originally developed for rough turbulent oscillatory flow over impermeable horizontal beds. Stresses on rubble-mound armour are also compared to the prevailing surf similarity (Iribarren) parameter. The largest stresses are found to result from waves that form collapsing breakers.

1 Introduction

Fluid flows on a rubble-mound under wave attack are highly variable in space and time. The spatial and temporal distribution of kinematics depends in a complex manner on the character of the incident waves, the type of wave breaking, and properties of the structure such as slope, roughness and permeability. Not surprisingly, the normal and shear stresses induced by such kinematics are also temporally and spatially variable.

Kobayashi et. al. (1990a, 1990b, 1990c) describe and present results from a numerical model of wave interaction with rough permeable slopes that includes an analysis of the stability of armour stones. In this model, the hydrodynamic forces acting on armour stones in the surface layer are separated into drag, inertia and lift forces that are calculated in terms of the fluid velocity and acceleration parallel to, and above, the surface of the structure. In this formulation, the forces on armour stones are independent of the kinematics normal to the surface of the rubble-mound.

Tørum (1994) measured regular wave loads on a single irregularly shaped armour stone with a mass of 0.152 kg located 10.3 cm below the still waterline on the

*National Research Council of Canada, Institute for Marine Dynamics, Building M32, Ottawa, Ontario, Canada K1A 0R6

surface of a reshaped berm breakwater. Measurements of fluid kinematics close to the stone were used to model the slope-parallel force component as a summation of drag and inertia forces calculated from slope-parallel kinematics. The slope-parallel force was found to be drag dominated, such that the force peaks occurred in phase with the peaks of slope-parallel velocity. The slope-normal force was modelled using a similar drag and inertia formulation, augmented by an additional term to represent lift force. In this case, drag and inertia forces were calculated from slope-normal kinematics, while the lift force was computed from the slope-parallel velocity. The peaks of the slope-normal force led those of the slope-parallel force, and were significantly smaller in magnitude. Tørum concludes that the slope-normal force is not dominated by lift, and could not be adequately modelled by the assumed augmented formulation of the Morison equation.

Shear stresses on an impermeable horizontal bed under oscillatory flow are commonly expressed in terms of a wave friction factor f_w , defined by

$$f_w = \frac{2\tau_{\max}}{\rho u_{\max}^2} \quad (1)$$

where $\tau_{\max}$ is the maximum shear stress at the bed, and $u_{\max}$ is the maximum orbital velocity just outside the boundary layer. Riedel et. al. (1972) and Kamphuis (1975) report results on f_w obtained with a shear plate in an oscillating water tunnel as functions of the maximum amplitude Reynolds number $Re = u_{\max}a/\nu$ and the relative roughness a/k_s where ν is the kinematic viscosity, a is the amplitude of water particle orbits just outside the boundary layer, k_s is the Nikuradse sand grain roughness, given by $k_s \simeq 2D_{90} \simeq 2.5D_{n50}$, where D_{n50} is the nominal diameter of particles on the bed. Different flow regimes were delineated, corresponding to laminar, smooth turbulent, and rough turbulent flow. For rough turbulent flow, the wave friction factor was found to be independent of Re and could be well represented by the simple expression

$$f_w = \frac{2}{5} \left(\frac{a}{k_s} \right)^{-3/4} \quad \text{for } \frac{a}{k_s} \leq 100. \quad (2)$$

2 Experiments

Experiments to measure the wave stresses on rubble-mound armour were performed in the 'Coastal Wave Basin' of the National Research Council of Canada at Ottawa. Figure 1 shows a sketch of the lay-out for these experiments. Three separate test channels, each 0.65 m wide, were constructed near the centre of a 14 m wide wave flume. The entrances to each channel were calibrated to ensure very similar wave conditions. Two dimensional sections of a rubble-mound breakwater were constructed in two of the three test channels. The third test channel was left open to record incident wave conditions using an array of capacitance wave probes. Stresses on the armour layer due to waves were measured in the central test channel using a pair of instrumented armour panels, described below

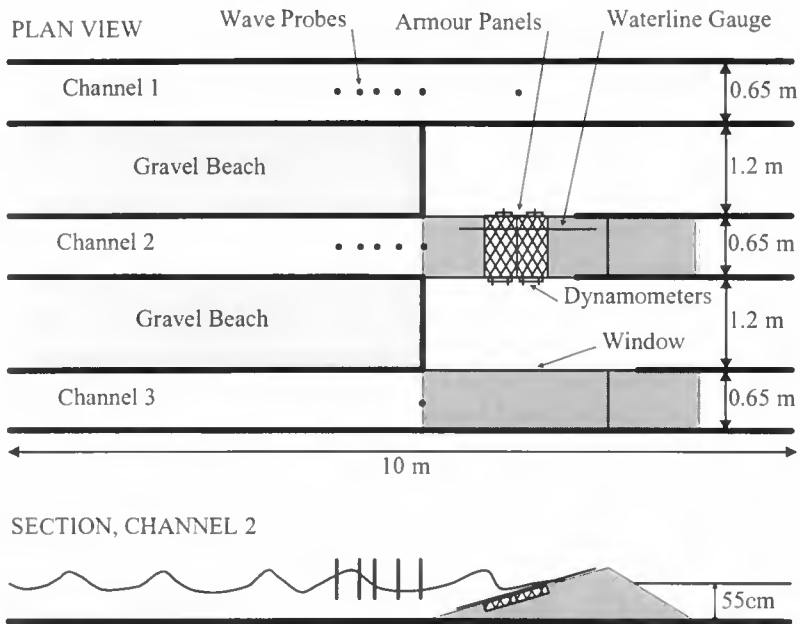


Figure 1: Sketch of the experiment lay-out.

in more detail. The third test channel was used to monitor damage to an identical rubble-mound constructed entirely from loose stones. The end of the wave flume was lined with a porous gravel beach that limited wave reflections to 5% or less. Similar absorbing beaches were installed between the three test channels. This set-up allowed simultaneous measurement of the incident waves, stresses on armour stones, and armour damage, relatively uncontaminated by spurious wave reflections.

Results for two different structures (test series 4 and 5) are reported here. Pertinent characteristics are summarized in Table 1. Cross-sections are sketched in Figures 2 and 3. The armour for both structures consists of finely graded, angular, granite rock with nominal diameter $D_{n50} = (M_{50}/\rho_a)^{1/3} = 4.2 \text{ cm}$ placed in two layers. The series 4 rubble-mound features a sea-ward slope of $\cot \alpha = 1.75$, a permeable core with $D_{n50} = 2.6 \text{ cm}$, $D_{85}/D_{15} = 1.3$ and no filter layer. The series 5 structure features a milder slope of $\cot \alpha = 3$, a thin filter layer with $D_{n50} = 1.9 \text{ cm}$, $D_{85}/D_{15} = 1.3$ over an impermeable core.

2.1 Armour panels

Each instrumented armour panel consists of 50 individual, irregularly shaped, aluminum model rocks, bonded together by spot-welds into a rigid, porous, rect-

	Series 4	Series 5
water depth (cm)	55	55
slope, $\cot(\alpha)$	1.75	3
core	permeable	impermeable
filter thickness (cm)	-	4
filter D_{n50} (cm)	-	1.9
filter D_{85}/D_{15}	-	1.3
armour thickness (cm)	8	8
armour D_{n50} (cm)	4.2	4.2
armour D_{85}/D_{15}	1.1	1.1
panel 1, centroid elev. (cm)	43.5	41.5
panel 2, centroid elev. (cm)	31.5	33.5

Table 1: Characteristics of the series 4 and 5 rubble-mounds.

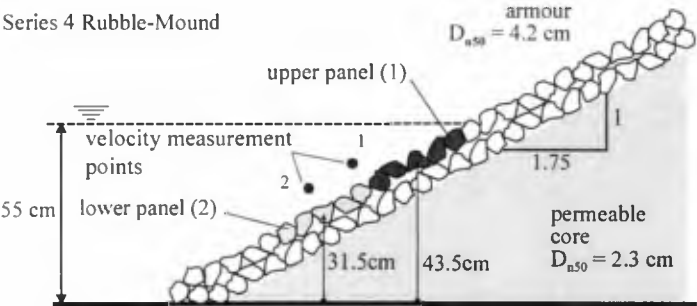


Figure 2: Cross-section sketch of the series 4 rubble-mound.

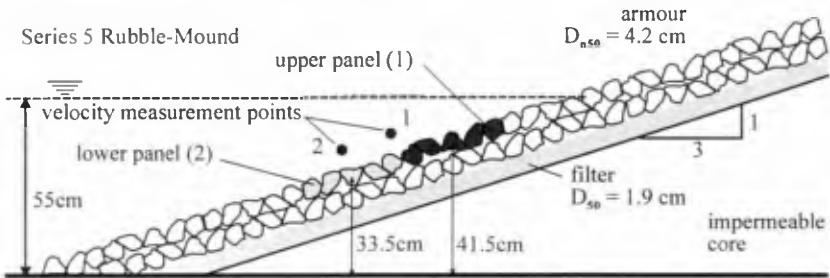


Figure 3: Cross-section sketch of the series 5 rubble-mound.

angular mat of armour stones in a single layer with approximate overall dimensions 64 cm by 23 cm by 5 cm . Each panel represents a 0.15 m^2 rectangular patch of armour stones. The panels were installed on the outer surface of the rubble-mound test sections, just below the still waterline, as sketched in Figures 2 and 3. The upper and lower panels are denoted as panels 1 and 2, respectively. The elevations of the panel centroids are included in Table 1. Each panel was connected at three points to a custom-built, five degree-of-freedom dynamometer located outside the walls of the test channel. Armour stones immediately surrounding the panels were glued in place so that the panels were isolated from adjacent materials by a thin ($\sim 2\text{ mm}$) gap that followed their irregular shape. Away from the armour panels, wire mesh was placed over the surface of the rubble-mound to restrain the motion of loose armour stones. This set-up was adopted as a trade-off between the need to isolate the panels from the surrounding breakwater materials, and the desire to minimize distortions in the modelling of the rubble-mound.

The total fluid force $\mathbf{F}_f$ acting on an armour panel can be written $\mathbf{F}_f = \mathbf{F} + \mathbf{F}_b$ where $\mathbf{F}$ is the hydrodynamic force and $\mathbf{F}_b$ is the buoyancy force. For a panel that remains fully submerged, the buoyancy force is constant and equals the mass of fluid displaced by the panel and some dynamometer hardware. In this case, isolation of the hydrodynamic force is relatively straightforward. When the submergence of the panel varies with time, the magnitude of the time varying buoyancy force $F_b(t)$ can be estimated from the instantaneous elevation of the waterline on the surface of the rubble-mound $\eta_s(t)$ as

$$F_b(t) = |\mathbf{F}_b(t)| = \begin{cases} F_{b,\max} & \text{for } \eta_s(t) > \eta_{s,0} + l \sin \alpha, \\ F_{b,\max} \left(\frac{\eta_s(t) - \eta_{s,0}}{l \sin \alpha} \right) & \text{for } \eta_{s,0} \geq \eta_s(t) \leq \eta_{s,0} + l \sin \alpha, \\ 0 & \text{for } \eta_s(t) < \eta_{s,0}, \end{cases} \quad (3)$$

where $F_{b,\max}$ is the buoyancy force when fully submerged, $\eta_{s,0}$ is the largest value of η_s for which the panel remains entirely dry, and $l \sin \alpha$ is the effective vertical distance between the lowest and highest parts of the panel. Appropriate values of l and $\eta_{s,0}$ for each panel were determined by experiment.

Both panels were located entirely below the still waterline. The lower panel remained entirely submerged during all wave conditions, while the upper panel became partially submerged during attack by larger waves. When necessary, (3) was applied to estimate the time-varying buoyancy force in order to isolate of the hydrodynamic force acting on the upper panel.

2.2 Waves, waterline motion and kinematics

The rubble-mounds were exposed to a variety of regular and irregular waves; however, results in regular waves alone are reported here. Incident wave characteristics were computed by zero-crossing analysis of the water surface elevation $\eta(t)$ recorded in the side channel at a location corresponding to the toe of the rubble-mounds. The regular waves ranged in height from 10 cm to 22 cm at peri-

ods of 1.5, 2.0 and 3.0 s. For each wave condition, reported values were averaged over 100 wave cycles.

The vertical motion of the waterline $\eta_s(t)$ on the surface of the instrumented rubble-mound was recorded using a capacitance wire wave gauge inclined parallel to, and located approximately 1 cm above, the surface of the structure.

Fluid velocities were measured using a pair of bi-directional electromagnetic velocimeters located 4 cm above the upper surface of the instrumented armour panels at the positions shown in Figures 2 and 3. These locations are believed to be outside the boundary layer. Sleath (1992) gives the approximate displacement thickness δ of an oscillatory boundary layer as $\delta \simeq 0.5 f_w a$. For rough turbulent flow, (2) may be re-arranged to give $a = 0.295 k_s f_w^{-4/3}$, whence the displacement thickness can be estimated by

$$\delta \simeq 0.147 k_s f_w^{-1/3} . \quad (4)$$

With $k_s = 2.5 D_{n50} = 0.105$ m, and $f_w = 0.15$, this equation gives $\delta \approx 0.029$ m, which suggests that the positions of velocity measurement are outside the boundary layer. Velocity components parallel and normal to the surface of the rubble-mound are denoted by u and w , respectively, where u is positive up-slope and w is positive away from the structure.

Water particle orbits are estimated by integration of the Eulerian velocity signal. In particular, the water particle displacement parallel to the face of the structure at time t_1 is given by

$$s(t_1) = \int_0^{t_1} u(t) dt . \quad (5)$$

3 Shear and Normal Stresses

The armour panels provided a steady, repeatable measure of the fluid forces exerted on a patch of surface layer armour stones due to wave attack. The hydrodynamic component of the fluid force was isolated by compensating the measured force for the buoyancy of the armour panel. The hydrodynamic force was separated into orthogonal components F_P and F_N , acting parallel and perpendicular to the surface of the rubble-mound. F_P was defined positive up-slope, while F_N was defined positive away from the structure. These hydrodynamic force components can be expressed as a shear stress τ and a normal stress σ , defined by

$$\tau(t) = \frac{F_P(t)}{A} , \quad \sigma(t) = \frac{F_N(t)}{A} \quad (6)$$

where $A = 0.15$ m² is the surface area occupied by a panel. The surface area can be expressed in terms of the number of armour stones N_a , and the porosity of the armour n , as $A = N_a D_{n50}^2 / (1 - n)$. The average hydrodynamic force components acting on a single armour stone located within the patch are denoted by f_P and

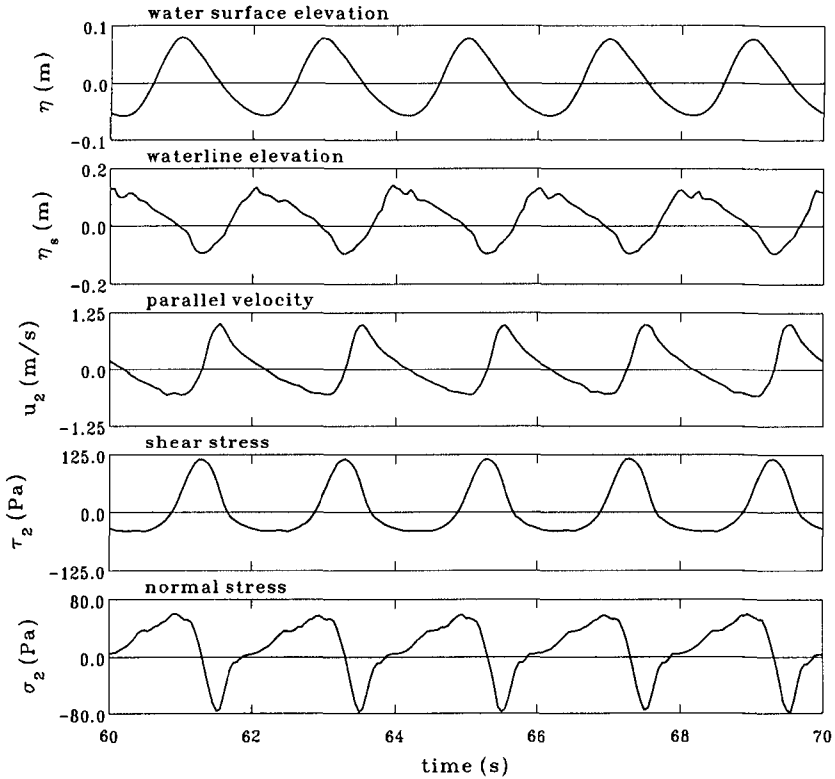


Figure 4: $\eta(t)$, $\eta_s(t)$, $u_2(t)$, $\tau_2(t)$, $\sigma_2(t)$ on the series 5 rubble-mound ($H = 14$ cm, $T = 2$ s and $\xi = 2.2$).

f_N , and can be estimated in terms of the shear and normal stresses as

$$f_P(t) = \frac{D_{n50}^2}{1-n} \cdot \tau(t) \quad , \quad f_N(t) = \frac{D_{n50}^2}{1-n} \cdot \sigma(t) \quad . \quad (7)$$

The shear and normal stresses defined by (6) include contributions from all hydrodynamic forcing mechanisms, including drag, inertia and lift forces.

Short segments of $\eta(t)$, $\eta_s(t)$, $u_2(t)$, $\tau_2(t)$ and $\sigma_2(t)$ from a series 5 regular wave test with $H = 14$ cm and $T = 2$ s are presented in Figure 4. The velocity signal features an asymmetric 'saw-tooth' shape that indicates large positive (up-slope) accelerations immediately prior to the maximum up-slope velocity. Negative (down-slope) accelerations are less intense, but prevail for a longer duration during each flow cycle. The shear stress time series features sharp positive (up-slope) peaks in approximate phase with the large positive accelerations, which

suggests that accelerations are a dominant forcing mechanism. Negative shear stress peaks are significantly smaller and more broad, which parallels the character of the negative fluid accelerations. In these wave conditions, the maxima and minima of the normal stress coincide with the negative and positive velocity peaks, respectively. Significant seepage flows normal to the surface of the rubble-mound, associated with drainage of the permeable outer layers, were observed during the tests, even for structures with an impermeable core. These slope-normal flows are an important factor contributing to the normal stresses on the surface layer of armour.

Individual flow cycles on the rubble-mound were defined to start and finish at the times of maximum runup. During each flow cycle, the maximum and minimum values of velocity, fluid particle displacement, shear stress, and normal stress were obtained and then averaged over 100 regular waves to give representative quantities, which are denoted by the subscripts $_{\min}$ and $_{\max}$; i.e. $u_{\min}$, $u_{\max}$, etc. Peak-to-peak values are denoted by subscript $_{pp}$, and are defined as the positive difference between the maximum and minimum value; i.e. $u_{pp} = u_{\max} - u_{\min}$.

For a given wave height incident to a given rubble-mound structure, the wave period influences the type of wave breaking that prevails. The effect of wave period on the type of wave breaking, and on the stability of rock armour, is commonly quantified in terms of the surf similarity (or Iribarren) parameter $\xi = \tan \alpha \sqrt{gT^2 / (2\pi H)}$. Battjes (1974) observed that the character of wave breaking on a slope depends on ξ , such that collapsing breakers prevail for $\xi \sim 3$, while plunging breakers prevail for $\xi < 2$, and surging breakers prevail for $\xi > 4$.

Van der Meer (1988) presented separate design formulae for the stability of rubble-mound armour under plunging and surging wave attack. His equations suggest that minimum stability occurs under collapsing breakers and that wave period has a strong influence on stability for plunging waves ($\xi < 2$), but has much less influence on stability for surging waves ($\xi > 4$). Van der Meer's equations also describe a dependence between armour stability and the permeability of a rubble-mound, such that stability is enhanced on more permeable structures.

Figure 5 shows peak-to-peak values of **shear** stress as a function of surf similarity for the upper and lower panels of the series 4 and 5 rubble-mounds. Here, τ_{pp} has been made non-dimensional by the factor $\rho g H$, which represents the pressure under a static column of water with height H . Several observations can be made.

- Panel 1 (the upper panel) always experiences greater shear stresses. This result is consistent with the numerical simulations of Kobayashi et al. (1990c) which suggest that armour damage is most likely to occur at an elevation approximately $0.75H$ below the still waterline, and that the likelihood of damage decreases below and above this elevation.
- On both panels, the normalized peak-to-peak shear stress is maximized for $\xi \sim 2.5$. For a given wave height, the shear stress exerted on a patch of armour stones is greatest for waves with periods such that $\xi \sim 2.5$. These

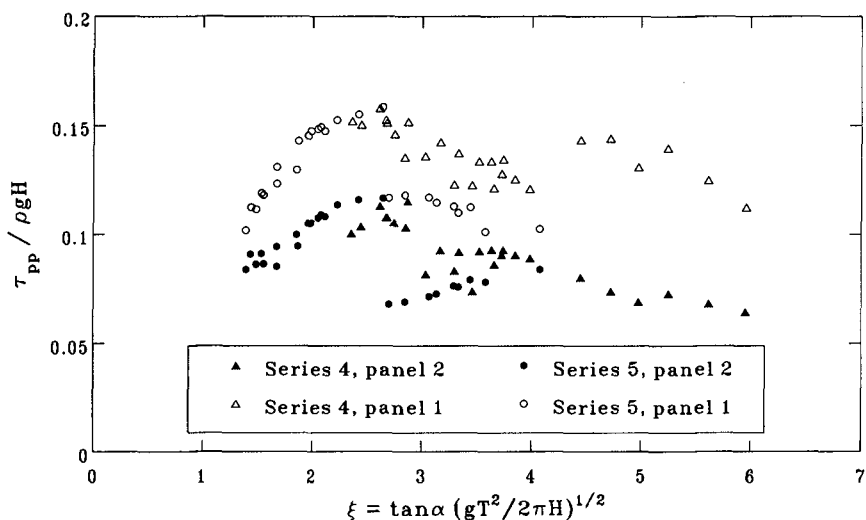


Figure 5: Non-dimensional peak-to-peak shear stress.

conditions are close to those associated with collapsing breakers. This result is consistent with idea that stability is minimized for collapsing breakers, and supports the form of stability design equation proposed by van der Meer (1988).

- Surf similarity has a strong effect on the normalized peak-to-peak shear stress in plunging waves ($\xi < 2$), but has relatively little influence on the shear stress in surging waves ($\xi > 4$). This result is also consistent with the design equations of van der Meer (1988), and suggests that plunging and surging breakers represent distinctly different processes.
- Tørum (1994) computes a shear stress of $\tau = 216 \text{ Pa}$ for a specific regular wave with $H = 0.2 \text{ m}$ and $T = 1.8 \text{ s}$. In non-dimensional terms, this is equivalent to $\tau / \rho g H = 216 / (9810 \cdot 0.2) = 0.11$, which is in general agreement with the shear stresses reported here.

A similar plot of non-dimensional maximum **normal** stress $\sigma_{\max} / \rho g H$ on the lower and upper panels of the series 4 and 5 rubble-mounds is presented in Figure 6. Positive normal stresses act to lift armour stones out of the surface layer and thus are critical to the stability and initial motion of armour units. In general, the variation in maximum normal stress measured on these two structures in various wave periods are less well described by ξ alone. In spite of this, the following observations can be made.

- Normal stresses are greater on the upper panel. Again, this supports the result that armour stones just below the still waterline are more susceptible

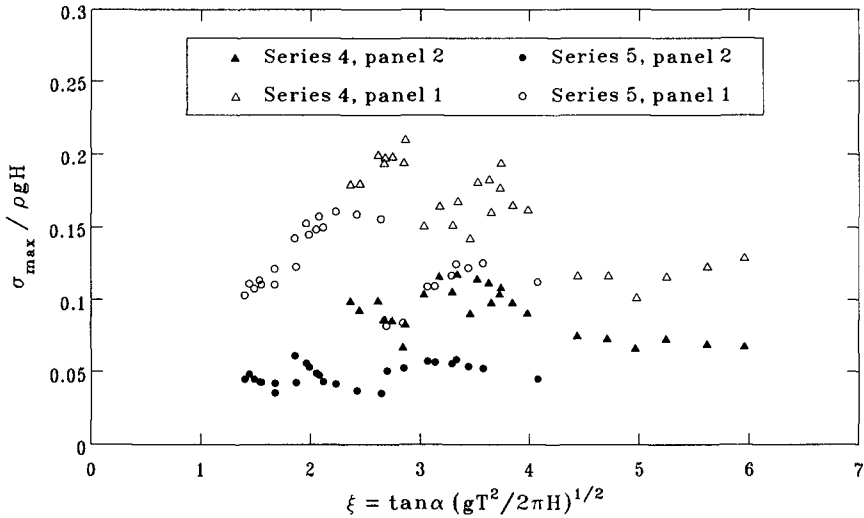


Figure 6: Non-dimensional maximum normal stress.

to damage than those lower down the rubble-mound. The normal stress maxima measured on the upper panel generally exceed the peak-to-peak shear stress in the same wave condition.

- For the same value of ξ , larger normal stresses are exerted on the armour of the steeper, more permeable series 4 rubble-mound.
- Surf similarity has a significant effect on the normal stress exerted on the upper panel, but has only a small influence on the normal stress acting on the lower panel.

4 Friction Factors for Rubble-Mound Armour

Flows on the surface of a rubble-mound under wave attack exhibit similarities and differences compared to wave driven flows on a horizontal seabed. While both flows are fundamentally oscillatory, flows on a rubble-mound tend to exhibit proportionately larger high-frequency content that produces ‘saw-tooth’ velocity fluctuations and larger accelerations. Flows on a rubble-mound also vary greatly with position. Near the toe, flows are similar to those on a horizontal seabed at similar depth, however, approaching the still waterline, velocities and accelerations are significantly amplified. Above the point of minimum rundown, the surface of the rubble-mound is only intermittently submerged. In this zone, the depth of flow can vary greatly throughout a wave cycle. When the water level outside the structure exceeds the level of the internal phreatic surface, water flows into the

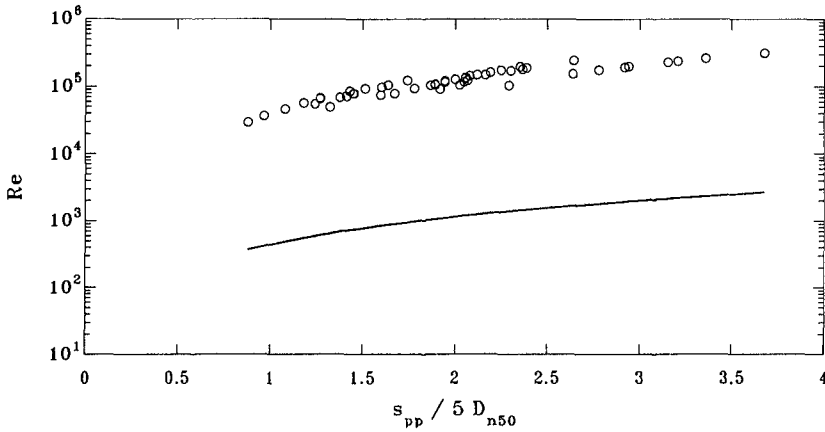


Figure 7: Orbital amplitude Reynolds number at velocimeter 2.

permeable outer layers. As the external water level recedes below the internal phreatic surface, water flows out of the permeable outer layers. Infiltration flow generally occurs above the mean water level during uprush while seepage flow is concentrated below the mean water level during downrush.

Kamphuis (1975) suggests the following criterion for the lower limit of rough turbulent flow:

$$Re = \frac{u_{\max} a}{\nu} \geq 200 \frac{a}{k_s} \sqrt{\frac{f_w}{2}} \quad \text{for rough turbulent flow.} \quad (8)$$

Substituting (2) for f_w yields

$$Re \geq 447 \left(\frac{a}{k_s} \right)^{1.375} \quad \text{for } \frac{a}{k_s} \leq 100. \quad (9)$$

An equivalent orbital amplitude Reynolds number for the surface of a rubble-mound can be written

$$Re_{pp} = \frac{\frac{u_{pp}}{2} \frac{s_{pp}}{2}}{\nu} = \frac{u_{pp} s_{pp}}{4\nu}. \quad (10)$$

The relative roughness of the flow on a rubble-mound can be written

$$\frac{s_{pp}/2}{2.5D_{n50}} = \frac{s_{pp}}{5D_{n50}} \quad (11)$$

which is equivalent to the quantity a/k_s . Figure 7 shows values of Re_{pp} at velocimeter 2, in which the solid line indicates the threshold criterion for rough turbulent flow given by (9). The data span the range of $s_{pp}/5D_{n50}$ (relative roughness) from 0.8 to 3.7 and indicate that rough turbulent flow prevails for all test conditions.

Measurements of the shear stress on panel 2 and the kinematics at position 2 have been used to construct friction factors for rubble-mound armour. Separate

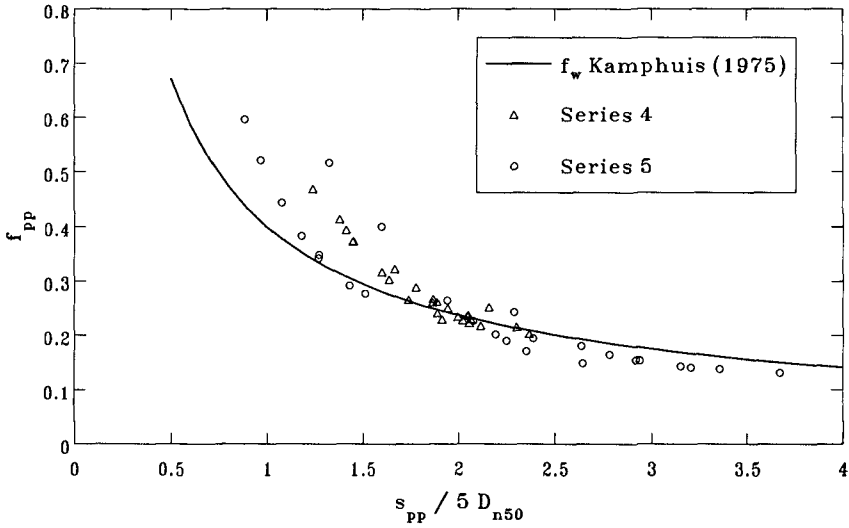


Figure 8: Friction factors on panel 2 for the series 4 and 5 rubble-mounds.

friction factors are computed for the uprush and downrush portions of the flow cycle according to

$$\begin{aligned} f_u &= \frac{2\tau_{\max}}{\rho u_{\max}^2} && \text{for uprush} \\ f_d &= \frac{-2\tau_{\min}}{\rho u_{\min}^2} && \text{for downrush.} \end{aligned} \quad (12)$$

A friction factor for the complete flow cycle is defined in terms of the peak-to-peak velocity u_{pp} and peak-to-peak shear stress τ_{pp} as

$$f_{pp} = \frac{2(\tau_{pp}/2)}{\rho(u_{pp}/2)^2} = \frac{4\tau_{pp}}{\rho u_{pp}^2} \quad \text{for the complete flow cycle.} \quad (13)$$

Friction factors for the complete flow cycle f_{pp} measured on panel 2 during regular wave attack of the series 4 and series 5 rubble-mounds are favorably compared in Figure 8 to estimates of f_w from (2). Data from the two different rubble-mounds show good agreement, however, there is evidence of a stronger variation with relative roughness than predicted by (2). Overall, these results suggest that the relation between friction factor and relative roughness on a rubble-mound under wave attack is similar to that for a rough, impermeable, horizontal bed under oscillatory flow. Moreover, these results suggest that wave friction factors might be used to estimate the shear stresses acting on rubble-mound armour exposed to wave attack.

Friction factors for the uprush and downrush portions of the flow cycle are presented in Figures 9 and 10. Most of the data for the uprush friction factor suggest that $f_u < f_w$, while most of the data for the downrush friction factor

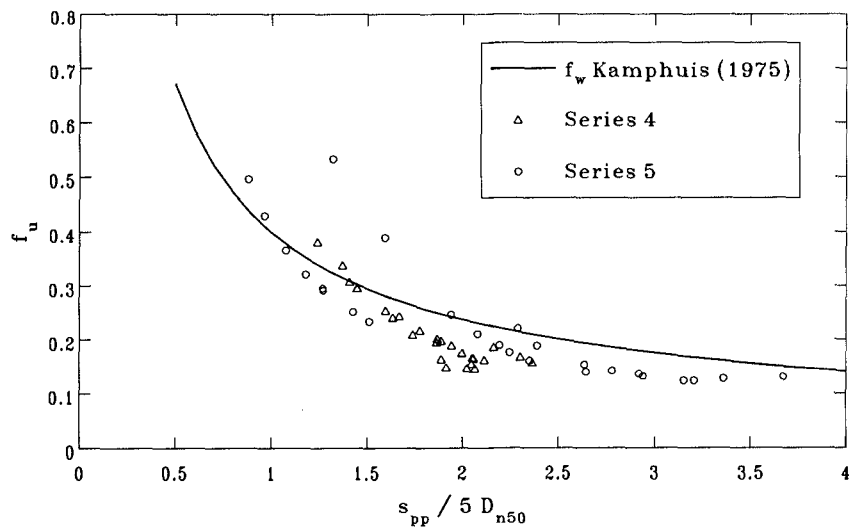


Figure 9: Uprush friction factors on panel 2 for the series 4 and 5 rubble-mounds.

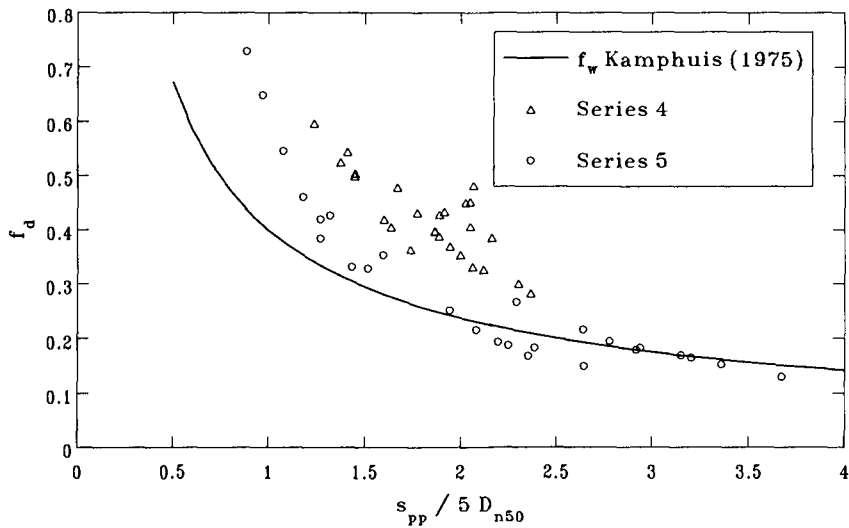


Figure 10: Downrush friction factors on panel 2 for the series 4 and 5 rubble-mounds.

suggest that $f_d > f_w$. In other words, the peak shear stress acting down-slope is generally larger than that predicted by (2), while the peak shear stress acting up-slope is generally smaller. The trend is particularly evident for the steeper, more permeable, series 4 rubble-mound. This behavior is not entirely surprising, considering the distorted, asymmetric, quasi-oscillatory flow cycles that prevail on the surface of a rubble-mound (see Figure 4), in contrast to the more symmetric flow cycles that prevail over horizontal beds under waves.

Kobayashi et. al. (1990a, 1990b, 1990c) assumed a constant friction factor of $f_w = 0.3$ in numerical simulations of regular and irregular waves on conventional rubble-mounds. Tørum and van Gent (1992) used a constant wave friction factor $f_w = 0.15$ in their numerical simulations of regular wave interaction with a permeable berm breakwater, while Tørum (1994) used $f_w = 0.3$. These values are in general agreement with the results reported here. However, the present results confirm that the friction factor for rubble-mound armour is a variable quantity that, for rough turbulent flow, depends on the relative roughness in a manner that is similar to that for oscillatory flow over a rough, impermeable, horizontal bed, and can be reasonably well predicted by (2).

5 Conclusions

Measurements of the shear and normal stresses acting on rubble-mound armour under regular wave attack have been presented. The stresses were measured on a pair of rigid porous armour panels, each supported by a five degree-of-freedom force dynamometer, and installed to minimize distortions to internal flows within the test structures. Results from two different structures have been considered.

- Greater shear and normal stresses were measured just below the still water-line than lower down the rubble-mound.
- Shear and normal stresses are maximized in waves for which $\xi \sim 2.5$, suggesting that collapsing breakers exert greater forces on armour stones and are therefore most critical to the their stability.
- The influence of wave period on shear stress is strong for plunging waves ($\xi < 2$), and relatively weak for surging waves ($\xi > 4$).
- The shear stresses reported here are in general agreement with the force measurements on a single armour stone reported by Tørum (1994).
- In some cases, normal stresses were measured that exceed the shear stresses.
- Greater normal stresses were recorded on the steeper, more permeable, series 4 rubble-mound.
- Friction factors for rubble-mound armour under regular wave attack are in general agreement with the wave friction factors f_w for rough turbulent flow computed from (2).

- Friction factors for uprush are generally less than f_w , while friction factors for downrush are generally greater. This behavior is related to the asymmetric, quasi-oscillatory nature of surface flows on a rubble-mound.

6 Acknowledgment

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