2009

Stability of Articulated Revetments Against Wave Attack on Shallow Soft Soil Slopes

Edmond Joseph Russo

Louisiana State University and Agricultural and Mechanical College, edmond.j.russo@usace.army.mil

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STABILITY OF ARTICULATED REVETMENTS
AGAINST WAVE ATTACK ON SHALLOW SOFT SOIL SLOPES

A Dissertation

Submitted to the Graduate Faculty of the
Louisiana State University and
Agricultural and Mechanical College
in partial fulfillment of the
Requirements for the degree of
Doctor of Philosophy

in

Department of Civil and Environmental Engineering

by

Edmond J. Russo, Jr.
B.S., Louisiana State University, 1990
M.S., University of New Orleans, 1997
December, 2009
To Alisa
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  - Ms. Debbie George
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## SYMBOLOGY

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<tr>
<td>a, b</td>
<td>Regression coefficients of $R_{max}$</td>
</tr>
<tr>
<td>$a_{hw}$</td>
<td>Harmonic wave amplitude</td>
</tr>
<tr>
<td>$a_n$</td>
<td>Cosine waveform coefficient</td>
</tr>
<tr>
<td>$a_0$</td>
<td>Constant average value of waveform</td>
</tr>
<tr>
<td>A, B</td>
<td>Variables of $H_w$</td>
</tr>
<tr>
<td>$A_f$</td>
<td>Cross sectional area of flow</td>
</tr>
<tr>
<td>$b_n$</td>
<td>Sine waveform coefficient</td>
</tr>
<tr>
<td>d</td>
<td>Water depth</td>
</tr>
<tr>
<td>$d_b$</td>
<td>Depth of water at breaking</td>
</tr>
<tr>
<td>$d_{15\text{filter}}$</td>
<td>Sieve 15 percent passing mean diameter of filter</td>
</tr>
<tr>
<td>$d_{15\text{soil}}$</td>
<td>Sieve 15 percent passing mean diameter of soil</td>
</tr>
<tr>
<td>$d_{85\text{soil}}$</td>
<td>Sieve 85 percent passing mean diameter of soil</td>
</tr>
<tr>
<td>$d_{15\text{upper}}$</td>
<td>Sieve 15 percent passing mean diameter of upper layer material</td>
</tr>
<tr>
<td>$d_{85\text{under}}$</td>
<td>Sieve 85 percent passing mean diameter of under layer material</td>
</tr>
<tr>
<td>$d_{#\text{material}}$</td>
<td>Sieve percent passing mean diameter of material</td>
</tr>
<tr>
<td>$C_q$</td>
<td>Combined drag and virtual mass coefficient</td>
</tr>
<tr>
<td>D</td>
<td>Time duration of a set of “n” total observations</td>
</tr>
<tr>
<td>$D_{50}$</td>
<td>Aggregate mean gradation diameter</td>
</tr>
<tr>
<td>$D_f$</td>
<td>Flow depth</td>
</tr>
<tr>
<td>$D_{eff}$</td>
<td>Effective “pipe” pathway diameter of ACM structure area of entry / exit due to uniform gaps present between adjacent blocks across the system in scale physical model</td>
</tr>
<tr>
<td>E(f)</td>
<td>Variance density spectrum</td>
</tr>
<tr>
<td>$f$</td>
<td>Darcy-Weisbach friction factor</td>
</tr>
</tbody>
</table>
Fr  Froude Number

g  Gravitational acceleration

$g_{m, p}$  Gravity of the model and prototype, respectively

$h_a$  Work performed by run up waves in displacing ACM armor elements in scale physical model

$h_i, h_{i+1}$  Elevation differences between instrument stations in scale physical model

$h_L$  Computed head losses for wave water penetrating the armor layer during run up in scale physical model

$h_t$  Water depth at the structure toe

$H$  Horizontal dimension

$H_{2\%t}$  Highest 2% of waves at the structure toe

$H_b$  Critical breaking wave height

$H_{j-1}$  Time-shifted deep water wave gauge reading preceding wave run up at time “j”

$H_{mo}$  Energy-based wave height of the zeroth moment

$H_o$  Deep water wave height

$H_o / L_o$  Deep water wave steepness

$H_{oFDS}$  FDS deep water wave height

$H_o$  Null hypothesis

$H_{oD=0}$  “No damage” wave height

$H_{DDW}$  Deep water wave height

$H_{DDW_s}$  Significant deep water wave height

$H_{RIR_j}$  Relative instantaneous wave run up at time “j” during experimentation

$H_{RIRs}$  Significant relative instantaneous run up

$H_s$  Significant wave height

$H_{st}$  Significant wave height near the structure toe
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_w$</td>
<td>Wave height</td>
</tr>
<tr>
<td>$j$</td>
<td>Individual sequential integer time observation from 1 to $n$</td>
</tr>
<tr>
<td>$k$</td>
<td>Wave number of deep water wave</td>
</tr>
<tr>
<td>$k_b$</td>
<td>Wave number of breaking wave</td>
</tr>
<tr>
<td>$k_s$</td>
<td>Equivalent sand roughness of material at flow boundary layer</td>
</tr>
<tr>
<td>$k_v l_a^3$</td>
<td>Armor unit volume</td>
</tr>
<tr>
<td>$k$</td>
<td>Permeability of the sub-base (m/s)</td>
</tr>
<tr>
<td>$k'$</td>
<td>Permeability of the armor layer (m/s)</td>
</tr>
<tr>
<td>$K$</td>
<td>Flow transition energy loss coefficient</td>
</tr>
<tr>
<td>$K^2$</td>
<td>D’Agostino’s omnibus test for probability distribution normality</td>
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<tr>
<td>$K_D$</td>
<td>Stability coefficient of Hudson Equation representing contributions due to interaction/interlock between individual armor elements of a structure</td>
</tr>
<tr>
<td>$K_h$</td>
<td>Depth parameter, to include armor surface roughness</td>
</tr>
<tr>
<td>$K_T$</td>
<td>Type and magnitude of turbulence experienced</td>
</tr>
<tr>
<td>$l_a^2$</td>
<td>Area of armor unit impacted by wave water</td>
</tr>
<tr>
<td>$l_a$</td>
<td>ACM block length</td>
</tr>
<tr>
<td>$L$</td>
<td>Linear distance in x-y 2-dimensional vertical space between $j^{th}$ time steps</td>
</tr>
<tr>
<td>$L_1$</td>
<td>Horizontal distance between locations of $\phi_b$ and the SWL</td>
</tr>
<tr>
<td>$L_2$</td>
<td>Horizontal distance between locations of $\phi_b$ and $\phi_u$</td>
</tr>
<tr>
<td>$L_b$</td>
<td>Breaking wave length</td>
</tr>
<tr>
<td>$L_f$</td>
<td>Characteristic length across flow path</td>
</tr>
<tr>
<td>$L_o$</td>
<td>FDS wave length</td>
</tr>
<tr>
<td>$L_{m, p}$</td>
<td>Length scale of the model and prototype, respectively</td>
</tr>
<tr>
<td>$m$</td>
<td>Number of data bin segments created from “n” total data series points</td>
</tr>
<tr>
<td>mm</td>
<td>Millimeters</td>
</tr>
</tbody>
</table>
Area under the signal of $\eta^2$ over $\Delta f_i$ up to $\lambda_o$

Second-order moment about the mean

Total integer number count of a defined quantity

Filter porosity

Froude Number scale ratio

Gravity scale ratio

Stability number for a given set of tested conditions, e.g., specific slopes of seaward approach and structure, structure armor unit size, shape, and composition, etc.

Length scale ratio

Time scale ratio

Velocity scale ratio

Typical width of openings between adjacent ACM blocks

Nominal opening size of armor

Statistical $p$-value for null hypothesis testing

Computed dynamic pressure gradients between stations within the filter layer in scale physical model

Wetted perimeter of flow

Flow inducing ACM uplift due to a combination of wave run up discharge through ACM openings, as well as return water exiting the filter layer down slope at the location of ACM uplift between stations in scale physical model

Pearson’s product moment coefficient

Population correlation

Hydraulic radius for open channel flow

Hydraulic radius for a perfectly symmetrical characteristic length of a cylindrical pipe (CP) diameter
R_{max}  
Maximum wave run up on rough quarry stone for irregular waves

R_{u2%}  
Highest 2% wave run up

R^2  
Coefficient of determination

R_j  
Wave run up at time “j”

R_{max}  
Maximum wave run up observed in the time series

R_{min}  
Minimum wave run up observed in the time series

Re  
Reynolds Number computed between stations in scale physical model

s_{x,y}  
Sample standard deviation for sample data series “x” and “y”

S  
Number of bin segments in spectral analysis

S_b  
Stability coefficient as a function of the relative permeabilities of the armor and under layers

S_r  
Ratio of unit weight of armor unit to unit weight of water

t  
Constant time step increment

\bar{t}  
Statistical test of R that R is zero

t_a  
Armor thickness, representing the length of the “pipe” pathway for flow of water into and out of structure filter layer

t_{a\text{(eff)}}  
Weighted geometric mean of an individual armor block with respect to tested hydromechanic potential

t_f  
Filter thickness

T  
Waveform period

T_p  
Peak period of signal spectrum

T_w  
Wave period

T_{DWWp}  
Peak period of the deep water wave

T_{RIRp}  
Peak period of the relative instantaneous run up

T_{sp}  
Peak period of the spectral hydromechanic potential
Critical vertically-averaged flow velocity on the revetment slope

Run down velocity

Run up velocity

Depth-averaged velocity computed between stations in scale physical model

Velocity of the model and prototype, respectively

Computed depth-averaged velocities of wave run up water entering the armor layer between stations in scale physical model

Vertical dimension

Volume of water during a time step increment present between the uplifted ACM layer and top of the filter layer between stations in scale physical model

ACM block width

Weight of a single ACM unit

Flow width

Weight of individual armor unit

Uniform distance between stations in scale physical model

5% confidence interval estimate

95% confidence interval estimate

Paired sample data series

Armor layer vertical displacement between stations in scale physical model

Location on slope relative to the SWL

“z”-score for confidence interval estimates

Statistical significance level

Slope angle of the water bottom

Coastal structure slope angle

Near shore slope ratio
$\Delta$ Relative density of armor material to density of wave water

$\Delta f, \Delta f_k$ Discrete progressive bandwidth frequency

$\Delta t, \Delta t_j$ Constant observation time increment

$\gamma$ Unit weight of water

$\gamma_{am}$ Model armor specific weight

$\gamma_{ap}$ Prototype armor specific weight

$\gamma_b$ Berm reduction factor

$\gamma_f$ Slope roughness reduction factor

$\gamma_h$ Shallow foreshore reduction factor

$\gamma_r$ Unit weight of armor unit

$\gamma_w$ Unit weight of water

$\gamma_\beta$ Oblique wave attach reduction factor

$\eta$ Randomly-generated, time-averaged water waveform surface observation

$\eta^2$ Individual wave component

$\eta(t)$ Fourier series of waveform surface elevation as a function of time

$\lambda_0$ Nyquist frequency

$\mu$ Sample arithmetic mean

$\nu$ Number of degrees of freedom

$\nu_f$ Kinematic viscosity of flow fluid

$\xi_{eq}$ Equivalent breaker parameter for a slope with a berm

$\xi_o$ Iribarren number, otherwise termed the breaker parameter, or surf parameter

$\pi$ $\Pi \sim 3.14159$

$\rho_w$ Mass density of fresh water

$\rho_{am}$ Mass density of the armor in the model
\( \rho_{ap} \) Armor mass density of the prototype

\( \rho_{wm} \) Mass density of (fresh) water of the model

\( \rho_{wp} \) Mass density of water of prototype in coastal waters, assumed to be salt water

\( \sigma \) Population standard deviation

\( \sigma_{\text{crit}}^2 \) Critical value of magnitude squared coherence to test the null hypothesis that there is no correlation between two autospectra

\( \sigma_{\eta}^2 \) Statistical variance of \( \eta \)

\( \varphi \) Force-displacement function termed the “hydromechanic potential,” and is effectively equivalent to the number of individual ACM blocks mobilized

\( \varphi_s \) Significant spectral hydromechanic potential

\( \phi_b \) Maximum piezometric head in filter

\( \phi_u \) Piezometric head in filter exerting armor uplift

\( \Phi \) Stability parameter depending on armor design type and shape

\( X^2 \) Chi Squared distribution

\( X^2_{99.5} \) \( X^2 \) probability at the 99.5 percentile cutoff value

\( \Psi \) Critical Shield’s parameter for the armor design type and shape

\( \omega \) Waveform fundamental frequency
## TERMINOLOGY

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<th>Term</th>
<th>Definition</th>
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<tr>
<td>ACM</td>
<td>Articulated Concrete Mattress</td>
</tr>
<tr>
<td>BOUSS-2D</td>
<td>Hydrodynamic wave model based on fully non-linear form of Boussinesq-type equations</td>
</tr>
<tr>
<td>CP</td>
<td>Cylindrical pipe</td>
</tr>
<tr>
<td>DFT</td>
<td>Discrete Fourier Transform</td>
</tr>
<tr>
<td>DWW</td>
<td>Deep Water Wave</td>
</tr>
<tr>
<td>EMD</td>
<td>Empirical Mode Decomposition</td>
</tr>
<tr>
<td>ERDC CHL</td>
<td>US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory</td>
</tr>
<tr>
<td>FDS</td>
<td>Fully Developed Sea</td>
</tr>
<tr>
<td>FEMA</td>
<td>Federal Emergency Management Agency</td>
</tr>
<tr>
<td>FFT</td>
<td>Fast Fourier Transformation</td>
</tr>
<tr>
<td>GSSHA</td>
<td>Gridded Surface and Subsurface Hydrologic Analysis Model</td>
</tr>
<tr>
<td>HHT</td>
<td>Hilbert-Huang Transformation</td>
</tr>
<tr>
<td>HSDRRS</td>
<td>Hurricane and Storm Damage Risk Reduction System</td>
</tr>
<tr>
<td>Hz</td>
<td>Hertz</td>
</tr>
<tr>
<td>IPET</td>
<td>Interagency Performance Evaluation Team</td>
</tr>
<tr>
<td>JONSWAP</td>
<td>Joint North Sea Wave Project</td>
</tr>
<tr>
<td>LACPR</td>
<td>Louisiana Coastal Protection and Restoration Project</td>
</tr>
<tr>
<td>LIDAR</td>
<td>Light Detection and Ranging</td>
</tr>
<tr>
<td>MSU</td>
<td>Mat Sinking Unit</td>
</tr>
<tr>
<td>NAVD</td>
<td>North American Vertical Datum</td>
</tr>
<tr>
<td>N.T.S.</td>
<td>Not to scale</td>
</tr>
</tbody>
</table>
PDF  Probability Density Function
PSD  Power Spectral Density estimate
RIR  Relative Instantaneous Run up
R&D  Research and development
SERDP Strategic Environmental Research and Development Program
SLR  Sea level rise
SWL  Still water level
USGS US Geological Survey
USACE US Army Corps of Engineers
1-D  One-dimensional
2-D  Two-dimensional
UNIT EQUIVALENTS

The following unit equivalents were used in this research, which involved conversions between field prototype and scale model dimensions, as well as in working with units of the laboratory instrumentation output as that pertained to model scale.

Length

1 foot (ft) = 12 inches (in.) = 0.3048 meter (m) = 304.8 millimeters (mm)

Acceleration

32.2 feet/second (ft/s) = 9810 millimeters/second (mm/s)

Mass density

1 pound-second²/feet⁴ (lb-s²/ft⁴) = 5.15 x 10⁻⁷ kilogram/millimeter³ (kg/mm³)
ABSTRACT

Continuously-connected, articulated revetment systems have potential to decrease the weight of armor cover in resisting wave attack, compared to traditional designs. Modes of instability for sloping revetments include uplift, sliding, and toe roll-up. Design methods are summarized by McDonnell (1998), Pilarczyk (1998), and Herbich (1999). Russo (2003) conducted a field prototype scale investigation on performance of Articulated Concrete Mattresses (ACMs) in coastal Louisiana, which demonstrated this structure’s ability to resist a range of wave loading conditions, and inspired scoping of further research to quantify structure performance beyond known limits.

Present research expanded earlier works by examining fundamental physical processes of wave loading near the theoretical threshold of structure incipient motion. The motivation for further investigation and modeling modes of failure is to:

- demonstrate a method to support the design selection process,
- optimize revetment dimensions when articulated block is considered the most appropriate application, and
- meet earthen slope protection requirements with relatively low ground pressures exerted by the armor layer for use in soft soil conditions.

A new structure performance metric is derived as the physically dimensionless “hydromechanic potential,” which is used to quantify structure movement as an interconnected system under wave attack. Research involved using a spectral hydromechanics analytical approach, with instrumented physical model results, to demonstrate a capability for constraining uncertainty on the behavior of revetments in specified conditions. Physical modeling was conducted based on dimensional analysis and similitude criteria. Physical modeling and spectral analysis were based on principles of hydrodynamics and structure mechanics of articulated revetment system configurations at incipient motion under irregular wave conditions.
Theoretical equilibrium exists when destabilizing wave loading forces are in balance with restoring gravitational forces of the structure. Tests of prior works, conducted through traditional methods, were generally able to measure structure performance under wave attack to between 3.7 and 8 of the ratio of destabilizing-to-restoring forces. Despite being the best available physical data measurable to-date, Herbich (1999) characterized structure performance in this range for design as “doubtful”. Results of this dissertation research indicated that a new lower limit is detectable at the threshold of equilibrium based on hydromechanic potential.
CHAPTER 1. INTRODUCTION.

1.1 Background. Louisiana’s coastal wetlands perform many diverse functions that provide value to our Nation’s people, economy, and environment. Their ability to serve as emergent vegetated buffers from wave action between open waters and low-lying assets and inhabitations is valued in flood risk management. Composition and performance of flood risk management structure designs, such as vegetated earthen levees, traditionally take into account initial absorption of wave loadings by the presence of flood-side wetland buffers (Figure 1-1, USACE (2009a)). This reliance is a concern, considering historic and predicted future loss of wetland features in coastal Louisiana.

![Coastal wetlands and vegetated earthen levee](image)

**Figure 1-1. Interconnected levee and wetland systems, Larose to Golden Meadow Hurricane and Storm Damage Risk Reduction System (HSDRRS) levee, Louisiana (USACE, 2009a).**

Current and predicted future coastal Louisiana landscape losses remain largely unabated (Figure 1-2, US Geological Survey (USGS), 2003), despite coastal restoration efforts conducted to-date (US Army Corps of Engineers (USACE), 2009b). The combined relative effect of sea level rise and regional geological subsidence is a driving factor for continued coastal landscape losses in the future. Table 1-1 presents a summary of relative sea level rise projections for coastal Louisiana from the years 2010 to 2060 (USACE, 2009). Projections 1 and 2 of Table 1-1 are based on information from Meehl (2007) and the National Research Council (NRC, 1987), respectively.
Table 1-1. Relative sea level rise projections for coastal Louisiana between 2010-2060 (USACE, 2009).

<table>
<thead>
<tr>
<th>Basis for Value</th>
<th>Pontchartrain Basin (Planning Unit 1)</th>
<th>Delta Plain (Planning Units 2, 3a, and 3b)</th>
<th>Chenier Plain (Planning Unit 4)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Historic rate (for comparison only)</td>
<td>0.2 m (0.7 ft)</td>
<td>0.4 m (1.3 ft)</td>
<td>0.2 m (0.7 ft)</td>
</tr>
<tr>
<td>Future Projection 1 (based on Intergovernmental Panel of Climate Change values)</td>
<td>0.4 m (1.3 ft)</td>
<td>0.6 m (1.9 ft)</td>
<td>0.4 m (1.3 ft)</td>
</tr>
<tr>
<td>Future Protection 2 (based on National Research Council values)</td>
<td>0.8 m (2.6 ft)</td>
<td>1.0 m (3.2 ft)</td>
<td>0.8 m (2.6 ft)</td>
</tr>
</tbody>
</table>

If sea level rise projections in these ranges are realized, waters of the Gulf of Mexico across south Louisiana will generally encroach inland from present conditions (Figures 1-3a through 1-3d, Swenson, 2009). As a compounding factor, any potential future increases of coastal storm frequency and intensity that is realized due to inter-decadal mode variability and climate change effects to the earth’s atmosphere and oceans could result in increased surge and wave conditions around near shore shallow open waters and low-lying coastal areas (Scavia, et al, 2002). In these conditions, without maintenance and restoration of natural coastal buffering features for initial wave energy absorption, there is an anticipated need in providing armoring for earthen levees having greater wave exposure to the Gulf of Mexico.
1.2 Problem. Earthen levee structure designs that become exposed to open seas across coastal Louisiana will require modification to increase their resistance to direct wave attack. Earthen levees are vulnerable to deterioration at the seaward slope and crown due to chronic wave attack under daily ambient conditions, as well as under periodically-occurring coastal storm conditions. Besides the concern the latter conditions pose on structure breaching and catastrophic system failure, there would
be increased maintenance and repair costs over the project lifespan without additional seaward frontal protection from daily prevailing direct wave exposure. Compounding the problem of earthen levee seaward frontal protection design are the relatively poor subsurface soil conditions in coastal Louisiana, which are subject to significant amounts of consolidation under massive static downward vertical surcharge loadings of earthen levees. These conditions pose a challenge in using traditional
armoring techniques to cost effectively achieve levels of service required. In this case, the armor design relies significantly on mass for stability against wave attack, adding to levee foundation soils surcharging.

1.3 Significance. There are hundreds of miles of earthen levees incorporated into flood risk management systems across south Louisiana. These structures may become subject to increased wave exposure due to current and future potential coastal landscape changes. While efforts are underway for maintaining and restoring coastal wetlands as part of flood risk management structure design conditions, plans should be prepared to provide engineered seaward frontal protection to earthen levees that is currently required and/or may become required, depending on local coastal storm surge and wave design conditions and future relative sea level rise scenarios. Engineering procedures require further development beyond current knowledge to enhance performance quantification of articulated revetments under wave attack for comparison of life cycle costs with other alternative measures. Such engineering procedures are required to support decisions on selection of approaches to protect earthen levees. Given the implication of costs into the billions of dollars to enact earthen levee armoring plans, development of engineering procedures to quantify performance of articulated revetment designs under wave attack is prudent and desirable to identify the solution with greatest value to flood risk management.

1.4 Needs. Extensive research and development (R&D) has been conducted on analysis and design of rubble mound structures, rip rap, and manufactured concrete armor units for coastal applications. The emphasis of this work has been on increasing structure constructability and performance, as well as for decreasing implementation costs. The performance advantage of concrete armor unit designs over that of rubble mounds and rip rap is based on greater stability due to shape efficiency and interlocking in constructed configurations. The result is an ability to decrease individual unit size and weight in application, thus leading to decreased project implementation costs (USACE, 2006). There is a tradeoff with loss in armor unit stability under given water level and wave
loading conditions as the weights of individual armor units with given shapes are decreased.
Continuously-connected, i.e., articulated armor unit systems offer increased stability potential under wave attack. USACE (1989) model tested Articulated Concrete Mattresses (ACMs) for stability in an application for earthen dike protection at Lake Okeechobee, Florida. Testing involved identification of modes of toe and slope instability under wave action. The work was aimed in establishing limiting wave heights for stability of single, double, and triple layer mat protection, offering options that effectively decrease armor unit size for comparable designs of individually-placed armor units. Extensive research in the field has also been performed in the Netherlands (Pilarczyk, 1998).

Traditional practice for evaluating structure stability under specified wave loading conditions is based on “no damage” criteria to the structure (Pilarczyk, 1998). As a practical matter, “no damage” involves a threshold limit of very minimal damage rates, which are considered repairable during routine maintenance (USACE, 2006). This approach could be a concern if structure maintenance is not able to be performed between damaging coastal storm events, as is generally assumed possible by traditionally-observed practices.

1.5 Requirements. USACE (1995) describes revetment armor as generally being relatively massive to resist wave action. Consequently, it imparts relatively large ground pressures that may induce significant total and differential ground settlements below the structure compared to flexible armor. A decrease in individual unit mass is anticipated to become a need for applications in Louisiana coastal protection and restoration, with potentially-increased coastal storminess and relative sea level rise, and considering soft soil foundation conditions that limit design options for geotechnical stability under expected hydrodynamic wave loadings. Discrete element armoring options have limited technical feasibility for implementation in these challenging field conditions, and/or they have potential to be too cost prohibitive over the life cycle to make projects viable. As improved coastal engineering armoring solutions for these conditions are developed, seaward frontal protection designs could be optimized to maintain expected earthen levee performance levels in the future. An articulated
revetment design, which can be competitive in life cycle performance and cost to traditional methods (e.g., individually-placed armor units), has the potential to meet the need of stabilizing earthen levee structures from deterioration under wave attack.

Critical factors for success in modeling an articulated revetment system include identifying armor unit stability against design forces, as well as ensuring system design integrity during construction. Filter stone and geotextile fabric underlying the armor layer act as supporting elements for stable design. These features must:

- maintain separation between the armor and bank soils,
- manage buildup of hydrostatic forces under the revetments during wave attack, and
- retain bank soils from erosion through the filter layer and armor blocks (McConnell, 1998).

Consideration must be given to geotechnical slope stability, bearing capacity, and settlement of the foundation soils, for structural support as well as to facilitate the construction process (Terzaghi, Peck, and Mesri, 1996).

Existing literature describes principles of conservation of mass and momentum resulting in structural failure modes and mechanics of articulated revetment systems to wave loadings (ASCE, 2003). However, these methods use many experimentally-derived factors quantitatively link the structure response to the hydrodynamics of wave loadings. A methodology based on hydrodynamics and structure mechanics process interactions, supported by spectral analysis and physical modeling that is performed in this research, advances the state of understanding in this field using fewer experimental variables. Statistical expression on the confidence and variability in results is also provided in this new research.

This research expands the knowledge base on performance of articulated revetments that are subject to wave attack. An approach termed “spectral hydromechanics” is demonstrated for its applicability in the design of armor for erodable sloping structures, such as earthen levees used in coastal flood risk management. Additional research at greater levels of detail would enable use of the
methodology in planning, engineering, and design phases for earthen levee armoring in cases
articulated revetment was found most economical among competing options.

This dissertation is organized as follows:

- In Chapter 3, an experimental design approach is explained, which is used to guide laboratory test planning, design, execution, data collection, data analysis, spectral modeling, hypothesis testing, and statistical analyses.
- Relevant physical processes of wave loading and structure response are summarized in Chapter 4 for describing the problem and research context.
- Chapter 5 contains a description of the laboratory wave flume physical scale model planning, design, and operation.
- The laboratory instrumentation, data collection, and data analysis effort is explained in Chapter 6.
- In Chapter 7, the spectral hydromechanics research is discussed for the program of completed laboratory testing.
- Physical process mathematical and statistical modeling is summarized in Chapter 8, in context of prior works in the field.
- Chapter 9 presents findings and conclusions of the research, summarizing the importance of a demonstrated new capability for detecting and measuring structure incipient motion.
- Chapter 10 contains future research recommendations beyond the scope of the newly demonstrated method.
CHAPTER 2. LITERATURE REVIEW.

The following is a summary of governing physical processes and properties that support formulation of assumptions, advancement of research, and model development, towards quantifying performance of articulated revetments in coastal applications.

2.1. Water Level and Wave Modeling. Figure 2-1 illustrates the relationships of the basic processes that govern coastal wave action, sediment movement, and structure interaction (Balsillie and Berg, 1972; Dean and Dalrymple, 2002; NOAA Coastal Services Center, 2009).

![Figure 2-1. Coastal wave influence processes (Balsillie and Berg, 1972; Dean and Dalrymple, 2002; NOAA Coastal Services Center, 2009).](image)

Prevailing winds, storm winds, physical boundaries of the wave generating area, and bottom configuration, all contribute to wave period, height, and angle with respect to the shoreline. These wave characteristics result in long and cross shore waves and currents being generated, which together with tidal effects, influence the velocity magnitude and direction of water movement. Water
movements induce sediment bed load transport, and when these phenomena interact, there is an effect on sediment movement rates and directions. Sediment movement rates and directions also are impacted by the configuration of the coastline and the presence of any coastal structures that may be in place in the area of influence. The net sediment transport rate and direction with respect to emergent features is related to whether these features are eroding or accreting.

Skafel and Bishop (1994), and Davidson-Arnott and Ollerhead (1995), state that open, cohesive coastlines often experience liberation of fine-grained sediments from the water bottom into the water column due to near shore waves. Tidal currents re-distribute fine-grained sediments over great cyclic excursion distances across estuarine basins, due to the low settling potential of these materials. Wave breaking on cohesive, fine-grained, shallow- to very shallow-sloping and vegetated coastlines, such as the Mississippi River Deltaic Plain in Louisiana, is a primary factor causing erosion (Coleman, 1988). Daily, chronic wave action that occurs in normal tide ranges of coastal Louisiana cause a significant amount of erosion of emergent coastal features. Rapid rates of soil mass erosion may also occur during coastal storm conditions. USACE (2006) states that the sea state in nature during coastal storms is always short-crested and irregular. The effects of wave-induced erosion may be observed in the Louisiana Coastal Plain through patterns of land loss since about 1900 (Britsch and Dunbar, 1993, and Gagliano, et al, 1981).

A simplified, deterministic approach may be used to roughly estimate near shore, short-crested, wind-generated wave actions, beginning with establishment of the incipient wave generated in deep waters to the breaking wave transformations in shallow waters. Resio, et al (2003) gathered and analyzed wave data in the Joint North Sea Wave Project (JONSWAP). This work resulted in nomograms relating wind-wave parameters. At Fully Developed Sea (FDS) conditions, wave parameters are at their maximum achievable levels, considering wind generating potential. An analysis by Le Roux (2007) of this data for a FDS state yielded identification of the following relationship:
\[ H_{oFDS} = L_o / 9\pi \]  

where:

\( H_{oFDS} \) = FDS wave height, which is the maximum-achievable value of \( H_o \), the deep water wave, and \( L_o \) = FDS wave length.

Le Roux (2007), after Sakai and Battjes (1980) and Cokelet (1977), established the following relationship for the wave height (\( H_w \)) in deep water is as a function of the deep water depth (\( d \)):  
\[ H_w = H_o \{ A \exp \left[ \frac{H_o}{L_o} B \right] \} \]

where:

\[ A = 0.5875 \left( \frac{d}{L_o} \right)^{-0.18} \quad \text{for} \quad \frac{d}{L_o} \leq 0.0844 \]  
\[ = 0.9672 \left( \frac{d}{L_o} \right)^2 - 0.5013 \left( \frac{d}{L_o} \right) + 0.9521 \quad \text{for} \quad 0.0844 < \frac{d}{L_o} \leq 0.6 \]
\[ = 1 \quad \text{for} \quad \frac{d}{L_o} > 0.6 \]  
\[ B = 0.0042 \left( \frac{d}{L_o} \right)^{2.3211} \]

Based on data contained in USACE (2006), the critical breaking wave height (\( H_b \)) was determined by Le Roux (2007) as a function of breaking depth (\( d_b \)) and its bottom slope (\( \alpha_b \)), as follows:
\[ H_b = d_b \left( -0.0036 \alpha_b^2 + 0.0843 \alpha_b + 0.835 \right). \]

The values of \( H_b \) and \( d_b \) may be found iteratively using the equations shown above for \( H_o \), \( H_w \), and \( H_b \), for shores with shallow depths and slopes. Based on JONSWAP data analysis, Le Roux (2007) found the following relationship for \( H_{oFDS} \):
\[ H_{oFDS} = 0.0542 T_w^{2.0156}. \]

where:

\( T_w \) = Wave period.

Alternatively, a simple relationship for obtaining \( H_b \) by Miche (1951) is:
\[ H_b = (0.64 / k) \tanh(k d_b) \]

and
\[ k = \frac{2 \pi}{L_o} \]  

where:

\( k \) = Wave number in deep water.

This equation reduces to the following for shallow water conditions where the quantity “\( k d_b \)” becomes small (Smith, et al, 1999):

\[ H_b \leq 0.64 d_b. \]

The approaches described above assume perpendicular wave attack of the shore. Wave angles from any other direction on a straight shoreline will pose a less severe impact than a normal angle of approach. These methods alone are unable to be used for determining the affects of wave refraction/diffraction along complex shore shapes. In these conditions, waves and water levels may become amplified, especially as waters become shallow.

For advanced planning, analysis, and engineering design studies of flood risk management systems, probabilistic water level and wave characteristics must be defined as forcing functions. A combination of measured and calculated values are typically used in this process to establish design water levels, wave heights, and wave periods for determination of wave run up and loading on coastal structures. Local foreshore conditions have an affect on these parameters in the vicinity of the shore (Ritzen, Wolters, Berger, Seijffert, and Rijks, 2001). When determining wave and water level conditions, future uncertainties of climate change effects should be considered for the intended life cycle of the project (USACE, 2009b).

A highly advanced suite of predictive models has been developed and used in a joint probabilistic simulation technique with optimal statistical sampling to obtain water level and wave information for coastal flood risk management system planning, analysis, and design, as described in Appendix A. Formulation of these models are based on first principles for conservation of mass and momentum. The equations are implemented using finite element techniques for numerically solving partial differential equations across a large-domain grid of the Atlantic Ocean and Gulf of Mexico. This is
the most advanced approach available to determine water level and wave parameter probabilities at specific locations along the coast.

2.2. Wave, Shoreline, and Structure Interaction. Foreshores of coastal structures in Louisiana are normally shallow to very shallow. Demirbilek (2007) estimates that in typical coastal Louisiana conditions, the foreshore may range from approximately 1 V (vertical dimension) : 100 H (horizontal dimension) to 1 V : 500 H. Earthen levee designs analyzed by Hughes (2008) have a 1 V : 4.25 H seaward slope, which is considered generally typical in south Louisiana.

Shallow to very shallow foreshore conditions result in wave depth limitations for incident wave breaking and run up onto coastal structures. Most types of waves tend to break within one wavelength in these conditions, which may be considered the lower limit condition of wave impact. In a shallow foreshore condition, wave heights decay with breaking. However, the wave spectrum is maintained with approximately the same properties as that of the incident wave. For wave breaking in very shallow foreshores, there are considerable changes in spectrum, with loss of detectable peak (Van der Meer, 2002). With this transition, there is development of multiple small waves of differing periods. There is no exact definition for transition from shallow to very shallow conditions. However, this transition may be characterized where the incoming incident wave height during breaking decreases by approximately 50% or greater.

The Iribarren number, also termed the breaker parameter or surf parameter \( \xi_o \), is used to describe breaking waves, based on \( H_b, L_o, \) and near shore slope ratio \( \delta \), as follows (Battjes, 1974):

\[
\xi_o = \delta \left( \frac{H_b}{L_o} \right)^{1/2}. \tag{2-10a}
\]

The value of \( \xi_o \) may also be computed as:

\[
\xi_o = \tan \alpha_s \left( 2 \pi \frac{H_s}{g T_p^2} \right)^{1/2}, \tag{2-10b}
\]

where:

\( \alpha_s = \) Slope angle of the coastal structure,

\( H_s = \) Significant deep water wave height, and
$T_p = \text{Peak period of the deep water wave energy density spectrum.}$

Spilling waves occur on mild slopes without breaking when $\xi_o$ is equal to approximately 0.2 (Herbich, 1999). Battjes (1974) states that a turbulence splitting-dominant threshold is present when $\xi_o < 0.4$ to 0.5, which is primarily due to the presence of a surface roller vortex (Peregrine and Svendsen, 1978; Basco, 1985; Diegaard, et al., 1986). Herbich (1999) states that plunging breakers occur in general for short waves and medium slopes, with values of $\xi_o = 1$ to 2.5. Wave breaking on slopes occur with $\xi_o = 2$ to 2.5, which is generally the case for slopes of 1 V: 3 H or milder. Greater values of $\xi_o$ generally occur with wave surging and collapsing on steeper slopes, where wave breaking does not occur. USACE (1995) states that $\xi_o = 5$ for a surging wave; $= 3$ for a collapsing wave; $= 1.5$ for a plunging wave. The transition zone for shallow to very shallow water depth conditions exists when $\xi_o$ exceeds 5 to 7 (Battjes, 1974).

Battjes and Janssen (1978), Thornton and Guza (1983), and Lippmann, et al. (1996), apply linear wave theory to surf zone hydrodynamics for wave energy transformation under turbulence splitting conditions. As turbulence splitting reaches the coastline, its effects impact the water bottom (Zhang, et al., 1998). Turbulence splitting determined using linear wave theory was shown to be applicable in estimating bed shear stresses by Cox, et al. (1996). The velocity field caused by turbulence splitting during wave breaking and decay on the near shore slope is fairly constant (Peregrine and Svendsen, 1978). These velocities exert inertial and drag forces on shoreline sediment particles, which are resisted from movement by gravitational force exerted on the soil particle mass, as well as by soil particle interlock (Julien, 1998). Cohesive forces between soil particles, when present, provide added resistive forces to erosion (Ravens and Gshwend, 1999). An active area of research is the study of resistive forces to erosion afforded by biomechanical interlock and biochemical attraction of the organic matter component that may be present in the soil matrix.

Pilarczyk (1998) explains that wave action impacts have potential to result in soil erosion on the seaward faces and crown of exposed earthen dike (i.e., levee) structures. Figure 2-2 illustrates the
effect of near shore seaward slope wave breaking and run up on a vegetated earthen levee, which caused stripping of the vegetation layer and embankment soil loss (USACE, 2008). The location of this levee erosion is in Eastern New Orleans, as shown in the map of Figure 2-2. In the pictured image of Figure 2-2, the levee structure ascends to the left. The levee toe ascends from the lower right in this depiction. Between the levee structure and toe resides the geotechnical stability berm. This design element is required to provide the levee structure sufficient bearing capacity to maintain its geometrical shape and elevations, considering the challenging soft soil foundation conditions. Figure 2-2 shows:

- the debris line on the levee, which indicates the approximate level of wave run up, and
- the very close proximity of seaward open water to the toe of the levee.

Figure 2-2. Effect of near shore seaward slope wave breaking on a vegetated earthen levee, Lake Pontchartrain and Vicinity HSDRRS, Louisiana (USACE, 2008).
The levee damage shown in Figure 2-2 occurred during Hurricane Gustav in 2008. Given the close proximity of the toe of this levee to open water, it is conceivable that similar modes of levee surface erosion could happen on wider scales across the flood risk management system without the necessary erosion abatement actions. This problem will become exacerbated by ensuing conditions shown in Figures 1-2 and 1-3. Progressive damage to vegetated earthen levees has potential to occur through experiences of successive storm events if sufficient repairs are not completed after the first storm strikes (Pilarczyk, 1998).

Successive storms occurring relatively close together in time have not been uncommon in the recent past across the Northern Central Gulf of Mexico. Those series of events occurring most recently include Hurricanes Gustav and Ike during 2008, Hurricanes Katrina and Rita occurring in 2005, as well as Hurricanes Isidore and Lilly during 2002. With a very short duration between back-to-back hurricanes, repair of erosion on vegetated earthen levees to restore pre-storm integrity is logistically very difficult.

Seaward-facing slope protection is a risk-reduction measure worth considering for minimizing the potential of initial and progressive damage to earthen levees due to wave loadings. Revetments lining seaward slopes are options to retard soil erosion against hydrodynamic wave loadings (McConnell, 1998). Revetment designs are low ground pressure applications. These are desirable for use in cases foundation soils are soft and compressible under loading, such as in the case of levees located in south Louisiana. The revetment structure typically consists of:

- an armor layer to resist wave action and control run up,
- a filter under layer, which relieves piezometric head buildup below the armor layer, and separates the armor layer from the earthen slope soils, and
- anchorage at the head, toe, and optionally, along the slope.
Russo (2003, 2006) described low-crested articulated revetment applications that have been successful in south Louisiana conditions for abating wave-induced shoreline edge erosion, as well as erosion from wave action due to passing vessels.

Wave run up on coastal structures has been traditionally calculated to set the structure slopes and crown elevations. For levees, the objective is to minimize the potential for overtopping under design wave conditions. Seaward-facing levee revetment armor is designed using the design wave run up conditions. USACE (1995) proposed the following equation for estimating maximum wave run up ($R_{\text{max}}$) on rough quarry stone for irregular waves:

$$R_{\text{max}} = H_{\text{mo}} \frac{\xi_0}{(1 + b \xi_0)^{2-11}}$$

where:

$H_{\text{mo}} = \text{energy-based wave height of the zeroth moment, which in deep water is approximately equal to the significant wave height, } H_s$. The significant wave height is defined as the average of the highest one-third of all waves in a wave train. USACE (1995) recommends that $H_{\text{mo}}$ be replaced by $H_b$ when shallow water conditions govern.

$a = \text{regression coefficient} = 1.022$

$b = \text{regression coefficient} = 0.247$

To estimate run up on smooth surfaces with regular waves, USACE (1995) suggests dividing the result of $R_{\text{max}}$ by $\sim 0.60$. A correction factor is also recommended for block revetment slopes equal to 0.93 for multiplication with $R_{\text{max}}$.

Van der Meer and Janssen (1995) conducted an analysis of wave run up for obliquely approaching, short-crested waves with slopes that are smooth and straight, as well as for berms and rough surfaces, which resulted in the following relationship, with a maximum of $3.0 \gamma_h \gamma_f \gamma_{\beta}$

$$R_{u2\%} / H_{st} = 1.5 \gamma_h \gamma_f \gamma_{\beta} \xi_{sq}$$

where:

$R_{u2\%} = \text{Highest 2\% wave run up,}$
\text{H}_{st} = \text{Significant wave height near the structure toe},

\gamma_h = \text{Shallow foreshore reduction factor},

\gamma_f = \text{Slope roughness reduction factor},

\gamma_\beta = \text{Oblique wave attack reduction factor},

\xi_{eq} = \text{Equivalent breaker parameter for a slope with a berm} = \xi_o \gamma_b, \text{ and}

\gamma_b = \text{Berm reduction factor}.

Herbich (1999) further explains design use of Equation 2-12, stating that a tentative empirical formulation of \( \gamma_h \) for a relatively shallow foreshore slope of 1 V : 100 H is as follows:

\[
\gamma_h = \frac{H_{2\%t}}{(1.4 \ H_{st})} = 1 - 0.03(4 - \frac{h_t}{H_{st}})^2 \text{ for } 1 < \frac{h_t}{H_{st}} < 4 \tag{2-13}
\]

\[
\gamma_h = 1 \text{ for } \frac{h_t}{H_{st}} \geq 4 \tag{2-14}
\]

where:

\( H_{2\%t} = \text{Highest 2\% of waves at the structure toe}, \) \text{ and}

\( h_t = \text{Water depth at the structure toe}. \)

The complexity of wave propagation, breaking, run up, and run down on sloping structures with shallow foreshores are further compounded with wave water movement through a semi-porous revetment structure with porous filter bedding laid on an impervious slope. Consideration must be given to flow separation occurring during run up on the structure and flow into the filter layer, with return of wave water down the top surface and through the filter media. These interactions for successive incoming wave cycles become superimposed and increase hydraulic loadings on the structure.

The concept of a “reservoir effect” was presented by Burcharth and Thompson (1983), regarding wave run up flows through armor layers with cores having varying porosities. For waves of increasing length acting on armored slopes with granular cores, there is decreasing structure stability. The reason is that with the rate of wave run up on the structure, there is insufficient time for water that penetrates
the armor layer to percolate into the core, resulting in armor layer uplift to accommodate the incoming incompressible water. For armor layers with coarse bedding materials:

- cycles of short waves have less time to penetrate the core, having a tendency with progressive attack to result in water accumulation near the top of the core material (i.e., inducing internal set up within the core), backing up in armor layer uplift, and
- cycles of long waves have more time to penetrate and dissipate into the core, reducing the amount of water acting in armor uplift when compared relatively to the effect of short waves.

Based upon physical model studies performed by Hedar (1960 and 1986), van der Meer (1988), and Burcharth, Christensen, Jensen, and Frigaard (1998), it was observed that single armor layer masses with relatively low pore volume are increasingly unstable against wave attack with decreasing core material porosity. Evolution of damage under wave loading progresses more rapidly with this trend as well. While armor layer stability is significantly improved with the reverse trend, when failure occurs, it is more sudden.

Van Gent (1994) investigated surface profiles and velocities of long waves on coastal structure run up and transmission within its porous layers. An outcome of these model studies was confirmation of the trend that with decreased core material porosity, there is increased internal set up. Calculation of run up on porous coastal structures as may also be conducted using an advanced approach by Nwogu and Demirbilek (2001), entitled “BOUSS-2D,” which is a hydrodynamic wave model based on fully non-linear form of Boussinesq-type equations (Demirbilek, et al, 2009; Nwogu and Demirbilek, 2007; Nwogu and Demirbilek, 2006; Asmar and Nwogu, 2006; Demirbilek, et al, 2005a and 2005b; Nwogu, 1996 and 1993). Borsboom, Groeneweg, Doorn, and van Gent (2000) developed boundary conditions for a two-dimensional (2-D) Boussinesq-type numerical model for wave propagation and interaction with porous structures. The numerical model was validated using physical model tests with regular solitary waves. The effects of a shallow foreshore for interaction of short waves with a porous sloping structure were explored through numerical model investigations by van Gent and Doorn (2000). Lara
developed and validated a 2-D numerical model using physical model results for simulation of regular/irregular, linear/non-linear waves interacting with porous/impervious, submerged/emerged, single/multi-layered coastal structures. Lara’s 2005 research characterized wave-induced turbulent flow regimes on top of and in the porous layers of structures, which resulted in turbulence dissipation within these under layers. According to Herbich (1999), there is a need for further investigation of the influence of the shallow foreshore on wave breaking and run up on sloping revetment structure designs. An active field of research is development and application of first principles Reynolds Averaged Navier-Stokes equations for this purpose. The present research has potential to inform mechanistic hydrodynamic modeling on wave-revetment structure interaction.

2.3. Structure Response to Wave Loadings. Water particle motion of short period waves impacting armored coastal structures is governed by the deep water wave steepness \( \frac{H_o}{L_o} \), its angle of approach to shore, the water bottom geometry approaching shore, coastal structure shape, and its material composition. The mass, shape, and interlock/articulation of armor units provide stability against buoyant weight lifting and rotation, which may occur as a result of drag and inertia forces associated with short period wind wave impact with the coastal structure. The position of the armor unit above the still water level for wave breaking, as well as the thicknesses and porosities of the armor and under layers, are factors in structure stability. The seaward approaching water bottom slope, structure slope angle, and its crown elevation, have influences on armor unit stability. Another factor on stability is the method of armor unit assembly in composing the structure, i.e., randomly dumped or arranged in an ordered manner. Hudson (1959) and Hudson, et al (1979) performed laboratory investigations of rubble-mound breakwaters to discover these processes and correlate the laboratory data.

Hudson (1959) combines the drag and inertial forces of wave breaking on the structure and equates that expression to the buoyant weight of an individual armor stone to represent incipient stability of the armor unit under wave loading:
\[ k_v l_a^3 (\gamma_r - \gamma_w) = C_q l_a^2 \gamma_w H_b / k_b \quad 2-15 \]

where:

- \( k_v \) = Armor unit volume,
- \( l_a^3 \) = Unit weight of armor unit,
- \( \gamma_r \) = Unit weight of water,
- \( \gamma_w \) = Unit weight of water,
- \( C_q \) = Combined drag and virtual mass coefficient,
- \( l_a^2 \) = Area of armor unit impacted by wave water, and
- \( k_b \) = Wave number of the breaking wave, which is a function of \( H_o / L_o \).

With substitutions and re-arrangement, the following expression is obtained:

\[ k (k_v)^{\frac{2}{3}} / C_q = \gamma_r^{\frac{1}{3}} H_b / (S_r - 1) \ W_r^{\frac{1}{3}} \quad 2-16 \]

where:

- \( S_r \) = Ratio of unit weight of armor unit to unit weight of water, and
- \( W_r \) = Weight of individual armor unit.

Both the left and right sides of Equation 2-16 are dimensionless. Each side is a function of variables of the wave transitioning toward structure impact, the geometry of the approach to shore, structure geometry, as well as the structure composition. It is assumed by this formulation that breaking and non-breaking waves have similar orders of magnitude of structure loading upon impact, and that they are a function of the deep water wave. Given these definitions and assumptions, different parameters can be chosen to test for their correlation with the following:

\[ N_s = \gamma_r^{\frac{1}{3}} H_{oD=0} / (S_r - 1) \ W_r^{\frac{1}{3}} \quad 2-17 \]

where:

- \( N_s \) = Stability number for a given set of tested conditions, e.g., specific slopes of seaward approach and structure, structure armor unit size, shape, and material composition, etc., and
- \( H_{oD=0} \) = “No damage” deep water wave height, which replaces \( H_b \) per the aforementioned rationale, and is defined as 1% and 3.5% permanent distortion of the initial structure configuration, respectively.
by Hudson (1959) and USACE (1995). USACE (2006) expands on these criteria for armor unit displacement, rocking in place, and breakage, for a range of coastal structure types and design conditions.

The experimental values of $N_s$ for given tests are fit via regression analysis to a log-log expression, as follows:

$$N_s = K_D^{\frac{1}{3}} \cot \alpha_s^{\frac{1}{3}}$$  \hspace{1cm} (2-18)

where:

$K_D$ = Stability number, representing interaction/interlock between individually-placed armor elements of a structure against wave attack.

Combining the results from Equations 2-17 and 2-18, the work of Hudson, et al (1979) resulted in development of the Hudson Equation:

$$W_r = \gamma_r \frac{H_{oD=0}^{3}}{K_D} (S_r - 1)^3 \cot \alpha_s$$  \hspace{1cm} (2-19)

The Hudson Equation is applicable to a wide variety of individual armor unit designs, with many rated values of $K_D$ available for design based on testing. When using the Hudson Equation for “no-damage” criteria, depending on the particular structure design, the low-levels of damage may not substantially change the overall condition of the structure relative to the as-built design. Additionally, the residual performance capability to resist wave attack may substantially be retained. Given this fundamental quality of resiliency for individually-placed armoring structures, there is often non-immediate urgency to perform maintenance to the structure between coastal storm events that cause structure deterioration, until such time its condition and performance potential deteriorates to below a level to meet minimum performance requirements (USACE, 2006).

Melby and Kobayashi (1996) and Melby and Hughes (2004), respectively advance the state of understanding by modeling incipient motion and stability of individually-placed breakwater armor units based on principles of hydrodynamics. In the work of Melby and Kobayashi (1996), a critical vertical velocity is related to incipient motion of a single spherical armor unit, which is nested in a
sloping structure with the other armor units fixed in place to isolate study to the motion of a single armor unit. The work of Melby and Hughes (2004) improve on the Hudson Equation with assumption that wave momentum flux at the structure toe is proportional to the maximum wave forces exerted on armor units in the structure. Their equation is fit to results of laboratory testing that was conducted by Van der Meer (1988), which involved the study of incipient motion of individual breakwater armor units.


Revetment armor units are composed of naturally-occurring and/or manufactured materials, which may take a variety of geometrical shapes. Interconnected block revetment is available from a variety of non-endorsed sources. Figure 2-3 depicts examples of U.S. government and commercial revetment applications (USACE, undated; International Erosion Control Systems, Inc., 2002). Commonly available designs fundamentally consist of a system of blocks that are integrated via cables or wires to form a continuously connected unit for covering slopes at the water’s edge. The blocks are ordinarily composed of concrete, arranged in a planar rectangular configuration for slope coverage. Continuous interconnections, often made of stainless steel wire or braided cable, extend in the principal planar
directions of the mat structure. Sections laid along a slope are at times overlapped, and are normally tied together at the edges between pre-fabricated sections that are installed. These measures are intended to achieve continuous stability in the longitudinal direction of the structure against wave action in this general direction.

![Diagram of revetment structure](image)

**Figure 2-3. U.S. government and commercial revetment applications (USACE, undated; International Erosion Control Systems, Inc., 2002).**

For greater stability in both the transverse and longitudinal directions, the installed mat may be anchored at regular spacings along the toe, on the slope, and at the crown of the slope. Anchoring may be used in the design for increased stability against wave action. Anchors or a stone cross section are often placed at the toe, burial into the water bottom, to resist rollup from incoming waves. Anchors used at the crown of the slope pin the top terminal end of the revetment down to prevent sliding of the revetment mat down slope with wave return down slope after run up. Anchors applied on the slope can be used for controlling uplift in lieu of using increasingly larger-mass revetment blocks. This
study was limited to use of anchoring at the slope crown and toe to isolate the models of slope armor block instability for investigation.

Sloped revetment is typically installed over a filter media, which may be composed of aggregate underlain with a polyester non-woven geotextile base material. The needle punched property of geotextile allows water collection beneath the armor layer to drain, preventing build up of hydraulic pressures below the revetment face that can cause instability in uplift. A geotextile layer also provides separation between the armor layer and embankment foundation soils. The mat revetment system protects the filter and subgrade material from direct exposure to damaging, erosive wave action and localized high water velocities. Under these forcings, the subgrade material is retained in the embankment by the filter layer, which is held in place by the weight of the revetment and any supplementary resistive measures, such as anchors or toe stone.

Nonwoven geotextiles resemble felt fabric in appearance. These may be alternatives to use of graded aggregate filters based on comparative economics, as well as for avoiding schedule and constructability issues associated with acquiring, transporting, and placing aggregate filters. Medium weight nonwoven fabrics are commonly used for erosion control (Koerner, 1990).

The design schematic plan view of ACM shown in Figure 2-3 is the system that is investigated in this research as a representative form of an articulated revetment system. The challenge in articulated revetment design is in ensuring sufficiently stable structure conditions in the shallow foreshore wave regime. A fundamental starting point is identifying the modes of structure motion under wave loading. Observations were made from laboratory physical scale model tests (USACE, 1989) that suggest the ACM system experiences mobility in two major modes under wave loadings: (a) uplift along the slope, and (b) roll up at the toe (Figure 2-4). Figure 2-5 (USACE, 1989) is an illustration of post-testing for a specific test design, which was used to quantify damage to structure elements as a result of wave loading using this information. A performance evaluation metric in this case to quantify armor layer
Figure 2-4. ACM system primary failure modes under wave loadings (USACE, 1989).

Figure 2-5. Illustration of post-tested design for quantifying damage to structure elements as a result of wave loading (USACE, 1989).
damage is percent of displaced and broken revetment blocks. Figure 2-6 illustrates filter layer sloughing resulting in permanent armor layer distortion (Pilarczyk, 1998).

Interpreting findings from prior and this new revetment research, Figures 2-7 and 2-8 display elevation cross-section views of ACM with filter bedding laid on a slope under wave run up and run down, respectively. In Figure 2-7, the dominant uplift forcing on the revetment armor is water being driven up slope in the filter layer by the incoming wave. Figure 2-8 illustrates the effect of wave back wash down slope, where saturated flow in the filter layer has potential to induce armor layer uplift.

![Figure 2-6. Scour and slumping of a linked-block revetment granular filter layer under wave attack (Pilarczyk, 1998).](image)

Under short period irregular wave run up and run down loadings, variables believed important in modeling uplift motions of a non-linked or linked block revetment that overlays an aggregate filter layer with impervious sloping base include (Pilarczyk, 1998):

- Armor mass, expressed in terms of armor layer thickness and material density relative to the density of wave water,
- Armor geometry and surface roughness,
- Armor system connectivity,
- Armor layer wave water flow conveyance capacity, defined as porosity,
- Filter layer flow conveyance capacity, described with variables of porosity and layer thickness,
- Structure slope geometry fronting wave attack, which is simplified in this case as a single straight slope with respect to the horizontal,
- Still water level (SWL) with respect to structure slope and its crown elevation,
- Wave height, period, and direction with respect to the structure, and
- Construction quality and post-construction structure maintained condition with respect to design specifications.

![Figure 2-7. Elevation cross section view of wave run up on sloped ACM structure.](image)

First-principles mechanistic hydrodynamic models are challenging to formulate, calibrate, and validate for quantifying ACM structure performance under wave loadings, since the physics are very complex and not completely understood. Using system parameters that have relatively significant contributions to block revetment system performance, Klein Breteler and Bezuijen (1991) developed the following equation for non-dimensionally describing stability of concrete blocks and slabs placed as sloping revetment:

\[
\frac{H_s}{\Delta t_a} = S_b \xi_o^{-\frac{3}{5}}
\]

where:

- \(H_s\) = Breaking wave height
- \(L_b\) = Breaking wave length
- \(n\) = Filter porosity
- \(O_a\) = Nominal opening size of armor
- \(R_{\text{max}}\) = Maximum wave run up
- \(\text{SWL}\) = Still water level
- \(t_a\) = Armor thickness
- \(t_f\) = Filter thickness
- \(\alpha_s\) = Slope angle
- \(\phi_b\) = Maximum piezometric head in filter

Legend:
- Phreatic surface
- N.T.S. (Not to scale)
\[ \Delta = \left( \frac{\rho_s}{\rho_w} \right) - 1, \]  
the relative density of armor material to wave water,

\[ \rho_s = \text{Density of the armor material}, \]

\[ \rho_w = \text{Density of wave water}, \]

\[ t_a = \text{Block thickness}, \]

S_b = Stability coefficient as a function of the relative permeabilities of the armor and under layers.

The value of S_b equals 3.7 at the upper limit of stability for no damage, and equals 8 at the lower limit of instability before structure unraveling occurs.

**Figure 2-8. Elevation cross section view of wave run down on sloped ACM structure.**

Implicitly incorporated into the term “\( \Delta t_a \)” are the combined affects of:

- Block type, i.e., shape, interlock/articulation,
- Formulation of inertial and drag forces,
- Type and magnitude of turbulent regime,
- Location of block on the slope with respect to the SWL, to include block roughness, and
Frictional resistance of the block on the slope against sliding.

A method is required to estimate the term “Δ tₘ” for specific linked and non-linked block revetment designs. Klein Breteler, Pilarczyk, and Stoutjesdijk (1998) determine the term “Δ tₘ” as follows, assuming the flow velocity is known:

\[
Δ tₘ = 0.035 \Phi K_T K_h u_{cr}^2 / 2 g \Psi K_s
\]

where:
- \( \Phi \) = Stability parameter depending on armor design type and shape,
- \( K_T \) = Type and magnitude of turbulence experienced,
- \( K_h \) = Depth parameter, to include armor surface roughness,
- \( u_{cr} \) = Critical vertically-averaged flow velocity on the revetment slope,
- \( g \) = Gravitational acceleration,
- \( \Psi \) = Critical Shield’s parameter for the armor design type and shape, and
- \( K_s \) = Slope parameter for estimation against revetment sliding on the slope, considering slope angle and angle of internal friction of the revetment on the filter material.

The run up velocity (\( u_{up} \)) and run down velocity (\( u_d \)) are respectively estimated as follows (van der Meer and Breteler, 1990):

\[
u_{up} = \left[ 2 R_{max} / H_o (1 - z / R_{max}) \right]^{1/2}
\]

\[
u_d = \left[ 1 / 2 \pi (H_o / L_o) (1 - z / R_{max}) \right]^{1/3}
\]

where:
- \( z \) = Location of block on the slope relative to the SWL.

Figure 2-9 summarizes test results available prior to this research, which mainly lie between 3.7 (i.e., stable upper limit) < \( S_b \) < 8 (i.e., unstable lower limit).

Equation 2-20 is a non-dimensional physics-based systems performance model that is consistent with the formulation of the Hudson Equation (Hudson, 1979). It is applicable for evaluation of laboratory tests, existing structure performance assessment, and preliminary design of new structures,
regarding stability of cases under wave attack. Theoretically, the condition of equilibrium exists for $S_b = 1$, when destabilizing inertial and drag forces from wave action are in balance with stabilizing gravitational forces of the structure. The larger the value of $S_b$ decided upon for use, the higher the allowed wave forcings for a structure of a specific configuration, and thus, the greater risk of structure mobility/failure under wave attack.

![Figure 2-9. Summary of revetment block stability test results prior to current research (Herbich, 1999).](image)

The criteria for “normal” stability in use of Equation 2-20 is described by Herbich (1999) for linked revetment blocks on a granular filter layer, which is:

\[
0.5 \text{ to } 1 > \left( \frac{k'}{k} \right) \left( \frac{t_a}{t_f} \right) > 0.05 \text{ to } 0.1
\]

where:

$k'$ = Permeability of the armor layer (m/s), and

$k$ = Permeability of the sub-base (m/s).

These filter design procedures involve use of granular materials that consist of sands and gravels, and call for minimization of filter layer thickness. The intent is to minimize hydraulic gradients under the armor layer (Thorne, et al, 1995).
For use of the general design configuration shown in Figure 2-10, USACE (1995) recommends use of gravel or stone meeting the following criteria for filter sizing.

- General form:
  \[ \frac{d_{15\text{upper}}}{d_{85\text{under}}} \]  \[ \leq 2-25 \]

- Multi-layer filters and filter-to-slope soil:
  \[ \frac{d_{15\text{filter}}}{d_{85\text{soil}}} < 4 \text{ to } 5 < \frac{d_{15\text{filter}}}{d_{15\text{soil}}} \]  \[ \leq 2-26 \]

where:

\[ d_{\#\text{material}} = \text{sieve percent passing mean diameter of material.} \]

Figure 2-10. General design configuration for block revetments based on USACE (1995).

Sand and gravel filter designs have permeability values assigned to them, which infer capability to handle laminar flow regimes. Given that wave action on revetment slopes can potentially have relatively high velocity regimes, turbulent flow conditions on the slope and in the structure often persist. The combination of a granular filter and high flow conditions result in flow separation, with:

- backwash water that is not able to return down slope on top of the revetment structure, creating a void between the armor layer and filter layer to escape, and

- scour and slumping of the filter layer under the revetment structure.

Figure 2-6 illustrates the potential consequences of this type of design (Pilarczyk, 1998). Consistent with the findings of Lara (2005), the intent of the filter design of an articulated revetment system is to afford turbulent dissipation of wave run up water, with management of the backwash water phreatic surface to levels below the armor layer. Prototype ACM designs implemented in coastal Louisiana
employed use of crushed stone aggregate for filter layer construction (Russo, 2003, 2006) based on USACE (1995). This design intended to minimize the potential for the failure mode illustrated in Figure 2-6.

Prior references on revetment research describe the wave forcings and articulated revetment structure response with respect to “no-damage” criteria as defined for $H_{oD}=0$, but do not extend experimentation, measurements, analyses, and modeling to lower energy levels at the threshold of incipient motion with true zero damage. According to Pilarczyk (1998), block revetment functions optimally if no structure movement is allowed under wave attack. No matter how competent the design, the experience of no structure movement over the project life cycle is unlikely, however. USACE (1986) states that virtually every implemented structure design will experience exceedance of design wave action and water levels at some point during its life cycle, which from a practical perspective, means that the structure will become mobilized. Thus, structural resiliency and ease of repair following design exceedance events are important qualities for revetments. When revetment armor layer movement occurs under design exceedance wave attack, the underlying filter material, when granular, is allowed to move. The result is potentially large deformations of the filter and overlying revetment layer (Pilarczyk, 1998). Unlike structures composed of individual armor units, revetment systems with granular filters suffering deformation failures demonstrate their relatively lower resiliency potential. Structure repairs are more time consuming and expensive in this case, as compared to structures composed of individual armor units, since the revetment must be removed and replaced during the process of repairing the filter layer.

Pilarczyk (1998) states that:

- no theories have been explicitly developed for use of the block revetment stability formula (Equation 2-20) with linked blocks,
- it is assumed that stability equation applies to linked blocks,
- when linked blocks are mobilized, there is large resistance in uplift of adjacent blocks, and
• laboratory tests are rare for linked blocks, relative to the stability equation.

It does not appear from existing literature on the topic whether prior research included:

• the steps necessary for hypothesis testing of the stated relationships, with statistical representation of mean values and confidence intervals to constrain predictive uncertainty, and

• sensitivity analysis/interpretation of design variables.

There is a gap in the body of knowledge on articulated revetment structure performance under wave loading between no movement and “no-damage” criteria. A spectral hydromechanics approach was developed and demonstrated possible via this research to detect and mathematically/physically model effects with interconnected revetment configurations at the threshold of incipient motion under wave loading on shallow slopes. The motivation was to constrain uncertainties in structure performance between a physically-established lower limit of structure motion under wave loading near the theoretical threshold of incipient motion, and the formerly established upper limit of stability for “no-damage,” i.e., \( S_e = 3.7 \).

2.4. Prototype Experiences. Figure 2-11 illustrates a full-scale prototype ACM structure under construction during spring, 2004 along the Louisiana coast, near Hopedale, southeast of New Orleans (Russo, 2003 and 2006). It is common for articulated revetment systems to be delivered to installation sites pre-cast and assembled into multiple-block groupings via interconnecting wires or cables. These install-ready components are normally placed on sloped banks in sections. In Figure 2-11 (a), the Mat Sinking Unit (MSU) is preparing to launch the ACM. Bull dozers on the bank pull the ACM onto the filter bed using lead wires, as shown in Figure 2-11 (b). As this occurs, the MSU backs away from the bank line, laying mat down onto the water bottom. A completed ACM reach is shown in Figure 2-11 (c). Figure 2-12 presents an aerial image that was taken on October 20, 2005 of the ACM prototype structure. This view shows the effects from passage of Hurricane Katrina over the site, which occurred on August, 29, 2005. Note that the small cove of water along the ACM bank line in Figure 2-12 is the same location where ACM was being installed as shown in Figure 2-11.
In general, the post-event inspection of the structure revealed that it survived well under extreme wave and surge conditions. Damage was observed where waves were concentrated in bank line coves, as well as where rollup occurred longitudinal to the structure alignment along the bank where it was not tied down between launches. This post-inspection rendered the determination that the bank line should be graded as straight as practicable, and that longitudinal ties are required for maintaining structure integrity during high wave energy events.
Figure 2-12. ACM prototype structure following passage of Hurricane Katrina over the site in 2005 (view looking in southerly direction toward Hopedale, Louisiana).
CHAPTER 3. EXPERIMENTAL DESIGN.

3.1 Purpose, Scope, Goal, and Objective. The purpose of this research work was to investigate the stability of articulated revetment structures at the threshold of incipient motion under short period irregular wave action. The scope of analysis involved demonstration of a new spectral hydromechanics approach for analyzing systems-scale performance. The goal was to quantify a new lower limit of articulated revetment stability under wave loading. The objective was to begin the process of addressing the anticipated planning, engineering, and design needs in coastal Louisiana, as described in Chapter 2. Further research is required to refine this new method for these uses.

3.2 Null Hypothesis. Physical scale model laboratory testing and data analysis were conducted to inform mathematical model development. The following steps were executed to govern the process. The generalized null hypothesis \( H_0 \) for the research is as follows:

“Articulated revetment armor block mechanical movements at delineated positions along the slope in the physical scale model results are not physically related to short period irregular wave loading conditions”

3.3 Tests for Statistical Significance. Spectral analyses were performed on the time series data to support testing the null hypothesis for statistical significance. The magnitude squared coherency and phase spectrum were determined from Fourier transformation of the time series data into the frequency domain, where a critical value

\[
\sigma_{\text{crit}}^2 = 1 - \alpha^{(2 / (v-2))}
\]

is found statistically to test \( H_0 \) for zero coherence with \( v \) degrees of freedom of the wave-structure response cross-spectra at a specified statistical significance level \( \alpha \) (Figure 3-1). The value of \( v \) equals two times the number of bin segments “S” in the Fourier transformation. The critical value reveals whether the coherency signal at any frequency displays a linear relationship between the wave forcing and structural response (Brockwell and Davis, 1987; Priestly, 1981). The desired outcome is achievement of a statistical significance \( \alpha \) at evaluation points of the system equal to or less than a
selected $p$-value (i.e., 1 chance in $1/p$ or less that the rejection of the null hypothesis is the wrong finding).

Previous research in this field has demonstrated that capability in correlating mathematical model forcings and responses with respect to physical model results is better in some conditions than others (Van Gent and Doorn, 2000). Variations may arise across the modeled system based on how well the hydrodynamics and structural mechanics are mathematically formulated, as well as to what degree laboratory effects introduced during testing undermine physical test values. Ideal achievement of $\alpha = 0.05$ or better at evaluation points across the system suggests that no further testing is required to affirmatively reject the null hypothesis. For results of $\alpha$ greater than 0.05, judgments must be made to explain the reasons for such findings, with recommendations for future research in ways to possibly improve strength in correlation (Holman, 1978).

![Critical cutoff value for squared coherency not different than zero.](image)

**Figure 3.1.** Critical cutoff value for squared coherency not different than zero.

### 3.4 Descriptive Statistics

Characterization of time series data normality, i.e., extent the data possesses Gaussian properties, were performed using D’Agostino’s Omnibus $K^2$ test (D’Agostino, Belanger, and D'Agostino, Jr., 1990). The lengthy formulation of this significance test is not presented in this text. This test quantifies the departure of data set distribution from normality based on combined (i.e., “omnibus”) analysis of kurtosis and skewness. For testing whether the null hypothesis
for normality is true, use is made of the Chi Squared distribution \((X^2)\) with two degrees of freedom. The \(X^2\) probability at the 99.5 percentile cutoff value \((X^2_{99.5})\) for two degrees of freedom is 10.6.

For spectral signal computations made at each frequency in the test calculations of the experimental design framework, there is a 5% confidence interval estimate, 50% (i.e., arithmetic mean), and 95% confidence interval estimate given, assuming a normal distribution, for the best level of \(\alpha\) that can be attained system wide during mathematical model calibration. The 5% and 95% values are computed as:

\[
(x_{5\%}, x_{95\%}) = \mu \pm z \left( \frac{\sigma}{\sqrt{n}} \right)
\]

where:

\(x_{5\%}\) = 5% confidence interval estimate,

\(x_{95\%}\) = 95% confidence interval estimate,

\(\mu\) = Sample arithmetic mean,

\(z\) = “z”-score = 1.96 for 5% and 95% confidence interval estimates,

\(\sigma\) = Population standard deviation, and

\(n\) = Total number of values in sample set.

3.5 Regression Analyses. The information arising from the tests for statistical significance were used in a non-dimensional mathematical systems model for constraining uncertainty in forcing-response performance. Pearson’s product moment coefficient (R) was used for determining correlation trends between forcings and responses in the time and frequency domains. The value of R was computed for paired \(x\) and \(y\) value data sets as:

\[
R = \left( \frac{\sum xy - \left( \frac{1}{n} \sum x \sum y \right)}{\sqrt{n} - 1 \sigma_x \sigma_y} \right)
\]

where:

\(\sigma_{x,y}\) = Sample standard deviation.
The statistical significance of $R$ for a linear relationship may be looked up for $n - 2$ degrees of freedom commensurate to the value of “$t$,” as calculated below, testing the null hypothesis that $R$, the population correlation, equals zero:

$$t = R \left[ \frac{n - 2}{1 - R^2} \right]^{\frac{1}{2}}.$$

The coefficient of determination ($R^2$) was computed to indicate the variability that each paired variable shares with the other.
CHAPTER 4. PHYSICAL PROCESS RESEARCH.

4.1. Physical Process Discovery. Knowledge of prior studies and physical process observations of this research supported the development of assumptions, description of newly required laboratory investigations, and mathematical modeling research. A sloping articulated revetment physical model under wave attack on an impervious slope (USACE, 1989), presented in Figure 2-4a, was qualitatively viewed to begin understanding modes of armor layer instability. The scope of testing included weighting the toe down against movement for waves sufficiently large to induce such movements. This physical model testing indicated that a single-thickness mat layer (i.e., 3-in. thick at prototype scale) begins to experience damages from prototype wave heights approaching 6 ft. In that examination, armor damages were observed on the slope, along with movements of toe stone from their original positions. The study did not include testing with filter bedding underlying the armor layer to understand any potentially related armor stabilizing effects. The stabilizing effects of a filter layer positioned between the armor layer and impervious slope were considered in this new research for management of wave run up and piezometric head buildup.

Figures 4-1 through 4-7 present still frames from the laboratory wave flume experimentation of this new research. These figures illustrate a time series of sloping ACM structure movements under wave attack. In the progression of these figures, the wave builds, breaks, and dissipates in run up on the slope, with the ACM system responding respectively in uplift that propagates up the slope. Typically, the maximum uplift was observed to occur between Stations 3 and 5 in this physical modeling study. This is the location on the slope just below the SWL.

McConnell (1998) states that hydraulic uplift pressures generated in the filter layer in uplift on the armor may be quasi-static or dynamic. Quasi-static uplift may occur from a lag in ground water level subsidence following a storm surge event relative to subsided free surface water levels. Dynamic uplift may be experienced due to ship-or wind-generated wave action that result in run up on the structure slope. Herbich (1999) indicates that these uplift pressures are likely highest at the point of
Figure 4-1. Wave run down as initial condition for next wave run up.

Figure 4-2. Wave building on the slope.

Figure 4-3. Maximum wave building on the slope.
Figure 4-4. Wave breaking with initial ACM uplift.

Figure 4-5. Continued wave breaking up slope with ACM uplift progressing up slope.

Figure 4-6. Wave beginning to dissipate with ACM uplift diminishing up slope.
Figure 4-7. Wave run up and ACM uplift dissipation before run down.

maximum wave run-down. At this location, a piezometric head builds up in the filter in cyclic lagging of the run-down process. Critical uplift pressures on the revetment armor cause instability and structure motions.

4.2. Research and Modeling Assumptions. The following assumptions provide context for remaining chapters on this research.

- A volume of water in slope run up enters the porous revetment armor layer into an underlying porous filter media with impervious base. Based on conservation of mass via the continuity equation, water that enters from the sea side must exit seaward, i.e., no water of consequential volumes for these computations effectively enter the impervious base.

- The waves acting on the revetment structure resulting in run up on the slope have statistically stationary parameters, and the still water level is not changing with time during testing. The short crested irregular deep water wave movement during testing is a linear process with a Gaussian probability distribution. The wave breaking transformation and run up onto the slope are non-linear, nearly Gaussian processes.

- The mass of water in the flume is conserved during testing, in that wave run up is not allowed to overtop the sloping structure undergoing testing and exit the portion of the tank experiencing wave motion.
• Incipient motion is defined as movement of the armor layer under wave attack that does not result in breakage and/or permanent deformation of the armor layer and filter layer. Armor instability in the design wave climate will result in progressive structure failure.

• While some water enters the revetment filter media through the armor layer, significant wave run up discharge runs back down-slope over the top of the armor layer, returning to open water.

• A turbulent flow regime, as defined by the Reynolds Number, applies for porous filter water movement. A phreatic surface is generated in the filter layer below the elevation of the maximum wave run up, descending non-linearly away from the lower boundary impervious base seaward towards the armor layer, resulting in filter water discharge through the armor layer some distance on the slope above the SWL to the SWL (Bear, 1972). Along the direction of flow in the filter, the saturated thickness diminishes with increasing hydraulic gradient to the seaward exit point through the armor layer. The condition exists at maximum instantaneous critical conditions for armor stability, and can be designed for managing piezometric head of both the model and prototype.

• Measures are in place to fix the revetment head and toe at its terminal ends so that armor stability investigations are limited to incipient motion on the slope.

4.3 Physical Process Model for Laboratory Data Processing. A method founded in hydrodynamics and structure mechanics is required to process key data collected in laboratory experimentation near the threshold of incipient motion. The goal is to leverage use of these supporting calculations to enable collection of select time series data streams to inform the solution.

Based on the importance of the deep water wave forcing related to an associated structure run up, as described in Chapter 2, the forcing parameter for this research is defined as the relative instantaneous wave run up \( H_{RIR,j} \) at time “j” during the experiment. This quantity is computed as the time-dependent position of the wave run up elevation on the slope relative to the time-shifted deep water wave elevation that induced the respective run up motion, as follows:

\[
H_{RIR,j} = H_{j-1} \left[ \frac{R_j}{(R_{max} - R_{min})} \right]
\]
where:

\[ H_{j-1} = \text{Time-shifted deep water wave gauge reading preceding the wave run up at time } j, \]
\[ R_j = \text{Wave run up at time } j, \]
\[ R_{\text{max}} = \text{Maximum wave run up observed in the time series, and} \]
\[ R_{\text{min}} = \text{Minimum wave run up observed in the time series.} \]

An \( H_{RIR} \) time series time lag shift was applied for test signal analysis by observation of the time periods for generated deep water waves to traverse the flume and impact the structure at the run up wave gauges. This time lag was approximately 6, 4, and 3 sec at model scale between these two sets of gauges for 0.6, 1.2, and 1.8 sec wave periods tested at model scale, respectively.

Removal of the reflected wave from the deep water wave signal is typically done when using physical modeling results for design application purposes. The reflected wave is not removed from the results of this research for the following reasons.

- Since the thrust of this research is to establish new force-response physical relationships, it is important to conserve momentum in the laboratory flume when processing the data. Therefore, in conserving momentum, wave reflection was not removed from the deep water wave signal. Removing the reflected wave energy from the deep water wave signal in this work would corrupt the spectral analysis and attendant force-response relationships upon which conservation of momentum is dependent.

- Reflected wave energy makes the wave spectrum relatively wider across the frequency domain than when it is removed. Removal of the reflected wave has been traditional practice in design for expressing wave parameters of physical modeling studies that have sufficient energy to cause structure damage. Hydrodynamic modeling used for design, as described in Appendix A, the irregular incident wave with reflected wave are both modeled for determining joint probabilistic water level return frequencies at specific locations in the modeling domain. The current research expresses wave parameters consistent for use in this modeling approach.
Equation 2-21 contains several parameters that require specialized tests to quantify. According to the Principle of Parsimony, model calibration and verification should be performed with as few physically meaningful parameters tied to the underlying processes of interest. As the number of parameters increase, there is less certainty in model simulations (Martin and McCutcheon, 1999). According to the Principle of Parsimony, this new research attempts to reduce the data requirements for use of Equation 2-20, commensurate with similar physics modeled by Equation 2-21.

Figures 4-1 and 4-7 show the wave run up forcings on the ACM structure, with focused interest placed on the leading phreatic surface wave moving in the filter layer. In Figure 4-8 (a), this concept is shown for wave run up impact, ACM element displacement, new ACM position, and changed hydrodynamics for repetitive wave cycles. To mathematically model this physical process in context of the laboratory experiments, use is made of the Energy Equation. Inputs required from the laboratory time series data include the deep water and run up wave time series forcings, as well as the following time series data between adjacent structure slope instrumentation stations: pressure gradients, velocity gradients, water surface elevation differences, head losses, and work performed by deep water waves in displacing ACM armor elements.

Static points for evaluation of required inputs are between gauge stations. Piezometers provide the ability to compute dynamic pressure gradients between "ith" stations (p_i, p_{i+1}) within the filter layer. Depth-averaged velocities of wave run up water entering the armor layer between stations (v_i, v_{i+1}) are computed using the relationship shown in Figure 4-1 (b). The distance “L” in Figure 4-1 (b) is defined as the linear distance in x-y 2-Dimensional (2-D) vertical space between jth time steps. The piezometer readings do not reflect the actual changes in water levels between stations in the filter layer. The assumption is that the rates of incompressible, substantially saturated water parcel exchanges in the filter, as reflected in the fluctuations in the piezometers, are valid proxy source term inputs to compute the velocity signal.
Water surface elevation differences between stations ($\Delta h_{(i,j)}$) were computed at each time step using the following rules, relative to ACM layer uplift potential:

- Activated when the upstream piezometer gauge water level was at or below the plane of the ACM layer, meaning there was only phreatic surface hydrostatic pressure head in the filter. At these instances, the water surface elevation crossed the armor layer between the phreatic surface at the upstream piezometer gauge and the downstream run up gauge. When activated, $\Delta h_{(i,j)}$ was computed as the difference between the water surface elevation in the upstream piezometer and the downstream run up gauge.

- Deactivated when the upstream piezometer gauge water level was above the plane of the ACM layer, meaning there was pressure in the filter layer greater than free surface hydrostatic pressure that can exist up to the vertical thickness of the filter layer. The assumption in this case is that the ACM is fully submerged below the free surface wave run up levels between stations, not contributing to the uplift forcings.

**Figure 4-8. Physical process conceptualization.**

- Deactivated when the upstream piezometer gauge water level was above the plane of the ACM layer, meaning there was pressure in the filter layer greater than free surface hydrostatic pressure that can exist up to the vertical thickness of the filter layer. The assumption in this case is that the ACM is fully submerged below the free surface wave run up levels between stations, not contributing to the uplift forcings.
Existing relationships that are described below are used to compute head losses \((h_L)\) for wave water penetrating the armor layer during run up. A derived relationship is then used to quantify work \((h_a)\) performed by run up waves in displacing ACM armor elements. There are approximately \(n = 2.57\) armor blocks between stations in the physical model, which is the structure mass experiencing vertical movement due to water forcings between gauge stations.

The Energy Equation is used to express the head conditions between any two locations “\(i\)” and “\(i+1\)” for wave run up water entering through the ACM armor layer into the filter layer. This causes piezometric head buildup in the filter layer, with the effect of ACM element uplift at forward time step increments \(\Delta t = t_{j+1} - t_j\):

\[
p_i / \gamma + v_i^2 / 2g + h_i = p_{i+1} / \gamma + v_{i+1}^2 / 2g + h_{i+1} + n \sum h_L + h_a \quad 4-2
\]

\[
h_a = \phi \ n \ W_a \ y_a / Q \ \gamma \ t \quad 4-3
\]

\[
Q = V / t \quad 4-4
\]

where:

\[
\sum h_L = (K + f_{t_a} / D_{eff}) v^2 / 2g \quad 4-5
\]

\(K =\) Loss coefficient of expansion during flow transition (For assumed sudden flow expansion, which can range from \(\sim 0.1\) to \(1.0\), with \(0.5\) chosen to represent entrance of water through revetment opening as trial in demonstration of the method, Robertson and Crowe, 1985)

\[
f = 0.25 / [ \log (k_s / 3.7 D_{eff} + 5.74 / Re^{0.9}) ]^2 \quad 4-6
\]

\(k_s \sim 0.1\) mm, Nikuradse equivalent sand roughness of material at flow boundary layer, assumed as similar to asphalted cast iron (Pilarczyk, 1998), and used for the brick that was sawed with a water jet to fabricate the scale model revetment blocks for this research, as discussed in Chapter 5

\[
Re = Reynolds \ Number = v \ D_{eff} / \nu \quad 4-7
\]

and

\(\gamma =\) Unit weight of water

\(W_a =\) Weight of a single ACM unit
φ n Wₜ represents the time step increment of articulated armor system weight vertically displaced, with “φ” being the number of blocks effectively mobilized, and “n” being the number of blocks between stations.

\( yₐ \) = Armor layer vertical displacement between stations being evaluated

\( Q \) = Flow inducing ACM uplift due to a combination of wave run up discharge through ACM openings, as well as return water exiting the filter layer down slope at the location of ACM uplift

\( \Phi \) = Volume of water during a time step increment present between the uplifted ACM layer and top of the filter layer

\( D_{eff} \) = Effective “pipe” pathway diameter of ACM structure area of entry/exit due to uniform gaps present between adjacent blocks across the system

\( tₐ \) = thickness of the ACM block, representing the length of the “pipe” pathway

Pilarczyk (1998) states that for structure design in the field at prototype scale, the value of \( k_s \) ranges from approximately 1 mm to 10 mm, respectively, for flat surfaces and well grown-through revetments/very rough revetments. These values are computed as 0.04 mm and 0.4 mm, respectively at 1:25 model-to-prototype scale, which is the scale used in this research for physical model testing, as described in Chapter 5. The latter of these two values is comparable in order of magnitude to the value of \( k_s \) used to represent the model revetment units at 1:25 model-to-prototype scale of this research (i.e., 0.1 mm).

The value of \( Re \) was found to be on the order of \( 10^4 \) for the laboratory tests, as described in Chapter 5. Considering this and the value of \( k_s / D \sim 0.01 \), the use of Equation 4-6 is valid for use as being within range of completely turbulent flow.

Figure 4-9 illustrates the physical process and method of flow volume estimation of \( \Phi \) using the time series displacement record of the ACMs at incipient motion between stations being evaluated. The actual volume, shown as a red outlined polygon, is estimated at each time step by the purple shaded polygon, using a Daniell (1946) 15-point moving average over time steps of the ACM.
displacements between respective stations. Use of the Daniell moving average is explained also in Chapter 7 for application during spectral analyses.

Figure 4-10 presents the physical definitions for “pipe” pathway flow through the openings between adjacent ACM blocks. See Figure 2-3 for design details of the ACM system. When the layout of a single ACM block pattern with its opening is fit into a repeating pattern, there is full representation of the ACM system of connected blocks and openings. Accordingly, the following mathematical derivation is used to represent this condition, expressed in terms of an effective diameter of a pipe for use in the physical process equations:

\[
\pi \frac{D_{\text{eff}}^2}{4} = w_a o_a + l_a o_a + o_a^2
\]

\[
D_{\text{eff}} = \frac{4}{\pi (w_a o_a + l_a o_a + o_a^2)^{\frac{1}{2}}}
\]

**Case of Uplift:**

**Figure 4-9. Time step flow volume estimation of \( \Delta V_j \).**

where:

\( w_a = \) ACM block width,

\( l_a = \) ACM block length, and

\( o_a = \) Typical width of openings between adjacent ACM blocks.

N.T.S.
Figure 4-10. Definitions for “pipe” pathway flow between adjacent ACM blocks.

The physically dimensionless parameter, $\varphi$, is back solved in Equation 4-9a and subsequent relations for analysis between stations of the laboratory time series data. A buoyancy rule is applied to the term “$n W_a$” for time steps when the elevation of the up-slope station run up gauge measurement is higher than the undisturbed elevation of the ACM layer between stations. According to this buoyancy rule, the blocks between stations are either designated as fully submerged or not, for the purpose of calculation.

$$
\varphi_{(i,j)} = \left(\frac{\Delta V_{(i,j)}}{n W_a \Delta y_{(i,j)}} \frac{\gamma}{\Delta p_{(i,j)} + \Delta v_{(i,j)}^2 / 2g + \Delta h_{(i,j)} - n \Sigma \Delta h_{L(i,j)}}\right) 4-9a
$$

The $\varphi$ relationship is a force-displacement function, and is termed the “hydromechanic potential”. This term is effectively equivalent to the number of individual ACM blocks mobilized, according to the free body diagram of Figure 4-11. Thus, $t_{a(\text{eff})}$ is computed as a weighted geometric mean, as follows, and is similar in nature to the term “$\Delta t_a$” of Equation 2-21:
\[ t_{a\text{(eff)}} = (\phi_s l_a w_a t_a)^{1/3}. \quad 4-9b \]

In Figure 4-11, the hydrodynamic uplift forces shown are distributed in nature. The \( \phi \) function captures the complex time dependent cycles of:

- wave run up, back wash, and piezometric hydraulic forces working at irregular frequencies and phases resulting in intermittent uplift of a progressive series of adjacent blocks on the slope, and
- cavity expansion and contraction between the intermittent vertical movement time progression of armor units over the filter bed in which the incoming water flows, causing structure instability.

The hydromechanic approach is a “quasi-on/off” function, in that for short bursts in time, there are sequences of vertical structure uplift motion. These bursts are a progression of uplift and relaxation of the structure along the slope ascent for very short periods of time. In between short time bursts of

---

**Figure 4-11. Free body diagram of initial movement for incipient motion of the armor layer.**

- wave run up, back wash, and piezometric hydraulic forces working at irregular frequencies and phases resulting in intermittent uplift of a progressive series of adjacent blocks on the slope, and
- cavity expansion and contraction between the intermittent vertical movement time progression of armor units over the filter bed in which the incoming water flows, causing structure instability.

The hydromechanic approach is a “quasi-on/off” function, in that for short bursts in time, there are sequences of vertical structure uplift motion. These bursts are a progression of uplift and relaxation of the structure along the slope ascent for very short periods of time. In between short time bursts of
motion, the hydromechanic potential signal is zero, i.e., representing a motionless structure laying on the slope.

The value of $\varphi$ may be positive and negative during wave cycles. When positive, $\varphi$ represents hydraulic uplift pressure action. When $\varphi$ is negative, it implies that a downward hydraulic pressure is acting on the ACM system.

Only the positive $\varphi$ values are of interest in evaluating structure system stability under wave loadings. The value of $\varphi$ may be considered analogous to the value of $K_D$ of the Hudson Equation (1979). It should be noted, however, that the value of $K_D$ is for stability contributions due to interaction/interlock between individual armor elements of a structure, not system-wide structure performance.

According to the Principle of Parsimony, there are two empirical input variables to manage in the use of Equation 4-9b for specific design conditions, whereas there are six empirical input variables in Equation 2-21. Klein Breteler, Pilarczyk, and Stoutjesdijk (1998) state that a disadvantage in use of Equation 2-21 is that it produces a large scatter of data points during plotting relationships of test results due to the large number of experimental input parameters.

Figures 4-12a and 4-12b present an exploration of variable sensitivity in computation of $\varphi$ with variation of $K$ and $k_s$, which is performed for STA 3-4, Test A3F12T9H2. According to Robertson and Crowe (1985), the value of $K$ ranges from 0.1 to 1.0 for flow expansion during transition. Thus, sensitivity was explored for this range. The range of values of $k_s$ is explored in accordance with values suggested by Pilarczyk (1998) for block revetments, which are adjusted in this research to cover the range of values from $\sim 0.1$ to 1.0 mm at 1:25 model-to-prototype scale.

It can be seen from Figures 4-12a and 4-12b that computation of $\varphi$ is relatively sensitive to changes in the variable $K$, and relatively insensitive to changes in the value of $k_s$. The value for $K$ used in this research could not be much different than the value of 0.5 used for structure movements tested at incipient motion, since:
- The data point for this test (STA 3-4, Test A3F12T9H2) ad a reasonably good statistical confidence level in hypothesis testing, as described in Chapter 7, and
- The data point falls relatively close to theoretical incipient motion that is explained in Chapter 8. Choosing a much higher value of K would push this point below the theoretical threshold of incipient motion, which is a non-existent condition. Use of a lower value of K would distance the point further from the theoretical threshold of incipient motion.

Figure 4-12a. Exploration of variable sensitivity of K with constant ks, STA 3-4, Test A3F12T9H2.

It is possible that the value of K could vary as a function of revetment design characteristics, despite that value being held constant for all tests of this research. With iterations involving modification of K for varying block thicknesses in the research explained in Chapters 7 and 8, improvement in agreement of test data points along the theoretical threshold of incipient motion might be possible.

4.4. Conditions Beyond Research Scope. Research was not conducted for conditions where the structure becomes completely submerged. While recognized as important to structure stability in some cases, investigation of long waves and wave groups were not considered beyond the scope of this study.
Figure 4-5b. Exploration of variable sensitivity of $k_s$ with constant $K$, STA 3-4, Test A3F12T9H2.
CHAPTER 5. LABORATORY PHYSICAL MODELING.

5.1. Wave Flume Requirements. A wave flume scale physical model was used to understand and quantify structural performance of sloping articulated revetments against wave attack. Figure 5-1a depicts the two-dimensional glass-walled laboratory wave flume used for testing. The flume width, depth, and length are 0.91 m (3 ft), 0.91 m (3 ft), and 45 m (148 ft), respectively. The flume is outfitted with a computerized electro-hydraulic wave generator, which is able to produce irregular short period waves with a maximum wave height of 0.23 m (0.75 ft), and wave periods of 0.50-10.0 secs (Melby, 2003).

Figure 5-1a. Laboratory wave flume.

Figure 5-1b (1) shows an elevation view schematic of the laboratory wave flume design. Figure 5-1b (2) presents an elevation view of the prototype design that was modeled in the flume.

Figure 5-2a presents an elevation cross section view of the base structure that was constructed at 1:25 model-to-prototype scale in the flume, after Hughes (2008).
These slopes and grades are typical geometry of an earthen levee structure placed in a typical setting along the shore of a shallow open water foreshore in coastal Louisiana. The base structure slopes and grades were held constant for all tests performed. The base structure slopes were composed of an impervious high density foam board that was secured in place on top of a graded sand and gravel bed. During preliminary test trials, it was discovered that some run up waves overtopped the structure crown, which was set at El. 20 ft. To ensure that all run up water ran back down the slope with no overtopping, the physical model slope was extended upward at the same slope angle to a new crown elevation of approximately 26 ft. Figure 5-2b shows the ascending slope from deep to shallow water,
terminating at the levee structure in the flume. Figure 5-2c contains a picture of the base configuration constructed in the flume.

![Diagram of wave flume cross section](image)

* Extended to avoid wave overtopping that was discovered during preliminary trials.

(a) Structure cross section design, prototype scale dimensions (modified after Hughes, 2008).

(b) Constructed cross section in flume looking in direction from wave generator to levee model, shown with tank empty.

(c) Constructed cross section in flume, shown with tank filled to a still water line the on slope.

**Figure 5-2. Elevation cross section of structure slope in wave flume.**

5.2. Physical Model Study Scope. Data collection using a wave flume physical model was required in support of research for quantifying structure performance under wave run up loadings for a range of revetment structure design configurations. Testing was required at wave heights and periods corresponding to observed thresholds of articulated revetment structure incipient motion. An instrumentation layout of wave gauges and piezometers were required to operate in concert with the structure design in the wave flume, which is described in Chapter 6. Tests were designed to have a constant SWL as the zero datum elevation at approximately half-depth between the range of instrumented elevations on the structure slope, for combinations of the prototype structure configuration, as follows:

- 3, 6, and 9-in.-thick ACM block thicknesses
- 6, 9, and 12-in.-thick stone filter media bedding
The following data format is followed to reference tests conducted: AwFxTyHz, where A, F, T, and H respectively identify the prototype ACM thickness, filter layer thickness, wave period, and wave height. The variables w, x, y, and z respectively represent the specific test values of these parameters. Table 5-1 presents a summary of tests configurations and wave loadings conducted within the scope of the physical model phase of this research. The variety of tests conducted was intended to span a sampling range of possible wave forcings and structure configurations.

### Table 5-1. Summary of physical model test configurations and wave loadings, prototype scale.

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<th>Test Identifier</th>
<th>ACM Thickness (in)</th>
<th>Filter Thickness (ft)</th>
<th>Wave Period (T) (sec)</th>
<th>Wave Height (H) (ft)</th>
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5.3. Physical Model Material Properties Scaling. Proportioning of the laboratory physical model components was conducted based on guidelines described in Hughes (1993) to determine the requirements for design and fabrication of the physical model inside of the laboratory wave flume. The mass density of water ($\rho_m$) of the model ($m$), was $\rho_{wm} = 1.94 \text{ lb-sec}^2/\text{ft}^4$, which is fresh water. The model material elements were scaled proportionally, considering a coastal prototype ($p$) would have a salt water mass density of $\rho_{wp} = 1.99 \text{ lb-sec}^2/\text{ft}^4$. Of note, near shore waters in Louisiana with erosion problems that this research addresses often reside within estuaries, which may have water salinities ranging from fresh, intermediate, brackish, to salt. For the purpose of this exercise, salt water is conservatively assumed for research study design. The armor mass density ($\rho_a$) of the prototype ($p_{ap}$)
is 4.60 lb-sec²/ft⁴, equivalent to a prototype armor specific weight \( (\gamma_{ap}) \) of 148 lb/cf, being representative of classes of concrete typically used in industry manufacturing. With the adjustment from salt to fresh water for laboratory experimentation purposes, the mass density of the armor in the model \( (\rho_{am}) \) is 4.49 lb-sec²/ft⁴. This is equivalent to a model armor specific weight \( (\gamma_{am}) \) of 144.6 lb/cf. Gravitational acceleration \( (g) \) is assumed to be the same between the model \( (g_m) \) and prototype \( (g_p) \).

5.4. **Physical Model Similitude.** The physical model structure configuration must obey Froude similitude for representation of physical processes at prototype full scale in the field. An undistorted geometric scale factor of 1:25 (model-to-prototype) was used to fabricate the laboratory physical model in a 3-ft-wide flume. The prototype ACM block length, width, and thickness are 3.85 ft, 1.48 ft, and 3.0 in., respectively. With undistorted geometric scaling, the model ACM block length, width, and thickness are 1.85 in., 0.71 in., and 0.12 in., respectively. Mat armor is typically fabricated for field installation as a “launch”, which consists of 16 armor blocks cast together with stainless steel wire embedded throughout (Figure 2-3) to form a continuous length of 25 ft. In the scale model, a single launch measured 1 ft long. The gaps between these mat blocks are approximately 1-2 in. in the prototype, so by geometric scaling, the scale model has gaps of approximately 1-2 mm.

Tests were conducted by Hughes (2008) in a 3-ft-wide flume using this design for testing articulated revetment performance on the protected side of a levee in overtopping. Hughes (2008) did not use a filter layer between the ACM model revetment layer and the impermeable slope upon which it was laid for testing under wave attack. Tests in the current research were made using new mats very similar in design to those of Hughes (2008), as well as the model cross sectional configuration in the 3-ft-wide flume.

For practical purposes, fire brick material was used to fabricate the scale model armor blocks. The prototype concrete and model fire brick have material densities of 148 and 136 lb/cf, respectively. Considering salt-fresh water density adjustments between the prototype and model, respectively, the model would have a material density of 144.6 lb/cf. This being the case, the 3-in. thick prototype
block thickness for a single armor layer was adjusted by the ratio $\frac{144.6 \text{ lb/cf}}{136 \text{ lb/cf}} = 1.06$, resulting in a block thickness of 0.127 in., or approximately $\frac{1}{8}$ in. The 3-in. thick prototype ACM block weighs about 209 lbs (in air), and each model ACM block, 3-in. wide, will weigh approximately 0.013 lb, or 0.21 oz (in air). These values are proportionally larger for model blocks twice and three times thicker, $\frac{1}{4}$-in., and $\frac{3}{8}$-in., respectively, than the 3-in. thick prototype. The designation for the $\frac{1}{8}$-in., $\frac{1}{4}$-in., and $\frac{3}{8}$-in. armor units is A3, A6, and A9, respectively.

The A3, A6, and A9 class model revetment mats were cut from the fire brick material using a water jet, then assembled into continuous model mat layers, 57 rows long and 19 columns wide, as shown in Figure 5-3. Each revetment layer fabricated was composed of 1083 blocks each.

![Figure 5-3](image.png)

**Figure 5-3. Model components being assembled to form an articulated mat structure.**

A fabric mesh material was secured to the back sides of these revetment mats using waterproof glue to hold them together, representing the stainless steel embedded wire of the prototype. A stainless steel template, cut to size using a water jet, was used to lay out the loose ACM blocks for gluing fabric mesh to the under-side. A marine-grade glue was used that holds under water. Sufficient spacing between blocks was provided for the $\frac{1}{4}$-in., and $\frac{3}{8}$-in. mat thicknesses so that there was an unconstrained range of rotational motion under wave attack. This approach discounts any resistance to
rotational motion under wave loading that the stainless steel wire connecting the prototype blocks in the system might provide.

Balancing inertial and gravity forces during wave motion impact of the structure, the Froude Number scale ratio ($N_{Fr}$) must remain equal to one:

$$N_{Fr} = N_v / (N_g N_L)^{1/2} = 1$$  \hspace{1cm} 5-1

where:

$$N_v = v_p / v_m$$  \hspace{1cm} 5-2

$$N_g = g_p / g_m = 1 \text{ (assumed)}$$  \hspace{1cm} 5-3

$$N_L = L_p / L_m = 25$$  \hspace{1cm} 5-4

$v_{m,p} = \text{velocity of the model and prototype, respectively}$

$g_{m,p} = \text{gravity of the model and prototype, respectively}$

$L_{m,p} = \text{length scale of the model and prototype, respectively}$

Therefore, $N_v = 5$, meaning the prototype should have velocities 5 times greater than measured in the model. Since $N_T = N_L / N_v$, $N_T = 5$, in estimating the proportion of the model-to-prototype time scale. The length and time scales are important when converting model-to-prototype wave heights and periods, which are commonly-used parameters.

A filter layer was incorporated into the current research for its evaluation as part of structure design performance. At prototype scale, the flow field is turbulent in the filter layer (Hughes, 1993). The model filter layer must be capable of maintaining a turbulent flow field under wave loading as it would in prototype conditions. This can be a challenge, since scaling down of filter aggregate will reduce the media permeability and induce a laminar flow field (Darcy, 1856). Trials were conducted to ensure the selected gradation of the aggregate used for the model filter would meet this criterion. An approach was developed to conduct this testing, as follows. The Reynolds Number (Re) is defined as follows:

$$Re = v L_f / v_f$$  \hspace{1cm} 5-5
where:

\( v \) = velocity of flow,

\( L_f \) = characteristic length across flow path, and

\( \nu_f \) = kinematic viscosity of flow fluid.

The hydraulic radius (\( R_h \)) for open channel flow is:

\[
R_h = \frac{A_f}{P_f} \tag{5-6}
\]

where:

\( A_f \) = Cross sectional area of flow, and

\( P_f \) = Wetted perimeter of flow.

The perfectly symmetrical characteristic length of a cylindrical pipe (CP) is its diameter. Used to develop a relationship with the hydraulic radius, \( R_{h\text{CP}} = \frac{D}{4} \), thus, \( D = 4 \, R_{h\text{CP}} \), and:

\[
Re = 4 \, R_{h\text{CP}} \frac{v}{\nu_f}. \tag{5-7}
\]

For a rectangular channel:

\[
A_f = D_f \, W_f \tag{5-8}
\]

\[
P_f = 2 \, D_f + W_f \tag{5-9}
\]

\( D_f \) = flow depth, and

\( W_f \) = flow width.

For a wide rectangular channel, \( R_h = \frac{D_f}{1 + 2D_f/W_f} \sim D_f \), resulting in:

\[
Re = 4 \, D_f \frac{v}{\nu_f}. \tag{5-10}
\]

For the case of flow along the physical model slope in the porous filter media, assuming open channel flow conditions of a wide channel, the term “\( t \, n = D_f \)” was used as a surrogate for depth, where \( n \) is filter media porosity. Then:

\[
Re = \left( 4 \, v \, t_f \, n \right) / \nu_f. \tag{5-11}
\]
It is assumed the filter layer acts as a rough channel, after Chow (1959), with criteria that must be met as follows for flow turbulence: (1) sub-critical flow regime, and (2) negligible surface tension influence. Sub-critical-turbulent flow is defined by Chow (1959) as:

\[
Fr < 1 \text{ and } Re > 2000, \text{ where:}
\]

\[
Fr = \frac{v}{(g t_f n)^{1/2}} = \text{Froude Number.}
\]

Laboratory tests were conducted to estimate the porosity value for model aggregate to be tested for filter flow turbulence with the articulated revetment layer in the wave flume. Figure 5-4 illustrates the procedure for estimating aggregate porosity for ¼-to-⅜-in filter media. The calculation of porosity is the volume of voids water (Figure 5-4b) divided by the total volume of water and solids (Figure 5-4a), i.e., \( \frac{220}{500} \text{ ml} \approx 0.4 \), read to the nearest 100 ml in the measuring device. Since the value of porosity estimated with this degree of precision is used in the numerator of the Reynolds Number computation to the first power, differences in porosity read with this precision affect the results in determining whether flow is laminar or turbulent with \( \pm 10\% \). Visually reading to the nearest 100 ml was planned to ensure the best accuracy in results, i.e., no attempt was made to estimate between 100 ml markings on the measurement device to compute the value of porosity.

Figure 5-4. Procedure for estimating aggregate porosity.

(a) Aggregate submerged in water to the 500 ml level of the measuring cup.

(b) Water poured off from the voids of the aggregate into a measuring cup, which measures 220 ml.
Using the aforementioned Reynolds Equations developed to represent filter flow, a procedure was required to make a determination on filter flow turbulence in the physical model. Based on qualitative wave flume test observations, it was found that in general, the filter flow with an incoming run up wave moves faster than filter flow in wave run down. Therefore, it is most conservative to measure the flow turbulence in wave run down. To validate use of \( t_r/n \), it is assumed that waves in run down on revetment slope move faster than the return flow running down within the filter, effectively leaving a discernable flow stream in the filter for estimating Re at a point where water exits through the armor layer.

The procedure used for estimating Re in the ACM physical model filter was followed, as described below:

- Scale off the slope in tenths of an inch along the flume glass along the slope.
- Video a large solitary wave in run up on the slope at a given water level while achieving incipient motion of the ACM. While videoing, inject a small slug of dye in near the highest point of wave run up at that instant in time into the filter on the slope right at the flume glass wall.
- Review the video to determine the time taken for the dye to migrate down slope within the filter layer over a given scaled-off distance to estimate the respectively observed velocity.
- Compute Fr and Re from Equations 5-12 and 5-13 to determine whether the flow meets subcritical-turbulent criteria for the computed velocity, as shown in columns of Table 5-2.
- Use a larger aggregate gradation and repeat test if subcritical-turbulent criteria are not met.

At model scale, the ¼-in.-diameter gradation aggregate was proportional to the F6 filter layer thickness. The ¼-in.-diameter gradation aggregate was found to exhibit laminar flow properties at the upper end of the Reynolds criterion. The physical model tests were run with this stone gradation recognizing the laminar flow shortcoming, since there was no other physical way to test the F6 gradation at the required model filter layer thickness. In other words, a larger average diameter aggregate would increase the modeled thickness of the filter layer, not accomplishing simulation of a
F6 model scale filter. The \(\frac{3}{8}\)-in.-diameter gradation stone was found to have turbulent flow properties beyond the accuracy and precision limitations on the estimation of porosity as it relates to the criteria of Equations 5-12 and 5-13. This facilitated testing the F9 and F12 filter thickness at model scale with no concerns of not meeting the Reynolds criterion.

Table 5-2. Direct measurement of Reynolds Number for flow in filter layer.

<table>
<thead>
<tr>
<th>(v) (ft/sec)</th>
<th>(\frac{1}{v}) (sec/ft)</th>
<th>Re</th>
<th>Fr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Prototype</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>0.5</td>
<td>0.250</td>
<td>0.250</td>
<td>2.857</td>
</tr>
<tr>
<td></td>
<td>0.350</td>
<td>0.350</td>
<td>0.450</td>
</tr>
<tr>
<td>1.0</td>
<td>0.250</td>
<td>0.250</td>
<td>0.350</td>
</tr>
</tbody>
</table>

| Model          |                         |    |    |
| 0.5            | 0.250                    | 0.250 | 2.857 | 2.222 | 1.818 | 1.538 |
|                | 0.350                    | 0.350 | 0.450 | 0.550 | 0.650 | 0.750 | 0.850 |
| 1.0            | 0.250                    | 0.250 | 0.350 | 0.450 | 0.550 | 0.650 | 0.750 | 0.850 |

Three different size thickness filter layers, classified as F3, F6, and F9, were used in combination with the three armor thickness sizes in flume testing. These respectively correspond to 6, 9, and 12-in. thick prototype equivalents at 1:25 model-to-prototype scale, or 6.1, 9.1, and 12.2 mm, in that order. The F3 class filter had an aggregate mean gradation diameter \((D_{50}) = \frac{1}{4}\) in. The F6 and F9 class filters had an aggregate \(D_{50} = \frac{3}{8}\) in.

5.5. Physical Model Setup. The first step in preparing individual filter and armor configurations was placing and uniformly grading the aggregate on the seaward slope. Parallel steel rods were laid on the slope amongst the loose aggregate to serve as guides for leveling to uniform thickness. The steel rods were removed and the depressions they left were filled with final touch-up grading performed in preparation for ACM placement (Figure 5-5).
ACM model revetments were placed on top of filter layer combinations, one-by-one, for each battery of tests wave loading tests. The model ACM revetments had to be handled with care to not tear the fabric mesh. Anchorage of the ACM system at the top and bottom ends were made approximately every 0.5 ft across the 3-ft-wide flume, from end-to-end. The model testing assumed that the anchorage made in the field is sufficiently strong so that pull-out during wave loadings would occur long after violent instabilities of the armor layer. Therefore, in the flume, anchorage will be made so that no pull-out conditions will occur during wave loading. These measures were taken to simulate standard practice in the field of anchoring the revetment at top-of-bank and the toe. Figure 5-6 illustrates a typical revetment design configuration of physical model elements assembled in the flume for a typical setup.
Figure 5-6. Typical revetment test design configuration assembled in the flume.
CHAPTER 6. LABORATORY DATA COLLECTION AND ANALYSIS.

6.1. Wave Gauge Requirements. Capacitance gauges were used to collect time series wave water surface elevation changes at 50 samples per second, i.e., Hertz (Hz), during testing. For the deep water and breaking wave gauges, capacitance wire instruments were mounted on Jordan controllers. These gauges were custom fabricated to meet wave flume measurement and data collection needs. The capacitance gauges function by sensing the change in capacitance in a thin insulated vertical wire as the water elevation varies on the wire. Each gauge captures a time series of information that can be converted into water surface elevations at that location. The time series can then be analyzed to obtain wave information. Jordan controllers are remotely-controlled motorized devices used to precisely raise and lower the gauges in the flume setup and calibration processes in preparation of flume operations. Instrument setups were located at fixed points centered along the longitudinal flume axis. Figure 6-1 illustrates an example of the gauge equipment and setup in the flume (Hughes, 2008).

Figure 6-1. Wave gauge instrumentation design.

(a) Resistance rod.  
(b) Typical gauge mounting on Jordan controller.
Deep water wave propagation is a linear harmonic process. This being the case, only one gauge setup was required to measure the deep water wave. Breaking wave action is a non-linear wave transformation process (Holthuijsen, 2007). To measure the spatial and temporal trends of non-linearity, multiple gauges were used at equidistant spacings. Three gauge setups formed an array to measure the breaking wave. Figures 6-2a and 6-2b display these instrumentation layouts in the model for the deep water and breaking waves, respectively.

6.2. Run Up Gauge Requirements. Resistance rods were used to collect time series run up wave water surface elevation changes at 50 Hz during testing. The resistance rods operate in a similar manner as capacitance gauges, detecting resistance changes with water elevation changes over time. The resistance rods were custom fabricated to meet wave flume measurement and data collection needs, according to the description provided in Figure 6-1. Each instrument setup was located at static points centered along the longitudinal axis of the flume.

Run up wave action is a non-linear wave transformation process (Holthuijsen, 2007). Following techniques used by Davis and Nielsen (1988) and Nielsen and Dunn (1998), multiple gauges were used at equidistant spacings to capture data for characterizing these processes on the structure slope.
Three gauge setups formed an array to measure the run up wave. These gauges were mounted on the structure slope at Stations 1, 4, and 7, commensurate to the layout described in Section 6.3 for piezometer instrumentation. An open space was left in the plane of the revetment armor layer at the run-up gauge station spacings, such that the terminal end of capacitance gauge rods for each station fit vertically flush with the top surface of the filter layer. This allowed the resistance rod to be exposed to very small water level changes on the slope across the surface of the revetment layer. The run up gauges at Stations 1 and 4 were partially submerged below the SWL. The gauge at Station 7 was mounted in the dry above the SWL. Linear interpolation was made to estimate run up signals at slope stations between gauged run up stations. Figure 6-3 displays the run up wave instrumentation array in the model.

Figure 6-3. Run up wave instrumentation array in the model.

6.3. Piezometer Requirements. Resistance rods were used to collect time series piezometer water surface elevation changes in the filter layer at 50 Hz during testing. The resistance rods were custom fabricated to meet wave flume measurement and data collection needs, according to the description provided in Figure 6-1. Piezometric flows under the saturated phreatic surface in the filter media is a non-linear process. The array of gauges established to capture data for characterizing these processes on the structure slope was formulated after techniques used by Davis and Nielsen (1988) and
Nielsen and Dunn (1998). Their application was for performing field measurements in coastal shoreline hydrodynamics investigations.

Each instrument setup was required to be compact in size, as well as be capable of measuring very small changes in piezometric head changes in time. Commercial instruments meeting these criteria are very expensive, thus were prohibitive to acquire and use. Consequently, the instrument setup for collecting piezometric data in the filter layer posed a data collection challenge. An alternative approach was taken to develop a custom design for instrument fabrication. Figures 6-4a, 6-4b, and 6-4c display the innovative, original piezometer instrumentation array design used to guide fabrication and construction.

Figure 6-4a. Piezometer slope port structure design in the model.

Ten gauge setups formed an array to measure the piezometric gradients between stations, as shown in Figure 6-4a. These gauges were mounted on the structure slope at Stations 1 through 10. The gauges at Stations 1 through 6 were partially submerged below the SWL. The gauges at Stations 7 through 10 were mounted in the dry above the SWL.

The piezometric head measuring ports were located at ten stations along the slope, which were accessed using flexible clear plastic diameter tubing. Pairs of tubes were used for each station to
Figure 6-4b. Piezometer slope ports and gauge instrumentation array in the model.

- Glass pipettes:
  - 5/16 in. OD x 1/4 in. ID (top)
  - 1/4 in. OD x 1/8 in. ID (bottom)

- Capacitance rod:
  - 5 and 9/16-in.-long rods (approx.), typical
  - Top rod: 0.148 in.-dia.
  - Bottom rod: 0.0625 in.-dia.

- Clear vinyl tubing:
  - 3/8 in. OD x 1/4 in. ID (top)
  - 1/4 in. OD x 1/8 in. ID (bottom)

- Vinyl tubing fits snugly on glass pipettes for respectively proportional sizes

Piezometer gauge instrumentation.

Figure 6-4c. Piezometer slope ports and gauge instrumentation array in the model.
provide separate wells at the piezometer instrumentation for each rod to complete the electrical circuit loop. The clear plastic tubes ran from the slope ports along the underside of a 1-¼-in.-thick foam board, which acted at the impervious slope surface in the flume. The open ends of the tubes were flush-mounted into the foam board top side. The tubes were bundled into channels flush along the underside of the foam board.

![Constructed piezometer instrumentation.](image)

**Figure 6-4d. Piezometer slope ports and gauge instrumentation array in the model.**

The slope port board was fixed onto the front levee slope in the flume and secured in the tank using caulk sealant. The bundle of tubes exited the underside channel at the topside crown of the slope, and was draped over the side of the flume. The clear plastic tube pairs running out of the flume were connected outside of the flume to an array via vertically-mounted clear glass tube pairs. The capacitance rods for each station were respectively placed into these glass tube pairs and fixed in position on a vertically-mounted board along the outside of the flume near the model, forming a piezometer array. The lines were bled to bring water from the slope ports to the glass tubes,
eliminating air bubbles in the process. Blue dye was injected into the glass tubes to enhance visual readability and verification of piezometer gauge measurements.

6.4. **Timer Requirements.** A laptop computer with XNote™ Timer software installed was mounted in front of the model structure in the wave flume. The timer program was able to read hours, minutes, seconds, and hundredths of a second. A timer activation device was run from the flume wave generator to the timer. At the instant the wave generator was activated during initialization of each test, all water level gauges began recording, and simultaneously, the timer was activated for the test. Figure 6-5 illustrates the timer device used in the laboratory experimentation.

![Timer](image)

**Figure 6-5. Timer.**

6.5. **Video Camera Requirements.** Two high definition video cameras that operated at 100 Hz were used to record the wave-structure interaction in the flume, as well as to record the visual piezometric head changes in the measurement tubes. The camera model used was a Canon Vixia HG10 HD AVCHD HDD.

The model structure was videoed from stationary, orthogonal locations with respect to the flume model and instrumentation setup during the battery of testing to record run up and occurrences when incipient motion are visually detected as the water level and wave parameters increased from the smallest to the largest executed. Video production for each test began with identification of respective
test configuration and wave parameters to be run using a placard. Once the test was identified, the test equipment was initialized. Video production was stopped after water level and wave parameter testing were terminated for each increasing sequence at those forcing conditions causing incipient motion of the armor layer.

6.6 Equipment Integration, Control, and Data Collection Requirements. Figure 6-6 shows the instrumented model in the wave flume ready for testing. The gages at each location were linked to an instrument data collection center for synchronizing and recording the time series data. The facility used an automated wave flume/instrument control and data acquisition system, which integrated all 17 channels of gauge data collected during experimentation (Figure 6-7). A Buffalo 2 TB DriveStation Quattro TurboUSB external hard drive was used to store all gauge and video data from the testing.

6.7. Data Collection Procedures. Data was recorded during laboratory testing for analysis of the relationship of wave loadings and incipient structure movements. The physical model and instrumentation setup was initially used as follows:

- Qualitative process modeling observations were made in the beginning for the purpose of visually validating the initial assumptions used to justify and support problem formulation and subsequent spectral hydromechanics analyses for quantitatively describing wave-structure performance,
- Confirmation was made to ensure that physical modeling techniques were being applied properly to achieve similitude for the armor layer and filter layers, and to control/minimize laboratory effects. Adjustments to the approach were made as necessary based on these results in preparation of quantitative modeling and data collection, and
- Testing of the instrumentation setup was made for adjustment to ensure data streams intended for collection was going to be achieved, and for calibration across the ranges of measurements required.

Deep water and breaking wave gauges were calibrated daily with the water in the tank motionless at approximately half-depth on the structure slope. The gauges were raised initially at 10 equal
increments, then lower by 20 equal increments, and finally raised by 10 equal increments, to bring the
gauges back to their original vertical positions. Data was collected at each stopping point and
analyzed to establish the relationship (usually linear) between water elevation at the gauge and
frequency output by the gauge. Calibration was conducted for the range of expected water level
changes at each gauge location such that the error tolerance in water surface elevation measurement
was limited to 0.9 mm. Provided all gauges had the expected calibration, the calibration relationships
were saved in a file for application to the measured raw wave data collected the same day as the
calibration (Hughes, 2008).
Run up and piezometer gauge were calibrated daily with the motionless water in the flume, using a two-point calibration up the slope such that the gauges intersected the still water line at these two points. Calibration was conducted for the range of expected water level changes at each gauge location such that the error tolerance in water surface elevation measurement was limited to 0.9 mm. Accuracy of instrument readout was independently verified by using a vertically-mounted measurement scale of the motionless water levels in the flume during the calibration process (Figure 6-8). Precision of instrument readings was verified during calibration by ensuring that the error tolerance specified above was able to be reproduced across the range of water level changes expected during testing (Holman, 1978).

Each test lasted about 5 minutes each, which produced approximately 15,000 time series data points for each gauge. The tests were run for each structure configuration beginning with a 1-ft-high
prototype wave for a given prototype period, incrementing the wave height upward by 1 ft (prototype) until incipient motion was observed.

Using the video that was time-synchronized with the gauge instrumentation of the physical model tests, time series records were prepared at 50 Hz each for the ACM displacements occurring in the vertical center between gauge stations on the structure slope. Armor displacements were read and recorded to the nearest millimeter of vertical movement at each time step. Readings were taken using a regular square grid superimposed on the computer video screen during the data transcription process. Accuracy of vertical movement readings of the structure was independently verified by ensuring that readings taken using the regular square grid corresponded to the measurement scale mounted along the model slope stations in the flume. Precision of these data corresponded to the least count readability of the regular square grid (Holman, 1978).

A goal of reaching a state of instability (i.e., incipient motion) was adopted to acquire data near the threshold of theoretical equilibrium, as detected by visual inspection and documented during dimensionally and temporally scaled video taping for later use in analyses. This level of movement is below the previously-established “no-damage” criteria threshold under wave loading (USACE, 2006). During post-testing of each set up, the revetment system and individual armor blocks were inspected for integrity that existed before testing began. No damages were incurred during any testing, given the incipient motion testing goal. The time series data records for the testing batteries conducted are presented in Appendix B.

6.8. Statistical Analysis of Experimental Measurements. An example application of the D’Agostino’s Omnibus $K^2$ test for data distribution normality was performed for Test A3F9T6H2. The computed values of $K^2$ are approximately 70,000, 116,000, and 135,000, respectively, for data from the deep water wave gauge, run up gauge at Station 4, and piezometer gauge at Station 4. These computed values are much higher than $X^2_{99.5} = 10.6$ for two degrees of freedom, which demonstrates a high degree of distribution normality. This is supported by the Central Limit Theorem, which states
that for increasingly large data sets, the sampling distribution of the mean approaches normal distribution, no matter the population variable distribution (Hill and Lewicki, 2006). All gauge data collected had numerical counts of approximately 15,000 data points, which is very high.

Hill and Lewicki (2006) state that quantitative tests for significance of distribution normality cannot entirely substitute visual inspection of a normal “bell-shaped” curve in the probability distribution of the data. By visual inspection of the probability distributions of the data for all tests, as shown in Appendix C, there is consistency in the distribution normality. Quantitative significance testing for normality for all tests was not performed for this research. Performance of quantitative significance testing for normality would be a concern for the remaining gauge data if: (1) the shapes of the probability distributions for all of the tests were not very similar in shape, and (2) the example test for normality as presented above was close to the cutoff value.

The data records for vertical movements of the ACM under wave loading are not normally distributed. Very often in time during testing, there were short periods of no structure motion. In between, there where varying degrees of very short vertical temporal movements of the structure in upward motion, then returning back down to original position on the slope. These structure vertical measurements exhibited intermittency of near-wall turbulent flow regimes. They appeared to occur temporally during turbulent bursts between moments of laminar and transition flows during testing, as described by Bossey and Lumley (2001). Due to this phenomenon, there is a challenge with ensuring that enough energy is imparted by wave loadings during testing such that the structure incipient motion is not so low as to diminish the significance of correlations between these variables. This phenomenon and its effects on the research are explained further in Chapters 7 and 8.
CHAPTER 7. SPECTRAL HYDROMECHANICS RESEARCH.

infinitely large number of randomly-generated, time-averaged water waveform surface observations
(\eta), its statistical variance (\sigma_\eta^2), is equivalent to individual wave components (\eta^2) integrated over
discrete progressive bandwidth frequencies \Delta f_k. The subscript “k” is the sequential frequency
increment starting from a value of \Delta f, given by:

\[
\Delta f = 1 / D.
\]  

where:

D = Time duration of a set of “n” total observations = \sum_{j=1}^{n} \Delta t_j,
\Delta t_j = Constant observation time increment, and
j = Individual sequential integer time observation from 1 to n.

The number of output variables measured during a test per unit time is commonly expressed in
samples per second, i.e., Hertz (Hz). It follows that the spectral resolution is equal to the quotient of
the frequency bandwidth divided by the number of frequency domain output variables. In this case,
the output variables are the individual values of \eta^2. The Nyquist frequency (\lambda_o) is computed as

\[
\lambda_o = 1 / 2 \Delta t,
\]  

which is the highest frequency that is able to be detected in spectral analyses for a specified \Delta t. The
frequency range from \Delta f to \lambda_o is termed the one-sided Power Spectral Density (PSD) estimate, which
contains total power of the spectrum, and was used in the analyses of this research (Priestly, 1981).
Since \Delta f = 50 Hz for all instrumented channels of testing, \lambda_o = 25 Hz.

For a harmonic wave with amplitude “a_{hw},”

\[
\eta^2 = \frac{1}{2} a_{hw}^2 / \Delta f.
\]  

In the frequency domain, the signal of \eta^2 is the variance density spectrum (E(f)). Assuming fluid
incompressibility, the wave energy of linear surface gravity waves in water is proportional to the
individual wave component $\eta^2$. The variance density spectrum may be distributed over $\Delta f_k$ up to $\lambda_o$ to obtain a constant variance density $\frac{1}{2} a_{hw}^2 / \Delta f$ at individual frequencies. Thus, the area under the signal of $\eta^2$ over $\Delta f_i$ up to $\lambda_o$, termed $m_0$, is equivalent to the value of $\sigma_\eta^2$ of the time series data record. These assumptions and relationships allow identification of $\eta^2$ with physical waveform properties, particle velocities, and pressure variations. As the value of $\Delta f$ approaches zero, $E(f)$, becomes:

$$E(f) = \lim_{\Delta f \to 0} \frac{1}{2} a_{hw}^2 / \Delta f.$$  \hspace{1cm} 7-4

The integral from zero to infinity of $E(f) \, df$ is equivalent to the zeroth-order moment about the mean ($m_0$). The signal of $\frac{1}{2} a_{hw}^2 / \Delta f$ corresponding to initial value of $\Delta f_k$, and sequentially incremental up to $\lambda_o$, may be used to determine the vertical scale of wave heights. The summation of thin vertical areas of $\frac{1}{2} a_{hw}^2$ under the signal from an initial value of $\Delta f_k$, sequentially up to $\lambda_o$ for a single-sided spectrum, is an estimate of $m_0$. In the frequency domain, Holthuijsen (2007) defines the significant wave height, $H_s$, as:

$$H_s = 4 \, (m_0)^{\frac{1}{2}}.$$  \hspace{1cm} 7-5a

In the time domain, Holthuijsen (2007) defines $H_s$ as the mean of the highest one-third of waves in the time series wave record. The value of $H_s$ may be computed in the time domain, according to USACE (2006), as:

$$H_s = 3.8 \, \eta_{rms} \approx 4 \, \eta_{rms}$$  \hspace{1cm} 7-5b

and

$$\eta_{rms} = [ \frac{1}{n} \sum_{i=1}^{n} \eta_i^2 ]^{\frac{1}{2}}$$  \hspace{1cm} 7-5c

where:

$\eta_{rms}$ = Root mean square of the individual time series of water surface elevations, $\eta_i$.

For a statistically stationary short-term record of wind sea waves, the water surface elevations pass through a zero crossing, i.e., a mean sea surface elevation up and down through space and time. The value of $\eta_i$ for a wide spectrum, i.e., irregular wave patterns, is taken as the maximum crest height per
wave relative to the zero crossing elevation of a statistically stationary wave record (Holthuijsen, 2007).

Since execution of a spectral analysis technique results in a frequency domain model of the time series data, the significant wave height computed using Equation 7.5a is an approximation of the value as computed via Equation 7.5b. Section 7.7 includes a discussion of the technique used in-part for progressive improvement of the spectral estimate of significant wave height, relative to the comparable value computed using the time domain signal. Holthuijsen (2007) describes the peak wave period ($T_p$) as the mean period commensurate to the highest one-third of waves in the time series wave record. For consistency, the definition of significant wave height and peak period are used to compute the commensurate values for wave run up and hydromechanic potential.

### 7.2. Spectral Waveform Model

Time series data possess a unique quality of having order in arrangement as a function of time, which is valuable for enabling mathematical modeling of underlying processes representing the data. Modeling can be performed in the time and frequency domains. The zero crossing method (USACE, 2006) is applicable in the time domain. The approach is a wave-by-wave analysis of attendant heights and lengths at a stationary location based on temporal passing of the water surface elevation above and below the mean sea level, which is taken as the zero crossing line. For irregular waves, if the vertically-moving water surface elevation at a static location comes close to but does not cross the zero crossing line before reversing to the opposite direction in the next wave cycle, the height and length of such waves are statistically absorbed into a wave of effectively larger descriptive parameters. The result is an inaccuracy in wave parameter description due to a methodological shortcoming. A random phase/amplitude spectral model is applicable to stationary data time series as the sum of infinitely large number of statistically independent harmonic waves. This approach is superior to the zero crossing approach in describing wave parameters based on data analysis. Beyond this quality, the spectral analysis technique affords the ability to perform statistical hypothesis testing of cause and affect between paired time series signals, as well as the
ability to express statistical confidence intervals in the results. For these reasons, spectral analysis was the preferred method of data analysis for use in this research.

A Fourier analysis technique was chosen for transforming the waveform time series data into the frequency domain using a series of sinusoidal terms, as follows (Bloomfield, 2000):

\[ \eta(t) = a_0 + a_n \cos(\omega t) + b_n \sin(\omega t) \]  \hspace{1cm} 7-6

where:

\( \eta(t) = \) Fourier series of waveform surface elevation as a function of time, normalized to radian scale by dividing all frequencies by the waveform fundamental frequency

\( t = \) Constant time step,

\( \omega = \) Waveform fundamental frequency = \( \frac{2\pi}{T} \),  \hspace{1cm} 7-7

\( T = \) Waveform period,

\( a_0 = \) Constant average value of waveform = \( \frac{1}{2\pi} \int_0^{2\pi} \eta(t) \, dt \), \hspace{1cm} 7-8

\( a_n = \) Cosine waveform coefficient = \( \frac{1}{2\pi} \int_0^{2\pi} \eta(t) \cos(jt) \, dt \) \[ j \geq 1 \],  \hspace{1cm} 7-9

\( b_n = \) Sine waveform coefficient = \( \frac{1}{2\pi} \int_0^{2\pi} \eta(t) \sin(jt) \, dt \) \[ j \geq 1 \], and  \hspace{1cm} 7-10

\( j = \) “jth” observation in time series order of sequence.

During analysis and modeling of time series data, the time and frequency domain values are traditionally represented as abscissa axis values, which is considered the independent process variable. Signal amplitudes are usually expressed as ordinate values, i.e., the dependent variable (Fuller, 1976).

To develop spectral estimates using Equation 7-6 when modeling time series data, the Equations of 7-8 through 7-10 are implemented by replacing the integrals of these equations with discrete summations. This is termed the Discrete Fourier Transform (DFT). The number of data points of the DFT summation equations are considered an unknown until fixed commensurately with the signal length of time series data to be modeled. There is a difficulty in implementing this approach to model
time series data, which requires a mathematical strategy for solution (Priestly, 1981). For given time
series data set:

- without prior knowledge, the waveform fundamental frequency and coefficients of the spectral
  model of Equations 7-6 through 7-10 are typically not known, and
- Unless the time series data are perfectly stationary in its descriptive statistics, Gaussian in
  probabilistic distribution, and linearly harmonic, there will be quantifiable error between the
  ordinate values of the time series data signal and the spectral model signal. This error is termed as
  signal “noise,” and involves naturally random processes, i.e., processes that are not fully
  understood for mathematical/physical explanation in the solution.

The Fast Fourier Transform (FFT) algorithm is a trial-and-error process for determining the
coefficients and fundamental frequency of the spectral model with respect to a time series data signal.
The FFT uses the DFT summation approach for spectral modeling of time series data. During the FFT
trial-and-error process, trial waveform fundamental frequencies are incrementally used to model the
time series data set. The result is back-calculation of trial coefficients commensurate to those of
Equations 7-8 through 7-10. These trial models are termed periodograms.

Given the assumption that random signal noise is present in the time series data signal, the spectral
model signal component of the trial periodogram will typically reveal varying amounts of residual
error of the ordinate values continuously in order point-by-point with respect to the data time series
signal. The spectral model signal component of the periodogram is called a uniform “white noise”
signal or a “purely discrete spectrum”. The random noise component is termed as a non-uniform
“colored noise” or “mixed spectrum” signal.

Using the mathematical formulation of the periodogram in a multiple linear regression approach,
the least squares residual error is minimized for incremental trial fundamental waveform frequencies,
iterating on computation of the spectral model coefficients. During the trial process, the sum of the
squared iterative coefficients is computed for each incremental waveform fundamental frequency. For
iterative trial increments where the normalized values of the coefficients become appreciably greater than zero, the least squares residual error is minimized, and the best fit of the data is converged upon. The solution is simplified and made less computationally intensive when the data time series length is sized as an integral multiple of the periods of the sine and cosine terms, i.e., \(2 \pi / n\), of this error minimization process. The FFT implements these procedures to identify the spectral model fundamental waveform frequency and coefficients that best represent the time series data signal (Priestly, 1981).

The DFT is a complex function, producing a variance density spectrum, also termed a PSD function (Bendat and Piersol, 1980). The PSD may be an autospectrum or a cross spectrum, as explained in that to follow.

An autospectrum is the signal modeled from a single time series data set. The autospectrum for paired \(x\) and \(y\) time series data sets are respectively termed as \(S_{xx}(f)\) and \(S_{yy}(f)\) (two-sided, ranging from 0 to \(2\pi\) in the frequency domain) and \(G_{xx}(f)\) and \(G_{yy}(f)\) (one-sided, ranging from 0 to \(\pi\) in the frequency domain). The two-sided autospectrum is half the magnitude value of the single-sided spectrum. The autospectrum is formulated as shown below.

\[
2 \ S_{xx} = G_{xx}(f) \text{ and } 2 \ S_{yy}(f) = G_{yy}(f)
\]

The autospectrum is composed of real even numbers only, since a single time series data set is always in perfect phase with itself in the frequency domain (Bendat and Piersol, 1980).

“Paired” \(x\) and \(y\) time series data sets are herein defined as those where the ordinate values \(x\) and \(y\) of two different signals are sampled synchronously, i.e., at the same moments together in time. The two-sided cross spectrum \((S_{xy}(f))\) is modeled after paired \(x\) and \(y\) time series data sets. The single-sided cross spectrum, twice the magnitude value of \(S_{xy}(f)\), is termed \(G_{xy}(f)\). Since two different time-synchronized signals may not be in perfect phase with each other in the frequency domain, the cross spectrum is composed of real (coincident spectrum, or cospectrum), i.e., \(C_{xy}(f)\), and imaginary, \(Q_{xy}(f)\),
(quadrature spectrum, or quadspectrum) parts of a complex number, capturing the phase differences between signals (Bendat and Piersol, 1980).

The cross spectrum is expressed as follows:

\[ 2 S_{xy}(f) = G_{xy}(f) = C_{xy}(f) - i Q_{xy}(f) \] 7-12

and

\[ |G_{xy}(f)| = \left[ C_{xy}(f)^2 - Q_{xy}(f)^2 \right]^{\frac{1}{2}} \] 7-13

7.3. **Waveform Variance Density Spectrum Relation with Spectral Model.** The variance density spectrum provides a complete statistical description of the wave propagation processes that take place. For spectral analyses, the data must have a Gaussian distribution, as a representation of naturally and randomly-occurring phenomena according to the Central Limit Theorem (i.e., the sum of a large number of independent random variables without one being dominant) (Holthuijsen, 2007). Assuming the presence of statistically stationary, Gaussian processes, the Fourier transform in the frequency domain is formally defined as the integral over infinite time of the average product of the water waveform surface elevations at each moment in time with constant time lag relative to the mean water waveform surface elevation. Since all joint probability density functions are represented in this computation, there is a complete statistical description of the processes taking place, as in the computation of the variance density spectrum. Conserving total variance, these two computational approaches both provide its distribution over all frequencies, and are thus equivalent (Holthuijsen, 2007).

7.4. **Implementation of Spectral Modeling Techniques in Data Analysis.** The Fourier analysis approach assumes the harmonic waves analyzed are characteristically linear. In the laboratory test, the deep water waves obey a linear harmonic process. The breaking waves, run up waves, and piezometric head buildup in the filter layer of the structure are non-linear harmonic processes. Understanding of non-linear harmonic wave processes and structure response occurring on the slope
was accomplished by use of multiple gauges placed at equidistant spacings at discrete intervals on the structure slope (Davis and Nielsen, 1988; and Nielsen and Dunn, 1998).

Test protocols were executed for irregular waves in each laboratory test. The rationale for specifying irregular waves was to achieve within a relatively short testing time period per test (i.e., 5 minutes), a large range of variability over detectable frequencies. In addition, irregular waves in the flume exhibited a Gaussian distribution, which are considered the type of wave loadings in nature for prototype structure analysis and design.

For each test, the water level in the wave flume was held constant for application of irregular wave conditions of a specified period and height that induced the onset of incipient motion of the ACM structure. For this reason, no de-trending was necessary to attain stationarity in the data, which is required for performing spectral analysis in representation of “short term” relatively constant coastal wave conditions in nature.

7.5. **Data Organization.** The channels of gauge data were prepared in files for each test, as follows. Refer to Figure 5-1b and Figure 6.4a for gauge locations in the flume. Wave gauge #11 measured the deep water wave. Wave gauges #12-14 measured the breaking wave. Wave gauges #15-17 measured the wave run up. The data stream entitled “3 foot Flume Program” and “3 foot Flume Displacement” compare the calculated SWL from the gauge instrumentation with the actual initial measurement of the SWL to ensure and document that the water in the tank is being conserved during testing, i.e., not spilling over the structure in overtopping. The term “slope waverod” stands for “piezometer gauge,” and there are 10 stations of piezometers on the physical model slope. All tests were run in fresh water. The designation “In-H2O” means that the data from these gauges were collected in inches.

- Database Column 1: wave gauge #11 (m)
- Database Column 2: wave gauge #12 (m)
- Database Column 3: wave gauge #13 (m)
7.6. **Data Pre-conditioning.** The time series raw data from physical model testing was inspected for any missing values and analyzed for outliers. A procedure was adopted for removing unexplained outliers, or, retaining outliers with supporting explanation based on linkage to physical processes. The criteria established prior to testing for removing unexplainable outliers was any value falling outside of two standard deviations of the data set. The protocol adopted before testing began that would be used for filling missing values, as well as replacing removed outliers, was averaging between adjacent time series laboratory test data (McAnally, 2008). The quality of the data streams collected was found to be very high, requiring no modification of the data using these techniques. Descriptive statistics were
computed for inspection and confirmation of statistical stationarity and probabilistic distribution as Gaussian in nature, as explained in Chapter 6.

The data records for STAs 3 and 4 of test A3F9T6H2 were chosen for analysis to demonstrate the spectral hydromechanics research algorithm in the text that follows. Energy builds and plateaus in the wave flume during the initial wave generation process. In an effort to manage statistical stationarity in test data collection, the length of the time series record closest to the end of the test where energy levels plateau was used in spectral analyses.

Free surface piezometer readings at each station were adjusted relative to their elevations on the slope with respect to the SWL. Laboratory measurements were converted to millimeters (mm) for purposes of analysis and results presentation. Since pressure changes in the filter media induce ACM structure uplift during wave run up, the piezometer gauge time series at Station 3 was subtracted from that of Station 4 to obtain the pressure head gradient due to wave run up action in the filter media below the armor layer (See Figure 6-4a).

ACM structure displacement video observations were converted into digital format at 50 Hz for each 5-minute-long test record, which resulted in creation of a displacement time series for each measurement station along the physical model revetment slope. The raw ACM displacement data time series was scaled according to structure video animation ratio of on-screen distances measured to the videoed flume measurement standard. Measurements were converted from meters to millimeters in the laboratory results analysis for enhanced comprehension of the relatively small movements detected during testing at the 1:25 model-to-prototype scale. Since this research is focused on physical process discovery, not design, results explained in the following are placed in context of the laboratory model test setting, i.e., not in field prototype dimensions. In any case, the results in Chapter 8 are dimensionless, making no difference on which scale is used for analysis.

Figure 7-1a illustrates the deep water wave record relative to the SWL for Test A3F9T6H2. Figure 7-1b presents the Probability Density Function (PDF) for this wave record. Figures 7-2 and 7-4,
present the time series record and PDFs for the run up gauges at STAs 3 and 4, respectively, for Test A3F9T6H2.

Figure 7-1a. Deep water wave time series record, Test A3F9T6H2.

Figure 7-1b. Deep water wave time series PDF, Test A3F9T6H2.

Figure 7-2a. Wave run up time series record at STA 3, Test A3F9T6H2.
Figure 7-2b. Wave run up time series PDF at STA 3, Test A3F9T6H2.

Figures 7-3 and 7-5 present the time series record and PDFs for the piezometer gauges at STAs 3 and 4, respectively, for Test A3F9T6H2. The PDFs of these figures illustrate the Gaussian nature of this pre-conditioned data, as described in Section 6.8. Figure 7-6 presents a time record of ACM displacements for Test A3F9T6H2 half way between STAs 3 and 4. Displacement data was collected at half-way points between gauges since the research methodology relies on time series gradients between gauges.

Figure 7-3a. Piezometer time series record at STA 3, Test A3F9T6H2.

Figures 7-7a and 7-7b respectively illustrate the test time interval and a sample time interval of data processed using the values of $H_{RIR}$ and $\varphi$, for Test A3F9T6H2. There is an implication of causality between the values of $H_{RIR}$ and $\varphi$ in Figure 7-7a, which is exemplified by relatively low and high
amplitudes running to varying extents together over time. Appendices B and C contain plots of the time series data records and statistics for all test batteries.

Figure 7-3b. Piezometer time series PDF at STA 3, Test A3F9T6H2.

Figure 7-4a. Wave run up time series record at STA 4, Test A3F9T6H2.

Figure 7-4b. Wave run up time series PDF at STA 4, Test A3F9T6H2.
Figure 7-5a. Piezometer time series record at STA 4, Test A3F9T6H2.

Figure 7-5b. Piezometer time series PDF at STA 4, Test A3F9T6H2.

Figure 7-6. ACM displacement time series record at mid-point between STAs 3 and 4, Test A3F9T6H2.

7.7. Spectral Hydromechanics Analysis of System Performance. The pre-conditioned data set were filtered and transformed from the time domain to the frequency domain using Fourier
Figure 7-7a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STA 3-4, Test A3F9T6H2.

Figure 7-7b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STA 3-4, Test A3F9T6H2.

transformation techniques, as described in Sections 7.1 through 7.6. Cross spectral analyses were conducted between incipient wave parameters and structure response parameters to determine spectral relationships across the range of testing. Results of spectral analyses were used to portray revetment system instabilities at respectively tested wave forcings. A research algorithm was developed to quantify spectral hydromechanics performance of the tests. Automated spectral analysis matrix operations were coded in Matlab Version R2008b to process pre-conditioned time series data, as
described in the following. These spectral outputs serve as the foundation for informing a mathematical model to constrain the understanding of uncertainty in performance of the ACM structure at the threshold of stability for the executed range of test structure configurations and wave conditions.

A Daniell 15-point equal-weighted moving average data window transformation was produced over the entire data time series for use in smoothing to diminish random signal noise that may obscure meaningful periodic cycles in the periodogram at higher frequencies. This corresponds to a filter frequency of 3.33Hz, considering that data was collected at 50 Hz.

A Hamming filter was applied on bin data time series segments to:

- identify the greatest spectral densities (i.e., regions of the record consisting of many adjacent frequencies) that contribute most to overall periodic behavior of the series, among periodogram values that may be subject to substantial random fluctuations,
- reduce spectral leakage side lobes of the periodogram values to adjacent frequencies, and
- enhance detection of periodicities of related data sets during spectral analysis.

The data sets were each sized to a multiple of two, equal to 8192 points each, for computationally efficient use in FFT. This total bin length was set as close to the end of the time series record as possible to best manage achieving statistical stationarity. The reason for this approach is that earlier in the record from the observed test beginning, i.e., from the first wave run up attack, energy begins to build in the wave tank with successive wave generation until it plateaus at a relatively steady state. Ensemble averages were produced by subdividing each data series into bin segments with a number of data points sized to the power of two. This procedure included an experimental component for establishing the number of bin segments to attain a minimally-acceptable confidence interval and statistical significance of the coherency function, as described in Chapter 3, entitled “Experimental Design”.

Spectral analyses using FFTs were conducted to obtain auto spectra in the frequency domain from
the $H_{\text{DW}}$ (deep water wave), $H_{\text{RIR}}$, and $\varphi$ time series. Figures 7-8a through 7-8e respectively present the spectral analyses from 0 to 5 Hz with increasing bin segmentations for STA 3-4, Test A3F9T6H2. Values above 5 Hz are not shown since high-frequency noise was removed during time domain smoothing of the signals before conducting spectral analyses. Figure 7-8a is for a single bin length of 8192 points for computation of the raw estimate. The ensemble averages of Figures 7-8b through 7-8e respectively show the 95% confidence intervals of the auto spectra. The confidence limits were calculated using Equation 3-2.

Significant wave heights and peak periods where computed as described in Section 7.1. The values of $H_{\text{DW}}$ (significant deep water wave height), $H_{\text{RIR}}$ (significant relative instantaneous run up) and $\varphi$ (significant spectral hydromechanic potential), computed based on frequency domain estimation techniques, are shown in Figures 7-8a through 7-8e. These plots also present the peak periods of the deep water wave ($T_{\text{DW}}$), relative instantaneous run up ($T_{\text{RIR}}$), and spectral hydromechanic potential ($T_{\varphi}$). As described by Holthuijsen (2007), computation of significant wave height from the raw estimate may have an order of magnitude error of 100%. This is due to the “grassy” signal, i.e., the wide vertical variability from frequency-to-frequency along the signal. As ensembles with increasing data record segmentation are computed, this resolution in the signal is lost, which results in a smoothing of this variability across frequencies of the signal. With smoothing, the value of significant wave height increases in approach to the value of significant wave height computed in the time domain. Progressive improvement of these estimates are made with respect to the significant wave height computed in the time domain using Equation 7.5b, which is equal to 25.7 mm.

The magnitude squared coherency function, also referred to in literature as the squared coherency, or squared coherence, was computed to indicate the degree a linear relationship exists between the auto spectra of $H_{\text{RIR}}$ and $\varphi$. The squared coherency function in spectral analysis is similar to the correlation coefficient used in linear regression analysis. A value of “0” means no linear correlation.
and the result of “1” indicates a perfect linear correlation. The magnitude squared coherency spectrum $(\gamma_{xy}^2(f))$ is computed as follows (Bendat and Piersol, 1980):

$$\gamma_{xy}^2(f) = \frac{|S_{xy}(f)|^2}{S_{xx}(f)S_{yy}(f)} = \frac{|G_{xy}(f)|^2}{G_{xx}(f)G_{yy}(f)}$$  \hspace{1cm} 7-14

The 95% confidence interval of $\gamma_{xx}^2(f)$ is computed after Bloomfield (2000), as follows:

$$\tanh^{-1}(\gamma_{xx}^2(f)) \pm 1.96 \sigma_s$$  \hspace{1cm} 7-15

where:

$\sigma_s$ is computed for values of $\tanh^{-1}(\gamma_{xx}^2(f))$.

The cospectrum and quadspectrum respectively represent in-phase and out-of-phase components of the signal in the frequency domain. The phase difference as a function of frequency between two time series data sets is revealed by computation of the phase function. The phase function $(\theta_{xy}(f))$ is calculated as shown below (Bendat and Piersol, 1980):

$$\theta_{xy}(f) = \tan^{-1}\left[\frac{Q_{xy}(f)}{C_{xy}(f)}\right]$$  \hspace{1cm} 7-16

The 95% confidence interval of $\theta_{xy}(f)$ is computed below based on Bloomfield (2000):

$$\theta_{xy}(f) \pm \left[\frac{1.96}{2\pi}\sigma_s(\gamma_{xx}^2(f))^{-1}\right] - 1$$  \hspace{1cm} 7-17

It is important to note that the squared coherency and phase functions, and their respective confidence intervals, are computed using ensemble average values as inputs, with the intent of estimating correlation variability and uncertainty as a function of frequency.

Cross spectra, squared coherency, and phase functions of the $H_{RIR}$ and $\varphi$ data series for STA 3-4, Test A3F9T6H2, were produced with 95% confidence intervals as shown in Figures 7-9a through 7-9d, which are respectively commensurate to bin segments of Figures 7-8b through 7-8e.

In Figures 7-9a through 7-9d, there is progressive improvement in constraint of uncertainty of the mean signals, as the confidence intervals become more defined with greater segmentation and ensemble averaging. In the process, the squared coherency signal becomes less variable over the frequency range shown in tradeoff with progressive shifting downward along the ordinate axis. With downward shift, the signal approaches zero, at which point, there is no relationship.
Appendix D presents the results for the tests analyzed in this research. The null hypothesis is tested at mean squared coherency signal values commensurate to $T_{RIRp}$, using data from Appendix D, which is summarized in Table 7-1. This is the first known attempt to perform hypothesis testing on relationships between wave loading and revetment structure performance. In some of the test results, the mean squared coherency signals fall above the $\sigma^2 = 0.181$ cutoff value, where the null hypothesis may be rejected at a statistical significance level of $\alpha = 0.05$. The remaining tests had statistical significance levels between 0.05 and < 0.30, as shown in Table 7-1. Bloomfield (2000) states that if the null hypothesis is able to be rejected in the frequency range that the squared coherency signal was analyzed:

- confidence intervals are valid for that frequency range, and
- the attendant autospectra may be directly compared in that frequency range.

In a similar manner, the phase is only comparable at frequencies where the spectra are coherent. With greater numerical value of statistical significance for each test analyzed, the probability that rejection of the null hypothesis is the wrong finding increases, as further described in Chapter 3. Chapter 8 further analyzes these results as a group in demonstration of an approach to constrain uncertainty in structure performance quantification between the thresholds of incipient motion and no damage.

In Figures 7-9a through 7-9d, the signals between $H_{RIR}$ and $\varphi$ are in and out of phase intermittently, but the signal generally oscillates out of phase about zero equally across the frequency domain, i.e., not trending in either direction.

In summary, the generalized algorithm is as follows, which as described may be applied to any two physical process-related generic time series signals “$x$” and “$y$” that meet the conditions for spectral analysis:

- Plot and inspect the single column time series arrays “$x$” and “$y$” for missing values and outliers
- Explain outliers considered valid in the experimental processes, if required
• If required, remove outliers that cannot be explained as valid in the experimental processes that lie two standard deviations beyond the mean

• Replace removed outliers and missing values with averages of adjacent time series data

• Inspect the single column time series arrays “x” and “y” for statistical stationarity and Gaussian distribution

• De-trend single column time series arrays “x” and “y,” if required

• De-mean single column time series arrays “x” and “y” to eliminate a sharp spike in application of the first cosine function in spectral analyses at low frequency

• Smooth arrays “x” and “y” using data windows to remove signal noise and manage spectral leakage across frequencies

• Divide arrays “x” and “y” into a trial number of “m” segments each, with each segment consisting of “n” data points per segment consisting of string data point lengths to a power of 2 each in preparation of FFT

• Compute the statistical significance level required to reject the null hypothesis that the square coherency is zero, for trial “m” segments a desired value of α

• Produce a single column frequency array for equally-spaced, constant time step “t,” for “n” points of segmented single column time series arrays “x” and “y”

• Compute the Nyquist frequency as maximum frequency for signal plots

• FFT each segment of each array to produce auto spectra

• FFT each segment of each array to produce cross spectra

• Produce ensemble averages for auto spectra of each signal and the cross spectrum

• Produce ensemble average squared coherency function

• Produce ensemble average phase function

• Produce ensemble average amplitude response function
• Produce percent confidence intervals for ensemble averages of the auto and cross spectra, squared coherency function, phase function, and amplitude response function

• Plot the frequency domain output signals against frequencies up to no higher than the Nyquist frequency, along with the signal confidence intervals, and the statistical significance level required to reject the null hypothesis that the square coherency is zero

• Inspect the squared coherency plot to determine whether the signal and confidence intervals fall above the statistical significance level required to reject the null hypothesis that the square coherency is zero
  – If the null hypothesis is able to be rejected, consider the spectral analysis process final
  – If the null hypothesis is unable to be rejected, increase the number of segments “m” and repeat the above-described process

• Compute the significant amplitude height and period of the auto spectra

Table 7-1. Summary of tests for statistical significance.

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<th>Test Designation</th>
<th>STA</th>
<th>α</th>
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Figure 7-8a. Variance density auto spectra, raw estimates, STA 3-4, Test A3F9T6H2.
Figure 7-8b. Variance density auto spectra, 2-segment ensemble, STA 3-4, Test A3F9T6H2.
Figure 7-8c. Variance density auto spectra, 4-segment ensemble, STA 3-4, Test A3F9T6H2.
Figure 7-8d. Variance density auto spectra, 8-segment ensemble, STA 3-4, Test A3F9T6H2.
Figure 7-8e. Variance density auto spectra, 16-segment ensemble, STA 3-4, Test A3F9T6H2.
Figure 7-9a. $\varphi$ - H$_{RIR}$ variance density cross spectrum, squared coherency function, and phase function, 2-segment ensemble, STA 3-4, Test A3F9T6H2.
Figure 7-9b. $\varphi - H_{RIR}$ variance density cross spectrum, squared coherency function, and phase function, 4-segment ensemble, STA 3-4, Test A3F9T6H2.
Figure 7-9c. $\varphi - H_{RIR}$ variance density cross spectrum, squared coherency function, and phase function, 8-segment ensemble, STA 3-4, Test A3F9T6H2.
Figure 7-9d. $\varphi - H_{RIR}$ variance density cross spectrum, squared coherency function, and phase function, 16-segment ensemble, STA 3-4, Test A3F9T6H2.
CHAPTER 8. STATISTICAL AND MATHEMATICAL MODELING.

8.1. Revetment Stability Under Wave Loading. Herbich (1999) summarized the results of block revetment tests for a variety of cases in the context of Equation 2-20, including loose and connected blocks, as well as blocks laid on pervious and impervious slopes. The least amount of research exists for connected blocks, and there is significant uncertainty in existing literature on its performance. Herbich (1999) recommends conservatism for design use of this information in the absence of additional performance data, which has been missing from prior research in the lower ranges of stability where it is technically the most difficult to acquire performance data.

In the ACM structure research, an approach was followed to demonstrate a method for reducing the knowledge gap in the performance potential of connected block revetments under wave attack near the threshold of incipient motion. The analysis requires modification of Equation 2-20 to incorporate the effects discovered in spectral hydromechanics research. The use of \( \phi_s \) in Equation 2-20 is introduced via substitution of Equation 4-9b into it, as shown in Equation 8-1, which quantifies the spectral hydromechanics systems-scale affects of connected blocks mobilized in resistance to wave loading.

\[
S_b = \frac{H_s}{\Delta (\phi_s l_a w_a t_a)^{\frac{1}{3}} \xi_o^{-\phi_s}}
\]

In Equation 8-1, the interpretation is that the number of blocks mobilized is equated to an effective block geometry, i.e., capturing the total mass of blocks instantaneously mobilized at systems-scale in resistance to wave attack. This formulation attempts to provide a mechanism to capture the articulation affects that Pilarczyk (1998) considered missing from Equation 2-20, as modified by Equation 2-21.

8.2. Spectral Hydromechanics Revetment Stability Statistical Results. Significant values of signal amplitude and peak period were found via spectral hydromechanics analyses, as described in Chapter 7. Tables 8-1a, 8-1b, and 8-1c, respectively present summary statistics for the 95% upper confidence limit, average values, and 95% lower confidence limit of these data.
Table 8-1a. Summary statistics of spectral hydromechanics analyses, 95% upper confidence
limit, frequency domain.

Table 8-1b1. Summary statistics of spectral hydromechanics analyses, average values, frequency
domain.

The ability to produce confidence interval values arises from use of ensemble averages of the spectral

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hydromechanics research. Therefore, the 95% upper and lower confidence interval summary statistics are only available for spectral hydromechanics research performed in the frequency domain.

Summary statistics are presented in both the frequency and time domains for average values.

Table 8-1b2. Summary statistics of spectral hydromechanics analyses, average values, time domain.

| Test Designation | STA | \( t \) (mm) | \( S \) (mm) | \( \sigma \) (mm) | \( 
\sum \sigma \) (mm) | \( H_{\text{low}} \) (mm) | \( H_{\text{high}} \) (mm) |
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Table 8-1c. Summary statistics of spectral hydromechanics analyses, 95% lower confidence limit, frequency domain.

| Test Designation | STA | \( t \) (mm) | \( S \) (mm) | \( \sigma \) (mm) | \( 
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* \( \text{tan}(\theta) = \cfrac{\text{vertical displacement}}{\text{horizontal displacement}} \)

\( \text{tan}(\theta), \text{sec} = \cfrac{\text{vertical displacement}}{\text{horizontal displacement}} \)

\( \text{sum} = \sum_{i=1}^{n} \text{data} \)

\( \text{mean} = \frac{\sum_{i=1}^{n} \text{data}}{n} \)

\( \text{std} = \sqrt{\frac{\sum_{i=1}^{n} (\text{data}_i - \text{mean})^2}{n}} \)

\( \text{var} = \frac{\sum_{i=1}^{n} (\text{data}_i - \text{mean})^2}{n} \)

\( \text{cov} = \frac{\sum_{i=1}^{n} (\text{data}_i - \text{mean})(\text{data}_j - \text{mean})}{n} \)

\( \text{corr} = \frac{\text{cov}}{\text{std} \times \text{std}} \)

\( \text{RMS} = \sqrt{\frac{\sum_{i=1}^{n} \text{data}_i^2}{n}} \)

Very low value of \( \text{var} \) makes computation beyond meaningful characterization.
In Tables 8-1a, 8-1b, and 8-1c, results and computations are shown for the irregular Deep Water Wave (DWW), Relative Instantaneous Run up (RIR) for irregular waves, and hydromechanic structure response (φ) for irregular waves. All data are presented at model scale of the laboratory tests.

Comparative analyses of significant amplitude signal response and peak periods are investigated in the time and frequency domains via plots that are presented in Figures 8-1a, 8-1b, and 8-1c. These figures respectively present the relationship between time and frequency domain statistical estimates of significant amplitudes for the irregular deep water wave, relative instantaneous run up for irregular deep water waves, and hydromechanic potential for irregular deep water waves.

Figure 8-1a. Relationship between time and frequency domain statistical estimates of significant amplitudes for the irregular deep water wave.

Differences in comparable data, respectively shown in Figures 8-1a, 8-1b, and 8-1c, result from:

- imperfections in the laboratory physical scale model,
- limitations on the accuracy and precision of instrumented data collection, and
- limitations on use of the linear harmonic random phase/amplitude model for Fourier transformation of time domain data into the frequency domain to model processes that are not perfectly statistically stationary and Gaussian in probability distribution.
Figure 8-1b. Relationship between time and frequency domain statistical estimates of significant amplitudes for the relative instantaneous run up for irregular deep water waves.

Figure 8-1c. Relationship between time and frequency domain statistical estimates of significant amplitudes for the spectral hydromechanic potential for irregular deep water waves.

Differences between time and frequency domain estimates are also compounded by loss of spectral resolution with increasing ensemble averaging. Given the knowledge of the statistical differences in
the trends of the relationships contained in Figures 8-1a, 8-1b, and 8-1c, for consistency in this research, results of frequency domain analyses are used in subsequent investigations.

New relationships based on this research were quantified to associate irregular deep water wave parameters with relative instantaneous run up for irregular deep water waves, with respect to the still water level, for testing using a structure slope equal to a geometric proportion of 1 V : 4.25 H. These results are presented in Figures 8-2a, 8-2b, and 8-2c, for the 95% upper confidence limit, average, and 95% lower confidence limit values, respectively. Since the values of $T_p$ for the deep water wave and relative instantaneous run up are respectively similar in the tested irregular wave conditions, as shown in Table 8-1, an estimate for the value of $H_{RIRs}$ may be found using the $T_{pDWW}$ and results shown in Figures 8-2a, 8-2b, and 8-2c. The value in these relationships are most advantageous in:

- estimating the height of run up for preliminary design of the structure crown, considering freeboard, and
- establishing the confidence limits of wave run up for consideration in design to constrain uncertainties in structure performance for limiting wave run up height average return period exceedance.

8.3. Comparison of Revetment Stability Research with Prior Work. Figure 8-3 presents an evaluation of test cases of linked revetment blocks on a granular filter layer for “normal” criteria, which was presented by Herbich (1999) for regular wave forcings. Refer to Section 2.3 for definition of “normal” criteria. The results from the spectral hydromechanics research was conducted using irregular wave loadings and are plotted in Figure 8-3 along with the data of Herbich (1999). The value of $H_s$ for irregular waves may be adjusted to be equivalent to $H_s$ for regular waves via the following relationship (Pilarczyk, 1998):

$$\frac{H}{H_s} = 1.4 \quad 8-2$$

Equation 8-2 is for wave heights at the threshold of structure damage, based on measurements of piezometric head on the slope for regular and irregular wave attack. It is assumed that Equation 8-2 is
applicable for adjusting irregular waves to regular waves for the threshold of incipient motion. However, no attempt has been made to apply Equation 8-2 to the results of this research for conversion from irregular to regular wave conditions, since the affect is unknown for making an equitable adjustment of \( \phi \) in use of Equation 8-1.

The stable upper limit, as described by Herbich (1999), is for preliminary design of new structures, and is commensurate to \( S_b = 3.7 \). The unstable lower limit (Herbich, 1999) is for preliminary design of new structures, which corresponds to \( S_b = 8 \). The stable upper limit for preliminary verification of old structures is not shown, but is an approximately 15% less-conservative upward shift of the stable upper limit line for preliminary design of new structures. The region lying between the stable upper limit and unstable lower limit is a zone of what Herbich (1999) termed as “doubtful” performance, as evaluated under test conditions of prior works.

Figure 8-2a. Relationship of deep water wave parameters with relative instantaneous run up for structure slope equal to \( 1 \, V : 4.25 \, H \), 95% upper confidence limit values, frequency domain.

This is a large range of uncertainty in structure performance under wave loadings in the data of prior works in the context of the stability number of Equation 2-20, particularly below the stable upper limit.

Since the goals of engineering design are to be as efficient and effective as possible, this new research explores lower stability limits than achieved and explained in prior works. The intent is to demonstrate how the design process could be better informed for reducing knowledge gaps in this
field, considering new analytical information and measures for management of remaining uncertainties in structure performance below the traditional stable upper limit for no damage.

Figure 8-2b. Relationship of deep water wave parameters with relative instantaneous run up for structure slope equal to 1 V : 4.25 H, average values, frequency domain.

Figure 8-2c. Relationship of deep water wave parameters with relative instantaneous run up for structure slope equal to 1 V : 4.25 H, 95% lower confidence limit values, frequency domain.

Figures 8-3a, 8-3b, and 8-3c each contain the stability limit curve for theoretical equilibrium of the destabilizing forces of wave attack, in balance with the restoring force of the sloping revetment structure elements, i.e., for $S_b = 1.0$ for all range values of experimental testing, as processed using Equation 8-1. Figures 8-3a, 8-3b, and 8-3c present test data results of this research for calculations
using the 95% upper confidence limit, average values, and 95% lower confidence limit.

No detectable patterns were able to be identified between the performance of varying ACM structure designs as a function of filter layer thickness, given the design approach of ensuring the filter layer is able to behave as a porous structure with turbulent flow potential.

Figure 8-3a. Past and current body of test results for evaluation of linked revetment blocks on granular filter, 95% upper confidence limit values, frequency domain.

Figure 8-3b. Past and current body of test results for evaluation of linked revetment blocks on granular filter, average values, frequency domain.
Figure 8-3c. Past and current body of test results for evaluation of linked revetment blocks on granular filter, 95% lower confidence limit values, frequency domain.

For the data points of this research shown in Figures 8-3a and 8-3b, the plots contain data points with a range of statistical significance, as described in Chapters 3 and 7. Considering the phenomenon of intermittency described in Section 6.8, it is speculated that points of the remaining tests with statistical significance values greater than $\alpha = 0.05$ may be the result of an insufficient amount of wave loading energy to generate sufficient PSD in the structure response for bearing out higher, more desirable levels of statistical significance in the spectral analyses. Data points of this research shown in Figures 8-3a and 8-3b with statistical significance values greater than $\alpha = 0.05$ have similarity with the tests with results of $\alpha = 0.05$ in experimental trends of the spectral analyses. All data points of this research that are shown in Figures 8-3a, 8-3b, and 8-3c lie between the stable upper limit and theoretical equilibrium curves. These observations provide rationale that suggests the collection of data points developed through this research, which have a range of statistical significance values, are more compelling to believe as true when taken as a whole than when considering any of the test results individually.
Figures 8-4a and 8-4b respectively show the test analysis results for Stations 3-4 and 4-5, average values, frequency domain. The points for Stations 3-4 lie just above the curve $S_b = 1.0$. Approximately two-thirds of the points for Stations 4-5 reside close to the curve $S_b = 1.0$, with the remainder falling generally between the curves for $S_b = 1.0$ and 3.7. While both station intervals lie just below the SWL, the latter is slightly closer. The observation of consistent trends in the separate calculations respectively made at these to closely-lying station intervals is confirmatory on the repeatability of the demonstration analyses.

![Figure 8-4a](image)

*Figure 8-4a. Past and current body of test results for evaluation of linked revetment blocks on granular filter, average values, frequency domain, Stations 3-4 only for ACM on aggregate filter.*

**8.4. Revetment Stability Parametric Constraint and Performance Simulation.** It is apparent from plotting the points in Figures 8-3a, 8-3b, and 8-3c for the ACM on aggregate filter that a new lower limit is physically observed to exist, commensurate to the range between theoretical equilibrium of $S_b = 1.0$ and the stable upper limit of $S_b = 3.7$. Given that laboratory experimentation was conducted at incipient structure motion under wave loading, synthesis of the spectral hydromechanics research using Equation 8-1 demonstrates its potential to support constraining uncertainty in quantifying stable structure performance for preliminary design to values of $S_b$ between 1.0 and 3.7.
Figure 8-4b. Past and current body of test results for evaluation of linked revetment blocks on granular filter, average values, frequency domain, Stations 4-5 only for ACM on aggregate filter.

With this method demonstrated, it is now possible to specify additional tests that by design should fall along equi-potential lines of $S_h$. In this process, with increasing equi-potential lines $S_h$ from 1.0 to 3.7, the model structure would be examined following testing for armor layer damage (i.e., cracks, breaks, and missing pieces). For this, the ACM model design will have to more closely mimic the prototype’s geometrical and material characteristics. The equi-potential lines of $S_h$ up to 3.7 would be annotated with average percent damage observed for number of wave cycles, commensurate to reasonably expected design wave event durations. In the process, iteration may be required to optimize the value of K in computation of $\varphi$ during time series data analysis. This information could be used by designers in explaining the life cycle tradeoff of varying revetment designs and costs from more to less stable, to include average return period damage repairs and maintenance with incrementally reduced stability.
CHAPTER 9. EPILOGUE.

Chapter and paragraph numbers are shown in parentheses in the findings and conclusions for cross reference to these supporting details.

9.1. Findings. Stability testing has been traditionally characterized with respect to no-damage criteria (2.3, 6.7). The range of structure performance between the thresholds of incipient motion and no-damage criteria has not previously been quantified. A hydromechanics approach was found useful in designing and executing laboratory experiment protocols for observing structure movements near the threshold of incipient motion, as well as for use in analyzing the data collected from these tests. A new term, the hydromechanic potential ($\varphi$), was derived for use in this research as an enabler to detect and measure incipient motion of the ACM structure under wave loading (4.3).

Through the use of the equations used to compute $\varphi$, the localized effect of drag and inertia forces are implicitly introduced into the stability equation for block revetments. It is possible that the value of $K$ for ACM blocks of different thicknesses differ, despite use of a constant value of $K = 0.5$ for all analysis of this research. Larger values of $K$ would be expected for increasing block thicknesses, which if explored for use in further study, may result in increased agreement of test data points along the incipient motion threshold curve (4.3).

It is important to manage similarity in model design and execution using the Froude Number for armor structure sizing, and simultaneously, the Reynolds Number for management of a turbulent filter flow regime by selecting appropriate aggregate sizes. A new method is provided via this research to demonstrate how Reynolds similarity was achieved in filter aggregate selection at model scale for testing, simultaneously with achieving Froude similarity (5.4).

Non-linear processes of wave transformation in structure run up are manageable in approach to linearity by applying limit theory from calculus for governing data collection during testing (6.1-6.3).

This research produced for the first time known hypothesis testing on the relationships between wave loading and structure response, with expression of mean values and confidence intervals (7.7).
It is recognized that the presence of a filter layer to provide a porous flow regime potential beneath the revetment structure is effective in managing armor layer stability. By inspection and trial statistical analyses of the summary data in Table 8-1, there is no apparent distinction in filter layer performance in the performance of armor layer stability for the range of structure configurations and combinations of short-wave loading parameters tested (8.2).

There is a relationship between significant amplitudes and periods of wave and structure parameters, as computed in the time and frequency domains. Statistical stationarity, Gaussian nature, linearity, and spectral resolution have affects on these results, relative to the respective time domain computations. This research demonstrated how quantification of wave and structure parameters varies depending on derivation from the time and frequency domains (8.2).

A new relationship has been developed between the Iribarren numbers for the deep water wave and relative instantaneous run up wave with respect to the location of the still water level, based on the results of the spectral analyses, which may be used to estimate structure crown elevation requirements. A geometric mean, weighted using values of $\varphi_s$, was used to identify performance trends in the laboratory experiment results. (8.2).

The spread of data plotted in Figure 8-3a, 8-3b, and 8-3c for the ACM research for the 95% confidence interval and average values demonstrate that a theoretical lower limit threshold exists, is measurable, and quantifiable between values of $S_b$ of 1.0 and 3.7. A number of these points had statistical significance of $\alpha = 0.05$. Considering the phenomenon of intermittency, there is speculation that points of the remaining tests may not have had an amount of wave loading energy to generate sufficient power in the structure response for bearing out this high level of statistical significance in the spectral analyses (6.8, 8.2-8.3).

9.2. Conclusions. A high degree of expertise and specialized laboratory resources are required to achieve the physical model planning, design, testing, instrumentation, and data collection, for the scope of this research at such low force-displacement detection levels. For the first time, detailed,
quantitative spectral hydromechanic correlations have been found between wave loading and structure response due to these achievements. This greatly surpasses traditional methods on the analysis of coastal structure performance, which principally rely on before and after test conditions of the structure, in terms of damage due to wave loading. The difference between traditional methods and the new spectral hydromechanics approach made the difference in the ability to test structure configurations at incipient motion under wave loading.

Consideration should be given to adopting a filter design approach that enables turbulent porous flow, which has potential as indicated in this research of limiting problems in armor layer uplift and filter layer scour/sloughing during wave backwash down slope, which is a potential failure mode when granular filters are used in design.

Developing this newly-demonstrated spectral hydromechanics approach for use in planning and design of structure armoring options will become increasingly important with greater exposure of earthen levee structures to the open coast, considering coastal Louisiana wetlands loss rates, sea level rise, and the potential for increased coastal storm activity.

9.3. Recommendations. For use in preliminary design, given the spectral hydromechanics approach demonstration, consideration should be given to using the lower and upper limits of the stability coefficient, \( S_b \), of 1.0 and 3.7, respectively, with the average value of \( S_b = 2.35 \). This will result in an expression of the solution in terms of the 95% confidence interval, with the average value, which are useful in constraining uncertainty in both structure design performance and related construction cost implications. Commensurately, damage rates for equi-potential curves of \( S_b \) should be estimated into the future to inform the design development and selection process.

When potentially applying this new approach, if developed further, comparison of coastal structure protection options should be made with alternatives of maintaining and restoring coastal wetland buffers residing seaward of earthen levee systems, with a view towards minimizing the direct exposure of these levees to the open coast that could otherwise lead to increased maintenance costs.
CHAPTER 10. FUTURE RESEARCH RECOMMENDATIONS.

An investigation in different structure slope angles with longer period waves would be useful in identifying where the function of varying filter thicknesses and porosities/permeabilities become distinguishable for varying granular and stone equivalent gradations, beyond research conducted to-date, regarding its effect on stability of the armor layer.

Future research should be performed on idealized tests to examine regular wave action on ACM displacement to gain a clearer understanding of fundamental physical processes of force-displacement, considering single and multiple gauge station windows of evaluation.

More consideration should be given to instrumentation setup, measurement, and computation of velocity in the filter layer. Use of shallow water equations might be useful to refine velocity computations in the filter layer.

Refinements to the value of $K$ used in computation of $\varphi$ for varying block thicknesses should be attempted for potential improvement in agreement of test results along the theoretical incipient motion curve. Two aspects should be considered:

- Conducting idealized tests for various design configurations (e.g., block thickness/opening width, block roughness ($k_s$), filter thickness/porosity) that isolate the process of water flow through the porous armor and filter layers for process discovery and quantification (e.g., entry, transition, and exit loss components of “$K$”), and
- Developing aggregate values of “$K$” for systems-level performance for various design configurations and statistically relate these values to the sum of the individual loss components of respective equivalents in the idealized tests.

Given the spectral hydromechanics method successfully demonstrated in this research as a basis, further experimentation using the laboratory flume model and instrumentation should be conducted to collect data targeted in the region between $S_b = 1.0$ to 3.7 with a goal of developing equi-potential stability curves, respectively commensurate with damage estimates. This information could be used to
inform designers on the percentage of blocks that become unstable as structure-impacting wave energy increases, to understand where the values of $S_b$ for specific designs are commensurate with damage during structure movement under loading. In any future work, it is recommended that the laboratory test results for each test battery be checked using the automated spectral hydromechanics computer code before moving onto the next test setup. This should ensure that intermittency is managed and an amount of wave loading energy is provided to generate sufficient power in the structure response for bearing out a high level of statistical significance in the spectral analyses. An automated displacement recognition technology would be required to rapidly obtain the structure displacements in facilitation of this step, rather than experiencing undue delay by manually transcribing the analog displacement data to digital form for each test.

Consideration of using alternative spectral analysis techniques should be explored for handling of non-stationary, non-linear, non-Gaussian, and intermittent data sets to determine whether there could be any improvements in the data analysis. Alternative approaches include:

- **Continuous wavelet analyses**, which involves use of alternative functions in the spectral model such as a linear harmonic decay (“Mexican Hat”), or a fractal decay. These functions have potential for enhancing understanding of time series phenomena by reducing what would otherwise be considered as unresolved random signal noise when using linear harmonics in spectral analysis (Lewalle, 1995). Relevant application of this approach was for sediment transport under breaking wave conditions (Scott and Hsu, 2008).

- **Empirical Mode Decomposition (EMD) with Hilbert-Huang Transformation (HHT) spectral analyses**, which has been used to identify intrinsic modes and characteristic scales of high Reynolds Number turbulent intermittency (Huang, Schmitt, Lu, and Liu, 2008).

Formulation of additional testing to inform a statistical correlation between wave loading and structure response parameters, with a confidence interval, would increase understanding of variability and uncertainty in relevant physical processes of interest. This would also be useful for informing
formulation and development of mechanistic hydrodynamic numerical modeling of sloping revetment structures under wave attack. With such a model, it would be possible to simulate a wider and more complex range of structure configurations under wave loading.
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The US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory (ERDC CHL), has developed a model suite and simulation technique for determination of water level probabilities for coastal flood risk assessment and management studies. These state-of-the-art methods and models have been applied to the Interagency Performance Evaluation Team (IPET) analyses, 100-year design study for the hurricane risk reduction system around New Orleans, and the Louisiana Coastal Protection and Restoration (LACPR) Project (USACE, 2009), and others. At the time of this writing, ERDC CHL is using an evolved approach for demonstration of these methods and models for “Risk Quantification for Sustaining Coastal Military Installation Assets and Mission Capabilities, Project Number SI-1701,” under the Strategic Environmental Research and Development Program (SERDP) (Russo, 2009). These methods and models are the most advanced available to estimate water levels and wave parameters for determining run up on coastal structures for planning, analysis, and design purposes, in the context of a comprehensive systems-scale flood stage-frequency analysis.

On SERDP Project SI-1701, hydrologic modeling studies are being conducted to characterize shift the entire range of water-level probabilities over the life cycle period of analysis due to rises in mean sea level. It is very important to understand the impacts of sea level rise (SLR) on the entire range of water level probabilities, due to its potential impact on a wide range of factors ranging from the succession of ecological zones to increased frequency and/or severity of hazards to military installation assets and mission capabilities (training grounds, facilities, road networks, etc.). The objective is to develop efficient methods for accurately characterizing expected variations in water level hazards at coastal sites in response to the SLR scenarios of SERDP Project SI-1701. In this context, both the role of wind-driven surges/waves and the impact of SLR on inundation depths will be considered.
The hydrodynamic modeling process to accomplish the aforementioned scope involves analysis of several variables for existing conditions in the Hampton Roads region of the Atlantic Ocean coastline to generate stage-frequency outputs. This information is being used to support installation asset and mission capability performance assessment across a range of metrics as sea levels rise. Static inputs to the hydrodynamic modeling process will include ground elevations, bathymetry, and pumping/storage capacity inside of existing flood risk reduction systems. Variable inputs for analysis include:

- Storm intensity, path, and frequency,
- SLR and localized land subsidence rates,
- Base and future changed conditions of the coastal landscape outside of existing levee/seawall/floodwall/breakwater/gated (i.e., structural flood risk reduction) systems,
- Potential improvements to the coastal landscape outside existing structural flood risk reduction systems,
- Storm surge elevation/duration,
- Wave characteristics,
- Existing structural flood risk reduction system elevations and locations, and
- Rainfall volumes/durations.

State-of-the-art hydrodynamic modeling is being used to simulate flooding conditions for a specified range of coastal storm average return periods in the study area. Existing data sources and codes are being used in well-tested analytical and numerical models and tools for analysis of tropical and extra-tropical cyclone winds, surges, waves, run-up, overtopping, and interior flood routing/drainage of installations, from storm origin through landfall and system decay (Figure A-1). The ERDC CHL has: (1) either developed and/or participated in development of these models, (2) high performance computers capable to run these models, and (3) has extensive experience and has internationally recognized expertise for adapting and applying these types of models on projects and studies world wide as part of the ERDC mission.
Figure A-1. ERDC CHL simulation technique and models for inundation stage-frequency analysis.

For the Hampton Roads region, analysis will include the 1- and 10-yr precipitation hydrographs combined with 50- and 100-year average return period tropical and extra-tropical coastal storm events as sea levels rise. The hydrodynamic modeling process applied to the Hampton Roads region will determine the behavior of the surge and waves and resultant stages for the specified range of storm event probabilities in the Hampton Roads region at individual military installation sites located:

- on the outside of any existing structural flood risk reduction systems, considering the interaction between these measures and existing coastal geomorphologic features,
- inside existing flood risk reduction systems from overtopping and rainfall during a storm event, and
- inside of military installation areas having no existing flood risk reduction systems.

The team is acquiring and converting historical data, such as gauge data, high water mark data, etc., into the most currently used datum for the Hampton Roads region, taking into account the
potential for historical data references to multiple datums spanning numerous leveling epochs. The North American Vertical Datum (NAVD) 88 datum is used as the reference for all elevations in the study unless otherwise stated as being a different datum. The team is acquiring and using existing topographic, bathymetric, and water surface elevation gauge data that is available from a variety of sources, complementing data already belonging to the team. No new field data collection efforts is being undertaken to generate new data for this study. Existing grids for the study area are being modified for use to conduct analyses. This effort is being accomplished using spatial database methods and tools, which entails developing a physical characterization and inventory of installation asset/mission capability in the Hampton Roads region. This data will show which sites in the region have flood risk reduction structures in place at their coastal boundaries, as well as those exposed to the open coast. The team is partnering this effort with other related efforts being conducted in the region, such as coastal flood elevation re-mapping by ERDC CHL for the Federal Emergency Management Agency (FEMA).

Development and hydrodynamic simulation of a full storm suite to estimate future water level probabilities is beyond the scope of the present effort, as was performed for the LACPR Project. For the level of analysis required for SERDP Project SI-1701, FEMA maps are first being used and adjusted for the impact of SLR. Sea level rise does not result in linear changes to the stage-frequency curves across the coastal landscape. Therefore, a limited suite of storms will be selected and a sensitivity modeling analysis performed for the existing and future life cycle no-action conditions. Deviations of the surge and wave responses from existing conditions will be computed for each storm in the sensitivity suite. The rank order of the storms is assumed as a constant from the existing condition at each location of interest to estimate the change in the water level and commensurate inundation depth probabilities due to SLR under existing and no-action life cycle conditions. The proposed sensitivity modeling approach is being applied to demonstrate the risk quantification capability for the scope of analysis on SERDP Project SI-1701. An initial assessment based on known
historical water level data for the targeted region suggests that the range of regional inundation depths commensurate to annual, 10-yr, 50-yr, and 100-yr average return periods, for combination with required SLR future scenarios, may bracket the water levels of concern for moderate inundation losses in transition across a threshold into severe flooding due to SLR and coastal storms effects, for baseline and life cycle no-action conditions.

Coastal storm surge and wave modeling is being conducted for the four specific SLR scenarios of SERDP Project SI-1701 to produce maps that show installations and areas experiencing over-wash with surge and/or waves, specifically where there are no flood risk reduction structures in place. The coastal storm modeling can only show surge and/or wave height for the 50 and 100 year return periods available from the FEMA modeling work. These outputs will be superimposed with the annual and 10-year precipitation hydrographs. Flood waters determined to flow into exposed areas due elevated SLR and/or coastal storm surges/waves will serve as inputs to next phases of modeling, which include: (1) surface flood water inundation modeling of infrastructure exposed to the open coast, and (2) wave run up, overtopping, and surface flood water modeling of military installations protected from the open coast (for example, by sea walls).

Flooding in urban environment is very complex and requires the simulation of flooding infrastructure (e.g., levees, pumps, canals, internal drainage structures, etc.). For the interior flood modeling approach, a dynamic, state-of-the-art surface water model will be used to estimate flood levels inside of existing flood risk reduction systems due to overtopping and rainfall. Flood water routing will be conducted to simulate how the water moves through military installations during progressively higher inundation events. This includes the simulation of surface flows in two dimensions, the determination of water surface elevation as well as water velocity over time and the ability to simulate pumping stations and the incorporation of pressurized pipe network flow and open channel canal flow. The land surface will be divided into equal area intervals (or rasters) that have consistent hydrologic parameters. A raster based model of the urban environment in military
installation areas will be developed that factors in urban infrastructure such as drainage ditches, canals and storm drainage networks. This model will utilize any high resolution (<30m) digital elevation data available for the site. This model will be used to simulate the flooding response to two storm surge/wave conditions identified during previous components of this project illustrate threshold risks for contrasting moderate and severe condition impacts. The modeling results for stage and flow at various locations will then be used in probabilistic asset/mission capability impact assessment.

A step-wise procedure will be adapted for integrating all of the hydrodynamic modeling analyses, after the methodology described in the report entitled “Elevations for Design of Hurricane Protection Levees and Structures,” prepared by the USACE, New Orleans District, dated October 9, 2007.

**Hydrodynamic Modeling Step 1: Surge Levels and Wave Characteristics.** Wind-induced setup by strong winds of coastal storms is a significant contributor to the surge level variation for coastal flooding and inundation. Intensity of wave breaking increases at higher wind speeds and surge levels to cause more waves to break in deepwater by spilling than plunging. Since nearshore waves strongly depend on storm surge levels, coupled wave-surge modeling is necessary with a combination of regional, coastal bay, and local area scales. Spectral wave models provide wave input to circulation models to calculate combined water levels and currents generated by winds and waves. Computation for the surge levels and the wave characteristics will be executed with numerical hydrodynamic models. The Advanced Circulation (ADCIRC) model is being used to compute surge levels and Wave Analysis Model (WAM) / Spectral Wave (STWAVE) model will be used to calculate wave characteristics. A Boussinesq (1-D and 2-D) model will be used to calculate nearshore waves for estimate of wave run up and inundation lines at the military installation. Sea level rise is included in both deepwater and nearshore numerical modeling.

A set of hurricane conditions is being evaluated with the modeling suite ADCIRC / STWAVE for existing conditions. The modeled storms are different in terms of the hurricane tracks, minimum pressure, and radius, among others. Historic hurricanes hind-casts will be performed to validate the
ADCIRC / STWAVE grids to measured events. Existing conditions will be represented with available bathymetric and topographic contours, coastal wetlands, barrier islands, shorelines, as well as the built water and land side settings. Coastal storm modeling computations include evaluation of the future effects of sea level rise, coastal land subsidence, and coastal geomorphologic evolution rates.

**Hydrodynamic Modeling Step 2: Frequency Analysis.** Based on the results from ADCIRC and STWAVE in Step 1, a frequency analysis will be performed to integrate the surge levels and wave characteristics spatially across the study area for the specified range of average return periods. The method adopted for the frequency analysis is a modification to the Joint Probability Method with Optimal Sampling (JPM-OS), which takes into account the joint probability of forward speed, size, minimum pressure, angle of approach, and geographic distribution of the coastal storms (Resio, et. al., 2007). Characteristic probabilities of forward speed, minimum pressure, etc. will be based upon frequency analyses of historical storms affecting the study area. Based on this general approach, for the purposes of this research, a demonstrative sampling of storms will be used for statistical and probabilistic modeling to establish the frequency curves for surge and waves. The levels of confidence in predicted water level for a given frequency of storm will be set at the 10%, 50% and 90% and achieved statistically.

The frequency analysis will result in development of stage frequency outputs for the exterior areas, i.e. the areas that are not protected by any existing flood risk reduction systems. In addition, this analysis will provide surge levels and the wave characteristics for different average return periods along any existing flood risk reduction systems as needed as inputs for computing overtopping volumes in Hydrodynamic Modeling Step 3.

**Hydrodynamic Modeling Step 3: Coastal Structure Overtopping Volumes.** For enabling the assessment of adverse impacts to military installation assets and capabilities, existing conditions will be evaluated to identify evolving risk levels commensurate to storm event average return frequencies,
as sea levels rise. For existing flood risk reduction systems in place, the overtopping volumes will be computed for the specified average return periods of the outside surge level and wave characteristics. This procedure will be applied as follows:

- Obtain the surge level and wave characteristics from hydrodynamic modeling for existing conditions as sea levels change.
- Determine the overtopping rate using empirical formulations. A Monte Carlo Simulation will be adopted to compute the uncertainty in the overtopping rate given the engineering uncertainties in the hydraulic boundary conditions and the empirical coefficients in the overtopping formulations.
- Determine whether the overtopping rate is less than 0.1 cubic feet per second per foot with a 90 percent confidence level as a decision rule on whether the interior flood routing model will involve flood water inundation from surge and waves, or exclusively from rainfall.

The overtopping volumes will be computed using information on the surge level hydrographs from ADCIRC / STWAVE outputs. Based on a statistical analysis, a correlation will be established between the duration of the surge and the maximum surge level. This correlation will be applied to compute the overtopping rate during the storm assuming that the wave characteristics are constant around the peak of the storm.

**Hydrodynamic Modeling Step 4: Interior Stage Frequency.** The final step will be to determine the interior stage frequency for military installation subunits, which will be delineated spatially within the study area according to geographical and hydrological characteristics.

Flood stages in interior areas will be simulated using the Gridded Surface and Subsurface Hydrologic Analysis (GSSHA) Model (Downer and Ogden, 2004; Downer, Ogden, Niedzialek, and Liu. 2006). GSSHA is applicable to urban, agrarian and riparian hydrologic studies. GSSHA is a state-of-the-art raster-based model in which the land surface is divided into equal area intervals (i.e., “rasters”) that have consistent hydrologic parameters. A raster-based GSSHA model of the urban environment in the military installation sites selected by SERDP Program Managers during Phase II of
this study will be developed that factors in urban infrastructure such as drainage ditches, canals, waterways, and storm drainage networks. This model will utilize any existing high-resolution digital elevation data available for the site, including Light Detection and Ranging (LIDAR) data. The interior stage frequency will be simulated by the GSSHA model using the sum of the overtopping volume from Hydrodynamic Modeling Step 3 together with the 1- and 10-year rainfalls in the subunits. The effect of forced drainage pumping will be taken into account as applicable for existing conditions.

Hydrodynamic modeling outputs will be used to determine the probability of damage inside and outside of existing flood risk reduction systems, as well as for conditions with no flood risk reduction systems in place. Outputs of the hydrodynamic modeling process will be used to develop metrics for performance assessment of existing military installation assets and mission capabilities. For example, storm-stage frequencies (the percent chance that a specific inundation level is expected to occur for a given average return period) in combination with stage-damage relationships (damage expected for a given inundation level), will be used to estimate residual damages, which is a monetary damage metric.
APPENDIX B. TIME SERIES DATA RECORDS.

Figure B-1. Deep water wave time series record, Test A3F9T6H2.

Figure B-2. Wave run up time series record at STA 3, Test A3F9T6H2.

Figure B-3. Piezometer time series record at STA 3, Test A3F9T6H2.
Figure B-4. Wave run up time series record at STA 4, Test A3F9T6H2.

Figure B-5. Piezometer time series record at STA 4, Test A3F9T6H2.

Figure B-6. ACM displacement time series record at mid-point between STAs 3 and 4, Test A3F9T6H2.
Figure B-7a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A3F9T6H2.

Figure B-7b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A3F9T6H2.

Figure B-8. Wave run up time series record at STA 5, Test A3F9T6H2.
Figure B-9. Piezometer time series record at STA 5, Test A3F9T6H2.

Figure B-10. ACM displacement time series record at mid-point between STAs 4 and 5, Test A3F9T6H2.

Figure B-11a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A3F9T6H2.
Figure B-11b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A3F9T6H2.

Figure B-12. Deep water wave time series record, Test A3F12T9H2.

Figure B-13. Wave run up time series record at STA 3, Test A3F12T9H2.
Figure B-14. Piezometer time series record at STA 3, Test A3F12T9H2.

Figure B-15. Wave run up time series record at STA 4, Test A3F12T9H2.

Figure B-16. Piezometer time series record at STA 4, Test A3F12T9H2.
Figure B-17. ACM displacement time series record at mid-point between STAs 3 and 4, Test A3F12T9H2.

Figure B-18a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A3F12T9H2.

Figure B-18b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A3F12T9H2.
Figure B-19. Wave run up time series record at Station 5, Test A3F12T9H2.

Figure B-20. Piezometer time series record at STA 5, Test A3F12T9H2.

Figure B-21. ACM displacement time series record at mid-point between STAs 4 and 5, Test A3F12T9H2.
Figure B-22a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A3F12T9H2.

Figure B-22b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A3F12T9H2.

Figure B-23. Deep water wave time series record, Test A6F9T3H4.
Figure B-24. Wave run up time series record at STA 3, Test A6F9T3H4.

Figure B-25. Piezometer time series record at STA 3, Test A6F9T3H4.

Figure B-26. Wave run up time series record at STA 4, Test A6F9T3H4.
Figure B-27. Piezometer time series record at STA 4, Test A6F9T3H4.

Figure B-28. ACM displacement time series record at mid-point between STAs 3 and 4, Test A6F9T3H4.

Figure B-29a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A6F9T3H4.
Figure B-29b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A6F9T3H4.

Figure B-30. Wave run up time series record at STA 5, Test A6F9T3H4.

Figure B-31. Piezometer time series record at STA 5, Test A6F9T3H4.
Figure B-32. ACM displacement time series record at mid-point between STAs 4 and 5, Test A6F9T3H4.

Figure B-33a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A6F9T3H4.

Figure B-33b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A6F9T3H4.
Figure B-34. Deep water wave time series record, Test A6F9T6H2.

Figure B-35. Wave run up time series record at STA 3, Test A6F9T6H2.

Figure B-36. Piezometer time series record at STA 3, Test A6F9T6H2.
Figure B-37. Wave run up time series record at STA 4, Test A6F9T6H2.

Figure B-38. Piezometer time series record at STA 4, Test A6F9T6H2.

Figure B-39. ACM displacement time series record at mid-point between STAs 3 and 4, Test A6F9T6H2.
Figure B-40a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A6F9T6H2.

Figure B-40b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A6F9T6H2.

Figure B-41. Wave run up time series record at STA 5, Test A6F9T6H2.
Figure B-42. Piezometer time series record at STA 5, Test A6F9T6H2.

Figure B-43. ACM displacement time series record at mid-point between STAs 4 and 5, Test A6F9T6H2.

Figure B-44a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A6F9T6H2.
Figure B-44b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A6F9T6H2.

Figure B-45. Deep water wave time series record, STAs 4 and 5, Test A6F9T9H1.

Figure B-46. Wave run up time series record at STA 3, Test A6F9T9H1.
Figure B-47. Piezometer time series record at STA 3, Test A6F9T9H1.

Figure B-48. Wave run up time series record at STA 4, Test A6F9T9H1.

Figure B-49. Piezometer time series record at STA 4, Test A6F9T9H1.
Figure B-50. ACM displacement time series record at mid-point between STAs 3 and 4, Test A6F9T9H1.

Figure B-51a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A6F9T9H1.

Figure B-51b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A6F9T9H1.
Figure B-52. Wave run up time series record at STA 5, Test A6F9T9H1.

Figure B-53. Piezometer time series record at STA 5, Test A6F9T9H1.

Figure B-54. ACM displacement time series record at mid-point between STAs 4 and 5, Test A6F9T9H1.
Figure B-55a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A6F9T9H1.

Figure B-55b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A6F9T9H1.

Figure B-56. Deep water wave time series record, Test A6F12T6H3.
Figure B-57. Wave run up time series record at STA 3, Test A6F12T6H3.

Figure B-58. Piezometer time series record at STA 3, Test A6F12T6H3.

Figure B-59. Wave run up time series record at STA 4, Test A6F12T6H3.
Figure B-60. Piezometer time series record at STA 4, Test A6F12T6H3.

Figure B-61. ACM displacement time series record at mid-point between STAs 3 and 4, Test A6F12T6H3.

Figure B-62a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A6F12T6H3.
Figure B-62b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A6F12T6H3.

Figure B-63. Wave run up time series record at STA 5, Test A6F12T6H3.

Figure B-64. Piezometer time series record at STA 5, Test A6F12T6H3.
Figure B-65. ACM displacement time series record at mid-point between STAs 4 and 5, Test A6F12T6H3.

Figure B-66a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A6F12T6H3.

Figure B-66b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A6F12T6H3.
Figure B-78. Deep water wave time series record, Test A6F12T9H2.

Figure B-79. Wave run up time series record at STA 3, Test A6F12T9H2.

Figure B-80. Piezometer time series record at STA 3, Test A6F12T9H2.
Figure B-81. Wave run up time series record at STA 4, Test A6F12T9H2.

Figure B-82. Piezometer time series record at STA 4, Test A6F12T9H2.

Figure B-83. ACM displacement time series record at mid-point between STAs 3 and 4, Test A6F12T9H2.
Figure B-84a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A6F12T9H2.

Figure B-84b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A6F12T9H2.

Figure B-85. Wave run up time series record at STA 5, Test A6F12T9H2.
Figure B-86. Piezometer time series record at STA 5, Test A6F12T9H2.

Figure B-87. ACM displacement time series record at mid-point between STAs 4 and 5, Test A6F12T9H2.

Figure B-88a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A6F12T9H2.
Figure B-88b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A6F12T9H2.

Figure B-89. Deep water wave time series record, Test A9F9T3H4.

Figure B-90. Wave run up time series record at STA 3, Test A9F9T3H4.
Figure B-91. Piezometer time series record at STA 3, Test A9F9T3H4.

Figure B-92. Wave run up time series record at STA 4, Test A9F9T3H4.

Figure B-93. Piezometer time series record at STA 4, Test A9F9T3H4.
Figure B-94. ACM displacement time series record at mid-point between STAs 3 and 4, Test A9F9T3H4.

Figure B-95a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A9F9T3H4.

Figure B-95b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A9F9T3H4.
Figure B-96. Deep water wave time series record, Test A9F9T6H4.

Figure B-97. Wave run up time series record at STA 3, Test A9F9T6H4.

Figure B-98. Piezometer time series record at STA 3, Test A9F9T6H4.
Figure B-99. Wave run up time series record at STA 4, Test A9F9T6H4.

Figure B-100. Piezometer time series record at STA 4, Test A9F9T6H4.

Figure B-101. ACM displacement time series record at mid-point between STAs 3 and 4, Test A9F9T6H4.
Figure B-102a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A9F9T6H4.

Figure B-102b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A9F9T6H4.

Figure B-103. Wave run up time series record at STA 5, Test A9F9T6H4.
Figure B-104. Piezometer time series record at STA 5, Test A9F9T6H4.

Figure B-105. ACM displacement time series record at mid-point between STAs 4 and 5, Test A9F9T6H4.

Figure B-106a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A9F9T6H4.
Figure B-106b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A9F9T6H4.

Figure B-107. Deep water wave time series record, Test A9F9T9H3.

Figure B-108. Wave run up time series record at STA 3, Test A9F9T9H3.
Figure B-109. Piezometer time series record at STA 3, Test A9F9T9H3.

Figure B-110. Wave run up time series record at STA 4, Test A9F9T9H3.

Figure B-111. Piezometer time series record at STA 4, Test A9F9T9H3.
Figure B-112. ACM displacement time series record at mid-point between STAs 3 and 4, Test A9F9T9H3.

Figure B-113a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A9F9T9H3.

Figure B-113b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A9F9T9H3.
Figure B-114. Wave run up time series record at STA 5, Test A9F9T9H3.

Figure B-115. Piezometer time series record at STA 5, Test A9F9T9H3.

Figure B-116. ACM displacement time series record at mid-point between STAs 4 and 5, Test A9F9T9H3.
Figure B-117a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A9F9T9H3.

Figure B-117b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A9F9T9H3.
Figure B-118. Deep water wave time series record, Test A9F12T6H4.

Figure B-119. Wave run up time series record at STA 3, Test A9F12T6H4.

Figure B-120. Piezometer time series record at STA 3, Test A9F12T6H4.
Figure B-121. Wave run up time series record at STA 4, Test A9F12T6H4.

Figure B-122. Piezometer time series record at STA 4, Test A9F12T6H4.

Figure B-123. ACM displacement time series record at mid-point between STAs 3 and 4, Test A9F12T6H4.
Figure B-124a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A9F12T6H4.

Figure B-124b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A9F12T6H4.

Figure B-125. Wave run up time series record at STA 5, Test A9F12T6H4.
Figure B-126. Piezometer time series record at STA 5, Test A9F12T6H4.

Figure B-127. ACM displacement time series record at mid-point between STAs 4 and 5, Test A9F12T6H4.

Figure B-128a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A9F12T6H4.
Figure B-128b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 4 and 5, Test A9F12T6H4.

Figure B-129. Deep water wave time series record, Test A9F12T9H3.

Figure B-130. Wave run up time series record at STA 3, Test A9F12T9H3.
Figure B-131. Piezometer time series record at STA 3, Test A9F12T9H3.

Figure B-132. Wave run up time series record at STA 4, Test A9F12T9H3.

Figure B-133. Piezometer time series record at STA 4, Test A9F12T9H3.
Figure B-134. ACM displacement time series record at mid-point between STAs 3 and 4, Test A9F12T9H3.

Figure B-135a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A9F12T9H3.

Figure B-135b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STAs 3 and 4, Test A9F12T9H3.
Figure B-136. Wave run up time series record at STA 5, Test A9F12T9H3.

Figure B-137. Piezometer time series record at STA 5, Test A9F12T9H3.

Figure B-138. ACM displacement time series record at mid-point between STA s 4 and 5, Test A9F12T9H3.
Figure B-139a. Test time interval for data processed using relative instantaneous run up and hydromechanics relationships, STA s 4 and 5, Test A9F12T9H3.

Figure B-139b. Close up sample time interval for data processed using relative instantaneous run up and hydromechanics relationships, STA s 4 and 5, Test A9F12T9H3.
APPENDIX C. TIME SERIES DATA STATISTICS.

Figure C-1. Deep water wave time series PDF, Test A3F9T6H2.

Figure C-2. Run up wave time series PDF at STA 3, Test A3F9T6H2.

Figure C-3. Piezometer time series PDF at STA 3, Test A3F9T6H2.
Figure C-4. Run up wave time series PDF at STA 4, Test A3F9T6H2.

Figure C-5. Piezometer time series PDF at STA 4, Test A3F9T6H2.

Figure C-6. Run up wave time series PDF at STA 5, Test A3F9T6H2.
Figure C-7. Piezometer time series PDF at STA 5, Test A3F9T6H2.

Figure C-8. Deep water wave time series PDF, Test A3F12T9H2.

Figure C-9. Run up wave time series PDF at STA 3, Test A3F12T9H2.
Figure C-10. Piezometer time series PDF at STA 3, Test A3F12T9H2.

Figure C-11. Run up wave time series PDF at STA 4, Test A3F12T9H2.

Figure C-12. Piezometer time series PDF at STA 4, Test A3F12T9H2.
Figure C-13. Run up wave time series PDF at STA 5, Test A3F12T9H2.

Figure C-14. Piezometer time series PDF at STA 5, Test A3F12T9H2.

Figure C-15. Deep water wave time series PDF, Test A3F12T9H2.
Figure C-16. Run up wave time series PDF at STA 3, Test A3F12T9H2.

Figure C-17. Piezometer time series PDF at STA 3, Test A3F12T9H2.

Figure C-18. Run up wave time series PDF at STA 4, Test A3F12T9H2.
Figure C-19. Piezometer time series PDF at STA 4, Test A3F12T9H2.

Figure C-20. Run up wave time series PDF at STA 5, Test A3F12T9H2.

Figure C-21. Piezometer time series PDF at STA 5, Test A3F12T9H2.
Figure C-22. Deep water wave time series PDF, Test A6F9T3H4.

Figure C-23. Run up wave time series PDF at STA 3, Test A6F9T3H4.

Figure C-24. Piezometer time series PDF at STA 3, Test A6F9T3H4.
Figure C-25. Run up wave time series PDF at STA 4, Test A6F9T3H4.

Figure C-26. Piezometer time series PDF at STA 4, Test A6F9T3H4.

Figure C-27. Run up wave time series PDF at STA 5, Test A6F9T3H4.
Figure C-28. Piezometer time series PDF at STA 5, Test A6F9T3H4.

Figure C-29. Deep water wave time series PDF, Test A6F9T6H2.

Figure C-30. Run up wave time series PDF at STA 3, Test A6F9T6H2.
Figure C-31. Piezometer time series PDF at STA 3, Test A6F9T6H2.

Figure C-32. Run up wave time series PDF at STA 4, Test A6F9T6H2.

Figure C-33. Piezometer time series PDF at STA 4, Test A6F9T6H2.
Figure C-34. Run up wave time series PDF at STA 5, Test A6F9T6H2.

Figure C-35. Piezometer time series PDF at STA 5, Test A6F9T6H2.

Figure C-36. Deep water wave time series PDF, Test A6F9T9H1.
Figure C-37. Run up wave time series PDF at STA 3, Test A6F9T9H1.

Figure C-38. Piezometer time series PDF at STA 3, Test A6F9T9H1.

Figure C-39. Run up wave time series PDF at STA 4, Test A6F9T9H1.
Figure C-40. Piezometer time series PDF at STA 4, Test A6F9T9H1.

Figure C-41. Run up wave time series PDF at STA 5, Test A6F9T9H1.

Figure C-42. Piezometer time series PDF at STA 5, Test A6F9T9H1.
Figure C-43. Deep water wave time series PDF, Test A6F12T6H3.

Figure C-44. Run up wave time series PDF at STA 3, Test A6F12T6H3.

Figure C-45. Piezometer time series PDF at STA 3, Test A6F12T6H3.
Figure C-46. Run up wave time series PDF at STA 4, Test A6F12T6H3.

Figure C-47. Piezometer time series PDF at STA 4, Test A6F12T6H3.

Figure C-48. Run up wave time series PDF at STA 5, Test A6F12T6H3.
Figure C-49. Piezometer time series PDF at STA 5, Test A6F12T6H3.

Figure C-50. Deep water wave time series PDF, Test A6F12T6H3.

Figure C-51. Run up wave time series PDF at STA 3, Test A6F12T6H3.
Figure C-52. Piezometer time series PDF at STA 3, Test A6F12T6H3.

Figure C-53. Run up wave time series PDF at STA 4, Test A6F12T6H3.

Figure C-54. Piezometer time series PDF at STA 4, Test A6F12T6H3.
Figure C-55. Run up wave time series PDF at STA 5, Test A6F12T6H3.

Figure C-56. Piezometer time series PDF at STA 5, Test A6F12T6H3.

Figure C-57. Deep water wave time series PDF, Test A6F12T9H2.
Figure C-58. Run up wave time series PDF at STA 3, Test A6F12T9H2.

Figure C-59. Piezometer time series PDF at STA 3, Test A6F12T9H2.

Figure C-60. Run up wave time series PDF at STA 4, Test A6F12T9H2.
Figure C-61. Piezometer time series PDF at STA 4, Test A6F12T9H2.

Figure C-62. Run up wave time series PDF at STA 5, Test A6F12T9H2.

Figure C-63. Piezometer time series PDF at STA 5, Test A6F12T9H2.
Figure C-64. Deep water wave time series PDF, Test A9F9T3H4.

Figure C-65. Run up wave time series PDF at STA 3, Test A9F9T3H4.

Figure C-66. Piezometer time series PDF at STA 3, Test A9F9T3H4.
Figure C-67. Run up wave time series PDF at STA 4, Test A9F9T3H4.

Figure C-68. Piezometer time series PDF at STA 4, Test A9F9T3H4.

Figure C-69. Deep water wave time series PDF, Test A9F9T6H4.
Figure C-70. Run up wave time series PDF at STA 3, Test A9F9T6H4.

Figure C-71. Piezometer time series PDF at STA 3, Test A9F9T6H4.

Figure C-72. Run up wave time series PDF at STA 4, Test A9F9T6H4.
Figure C-73. Piezometer time series PDF at STA 4, Test A9F9T6H4.

Figure C-74. Run up wave time series PDF at STA 5, Test A9F9T6H4.

Figure C-75. Piezometer time series PDF at STA 5, Test A9F9T6H4.
Figure C-76. Deep water wave time series PDF, Test A9F9T9H3.

Figure C-77. Run up wave time series PDF at STA 3, Test A9F9T9H3.

Figure C-78. Piezometer time series PDF at STA 3, Test A9F9T9H3.
Figure C-79. Run up wave time series PDF at STA 4, Test A9F9T9H3.

Figure C-80. Piezometer time series PDF at STA 4, Test A9F9T9H3.

Figure C-81. Run up wave time series PDF at STA 5, Test A9F9T9H3.
Figure C-82. Piezometer time series PDF at STA 5, Test A9F9T9H3.

Figure C-83. Deep water wave time series PDF, Test A9F12T6H4.

Figure C-84. Run up wave time series PDF at STA 3, Test A9F12T6H4.
Figure C-85. Piezometer time series PDF at STA 3, Test A9F12T6H4.

Figure C-86. Run up wave time series PDF at STA 4, Test A9F12T6H4.

Figure C-87. Piezometer time series PDF at STA 4, Test A9F12T6H4.
Figure C-88. Run up wave time series PDF at STA 5, Test A9F12T6H4.

Figure C-89. Piezometer time series PDF at STA 5, Test A9F12T6H4.

Figure C-90. Deep water wave time series PDF, Test A9F12T9H3.
Figure C-91. Run up wave time series PDF at STA 3, Test A9F12T9H3.

Figure C-92. Piezometer time series PDF at STA 3, Test A9F12T9H3.

Figure C-93. Run up wave time series PDF at STA 4, Test A9F12T9H3.
Figure C-94. Piezometer time series PDF at STA 4, Test A9F12T9H3.

Figure C-95. Run up wave time series PDF at STA 5, Test A9F12T9H3.

Figure C-96. Piezometer time series PDF at STA 5, Test A9F12T9H3.
APPENDIX D

SPECTRAL HYDROMECHANICS ANALYSES
Figure D-1. Variance density auto spectra, STA 3-4, Test A3F9T6H2.
Figure D-2. Cross spectra, squared coherency, and phase function, STA 3-4, Test A3F9T6H2.
Figure D-3. Variance density auto spectra, STA 4-5, Test A3F9T6H2.
Figure D-4. Cross spectra, squared coherency, and phase function, STA 4-5, Test A3F9T6H2.
Figure D-5. Variance density auto spectra, STA 3-4, Test A3F12T9H2.
Figure D-6. Cross spectra, squared coherency, and phase function, STA 3-4, Test A3F12T9H2.
Figure D-7. Variance density auto spectra, STA 4-5, Test A3F12T9H2.
Figure D-8. Cross spectra, squared coherency, and phase function, STA 4-5, Test A3F12T9H2.
Figure D-9. Variance density auto spectra, STA 3-4, Test A6F9T3H4.
Figure D-10. Cross spectra, squared coherency, and phase function, STA 3-4, Test A6F9T3H4.
Figure D-11. Variance density auto spectra, STA 4-5, Test A6F9T3H4.
Figure D-12. Cross spectra, squared coherency, and phase function, STA 4-5, Test A6F9T3H4.
Figure D-13. Variance density auto spectra, STA 3-4, Test A6F9T6H2.
Figure D-14. Cross spectra, squared coherency, and phase function, STA 3-4, Test A6F9T6H2.
Figure D-15. Variance density auto spectra, STA 4-5, Test A6F9T6H2.
Figure D-16. Cross spectra, squared coherency, and phase function, STA 4-5, Test A6F9T6H2.
Figure D-17. Variance density auto spectra, STA 3-4, Test A6F9T9H1.
Figure D-18. Cross spectra, squared coherency, and phase function, STA 3-4, Test A6F9T9H1.
Figure D-19. Variance density auto spectra, STA 4-5, Test A6F9T9H1.
Figure D-20. Cross spectra, squared coherency, and phase function, STA 4-5, Test A6F9T9H1.
Figure D-21. Variance density auto spectra, STA 3-4, Test A6F12T6H3.
Figure D-22. Cross spectra, squared coherency, and phase function, STA 3-4, Test A6F12T6H3.
Figure D-23. Variance density auto spectra, STA 4-5, Test A6F12T6H3.
Figure D-24. Cross spectra, squared coherency, and phase function, STA 4-5, Test A6F12T6H3.
Figure D-25. Variance density auto spectra, STA 3-4, Test A6F12T9H2.
Figure D-26. Cross spectra, squared coherency, and phase function, STA 3-4, Test A6F12T9H2.
Figure D-27. Variance density auto spectra, STA 4-5, Test A6F12T9H2.
Figure D-28. Cross spectra, squared coherency, and phase function, STA 4-5, Test A6F12T9H2.
Figure D-29. Variance density auto spectra, STA 3-4, Test A9F9T3H4.
Figure D-30. Cross spectra, squared coherency, and phase function, STA 3-4, Test A9F9T3H4.
Figure D-31. Variance density auto spectra, STA 3-4, Test A9F9T6H4.
Figure D-32. Cross spectra, squared coherency, and phase function, STA 3-4, Test A9F9T6H4.
Figure D-33. Variance density auto spectra, STA 4-5, Test A9F9T6H4.
Figure D-34. Cross spectra, squared coherency, and phase function, STA 4-5, Test A9F9T6H4.

\[ \sigma_c = 0.077 \]
Figure D-35. Variance density auto spectra, STA 3-4, Test A9F9T9H3.
Figure D-36. Cross spectra, squared coherency, and phase function, STA 3-4, Test A9F9T9H3.
Figure D-37. Variance density auto spectra, STA 4-5, Test A9F9T9H3.
Figure D-38. Cross spectra, squared coherency, and phase function, STA 4-5, Test A9F9T9H3.
Figure D-39. Variance density auto spectra, STA 3-4, Test A9F12T6H4.
Figure D-40. Cross spectra, squared coherency, and phase function, STA 3-4, Test A9F12T6H4.
Figure D-41. Variance density auto spectra, STA 4-5, Test A9F12T6H4.
Figure D-42. Cross spectra, squared coherency, and phase function, STA 4-5, Test A9F12T6H4.
Figure D-43. Variance density auto spectra, STA 3-4, Test A9F12T9H3.
Figure D-44. Cross spectra, squared coherency, and phase function, STA 3-4, Test A9F12T9H3.
Figure D-45. Variance density auto spectra, STA 4-5, Test A9F12T9H3.
Figure D-46. Cross spectra, squared coherency, and phase function, STA 4-5, Test A9F12T9H3.
Mr. Edmond J. Russo, Jr., is currently Chief, Ecosystem Evaluation and Engineering Division, Environmental Laboratory, at the US Army Engineer Research and Development Center (ERDC), Vicksburg, Mississippi. From 2005-2009, he served as Chief, Coastal Engineering Branch, Coastal and Hydraulics Laboratory, ERDC. On special assignment following the 2005 Atlantic hurricane season, he served as Project Manager for the Louisiana Coastal Protection and Restoration (LACPR) Project, completing this role in December 2007. In this role, Mr. Russo led technical exchanges held in the United States and The Netherlands with Rijkswaterstaat, The Delft University, and Dutch consultants, on developing and implementing a civil works approach for multi-objective systems-scale life cycle coastal engineering risk assessment and management. He is currently the Principal Investigator for Risk Quantification for Sustaining Coastal Military Installation Assets and Mission Capabilities, Project Number SI-1701, under the Strategic Environmental Research and Development Program (SERDP), advancing research on coastal engineering risk assessment and management for military installations.

Formerly, Mr. Russo performed management and engineering work on navigation and ecosystem restoration projects and studies at US Army Corps of Engineers, New Orleans District, New Orleans, Louisiana, in the planning, project management, and operations arenas, between 1992-2005. Mr. Russo has led, managed, and been a team member on over $100 million in completed projects during this time. During 1991-1992, he was employed in New Orleans by Fugro-McClelland (Southeast, Inc.), as a Project Engineer on geotechnical and environmental projects. From 1985-1990, Mr. Russo
was a civil engineering technician in the Pile Driving Department of Boh Bros. Construction Company in New Orleans.

In his career, Mr. Russo has appeared in front of the following bodies to present and testify as a subject matter expert on water resources projects: Policy Director, Office of the President of the United Stated of America; The President’s Gulf Coast Rebuilding Office; Office of Management and Budget; Members of the Louisiana Delegation, Members of the Authorization Committee, and Members of the Appropriations Committee, United States Congress; Assistant Secretary of the Army for Civil Works; Commanding General for Civil Works, USACE Headquarters, Commanding General, Mississippi Valley Division, and Commander, New Orleans District; USACE Mississippi River Commission; USACE Coastal Engineering Research Board; USACE Environmental Advisory Board; United States District Court, Eastern District of Louisiana; Special Hearing on USACE Activities, Louisiana Legislature; Members, Louisiana Coastal Protection and Restoration Authority; Task Force, Coastal Wetlands Planning Protection and Restoration Act, and elected officials across parishes of the Louisiana Coastal Zone.

Mr. Russo is a licensed Professional Engineer in the State of Louisiana. He is Vice-Chair and Secretary for the Environmental Commission of the Permanent International Association of Navigation Congresses (PIANC). Mr. Russo is a member of the American Society of Civil Engineers (ACSE) Coasts, Oceans, Ports, and Rivers Institute (COPRI) Wetlands Restoration Committee, Sub-Committee on Beneficial Uses of Dredged Materials; Environmental Committee of the Western Dredging Association; and Committee on Marine Transportation, Interagency Action Team. He served as Chair of the Publications Committee – DePaepe-Willems Annual Award Contest, and editor of the Quarterly Bulletin for the US Section of PIANC, from 2004-2009. During 2003-2004, Mr. Russo served on the Board of Graduate Examiners, Faculty of Civil Engineering and Geoscience Civil Engineering, Delft University of Technology, The Netherlands.
A fourth-generation native of New Orleans, Mr. Russo graduated from Louisiana State University, Baton Rouge, Louisiana, in 1990 with a Bachelor of Science in Civil Engineering Degree. He received a Master of Science in Civil Engineering Degree from University of New Orleans in New Orleans during 1997. In 1999, Mr. Russo graduated from the Army Management Staff College, Washington, D.C., in the Sustaining Base Leadership and Management curriculum. From 2002-2005, he completed doctoral course work at Tulane University’s Department of Civil and Environmental Engineering. Following closure of Tulane University’s School of Engineering with the devastation of Hurricane Katrina in 2005, Mr. Russo transferred his doctoral studies to the Department of Civil and Environmental Engineering at Louisiana State University.

The degree of Doctor of Philosophy will be conferred on Mr. Russo at the December 2009 Commencement.