

COASTAL RUBBLE MOUND DESIGN AND LIFE-CYCLE PERFORMANCE EVALUATION - RECENT RESEARCH RESULTS AND THE CHS/CHESS PARADIGM

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U.S. Army Engineer Research and
Development Center, Coastal and Hydraulics
Laboratory





CONTENTS – OVERVIEW



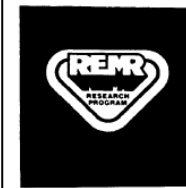
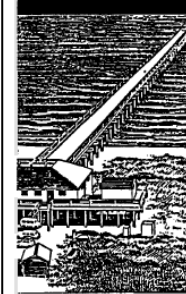
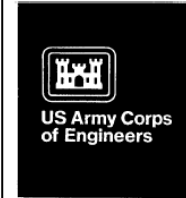
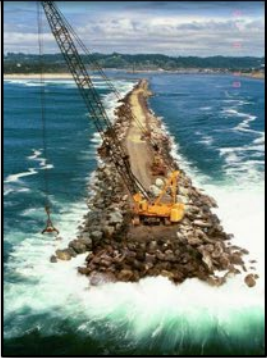
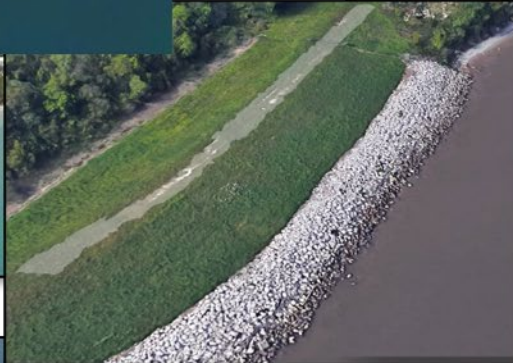
- Problem
- CHS/CHESS (Coastal Hazards, Engineering, and Structures System)
 - Runup, overtopping, wave transmission
 - Seaside stability and damage progression
 - Leese side stability and damage progression
 - Low-Crested and submerged stability and damage
- Present work



USACE COASTAL RUBBLE MOUND INTEREST



USACE manages over 1000 coastal rubble mound structures (primarily breakwaters & jetties).



REPAIR, EVALUATION, MAINTENANCE AND
REHABILITATION RESEARCH PROGRAM

TECHNICAL REPORT REMR-CO-3

CASE HISTORIES OF CORPS BREAKWATER
AND JETTY STRUCTURES

Report 2
SOUTH ATLANTIC DIVISION

by
Francis E. Sargent
Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39181-0631



September 1988
Report 2 of a Series

Approved For Public Release; Distribution Unlimited

Prepared for DEPARTMENT OF THE ARMY
US Army Corps of Engineers
Washington, DC 20314-1000

Under Work Unit 32278 and Work Unit 31269





PROBLEM

1984 Rehab, crest el 10 ft MLLW
(8.26 ft MSL, 2.52 m), W50=12-15 t



Point Judith Breakwater

ERDC/CHL TR-15-13



US Army Corps
of Engineers®
Engineer Research and
Development Center

ERDC
INNOVATIVE SOLUTIONS
for a safer, better world

Point Judith, Rhode Island, Breakwater Risk Assessment

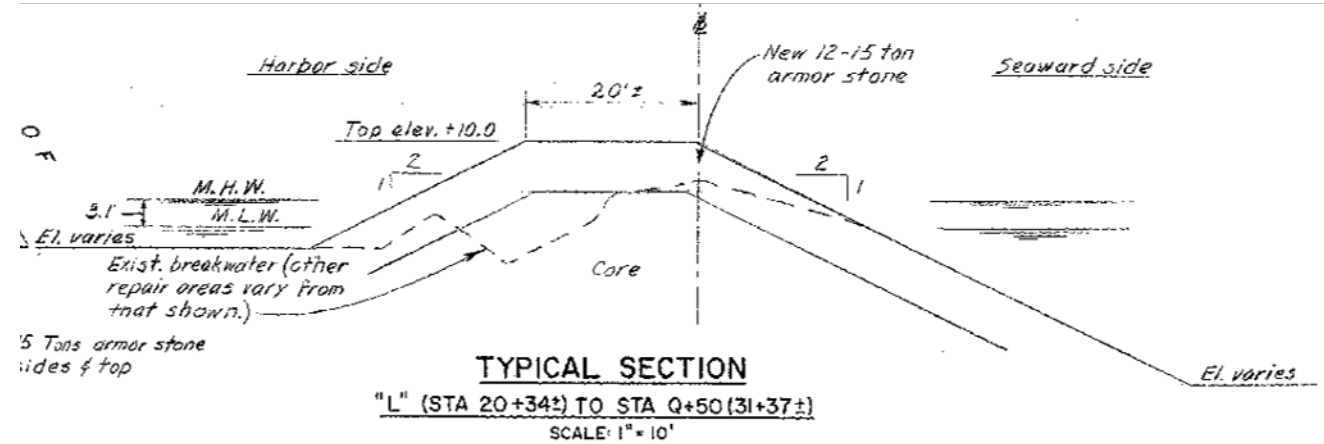
Jeffrey A. Melby, Norberto C. Nadal-Caraballo,
and John Winkelman

August 2015

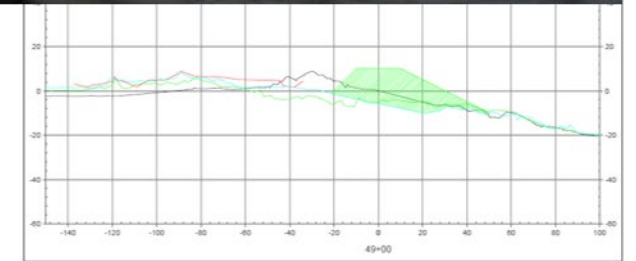
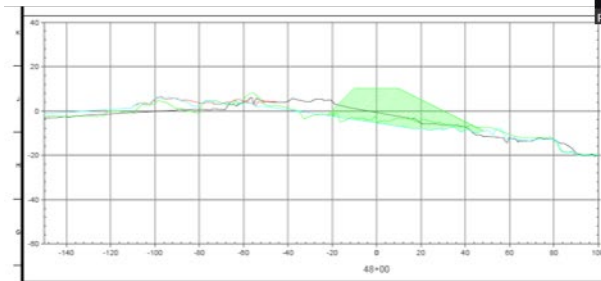


Coastal and Hydraulics Laboratory

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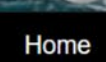


Tide range:
MHHW-MLLLW =
3.85 ft (1.2 m)





SP	Lat	Ln	Depth, m, MSL
889	41.3564	-71.5196	9.28
876	41.3507	-71.5106	9.59
874	41.3488	-71.499	10.59
877	41.3507	-71.4875	9.93
213	41.3550	-71.4948	10.14



**COASTAL HAZARDS
SYSTEM, V2.0**

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[Studies](#)
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Regional Coastal Studies

Instructions

CHS - North Atlantic Coast Comprehensive Study (CHS-NACCS)

Save Point (Model Condition/Area)

Instructions

Apply Filter

Name	Select	Results	Statistics
▼ CHS-NA (VA to ME)			
Base Conditions i	☒		3
Base Conditions, Tides i	☒		3
Base Conditions, 96 Random Tides i	☒		3
Base Conditions + SLC1 i	☒		3
Validation, Historical, Tides i	☒		3



STWAVE Grid 8

Lat/Lon 41.356 -71.520 | Scale 1:36112 | Zoom 14

889	41.356
876	41.350
874	41.348
877	41.350
213	41.355



PROBLEM – ANALYSIS OF STORM



Tide range: MHHW-MLLW = 3.85 ft (1.2 m)

Toe depth on east arm 21 – 32 ft, MSL (6.4 – 9.7 m)

Consider a storm with AEP of about 1% (100 yr ARI, 84% CL)

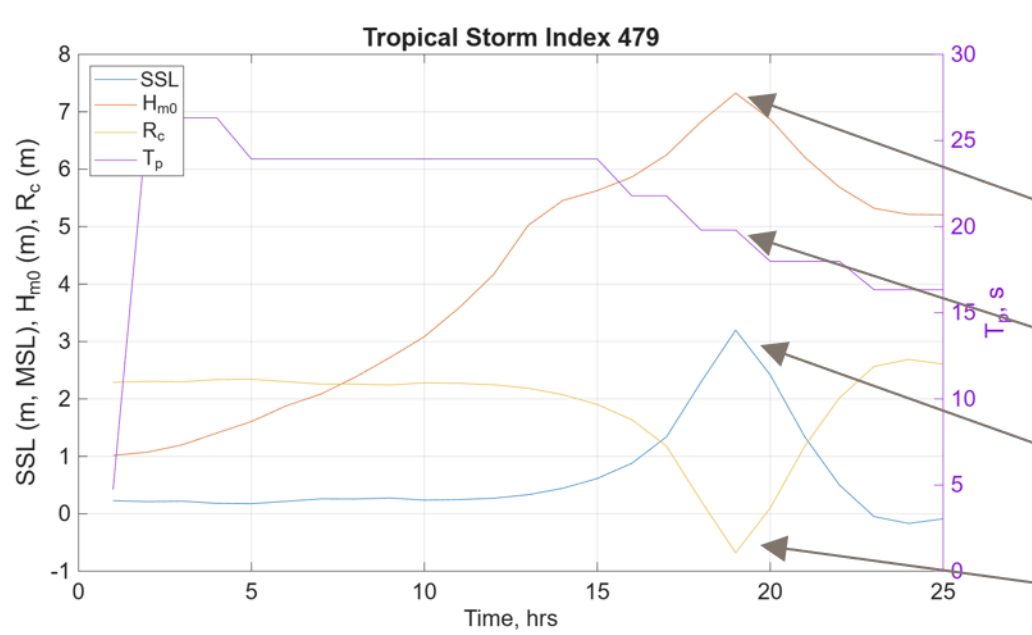
SP 213, $d=10.1$ m, MSL, SSL=2.53 m MSL, $H_{m0}=7.0$ m, $T_p=19.9$ s

(includes skew tide, conditional wave)

12 tropical storms with SSL>2.5 m.

Of those, 7 storms have $H_{m0} > 7$ m. They all look similar to the storm below.

Storm Name	Peak Date	Peak H_{m0} (m)	Peak Surge (m)
Hurricane Bob	8/19/1991	10.1	1.6
Hurricane Carol	8/31/1954	8.5	2.4
Hurricane Edna	9/11/1954	8.4	NaN
Hurricane Donna	9/12/1960	8.3	1.3
Hurricane Gloria	9/27/1985	8.3	1.3
Hurricane Esther	9/21/1961	7.8	0.7
Not Named	12/12/1992	7.5	0.9



Peak $H_{m0} \sim 7$ m

Coincident $T_p \sim 20$ s

Peak SSL ~ 3 m

Min $R_c = -0.68$ m



PROBLEM – LOW-CRESTED AND SUBMERGED RM STRUCTURE RESEARCH



1978 Dattari, J., H. Raman, N. J. Shankar. 1978. Performance Characteristics of Submerged Breakwaters. Coastal Engineering.

1983 Allsop, N. 1983. Low Crested Breakwaters, Studies in Random Waves.

1985 Powell, K. and N. W. H. Allsop. 1985. Low-crest breakwaters, hydraulic performance and stability.

1989 Ahrens 1989 reef breakwaters

1990 van der Meer, J. W. and K. W. Pilarczyk. 1990. Stability of low-crested and reef breakwaters. Coastal Engineering.

1991 Vidal, C. M. A. Losada, R. Medina. 1991. Stability of Mound Breakwater's Head and Trunk. J. Waterway, Port, Coastal, Ocean Eng. 117(6):570-587.

1992 Losada, M., N. Kobayashi, F. S. Martin. 1992. Armor Stability on Submerged Breakwaters. J. Waterway, Port, Coastal, Ocean Eng. 118(2):207-212.

1992 Vidal, C., M. A. Losada, et al. 1993. A Universal analysis for the stability of both low-crested and submerged breakwaters. Coastal Engineering.

1994 van der Meer, J. W., I. F. R. Daemen. 1994. Stability and wave transmission at low-crested rubble mound structures. J. Wtwy, Port, Coast., Oc. Eng. 120(1):1-19.

1995 Burger, G. 1995. Stability of low-crested breakwaters. MSc. Thesis Delft University, report H1878/H2415 and Final Proceedings, MAST

1996 Kudale, M. D., N. Kobayashi. 1996. Hydraulic stability analysis of leeside slopes of overtopped breakwaters. Coastal Engineering.

1996 van der Meer, J. W., W. H. Tutuarima, G. Burger. 1996. Influence of rock shape and grading on stability of low crested structures. Coastal Engineering.

1998 Vidal, C., I. J. Losada, F. Martin. 1998. Stability of near-bed rubble-mound structures. Coastal Engineering.

2000 Vidal, C., R. Medina, F. L. Martin. 2000. A methodology to assess the armor unit stability of low-crested and submerged RM breakwaters. Coastal Structures.

2003 Kramer, M., H. Burcharth. 2003. Stability of Low-Crested Breakwaters in Shallow Water Short Crested Waves. Coastal Structures.

2005 Kramer, M., B. Zanuttigh, J. W. van der Meer, C. Vidal, F. X. Gironella. 2005. Laboratory experiments on low-crested breakwaters. Coastal Engineering 52:867-885.

2005 Lamberti, A., R. Archetti, M. Kramer, et al.. 2005. European experience of low crested structures for coastal management. Coastal Engineering 52:841-866.

2005 Taveira-Pinto, F. 2005. Analysis of submerged breakwaters stability design.

2006 Kramer, M. 2006. Structural stability of low-crested breakwaters. Dept. of Civil Eng. Aalborg University. DCE Thesis No. 1.

2006 Burcharth, H. F., M. Kramer, A. Lamberti, B. Zanuttigh. 2006. Structural stability of detached low crested breakwaters. Coastal Engineering 53:381-394.

2013 Kobayashi, N., J. Pietropaolo, J. A. Melby. 2013. Deformation of Reef Breakwaters and Wave Transmission. J. Wtwy, Port, Coast., and Oc. Eng. 139(4):336-340.

2013 Kudumula, S. R., Mutukuru, M. R. G. 2013. Experimental Studies on Low crested Rubble Mound, The Intl Jour. of Oc. and Clim. Sys. Sep. 2013:213-226.

2015 Garcia, R., N. Kobayashi. 2015. Trunk and Head Damage on a Low-Crested Breakwater. J. Waterway, Port, Coastal, Ocean Eng. 141(2):04014037

2018 Argente, G., M. E. Gómez-Martín, J. R. Medina. 2018. Hydraulic Stability of the Armor Layer of Overtopped Breakwaters. J. Marine Science and Eng.

2021 Burgess, C. P. 2021. Stability of Low Crested and Submerged Breakwaters: MS Thesis, Civil & Environmental Engineering, Old Dominion University.

2021 Mares-Nassare, P., G. Argente, et al. 2021. Armor damage of overtopped mound breakwaters in J. Mar. Sci. and Eng. 9, 952.

CEM

DELOS



PROBLEM – SEASIDE AND LEESIDE

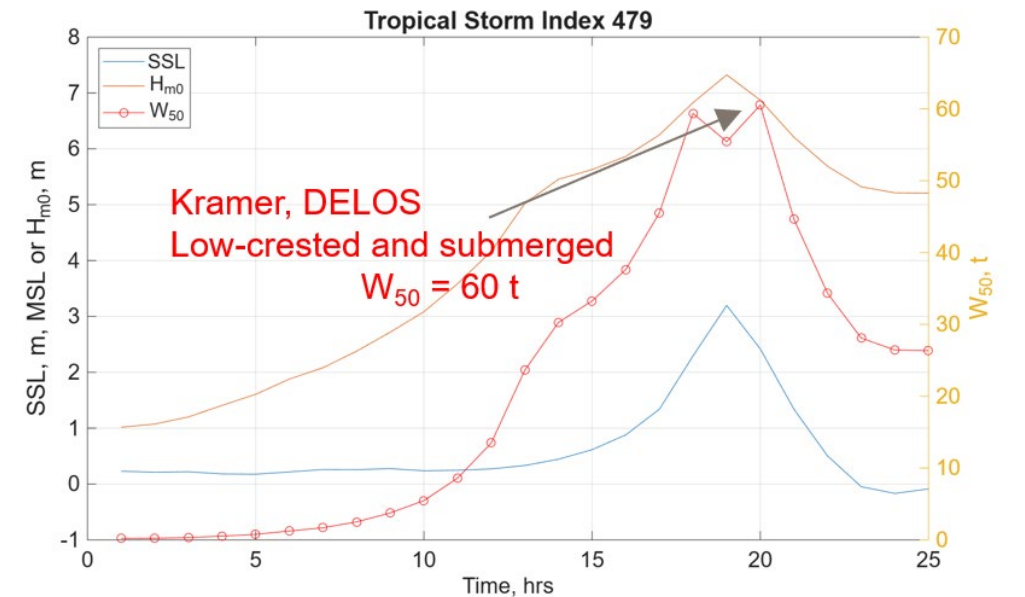
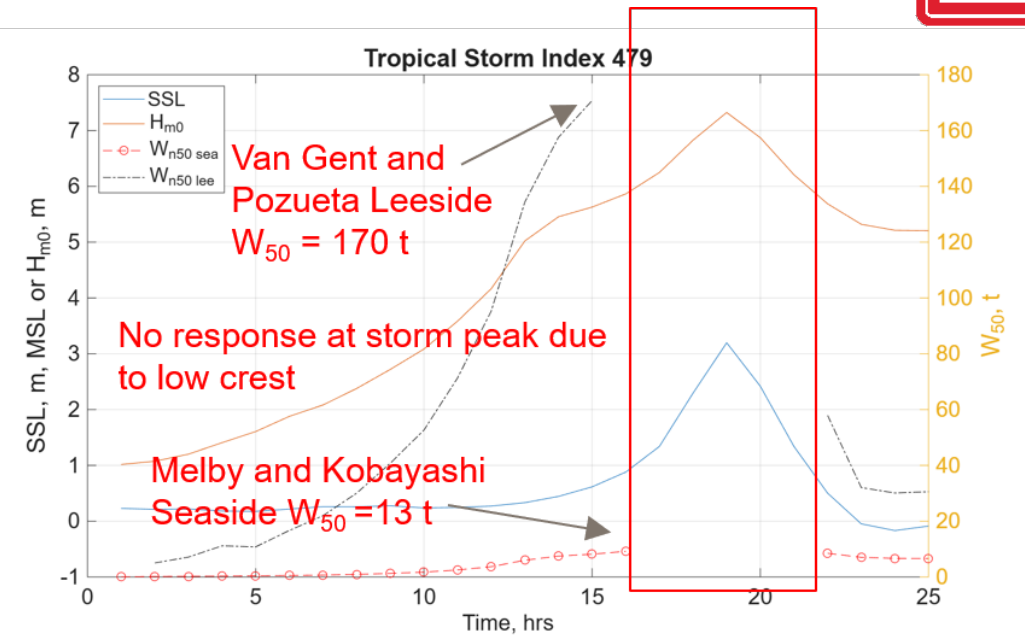
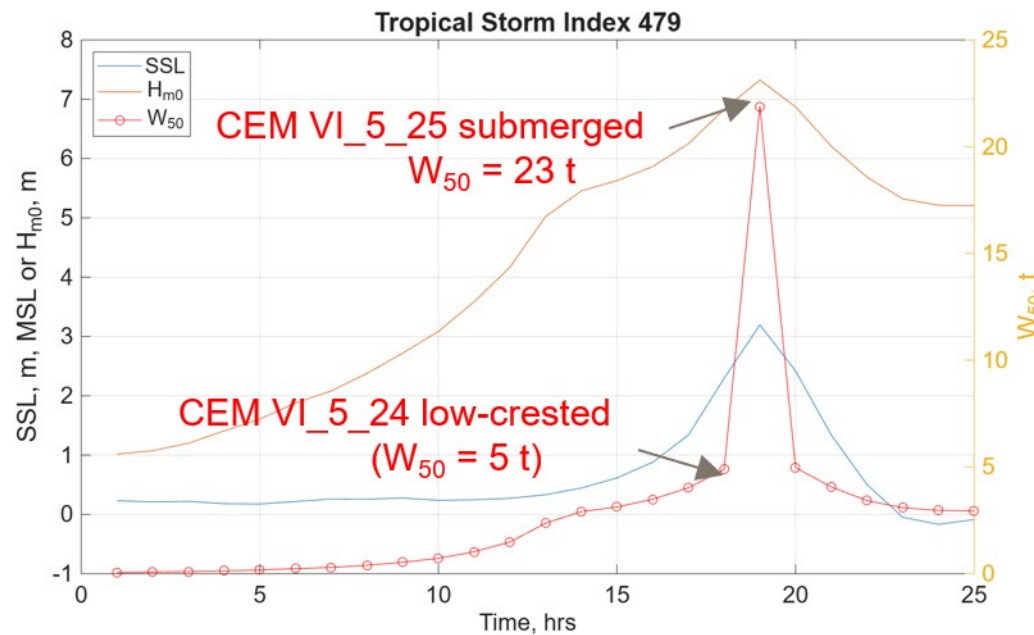


Traditional Seaside and Leeside stability equations

Seaside $W_{50} = 13$ t reasonable

Leeside $W_{50} = 170$ t !!!

Both drop at relatively low freeboards at storm peak





PROBLEM – EVALUATION



- Various available armor stability equations
 - Give wildly different answers in overlap areas
 - Equations have different context (damage initiation, mean S, duration, envelop or mean fit)
 - No good solution for low-crested and submerged structures and most USACE rubble mound structures are low-crested or submerged during design conditions
 - Low crested structures may have worst case at an odd non-intuitive parameter combination (e.g. low water level and high wave). Wave period and other correlated parameters make probabilistic response problem complex.
 - CSHORE is giving poor prediction of RM damage
-
- StormSim/CHESS computes response at every time step of every storm for both PROS and life-cycle simulation.
 - This takes care of multivariate problem
 - But the low-crest problem is unsolved. How does user know when the computation around the peak of storm is dropping out due to low crest and what can they do about it?



CONTENTS – CHS/CHESS



- Problem
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Coastal Hazards System (CHS)

What is the CHS?

A national-scale, multi-agency initiative for accurate, efficient, and consistent quantification of coastal storm hazards along U.S. coastlines and other strategic locations critical to our national security.

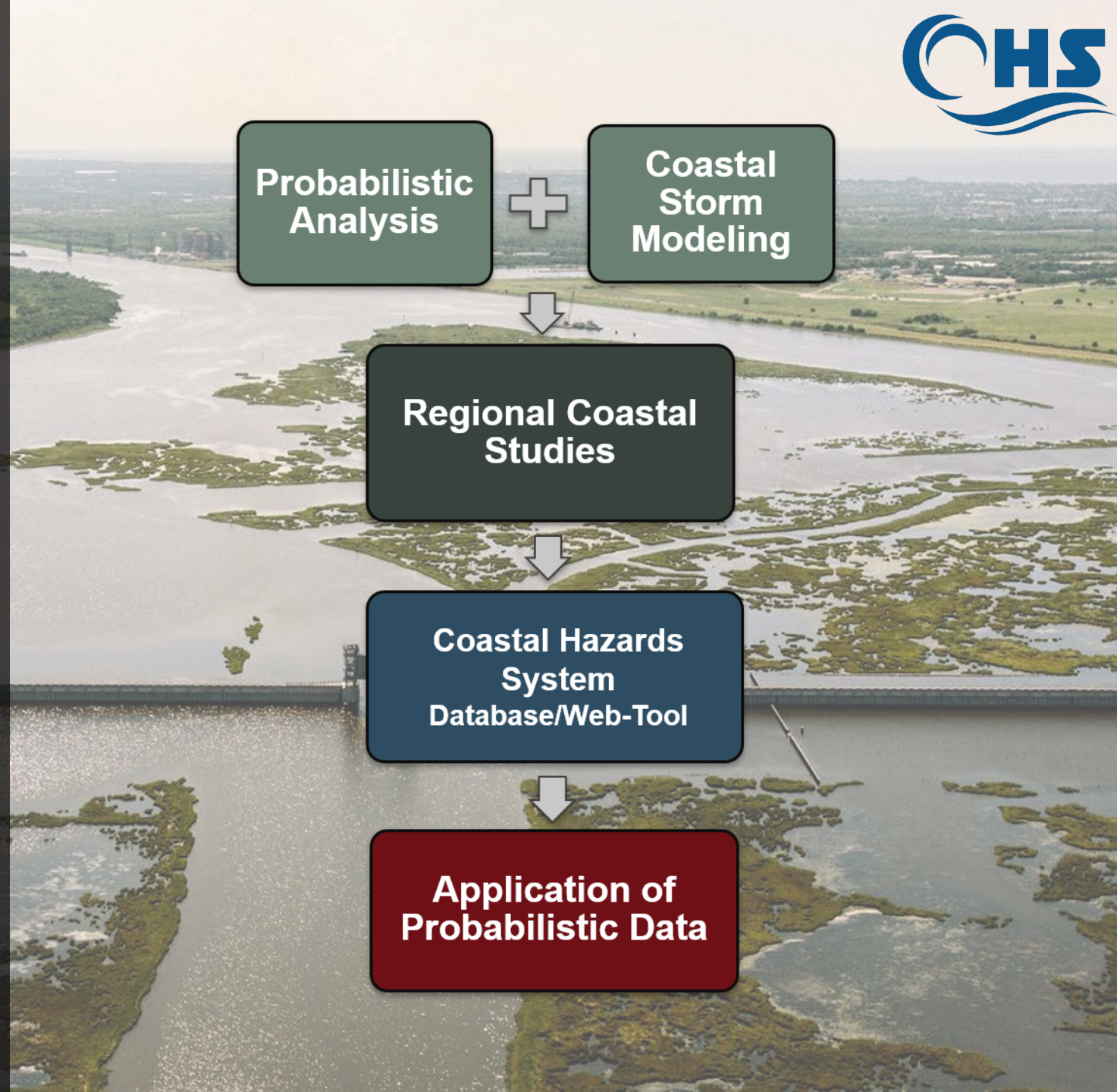
Goal:

Provide high-fidelity, high-resolution state-of-the-art hydrodynamic and probabilistic modeling and companion tools in a multivariate statistical context for coastal planning, engineering, and operations and maintenance.

Impact to the Nation:

Methods, data, and tools within the CHS serve as the basis for coastal engineering by providing high-fidelity, probabilistic coastal hazards on a national scale.

<https://chs.erdcdren.mil>





Synthetic Storm Suite

(4,355 unique TCs)

NACCS

1050 TCs (green tracks)

SACS: NCSFL

1,060 TCs (orange tracks)

SACS: SFLMS

1,085 TCs (cyan tracks)

SACS: PR/USVI

300 TCs (red tracks)

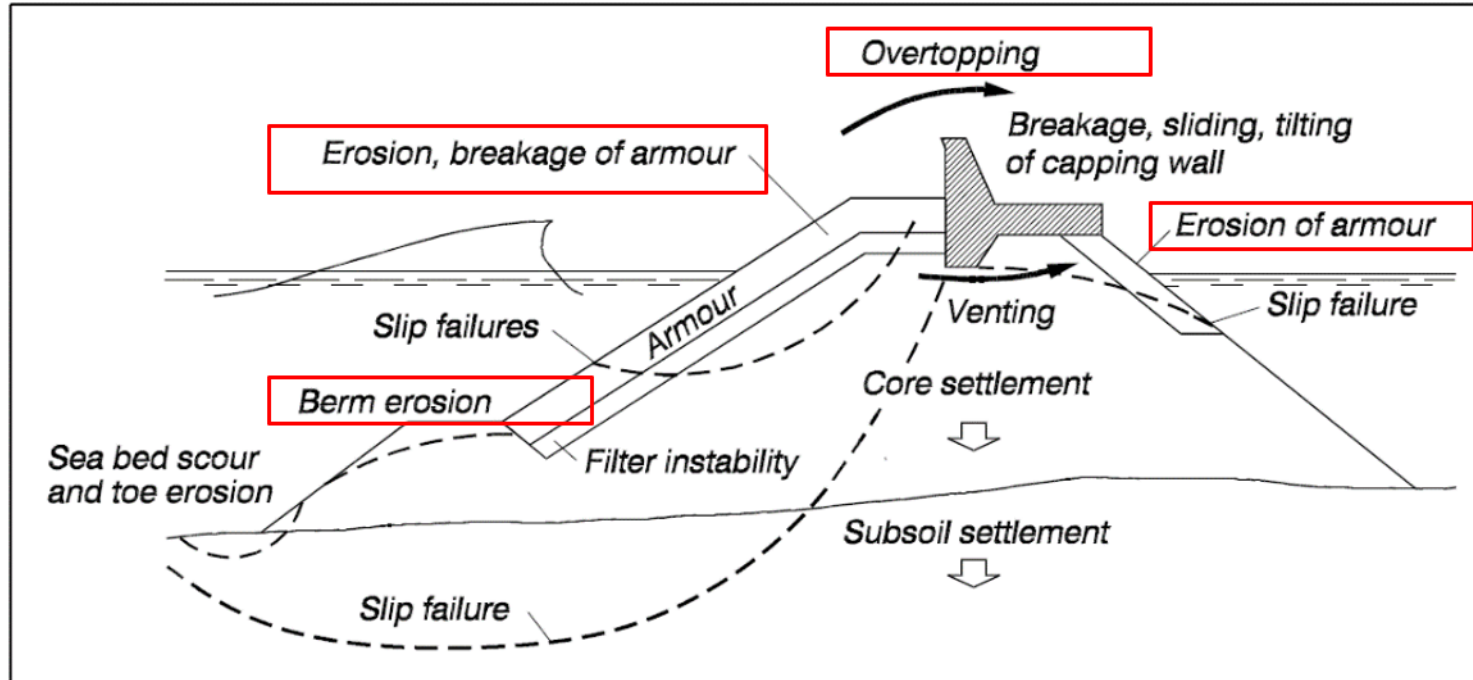
LACS

645 TCs (purple tracks)

TXCS

660 TCs (yellow tracks)

CHESS PRIMARY STRUCTURE RESPONSES



CHESS Primary RM Responses

- Runup
- Overtopping/Overflow
- Seaside armor stability/damage
- Toe stability/damage
- Leaside stability/damage
- Low-crested/submerged stability



CHESS

StormSim_Library\Structure_Reponses

call_project_response.m

compute_structure_response.m

- low_crested_stability2026.m
- melby_Dn50_seaside_stability.m
- u1p_calc.m
- z1p_calc.m
- submerged_CEM_VI_5_25.m
- van_gent_Dn50_leeside_stability.m
- low_crested_stability2026_original.m
- LeeDamFunc_v2.m
- goda_forces_on_vertical_p2p3.m
- Eurotop_r2p_q_Final.m
- goda_forces_on_vertical_p1.m
- damaging_depth.m
- floodwall_nappe_response.m
- load_empirical_coefficients.m
- seaside_armor_stone_damage.m
- melby_low_crested_stability.m



StormSim: PROS Responses

Storm Hazards

Eurotop_r2p_q_Final.m
General Run-up
EurOtop 2018

Eurotop_r2p_q_Final.m
Overtopping Discharge Rate
Combined Overtopping Discharge Rate
($q_{wave_ot} + q_{overflow}$)
Overtopping Discharge Volume
EurOtop 2018

melby_Dn50_seaside_stability.m
Seaside Armor Stone Size
Melby and Kobayashi 2011

van_gent_Dn50_leeside_stability.m
Leeside Armor Stone Size
Van Gent and Pozueta 2003

melby_low_crested_stability.m
Low Crested Median Seaside Stone Size
Kramer 2009 with Melby mod 2019

eurotop_wave_transmission.m
Wave Transmission
EurOtop 2018

goda_forces_on_vertical_p1.m
goda_forces_on_vertical_p2p3.m
floodwall_nappe_response.m
Goda Wall Pressures and overflow nappe
Goda 2000, Hughes 2011

damaging_depth.m
Damaging Depth
Melby et al., 2026

StormSim: LCS Responses

seaside_armor_stone_damage.m
Seaside Armor Damage
Melby and Kobayashi 2011

leeside_armor_stone_damage.m
Leeside Armor Damage
Van Gent and Pozueta 2003

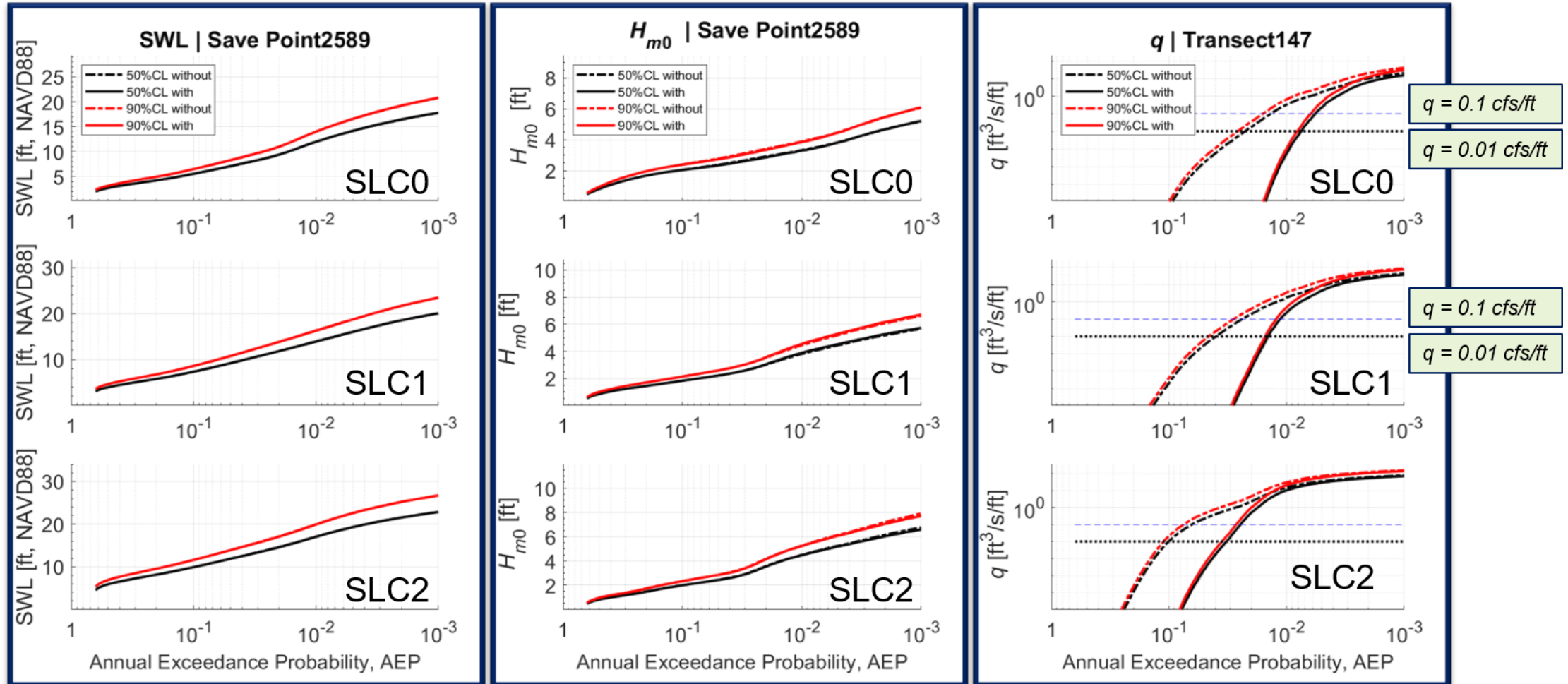
submerged_armor_stone_damage.m
Low Crested Armor Damage
Kramer 2009 with Melby mod 2019



CHESS EXAMPLE OUTPUT – SABINE TO GALVESTON



Sabine to Galveston CSRM PED, SWG

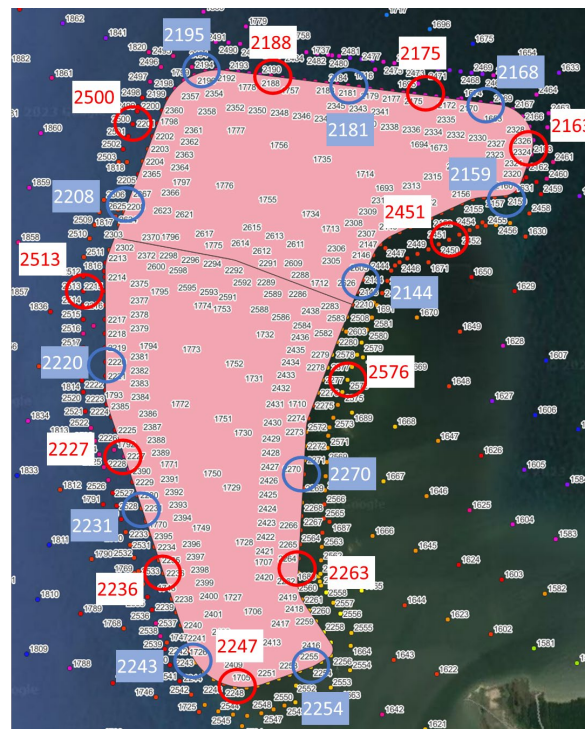
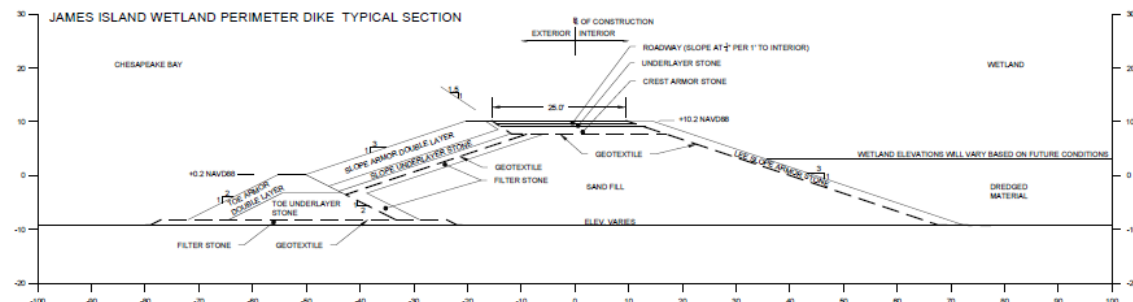




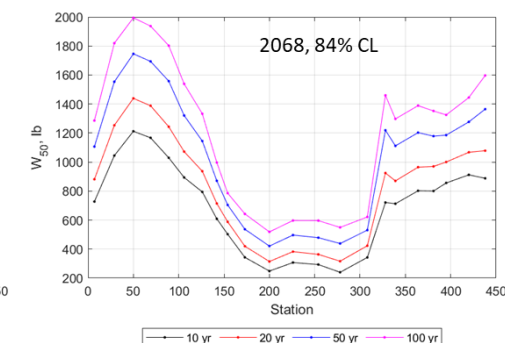
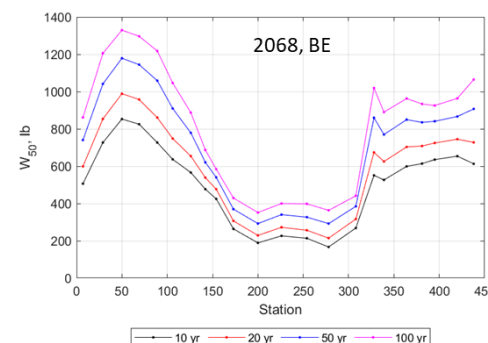
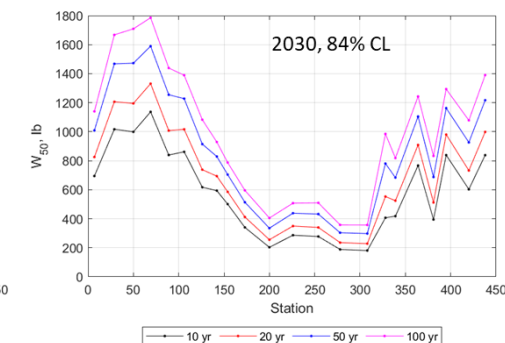
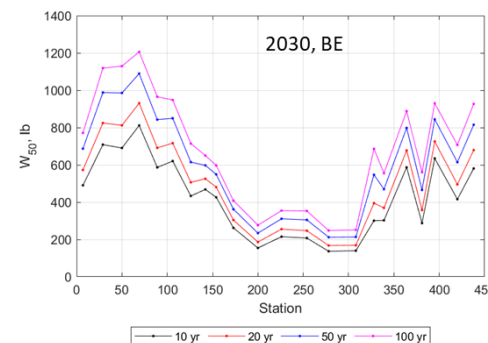
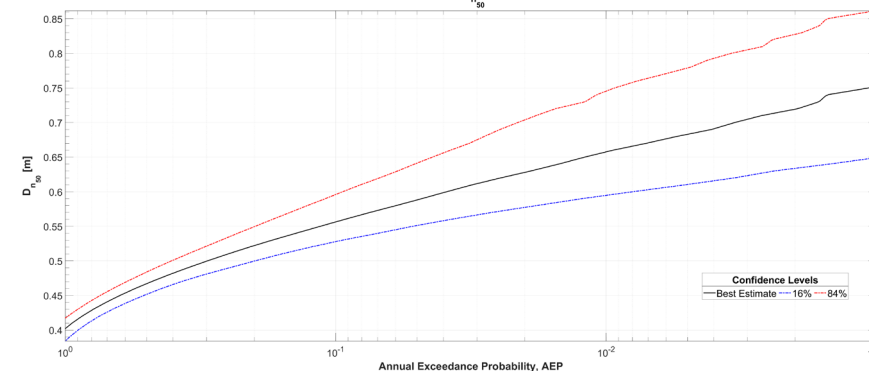
CHESS EXAMPLE OUTPUT – JAMES ISLAND



James Island, MD, NAB



StormSim: PROS - RB3 | Case1 | Sta50 2195
CC | D_{50} [m]

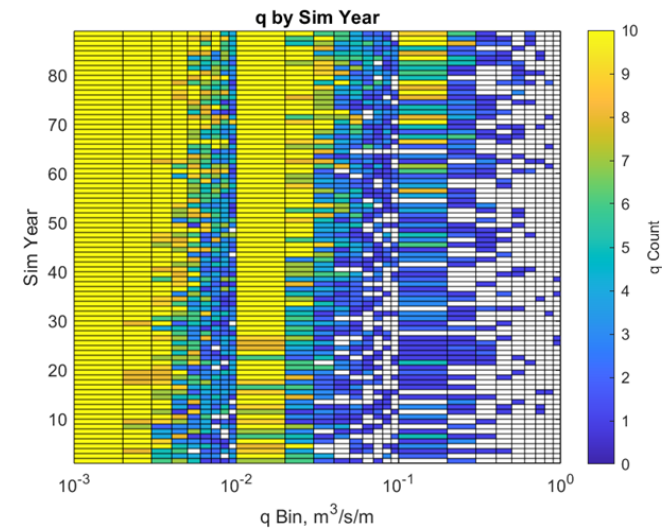
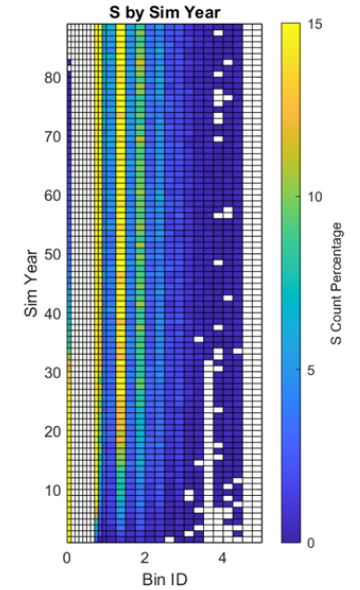
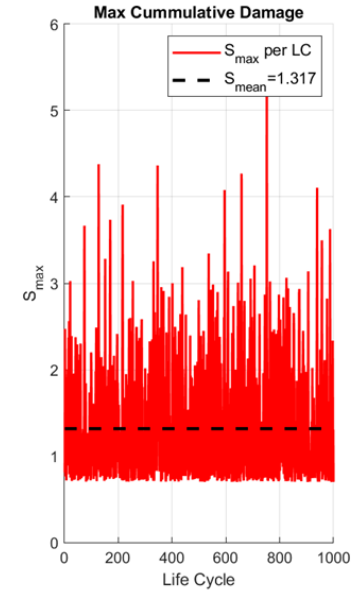
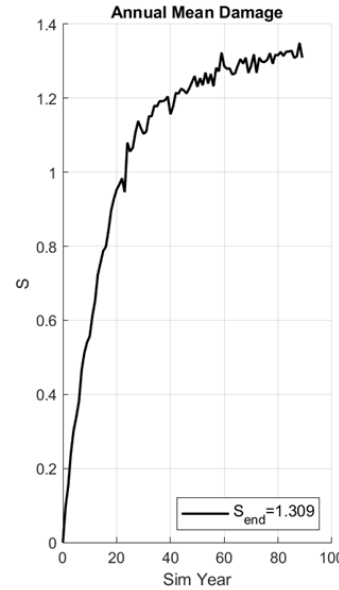
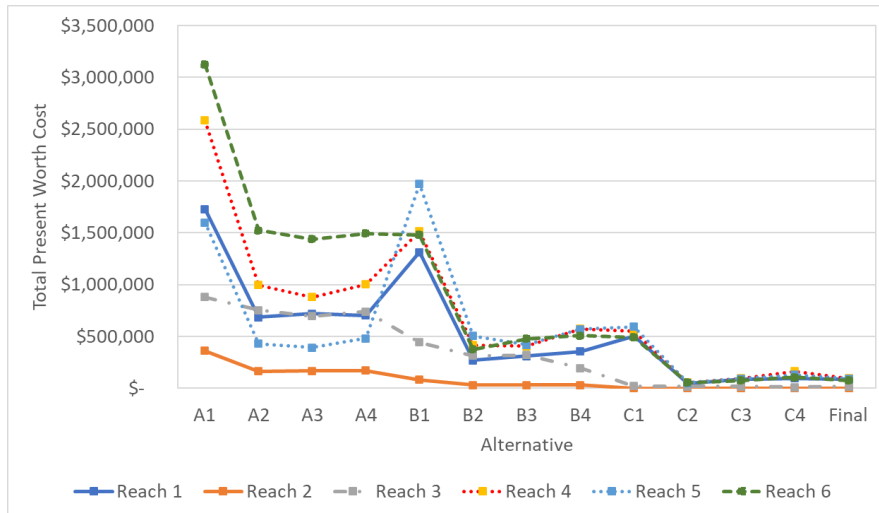




CHESS EXAMPLE OUTPUT – LIFE-CYCLE



James Island Life-Cycle Simulation





CONTENTS – RUNUP



- Problem
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RUNUP, OVERTOPPING, WAVE TRANSMISSION



EurOtop

Manual on wave overtopping of sea defences and related structures

An overtopping manual largely based on European research, but for worldwide application

Second Edition 2018



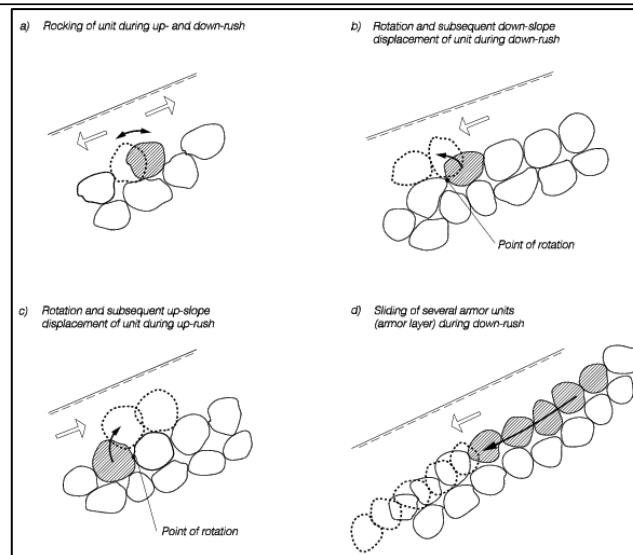
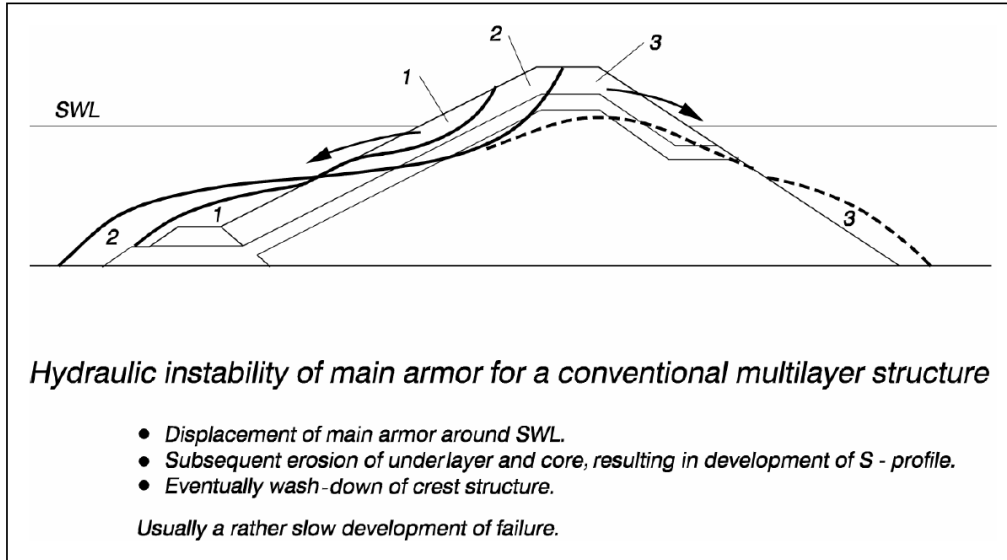
www.overtopping-manual.com



CONTENTS – SEASIDE STABILITY



- Problem
- CHS/CHESS (Coastal Hazards, Engineering, and Structures System)
 - Runup, overtopping, wave transmission
 - **Seaside stability and damage progression**
 - Leese side stability and damage progression
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- Present work





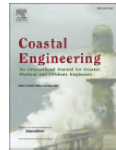
SEASIDE STABILITY, LITTLE TO NO OVERTOPPING



Contents lists available at [ScienceDirect](https://www.sciencedirect.com)

Coastal Engineering

journal homepage: www.elsevier.com/locate/coastaleng



On the uncertainties in stone armor stability

Giulio Scaravaglione^{a,1,*}, Jeffrey A. Melby^{b,1}, Giuseppe R. Tomasicchio^{c,1},
Marcel R.A. van Gent^{d,1}, Alessandra Saponieri^{c,1}

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ARTICLE INFO

Keywords:

Armor stone stability
Homogeneous database
Uncertainty
Shallowness conditions
Damage

ABSTRACT

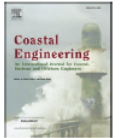
The present research aims to investigate the uncertainties in the evaluation of stone armor stability. Data synthesis was achieved by collecting and homogenizing data from 4 distinct studies, considering the inherent variability of the original data. Established stability equations are then applied to the synthesized database to assess both the strengths and limitations of different approaches across deep, shallow, and very shallow water. The results indicate that while nearly all formulations perform well in deep water, some inadequacies emerge in shallow and very shallow water. To address these limitations, the stability equations were recalibrated using the new database, with a focus on error and uncertainty quantification. The refitted Etemad-Shahidi et al. (ES, 2020) and Modified ES (Scaravaglione et al., 2025) equations consistently demonstrate better predictive capability across all water depths. However, damage assessment reveals persistent uncertainties across all formulations, rendering the selection of a single equation inconclusive, mainly due to the high uncertainty of the available laboratory data. Further synthesizing and homogenizing require additional modeling given the varying modeling approaches, the non-homogenous nature of the parametric data, and the limited understanding possible of the detailed laboratory techniques and data analysis carried out.



Contents lists available at [ScienceDirect](https://www.sciencedirect.com)

Coastal Engineering

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A comprehensive review of the primary sources of uncertainty in stone armor stability

Giulio Scaravaglione^{a,*}, Jeffrey A. Melby^b

^a Department of Civil, Environmental, Land, Building Engineering and Chemistry (DICATECh), Polytechnic University of Bari, Bari, Italy

^b Coastal Structures Group, Coastal and Hydraulic Laboratory, U.S. Army Engineer Research and Development Center, USA

ARTICLE INFO

Keywords:

Stone armor stability
Aleatory and epistemic uncertainty
Coefficient of variation
Laboratory data uncertainty
Predictive model uncertainty
Error-related uncertainty

ABSTRACT

Coastal rubble mound armor stability prediction uncertainty is relatively high in the field of civil engineering. The present study aims to provide an in-depth review of the principal sources of stone armor stability uncertainty derived from laboratory experiments. The study delineates the contribution of each source and sub-class to the total uncertainty based on the body of knowledge from the literature and data analysis. Uncertainty is first classified into two main components: aleatory (intrinsic), which is irreducible and arises from the inherent randomness of natural processes, and epistemic uncertainty, which relates to limited knowledge of physical processes, observations, and predictive methods, and can be reduced with appropriate precautions. Epistemic uncertainty is further subdivided into three main categories: data uncertainty (waves and damage), predictive model uncertainty, and experimental errors. The focus is on empirical stability equations and the underlying data and experiments. For each category and sub-class, a semi-quantitative estimation of the coefficient of variation is provided to convey a sense of the magnitude of the component contribution to the total epistemic uncertainty in stability predictions. Results indicate that data uncertainty, particularly related to damage assessment, is the dominant contributor, followed by predictive model uncertainty, while error-related uncertainty have a smaller impact. The findings highlight the importance of improving data quality and standardization to reduce epistemic uncertainty, thereby enhancing the reliability of empirical design models, and supporting more consistent probabilistic design of rubble mound structures.

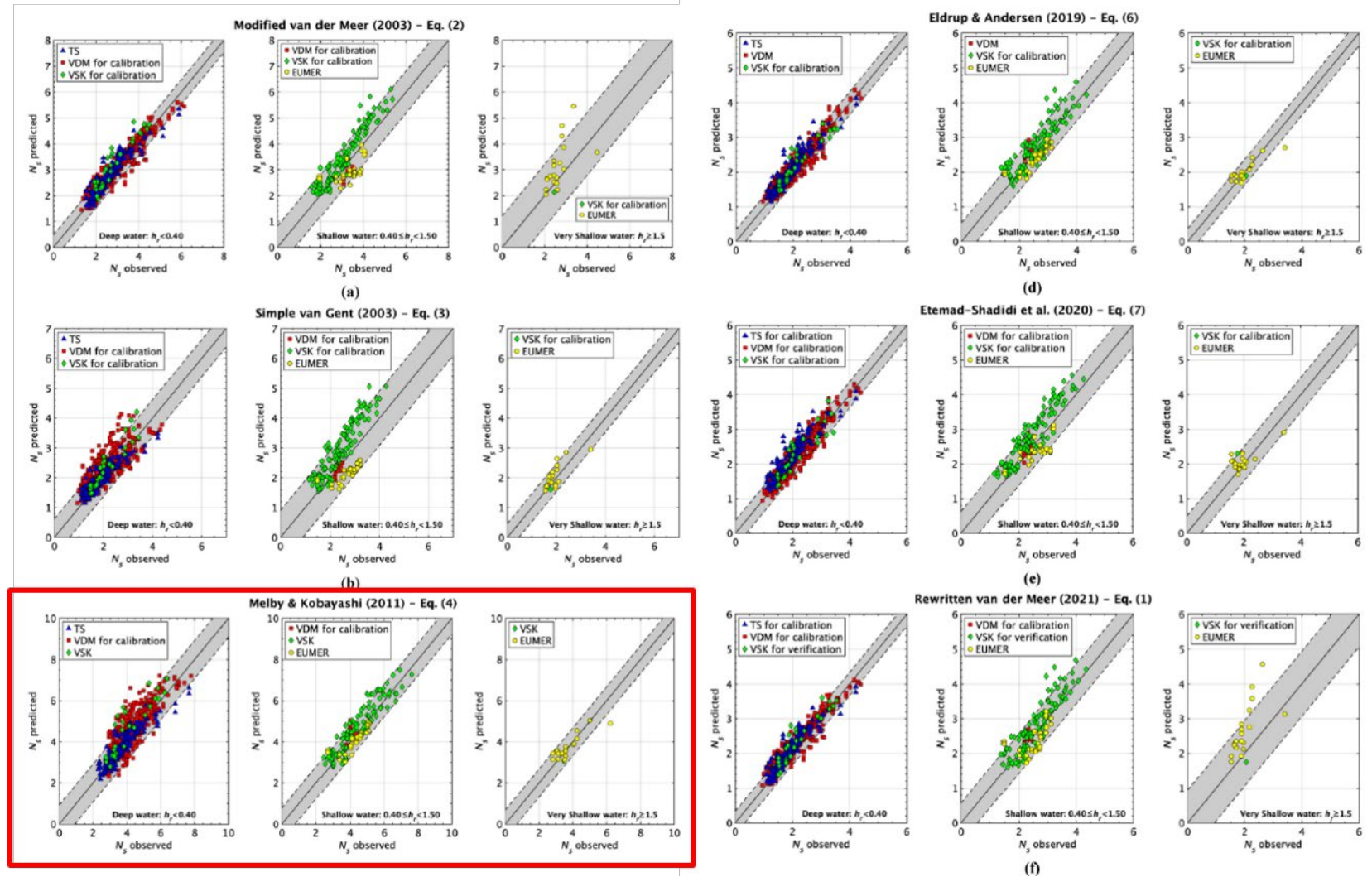
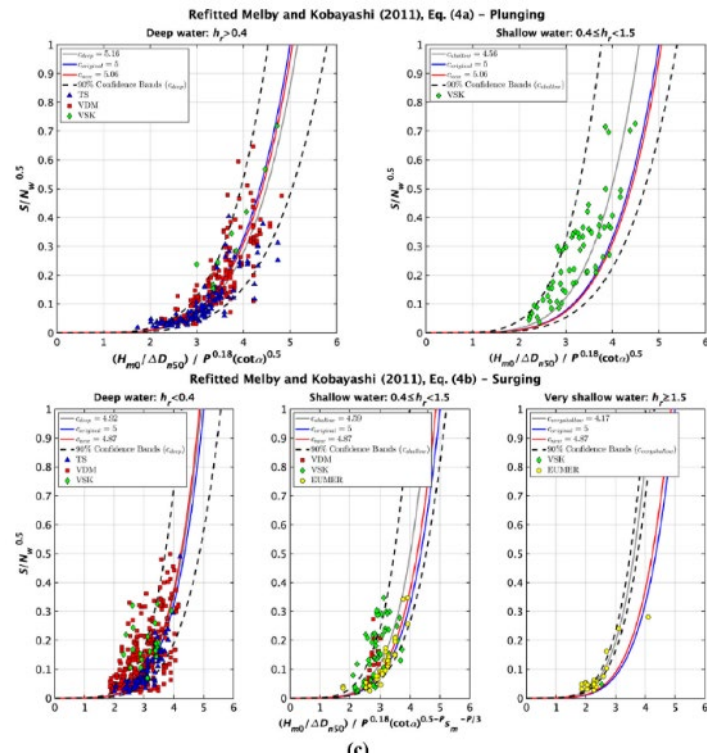
Giulio gathered data from 7 seaside armor stability experiments. Curated data & reproduced studies with COULWAVE. Compared to equations of van der Meer (1988, 2003), van Gent (2003, 2025), Melby and Kobayashi (2011), Eldrup and Anderson (2019), Etemad-Shahidi et al. (2020, 2025). Refit equations to large dataset.



SEASIDE STONE ARMOR STABILITY



Equations all did pretty well across deep, shallow, and very shallow water regimes. Some are tuned to deep water with flat bathymetry and do poorly in shallow water. Some do better for damage.





SEASIDE STONE ARMOR STABILITY – EQUATIONS



• Melby, J. A., & Hughes, S. A. (2004). Armor stability based on wave momentum flux. In Coastal Structures 2003 (pp. 53-65).

• Melby, J. A., & Kobayashi, N. (2011). Stone armor damage initiation and progression based on the maximum wave momentum flux. Journal of Coastal Research, 27(1), 110-119.

$$D_{n50} = h a_m \left(\frac{S}{K_s \sqrt{N_z}} \right)^{-\frac{1}{5}} \left(\frac{(M_F)_{max}}{\rho_w g h^2 \Delta} \right)^{\frac{1}{2}}$$

$$a_m = \frac{S_m^{P/3}}{K_{m2} P^{0.18} (\cot \theta)^{0.5-P}} \quad S_m < S_{mc}$$

$$a_m = \frac{1}{K_{m1} P^{0.18} \sqrt{\cot \theta}} \quad S_m \geq S_{mc} \quad S_{mc} = (\cot \theta)^{-3}$$

K Coefficients are determined from best fit of equation to lab data

Permeability

P=0.1 impermeable
P=0.4 multilayer
P=0.6 homogeneous

$$\frac{(M_F)_{max}}{\rho_w g h^2} = A_0 \left(\frac{h}{g T_m^2} \right)^{-A_1}$$

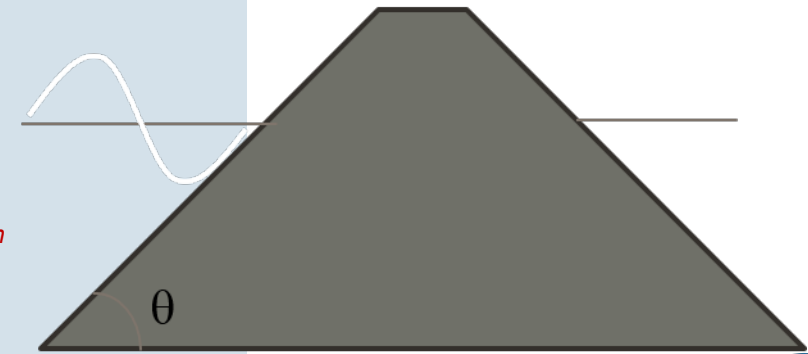
$$A_0 = 0.639 \left(\frac{H_{m0}}{h} \right)^{2.026}$$

$$A_1 = 0.180 \left(\frac{H_{m0}}{h} \right)^{-0.391}$$

Definitions:

- g = acceleration of gravity
- ρ_w = density of water
- h = water depth at structure toe
- θ = structure slope
- $\Delta = S_r - 1$, S_r = specific gravity of armor stone
- $S_m = H_{m0} / L_m$, L_m is wave length from linear dispersion equation using T_m
- H_{m0} = spectral energy significant wave height
- T_m = local mean wave period near structure toe
- N_z = number of waves = duration/ T_m

CEM pg VI-5-16





SEASIDE STONE ARMOR STABILITY – FLUX



Depth Integrated wave momentum flux per unit crest length

$$(M_F)_{\max} = \int_{-h}^{H/2} (p_d + \rho u^2) dz$$

Maximum wave force proportional to $M_{\max}$

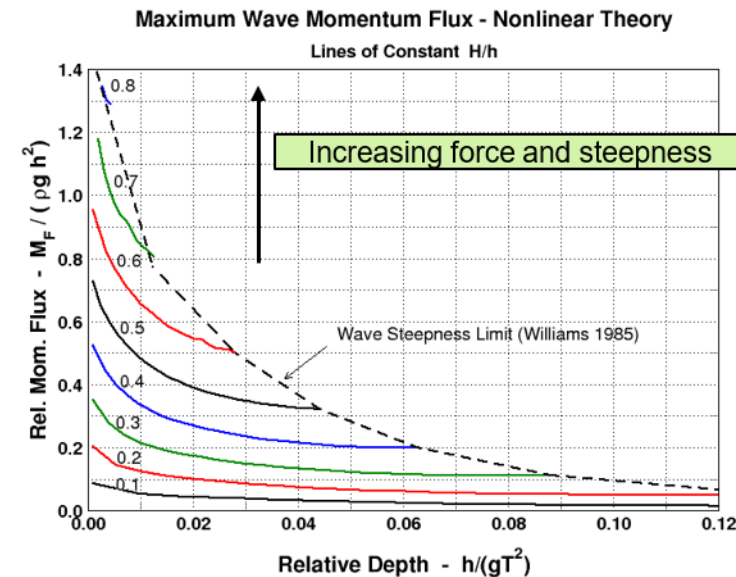
$$\left(\frac{M_F}{\rho_w g h^2} \right)_{\max} = \frac{1}{2} \frac{H}{h} \frac{\tanh kh}{kh} + \frac{1}{8} \left(\frac{H}{h} \right)^2 \left(1 + \frac{2kh}{\sinh 2kh} \right)$$

Dynamic
Pressure
Term

Velocity
Term

Low steepness waves: 5% contribution from velocity term
High steepness waves: 30% contribution from velocity term
Expect wave period for low steepness long waves to be very influential on stability

H =wave height, h =local water depth, ρ =water density,
 g =accel of gravity, k =wave number, T =wave period





SEASIDE STONE ARMOR DAMAGE PROGRESSION

Damage Initiation

$$S(t_1) = 0.5 + 1.3\sqrt{N_{ze} + (N_z)_1} (a_m N_m)_1^5 \quad \text{for } n = 1 \text{ and } S(t_1) > 0.2$$

Progressive Damage

$$S(t_n) = 1.0\sqrt{N_{ze} + (N_z)_n} (a_m N_m)_n^5 \quad \text{for } n > 1$$

$$N_m = \left(\frac{(M_F)_{\max}}{\rho_w g h^2 \Delta} \right)^{1/2} \frac{h}{D_{n50}} \quad N_{ze} = \left(\frac{S_{n-1}}{K_s (a_m N_m)_n^5} \right)^2 \quad (N_z)_n = (t_n - t_{n-1}) / (T_m)_n$$

$S(t_n)$ = cumulative damage at time t_n

N_{ze} = equivalent prior number of waves

$(N_z)_n$ = number of waves at T_m for time step n

N_m = momentum flux stability number

$t_n - t_{n-1}$ = time step n (typically 3 hr)

$S(t_{n-1})$ = cumulative damage at time t_{n-1}

CEM pg VI-5-58 to 62; Melby, J.A. and Kobayashi, N.K. (1998). "Progression and variability of damage on rubble mound breakwaters," ASCE Journal of the Waterway, Port, Coastal and Ocean Division, V. 124, 6:286-294.

Updated: Melby, J.A., and Kobayashi, N.K. (2011). "Stone armor damage initiation and progression," Journal of Coastal Research 27(1), Allen Press, 110-119.



CONTENTS – LEESIDE



- Problem
- CHS/CHESS (Coastal Hazards, Engineering, and Structures System)
 - Runup, overtopping, wave transmission
 - Seaside stability and damage progression
 - **Leeside stability and damage progression**
 - Low-Crested and submerged stability and damage
- Present work



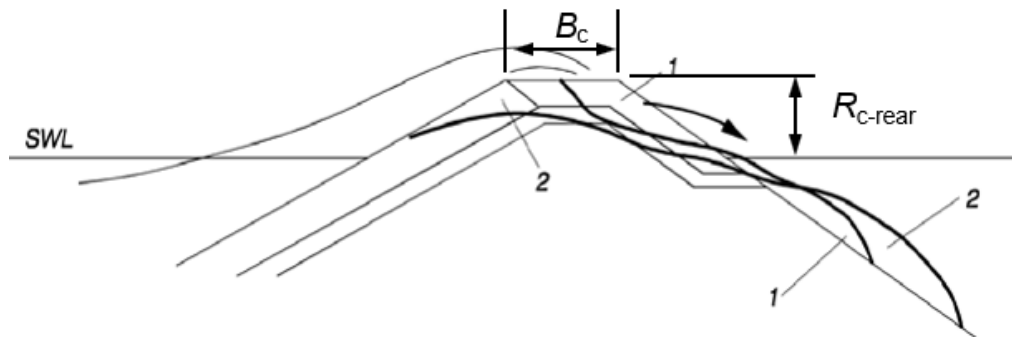
LEESIDE STONE ARMOR STABILITY



$$D_{n50} = a_{ls} \left(\frac{S_{ls}}{K_{ls} \sqrt{N_z}} \right)^{-1/r} \left(\frac{u_{1\%} T_{m-1,0}}{125 \sqrt{\Delta}} \right)$$

$$a_{ls} = (\cot \phi)^{-2.5/r} [1 + 10 \cdot \exp(-R_{c-rear} / H_{m0})]^{1/r}$$

$$\frac{u_{1\%}}{\sqrt{gH_{m0}}} = \frac{1.7(\gamma_{f-c})^{0.5} \left(\frac{Z_{1\%} - R_c}{\gamma_f H_{m0}} \right)^{0.5}}{\left(1 + 0.1 \frac{B_c}{H_{m0}} \right)}$$



$$R_c/H_s > 0.3, D_{lee} < D_{sea}$$

S_{ls} = leeside damage

K_{ls} = empirical coefficient

N_z = number of waves at T_m

$u_{1\%}$ = hydro velocity on crest for 1% exceedance

$\Delta = S_r - 1$

$T_{m-1,0}$ = zeroth moment wave period $\sim T_p/1.1$ (EurOtop pg 4)

D_{n50} = median leeside armor stone equivalent cube length

R_{c-rear} = leeside freeboard

R_c = seaside freeboard

ϕ = leeside slope

$Z_{1\%}$ = runup 1% exceedance

γ_f = friction factor

γ_{f-c} = friction factor on crest

B_c = crest width

$K_{ls} = 1$

$r = 6$

Van Gent, M. and Pozueta, B. (2004). "Rear-side stability of rubble mound structures," Proc. 29th Intl Conf on Coast. Eng., ASCE, V4, 3481-3493.



LEESIDE STONE ARMOR DAMAGE PROGRESSION

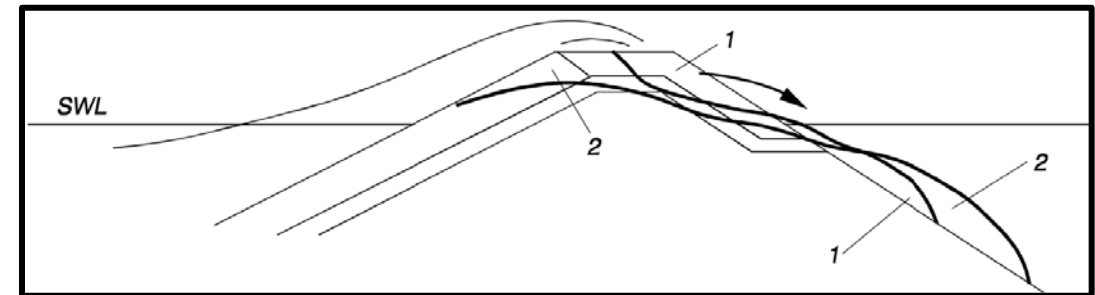
$$\bar{S}(t_n) = K_{ls} \sqrt{N_{ze} + (N_z)_n} (a_{ls} N_{ls})_n^r \quad \bar{S}(t_n) \geq 2$$

$$(N_{ze})_{ls} = \left(\frac{\bar{S}(t_{n-1})}{K_{ls} (a_{ls} N_{ls})_n^r} \right)^2 \quad (N_z)_n = \frac{t_n - t_{n-1}}{(T_m)_n}$$

$$N_{ls} = \left(\frac{u_{1\%} T_{m-1,0}}{125 (D_n \sqrt{\Delta})_{ls}} \right)$$

$$\bar{S}_{lee} \leq 2: \quad K_{ls} = 0.4, \quad r = 6$$

$$\bar{S}_{lee} > 2: \quad K_{ls} = 0.04, \quad r = 1$$





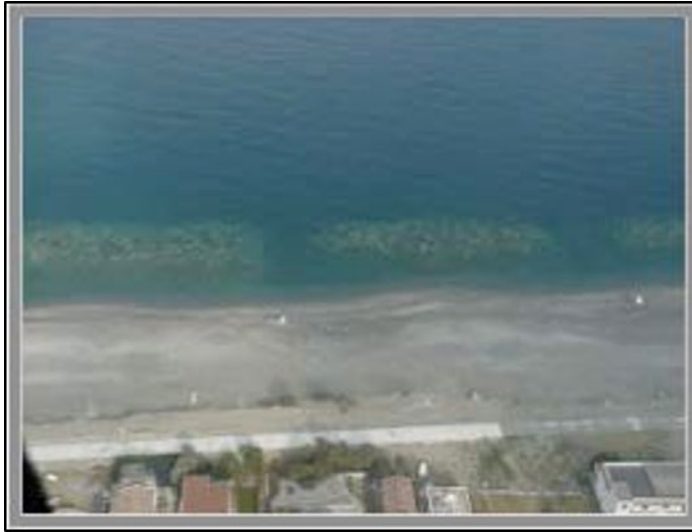
CONTENTS – LOW-CRESTED AND SUBMERGED



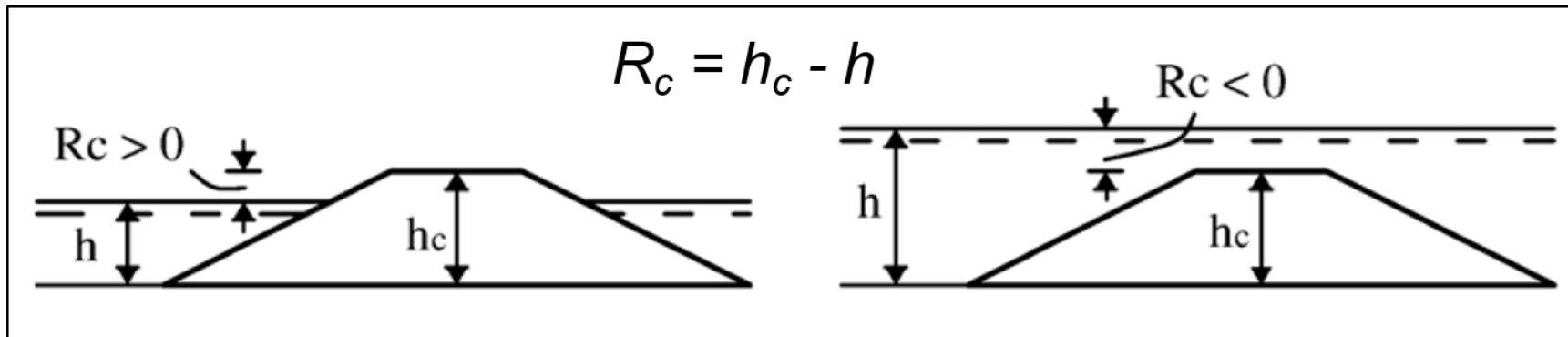
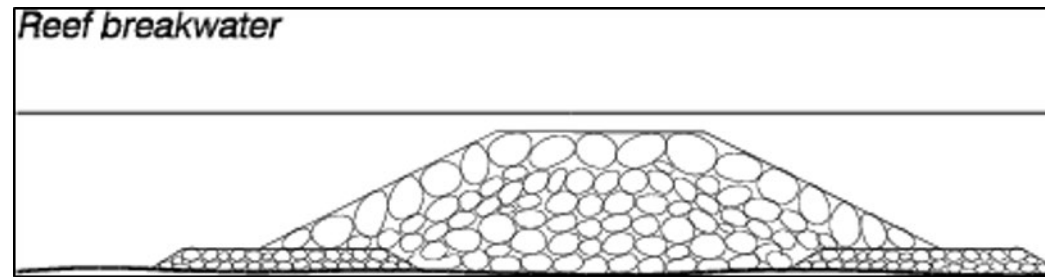
- Problem
- CHS/CHESS (Coastal Hazards, Engineering, and Structures System)
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- Present work



LOW-CRESTED & SUBMERGED STONE ARMOR STABILITY



Low crested: $R_{2\%} > R_c$



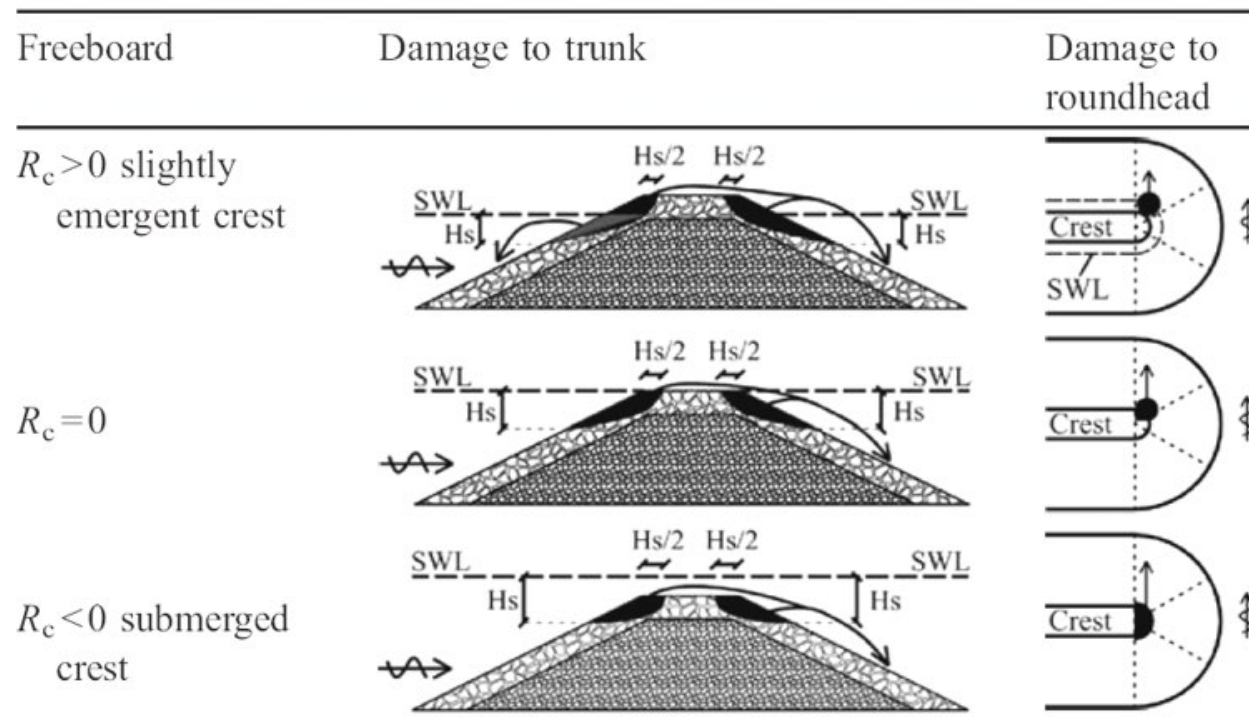


LOW-CRESTED & SUBMERGED STONE ARMOR STABILITY DAMAGE



In addition to front-slope instability, damage can occur on crest and backside as well

- For emergent structures, armor on the seaward slope is least stable
- For submerged and slightly emergent structures, crest is least stable





LOW-CRESTED & SUBMERGED STONE ARMOR STABILITY DOCUMENTATION



CEM Mostly written in late 1990's

Part V, Chapter VI

– Design:

- Little to no OT crest elevation: vdM
- Little for leeside stability/damage
- Low crested & submerged
 - VI-5-24, VI-5-25
 - Combination of regular and irregular experiments, different damage measurements, different wavelengths, different data fits
 - No continuity

– Performance Evaluation

- Cumulative damage for seaside armor
 - Dated
- Nothing for LC or submerged

EM 1110-2-1100 (Part VI)
1 Jun 06

Table VI-5-23
Rock, Two-Layer Armored Non-Overtopped Slopes (van der Meer 1988)

Irregular, head-on waves			
$\frac{H_s}{\Delta D_{n50}}$	$= 6.2 \cdot S^{0.2} P^{0.18} N_z^{-0.1} \xi_{sm}^{-0.5}$	Plunging waves : $\xi_m < \xi_{mc}$	(VI-5-68)
$\frac{H_s}{\Delta D_{n50}}$	$= 1.0 \cdot S^{0.2} P^{-0.13} N_z^{-0.1} (\cot \alpha)^{0.5} \xi_{sm}^P$	Surging waves : $\xi_m > \xi_{mc}$	(VI-5-69)
	$\xi_{mc} = s_m^{-0.5} \tan \alpha$	$\xi_{mc} = (6.2 P^{0.31} (\tan \alpha)^{0.5})^{1/(P+0.5)}$	
where H_s	Significant wave height in front of breakwater		
D_{n50}	Equivalent cube length of median rock		
ρ_s	Mass density of rocks		
ρ_w	Mass density of water		
Δ	$(\rho_s / \rho_w) - 1$		
S	Relative eroded area (see Table VI-5-21 for nominal values)		
P	Notional permeability (see Figure VI-5-11)		
N_z	Number of waves		
α	Slope angle		
s_m	Wave steepness, $s_m = H_s / L_{om}$		
L_{om}	Deepwater wavelength corresponding to mean wave period		

Validity:

- 1) Equations VI-5-68 and VI-5-69 are valid for non-depth-limited waves. For depth-limited waves H_s is replaced by $H_{2\%}/1.4$.
- 2) For $\cot \alpha \geq 4.0$ only Eq VI-5-68 should be used.
- 3) $N_z \leq 7,500$ after which number equilibrium damage is more or less reached.
- 4) $0.1 \leq P \leq 0.6$, $0.005 \leq s_m \leq 0.06$, $2.0 \text{ tonne/m}^3 \leq \rho \leq 3.1 \text{ tonne/m}^3$
- 5) For the 8 tests run with depth-limited waves, breaking conditions were limited to spilling breakers which are not as damaging as plunging breakers. Therefore, Eqs VI-5-68 and VI-5-69 may not be conservative in some breaking wave conditions.

Uncertainty of the formula: The coefficient of variation on the factor 6.2 in Eq VI-5-68 and on the factor 1.0 in Eq VI-5-69 are estimated to be 6.5% and 8%, respectively.

Test program: See Table VI-5-4.

EM 1110-2-1100 (Part VI)
1 Jun 06

Table VI-5-24
Rock, Two-Layer Armored Overtopped, but Not Submerged, Low-crested Slopes

Powell and Allsop (1985) analyzed data by Allsop (1983) and proposed the stability formula

$$\frac{N_{od}}{N_a} = a \exp \left[b s_p^{-1/3} H_s / (\Delta D_{n50}) \right] \quad \text{or} \quad \frac{H_s}{\Delta D_{n50}} = \frac{s_p^{1/3}}{b} \ln \left(\frac{1}{a} \frac{N_{od}}{N_a} \right) \quad (\text{VI-5-70})$$

where values of the empirical coefficients a and b are given in the table as functions of freeboard R_c and water depth h . N_{od} and N_a are the number of units displaced out of the armor layer and the total number of armor layer units, respectively.

Values of coefficients a and b in Eq VI-5-70.

R_c/h	$a \cdot 10^4$	b	wave steepness H_s/L_p
0.29	0.07	1.66	<0.03
0.39	0.18	1.58	<0.03
0.57	0.09	1.92	<0.03
0.38	0.59	1.07	>0.03

van der Meer (1991) suggested that the van der Meer stability formulae for non-overtopped rock slope, Eqs VI-5-68 and VI-5-69, be used with $f_i D_{n50}$ substituted for D_{n50} . The reduction factor f_i is given as

$$f_i = \left(1.25 - 4.8 \frac{R_c}{H_s} \sqrt{\frac{s_{op}}{2\pi}} \right)^{-1} \quad (\text{VI-5-71})$$

where R_c is the freeboard, $s_{op} = H_s / L_{op}$, and L_{op} is deepwater wavelength corresponding to the peak wave period. Limits of Eq VI-5-71 are given by

$$0 < \frac{R_c}{H_s} \sqrt{\frac{s_{op}}{2\pi}} < 0.052$$

- Some of the earlier results were obtained using monochromatic waves, whereas most of the more recent model tests used irregular waves. Numerous studies have suggested that the monochromatic wave height leading to armor instability roughly corresponds to the significant wave height of irregular waves; however, not all studies have found this correspondence. For preliminary design for nonbreaking wave conditions always use a stability formula based on irregular wave testing if possible. For breaking wave conditions monochromatic wave stability results will be conservative.
- It is generally thought that the higher waves associated with wave groups are responsible for armor layer damage. Typically irregular wave stability model tests use wave trains with assumed random phasing of the spectral components. Over the course of the testing wave groups of differing characteristics impact the structure, and the assumption is that these wave groups are representative of nature. However, it is possible that nonrandom phasing occurs in nature, particularly in shallow water (Andrews and Borgman 1981). Therefore, use of regular wave stability results will be appropriate in some cases.



LOW-CRESTED & SUBMERGED STONE ARMOR STABILITY DELOS

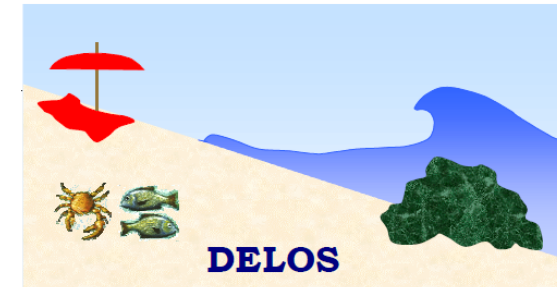


DELOS low-crested and submerged
2D and 3D experiments
Gathered data from 5 prior experiments

- DELOS Lab reports
- Kramer, M. 2006. Structural Stability of Low-Crested Breakwaters. Department of Civil Engineering, Aalborg University
- Burcharth, H. F., M. Kramer, A. Lamberti, B. Zanuttigh. 2006. *Structural stability of detached low crested breakwaters*. Coastal Engineering 53:381-394.
- Conference papers

EU Fifth Framework Programme 1998-2002
Energy, Environment and Sustainable Development

Environmental Design of Low Crested Coastal Defence Structures



D31

Wave basin experiment final form



LOW-CRESTED & SUBMERGED STONE ARMOR STABILITY

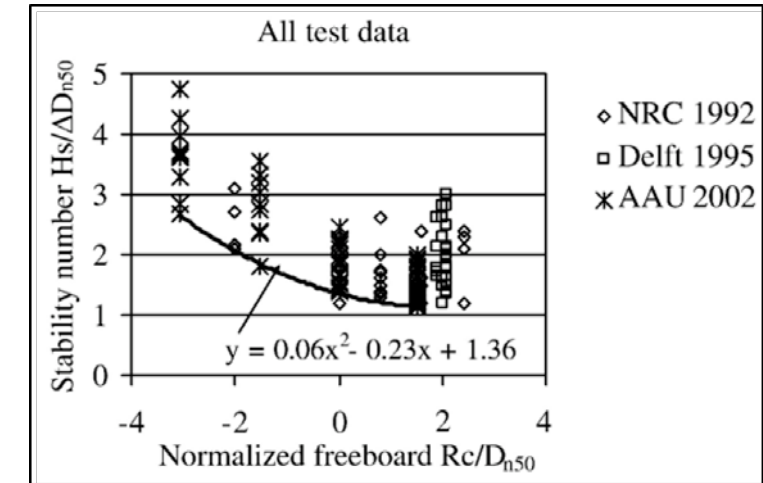
DELOS RESEARCH PROGRAM PRIMARY PRODUCT



$$\frac{H_s}{\Delta D_{n50}} = 0.06 \left(\frac{R_c}{D_{n50}} \right)^2 - 0.23 \left(\frac{R_c}{D_{n50}} \right) + 1.36$$

$$-3 \leq \frac{R_c}{D_{n50}} < 2$$

H_s – significant wave height in front of structure [= H_{mo}]
 Δ – armor immersed relative density [= $(\rho_s/\rho_w - 1)$]
 ρ_s – armor stone mass density
 ρ_w – fluid mass density
 D_{n50} – equivalent cube length of median armor stone
 R_c – structure freeboard



Observations

- Damage was stone counts – assumed “no-damage” criterion
- Polynomial fit
 - Suggests no physics, Best fit is a lower envelope
 - Quadratic has 2 solutions
 - Lower envelope demands deterministic design approach



CONTENTS



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 - Low-Crested and submerged stability and damage
- **Present work**



PRESENT WORK



Primary goals:

Validated methods

Process models

Continuity across different rubble mound structure and forcing regimes

- Varying structure types

- Varying structure locations (seaside, leese side)

- Varying structure responses (incipient stability, progressive damage)

- Varying relative depth, relative freeboard (emergent, low-crested, submerged)



PRESENT WORK – APPROACH



1. Validation:

- Hindcast projects: Point Judith, Indian River, Perdido Pass, St Lucie, Wells Harbor, Oswego, Poplar Island, Portland Main
- Giulio's validation

- ## 2. Assembled Giulio's and DELOS data along with my own data, over 1500 observations and climbing
- Curated data – normalized broad range of wave and damage measurements

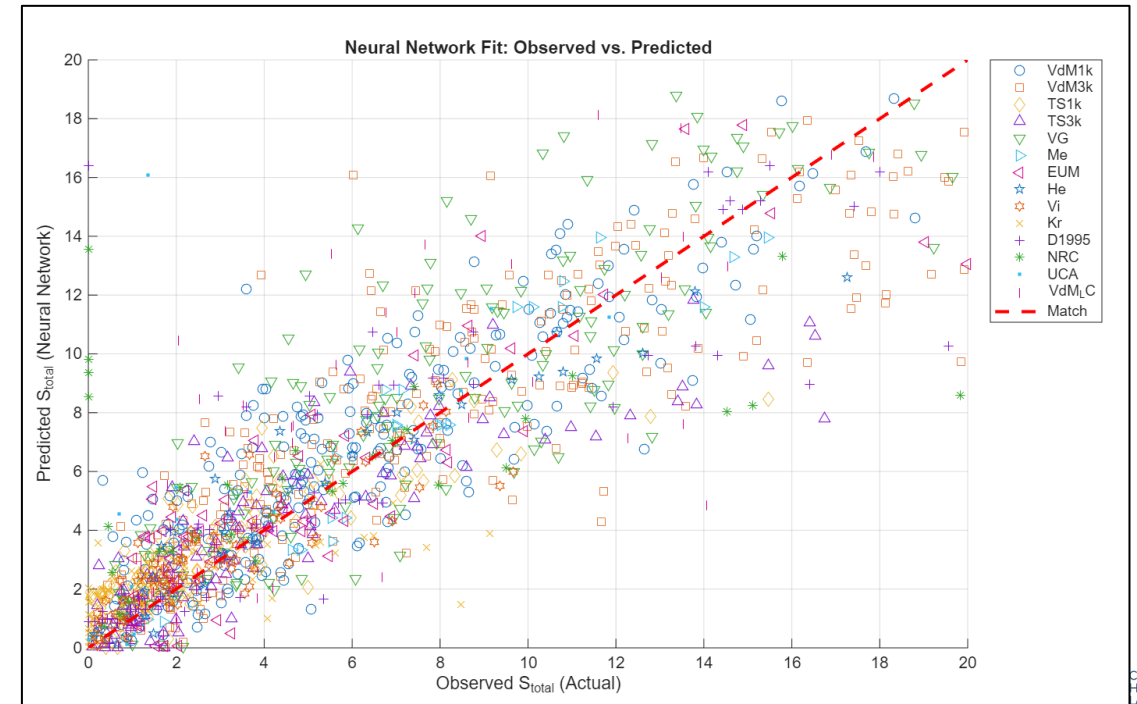
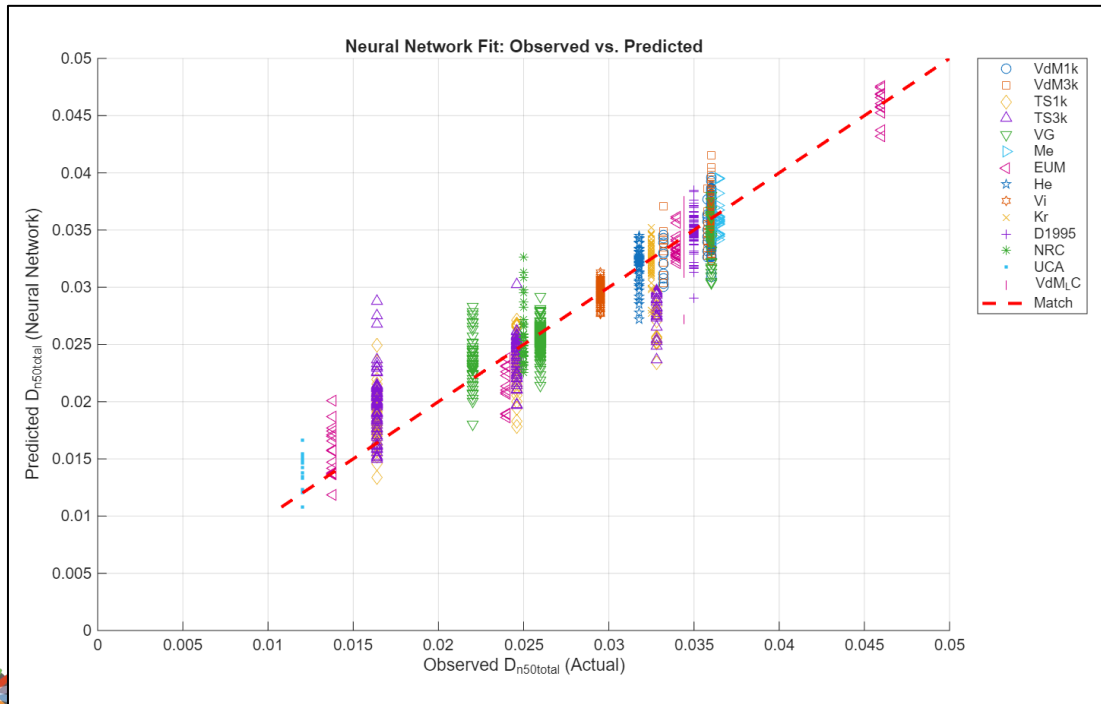
	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X
1	Group	Test_Num	Global_Num	JAM_Test_No	Description	P	hc_m	B_m	d_m	Dn50_m	Delta	cot_a	cot_b	Hs_m	Hm0_m	Tp_s	Tm_s	Tm-10_s	Nwaves	S_SS	S_C	S_LS	S_C+S_LS	S_Total
2	G01	1	2	VdM1k_G01_1	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.086	0.086	2.080	1.85	2.03	1000	3.45	0.00	0.00	0.00	3.45
3	G01	2	3	VdM1k_G01_2	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.046	0.046	2.130	1.85	2.03	1000	0.35	0.00	0.00	0.00	0.35
4	G01	3	4	VdM1k_G01_3	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.077	0.077	2.130	1.85	2.03	1000	3.26	0.00	0.00	0.00	3.26
5	G01	4	5	VdM1k_G01_4	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.070	0.070	2.130	1.85	2.03	1000	0.81	0.00	0.00	0.00	0.81
6	G01	5	6	VdM1k_G01_5	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.073	0.073	2.560	2.19	2.40	1000	2.04	0.00	0.00	0.00	2.04
7	G01	6	7	VdM1k_G01_6	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.083	0.083	2.530	2.18	2.39	1000	4.78	0.00	0.00	0.00	4.78
8	G01	7	8	VdM1k_G01_7	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.066	0.066	2.600	2.19	2.40	1000	1.17	0.00	0.00	0.00	1.17
9	G01	8	9	VdM1k_G01_8	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.056	0.056	2.560	2.19	2.40	1000	0.56	0.00	0.00	0.00	0.56
10	G01	9	10	VdM1k_G01_9	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.080	0.080	2.560	2.19	2.40	1000	2.14	0.00	0.00	0.00	2.14
11	G01	10	11	VdM1k_G01_10	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.075	0.075	3.130	2.69	2.95	1000	1.24	0.00	0.00	0.00	1.24
12	G01	11	12	VdM1k_G01_11	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.066	0.066	3.130	2.69	2.95	1000	0.56	0.00	0.00	0.00	0.56
13	G01	12	13	VdM1k_G01_12	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.055	0.055	3.130	2.69	2.95	1000	0.76	0.00	0.00	0.00	0.76
14	G01	13	14	VdM1k_G01_13	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.084	0.084	3.170	2.68	2.93	1000	3.59	0.00	0.00	0.00	3.59
15	G01	14	15	VdM1k_G01_14	Rock impermeable	0.1	1.15	0.108	0.80	0.0360	1.630	2	1.5	0.093	0.093	3.170	2.68	2.93	1000	32.63	0.00	0.00	0.00	32.63



PRESENT WORK – MACHINE LEARNING



3. Trained machine learning base models for total S , seaside S , leeside S , D_{n50} and N_s
- Matlab, many different training and validation approaches
 - All training has been successful, generally low errors (except S)
 - Presently working on augmenting data with weighted synthesized results
 - Parameters vary by experiments – some parameters are missing
 - Some parameters have limited support (h_c , D_{n50})





PRESENT WORK – CSHORE



4. Validated CSHORE for armor damage, trained machine learning model for profile evolution, detailed damage information (mean S , std dev S , max S , etc)

Coastal Engineering 204 (2026) 104897



Contents lists available at ScienceDirect

Coastal Engineering

journal homepage: www.elsevier.com/locate/coastaleng



Effects of design parameters and sea level rise on lifecycle performance of rubble-mound breakwaters: A computational-statistical framework

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ARTICLE INFO

Keywords:

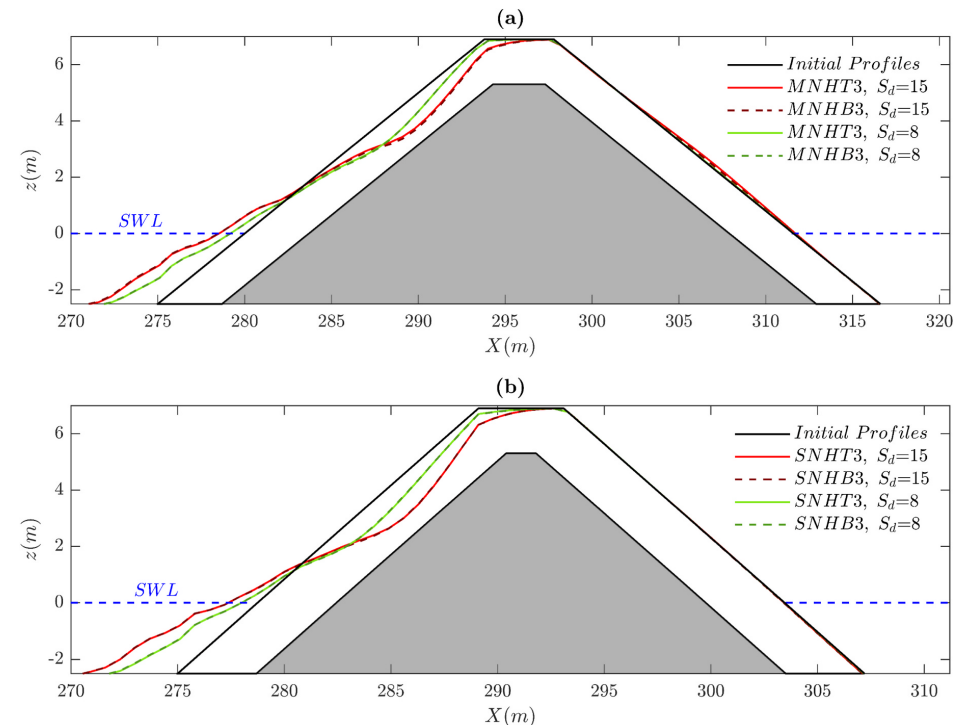
Probabilistic modeling
Cumulative damage
Performance-based design
Survival analysis
Physics-based simulation
Extreme events
Coastal infrastructure resilience
Statistical classification
Synthetic storm generation
Hydrodynamic forcing
Fragility analysis

ABSTRACT

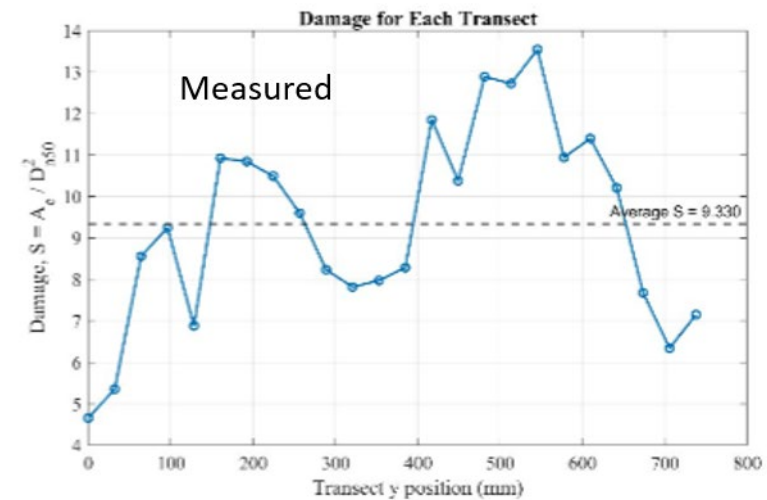
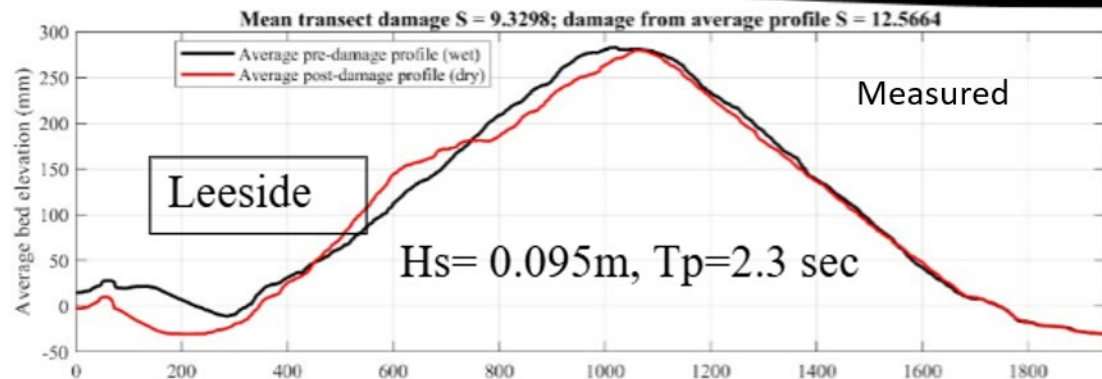
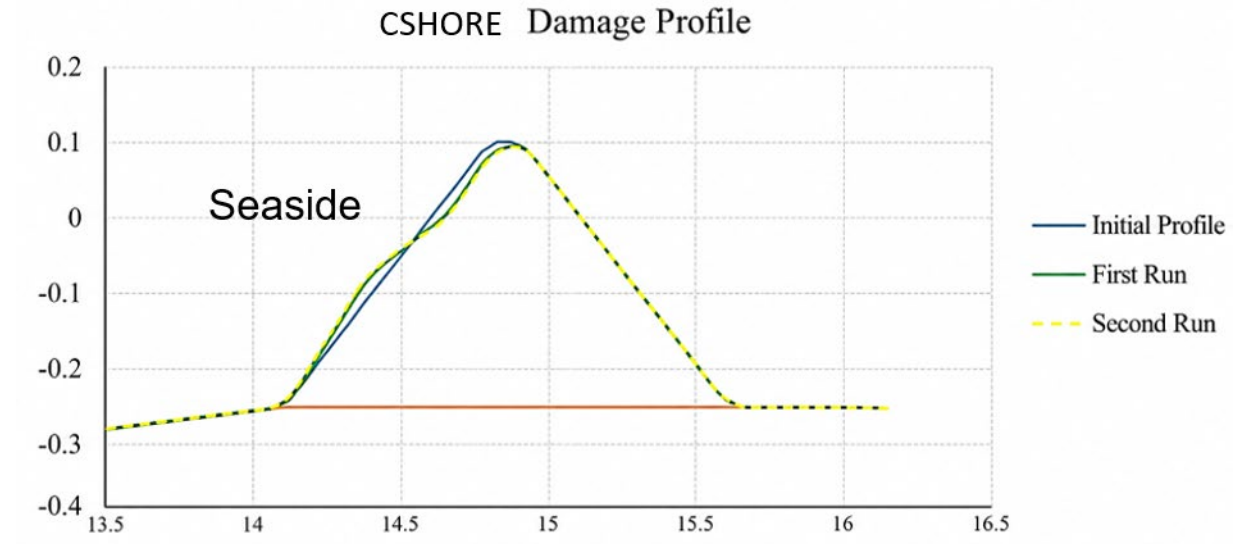
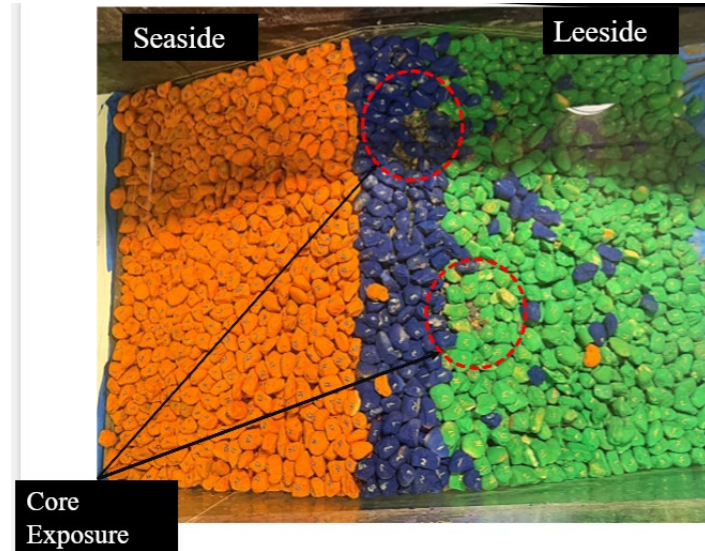
This study presents a simulation-based framework to evaluate the long-term performance of rubble-mound breakwaters under hydrodynamic forcing and sea level rise (SLR). The framework integrates synthetic storm generation (StormSim), process-based damage modeling (CSHORE), and suite of statistical methods, including Cox hazard models, Principal Component Analysis (PCA), and machine learning classification, to identify performance-critical parameters and assess structural resilience under non-stationary forcing conditions.

The methodology is demonstrated through application to 32 breakwater configurations at a site within the North Atlantic Coast Comprehensive Study region, with 1000 synthetic lifecycles spanning 50 years. Key findings for the tested wave climate include: steeper slopes exhibited 20-fold higher initial failure risk than milder slopes, though this differential decreased over time; armor stone size and porosity strongly influenced time-to-repair; spectral wave height and water level dominated damage prediction (odds ratios, OR = 15.6 and 4.6); and SLR incorporation accelerated damage progression, particularly after Year 30.

The framework successfully quantified time-dependent reliability, extracted spatial damage patterns via PCA, and ranked forcing parameters by importance. While quantitative results are specific to North Atlantic conditions and selected breakwater configurations, the methodology is transferable given appropriate storm climatology, SLR projections, and structural parameters. The results demonstrate the importance of integrating sea level rise into performance-based coastal infrastructure design through physics-based modeling coupled with comprehensive statistical assessment.



Givehki progress report.



5. Reproduce all experiments with Boussinesq model to produce consistent set of hydrodynamics

- Validated model
- Developed analysis software with reflection analysis, filtering
- Described infragravity development in shallow water

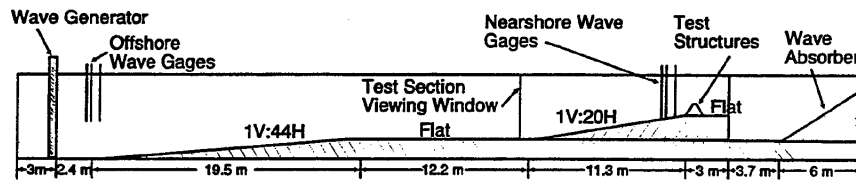
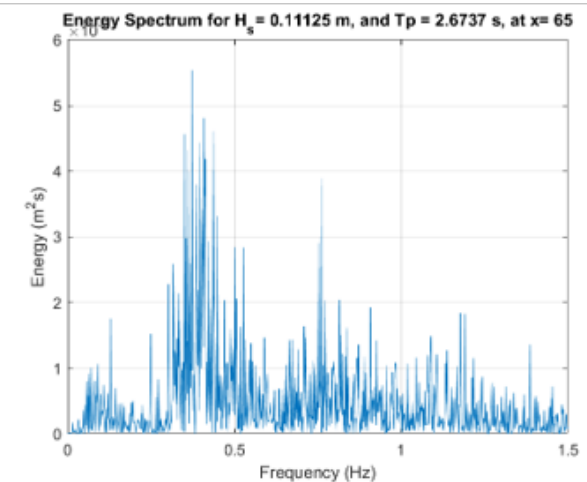
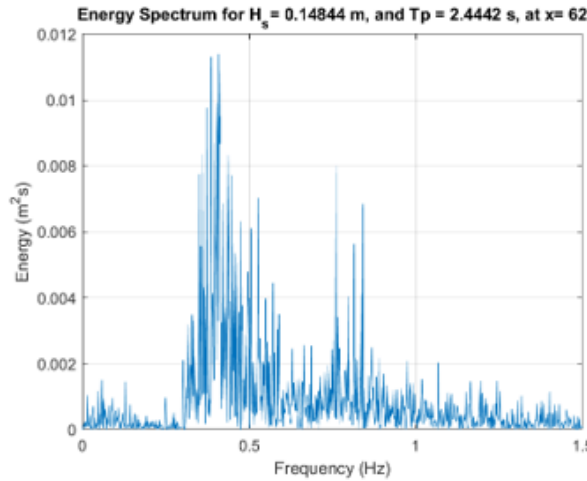
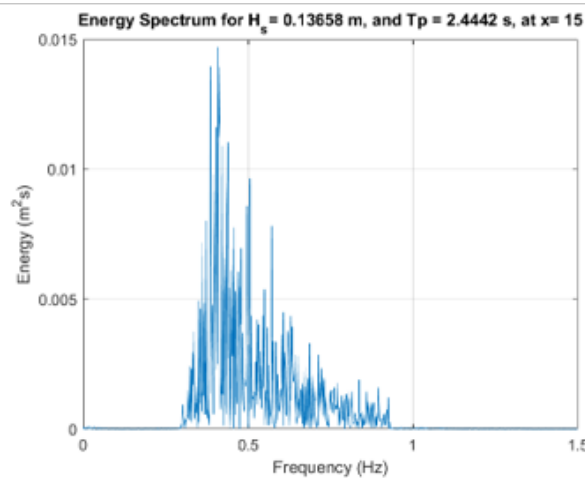
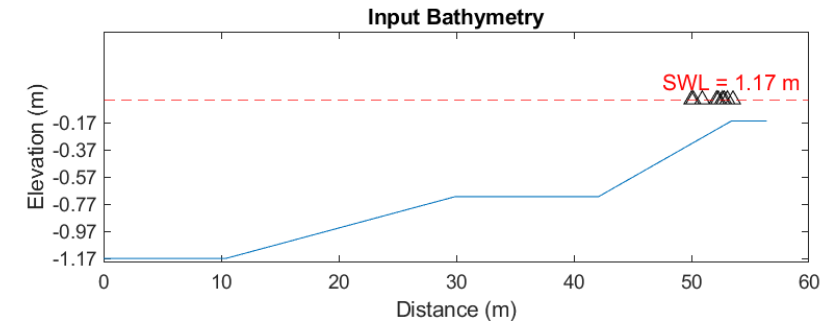


Figure 4.1 Flume profile for damage experiments





SUMMARY



- USACE manages over 1000 rubble mounds most of which are low-crested under extreme loading
- CHS/CHESS system provides hazards and probabilistic analysis software for coastal structures that uses up-to-date methods for rubble mound design and life-cycle simulation
- Seaside stability/damage methods have been recently and independently validated in leading journals
- Leaside methods have limited data support, validation
- CEM and DELOS low-crested or submerged methods are not very useful
- Present work is focused on hindcasting with CHESS, developing continuous physics and validating process models
- Neural network continuous physics model appears to solve primary problems
- CSHORE process model RM damage prediction is being improved



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Questions?

