THE EFFICACY AND DESIGN OF COASTAL PROTECTION USING LARGE WOODY DEBRIS

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Abstract

Those who frequent the coastline may be accustomed to seeing driftwood washed onshore, some of it having seemingly found a home there for many years, others having been freshly deposited during the last set of storms; However, if a passerby were to take a closer look at the driftwood on the coastline, they may notice that some of these logs – also known as Large Woody Debris (LWD) – are anchored in place, a practice which is generally used for the purpose of stabilizing the shoreline or reducing wave-induced flooding. Records of existing anchored LWD project sites date back to 1997 and anecdotal evidence suggests that the technique has been used since the mid-1900's in coastal British Columbia (BC), Canada, and Washington State, USA. Now, with an increased demand for natural and nature-based solutions, the technique is again gaining popularity. Despite this, the design of anchored LWD has largely been based on anecdotal observations and experience, as well as a continuity of design practices from the river engineering field. To date, there is no known peer-reviewed literature on the design or efficacy of LWD protection systems in a coastal environment.

In 2019, the "Efficacy and Design of Coastal Protection using Large Woody Debris" research project was initiated to determine if LWD are effective at stabilizing the shoreline under wave action, if they are effective at reducing wave run-up, and if they are durable enough to meet engineering requirements for shore protection. In addition, the project aimed to determine the optimum configuration of LWD for design purposes. To meet these objectives, this study included the following work: (1) field studies of existing LWD installations, (2) experimental modeling of beach morphology with and without LWD structures, (3) experimental modeling of wave run-up with and without LWD structures, and (4) development of preliminary design guidance.

The first phase of the project included field investigations at 15 existing anchored LWD sites in coastal BC and Washington State. Site characteristics, design techniques, and durability indicators were examined and correlated to a new design life parameter: 'Effective Life'. Six primary installation techniques were observed: Single, Multiple, Benched, Stacked, Matrix, and Groyne. Observed durability and/or performance issues included: missing LWD, erosion, arson, wood decay, and anchor corrosion/damage. The Effective Life of anchored LWD was found to be strongly correlated to the tidal range and the upper beach slope for all installation types, and the LWD placement elevation relative to the beach crest elevation for single, shore-parallel structures.

The many noted durability issues and ineffectiveness as mitigating erosion indicates that existing design methods for anchored LWD have not generally been effective at providing coastal protection and meeting engineering design life requirements.

A comprehensive set of over 60 experimental tests were completed as part of the overall research program. Thirty-two (32) tests were analyzed as part of this study relating to the morphological response of a gravel beach with and without various LWD configurations. The tests were conducted within a wave flume at the National Research Council's Ocean, Coastal and River Engineering Research Centre (NRC-OCRE), at a large scale (5:1) based on site characteristics and LWD design characteristics made during the previous field investigations. Tests were also conducted to assess experiment repeatability, sensitivity to test duration, sensitivity to wave height, wave period, and relative water level, influence of regular waves, and influence of log roughness. The position of the most seaward LWD (whether considering distance or elevation) was found to be strongly linked to morphological response. A theoretical relationship was developed between LWD elevation and sediment volume change. Configurations which included LWD placement below the still water level, such as the Benched configuration, were found to be most effective at stabilizing the beach profile.

As part of the experimental modeling program, 24 tests were also conducted for the purpose of estimating the effect of LWD design configuration on wave run-up. In total, six different beach and LWD configurations were tested under a base set of four regular wave conditions. The study findings indicated that anchored LWD may increase wave run-up relative to a gravel beach with no structures. In particular, configurations with more logs tended to result in higher wave run-up. However, additional research is needed on the effect of LWD on wave run-up to confirm and expand these findings.

There are a number of potential engineering, ecological, social, and economic benefits associated with anchored LWD installations if designed, installed, and monitored appropriately for the site conditions and user needs. To realize these potential benefits, significant additional research is needed on the topic. One of the most significant barriers to usage is a lack of information on how to effectively anchor LWD structures. However, this research project provides a baseline for future comprehensive studies on the effect and design of coastal protection using LWD. The project provides preliminary design considerations for the usage of LWD as coastal protection and contributes to the growing body of literature on nature-based solutions.

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Symbols

C_r Reflection coefficient [%]

C_u Coefficient of uniformity of soil [-]

C_z Coefficient of gradation of soil [-]

d Water depth [m]

d_b Water depth at wave breaking [m]

dh Change in beach profile elevation [m]

dh_{max} Maximum change in elevation, analogous to the accretion height at the beach crest [m]

dh_{min} Minimum change in elevation, analogous to the depth of erosion at the beach trough [m]

D Sediment grain size diameter [mm]

D_{50} Mean grain size diameter corresponding to 50% finer [mm]

D_{av} Average diameter of Large Woody Debris [m]

D_{LWD} Diameter of Large Woody Debris model [m]

D_x Grain size diameter corresponding to X% finer [mm]

e Void ratio [-]

f_p Peak frequency [Hz]

Fr Froude Number [-]

g Gravitational acceleration constant, equal to 9.81 [m/s^2]

G_s Specific gravity [-]

H Wave height [m]

H_0 Deepwater wave height [m]

$H_{1/3}$ Estimate of significant wave height, mean height of the highest 3rd of waves [m]

h_c Crest Height [m]

h_{LWD} Crest elevation of LWD [m, msl]

H_m Average wave height [m]

H_{m0} Estimate of significant wave height from spectral analysis, calculated as $4\sqrt{m_0}$ [m]

H_s Significant wave height [m]

K Hydraulic Conductivity [m/s]

L Wave length [m]

L_0 Deep water wave length [m]

L_{av} Average length of Large Woody Debris [m]

L_m Characteristic length in model [m]

L_p Characteristic length in prototype [m]

L_{photo} Long axis of particle within photo sieve analysis [m]

M Mass [kg]

n Porosity [%]

n Sample size [-]

N Number of waves [-]

r Correlation coefficient [-]

R Wave run-up, above the still water level [m]

$R_{2\%}$ Wave run-up exceeded by 2% of waves, above the still water level [m]

Re Reynolds Number [-]

R_{max} Maximum wave run-up, above the still water level [m]

s Beach slope, H:V [-]

s_{lower} Slope of the upper beach [-]

s_{upper} Slope of the lower beach [-]

S Wave steepness [-]

S_{om}	Wave steepness associated with mean wave period [-]
S_{photo}	Short axis of particle within photo sieve analysis [m]
$S(f)$	Wave spectrum for a given frequency [-]
t	Time [s]
t_{eqlb}	Time until beach equilibrium criteria is met [s]
T	Wave period [s]
$T_{1/3}$	Significant wave period, associated with the mean of the highest 3rd of waves [s]
T_m	Average wave period [s]
T_p	Peak wave period, associated with the peak of the spectrum [s]
u	Wind speed [m/s]
U	Flow velocity [m/s]
V	Volume [m^3]
V_{avg}	Average volume change in the beach profile, based on the average of V_{crest} and V_{trough} [m^3]
V_{crest}	Volume change on the upper slope of the beach profile (inland of X = 41.65 m) [m^3]
V_E	Volume of ellipsoid revolving around the long axis of a sediment particle [m^3]
V_{trough}	Volume change on the lower slope of the beach profile (seaward of X = 41.65 m) [m^3]
w	Moisture content [-]
X	Distance in the x-direction, increasing away from the wave paddle at the north end of the flume [m]
X_{crest}	Location of dh_{max} in the X-direction[m]
X_{trough}	Location of dh_{min} in the X-direction [m]
Y	Distance in the y-direction, increasing away from the east wall of the flume [m]
Z	Distance in the z-direction, increasing upward from the still water level (SWL) [m]
Z_{crest}	Vertical elevation of dh_{max} [m]

Z_{LWD} Elevation of LWD above water surface [m]

Z_{trough} Vertical elevation of dh_{min} [m]

α Beach slope angle relative to horizontal [°]

γ Peak enhancement factor, JONSWAP spectrum [-]

γ_b Wave breaking index [-]

γ_d Dry specific weight of sediment sample [kN/m^3]

γ_{Td} Specific weight of wood [kN/m^3]

γ_w Specific weight of water [kN/m^3]

γ_f, γ_k Influence factors for wave run-up based on surface roughness and hydraulic conductivity [-]

η Water surface elevation relative to the still water level [m]

λ Model scale ratio [-]

ν Kinematic fluid viscosity [m^2/s]

ξ Iribarren number, also referred to as the surf similarity parameter [-]

ξ_0 Iribarren number for deep-water conditions [-]

ξ_{av} Iribarren number as calculated for regular waves, based on H_m and T_m [-]

ξ_p Iribarren number as calculated for irregular waves, based on H_s and T_p [-]

π The numerical value, approximately equal to 3.141592

ρ_s Density of dry sediment [kg/m^3]

ρ_w Density of water [kg/m^3]

σ Standard Deviation [-]

τ Shear stress [N/m^2]

φ Dimensionless hydraulic conductivity [-]

ω Angular frequency, calculated as $2\pi/T$ [m]

Abbreviations

ADV	Acoustic Doppler Velocimeter
AEP	Annual Exceedance Probability
ASL	Above still-water level
AWA	Active Wave Absorption
BC	British Columbia, Canada
CHS	Canadian Hydrographic Service
FFT	Fast Fourier Transform
GPS	Global Positioning System
HHWLT	Higher High Water Large Tide (in Canada)
JONSWAP	Joint North Sea Wave Analysis Project
LLWLT	Lower Low Water Large Tide (in Canada)
LWD	Large Woody Debris
MHHW	Mean Higher High Water (in USA)
MLLW	Mean Lower Low Water (in USA)
MSL	Mean Sea Level
NOAA	National Oceanic and Atmospheric Administration
NBS	Nature-Based Solutions
NNBF	Natural and Nature-Based Features
NRC	National Research Council of Canada
OT	Overtopping
RMSE	Root Mean Square Error
RTK	Real-Time Kinematic
SLR	Sea Leve Rise

SWL	Still Water Level
WA	Washington State, USA
WG	Wave Gauge
UTM	Universal Transverse Mercator Coordinate System
VRS	Virtual Reference Station

Chapter 1. Introduction

1.1 Background

Natural accumulations of Large Woody Debris (LWD) are a pervasive feature of coastal British Columbia, Canada, and Washington State, USA (Braudrick et al., 2001; Grilliot, Walker, & Bauer, 2018; Heathfield & Walker, 2011; Rich et al., 2014) (Figure 1). They are understood to have important ecological benefits, such as the moderation of substrate temperatures and provision of habitat complexity (Brennan et al., 2009; Gonor et al., 1988; Heathfield & Walker, 2011; Lee et al., 2018; Sass, 2009). Some studies have found that natural LWD accumulations promote the accretion of fine sand in beach dune systems during aeolian sediment transport (Eamer & Walker, 2010; Gonor et al., 1988; Heathfield & Walker, 2011). Natural LWD matrices may also serve to trap coarse sediment on the upper beach and prevent wave run-up (Kennedy & Woods, 2012).

Figure 1 – Natural accumulations of woody debris in Richmond, BC

Attempts have been made to anchor LWD into the shoreline to harness these benefits and reduce coastal erosion and wave-induced flooding, while providing ecological services. Zelo et al (2000) and Johannessen et al. (2014) provide records of project sites where anchored LWD was installed as coastal protection as early as 1997 in Puget Sound, Washington, USA (Figure 2). Anchored LWD projects are also prevalent throughout the Strait of Georgia, BC, Canada (e.g. Northwest Hydraulic Consultants, 2014), but are not as well documented. Despite the frequent and historical usage of anchored LWD as coastal protection, there is no known peer-reviewed literature on the design or efficacy of LWD protection systems in a coastal environment.

Figure 2 – Coastal protection using anchored LWD in Comox, BC

To date, the design of anchored LWD has been largely based on anecdotal observations and experience, as well as a continuity of design practices from the river engineering field, where significant literature is available on its function and design (Bocchiola, 2011; Douglas Shields Jr., Morin, & Cooper, 2004; Gonor, Sedell, & Benner, 1988; Hilderbrand, et al., 1998; Hills & Street, 2007; Kail et al., 2007; Rafferty, 2017; Sass, 2009). Despite the significant physical and ecological differences between fluvial and marine environments, design guidance and permitting documents promote the usage of LWD as an ecologically friendly and effective means of coastal protection (eg. Johannessen et al., 2014; Washington Department of Fish and Wildlife, 2016). However, these works do not provide a basis for anchoring LWD to prevent erosion or wave run-up and they do not provide sufficient engineering guidance or best practices for doing so. As such, Jones et al. (2016) recommended avoiding the use of LWD in projects in tidal or coastal environments unless the need was explicitly justified, and to treat its use as experimental paired with rigorous monitoring.

Before anchored LWD can be reliably used as coastal protection and serve as an alternative to more conventional techniques, research is needed to understand the geomorphological and hydraulic behaviour of LWD under wave-action. Research is specifically needed to determine the efficacy of anchored LWD in stabilizing the shoreline and reducing wave-induced flooding in a coastal environment (Zelo et al., 2000; Shipman, personal comm, Sept. 20, 2018).

In 2019, a collaborative research project was initiated between the University of Ottawa and the Ocean, Coastal, and River Engineering Research Centre of the National Research Council of Canada (NRC-OCRE) to undertake the first research project focused on understanding the design and efficacy of coastal protection using LWD. The comprehensive research project included: (1) field studies of existing LWD installations, (2) experimental modeling of beach morphology with and without LWD structures, (3) experimental modeling of wave run-up with and without LWD structures, (4) experimental modeling of beach morphology and overtopping of vertical structures (stacked log wall and seawall), (5) preliminary numerical modeling, and (6) development of design guidance and best practices. The present study focused on items 1, 2, 3, and 6.

1.2 Objectives

To support the future usage of LWD structures, the present study aimed to answer the following questions:

1. Are LWD effective at stabilizing the shoreline under wave action?
2. Are LWD effective at reducing wave run-up?
3. Are LWD durable enough to meet engineering requirements for shore protection?
4. What is the optimum configuration of LWD for design purposes?

An extensive field-based study and experimental modeling program was developed and undertaken with the objective of answering these questions. The objectives of the field study were to survey existing LWD installations to better understand the design methods currently being employed, determine the durability and longevity of such structures, and provide a first indicator of the efficacy of LWD structures at stabilizing the shoreline and reducing wave run-up. The field study also served as a basis for the development of the experimental set-up and testing program. The objectives of the experimental program were to understand better the influence of anchored LWD on beach morphology and wave run-up, under a range of wave and water levels, in a controlled setting. The experimental program also allowed for the testing of multiple LWD configurations to help optimize designs. Ultimately, this study aimed to provide a starting point for future research on coastal protection using LWD and provide a basis to develop evidence-based design guidance and best practices for coastal protection using anchored LWD.

1.3 Scope

This study investigated the efficacy of coastal protection using anchored LWD in stabilizing the shoreline and reducing wave-induced flooding, and provided preliminary design guidance for the best usage of this technique. The study involved both field studies and an extensive experimental modeling program.

1.3.1 Field Studies

Field studies focused on assessing the efficacy of existing coastal protection structures using LWD in both BC, Canada, and Washington State, USA. Based on a desktop review and anecdotal observations, 28 project sites were identified as having used LWD as a component of the project. The selected sites generally aimed to reduce wave-induced flooding and/or shoreline erosion primarily through the use of shore-parallel anchored LWD. Between June – August 2019, the author attempted to conduct field investigations at all 28 sites. The status of these site visits is summarized below and detailed in Appendix A.

- In-depth surveys at 8 sites in BC (using an RTK GPS)
- Partial surveys at 7 sites in WA (using a laser level and rod, where site access permitted)
- 5 sites visited, but no anchored LWD was identified
- 6 sites not visited due to access issues
- 2 sites visited, but no permission granted, so surveys were not completed and data could not be reported

Measurements of the LWD placement elevation and beach slope were made using an RTK GPS within BC, and a rotary laser level and rod within Washington where site-access permitted (Figure 3). Surficial sediment sizes were estimated using digital grain size analysis (see Appendix C). Additional observations were made on log size, anchoring technique, signs of beach erosion/accretion, and signs of damage to the LWD or anchoring system. The template field survey protocol used at each site is included in Appendix A.

Figure 3 – Rotary laser level in place above a beach transect in Bremerton, Washington

Field survey results were used to develop a preliminary understanding of anchored LWD durability, efficacy, and design methods. Results were also used as a basis for the development of the experimental modeling program, such that tests could reflect authentic site conditions and LWD design characteristics. A summary of the field study locations, names, and survey status for each site are included in Appendix B.

1.3.2 Experimental Modeling

The experimental modeling program was planned by the author based on the field survey results and for implementation in the steel-wave flume (SWF) at the National Research Council's Ocean, Coastal and River Engineering centre (NRC-OCRE) in Ottawa, Canada. The overall experimental research program can be broken into the following three main research topics, the first two of which are assessed in this thesis:

1. Beach morphology with and without LWD structures
2. Wave run-up with and without LWD structures
3. Wave reflection and overtopping for vertical structures

The experiments included constructing a sloping gravel beach, installing a range of structures on the beach, and observing hydro-morphological behaviour under a set of three base wave conditions. Seven beach-structure configurations were tested, including one configuration with no structures, five configurations using simulated anchored LWD, and two configurations with vertical structures (Figure 4)[1]. Experiments were also conducted to assess test repeatability, influence of test duration, sensitivity to wave height, sensitivity to wave period, influence of relative water level, influence of regular waves, and influence of log roughness. Experiments were conducted by the author and P. Falkenrich over a four-month period (see Appendix E. for the daily log of experiments), resulting in over 60 completed simulations. A detailed list of experiments is included in Appendix F.

[1] This thesis focuses on the first two research topics, which included assessment of test configurations illustrated in Figure 4 a-e. The third research topic forms the thesis of P. Falkenrich (2020), which assesses and compares configurations shown in Figure 4 a, b, f, and g.

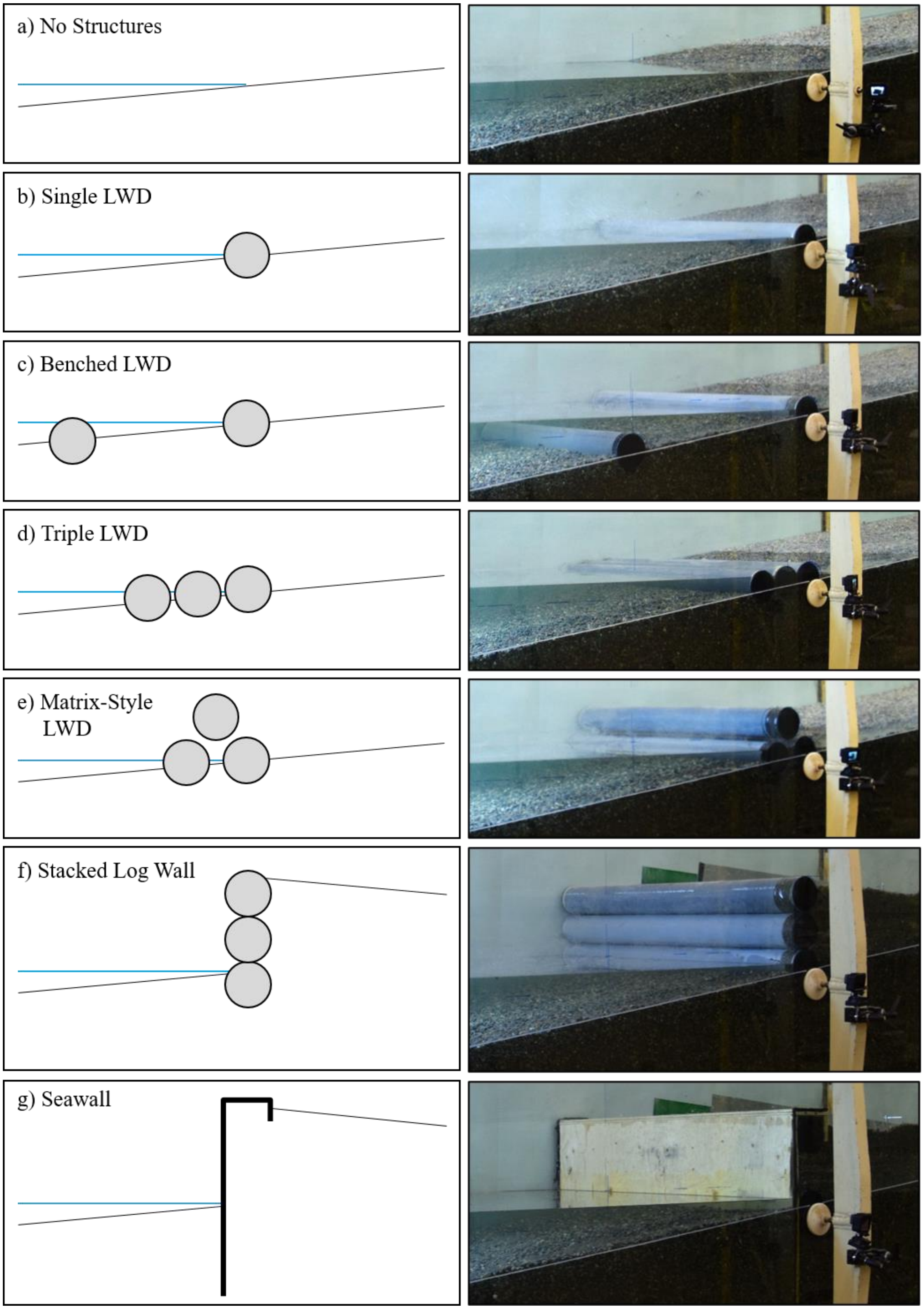

Figure 4 – Configurations tested during the overall experimental program. Not to scale. Configurations a-e were assessed in detail as part of this study.

The overall experimental program focused on assessing the hydro-morphological impacts of various shore-parallel LWD structures in comparison to more traditional shoreline protection techniques (e.g. a beach with no structures and a seawall). As a result of physical and temporal constraints, the experimental program had several limitations, including the following:

- Only one sediment type and size were tested. The resulting beach was highly porous and homogeneous, whereas many of the beaches in BC and Washington State have mixed-sediment and/or semi-impermeable layers.
- A consistent initial beach slope of 8H:1V was used throughout the tests, reflective of the average beach slope observed during field studies.
- Model logs were constructed with PVC pipe with a 0.114 m diameter to simulate a prototype log with a 0.6 m diameter, typical of that observed during the field studies. No additional log diameters were tested.
- Model logs were fixed in place on the beach by applying lateral pressure to the side walls of the flume, resulting in a static structure throughout the test duration. Although this behaviour is representative of anchored LWD at the time of initial installation, all anchoring systems observed during the field studies would allow for some dynamic movement of the LWD if sufficient undermining/scouring occurred.
- Due to the 2-Dimensional nature of the wave flume, all waves were normally incident to the beach and all tested configurations were shore parallel. These tests therefore do not consider longshore transport, which is understood to play a significant role in erosion potential at many of the study sites (Finlayson, 2006).
- All tests were completed with smooth logs. One simulation was completed to test the sensitivity of beach morphology and run-up results to a rough log.

The implementation of various measurement techniques and instrumentation formed a significant part of the experimental scope of work. A summary of the measurement techniques/analyses used as part of this program are as follows:

- Sieve analyses, specific gravity analyses, and dry density tests (see Appendix D).
- A Total Station to measure the precise location of instrumentation and reference markers
- Eight capacitance wire wave gauges to measure water surface elevations

- One Acoustic Doppler Velocimeter (ADV) to measure water velocities[2]
- Photogrammic reconstruction of the beach surface to measure elevations pre- and post-simulations
- Laser scanner to measure the beach surface elevations for comparison with the photogrammetry technique
- Digitized and rectified time-lapse photos of the beach profile (through the glass wall of the flume) to estimate beach profile changes throughout the simulations
- Manual identification of the water surface elevation from digitized video frames (through the glass wall of the flume) to estimate maximum wave run-up
- One GoPro to take high-speed videos and photos during the simulations
- A metal chute, bin, and two capacitance wire wave gauges to measure wave overtopping of vertical structures[2]

A description of the acquisition and analysis for the wave gauge and ADV data is provided in Appendix G. A comparison of methods used to measure breach profile elevations (photogrammetry, laser scanner, and digitized profile photos) is also included in Appendix H.

1.4 Contributions and Novelty of Study

This is the first systematic research project focused on understanding the design and efficacy of coastal protection using anchored Large Woody Debris. It encompasses both field studies and an extensive experimental modeling program to develop the basis for the first evidence-based design guidance for this technique.

No known previous research has been conducted on the hydraulic or morphological interactions of anchored LWD under wave action. Previous research on LWD has focused only on natural accumulations exposed wave action and aeolian forcing (Kennedy & Woods, 2012) or aeolian sediment transport around natural accumulations (e.g. Eamer & Walker, 2010; Grilliot, Walker, & Bauer, 2019; Heathfield & Walker, 2011).

Several reports have included limited field studies on coastal protection projects using anchored LWD in Washington, USA (Gerstel & Brown, 2006; Johannessen et al., 2014; Zelo et

[2] Note: This data was collected to facilitate future research projects and not analyzed as part of this thesis.

al., 2000). However, to the author’s knowledge, this study is the first systematic (using a defined set of measurement techniques and comparative metrics) and independent (where the author was not involved in design or construction of the initial installations) field review of existing projects using anchored Large Woody Debris. The conducted field study also included 15 sites, spanning both British Columbia and Washington, making it the most expansive field study to date. The field study protocol (Appendix A) provides a template for future low-effort and low-cost field studies, allowing for replication and extension of this research. In addition, the field study provides the first dataset on anchored LWD design practices in a coastal environment and preliminary assessment of their efficacy and durability.

Numerous experimental models have been completed on LWD in rivers (e.g. Bocchiola, 2011; Braudrick et al., 2001; Hygelund & Manga, 2003; Perry et al., 2018; Wallerstein et al., 2001). This is the first study to include an experimental testing program on Large Woody Debris under wave action. It is differentiated from experimental modeling of pipelines under wave action because of the coarse sediment size and location of the structure within the swash zone, as opposed to a submerged structure which is typical of pipeline research. Significant experimental research has also been completed on beach morphology; however, research is still limited for gravel beaches specifically. Because the experimental set-up closely resembles that of Atladottir (2008), this study both contributes to the limited body of work on gravel beach behaviour and as an independent replication of previous study results. This study further contributes to existing experimental research on wave run-up on highly porous beaches, which is of significant importance in the assessment of coastal flood potential and development of flood protection in the Pacific Northwest. The experimental set-up and program are also closely linked to field study observations of the site characteristics and LWD design characteristics, allowing for direct engineering application.

The experimental testing program also used three techniques to measure the beach surface elevation and elevation changes within the flume, including the use of a relatively novel technique (structure from motion photogrammetry). The accuracy of these three techniques are compared and discussed. Because existing definitions of ‘beach equilibrium’ are not well defined in previous experimental programs (e.g. Atladottir, 2008), this study also presents a novel and systematic method for quantifying beach equilibrium (Appendix I.3).

It is expected that numerical modeling will play an important role in future research on LWD under wave action, due to the physical and economic restrictions of experimental modeling. To date, limited numerical work has been conducted on either anchored or natural LWD, with the exception of one study on natural LWD in estuaries (Huff, 2015). As part of the experimental set-up, an ADV was installed to measure velocities at the toe of the beach. Although the velocity data was not analyzed as part of this thesis, the program resulted in an extensive experimental dataset, from which future numerical simulations may be calibrated and validated.

Previously, the design of coastal protection using LWD has been based on 'grey' literature (e.g.; Zelo et al., 2000), design guidance for rivers (e.g. D'Aoust & Millar, 2000; Hilderbrand et al., 1998; Hills & Street, 2007; Kail et al., 2007; Rafferty, 2017; USBR and ERDC, 2016), and anecdotal experience. This research project provides the first systematic research and preliminary design guidance related to coastal protection using LWD (see Section 6.1.4).

This research project ultimately provides a thorough baseline for future comprehensive studies – including field studies, experimental modeling, and numerical modeling – on the effect and design of coastal protection using LWD. The project provides an extensive dataset as a base for the development of best practices for the design and usage of LWD as coastal protection. It also contributes to the growing body of literature on nature-based solutions (otherwise known as natural and nature-based features, NNBF).

1.5 Publications

The following are publications affiliated with this research project, of which the author is a contributor.

<u>Conference Proceedings</u>

1. **Wilson, J.,** Nistor, I., & Mohammadian, M. (2019). *Nature-based coastal protection using Large Woody Debris: What is it and does it work?.* Poster Presentation at the Cold Regions Living Shoreline Community of Practice Workshop. Halifax, BC, Canada.

2. **Wilson, J.,** Nistor, I., Mohammadian, M., & Cornett, A. (2020). *Assessing the Legacy of Large Woody Debris as Coastal Protection in BC and Washington.* Salish Sea Ecosystem Conference. Vancouver, BC, Canada.

3. **Wilson, J.,** Nistor, I., Mohammadian, M., & Cornett, A. (2020). *Nature-based coastal protection using Large Woody Debris.* 37th International Conference on Coastal Engineering. Sydney, Australia. **(Abstract accepted for oral presentation and conference proceedings).**

Journal Publications

1. **Wilson, J.,** Nistor, I., Mohammadian, M., Cornett, A., & Lamont, G. (2020). *Field Investigation of Anchored Large Woody Debris Coastal Protection Structures*. **(Submitted to the Journal of Ecological Engineering, Elsevier).**

2. **Wilson, J.,** Falkenrich, P., Cornett, A., Nistor, I., & Mohammadian, M. (2020). *Experimental Modeling of Coastal Protection using Large Woody Debris: Morphological Response.* **(Submitted to Coastal Engineering, Elsevier).**
3. **Wilson, J.,** Falkenrich, P., Mohammadian, M., Cornett, A., & Nistor, I. (2020). *Experimental Modeling of Coastal Protection using Large Woody Debris: Wave Run-Up.* **(In preparation for submission to the Canadian Journal of Civil Engineering, CSCE).**
4. P. Falkenrich, **Wilson, J.,** Nistor, I., Goseberg, N., Cornett, A., Mohammadian, M. (2020). *Nature-Based Coastal Protection using Large Woody Debris (LWD) and vs Seawalls: A Physical Model Study of Beach Morphology and Wave Reflection*. **(In preparation).**

1.6 Outline of Thesis

Chapter 1 provides an overview of the background, objectives, scope, and novelty of this project. It also outlines publications affiliated with this project.

Chapter 2 provides a comprehensive literature review related to coastal protection using LWD. The chapter first provides background on the history and usage of both conventional and nature-based methods of coastal protection. The chapter then summarizes research related to LWD in the natural and built environments, outlines previous case studies, reviews existing design guidance and recommendations for anchored LWD in both riverine and coastal settings, and summarizes key ecological considerations. The chapter reviews background information on wave-related theories and key processes, and on experimental modeling methodologies and considerations. Finally, based on the literature review, a discussion of knowledge gaps, research needs, and key considerations is provided.

Chapter 3 includes a manuscript titled "*Field investigation of Anchored Large Woody Debris Coastal Protection Structures*", which has been submitted to the *Special Issue on Nature Based Protection* in the *Journal of Ecological Engineering (Elsevier*). The manuscript presents the results of field studies conducted by the author at fifteen existing anchored LWD sites in coastal

British Columbia (BC), Canada, and Washington State, USA. The field studies were focused on documenting current design practices, site characteristics, and potential durability concerns. Six primary design configurations (single, multiple, benched, stacked, matrix-style, or groyne-style) and two primary methods for anchoring were characterized (cable or chain attached to rock ballast). Significant durability issues were outlined, including: missing LWD, erosion, arson, wood decay, and anchor corrosion/damage. Based on the findings, preliminary design considerations and future research recommendations were made.

Chapter 4 includes a manuscript titled "*Experimental Modeling of Coastal Protection using Large Woody Debris: Morphological Response*", which has been submitted to *Coastal Engineering* (Elsevier). The manuscript presents the results of a portion of experimental modeling results related to the geomorphological response of a gravel beach with anchored LWD structures under wave action. The test set-up and tested configurations of LWD structures was designed based on the author's findings from the previous field investigation phase of the research project. Tests were conducted to assess experiment repeatability, sensitivity to test duration, sensitivity to wave height and period, influence of the relative water level, influence of regular waves, influence of LWD configurations, and influence of log roughness. The elevation or position of the most-seaward LWD was found to be strongly linked to morphological response and a theoretical relationship was developed between LWD elevation and sediment volume change. Additional design considerations and future research recommendations were also outlined.

Chapter 5 includes a manuscript titled "*Experimental Modeling of Coastal Protection using Large Woody Debris: Wave Run-Up*", which is currently in preparation with intent to submit to the *Canadian Journal of Civil Engineering (CSCE).* The manuscript presents the results of the experimental study conducted by the author on the influence of various anchored LWD configurations on wave run-up under a base set of four regular wave conditions. Similarly to the morphological experiments, the tested LWD configurations reflected the author's findings from the field investigations. Anchored LWD was found to result in higher wave run-up than a gravel beach with no structures. Notably, configurations with more logs were generally found to produce the highest run-up. Study limitations, future research recommendations, and preliminary design guidance related to wave run-up are outlined.

Chapter 6 presents the main conclusions of this research project, outlines preliminary design guidance, and provides recommendations for future research.

Appendices A – K provide supplemental information on the procedures and results of the field study and experimental modeling programs.

Chapter 2. Literature Review

2.1 Background on Coastal Protection

2.1.1 Conventional Approaches

Traditionally, ‘hard’ engineering approaches have been used to mitigate shoreline erosion, promote sediment accretion, prevent flooding, and extend upland areas seaward of the natural boundary (Bruun, 1972; USACE, 2002).

The Coastal Engineering Manual (USACE, 2002) defines ‘hard’ approaches as a “general term applied to impermeable coastal defense structures of concrete, timber, steel, masonry, etc. which reflect a high proportion of incident wave energy”. This definition draws a distinction between impermeable and permeable structures, the material type, and the level of wave reflection. Conversely, others (e.g. Gittman et al., 2016; Morris et al., 2018) generally define ‘hard’ approaches as the installation of engineered structures with the purpose of preventing or stabilization shorelines or providing flood protection. However, this definition is so broad that it tends to encompass any material designed and placed on the shoreline with an engineering intent, including a gravel beach fill, which is generally considered a ‘soft’ approach. Alternatively, Gianou, (2014) states that the distinction between ‘hard’ and ‘soft’ approaches is unclear, but is more related to the intent of a project as opposed to its specific characteristics.

Despite the range of definitions, there is a general consensus that ‘hard’ approaches include conventional coastal engineering structures such as seawalls, bulkheads, jetties, breakwaters, and rip-rap revetments (eg. Bilkovic, Michell, La Peyre, & Toft, 2017; Chapman & Underwood, 2011; Gianou, 2014; Gittman et al., 2016; Johannessen et al., 2014; Lee et al., 2018; USACE, 2002; Zelo et al., 2000).

Although ‘hard’ approaches are the conventional means of protecting coastal areas, there is a growing body of literature dedicated to demonstrating that these types of structures may exacerbate erosional processes and degrade coastal habitat (e.g. Bulleri & Chapman, 2010; Firth et al., 2014; Perkins, Ng, Dudgeon, Bonebrake, & Leung, 2015; Toft, Ogston, Heerhartz, Cordell, & Flemer, 2013). Zelo et al. (2000) summarized many of the perceived issues associated with 'hard’ coastal structures, including cutting off upland sediment supply, increasing wave reflectivity at the shoreline, coarsening of the beach due to increased wave energy from reflection, and loss of upland vegetation.

Coyle & Dethier (2006) undertook a literature review of both peer-reviewed and 'grey' literature on the topic of shoreline armoring, with specific application to Puget Sound, Washington. They found that on shorelines already undergoing long-term erosion, the presence of a seawall ultimately resulted in the further narrowing of the foreshore. However, whether seawalls actively contributed to the erosion through increased wave reflection or loss of sediment supply remained uncertain in the literature. Notably, the authors found little available literature on the effects of hard structures on beaches with coarse sediments (i.e. gravels). In addition, the most clear and direct ecological impacts of 'hard' armoring were found to come from placement loss, when part of a beach is replaced or covered by a new structure. Placement losses were shown to lead to loss of spawning and foraging habitat for fish and bird populations and loss of organic debris (beach wrack, logs, etc.). Notably, this effect may also be present with so-called 'soft' approaches although often on a less permanent basis. The authors hypothesized about additional indirect effects of armouring, such as loss of backshore vegetation, increased shading, increased coastal squeeze (loss of intertidal area), and coarsening of beach sediments; however, they noted that there was not sufficient evidence to reach a consensus on these claims.

Heerhartz et al. (2014) investigated the effects of bulkheads and seawalls on mixed sediment beaches in Puget Sound, Washington. They found that they were associated with lowered intertidal areas and lower beach wrack subsidies than their non-armoured counterparts. However, hard structures are often placed in lower elevation areas that are actively eroding, and a distinction between the cause and effect of constructing the shoreline armouring was not made.

Gittman et al (2016) undertook a meta-analysis of studies that compared ecological factors along armoured vs. non-armoured shorelines. Armoured shorelines were divided into (1) seawalls or bulkheads, (2) sloped revetments, or (3) breakwaters/sills. They found that, although the design of 'hard' armoured shorelines varied widely, there were some clear ecological consequences of armouring. Seawalls supported 23% less biodiversity and 45% fewer organisms than non-armoured shorelines. There were no significant differences between revetment or breakwater structures and non-armoured shorelines in terms of biodiversity and organism abundance; however, the authors noted that the number of studies focused on these types of structures is significantly less than for seawalls.

Lee et al. (2018) undertook a field assessment at six sites in Puget Sound, Washington, to study the effects of shoreline restoration on coastal biota. Specifically, they reviewed opportunistic pre- and post-restoration surveys at six sites where armouring (either with concrete or wooden bulkheads or riprap structures) was removed. The authors assessed changes in (1) percent beach wrack cover, (2) percent salt marsh cover, (3) number of logs (LWD), (4) macroinvertebrate abundance, and (5) macroinvertebrate richness. The authors also assessed the elevation change at the location on the foreshore where the base of the armouring was previously located. However, data on all of the considered metrics were not available for all of the six sites. The authors acknowledged that the limited sample size was a challenge for generalizing the results of the study and had the potential to increase errors and distort interpretations. Despite this, they concluded that biota metric responded strongly and positively to armour removal.

2.1.2 Nature-Based Solutions

The mid-late 1900’s marked the beginning of a movement away from conventional ‘hard’ approaches and towards ‘soft’ engineering approaches (Pontee et al, 2016), often also called ‘nature-based solutions’ or ‘living shorelines’. This movement was driven by claims that these ‘soft’ approaches could provide the same level of protection while minimizing harm to, or, in some cases, restoring natural ecological systems (Morris et al., 2018; Pontee et al., 2016). The concept of ‘nature-based solutions’ is explored herein.

2.1.2.1 Terminology

Numerous terms exist to describe coastal works that aim to mimic or restore natural coastal ecosystems through the use of natural processes and materials. ‘Soft engineering’, ‘living shorelines’, ‘nature-based solutions’, ‘greenshores approaches’, ‘building with nature’, ‘green infrastructure’, and ‘ecological engineering’ are all now common terms in the coastal engineering vernacular (Bilkovic et al., 2017; Pontee et al., 2016). However, each term comes with its own set of connotations regarding the primary function, the level of design, and amount of artificial components. The range of terminology in the literature may serve to introduce uncertainty around these types of approaches and their potential effects (Morris et al., 2018).

Arguably, the most natural method would be to have no intervention and to let a shoreline naturally erode and accrete. Erosion, after all, is also often a natural process (Bilkovic et al., 2017), and by limiting it, humans may be interfering with a natural process. As such, it is necessary to recognize that there are reasons, both ecological and societal, that interventions are required in the form of coastal works.

Coastal works that use engineering techniques to meet ecological needs may be termed 'restoration' or 'ecological engineering' (Chapman & Underwood, 2011; Morris et al., 2018; Perkins et al., 2015). Conceptually, these are projects where the 'client' is the coastal ecosystem rather than society. Morris et al. (2018) further divided ecological engineering into 'soft' and 'hard' ecological engineering practices.

Bilkovic et al. (2017) used the term 'living shorelines' to describe shoreline protection techniques with the restoration at the core. Conceptually, these are projects where the 'client' is both the coastal ecosystem and society. This term, however, comes with a connotation that the technique must use 'living' features; however, Bilkovic et al. (2017) also used it to describe a beach fill project with rock headlands, which is arguably both artificial and not 'living'.

'Soft engineering' has come to describe coastal protection techniques that are perceived to behave symbiotically with the natural coastal processes (Pontee et al., 2016). Conceptually, these may be projects where the client is society, and benefits to the coastal ecosystem are secondary but present nonetheless. This conceptualization may match more closely to the reality of why projects are undertaken. The term, however, applies negative connotations to approaches that are 'hard', regardless of their fit into the natural ecosystem. For example, a vertical seawall may mimic natural coastal processes more readily in a rocky exposed coastline than a beach fill project.

'Nature based solutions' (NBS) or 'nature based approaches' are used to describe techniques that aim to meet societal needs and challenges by using natural methods and materials, and does not apply strictly to coastal or water engineering (Eggermont et al., 2015). For the purpose of this document however, NBS are defined as follows:

Nature-based solutions (NBS) aim to solve societal challenges by mimicking specific processes and materials found in the local natural environment.

By this definition, NBSs are context and site dependent. In fact, by this definition, a large vertical rock wall that mimics the outcrops present along much of western Vancouver Island, BC, may be considered a NBS. Furthermore, any system that stops erosion such that it limits the natural input of sediment into the coastal system, whether made from wood or concrete, may not be considered a NBS. In some cases, this definition may also mean that a NBS cannot actually meet societal challenges. As such, there must be an understanding that 'win-win' solutions are not always possible (Eggermont et al., 2015; Nesshöver et al., 2017).

Notably, the term ‘Natural and Nature-Based Features’ (NNBF) has also gained recent popularity (Bridges et al., 2015), extending the NBS term to be more inclusive towards restoration projects and remove connotations associated with implying these techniques are ‘solutions’. Although generally interchangeable, NBS is preferred for this study since the purpose of using anchored LWD is primarily for the purpose of coastal protection, not restoration.

2.1.2.2 Efficacy

Many studies have been published that support the usage of Nature Based Solutions. Bulleri & Chapman (2010) stated that incorporated natural elements into shoreline protection designs can reduce ecological impacts without compromising the engineering efficacy. Studies on the impact of ‘hard’ vs. ‘soft’ shoreline protection techniques on invertebrate communities have found mixed results. Some studies found altered communities of invertebrates, while others found greater diversity and density of organisms in hybrid living shoreline sites (Bilkovic et al., 2017; Davis, Takacs, & Schnabel, 2006). Komar & Allan (2009) developed a cobble beach berm to mitigate ongoing shoreline erosion in Oregon, USA. They found that the technique was effective at mitigating erosion and reducing wave run-up. Even with periodic maintenance, Komar & Allan concluded that the total project cost was significantly less than that of a bulkhead.

However, the success of Nature Based Solutions as a whole is highly dependent on the type of project and its suitability to the given site. Morris et al. (2018) compared the outcomes of conventional ‘hard’ shoreline protection techniques to ‘soft’ techniques and found that they were generally difficult to compare because they were not tested/evaluated under the same environmental conditions. For example, a higher percentage of ‘soft’ engineering works were applied to smaller tidal and wave energy conditions than ‘hard’ techniques.

Furthermore, there is some debate about what constitutes a successful NBS. For example, since the purpose of many projects is often to minimize erosion, these techniques have the potential to limit sediment transport processes in a similar fashion to conventional techniques. Despite this, there is a general consensus, at least amongst policy makers, that NBS provide an effective alternative to conventional ‘hard’ techniques (Bilkovic et al., 2017).

2.1.2.3 Design Guidance

There has been a growing trend to the usage of NBSs in the coastal environment due to (1) a growing body of literature indicating that traditional coastal protection may damage coastal ecosystems, (2) the loss of existing coastal ecosystems, particularly near population centres, and (3) desire for more dynamic and adaptive measures (Bilkovic et al., 2017). The uptake has occurred despite the fact that design guidance and best practices are limited for NBSs (Bilkovic et al., 2017).

Guidance for NBSs is limited partially because they comprise a relatively new type of coastal engineering practice and also because they span such a wide variety of techniques. For example, NBSs may include salt marsh development (Davis et al., 2006), oyster reef construction (Morris et al., 2019), coir log installation (Pontee et al., 2016), beach nourishment (Komar & Allan, 2009), and Large Woody Debris (Zelo et al., 2000), amongst other techniques. Additionally, each technique requires its own set of understanding regarding its interactions with coastal processes, ecological effects, and technical feasibility.

The design guidance that does exist is limited and not well established (Pontee et al., 2016). Because of a lack of peer-review literature, development of design guidance often relies upon 'grey' literature (i.e. technical documents that have not undergone a formal peer-review process) (eg. Envirovision, Herrera Environmental Consultants, & AHG, 2010; Gerstel & Brown, 2006; Gianou, 2014; Zelo et al., 2000). While evaluating ecological adaptations to 'hard' structures, Chapman & Underwood (2011) also encountered this problem, stating that "It is not always clear whether the proposal or the work itself has been peer-reviewed and, if so, how well it was reviewed… There seems to be often more effort put into doing something, rather than evaluating whether it worked or not". As such, it is imperative that peer-reviewed research be conducted, from which design guidance for NBS can be established.

2.2 Large Woody Debris

2.2.1 Naturally Accumulated LWD

A growing body of literature suggests that natural deposits of LWD on estuaries or beaches may promote additional sediment accretion and reduce wave run-up (Eamer & Walker, 2010; Gonor et al., 1988; Grilliot et al., 2018; Heathfield & Walker, 2011; Huff, 2015; Kennedy & Woods, 2012). Gonor et al. (1988) reviewed and summarized previous technical documents (both peer reviewed and 'grey' literature) regarding the role and presence of large wood and trees in coastal areas. The authors found that LWD may passively stabilize beach dune fronts. LWD was observed to behave differently along rocky shores, where winter storm waves batter and pile-up LWD against the rocks and headlands. Additionally, LWD was observed to batter and erode a marsh shoreline at high water levels during winter storms. In Oregon, USA, the natural cycle of LWD erosion and deposition was observed to follow the below sequence:

1. During storm events, waves reach the backshore and erode
2. During following high water events, LWD deposits at the base of the eroded bank
3. Low energy summer waves develop a summer berm profile
4. Deposited LWD matrices act to trap sand through wave action and aeolian transport
5. Aeolian transport continues to re-establish the previously eroded foredune which generally becomes colonized by vegetation
6. Episodic erosion occurs again and the cycle repeats

Brennan et al. (2009) summarized the current science regarding ecological functions of marine riparian areas and developed management recommendations to protect these areas. The authors found that LWD provided several important functions, including:

1. Moderation of substrate temperature
2. Increased accumulation of detritus
3. Structural complexity and support for fish, wildlife, and vegetation
4. Bank/shoreline stabilization

In regards to bank/shoreline stabilization, Brennan et al. (2009) stated (without citing a reference) that LWD helps buffer the effects of storm waves and longshore currents by dissipating wave energy and retaining sediments. The authors also stated that the longevity (in terms of residence time) of LWD was controlled by the decomposition rate (dependent on the type of wood), the size characteristics, their ability to become naturally anchored in place, and metocean exposure.

Eamer & Walker (2010) undertook a study on the sedimentological role of naturally occurring large woody debris in a high-energy sand beach on Haida Gwaii, BC. The authors specifically studied the role and storage potential capacity of LWD in trapping sediment during aeolian (windborne) sand transport using high-resolution LiDAR and orthophotographs. Eamer and Walker found that significant amounts of sand were stored within LWD complexes on the backshore. The found that LWD promotes aeolian sand accretion within the dune by acting as accretion sink during aeolian transport, but may limit accretion landward (limited sediment supply).

Heathfield & Walker (2011) used air-photo analyses to study long term trends in LWD and vegetation cover related to changes in shoreline position at Long Beach, near Tofino, BC. The authors found a seaward progradation (by 28% on average) of the dunes at all sites over the 34-year observation period. They contended that progradation was due to LWD deposits and vegetation stabilizing the dunes. However, they also found that LWD coverage had decreased 61% on average.

Grilliot, Walker, & Bauer (2018) undertook the first study to quantify the impacts of naturally accumulated LWD matrices on near-surface airflow near coastal dunes, with specific application to aeolian sediment transport. The field study was conducted on Calvert Island, central BC. The authors used 3D anemometers to measure wind speed and direction along a transect within a LWD complex and along a reference transect with no LWD. The study results indicated that LWD effects flow close to the beach surface (within 0.5m) and to a much lesser extent than the foredune itself. Specifically, the presence of LWD reduced the wind speed, Reynolds normal stress, and turbulent energy near the beach surface. The authors determined that LWD serves as a momentum sink for sediment deposition and that variations in matrix shape, size, structure, density, orientation, height, and porosity, can alter flow characteristics and sediment transport pathways. The authors also noted that shore parallel LWD alignments may create a physical barrier that steers flow alongshore.

Kennedy & Woods (2012) undertook the only peer-reviewed study focused on the effects of LWD on beach morphology under wave action. They quantified the size and volume of large woody debris present on two primarily gravel beaches in New Zealand exposed to waves up to 2.25m high (significant wave height). Each beach was characterized by two sections: one section with a significant build-up of LWD and another section devoid of LWD due to harvesting. The beaches were surveyed and sediment samples were collected 20 times between February and July 2010. The measured beach height was on average 0.5–1.0 m greater and the beach face was almost two times steeper on beaches with LWD compared to the beaches without. Kennedy & Woods (2012) concluded that LWD is deposited on the storm berm during high wave events and subsequently traps sediments at the uppermost limit of the swash zone. The LWD remains inactive until another storm occurs that is large enough to mobilize the upper beach. In addition, the study concluded that the LWD prevents the inland penetration of waves (i.e. reduces run-up). These results suggest that LWD may serve to trap sediment on the upper beach, outside of regular wave action, and prevent it from returning to the active portion of the beach post-storm. This may provide some insight into the elevation or location on the beach profile where placed LWD could be most effective. However, it is unclear if LWD alone is responsible for the observations in the study, or if the presence of small woody debris and beach wrack in collaboration with the LWD contributed significantly to beach behavior. This is an important distinction because engineered LWD structures are typically monolithic and devoid of smaller materials. In addition, none of the LWD in the study was anchored or engineered/placed.

Huff (2015) undertook a numerical modeling study of the Brazos River delta in Texas, USA. The purpose of the study was to better understand LWD influence on longshore sediment transport and its' effect of delta morphology. The influence of LWD was incorporated into the model by calculating the skin friction stress based on LWD concentration and drag. Field surveys and photographs were used to compare/validate the numerical model results with numerical results. The results of the study indicated a correlation between decreased sediment transport (and therefore a more stable delta) and increasing LWD concentration. The authors suggest that LWD (natural, as opposed to anchored) may be an effective buffer against wave action, because LWD is buoyant and therefore minimizes undercutting during storm events (compared to breakwaters, for example). The author does not consider other observations (see Zelo et al., 2000) that indicate that mobilized LWD may cause shoreline damage.

2.2.2 Placed LWD

Little work exists that explicitly evaluates the efficacy of placed LWD in providing shoreline protection in a coastal environment and – similarly to the literature on NBS in general – the documentation that does exist is generally only found in 'grey' literature. As such, Simenstad el at. (2004; as cited in Jones et al., 2016) stated that research is still needed to understand the role and influence of large woody debris.

Zelo et al. (2000) reviewed fifteen projects in Puget Sound that used alternative or 'soft' methods as shoreline protection. Eight of the projects utilized engineered or 'placed' LWD as a structural component of the work, six of which used anchors to secure the LWD. Anchoring methods, elevation of placement, and size of LWD varied between projects. The authors found that anchoring provided a sense of durability, but may cause undesirable consequences. For example, during high water levels and storm events, the LWD may become partially mobile and cause scour. Additionally, cables, anchors, and dead-men may work loose and become exposed. The report also raised the question of whether large woody material should be anchored at all – a question that is not yet answered by existing literature. All of the LWD projects were also accompanied by some type of beach nourishment, typically using gravel. The study found that most of the projects were performing well, however, some of the projects had yet to be constructed or had only recently (within 0-5 years) been constructed. Additional discussions on case study locations and findings are included in Section 2.2.3.

Rich et al. (2014) studied the distribution of naturally occurring LWD near the Elwha River Estuary in Washington State, USA., and made restoration recommendations. The recommendations included first removing bulkheads and rip-rap structures which they asserted prevent LWD accumulation. Additionally, they noted that the installation of LWD with beach nourishment (gravel) has been a successful alternative to ‘hard’ structures. They stated that this type of approach breaks-up wave energy, provides shading, serves as a substrate for vegetation, and increases shoreline complexity; however, they caution that the ability of placed LWD to serve these functions is not fully understood and poorly researched.

Johannessen et al. (2014) developed the Marine Shoreline Design Guidelines, which were aimed at providing guidance for selecting and designing ‘soft’ protection approaches. In preparing the guidelines, the authors undertook case studies of 25 projects in Puget Sound, Washington State, USA. Five of the projects evaluated included placed LWD as the primary component of the work. They found that 4 of the 5 projects were effective at mitigating erosion; however it does not mention if the shorelines had erosion issues before installation of the LWD or to what extent the projects were effective. The authors concluded that LWD is an appropriate technique for sites with low-moderate risk, low-moderate wave energy, all shore types (except for rocky headlands/shores) and moderate-high backshore width (to reduce risk). Interestingly, in the case studies, LWD was consistently used on shores with higher risk than the other soft shore techniques. Notably, the lead author of the publication was also the designer on at least two of the five case study projects on LWD. Additional discussion on the case studies is included in Section 2.2.3.

Jones et al. (2016) studied the structure and function of LWD in estuaries and made recommendations for restoration using placed LWD. Based on their observations, the authors questioned “whether these types of structures either mimic natural forms in streams or estuaries, or provide additional benefit to channel forming processes or ecology of the stream”. In addition, because of the lack of research in this area of study, they recommended avoiding the use of LWD in habitat restoration projects in tidal environments unless the need is explicitly justified, and to treat its use as an experiment paired with rigorous monitoring.

2.2.3 Previous Case Studies

Several anchored LWD projects in Washington, USA, have been documented in ‘grey’ literature and comprise most of the previously available information on coastal protection using LWD. Zelo & Shipman (2000) documented several projects where LWD was installed using a variety of techniques throughout Puget Sound, Washington, and has served as a reference document for the design of LWD projects since its publication. Gerstel & Brown (2006) developed a document outlining alternative methods for shoreline protection, which included placement of anchored LWD. Three anchored LWD projects are described within the document. The Washington State Marine Shoreline Design Guidelines (Johannessen et al., 2014) developed general design guidance for coastal protection using anchored LWD. The guidance was based on a number of site visits conducted to previously installed anchored LWD projects throughout Puget Sound, Washington. These previous project case studies are summarized in Table 1.

Table 1 – Summary of case studies on LWD

Project Name	Const. Year	LWD Characteristics				Anchor Description	Additional Components	Monitoring Results
		Elevation	Diam.	Type	Orientation			
Blake Island State Park, WA[1]	2000	~2.5 ft above HHWLT	~3.0 ft	Single	Shore-parallel	Buried eco-blocks	Coarse fill & vegetation planting	None
Blomquist Residence, WA[1]	1999	0.0 – 4.0 ft above HHWLT	1.5 – 2.0 ft	Multiple	Shore-parallel	Buried concrete curbs or eco-blocks	7/8" gravel fill	None
Dick Residence, WA[1]	1998	At or below HHWMT	-	Single	Shore-parallel	Cable with deadman	Sand and gravel fill	"Logs helped retain beach seidment"
Dully Residence, WA[1]	1998	~1.0 ft above HHWLT	1.5 – 2.0 ft	Single	Shore-parallel	Manta ray earth anchors	Burried rock revetment with some gravel fill	Small erosion problem remains at one end; gravel deposited above LWD
Indian Island, WA[1]	1997	-	-	Multiple	Shore-parallel	Derivable screw anchors	Fill, revetment, live stakings, etc.	Considered successful but details not provided
Bainbridge Island, WA[2]	1998	-	-	Single	Shore-parallel	Anchored (no details)	Geotextile and hand-placed rock	None
Hood Canal, WA[2]	1999	-	-	Single	Shore-parallel	Anchored (no details)	Underlayn with rip-rap	"This installation held up well in the February, 2006 storm"
Narrows County Park, WA[2]	2004	-	-	Single	~30^{o} to parallel	Pin anchors and chain	Beach fill (sand)	"Erosion of the imported beach sand is occurring around the log anchor chains"
Birch Point, WA[3]	2001	1.0 – 3.0 ft above HHWMT	2.5 – 3.0 ft	Multiple (pairs)	Shore-parallel	Eco-blocks buried >2.5ft below grade	Cobble fill and vegetation planting	Not effective due to high wave-energy; required maintenance within several years
Dabob Bay, WA[3]	2002	~3 ft abvoe HHWMT	-	Single	Shore-parallel	Eco-blocks buried >2.5ft below grade	Cobble fill and vegetation planting	Performed well and in good condition, but success may be due to presence of long barrier spit

Project Name	Const. Year	LWD Characteristics				Anchor Description	Additional Components	Monitoring Results
		Elevation	Diam.	Type	Orientation			
East Eld Inlet, WA[3]	2009	Mostly above HHWMT	1.5 – 2.5 ft	Single	Mixed shore-parralel and angled	Auger/screw	Large boulders	Inititially installed using pins and reinstalled using augers; appears to be effective at slowing erosion
NW Whidbey Island, WA[3]	2008	~4 ft abvoe HHWMT	2.0 ft	Single	Shore-parallel	Eco-blocks buried 4.0 – 4.5 ft below grade	Intertidal large boulders	Erosion was prevented and additional LWD was natrually recruited
Oak Bay, WA[3]	1999	1.0 – 2.0 ft above HHWMT	2.0 – 3.0 ft	Single	Shore-parallel	Buried eco-blocks	Cobble fill and vegetation planting	LWD showed signs of rot. No significant erosion since installation

[1] Source: Zelo et al. (2000)
[2] Source: Gerstel & Brown (2006)
[3] Source: Johannessen et al. (2014)

2.2.4 Existing Design Guidance and Recommendations

Despite a lack of research on LWD in a coastal environment, LWD has nevertheless been promoted and installed as shoreline protection, or an element of shoreline protection, particularly in Washington State, USA, and in British Columbia, Canada (Elliott et al., 2016; Zelo et al., 2000).

Zelo & Shipman (2000) documented several anchored LWD projects, which has served as a reference document for the design of LWD projects since its publication. In each of the documented projects, the LWD varied in physical setting, placement density, elevation, burial depth, log size and character, and anchoring technique, demonstrating a lack of, and need for, engineering guidance in the usage of LWD. The report generally recommends placing LWD on the upper, flatter portion of the foreshore, rather than on the main sloping face. It suggests that LWD may generally stabilize the beach and promote accretion, but may also exacerbate erosion and damage the shoreline during high water levels and storm events. The report notes that anchoring may provide a sense of durability, but may again enhance erosion during high water levels and storm events, due to increased scouring. Additionally, cables and anchors may become loose or exposed. They state that any anchored LWD design should specifically consider wave action and fluctuations in the beach profile (which is not included in design guidance for LWD in rivers). Zelo et al. (2000) recommended that "ultimately, we need design standards and well-documented demonstrations of these technologies".

Menashe (2001) proposed a composite structure, called the 'root wall' composed largely of LWD and native vegetation. Although the conference paper stated that the root wall would reduce reflection and refraction of wave energy, thereby reducing sediment suspension, "scour" and transport, the paper cited no published literature or numerical/physical/empirical basis for the claims. Although the root wall would effectively be biodegradable, the paper also gave no indication of project longevity.

Gerstel & Brown (2006) developed a document outlining alternative methods for shoreline protection, which included placement of anchored LWD. They state that the typical design includes anchoring LWD with steel cables or ports to rock or concrete blocks buried into the beach sediments. Additionally, this document states that beach nourishment and revegetation are typically included in the design. The authors state that large cedar pieces with root wads make the most desirable wood. Gerstel & Brown (2006) also noted that installing anchored LWD into the beach can cause significant disturbances to the substrates and may in fact result in long-term damages to the natural beach processes and shore ecology. Finally, they comment that anchoring LWD may not be recommended where coastal processes allow for the natural recruitment of LWD but the area is supply limited.

The Mason County Department of Community Development recommends using LWD as an alternative to bulkheads (Mason County, 2010). The document states that LWD can slow wave induced erosion, promote sediment accretion, and provide substrate for vegetation growth. However, the only guidance provided for design is that LWD should be kept in place through the use of cables or by partially burying the logs within the beach.

The Washington State Marine Shoreline Design Guidelines (Johannessen et al., 2014) provides the most comprehensive design guidance for LWD as coastal protection. The authors state that LWD often occurs in conjunction with other nature-based solutions, such as beach nourishment or vegetation. The authors recommend that LWD is only suitable in areas without high levels of erosion, with low risk to structures, with moderate-high backshore width, and low-moderate wave energy. Johannessen et al. (2014) also list LWD elevation, orientation, characteristics, and anchoring as key design considerations. In regard to elevation, the authors recommend installing at or above Higher High Water Mean Tide (HHWMT). The authors state that LWD should be generally placed shore-parallel for erosion protection and may be placed as groynes (angled) if the aim is to trap additional LWD. In regard to characteristics, the authors note that the log diameter in the case studies ranged from 19-34 inches and that small pieces will degrade more quickly. Additionally, the authors state that Douglas Fir and Western Red Cedar logs will have increased longevity compared to other types of wood. Johannessen et al. (2014) state that LWD placed for erosion control should be anchored in place. No quantitative methods to size anchors are included in the guidelines, however some qualitative guidance is provided for the type of anchor to choose. In summary, the authors do not provide any quantitative guidance/equations on how to determine the size, length, elevation, design life, anchor type, or anchor size. The authors do however, state that areas of additional research are: (1) wave and LWD interactions, and (2) effects of anchoring on sediment transport and LWD stability.

Brzozowski & Nichols (2014) developed 'A Landowner's Guide to Protecting Shoreline Ecosystems' largely based on the Washington State Marine Shoreline Design Guidelines (Johannessen et al., 2014). The guide lists various methods of soft shore protection, including the use of LWD to add habitat complexity, increase beach elevation, and to "hold" beach material. The guide recommends partially burying LWD within beach fill material and to avoid anchoring LWD if possible. Anchoring LWD to buried boulders or concrete blocks should be designed by a qualified professional. Additionally, the authors specify that the LWD should generally be placed above the high-tide line, but have sometimes been successful at lower elevations at sites with low wave energy.

The Greenshores Design and Rating System (Stewardship Centre for British Columbia, 2016) states that anchored LWD is an effective strategy for protecting the shoreline under relatively sheltered (low wave energy) conditions. Although the document does not provide design guidance specifically, the greenshores website[3] highlights several projects in the BC area that incorporate this technique as a means of shoreline protection.

Bilkovic et al. (2017) developed an overview of living shoreline techniques. Although they did not identify any specific guidance for LWD placement, they did identify a large gap in knowledge regarding design standards. Specifically, the stated that "guidelines for structural component material, size, height, shape, and placement; intertidal slope; and other design features based on site characteristics such as energy regime, substrate, bathymetry, and topography" were needed. Additionally, they stated that in order to develop such guidelines, experimental studies need to be conducted to determine the risks associated with different design characteristics for each type of shoreline protection technique.

2.2.5 LWD in Rivers

Large Woody Debris as a shoreline stabilization measure in the coastal environment likely stems from its frequent usage in the fluvial environment for decades (Menashe, 2001), where significant literature is available on the function and design of LWD (Bocchiola, 2011; Douglas, Morin, & Cooper, 2004; Gonor, Sedell, & Benner, 1988; Hilderbrand, et al., 1998; Hills & Street, 2007; Kail et al., 2007; Rafferty, 2017; Sass, 2009). In a study on LWD usage in river restoration projects in the UK, Cashman et al. (2018) found 912 projects involving LWD, from an archive of 4894 potential projects. The authors also anticipate increased and more widespread usage of LWD in the future for the purpose of habitat restoration and flood management.

[3] http://stewardshipcentrebc.ca/Green_shores/cdrs/

Hilderbrand et al. (1998) studied various factors influencing LWD stability in streams, with application to the design of LWD in-stream enhancement projects. They found that log length was the most important factor for stability, as opposed to initial orientation which played little role. Regardless of orientation, when LWD was ‘ramped’, scour was observed in the channel both upstream and downstream of the log.

Shields et al. (2004) outlined a method for reducing channel erosion and providing habitat in sand-bed channels using LWD. The method involves lacing/interlocking LWD into a mat-like structure. Shields et al. summarize the general design problem as follows:

1. Use of buoyant materials
2. Use of organic materials that will decay over time
3. Meeting dual objectives of reduced erosion and habitat enhancement

In a *‘Primer on Large Woody Debris Management’*, Hills & Street (2007) detail types of LWD installation techniques (including groyne type structures, crib structures, and laced-matted structures), and provide summary recommendations on placement and anchoring of LWD in rivers. They related log diameter to bankfull width and outline four general classifications of anchors: (1) no anchors, (2) passive, (3) flexible, and (4) rigid.

D’Aoust & Millar (2000) outlined a theoretical force-balance approach for estimating ballast requirements of LWD structures in rivers, with and without root wads. Based on field studies at 90 existing anchored LWD sites, this approach was able to successfully predict the stability of single LWD structures, but not the more complicated multiple LWD structures,

The U.S. Army Engineer Research and Development Center (2016) developed the *‘National Large Wood Manual’*. The manual outlines seven different types of LWD structures in rivers. Suggested LWD dimensions are provided in relation to bankfull discharge depth and width of the stream. Imporantly, the authors compile data on decay rates for stream structures based on location and type of wood; however, their data does not consider LWD in a coastal (saline) environment. The authors also detail procedure and formula for a force and moment analysis, considering buoyancy, friction between the logs and the stream bed, fluid drag and lift, and geotechnical (ballast) forces. Qualitative guidance for anchoring/securing the wood is also provided.

Most recently, Rafferty (2017) developed a technical note outlining a design tool (in excel) for evaluating the stability of LWD structures in rivers. Rafferty states that the log should be assumed to be dry prior to a high water-level event (at a 12% moisture content volume basis).

Using the dry density is also thought to represent the wood better as it decays. The specific weight of air-dried timber, γ_{Td}, can therefore be calculated as follows:

$$\boldsymbol{\gamma_{Td} = G_s \times \gamma_w \times (1 + w)} \tag{1}$$

Where G_s is the average oven-dried specific gravity of wood, γ_w is the specific weight of water, and w is the moisture content of the wood.

Rafferty (2017) also outlines formulas for the calculations of vertical forces, including buoyancy, gravitational, lift, soil ballast, and anchor forces. These additional formulas for the calculation of horizontal forces and moments for application in rivers can be found in Rafferty (2017).

Numerous physical studies have been completed on LWD in rivers (compared to no known experimental studies of LWD in the coastal environment). Braudrick et al. (2001) undertook a gravel-bed flume study to better understand how far LWD was transported in the river environment and where it deposited. Wallerstein et al. (2001) undertook a sand-bed flume study on the effect of LWD on morphology and hydraulics. Hygelund & Manga (2003) undertook field measurements with simulated logs to better understand drag coefficients associated with LWD in rivers. Bocchiola (2011) studied flow modifications due to LWD in a flume setting, and then analyzed resulting changes in habitat suitability for fish species. Perry et al. (2018) undertook a large scale 3D experimental model on LWD accumulation at a flow control structure. Several other previous physical model studies involving LWD in rivers are summarized in Gallisdorfer et al. (2013).

Numerous other studies of LWD in rivers has focused on transport, impact forces, contribution to flooding, ecological values, and morphological effects (e.g. Bertoldi & Ruiz-Villanueva, 2017; Kramer & Wohl, 2015; Ruiz Villanueva et al., 2014; Wohl, 2017)

2.2.6 Ecological Considerations

Much literature exists as to the ecological importance of natural LWD deposits in providing habitat, particularly if LWD is in various states of decomposition (Gonor et al., 1988) and in riverine systems (Maloy, 2013). Brennan et al. (2009) summarized the ecological benefits of LWD in providing temperature control, accumulation of detritus serving as both habitat and food for invertebrates, providing substrate for terrestrial vegetation, and structural complexity for fish and wildlife. Sass (2009) found that LWD provided food, refuge, spawning substrate, and nursery and rearing habitat for at least 85 species of fish. Heathfield & Walker (2011) found that natural accumulations of LWD on an exposed beach in BC provided substrate for vegetation growth (specifically native dune grass and invasive beach grasses). Heerhartz et al. (2014) studied the effects of shoreline armouring on riparian habitat, specifically studying the effect on beach wrack and naturally accumulated LWD. Lee et al. (2018) stated that LWD reduced beach erosion and enhanced beach wrack accumulation provided by LWD serves to enhance saltmarsh growth and improve riparian connectivity.

Conversely, Fujisaki & Lamont (2016) studied the influence of large debris on sea turtle nesting behaviour at a beach in Gulf County, Florida, USA. Debris was composed of both man-made and natural materials, and most the natural material was LWD. The authors found that removal of the debris resulted in an increase in sea turtle nesting behaviour, and, as such, the removal of debris may be an effective restoration strategy.

The ecological benefits of engineered or 'placed' LWD, as opposed to naturally accumulated LWD, is still debated. Referring to ecological modifications to 'hard' shoreline protection works, Chapman & Underwood (2011) concluded that there is little data (quantitative or experimental) to support the idea that using natural materials (such as LWD) will support biota assemblages more similar to natural ones than using 'man-made' materials. (Rich et al., 2014) noted that 'soft' shoreline protection techniques do not serve as true habitat restoration and care should be taken to differentiate the ecological benefits that these types of methods may provide. Jones et al. (2016) questioned whether placed LWD mimic the performance of naturally recruited LWD in estuaries.

2.3 Theoretical Background

2.3.1 Wave Theory

2.3.1.1 Regular Waves

The simplest method to mathematically represent waves is through linear wave theory, also known as sinusoidal or Airy wave theory. Along with several other assumptions and simplifications, this first-order theory assumes waves are two-dimensional, sinusoidal, small in amplitude, and regular (invariant) (Baker, 2018). It also allows for waves to be defined by their height, H, and period, T, in relation to water depth, d. Linear wave theory therefore allows for a simple, convenient method of representing simple wave conditions, although it is generally not an accurate representation of waves in shallow water nor of wave fields generated in real-world conditions (outside of the lab).

Non-linear (second and third order) wave theories often allow better representations of wave conditions, particularly in shallow water. These wave theories, such as Stokes or Cnoidal wave theories, allow for characterization of waves that are not purely sinusoidal, such that wave trains are more peaked, have a flatter trough, and allow for a water surface distribution that is skewed above the still water level (Baker, 2018; Isaacson, 2011).

2.3.1.2 Random Waves

Random wave theory assumes that the sea-state is composed of a series of regular waves trains with various frequencies. The wave spectrum, or spectral density, *S(f)* [m^2/Hz], describes the mix of wave frequencies within the sea-state. Various formulations have been developed to describe wave spectrums, some of which are described herein.

Modified Bretschneider – Mitsuyasu Spectrum

The *Bretschneider spectrum* was developed for predicting fully developed wind-waves in the open ocean. The *Bretschneider – Mitsuyasu spectrum* introduced adjustment coefficients. Later, the *modified Bretschneider – Mitsuyasu Spectrum* was proposed by Goda (2010), which re-wrote the equations in terms of wave height and period, and included additional adjustments of the coefficients:

$$\boldsymbol{S(f) = 0.205 H_{1/3}^{2} T_{1/3}^{-4} f^{-5} exp\left[-0.75\left(T_{1/3} f\right)^{-4}\right]} \qquad (2)$$

Where *S(f)* is the spectrum for a given frequency, $H_{1/3}$ it the significant wave height defined as the average of the highest one-third of waves in the wave train, $T_{1/3}$ is the average period associated with the highest one-third of the waves, and f is the given frequency defined as T^{-1}. Note that $T_p \cong 1.1 \times T_{1/3}$ for wind-waves, and $H_{1/3}$ is equivalent to H_s. Note that frequency is often also described as an angular frequency, which is defined as:

$$\boldsymbol{\omega = 2\pi f} \qquad (3)$$

Pierson-Moskowitz Spectrum

The most commonly used formulation is *the Pierson-Moskowitz (1964) spectrum*. It was developed based on the assumption that wind blows steadily across a water surface such that the sea-state becomes fully developed. The formula can be written in terms of the wave height and period (or frequency), as shown in the below equation (Isaacson, 2011).

$$\boldsymbol{S(f) = \frac{5H_s^2}{16f_p}\left(\frac{f}{f_p}\right)^{-5} exp\left[-\left(\frac{5}{4}\right)\left(\frac{f}{f_p}\right)^{-4}\right]} \quad (4)$$

Where f_p is the peak frequency, at which the spectral density ($S(f)$) if maximum, and is equal to $T_p = 1/f_p$.

JONSWAP Spectrum

Wind-wave that develop in more restricted bodies of water (developing sea-state), such as the Strait of Georgia and Puget Sound, typically demonstrate a more ‘peaked’ spectrum, which is not typically well represented by spectrum developed for the open ocean. The more peaked nature of these types of waves was included in the *JONSWAP spectrum*, which was rewritten in terms of wave height and period by Goda (2010) as shown in Figure 5 and described in the following equations:

$$\boldsymbol{S(f) = \beta_J H_{1/3}^2 T_p^{-4} f^{-5} exp\left[-1.25\left(T_p f\right)^{-4}\right]\gamma^{exp\left[-\frac{(T_p f-1)^2}{2\sigma^2}\right]}} \quad (5)$$

$$\boldsymbol{\beta_J = \frac{0.0624}{0.230+0.0336\gamma-0.0185(1.9+\gamma)^{-1}}[1.094-0.01915\ln\gamma]} \quad (6)$$

$$\boldsymbol{T_p \cong T_{1/3}/\left[1-0.132(\gamma+0.2)^{-0.559}\right]} \quad (7)$$

$$\boldsymbol{\sigma = \begin{cases}\sigma_a : f \leq f_p \\ \sigma_b : f > f_p\end{cases}} \quad (8)$$

$$\boldsymbol{\gamma = 1 \sim 7\ (mean\ of\ 3.3), \sigma_a \cong 0.07, \sigma_b \cong 0.09} \quad (9)$$

Where γ is the peak enhancement factor; for the North Sea, where the formula was developed, this factor is equal to 3.3, which is also generally the default value used in coastal engineering practice (Goda, 2010).

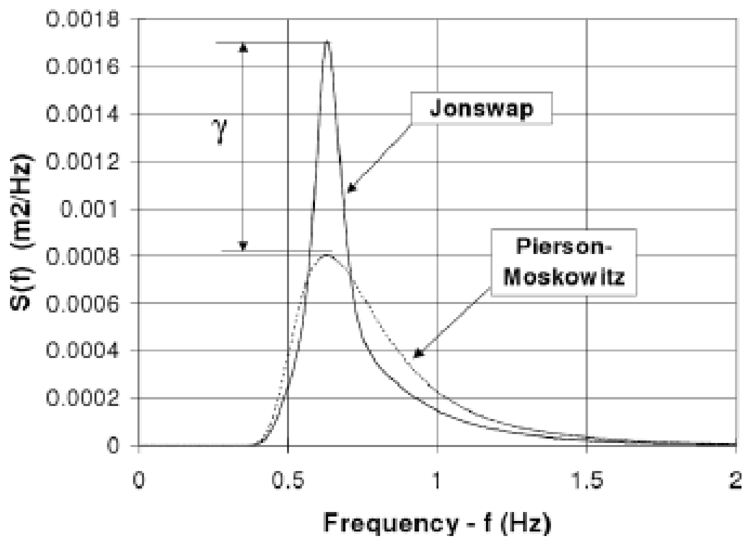

Figure 5 – Comparison of spectra for JONSWAP and Pierson-Moskowitz (Baker, 2018)

2.3.2 Breaking Waves

Wave breaking occurs when waves can no longer sustain their form due to increased wave steepness or decreased water depth, and energy is transformed into kinetic energy. It is understood that the criteria for wave breaking can be described in terms of Wave Steepness, *S*, and the Breaker Index, γ_b. As an approximation, wave breaking occurs when the following conditions are met (Kamphuis, 2000):

$$\boldsymbol{Wave\ Steepness\ (S)}:\ \ \boldsymbol{H/L > 1/7} \tag{10}$$

$$\boldsymbol{Breaker\ Index\ (\gamma_b)}:\ \ \boldsymbol{H/d > \sim 0.78} \tag{11}$$

For irregular waves, the wave height, *H*, is generally taken as the maximum wave height, H_{max}. In addition, the criteria for wave breaking based on the Breaker Index varies significantly depending on the beach slope and the breaker type (Goda, 2010). As such, many authors have proposed more complex relationships to describe the Breaker Index (Kamphuis, 2000).

The breaker type can be described by the ratio of beach slope to wave steepness, represented by the Surf Similarity Parameter or Iribarren Number, ξ (Battjes, 1974). Where α is the slope angle from horizontal, H is the representative wave height, and L is the wavelength.

$$\boldsymbol{Surf\ Similarity\ Parameter}:\qquad \boldsymbol{\xi = \frac{\tan\alpha}{\sqrt{H/L}}} \tag{12}$$

For irregular and regular waves, the Surf Similarity Parameter can be calculated as follows:

$$\boldsymbol{Regular\ Waves}:\qquad \boldsymbol{\xi_{av} = \tan\alpha \Big/ \sqrt{{2\pi H_m}\Big/{gT_m^2}}} \tag{13}$$

$$\boldsymbol{Irregular\ Waves}: \qquad \xi_p = \tan\alpha \Big/ \sqrt{\frac{2\pi H_s}{gT_p^2}} \tag{14}$$

For deep water, the breaker type can be categorized based on the Surf Similarity Parameter, as follows (Battjes, 1974):

$$\boldsymbol{Surging\ or\ Collapsing}: \qquad \xi_0 > 3.3 \tag{15}$$

$$\boldsymbol{Plunging}: \qquad 0.5 < \xi_0 < 3.3 \tag{16}$$

$$\boldsymbol{Spilling}: \qquad \xi_0 < 0.5 \tag{17}$$

2.3.3 Beach Profile Characteristics

Beaches are shaped by a long history of changing tides, currents, winds, and waves. Their shape and character are also directly connected to ecological and hydrological factors (Firth et al., 2014; Stark et al., 2016). During extreme storm conditions, sediments will erode from the beach face and be transported offshore forming a bar, having the overall effect of flattening the overall beach profile and steepening the beach face. During more nominal conditions, sediments will be transported towards the beach face again, serving to rebuild the original beach profile. Although these processes can lead to large seasonal variations in the beach profile, it is understood that – given sufficient time, forcing, and sediment supply – beaches will tend to revert to an average, characteristic beach profile with a number of typical features (Figure 6).

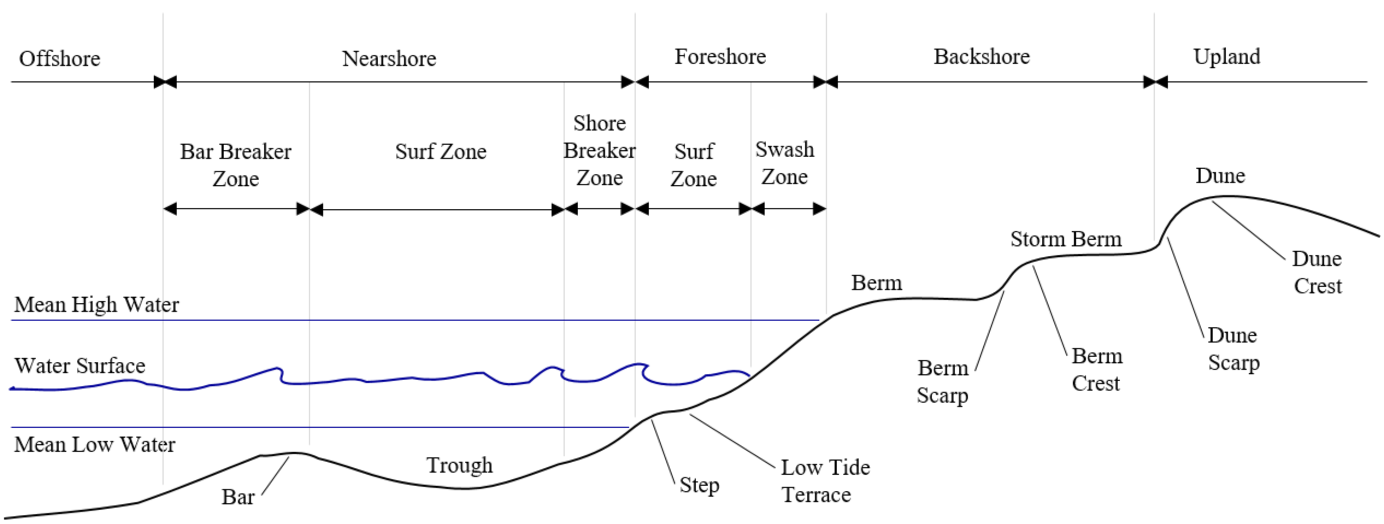

Figure 6 – Typical beach profile characteristics (adapted from USACE, 2002)

This characteristic base profile is described as the ‘equilibrium profile’ or state of ‘dynamic equilibrium’. Dean (1991) described key features of equilibrium beach profiles as follows:

1. The overall beach profile tends to be concave upwards
2. The beach face is approximately linear
3. Increased sediment size is associated steeper slopes
4. Steep waves generally result in milder slopes and offshore bar formation

Beach equilibrium can be defined as the beach profile at which the net transport capacity has become zero (Jentsje W. Van Der Meer, 1988). The concept has been heavily used in studies involving sand beach morphology and (to a lesser extent) to studies on gravel beach morphology (See Section 2.4.5). Beach equilibrium (or dynamic stability) has been defined as a function of incident wave and sediment characteristics (Kobayashi, Hicks, & Figlus, 2011).

Typical features observed for gravel beaches include the existence of the offshore step below water, a steeply sloping beach face, and an above water berm (Atladottir, 2008; Buscombe & Masselink, 2006; Kennedy & Woods, 2012; Kobayashi et al., 2011; Powell, 1990). Characteristic slopes of gravel beaches typically range between 6:1 – 10:1 (H:V), depending on the specific site; However, gravel beaches have been observed to support slopes steeper than 6:1 (Buscombe & Masselink, 2006).

Due to their steep slopes, gravel beaches tend to result in narrow surf zones where waves break later and closer to the shore than on sand beaches. The offshore step acts as a submerged slope break, located at the base of the swash zone (Buscombe & Masselink, 2006). At the step, wave bores develop, shoal, and collapse over the relatively shallow and flat beach section. The step is therefore understood to force and modulate wave-breaking. The step location and characteristics are thought to be directly related to wave height and the surf similarity parameter. The step allows waves to shoal in deep water, concentrating wave energy on the inshore edge of the step and creating the conditions to maintain a relatively steep and reflective step face. In part because of the energy concentration, a significant amount of sediment transport and beach profile changes occur near the step (eg. Atladottir, 2008; de San Román-Blanco et al., 2006; Kobayashi et al., 2011; Powell, 1990), and it is generally composed of sediment which is coarser than that of the adjacent beach profile.

On porous gravel beaches, larger sediments tend to move onshore and up-slope, while fine material tends to move down-slope (Buscombe & Masselink, 2006; Masselink & Turner, 2012). This phenomenon occurs because infiltration lowers velocities near the wave run-up limit (Buscombe & Masselink, 2006; D. Horn, Li, Hornt, & Lit, 2006). Coarser material requiring higher flow velocities for transport become stranded at the top of the beach face, whilst the wave run-down still has sufficient velocities to wash away finer materials. Over many swash cycles, this process leads to the upward migration of a lens of coarse sediment on the upper beach, known as berm building (Kobayashi et al., 2011). During storm events, berm building is particularly prevalent, and it is expected that natural LWD also tends to accumulate and influence berm building during this time (Kennedy & Woods, 2012). The introduction of an impermeable layer or an increase of the beach groundwater table relative to the ocean level can serve to interrupt this berm building process, resulting in higher wave run-down velocities and erosion (Bagnold, 1940). As such, a low beach groundwater table and porous material is generally considered to enhance sediment accretion on the shoreline, while a high groundwater table or impervious material is understood to inhibit accretion and promote erosion (Masselink & Turner, 2012).

Sediment transport mechanisms on gravel beaches are dominated by saltation, bedload, and sheet flow (Buscombe & Masselink, 2006). The primary method of transport, however, is bedload by shear due to the relatively large sediments with high fall velocities. The initiation of motion occurs when the drag forces on a sediment particle due to shear and turbulence exceed the resisting forces of gravity and friction. Notably, steep and shallow water waves are asymmetrical in shape, with higher orbital velocities at the wave crest than at the trough. Consequently, sediment is generally more mobile in the direction of wave propagation (onshore) than in the opposite direction (offshore).

Despite a general understanding of gravel beach behaviour and development, research is limited in comparison to that for sand beaches (Atladottir, 2008; D. Horn et al., 2006; Masselink & Turner, 2012). Models for predicting gravel beach evolution and equilibrium shape are also limited, and additional data is needed in order to validate models in development (Kobayashi et al., 2011), although recent numerical modeling advancements provide significant opportunities for better understanding (eg. Poate, McCall, & Masselink, 2016).

2.3.4 Wave Run-Up

The vertical distance that a wave rushes up a beach or structures is termed the wave 'run-up', as measured from the still water level (Arana, 2017; EurOtop, 2018; Goda, 2010). Run-up is driven by the momentum of incident waves and diminished by the porosity, roughness, and slope of the beach or structure (Arana, 2017).

For the purpose of defining and estimating wave run-up, two main variables are used:

1. **R_{max}** – The maximum limit of wave run-up. For the purpose of estimating R_{max} the beach profile needs to be approximately static (not changing in time), and the beach profile needs to extend past the maximum limit of wave run-up.
2. **$R_{2\%}$** – The run-up exceeded by only 2% of the incident waves. For the purpose of estimating $R_{2\%}$, the beach profile needs to be approximately static, the beach profile needs to extend past the maximum limit of wave run-up, and a statistically meaningful sample of waves is required.

Several formulas have been developed to characterize and estimate wave run-up; however, their applicability and accuracy to gravel beaches is still being investigated. Amongst the first attempts to estimate run-up, Battjes (1974) developed a simple formula for predicting wave-up under regular breaking waves on a smooth, impermeable slope, which related the non-dimensionalized run-up, R/H, to the Surf Similarity Parameter, ξ:

$$\boldsymbol{R}/\boldsymbol{H} = \boldsymbol{\xi} \qquad \text{for } 0.1 < \xi < 2.3 \qquad \textbf{(18)}$$

Later research found the Surf Similarity Parameter may be insufficient to describe wave-up for non-breaking waves, because it did not include consideration for wave kinematics (e.g. reflection and turbulence) and water depth (Arana, 2017; Poate et al., 2016). Dynamic movement of sediments and slope permeability are also understood to produce significantly different run-up patterns than static, impermeable slopes (Buscombe & Masselink, 2006; Powell, 1990). In response, newer formula developments have focused on incorporating variables and methods to account for these factors.

Of interest to this study in particular, Van Broekhoven (2011) proposed a hyperbolic function for porous structures, which was not specific to the breaker type:

$$\textit{Van Broekhoven, 2011:} \qquad \boldsymbol{R}/\boldsymbol{H} = \mathbf{2.25}\,\mathbf{tanh}(\mathbf{0.5}\boldsymbol{\xi}) \qquad \textbf{(19)}$$

Arana (2017) later proposed a formula for run-up of regular breaking waves on rough-permeable slopes based on data from Van Broekhoven (2011):

Arana, 2017: $$R/H = 3.74\,tanh(0.38\xi)\gamma_f\,\gamma_K \tag{20}$$

Where γ_f is the surface roughness coefficient and γ_K is the hydraulic conductivity coefficient approximately equal to $0.64*\xi^{-0.13}$ for gravel beaches. Arana (2017) proposed a new influence factor for hydraulic conductivity based on the dimensionless hydraulic conductivity, ψ.

Arana, 2017: $$\gamma_K = 0.94\psi^{-0.04}\xi^{-0.026\psi^{0.17}} \tag{21}$$

$$\psi = K^3/g\nu \tag{22}$$

Where K is the hydraulic conductivity (m/s) typically equal to 0.1 – 10 m/s for porous gravel substrate, g is the gravitational acceleration (m/s^2) and ν is the kinematic viscosity of water (m^2/s) approximately equal to $1.307*E^{-6}$ for freshwater and $1.3604*E^{-6}$ for salt water at 10^oC.

For the purpose of estimating $R_{2\%}$, for irregular waves, the beach profile needs to be approximately static and extend past the maximum limit of wave run-up at the same slope as the beach face, and a statistically meaningful sample of waves (length of test) is required. Because these conditions cannot generally be met for moveable bed tests, $R_{2\%}$ is difficult to obtain. Instead, the berm (or crest) height can be used as a proxy for the run-up height (van Hijum, 1975) and assumed to be approximately equal to $R_{2\%}$ (Van der Meer et al., 2016). The EurOtop Manual (Van der Meer et al., 2016) presents a basic formula for calculating the run-up of a shingle (or gravel) beach based on this assumption:

$$h_c/H_{m0} = 0.3S_{om}^{-0.5} \tag{23}$$

Where h_c is the beach crest height, H_{m0} is the spectral significant wave height, S_{om} is the wave steepness associated with the mean wave period.

It should be noted that the present understanding of wave run-up on gravel beaches is still considered to be poor, particularly because existing formulas are based on idealistic laboratory conditions for largely uniform or well-graded sediments (as opposed to more realistic mixed sediments) (J.W. Van der Meer et al., 2016).

2.4 Experimental Modelling

2.4.1 Introduction

The main factors influencing gravel beach profile morphology are: (1) wave height, (2) wave period, (3) wave duration, (4) beach material characteristics, and (5) angle of wave attack (de San Román-Blanco et al., 2006; Jentsje W. Van Der Meer, 1988). Additionally, LWD influence is greatly affected by the (6) elevation, (7) orientation, (8) size, and (9) arrangement (Johannessen et al., 2014). Test variables applicable to experimental modeling of LWD are discussed herein.

2.4.2 Scaling

2.4.2.1 General

Differences between model and prototype behaviour can result from three sources: (1) model effects, (2) measurement differences, and (3) scale effects (Heller, 2011). Model effects arise from incorrect replication of the prototype features, such as differences in geometry or wave generation methods. Measurement errors arise when using different data collection techniques at prototype-scale than at model-scale. Scaling errors arise when relevant force ratios are not kept constant between the prototype and model. Generally, scale effects increase as the ratio between the prototype and model scale (scale factor, λ) is increased:

$$\lambda = \frac{L_p}{L_m} \tag{24}$$

Where L_p is the characteristic length in the prototype and L_m is the corresponding length in the model.

In order to avoid scale effects that result from a scale factor not equal to one, a physical model would be required to achieve geometric (similarity of shape), kinematic (similarity of motion, i.e. time, velocity, acceleration, etc.), and dynamic (similarity of force ratios) similarity (Heller, 2011). Dynamic similarity requires that all relevant force ratios be equal in the prototype and the model. In fluid dynamics, these relevant force ratios include Froude, Reynolds, Weber, Cauchy, and Euler numbers. However, achieving similarity for all the force ratios is not practically possible and researchers must instead select force ratios that are most relevant, such that scale effects from the other force ratios are negligible.

2.4.2.2 Froude Scaling

For experimental modeling of open channel or free surface flows, similarity of the Froude number, Fr, is most often used (Sutherland & Soulsby, 2010), as per:

$$\boldsymbol{Fr} = \frac{\boldsymbol{U}}{(\boldsymbol{gd})^{\mathbf{0.5}}} \tag{25}$$

Where U is the flow velocity, g the gravitational acceleration, and d is the water depth. The resulting Froude number indicates the state of the fluid flow, where a Fr < 1 indicates sub-critical flow, Fr = 1 indicates critical flow, and Fr >1 indicates super-critical flow.

Froude scaling assumes that gravitational and inertial forces are dominant, and that viscous (Reynolds) effects are small. Generally, Froude scaling method is suitable for free-surface flows and large scale and/or turbulent models. Because surface waves are gravity-driven, Froude similarity will ensure that most wave-related phenomena are reproduced (NTNU, n.d.; Sutherland & Soulsby, 2010). For example, surface tension will have a less than 1% effect on linear wave propagation provided that the wave period is larger than 0.35s (NTNU, n.d.). During wave breaking, the breaker type and wave decay can also be accurately reproduced, provided waves are sufficiently large to minimize viscous and surface tension effects (Sutherland & Soulsby, 2010). To avoid inaccuracies within the surf zone, it is recommended to check the Reynolds number to ensure that turbulent flow is achieved in the model.

Scaling of several commonly used parameters using Froude similarity are provided in Table 2.

Table 2 – Scaling Ratios using Froude Similarity (Beulink, 2018 and NTNU, n.d.)

Parameter	Unit	Multiplication Factor
Length	[L]	λ
Area	[L^2]	λ^2
Volume	[L^3]	λ^3
Mass	[M]	λ^3
Time	[T]	$\lambda^{1/2}$
Velocity	[LT^{-1}]	$\lambda^{1/2}$
Acceleration	[LT^{-2}]	1
Discharge	[L^3T^{-1}]	$\lambda^{5/2}$

Note that because the prototype LWD structures for this study are located in a coastal environment, they are exposed to seawater with a typical density of 1025 kg/m^3. However, experimental modeling was undertaken using freshwater (for ease of availability and to avoid damaging pump systems) with a density closer to 1000 kg/m^3. As such, there is non-negligible scaling error induced for the density of water.

Scale effects become more significant with increasing scaling ratios. With a scale ratio of 1:5 (as in this study), scale effects are expected to be minimal.

2.4.2.3 Mobile-Bed Materials

Some sediment-related quantities can be correctly scaled using Froude similarity, provided that two general requirements are met (Sutherland & Soulsby, 2010). Specifically, bed material must have the same density as the prototype and the sediment size must be geometrically scaled. If these requirements are met, the following two quantities can be maintained between the model and prototype:

- *Shield's parameter* in rough, turbulent flow
- *Threshold Shield's parameter* for sediment with a non-dimensional grainsize, D*, greater than 120 (approx. D50 > 5mm)

Generally, this means that for gravel or cobbles, Froude scaling can be directly applied. In this case, the bedload transport rate, q_b, is scaled using a multiplication factor of $\lambda^{3/2}$ for units of $[L^2T^{-1}]$ (Sutherland & Soulsby, 2010). Alternatively, when undertaking mobile-bed experiments for finer sediments (i.e. $D_{50} < 5mm$) with Froude scaling, it is common to scale sediment size according to the fall velocity (e.g. Dean, 1991). Additional considerations for model sediment include consideration of electrochemical cohesive forces which occur for sediment with a diameter less than approximately 100μm and the presence of clays which can cause additional cohesion at model scale which is not present at prototype scale (Sutherland & Soulsby, 2010).

For this project, sediment has been scaled geometrically and rough, turbulent flow was achieved, in accordance with Froude scaling. Despite this, the modeled morphological response of the beach may still reflect scale effects caused by inaccurate representation of percolation/infiltration flows at low Reynolds numbers within the beach (Hughes, 1993). These effects are not well understood, but are expected to be small for large scale models.

2.4.3 LWD Characteristics

Several experimental (physical) studies have been conducted on LWD in rivers. These have typically been completed in 2D hydraulic flumes (Bocchiola, 2011; Braudrick et al., 2001; Wallerstein et al., 2001), but one recent study evaluated LWD in a 3D experimental model (Perry et al., 2018). Most of the previous studies used wooden dowels to model LWD (eg. Bocchiola, 2011; Braudrick et al., 2001); however, Hygelund & Manga (2003) used PVC piping, Wallerstein et al. (2001) used "cylindrical aluminium stock", and Perry et al. (2018) tested two different types of material: natural branches and fabricated cylindrical dowels. A major finding of the study by Perry et al. (2018) was that fabricated dowels exhibited different behaviour than natural wood (branches), suggesting that roughness plays an important role in transport and dam formation. As such, the authors recommended using natural branches/logs to model LWD in future experimental modeling programs.

Gallisdorfer et al. (2013) undertook an experimental testing program on engineered log jams in rivers, involving LWD. They outlined the design of their physical-scale model and use of physical scaling theory as directly applied to LWD in rivers. A dimensional analysis was conducted to determine important parameters for constructing the physical model, and scaling ratios are presented. Notably, they considered scaling effects on surface roughness of the LWD and found that the drag coefficient in the model and the prototype should be kept equal.

2.4.4 Beach Material and Slope

Van Der Meer (1988) conducted the most extensive set of tests on gravel beaches to date using a 50m long wave flume at Delft University, the Netherlands, and repeating some tests in a larger 230m long flume to evaluate scale effects. The nominal grain size diameter, D_{n50}, was varied between 4.0 to 27.2mm, and the beach slope was varied between 1:1.5 to 1:10. Notably, Van Der Meer (1988) found that the shape of the material (whether rounded, angular, etc.) made no or minor differences in the beach profile change. The gradation of material was observed to make a difference in the beach profile below the still water level, but made little influence on the profile above (within the run-up zone).

Powell (1990, as cited in Altadottir, 2008) also undertook an experimental study on gravel beach morphology in a wave flume in Wallingford, UK. Gravel beach material varied in size between 10-30mm (D_{50}), with a beach slope of 1:7 (H:V).

In a field study of beaches in New Zealand, Jennings & Shulmeister (2002) found an average beach slope of 1:5.6 (H:V) for gravel beaches. For mixed sand and gravel beaches, the average slope was approximately 1:10. The pure gravel beaches had an average grain size of 18.5 mm and the mixed sand/gravel beaches had an average size of 15.2 mm. Note that grain size analysis was completed based on the top 10 cm of the bed. For mixed sand/gravel beaches, where sorting and armouring may occur, this sampling technique may skew the reported mean grain size.

In the large-scale physical model study by de San Román-Blanco et al. (2006), the authors used gravel with a mean diameter of 21 mm on a 1:8 slope. The beach porosity was approximately 0.44. It was noted that, although the gravel material was not as rounded as that found on natural beaches, it was considered to be "within acceptable limits of angularity" (de San Román-Blanco et al., 2006).

Lee et al. (2007, as cited in Altadottir, 2008) conducted an experimental study in a 30 m wave flume. The modeled gravel beach had a 1:7 slope and an average grain size of 5 mm.

Altadottir (2008) conducted physical model studies on gravel beach morphology in the 64 m long steel wave flume at the National Research Council (NRC) Hydraulics Centre in Ottawa. A grain size of 8 mm was used in the study and a porosity of 0.37 was achieved. Slopes of 1:10 and 1:7 were both tested. Increases in the wave height and wave period were found to increase both erosion below the water level (e.g. trough depth) and berm crest height. The test results also indicated that gravel beaches were both highly reflective and effective at dissipating wave energy. The experimental set-up used in this research program closely resembles Atladottir (2008).

2.4.5 Beach Equilibrium Time

Beach equilibrium is generally defined as the point at which the net transport capacity has become zero (Jentsje W. Van Der Meer, 1988). The concept has been heavily used in studies involving sand beach morphology (e.g. Dean, 1991; Horn, 1992; Mcfall, 2019; Miller & Zeigler, 1958) and has been applied (to a lesser extent) to experimental studies on gravel beach morphology (Atladottir, 2008; de San Román-Blanco et al., 2006; Frandsen et al., 2015; Kobayashi et al., 2011; Lee et al., 2007; Masselink & Turner, 2012; Powell, 1990; Van Der Meer, 1988; van Hijum, 1975; van Hijum & Pilarczyk, 1982).

Van Hijurn and Pilarczyk (1982, as cited in Van Der Meer, 1988) used both regular and random waves. Random wave tests were generally found to reach an equilibrium profile after between 3000 and 6000 waves.

In the study by Van Der Meer (1988) five different irregular wave spectra were tested. Notably, the spectral shape had little to no influence on the beach profile morphology. Wave height and period were found to have a large influence on profile changes, with longer wave periods (and lengths) resulting in wider and higher beach profiles. Most of the beach profile changes occurred within the first few hundred waves. The author also noted that in most cases, reshaping of the beach was complete after between 1000-3000 waves.

In the 1990 study by Powell (as cited in Altadottir, 2008), tests were conducted using irregular waves with a JONSWAP spectrum and a water depth of 0.8m. It was found that wave height and period were amongst the principle variable effecting beach profile changes and 80% of the volumetric changes in the gravel beach profile occurred within the first 500 waves.

De San Román-Blanco et al. (2006) also used the JONSWAP spectrum for irregular waves. Tests usually comprised of 3000 waves, but were not specifically run until equilibrium of the beach profile was reached. Generally, however, the beach reached equilibirium after 2000-3000 waves. Notably, Ibrahim et al. (2006, as cited in Altadottir, 2008) investigated gravel beach morphodynamics with field measurements in England and compared the data to the experimental data by de San Román-Blanco et al. (2006). Although the experimental beach profile reached dynamic equilibrium, Ibrahim noted that the field beach profile never reached such a state.

Atladottir (2008) undertook their experimental study using both regular and random (irregular waves). For regular waves, wave trains had a period of 2.0s and heights of 15, 19, 21 and 25 cm. Six different random wave signals were created following the JONSWAP spectrum. Morphologic changes were rapid during the early stages of the test and generally reached equilibrium after 2500 waves for regular wave tests and after approximately 5000 waves for the irregular wave tests.

Frandsen et al. (2015) undertook an experimental modeling program on a sang/gravel/cobble beach under irregular wave conditions at a 1:3 scale. Significant wave heights ranged from 1.1 – 1.5 m and the peak period was set to 6 s. They found a much longer time for beach equilibrium than previous studies (approximately 10,000 waves); however, they only measured the beach profile after 2400 and 9800 waves, finding that beach equilibrium had only been met after the later of the two measurements. The measurement interval implies that beach equilbrium may have been met anywhere between 2400-9800 waves.

Note that it is difficult to compare beach equilibrium results between previous experiments due to a lack of continuity between test conditions (e.g. wave spectra, wave height, wave period, beach slope, and sediment diameter), the frequency at which beach profile measurements are taken, as well as a lack of discussion regarding the metrics used for defining beach equilibrium. To the authors' knowledge, the previous experimental studies on gravel beach morphology did not include – or document – any systematic checks and thresholds for beach equilibrium (eg. Atladottir, 2008; de San Román-Blanco et al., 2006; Frandsen et al., 2015; Kobayashi et al., 2011; Lee et al., 2007; Masselink & Turner, 2012; Van Der Meer, 1988; van Hijum & Pilarczyk, 1982).

Despite this, it can generally be concluded that previous studies of irregular waves on mildly sloping gravel beaches observed beach equilibrium between 3000 - 6000 waves (Atladottir, 2008; Kobayashi et al., 2011; van Hijum & Pilarczyk, 1982) or 1000 – 3000 waves (de San Román-Blanco et al., 2006; Masselink & Turner, 2012; Van Der Meer, 1988). Previous studies for regular waves found that beach equilibrium was achieved after an estimated 1000 – 1800 waves (Lee et al., 2007) and 2500 waves (Atladottir, 2008).

2.4.6 Comments on Modeling of Submerged Pipelines

Numerous experimental modeling studies have been conducted on scour formation around submerged pipelines. Notably, experimental modeling of LWD using PVC piping or wooden dowels may be comparable to modeling a pipeline on a beach if similar experimental set-ups are used. A brief review of literature on this topic indicated that, although some research exists on scour around pipelines under wave action, much of the research is focused on steady flow conditions (e.g. Cataño-Lopera & García, 2007; Cheng et al., 2014; Myrhaug & Ong, 2009; Zang et al., 2019; Zhang et al., 2016; Zhao et al., 2018). Of the studies focused on pipelines under wave action, they generally consider fully submerged pipes on fine-grained sand beds, which limits their applicability to this study.

For example, Sumer & Fredsøe (1990) undertook some of the first experimental modeling on submerged pipelines exposed to waves. The pipe was placed on a horizontal sand bed with a mean sediment size, D_{50}, between 0.18 – 0.58 mm. The bed was located 0.40 m below the still water surface and wave height was not reported.

Çevik & Yüksel (1999) later undertook similar experiments of normally incident regular waves acting on submerged pipeline placed on the surface of horizontal, 5:1 (H:V), and 10:1 (H:V) beds. A sand bed was used with a mean sediment size, D_{50}, of 1.28 mm and 1.89 mm. The pipes were placed in the "shoaling zone" at unspecified depths for regular wave heights up to 0.16 mm.

In 2008, Myrhaug et al. developed a method to estimate the scour depth below pipelines for shoaling conditions on sand slopes. The authors noted that no existing data was available for irregular wave conditions on sloping beds.

Later studies focused on experimental modeling under irregular waves. For example, Kizilöz et al. (2013) modeled scour around a submerged pipeline under irregular waves, on both a horizontal and 10:1 (H:V) sloping sand bed with a median diameter, D_{50}, of 1.28 mm.

Zang et al. (2019) studied scour formation around a partially buried and fully submerged pipeline under oblique currents and waves. The horizontal bed was located 0.45 m below the water level, with regular waves reaching up to 0.17 m in height. Siliceous sand was used with a median diameter, D_{50}, of 0.37 mm.

In addition to not being specifically applicable to the experimental set-up of this study, the existing research on pipelines also focuses on scour directly around the pipeline and not on the nearby bed evolution. None of the studies focus on the influence of the pipes on wave run-up.

2.5 Discussion

Traditionally, 'hard' engineering shore protection solutions, such as seawalls or revetments, have been used to mitigate shoreline erosion, promote sediment accretion, prevent flooding, and extend upland areas seaward of the natural boundary (Bruun, 1972; USACE, 2002). However, there is a growing body of literature demonstrating that these types of structures may exacerbate erosion, limit sediment supply, and degrade coastal habitat (Chapman & Underwood, 2011; Heerhartz et al., 2014)

The mid-1900's marked the beginning of a movement to utilize 'soft' engineering approaches, often also called 'nature-based solutions'. This movement was driven by claims that these 'soft' approaches could provide the same level of protection while minimizing harm to, or, in some cases, restoring natural ecological systems (Morris et al., 2018; Pontee et al., 2016).

Recent literature suggests that natural deposits of LWD on estuaries or beaches may promote additional sediment accretion and reduce wave run-up. Some studies found that the presence of

LWD promotes the accretion of fine sand in beach dune systems during aeolian sediment transport by acting as “momentum sinks”; however, they may limit the volume of sediment transported further inshore (Eamer & Walker, 2010; Gonor et al., 1988; Heathfield & Walker, 2011). In a study on the effect of natural LWD deposits on gravel beaches, Kennedy & Woods (2012) concluded that LWD may trap sediment on the upper beach and prevent the inland penetration of waves. In part because of this understanding of natural accumulations, anchored LWD has become a promising nature-based solution and is frequently installed in conjunction with other more natural techniques (Elliott et al., 2016; Zelo et al., 2000). Increased usage of anchored LWD in the coastal environment is being driven by the following factors:

- Decreasing quantities of natural accumulations of LWD
- Increasing demand for nature-based techniques
- Continuity of design and construction practices from river engineering
- Relatively economical installation

No peer-reviewed literature was found on the design or efficacy of LWD protection systems in a coastal environment. Despite this, guidance documents promote anchored LWD as a cost-effective and environmentally-friendly method to protect coastal areas (eg. Johannessen et al., 2014; Stewardship Centre for British Columbia, 2016; Washington Department of Fish and Wildlife, 2016). They generally recommend the following:

- LWD as coastal protection is most suitable for low to moderate wave energy environments
- LWD should be placed above the high water mark, but within the maximum wave run-up limit (to mimic the location of natural accumulations).
- LWD should be embedded or buried into the beach substrate.
- LWD can be secured in place using surface rocks, root wads, or more traditional anchoring techniques, such as cables and buried blocks. Despite this, they recommend avoiding completely rigid anchoring systems.

However, these works do not provide a basis for anchoring LWD on a foreshore to prevent erosion and they do not provide engineering guidance or best practices for doing so. More specifically, the following general issues exist regarding the usage and understanding coastal protection using LWD:

- Multiple public documents already recommend using LWD as coastal protection
- Design guidance for rivers is generally not applicable to a coastal environment
- No systematic field studies have been completed in a coastal environment
- No academic research exists on anchored LWD under wave action
- Only limited research exists on natural accumulations of LWD

Research is specifically needed to understand the efficacy of coastal protection using LWD in mitigating erosion, promoting accretion, and reducing wave run-up and wave-induced flooding in a coastal environment (Zelo et al., 2000). In addition, evidence-based design guidance needs to be developed before this technique can be used in the Coastal Engineering community with any level of certainty.

Chapter 3. Field Investigation of Anchored Large Woody Debris Coastal Protection Structures

3.1 Introduction

The field of Coastal Engineering arose to better understand and, in many cases, manipulate coastal processes in order to make coastal areas more suitable for anthropogenic uses (Bruun, 1972). ‘Hard’ approaches, such as seawalls or revetments, have traditionally been used to meet these needs (Bruun, 1972; Bulleri & Chapman, 2010; USACE, 2002). However, there is a growing body of literature demonstrating that these types of structures may exacerbate erosion, limit sediment supply, and degrade coastal habitat (Bulleri & Chapman, 2010; Chapman & Underwood, 2011; Heerhartz et al., 2014). Coastal areas are also increasingly threatened by the combined effects of climate change and increased anthropogenic usage (e.g. Hansen et al., 2015; Heerhartz et al., 2014; Prosser et al., 2018; Readshaw et al., 2016). Coastal engineering approaches using natural and nature-based features (NNBF) have increasingly gained international focus as potential methods to mitigate these challenges (Bilkovic et al., 2017; Bridges et al., 2015; Eggermont et al., 2015).

NNBF (also referred to as Nature-Based Solutions or Soft Approaches or Green Infrastructure) are features which are either naturally occurring themselves or mimic naturally occurring conditions (Bridges et al., 2015). They include a wide variety of techniques to protect coastal areas, including beach nourishment (Komar & Allan, 2009), salt marsh development (Davis et al., 2006), oyster reef construction (Morris et al., 2019), and Large Woody Debris (LWD) installation (Zelo et al., 2000), amongst other techniques. Peer-reviewed research and evidence-based design guidance for some of these techniques, such as beach nourishment design, are already available (eg. Bilkovic et al., 2017; Dette et al., 2002; Komar, 2007; USACE, 2002). However, guidance is limited for many NNBF as they comprise a relatively new field of coastal engineering practice and because they span such a wide variety of techniques (Pontee et al., 2016).

LWD have recently gained popularity as a NNBF, particularly in the Pacific Northwest where natural accumulations of LWD have historically been prevalent (Brennan et al., 2009; Gonor et al., 1988; Heathfield & Walker, 2011; Sass, 2009) (Figure 7a,b). LWD are referred to under various names, including: drift logs, drift wood, coarse woody debris/material, coarse woody habitat, coarse woody structure, and large woody debris (Sass, 2009; Zelo et al., 2000). Coastal protection using LWD generally includes logs or driftwood larger than 0.3 m in diameter and 2.0 m in length, anchored and/or partially buried into the shoreline, with or without root masses (Figure 8) (Johannessen et al., 2014; Wilson, Nistor, & Mohammadian, 2019; Zelo et al., 2000).

Figure 7 – Examples of Large Woody Debris anchored into the foreshore using cable and rock ballast in Comox, BC. (a) View looking alongshore and (b) close-up of the anchoring system.

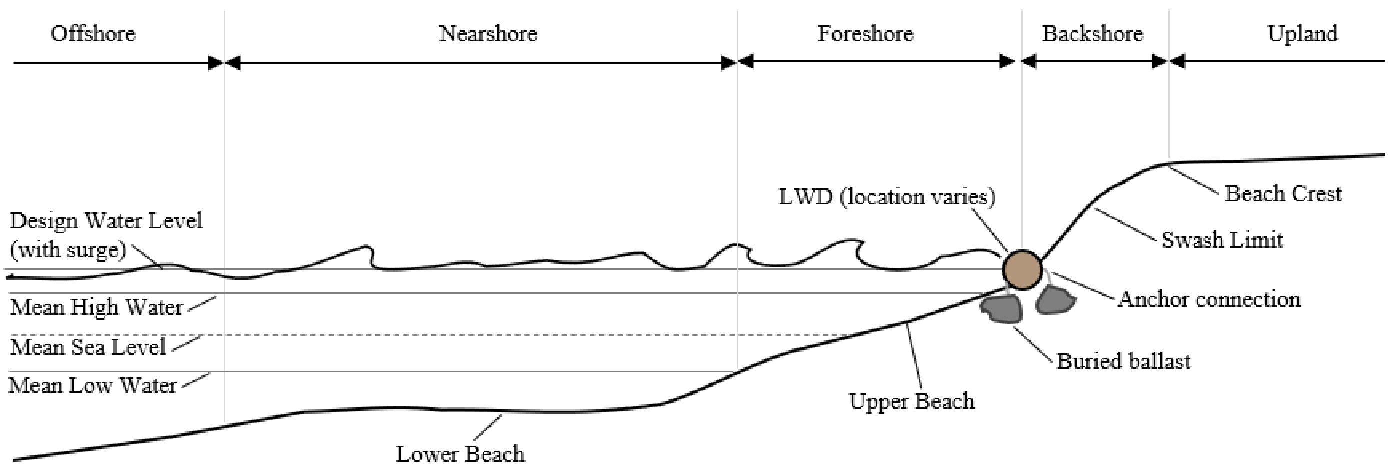

Figure 8 – Schematic of anchored LWD on a beach face and key features

Using Large Woody Debris as a shoreline stabilization measure in the coastal environment likely stems from it's frequent deployment in the fluvial environment (Menashe, 2001). In a study on LWD usage in river restoration projects in the UK, Cashman et al. (2018) found 912 projects involving LWD, from an archive of 4,894 potential projects. The authors also anticipated increased and more widespread usage of LWD in the future for the purpose of habitat restoration and flood management. Because LWD in river restoration is commonplace, there is significant literature available on its function and design (Bocchiola, 2011; Shields, Morin, & Cooper, 2004; Gonor et al., 1988; Hilderbrand et al., 1998; Hills & Street, 2007; Kail et al., 2007; Rafferty, 2017; Sass, 2009). Rafferty (2017) developed a computational tool for evaluating and designing the stability of anchored LWD structures in river environments. The U.S. Army Engineer Research and Development Center (USBR and ERDC, 2016) developed a comprehensive outline of planning, design, maintenance, and construction guidance for LWD in river restoration and shoreline protection projects. Shields et al. (2004) suggested that the design problems associated with LWD in the river environment include: (1) the use of buoyant materials, (2) the use of organic material that will decay over time, and (3) meeting the dual objectives of reducing erosion and habitat enhancement. These design problems are expected to be further exacerbated in the coastal environment by the presence of multi-directional currents and sediment transport, rapidly varying water levels, and wave effects.

Natural accumulations of LWD are understood to have important coastal ecological functions, including moderation of substrate temperatures, increased detritus accumulation, provision of substrate for vegetation growth, and improved foreshore habitat complexity (Brennan et al., 2009; Heathfield & Walker, 2011; T. S. Lee et al., 2018; Sass, 2009). The ecological benefits of placed or anchored LWD, as opposed to naturally accumulated LWD, are also still under debate. For example, Jones et al. (2016) also questioned whether placed or anchored LWD mimic the performance and ecological function of naturally recruited LWD in estuaries.

Natural LWD accumulations have been shown to have potential shoreline stabilization benefits (Eamer & Walker, 2010; Gonor et al., 1988; Grilliot et al., 2018; Heathfield & Walker, 2011; Huff, 2015; Kennedy & Woods, 2012). Kennedy & Woods (2012) demonstrated that LWD on gravel beaches in New Zealand may serve to trap sediment on the upper beach and prevent the inland penetration of waves (i.e. reduces run-up). LWD matrices have also been shown to serve as roughness elements on sandy foredunes, promoting accretion in their vicinity during aeolian transport and correspondingly limiting the volume of sediment transported to dunes further inshore (Eamer & Walker, 2010; Gonor et al., 1988; Heathfield & Walker, 2011). Huff (2015) found a correlation between decreased sediment transport and increasing LWD concentration in a river delta setting, suggesting that LWD may be an effective buffer against wave action. However, no peer-reviewed studies on anchored LWD in a coastal setting have been completed to date, and their potential to provide similar benefits to natural accumulations is not understood. In addition, Zelo et al. (2000) indicated that once mobilized, LWD may actually cause shoreline damage.

Zelo et al. (2000) and Johannessen et al. (2014) made efforts to survey and assess existing coastal protection projects that utilized anchored LWD. These works were not peer-reviewed and the evaluation metrics were not well-defined. Notably, restoration projects without rigorous and well-defined evaluation strategies are more likely to draw positive conclusions (Morandi, Piégay, Lamouroux, & Vaudor, 2014). Because of a lack of research, Jones et al. (2016) recommended generally avoiding the use of LWD in habitat restoration projects in tidal or coastal environments, and to treat its use as an experiment paired with rigorous monitoring.

Despite a lack of peer-reviewed literature or evidence-based design guidance on the use of anchored LWD as coastal protection, they have been utilized in low-cost coastal protection works for decades in Washington state, USA, and in British Columbia (BC), Canada (Wilson, Nistor, Mohammadian, Cornett, & Lamont, 2020). LWD are increasingly referred to and advocated as an ecologically beneficial NNBF (Brzozowski & Nichols, 2014; Coyle & Dethier, 2006; Elliott et al., 2016; Gianou, 2014; Johannessen et al., 2014). Government agencies are also increasingly referencing this technique as a method of stabilizing the shoreline while meeting environmental and permitting requirements (eg. Adamus Resource Assessment Inc et al., 2011; Government of Alberta, Canada, 2011; Washington Department of Fish and Wildlife, USA, 2016). To inform future usage, research is urgently needed to understand the role and influence of anchored LWD in mitigating erosion, promoting accretion, and reducing wave run-up in a coastal environment (Grilliot et al., 2019; Kennedy and Woods, 2012; Zelo et al., 2000; Shipman, personal communication, September 20, 2018).

The objectives of this study include determining: (1) if LWD are effective at stabilizing a shoreline under wave action and reducing erosion and wave run-up, (2) if LWD are durable enough to meet engineering requirements for shore-protection, and (3) what are best practices for designing coastal protection using LWD. This paper presents results of the first phase of this research project, which involved field surveys of numerous existing anchored LWD projects throughout coastal British Columbia, Canada and Washington State, USA. The aim of this paper is to provide an initial understanding of typical anchored LWD design characteristics, potential benefits and issues related to their usage, and identify future research needs.

3.2 Study Sites

3.2.1 Physical Setting

Vancouver Island and the Olympic Peninsula separate the relatively sheltered waterbodies of southern Province of British Columbia (BC), Canada and northern Washington State from the Pacific Ocean. Collectively, these waterbodies (the Juan de Fuca Strait, the Strait of Georgia, Puget Sound, and their connecting regions), are referred to as the Salish Sea. Twenty-eight (28) potential study sites were identified within the Salish Sea region where anchored LWD are used in coastal protection works; however, for various reasons, field work could only be conducted at fifteen (15) of these.

The Salish Sea and the surrounding landmasses are the product of glaciation during the Pleistocene, followed by glacial retreat, isostatic rebound, eustatic sea level rise, and tectonic stresses (Finlayson, 2006). Rivers and streams have built deltas and estuaries along these shorelines. Wave action has re-shaped the coastline across a range of sea levels. The complicated interplay of these processes in the region has led to an extremely varied coastal landscape, ranging from steep rock fjords to extensive sand beaches. Mixed sand, gravel, and cobble beaches, however, are dominant features in the Salish Sea (Curtiss, Osborne, & Horner-Devine, 2009; Shipman, 2010). Coastal morphology is the product of a number of forcing mechanisms, including tides, surges, currents, winds, wind-waves, boat-wakes, groundwater conditions, overland run-off, and sediment supply (Finlayson, 2006; Osborne et al., 2019). Because anchored LWD are often installed in conjunction with beach nourishment using coarse sediments (Johannessen et al., 2014; Zelo et al., 2000), water levels and incident wave characteristics are expected to play particularly important roles in beach morphology and anchored LWD efficacy at the study sites.

3.2.2 Water Levels

The Salish Sea region generally experiences mixed semi-diurnal tides, with the exception of a localized area near Victoria, BC, in the Eastern Juan de Fuca, which experiences mixed-diurnal tides (Davenne, Masson, & Canada, 2001; Finlayson, 2006; Thomson, 1981). The shallow and narrow form of Puget Sound results in a relatively small tidal range at the Northern entrance and tidal amplification towards the Southern ends. Conversely, the Strait of Georgia experiences larger and more uniform tides throughout. Tidal ranges at each of the potential study sites are shown in

Figure 9a. The tidal ranges for sites in BC are calculated as the difference between Higher High Water Large Tide (HHWLT) and Lower Low Water Large Tide (LLWLT) at the nearest Secondary Port, based on CHS (2019). The tidal ranges for sites along the coast of Washington State are calculated as the difference between Mean Higher High Water (MHHW) and Mean Lower Low Water (MLLW) at the nearest Datum Station based on NOAA (2019).

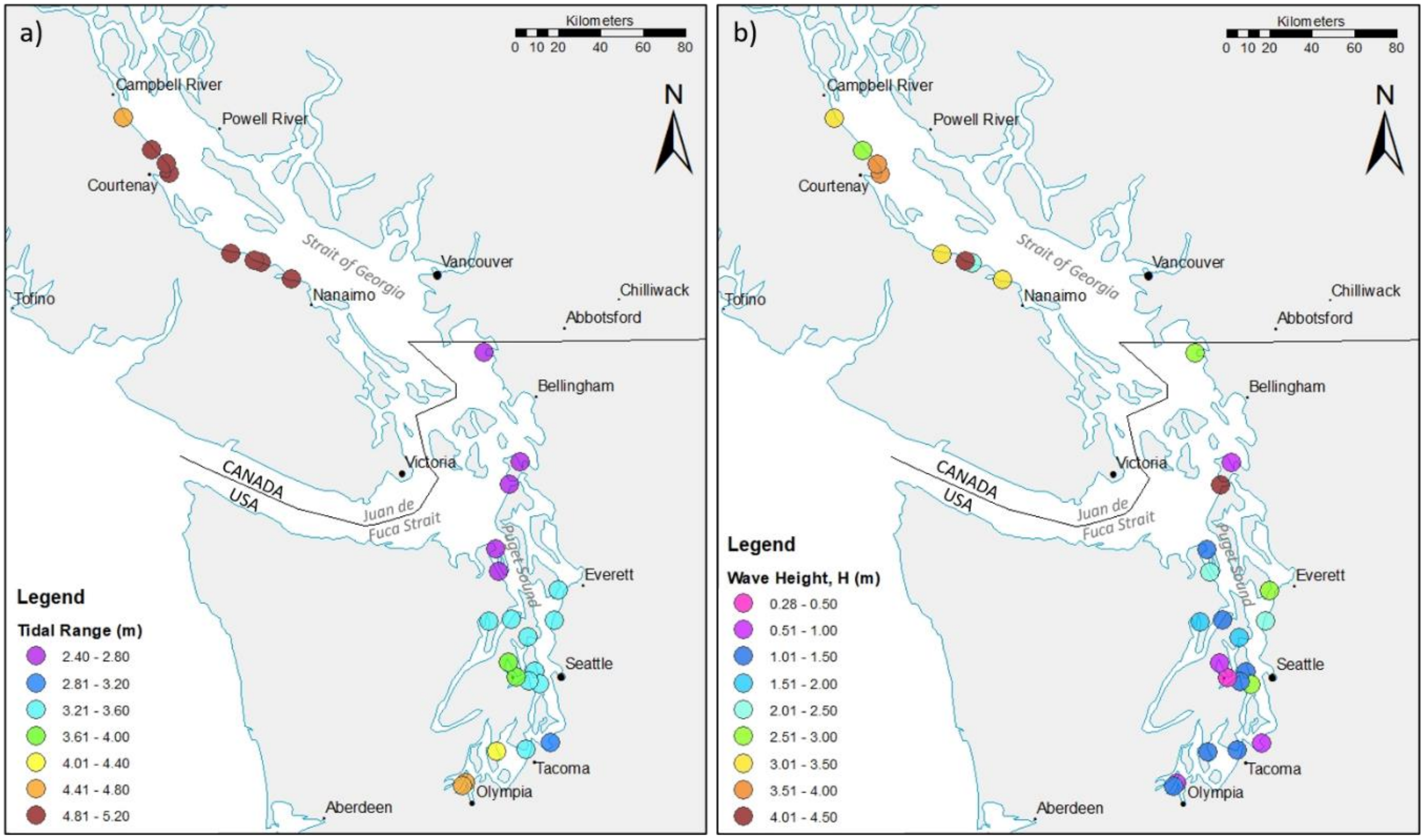

Figure 9 – (a) Tidal ranges and (b) fetch-limited wave height estimates at the 28 potential study sites

Storm surge due to low atmospheric pressure and wind set-up can raise the sea level above the astronomical tide level. In the Strait of Georgia, storm surge is estimated to be 0.75 m to 1.25 m for annual and 1/200 year Annual Exceedance Probabilities (AEPs), respectively (Ausenco Sandwell, 2011). Slightly smaller storm surge magnitudes have been reported for the Juan de Fuca Strait and Puget Sound regions, with the exception of some smaller connecting waterbodies, where the surge is expected to be amplified due to bathymetric forcing (Finlayson, 2006; Shipman, 2010; Yang, Wang, & Castrucci, 2019).

3.2.3 Wave Exposure

Wave exposure is recognized as a major driver of sediment transport in coastal areas (Buscombe & Masselink, 2006; Soulsby, 1997; USACE, 2002). However, erosion potential is also significantly complicated by other factors, such as longshore currents, sediment supply, the correlation of storm events with high water levels, and storm frequency, amongst other factors (Buscombe & Masselink, 2006; Kennedy & Woods, 2012; Kobayashi et al., 2011; Soulsby, 1997).

For this study, a simple wave hindcast was used as an indicator of wave exposure for each site following the Shore Protection Manual (SPM) method (USACE, 1984). For deep-water conditions, the effective fetch is the minimum of the equivalent fetch length for a given duration (duration-limited) and actual fetch length (fetch-limited). The estimated open-water fetch length varied between 1 – 110 km at the 28 potential study sites. The estimated fetch-limited wave heights were calculated for a wind speed equivalent to gale force (e.g. 34 knots or 17.5 m/s). Gale force winds are known to occur frequently (e.g. several times a year) throughout the study region (e.g. Finlayson, 2006; Johannessen et al., 2014; Northwest Hydraulic Consultants, 2014, 2004; Readshaw et al., 2016; Thomson, 1981), but their duration and directionality in relation to local topography is not always sufficient to produce these wave conditions (Finlayson, 2006). The hindcast also assumes deep-water and does not account for local wave transformation phenomena, such as refraction, diffraction, shoaling, and wave breaking (USACE, 1984).

Estimated fetch-limited wave heights for all 28 potential study sites are illustrated in Figure 9b. The wave climate in Puget Sound is generally fetch-limited, with significant wave heights of less than 3.0 m and peak periods of less than 6.5 s for gale force winds. For more nominal conditions, waves rarely exceed 1-2 m height and 3 s period (Finlayson, 2006; Johannessen et al., 2014). In the Strait of Georgia and the Juan de Fuca strait, sites are exposed to a more extreme wave climate associated with longer fetch lengths and may be duration limited for storms lasting less than approximately seven hours. For gale force winds and fetch-limited conditions, significant wave heights in these areas are estimated to reach up to 4.0 m with a peak period of 8.5 s. Sites in the Juan de Fuca Strait, in particular, are also exposed to long period swell from the Pacific Ocean (Thomson, 1981), which may be a particularly important driver of sediment transport.

3.3 Methods

3.3.1 General

Twenty-eight (28) potential study sites were identified where anchored LWD was used as coastal protection in northern Washington and southern British Columbia (Figure 10). Attempts were made to visit all the potential sites between June and August 2019. Sites in BC were scoped in advance of the site visits to confirm the site locations and site access. For accessibility and logistical reasons, this could not be completed in advance for sites located in Washington State.

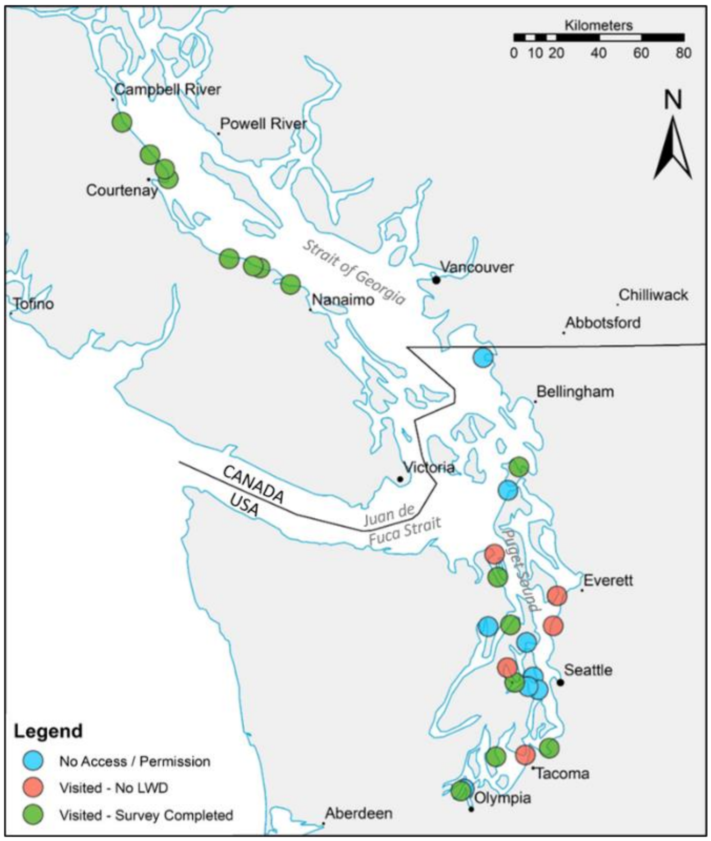

Figure 10 – Locations of LWD study sites and status of surveys in BC, Canada, and Washington State, USA

At five (5) of the locations in Washington State, no anchored LWD could be found and the installation site could not be identified. It is possible that the precise site location was not identifiable based on the available background documents, or that the LWD installations had since been removed or destroyed. An additional eight (8) sites in Washington could not be surveyed due to site access or permission constraints. Site visits were successfully completed at eight (8) sites in BC and seven (7) sites in Washington.

Site visits were conducted in-person and planned to coincide with the time of low tides when possible. The survey protocol included taking qualitative observations of the site and LWD structure, measuring key characteristics of the LWD structure and anchor systems, measuring key elevations and slopes at a central location within the footprint of the LWD installations, as well as photographing key features and the beach surface for a digital grain size analysis of the surficial beach sediment. Qualitative observations were made on the installation type, installation orientation, anchor attachment type, anchor ballast type, and the presence of root wads and bark. Observations were also made on the condition of the beach, LWD structure, and anchor system.

A standard steel measuring tape with a 1/16 inch (1.588 mm) precision was used to measure the length, end diameters, and exposed end heights of individual logs. Measurements of the anchor attachment (e.g. cable or chain) diameters were made using a digital caliper with 0.01 mm precision.

3.3.2 Slopes and Elevations

A transect line was laid along a representative section of the foreshore at each site and elevation measurements were taken on a 1.5 – 3.0 m interval to estimate the beach slope (Figure 11a). Multiple transects were used for large and/or complex sites to ensure the estimated slopes were representative.

A Real-Time Kinematic (RTK) GPS connected to the Can-Net Virtual Reference Station (VRS) Network was used to measure the elevation and location of the beach crest, the elevation and locations of LWD, and beach slopes at sites in BC. During the site visits, the RTK GPS had a reported vertical accuracy of less than 0.10 m (mean of 0.04 m) and a horizontal accuracy of less than 0.09 m (mean of 0.03 m). This system could not be used for sites in Washington State due to network access restrictions. Instead, a Spectra Precision rotary laser level (LL300N) and rod (Figure 11b) was used to measure slopes where site-access permitted, and elevation measurements were not made. The laser level has a reported accuracy of 2.2 mm at 30 m.

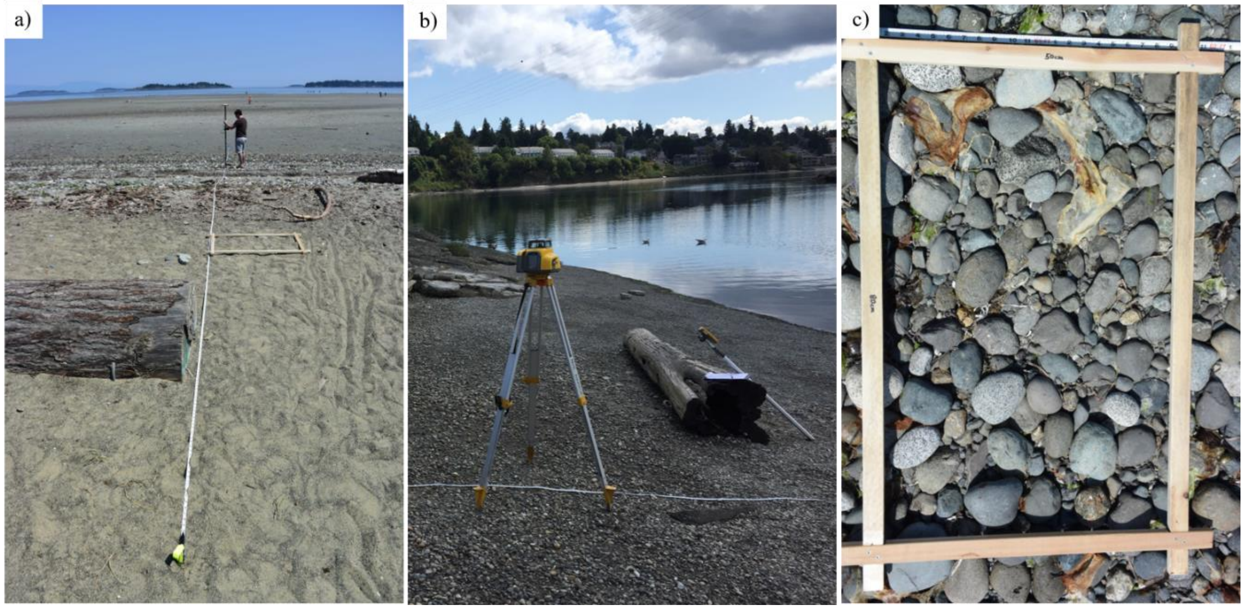

Figure 11 – (a) Transect line on foreshore used as the reference point for measurements, (b) rotary laser level and rod used to measure beach slope for accessible sites in Washington State, and (c) rectangular wooden frame with known dimensions used to support digital surficial grain size analysis

3.3.3 Grain Size Distribution

A digital grain size analysis was undertaken to estimate the grain size distribution of surficial sediments in-situ. Plan-view photographs were taken at representative locations on the beach surface using a 0.5 m x 0.8 m rectangular frame (Figure 11c). The images were orthorectified based on the known dimensions of the frame to adjust for image distortion and scale. A 0.1 m x 0.1 m grid was digitally overlaid on the images resulting in 28 intersection nodes per image, with several representative images of the upper beach slope at each site. The surface areas of sediment at each node were digitized and converted to an equivalent ellipse diameter (see Appendix C). A significant limitation of employed digital grain size analysis is that the resolution of the photograph limits the size of sediments that can be observed and therefore measured. Only sediments above approximately 2 mm were visible in images due to pixel resolution. As such, sediments smaller than 2 mm were accounted for in the grain size analysis based on the percentage of nodes with fine sediments. Debris and organic materials also reduced substrate visibility. The surficial grain size distribution may differ from the substrate grain size distribution, but is understood to be a good indicator of sediment mobility.

To compare the resulting grain size distribution from this method with other types of sediment sampling methods (e.g. bulk samples), it was necessary to convert the frequency distribution. The estimated grain sizes were converted from an area-by number to a volume-by-weight distribution, using a conversion factor of D^2 (Bunte & Abt, 2001).

3.3.4 Damage and Durability

Observations were made and recorded on the condition of the beach, LWD structure, and anchor system as indicators of both the efficacy and durability of the structures. Specific observations were made regarding missing LWD, beach erosion, scouring beneath the LWD, wood decay, anchor corrosion or failure, and other indicators of problematic design features or loss of functionality. A quantitative metric, Remaining Life (%), was assigned based on a qualitative assessment of durability issues observed at each of the study sites, following Table 3.

Table 3 – Rating scale to determine Remaining Life of anchored LWD structures

Remaining Life	Qualitative Description of Durability
100%	No signs of degradation
75%	Structure is functioning as intended with minor issues
50%	Structure is functioning partially as intended, or significant maintenance has been required
25%	Structure functionality is severely compromised
0%	Structure is missing or installation has failed

3.4 Results

3.4.1 Site Characteristics

Site visits were successfully undertaken at 15 of the 28 of the potential study sites. A summary of background information and site characteristics for each of the 15 surveyed study sites is included in Table 4.

Approximately one-third (10/28) of the potential study sites were privately owned. These privately owned sites were under-represented in the completed surveys due to constraints related to site access and permissions. Only two (2/15) of the completed surveys were on private property. The remaining study sites (13/15) were located on public property, primarily in municipal or provincial/state parks.

The age of the anchored LWD projects was determined based on available background documentation and are considered to be accurate to ±1 year. Approximately one-third of the study sites (4/15) were constructed more than 10 years prior to the date of the survey, with the oldest installation constructed in 1999 (Site 17). At the time of the field study, at least 50% of the original anchored LWD at Site 17 had already been replaced, several additional pieces of LWD had been added since the original construction, and the owner was planning to replace the remaining original anchored LWD. Nearly half of the study sites were between 5 to 10 years old at the time of survey (constructed between 2009 – 2014). According to local government officials, Site 7, which was constructed in 2011, was damaged and removed within a few years of installation. The remaining sites (4/15) were constructed within 5 years prior to the survey. Site 5, which was constructed in 2017, experienced significant erosion within two years of construction and additional beach fill was placed in 2019 as maintenance. Notably, usage of the year of construction as a proxy for structure durability was made difficult due to the unavailability of reliable information regarding timing, frequency, and extent of maintenance at each of the sites.

Tidal ranges and wave exposure were highest for sites in British Columbia, indicating that these sites may experience a larger range of metocean conditions than sites in Washington State. The mean tidal range (HHWLT – LLWLT) is 5.1 m for the surveyed sites in BC, compared to 3.4 m for those in Washington. The mean fetch-limited wave-height is H_{m0} = 2.6 m for sites in BC, compared to H_{m0} = 0.9 m in Washington.

The beach crest elevation was qualitatively determined in the field based on slope breaks, presence of upland infrastructure, and presence of terrestrial vegetation. The beach crest elevations ranged between 2.0 and 4.2 m above mean sea level (MSL); however, these elevations were highly variable based on the upland treatment and shoreline structure design. At each of the sites, a beach face and a low-tide terrace were generally identifiable by distinct changes in the beach slope and/or substrate material size. These two features are referred herein to as the upper and lower beach. The upper beach slope varied between 4.4:1 (H:V) to 10.2:1 with an average slope of 7.6:1. Notably the site with an upper beach slope of 4.4:1 (Site 6) was experiencing significant erosion and undermining of the LWD structure. The mildest upper beach slope of 10.2:1 (Site 2) was surveyed on a primarily sand beach. The lower beach slope varied between 8:1 and 500:1 (effectively horizontal/flat).

The substrate on the upper beach of the study sites was typically composed of mixed sediment, ranging from coarse sand to boulders. The mean sediment size, D_{50}, ranged from 18 mm (gravel) to 140 mm (cobble), with an average sediment size of 73 mm (cobble).

Table 4 – Overview of surveyed site characteristics

Site No.	Location	Site Description	Background Information				Surveyed Characteristics			
			Ownership	Construction Year	Tidal Range [m]	Estimated Wave Height, H [m]	Beach Crest Elevation, h_{beach} [m, MSL]	Upper Beach Slope, s_{upper} [-]	Lower Beach Slope, s_{lower} [-]	Median Grain Size on Upper Beach Slope, D_{50} [mm]
1	Parksville, BC	Seawall removal, beach nourishment, anchored LWD installation, and vegetation enhancement	Public	2016	5.0	1.8	2.6	0.11	0.00	57
2	Comox, BC	Riprap placement, anchored LWD installation on foreshore and backshore, and vegetation enhancement		2016	5.2	3.9	3.5	0.10	0.01	120
3	Comox, BC	Beach nourishment, anchored LWD installation, and vegetation enhancement		2011	5.2	2.2	3.9	0.11	0.05	139
4	Oyster River, BC	Anchored LWD installation and vegetation enhancement		2013	4.7	2.5	4.2*	0.14	0.13	93
5	Comox, BC	Beach nourishment, anchored LWD installation, rock groyne installation, vegetation enhancement		2017	5.2	2.8	3.5	0.13	0.06	91
6	Parksville, BC	Seawall removal, riprap placement, beach nourishment, and anchored LWD installation		2017	5.0	3.0	3.2	0.23	0.03	108
7	Qualicum Beach, BC	Beach nourishment, anchored LWD installation, and minor riprap placement		2011	5.1	2.3	4.2*	0.13	0.02	40
8	Lantzville, BC	Beach nourishment, anchored LWD installation, and riprap placement around culvert		2013	5.0	2.3	2.8	0.11	0.03	90
10	Fidalgo Bay, WA	Beach nourishment, LWD installation, and vegetation enhancement	Private	2009	2.5	0.7	-	-	-	27
11	Lofall, WA	Seawall removal, beach nourishment, and anchored LWD	Public	2011	3.3	0.8	-	-	-	29
12	Tacoma, WA	Concrete and riprap beach access, and anchored LWD installation (various orientations)		2011	2.9	0.7	-	-	-	62
17	Port Hadlock, WA	Beach nourishment (cobble), anchored LWD installation, and vegetation enhancement.	Private	1999	2.8	1.7	2.2*	-	0.03	28
25	Bremerton, WA	Beach nourishment and anchored LWD installation	Public	2008	3.6	0.3	-	-	0.03	18
27	Lakebay, WA	Seawall and riprap removal, beach nourishment, and anchored LWD installation		2012	4.1	1.1	2.0*	0.2*	-	55
28	Gravelly Beach, WA	Seawall removal, beach nourishment (boulders), and anchored LWD		2008	4.4	0.8	-	-	-	141

** Data was obtained from previous survey data or drawings, not collected during the current survey work*

Table 5 – Overview of LWD design characteristics and durability

Site No.	LWD Description	Design Characteristics						Durability Observations					
		Installation Type	Anchor Type	Ballast Type	Mean Diameter, D_{av} [m]	Mean Length, L_{av} [m]	Mean LWD Crest Elev., h_{LWD} [m, msl]	Missing LWD	Erosion / Deflation	Wood Decomp.	Anchor Corrosion / Failure	Other	Remaining Life [%]
1	Groups of logs buried into the beach, placed above the beach crest and wrack line. Large installation within a bay.	Double	Cable	Rock	0.82	7.7	3.1	-	-	-	-	No issues noted.	100%
2	Mixed single and double LWD, placed just below the beach crest, varied shoreline shape, with significant natural LWD accumulation	Single & Double	Cable	Rock	0.47	9.4	3.2	-	X	-	-	Significant natural accumulations of LWD.	75%
3	Complex single and matrix LWD, placed below the beach crest. Large installation on an exposed, straight shoreline.	Single & Matrix	Cable	Rock	0.45	9.5	2.8	-	X	-	X	Beach deflation.	50%
4	Double and matrix LWD, placed seaward of a riprap revetment. Short installation on a straight shoreline.	Double & Matrix	Cable	Rock	0.39	5.1	3.3	-	-	X	X	Beach deflation.	75%
5	Benched single and double LWD. Large installation on a straight, exposed shoreline. Repairs/ nourishment undertaken directly prior to site visit to resolve backshore erosion.	Benched	Cable	Rock	0.61	10.1	2.8	-	X	-	-	Erosion noted during a preliminary site visit.	50%
6	Single LWD placed above the beach crest on a steep slope. Short installation on an exposed section of shoreline, adjacent to rip-rap slope.	Single	Cable	Rock	0.86	8.1	3.7	-	X	-	X	-	25%
7	Benched single LWD, placed below beach crest. Short installation on straight, exposed shoreline. LWD damaged and removed prior to survey.	Benched*	Chain*	Pin*	0.75*	-	3.0*	X*	X*	-	X*	Installation had failed and was removed within a few years of installation.	0%
8	Single LWD, placed below the beach crest. Short installation of only two logs on exposed, straight section of shoreline.	Single	Cable	Rock	0.46	4.6	2.3	-	X	X	X	Arson noted.	50%

Site No.	LWD Description	Design Characteristics						Durability Observations					
		Installation Type	Anchor Type	Ballast Type	Mean Diameter, D_{av}	Mean Length, L_{av}	Mean LWD Crest Elev., h_{LWD}	Missing LWD	Erosion / Deflation	Wood Decomp.	Anchor Corrosion / Failure	Other	Remaining Life
10	Groyne installation using single and triple LWD. Clear signs of longshore transport.	Groyne	Chain	-	0.53	6.8	N/A	-	X	X	-	Erosion limited to one end of the site; the other end was largely buried.	75%
11	Mixed single shore-parallel and groyne LWD installation. Mild slope and straight, moderately exposed section of shoreline.	Single & Groyne	Chain	-	0.51	7.8	N/A	-	X	X	-	Wood decay noted for lower intertidal LWD. Minor backshore erosion.	75%
12	Mixed single shore-parallel and groyne LWD installation. Straight section of shoreline, exposed to regular vessel wake from nearby harbour.	Single & Groyne	Cable & Chain	-	0.37	7.1	N/A	-	X	X	X	Arson noted.	25%
17	Originally an installation of two single LWD cabled to concrete blocks. Additional small logs added by owner as recently as 2016. Mild slope on a straight, moderately exposed section of shoreline.	Triple	Cable, Chain, & Rope	Conc. Block	0.61	11.4	2.2**	-	-	X	X	Oldest (original) logs severely degraded.	50%
25	Mixed single and stacked LWD. Single LWD placed below beach crest elevation. Mild slope on a sheltered section of shoreline within a bay.	Stacked & Single	Cable	-	0.57	5.4	N/A	-	X	X	X	-	50%
27	Assumed single, shore-parallel installation. No LWD still anchored in their original position at the time of survey. Moderately exposed section of shoreline.	Single	Cable	-	0.44	7.2	2.0**	X	X	-	X	No LWD have remained anchored.	0%
28	Complex matrix-style LWD. LWD placed below beach crest elevation. Sheltered, straight shoreline.	Matrix	Chain	Rock	0.27	4.2	N/A	X	X	X	X	Beach deflation noted. Chains have caused damage to the wood.	0%

** Data was obtained from previous survey data or drawings, not collected during the current survey work*

*** Estimated based on the still water level at the time of the survey, not measured directly*

3.4.2 LWD Design Characteristics

A summary of anchored LWD design characteristics for each of the 15 visited study sites is included in Table 5. The observed LWD installations were grouped into six general categories (Figure 12):

1) *Single* – a single or individual log placed shore-parallel,
2) *Multiple* – two or more logs placed closely together in a shore-parallel manner, forming a matt-like structure,
3) *Benched* – multiple logs placed on the beach face at differing elevations, forming a stepped structure,
4) *Stacked* – multiple logs stacked vertically, similar to traditional bulkheads or seawalls,
5) *Matrix* – multiple logs placed in a complex matrix with variable density and orientation, similar to large natural accumulations of LWD, and
6) *Groyne* – single or multiple logs placed in a shore-perpendicular manner, similar to traditional groyne structures.

The majority of the study sites (9/15) used more than one of these LWD installation types. 53% of the visited study sites used Single LWD (8/15), 27% used Multiple LWD (4/15), 13% used Benched LWD (2/15), 7% used Stacked LWD (1/15), 20% used Matrix-Style LWD (3/15), and 20% used LWD as Groynes (3/15).

LWD were secured to ballast using three primary anchoring methods, including chain, cable, and rope. Only two of the study sites used multiple techniques. Cable was the most prevalent anchor type, observed at 73% of the study sites (11/15) (Figure 13a). Chains were also used at 40% of the study sites (6/15) (Figure 13b). Rope was observed at only one study site (1/15), where the owner had undertaken low-effort modifications to the original LWD design. One site also weighted the ends of the logs with large boulders without any anchor attachments (Figure 13c). Notably, all except for one site in BC used cables to anchor the LWD.

Ballast used to weigh-down the LWD generally included large rocks or boulders, concrete blocks, or soil pins. Ballast is generally buried into the beach during construction and was only visible at sites where some deflation of the beach had occurred or where the ballast had not been fully buried at the time of construction. Additional information was obtained from reference documents and drawings, where available. Ballast type was determined for ten of the study sites, eight of which used boulders, one of which used concrete blocks, and one of which used soil pins.

Study sites had an average LWD diameter, D_{av}, ranging from 0.27 to 0.86 m, with a mean of 0.54 m (Figure 14a). Average LWD length, L_{av}, ranged from 4.2 to 11.4 m, with a mean of 7.5 m (Figure 14b). The average crest elevation measured along the top of LWD, h_{LWD}, ranged from 2.0 m to 3.7 m above MSL, with a mean of 2.8 m above MSL (Figure 14c).

All LWD at three sites had root wads attached and a portion of LWD had root wads at five additional sites. The size and complexity of root wads was highly variable, ranging from 0.5 m up to 2.5 m in diameter. No root wads were observed at six sites. The presence of root wads was unknown for one site where the installation had been removed prior to the survey (Site 7).

Design drawings and as-built records were not available for most of the survey sites. However, it is generally understood that LWD structures were partially buried or backfilled into the beach at the time of installation. Despite this, all except two sites showed signs of undermining of the LWD structures, resulting in logs elevated up to 0.45 m above the beach grade. These observations suggest local scour around the LWD structures or deflation of the beach profile.

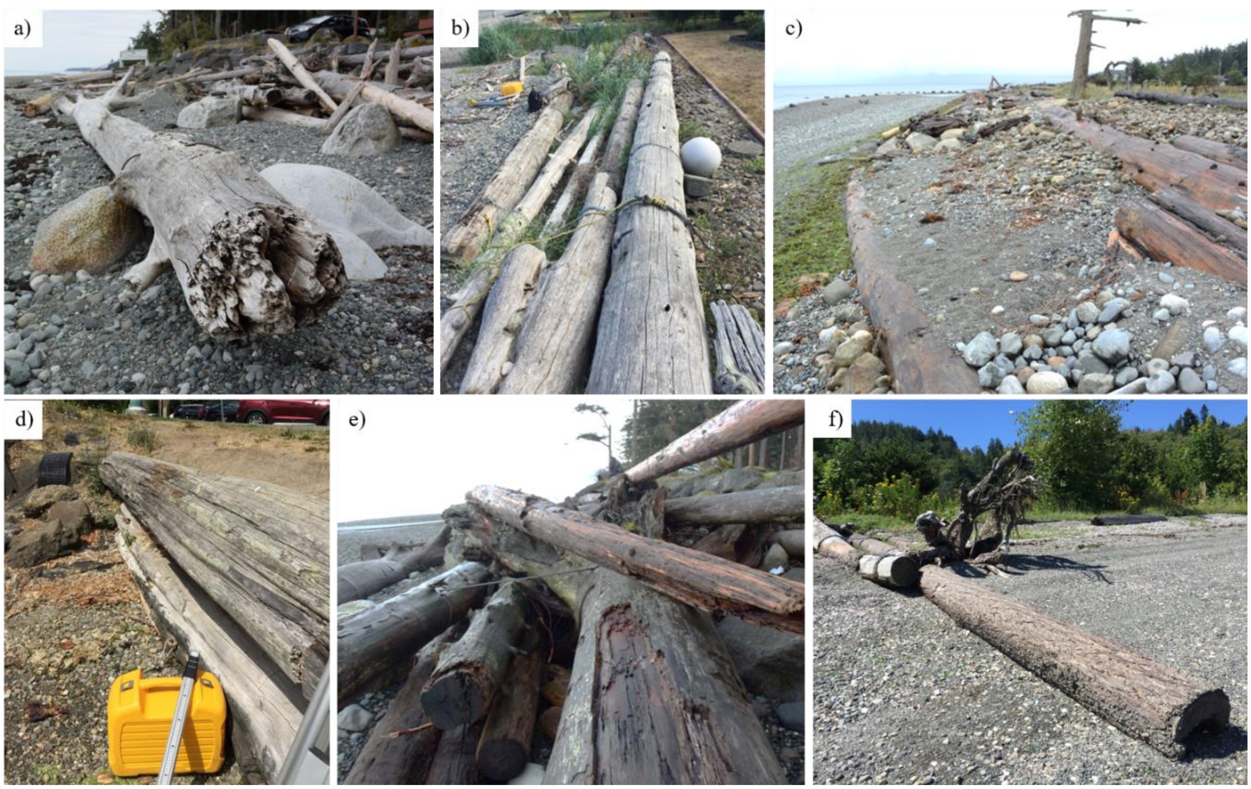

Figure 12 – LWD installation types observed during field surveys: (a) single, (b) multiple, (c) benched, (d) stacked, (e) matrix-style, or (f) groyne

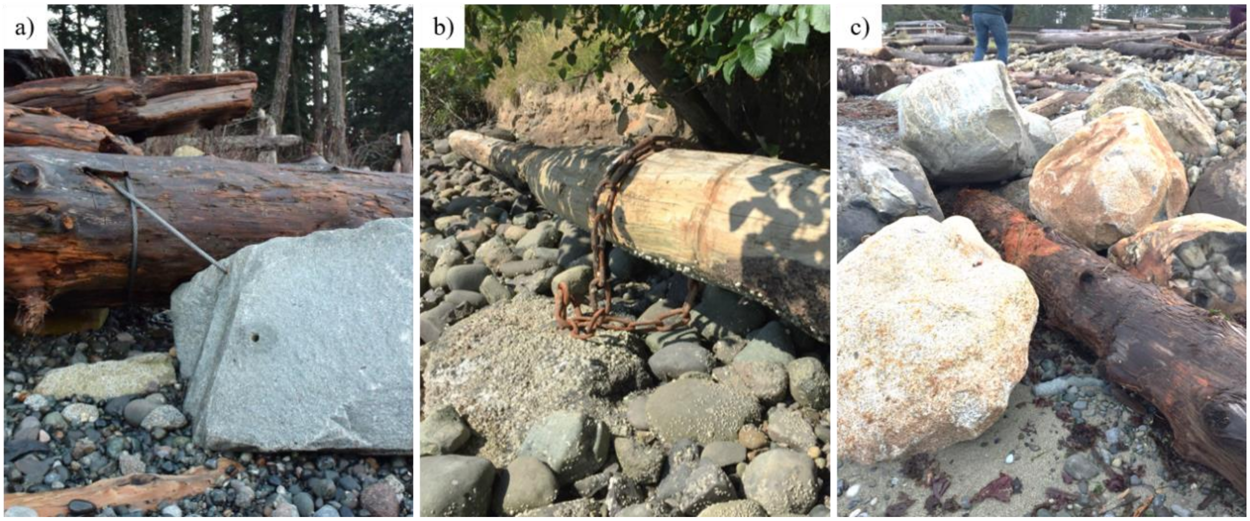

Figure 13 – LWD ballasting techniques: (a) steel cable drilled through LWD and fixed to a boulder, (b) chain wrapped around LWD and fixed to a boulder or concrete block, and (c) LWD weighted with boulders, without anchoring

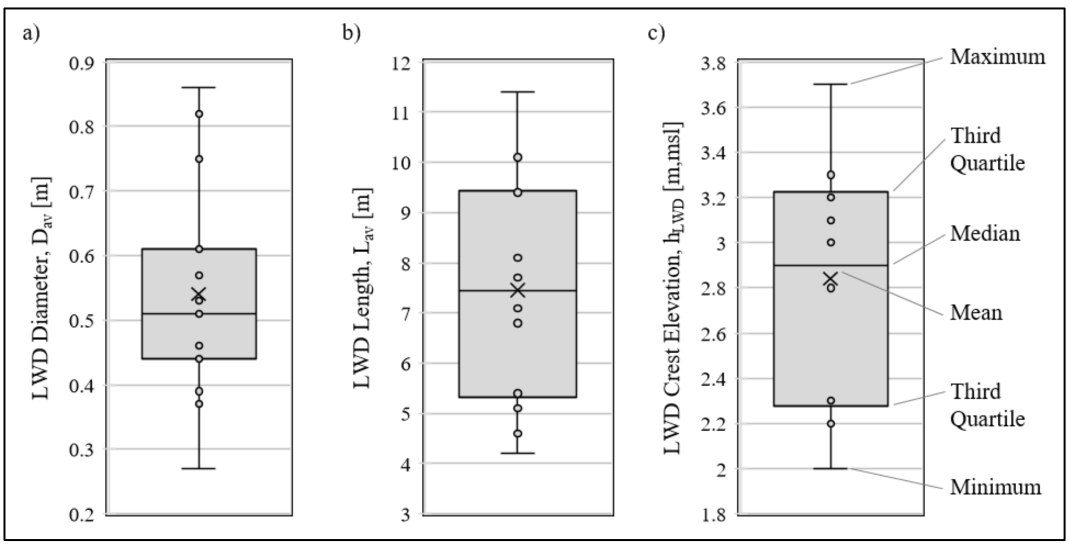

Figure 14 – (a) Range of LWD Diameters, D_{av}, (b) range of LWD Lengths, L_{av}, and (c) range of average LWD crest elevations, h_{LWD} measured at each of the study sites

3.4.3 LWD Durability

Evidence of damage or degradation to the site, LWD, or anchor and ballast was noted during the site surveys. Only one site (Site 1) had no evidence of damage or durability issues and was assessed to have 100% Remaining Life. Durability issues noted at the remaining sites included missing or loose LWD, erosion (backshore erosion, undermining or scouring of the LWD, or beach deflation), arson, wood decomposition (mechanical and biological), and anchor damage (breaks and corrosion) (Figure 15). Of the study sites, 20% had missing or loose LWD, 80% of the sites had evidence of erosion, 53% of the sites showed evidence of wood decay, and 67% of the sites had corroded or damaged anchors. Twelve (12) sites exhibited more than one durability issue.

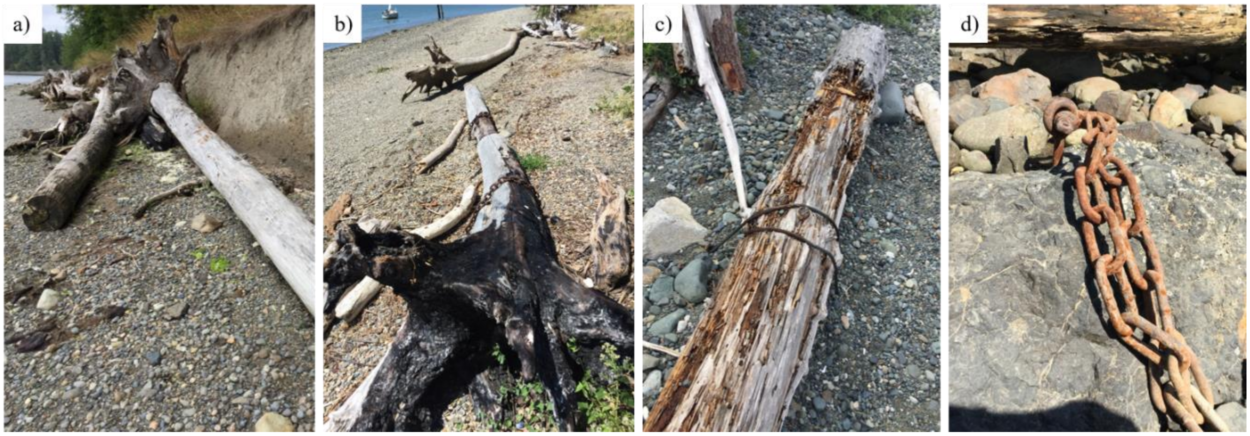

Figure 15 – Durability issues observed during site visits: (a) erosion, (b) arson, (c) wood decay, and (d) anchor corrosion/damage

3.4.4 Correlation of Site and Design Characteristics to Durability

To compare the durability of structures that were constructed in different years and had differing remaining useful lives, it was necessary to develop a secondary metric for durability, referred to here as the Effective Life. The Effective Life [years] is estimated as the Remaining Life [%] multiplied by an assumed 20-year Design Life plus the age of the structure at the time of the survey.

$$\boldsymbol{Effective\ Life = (Remaining\ Life \times 20\ years) + Age} \qquad (26)$$

The Pearson Product-Moment Correlation Coefficient was used to determine the linear correlation between various site and structure characteristics and the Effective Life. The correlation coefficient, r, is calculated as follows for a pair of samples:

$$\boldsymbol{r = \frac{\sum(x-\bar{x})(y-\bar{y})}{\sqrt{\sum(x-\bar{x})^2 \sum(y-\bar{y})^2}}} \qquad (27)$$

Where x and y are the individual points of samples one and two, and $\bar{x}$ and $\bar{y}$ are the means of samples one and two, respectively. Correlation coefficients for various site and LWD design characteristics and Effective Life are provided in Table 6. The correlation coefficient, r, ranges from -1.0 to +1.0, where -1.0 implies a perfect negative linear correlation and +1.0 implies a perfect positive linear correlation. Statistical significance of the correlation was determined for 90% and 95% confidence intervals. The tidal range, and upper beach slope showed statistically significant correlation to the Effective Life when considering all of the study sites and installation types. The upper beach slope and the relative elevation of the LWD compared to the beach crest elevation also showed statistically significant correlation to the Effective Life when considering only sites with single, shore-parallel LWD structures. Additional correlations to other installation types were not completed due to insufficient sample size.

Table 6 – Correlations between observed characteristics and Effective Life

Characteristic	Correlation, r, to Effective Life	
	All Installation Types	Single, Shore-Parallel Only
Tidal Range [m]	-0.49 (n = 15) *	-0.19 (n = 8)
Est. Wave Height, H [m]	-0.21 (n = 15)	-0.20 (n = 8)
Beach Crest Elevation, h_{beach} [m, msl]	-0.22 (n = 10)	+0.67 (n = 8)
Upper Beach Slope, s_{upper} [-]	-0.65 (n = 9) *	-0.94 (n = 5) **
Lower Beach Slope, s_{lower} [-]	+0.08 (n = 9)	-0.00 (n = 5)
Mean Grain Size, D_{50} [-]	-0.37 (n = 15)	-0.25 (n = 8)
LWD Diameter, D_{av} [m]	-0.06 (n = 15)	-0.31 (n = 8)
LWD Length, L_{av} [m]	+0.22 (n = 14)	-0.04 (n = 8)
LWD Crest Elevation, h_{LWD} [m, msl]	-0.15 (n = 10)	-0.01 (n = 8)
Relative LWD Elevation, h_{LWD} / h_{beach} [-]	+0.15 (n = 10)	-0.84 (n = 5) *
Relative LWD Diameter, D_{av} / H [-]	+0.26 (n = 15)	+0.39 (n = 8)

(n = _) Indicates the sample size, n

** Indicates that the r-value is statistically significant within a 90% Confidence Interval*

*** Indicates that the r-value is statistically significant within a 95% Confidence Interval*

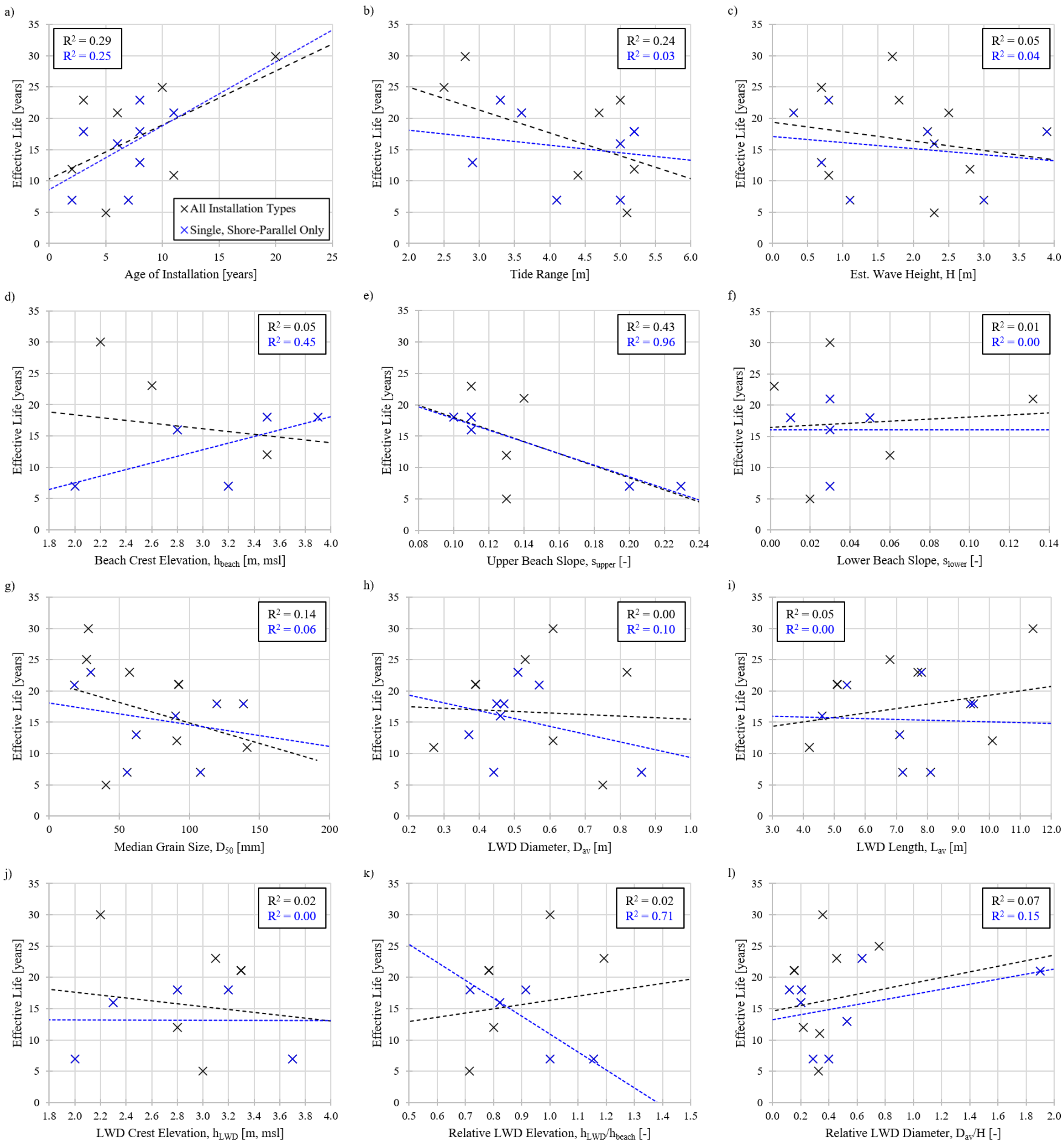

Figure 16 – Comparison of observed site and LWD design characteristics to Effective Life for all installation types (black symbols) and single, shore-parallel only structures (blue symbols). (a) Age of installation, (b) Tidal range, (c) Estimated wave height, H, (d) Beach crest elevation, h_{beach}, (e) Upper beach slope, s_{upper}, (f) Lower beach slope, s_{lower}, (g) Mean grain size, D_{50}, (h) LWD Diameter, D_{av}, (i) LWD Length, L_{av} (j) LWD crest elevation, h_{LWD}, (k) Relative LWD elevation, h_{LWD} / h_{beach}, and (l) Relative LWD Diameter, D_{av} / H. Indicated R^2 values are based on linear correlation for all installation types (black) and single, shore-parallel structures only (blue).

3.5 Discussion

3.5.1 Drivers for Usage

Anchored LWD have been utilized as a low-cost coastal protection method for decades in BC and Washington. Usage of anchored LWD in the coastal environment is understood to be driven by the following factors:

1. Demand for coastal protection to stabilize the shoreline and/or reduce wave run-up.
2. Increasing demand for nature-based techniques, in part, due to permitting and approval agencies promoting these techniques (eg. Adamus Resource Assessment Inc et al., 2011; Government of Alberta, 2011; Washington Department of Fish and Wildlife, 2016).
3. Continuity of design and construction practices from river engineering (e.g. USBR and ERDC, 2016).
4. Perception of decreasing quantities of natural LWD accumulations.
5. Potential for co-benefits such as: habitat improvements, potential recreational usage, and a more natural aesthetic.
6. Increased public awareness and support towards accepting less environmentally intrusive protection structures.
7. Perceived lower construction and maintenance costs compared to more traditional coastal protection methods.

Notably, most of the identified potential study sites were located on public property. This is potentially because nature-based coastal protection methods are associated with higher risk or uncertainty and public properties with little permanent infrastructure are better able to absorb these risks. In addition, private property owners may utilize these techniques but may be less likely to make public the construction, design, or monitoring details.

3.5.2 Site Characteristics

The surveyed sites are exposed to a range of water levels, driven primarily by tides, storm surge, and local wind/wave effects. Notably, the tidal range was found to have a significant negative correlation with the Effective Life of the structures. It is expectedly difficult to design anchored LWD structures for equal levels of functionality at the full range of expected water levels, due to the structure’s buoyancy and relatively small debris diameter or height in comparison to the tidal range. Long term, these sites will also be exposed to local sea level changes, which will provide an additional design challenge.

A beach face and low-tide terrace were generally identifiable at each of the sites, separated by a slope break. The average upper beach slope (7.6:1) is typical of mixed sand, gravel, and cobble beaches within the Strait of Georgia and Puget Sound (Finlayson, 2006; Thomson, 1981). Characteristic slopes of gravel and cobbles beaches also typically range between 6:1 – 10:1; However, gravel beaches have been observed to support slopes steeper than 6:1 (Buscombe & Masselink, 2006). The relatively flat and extensive low-tide sand terraces are a pervasive feature throughout the western Strait of Georgia and parts of Puget Sound (Curtiss et al., 2009; Finlayson, 2006; Thomson, 1981). The offshore step acts as a submerged slope break, located at the base of the swash zone (Buscombe & Masselink, 2006). At the step, wave bores develop, shoal, and collapse over the relatively shallow and flat beach section. The step is therefore understood to force and modulate wave-breaking. The step location and characteristics are thought to be directly related to the local incident wave height, and therefore to wave run-up and erosion potential.

The limited available information on anchored LWD design suggests that these structures are most suitable for low to moderate wave energy environments (Johannessen et al., 2014). Wave-related parameters were expected to play a larger role in LWD efficacy, as they are widely understood to be fundamental to coastal processes and beach profile development (Dean, 1991; Gomes da Silva, Medina, González, & Garnier, 2019; Jentsje W. Van Der Meer, 1988; van Hijum, 1975); however, the estimated wave height offshore of the project sites had a weak negative correlation (not statistically significant) with Effective Life. Notably, the estimated offshore (gale-force) wave height does not take into account local wave transformations, such as wave shoaling, diffraction, refraction, and depth-limited wave breaking, resulting in a potentially inaccurate representation of the wave energy at the shoreline and thus a weaker correlation than anticipated. In addition, many coastal processes are related to long-term trends driven by more nominal metocean conditions, and therefore may not be directly correlated to more extreme, but episodic storms. Notably, the long-term erosional or depositional behaviour of each site is unknown.

The influences of longshore and cross-shore sediment transport were not explicitly investigated in this paper; however, evidence of longshore sediment transport was noted at several of the sites. Because these mechanisms play an important role in erosion and accretion on beaches and therefore structure design and efficacy, these processes and their interaction with, and effect on, anchored LWD should be investigated further. In areas with significant aeolian sediment transport, LWD are expected to be a driver of sediment deposition due to increased surface roughness (eg. Eamer & Walker, 2010; Gonor et al., 1988; Heathfield & Walker, 2011); however, none of the currently investigated study sites were dominated by aeolian sediment transport.

The estimated beach crest elevations ranged between 2.0 – 4.2 m MSL. Some error in these estimations is expected because of difficulties determining the natural beach crest due to upland treatment and shoreline structure design. In addition, because substrates were generally composed of coarse sediment (mixed sand, gravel, and cobble material), the groundwater table and any compacted semi-impermeable substrates are expected to play a significant role in wave run-up and run-down. For example, the introduction of an impermeable layer or an increase of the beach groundwater table relative to the ocean level can serve to interrupt the berm building process, resulting in higher wave run-down velocities, increased erosion, and lower berm crests (Bagnold, 1940; Masselink & Turner, 2012).

3.5.3 Design Characteristics

Design drawings, as-built records, and monitoring information were not available for most of the survey sites. This lack of information made it challenging to understand the intended design and performance of the anchored LWD structures; however, general design characteristics, intended functionality, and Effective Life can be interpreted from the survey findings.

Six general LWD installation types were identified. The most prevalent installation type was a single, shore-parallel structure. 60% of the study sites used more than one of these LWD installation types. In addition, the LWD designs were almost always installed in conjunction with other natural and nature-based features, such as beach nourishment and revegetation, which is consistent with findings from previous case studies (Gerstel & Brown, 2006; Johannessen et al., 2014; Zelo et al., 2000). Some sites were also associated with adjacent or buried rip-rap structures, constructed to protect upland areas that are not able to tolerate higher risk or provide another layer of protection should the anchored LWD fail.

LWD structures were generally partially buried or backfilled into the beach at the time of installation (Johannessen et al., 2014; Northwest Hydraulic Consultants, 2014; Zelo et al., 2000). Despite this, all except two sites showed elevated or undermined LWD structures, with logs elevated up to 0.45 m above the beach grade at the time of the surveys. Because of a lack of information on long-term geomorphological trends at the study sites, it is not clear whether this finding is indicative of long-term erosional trends or an effect of LWD placement.

LWD were secured to ballast using primarily cable or chain. Cable was the most prevalent anchor type, associated with 73% of the installations. Chains were associated with 40% of the study sites. Notably, two of the study sites used two or more techniques, which is understood by the author to be, at least in part, due to inconsistencies in maintenance. Ballast used to weigh-down the LWD generally included large rocks or boulders, concrete blocks, or soil pins. Large rock/boulders were the primary method for ballasting the LWD, however, concrete blocks and soil pins were also used (less frequently).

The average LWD diameter, D_{av}, and the relative LWD diameter, D_{av} / H, were both not significantly correlated with Effective Life. This finding suggests that LWD diameter is not a governing factor in erosion potential or Effective Life; however, this may be due to the wide variety of design and site characteristics (such as the sediment grain size) and uncertainty in the local incident wave climate at each site. However, the finding does not provide insight into the role in LWD diameter in reducing wave run-up, which is one of the primary drivers behind anchored LWD usage. In addition, a larger diameter is expected to decay more slowly when exposed to biological and mechanical mechanisms for degradation; however, this hypothesis could not be studied within the existing dataset.

As expected, LWD length had little correlation with Effective Life. Length would be expected to correlate with erosion potential and Effective Life for groyne-style installations (Seed, 1997; Yossef, 2002), but the sample size for this installation type was not large enough to draw any conclusions in this regard.

The average crest elevation measured along the top of LWD, h_{LWD}, was not strongly correlated with Effective Life. This finding may be due to differences in the site characteristics (e.g. wave exposure and water level range) and design characteristics (e.g. installation type and wood type) at each of the sites. For single LWD installations however, the Relative LWD Elevation, defined as h_{LWD} / h_{beach}, was found to be significantly negatively correlated with Effective Life. This finding suggests that there are potential benefits associated with single LWD installations when they are anchored below the beach crest elevation, and within the swash zone; However, the finding cannot be extrapolated to LWD placed below the high-water mark, where it is understood that LWD will become regularly submerged, interact differently with wave and morphological processes, and decay more rapidly.

The influence of anchored LWD on wave run-up remains unknown; however, LWD did not generally appear to inhibit backshore erosion, indicating that waves were still able to reach the backshore and erode material.

Root wads are frequently featured in LWD installations, generally with the intent of improving stability or structure complexity. The variability of root wad sizes and installation types made it challenging to determine whether this intent was met. Generally, a reduction in foreshore and backshore erosion was not observed to be associated with the use of root wads. One possible explanation for this may be that the root wads help to stabilize the LWD on the foreshore, and thusly does not allow the LWD to settle and match the grade of the foreshore. The root wads consequently result in LWD that are supported by the root wad and elevated above the beach grade as the beach erodes or reshapes.

Bark was noted to some extent on LWD at the majority of the sites (11/15). There was no correlation found between the percentage of remaining bark, the structure age, or the Effective Life. However, it is expected that the presence of bark is closely tied to the origin of the LWD, whether procured from fresh, terrestrial sources or captured from natural LWD accumulations. In addition, as LWD are exposed to wave forcing and abrasion, any bark present at the time of installation will slowly be eroded.

3.5.4 Durability

At five of the potential study sites, no anchored LWD could be found and the installation site could not be identified, either due to a lack of available information on location or because the LWD installation had since been removed or destroyed. Of the surveyed sites, evidence of damage or degradation was noted at all except one site. Durability issues noted at the remaining sites included missing or loose LWD, erosion, arson, wood decay, and anchor damage. Arson was particularly noted for sites close to population centers. Wood decay was particularly noted for LWD placed lower in the intertidal zone and is expected to be related to usage of tree species with high rates of biodegradation (USBR and ERDC, 2016; Charles et al., 2016).

The tidal range, upper beach slope, and relative LWD elevation were all found to be strongly correlated to durability and Effective Life. The sediment size, relative LWD diameter

(D/H), and wave exposure may also be correlated to durability. In addition to these factors, it is anticipated that the following design practices and site characteristics may also affect durability and efficacy of LWD installations: maintenance techniques and frequency, wood type, sediment budget, predominance of longshore or cross-shore transport, sediment gradation and uniformity, groundwater conditions, installation type, anchor and ballast type, usage of associated techniques (e.g. beach nourishment, revegetation, or rock revetments), and anthropogenic uses.

Natural or nature-based approaches are advocated as providing the same level of protection while minimizing harm to, or, in some cases, restoring natural ecological systems (Morris et al., 2018; Pontee et al., 2016). However, without further study, it remains unknown whether installation of these LWD structures resulted in more or less stable coastlines and lower or higher flood and erosion risks, compared to the natural beach systems. Moreover, the basic services of coastal protection and ecological restoration cannot be provided if the implemented measures are not durable enough to meet the design life required of comparable conventional coastal protection structures, which is typically between 20 to 100 years. The mean Effective Life for the surveyed structures was only 17 years. The oldest structure surveyed was 20-years old at the time of the survey and had undergone significant repairs and maintenance. These findings suggest that anchored LWD installations generally have not been effective at providing coastal protection and meeting engineering design life requirements. It should be noted, however, that these results cannot conclude if these structures slowed or increased the absolute rate of erosion at the project sites in comparison to conditions without anchored LWD, only that significant erosion and durability issues were evident.

3.5.5 Preliminary Design Considerations

There are potential engineering, ecological, social, and economic benefits associated with anchored LWD installations if designed, installed, and monitored appropriately for the site conditions. This study provides initial insight into potential design flaws and durability issues associated with coastal protection using LWD. Potential remedies for these issues and preliminary design considerations include the following:

1. LWD will decay over time and, hence, tree species with low decay rates should be considered when exposed to wetting and drying cycles, and salt-water (USBR and ERDC, 2016; Charles et al., 2016; Shields et al., 2004).

2. LWD may degrade slower when placed above the beach crest, but are also expected to have a larger impact on beach stability when placed below the beach crest. It is not recommended to place LWD below the typical high-tide level due to consideration of buoyancy and decay.
3. LWD installations using chains were generally associated with higher failure rates and corrosion than installations with cables. Consideration should be given for marine-grade cables as a ballasting method.
4. Anchored LWD installations are expected to be better suited to areas with smaller tidal ranges or a smaller range of expected water levels.
5. Anchored LWD installations are better suited to sites with mildly sloping upper beaches. Notably, these sites may already be associated with a more inherently stable beach slope and may not require any intervention.
6. Anchored LWD's orientation and installation type should be selected with consideration of incident wave characteristics, sediment budget, and sediment transport conditions.
7. Anchored LWD may be suitable for sites with significant aeolian sediment transport (Grilliot et al., 2018).
8. Anchored LWD placed on sites near population centers may be susceptible to arson or tampering.
9. Anchors may become loose or exposed, posing a safety hazard and potentially resulting in LWD movement (Zelo et al., 2000). It is critical that inspections and maintenance are planned and completed.
10. There is still little information available on whether anchored LWD will enhance shoreline stabilization or protection from wave run-up. Any usage should be treated as experimental (Jones et al., 2016).

3.6 Conclusions

This paper presents the findings from site investigations of existing anchored LWD project sites in BC, Canada, and Washington State, USA. Based on this preliminary assessment of existing installations, the following conclusions can be made:

1. Anchored LWD have been used as coastal protection in these areas since 1999 or earlier.
2. Existing projects are generally lacking both design and construction documentation, long-

term monitoring, and adequate maintenance records.

3. LWD designs are highly variable, but dominated by single, shore-parallel structures.
4. Increased efficacy and design life are associated with sites with a smaller tidal range and milder upper beach slope for all structure designs, and are correlated to the LWD placement elevation relative to the beach crest for single, shore-parallel structures.
5. The effect of LWD on beach morphology is often site-specific and modified by exposure to complex forcing mechanisms, local sediment transport pathways and sediment budget, the inclusion or presence of other coastal protection techniques, and local substrate characteristics.
6. Typical durability issues included missing/loose LWD, failed or corroded anchors, erosion and beach deflation, arson, and wood decay.
7. Preliminary evidence suggests that anchored LWD installations have not been effective at providing coastal protection and have not met typical engineering design life requirements.

Additional research is needed to understand the role and influence of anchored LWD in mitigating erosion, promoting accretion, and reducing wave run-up in a coastal environment (Grilliot et al., 2019; Kennedy and Woods, 2012; Zelo et al., 2000). Aspects that require research include the following:

1. Influence of site characteristics, such as wave exposure, aeolian transport, substrate characteristics, groundwater conditions, and water levels on beach morphology and wave run-up
2. Influence of anchored LWD design characteristics, such as installation type, orientation, elevation, and diameter on beach morphology and wave run-up
3. Influence of anchored LWD stability or dynamic movements on sediment mobility and scouring
4. Anchoring/ballast system requirements
5. Relative effects of anchored LWD in comparison to more traditional coastal protection structures
6. Durability of anchored LWD structures in the coastal environment
7. Development of evidence-based design guidance
8. Potential dual benefits of anchored LWD, such as ecological, social, or economic benefits

Chapter 4. Experimental Modeling of Coastal Protection using Large Woody Debris: Morphological Response

4.1 Introduction

Evidence-based design relies heavily upon field observations and investigations, experimental modeling, numerical modeling, and empirical equations (Hughes, 1993). Traditional coastal engineering techniques, such as seawalls, revetments, and breakwaters, have been studied extensively, allowing for the development of reliable design guidance (e.g. USACE, 2002). However, there has been a growing trend towards the usage of more novel and nature-based techniques, due to an increased understanding that (1) traditional "hard" techniques may damage coastal ecosystems, (2) existing coastal ecosystems are shrinking and degrading, and (3) nature-based techniques may be more dynamic and adaptive (Bilkovic et al., 2017). The uptake has occurred despite the fact that design guidance and best practices are limited for many nature-based techniques (Bilkovic et al., 2017; Pontee et al., 2016), including for coastal protection using Large Woody Debris (LWD) (Wilson et al., 2020; Zelo et al., 2000)

Coastal protection using anchored LWD generally includes logs or driftwood larger than 0.3 m in diameter and 2.0 m in length, anchored and/or partially buried into the shoreline, with or without root masses (Johannessen et al., 2014; Wilson et al., 2019; Zelo et al., 2000). LWD have recently gained popularity as a nature-based technique, particularly in regions where natural accumulations of LWD have historically been prevalent (Brennan et al., 2009; Gonor et al., 1988; Heathfield & Walker, 2011; Sass, 2009). Decreasing coverages of naturally accumulated LWD and increasing demand for ecologically sensitive coastal protection measures has led to the promotion and usage of LWD as a viable nature-based coastal protection technique (e.g. Johannessen et al., 2014). However, it has been used as natural and economic means of shore protection at least since 1999, with anecdotal records and evidence as early as the 1970's. There installations have been constructed generally with the specific aim of reducing wave induced erosion and limiting wave run-up.

Despite its frequent usage, there is currently limited peer-reviewed literature on the design or efficacy of coastal protection using LWD. Simenstad el at. (2004; as cited in Jones et al., 2016) stated that research is still needed to understand the role and influence of Large Woody Debris.

Similarly to the literature on novel nature-based techniques in general, the documentation that does exist is generally only found in 'grey' literature (e.g. Johannessen et al., 2014; Zelo et al., 2000). For this reason, the design of anchored LWD to date has generally been based on (1) anecdotal observations and experience, (2) research that suggests that natural deposits of LWD may promote additional sediment accretion and reduce wave run-up (Eamer & Walker, 2010; Gonor et al., 1988; Grilliot et al., 2018; Heathfield & Walker, 2011; Huff, 2015; Kennedy & Woods, 2012), and (3) a continuity of design practices from the river engineering field, where significant literature is available on the function and design of LWD (Bocchiola, 2011; Douglas Shields Jr., Morin, & Cooper, 2004; Gonor, Sedell, & Benner, 1988; Hilderbrand, et al., 1998; Hills & Street, 2007; Kail et al., 2007; Rafferty, 2017; Sass, 2009).

Numerous numerical and physical models have been completed on anchored LWD in river environments. Braudrick et al. (2001) undertook a gravel-bed flume study to better understand how far LWD was transported in the river environment and where it deposited. Wallerstein et al. (2001) undertook a sand-bed flume study on the effect of LWD on morphology and hydraulics. Hygelund & Manga (2003) undertook field measurements with simulated logs to better understand drag coefficients associated with LWD in rivers. Bocchiola (2011) studied flow modifications due to LWD in a flume setting, and then analyzed resulting changes in habitat suitability for fish species. Perry et al. (2018) undertook a large-scale 3D experimental model on LWD accumulation at a flow control structure. Several other previous physical model studies involving LWD in rivers are summarized in Gallisdorfer et al. (2013). However, to date, no known experimental or numerical studies have been conducted on anchored LWD in a coastal environment.

The present study was completed as part of a collaborative research project between the University of Ottawa and the Ocean, Coastal, and River Engineering Research Centre of the National Research Council of Canada (NRC-OCRE). The aim of the study was to better understand the efficacy of coastal protection using LWD and to develop evidence-based design guidance for its' usage. Chapter 3 presented the results of the first phase of the project, which included extensive field surveys of existing anchored LWD project sites in British Columbia (BC), Canada, and Washington State, USA. The second phase of the project included a broad experimental wave modeling program to investigate the efficacy of LWD at meeting the design goals of stabilizing the shoreline and reducing wave run-up. The program also served to investigate the most effective configurations of anchored LWD for meeting these goals. This Chapter outlines the experimental modeling program and presents the study results related to beach morphology.

4.2 Methodology

4.2.1 Experimental Set-Up

The experiments were conducted in the steel-wave flume (SWF) at the NRC-OCRE in Ottawa, Canada. The SWF is approximately 63.0 m long, 1.22 m (4 ft) wide and 1.22 m (4 ft) deep. The flume is equipped with a piston-type wave generator at the north end and uses an active wave absorption (AWA) system to suppress wave energy reflected from the beach and LWD structure during the tests.

The coordinate system adopted for this experimental program is shown in Figure 17 and is consistent with previous experiments on gravel beach morphology conducted in the SWF by Atladottir (2008). The *x*-direction points away from the wave paddle located at the north end of the flume, the *y*-direction points away from the east wall of the flume, and the z-direction is directed up, above the still water level (SWL). A constant water depth of 0.6 m was maintained during all experiments and this still water level was set as the datum for all elevation measurements referred to throughout this study.

The program was completed at a 5:1 (prototype to model) scale, according to Froude scaling. The experimental set-up was developed based on observations of the site characteristics and LWD design characteristics made during the field study as part the first phase of the research project (see Chapter 3). A gravel beach was installed such that the entirety of the beach was visible within a section of glass walls which extended from approximately x = 37.0 m to x = 40.6 m on both sides of the flume. The beach had an 8:1 (H:V) slope with a median grain size, D_{50}, of 7.9 mm (prototype D_{50} of approximately 40 mm, cobble). The gravel material had a Uniformity Coefficient, $C_u = D_{60}/D_{10}$, of 1.491. The set-up generally coincided previous field observations, which found that anchored LWD structures were generally installed on the upper beach with a slope of approximately 7.6:1 and a sediment size between 18 mm (gravel) to 140 mm (cobble). The toe of the beach was located at x = 37.0 m and the crest at x = 45.0 m, 1.0 m above the bed of the flume (z = +0.4 m), as shown in Figure 18.

Model logs were constructed of hollow PVC pipes with steel and rubber end caps (Figure 18). Each model log was 0.114 m in diameter to simulate a 0.56 m diameter prototype log typical of the LWD observed at sites previously surveyed by the author (see Chapter 3). The logs were manufactured with one extendable end, such that each log could be fixed in place on the beach by applying lateral pressure to the side walls of the flume. The logs had a minimum length of 1.21 m and the capacity to extend of approximately 0.02 m. In total, three logs were manufactured to allow for arrangements into various LWD configurations. The test configurations included five of the six distinct installation types characterized in Chapter 3, as well as a beach with no LWD installed (Figure 19). Note that logs were generally positioned at the still water level within the flume, which is analogous to placement on the upper beach slope during design storm and water level conditions for the prototype.

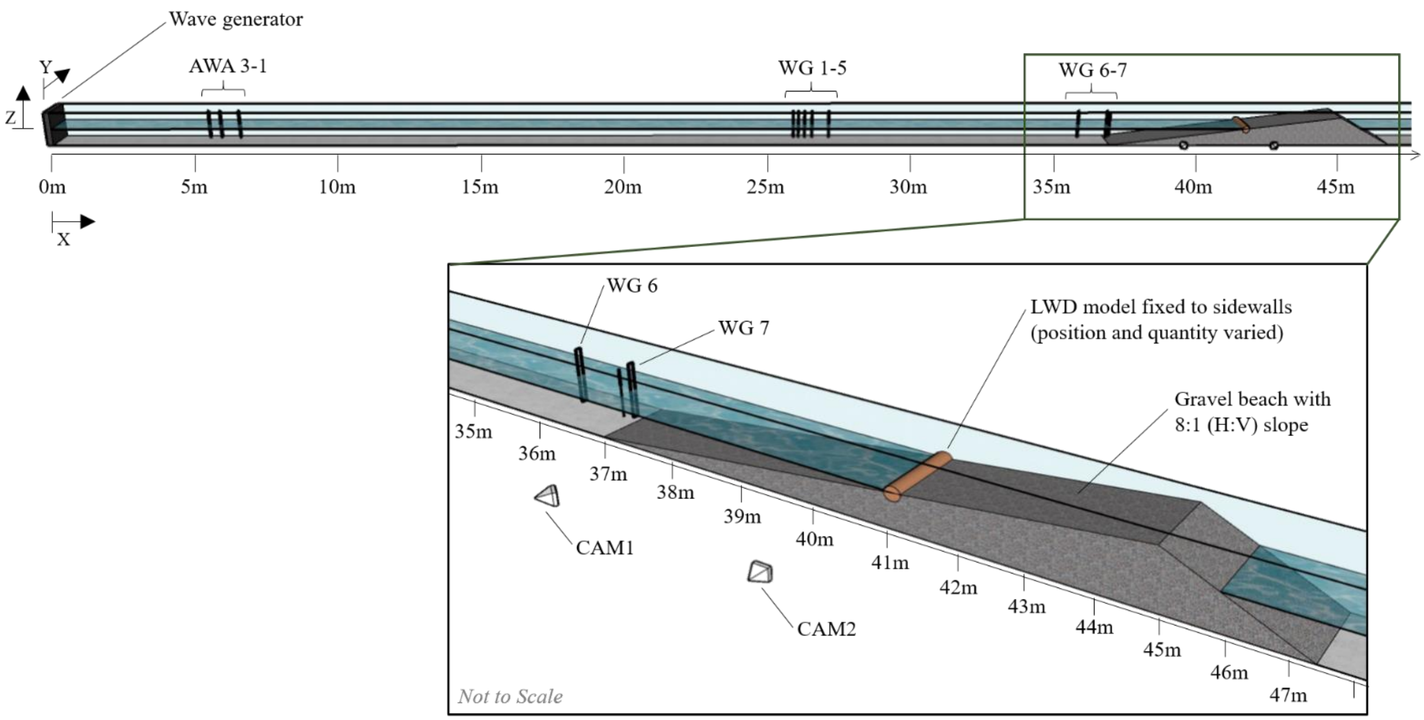

Figure 17 – Schematic of experimental set-up in the Steel Wave Flume at NRC-OCRE Ottawa

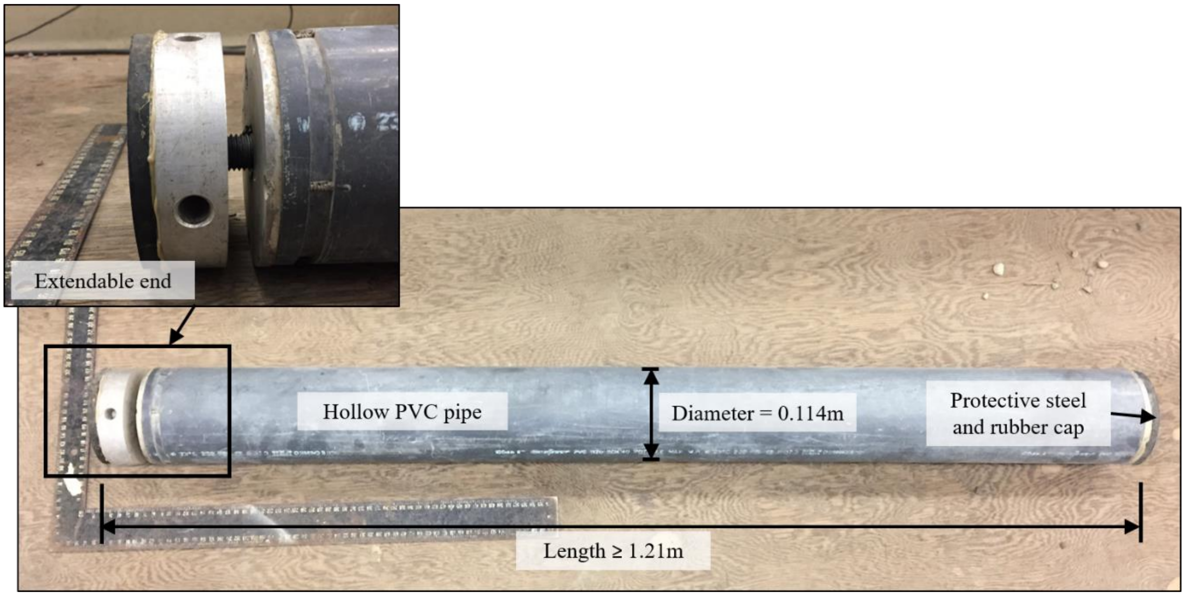

Figure 18 – Model log, manufactured with hollow PVC piping, steel caps, and rubber ends. One end extends to provide lateral pressure and fix the log in place.

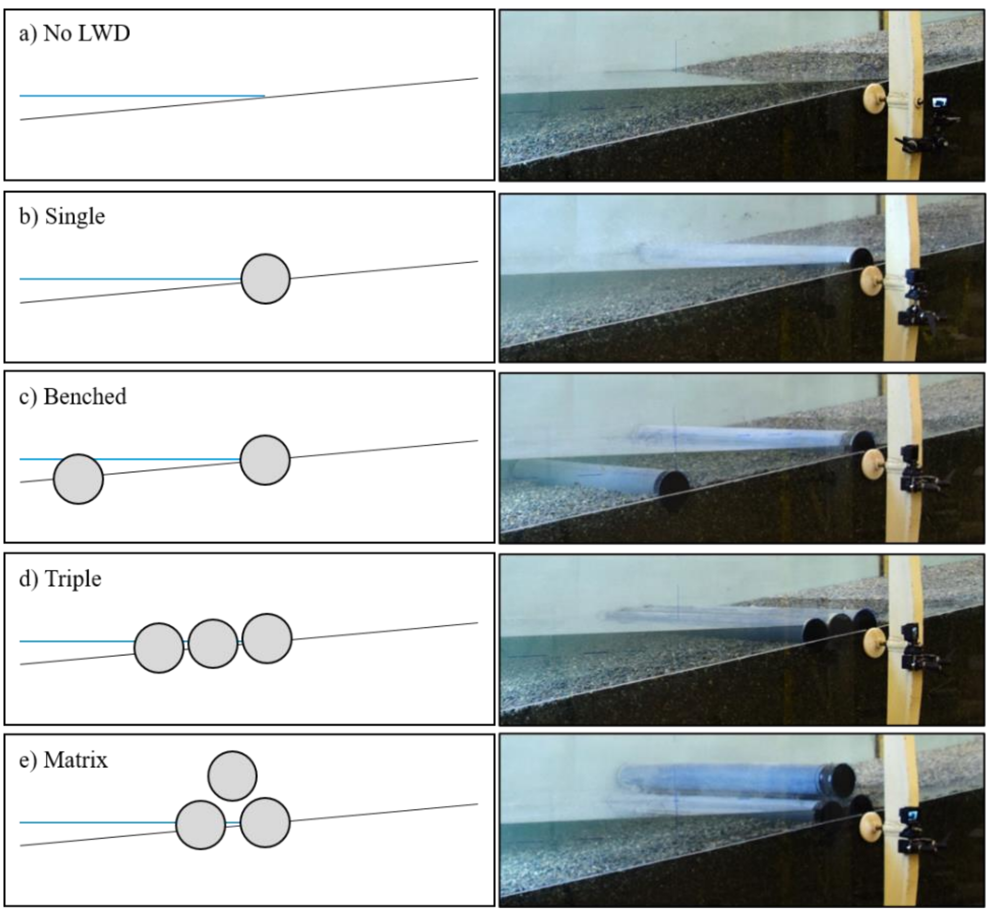

Figure 19 – Test configurations. Conceptual sketches (left) and photographs of the experimental set-up (right).

4.2.2 Instrumentation

Capacitance-wire wave gauges were used to measure the time-history of the water surface elevations (η). An array of three wave gauges (AWA 1-3) were installed directly in front of the wave generator paddle to assist with the wave reflection analysis which was used in real-time by the NRC's Active Wave Absorption (AWA) system. An array of five wave gauges (WG 1-5) were installed mid-flume for the purpose of reflection analysis post-simulation. Additionally, two wave gauges (WG 6-7) were installed at the toe of the beach to measure the incident wave conditions. Wave gauges were calibrated prior to starting the test program and the calibration was checked prior to every test. Wave gauges were re-calibrated periodically during the test program such that the maximum error was less than ±0.5% of the calibration range.

The 3D beach surface was captured before and after each run using a digital photogrammetry technique. Agisoft Metashape Professional and CloudCompare software were used to construct point-clouds and surfaces from the photographs (Agisoft, 2020; CloudCompare, 2020). Scaling and conversion to the local coordinates system was made possible by using both stationary and mobile location markers. The stationary markers were fixed to the side walls of the flume for the duration of the experiments and were surveyed using a total station to obtain their precise positions. Additional mobile markers were placed directly onto the beach surface and removed prior to filling the flume with water for the simulation or regrading for the next tests. Beach profiles were exported from the developed 3D surface and averaged to smooth local anomalies (omitting the two profiles closest to either side of the flume to avoid capturing wall effects). This process resulted in an averaged beach profile before and after each simulation.

Several beach surfaces were also captured using a FARO Focus-3D laser scanner for comparison with the photogrammetry technique. A comparison between the beach surfaces created from these two different techniques showed an average vertical difference of approximately 0.003 m and a maximum vertical difference of 0.026 m. Maximum vertical differences were located near the LWD structure, particularly where scouring had developed in front of and underneath the LWD structure. A qualitative review of the data indicated that the photogrammetry technique resulted in a more accurate representation of the beach surface in this area because photograph density was easily increased in the vicinity in order to appropriately resolve these features.

A stationary camera (CAM1 in Figure 17) captured time lapse photos of the beach profile change along the flume wall. Photographs were rectified and the beach profile was manually digitized on a 10-minute interval. The digitized beach profiles resulted in a mean vertical difference of 0.007 m and a maximum difference between 0.018-0.033 m when compared to the averaged beach profile from the photogrammetry technique for two post-simulation beach profiles. However, when compared to the profile along the sidewall using the photogrammetry technique, the digitized beach profiles resulted in a mean difference of between 0.005-0.006 m and a maximum difference between 0.018-0.023. The digitized beach profile technique therefore provides a better representation of the profile along the sidewall of the flume than the average beach profile because it incorporated wall effects. The digitized beach profile technique was therefore used as an indicator of beach profile evolution for this project, but was not relied upon for absolute values of profile changes for each test.

An additional stationary camera (CAM2 in Figure 17) was positioned to capture videos of the wave run-up and wave-structure interactions. A mobile GoPro camera was also used to capture close-up images and take slow-motion videos. Details of all instrumentation used for this set of experiments are summarized in Table 7.

Table 7 – Instrumentation types, locations, and sampling frequency

Name	Type / Model	x-location	y-location	Sampling Frequency
AWA3	Capacitance wire wave probes and Akamina AWP-24-2 wave gauges	5.317	0.629	50 Hz
AWA2		5.719	0.634	50 Hz
AWA1		5.982	0.638	50 Hz
WG1		25.929	0.626	50 Hz
WG2		26.109	0.626	50 Hz
WG3		26.329	0.626	50 Hz
WG4		26.589	0.626	50 Hz
WG5		27.179	0.626	50 Hz
WG6		36.000	0.620	50 Hz
WG7		36.989	0.620	50 Hz
CAM1	Nikon D5300	38.8	3.8	1 Frame/min
CAM2	Nikon D5300	41.8	3.4	30 Frame/s
CAM3	Nikon D5100	Moveable	Moveable	N/A
GOPro camera	HERO 6 Black	Moveable	Moveable	N/A
Laser Scanner	FARO Focus3D S 120	36.5 & 45.0	N/A	N/A

4.2.3 Experimental Program

Over 60 experimental tests were conducted by the authors as part of the "Efficacy of LWD as Coastal Protection" research project. A subset of 32 experiments are discussed within this paper.

All tests were conducted until beach equilibrium was reached (see Section 4.2.4 for additional details). A base set of three target random wave conditions (JONSWAP spectra, γ =3.3) were tested for each LWD configuration: (1) H_s = 0.10 m, T_p = 1.78 s, (2) H_s = 0.15 m, T_p = 2.17 s, (3) H_s = 0.20 m, T_p = 2.51 s. Tests were also conducted to assess repeatability, sensitivity to test duration, sensitivity to wave height, wave period, and LWD elevation relative to the water level, influence of regular waves, and influence of log roughness. A complete list of the test variants and wave conditions are included Table 8.

4.2.4 Analysis of Beach Equilibrium

All tests were conducted until it was visually observed that little to no beach profile development was occurring. After each test was complete, a more rigorous process was used to systematically check that beach equilibrium was achieved. Because previous experimental studies on gravel beach morphology did not detail systematic checks and thresholds used to determine beach equilibrium (e.g. Atladottir, 2008; de San Román-Blanco et al., 2006; Frandsen et al., 2015; Kobayashi et al., 2011; Lee et al., 2007; Masselink & Turner, 2012; Van Der Meer, 1988; van Hijum & Pilarczyk, 1982), a new method was developed for this study.

The beach profile evolution was monitored using the digitized time-lapse photos of the beach profile at 10-minutes interval. The maximum absolute 10-minute elevation change, |dh|, was determined and plotted over time for each test. Because the areas directly in front of and behind the structures were subject to rapid scouring or deposition from individual waves, these areas were not included in the analysis. A threshold was set such that once the maximum |dh| remained below the threshold for three consecutive 10-minute intervals, the beach profile was considered to have reached equilibrium, although some minor profile development continued (Figure 20). Thresholds were varied in relation to the measured wave height because the total beach profile change was found to be highly dependent on the wave height. The beach equilibrium thresholds are as follows:

- For 0.08 m $\leq$ H < 0.13 m, beach equilibrium threshold = 0.010 m
- For 0.13 m $\leq$ H < 0.18 m, beach equilibrium threshold = 0.015 m
- For 0.18 m $\leq$ H , beach equilibrium threshold = 0.020 m.

Table 8 – Experimental model variants at model scale. For irregular waves, the wave height is the spectral significant wave height, H_{m0}, and the wave period is the associated peak wave period, T_p. For regular waves, the listed wave height is mean wave height, H_m, and the wave period is the mean wave period, T_m. Actual wave height, wave period, and the number of waves are based on measurements at WG5. For comparison purposes, the base case (Test 04-v2) is highlighted in grey in the table.

Purpose	Test No.	LWD Configuration	Log Elevation [m]	Spectra Type	Target		Actual	
					Wave Height H [m]	Wave Period T [s]	Wave Height H [m]	Wave Period T [s]
Repeatability	01-v1	Single	0.0	Random	0.18	2.38	0.18	2.44
	02-v1	Single	0.0	Random	0.18	2.38	0.18	2.44
Simulation duration	04-v1	Single	0.0	Random	0.20	2.51	0.20	2.59
	04-v2	Single	0.0	Random	0.20	2.51	0.20	2.64
	20-v1	Single	0.0	Random	0.20	2.51	0.20	2.68
Wave height and period	10-v1	Single	0.0	Random	0.20	3.00	0.20	3.02
	09-v1	Single	0.0	Random	0.20	2.80	0.20	2.80
	03-v2	Single	0.0	Random	0.20	2.38	0.20	2.48
	05-v1	Single	0.0	Random	0.15	2.38	0.15	2.42
	06-v1	Single	0.0	Random	0.15	2.17	0.15	2.13
	07-v1	Single	0.0	Random	0.10	2.38	0.11	2.42
	08-v1	Single	0.0	Random	0.10	1.78	0.11	1.77
Regular waves	12-v1	Single	0.0	Regular	0.25	2.25	0.26	2.25
	13-v1	Single	0.0	Regular	0.20	2.01	0.19	2.00
	14-v1	Single	0.0	Regular	0.15	1.74	0.15	1.75
	15-v1	Single	0.0	Regular	0.10	1.42	0.10	1.42
Elevation relative to water level	19-v1	Single	+0.1	Random	0.20	2.51	0.20	2.69
	22-v1	Single	-0.1	Random	0.20	2.51	0.20	2.52
	23-v1	Single	-0.2	Random	0.20	2.51	0.20	2.61
Roughness	59_v1	Single-Rough	0.0	Random	0.20	2.51	0.20	2.69
No LWD	18-v2	None	0.0	Random	0.20	2.51	0.20	2.61
	16-v1	None	0.0	Random	0.15	2.17	0.15	2.14
	21-v1	None	0.0	Random	0.10	1.78	0.10	1.78
Benched	39-v1	Benched	0.0	Random	0.20	2.51	0.19	2.50
	40-v1	Benched	0.0	Random	0.15	2.17	0.15	2.17
	41-v1	Benched	0.0	Random	0.10	1.78	0.11	1.75
Triple	28-v3	Triple	0.0	Random	0.20	2.51	0.20	2.58
	29-v1	Triple	0.0	Random	0.15	2.17	0.15	2.10
	30-v1	Triple	0.0	Random	0.10	1.78	0.11	1.78
Matrix	46-v1	Matrix	0.0	Random	0.20	2.51	0.20	2.72
	47-v1	Matrix	0.0	Random	0.15	2.17	0.15	2.11
	48-v1	Matrix	0.0	Random	0.10	1.78	0.11	1.80

Following this analysis, two tests (test 03-v1 and 04-v1) were re-done with longer durations in order to reach beach equilibrium.

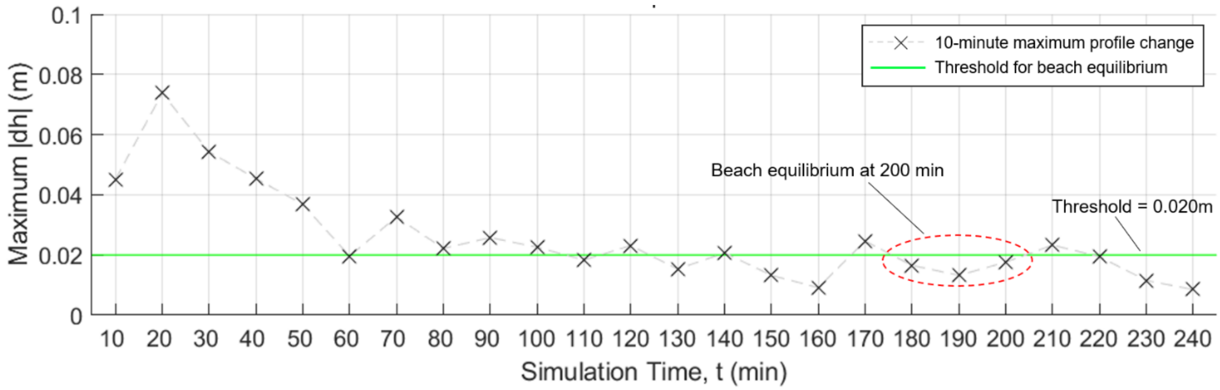

Figure 20 – Example of maximum beach profile change over 10-minute interval based on time lapse photos along flume wall for test 04-v2. Beach equilibrium was met when the maximum profile changed remained below the threshold for 30 consecutive minutes. The threshold varies between 0.010 – 0.020 m based on the incident wave height.

4.2.5 Beach Profile Characteristics

Typical gravel beach profile features include an offshore trough, step, and terrace below the still water level, a steeply sloping beach face, and an above water berm (Atladottir, 2008; Buscombe & Masselink, 2006; Kennedy & Woods, 2012; Kobayashi et al., 2011; Powell, 1990). Due to their steep slopes, gravel beaches tend to result in narrow surf zones where waves break later and closer to the shore than on sand beaches. The offshore step acts as a submerged slope break, located at the base of the swash zone (Buscombe & Masselink, 2006).

For the purpose of this study, a number of variables were defined in order to characterize and compare key beach profile characteristics (Figure 21). dh_{max} and dh_{min} are the change in elevations of maximum accretion (at the beach crest) and maximum erosion (at the beach trough), respectively, measured vertically relative to the original beach profile. X_{crest} and Z_{crest}, and X_{trough} and Z_{trough} are the horizontal and vertical locations of dh_{max} and dh_{min}, respectively. V_{crest} and V_{trough} are the volume changes (measured in m^3 per m width of flume) measured on the upper and lower slopes, respectively, relatively to the original beach profile. The upper and lower beach slope were delineated at X = 41.65m, which is approximately the center location of LWD placement. The average of these two volume measurements, V_{avg}, is used as an indicator of the magnitude of cumulative sediment transport throughout each test.

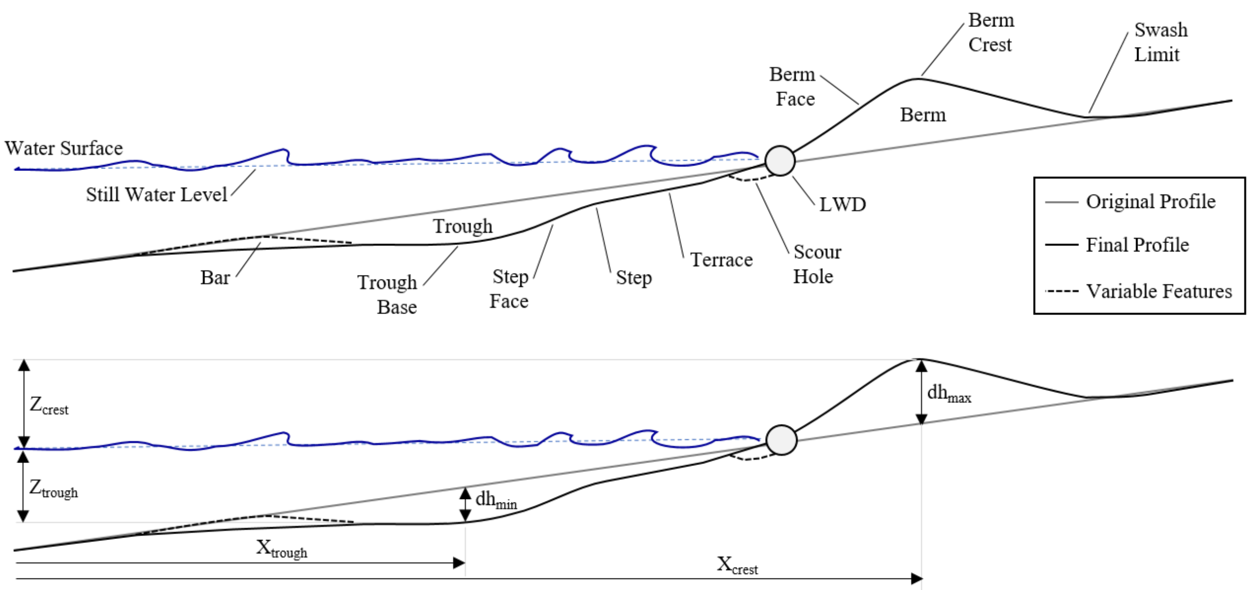

Figure 21 – Schematic of key gravel beach profile characteristics (top) and variables (bottom) (adapted from USACE, 2002)

4.3 Results

4.3.1 Repeatability

Two simulations (test 01-v1 and 02-v1) were conducted with the identical wave signal and flume set-up to test repeatability, the results of which are shown in Figure 22. The overall beach shape closely matched for both tests. The mean absolute vertical difference over the entire beach profile was 8.6 mm, with a standard deviation of 6.2 mm. Comparatively, this is the equivalent difference of one typical sediment grain given that the median grain size, D50, was 7.9 mm. The measured profiles had a maximum difference of 45.1 mm. The largest differences were located on the beach berm face where there was a steep change in elevation, and directly seaward of the LWD structure, where scouring occurred.

Key profile characteristics were also calculated and compared (Table 9).

Notably, the beach crest location was shifted inland approximately 60 mm for test 02-v1. Despite this, the maximum erosion and maximum accretion, dh_{max} and dh_{min}, were within 0.8% and 5.7%, respectively. Additionally, the volume change on the upper and lower slopes were closely matched between the two tests, within 1.3% and 0.7%, respectively.

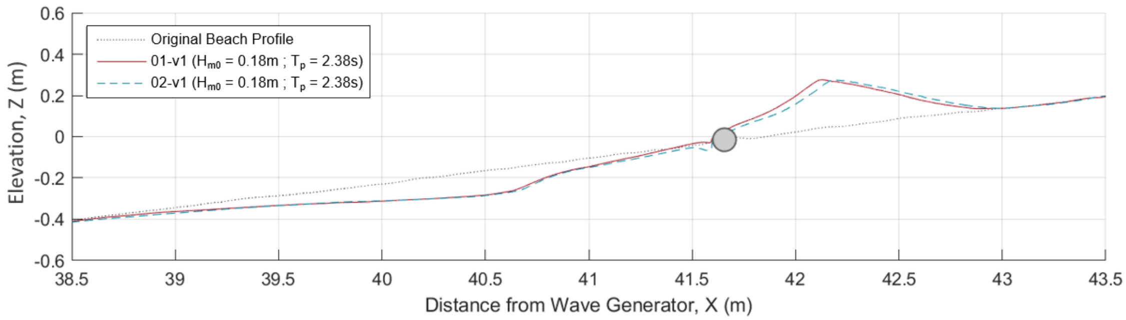

Figure 22 – Comparison of the equilibrium beach profile for repeated tests for a Single LWD structure (01-v1 and 02-v1).

Table 9 – Comparison of beach profile characteristics for repeated tests 01-v1 and 02-v1

Test No.	Upper Slope Characteristics				Lower Slope Characteristics			
	dh_{max}	Z_{crest}	X_{crest}	V_{crest}	dh_{min}	Z_{trough}	X_{trough}	V_{trough}
[-]	[m]	[m]	[m]	[m^3/m]	[m]	[m]	[m]	[m^3/m]
01-v1	0.233	0.276	42.121	0.151	-0.119	-0.281	40.518	-0.153
02-v1	0.231	0.274	42.181	0.153	-0.113	-0.268	40.629	-0.154
Absolute Difference	0.002	0.002	0.060	0.002	0.007	0.013	0.111	0.001
Percent Difference	0.8%	-	-	1.3%	5.7%	-	-	0.7%

4.3.2 Effect of Test Duration

The base case scenario (single LWD with target wave characteristics: H_s = 0.2 m and T_p = 2.51 s) was tested for three different durations to assess the influence of test duration and the number of waves on beach morphology (Figure 23): (1) 3 hours (test 04-v1), (2) 4 hours (test 04-v2), and (3) 6 hours (test 20-v1). Note that test 04-v1 did not reach beach equilibrium.

The average volume of material that moved from the lower to upper slope, V_{avg}, was 0.169, 0.197, and 0.211 m^3/m for tests 04-v1, 04-v2, and 20-v1, respectively. From comparing all three tests, it can be observed that the rate of volume change was four times higher between hours 3 - 4 (waves 5,370 – 7,230) than hours 4 – 6 (waves 7,230 – 10,820), indicating that the rate of change was highly non-linear. Similarly, the elevation increase at the berm crest, dh_{max}, was 0.262, 0.298, and 0.310 m while the elevation decrease at the beach trough, dh_{min}, was -0.154, -0.173, and -0.193 m for tests 04-v1, 04-v2, and 20-v1, respectively. The rate of change for these two parameters also decreased in non-linear fashion as test duration increased. The horizontal location of the berm crest stayed at approximately the same location; however, the trough continued to move offshore as the number of waves increased.

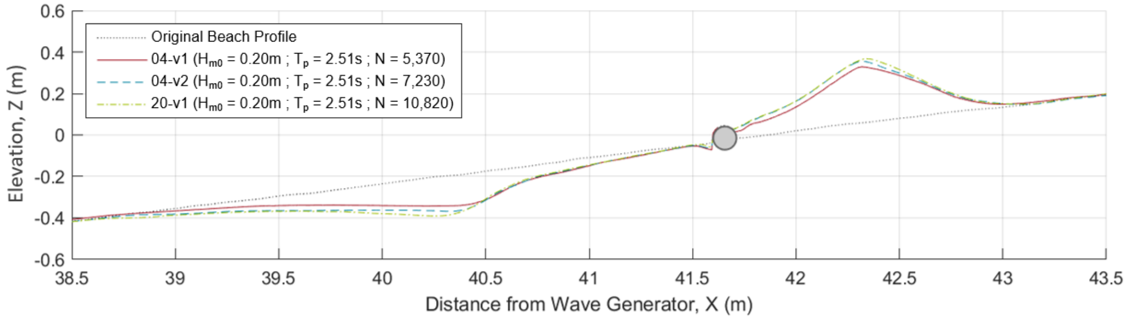

Figure 23 – Influence of simulation duration on the equilibrium beach profile. Tests with varying duration and constant incident wave conditions for a Single LWD structure (04-v1, 04-v2, 20-v1).

4.3.3 Effect of Wave Height & Wave Period

Simulations were conducted to test the influence of wave height and wave period independently on beach morphology for a Single LWD structure, the results of which are shown in Figure 24 and Figure 25, respectively. The beach crest and trough both increased in height/depth and increased in distance away from the LWD structure as a result of increased wave height. The beach terrace maintained elevation, but increased in width with an increase in wave height. Wave period was found to have less of an influence on beach profile behaviour, although an increase resulted in a marginally deeper beach trough and higher beach crest. The beach terrace maintained position and width, regardless of the incident wave period.

The average volume change, V_{avg}, for each test is also shown in Figure 26. The volume of sediment that moved upslope increased proportionally to the incident wave height. The volume change was only weakly dependent on the wave period.

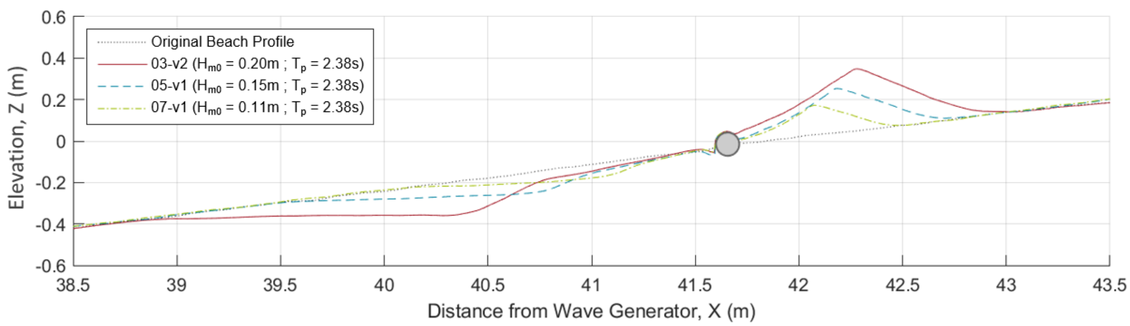

Figure 24 – Influence of wave height on the equilibrium beach profile. Tests with varying wave height and constant wave period for a Single LWD structure (03-v2, 05-v1, 20-v1).

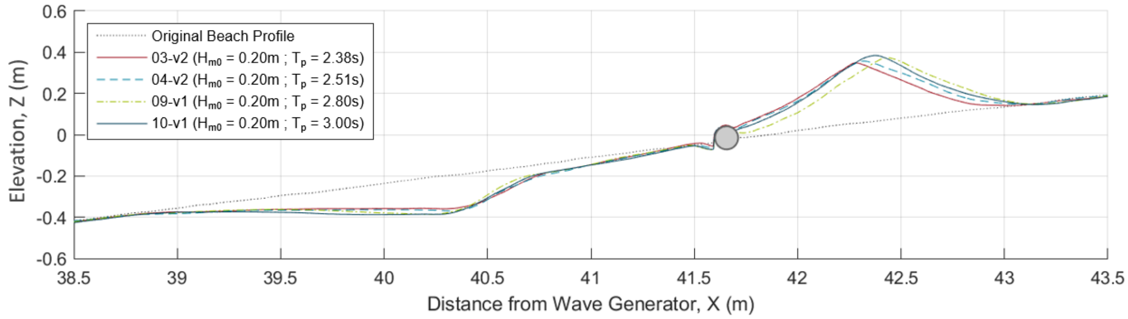

Figure 25 – Influence of wave period on the equilibrium beach profile. Tests with varying wave period and constant wave height for a Single LWD structure (03-v2, 04-v2, 09-v1, and 10-v1).

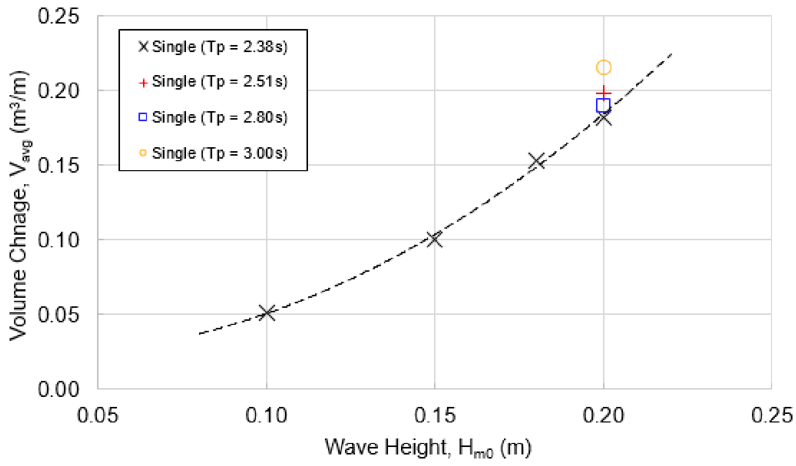

Figure 26 – Influence of wave height, H_{m0}, and wave period, T_p, on volume change, V_{avg} for a Single LWD structure. Dashed line is a fitted second-order polynomial for T_p = 2.38s.

4.3.4 Regular Waves

Simulations with regular waves were also conducted for a Single LWD structure (Figure 27). The regular wave tests produced significant 3-D effects in the flume, resulting in a convex beach crest (Figure 28). This observation may, in part, be due to resonance within the flume under regular wave conditions. Additional scouring was also observed during regular wave tests in comparison to random wave tests, in some cases resulting in complete undermining of the LWD, particularly near the flume walls.

The volume of sediment transported upslope increased non-linearly with an increase in wave height. V_{avg} was 0.005, 0.021, 0.042, and 0.121 m^3/m for wave heights, H, of 0.10, 0.15, 0.20, and 0.25 m, respectively. Similarly, the elevation increased at the berm crest, dh_{max}, was 0.041, 0.085, 0.131, and 0.220 m while the elevation decrease at the beach trough, dh_{min}, was -0.021, -0.051, -0.071, and -0.148 m. The location of the berm crest and the trough progressively moved away from the LWD structure. Notably, for the largest tested wave heights, the beach terrace experienced accretion, despite significant scouring directly seaward of the LWD.

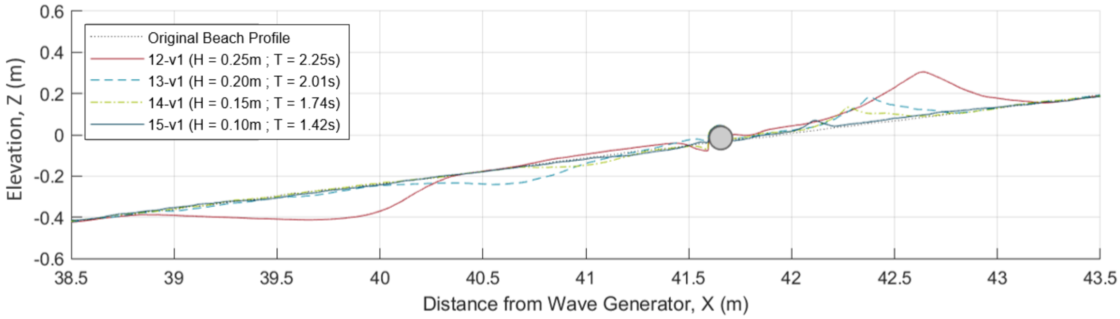

Figure 27 – Influence of regular waves on the equilibrium beach profile. Regular wave tests with varying wave heights and periods (constant wave steepness) for a Single LWD structure (12-v1, 13-v1, 14-v1, and 15-v1).

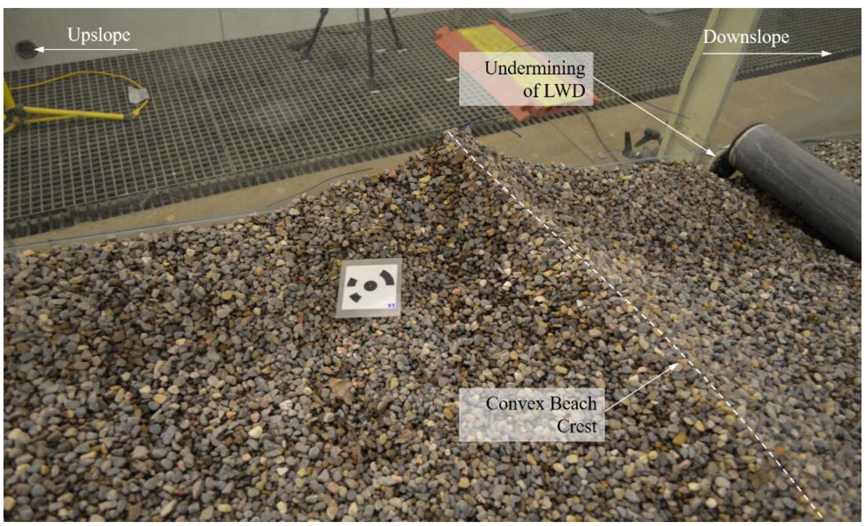

Figure 28 – Example of 3D effects observed during regular wave test 13-v1, including a convex beach crest and undermining of the LWD near the wall.

4.3.5 Influence of LWD Placement Elevation

Four LWD placement elevations were tested: +0.1 m, 0.0 m, -0.1 m, and -0.2 m relative to the still water level (Figure 29). The LWD placed at 0.0 m and +0.1 m elevation resulted in nearly identical beach profile shapes and volume changes of approximately 0.200 m^3 per m width. The LWD placed at -0.1 m resulted in decreased trough erosion and consequently in a much smaller change in volume of approximately 0.088 m^3 per m width. The resulting profile was flatter, with a small offshore trough, a wide beach step, significant scouring directly in front of the LWD, and a depressed beach crest. The LWD placed at -0.2 m was located approximately where the beach trough would normally form if no structure were present. Placing the LWD at this position resulted in reduced erosion at the beach trough; however, the placement resulted in erosion of the beach step in the lee of the LWD, producing significant sediment transport upslope and subsequent berm building. Although the total volume change was less than the two highest LWD placements, the -0.2 m placement resulted in 50% more transport than the -0.1 m placement.

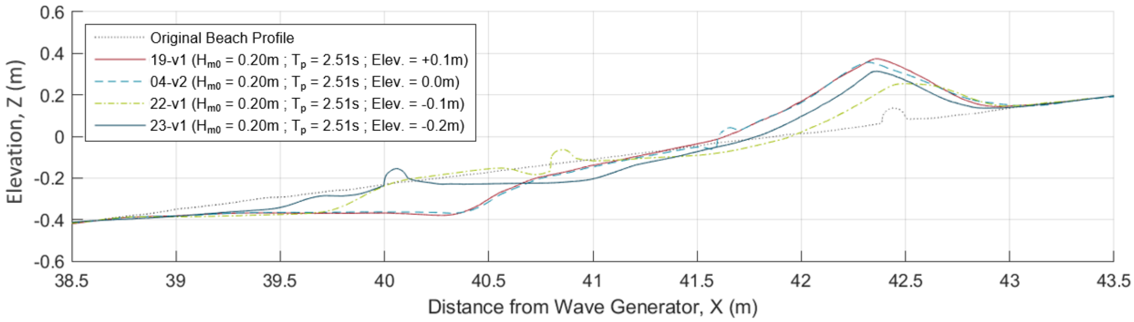

Figure 29 – Influence of LWD elevation relative to the still water level on the equilibrium beach profile for a Single LWD structure (19-v1, 04-v2, 22-v1, and 23-v1).

4.3.6 Effect of LWD Roughness

During the first phase of the research project, existing anchored LWD projects were found to include both logs with and without bark. A sensitivity test was conducted to test the effect of log roughness from bark. This was done by adding a 2 mm thick layer to the LWD model and incising 4 mm deep roughness elements into the surface of both the layer and the underlying PVC pipe. The roughness elements were designed to approximately mimic the roughness of a mature Douglas Fir while maintaining the model LWD diameter (Figure 30).

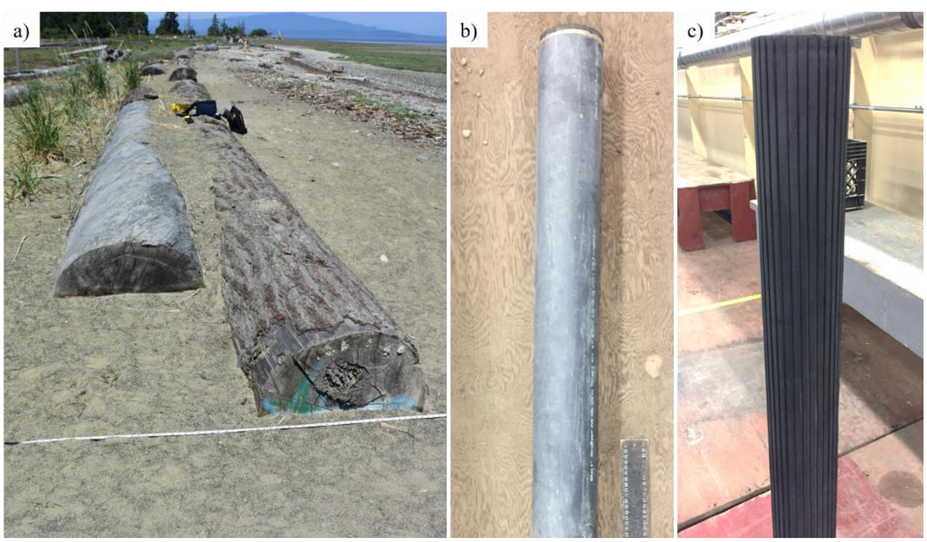

Figure 30 – (a) Photograph of anchored LWD with and without bark, (b) modeled LWD without bark, and (c) modeled LWD with bark

The beach crest and trough heights did not change significantly due to the addition of roughness elements (Figure 31). However, the volume change, V_{avg}, was reduced by 4.7% through the addition of roughness elements. Consequently, the beach crest was positioned farther onshore due to less sediment supply onto the beach face. Although some minor differences were observed, they are generally considered to be within the margin of error. Similarly, surface roughness was not found to play a role in scour formation around submerged pipelines in sandy beds (Sumer & Fredsøe, 1990).

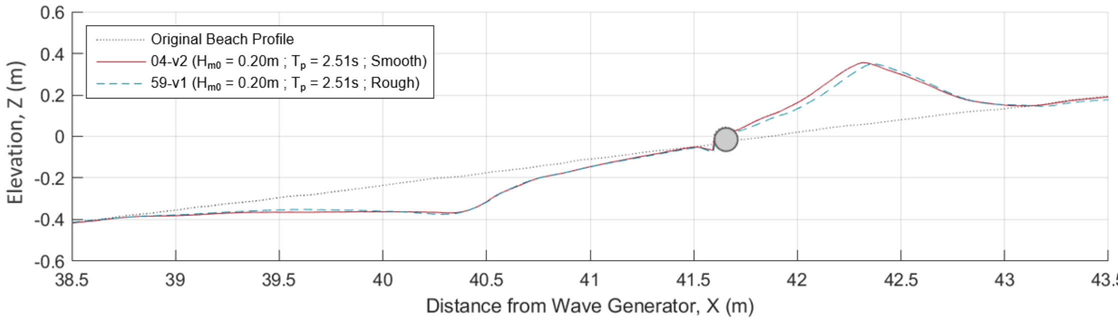

Figure 31 – Influence of LWD roughness on the equilibrium beach profile for a Single LWD structure (04-v2 and 59-v1).

4.3.7 Influence of LWD Design Configurations

Morphological experiments were conducted for four (4) LWD design configurations and a beach with no structures for the three (3) base wave conditions, in order to determine the efficacy of LWD structures at stabilizing the beach profile (Figure 32). Comparisons of the volume change, $V_{avg,}$ and the maximum elevation changes at the beach crest, dh_{max}, and trough, dh_{min}, are provided in Figure 33 and Figure 34.

The Benched configuration demonstrated the smallest elevation changes at the crest and trough for all wave heights, seconded by the Triple configuration. Notably, the beach trough was located farther offshore and the beach crest farther onshore for these two configurations, resulting in a flatter beach profile overall. The Single configuration resulted in a similar beach shape to the beach with no structures, with a deeper beach trough and a beach crest with a comparable elevation and position. Notably, the Single configuration had a larger influence on beach morphology for the smallest tested wave height in comparison to a beach with no LWD. This finding suggests that diameter relative to wave height may be an important factor in design. The Matrix configuration shifted the beach trough and crest approximately 0.2 – 0.3 m offshore in comparison to the beach with no structures.

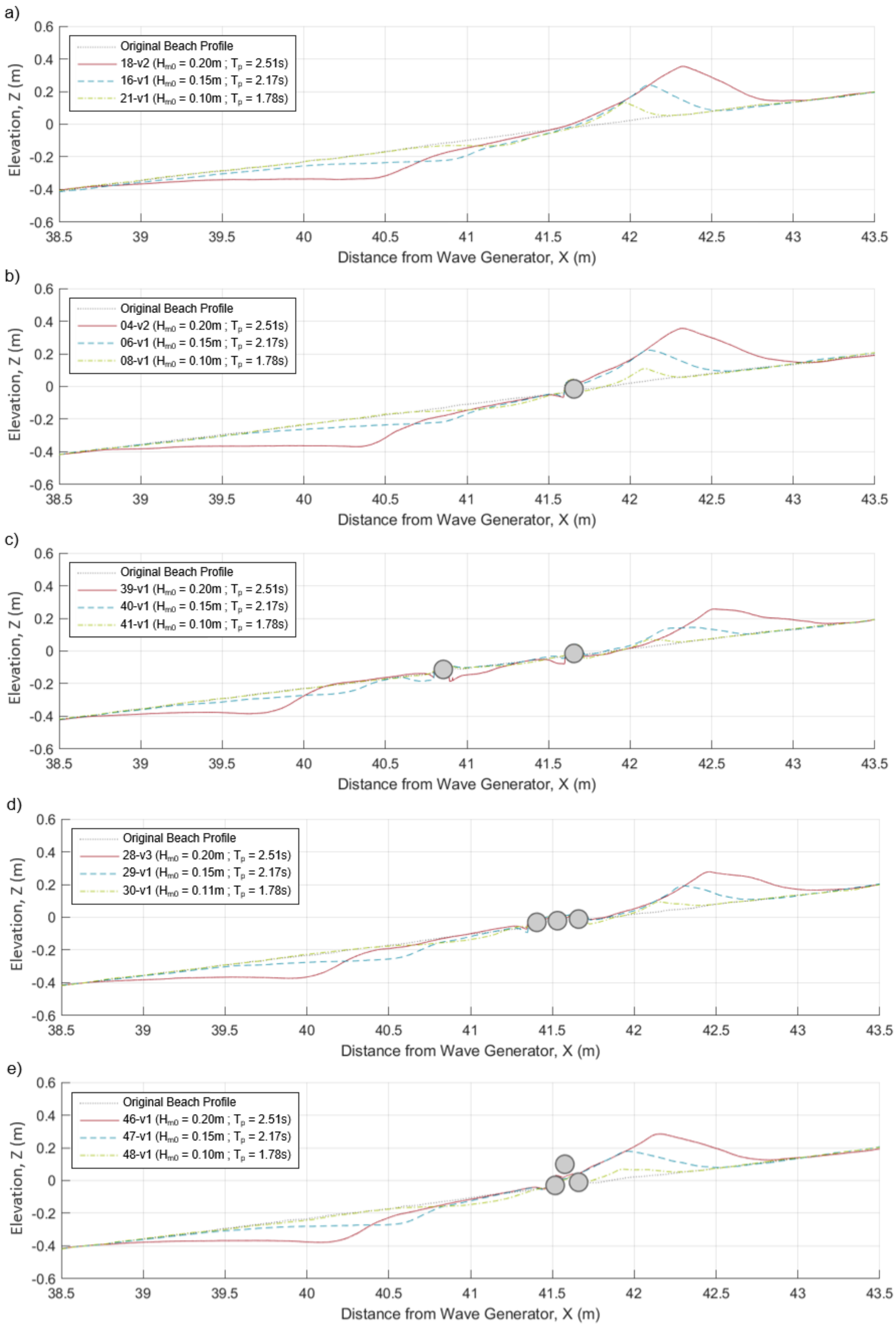

Figure 32 – Influence of the LWD design configurations on the equilibrium beach profile under the three base wave conditions, for: (a) None, (b) Single, (c) Benched, (d) Triple, and (e) Matrix-style LWD structures

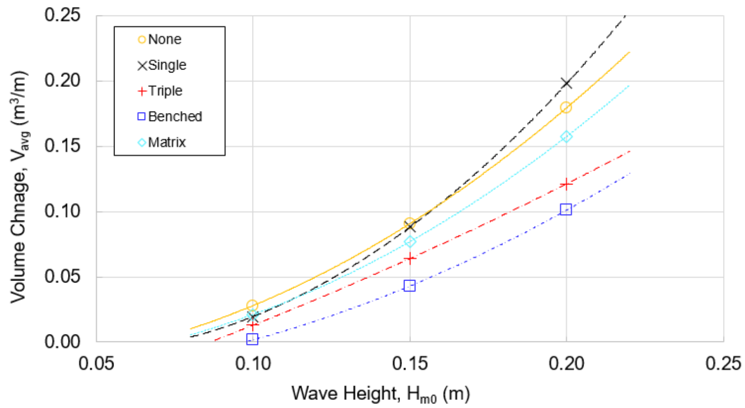

Figure 33 – Comparison of volume change, Vavg, for each LWD configuration under the base set of three wave conditions. Dashed lines are fitted second-order polynomials.

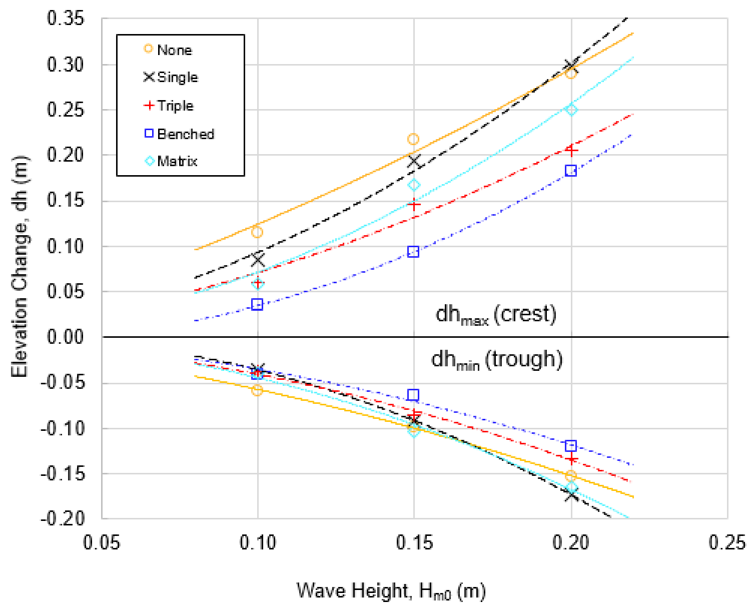

Figure 34 – Comparison of maximum and minimum elevation changes, dh_{max} (top) and dh_{min} (bottom) for each LWD configuration under the base set of three wave conditions. Trend lines are fitted second-order polynomials.

At the highest tested wave height, a Single LWD structure was found to produce 10% more volume change than a beach with no such structure. The other configurations provided some beach stabilization benefits at all tested wave conditions. The Benched configuration, however, produced almost no measurable volume change at the smallest wave height (0.002 m^3 per m), and approximately half the volume change observed for the beach with no structure at the two higher tested wave heights. Although not as effective, the Triple configuration was also effective at reducing volume change when compared to other options, particularly for the highest tested wave height where it resulted in 26% less volume change than the beach with no structures.

4.4 Discussion

4.4.1 Beach Equilibrium

Beach equilibrium can be defined as the beach profile at which the net transport capacity has become zero (Jentsje W. Van Der Meer, 1988). In this study, beach equilibrium was assessed visually during the simulations and confirmed post-simulation by analyzing time-lapse photos of the beach profile on a 10-minute interval. For irregular waves, beach equilibrium was achieved between 1500 – 6200 waves. This is consistent with most previous studies of irregular waves acting on mildly sloping gravel beaches, which generally found that beach equilibrium occurred between 3000 - 6000 waves (Atladottir, 2008; Kobayashi et al., 2011; van Hijum & Pilarczyk, 1982) or 1000 – 3000 waves (de San Román-Blanco et al., 2006; Masselink & Turner, 2012; Van Der Meer, 1988). Frandsen et al. (2015) found a much longer time for beach equilibrium (approximately 10,000 waves); however, they tested a mixed sand/gravel/cobble beach and only measured the beach profile after 2400 and 9800 waves, finding that beach equilibrium had only been met after the later of the two measurements.

For regular waves, beach equilibrium was generally achieved between 2500 – 4000 waves. These results are generally higher than previous studies, which found that beach equilibrium was achieved after an estimated 1000 – 1800 waves (Lee et al., 2007) and 2500 waves (Atladottir, 2008).

Note that it is difficult to compare the current results on beach equilibrium with previous experiments due to a lack of continuity between test conditions (e.g. wave spectra, wave height, wave period, beach slope, and sediment diameter), measurement/analysis frequency, and a general lack of discussion regarding the metrics used for defining beach equilibrium in the available literature. Notably, Atladottir (2008) used a similar experimental set-up and test program for gravel beaches with no LWD and found similar results as this study for both irregular (5000 waves) and regular (2500 wave) wave conditions.

4.4.2 General Observations

The repeated tests resulted in a close match in the overall beach profile shape and the volume of material that moved upslope. The greatest differences in the beach profile were noted on the beach berm face and directly seaward of the LWD structure, where scouring occurred. Scouring was observed to be highly dependent on the interaction of individual incoming waves and the run-down from the preceding wave (Figure 35). The resulting shape and depth of the scour hole therefore varied rapidly throughout the duration of tests. Despite this, there was only a 1% difference in the average volume change, V_{avg}, between the repeated tests. It can be concluded that the morphological response of the beach profile was both comparable and repeatable.

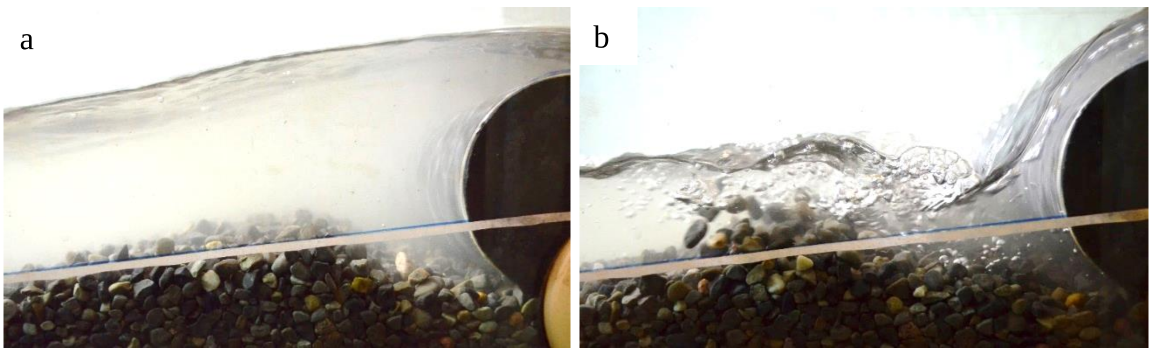

Figure 35 – Close-up image of scour formation seaward of a LWD structure, during (a) wave run-up and (b) run-down

Typical gravel beach profile features (offshore trough and step, a steeply sloping beach face, and berm) were all observed during all the simulations, regardless of the presence of LWD structures. The elevation of the LWD structure and the LWD configuration type had a significant impact on the extent and location of the beach trough, the width of the beach step, and the elevation of the beach crest above water.

The wave height was found to significantly influence the beach profile development for the case with a Single LWD at the still water level for an 8:1 (H:V) initial beach slope. For this case, the maximum and minimum elevation changes, dh_{max} and dh_{min}, were found to follow the two linear relationships below, with R^2 values of 99.8 and 99.4%, respectively.

$$dh_{max} = 1.4424 \times H_{m0} \tag{28}$$

$$dh_{min} = -0.7528 \times H_{m0} \tag{29}$$

Previous research on gravel beach morphology also indicated that wave height was a governing factor in beach profile development (Atladottir, 2008; Powell, 1990; Jentsje W. Van Der Meer, 1988). Based on this previous experimental work on gravel beaches with no structures (e.g. Atladottir, 2008; Powell, 1990; Van Der Meer, 1988), wave period was also expected to be an important factor in beach development with LWD. However, a strong correlation was not found between wave period and the maximum or minimum elevation changes, nor the volume change. The lack of correlation may be because (1) the wave height (H_{m0} = 0.2 m) was relatively large and governed profile development, (2) the presence of LWD resulted in a beach response that was more resilient to the wave period, or (3) only a small range of wave periods were tested in comparison to previous studies.

Although not representative of field conditions, regular waves were tested as part of this study for the purpose of comparing results with previous studies that included regular wave tests (e.g. Atladottir, 2008; van Hijum and Pilarczyk, 1982). Regular waves resulted in a wider and flatter beach profile than irregular waves of a similar height and period. More scouring and, in some cases, full undermining of the Single LWD was observed due to a concentration of wave energy in one location. Notably, significant 3-D effects were noted during regular wave tests, particularly near the LWD structure and on the above-water portion of the beach face and crest. This was in contrast to Atladottir (2008), who did not observe 3D effects during experiments on gravel beaches with no structures for the same flume and similar wave conditions. The introduction of a structure into the flume may have led to increased wall-related effects, compared to a gravel beach alone. However, three dimensionality due to wall effects was not observed to be significant for irregular wave tests.

In a recent study on LWD transport in rivers, Perry et al. (2018) tested the behaviour of rough, natural log models in comparison to smooth models. They found that fabricated dowels exhibited different behaviour than natural wood (branches), suggesting that roughness played an important role in LWD transport and dam formation. As such, roughness elements were modeled to mimic and test the role of tree bark on beach morphology. LWD roughness was not found to be a major variable in beach profile development, although it may moderately reduce sediment transport upslope. Additional research is required to confirm this observation due to the limited number of tests conducted.

4.4.3 Design Considerations

Gravel beaches tend to result in narrow surf zones where waves break later and closer to the shore than on sand beaches. The offshore step acts as a submerged slope break, located at the base of the swash zone (Buscombe & Masselink, 2006). At the step, wave bores develop, shoal, and collapse over the relatively shallow and flat beach section. The step allows waves to shoal in deep water, concentrating wave energy on the inshore edge of the step and creating the conditions to maintain a relatively steep and reflective step face. In part because of the energy concentration, a significant amount of sediment transport and beach profile changes occur near the step (e.g. Atladottir, 2008; de San Román-Blanco et al., 2006; Kobayashi et al., 2011; Powell, 1990), and it is therefore understood to force and modulate wave-breaking. The experimental modeling results indicate that the position of LWD is a major variable which controls the volume change and the beach profile shape. The most significant stabilization benefits are observed when LWD is anchored near the seaward edge of this step.

Theoretically there is a lower placement elevation at which the LWD will not influence wave interactions with the beach profile. This lower limit conceivably lies somewhere between the -0.2 m LWD placement and the maximum trough depth of -0.4 m. Insufficient data is available to develop an empirical equation characterizing the relationship between volume change and LWD placement elevation; however, a theoretical relationship between LWD elevation and volume change was hand-fitted based on the available data and the authors' understanding of the gravel beach profile development (Figure 36).

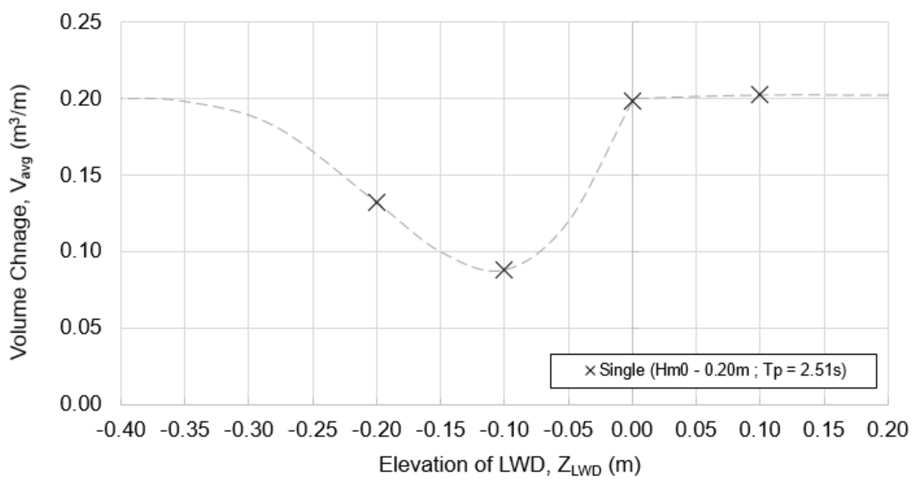

Figure 36 – Influence of LWD elevation on volume change, Vavg, for the base case scenario (single LWD with target wave characteristics: Hs = 0.2 m and Tp = 2.51 s). The gray dashed line is a hand-fitted theoretical relationship between LWD elevation and volume change.

Notably, LWD configurations placed at or above the still water level did not provide significant shore stabilization benefits. For the highest wave heights, there was a potential for LWD (particularly for the Single LWD configuration at the still water level) to effect larger volume changes than a beach with no structure. Configurations which extended seaward or below the still water level, such as the Triple or Benched configurations, provided the largest benefits in terms of beach profile stabilization. Notably, Single LWD placed at -0.1 m resulted in approximately the same volume changes as the Benched structure for a wave height, H_{m0}, of 0.2 m, indicating that upper log in the Benched configuration provided little stabilization benefits.

It should be noted that placement of LWD structures near the beach step to maximize shore stabilization benefits will also locate them in regions of high wave energy, active erosion, and below the still water level. In practice, this creates a design problem of how to anchor a buoyant and mobile material on a dynamically changing slope and, if it becomes mobile, will it cause more damage than benefits.

4.4.4 Study Limitations

The experimental program focused on assessing the morphological response of a gravel beach due to various shore-parallel LWD structures in comparison to a beach with no structures. Limitations of the study include the following:

- Only one sediment type and gradation were tested, resulting in a beach that was both porous (porosity, n = 40.5%) and homogeneous; however, many of the beaches where these structures are frequently constructed in British Columbia, Canada, and Washington State, USA, consist of mixed-sediment beaches. A low beach groundwater table and porous material is generally considered to enhance sediment accretion on the shoreline, while a high groundwater table or impervious material is understood to inhibit accretion and promote erosion (Masselink & Turner, 2012). As such, introducing a semi-impermeable layer and changing the beach porosity or adjusting the groundwater table may significantly modify the beach profile development (Bagnold, 1940).
- On porous gravel beaches, larger sediments tend to move onshore and up-slope, while fine material tends to move down-slope (Buscombe & Masselink, 2006; Masselink & Turner, 2012). This phenomenon occurs because infiltration lowers velocities near the limit of wave run-up (Buscombe & Masselink, 2006; D. Horn et al., 2006). Coarser material

requiring higher flow velocities for transport become stranded at the top of the beach face, whilst the wave run-down still has sufficient velocities to wash away finer materials. Although steps were taken to remix sediments during the testing program, these physical processes led to some sediment sorting across the beach profile.

- Model logs were fixed in place on the beach by applying lateral pressure to the side walls of the flume, resulting in a static structure throughout the test duration. Although this behaviour is representative of anchored LWD at the time of initial installation, anchoring systems do generally allow for some dynamic movement of the LWD if sufficient undermining/scouring occurred.
- Due to the 2-Dimensional nature of the wave flume, all waves were normally incident to the beach and all tested configurations were shore parallel. These tests therefore do not consider longshore transport, which is understood to play a significant role in erosion potential at many of the study sites (Finlayson, 2006).
- The models were completed at a 5:1 scale, according to Froude scaling. Generally, Froude scaling is suitable for free-surface flows and large scale and/or turbulent models (Sutherland & Soulsby, 2010). Errors from Froude scaling occur when the fluid is not turbulent and viscous effects are non-negligible (e.g., distortion of the Reynolds number) (Heller, 2011). No full-scale tests were completed to quantitively assess scale-effects; however, for this test program, the effect of viscous forces was expected to be minimal since turbulent conditions were maintained near the beach and the tests were conducted at a relatively large scale.

4.5 Conclusions

A comprehensive set of over 60 experimental tests were completed as part of the "Efficacy of Large Woody Debris as Coastal Protection" research program. This work represents the first experimental study on Large Woody Debris under wave action. A subset of 32 tests were explored in this paper related to the morphological response of a gravel beach with and without various LWD configures.

From this study, it can be concluded that anchored LWD structures placed below the still water level, near or on the beach step, resulted in the largest shore stabilization benefits. Of the four LWD configurations tested, the Benched configuration resulted in the most stable beach profile, seconded by the Triple configuration. A Single LWD configuration provided similar beach stabilization benefits as the Benched configuration when placed below the still water level, but may be detrimental when placed on the upper beach for large wave heights. The Matrix configuration provided only a small reduction in sediment transport. Given the study limitations outlined in Section 4.4.4, caution should also be taken in applying the results of this study outside of the tested conditions.

Despite potential benefits to shore stabilization of coastal protection using Large Woody Debris, a significant barrier to garnering this potential is how to effectively anchor LWD. As such, to expand this work for engineering application and for the development of design guidance, additional research is required in the following areas:

- Additional experimental modeling to include variations in the initial beach slope, water level, log diameter, and sediment characteristics, including porosity, permeability, and gradation.
- Expand the existing experimental program to include a wide range of wave characteristics and allow for 3-D testing in a wave-basin for longshore sediment transport.
- Study the behaviour of dynamic (as opposed to fixed-in-place) anchored LWD structures, which mimic the anchoring techniques observed during field investigations of existing LWD structures.
- Test the influence of LWD structures on wave run-up and overtopping.
- Compare the impact of anchored LWD structures in comparison to more traditional coastal protection structures, such as seawalls and revetments.

Chapter 5. Experimental Modeling of Coastal Protection using Large Woody Debris: Wave Run-Up

5.1 Introduction

Nature-based solutions (NBS) aim to solve societal challenges by mimicking materials and processes found in the local natural environment (Eggermont et al., 2015; Pontee et al., 2016). In the face of global sea level rise (Chen et al., 2017; Hansen et al., 2015; IPCC, 2014) and anthropogenic effects along the coastal margin, there is a growing demand for NBS that serve as coastal flood protection (Arkema et al., 2013; Pontee et al., 2016; Saleh & Weinstein, 2016). Coastal flooding is driven by the combined effects of sea level rise, tides, storm surge, wind and wave set-up, and wave run-up or overtopping. Coastal flood protection generally aims to prevent flooding through raising the land elevation and/or dissipating wave energy to reduce the inland penetration of waves (Bruun, 1972; EurOtop, 2018; Stark et al., 2016). The extensive diking systems that protect much of the Netherlands or New Orleans have historically focused on the former technique, although there have been recent initiatives to set-back dikes to capitalize on the ecological services provided by marsh habitats (Arkema et al., 2013; Bridges et al., 2015; Bruun, 1972; Saleh & Weinstein, 2016). Conversely, coastal flood protection for many smaller projects, or for areas where the land is not particularly low-lying, mainly rely on dissipating or reflecting wave energy at the shoreline to reduce wave run-up and wave-induced flooding.

Numerous formulas have been developed to characterize and estimate wave run-up. Amongst the first attempts, Hunt (1959) developed a simple formula for predicting wave-up under regular breaking waves on a smooth, impermeable slope, which related the non-dimensionalized run-up, R/H, to the Surf Similarity Parameter, ξ (Battjes, 1974):

$$R/H = \xi \qquad \text{for } 0.1 < \xi < 2.3 \tag{30}$$

Later research found the Surf Similarity Parameter alone was insufficient to describe wave run-up for non-breaking waves, because it did not include consideration for reflection, turbulence, and water depth (Arana, 2017; Poate et al., 2016). Dynamic movement of sediments and slope permeability are also understood to result in different run-up patterns than static, impermeable slopes (Buscombe & Masselink, 2006; Powell, 1990). Gravel beaches, in particular, effectively dissipate wave energy through the mechanisms of surface roughness, infiltration, and beach reconfiguration (Arana, 2017; Bagnold, 1940; Buscombe & Masselink, 2006). Newer formulas have focused on incorporating variables and methods to account for these factors (e.g. Arana, 2017; Van der Meer et al., 2016). Advances in numerical modeling have also allowed for improved insight and prediction of wave run-up on gravel and mixed beaches (e.g. Lee et al., 2007; Poate et al., 2016), but understanding is still considered to be poor in comparison to the understanding for sandy beaches (J.W. Van der Meer et al., 2016).

In a study on the effects of Large Woody Debris (LWD) on gravel beaches, Kennedy & Woods (2012) concluded that natural deposits of LWD may serve to build up the storm berm on gravel beaches, further inhibiting wave run-up and coastal flooding. Additional studies have found that natural LWD serves as roughness elements on beaches, promoting accretion of aeolian sand (Eamer & Walker, 2010; Grilliot et al., 2019). Studies such as these have been cited as a basis for using LWD as coastal protection in order to stabilize the shoreline and reduce wave run-up (e.g. Johannessen et al., 2014; Washington Department of Fish and Wildlife, 2016). In addition, because there is a lack of evidence-based design guidance for coastal protection using LWD, design practitioners have relied on anecdotal observations and experience, as well as a continuity of design practices from the river engineering field, where significant literature is available on the function and design of LWD (e.g. Bocchiola, 2011; Gonor et al., 1988; Hilderbrand et al., 1998; Hills and Street, 2007; Kail et al., 2007; Rafferty, 2017; Sass, 2009; Shields et al., 2004; USBR and ERDC, 2016). However, LWD in rivers are not generally exposed or designed for important coastal processes, such as wave action. In order to reliably implement coastal protection using LWD, research is needed to understand the geomorphological and hydraulic behaviour of anchored LWD under wave-action. To date, no such research is known to exist.

The “Efficacy of Large Woody Debris as Coastal Protection” research project was initiated to better understand the efficacy of coastal protection using anchored LWD. The first phase of the project involved extensive field investigations of existing sites with anchored LWD installations throughout coastal British Columbia (BC), Canada, and Washington State, USA (Chapter 3). This work provided insight into current design practices and typical site characteristics, as well as common durability issues. The second phase of the project included a comprehensive experimental modeling program investigating the influence of anchored LWD on beach morphology and wave run-up. Chapter 4 outlined the experimental study results related to beach morphology, finding that LWD elevation, rather than configuration, was a governing factor in beach stabilization. This Chapter presents the experimental study results related to the effect of LWD on wave run-up.

5.2 Methodology

5.2.1 Prototype Overview

The experimental program was developed to mimic typical site and LWD design characteristics of existing anchored LWD installations in coastal British Columbia, Canada, and Washington State, USA (Chapter 3). The deep-water fetch-limited wave height at each of the investigated sites, H_{m0}, was estimated to range from 0.3 – 3.9 m. The upper beach (beach face) was found to have a slope between 4.4:1 (H:V) and 10.2:1 with an average slope of 7.6:1, where the site with a 4.4:1 slope included an eroding scarp. The lower beach slope varied between 8:1 and 500:1 (effectively horizontal). The beach substrate ranged from coarse sand to boulders, generally with a mean sediment size, D_{50}, between 18 to 140 mm. Installed LWD had an average diameter of 0.56 m.

Six installation techniques from the field investigations were characterized (see Chapter 3), including: (1) Single, (2) Multiple, (3) Benched, (4) Stacked, (5) Matrix, or (6) Groyne-style. An example of a typical site with a mixed gravel and cobble upper beach and a Single LWD structure is shown in Figure 37. The study was not able to distinguish the influence of the various LWD configurations on wave run-up and wave-induced flooding.

Figure 37 – Typical Single LWD structure anchored on the upper slope of a mixed gravel/cobble beach in British Columbia, Canada.

5.2.2 Experimental Set-Up

The experimental model tests were undertaken in a 63.0 m long, 1.22 m wide, and 1.22 m deep wave flume at the National Research Council's Ocean, Coastal and River Engineering Research Centre (NRC-OCRE) in Ottawa, Ontario, Canada, as part of a joint research project between researchers are the University of Ottawa and NRC-OCRE. A piston-type wave generator was installed at the far end of the flume with an active wave absorption system. The flume included an approximately 3.6 m long section of glass walls on either side, allowing for side-viewing and recording of processes within the flume.

A model scale of approximately 5:1 (prototype to model) was used, based on Froude scaling. The resulting experimental set-up included a gravel beach with a uniform slope of 8:1 (H:V), installed such that the entirety of the beach was visible through the glass walls of the flume. A water depth of 0.6 m was maintained during all experiments and this still water level was set as the elevation datum (z-direction) measurements. The beach toe was positioned at x = 37.0 m (measured from the face of wave paddle) and the crest extended 1.0 m above the flume bed (z = +0.4 m and x = 45.0 m), as shown in Figure 38. The model included a rounded gravel substrate with a median grain size, D_{50}, of 7.9 mm, a Gradation Coefficient, C_z, of 0.96, and a Porosity, n, of 40.5%.

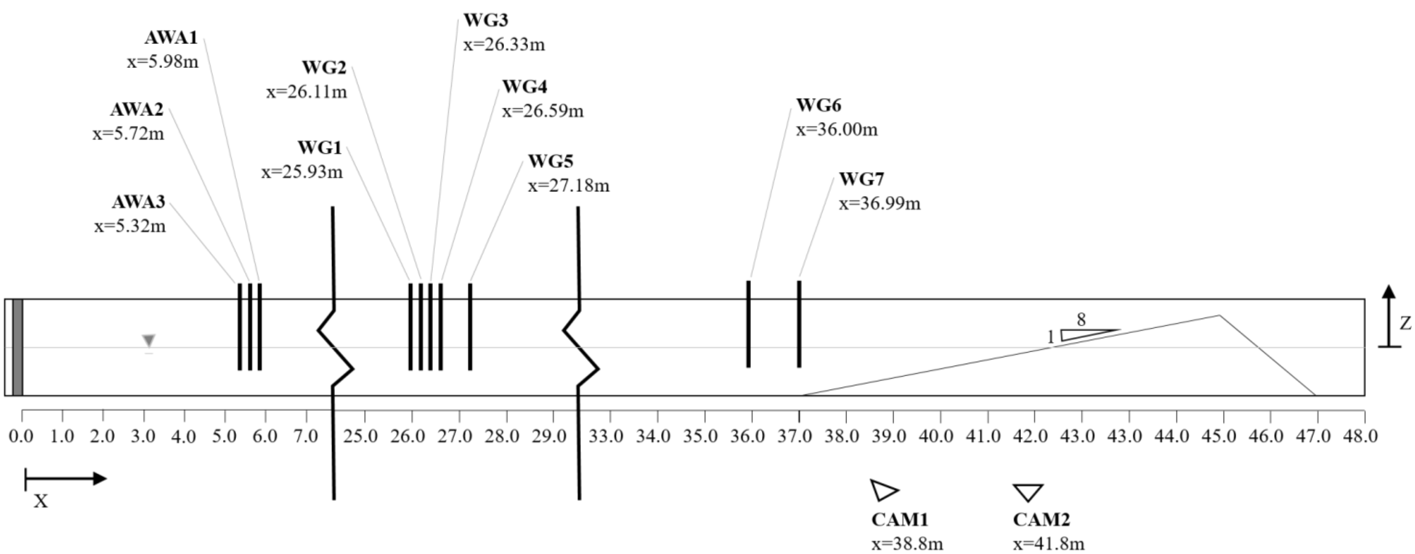

Figure 38 –Schematic of experimental set-up. The x-direction increases away from the wave paddle towards the beach, the y-direction increases into the page, and the z-direction increases above the still water level (SWL).

LWD structures were constructed using PVC pipes with a 0.114 m diameter. The LWD models were each manufactured with one end that could be extended in order to apply lateral pressure to the flume walls, fixing the LWD in-place to mimic a rigid anchoring system. The model logs were placed in various arrangements to achieve four different LWD shore-parallel configurations: Single, Benched, Triple, and Matrix. A beach with no LWD structures and a single LWD structure with roughness elements were also tested (Figure 39). The roughness elements were included to test the influence of tree bark on wave run-up. This was achieved through gluing a 2 mm thick layer of plastic material to the LWD and incising 4 mm deep grooves in the model in order to maintain the LWD diameter.

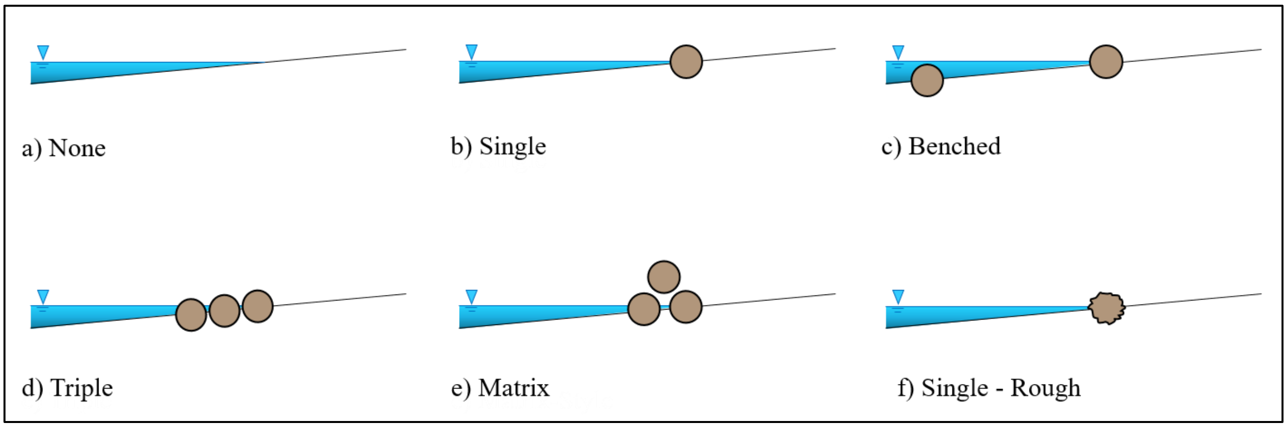

Figure 39 –Sketches of beach set-up variants, including: (a) no LWD structures, (b) single LWD structure, (c) benched LWD structure, (d) triple LWD structure, (e) matrix-style LWD structure, and (f) single LWD structure with roughness elements.

5.2.1 Instrumentation

Ten (10) capacitance-wire wave gauges were used to measure the water surface elevation (Figure 38). AWA 1-3 were used for real-time analysis for active wave absorption, WG 1-5 were used to support wave reflection analysis, and WG 6-7 provided information on wave transformation and characteristics near the beach toe. Wave gauge calibration was checked prior to every test and recalibration was undertaken if the maximum error exceeded ±0.5% of the calibration range.

A stationary camera (CAM1 in Figure 38) was positioned to capture time lapse photos of the beach profile change along the flume wall on a 1-minute interval. This allowed for quantification of the rate and magnitude of beach profile changes during the simulations. An additional stationary camera (CAM2 in Figure 38) was positioned to capture videos of the wave run-up and wave-structure interactions. The wave run-up analysis procedure is described in Section 5.2.3. A mobile GoPro was also used to capture close-up images and take slow-motion video. Additional details on the flume set-up and instrumentation are provided in Chapter 4.

5.2.2 Experimental Program

The run-up height exceeded by 2% of the incident waves, $R_{2\%}$, is frequently used as an indicator of wave run-up severity and for the purpose of engineering design (EurOtop, 2018). In order to estimate $R_{2\%}$, irregular waves must be generated with a statistically meaningful sample of waves. The beach profile must also be approximately static and extend past the maximum limit of wave run-up. Because these conditions cannot generally be met for moveable bed tests, $R_{2\%}$ is difficult to estimate reliably. Instead, regular waves were generated to investigate the wave run-up whilst the beach profile remained relatively constant at an 8:1 (H:V) slope over a set of 20 waves. A set of four base wave characteristics were tested for six different beach and LWD configurations, as outlined in Table 10. Repetition tests were completed for every test number.

Table 10 – Experimental model variants in model scale, including the target wave characteristics.

Test No.	LWD Configuration	Wave Height H [m]	Wave Period T [s]
24	None	0.10	1.42
25	None	0.15	1.74
26	None	0.20	2.01
27	None	0.25	2.25
35	Single	0.10	1.42
36	Single	0.15	1.74
37	Single	0.20	2.01
38	Single	0.25	2.25
42	Benched	0.10	1.42
43	Benched	0.15	1.74
44	Benched	0.20	2.01
45	Benched	0.25	2.25
31	Triple	0.10	1.42
32	Triple	0.15	1.74
33	Triple	0.20	2.01
34	Triple	0.25	2.25
49	Matrix	0.10	1.42
50	Matrix	0.15	1.74
51	Matrix	0.20	2.01
52	Matrix	0.25	2.25

Test No.	LWD Configuration	Wave Height H [m]	Wave Period T [s]
60	Single - Rough	0.10	1.42
61	Single - Rough	0.15	1.74
62	Single - Rough	0.20	2.01
63	Single - Rough	0.25	2.25

5.2.3 Analysis of Wave Run-Up

A video camera (CAM 2 in Figure 38) was positioned outside of the flume for the duration of the experiments to monitor wave run-up. Video frames were exported and corrected for camera distortion in MATLAB using a checkerboard-based correction (Figure 40) (Mathworks, 2020). The maximum wave run-up was manually selected in MATLAB for each of the undistorted images. The pixel location of the wave run-up was converted to X- and Z-coordinates based on the known positions of four scale markers. The scale markers were placed on the outside of the flume within the outer limits of the camera view and their position was surveyed prior to starting the tests with a Total Station (for additional details, see Chapter 4). Using this procedure, the maximum limit of wave run-up was determined over a sample of 20 waves near the beginning of each test, when the beach profile was still approximately flat. Each simulation was repeated once, for a total sample of 40 waves for each test. The standard deviation for each 40-wave test set was between 0.003 – 0.009 m.

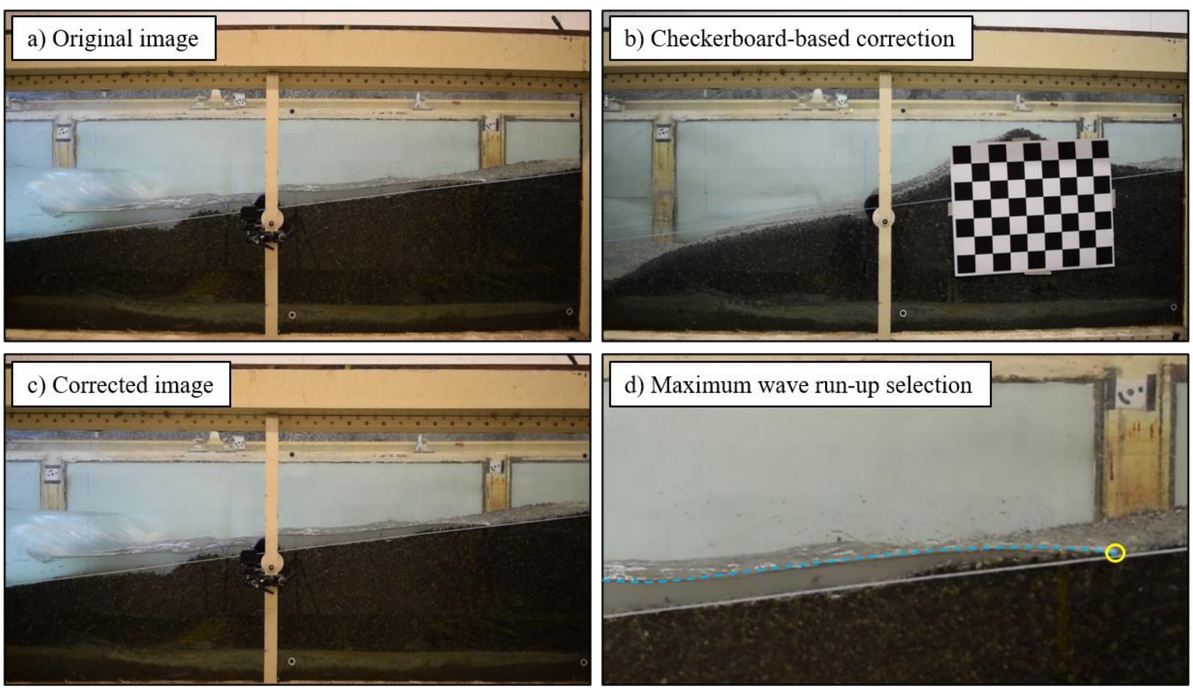

Figure 40 – Example of image processing method for Test No. 37 (Duplicate Set 1).

5.3 Results

5.3.1 Effect of Configuration on Wave Run-Up

The measured wave run-up, R, is plotted in Figure 41 against the target wave height, H, for LWD configurations: (a) None, (b) Single, (c) Benched, (d) Triple, and (e) Matrix. Apart from the smallest tested wave height, the LWD configurations result in higher wave run-up than a beach with no LWD structures. For the two largest wave heights, the Benched and Triple configurations resulted in approximately 10 – 25% more wave run-up than a gravel beach with no structures. The Single configuration resulted in approximately 10% higher run-up. Although it experienced generally higher run-up than a plain gravel beach, the Matrix configuration performance was within the margin of error.

In addition, almost all empirical formulas for estimating wave run-up relate the non-dimensionalized wave run-up, R/H, to the Surf Similarity Parameter or Iribarren Number, *ξ:*

$$\boldsymbol{R}/\boldsymbol{H} = \boldsymbol{f}(\boldsymbol{\xi}) \qquad (31)$$

The Surf Similarity Parameter can be calculated as the ratio of beach slope to wave steepness, as follows (Battjes, 1974):

$$\boldsymbol{\xi} = \frac{\boldsymbol{tan\,\alpha}}{\sqrt{\boldsymbol{H}/\boldsymbol{L}}} \qquad (32)$$

Where α is the slope angle from horizontal, H is the representative wave height, and L is the wavelength. For reference, the non-dimensionalized wave run-up, R/H, is also plotted against the Surf Similarity Parameter in Figure 42.

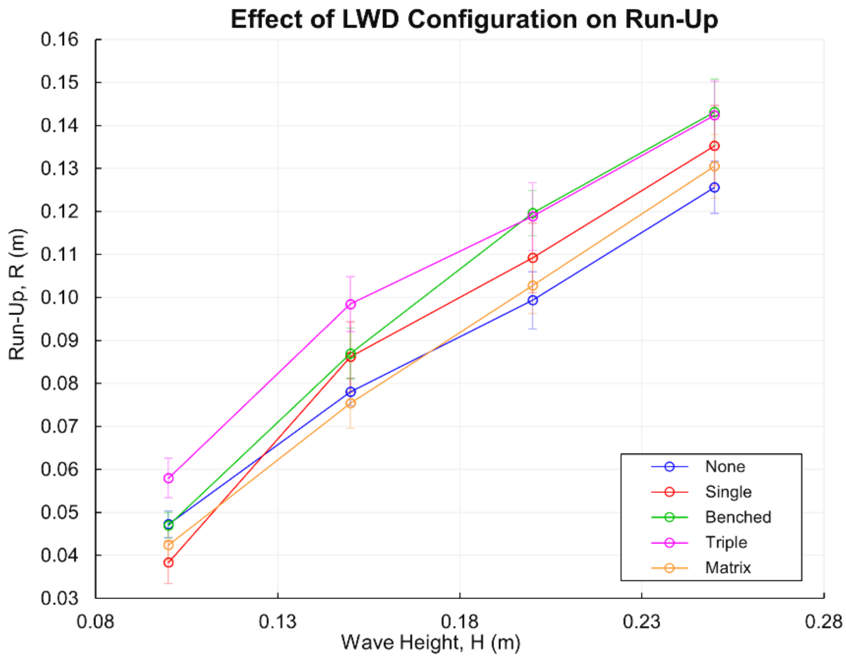

Figure 41 – Measured wave run-up versus wave height for configurations: none, single, benched, triple, and matrix. Error bars indicate one standard deviation.

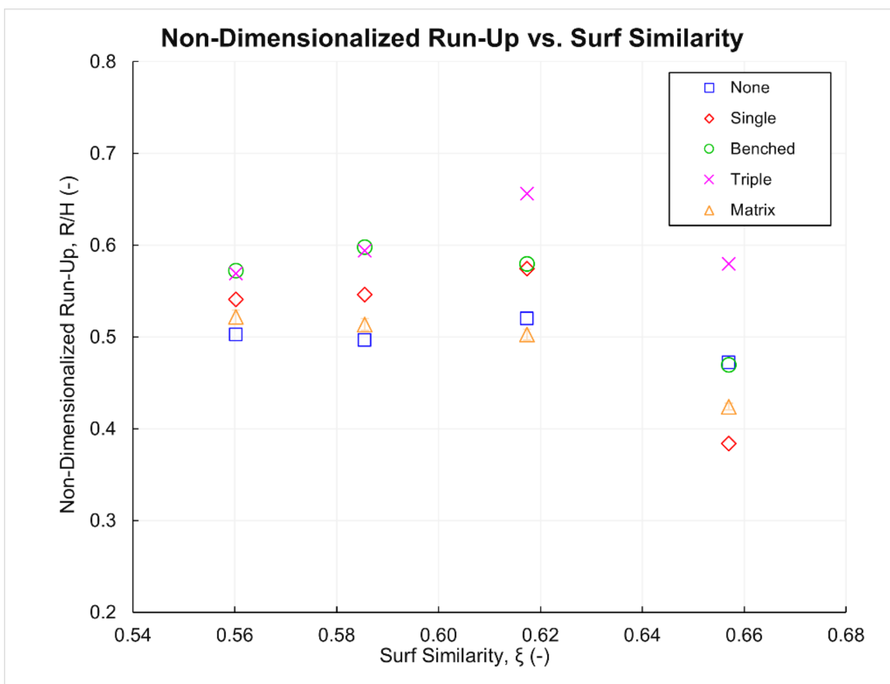

Figure 42 – Non-dimensionalized wave run-up versus Surf Similarity Parameter for configurations: none, single, benched, triple, and matrix.

5.3.2 Effect of Roughness on Wave Run-Up

To assess the effect of log roughness, the measured wave run-up is plotted against the target wave height in Figure 43 for the following configurations: (a) None, (b) Single, and (c) Single-Rough. See Section 5.2.2 for a description of the roughness elements added to the LWD model. Measured run-up for the Single-Rough configuration follows a similar trend as the Single configuration and are within the margins of error. It can therefore be concluded that log roughness does not play a significant role in wave run-up.

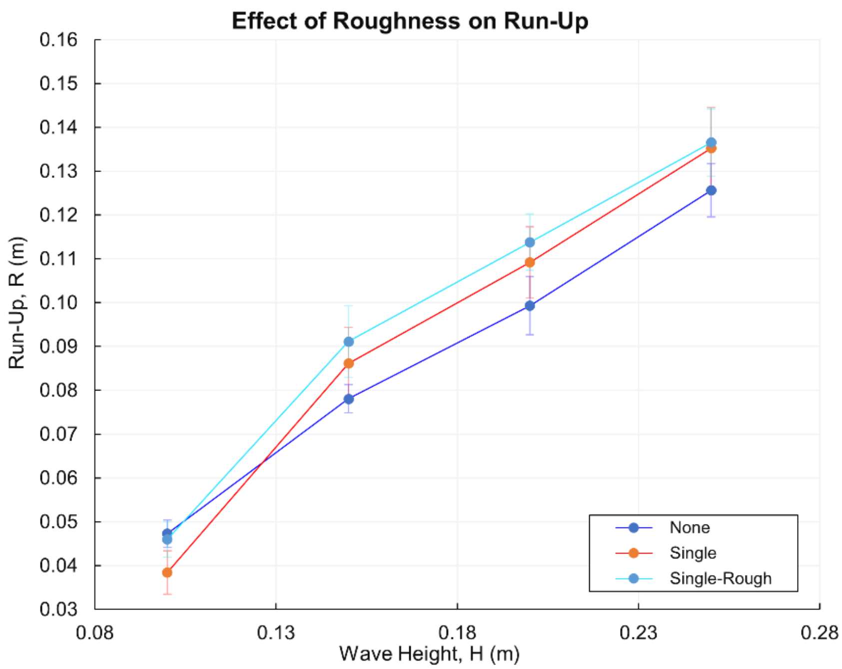

Figure 43 – Measured wave run-up versus wave height for configurations: none, single, and single-rough. Error bars indicate one standard deviation.

5.3.3 Comparison with Empirical Formulas

Several formulas have been developed to characterize and estimate wave run-up; however, their applicability and accuracy to gravel beaches are still being investigated. In addition, most of the existing empirical formulas were developed for irregular waves on static slopes. Of interest to this study, Van Broekhoven (2011) developed a hyperbolic function for porous structures of various breaker types:

$$R/_{H} = 2.25\,tanh(0.5\xi) \qquad (33)$$

Arana (2017) proposed a formula for run-up of regular breaking waves on rough-permeable slopes based on data from Van Broekhoven (2011):

$$\boldsymbol{R/H = 3.74\, tanh(0.38\xi)\gamma_f\, \gamma_K} \tag{34}$$

Where ξ is the Surf Similarity Parameter, γ_f is the surface roughness coefficient, and γ_K is the hydraulic conductivity coefficient approximately equal to $0.64*\xi^{-0.13}$ for gravel beaches. Arana (2017) also proposed new formulas for estimating the hydraulic conductivity.

$$\boldsymbol{\gamma_K = 0.94\psi^{-0.04}\xi^{-0.026\psi^{0.17}}} \tag{35}$$

$$\boldsymbol{\psi = K^3/gv} \tag{36}$$

Where ψ is the dimensionless hydraulic conductivity (-), K is the hydraulic conductivity (m/s), g is the gravitational acceleration (m/s^2), and ν is the kinematic viscosity of water (m^2/s). The non-dimensionalized wave run-up measured for all LWD configurations were compared against those predicted by empirical formulas by Van Broekhoven (2011) and Arana (2017) for gravel beaches without any structures (Figure 44). For this project, the hydraulic conductivity was assumed to be equal to 0.1 m/s.

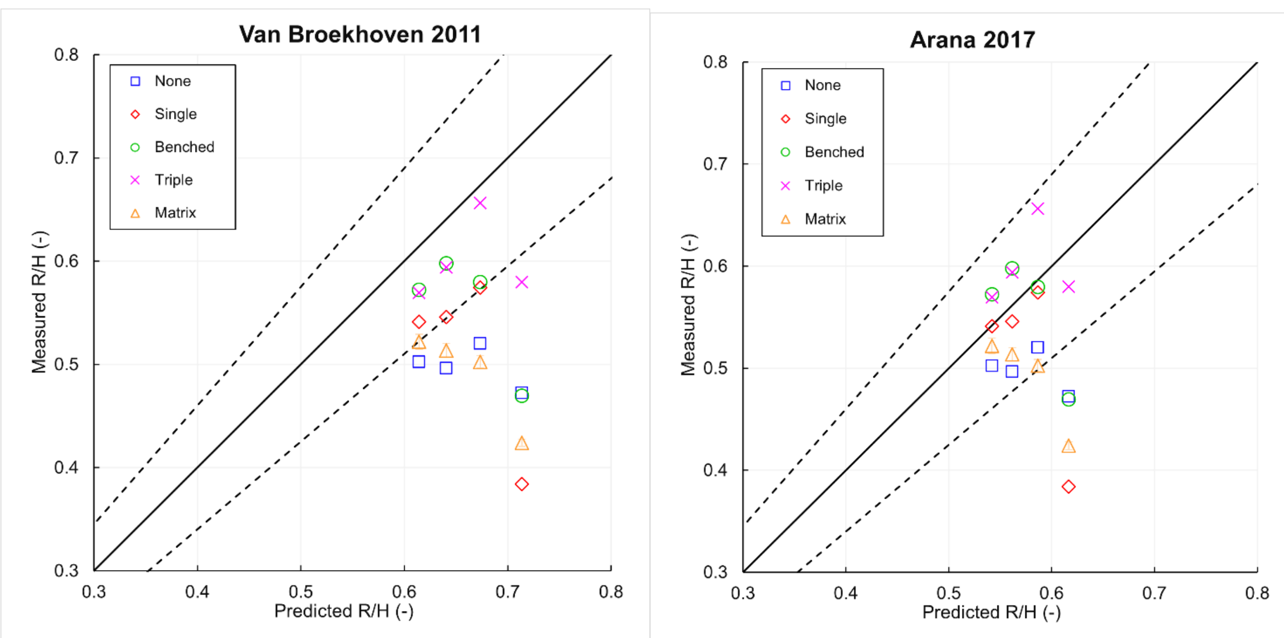

Figure 44 – Measured dimensionless wave run-up for a gravel beach with no structures vs predicted wave run-up following Van Broekhoven, 2011 (left) and Arana, 2017 (right). The solid black line indicates perfect agreement between measurements and predictions of R/H. The dashed lines indicate ±15% error.

Van Broekhoven (2011) tends to overpredict the wave run-up measured for the configuration with no LWD. Conversely, Arana (2017) shows good agreement with the measured data for a beach with no LWD. In addition, run-up measurements for all configurations are generally within 15% of that predicted by Arana (2017). Note that the smallest measured R/H for each configuration is consistently underpredicted by both equations. These data points correspond to the smallest tested wave height (H = 0.1 m) and highest surf similarity parameter (ξ = 0.66). Under these conditions, the rate of water infiltration and energy dissipation may be outside of that considered during the development of the empirical equations.

5.4 Discussion

5.4.1 Mechanisms of Increased Wave Run-Up for LWD Structures

With the exception of the smallest tested wave heights, more logs were generally associated with higher run-up regardless of the configuration. There are several possible reasons for this finding, including:

1. The logs were secured to the wall of the flume, which provided a barrier to water drainage both down the beach face and around the ends of the logs. This barrier resulted in ponding of water on the upland side of the LWD. It is possible that the ponded water reduced both turbulence and infiltration on upland side of the LWD, permitting subsequent waves to effectively ride over the ponded water and run-up farther on the beach (Figure 45).

2. The overall slope permeability was effectively reduced by adding impermeable logs on the slope. Reduced permeability of gravel beaches is understood to increase the wave run-up (Arana, 2017; Buscombe & Masselink, 2006; Powell, 1990; Van Broekhoven, 2011). This potential mechanism is supported by the fact that additional logs resulted in higher wave run-up. For example, the Triple configuration resulted in between 5 – 34% more run-up than the Single configuration.

Notably, if the LWD structures were placed near the upper limit of wave run-up such that they were not submerged, they may act as a barrier to wave run-up, thereby reducing run-up. However, this hypothesis could not be concluded as the effect of LWD elevation on run-up was not tested.

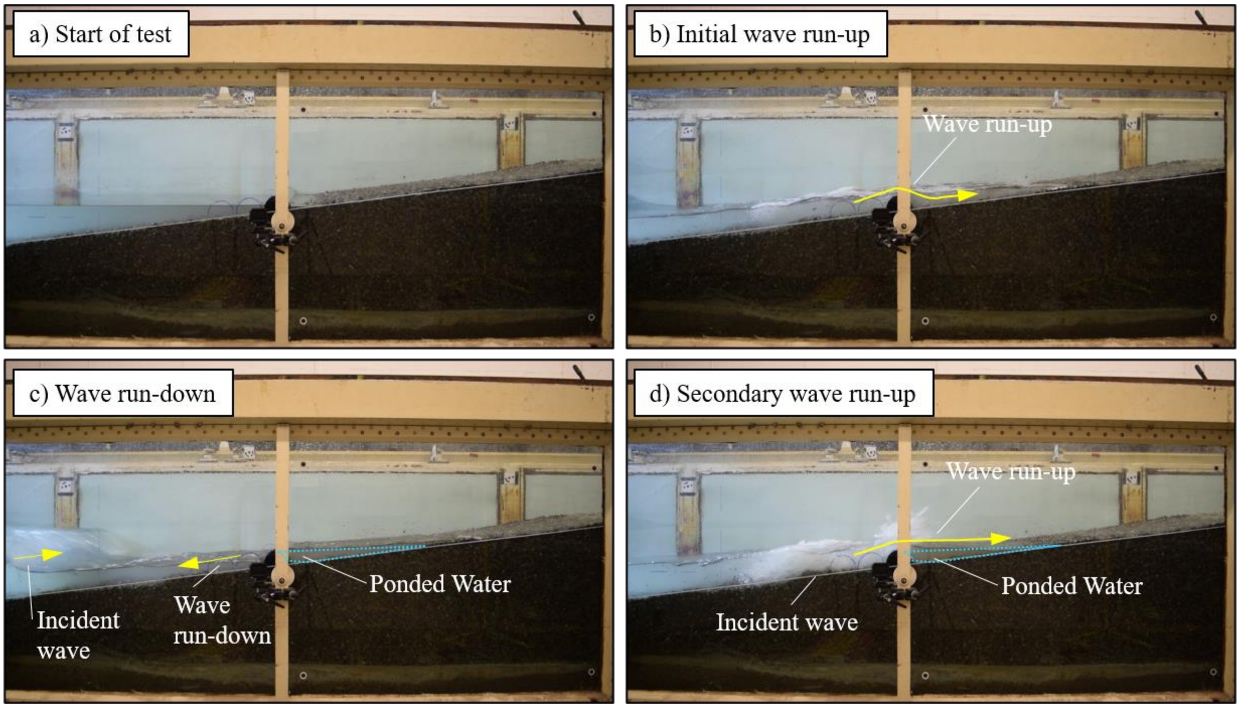

Figure 45 – Schematic of ponding water upslope of LWD causing increased wave run-up.

5.4.2 Design Considerations

No peer-reviewed literature exists on the design or efficacy of anchored LWD the coastal environment. However, some limited research suggests that natural deposits of LWD may serve to build up the beach crest and reduce the inland penetration of waves (Kennedy & Woods, 2012). This research does not directly translate to anchored LWD structures. Despite this, existing design guidance and conventional wisdom suggests that LWD may be used to reduce wave run-up and wave induced flooding (e.g. Gerstel and Brown, 2006; Johannessen et al., 2014; Zelo et al., 2000). This research, however, suggests that anchored LWD structures may actually serve to promote wave run-up on the shoreline. If anchored LWD structures are to be used as coastal protection, it is recommended that special attention be made for the following design considerations:

- LWD should not be relied upon to reduce wave run-up or provide coastal flood protection. It is therefore not recommended to use this technique for low-lying shorelines or flood prone areas.
- LWD should be placed in a discontinuous manner, avoiding placement of multiple logs in a mat-like fashion or end-to-end. This will allow for better infiltration of water into the beach slope and surface water to run-down between and around individual logs.

- Consideration should be made for LWD configurations that promote dissipation of wave energy on the shoreline through increased turbulence, such as matrix-style structures that closely resemble natural accumulations.
- LWD placement elevation is expected to play a significant role in wave run-up and beach stabilization. Theoretically, there is both lower and upper placement elevations at which the LWD will not influence wave shoaling or wave run-up. As preliminary guidance, LWD should be placed above the high-water mark, but within the maximum wave run-up limit (to mimic the location of natural accumulations).

5.4.3 Study Limitations & Future Research

This experimental program focused on the effect of various shore-parallel LWD configurations on wave run-up in comparison to a beach with no structures. Limitations of this study and areas for future research include the following:

- The current set of tests relied on a small set of regular wave tests to allow for sediment mobility whilst maintaining a relatively flat and continuous bed slope for wave run-up estimation. This would not have been possible using irregular waves due to the need to include a large sample of waves representative of the wave spectrum, leading to significant bed changes over the test duration. However, regular waves are known not to be representative of real wave climates. As such, future experimental modeling on wave run-up should include irregular wave tests.
- Sediment characteristics are understood to play an important role in wave run-up for gravel beaches (Arana, 2017; Bagnold, 1940; Poate et al., 2016; Powell, 1990). The existing experimental modeling program should be expanded to include variation in the beach slope and beach sediment characteristics (including sediment size, gradation, porosity, and permeability). For example, introducing a semi-impermeable layer or reducing the beach porosity may reduce infiltration, increase wave run-up and run-down velocities, and significantly modify the observed wave run-up.
- The Surf Similarity Parameter (Iribarren Number) is fundamental to the current understanding of wave run-up on gravel beaches. The existing experimental modeling

program should be expanded to include variations in the wave characteristics to test sensitivity to this parameter.

- Expand the existing experimental modeling program to include additional variations to important design characteristics, such as the relative LWD placement elevation, log diameter, log roughness.
- For this test program, LWD was fixed-in-place by applying pressure to the side walls of the flume resulting in a static structure. Although this behaviour is representative of some anchored LWD installations when first constructed, anchoring systems generally allow for some LWD mobility. Wave run-up behaviour may be significantly different if LWD is mobile or dynamic, potentially serving to absorb or amplify wave run-up. Additional studies should be undertaken to explore the influence of dynamic (as opposed to fixed-in-place) anchored LWD structures.

5.5 Conclusions

The "Efficacy of Large Woody Debris as Coastal Protection" research program aimed to investigate current design practices for coastal protection using LWD, assess their effectiveness at stabilizing the shoreline and reducing wave run-up, and develop preliminary design guidance. As part of this research, this study presented experimental modeling results related to the effect of LWD on wave run-up. The testing program considered regular waves to allow for sediment mobility whilst maintaining a relatively stable and continuous beach slope.

In contrast to conventional wisdom, these study findings suggest that LWD structures may result in higher wave run-up than a beach with no structures due, in part, from a decrease the effective beach permeability and increased water pooling on the up-slope side of the structures. Configurations with more logs resulted in the highest run-up, with the exception of the Matrix-style structure which resulted in similar levels of run-up as the plain gravel beach. The study results also suggest that LWD roughness is not a governing factor in wave run-up. Based on these findings, it is generally not recommended to use anchored LWD as a means to reduce wave run-up and wave induced flooding. However, given the study limitations (Section 5.4.3), significant additional research is needed to confirm these study findings and caution should be taken before applying these findings to engineering designs.

Chapter 6. Conclusions

6.1 General

The “Efficacy and Design of Coastal Protection using Large Woody Debris” research project represents the first systematic study focused on understanding the behaviour and usage of LWD coastal protection structures. This thesis was focused on a portion of the overall research project, related to: (1) field investigation of existing LWD installations, (2) experimental modeling of beach morphology, (3) experimental modeling of wave run-up, and (4) development of preliminary design guidance. Conclusions from this work are provided herein.

6.1.1 Field Investigation

The first phase of the project included field studies of existing anchored LWD project sites in BC and Washington State. The study sites were generally lacking both design and construction documentation, long-term monitoring, and adequate maintenance records. However, based on available information, it is understood that LWD coastal protection structures have been used in these areas since 1999 or earlier. The designs were found to be highly variable, but dominated by single, shore-parallel structures. Other installation types included multiple, benched, matrix, groyne, and stacked structures. Typical durability issues included missing or loose LWD, failed or corroded anchors, erosion and beach deflation, arson, and wood decay. Site characteristics, design techniques, and durability indicators were examined and correlated to a new design life parameter: ‘Effective Life’. Increased Effective Life was associated with sites with a smaller tidal range and milder upper beach slope for all LWD configurations, and was correlated to the LWD placement elevation relative to the beach crest for single, shore-parallel structures. The field investigation results suggest that existing anchored LWD installations have not been effective at providing coastal protection nor have they met typical engineering design life requirements.

6.1.2 Experimental Modeling: Morphology

A comprehensive set of over 60 experimental tests were completed as part of the overall research program. For the purpose of this study, 32 tests were analyzed relating to the morphological response of a gravel beach with and without various LWD configurations. Of the four LWD configurations tested, the Benched configuration resulted in the most stable beach profile, seconded by the Triple configuration. A Single LWD configuration provided similar beach stabilization benefits as the Benched configuration when placed below the still water level, but may be detrimental when placed on the upper beach for large wave heights. The Matrix configuration provided only a small reduction in sediment transport. More-so than the configuration type, the position of the most seaward LWD (whether considering distance or elevation) was found to be strongly linked to morphological response. As such, LWD design configurations which included LWD below the still water level, such as the Benched configuration, were found to be most effective at stabilizing the beach profile.

6.1.3 Experimental Modeling: Wave Run-Up

Twenty-four (24) tests were also conducted for the purpose of estimating the effect of LWD configurations on wave run-up. In total, six different beach and LWD configurations were tested under a base set of four regular wave conditions. The tests were each repeated once, to achieve a total sample of 40 waves per test. The study findings indicated that anchored LWD may serve to increase wave run-up relative to a gravel beach with no structures. Configurations with more logs tended to result in higher wave run-up. The mechanisms behind this finding may be that the effective reduction in beach permeability and infiltration from the presence of the logs on the beach surface, or that the fixed-in-place logs did not allow for adequate drainage resulting in water ponding on the upland side of the logs. Significant additional research is needed on the effect of LWD on wave run-up to confirm and expand these findings.

6.1.4 Preliminary Design Guidance

There are a number of potential benefits associated with anchored LWD installations if designed, installed, and monitored appropriately for the site conditions. However, to date, design guidance on how to appropriately design these structures has been limited and largely based on anecdotal evidence and practitioner experience. Based on the results of this study, preliminary design considerations include the following:

1. LWD will decay over time and, hence, tree species with low decay rates should be considered when exposed to wetting and drying cycles, and salt-water (USBR and ERDC, 2016; Charles et al., 2016; Shields et al., 2004).

2. LWD installations using chains were generally associated with higher failure rates and corrosion than installations with cables. Consideration should be given for marine-grade

cables as a ballasting method.

3. Anchored LWD installations are expected to be better suited to areas with smaller tidal ranges or a smaller range of expected water levels.
4. Anchored LWD installations are better suited to sites with mildly sloping upper beaches. Notably, these sites may already be associated with a more inherently stable beach slope and may not require installation of any coastal protection structures.
5. LWD may degrade slower when placed above the beach crest, but are unlikely to provide beach stabilization benefits in this location. For the purpose of realizing improved beach stability, LWD may be placed below the limit of wave run-up; however, extremely careful consideration should be made on the anchoring methods used and the design life of a structure placed below the high-tide level due to consideration of buoyancy and decay.
6. Anchored LWD's orientation and installation type should be selected with consideration of incident wave characteristics, sediment budget, and sediment transport conditions.
7. Consideration should be made for LWD configurations that promote dissipation of wave energy on the shoreline through increased turbulence, such as matrix-style structures that closely resemble natural accumulations.
8. LWD should be placed in a discontinuous manner, avoiding placement of multiple logs in a mat-like fashion or end-to-end.
9. Anchored LWD may be suitable for sites with significant aeolian sediment transport (Grilliot et al., 2018).
10. Anchored LWD located near population centers may be susceptible to arson or tampering. As such, these types of structures may not be suitable in high-population areas without proper signage and public awareness.
11. Anchors may become loose or exposed, posing a safety hazard and potentially resulting in LWD movement (Zelo et al., 2000). It is critical that inspections and maintenance are planned and completed.
12. The present study does not support using anchored LWD as protection from wave run-up and wave-induced flooding.

13. Because of a lack of other available literature, any usage of anchored LWD as coastal protection should be treated as experimental (Jones et al., 2016).
14. Despite potential shore stabilization benefits of coastal protection using LWD, a significant barrier to usage is how to effectively anchor the structures. Careful consideration must be made of the anchoring and ballast type and size. Because little information is available on how to design these types of anchoring systems for LWD under wave action, experimental or numerical modeling of designs should be considered.

6.2 Recommendations for Future Work

This study represents a starting point for research on coastal protection using LWD. As such, there are several areas that require additional research to fully understand anchored LWD behaviour, effects, and best design practices. Some areas for future research are as follows:

- The field investigation included a high-level study of fifteen sites with existing LWD installations. Detailed analysis of the wave exposure, long-term geomorphological trends, original design intent, and maintenance should be undertaken to better understand the efficacy of these structures.
- All of the field study sites were located in the Salish Sea region. In order to extrapolate the study findings beyond the metocean conditions and physical setting of the Salish Sea, investigations could be completed for sites outside of this area, including lakes.
- The research program included an extensive experimental modeling program. Despite this, due to time constraints, there were limitations on the number and range of variable that could be tested. Expansion of the existing experimental modeling program may include the following:
    - Study the combined effects of LWD configurations (including groynes) on longshore and cross-shore sediment dynamics using a wave basin.
    - Vary the initial beach slope
    - Vary the water level to simulate tides and/or variation in groundwater level
    - Vary the log characteristics, including diameter and roughness

    - Vary the beach sediment characteristics, including sediment size, gradation, porosity, and permeability
    - Study the behaviour of dynamic (as opposed to fixed-in-place) anchored LWD structures.

- Additional research is needed to expand the experimental modeling program to include wave run-up with irregular waves.
- Additional research should also be undertaken to study anchor forces and develop design guidance for the anchor and ballast system.
- Lastly, to allow for options analysis and policy development, the efficacy of anchored LWD structures (as well as other nature-based solutions) should be compared to more traditional coastal protection structures, such as seawalls and revetments.

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Appendix A. Field Protocol Template

FIELD PROTOCOL

Updated: May 29, 2019

SITE INFORMATION

Site Name		
Site Identifier		
GPS Location	N:	E:
Date		
Start Time	PST	
End Time	PST	
Weather		

POST PROCESSING

Path of saved photos:
Path of saved GPS files:
Distance to/from site (km)

GENERAL OBSERVATIONS

Note: If 'Other' is chosen under 'Selection' specify type in comment
If only some of the logs have root wads/bark/limbs - specify percentage in comment
For anchor attachment and ballast types: if present, add supplemental information on sizes/diameters

Description	Selection	Comment	Photo No.(s)
Installation Type	Singl. / Doub. / Matrix / Other		
Installation Orientation	Parallel / Groyne / Other		
Anchor Attach. Type	Cable / Chain / Rope / Other		
Anchor Ballast Type	Rock / Conc. Block / Other		
Root Wads	None / Some (%) / All		
Bark	None / Some (%) / All		
Tree Limbs	None / Some (%) / All		
Riparian Vegetation	None / Native / Invas. / Other		

GPS POINT COLLECTION

ID	Description	ID	Description
SP	Beach Slope (One transect; measurements at **5ft** intervals)		
WL	Water line		
CL	Beach crest line		
LW	Top line of most seaward and landward logs (minimum)		

SEDIMENT SIZE (PHOTOS SEIVE)

*Note: Transect to match GPS slope (SLP) measurements; Take photo along transect at every **10 ft** interval;*
*Section 0 located at **seaward edge** of first LWD and increasing seaward.*

Section	Comment	Photo No.(s)

LWD MEASUREMENTS/OBSERVATIONS

No.	Length (ft'in")	Diameter (ft'in")	Av. Ht Visible Above Ground (ft'in")	Damage Observations (Erosion / Wood Degrad. / Anchor Degrad.)	Photo No.(s)
1					
2					
3					
4					
5					
6					
7					
8					
9					
10					
11					
12					

SUPPLEMENTAL INFORMATION

No.	Measurement	Comment	Photo No.(s)
Extra Space if Required:			
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			
Extra Space if Required:			
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			
Extra Space if Required:			
1			
2			
3			
4			
5			
6			
7			
8			
9			
10			

ADDITIONAL COMMENTS

Appendix B. Site Summary Sheets

28 potential study sites were identified where anchored LWD was used as coastal protection in northern Washington and southern British Columbia (Table 11). Attempts were made to visit all the potential sites between June – August 2019. Sites in BC were scoped in advance of the site visits to confirm the site locations and site access. For accessibility reasons, this could not be completed in advance for sites in Washington.

At five of the locations in Washington, no anchored LWD could be found and the installation site could not be identified. It is possible that the precise site location was not identifiable based on the available background documents, or that the LWD installation had since been removed or destroyed. An additional eight sites in Washington could not be surveyed due to site access or permission constraints. Site visits were successfully completed at eight sites in BC and seven sites in Washington.

Table 11 – Status of site visits to potential study sites anchored LWD installations

Site Identifier	Site Name	Location
Site Investigation Completed		
1_BC_Rath	Rathtrevor Park	Parksville, BC, CA
2_BC_Lazo	Lazo Road	Comox BC, BC, CA
3_BC_Kitt	Kitty Coleman Park	Comox BC, BC, CA
4_BC_Stor	Stories Beach	Oyster River, BC, CA
5_BC_Kin	Kin beach	Comox, BC, CA
6_BC_Arbu	Arbutus Point	Parksville, BC, CA
7_BC_Qual	Qualicum Beach Shoreline	Qualicum Beach, BC, CA
8_BC_Oar	Oar Road Outfall	Lantzville, BC, CA
10_WA_Weav	Weaverling Spit	Fidalgo Bay, WA, USA
11_WA_Kits	Kitsap Memorial Park	Lofall, WA, USA
12_WA_Mari	Marine View Drive	Tacoma, WA, USA
17_WA_Oak	Oak Bay	Port Hadlock - Irondale, WA, USA
25_WA_Brem	Bremerton Evergreen Park	Bremerton, WA, USA
27_WA_Penr	Penrose Point	Lakebay, WA, USA
28_WA_Frye	Frye Cove Park	Gravelly Beach, WA, USA
Site Visited – No LWD Structures Found		
18_WA_Nor	Northwest Maritime Center	Port Townsend, WA, USA
20_WA_Narr	Narrows Park	Tacoma Narrows Park, WA, USA
22_WA_Edmo	Edmonds Waterfront	Edmonds, WA, USA
24_WA_Glen	Glendale Shoreline	Glendale, WA, USA
26_WA_Dyes	Dyes Inlet	Dyes Inlet, WA, USA
Site Not Visited – Permission or Site Access Constraints		
9_WA_Nort	North side of Blakely Harbour	Bainbridge Island, WA, USA
13_WA_Birc	Birch Point	Blaine, WA, USA
14_WA_Dabo	Dabob Bay	Dabob, WA, USA
15_WA_East	East Eld Inlet	Olympia, WA, USA
16_WA_NW W	NW Whidbey Island	Whidbey Island, WA, USA
19_WA_Blak	Blake Island State Park	Blake Island, WA, USA
21_WA_Suqu	Suquamish Pebble Beach	Port Madison, WA, USA
23_WA_Manc	Manchester Port Shoreline	Manchester, WA, USA

Appendix C. Field-Based Grain Size Distribution

C.1. Photographic Areal Sampling Overview

Photographic areal sampling is a technique used to measure the grain size distribution of surficial sediments in-situ (Rennie, 2019). The method involves taking a plan-view photograph of the bed surface and orthorectifying the photograph to account for lens distortion. Typically, a frame of known distances and angles is laid on the ground (as control points) before taking the photo to support orthorectification. Areal samples, and therefore the frame, typically cover an area of 0.1 – 1 m^2. GIS packages can be used to orthorectify the image to account for distortion and scale. Two main methods for obtaining particle size distributions can be used[4]:

- Photo sieving (Ibbeken & Schleyer, 1986): Involves digitizing the observable particle clasts, and measuring the long and short particle axis. A phi value is calculated based on the short axis measurement. Each particle is then assigned to a sediment grain-size class. The particle volume is calculated based on an ellipsoid fitted to the long and short-axis (as per the below equation). Each grain size class is converted to weight based on the particle density (typically 2650 kg/m^3). This method will result in an area-by-weight distribution.

$$\boldsymbol{V_E = \frac{\pi}{6} \times L_{photo} \times S_{photo}^2} \tag{37}$$

  Where V_E is the volume of the ellipsoid revolving around the long axis, L_{photo}, with the short axis equal to S_{photo}.

- Measure b-axes: If particles are assumed to be lying flat, then the a- and b-axes can be measured for all observable clasts. This method will result in an area-by-number frequency distribution if completed for all clasts within a given area, or a grid-by-

[4] A third method includes a photographic pebble count, which involves counting the number of clasts within the frame and converting to a pebble count D_{50} using a function calibrated to the site. This method is recognized to be relatively crude and is not considered to be an areal sample.

number frequency distribution if sediment is sampled by overlaying a grid onto the image.

A main limitation of photographic areal sampling is that the resolution of the photograph limits the size of clasts that can be observed and therefore measured. As such, an estimation of the area covered by material not included in the analysis must be made.

To compare with other types of sediment sample distributions, it is necessary to convert the frequency distribution. Typically, it is desired to convert to a volume-by-weight (e.g. bulk sample) sediment distribution (Rennie, 2019). For example, to convert from an area-by number to a volume-by-weight distribution, the conversion factor is D^2.(area to volume: D^{-1}; number to weight: D^3; area-by-number to volume-by-weight: $D^{-1} \times D^3 = D^2$). In practice, this means that each sediment fraction, p_i, is multiplied by the conversion factor based on the mean of the sediment fraction, D_i^x (Rennie, 2019).

Table 12 – Conversion factors based on sediment sampling method (Bunte & Abt, 2001)

Conversion from	Conversion to				
	Volume-by-weight	Volume-by-weight	Volume-by-weight	Volume-by-weight	Volume-by-weight
Volume-by-weight	1	1	D^3	D^{-2}	D
Grid-by-number	1	1	D^3	D^{-2}	D
Grid-by-weight	D^{-3}	D^{-3}	1	D^{-5}	D^{-2}
Area-by-number	D^2	D^2	D^5	1	D^3
Area-by-weight	D^{-1}	D^{-1}	D^2	D^{-3}	1

C.2. Methodology

A digital grain size analysis was undertaken to estimate the grain size distribution of surficial sediments in-situ. Plan-view photographs were taken at representative locations on the beach surface using a 0.5 m x 0.8 m rectangular frame (Figure 46). The images were orthorectified using ESRI ArcMap (Version 10.7.1) based on the known dimensions of the frame to adjust for lens distortion and scale. A 0.1 m x 0.1 m grid was overlaid resulting in 28 intersection nodes per image, for several representative images of the upper beach slope at each site. The surface areas of sediment clasts at each node were digitized and converted to an equivalent ellipse diameter.

Only sediments above approximately 2mm were visible in images due to pixel resolution. As such, sediments smaller than 2mm were accounted for in the grain size analysis based on the percentage of nodes with fine sediments. Debris and organic materials also reduced particle visibility.

To compare the resulting grain size distribution from this method with other types of sediment sampling methods (e.g. bulk samples), it was necessary to convert the frequency distribution. The estimated grain sizes were converted from an area-by number to a volume-by-weight distribution, using a conversion factor of D^2 (Bunte & Abt, 2001). The resulting grain size distribution curves are provided in the Section C3.

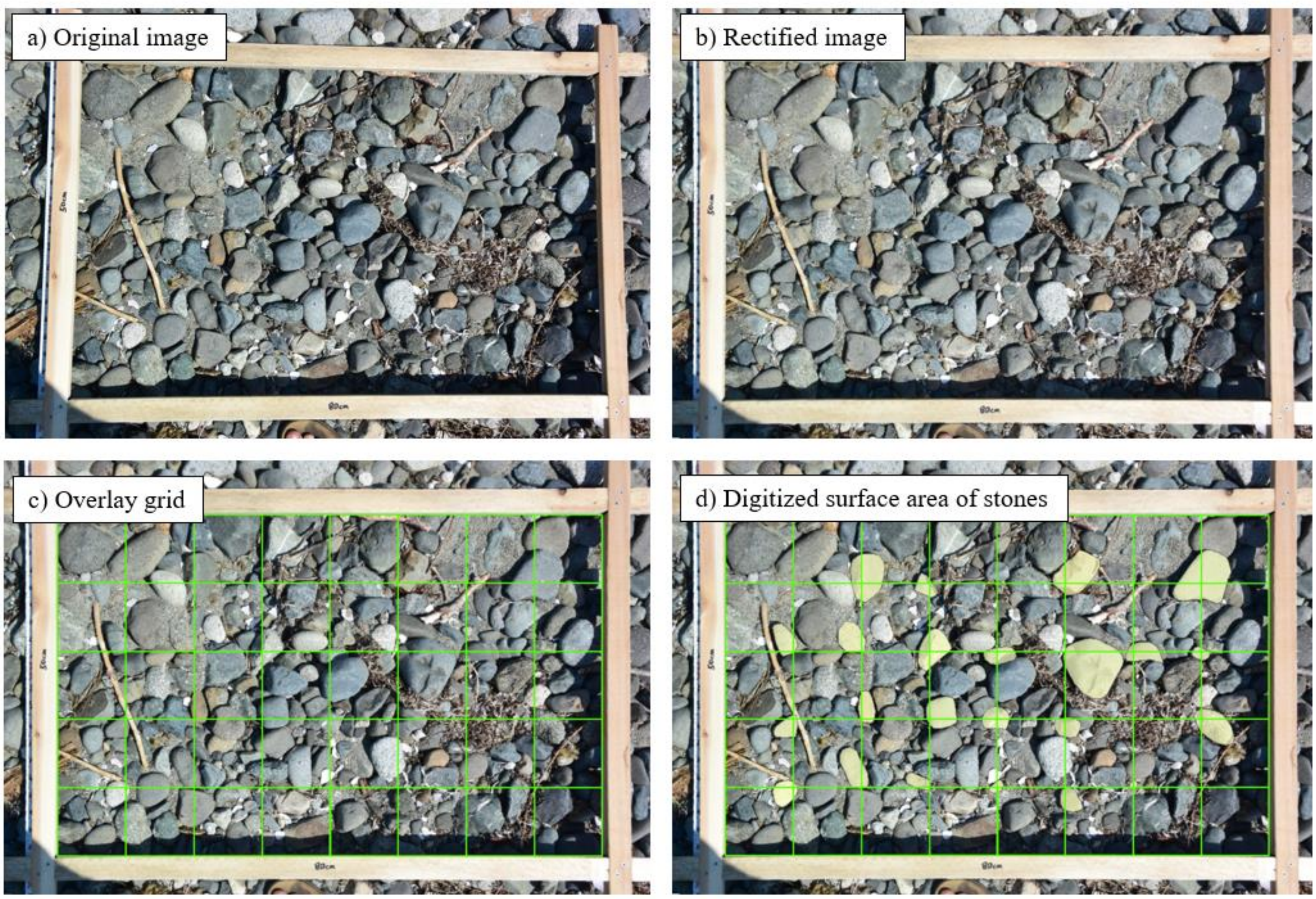
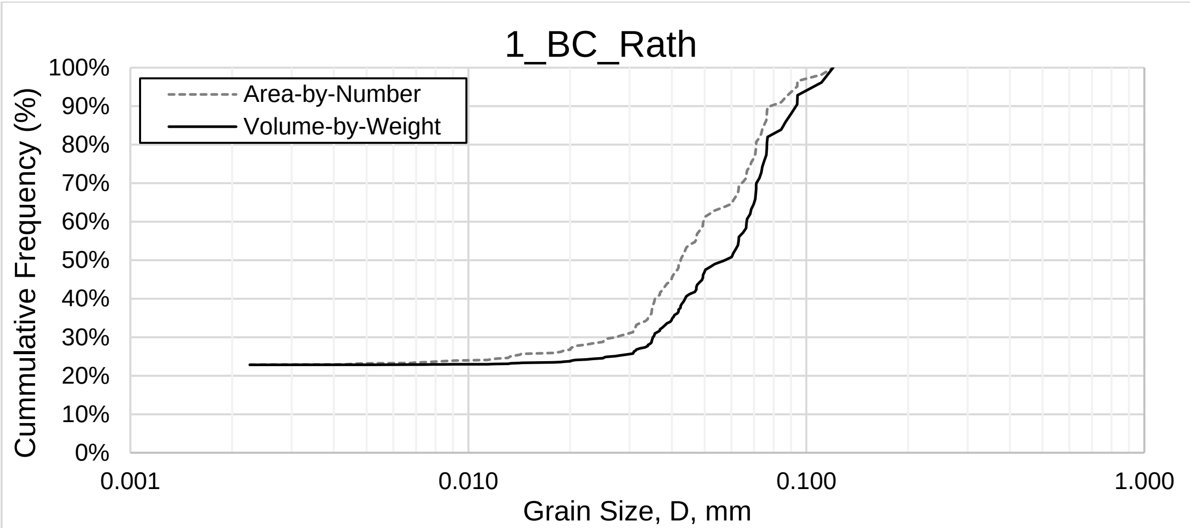
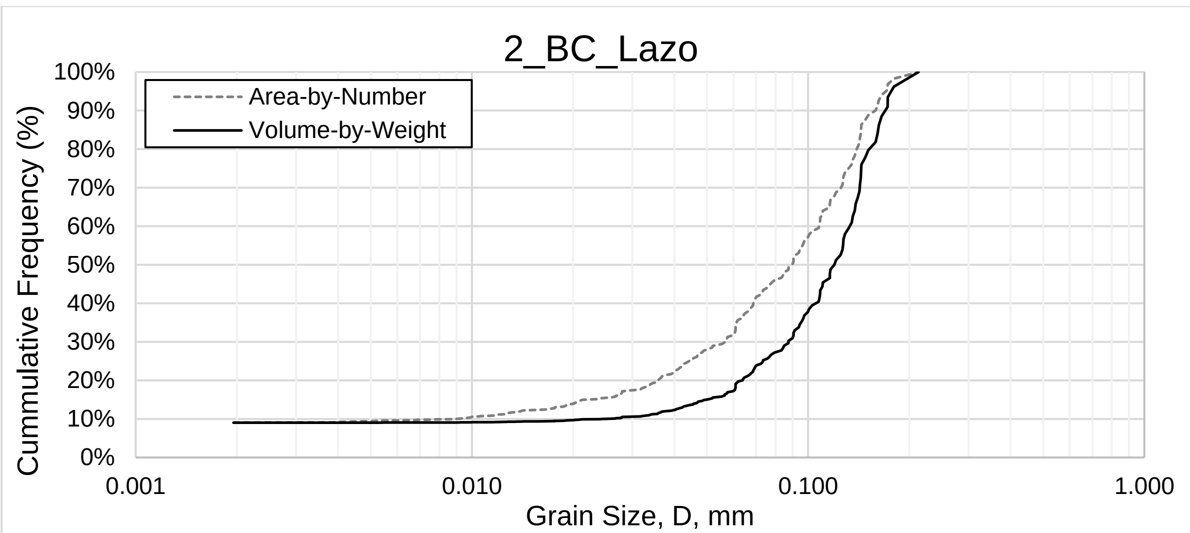
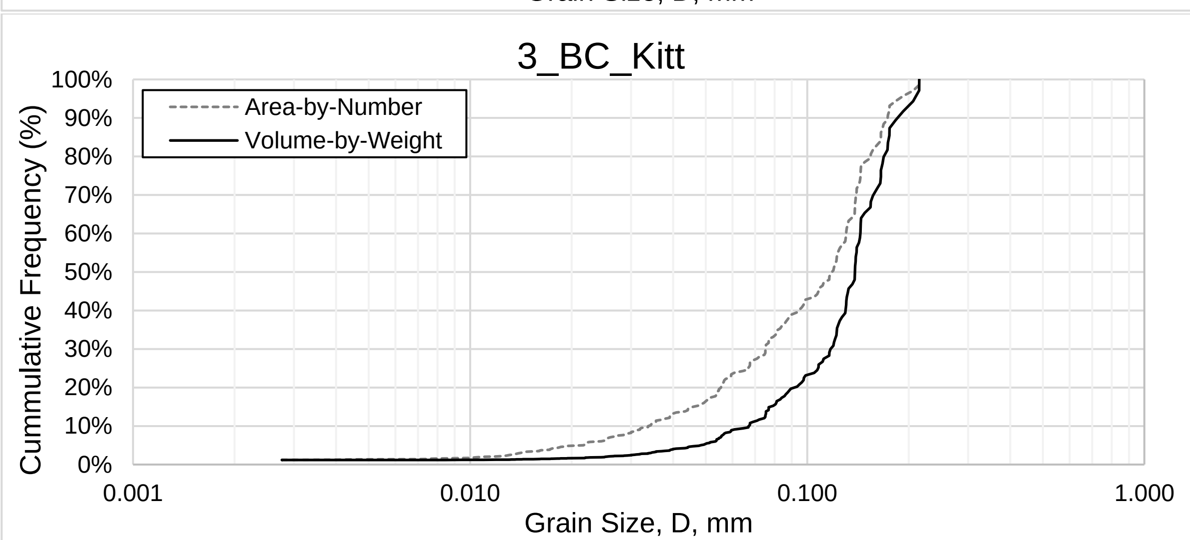


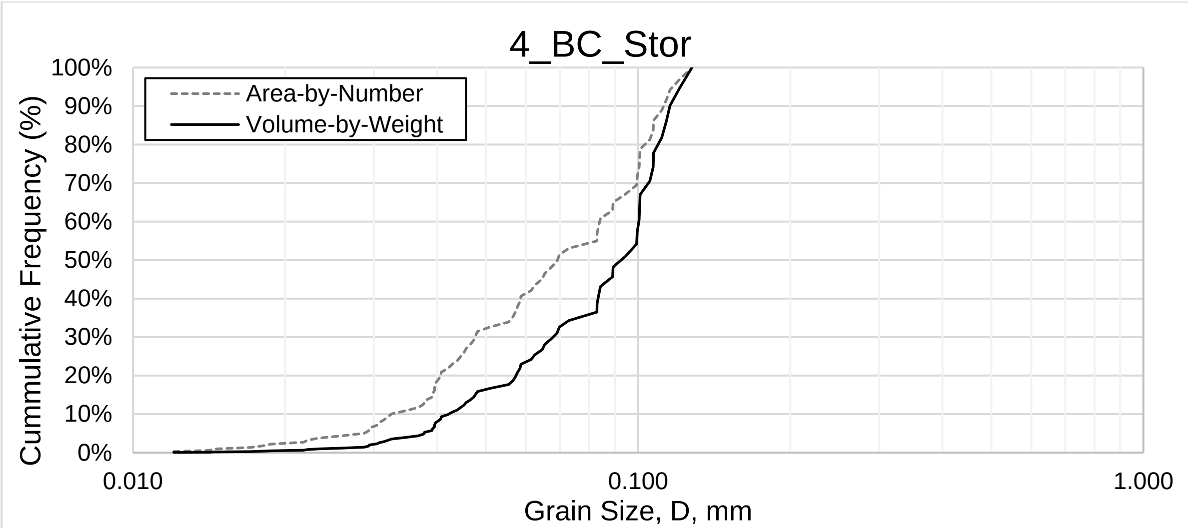
Figure 46 – Methodology for undertaking digital grain size analysis by grid overlay

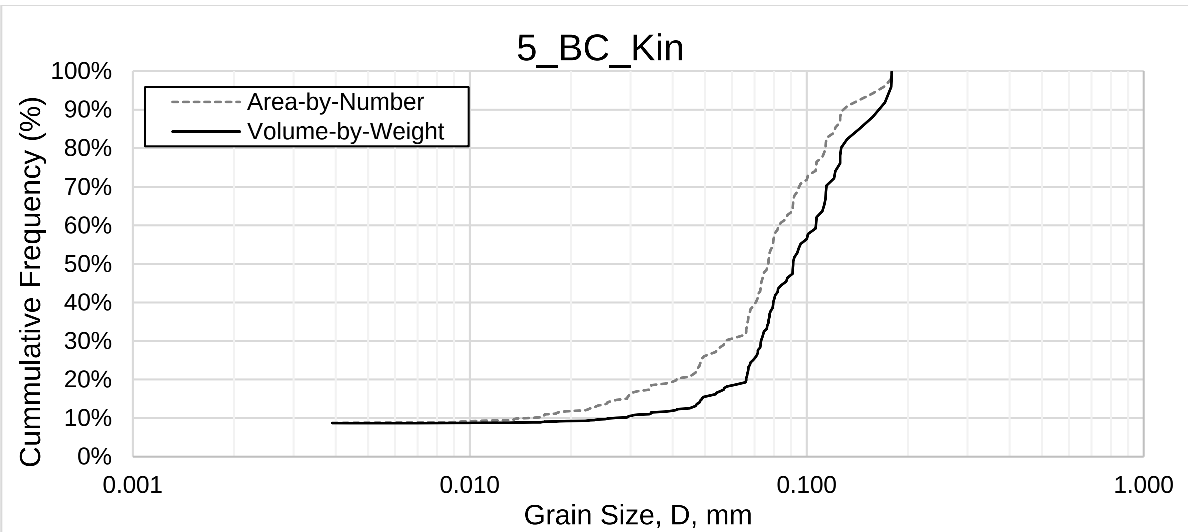
C.3. Digital Grain-Size Analysis Results

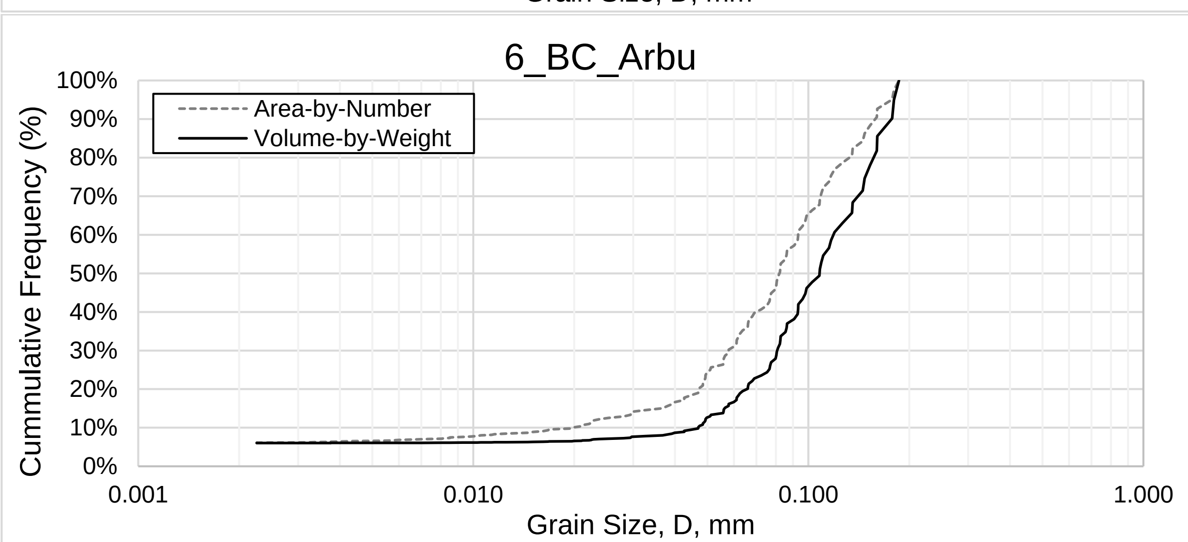


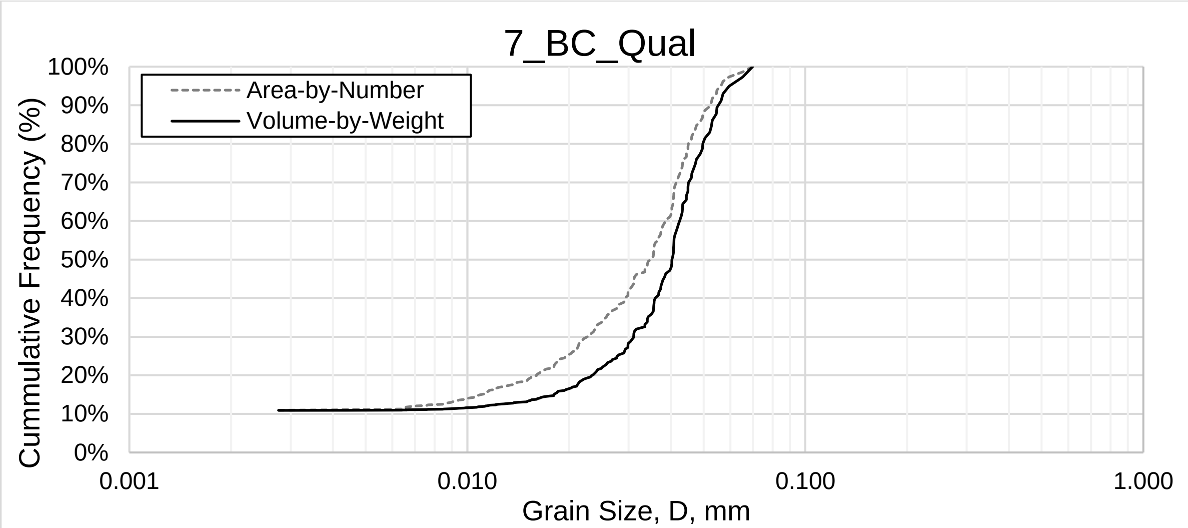


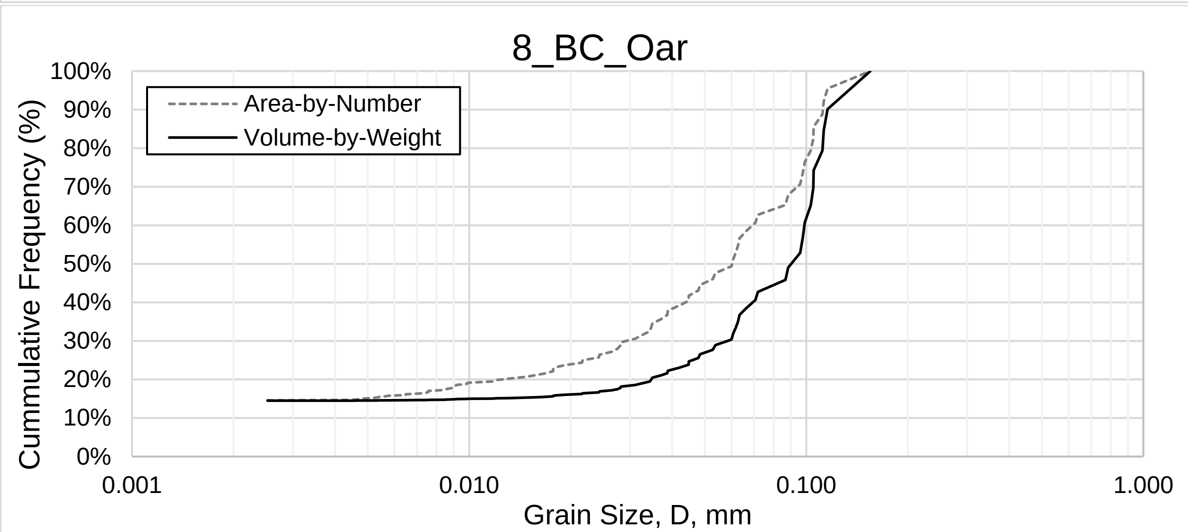


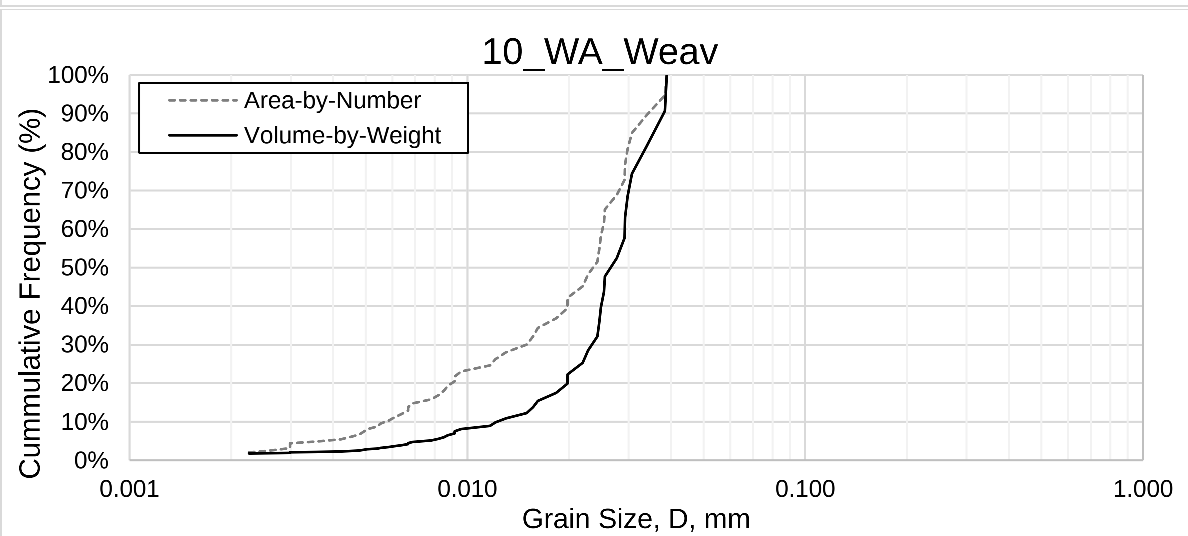

4_BC_Stor
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.010
0.100
1.000
Grain Size, D, mm

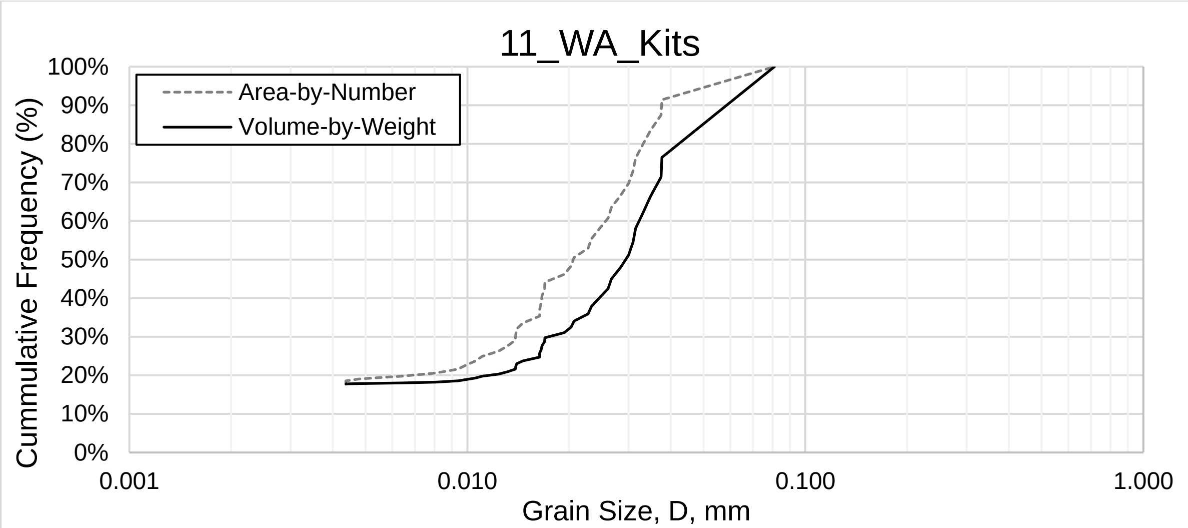

5_BC_Kin
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm

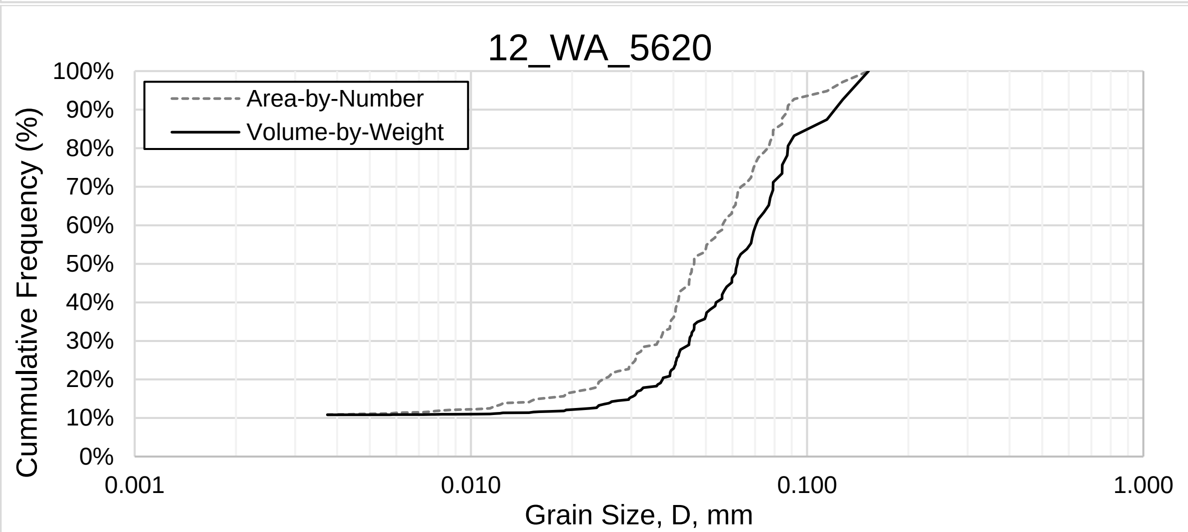

6_BC_Arbu
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm

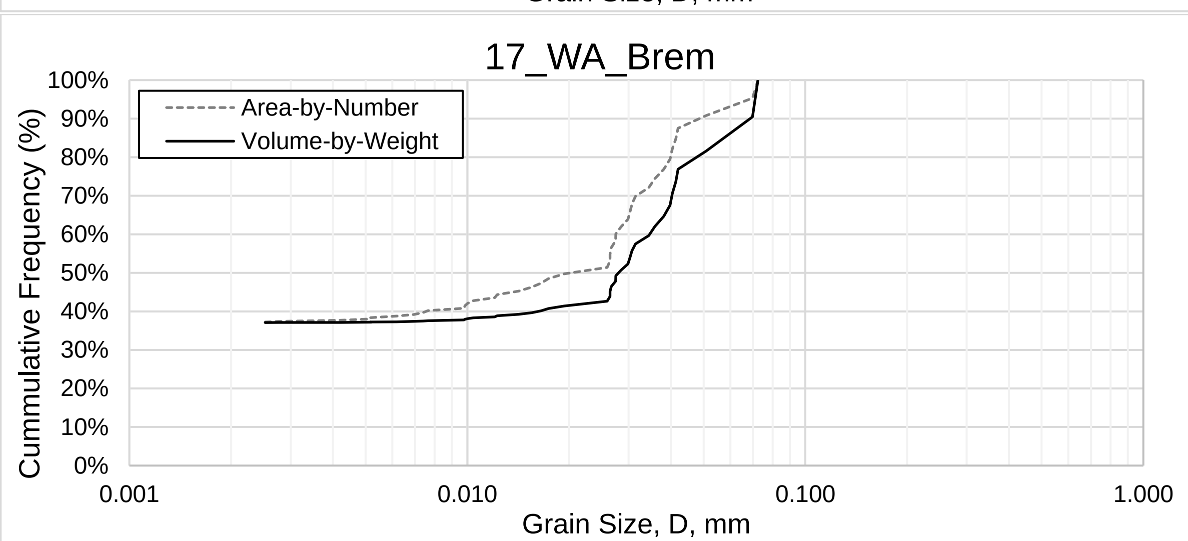

7_BC_Qual
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm

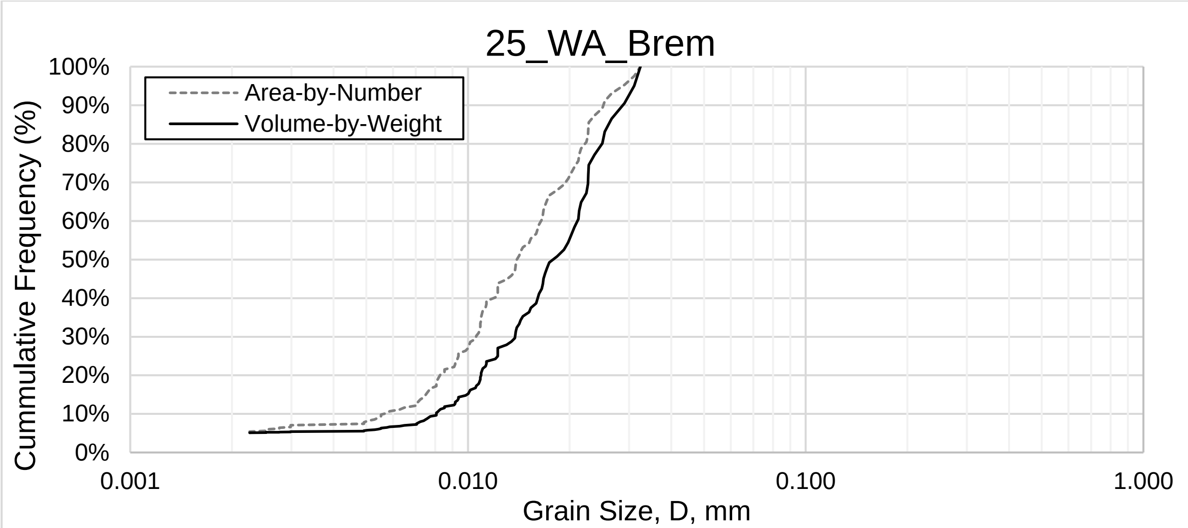

8_BC_Oar
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm


10_WA_Weav
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm

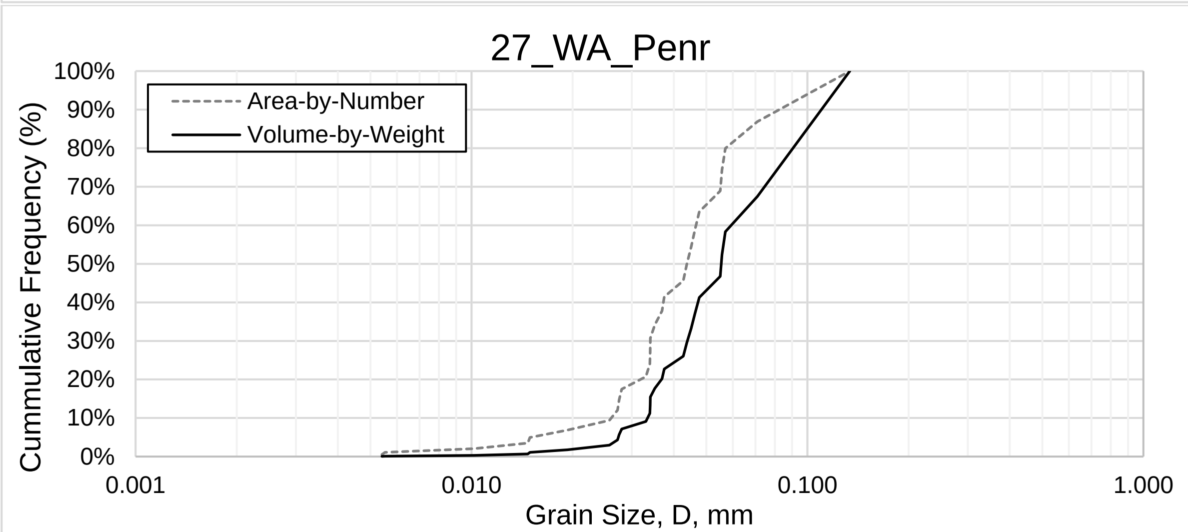

11_WA_Kits
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm


12_WA_5620
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm

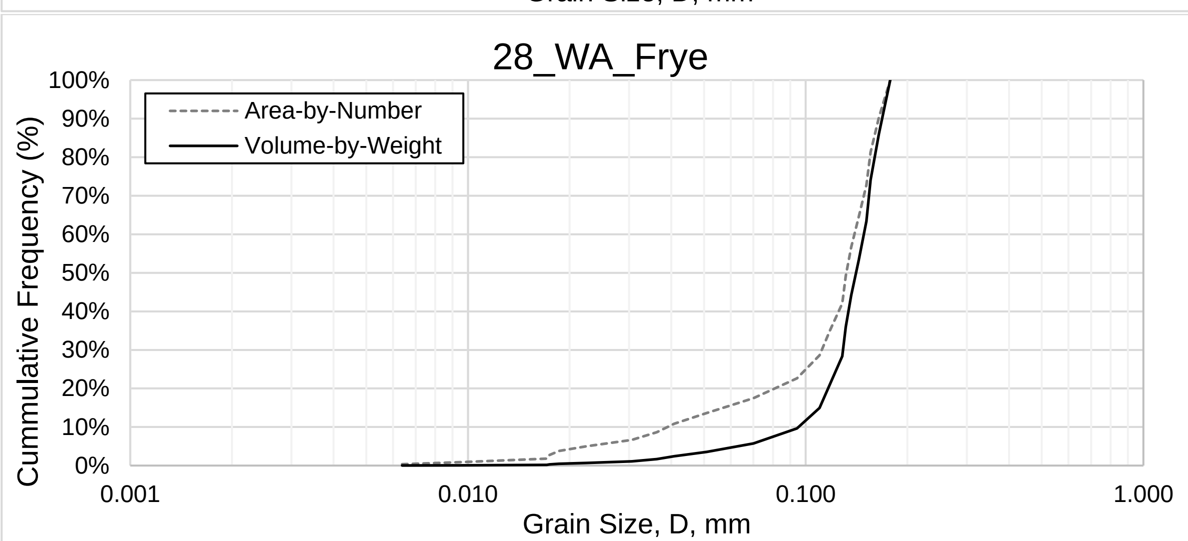

17_WA_Brem
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm



25_WA_Brem
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm



27_WA_Penr
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm



28_WA_Frye
Area-by-Number
Volume-by-Weight
Cummulative Frequency (%)
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
0.001
0.010
0.100
1.000
Grain Size, D, mm

Appendix D. Geotechnical Testing

The beach composition observed during the field investigations varied between sand and boulders, with the mean grain size (D_{50}) at each site varying between 18 – 141 mm (gravel – cobble), with an average D_{50} of approximately 73 mm (cobble). The observed mean grain size is the equivalent of a 4 – 28 mm gravel in the experimental model at a 1:5 geometric scale. In addition, the sediments were typically observed to be rounded, due to regular exposure to wave action.

For the purpose of experimental modeling, cost and availability of a sediment source had to be considered. Accordingly, a uniform river-washed gravel material known as 'Playground Stone' was sourced from Greely Sand and Gravel, located in Greely, Ontario. 11.8 metric tonnes (approximately 7 m^3) of material was purchased and delivered to the Canadian Hydraulics Centre of the National Research Council (NRC-OCRE) in Ottawa. The material had a reported grain size within the range of 2.0 to 17.5mm, a Uniformity Coefficient, C_u, of 1.491, and a Gradation Coefficient, C_z, of 0.937. Where C_u and C_z are defined as follows (Murthy, 2002):

Uniformity Coefficient : $$\boldsymbol{C_u = D_{60}/D_{10}} \qquad \textbf{(38)}$$

Gradation Coefficient: $$\boldsymbol{C_z = D_{30}^2/(D_{60}D_{10})} \qquad \textbf{(39)}$$

Where D_x is the sediment diameter of which x percent of the sample has a smaller grain size.

The reported geotechnical properties were confirmed and additional properties were tested at University of Ottawa's Geotechnical Laboratory on a random sample of the delivered material. Specifically, sieve analysis, specific gravity analysis, and dry density tests were performed, the results of which are outlined herein.

D.1. Grain Size Distribution

A dry sieve analysis was performed to obtain the grain size distribution of the sediment, as well as uniformity/gradation coefficients. Two representative samples were taken from the delivered material, with masses of approximately 1.5kg each. The sieve analysis used openings between 0.60 to 25.00mm, as outlined in Table 13. Each sample was shaken for 10-minutes prior to measuring the weight of each sieve. The percent passing for both samples is also included in Table 13. The resulting grain size distribution curves are included in Figure 47.

Table 13 – Sieve characteristics and measured percent passing

Sieve Characteristics		Percent Passing	
No.	**Opening Size (mm)**	**Sample 1**	**Sample 2**
1 inch	25.00	100.0	100.0
3/4 inch	19.00	100.0	100.0
3/8 inch	9.50	71.7	77.3
No. 4	4.75	1.5	2.5
No. 8	2.38	0.2	0.2
No. 10	2.00	0.2	0.2
No. 16	1.18	0.2	0.2
No. 30	0.60	0.2	0.2
Pan	-	0.0	0.0

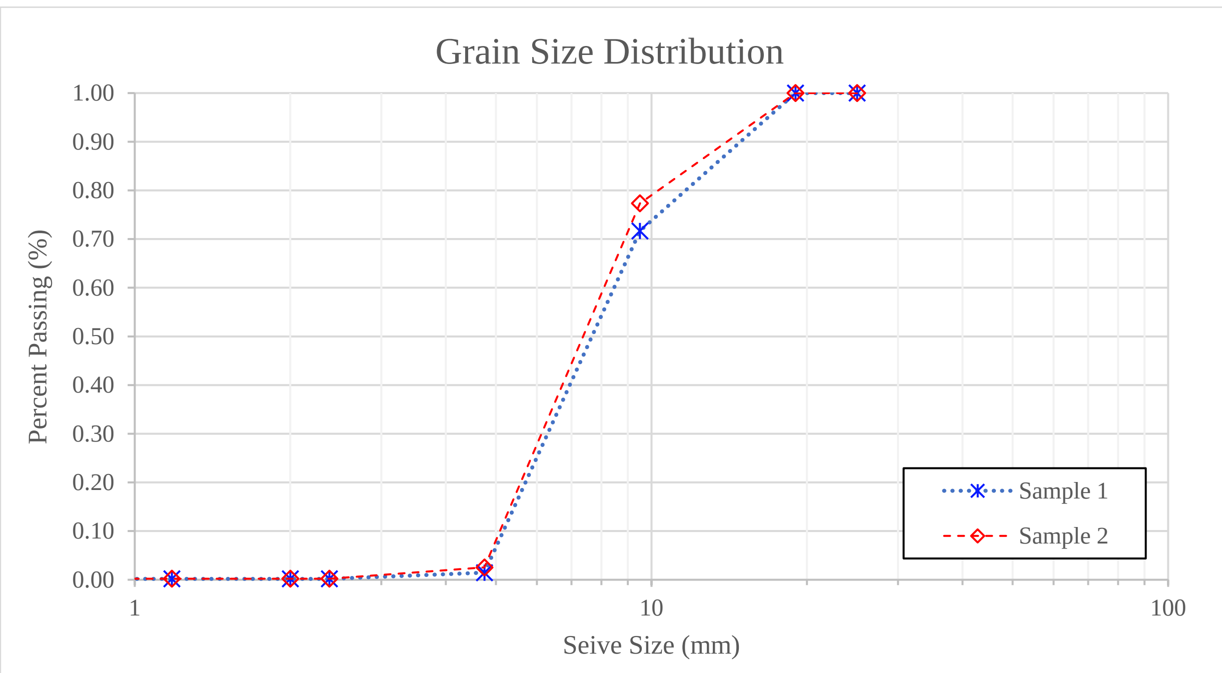


Figure 47 – Measured grain size distributions for Sample 1 and 2

Based on the grain size distribution curves, several sediment properties were estimated, including characteristic grain sizes, the Uniformity Coefficient, and the Gradation Coefficient. The samples had an average Median Diameter, D_{50}, of 7.90mm, which is on the upper limit of the fine gravel classification based on the Udden-Wentworth scale (after Wentworth, 1922). At prototype scale, this would be a pebble with a 40mm D_{50}, which is within the range of sediments observed during the field visits. The Uniformity Coefficient close to one, means that the samples are uniformly or poorly graded (Murthy, 2002). The Gradation Coefficient is also close to one, which indicates that the samples are very well sorted. Together these two properties indicate that the sediment is a single mass, as opposed to gap-graded.

Table 14 – Calculated sediment grain size properties

Property	Sample 1	Sample 2	Average
Diameter - 10% finer, D_{10} [mm]	5.33	5.22	**5.28**
Diameter - 30% finer, D_{30} [mm]	6.68	6.49	**6.59**
Median diameter, D_{50} [mm]	8.03	7.76	**7.90**
Diameter - 60% finer, D_{60} [mm]	8.71	8.40	**8.55**
Diameter - 90% finer, D_{90} [mm]	15.64	14.81	**15.23**
Uniformity Coefficient, C_u [-]	1.63	1.61	**1.62**
Gradation Coefficient, C_z [-]	0.96	0.96	**0.96**

D.2. Specific Gravity

Specific Gravity, G_s, is a dimensionless property that relates the relative density of sediment and water at a specific temperature. Specific Gravity is defined as follows (Murthy, 2002):

$$\text{Specific Gravity:} \qquad \boldsymbol{G_s = \frac{M_s}{V_s \cdot \rho_w}} \qquad \textbf{(40)}$$

Where M_s is the mass of the gravel sample, V_s is the volume of the gravel sample, and ρ_w is the density of water.

The Specific Gravity of two representative gravel samples were determined generally following the ASTM *Standard Test Method for Specific Gravity of Soil Solids by Water*

Pycnometer (ASTM, 2002). A pycnometer (flask) was filled to 500 mL and weighed, first with distilled water only, and then again with approximately 100 g gravel and distilled water. In both cases the water was heated on a hot plate to remove air (Figure 48). The calculated Specific Gravity was then adjusted to account for temperature according to ASTM (2002). Measurements and calculations to determine Specific Gravity are provided in Table 15. The gravel samples had an average Specific Gravity of 2.80.

Figure 48 – Two pycnometers and hot plate used for Specific Gravity analysis

Table 15 – Measurements and calculations for Specific Gravity

Description	Sample 1	Sample 2
Mass of flask + water filled to 500mL mark, M_1 [g]	496.60	496.26
Mass of flask + gravel + water filled to 500mL mark, M_2 [g]	561.26	566.36
Mass of dry gravel, M_s [g]	100.49	109.04
Mass of equal volume of water as the soil solids, $M_w = M_s + (M_1 - M_2)$ [g]	35.83	38.94
Specific Gravity at test temperature, $G_{s(@T)} = M_s/M_w$ [-]	2.80	2.80
Temperature of test, T [^{o}C]	25.8	29.7
Temperature correction factor, $K = \rho_{w@T} / \rho_{w@T=20}$ [-]	0.9986	0.9975
Specific Gravity $G_s = G_{s(@T)}*K$ [-]	2.80	2.79

D.3. Dry Density and Porosity

Dry Density, ρ, of the gravel was determined by filling a flask with a sample of gravel to a known volume and weighing the sample. For this set of tests, three samples were used varying between 1400 and 2000 mL in volume. Specific Weight, γ, was calculated as follows:

Specific Weight: $$\boldsymbol{\gamma = \rho g} \quad \textbf{(41)}$$

Measurements and calculations on each of the three samples are provided in Table 16. The average dry density for the gravel is 1664 kg/m^3 and the average porosity is 40.5%. Void Ratio, e, and Porosity, n, were determined following the below set of equations (Murthy, 2002):

Void Ratio: $$\boldsymbol{e = \frac{G_s \cdot \gamma_w}{\gamma_d} - 1} \quad \textbf{(42)}$$

Porosity: $$\boldsymbol{n = \frac{e}{e+1}} \quad \textbf{(43)}$$

Where γ_w is the specific weight of water, and γ_d is the dry specific weight of the gravel sample.

Table 16 – Measurements and calculations for Dry Density and Porosity

Description	Sample 1	Sample 2	Sample 3	Average
Volume of cylinder, V [mL]	2000	1400	2000	-
Mass of gravel, M_s [g]	3323.2	2370.7	3273.1	-
Dry density, ρ_d [kg/m^3]	1661.6	1693.3	1636.6	1663.8
Specific Weight, γ_d [N/m^3]	16300.2	16611.6	16054.7	16322.2
Void ratio, e [%]	68.3	65.2	70.9	68.1
Porosity, n [%]	40.6	39.5	41.5	40.5

Appendix E. Daily Experimental Log

Table 17 – Daily experimental log outlining major tasks and runs completed.

Day	Task/Details	Test Ref.
October 7, 2019	12 tonnes of gravel delivered to NRC	N/A
October 8, 2019	Bucket sample of gravel obtained for geotechnical testing	N/A
October 11, 2019	Geotechnical analysis completed at uOttawa	N/A
October 17, 2019	Hinged floor and concrete floor removed; start of cleaning flume	N/A
October 18, 2019	Beach profile drawn; flow return pipes extended; gravel fill imported and graded; 12 wave gauges tuned (2 extra);	N/A
October 21, 2019	LWD needs to be shortened to fit; wave gauges installed; ADV rack installed; beginning of camera positioning	N/A
October 23, 2019	Setting up markers and cameras; data acquisition unit set-up	N/A
October 24, 2019	Total station of flume set-up; adv installation; water overfilled to ~0.85m; wave gauges and adv signal calibrated	N/A
October 25, 2019	Flume drained to ~0.6m; preliminary calibration runs complete; testing of photogrammetry and instrumentation; beginning of post-processing; increased wave periods due to breaking	C1, C2, C3, C4, C5, C6
October 28, 2019	Regrade beach; white paneling up; new profile tape (white); reset wave gauge 7 and adv with sturdier beam; recalibrate WG 1, 3, 4; Run C7, C8, C3b, C6b; drain flume	C7, C8, C3b, C6b
October 29, 2019	Regrade beach; checked wave gauges - no calibration needed; installed log at ~0.65m; run C9 (longer test);	C9
October 30, 2019	Regrade beach; checked wave gauges; first production run without LWD (LWD18_v1) - wave generation stopped (NDAC froze) after approximately 3615s; drained and regraded beach	LWD18_v1
October 31, 2019	checked wave gauges; calibrated WG2 & 7; LWD18_v2 (b,c,d)	LWD18_v2
November 1, 2019	checked wave gauges; no calibration; LWD21_v1 (a,b); remaining 20 min drive signals made	LWD21_v1
November 4, 2019	checked wave gauges; no calibration; LWD01_v1 (a,b,c); lab tour/assessment	LWD01_v1
November 5, 2019	checked wave gauges; no calibration; repeatability test LWD02_v1 (a,b,c) - no photogrammetry taken (to do tomorrow); Nils Gosberg visit	LWD02_v1
November 6, 2019	checked wave gauges; no calibration; LWD03_v1 (a,b,c); Wave generator crash after run completion	LWD03_v1
November 7, 2019	checked wave gauges; no calibration; LWD04_v1;	LWD04_v1
November 8, 2019	checked wave gauges; calibrated 1-5; 6-7 OK; LWD05_v1; initial photogrammetry taken for following days test	LWD05_v1
November 12, 2019	checked wave gauges; no calibration; LWD06_v1; no photogrammetry taken - data not transferred from equipment; base case updated from 0.18m to 0.20m Hs	LWD06_v1

Day	Task/Details	Test Ref.
November 13, 2019	photogrammetry taken for previous day and today; checked wave gauges; no calibration; LWD07_v1; ADV data has low correlation (many anomalies) - water to clean - must turn over	LWD07_v1
November 14, 2019	photogrammetry for previous day and today; turned beach; recalibrated gauges 1-7; LWD08_v1; ADV correlation low in first few minutes of 08a - good after that once sediment in water.	LWD08_v1
November 15, 2019	photogrammetry for previous day and today; checked gauges 1-7; LWD20_v1 (6 1-hr runs);	LWD20_v1
November 18, 2019	photogrammetry for previous day and today; checked gauges 1-7; LWD09_v1 (4-hr run);	LWD09_v1
November 19, 2019	Data analyses - not in lab	N/A
November 20, 2019	photogrammetry for previous day and today; checked gauges 1-7; LWD10_v1 (5-hr run);	LWD10_v1
November 21, 2019	photogrammetry for previous day and today; checked gauges 1-7; LWD11_v1 (5-hr run); Significant issues with run: AWA2 became loose during first 30minutes, profile 3D, ADV turned slightly	LWD11_v1
November 22, 2019	photogrammetry for previous day and today; calibrated AWA1-3; checked gauges 1-7; LWD12_v1 (4-hr run);	LWD12_v1
November 25, 2019	photogrammetry for previous day and today; checked gauges 1-7; LWD03_v2 (rerun); photogrammetry for end of run, regrade, and photogrammetry for Nov 26 taken	LWD03_v2
November 26, 2019	Checked gauges 1-7; LWD15_v1 (only 2 hours needed to reach equilibrium); no photogrammetry	LWD15_v1
November 27, 2019	photogrammetry for previous day and today; checked gauges 1-7; calibrated WG 7; LWD04_v2 (rerun);	LWD04_v2
November 28, 2019	photogrammetry ; calibrated WG 1-5; checked WG 6-7; LWD13_v1	LWD13_v1
November 29, 2019	photogrammetry ; calibrated WG 1-5; checked WG 6-7; LWD14_v1; regrade and photogrammetry for next days simulation.	LWD14_v1
December 2, 2019	checked WG 1-7; redo of LWD13 for 3D effects (LWD13_v2); regrade and photogrammetry for next days simulation.	LWD13_v2
December 3, 2019	checked WG 1-7; LWD19_v1 ; regrade and photogrammetry for next days simulation.	LWD19_v1
December 4, 2019	checked WG 1-7; LWD22_v1 ; no photogrammetry	LWD22_v1
December 5, 2019	Not in lab	N/A
December 6, 2019	Photogrammetry from previous day; FARO scanner; checked WG 1-7; LWD23_v1 ; no end-of-day photogrammetry	LWD23_v1
December 9, 2019	Photogrammetry from previous and current day; FARO scanner from previous day only; checked WG 1-6; recalibrated WG 7; LWD16_v1; photogrammetry	LWD16_v1
December 10, 2019	No photogrammetry; Checked WG 1-7; Run LWD24-27 (v1 and v2); installed 3 logs (mat), laser scan and photogrammetry for tomorrows run	LWD24 -27 (v1 & v2)
December 11, 2019	Checked WG 1-7; Run LWD28; middle log pushed up from 'pore' pressure; laser scan and photogrammetry (post and pre- for tomorrows run)	LWD28_v1
December 12, 2019	Checked WG 1-7; Run LWD29; laser scan (post) and photogrammetry (post and pre- for tomorrows run)	LWD29_v1

Day	Task/Details	Test Ref.
December 13, 2019	Checked WG 1-7; Run LWD30_v1; no laser scan or photogrammetry	LWD30_v1
December 16, 2019	Photogrammetry from previous day; Checked WG 1-7; Run LWD28_v2 -> logs moved -> stopped after 1 hour, no photogrammetry; LWD35-38; re-zeroed wave gauges after LWD37a_v1; photogrammetry for following day complete	LWD28_v2; LWD35 - 38 (v1 & v2)
December 17, 2019	checked WG 1-7; LWD28_v3 (LWD re-fixed after 1st run); no photogrammetry at end-of-day	LWD28_v3
December 18, 2019	photogrammetry for previous day; checked WG 1-7; LWD31 - 34 (v1 & v2); no photogrammetry for run-up runs	LWD31 - 34 (v1 & v2)
January 6, 2020	Mixed gravel in flume; installed 'bench'; photogrammetry; checked wave gauges 1-7; LWD41_v1;	LWD41_v1
January 7, 2020	Photogrammetry from previous day; Checked WG 1-7; Run LWD40_v1 - added a 4th hour;	LWD40_v1
January 8, 2020	Photogrammetry from previous day; Checked WG 1-7; Run LWD39_v1 - added a 5th hour;	LWD39_v1
January 9, 2020	Photogrammetry from previous day; Checked WG 1-7; Run LWD42 - 45 (v1 & v2)	LWD42-45 (v1 & v2)
January 10, 2020	Installed matrix; Photogrammetry; Checked WG 1-7; Run LWD48_v1	LWD48_v1
January 13, 2020	Photogrammetry from previous day; Checked WG 1-7; Run LWD47_v1	LWD47_v1
January 14, 2020	Photogrammetry from previous day; Checked WG 1-7; Calibrated WG 7; Run LWD46_v1	LWD46_v1
January 15, 2020	Photogrammetry from previous day; Checked WG 1-7; Run LWD49 - 52 (v1 & v2); 51 (v3) run since v1 AWA was not turned on; set-up for 'stacked' configuration and overtopping measurements; photogrammetry for following day	LWD49-52 (v1 & v2), 52 (v3)
January 16, 2020	Tuned and Calibrated OT 1 & 2; Checked WG 1-7; Run LWD55_v1	LWD55_v1
January 17, 2020	Photogrammetry from previous day; Checked WG 1-7 and OT 1-2; Run LWD54_v1	LWD54_v1
January 20, 2020	Photogrammetry from previous day; Checked WG 1-7; Run LWD53_v1	LWD53_v1
January 21, 2020	Photogrammetry from previous day; installed seawall; photogrammetry; Checked WG 1-7 & OT 1-2; Run LWD58_v1	LWD58_v1
January 22, 2020	Photogrammetry; Checked WG 1-7, Calibrated OT 1-2 -> OT 1 error likely higher than OT 2; Run LWD57_v1	LWD57_v1
January 23, 2020	Photogrammetry; Checked WG 1-7, Checked OT 1-2 -> OT 1 error 3mm; Run LWD56_v1	LWD56_v1
January 24, 2020	Photogrammetry from previous day; remove seawall and OT 1 & 2; install rough log; no-pre photogrammetry to save time; Checked WG 1-7; Run LWD59 - 63	LWD59_v1; LWD60 - 63 (v1 &v2)

Appendix F. Experimental Matrices

Complete experimental matrices for experiments completed primarily for beach morphology monitoring and wave run-up monitoring are provided in Table 18 and Table 19, respectively. Actual wave characteristics are reported based on WG5. For irregular waves, the listed wave height is the spectral significant wave height, H_{m0}, and the wave period is the associated peak wave period, T_p. For regular waves, the listed wave height is mean wave height, H_m, and the wave period is the mean wave period, T_m. For comparison purposes, the base case (Test 04-v2) is indicated with an asterisk (*).

Table 18 – Experimental run matrix for beach morphology testing

Test No.	LWD Configuration	Log Elevation	Spectra Type	Target Wave Characteristics		Actual Wave Characteristics			Comments
				Wave Height	Wave Period	Wave Height	Wave Period	Number of Waves	
[-]	[-]	[m]	[-]	H [m]	T [s]	H [m]	T [s]	N [-]	
01-v1	Single	0.0	Irregular	0.18	2.38	0.18	2.44	4860	N/A
02-v1	Single	0.0	Irregular	0.18	2.38	0.18	2.44	4790	N/A
03-v1	Single	0.0	Irregular	0.20	2.38	0.20	2.47	5570	Test duration too short - Repeated
03-v2	Single	0.0	Irregular	0.20	2.38	0.20	2.48	7410	N/A
04-v1	Single	0.0	Irregular	0.20	2.51	0.20	2.59	5370	Test duration too short - Repeated
04-v2*	Single	0.0	Irregular	0.20	2.51	0.20	2.64	7230	N/A
05-v1	Single	0.0	Irregular	0.15	2.38	0.15	2.42	5120	N/A
06-v1	Single	0.0	Irregular	0.15	2.17	0.15	2.13	5350	N/A
07-v1	Single	0.0	Irregular	0.10	2.38	0.11	2.42	5920	N/A
08-v1	Single	0.0	Irregular	0.10	1.78	0.11	1.77	7500	N/A
09-v1	Single	0.0	Irregular	0.20	2.80	0.20	2.80	6460	N/A
10-v1	Single	0.0	Irregular	0.20	3.00	0.20	3.02	7900	N/A
11-v1	Single	0.0	Regular	0.30	2.45	0.31	2.45	7350	Active wave absorption unreliable (AWA2 failed during simulation)
12-v1	Single	0.0	Regular	0.25	2.25	0.26	2.25	6400	N/A
13-v1	Single	0.0	Regular	0.20	2.01	0.19	2.00	7200	3D observations – Repeated
13-v2	Single	0.0	Regular	0.20	2.01	0.19	2.00	7200	Results unreliable
14-v1	Single	0.0	Regular	0.15	1.74	0.15	1.75	8220	N/A
15-v1	Single	0.0	Regular	0.10	1.42	0.10	1.42	5070	N/A
16-v1	None	N/A	Irregular	0.15	2.17	0.15	2.14	6020	N/A

Test No.	LWD Configuration	Log Elevation	Spectra Type	Target Wave Characteristics		Actual Wave Characteristics			Comments
				Wave Height	Wave Period	Wave Height	Wave Period	Number of Waves	
[-]	[-]	[m]	[-]	H [m]	T [s]	H [m]	T [s]	N [-]	
18-v1	None	N/A	Irregular	0.20	2.51	N/A	N/A	N/A	Wave generator stopped part way through drive signal - Repeated
18-v2	None	N/A	Irregular	0.20	2.51	0.20	2.61	5320	N/A
19-v1	Single	+0.1	Irregular	0.20	2.51	0.20	2.59	7090	N/A
20-v1	Single	0.0	Irregular	0.20	2.51	0.20	2.68	10820	N/A
21-v1	None	N/A	Irregular	0.10	1.78	0.10	1.78	4900	N/A
22-v1	Single	-0.1	Irregular	0.20	2.51	0.20	2.52	7200	N/A
23-v1	Single	-0.2	Irregular	0.20	2.51	0.20	2.61	7200	N/A
28-v1	Triple	N/A	Irregular	0.20	2.51	N/A	N/A	N/A	Significant log movement - Repeated
28-v2	Triple	N/A	Irregular	0.20	2.51	N/A	N/A	N/A	Log movement, run ended prematurely
28-v3	Triple	N/A	Irregular	0.20	2.51	0.20	2.58	7130	N/A
29-v1	Triple	N/A	Irregular	0.15	2.17	0.15	2.10	5980	N/A
30-v1	Triple	N/A	Irregular	0.10	1.78	0.11	1.78	7420	N/A
39-v1	Benched	N/A	Irregular	0.20	2.51	0.19	2.50	8940	N/A
40-v1	Benched	N/A	Irregular	0.15	2.17	0.15	2.17	8070	N/A
41-v1	Benched	N/A	Irregular	0.10	1.78	0.11	1.75	7640	N/A
46-v1	Matrix	N/A	Irregular	0.20	2.51	0.20	2.72	7450	N/A
47-v1	Matrix	N/A	Irregular	0.15	2.17	0.15	2.11	6170	N/A
48-v1	Matrix	N/A	Irregular	0.10	1.78	0.11	1.80	7440	N/A
53-v1	Stacked	N/A	Irregular	0.20	2.51	0.21	2.66	7140	N/A
54-v1	Stacked	N/A	Irregular	0.15	2.17	0.16	2.09	6070	N/A
55-v1	Stacked	N/A	Irregular	0.10	1.78	0.11	1.79	7570	N/A
56-v1	Seawall	N/A	Irregular	0.20	2.51	0.21	2.62	7170	N/A
57-v1	Seawall	N/A	Irregular	0.15	2.17	0.16	2.09	6160	N/A
58-v1	Seawall	N/A	Irregular	0.10	1.78	0.11	1.79	7650	N/A
59-v1	Single-Rough	0.0	Irregular	0.20	2.51	0.20	2.59	7230	N/A

*Note: * Denotes the base case simulation, 04_v2*

Table 19 – Experimental run matrix for wave run-up monitoring

Test No.	LWD Configuration	LWD Elevation	Spectra Type	Target Wave Characteristics		Actual Wave Characteristics			
						Duplicate Set A		Duplicate Set B	
				Wave Height H [m]	Wave Period T [s]	Wave Height H [m]	Wave Period T [s]	Wave Height H [m]	Wave Period T [s]
24	No LWD	N/A	Regular	0.10	1.42	0.10	1.4	0.10	1.4
25	No LWD	N/A	Regular	0.15	1.74	0.15	1.7	0.15	1.7
26	No LWD	N/A	Regular	0.20	2.01	0.20	2.0	0.20	2.0
27	No LWD	N/A	Regular	0.25	2.25	0.26	2.2	0.26	2.2
31	Triple	N/A	Regular	0.10	1.42	0.09	1.4	0.09	1.4
32	Triple	N/A	Regular	0.15	1.74	0.15	1.7	0.15	1.7
33	Triple	N/A	Regular	0.20	2.01	0.20	2.0	0.20	2.0
34	Triple	N/A	Regular	0.25	2.25	0.26	2.2	0.26	2.2
35	Single	0.0	Regular	0.10	1.42	0.10	1.4	0.10	1.4
36	Single	0.0	Regular	0.15	1.74	0.15	1.7	0.15	1.7
37	Single	0.0	Regular	0.20	2.01	0.20	2.0	0.20	2.0
38	Single	0.0	Regular	0.25	2.25	0.25	2.2	0.25	2.2
42	Benched	N/A	Regular	0.10	1.42	0.09	1.4	0.09	1.4
43	Benched	N/A	Regular	0.15	1.74	0.15	1.7	0.15	1.7
44	Benched	N/A	Regular	0.20	2.01	0.20	2.0	0.19	2.0
45	Benched	N/A	Regular	0.25	2.25	0.26	2.2	0.26	2.2
49	Matrix	N/A	Regular	0.10	1.42	0.09	1.4	0.09	1.4
50	Matrix	N/A	Regular	0.15	1.74	0.15	1.7	0.15	1.7
51	Matrix	N/A	Regular	0.20	2.01	0.20	2.0	0.20	2.0
52*	Matrix	N/A	Regular	0.25	2.25	0.27	2.2	0.26	2.2
60	Single - Rough	0.0	Regular	0.10	1.42	0.09	1.4	0.09	1.4
61	Single - Rough	0.0	Regular	0.15	1.74	0.15	1.7	0.15	1.7
62	Single - Rough	0.0	Regular	0.20	2.01	0.20	2.0	0.20	2.0
63	Single - Rough	0.0	Regular	0.25	2.25	0.26	2.2	0.25	2.2

* *Note: First simulation failed as AWA was not turned on. As such, tests 52_v2 and 52_v3 were used as duplicate sets A and B.*

Appendix G. Data Acquisition & Analysis Overview

NRC-OCRE developed an in-house suite of software to provide experimental controls, data acquisition, and data analysis. These software packages were used throughout the experimental program and are briefly described herein.

G.1. GDAC & NDAC – Data Acquisition Packages

The GDAC software package is an experimental control and data acquisition system, which was primarily used in this project for data acquisition and sensor calibration of the wave gauges and ADV. The NDAC software is an older version of GDAC, which was required for experimental control of the wave generator and active wave absorption system for the Steel Wave Flume. Using these software packages, data sampling was set to begin when the wave maker was activated. Sampling for the wave gauges and ADV were set to a rate of 50 Hz and 25 Hz, respectively. Both the GDAC and NDAC software convert the recorded analogue signals into binary form which can be read by the GEDAP data analysis system.

G.2. GEDAP – Data Analysis Package

The GDAP software package was used for (1) creation of drive signals for both random and irregular wave generation and active wave absorption, (2) real time monitoring of sensors, and (3) preliminary analysis of the wave gauge and ADV data.

The first 25 seconds of each sample was discarded as part of the data analysis process, since sampling began when the wave maker was activated and the waves take some time to propagate down the flume. For the wave gauge data, the GEDAP software package was also used to compute the variance spectral analysis (VSD), conduct a zero crossing analysis (ZCA), compute a basic set of statistics (STAT1), conduct a probability analysis (STAT5), complete a reflection analysis (using REFLC, REFLS, or REFLM), and copy files to ASCII format for ease of later analysis (V2_TO_ASCII). For the ADV data, GEDAP was used to compute basic statistics (STAT1) and export 3D velocities to ASCII format (V2_TO_ASCII).

G.3. Reflection Analysis in GEDAP

The reflection analysis for the project was largely conducted by visiting Master's student, P. Falkenrich, but is summarized and discussed here for reference. Using the NRC-

OCRE's GEDAP software package, users can complete a reflection analysis using a variety of programs, including the following:

- REFLC – Automatically selected the best three wave probes from a five-probe array, performs a cross-spectral analysis, and conducts a reflection analysis based on the incident and reflected wave spectrums for irregular waves.
- REFLS – User specifies three wave probes and the program computes the Fourier transform (based on linear wave theory) for the incident and reflected waves inside a specified frequency domain Reflection analysis for irregular waves, based on the method outlined by Mansard & Funke (1980).
- REFLM – Similar to REFLS, but for regular waves, based on the method outlined by Mansard & Funke (1980).

Depending on the program, various parameters can be output, including the average reflection coefficient, C_r, which is defined as follows. Where σ_r and σ_i are the standard deviations for the reflected wave train and incident wave train, respectively.

$$\text{Reflection Coeffeicient:} \qquad \boldsymbol{C_r = \sigma_r/\sigma_i} \tag{44}$$

For regular waves, both REFLS and REFLM are expected to generate similar values for the Reflection Coefficient since both programs are based off the same method (Mansard & Funke, 1980). In theory, REFLS and REFLC should also generate similar values for irregular waves. Values calculated using both methods are reported in Falkenrich (2020); however, for the purpose of this research, REFLS was used to estimate the reflection coefficient.

Estimated reflection coefficients were generally between 8 – 25% for gravel beaches with no structures, between 14 – 23% for a single LWD structure, and between 18 – 28% for a stacked log wall. This was generally slightly higher than Atladottir (2008) which reported values of 15-17% for a similar experimental set-up with no structures under similar wave conditions.

Appendix H. Beach Profile Measurement Methods

The 3D beach surface was captured before and after each test using a digital photogrammetry technique. Because this technique is relatively novel, a FARO terrestrial laser scanner was also utilized to capture the beach surface before and after several runs to compare the accuracy of the techniques. The downside of both the photogrammetry and FARO laser scanner techniques is that they cannot capture the beach surface during the tests when water is in the flume. Instead, a time-lapse camera was set-up outside of the flume to capture profile changes on a 1-minute interval throughout the duration of each run. In this way temporal changes and beach equilibrium could be continuously monitored.

The three techniques employed for this project are described herein and a comparison of their accuracy is presented for two simulations.

H.1. Photogrammetry Technique

A Nikon D5100 camera (CAM3) was used to take between 150-200 photographs from various orientations and distances in order to photogrammically reconstruct the beach surface before and after each test. Photograph density was increased in the vicinity of complex beach features (e.g. scour holes) or structures (e.g. LWD) in order to better resolve these features. The Agisoft Metashape Professional (Metashape) was used to construct a point cloud from the photographs. Scaling and conversion to the project's coordinate system was made possible by using both stationary markers and mobile markers, which were then referenced in Metashape. The stationary markers were fixed to the walls of the flume in various orientations for the duration of the experiments and were surveyed using a total station to obtain their precise positions. Additional mobile markers were placed directly on the beach surface before the photographs were taken and removed prior to re-filling the flume or re-grading the beach surface (Figure 49). A mesh-surface was created from the point cloud in CloudCompare, using the Poisson Reconstruction method (Figure 50). A total of 47 beach transects / profiles were exported from the surface (on a 0.025m interval) and then averaged to smooth local anomalies and remove any wall effects. This process resulted in a single averaged beach profile before and after each test.

Figure 49 – Example of fixed markers attached to the flume's sidewall at known locations and mobile markers placed on the beach surface prior to taking photogrammetry photos

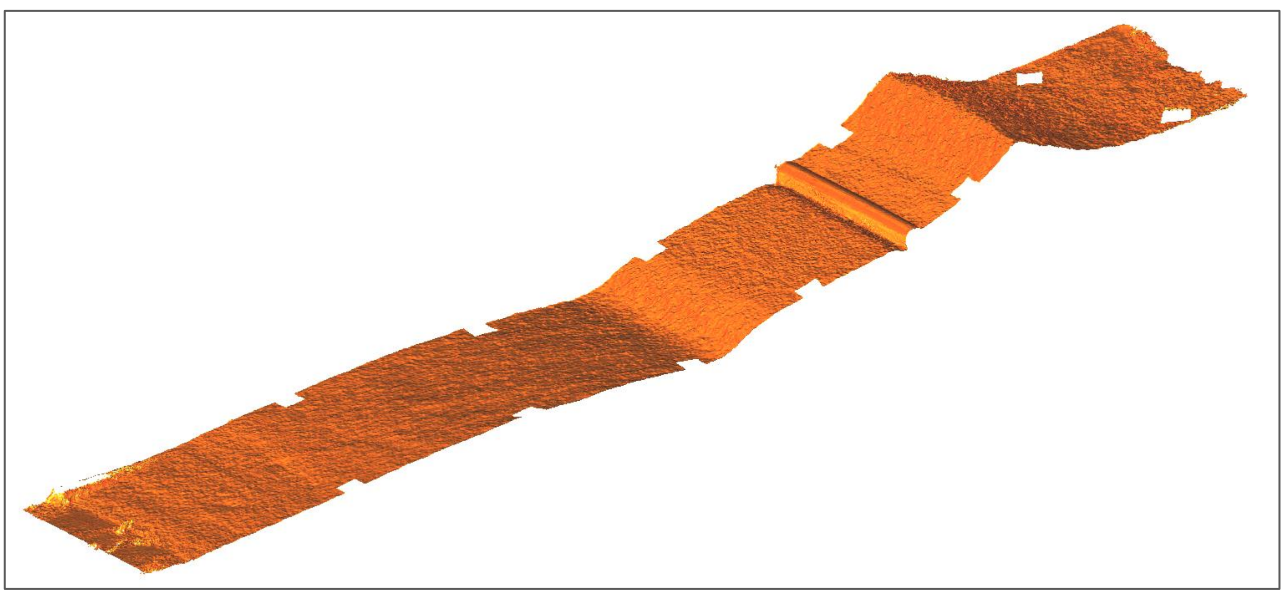


Figure 50 – Example of photogrammically reconstructed mesh surface for test 59-v1

H.2. FARO Laser Scanner

The FARO laser scanner (FARO Focus3D S 120) was utilized to capture the 3D beach surface before and after two simulations, Test 23-v1 and Test 29-v1. Two scans were taken in order to develop each surface, one scan from the beach toe and one scan from the beach crest. Space at the toe of the beach was severely constricted due to the close placement of existing instrumentation inside the flume, which limited placement options for the scanner (Figure 51). Scans were completed for a narrowed field of view (rather than a 360^{o} rotation), in black and white, and with approximately 3,000,000 – 4,000,000 points across the beach surface. Notably the total number of points is comparable to a typical 3,500,000 points for the photogrammetry technique; however, unlike the photogrammetry technique, the density of points from the FARO laser scanner technique was heavily biased to the crest and toe of the slope, closest to the two positions where the scanner was positioned (Figure 16). A similar process as the photogrammetry technique was used to create a mesh-surface, export transects, and average transects to create single smoothed beach profiles.

Figure 51 – FARO laser scanner set-up at the toe of the beach after draining the flume

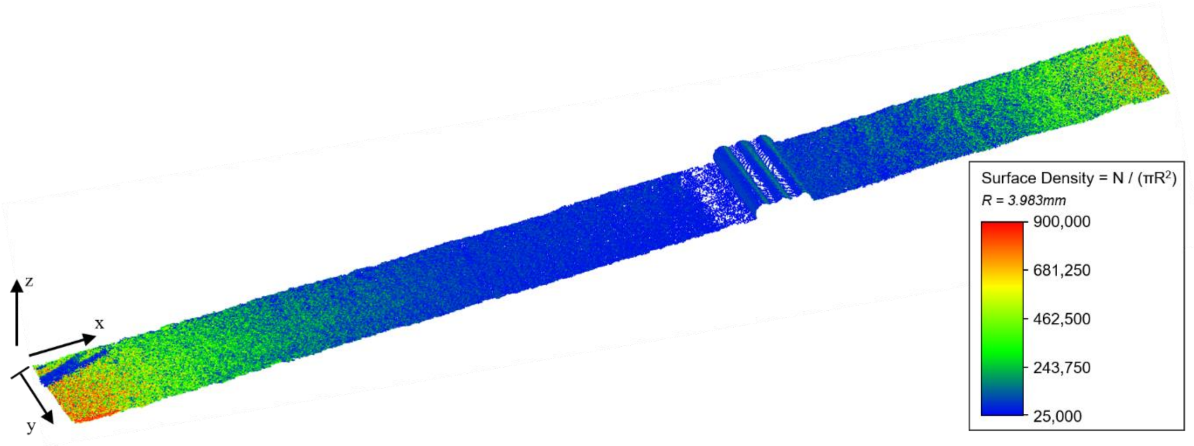


Figure 52 – Surface point density for the original beach profile for test 29-v1, showing a higher point density near the toe and crest of the beach, and a low point density in the center of the beach near and in front of the LWD structure.

H.3. Digitized Profile Photos

An additional Nikon D5300 camera (CAM1) was set-up at a stationary location outside of the flume to capture time-lapse photos of the beach profile change along the flume's glass wall. Due to space restrictions outside of the flume, the camera had to be set-up at an angle to the flume to capture the entire beach profile. Photographs were taken on a 1-minute interval throughout each test. Similarly, to the stationary markers used in the photogrammetry method, small markers were placed on the outside of the flume and surveyed using a total station to obtain their precise positions. This allowed photographs to be imported into ESRI ArcMap (Version 10.7.1) and rectified using the Projective Transformation technique based on the markers' locations. Beach profiles were then manually digitized on a 10-minute interval for every test and exported as points with a 0.005m spacing along each profile. A visualization of this process is shown in Figure 53 and an example of the final result of this process (e.g. beach profiles along the flume-wall on a 10-minute interval) is included in Figure 54 for test 20-v1.

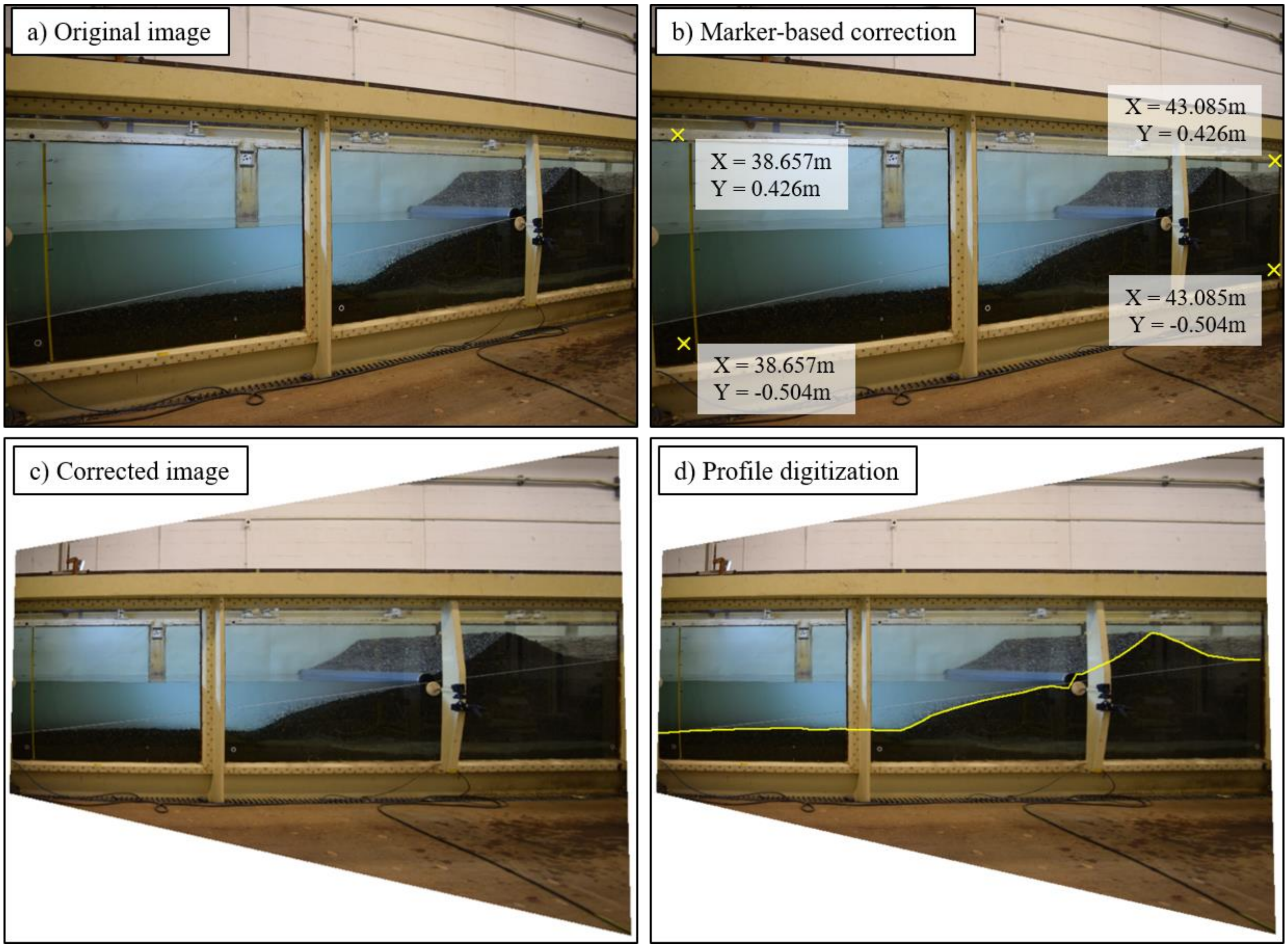


Figure 53 – Visualization of the process used to rectify and digitize beach profile photos

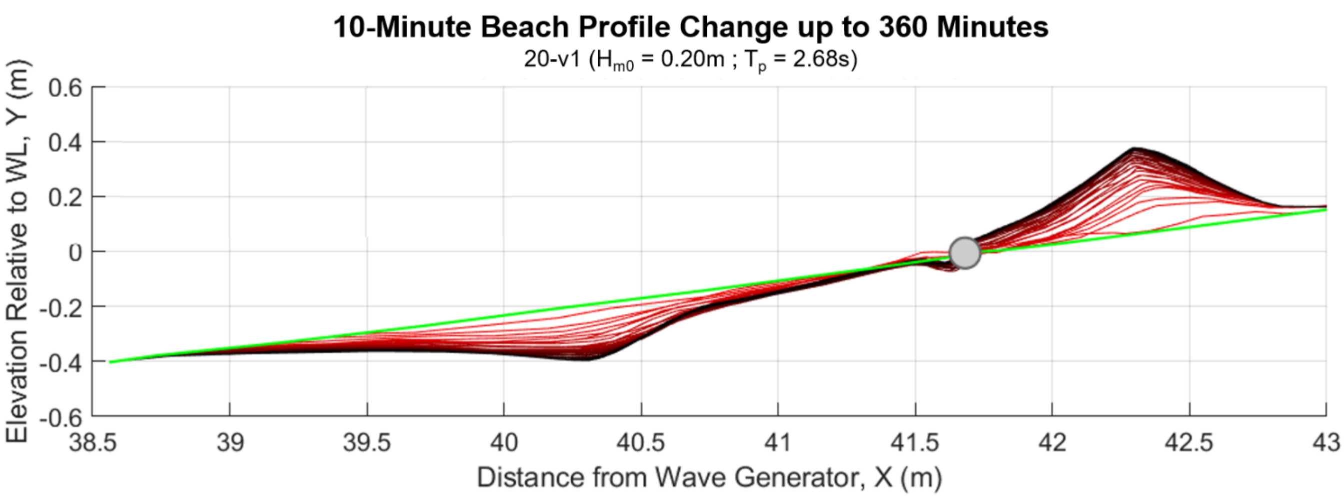


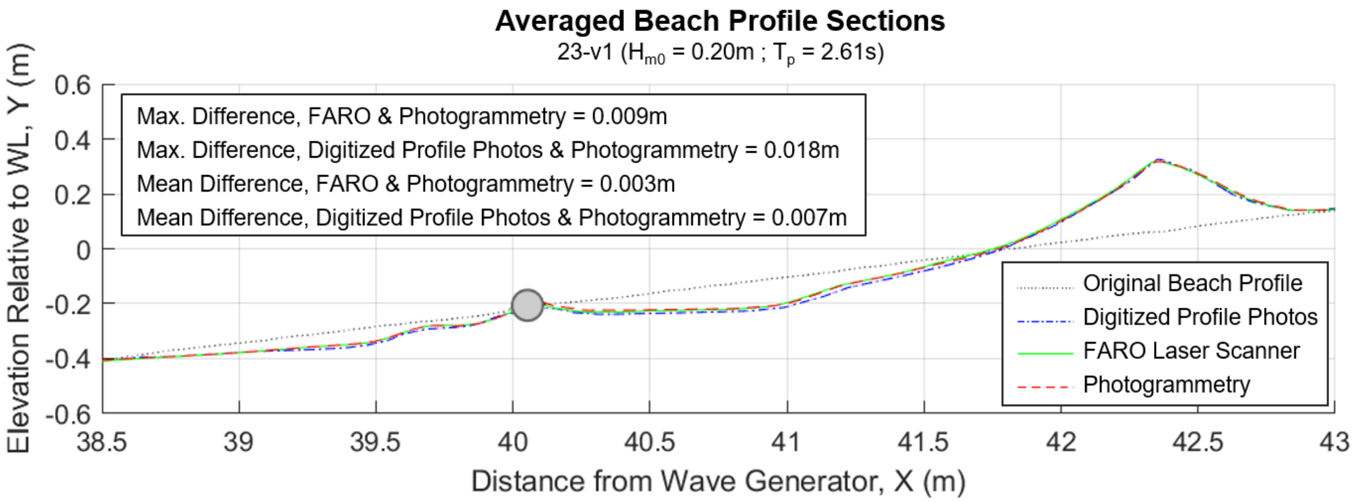
Figure 54 – Example of beach profiles digitized on a 10-minute interval for a total test duration of 360 minutes (Test 20-v1)

H.4. Comparison of Techniques

A comparison of the beach profiles obtained as a result of all three different methods is compared for Test 23-v1 and Test 29-v1 in Figure 55 and Figure 56, respectively. Note that in both figures, the top plots compare the mean profiles from the photogrammetry and FARO techniques to the digitized profile photo technique along the side wall, whereas the bottom plots compare the profile near the sidewall from the photogrammetry and FARO techniques directly to the digitized profile photo technique; in this way, the technique error can be differentiated from any error due to wall-effects.

The photogrammetry and FARO laser scanner had a mean difference between 0.002-0.003 m and a maximum difference between 0.009-0.026 for the post-simulation beach profiles of Test 23-v1 and 29-v1. Notably the largest differences were located near the log structure and on relatively steep slopes located in the middle of the beach profile. A close-up inspection of the results showed that the photogrammetry method more realistically captures features near the log, such as the scour hole, since the camera positioning could be adjusted to increase resolution near the more complex formations. In addition, on steep slopes in the middle of the profile, the FARO scanner is located farther away and provides a low density of points, therefore becoming less reliable. As such, the more novel photogrammetry technique was deemed to provide both more reliable beach profile measurements than the FARO technique and sufficient accuracy for the purpose of this project.

The digitized beach profiles resulted in a mean difference of 0.007m and a maximum difference between 0.018-0.033 when compared to the averaged profile for photogrammetry technique for the post-simulation beach profiles of Test 23-v1 and 29-v1. However, when compared to the profile along the sidewall using the photogrammetry technique, the digitized beach profiles resulted in a mean difference of between 0.005-0.006 m and a maximum difference between 0.018-0.023. It should be noted, however, that the photogrammetry technique is also less reliable along the sidewall. This analysis indicates that the digitized beach profile technique is a more accurate representation of the profile along the sidewall of the flume than the average profile, since wall effects captured in the digitized beach profiles decrease the relatively accuracy relative to the averaged beach profiles. The digitized beach profile technique was therefore used as an indicator of profile evolution for this project, but was not be relied upon for absolute values of profile changes for each test.



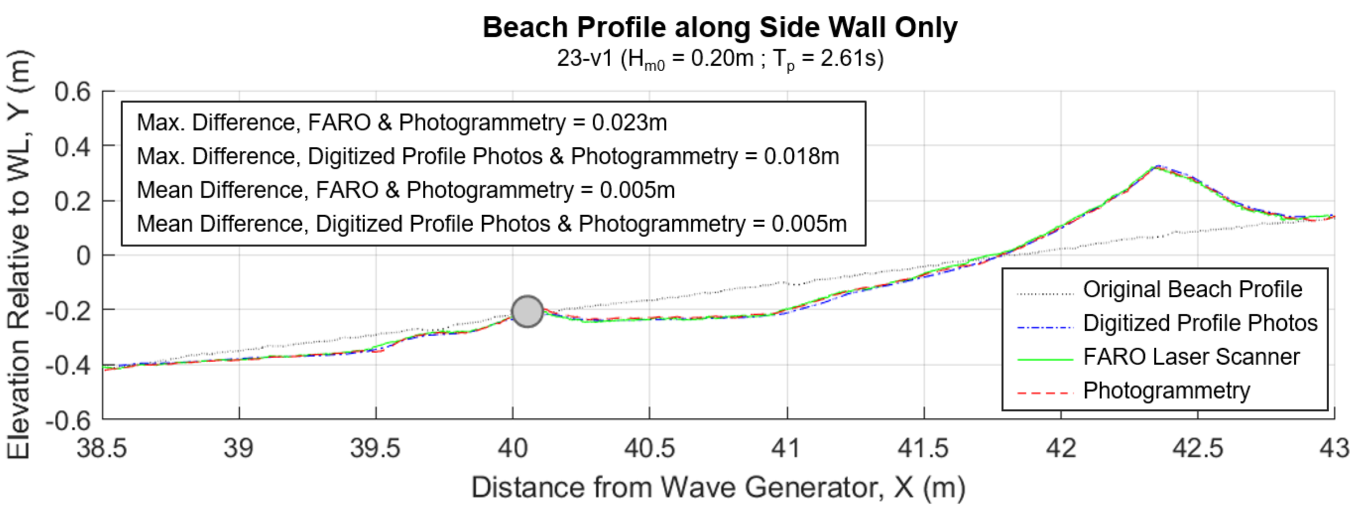


Figure 55 – Comparison of three measurement techniques for Test 23-v1. (Top) Digitized profile photos along the sidewall and averaged beach profiles across the flume width from surfaces based on the FARO laser scanner and photogrammetry techniques. (Bottom) Digitized profile photos along the sidewall and single beach profiles along the sidewall from surfaces based on the FARO laser scanner and photogrammetry techniques.

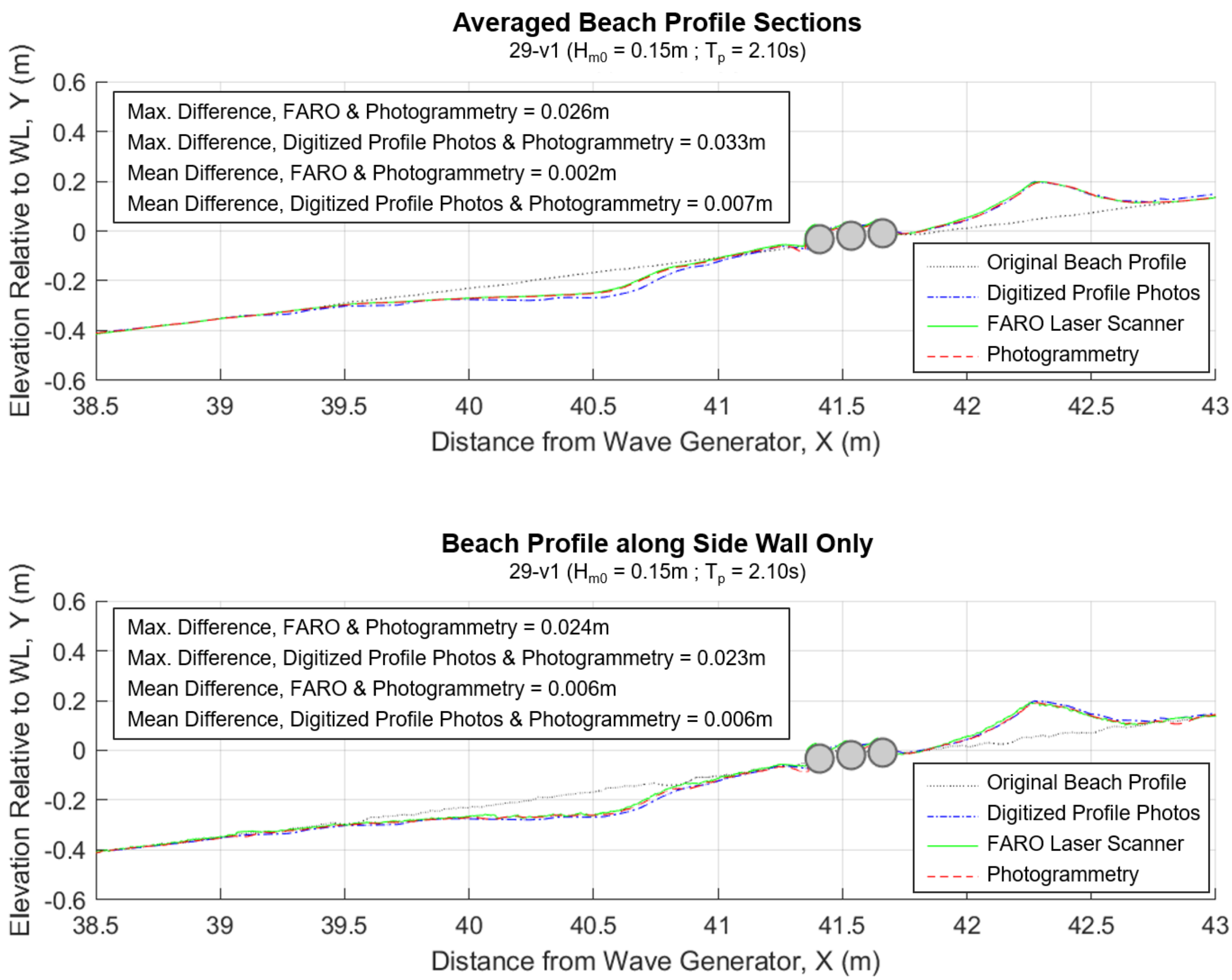


Figure 56 – Comparison of three measurement techniques for Test 29-v1.
(Top) Digitized profile photos along the sidewall and averaged beach profiles across the flume width from surfaces based on the FARO laser scanner and photogrammetry techniques.
(Bottom) Digitized profile photos along the sidewall and single beach profiles along the sidewall from surfaces based on the FARO laser scanner and photogrammetry techniques.

Appendix I. Test Duration & Beach Equilibrium

I.1. Beach Profile Characteristics

Several variables were defined in order to characterize and compare key beach profile characteristics (Figure 57). dh_{max} and dh_{min} are the elevations of maximum accretion (at the beach crest) and maximum erosion (at the beach trough), respectively, measured vertically relative to the original beach profile. X_{crest} and Z_{crest}, and X_{trough} and Z_{trough} are the horizontal and vertical locations of dhmax and dhmin, respectively. V_{crest} and V_{trough} are the volume changes (m^3 per m width of flume) measured on the upper and lower slopes, respectively, relatively to the original beach profile. The upper and lower beach slope were delineated at X = 41.65m, which is approximately the center location of LWD placement. The average of these two volume measurements, V_{avg}, is used as an indicator of the magnitude of sediment transport throughout each test.

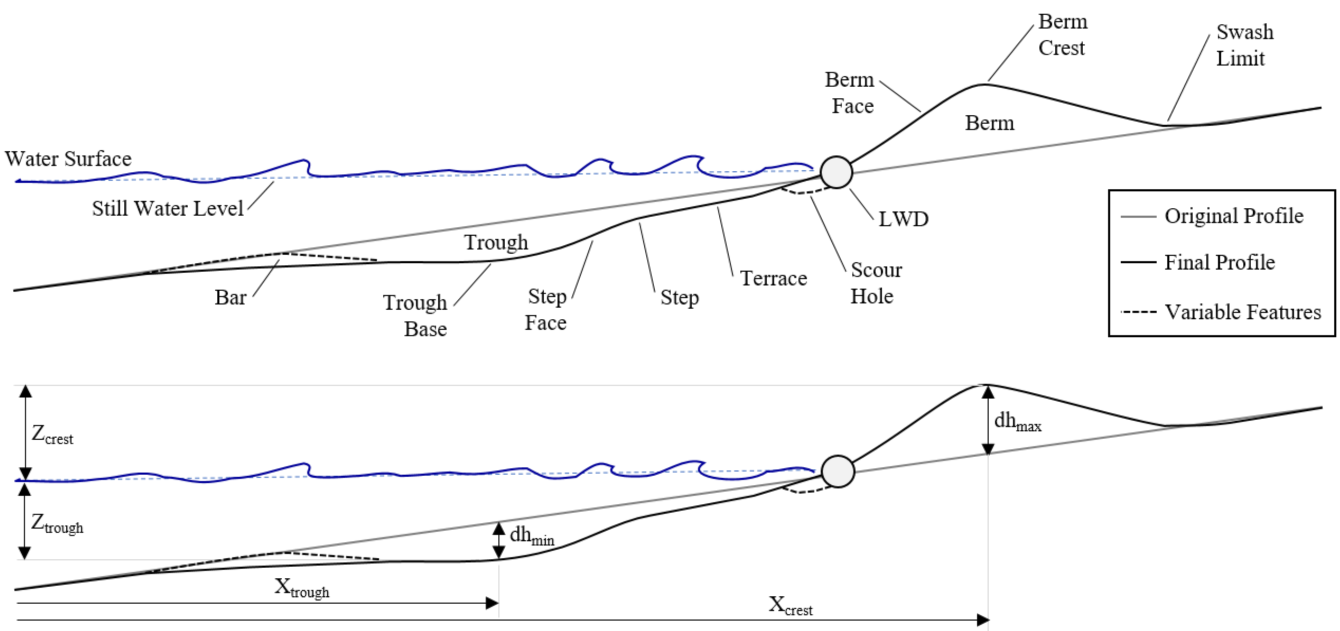


Figure 57 – Key beach profile characteristics and variables used for this study

I.2. Influence of Test Duration

To assess the influence of duration on beach morphology, the base case scenario (single LWD with target wave characteristics, H_s = 0.2 m and T_p = 2.51 s) was tested under three different durations, as follows:

- Test 04-v1: 3 hours (N = 5,370)
- Test 04-v2: 4 hours (N = 7,230)
- Test 20-v1: 6 hours (N = 10,820)

The final beach profiles for each of these tests are shown in Figure 58. After three hours of testing and 5,370 waves (test 04-v1), 89% of the total berm height and 88% of the total trough depth measured after 6 hours and 10,820 waves (test 20-v1) were observed. After four hours of testing and 7,230 (test-04-v2), 96% of the total berm height and 95% of the total trough depth after 6 hours were observed.

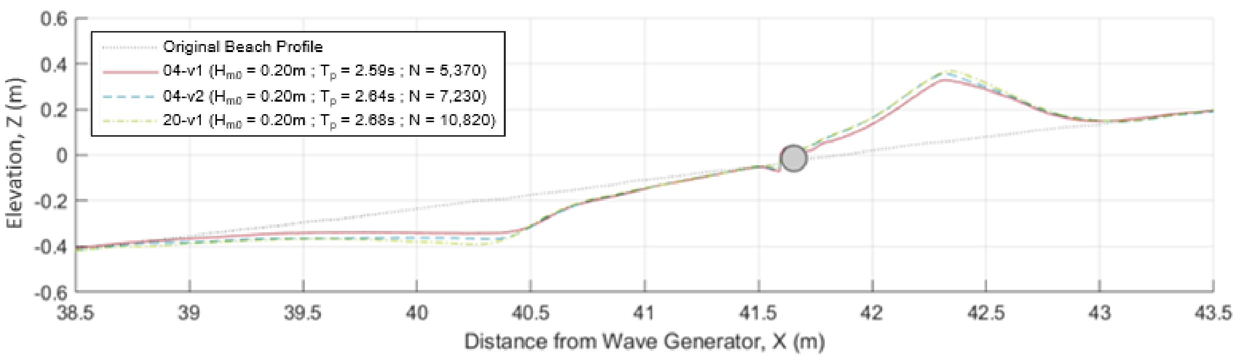


Figure 58 – Final beach profiles for tests 04-v1, 04-v2, and 20-v1 comparing the influence of test duration

In addition, during the first three hours of testing, the average rate of elevation change at the berm and trough was 0.062 mm/wave. Between hours three to four (test 04-v1 & 04-v2), the average rate of change decreased 77% to 0.014 mm/wave. Between hours four to six (test 04-v2 & 20-v1), the average rate of change decreased another 67% to only 0.005 mm/wave. Based on this information, it can be observed that the rate of change was highly non-linear, decreasingly substantially over time.

The average volume of material that moved from the lower to upper slope, V_{avg}, was 0.169, 0.197, and 0.211 m^3/m for tests 04-v1, 04-v2, and 20-v1, respectively. Similarly, the elevation increase at the berm crest, dh_{max}, was 0.262, 0.298, and 0.310 m and elevation decrease at the beach trough, dh_{min}, was -0.154, -0.173, and -0.193 m for tests 04-v1, 04-v2, and 20-v1, respectively. The rate of change for these two parameters also decreased in a non-linear fashion as duration increased. The horizontal location of the berm crest stayed at approximately the same location; however, the trough continued to move offshore as the number of waves increased.

I.3. Beach Equilibrium

Tests were generally run until it was visually observed that little to no changes in the beach profiles were occurring. Generally, this meant tests were run for between 3 – 4 hours. However, some tests were run for longer durations to monitor the influence of test duration (e.g. test 20-v1) or to ensure that beach equilibrium was met (e.g. tests 53-v1 and 56-v1). A number of the first tests were also run for slightly shorter durations before a more rigid protocol regarding test length was put in place (e.g. test 01-v1, 02-v1, and 21-v1).

In addition to a visual check that the beach profile was no longer changing, a more rigorous process was also used to systematically check that beach equilibrium was achieved after every test. Because previous experimental studies on gravel beach morphology did not include or detail systematic checks and thresholds for beach equilibrium (eg. Atladottir, 2008; de San Román-Blanco et al., 2006; Frandsen et al., 2015; Kobayashi et al., 2011; Lee et al., 2007; Masselink & Turner, 2012; Van Der Meer, 1988; van Hijum & Pilarczyk, 1982), a new method was developed for this study.

The beach profile evolution was monitored for every test by digitizing time-lapse photos taken of the beach profile from outside of the flume on a 10-minute interval (see Appendix H.3 for a discussion of this technique). The maximum 10-minute elevation change, $|dh|$, was determined and plotted over time for each test. Notably, because the areas directly in front of and behind the structures were subject to rapid scouring or deposition from individual waves, these areas were not included in the analysis.

A threshold was set such that once the maximum $|dh|$ remained below the threshold for 30 consecutive minutes (three 10-minute intervals), the profile was considered to have reached beach equilibrium. Because the total profile change was found to be highly dependent on wave height, three different thresholds for beach equilibrium were set related to the measured wave height:

- For 0.08 m ≤ H < 0.13 m, the beach equilibrium threshold = 0.010 m
- For 0.13 m ≤ H < 0.18 m, the beach equilibrium threshold = 0.015 m
- For 0.18 m ≤ H , the beach equilibrium threshold = 0.020 m

The plotted maximum |dh| and relevant thresholds for tests 04-v2, 06-v1, and 08-v1 are shown in Figure 59. Although all three profiles in Figure 59 successfully reached beach equilibrium prior to the end of the simulation, this analysis resulted in two tests (test 03-v1 and 04-v1) being re-done with longer durations in order to reach beach equilibrium.

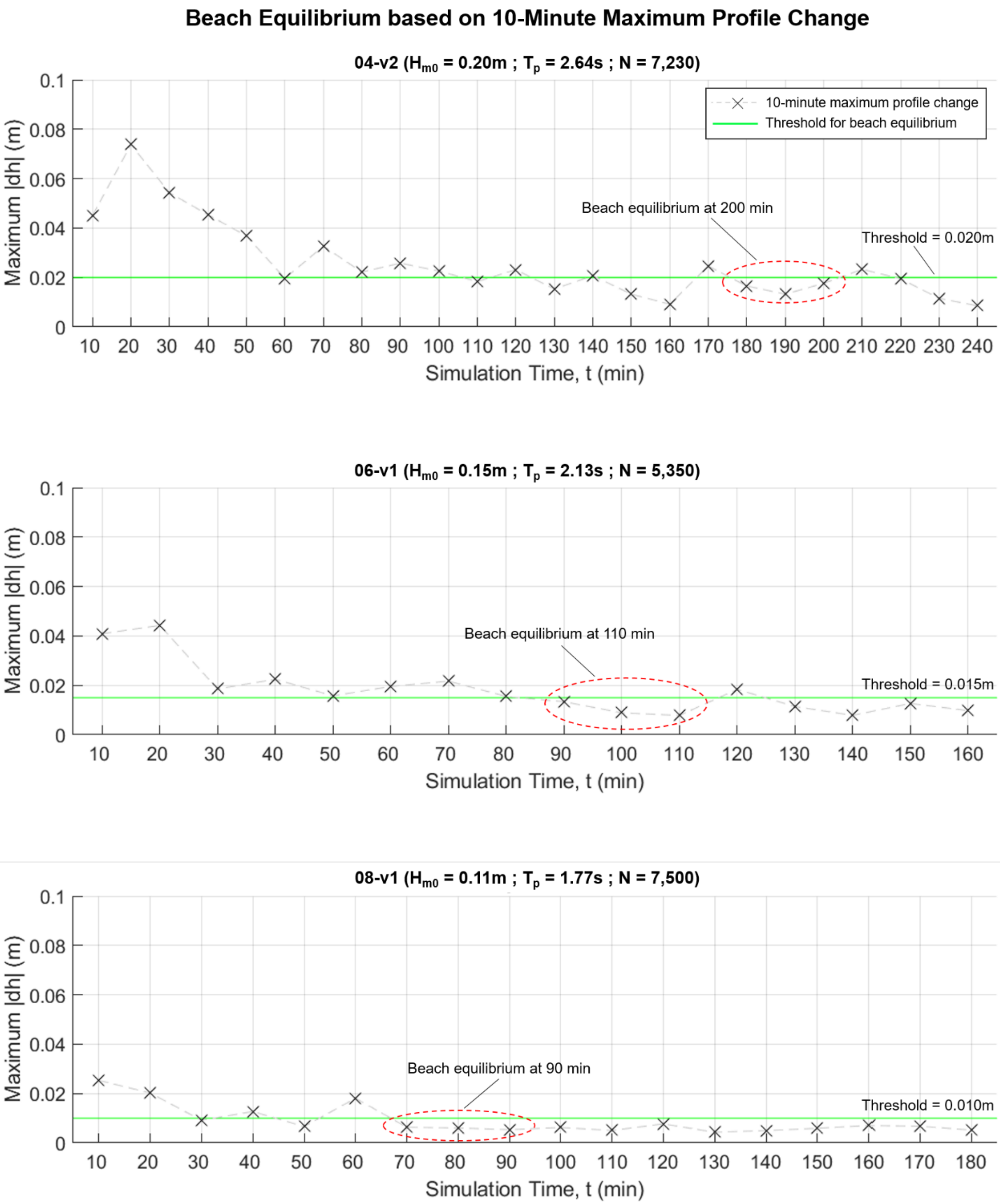


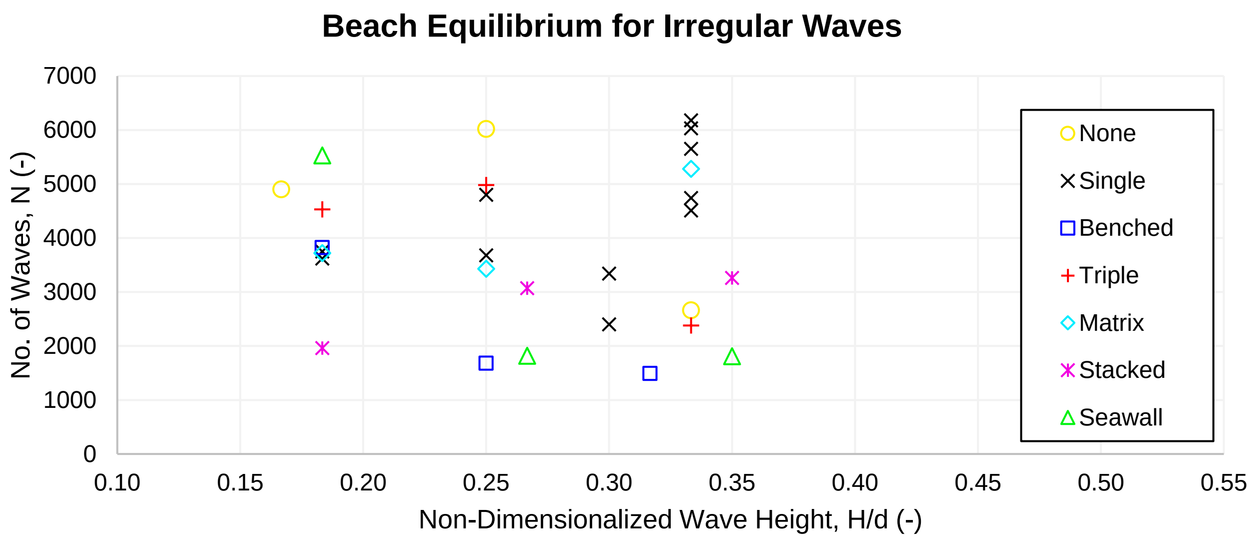
Figure 59 – Example of beach equilibrium thresholds and times for test 04-v2, 06-v1, and 08-v1

The number of waves required to reach beach equilibrium for both irregular and regular wave tests are plotted in Figure 60. A summary of the duration and equivalent number of waves required for beach equilibrium are also provided in tabular form in Table 20.

For irregular waves, beach equilibrium was achieved between 1500 – 6200 waves. This is consistent with most previous studies of irregular waves on mildly sloping gravel beaches, which generally found that beach equilibrium occurred between 3000 - 6000 waves (Atladottir, 2008; Kobayashi et al., 2011; van Hijum & Pilarczyk, 1982) or 1000 – 3000 waves (de San Román-Blanco et al., 2006; Masselink & Turner, 2012; Van Der Meer, 1988). Frandsen et al. (2015) found a much longer time for beach equilibrium (approximately 10,000 waves); however, they tested a mixed sand/gravel/cobble beach and only measured the beach profile after 2400 and 9800 waves, finding that beach equilibrium had only been met after the later of the two measurements.

For regular waves, beach equilibrium was generally achieved between 2500 – 4000 waves, with the exception of test 11-v1 (H = 0.31 m ; T = 2.45 s) which took approximately 6400 waves to reach beach equilibrium. Results from test 11-v1 may, however, be unreliable due to one the AWA wave gauges coming loose and potentially effecting active wave absorption during the simulation. These results are generally higher than previous studies, which found that beach equilibrium was achieved after an estimated 1000 – 1800 waves (Lee et al., 2007) and 2500 waves (Atladottir, 2008).

Note that it is difficult to compare beach equilibrium results between previous experiments due to a lack of continuity between test conditions (e.g. wave spectra, wave height, wave period, beach slope, and sediment diameter) and the beach profile measurement intervals, as well as a lack of discussion regarding the metrics used for defining beach equilibrium.



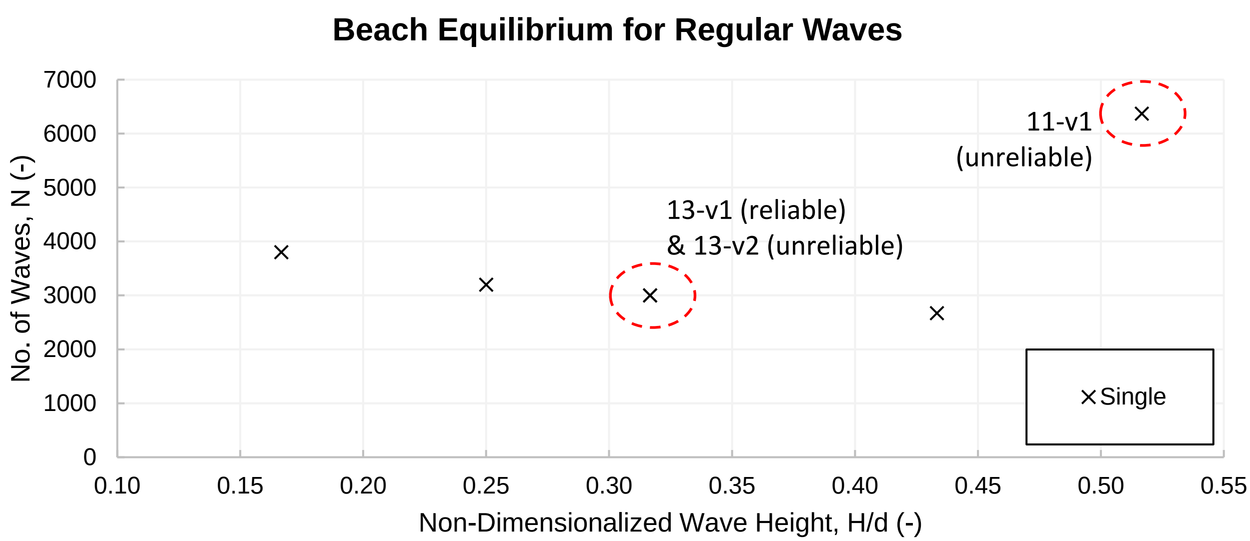


Figure 60 – Estimated number of waves required to reach beach equilibrium for (top) irregular and (bottom) regular waves.
Note that the results of tests 11-v1 and 13-v2 are not fully reliable due to complications during testing.

Table 20 – Beach equilibrium durations and equivalent number of waves required to meet beach equilibrium based on the maximum profile changes over a 10-minute interval. Tests highlighted in red are potentially unreliable due to problems during the experiments.

Test No.	LWD Configuration	Log Elevation	Spectra Type	Actual Wave Characteristics (WG5)		Actual Test Duration		Beach Equilibrium Criteria Met	
				Wave Height, H	Wave Period, T	Time, t	No. of Waves, N	Time, t_{eqlb}	Equivalent No. of Waves, N
[-]	[-]	[m]	[-]	[m]	[s]	[min]	[-]	[min]	[-]
01-v1	Single	0.0	Irregular	0.18	2.44	160	4860	110	3340
02-v1	Single	0.0	Irregular	0.18	2.44	160	4790	80	2400
03-v2	Single	0.0	Irregular	0.20	2.48	240	7410	200	6180
04-v2*	Single	0.0	Irregular	0.20	2.64	240	7230	200	6030
05-v1	Single	0.0	Irregular	0.15	2.42	160	5120	150	4800
06-v1	Single	0.0	Irregular	0.15	2.13	160	5350	110	3680
07-v1	Single	0.0	Irregular	0.11	2.42	180	5920	110	3620
08-v1	Single	0.0	Irregular	0.11	1.77	180	7500	90	3750
09-v1	Single	0.0	Irregular	0.20	2.80	240	6460	210	5650
10-v1	Single	0.0	Irregular	0.20	3.02	300	7900	180	4740
11-v1	Single	0.0	Regular	0.31	2.45	300	7350	260	6370
12-v1	Single	0.0	Regular	0.26	2.25	240	6400	100	2670
13-v1	Single	0.0	Regular	0.19	2.00	240	7200	100	3000
13-v2	Single	0.0	Regular	0.19	2.00	240	7200	100	3000
14-v1	Single	0.0	Regular	0.15	1.75	180	8220	70	3200
15-v1	Single	0.0	Regular	0.10	1.42	120	5070	90	3800
16-v1	None	N/A	Irregular	0.15	2.14	180	6020	180	6020
18-v2	None	N/A	Irregular	0.20	2.61	180	5320	90	2660
19-v1	Single	+0.1	Irregular	0.20	2.59	240	7090	180	5320
20-v1	Single	0.0	Irregular	0.20	2.68	360	10820	150	4510
21-v1	None	N/A	Irregular	0.10	1.78	120	4900	120	4900
22-v1	Single	-0.1	Irregular	0.20	2.52	240	7200	70	2100
23-v1	Single	-0.2	Irregular	0.20	2.61	240	7200	90	2700
28-v3	Triple	N/A	Irregular	0.20	2.58	240	7130	80	2380
29-v1	Triple	N/A	Irregular	0.15	2.10	180	5980	150	4980
30-v1	Triple	N/A	Irregular	0.11	1.78	180	7420	110	4530
39-v1	Benched	N/A	Irregular	0.19	2.50	300	8940	50	1490
40-v1	Benched	N/A	Irregular	0.15	2.17	240	8070	50	1680
41-v1	Benched	N/A	Irregular	0.11	1.75	180	7640	90	3820
46-v1	Matrix	N/A	Irregular	0.20	2.72	240	7450	170	5280

Test No.	LWD Configuration	Log Elevation	Spectra Type	Actual Wave Characteristics (WG5)		Actual Test Duration		Beach Equilibrium Criteria Met	
				Wave Height, H	Wave Period, T	Time, t	No. of Waves, N	Time, t_{eqlb}	Equivalent No. of Waves, N
[-]	[-]	[m]	[-]	[m]	[s]	[min]	[-]	[min]	[-]
47-v1	Matrix	N/A	Irregular	0.15	2.11	180	6170	100	3430
48-v1	Matrix	N/A	Irregular	0.11	1.80	180	7440	90	3720
53-v1	Stacked	N/A	Irregular	0.21	2.66	240	6520	120	3260
54-v1	Stacked	N/A	Irregular	0.16	2.09	180	6130	90	3070
55-v1	Stacked	N/A	Irregular	0.11	1.79	180	5050	70	1960
56-v1	Seawall	N/A	Irregular	0.21	2.62	240	7240	60	1810
57-v1	Seawall	N/A	Irregular	0.16	2.09	180	4090	80	1820
58-v1	Seawall	N/A	Irregular	0.11	1.79	180	7650	130	5530
59_v1	Single-Rough	0.0	Irregular	0.20	2.59	240	7230	170	5120

Appendix J. Morphology Results

J.1. Repeatability

Two simulations (test 01-v1 and 02-v1) were conducted with the identical wave signal and flume set-up to test repeatability. The overall beach shape closely matched for both tests. The mean vertical difference over the entire beach profile was 8.6 mm, with a standard deviation of 6.2 mm. Comparatively, this is the equivalent difference of one typical sediment grain given a median grain size, D_{50}, of 7.9 mm. The measured profiles had a maximum difference of 45.1 mm, located on the beach berm face where there is a steep change in elevation.

Measured wave characteristics and key profile characteristics are reported in Table 21. The absolute difference and percent differences are defined as follows:

$$\boldsymbol{Absolute\ Difference} = \ |\boldsymbol{A_1} - \boldsymbol{A_2}| \qquad (45)$$

$$\boldsymbol{Percent\ Difference} = \frac{|\boldsymbol{A_1} - \boldsymbol{A_2}|}{\left[\frac{\boldsymbol{A_1}+\boldsymbol{A_2}}{\boldsymbol{2}}\right]} \times \mathbf{100}\% \qquad (46)$$

The wave characteristics (H_{m0} and T_p) were within 1%. The maximum erosion and maximum accretion, dh_{max} and dh_{min}, were within 0.8% and 5.7%, respectively. Additionally, the volume change on the upper and lower slopes were closely matched between the two tests, within 1.3% and 0.7%, respectively.

Table 21 – Comparison of beach profile characteristics for repeated tests 01-v1 and 02-v1

Test No.	Actual Wave Characteristics (WG5)		Berm Characteristics				Trough Characteristics			
	H_{m0}	T_p	dh_{max}	Z_{crest}	X_{crest}	V_{crest}	dh_{min}	Z_{trough}	X_{trough}	V_{trough}
[-]	[m]	[s]	[m]	[m]	[m]	[m^3/m]	[m]	[m]	[m]	[m^3/m]
01-v1	0.185	2.440	0.233	0.276	42.121	0.151	-0.119	-0.281	40.518	-0.153
02-v1	0.183	2.439	0.231	0.274	42.181	0.153	-0.113	-0.268	40.629	-0.154
Absolute Difference	0.001	0.001	0.002	0.002	0.060	0.002	0.007	0.013	0.111	0.001
Percent Difference	0.8%	0.0%	0.8%	-	-	1.3%	5.7%	-	-	0.7%

J.2. Beach Profiles

Beach profile results were plotted and compared for the following sets of tests. Note that the reported wave characteristics are the measured characteristics at WG 5.

- Repeatability (tests 01-v1 and 02-v1) – Figure 61
- Influence of wave height (tests 03-v2, 05-v1, and 07-v1) – Figure 62
- Influence of wave period (tests 03-v2, 04-v2, 09-v1, and 10-v1) – Figure 63
- Influence of regular waves (tests 11-v1, 12-v1, 13-v1, 14-v1, and 15-v1) – Figure 64
- Influence of log elevation (19-v1, 04-v2, 22-v1, and 23-v1) – Figure 65
- Influence of log roughness (04-v2 and 59-v1) – Figure 66
- No structures (18-v2, 16-v1, and 21-v1) – Figure 67
- Single LWD (04-v2, 06-v1, and 08-v1) – Figure 68
- Benched LWD (39-v1, 40-v1, and 41-v1) – Figure 69
- Triple LWD (28-v3, 29-v1, and 30-v1) – Figure 70
- Matrix LWD (46-v1, 47-v1, and 48-v1) – Figure 71

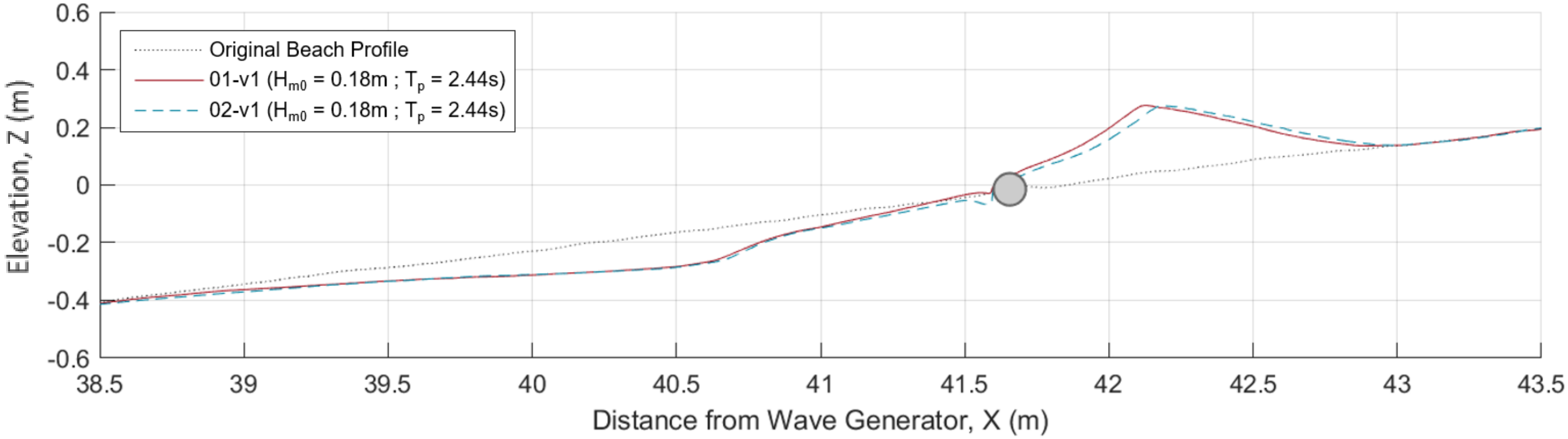


Figure 61 – Repeatability of beach profile observations

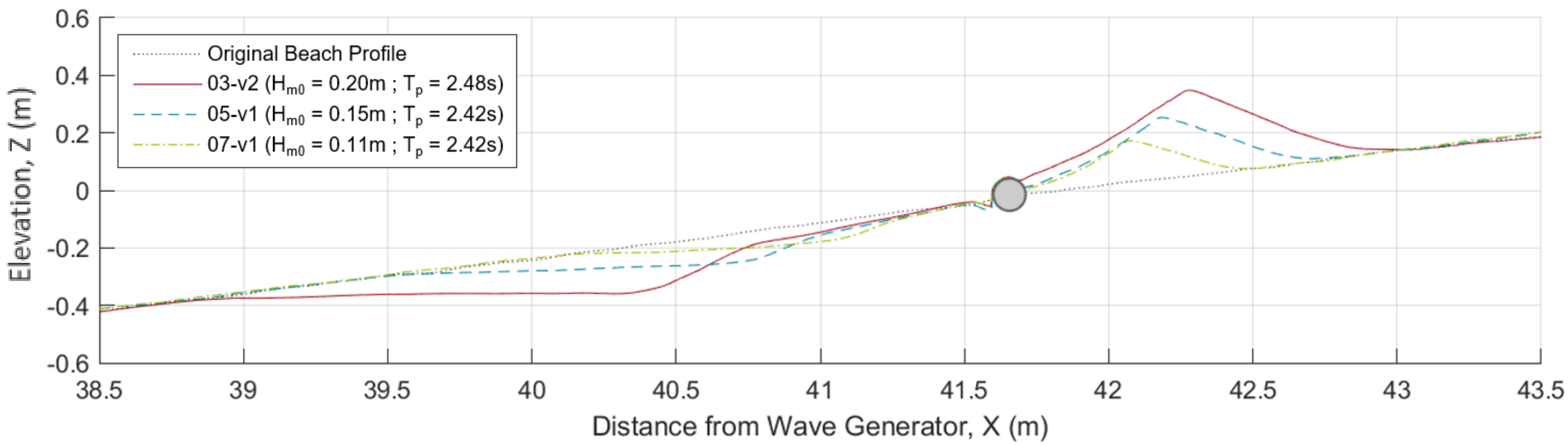


Figure 62 – Influence of wave height on beach profile behaviour

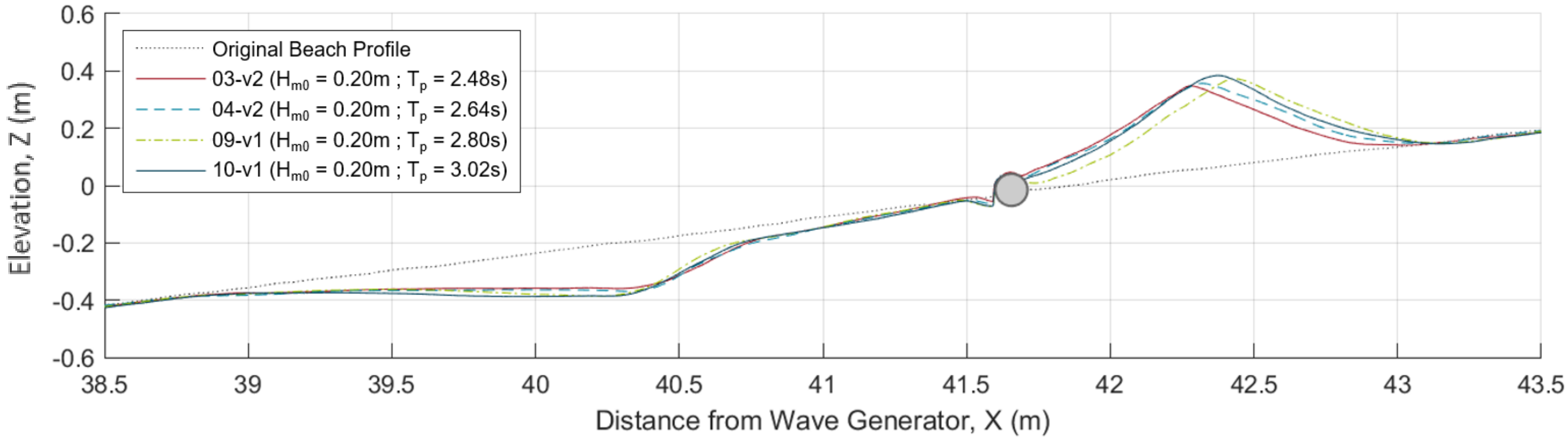


Figure 63 – Influence of wave period on beach profile behaviour

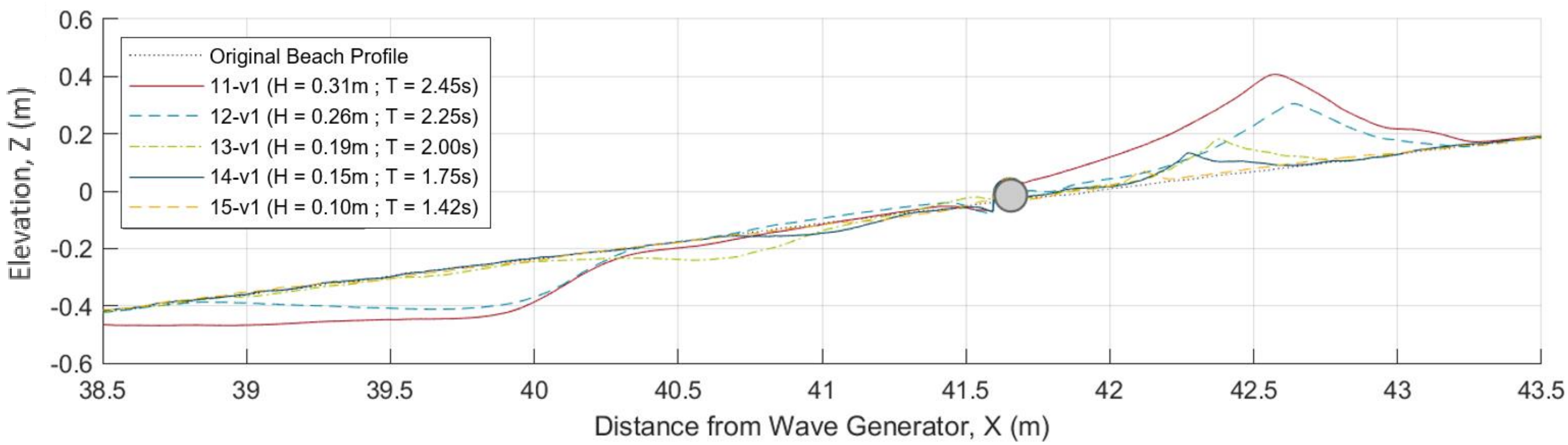


Figure 64 – Influence of regular waves on beach profile behaviour

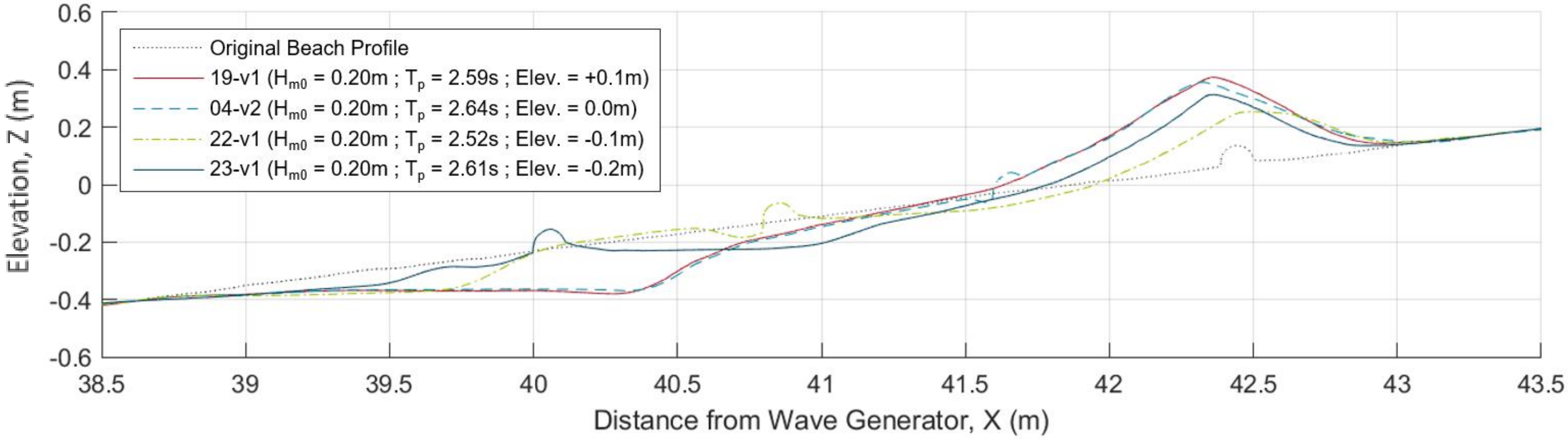


Figure 65 – Influence of log elevation on beach profile behaviour

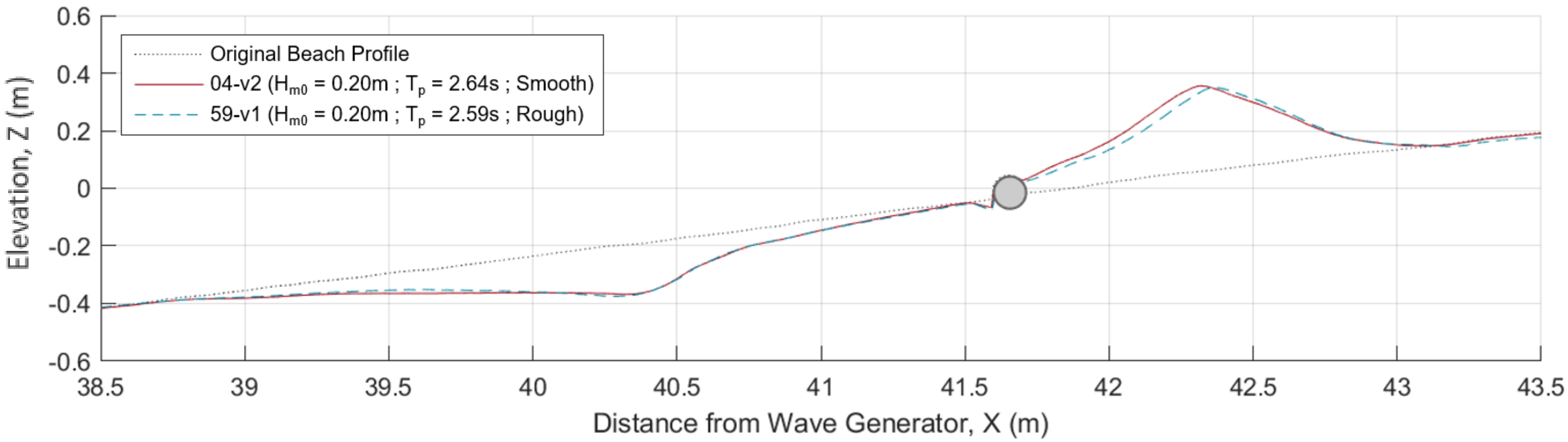


Figure 66 – Influence of roughness on beach profile behaviour

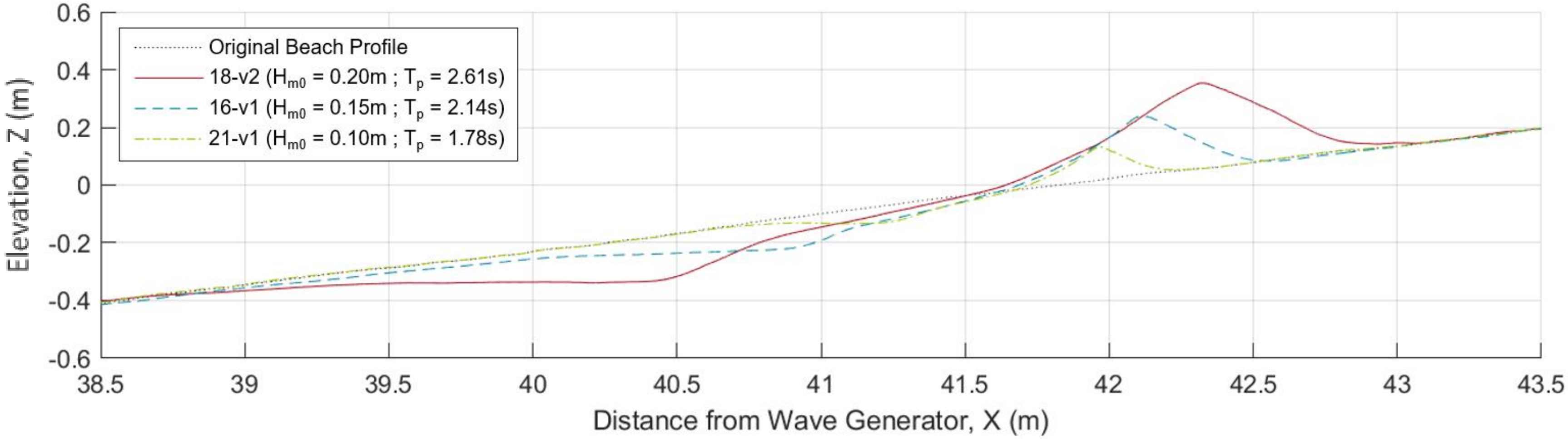


Figure 67 – Beach profile under three base incident wave conditions for configuration: No Structures

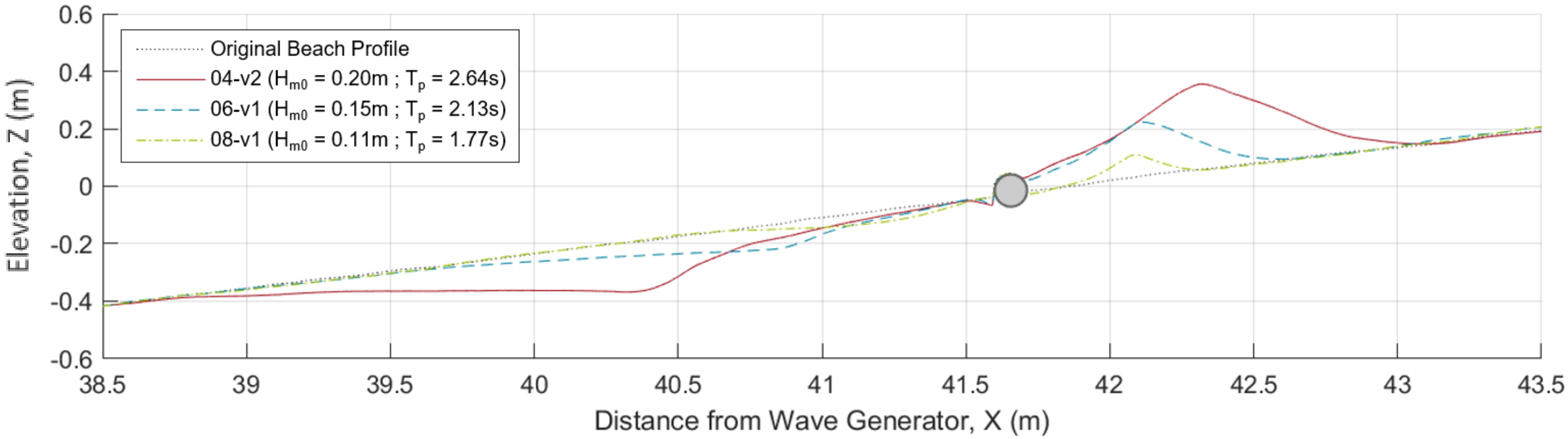


Figure 68 – Beach profile under three base incident wave conditions for configuration: Single

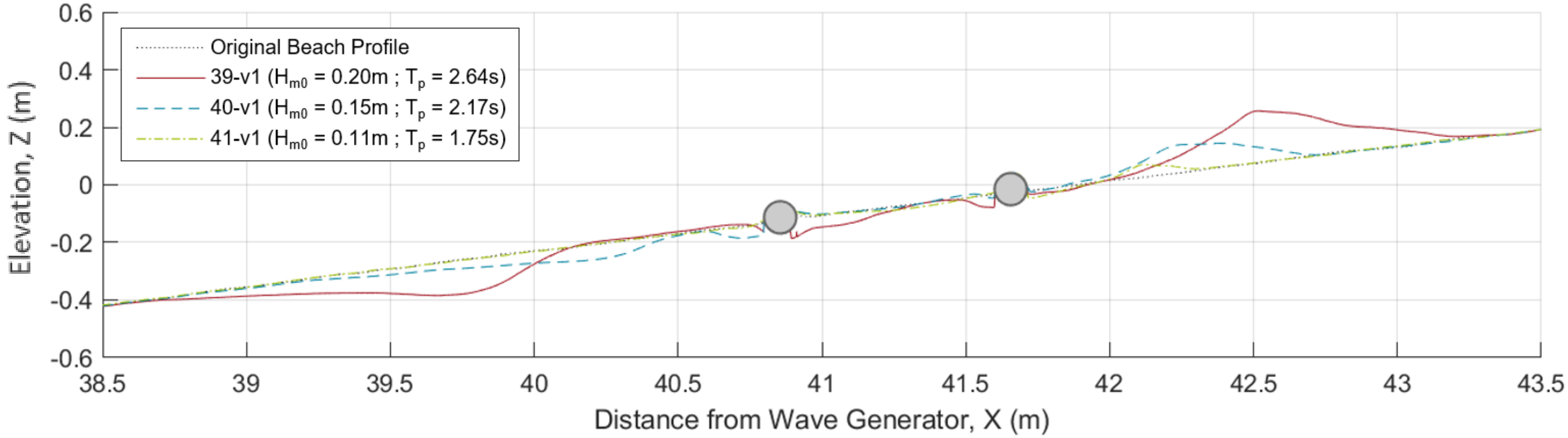


Figure 69 – Beach profile under three base incident wave conditions for configuration: Benched

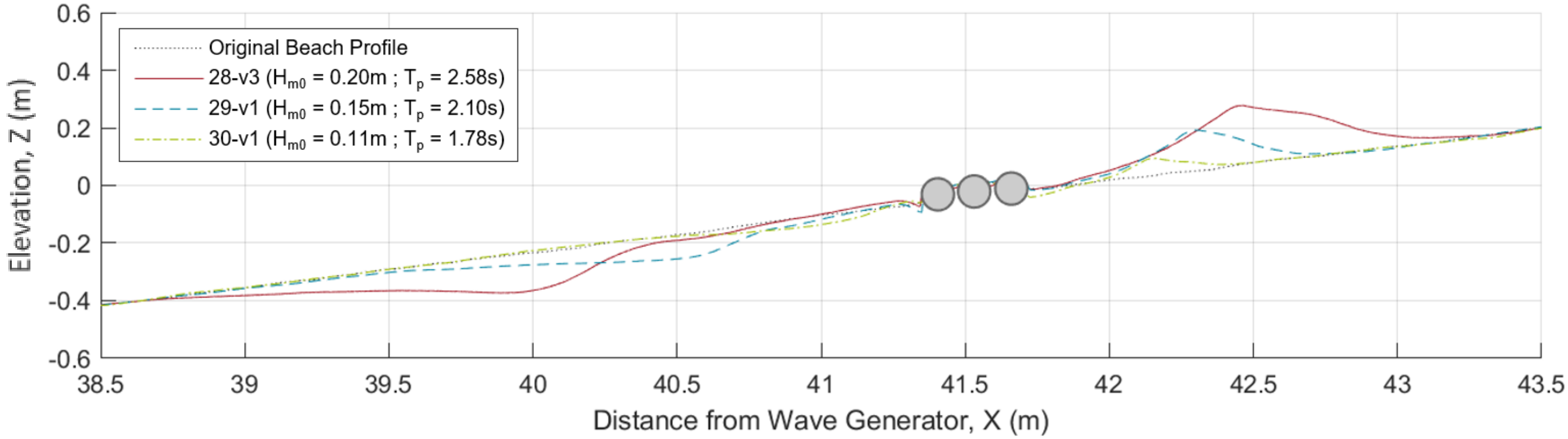


Figure 70 – Beach profile under three base incident wave conditions for configuration: Triple

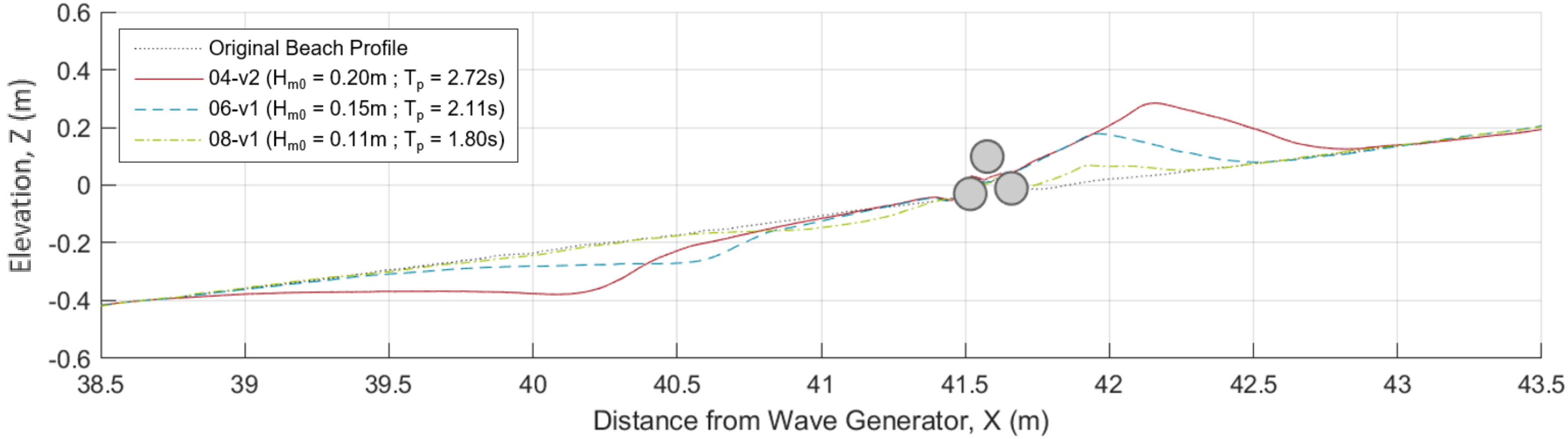


Figure 71 – Beach profile under three base incident wave conditions for configuration: Matrix

J.3. Key Characteristic

Key beach profile characteristics are provided for reference in Table 22 for all beach profiles included in Appendix J.2. Additional plots are also included illustrating the relationship between the target wave height and the maximum elevation change (Figure 72), crest and trough location (Figure 73), and volume change (Figure 74) for the LWD design configurations (not including the stacked log wall or seawall, which are not analyzed as part of this thesis).

Table 22 – Key beach profile characteristics for
Tests highlighted in red are potentially unreliable due to problems during the experiments.

Test No.	Beach Crest Characteristics			Beach Trough Characteristics			Volume Change
[-]	Z_{crest} [m]	dh_{max} [m]	X_{crest} [m]	Z_{trough} [m]	dh_{min} [m]	X_{trough} [m]	V_{avg} [m^3/m]
01-v1	0.276	0.233	42.121	-0.281	-0.119	40.518	0.152
02-v1	0.274	0.231	42.181	-0.268	-0.113	40.629	0.154
03-v2	0.347	0.299	42.276	-0.354	-0.161	40.354	0.182
04-v2*	0.355	0.298	42.312	-0.367	-0.173	40.350	0.198
05-v1	0.252	0.203	42.181	-0.247	-0.103	40.727	0.101
06-v1	0.223	0.194	42.110	-0.217	-0.091	40.852	0.088
07-v1	0.171	0.144	42.074	-0.180	-0.068	40.980	0.051
08-v1	0.109	0.085	42.091	-0.135	-0.036	41.116	0.020
09-v1	0.371	0.300	42.425	-0.384	-0.180	40.251	0.190
10-v1	0.383	0.318	42.366	-0.382	-0.185	40.301	0.216
12-v1	0.304	0.220	42.632	-0.399	-0.148	39.861	0.121
13-v1	0.181	0.131	42.376	-0.229	-0.071	40.703	0.042
14-v1	0.133	0.085	42.273	-0.072	-0.051	41.593	0.021
15-v1	0.070	0.041	42.115	-0.029	-0.021	41.724	0.005
16-v1	0.239	0.217	42.111	-0.220	-0.098	40.885	0.091
18-v2	0.354	0.290	42.319	-0.331	-0.153	40.429	0.179
19-v1	0.371	0.314	42.350	-0.377	-0.184	40.309	0.202
20-v1	0.368	0.310	42.340	-0.385	-0.193	40.327	0.211
21-v1	0.132	0.115	41.965	-0.129	-0.059	41.232	0.028
22-v1	0.252	0.180	42.467	-0.363	-0.096	39.703	0.088
23-v1	0.312	0.256	42.358	-0.213	-0.099	40.945	0.132
28-v3	0.278	0.205	42.455	-0.370	-0.133	39.968	0.121
29-v1	0.190	0.146	42.289	-0.253	-0.085	40.524	0.064
30-v1	0.094	0.060	42.148	-0.040	-0.038	41.721	0.013
39-v1	0.257	0.182	42.510	-0.380	-0.121	39.741	0.101

Test No.	Beach Crest Characteristics			Beach Trough Characteristics			Volume Change
[-]	Z_{crest} [m]	dh_{max} [m]	X_{crest} [m]	Z_{trough} [m]	dh_{min} [m]	X_{trough} [m]	V_{avg} [m^3/m]
40-v1	0.137	0.093	42.227	-0.177	-0.064	40.797	0.043
41-v1	0.069	0.036	42.106	-0.045	-0.040	41.717	0.002
46-v1	0.283	0.250	42.144	-0.372	-0.165	40.168	0.158
47-v1	0.178	0.167	41.953	-0.262	-0.103	40.577	0.077
48-v1	0.068	0.059	41.924	-0.152	-0.039	40.957	0.021
59-v1	0.350	0.287	42.360	-0.373	-0.178	40.337	0.189

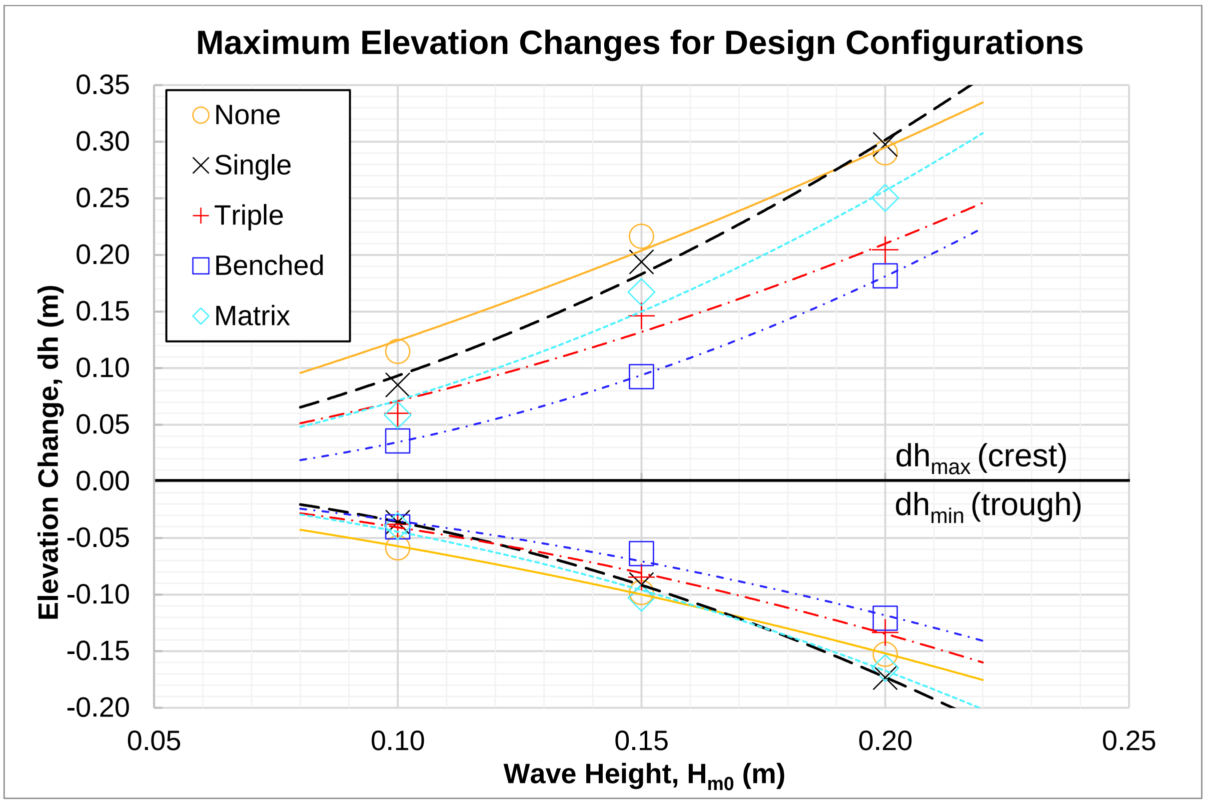


Figure 72 – Maximum elevation changes, dh_{min} and dh_{max}, versus the target wave height for various LWD design configurations

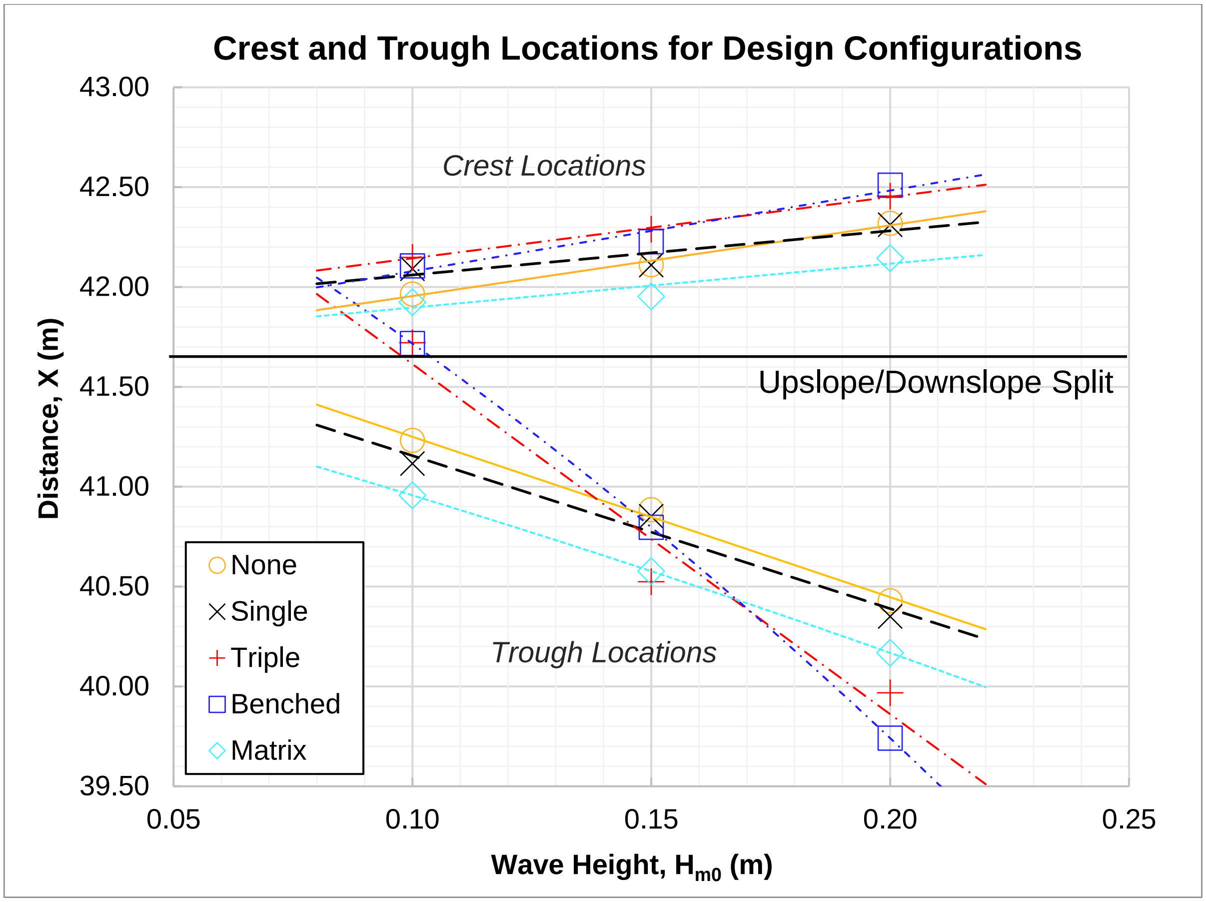


Figure 73 – Crest and trough location, X_{crest} and X_{trough}, versus the target wave height for various LWD design configurations

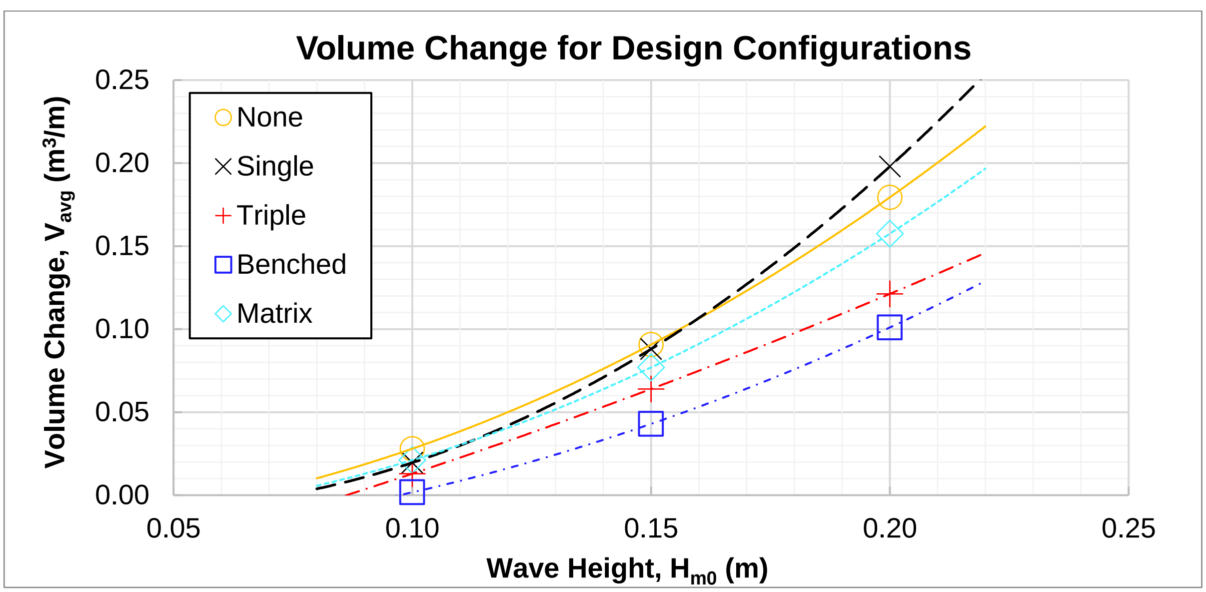


Figure 74 – Average volume change, V_{avg}, versus the target wave height for various LWD design configurations

Appendix K. Wave Run-Up for Regular Waves

K.1. Methodology

Video of wave run-up was captured from CAM2 outside of the flume. The video was exported into frames and frames with the maximum wave run-up near the beginning of the test (when the beach profile was approximately flat) were selected for further analysis. In total 20 frames were analyzed from each test set.

Each image was corrected for lens distortion and camera angle through a set of MATLAB commands which detects a checkerboard pattern and develops camera parameters based on the observed distortion of the checkerboard squares. The images were then undistorted in MATLAB and saved for additional analysis. Figure 75 illustrates this process for one image for test 37-v1.

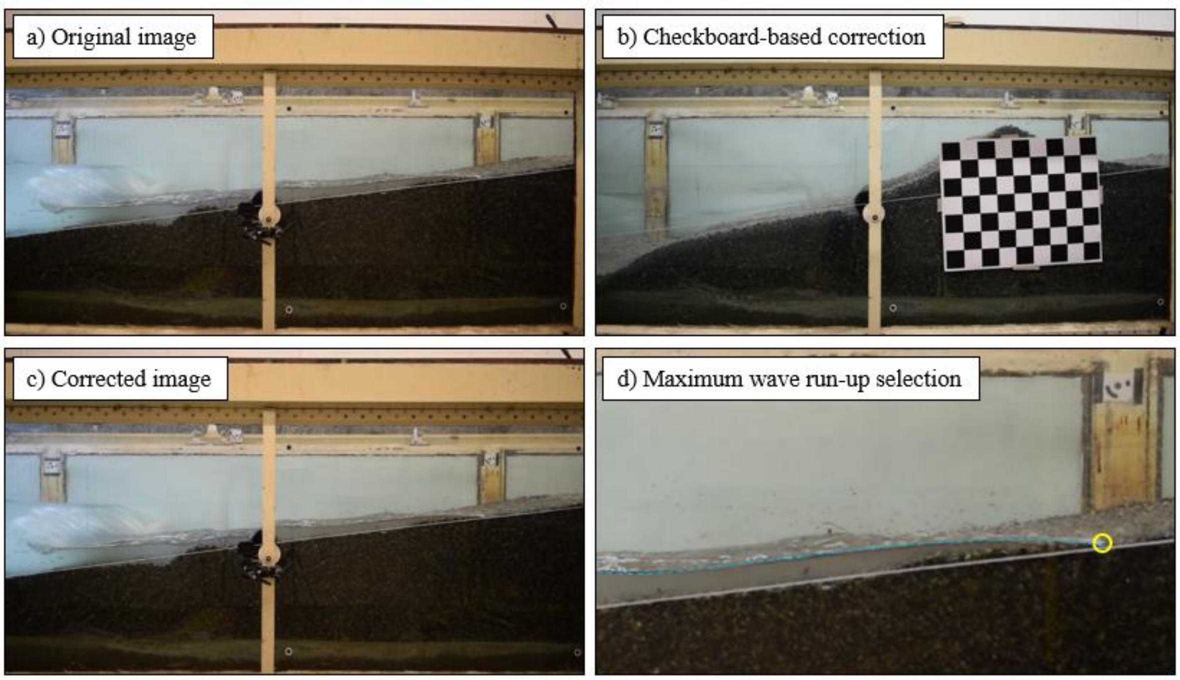


Figure 75 – Example of image processing method for wave run-up (test 37-v1)

For each image, the maximum location of wave run-up was manually selected in MATLAB. Based on the known surveyed coordinates of the four markers placed on the outside of the flume, the location and elevation of run-up was determined. This process resulted in run-up elevations from two sets of twenty waves for each test.

K.2. Run-Up Measurements

The variability of wave run-up estimates is shown in Figure 76 for each of the duplicate test sets. Generally, it can be seen that the interquartile range is typically within ±0.1 m, and that the number of outliers increases for higher wave heights.

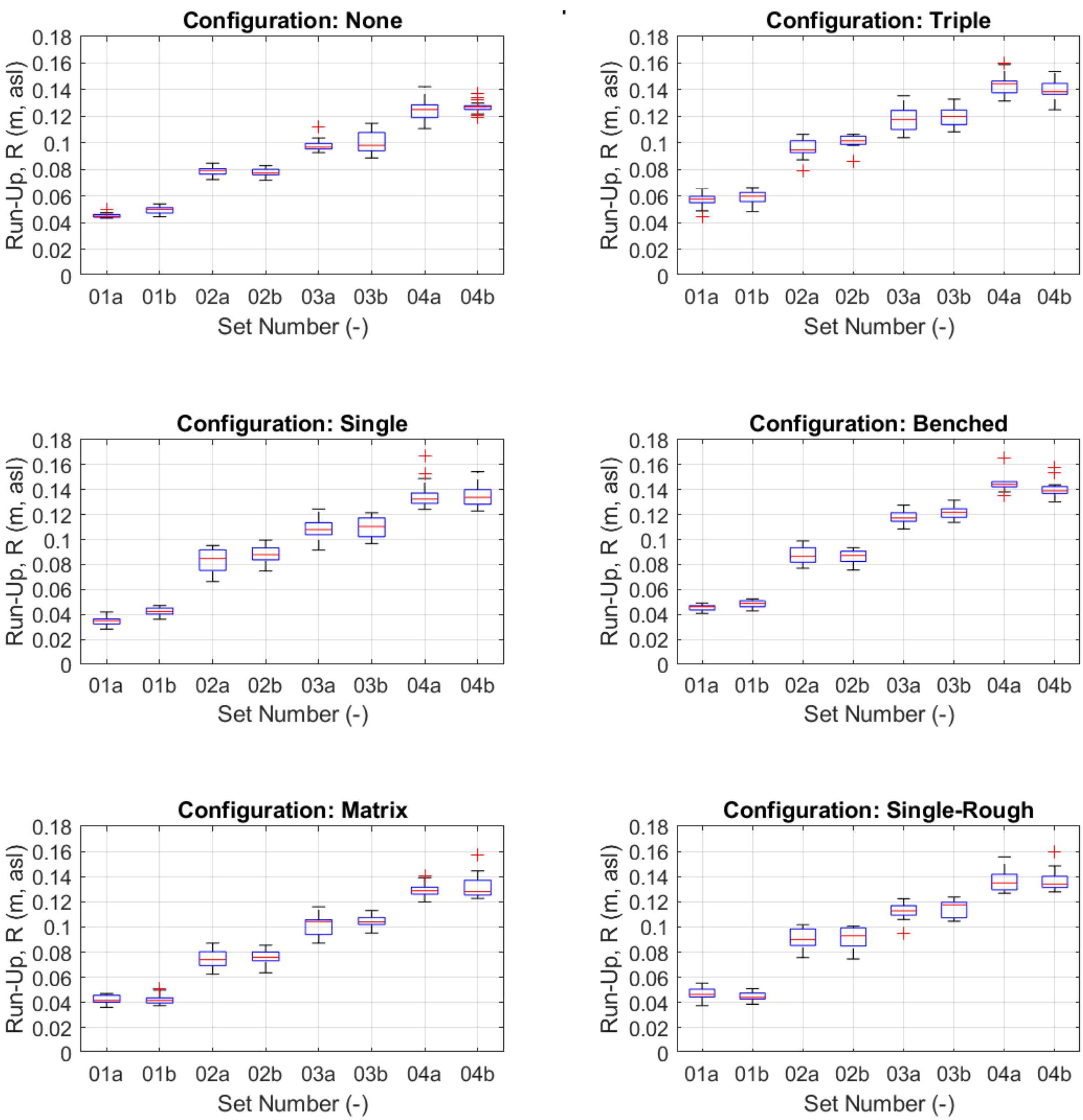


Figure 76 – Variability of measured wave run-up for two duplicate test sets (a and b) over a sample size, n, equal to 20 waves. Sets 1, 2, 3, and 4, have target wave heights of 0.10m, 0.15m, 0.20m, and 0.25m, respectively.