HYDRAULIC AND ENVIRONMENTAL DESIGN ASPECTS OF HARD COASTAL STRUCTURES

THE EXAMPLE OF STEPPED REVETMENTS

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SUMMARY

To continue safeguarding coastal communities and infrastructure in the face of climate change requires a range of coastal protection measures. Simultaneously, the greater awareness for sustainable development and the sense of urgency to preserve and restore the coastal environment, underline the importance of designing coastal protection measures sustainably. Hard coastal structures are effective coastal protection measures against wave overtopping and flooding, as they form physical barriers against waves. However, the effects of hard structures on hydrodynamics, sediment dynamics and habitats, bring unintended negative changes to the environment. The first part of this dissertation critically discusses the use of hard coastal structures and their environmental impacts. To minimise the environmental impacts of hard coastal structures or create new ecosystem services, environmental aspects should be incorporated in standard coastal engineering practice from the earliest design stages. Based on examples and recommendations in literature, this dissertation provides guidance on environmental aspects to be considered in the design of hard coastal structures to increase their sustainability, i.e. consider future environmental, social and economic needs.

The second part of this dissertation develops recommendations for the design of sustainable hard coastal structures, with the example of stepped revetments. Stepped revetments reduce wave overtopping effectively in comparison to smooth dikes, as their steps dissipate energy as waves interact with the structure. In addition to their primary function of ensuring coastal safety, stepped revetments offer opportunities for ecological enhancement and social benefits. The multi-functionality of stepped revetments make these structures especially suitable in urban and touristic settings. This dissertation improves design recommendations for stepped revetments by identifying environmental design aspects and improving design formulae for their hydraulic responses (wave reflection, wave run-up and wave overtopping).

Environmental design aspects are identified based on literature and include suggested nature-based adaptations to stepped revetment designs. For instance, the vertical and horizontal step surfaces of stepped revetments provide areas where roughness and surface complexity can be maximised to increase habitat variety and promote biodiversity. Additionally, their steps could be adapted to mimic habitats for intertidal organisms, e.g. by altering revetment steps to incorporate water retaining features like rock pools. The feasibility and success of these nature-based adaptations are highly dependent on the local environmental conditions, including hydrodynamics. With knowledge of environmental design aspects, coastal engineers gain a greater interdisciplinary understanding, thereby facilitating sustainable designs.

Hydraulic responses of stepped revetments are studied and analysed to improve and expand design recommendations for wave overtopping, wave run-up and wave reflection. In full-scale wave flume experiments, two stepped revetment cross-sections,

each with a slope of 1:3, were studied. The first cross-section had uniform step heights of 0.50 m, which was selected to add the secondary function of providing seating, i.e. serve as a bench. For the second cross-section, uniform step heights of 0.17 m were selected, as a typical height for walking up a staircase. Wave heights (H_{m0}) up to 1 m and wave periods ($T_{m-1,0}$) up to 6.5 s were generated. Based on the measurements of the physical model tests, empirical formulae were developed for estimating wave overtopping, wave run-up and wave reflection.

The tested stepped revetments effectively reduced wave overtopping in comparison to smooth dikes, resulting in influence factors for roughness (γ_f) between 0.43 and 0.73. Compared to smooth dikes, the energy dissipation of the revetment stairs reduces wave reflection and wave run-up. Within the tested range of boundary conditions, the stepped revetment with large steps (S_h=0.50 m) proved more effective in dissipating energy and reducing wave overtopping ($0.43 \leq \gamma_f \leq 0.54$). The higher effectiveness of the large steps is also confirmed with the measured wave reflection. Wave conditions were repeated for the large (S_h = 0.50 m) and small steps (S_h = 0.17 m) showing that reflection coefficients were 55 % higher at the small steps. Individual overtopping volumes at the tested stepped revetments are described by a two-parameter Weibull distribution, revealing a higher median shape factor (b=1.63) for stepped revetments compared to breakwaters, smooth dikes or vertical walls. The wave flume tests provide greater insight in the functioning of stepped revetments and enable the quantification of the hydraulic responses of stepped revetments.

The experimental work presented in this dissertation provides one of the first investigations into the hydraulic responses of stepped revetments at full scale. Compared to small-scale wave run-up and overtopping measurements, this study reveals that hydraulic responses measured in small scale are likely affected by scale effects. Small-scale studies overestimate the wave overtopping reduction (γ_f) by 2-31 % and underestimate relative wave run-up heights ($R_{u2\%}/H_{m0}$) by 31-51 %. As a result, basing the designs of stepped revetments on small-scale measurements could therefore lead to unsafe designs.

The gained knowledge on environmental aspects and hydraulic responses (wave reflection, run-up and overtopping) improves design recommendations for stepped revetments with regard to coastal safety and sustainability. In terms of coastal safety, the presented full-scale model tests provide reliable design recommendations that are not affected by scale. In terms of sustainability, the dissertation provides a review of environmental design aspects of coastal structures in general, and stepped revetments in particular. Hence, this dissertation contributes to recommendations for designing sustainable coastal structures.

Keywords: coastal protection, nature-based solutions, stepped revetment, physical model tests, wave overtopping, wave run-up, wave reflection

ZUSAMMENFASSUNG

Um die Küstengemeinden und küstennahe Infrastruktur angesichts des Klimawandels weiterhin zu schützen, ist eine Reihe von Küstenschutzmaßnahmen erforderlich. Gleichzeitig gewinnt durch das steigende Bewusstsein für eine nachhaltige Entwicklung sowie das Gefühl der Dringlichkeit, die Küstenumwelt zu erhalten und wiederherzustellen, die nachhaltige Gestaltung von Küstenschutzmaßnahmen an Bedeutung. Feste Küstenschutzbauwerke sind wirksame Küstenschutzmaßnahmen gegen Wellenüberlauf und Überflutung, da sie physische Barrieren gegen die Wellen bilden. Die Auswirkungen harter Strukturen auf die Hydrodynamik, die Sedimentdynamik und die Lebensräume führen jedoch zu unbeabsichtigten negativen Veränderungen in der Umwelt. Im ersten Teil dieser Dissertation werden der Einsatz harter Küstenstrukturen und ihre Umweltauswirkungen kritisch diskutiert. Diese sollten bereits in den frühesten Entwurfsphasen in der Standardpraxis des Küsteningenieurwesens einbezogen werden, um die Auswirkungen auf die Umwelt durch feste Küstenbauwerke zu minimieren oder neue Ökosystemleistungen zu schaffen zu können. Auf der Grundlage von Beispielen und Empfehlungen aus der Literatur bietet diese Dissertation einen Leitfaden für Umweltaspekte, die bei der Planung von harten Küstenschutzbauwerken zu berücksichtigen sind, um deren Nachhaltigkeit zu erhöhen und somit zukünftige ökologische, soziale und wirtschaftliche Bedürfnisse einzubeziehen.

Im zweiten Teil dieser Dissertation werden Empfehlungen für die Gestaltung nachhaltiger harter Küstenschutzbauwerke am Beispiel von gestuften Deckwerken entwickelt. Gestufte Deckwerke verringern im Vergleich zu glatten Deichen wirksam den Wellenüberlauf, da die Stufen bei der Interaktion der Wellen mit dem Bauwerk für eine Energiedissipation sorgen. Zusätzlich zu ihrer primären Funktion, der Gewährleistung eines wirksamen Küstenschutzes, bieten gestufte Deckwerke Möglichkeiten zur ökologischen Aufwertung und zum sozialen Nutzen. Durch die Multifunktionalität von gestuften Deckwerken sind diese Strukturen besonders geeignet für städtische und touristische Gebiete. Diese Dissertation verbessert die Empfehlungen für die Gestaltung und den Entwurf von gestuften Deckwerken durch die Identifikation der dabei zu berücksichtigenden ökologischen Aspekte sowie der Erweiterung der Formeln für die hydraulischen Auswirkungen an der Struktur (Wellenreflexion, Wellenauflauf und Wellenüberlauf).

Auf der Grundlage der Literatur werden ökologische Gestaltungsaspekte ermittelt und Vorschläge für naturbasierte Anpassungen von Deckwerkskonstruktionen unterbreitet. So bieten die vertikalen und horizontalen Stufenflächen der Deckwerke Bereiche, in denen Rauheit und Oberflächenkomplexität maximiert werden können, um die Lebensraumvielfalt zu erhöhen und die Biodiversität zu fördern. Darüber hinaus könnten die Stufen so angepasst werden, dass sie Lebensräume für intertidal lebende Organismen nachahmen, z. B. indem die Deckwerksstufen so verändert werden, dass

sie Wasserrückhalteelemente enthalten, die vergleichbar mit Felsbecken sind. Die Durchführbarkeit und der Erfolg dieser naturnahen Anpassungen hängen in hohem Maße von den örtlichen Umweltbedingungen, einschließlich der Hydrodynamik, ab. Mit dem Wissen um ökologische Gestaltungsaspekte gewinnen Küsteningenieure ein größeres interdisziplinäres Verständnis und können somit eine nachhaltige Gestaltung von Küstenbauwerken voranbringen.

Das hydraulische Verhalten von gestuften Deckwerken wird untersucht und analysiert, um die Entwurfsempfehlungen für Wellenüberlauf, Wellenauflauf und Wellenreflexion zu verbessern und zu erweitern. Mittels hydraulischer Modellversuche im Wellenkanal im Maßstab 1:1 wurden zwei gestufte Deckwerksquerschnitte mit einer Neigung von jeweils 1:3 untersucht. Der erste Querschnitt wies eine einheitliche Stufenhöhe von 0,50 m auf, welche ausgewählt wurde, um die sekundäre Funktion einer Sitzgelegenheit zu erfüllen, d. h. als Bank zu dienen. Für den zweiten Querschnitt wurde eine einheitliche Stufenhöhe von 0,17 m gewählt, da dies eine typische Höhe für das Begehen einer Treppe ist. Es wurden Wellenhöhen (H_{m0}) bis zu 1 m und Wellenperioden ($T_{m-1,0}$) bis zu 6,5 s generiert. Auf der Grundlage der physikalischen Modellversuche wurden empirische Formeln zur Abschätzung des Wellenüberlaufs, des Wellenauflaufs und der Wellenreflexion entwickelt.

Die getesteten gestuften Deckwerke reduzierten den Wellenüberlauf im Vergleich zu glatten Deichen effektiv, was zu Einflussfaktoren für die Rauheit (γ_f) zwischen 0,43 und 0,73 führte. Im Vergleich zu glatten Deichen wird durch die Deckwerkstreppe die ein Teil der Wellenenergie dissipiert und somit die Wellenreflexion und der Wellenauflauf reduziert. Innerhalb des untersuchten Bereichs von Randbedingungen erwies sich das gestufte Deckwerk mit großen Stufen (S_h=0,50 m) als effektiver bei der Energiedissipation und der Verringerung des Wellenüberlaufs (0,43 $\leq \gamma_f \leq$ 0,54). Die höhere Wirksamkeit der großen Stufen wird auch durch die gemessene Wellenreflexion bestätigt. Die Wellenbedingungen wurden für die großen (S_h = 0,50 m) und kleinen Stufen (S_h = 0,17 m) wiederholt, wobei sich zeigte, dass die Reflexionskoeffizienten bei den kleinen Stufen um 55 % höher waren. Die einzelnen Überlaufvolumina an den getesteten gestuften Deckwerken wurden durch eine zweiparametrige Weibull-Verteilung beschrieben, die einen höheren medianen Formfaktor (b=1,63) für gestufte Deckwerke im Vergleich zu Wellenbrechern, glatten Deichen oder vertikalen Mauern ergab. Die Versuche im Wellenkanal geben einen besseren Einblick in die Funktionsweise von gestuften Deckwerken und ermöglichen die Quantifizierung der hydraulischen Auswirkungen von gestuften Deckwerken.

Die in dieser Dissertation vorgestellten experimentellen Arbeiten sind eine der ersten Untersuchungen der hydraulischen Auswirkungen von gestuften Deckwerken im Maßstab 1:1. Im Vergleich zu kleinmaßstäblichen Messungen des Wellenauflaufs und des Wellenüberlaufs zeigt diese Studie, dass die in kleinem Maßstab gemessenen hydraulischen Auswirkungen wahrscheinlich durch Skaleneffekte beeinflusst werden. Kleinmaßstäbliche Studien überschätzen die Verringerung des Wellenüberlaufs (γ_f) um 2-31 % und unterschätzen die relativen Wellenauflaufhöhen ($R_{u2\%}/H_{m0}$) um 31-51 %. Die Zugrundelegung von kleinmaßstäblichen Messungen bei der Bemessung von gestuften Deckwerken könnte daher zu unsicheren Konstruktionen führen.

Die gewonnenen Erkenntnisse über Umweltaspekte und hydraulische

Auswirkungen (Wellenreflexion, Wellenauflauf und Wellenüberlauf) verbessern die Entwurfsempfehlungen für gestufte Deckwerke im Hinblick auf Küstensicherheit und Nachhaltigkeit. In Bezug auf die Küstensicherheit liefern die vorgestellten Modellversuche in Originalgröße zuverlässige Entwurfsempfehlungen, die nicht durch den Maßstab beeinflusst werden. Im Hinblick auf die Nachhaltigkeit bietet diese Dissertation einen Überblick über die Umweltaspekte von Küstenbauwerken im Allgemeinen und gestuften Deckwerken im Besonderen. Daher trägt diese Dissertation zu Empfehlungen für den Entwurf nachhaltiger Küstenbauwerke bei.

Schlüsselwörter: Küstenschutz, Ökosystembasierte Küstenschutzbauwerke, getreppte Deckwerke, physikalische Modellversuche, Wellenüberlauf, Wellenauflauf, Wellenreflexion

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SYMBOLS AND ABBREVIATIONS

α	Angle between structure slope and horizontal	$[°]$
Γ	Mathematical gamma function	$[-]$
γ_b	Influence factor for a berm	$[-]$
γ_f	Influence factor for roughness or roughness factor	$[-]$
γ_v	Influence factor for a wall at the end of a slope	$[-]$
γ_*	Influence factor for a storm wall or promenade	$[-]$
γ_β	Influence factor for oblique wave attack	$[-]$
$\xi_{m-1,0}$	Breaker parameter based on $s_{m-1,0}$ $(=\tan\alpha/\sqrt{s_{m-1,0}})$	$[-]$
a	Scale factor of Weibull distribution	$[m^3/m]$
b	Shape factor of Weibull distribution	$[-]$
c	Empirical coefficient in Eq. 4.12	$[-]$
C_r	Reflection coefficient	$[-]$
$cos\alpha \cdot S_h$	Characteristic step height	$[m]$
$cos\alpha \cdot S_h/H_{m0}$	Dimensionless step height	$[-]$
E_d	Dissipated wave energy	
E_i	Incident wave energy	
E_t	Transmitted wave energy	
f_{γ_f}	Scale correction factor for roughness $(=\gamma_{f(full\ scale)}/\gamma_{f(small\ scale)}$	$[-]$
g	Acceleration due to gravity	$[m/s^2]$
H	Wave height, regular waves	m
h	Water depth at toe of structure	$[m]$
$H_{m0,i}$	Incident spectral wave height	$[m]$
$H_{m0,r}$	Reflected spectral wave height	$[m]$
H_{m0}	Incident spectral significant wave height at toe of structure	$[m]$

H_{m0}/S_h	Step ratio	$[-]$
L_{0p}	Peak wavelength in deep water ($= g \cdot T_p{}^2/2\pi$)	$[m]$
$L_{m-1,0}$	Spectral wavelength in deep water ($= g \cdot T_{m-1,0}{}^2/2\pi$)	$[m]$
L_p	Peak wavelength based on local water depth at toe of structure	$[m]$
N_{ow}	Number of overtopping waves	$[-]$
N_w	Number of incident waves	$[-]$
P_{ow}	Probability of overtopping	$[-]$
P_v	Exceedance probability	$[-]$
q	Average overtopping	$[m^3/\text{s per m}]$
$q_{\gamma_f=1}$	Average overtopping according to Eq. 4.2 with $\gamma_f = 1$	$[m^3/\text{s per m}]$
r^2	Coefficient of determination	$[-]$
R_c	Crest freeboard	$[m]$
R_c/H_{m0}	Relative crest freeboard	$[m]$
$R_{u2\%}$	Wave run-up height exceeded by 2 % incident waves	$[m]$
$R_{u2\%}/H_{m0}$	Relative wave run-up	$[-]$
R_u	Run-up height caused by regular waves	$[m]$
s_{0p}	Wave steepness ($=H_{m0}/L_{0p}$)	$[-]$
S_h	Step height of stepped revetment	$[m]$
$s_{m-1,0}$	Wave steepness ($=H_{m0}/L_{m-1,0}$)	$[-]$
T	Wave period, regular waves	$[s]$
$T_{m-1,0}$	Spectral wave period	$[s]$
T_m	Average wave period	$[s]$
T_p	Spectral peak incident wave period	$[s]$
V	Individual overtopping volume	$[m^3]$
V_{max}	Maximum individual overtopping volume	$[m^3/m]$
X	Distance from wave maker ($X = 0\ m$)	$[m]$
Y	Elevation relative to flume bottom ($Y = 0\ m$)	$[m]$
1-γ_f	Slope roughness	$[-]$
RMSE	Root mean square error	$[-]$
SR	Stepped revetment(s)	

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Chapter 1

Introduction

1

1.1 Background and motivation

1.1.1 Challenges in coastal engineering

In the coming decades the effects of climate change will expose coastal communities and infrastructure to sea-level rise, elevated storminess and more frequent coastal flooding (Vousdoukas et al., 2020; Taherkhani et al., 2020; IPCC, 2019; Vitousek et al., 2017). Simultaneously the growing coastal population (Neumann et al., 2015) and expanding urban infrastructure (IPCC, 2019; Hawkins et al., 2021) increase the vulnerability of coastal areas to these effects. If no additional coastal protection solutions are applied, the annual projected cost of coastal flood damage in Europe is estimated to increase from € 1.25 billion in 2018 to between € 93 and 961 billion in 2100 (Vousdoukas et al., 2018). However, climate change associated damage is often underestimated due to the difficulty and uncertainty of quantifying economic damages suffered from e.g. loss of lives and livelihoods (DeFries et al., 2019). To limit coastal flood damage and ensure continued safety to coastal communities and infrastructure in the future we require effective coastal solutions.

The effects of climate change not only threaten coastal communities and infrastructure, but also the valuable coastal ecosystems along the world's shorelines. These ecosystems have already been declining due to human activities in the last decades (MEA, 2005). Coastal ecosystems directly or indirectly provide numerous benefits to human welfare, known as ecosystem services (Costanza et al., 1997). Examples of ecosystem services include tourism and recreation provided by coral reefs, coastal protection offered by salt marshes, erosion control of sand beaches and dunes, carbon sequestration of seagrass and maintenance of fisheries in mangroves (Barbier et al., 2011). Coastal ecosystems play an integral part in flood protection as they attenuate waves and reduce erosion. Globally the flood protection ecosystem service of coastal wetlands alone is worth an estimated $ US 447 billion/yr and saves an estimated 4,620 lives per year (Costanza et al., 2021). Besides flood protection, other ecosystem services provided by coastal wetlands are globally valued at $ US 135,000/ha/yr (Costanza et al., 2014). As a result, there is a large economic and social incentive for preserving and restoring coastal ecosystems (Arkema et al., 2013; Van Zelst et al., 2021). In addition to ensuring coastal safety in the face of climate change and a growing coastal population, we need coastal solutions to become sustainable, i.e. consider the environmental and social challenges of the future.

Traditionally the main task of coastal engineers has been to design cost-effective solutions with goals of coastal safety or enabling safe transport as commissioned by clients or governments (Kamphuis, 2006). In the past these solutions were often hard structures, such as breakwaters, groynes, seawalls or dikes. The designs of these structures are based on the hydraulic responses as well as the wave loading and related structural response (USACE, 2002). Hydraulic responses include wave overtopping over the structure, wave run-up, wave reflection and wave transmission by the structure. The wave loading and related structural response are based on the wave forces and structure stability. Hydraulic and structural responses are typically quantified with physical model tests in laboratory wave flumes or estimated by empirical formulae derived in these facilities. Hard structures form a barrier against wave action and stabilise shorelines, thereby altering hydrodynamic and morphological processes and introducing artificial hard habitats in coastal ecosystems. The resulting unavoidable changes in processes and habitats impact the environment (Hawkins et al., 2021). With coastal safety being the main focus in the past, the negative environmental (e.g. habitat changes) and social impacts (e.g. tourism) of hard structures tended to be minimised or mitigated reactively in coastal engineering projects (De Vries et al., 2021).

More recently, with the growing awareness of sustainable development, the challenge for coastal engineers has become to design cost-effective coastal solutions in line with environmental, stakeholder and client interests (Kamphuis, 2006; De Vries et al., 2021). Beyond the primary function of providing coastal safety, contemporary coastal solutions are therefore often multi-functional with secondary goals of creating opportunities for the environment (e.g. to enhance biodiversity) and society (e.g. recreational activities). Soft solutions, such as sediment nourishments or dune rehabilitation, are typically classified as green infrastructure as they are regarded as less damaging to the environment. Soft solutions are therefore increasingly preferred over hard structures. However, in some cases the use of soft solutions may be unsuitable due to local conditions, for instance spatial requirements, availability of building material (e.g. sediment) or equipment. In such cases hard structures, or a combination of hard and soft solutions are required instead. Although hard structures profoundly impact the environment, nature-based adaptations to conventional designs may minimise these impacts or create or restore ecosystem services. Hard structures can thus be made into green infrastructure if they are adapted to mimic ecosystem functions, induce or preserve connectivity in a similar way natural coastal ecosystems would. The design of hard structures as green infrastructure therefore commands interdisciplinary understanding and collaboration between coastal engineers, ecologists and biologists. Similarly, for recreational goals of coastal structures, interdisciplinary understanding and collaboration with social scientists and stakeholders are required. Ideally a proactive design approach is followed for designing multi-functional coastal structures in which environmental and social aspects are considered from the earliest phases. As such, the design basis of coastal engineers needs to extend beyond hydraulic response, wave loading and related structural response to include environmental and social aspects. This calls for the expansion of present design recommendations for coastal solutions.

This dissertation critically discusses the use of hard structures for coastal protection and outlines the conditions under which these structures are suitable. When hard structures are required, nature-based adaptations can be made to conventional hard structure designs, to minimise their environmental impacts and to create new ecosystem services, thereby enhancing the ecological value of hard coastal structures. The dissertation recommends options for nature-based adaptations which should be considered ideally from earliest design phases. Whereas the first part of the dissertation is concerned with hard structures for coastal protection in general, the second part focuses on one particular type of hard coastal structure, namely stepped revetments. Stepped revetments reduce wave overtopping effectively and are especially suitable in urban and touristic settings as they provide recreational benefits. This dissertation investigates stepped revetments as an example of a multi-functional coastal structure and expands design recommendations related to both hydraulic responses and environmental aspects.

1

1.1.2 Stepped revetments as multi-functional coastal structure

Stepped revetments are shore-parallel, sloped seawalls or dike covers consisting of stairs and are often constructed from concrete. Fig. 1.1 displays two examples of stepped revetments in Europe. When waves run up stepped revetments, their stairs interrupt the flow and dissipate wave energy, thus affecting wave run-up. Wave run-up height refers to the vertical distance between the still water level and the highest elevation a wave reaches on the structure. When the wave run-up height exceeds the crest height of the structure, wave overtopping occurs, i.e. water discharges over the structure's crest. Therefore, the design crest level of a structure is based on wave run-up heights and wave overtopping rates. As the stairs of stepped revetments induce energy dissipation, wave run-up heights and wave overtopping volumes reduce in comparison to those of a smooth slope or dike. From a hydraulic response point of view, the main benefit of stepped revetments is that their required crest level could be lower than that of a smooth slope to obtain the same level of coastal safety (Kerpen, 2017; Van Steeg et al., 2018). With rising sea levels, the reduction of wave run-up and overtopping is an important benefit of stepped revetments. Upon interaction with stepped revetments, wave energy is partly dissipated during wave run-up and run-down or transmitted over the structure crest when wave overtopping occurs. The remaining portion of wave energy is reflected, known as wave reflection. Wave reflection is another hydraulic response important in designs as it may exacerbate sea bed erosion (USACE, 2002) and affects sea states and navigation (Seelig and Ahrens, 1981).

Environmental design aspects comprise two categories, namely (1) minimising and mitigating adverse environmental impacts and (2) creating opportunities for the environment. To include environmental aspects in designs requires a sound understanding of the system (biotic and abiotic) and interdisciplinary collaboration, for instance between coastal engineers, scientists and ecologists. The implementation of stepped revetments or any coastal solution brings changes to the present system, defined as environmental impacts. Environmental impacts are site-specific and stretch over various time and spatial scales. For sustainable designs negative impacts need to be minimised and mitigated. Although these impacts are site-specific, a general understanding and awareness would aid coastal engineers to effectively collaborate with other disciplines and facilitate sustainability outcomes of stepped revetments. Beyond minimising and mitigating impacts, natural elements could be utilised to achieve the goals of coastal solutions or new opportunities for the environment could be created (PIANC, 2011). With their vertical and horizontal step surfaces, stepped revetments offer opportunities for ecological enhancement (Wiecek, 2009).

Stepped revetments also offer social opportunities, for instance their stairs could offer recreational benefits such as safe access (Fig. 1.1a) or seating (Fig. 1.1b). The potential multi-functionality of stepped revetments makes these coastal solutions particularly suitable in urban settings. In the face of climate change and the urgency for sustainable development, coastal solutions need to be reinforced or newly built in a sustainable way to safeguard coastal communities and infrastructure. With their multi-functionality and their ability to effectively reduce wave overtopping, stepped revetments show potential as a sustainable solution.

Research on stepped revetments as coastal protection dates back to the 1950s

Figure 1.1: Examples of stepped revetments: **(a)** Stepped revetment providing access to the beach in Margate, United Kingdom. Photo by Dean Barkley. **(b)** Promenade in Zadar, Croatia

and focused mainly on wave run-up and wave overtopping. Despite its long history, recommendations related to the hydraulic responses of stepped revetments are not included in design manuals such as USACE (2002) or EurOtop (2018). Previous studies are often based on small-scale physical model tests with regular waves (e.g. Xiaomin et al., 2013) or site-specific designs which include features such as a berm (Kerpen et al., 2014) or recurve wall (Heimbaugh et al., 1988). Hence translating data from these studies with divergent boundary conditions into empirical formulae was unsuitable.

More recent studies advanced knowledge on the hydraulic responses of stepped revetments. Small-scale data for wave reflection and run-up is presented by Kerpen (2017), while wave overtopping at stepped revetments was measured by Van Steeg et al. (2018), Kerpen et al. (2019) and Gallach-Sánchez (2018). For a wide range of boundary conditions, these studies proved the potential of stepped revetments to reduce wave run-up and overtopping in comparison to a smooth slope. Empirical formulae were developed, yet the underlying small-scale data presents considerable variability for similar boundary conditions. Due to this variability, it remains unverified whether the empirical formulae include all key processes affecting the hydraulic responses or whether the data is affected by scale. For instance, the small-scale overtopping measurements are likely affected by scale, since the majority of measurements had overtopping rates below 1 l/s per m (EurOtop, 2018). Based on full-scale physical model tests this dissertation seeks to gain a deeper understanding of the hydraulic response of stepped revetments to ultimately refine empirical formulae for estimating wave overtopping, run-up and reflection.

Since stepped revetments are mostly constructed from concrete, they are classified as hard coastal structures. Although recommendations specifically related to environmental aspects of stepped revetments are lacking, extensive research on environmental aspects of other hard coastal structures exists. Various studies highlight the negative environmental impacts of hard structures (e.g. Firth et al., 2016b; Bishop et al., 2017; Bulleri and Chapman, 2010; Cooper et al., 2016). Also opportunities for ecological enhancement of hard structures are broadly studied (e.g. Borsje et al., 2011; Firth et al., 2014; Airoldi et al., 2021). This dissertation compiles and discusses environmental aspects to be considered in stepped revetment designs.

1.2 Research objectives and methodology

Over recent years soft solutions are often recommended over hard structures, due to the adverse environmental impacts of hard structures. Yet, in some local conditions hard structures or a combination of hard structures and soft solutions would be more suitable. This dissertation aims to investigate when hard structures are suitable as coastal protection and how they can be adapted to minimise their environmental impacts and create new opportunities for the environment, whilst offering coastal safety. Hydraulic responses (i.e. wave reflection, run-up and overtopping) and environmental design aspects are considered for hard structures in general and more specifically investigated with an example for stepped revetments. The three objectives of the dissertation are listed and discussed below:

1. **Determine for which conditions hard coastal structures are most suitable and how they can be adapted to minimise environmental impacts or create opportunities for the environment**,

   This objective is achieved by means of a literature review. The review critically discusses the use of hard coastal structures on the foreshore (groynes, breakwaters and jetties) and onshore (dikes, seawalls and revetments). The purpose and design conditions for the structure types are described. Also the potential negative impacts and concerns which hard structures may pose on hydrodynamic, morphological and ecological conditions are discussed. Ideally coastal engineers need to consider these impacts in the earliest design phases to develop ecologically sensitive designs. Beyond minimising environmental impacts, nature-based adaptations can be made to hard structures to create new ecosystem services, thus enhancing the ecological value of hard structures. Based on literature, such nature-adaptations are proposed and described for foreshore and onshore hard structures.

2. **Identify environmental aspects to be considered in the design of sustainable stepped revetments**,

   To develop environmentally sensitive designs, a good understanding of environmental impacts and concerns is required. Based on what was found for hard structures in general, environmental aspects related specifically to stepped revetments are discussed. The first part describes the potential negative impacts and concerns which stepped revetments may pose on hydrodynamic, morphological and ecological conditions. The second part identifies and discusses adaptations for sustainable stepped revetments. These adaptations may reduce or mitigate the negative environmental impacts or create new ecosystem services. With these adaptations the ecological value and functionality of stepped revetments can be improved.

3. **Improve and supplement hydraulic design recommendations of stepped revetments for wave overtopping, wave reflection and wave run-up**,

   The selected methodology to achieve this objective is to review previous studies, identify knowledge gaps, conduct physical model tests as well as describe and interpret their results. Based on previous small-scale studies it was anticipated that scaling affects wave overtopping and run-up experiments at stepped revetments. In this study the model tests are conducted in a wave flume at full scale. Wave overtopping, reflection and run-up are measured at two stepped revetment cross-sections, each with a slope of 1:3. The first tested cross-section had larger step heights of 0.50 m, while the second had smaller step heights of 0.17 m. The measurements are analysed to gain insights in the hydraulic responses of stepped revetments. These insights and results are used to develop empirical equations for estimating the hydraulic performance of stepped revetments.

1.3 Overview of publications

This research takes the form of a cumulative dissertation, consisting of three peer reviewed articles in international scientific journals. The articles are presented in Chapters 2, 4 and 5. This subsection gives an overview of the published articles.

- **Chapter 2: Hard structures for coastal protection, towards greener designs**

  Estuaries and coasts 42, 1709-1729, 2019 doi.org/10.1007/s12237-019-00551-z

  T. Schoonees, A. Gijón Mancheño, B. Scheres, T. J. Bouma, R. Silva, T. Schlurmann, H. Schüttrumpf

  Over recent years, many coastal engineering projects have employed the use of soft solutions as these are generally less environmentally damaging than hard solutions. However, in some cases, local conditions hinder the use of soft solutions, meaning that hard solutions have to be adopted or, sometimes, a combination of hard and soft measures is seen as optimal. This chapter reviews the use of hard coastal structures on the foreshore (groynes, breakwaters and jetties) and onshore (seawalls and dikes). Groynes and breakwaters are constructed on the foreshore to prevent or compensate erosion, while jetties (and other breakwater types) protect navigational channels. Seawalls and dikes are built onshore to form the last line of defence against wave overtopping and flooding. The intended functions and for which conditions these structure types are suitable are further discussed.

  The use of hard coastal structures bring unintended negative effects on hydrodynamic, morphological and ecological conditions. These effects include wave reflection from the structure exacerbating scour, erosion on the down-drift side of the structure and habitat loss affecting ecological interactions. To reduce or mitigate these negative impacts, or to create new ecosystem services, the following nature-based adaptations are proposed and discussed: (1) applying soft solutions complementary to hard solutions, (2) mitigating morphological and hydrodynamic changes and (3) ecologically enhancing hard coastal structures. By applying soft solutions (including ecosystems and sediment nourishments) in the foreshore of seawalls and dikes, the hydrodynamic forcing on these structures is reduced. In addition, soft solutions retain or enhance ecosystem services. Structure designs can be optimised to reduce the negative effects on coastal processes. The ecological value of coastal structures can be enhanced by mimicking natural habitats, e.g. by incorporating micro-habitats like rock pools in designs. The selection and also the success of these potential adaptations are highly dependent on local conditions, such as hydrodynamic forcing, spatial requirements and socioeconomic factors.

The overview provided in this chapter aims to offer an interdisciplinary understanding, by giving general guidance on which type of solution is suitable for given characteristics, taking into consideration environmental aspects that are key for greener coastal designs. Overall, this chapter aims to provide guidance at the interdisciplinary design stage of nature-based coastal protection structures.

- **Chapter 4: Full-scale experimental study on wave overtopping at stepped revetments**

  Coastal Engineering 167, 103887, 2021 doi.org/10.1016/j.coastaleng.2021.103887

  T. Schoonees, N. B. Kerpen , T. Schlurmann

  Stepped revetments limit wave overtopping effectively. Based on small-scale physical model tests, previous studies provide insights in the design of crest-levels and step geometries of these structures. The validation of these insights at large scale, along with the analysis of individual overtopping volumes, is required to formulate recommendations for detail design. This chapter investigates wave overtopping at stepped revetments by means of full-scale flume experiments. Two cross-sections, each with a slope of 1:3, were studied with uniform step heights of 0.17 m and 0.50 m. Wave conditions were generated with wave heights (H_{m0}) up to 1 m and periods ($T_{m-1,0}$) up to 6.5 s. Water overtopping the structure crest was collected in a container, placed on load cells. The load cells enabled the measurement of individual wave overtopping volumes. Overtopping measurements were analysed to determine wave overtopping rates as well as the magnitude and distribution of individual wave overtopping volumes.

  The measured wave overtopping rates are substantially lower than those at smooth slopes. This reduction in wave overtopping can be attributed to the energy dissipation by the revetment stairs upon wave-structure interaction. To quantify the reduction in wave overtopping in comparison to smooth slopes, influence factors for roughness (γ_f) were calculated, ranging between 0.43 and 0.73. It was established that γ_f is influenced by relative wave overtopping, the height of the revetment steps and wavelength. An empirical formula for γ_f is proposed with which the wave overtopping rate at stepped revetments can be determined. The results also revealed that previous small-scale studies underestimate γ_f measured in this study by 2-31 %. Small-scale wave overtopping measurements are thus likely subjected to scale effects.

  Individual overtopping volumes were described by a Weibull distribution, revealing a higher median shape factor (b=1.63) for stepped revetments compared to breakwaters, smooth dikes or vertical walls. However this finding on individual wave overtopping is based on only 12 tests and should therefore be further investigated.

This chapter gives insight in the hydraulic design aspects of stepped revetments and provides recommendations for the crest-level design of stepped revetments. With observations and measurements a deeper understanding is gained in the functioning of stepped revetments.

- **Chapter 5: Full-scale experimental study in wave reflection and run-up at stepped revetments**

  Coastal Engineering 172, 104045, 2021 doi.org/10.1016/j.coastaleng.2021.104045

  T. Schoonees, N. B. Kerpen , T. Schlurmann

  Building on the findings in Chapter 4, this chapter further investigates hydraulic design aspects of stepped revetments. A deeper understanding of wave-structure interactions contributes to recommendations for optimised stepped revetment designs. The wave reflection and wave run-up processes are important as they indirectly provide insights in the wave energy dissipation of stepped revetments. In addition, knowledge on wave reflection aids designers, as wave reflections may induce scour and affect navigation. Wave run-up estimates are valuable for crest-level designs, planning maintenance and designing nature-based adaptations for stepped revetments. Only few studies have focused on wave reflection from stepped revetments, while the wave run-up process on stepped revetments has widely been studied, yet only with small-scale wave flume tests and mostly with regular waves. Since the wave overtopping rates in small-scale wave flume tests underestimate those measured in full-scale experiments, these studies are likely affected by scale. To address these knowledge gaps, this chapter investigates wave reflection from and wave run-up on non-overtopped stepped revetments by means of full-scale flume experiments.

  Two cross-sections, each with a slope of 1:3, were studied with uniform step heights of 0.17 m and 0.50 m. Irregular waves were generated with wave heights (H_{m0}) up to 1 m and wave periods ($T_{m-1,0}$) up to 6.5 s. In addition, wave run-up was studied with regular waves with wave heights (H) up to 1.6 m and wave periods (T) up to 7 s. Surface elevations were measured by wave gauges. With a reflection analysis based on 4 wave gauges, incident wave heights and reflection coefficients were determined. Wave run-up heights were measured with a 2D laser scanner. Both measured wave reflection and wave run-up are lower compared to that of smooth slopes reported in literature. This reduction is caused by the revetment steps that dissipate wave energy during the run-up and run-down process. The stepped revetments with small steps (S_h = 0.17 m) experienced 55% higher reflections than from the large steps (S_h = 0.50 m) under the same wave conditions. Wave reflection coefficients at stepped revetments are also influenced by the breaker parameter, together with the wave conditions in relation to step geometry. The results indicate that wave run-up on stepped revetments is predominantly influenced by the incident wave height. However, wave run-up is also influenced by the relation between wave conditions and step geometry. As was found for wave overtopping, wave run-up measurements in small-scale are likely affected by scale. Empirical predictions derived from previous small-scale model tests underestimate the measured relative run-up heights by 31-51%. Based on the measurements and findings, empirical formulae were derived to estimate wave reflection coefficients and wave run-up heights of stepped revetments. In addition to quantifying the hydraulic responses of non-overtopped stepped revetments, this chapter provides a deeper physical, process-based understanding of how waves interact with stepped revetments, thereby contributing to recommendations for optimised stepped revetment designs.

The publications presented in this dissertation build on prior co-authored studies on stepped revetments conducted in small-scale. These studies (including their abstracts) are listed below:

- **Wave overtopping prediction of a gentle sloped stepped revetment** *Conference Proceedings of International Conference on Coastal Engineering* 2018, doi.org/10.9753/icce.v36.papers.99

  T. Schoonees, N.B. Kerpen, T. Schlurmann

  Stepped revetments are multi-functional coastal structures offering protection against flooding. Despite the fact that these structures have been implemented for more than 60 years, comprehensive design guidance is lacking. Previous research studied overtopping of stepped revetments with slopes ranging between 1:1 to 1:4. To address the knowledge gap of predicting overtopping of stepped revetments with gentler slopes, this paper presents results of physical model tests for a 1:6 sloped stepped revetment with step heights of 0.05 m. The tests were conducted in a 110 m long, 2.2 m wide and 2.0 m deep wave flume. A fit through the overtopping results is compared with the reference curve for a smooth slope of EurOtop (2016), which allows the determination of the influence factor for roughness (γ_f) of the stepped revetment. A value of $\gamma_f = 0.74$ (r^2 of 0.94) is proposed to be used in combination with the overtopping prediction formula of EurOtop (2016) for slopes under breaking wave conditions. The results of the study indicate a high slope dependency for γ_f.

- **Wave impact pressures on stepped revetments**

  Journal of Marine Science and Eng. 6, 156, 2018 doi.org/10.3390/jmse6040156

  N. B. Kerpen, T. Schoonees, T. Schlurmann

  Horizontal and vertical wave forces on stepped revetments are investigated by means of small-scale physical model tests in a wave flume. Wave impacts on revetments with relative step heights of $0.3 < H_{m0}/S_h < 3.5$ and a constant slope of 1:2 are analysed with respect to (1) the probability distribution of the impacts, (2) the time evolution of impacts, and (3) the position of the maximum impact. The validity of the log-normal probability distribution for the largest wave impacts is experimentally verified for stepped revetments. The wave impact properties for stepped revetments are compared with those of vertical seawalls, showing that their impact rising times are within the same range. The impact duration for stepped revetments is shorter and decreases with increasing step height. Maximum horizontal wave impact loads are about two times larger than the corresponding maximum vertical wave impact loads. Measurements are compared with findings from literature for stepped revetments and vertical walls. A prediction formula is provided to calculate the maximum horizontal wave impact at stepped revetments.

- **Wave overtopping of stepped revetments**

  Water 11, 1035, 2019 doi.org/10.3390/w11051035

  N. B. Kerpen, T. Schoonees, T. Schlurmann

  Wave overtopping—i.e., excess of water over the crest of a coastal protection infrastructure due to wave run-up—of a smooth slope can be reduced by introducing slope roughness. A stepped revetment ideally constitutes a slope with uniform roughness and can reduce overtopping volumes of breaking waves up to 60 % compared to a smooth slope. The effectiveness of the overtopping reduction decreases with increasing Iribarren number. However, to date a unique approach applicable for a wide range of boundary conditions is still missing. The present paper: (i) critically reviews and analyzes previous findings; (ii) contributes new results from extensive model tests addressing present knowledge gaps; and (iii) proposes a novel empirical formulation for robust prediction of wave overtopping of stepped revetments for breaking and non-breaking waves. The developed approach contrasts a critical assessment between a smooth slope on the one hand and a plain vertical wall on the other. The derived roughness reduction coefficient is developed and adjusted for a direct incorporation into the present design guidelines. Underlying uncertainties due to scatter of the results are addressed and quantified. Scale effects are highlighted.

1.4 Outline

This dissertation consists of 7 chapters. Chapter 2 describes environmental aspects for designing sustainable hard coastal structures based on literature research. The described environmental design aspects include a discussion on environmental impacts of hard coastal structures, suggestions for compensating these impacts and recommendations for nature-based adaptations to enhance the ecological value of hard structures. Chapter 3 identifies environmental design aspects specifically for stepped revetments and is based on the knowledge gained in Chapter 2. The chapter describes the impacts that stepped revetments have on coastal processes and habitats as well as presents recommendations on developing environmentally sensitive designs for stepped revetments. Chapters 2 and 3 respectively address the first two research objectives related to environmental designs aspects of hard coastal structures and stepped revetments.

Chapters 4 and 5 address the last research objective with the focus on hydraulic design aspects of stepped revetments. With full-scale wave flume tests, the hydraulic responses of stepped revetments are investigated. The investigated hydraulic responses include wave overtopping, wave reflection and wave run-up. Chapters 4 and 5 describe the conducted wave flume tests, analyse measurements of the hydraulic responses and develop empirical formulae for estimating these responses. Chapter 4 is concerned with wave overtopping rates and individual overtopping volumes at stepped revetments, while Chapter 5 investigates wave reflection and wave run-up on stepped revetments.

Chapter 6 discusses the research findings of this dissertation. The discussion places the results in context and describes how the research findings are related. In addition, Chapter 6 highlights the research limitations and gives recommendations for future research. Lastly, Chapter 7 presents the conclusions of the dissertation.

Chapter 2

Hard Structures for Coastal Protection, Towards Greener Designs

This chapter is originally published in:

Estuaries and Coasts 42, T. Schoonees, A. Gijón Mancheño, B. Scheres, T. J. Bouma, R. Silva, T. Schlurmann, H. Schüttrumpf, 2019. **Hard structures for coastal protection, towards greener designs**, p. 1709-1729, Copyright Coastal and Estuarine Research Federation (2019), doi.org/10.1007/s12237-019-00551-z

2.1 INTRODUCTION

Predictions indicate that the percentage of world population living in coastal areas will continue to increase (Neumann et al., 2015). Concurrently, the effects of climate change, i.e. regional sea level rise and its consequences, pose risks to coastal communities and coastlines. As a result, coastal safety is of increasing importance (Temmerman et al., 2013; Firth et al., 2014). To ensure acceptable levels of coastal safety, coastal engineering solutions are implemented which can be classified into grey and green infrastructure (Fig. 2.1).

Grey infrastructure refers to conventional hard solutions (e.g. seawalls, dikes or breakwaters), while green infrastructure can be classified into three subcategories, namely (1) environmental-friendly grey infrastructure, (2) hybrid infrastructure and (3) soft infrastructure. Whereas soft solutions are typically green infrastructure, hard solutions can also be regarded as green infrastructure (under the label of environment-friendly grey infrastructure; Fig. 2.1) if they are designed to contribute to restoring, conserving or mitigating the loss of ecosystem services (David et al., 2016; Van der Nat et al., 2016; Pontee et al., 2016; Silva et al., 2017). This can be achieved if the infrastructure mimics ecosystem functions (e.g. habitat provision) or induces or preserves connectivity as a natural system would (Silva et al., 2017). Thus, adaptations may be made to conventional grey infrastructure to increase their value as green infrastructure by allowing organisms to settle, and thereby offering ecosystem services (Borsje et al., 2011; Firth et al., 2014). The combination of coastal habitat (e.g. salt marshes) with hard solutions, known as hybrid infrastructure, is another way to implement green infrastructure (Sutton-Grier et al., 2015; Pontee et al., 2016).

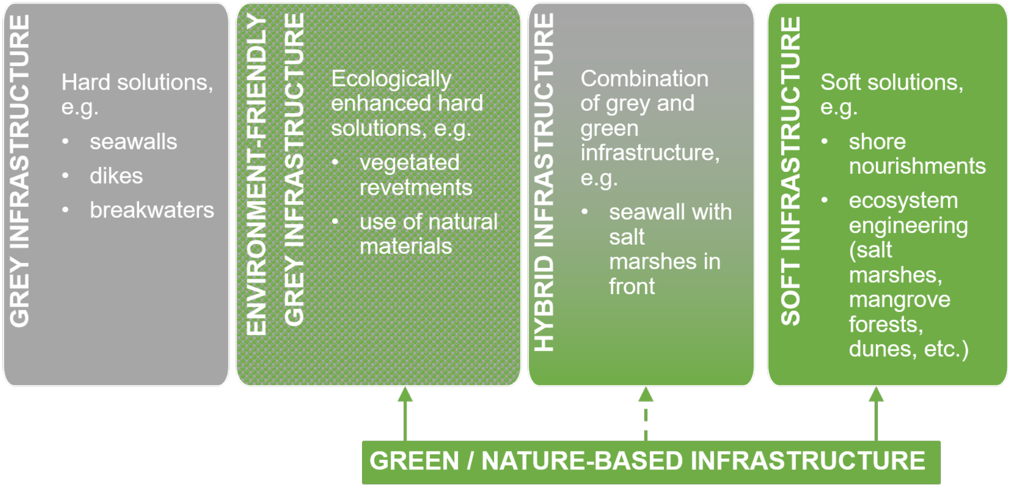

Figure 2.1: Definition of infrastructure types

Hard solutions, such as breakwaters, seawalls and dikes (Fig. 2.1), have been employed to offer coastal safety for years. Their designs are broadly based on recommended best practices and design guidelines developed over the last decades, e.g. in the Shore Protection Manual (CERC, 1984), Coastal Engineering Manual (USACE, 2002), Overtopping Manual (EurOtop, 2018), International Levee Handbook (CIRIA, 2013) or the Rock Manual (CIRIA, 2007); see Tab. 2.2. It is expected that the number of hard coastal structures will continue to rise in response to climate change (Moschella et al., 2005; Chapman and Underwood, 2011; Firth et al., 2013, 2016b). Simultaneously, with increasing awareness of the need for sustainable development and mitigating environmental impacts, implementing soft solutions, such as shore nourishments, has been increasingly considered (Capobianco and Stive, 2000). Consequently, the following questions arise: (i) where and when are hard structures still preferred over soft solutions and (ii) if hard solutions are needed, how can they be made into green infrastructure, taking into consideration ecological and engineering aspects?

These questions are answered with the aim of bridging gaps between disciplines and thereby enhancing future collaboration and the definition of joint objectives. Challenges between disciplines may lead to the unsuccessful implementation of green infrastructure. Coastal engineers often lack understanding of the far-reaching effects of coastal infrastructure and what constitutes successful nature-based solutions, while ecologists may need a greater awareness of the aspects required to ensure coastal safety and to determine acceptable risks. The first question of this review aims to clarify to ecologists how hard structures provide coastal safety and under which conditions these structures are suitable vs. unavoidable (Section 2.3). The focus of the second question aims to assist engineers in the consideration of the environmental impacts of hard structures (Section 2.4) and the incorporation of green infrastructure principles throughout the design process (Section 2.5).

This paper considers two groups/types of structures based on where they are employed. Breakwaters, groynes and jetties are grouped under the name of foreshore structures, while seawalls and dikes are classified as onshore coastal structures. The contents described are addressed through literature research and critically discussed in a review.

2.2 Guidelines and Policies on Nature-Based Coastal Engineering

The increasing recognition of green infrastructure has led to the development of policies and guidelines in a number of countries (e.g. the Netherlands, the UK, Germany and the USA) and international institutions. To give insight into existing approaches, Tab. 2.2 describes a selection of these guidelines and policies in terms of their main features and objectives.

In the Netherlands, the Building with Nature concept utilises the potentials of natural processes to develop multi-functional solutions that are aligned with the interests of nature and project stakeholders (De Vriend and Van Koningsveld, 2012). Similarly, the Engineering with Nature approach of the United States Army Corps of Engineers (USACE) is based on the intentional alignment of natural and engineering processes to facilitate economic, environmental and social benefits for water infrastructure through collaborative processes, such as stakeholder communication and engagement (USACE, 2012). On an international scale the Working with Nature philosophy of

PIANC, the World Association for Waterborne Transport Infrastructure likewise aims at an integrated, proactive process to find and exploit win-win solutions in terms of environment quality and engineering goals, for instance a breakwater that ensures safe navigation and creates habitat opportunities (PIANC, 2011).

A strategic approach for environmentally and economically sustainable management of coastal zones was introduced in 2002 by the European Parliament within their recommendation concerning the implementation of Integrated Coastal Zone Management in Europe (COM, 2002). In Germany, these recommendations were implemented within the National Strategy for Integrated Coastal Zone Management in Germany that again was adopted at federal state level, within the master plans for coastal protection (BMU, 2006). In the UK, sustainable policies for coastal management, such as the Managed Realignment Policy, are established through Shoreline Management Plans (SMPs) (DEFRA, 2006). Furthermore, Naylor et al. (2011) provide background on the principles of including ecological enhancements in hard coastal structures, such as incorporating rock pools in vertical seawalls (Chapman and Blockley, 2009) or using construction material with rougher surfaces (Moschella et al., 2005). Apart from general guidance, the guide offers a description of legal frameworks in the UK and Europe that supports these enhancements. In addition, Nesshöver et al. (2017) provide a review of the practice and policies of nature-based solutions within a European context.

Cheong et al. (2013) define the approach of Ecological Engineering that additionally considers societal functions and values as an integral part of nature-based solutions. The authors state that combined approaches to coastal adaptation, rather than a single strategy, such as seawall construction, is better preparation for the highly uncertain and dynamic coastal environment. They conclude that very few studies on practical implementations have examined the interactions, synergistic effects and co-benefits of combined approaches to adaptation.

The International Union for Conservation of Nature (IUCN, 2016) and the European Commission (2015) focus on nature-based solutions for infrastructure in general, whereas the other policies discussed focus more specifically on hydraulic and coastal engineering applications. Living Shorelines, an innovative US approach, presents nature-based shoreline stabilisation techniques for erosion control (SAGE, 2015).

Although the policies and guidelines in specific countries and institutions have different names, the desired outcomes of all are common: the use of natural processes and/or resources to achieve solutions that are socially, economically and environmentally beneficial. General guidelines are provided, while a need for detailed design guidelines in terms of effectiveness and performance of nature-based coastal solutions persists.

2.3 Structure Types and Their Design Conditions

This section presents a general description of two structure types (foreshore and onshore structures), and the design conditions relating to them. General design guidelines for the structure types are provided by different institutions, such as CIRIA (2007) and USACE (2002). A summary of the guidelines is included in Tab. 2.2, in terms of the structures included, design processes and the aspects covered.

Table 2.1: Summary of selected guidelines and policies on nature-based coastal engineering

Guideline/policy	Country/institution	Objective and main features
Building with Nature (De Vriend and Van Koningsveld, 2012)	Netherlands	Programme carried out by a consortium of private companies, government agencies and knowledge institutes. It aims to gather knowledge on ecosystems and to develop design rules for infrastructure in alignment with natural processes. This is done though pilot projects, which are implemented and monitored. Design guidelines are then developed based on the analysis of the gathered data.
Engineering with Nature (USACE, 2012; Bridges et al., 2015)	United States of America	Aims to incorporate natural features and nature-based features (created by human design) into traditional engineering. It presents a framework to identify stakeholders, analyse the system and its vulnerability and propose and evaluate different alternatives, accounting for ecosystem service creation and comparing them through performance metrics. The report includes practical examples of the tools and evaluation methods.
Living Shorelines (SAGE, 2015)	United States of America	Living shorelines presents coastal solutions that protect against storms and erosion while providing ecosystem services and preserving ecosystem connectivity. It considers different alternatives, consisting of vegetation alone or in combination with nourishments, stone fills and other structures. The alternatives are classified based on the environments where they are suitable, costs, advantages and disadvantages.
Shoreline Management Plans (DEFRA, 2006)	United Kingdom	Developing strategies for reducing the threat of flooding and erosion that are as far as possible beneficial to the environment, society and the economy. The environmental objective is to maintain, restore or where possible, improve environmental and historic assets.
National Strategy for Integrated Coastal Zone Management in Germany (BMU, 2006)	Germany	Balancing environmental, economic and social needs by developing and preserving coastal areas as ecologically intact, economically valuable and socially acceptable habitats.
Working with Nature (PIANC, 2011)	PIANC	Developing solutions for waterborne transport infrastructure, which are mutually beneficial to project and environmental stakeholders. Advocates an integrated proactive approach from conception to project completion, which deliver project objectives by maximising opportunities for nature, rather than minimising ecological impacts.

2

Guideline/policy	Country/institution	Objective and main features
Nature-Based Solutions and Re-Naturing Cities (European Commission, 2015)	European Commission	Solutions, which are inspired, supported or copied from nature and that provide environmental, social and economic benefits. Wide range of application areas. Aiming at: (1) enhancing sustainable urbanisation, (2) restoring degraded ecosystems, (3) developing climate change adaptation and mitigation, and (4) improving risk management and resilience.
Nature-Based Solutions (IUCN, 2016)	International Union for Conservation of Nature	Ecosystem-based measures to address societal challenges by protecting, managing or restoring ecosystems and making use of their services. Not restricted to one special application area. Includes different approaches, e.g. ecosystem restoration, ecosystem-based infrastructure, ecosystem-based management.
Ecological Engineering (Cheong et al., 2013)	Miscellaneous	Coastal adaptation strategies combining engineering structures and ecosystems under consideration of societal functions and values to use synergies of the different measures striving for increased and more flexible coastal protection.

2.3.1 Foreshore structures

Traditionally hard structures, such as groynes and breakwaters, are built on the foreshore to prevent or mitigate erosion (Hamm et al., 2002). A schematic overview of such structures and their effects on the physical environment is shown in Fig. 2.2.

Offshore breakwaters are built parallel to the shore and designed either to protect the coastline or to improve the recreational conditions behind them (Pilarczyk and Zeidler, 1996). They may be constructed from materials such as concrete, rocks, sand bags or geotextiles and are designed to allow a certain amount of wave transmission by flow through the porous structure or overtopping. Part of the wave energy is reflected seawards, and part is dissipated by wave breaking on the slope or by friction losses inside the breakwater body (Pilarczyk, 2003). The sheltering effect of breakwaters provides protection against storms but also affects the morphodynamics of the coast. The changes in the hydrodynamics induced by the presence of the structure (Mory and Hamm, 1997) alter the gradients in sediment transport and thus cause morphological changes (Van Rijn, 2013). The decrease of the longshore current behind the breakwater may induce a pattern of accretion on the updrift side, while the increase in flow velocity as the current leaves the sheltered area may result in erosion downdrift of the structure. Wave diffraction tends to increase erosion at the sides of the breakwater, and deposition in the middle part, since the diffracted waves curve towards the sheltered area and reduce in height inside of it (Hsu and Silvester, 1990). Wave-induced set-up currents are caused by the changes of wave height away from and behind the breakwater. Variables, such as the number of structures, the distance between them (along the shore), their distance to the coastline, the structure crest height and the width of the surf zone, may induce different patterns of erosion and accretion (Ranasinghe and Turner, 2006; Suh

Table 2.2: Summary of selected design guidelines for coastal structures

Guideline	Structure type	Country	Content
Shore Protection Manual (CERC, 1984)	Seawalls, revetments, groynes, breakwaters and jetties	International	Structure types, functions and limitations; Design conditions and practices; Construction materials
Recommendations for marine works [in Spanish] (ROM, 2009)	Seawalls, revetments, groynes, breakwaters and jetties	Spain	Design guidance (planning, site conditions and data collection, geometry, building materials, construction, environmental considerations)
The Rock Manual (CIRIA, 2007)	Rock structures including breakwaters, groynes, detached breakwaters, revetments and seawall toe	International	Structure types, functions and failure mechanisms; Design guidance (planning, site conditions and data collection, geometry, building materials, construction, environmental considerations); Maintenance and planning
Coastal Engineering Manual (USACE, 2002)	Seawalls, revetments, groynes, breakwaters (various types) and jetties	International	Structure types, functions and failure mechanisms; Design guidance (planning, site conditions, data collection, geometry, building materials, construction, environmental considerations); Maintenance and planning
Manual on wave overtopping of sea defences and related structures (EurOtop, 2018)	Coastal dikes, revetments, seawalls and breakwaters	International	Wave overtopping limits; Wave run-up and overtopping prediction including influence factors
The International Levee Handbook (CIRIA, 2013)	Coastal, estuarine and river dikes	International	General information (functions, forms and failure mechanisms); Design guidance (from site characterisation and data collection over design to construction); Maintenance and planning
Design Guidance for Coastal Structures [in German] (EAK, 2002)	Coastal and estuarine dikes	Germany	Loads on coastal structures; Geotechnical investigations and processes; Building materials; Design and construction

and Dalrymple, 1987).

Groynes are structures that extend towards the sea perpendicularly, or obliquely, to the shoreline (USACE, 2002) and are usually constructed of timber, rocks or concrete. Groynes prevent, or slow down, the longshore sediment transport, resulting in accretion of sediment on the updrift side of the structure and erosion on the downdrift side (Van Rijn, 2013), as illustrated in Fig. 2.2. Groups of groynes have been used to stabilise a stretch of coastline or for land reclamation purposes (Nienhuis and Gulati, 2002). Jetties are also structures which are perpendicular to the coast, but these extend further seaward and have the purpose of protecting navigational channels. They divert tidal currents offshore and restrict the lateral transfer of sediment, reducing channel dredging costs (Van Rijn, 2013).

Hard structures are designed to ensure structural integrity under extreme environmental conditions (not exceeding their design conditions). Different modes of failure are considered in the design, such as the erosion or breakage of the armour layer or geotechnical stability, i.e. sliding or settlement of the structure (e.g. Losada, 1990). The amount of overtopping under normal conditions is limited to ensure the operability in the area sheltered by the structure. Further information on the existing design guidelines is included in Tab. 2.2.

Foreshore structures have been seen to cause adverse longterm erosion effects on adjacent shores or nearby. These effects can be aggravated by implementing hard structures in unsuitable conditions or in cases where erosion is caused by socioeconomic practices, such as sand mining. Soft engineering approaches, such as sediment nourishments, can be applied alone, or in combination with traditional solutions (Capobianco and Stive, 2000; Hamm et al., 2002). The periodic addition of sediment has been used as a way to counteract structural erosion or in by-passing schemes between groyne compartments.

Although hard foreshore structures and sediment nourishments may increase wave energy dissipation and mitigate erosion in medium- to high-energy environments, they do not provide protection against storm surge and flooding. When higher levels of protection are required, they can be complemented or replaced by shoreline structures such as seawalls or sea dikes.

2.3.2 Onshore structures

Seawalls structures

Seawalls are shore-parallel, onshore structures designed to protect the coastal hinterland from overtopping and flooding. These structures fix the position of the shoreline, thereby preventing it from retreating landward. Seawalls can be vertical, curved, stepped or sloping structures. Revetments are sloping structures which cover the shoreline profile to offer erosion protection. Although there is a functional difference between seawalls and revetments (i.e. seawalls prevent inland flooding, while revetments prevent shoreline erosion), there are no distinct structural differences. Hence, in this paper, the collective term "seawalls" will be used. Seawalls are often considered the last line of defence on the coast and are typically constructed in areas where the natural protection of sandy or muddy beaches, ecosystems and/or other foreshore structures no longer provide an acceptable level of protection for landward

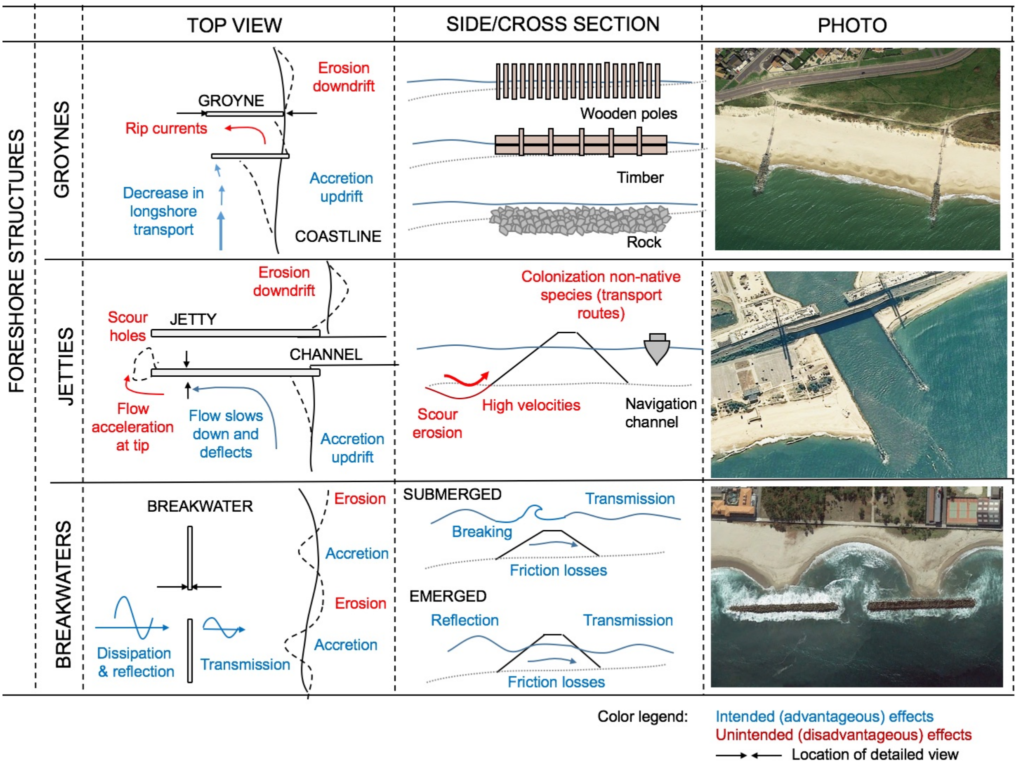

Figure 2.2: Overview of hydrodynamic and morphodynamic effects of groynes, jetties and breakwaters. The left column has schematic plans of each structure type and its effects. The black arrows indicate the location of detailed views of the structures, shown in the central column. The right column shows actual examples of these structures (photos: GoogleEarth, Landsat/Copernicus (groynes and breakwaters); Google Earth, USDA Farm Service Agency (jetties)

infrastructure, such as promenades and roads.

Various types of seawalls, ranging from vertical to sloping, have been constructed on shorelines around the world using a variety of construction materials: reinforced concrete, rocks, concrete armour units, rock-filled gabions, steel piles, wood and sand-filled bags. Seawalls are often classified as permeable (rock revetment; Fig. 2.3) or impermeable, designed either to absorb or to reflect the wave energy. General design guidelines are offered by CIRIA (2007) and USACE (2002), while EurOtop (2018) provides guidance on crest level design, Tab. 2.2. Seawalls are suitable for an open coast, exposed to high storm surges and large waves. They have the advantage of requiring little space, which makes them suitable for coastal defence in urban areas.

As seawalls form rigid barriers, they affect and alter coastal processes (see Section 2.4.1 and Fig. 2.3). A seawall often leads to increased wave reflection, which may result in scour, due to the (partially) standing waves in front of the structure. This makes seawalls, especially impermeable ones, likely to cause sediment loss and in turn leads to structural instabilities (USACE, 2002). To counteract the potential erosion effects, seawalls are

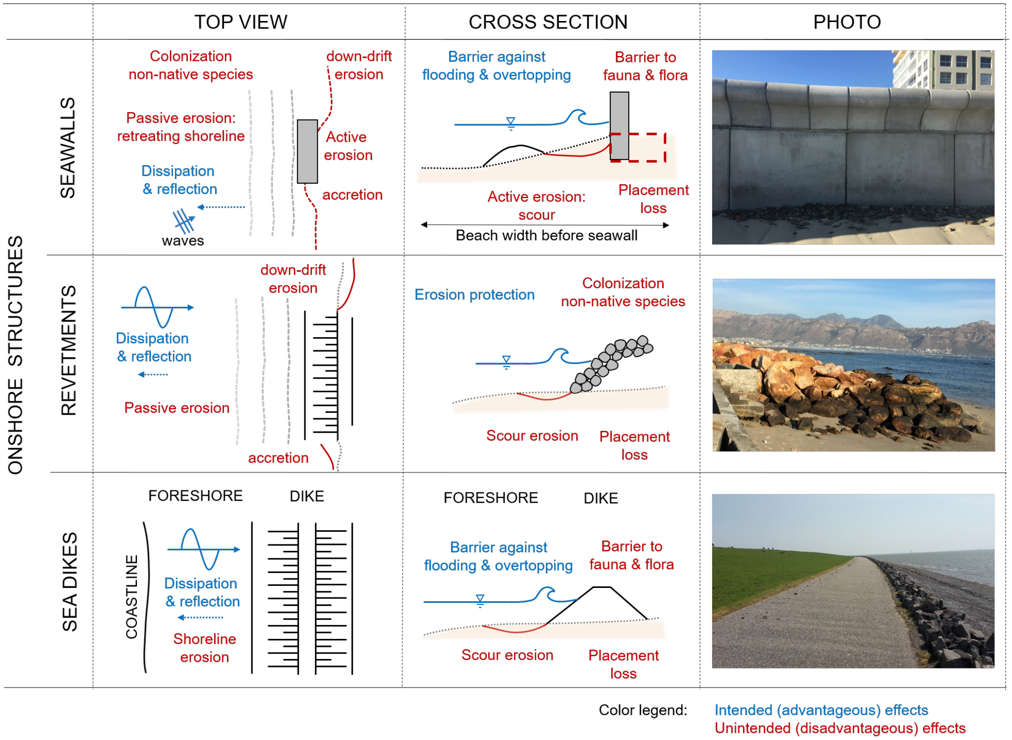

Figure 2.3: Overview of grey onshore coastal structures and their hydrodynamic and morphodynamic effects

often implemented in combination with foreshore structures such as groynes and shore nourishments, see Section 2.3.1. Apart from scour, other typical damage modes that have to be considered in the design of seawalls include overtopping, flanking (Section 2.4.1), rotational sliding, overturning and corrosion of reinforcement steel (USACE, 2002). A schematic overview of seawalls and their effects on the physical environment is shown in Fig. 2.3.

These structures provide short- to long-term protection (e.g. a sand-filled bag seawall: up to 5 years, a seawall with concrete armour units: up to 100 years). However, seawalls often cause a sudden shift in ecological balances at the site.

Sea dikes

As with seawalls, sea dikes are considered a last line of flood defence. Also known as embankments or levees, sea dikes are shore-parallel coastal barriers which protect the low-lying hinterland from flooding, thanks to their raised ground level. A schematic overview of sea dikes and their effects on the physical environment is shown in Fig. 2.3 below

Generally, dikes are composed of an earth-filled core with smooth slopes on both seaward and landward sides. Fig. 2.4 shows a typical dike cross-section with its design water level (DWL) and the mean high water (MHW). Sealings, revetments, berms, crest

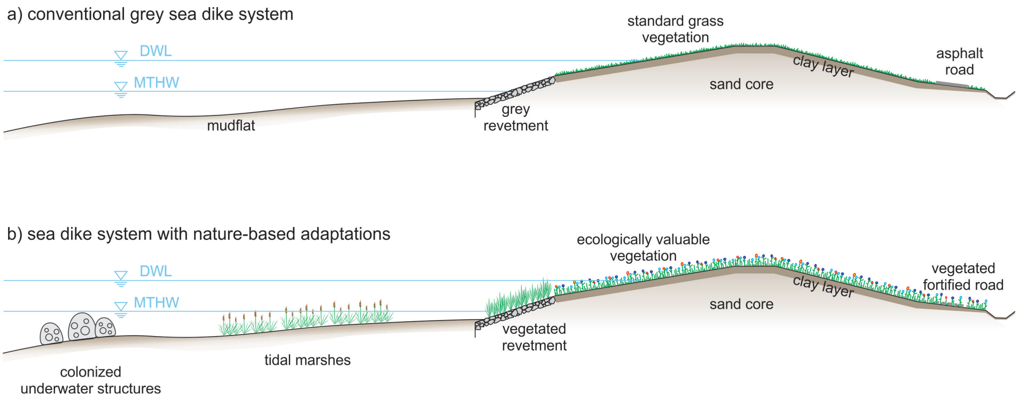

Figure 2.4: **(a)** Conventional grey sea dike system. **(b)** Sea dike system with nature-based adaptations

walls etc. can complement the basic structure. If the hydraulic loads are low, sea dikes are covered with a vegetated clay layer. In the case of higher loads, i.e. due to breaking wave impacts, hard revetments, e.g. rock filled, are applied to induce friction and increase energy dissipation, thereby ensuring dike safety (EAK, 2002; CIRIA, 2013). Following storm surge experiences, sea dikes in Germany were heightened and broadened, resulting in the current characteristic dike profiles, with seaward slopes of between 1:3 and 1:7 and landward slopes of 1:2 to 1:5 (Schüttrumpf, 2008). The footprint of a dike depends on the required crest height and slope angles, which in turn are determined by optimising dike stability (smooth slope), considering space restrictions and material consumption (CIRIA, 2013).

To ensure the main function of coastal protection, dikes must be designed in such a way that they resist external loads (high water levels, waves, currents, human activities etc.) and consequent damage from erosion and mass instability (sliding). Design methods (Tab. 2.2) consider hydraulic, morphological and geotechnical boundary conditions. Besides state-of-the-art design and construction, regular monitoring and maintenance are indispensable for dike safety (EAK, 2002; CIRIA, 2013). Sea dikes are generally built for long-term coastal protection, with a long design life, e.g. 50 years in the Netherlands (Pilarczyk, 2017).

2.4 Environmental Impacts and Concerns of Grey Coastal Infrastructure

Environmental impacts depend on many site-specific aspects, related to structural design as well as ecological and physical conditions of the local system (Chapman and Underwood, 2011; Nordstrom, 2014). The following section discusses the general impacts of the structures defined in the Section 2.3, based on (i) morphologic and hydrodynamic changes and (ii) changes in ecological communities, due to habitat modifications related to altered morphodynamics, substrate type and availability. Knowledge of environmental impacts and how they evolve through the lifetime of a structure is still very limited. Systematic interdisciplinary research is thus required to fully incorporate impact mitigation in future infrastructure design.

2.4.1 Morphologic and Hydrodynamic Changes

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The implementation of any coastal structure brings morphologic and hydrodynamic changes, such as alterations to wave regime, sediment dynamics and depositional processes (Dugan et al., 2011). Sometimes, they produce unintended morphodynamic effects, which may lead to structural deteriorations or sediment budget imbalances.

Wave reflection by emerged shore-parallel structures increases wave energy in front of the defence, increases scouring, and may result in erosion and the loss of intertidal areas (Douglass and Pickel, 1999; Winterwerp et al., 2013). Submerged detached breakwaters became popular because of their low visual impact, but the lack of understanding of their hydraulic behaviour often resulted in erosion problems; a literature review conducted by Ranasinghe and Turner (2006) concluded that the majority of existing submerged structures have induced shoreline erosion on their lee side. Sediment trapping by shore normal structures produces shoreline advance on one side, but induces coastline retreat downdrift of their position. Moreover, currents are deflected seaward, which may cause a loss of sediment offshore and a danger for recreation activities, if the flow velocities are high enough (Scott et al., 2016). In addition, flow contraction at the end of shore normal structures leads to flow acceleration and scour at the tip (Lillycrop and Hughes, 1993). Fig. 2.2 illustrates the morphologic and hydrodynamic changes due to groynes, jetties and breakwaters.

Periodic shore nourishments on the foreshore, or beach face, may mitigate structural erosion problems induced by hard structures. However, they produce environmental effects, such as biota burial and an increase in turbidity and sedimentation, at both the site of extraction and placement. Additionally, it is improbable that any benthic fauna which survives the dredging process will be capable of reestablishment on the seabed (ICES, 2016). The severity of the impact and the ability of the system to recover has not been systematically scrutinised, but depends on the intensity, duration and exposure of these effects and on the tolerance of the local species (Erftemeijer and Lewis III, 2006; Erftemeijer et al., 2012).

Seawalls and sea dikes are rigid structures that act as a boundary between the land and the sea. Generally, their physical impacts on a beach increase the lower on the beach profile they are built (Dugan et al., 2011). These impacts can be classified as (1) placement loss, (2) passive erosion and (3) active erosion (Griggs, 2010). Placement loss refers to the portion of beach that is lost when a structure is built, thus depending on the footprint of the structure. Although seawalls and dikes limit the extent of shoreline retreat, beach erosion in front of these structures continues on coasts subject to long-term net erosion. Consequently, the shoreline retreats towards (and possibly beyond) the structure, which results in gradual beach loss over time, known as passive erosion. This erosion is not a result of the structure itself, but continues as the structure does not address the underlying problems causing the erosion (Griggs, 2010). Where no structure is present, the beach width would remain mostly the same, but the beach would shift further landwards with time (Dean, 1987).

Active erosion occurs due to the interaction of the structure with coastal processes (Kraus and McDougal, 1996). Since seawalls and sea dikes may create a cross-shore barrier effect by blocking the sand movement between the beach and the backshore, they consequently affect aeolian sand transport, and as a result, the natural dune system (Jackson and Nordstrom, 2011; Nordstrom, 2014). Moreover, the interruption of the sand supply from the backshore can potentially cause erosion at locations down drift of the seawall (Dean, 1987; Kraus, 1988; Nordstrom, 2014). Analogous to emerged breakwaters, seawalls and sea dikes can further increase wave reflection and increase scour in front of the structure (USACE, 2002). Locally increased erosion at the ends of a seawall, known as flanking, may also occur. Fig. 2.3 illustrates the morphologic and hydrodynamic changes due to seawalls and sea dikes.

2.4.2 Effects Due to Changes on Habitat

The morphologic and hydrodynamic changes of coastal structures also have an impact on local species. The flow acceleration and scour at the end of shore normal structures (Section 2.4.1) may hinder the attachment of sessile organisms in the scour zone, and inhibit subtidal life near the base of the outer part of the structure (Britton and Morton, 1989).

Loss of habitat is initially due to the placement loss and increases as the beach becomes narrower, as a result of passive erosion or scour due to active erosion (Dugan et al., 2011; Nordstrom, 2014). Seawalls and sea dikes produce intertidal habitat loss, since the area flooded by the tide is obstructed by their presence, subsequently affecting the local biota (Dugan et al., 2008; Nordstrom, 2014). The cross-shore barrier effect of onshore structures not only interrupts sand movement (Section 2.4.1), but may also hinder the movement of fauna (Nordstrom, 2014). Consequently, fauna seeking refuge in the dunes are affected (Lucrezi et al., 2010; Nordstrom, 2014) and inland habitat may be isolated, which also affects neighbouring habitats (Chapman and Underwood, 2011). Dugan et al. (2008) found that seawalls may produce a significant decline in the abundance, biomass and size of macroinvertebrates in the upper intertidal zone as well as in the species richness and abundance of shorebirds. Structures with slopes steeper than the natural shore offer reduced habitat area, resulting in a loss of local biodiversity and finally leading to a reduction in the total population size in a region (Chapman and Underwood, 2011). In intertidal areas, a steeper slope means that species that used to live in different vertical zones are now located much closer together, causing changes in ecological interactions (Dugan et al., 2011; Nordstrom, 2014).

Artificial coastal defences transform and often fortify softshore coastlines into static, hard structures, allowing colonisation by rocky shore species (Firth et al., 2014). When placed close to harbours or shipping routes, there is a higher risk of invasion by non-native species (neobiota), which could spread along a chain of groynes or breakwaters (Bulleri et al., 2006; Keith et al., 2011; Firth et al., 2012; Airoldi et al., 2015). Thus, the impacts of structures are not only confined to their location. Large-scale impacts may result from changes to ecological connectivity, which in turn affects biodiversity, as well as the ecosystem services in coastal zones (Dugan et al., 2011; Firth et al., 2014; Bishop et al., 2017).

Seawalls introduce new hard substrata that are notably less dynamic than muddy, rocky or sandy habitats (Nordstrom, 2014). With reference to a number of studies, Chapman and Underwood (2011) reported that artificial habitat changes the mix of species, abundance, the size structures of population, reproductive output and competition or response to habitat. Sea dikes with a vegetated dike cover generally introduce less hard substrata than seawalls and therefore any ecological impacts are assumed to be less compared to seawalls, although, as yet, no scientific evidence of this is known to the authors. Nevertheless, alterations in the biotic structure through the construction of sea dikes are still to be expected as the dike vegetation usually diverges from the natural species (e.g. standard seeding mixtures in (EAK, 2002)).

2.5 Possible Nature-Based Adaptations for Grey Coastal Structures

Although the detrimental effects of hard solutions cannot always be avoided, certain nature-based adaptations can be made to mitigate or reduce these effects. This section describes (i) ecosystem engineering options as complement to traditional hard solutions, (ii) measures to mitigate morphologic and hydrodynamic changes and (iii) possible adaptations to ecologically enrich conventional grey structures. Nature-based measures should ideally be considered early on in project planning, to limit construction costs and to allow their implementation over larger spatial scales (Firth et al., 2013).

2.5.1 Using Ecosystems as Part of the Foreshore Structures Fronting Seawalls and Sea Dikes

General Considerations in Using Ecosystems Along the Foreshore

In planning ecosystems along the foreshore, two main generic principles, related to the habitat requirements of species, should first be considered. The first main principle is that species are typically either aquatic (seagrass meadows, mussel reefs, coral reefs) or terrestrial (e.g. salt marshes, mangroves), respectively, depending on whether they function the best underwater, tolerating regularly being shortly emerged from water, or whether they function best above water, tolerating flooding for part of the day. In general, species living higher in the intertidal system will be more efficient in attenuating waves (Bouma et al., 2014), as the surface area and structures they build are more likely to affect a larger part of the waves (see Bouma et al. (2014) for example calculations). The second main principle to keep in mind is that establishment and survival of species and ecosystems is typically limited by the maximum hydrodynamic forcing they can resist. This means that in green designs, the species should be selected based on their tolerance to hydrodynamic forcing. For example, corals and beach systems occur at higher energy levels than marshes and mangroves. In cases where hydrodynamic forcing creates unfavourable conditions for marshes and mangroves species to survive, hard structures or shore nourishments may be needed.

Ecosystems as Coastal Protection

Field studies show that coastal habitats have a significant potential for reducing wave heights that varies for habitat and site (Narayan et al., 2016). In general, coral reefs and salt marshes have the highest potential, producing an average of 70 % and 72 % attenuation, respectively.

Coral reefs are effective as they form a sharp transition from deep to shallow water

(Ferrario et al., 2014), whereas intertidal vegetation like salt marshes and mangroves are effective since they grow high in the intertidal area and consequently increase bathymetric elevation Bouma et al. (2014).

Reef-forming species, such as natural oyster reefs, act as natural breakwaters by attenuating waves (Meyer et al., 1997) and promoting sedimentation (Henderson and O 'Neil, 2003). Shellfish reef restoration has been implemented at a number of locations, such as the Oosterschelde in the Netherlands. Shellfish reefs were built in 2008 to prevent sediment loss from the tidal flats to the channels. Monitoring showed there was erosion prevention and accretion at the reefs compared to other, exposed locations (De Vriend et al., 2014). Scyphers et al. (2011) compared the bathymetric changes and vegetation retreat behind artificial oyster reefs with control locations, where reefs were not present. They concluded that although the experimental reefs created new habitat, they did not provide as much coastal protection as conventional solutions. This was partly attributed to their mesh not being rigid enough for the levels of wave energy. Natural reefs could also be installed in combination with hard structures to mitigate environmental impacts, such as loss of fish and shellfish habitat (Scyphers et al., 2011).

Submerged aquatic vegetation, such as seagrass, attenuates local currents (Gambi et al., 1990), dampens wave energy (Knutson et al., 1982; Fonseca and Cahalan, 1992; Möller et al., 2014) and promotes sedimentation (Callaghan et al., 2010; Shi et al., 2012), subsequently acting as a buffer against flooding and erosion. Seagrasses have been observed to dissipate 40 % of the wave energy, but have also shown negligible effects on waves when the water depth was considerably greater than the leaf length (Fonseca and Cahalan, 1992). Thus, the ability of aquatic vegetation to dissipate wave energy depends on a combination of the hydrodynamic conditions and plant properties (Bradley and Houser, 2009). Typically, the flexible structures of aquatic vegetation make them less effective in attenuating waves than the stiffer structures associated with salt marshes and mangroves (Bouma et al., 2005), although this may be compensated for by having a very high biomass (Bouma et al., 2010). It should also be noted that seagrass meadows may even contribute to flood safety when vegetation is extremely sparse and short, due to the fact that the dense rhizome mats will preserve an elevated bed level (Christianen et al., 2013).

Intertidal vegetation, like salt marshes, enhances sediment trapping and creates a platform that reduces the hydrodynamic loads behind them. Laboratory experiments by Möller et al. (2014) compared wave attenuation under storm surge conditions, with and without salt marshes, and observed that 60 % of wave height reduction was attributed to the effect of the vegetation. Marsh creation can be conducted by displacing historical dikes landwards or building sluices at existing dikes, subsequently allowing tidal flooding behind the structures (Temmerman et al., 2013). The cyclic dynamics of ecosystems like salt marshes means maintaining sufficient marsh width for coastal protection is difficult (Bouma et al., 2014, 2016). Measures that stimulate sediment accretion on the tidal flat fronting the marsh typically stimulate marsh expansion (Bouma et al., 2016; Cao et al., 2018). Mangrove ecosystems perform similar functions (protection against storms and sediment trapping) in tropical areas. Waves can be reduced between 13 and 66 % over 100 m of mangrove forest (McIvor et al., 2012). Bamboo and brushwood structures have been used in restoration schemes at eroding coastlines to provide the morphodynamic conditions required for mangroves to establish (Winterwerp et al., 2013; Schmitt et al., 2013). This approach goes back to the kwelderwerken (salt marsh works), where marshes were historically created using brushwood dams to enhance sediment accretion on the tidal flats fronting the marsh (Dijkema, 1987).

Even though coastal ecosystems provide multiple provisional and regulating ecosystem services, such as wave dampening or sediment trapping (Murray et al., 2002; Koch et al., 2009), it is still not known how to realistically design and assess green infrastructure (Costanza et al., 1997; MEA, 2005). Verified parametrisation of the efficiency and performance of green infrastructure is lacking, e.g. wave attenuation depending on coastal system boundary conditions.

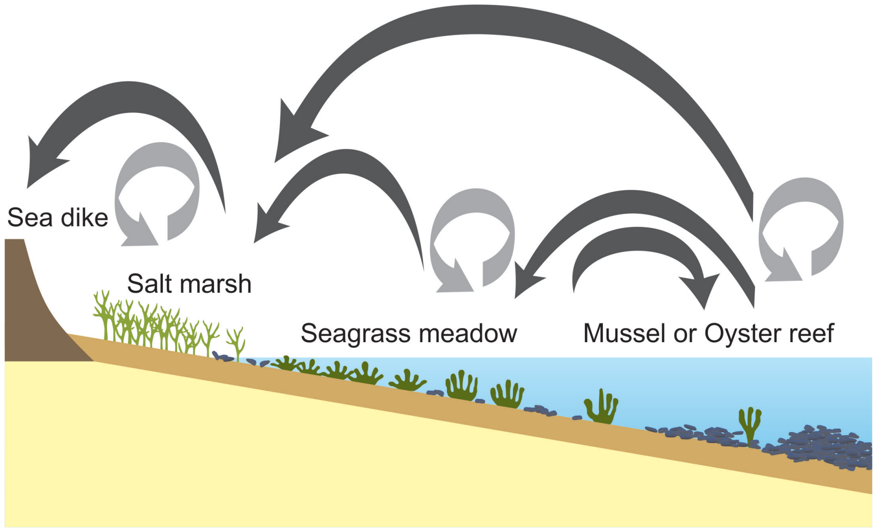

Figure 2.5: Gradient of ecosystem engineering that stabilise the sediment and thereby protect the coast. Arrows indicate positive interactions. Adapted and translated from De Vries et al. (2007) and Van Katwijk and Dankers (2001)

Towards an Integrated Approach by Combining Multiple Ecosystems

Although ecosystems located high in the intertidal zone, like salt marshes and mangroves, may contribute much more to flood safety than systems located lower, this does not mean the latter are unimportant. They can play an important role in stabilising and accreting upward the lower part of a foreshore. In doing so, these low intertidal ecosystems create favourable low-energy high-elevation conditions that allow the intertidal systems to expand (Bouma et al., 2014). This kind of mutual facilitative interaction was first suggested by De Vries et al. (2007); (see Fig. 2.5) and later demonstrated for tropical coastal ecosystems by Gillis et al. (2014). Using a set of ecosystems rather than any single ecosystem as a foreshore system may thus provide a more resilient defence.

The spatial requirements of ecosystem engineering restrict its application to locations where there is sufficient space between urbanised areas (Temmerman et al., 2013). Besides this, although coastal vegetation (wetlands, dune vegetation and seagrass) may dampen part of the wave energy, it is not an impermeable barrier and it will be flooded during extreme high water. Coastal vegetation cannot stop storm surges, which behave similarly to tidal forces that penetrate the vegetation and raise water levels over several hours (Feagin et al., 2010). Large waves can also propagate further onshore during these extreme water levels. Due to the limitations in wave dampening and the potential detrimental effects of high-energy events on the ecosystems (SAGE, 2015), ecosystem engineering is most suitable in environments of relative low energy. Whenever these conditions are not met, engineering measures may be needed to achieve the goals.

The long-term stability of coastal vegetation is an uncertainty that is still relatively poorly understood, which may hamper its implementation (Bouma et al., 2014). Fortunately, adaptations can also be implemented retrospectively, or incrementally, as discernible changes in design loads or other previously uncertain boundary conditions arise (such as sea level rise or increases in wave height impacts due to local morphological alterations).

2.5.2 Impact Mitigation for Morphologic and Hydrodynamic Changes

To ensure minimal physical effects, recommended design guidelines and standards (Tab. 2.2) should be followed. Numerical models predicting the shoreline response of coastal structures can help to optimise designs for mitigating physical changes on the adjacent coast. Moreover, monitoring of the physical impacts throughout the lifetime of the structure is important to identify unintended morphologic and hydrodynamic changes (USACE, 2002).

The morphodynamic effects of foreshore structures can be reduced or mitigated in different ways. The undermining of structures due to scour can be prevented by appropriate scour protection design. However, although typical scour protections of rock or concrete would create new habitat, they would not compensate for the loss of the soft habitat they are placed on. Alternatively, structures of porous materials could produce less scour than impermeable structures. The impacts of erosion could be minimised in the design phase by increasing the structure porosity or in the dimensioning of the structures. An alternative is to mitigate the impacts of the design by shore nourishments or sand bypass systems (USACE, 2002). However, sediment nourishments also produce negative environmental effects, as described in Section 2.4.1.

The physical impacts of seawalls can potentially be reduced by burying the structure, or changing its location on the cross-shore profile. In USACE (2002), an example is presented where a rock seawall/revetment was completely buried under a beach and dune, which meant that the revetment would only become exposed during storm conditions. This proved to be more environmentally beneficial and had lower costs than a conventional rock revetment and could also be expected to have less adverse physical effects. Since the physical impacts of seawalls on a beach increase the lower they are located on the beach profile (Dugan et al., 2011), seawalls should be built as high as possible on the beach profile. In contrast, to promote habitat variety and diversity, the structure should be placed as low down in the intertidal zone as possible (Firth et al., 2014). The barrier effect of seawalls can possibly be reduced by modifying or lowering certain stretches along the seawall to restore sediment movement and ecological connectivity between the beach and the backshore (Nordstrom, 2014).

The position of a sea dike influences how much wave energy reaches the dike and consequently how much the coastal processes are altered by the interaction with the dike structure. However, the process of selecting sea dike alignment involves various aspects, such as geomorphological and hydraulic conditions or land use, and is usually a compromise between the different issues (CIRIA, 2013).

Adaptations of the dike cross profile, such as the concept of wide green dikes with smooth seaward slopes (Van Loon-Steensma et al., 2014), can provide ecological enhancements and simultaneously reduce the effects on coastal processes, e.g. reflections at the structure (Dugan et al., 2011).

2.5.3 Ecological Enriching the Sub- and Intertidal Hard Substrates of Coastal Structures

As recommended by Firth et al. (2014), the selection of nature-based adaptations should be preceded by a thorough definition of the objectives and outcomes they should provide. A wide range of techniques for the ecological enhancements of artificial structures is described in literature (see Firth et al. (2016b) for a critical review) and is highlighted here for foreshore structures, seawalls and dikes.

Impact Mitigation of Hard Foreshore Structures

Heterogeneity, from surface roughness to larger irregularities in a structure, offers a greater variety of habitats and promotes higher biodiversity compared to a smooth surface (Firth et al., 2012; Martins et al., 2010; Hall et al., 2018; Chapman and Underwood, 2011; Naylor et al., 2011). Some heterogeneities are already present at coastal defences due to the construction procedure, such as holes/grooves in armour units, which retain water, or gaps between rocks or concrete blocks. Additional heterogeneities can also be incorporated in the design, for example, by adding tiles with different textures and microhabitats (Borsje et al., 2011; Coombes et al., 2015; Firth et al., 2016b). Drill-cored artificial rock pools can be an affordable, effective way to enhance the biodiversity of intertidal coastal structures (Firth et al., 2014; Evans et al., 2016).

Furthermore, Firth et al. (2013) suggest that rock structures should be constructed of both soft and hard rocks, since the weathering of carbonate rocks takes place faster than igneous rocks, thus creating additional surface roughness (Chapman and Underwood, 2011; Firth et al., 2012). Mixed rock sizes provide different habitats that can lead to greater species diversity and abundance (Wiecek, 2009). Porous coastal defences can form valuable habitats within their internal compartments, supporting greater species richness and diversity than external surfaces exposed to higher hydrodynamic forces (Sherrard et al., 2016). Artificial reef structures, for example Reef balls and WADs, have been built in many countries with this purpose. These elements are mound-shaped, concrete modules that imitate natural coral heads, providing a habitat for a variety of marine organisms (Barber, 1999). Precast habitat enhancement units, for example BIOBLOCKS (Firth et al., 2014), are another option to increase local biodiversity (Chapman and Underwood, 2011; Firth et al., 2013). BIOBLOCKS offer different, novel, micro-habitat types. To maximise species diversity, Firth et al. (2014) recommend that habitat enhancement units should include numerous novel habitat types, such as pits and pools of different depths and sizes, as well as ledges and overhangs.

Different types of artificial structures show different ratios of recruitment of non-indigenous versus native species, depending on their position and material (Glasby et al., 2007). Non-indigenous species were observed to recruit well on concrete surfaces near the water surface, while native species showed preference for locations closer to the shore and the seabed, such as rocky reefs and seawalls (Glasby et al., 2007). These factors should be analysed and considered in the design, to minimise threats to local biodiversity.

Adaptation Options for Seawalls

To minimise the environmental effects, adaptations for seawalls situated on rocky shores should ensure that the natural habitats are mimicked as far possible. As for seawalls on sedimentary coasts, their effects due to habitat changes should be mitigated as far as possible (Firth et al., 2014), e.g. lowering certain stretches of the seawall (Section 2.5.3). This subsection focuses on adaptation options that mimic rocky habitat.

Seawalls can be built or altered to enhance habitat diversity and complexity, without affecting the coastal safety offered, by maximising surface roughness and introducing micro-habitats (Wiecek, 2009; Borsje et al., 2011; Firth et al., 2014, 2016b; Hall et al., 2018). As with foreshore structures, smooth surfaces, such as concrete, should be avoided as far as possible (Wiecek, 2009; Firth et al., 2014; Coombes et al., 2015) or can be made rougher by casting irregular finishes (Wiecek, 2009) or by chiselling grooves or drilling holes (Martins et al., 2010; Nordstrom, 2014; Hall et al., 2018). Naylor et al. (2017) showed how a rock revetment can be ecologically enhanced by informed selection and intentional positioning of armour rocks. Rock pools can be incorporated in seawalls to provide habitat for intertidal organisms by adding water-retaining features (Chapman and Blockley, 2009; Firth et al., 2016a; Chapman and Blockley, 2009; Firth et al., 2014). As for hard foreshore structures, precast habitat enhancement units can be included into seawall design to offer a range of novel habitat types (Firth et al., 2014; Chapman and Underwood, 2011).

Since vertical seawalls decrease the area of intertidal habitat, sloping seawalls could be beneficial. A sloping structure, however, has a larger footprint and as such cannot be recommended over a vertical structure for this reason only (Chapman and Underwood, 2011). Another way to increase intertidal habitat for seawalls is to construct a stepped seawall or alternatively cavities can be left between the seawall blocks or rocks (Wiecek, 2009).

Careful consideration has to be given to the fact that invasive species favour hard structures and can use them as stepping stones (Airoldi et al., 2015; Bishop et al., 2017). For coasts with rocky shores, the probability of recruitment of local native species on seawalls can be increased by incorporating features that mimic their natural habitat (Airoldi et al., 2015). Measures can be taken to limit the colonisation of invasive species, such as using coating or smoother materials that hinder the settlement of fouling organisms (Airoldi et al., 2015; Coombes et al., 2015) or by seeding the structures with native species (Firth et al., 2016b; Bishop et al., 2017). A higher diversity of native species may offer a resistance to settlement by invasive species (Firth et al., 2014).

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Strain et al. (2018) conducted a quantitative meta-analysis and qualitative review of 109 studies to investigate which common nature-based adaptations (e.g. adding texture, crevices, pits and water-retaining units) have the greatest potential in increasing the biodiversity of key functional groups of organisms. The study found that adaptations in the intertidal zone that offer shade and provide moisture (e.g. crevices, pits and water-retaining features) had the largest influence on the richness of mobile and sessile organisms. Species whose body size was closest to the dimensions of the adaptations showed the greatest positive effects. Furthermore, intertidal adaptations that retain water had the largest impact on the richness of fish. Subtidal zone adaptations that added small-scale cavities resulted in higher abundances of sessile organisms, whereas elevated structures in the subtidal zone lead to higher numbers and abundances of fish. The results of the study provide valuable advice on selecting the most suitable adaptation type according to the desired goals, e.g. increasing biodiversity or enhancing certain ecosystem services.

Fig. 2.6 illustrates examples of possible nature-based adaptations for seawalls. It is essential that adaptation options are positioned in the correct tidal zone, to ensure that they are submerged during high tide. Therefore, adaptations are often placed below mean high water spring tide (MHWS) (but see Firth et al. 2016a). As seawalls are often located on coasts exposed to high wave energy, adaptations should be adequately attached (Browne and Chapman, 2011; Coombes et al., 2015; Hall et al., 2018). Additionally, the influence of adaptations need to be considered in the structural design, monitoring and maintenance of seawalls.

Adaptation Options for Greening Dikes

To establish vegetated sea dike covers, standard seeding mixtures of different grasses and, if desired, a small percentage of herbs are commonly used (in Germany: *Lolium perenne, Poa pratensis, Festuca rubra ssp. trichophylla, Festuca rubra* ssp. *rubra and Achillea millefolium*). These mixtures are known to ensure the main functions of the grass cover: protection against mechanical forces, such as wave impact, and the influence of weather (EAK, 2002). Although contemporary vegetated clay layers represent a green surface with a near-natural look, the ecological value of the dike cover is not sufficiently considered in the design process as indicated by the low biodiversity of the standard seeding mixtures. Nonetheless, the vegetated dike cover has the potential to become an ecologically valuable dike component by adapting the seeding mixtures towards more ecologically valuable plant compositions with an increased number of species and a higher amount of herbs and legumes. In doing so, the eligibility of the potential target vegetation, especially with regard to the erosion resistance and vegetation development under coastal conditions, has to be ensured (Scheres and Schüttrumpf, 2017). Grass reinforcement methods, e.g. High Performance Turf Reinforcement Mats (Pan et al., 2015), can support the dike vegetation and increase the overall erosion resistance. Vegetated geocellular containment systems (Meyer and Emersleben, 2009) increase the soil stability and allow the installation of dike paths with minimal soil sealing.

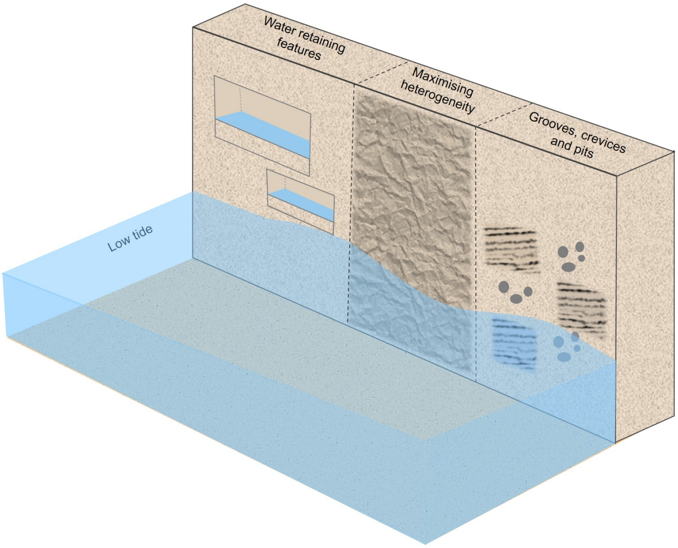

Figure 2.6: Schematization of possible nature-based adaptations for seawalls. Interventions should be positioned to ensure that they are submerged during high tide

Given past experiences, grey revetments are usually employed for dike safety, in case of high loads that exceed the resistance of a grass cover. Common materials are rip-rap, asphalt and concrete (CIRIA, 2013). Adaptations of the material of grey revetments, especially of the structure and texture, towards more natural, rougher and diverse-shaped surfaces allow for improved settlement and habitat conditions (Borsje et al., 2011). Alternatively, vegetated grey revetments, such as (partially) grouted rip-rap (Trentmann, 2011), cellular revetment blocks (Mohamed et al., 2006) or geosynthetic concrete mattresses (Wilke et al., 2012), can increase the ecological value of the system due to cavities or grooves within the revetments or reduced surface sealing that allows for vegetation.

Fig. 2.4b illustrates selected possible nature-based adaptations for sea dikes. When physically modifying the dike system, adaptations of the monitoring and maintenance strategies become necessary. In general, nature-based adaptations must not undermine the dike stability or its ability to offer coastal protection. Hence, a concept for ecologically valuable dikes should take into account conflicts between the ecological value and dike safety (Scheres and Schüttrumpf, 2017).

2.6 Discussion

Nature-based solutions for coastal protection should generally be considered before hard solutions in order to minimise environmental impacts (see Tab. 2.6). The two main considerations to determine the applicability of nature-based solutions are (1) the suitability in relation to hydrodynamic forcing and (2) the availability for the spatial requirements necessary to obtain the desired level of coastal safety. Other

2

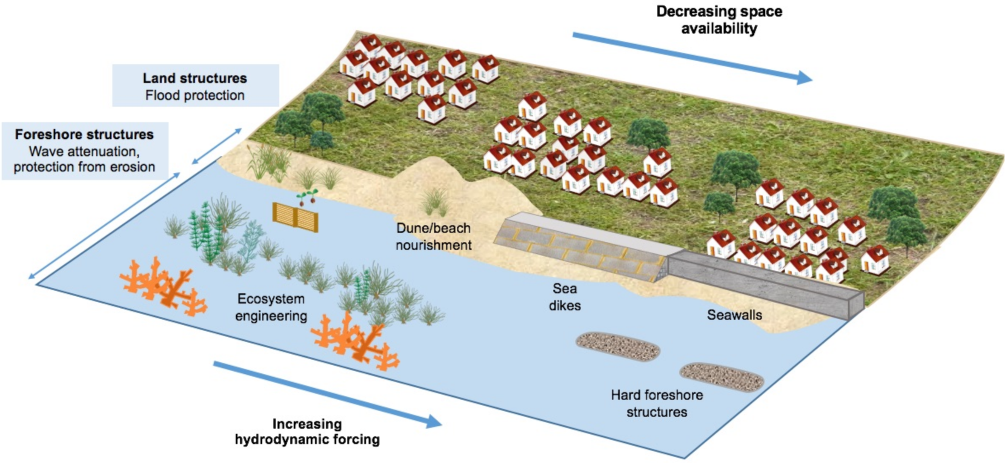

Figure 2.7: Implementation of hard solutions related to land use and hydrodynamic forcing

considerations include ecological impacts, costs, construction methods, maintenance required, stakeholder support, visual impacts and climate change consequences.

Fig. 2.7 illustrates in which conditions hard and soft solutions could be implemented. Ecosystem engineering may be suitable to provide coastal safety in areas with low to medium hydrodynamic forcing and elevated hinterlands (sufficiently high with respect to extreme water levels), given that there is enough space available. Wherever the wave energy is not sufficiently dissipated by coastal ecosystems, nourishments can increase the width of a beach and the amount of wave attenuation. In areas with structural erosion, periodic addition of sediment will be necessary to avoid net coastline retreat. Dunes, both natural and artificial, can provide additional protection against storms and serve as a physical barrier against flooding in lower lying areas.

Hard solutions should be considered a last resort and are suitable on coasts exposed to high wave energy and with reduced space available for changes in coastline position (e.g. urban areas). Beach nourishments can also be carried out in front of hard structures, in order to reduce the hydrodynamic loads acting on them. Hard foreshore structures offer wave attenuation and protection against erosion. For low-lying hinterlands, sea dikes/seawalls are generally the most appropriate solution, as these structures form a fixed barrier against inundation and provide a high level of safety. Besides building a hard barrier between the sea and hinterland, alternative coastal zone management approaches, such as retreat or elevated infrastructure, can be followed and may have ecological benefits, but is not discussed further in the present study.

When hard solutions are deemed necessary, it is advocated that they are ecologically enriched (Section 2.5.3) to either restore, mitigate or conserve ecosystem services. Although these adaptations may minimise environmental impacts, some effects cannot be mitigated completely. Also, artificial habitats have shown different community structures and functioning compared to natural rock habitats (Firth et al., 2016b). It is recommended that adaptations are considered early on in the design as possible and are designed in close collaboration with ecologists (Firth et al., 2016a; Naylor et al., 2017). Knowledge gaps (Section 2.7) in the technical design of adaptations, in terms of dimensioning related to boundary conditions, persist and hence complicate application.

Engineers and ecologists are not only faced with a challenge regarding the functional design of nature-based solutions; societal issues are also relevant for the implementation of coastal structure projects (Naylor et al., 2012). Great advances in the field of nature-based coastal infrastructure have been made, with literature mainly originating from Australia, the USA, and Europe (Strain et al., 2018; Morris et al., 2018; Salgado and Martinez, 2017), showing a bias to developed countries. Strain et al. (2018) suggest that, as is the case in terrestrial environments, socioeconomic status is a key indicator for the implementation of green infrastructure. The socioeconomic status depends in turn on factors such as level of education, willingness to invest in nature-based adaptations and available resources. This could explain the bias in literature. In the UK, Evans et al. (2017) studied stakeholder attitudes towards multi-functional coastal developments. The findings indicated that stakeholders favoured ecological benefits over social, economic or technical benefits. Perceptions of coastal authorities and societies towards nature-based coastal protection schemes for areas with different socioeconomic statuses are however still largely unknown.

Site-specific knowledge (e.g. ecologic analysis, assessment of the local resources and construction materials) is essential for the success of nature-based solutions (Salgado and Martinez, 2017). Considerable knowledge on green infrastructure was gained from pilot studies, monitoring and learning by doing principles. Reduced budgets and limited support may pose challenges for developing countries to gain site-specific knowledge through similar learning-by-doing approaches. Additionally, limited maintenance, monitoring (Silva et al., 2017) and social aspects (Salgado and Martinez, 2017) could hinder the application of ecological enhancement of hard coastal structures and the conservation of existing foreshore ecosystems in emerging countries. Progress in overcoming these challenges must be made, before nature-based solutions can be readily implemented in developing countries.

2.7 Conclusions and Future Avenues for Research

There is significant evidence of the effectiveness of nature-based solutions (Narayan et al., 2016; Hall et al., 2018; Strain et al., 2018), but knowledge gaps and persistent uncertainties pose challenges. Current guidelines and policies (Table 2.2) provide general recommendations, while technical design guidelines with respect to the performance of nature-based solutions under different boundary conditions (abiotic and biotic) are still lacking. Overall, a comprehensive design basis to assess the lifetime, applicability and maintenance needs of nature-based solutions is required to ensure that adaptations are readily included in designs in the future.

The uncertainties in the prediction of the wave height reduction of nature-based solutions are, firstly, due to the natural spatial variability in factors such as the plant height and diameter for different species. Secondly, current approaches to determine wave attenuation of plants are often based on a number of assumptions and simplifications and on site-specific calibration parameters. For example, the

Table 2.3: Summary of nature-based design considerations

	Soft solutions	Hard solutions		
Structure type	Ecosystem engineering	Foreshore structures	Seawalls	Sea dikes
Purpose	Protection against flooding and shoreline stabilization	Protection against flooding and shoreline stabilization	Protection against overtopping and flooding and shoreline stabilization	Protection against overtopping and flooding
Spatial requirements	High	Low	Low	Low - Medium
Hydrodynamic conditions	Sheltered to exposed coast; mild to high wave conditions (depending on the coastal ecosystem type)	Exposed coast; large storm surges and waves	Exposed coast; large storm surges and waves	Exposed coast; large storm surges and waves
Initial investment costs	Low- Medium	High	Medium - High	Medium - High
Maintenance costs	Low	Medium	Low	Medium
Design life	Short – Long (higher uncertainty due to limited research)	Long	Short - Long	Long
Environmental impacts and concerns	Possible negative effects if there is a replacement of a productive ecosystem by other (for instance when planting species of a different ecosystem); Positive impacts due to higher biodiversity, restoration of ecologic functions, ecosystem services.	Changes to coastal processes, erosion; Habitat loss; Impacts due to introducing hard substrata; Hazard for recreation; Increase of turbidity (if complemented by regular nourishments).	Changes to coastal processes, thus active and passes erosion; Habitat loss; Impacts due to introducing hard substrata; Interruption of ecological connectivity	Changes to coastal processes, thus active and passive erosion; Habitat loss; Changes in present plant communities; [Impacts due to introducing hard substrata]; Interruption of ecological connectivity
Nature-based adaptations	Building brushwood fences to emulate the effect of natural vegetation and restore ecosystems; Hydrologic restoration through excavation and/or filling; Sediment nourishments to increase wave damping in front of ecosystem; Planting seedlings/transplanting	Maximization of roughness and surface complexity; Incorporation of microhabitats; Combination with ecosystem restoration (coral reefs, sea grasses, salt marshes, mangroves); If complemented with nourishments, reduce their frequency in time	Maximization of roughness and surface complexity; Incorporation of microhabitats (e.g. rock pools; habitat enhancement units); Removal or alteration of seawall to restore sand movement; Implementation in combination with other solutions (ecosystem restoration, shore nourishments)	Ecosystems and/or nature-based structures in the foreshore; Adaptation of seeding mixtures towards more ecologically valuable vegetation; Vegetated revetments; Vegetated fortified dike paths; Dike geometry adaptations

schematization of coastal vegetation as a number of vertical layers consisting of rigid cylinders with different densities and sizes (Suzuki et al., 2012). Additionally, the temporal availability of ecosystems makes it hard to predict their effectiveness for long time scales, thus impeding their use in coastal protection schemes (Bouma et al., 2014; Morris et al., 2018). There is also additional uncertainty on how aquatic ecosystems will adapt to climate change and sea level rise. For hybrid solutions, this means that the boundary conditions for structures sheltered by or placed behind coastal ecosystems are hard to determine and it is therefore not always possible to optimise their design by including the protection provided by ecosystems.

Considerable laboratory and field work is necessary to formulate comprehensive guidelines which ecologically enrich hard coastal structures for particular objectives in specific locations. For instance, as pointed out by Strain et al. (2018), research has mainly focused on determining the success of only one type of adaptation and is usually not applied across the whole structure. As such, it is unclear which intervention or combination of interventions for seawalls or foreshore structures would deliver optimal ecological benefits and what the benefits in relation to the adaptation area are (Strain et al., 2018). As for vegetated dikes, the design of optimal seeding mixtures to support higher biodiversity needs further research. Another aspect that requires attention, is the analysis of the proportion of native, non-native and cryptogenic species when evaluating micro-habitat adaptations (Strain et al., 2018).

Systematic interdisciplinary research is required to holistically quantify the environmental impacts of hard structures and understand the underlying processes on different temporal and spatial scales (Bishop et al., 2017). For example, understanding the migration of species along networks of structures could lead to better planning of dimensioning and spacing of hard structures (Firth et al., 2016b). Economic and social aspects involved in implementing nature-based solutions present further challenges.

To ensure that green infrastructure is applied more widely, including countries with limited budgets, future research should include economic optimization of nature-based coastal protection methods. Further studies are also required on the cost-effectiveness of green infrastructure compared to grey infrastructure under the same environmental conditions(Narayan et al., 2016; Morris et al., 2018). Additionally, cost-benefit analyses should include ecosystem services provided by grey and green infrastructure (Morris et al., 2018).

This review provides descriptions on how hard solutions function, where, when and why their use is required, thereby highlighting requirements related to coastal safety. The environmental impacts of hard solutions and adaptation options to reduce them are discussed, thus advocating the importance of environmentally sensitive engineering designs. Overall, recent studies have made large progress in understanding the coastal protection benefits of ecosystems (Bouma et al., 2014; Möller et al., 2014; Narayan et al., 2016) and in researching adaptation techniques to enhance biodiversity of coastal structures (Chapman and Underwood, 2011; Firth et al., 2014; Strain et al., 2018). A greater interdisciplinary understanding is still required to move towards the inclusion of nature-based considerations in the standard practices and guidelines of coastal engineering, thus enabling a transition towards sustainable and environmentally friendly coastal solutions.

2

Chapter highlights

- Hard coastal structures form a barrier against wave action and stabilise shorelines, thereby altering hydrodynamic and morphological processes and introducing artificial hard habitats in coastal ecosystems. These changes to coastal processes and habitats bring negative environmental impacts.
- To reduce or mitigate these negative environmental impacts, or to create new ecosystem services, the following nature-based adaptations to hard coastal structures are proposed and discussed: (1) applying soft solutions complementary to hard solutions, (2) mitigating morphological and hydrodynamic changes and (3) ecologically enhancing hard coastal structures.
- Guidance is provided at the interdisciplinary design stage of nature-based coastal defence structures.

Acknowledgement

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Chapter 3

Environmental Design Aspects of Stepped Revetments

This chapter describes environmental aspects for the design of sustainable stepped revetments (SR). While Chapter 2 discusses environmental impacts and possible nature-based adaptations of hard coastal structures in general, this chapter summarises the findings specifically related to SR.

3.1 Environmental impacts and concerns of stepped revetments

SR are shore-parallel stepped slopes, designed to protect the coastal hinterland from flooding and erosion. The revetment stairs dissipate and reflect incident wave energy, thereby reducing wave overtopping and protecting the inland from flooding. Since SR are rigid structures that fix the position of the shoreline, they prevent the shoreline from retreating further inland. The intended effect of SR is to provide coastal safety by forming a barrier against wave action. However, SR also bring unintended negative environmental impacts as they affect hydrodynamic, morphological and ecologic conditions. These environmental impacts are site-specific and depend highly on the local environmental conditions. Moreover, the geometry of SR, their position on the beach profile and their building materials all affect how SR impact the environment. Like seawalls and other revetment types (Chapter 2), the environmental impacts of SR mainly stem from changes to coastal processes and habitats (Nordstrom, 2014). Fig. 3.1 illustrates the intended and unintended impacts of SR.

The implementation of SR influences sediment dynamics by means of (1) placement loss, (2) passive erosion and (3) active erosion (Griggs, 2010). The position and footprint of a SR influences its resulting placement loss, i.e. the portion of beach lost due to construction of the SR. When SR are built along coasts subjected to long-term net erosion, beach erosion will continue in front of the structure, causing the shoreline to retreat towards the structure (or even beyond). This gradual beach loss is referred to as passive erosion and could result in structure failure. Passive erosion is not caused by the structure itself, but due to the continued net erosion, experienced even before the structure was built. Without a SR or similar structure, the beach width would remain mostly constant, but the beach's position would move landward (Dean, 1987), provided that there is sufficient space without infrastructure.

SR are responsible for changing coastal processes, which in turn contribute to erosion. This type of erosion, known as active erosion, is caused by the structure itself (Kraus and McDougal, 1996). SR reflect a portion of incident wave energy, which potentially lead to additional transport of sediment seawards (USACE, 2002). This may in turn lead to scour at the structure toe, or result in steeper seabed profiles in front of the SR. Around the alongshore ends of the SR, erosion may also increase locally, leading to flanking. Furthermore, as SR form a physical barrier, they interrupt sediment movement in the cross-shore and longshore directions. Due to the retention of sediment, SR constructed on sandy coasts prevent sediment movement between the beach and the backshore, thereby affecting aeolian transport and the natural dune system (Jackson and Nordstrom, 2011; Nordstrom, 2014). This retention of sediment also affects the movement of sediment alongshore, potentially leading to erosion down-drift of SR.

The impacts of SR on coastal processes in turn result in loss of habitats and related

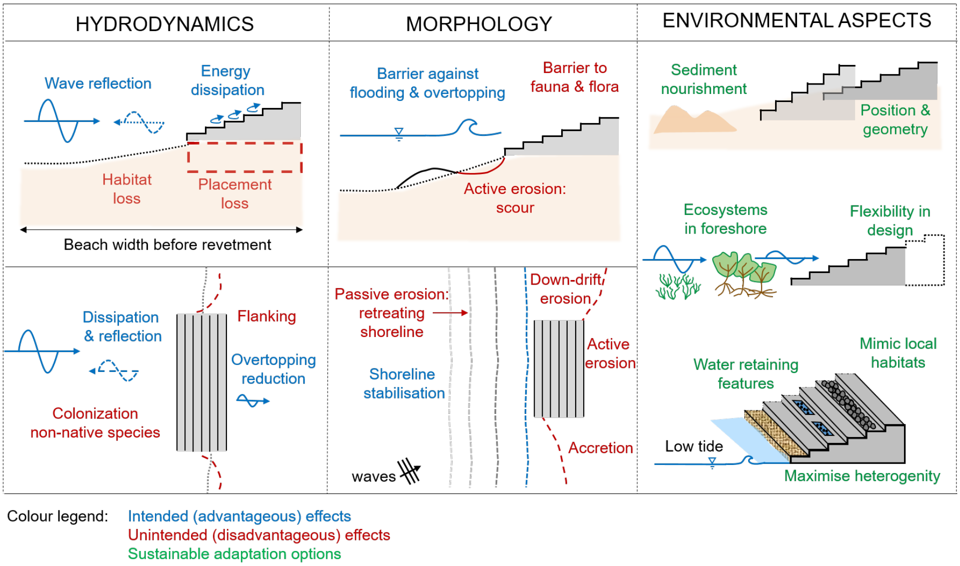

Figure 3.1: Stepped revetments: functions, environmental impacts and adaptations for sustainability

biota (Dugan et al., 2008; Nordstrom, 2014). This loss of habitat is initially due to placement loss and increases over time due to passive and active erosion. SR are not only barriers to sediment movement, but also interrupt ecological connectivity. Movement between the backshore and beach are essential to species seeking refuge or for nesting (Lucrezi et al., 2010; Nordstrom, 2014). According to Dugan et al. (2008), the implementation of seawalls have the largest impacts on the dry upper beach, as they result in a significant decline in the abundance, biomass and size of macroinvertebrates in the upper intertidal zone as well as reduced species richness and abundance of foraging shorebirds. Additionally SR may isolate inland habitats and affect neighbouring habitats (Chapman and Underwood, 2011). Further habitat loss is expected due to SR typically being steeper than the natural shore. Habitat loss leads to local losses in biodiversity and regionally affects population sizes (Chapman and Underwood, 2011). The steeper slope of SR also affects species-area relations. In intertidal areas, a steeper slope has the result that species that used to live in different vertical zones are now located much closer (Nordstrom, 2014; Bulleri and Chapman, 2010). In turn this affects ecological interactions since species that would usually not interact, will now affect each other. Ecological interactions can also be affected by species that easily colonise artificial structures (Chapman and Underwood, 2011).

The building material of SR brings changes to habitats as it introduces new rigid hard substrata with distinct differences to natural muddy, rocky or sandy habitats (Nordstrom, 2014). Artificial habitats affect mixes of species, abundance, size-structures of populations, reproductive outputs and competition or response to habitats (Chapman and Underwood, 2011). Moreover, when artificial habitats are introduced, native species may be found outside their natural habitat, while invasive species frequently colonise intertidal artificial structures (Chapman and Underwood, 2011). Both native and invasive species may use artificial structures to move along the coast (Firth et al., 2013; Chapman and Underwood, 2011). Environmental impacts of SR stretch across larger spatial scales due to changes to ecological connectivity (Bishop et al., 2017).

3.2 Nature-based considerations and adaptations for stepped revetments

The successful incorporation of nature-based adaptations requires sound knowledge of local environmental conditions. As such, these adaptations need to be developed in consultation with ecologists and environmental practitioners. Ideally environmental design aspects should be considered early on in project planning to limit construction costs and allow implementation over larger spatial scales (Firth et al., 2013). Fortunately some nature-based adaptations can also be implemented after construction, or incrementally as changes in design loads develop or other previously uncertain boundary conditions become clear (for instance due effects of climate change or increases in wave height impacts as a result of local seabed changes). Like all coastal solutions, monitoring and maintenance of nature-based adaptations are required for ensuring positive environmental outcomes. This subsection describes environmental design aspects to coastal engineers with the aim of improving the ecological value of their SR designs by minimising environmental impacts and creating new ecosystem services. Fig. 3.1 illustrates sustainable adaptation options for SR.

1. Assessing the functionality of stepped revetments to achieve project goals

As for all coastal engineering solutions, a first step for designing sustainable SR is to critically assess if the selected solution will achieve the desired project goals and outcomes. Prior to this step, other general design steps should be followed for implementing nature-based solutions, see World Bank (2017). This feasibility assessment relates to the SR's primary function of providing coastal safety as well as secondary benefits. Chapters 4 and 5 quantify the hydraulic responses of SR and review under which boundary conditions there is evidence of the effectiveness of SR to protect against wave overtopping and flooding. Although SR fix the position of the shoreline, they do not offer protection against passive erosion of the beach fronting them (see Section 3.1). The mechanisms for underlying erosion problems should therefore be identified.

2. Optimise the design to minimise unintended impacts on coastal processes

This step is typically included in coastal engineering practice, as physical impacts such as erosion and reflection also have consequences for the functionality of SR. To limit placement loss, a steeper SR slope can be selected for a smaller structure footprint. However, this must be in balance with the effect that steeper slopes have on habitats (see Section 3.1). Scour protection could compensate the undermining of the SR toe caused by local erosion induced by wave run-down and reflection. As scour protection is typically from hard material, it would in turn bring changes to habitats. In Chapter 5 it is shown that the revetment's step height in relation to incident wave conditions affect wave energy dissipation and therefore also wave reflection (Schoonees et al., 2022). Thus optimising SR geometry could also minimise unintended impacts on coastal processes and limit scour. Alternatively, adding porous sections in the revetment stairs could reduce wave overtopping, reflection and scour. The shoreline response to the implementation of SR should be assessed by numerical models. In this way, SR designs can be optimised for minimising impacts on adjacent coasts. Sediment nourishments in combination with SR compensates erosion and its consequences on the structure's functionality and habitat loss. Although it should be noted that sediment nourishments introduce other environmental impacts (Section 2.4.1)

3. Review the effect of the position and geometry on environmental impacts
The position of SR on the beach profile affects their exposure to waves and tides (Dugan et al., 2008). Lower on the beach profile, the exposure and therefore also the effects on coastal processes and consequent environmental impacts increase. When the goal is to minimise the environmental impacts on a beach, SR should be placed as high as possible on the beach profile. In contrast, when a SR is designed with the secondary benefit of habitat creation, a structure lower down in the intertidal zone has greater success in promoting habitat variety and promoting diversity (Firth et al., 2014). The position of the SR on coastal safety and erosion also need to be considered.

Another design parameter affecting habitat loss is the revetment slope. A gentler slope reduces loss of intertidal habitats, but at the same time results in increased placement and related habitat loss due to a larger structure footprint. Apart from coastal safety, the consideration of structure slope depends on the SR secondary benefits. Along a rocky coast a gentler sloped SR creates more surface area for incorporating and mimicking natural habitats. Along sandy beaches a steeper revetment slope may be more beneficial since it leads to a smaller footprint and introduces less hard substrata. However, the impact on movement of local species seeking refuge or nesting opportunities needs to be considered.

4. Consider the influence of building material on habitat changes
The habitat created by traditional concrete SR are poor surrogates for intertidal habitats due to their homogenous smooth surfaces (Wiecek, 2009). By maximising heterogeneity of the building material creates a range of habitats and promotes higher biodiversity compared to smooth surfaces (Firth et al., 2012; Martins et al., 2010; Hall et al., 2018; Naylor et al., 2011). Roughness and textures can be added to the revetment's step surfaces in a number of ways. Concrete can be cast with irregular finishes (Wiecek, 2009) or made rougher by drilling holes and chiseling grooves (Martins et al., 2010; Nordstrom, 2014; Hall et al., 2018). Alternatively tiles or slabs with a variety of textures could be attached to the structure (Borsje et al., 2011; Firth et al., 2014). Designs and construction need to ensure that tiles and slabs are sufficiently fixed, as these attachments are at risk to become detached due to hydraulic forces. Additionally, these adaptations need to consider the effect of wave interaction on the survival of species (Firth et al., 2014). In Chapters 4 and 5 physical model tests demonstrate that highly turbulent flows occur during wave run-up and run-down. To optimise the richness of mobile and sessile species in the intertidal zone, adaptations need to provide shade and retain moisture (Strain et al., 2018). Also the size of created grooves and pits influence the success of adaptations, showing greatest success to species with similar body size. Strain et al. (2018) provide valuable evidence on the most effective adaptations and their position in

the intertidal for particular project benefits, e.g. increasing biodiversity of target species or enhancing particular ecosystem services. Furthermore, concrete surfaces attract non-native species, especially near the water surface (Glasby et al., 2007). Instead of concrete, or in combination with concrete, other building materials can be considered, for instance incorporating rocks. Mixed rock sizes provide a variety in habitats for increasing species diversity and abundance (Wiecek, 2009), while including both hard and soft rocks create ranging surface roughness. The material type and the position of the structure need to considered to minimise impacts on local biodiversity.

5. Incorporate ecosystem and micro-habitats in stepped revetment design
Preserving or restoring ecosystems in the foreshore of SR retain or create ecosystem services. This includes wave attenuation and erosion control, leading to reduced wave energy reaching the structures. Combining ecosystems with hard coastal structures, called hybrid infrastructure (Sutton-Grier et al., 2015), could therefore reduce the design crest-level of SR. Ecosystems living higher in the intertidal, such salt marshes and mangroves, attenuate waves more effectively (Bouma et al., 2014). For ecosystems to survive, they need to be able to tolerate the hydrodynamic forcing. Further guidance on ecosystem types and their effectiveness in coastal protection has been described in Section 2.5.3.

Another way to create ecosystem services is to incorporate micro-habitats that mimic natural habitats in SR design. The stairs of SR can be adapted for this purpose. The vertical and horizontal surfaces of SR provide space for habitats to support a greater diversity of species (Wiecek, 2009). Along coasts where rock pools are common, adaptations which retain water can be applied to enhance the local biodiversity (Chapman and Underwood, 2011; Firth et al., 2013). Alterations to the revetment's stairs can be made to mimic rock pools, for instance to add porous sections containing rock. The inner compartments of the rock support greater species richness and diversity than external surfaces exposed to higher hydrodynamic forcing (Sherrard et al., 2016). Introducing porosity would also reduce wave overtopping, although it may bring other design challenges due to wave impacts. Another option is to repurpose the revetment steps, analogous to precast habitat enhancement units, for instance "BIOBLOCKS" (Firth et al., 2014). These units create a variety of novel micro-habitat types, such as pits and pools of varying depths and sizes as well as ledges and overhangs to retain water. This variety of habitats maximise species diversity (Firth et al., 2014). Adaptations need to be positioned to ensure they are submerged during high tide. Therefore adaptations are commonly placed below mean high water spring tide.

6. Allow flexibility in the design for future adaptation measures
Projections for climate change effects on sea-level rise and increased storminess contain large uncertainties. Designing SR based on worst case future scenarios will likely lead to overdimensioning and high costs (Van Gent, 2019). Overdimensioning leads to higher SR with wider footprints which, in turn, leads to increased environmental impacts. A solution for avoiding this is to include room for flexibility in designs (David, 2021), where designs can be reinforced incrementally as climate change effects can be estimated more accurately. This includes options to combine coastal solutions with SR or increase their crest-levels. This flexibility is not only for enhancing coastal safety, but could extend to the designs of nature-based adaptations.

Chapter highlights

- The intended effect of stepped revetments is to provide coastal safety by forming a barrier against wave action. However, stepped revetments also bring unintended negative environmental impacts as they affect hydrodynamic, morphological and ecologic conditions. These environmental impacts are site-specific and depend highly on the local environmental conditions.

- Adaptations to the designs of stepped revetments can be made to minimise environmental impacts and to restore or create ecosystem services. The ecological value of stepped revetments can be enhanced by mimicking natural habitats.

- The vertical and horizontal step surfaces of stepped revetments provide valuable areas where roughness and surface complexity can be maximised to mimic local habitats as closely as possible. Their steps could also be adapted to include micro-habitats, e.g. rock pools.

- Further considerations for adaptations include the position, geometry and building material of the stepped revetments. Additionally, designs need to be optimised to minimise erosion and associated habitat loss. By allowing flexibility in designs, overdimensioning for future worst-case scenarios can be avoided.

Chapter 4

Wave Overtopping at Stepped Revetments

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4.1 INTRODUCTION

Exposed to rising sea-levels and higher waves (IPCC, 2019; Young and Ribal, 2019; Taherkhani et al., 2020), flood defences need to be adapted, reinforced or newly built to protect coastal communities and infrastructure in the future (Vousdoukas et al., 2020). Especially in urban and touristic areas, increasing the elevation of existing structures is often not possible, nor desired, while space to develop green infrastructure is limited (Section 2.6 ; Schoonees et al. (2019)). The option to reduce wave overtopping by roughness elements on a slope is therefore highly relevant for the future management of coastal zones. One effective way to increase the slope roughness in this manner is by building stairs, i.e. stepped revetments (Van Steeg et al., 2018; Kerpen et al., 2019). In addition to supplying coastal safety, stepped revetments (SR) can be designed to be multi-functional, e.g. to provide seating to residents and tourists or offer safe access to water areas or beaches. Before SR can be applied in these settings, their coastal safety function must be understood and quantified for storm conditions: both in terms of average overtopping rates and individual overtopping events. Maximum overtopping volumes are key in determining tolerable overtopping limits, since they can be directly linked to structural instabilities, e.g. dike erosion or hazards to persons (Franco et al., 1994; Victor et al., 2012; Gallach-Sánchez, 2018).

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SR have been studied since the 1950s in more than 30 publications (reviewed in Kerpen and Schlurmann (2016); see also Section 4.2.3). For preliminary design purposes, Van Steeg et al. (2018) proposed a formula for predicting wave overtopping at SR based on small-scale physical model tests. A broad analysis by Kerpen et al. (2019) combined data sets from small-scale studies (Van Steeg et al., 2018; Kerpen, 2017; Goda and Kishira, 1976; Schoonees et al., 2018) to develop an empirical prediction for wave overtopping at SR. These predictions reveal the large potential of SR to reduce wave overtopping compared to a smooth slope and give first insights in the key processes that influence this reduction in overtopping. However, the wave overtopping reduction presents significant variability under similar test conditions. This raises the question if all key processes are included in the predictions. Overall the processes influencing the efficacy of SR to reduce wave overtopping are not yet fully understood and, as such, recommendations for detail design are lacking. Additionally, little is known about individual overtopping events at SR (Gallach-Sánchez, 2018).

Furthermore, it is anticipated that due to the slope roughness of SR, small-scale overtopping measurements are likely affected by scale effects, especially when overtopping rates are low (EurOtop, 2018). With full-scale flume tests, we seek to broaden our current understanding of the wave overtopping processes at SR and refine current empirical predictions. We investigate two SR cross-sections, both have a gentle slope ($\cot\alpha$=3), while each has a different step height (S_h=0.17; 0.50 m). The aims of our study are to (1) determine the average wave overtopping rate, (2) describe individual overtopping events, and (3) identify the processes that influence the reduction in wave overtopping when compared to a smooth slope.

Throughout the chapter, we refer to "overtopping reduction" to describe the reduction in wave overtopping rate between a SR and a smooth slope with the same boundary conditions.

4.2 Previous studies

4.2.1 Average wave overtopping rate

Guidance on wave overtopping is given in manuals from several European countries, e.g. Die Küste in Germany (EAK, 2002), the TAW report in the Netherlands (TAW, 2002) and the EA Overtopping Manual in the United Kingdom (Besley, 1999). National and European research was compiled in the EurOtop (EurOtop, 2007, 2018) to present the latest techniques and guidance on predicting wave overtopping at breakwaters, seawalls and other flood defences. Founded on model tests from various projects and institutions, the following empirical prediction was developed for the average wave overtopping rate q [m^3/s per m] at a slope (EurOtop, 2018; Van der Meer and Bruce, 2014):

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = \frac{0.023}{\sqrt{tan\alpha}} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot exp\left[-\left(2.7\frac{R_c}{\xi_{m-1,0} \cdot H_{m0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right)^{1.3}\right] \tag{4.1}$$

with a maximum of

$$\frac{q}{\sqrt{g \cdot H_{m0}^3}} = 0.09 \cdot exp\left[-\left(1.5\frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_*}\right)^{1.3}\right] \tag{4.2}$$

where g is the gravitational acceleration [m/s^2]; H_{m0} is the spectral significant wave height at the toe of the structure [m]; α is the structure's front face slope angle [°] and R_c is the crest freeboard of the structure [m]. The breaker parameter is defined as $\xi_{m-1,0}$=$tan\alpha/\sqrt{H_{m0}/L_{m-1,0}}$ and relates the structure slope to the wave steepness ($s_{m-1,0} = H_{m0}/L_{m-1,0}$). The dimensionless influence factors, ranging between 0 and 1 [-] are: γ_b the influence factor for a berm; γ_f the influence factor for roughness; γ_β the influence factor for oblique wave attack; γ_v the influence factor for a wall at the end of a slope and γ_* for a storm wall on a slope or promenade.

In this study only the influence factor for roughness γ_f, hereafter named roughness factor, is considered (hence γ_b=γ_β=γ_v=γ_*=1). The γ_f of a structure quantifies how effective wave overtopping rates are reduced by roughness, in comparison to a smooth sloped structure under the same boundary conditions. Dikes covered with grass (when H_{m0} > 0.75 m), asphalt or concrete are considered smooth slopes, such that γ_f = 1 (EurOtop, 2018). The steps of SR create slope roughness, thereby reducing wave overtopping (γ_f <1).

When γ_f of a structure is known, its wave overtopping discharge can be predicted by applying Eq. 4.1 or 4.2, which is valid for breaking ($\xi_{m-1,0}$ < 1.8) and non-breaking waves ($\xi_{m-1,0}$ > 1.8), respectively. Roughness factors listed in EurOtop (2018) are valid for breaking wave conditions. For non-breaking/surging wave conditions the roughness factor increases linearly with $\xi_{m-1,0}$, up to 1 when $\xi_{m-1,0}$ =10, such that:

$$\gamma_{f surging} = \gamma_f + \left(\xi_{m-1,0} - 1.8\right) \cdot \left(1 - \gamma_f\right)/8.2 \tag{4.3}$$

Apart from wave breaking, structural geometry also influences γ_f, as discussed in the next subsection.

4.2.2 Reduction of overtopping rates by slope roughness

A growing body of literature has examined roughness factors of slopes with various roughness types (e.g. Bruce et al., 2009; Capel, 2015; Chen et al., 2020). Earlier studies primarily determined roughness factors based on wave run-up experiments (TAW, 1972; Van der Meer and Janssen, 1994), while more recently roughness factors were determined by means of wave overtopping tests (Bruce et al., 2009; Chen et al., 2020). However, roughness factors for wave run-up and wave overtopping are similar (EurOtop, 2018) .

Generally, roughness factors for sloped structures are presented by a constant value, thus unaffected by wave conditions (e.g. EurOtop, 2018; Bruce et al., 2009). Yet, Eq. 4.3 is applied to correct γ_f for breaking waves. Since overtopping data scatters inherently, Bruce et al. (2009) consider constant γ_f values sufficient for design purposes when $2 \leq \xi_{m-1,0} \leq 4$. Their scale tests with rock slopes showed that γ_f varies with slope angle. For the flatter ($\cot\alpha = 2$) and steeper ($\cot\alpha = 1.5$) tested rock slopes, γ_f was 0.34 and 0.42, respectively. Accordingly, γ_f was mildly influenced by the type of wave breaking, consistent with Eq. 4.3, while γ_f increased with wave period.

In an experimental study by Capel (2015), and also more recently by Chen et al. (2020), it was established that γ_f for pattern block revetments is dependent on wave conditions and pattern block geometry, rather than being a constant value. Capel (2015) established that roughness elements become less effective (γ_f increases) when overtopping rates are large and waves are steep. Similarly, Chen et al. (2020) found that γ_f increases for lower freeboards and higher breaker parameters.

For steep slopes with ribs ($\cot\alpha$=0.58; 1) or blocks ($\cot\alpha$=1), Gallach-Sánchez (2018) observed that γ_f was unaffected by the breaker parameter. However, for the slopes with ribs, γ_f was lower for the milder slope ($\cot\alpha$=1), thus suggesting an influence of slope angle on γ_f. In contrast, for gentle slopes ($\cot\alpha$=3; 4) with ribs, Capel (2015) found a negligible effect of the slope angle on γ_f.

4.2.3 Slope roughness of stepped revetments

Recent reviews on wave overtopping of stepped revetments can be read in Kerpen and Schlurmann (2016) and Kerpen (2017), while background on previous studies are reported by Van Steeg et al. (2018), Kerpen et al. (2019) and Gallach-Sánchez (2018). Commonly SR are characterised by their step height (S_h) and slope ($\cot\alpha$) as defined in Fig. 4.1.

Physical model tests (scale of 1:10) were conducted to determine the wave overtopping at SR as part of a dike reinforcement project (Van Steeg et al., 2018; Van Steeg, 2012). Two SR slopes, 1:2 and 1:3 ($\cot\alpha$=2; 3), were investigated, each with larger (S_h=0.046 m (model values)) and smaller steps (S_h=0.023 m (model values)). Wave overtopping were measured at the SR and resulting roughness factors were determined with the TAW (2002) overtopping formulae (Eq. 4.18 and 4.19). Roughness factors between 0.37 and 0.54 were determined for the larger steps, while γ_f ranged between 0.43 and 0.65 for the smaller steps.

Van Steeg et al. (2018) were the first to highlight the importance of the characteristic step height (the step height perpendicular to the SR slope) relative to the wave height ($\cos\alpha \cdot S_h / H_{m0}$), hereafter called dimensionless step height, and developed the following

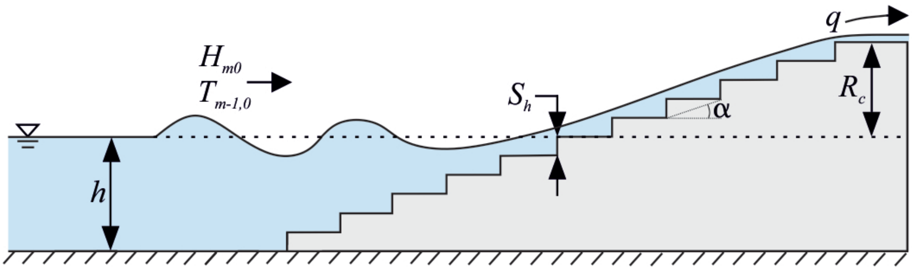

Figure 4.1: Definition sketch of Stepped Revetment parameters

prediction for γ_f:

$$\gamma_f = -0.190 \cdot ln\left(\frac{cos\alpha \cdot S_h}{H_{m0}}\right) + 0.257 \tag{4.4}$$

Eq. 4.4 is valid for the tested dimensionless step heights ($0.13 \leq \cos\alpha \cdot S_h/H_{m0} \leq 0.57$) and boundary conditions in Tab. 4.1. Further improvement to Eq. 4.4 was made by including the relative wave overtopping rate, obtaining Eq. 4.5, thereby improving the coefficient of determination r^2 from 0.77 to 0.81. With the improved prediction, γ_f has to be solved iteratively if the wave overtopping discharge is unknown:

$$\gamma_f = -0.17 \cdot ln\left[\frac{-cos\alpha \cdot S_h}{H_{m0}} \cdot ln\left(\frac{q}{\sqrt{g \cdot H_{m0}^3}}\right)\right] + 0.65 \tag{4.5}$$

Note that Equations 4.4 and 4.5 are valid in combination with the TAW (2002) overtopping formulae (Eq. 4.18 and 4.19).

Kerpen (2017) presents physical model tests with three SR slopes, 1:1, 1:2 and 1:3 (cotα=1; 2; 3), each with larger (S_h=0.30 m) and smaller steps (S_h=0.05 m). In a follow-up study, Kerpen et al. (2019) extended their data set by including results from Van Steeg et al. (2018), Goda and Kishira (1976) and Schoonees et al. (2018). The boundary conditions tested in Van Steeg et al. (2018) overlap partly with those of Kerpen (2017). Data from Goda and Kishira (1976) and Schoonees et al. (2018) extend the data set with very low step heights (S_h=0.006; 0.009 m) and a gentler slope (cotα= 6), respectively. An empirical formula for γ_f was developed based on the extended data set, revealing a maximum overtopping reduction when the dimensionless step height ($cos\alpha \cdot S_h/H_{m0}$) ranges between 0.5 and 2. Due to dissimilar test conditions (foreshore slopes and shallow water conditions), Goda and Kishira (1976) is not further discussed in the present study.

Gallach-Sánchez (2018) measured wave overtopping at two steep SR (cotα=0.58; 1), each with two low step heights (S_h=0.053; 0.106 m). For the steeper slope he found an average γ_f of 0.91, which was not influenced by the breaker parameter. For the flatter slope γ_f increased linearly with the breaker parameter ($0.57 \leq \gamma_f \leq 0.86$). In contrast to Kerpen et al. (2019), no influence of the dimensionless step height on γ_f was found.

Schoonees et al. (2018) reported on model tests conducted on a gentle slope SR (cotα=6) with a step height of 0.05 m. A reanalysis of the results (including the deep water wavelength instead of the local wavelength) yielded a best fit γ_f of 0.65 (r^2=0.72). The gentler-sloped SR is found more effective in reducing wave overtopping than previously reported.

The boundary conditions of previous SR studies are summarised in Tab. 4.1. All mentioned studies agree that SR offer substantial reduction in wave overtopping. This reduction, presented by γ_f, was found to be influenced by wave conditions (Van Steeg et al., 2018; Kerpen et al., 2019; Gallach-Sánchez, 2018). For gentler slopes ($\cot\alpha \geq 2$) the dimensionless step height substantially affects the slope roughness (Van Steeg et al., 2018; Kerpen et al., 2019), while for steeper slopes this effect was negligible (Gallach-Sánchez, 2018). Additionally, Van Steeg et al. (2018) highlighted the important influence of the wave overtopping rate on the overtopping reduction. In instances of high overtopping volumes, the steps become less effective as roughness elements.

4.2.4 Individual wave overtopping

Apart from the average overtopping rate, the volume of individual overtopping events is also an important design parameter. The same average overtopping rate could either consist of a smaller number of larger overtopping events, or a greater number of more similar overtopping volumes. Average overtopping rates have been studied extensively, while comparatively few studies investigated individual overtopping volumes. With only Gallach-Sánchez (2018) researching individual overtopping volumes at SR.

The probability distribution of individual overtopping volumes is well represented by a two-parameter Weibull distribution (Van der Meer and Janssen, 1994; Franco et al., 1994; Victor et al., 2012). It follows that the exceedance probability (P_v) of a specified individual overtopping volume (V_i) is:

$$P_v = P[V_i \geq V] = exp\left[-\left(\frac{V}{a}\right)^b\right] \tag{4.6}$$

where a is a dimensional scale factor $[m^3/m]$ that normalises the distribution and b is the dimensionless shape factor that defines the extreme tail of the distribution (EurOtop, 2018). A larger value of a corresponds to higher average overtopping rates and is defined as:

$$a = \left(\frac{1}{\Gamma(1+\frac{1}{b})}\right)\left(\frac{\Sigma V_i}{N_{ow}}\right) = \left(\frac{1}{\Gamma(1+\frac{1}{b})}\right)\left(\frac{q \cdot T_m}{P_{ow}}\right) \tag{4.7}$$

where T_m [s] is the average wave period, Γ is the mathematical gamma function and N_{ow} is the number of waves that result in overtopping. The probability of an overtopping

Table 4.1: Previous studies on wave overtopping at stepped revetments (S_h given in model values)

Reference	$\mathbf{S_h}$ **[m]**	**cot**α [-]	$\xi_{m-1,0}$ [-]	$S_h \cdot cos\alpha / H_{m0}$ [-]	γ_f [-]
Van Steeg et al. (2018)	0.023; 0.046	2; 3	1.7 - 3.7	0.13 - 0.57	0.37 - 0.65
Kerpen et al. (2019)	0.05; 0.30	1; 2; 3	1.5 - 9.4	0.40 - 4.82	0.35 - 0.85
Gallach-Sánchez (2018)	0.053; 0.106	0.58; 1	4.0 - 14.7	0.20 - 1.00	0.57 - 0.91
Schoonees et al. (2018)	0.05	6	0.9 - 1.5	0.21 - 0.32	0.54 - 0.73

wave P_{ow} is described by:

$$P_{ow} = \frac{N_{ow}}{N_w} \tag{4.8}$$

in which N_w is the number of incident waves.

Based on wave overtopping measurements at smooth mild slopes ($cot\alpha = 3;4$), Van der Meer and Janssen (1994) found a constant shape factor of b=0.75, which indicates that a small number of large overtopping events contribute to the average overtopping rate. In contrast, larger shape factors mean that the average overtopping rate is compiled by a larger number of more similar overtopping events. For emerged structures, the shape factor is smaller than 2 (EurOtop, 2018).

Franco et al. (1994) established that b=0.75 is also valid for vertical and composite breakwaters. Besley (1999) reported that b is influenced by the offshore wave steepness (s_{op}). For sloped structures, they determined b=0.76 for s_{op}=0.02 and b=0.92 for s_{op}=0.04. Furthermore, Bruce et al. (2009) investigated the overtopping performance of various armour units on rubble mound breakwaters and found on average b=0.74. They determined b by fitting all overtopping volumes in the distribution, rather than the higher overtopping volumes alone, thereby putting less emphasis on the largest events.

Victor et al. (2012) researched the characteristics of individual wave overtopping volumes at steep smooth slopes ($0.36 < cot\alpha < 2.75$). In contrast to Bruce et al. (2009), they fitted the shape factor only to the overtopping volumes that exceeded the average. The shape factor was found to increase with decreasing relative freeboard and slope angle. For large relative freeboard ($R_c/H_{m0} > 1.7$), they found a shape factor b=0.75, confirming the findings of Van der Meer and Janssen (1994).

Hughes et al. (2012) proposed a new prediction for b for smooth slopes, based on the data of Victor et al. (2012) (steep slopes with low relative freeboard), Van der Meer and Janssen (1994) (mild slopes) as well as Hughes and Nadal (2009) (mild slopes, $cot\alpha$=4.25 with negative freeboard). The shape factor was determined by fitting the largest 10 % overtopping volumes and developed a prediction as function of relative freeboard only. The authors note that for cases with low freeboard and steep slopes, the prediction of Victor et al. (2012) is preferred.

Prior studies focused primarily on smooth impermeable structures (with the exception of Bruce et al., 2009). Zanuttigh et al. (2013) were the first to undertake a joint analysis of smooth and rough permeable slopes. Rather than deriving a formula based on relative freeboard and slope angle as Victor et al. (2012), they opted for a prediction based on average overtopping rate, thereby implicitly including slope angle and wave steepness. For rough slopes the shape factors scatter more than for smooth slopes. They also found a small number of tests with low percentage of overtopping waves ($P_{ow} <$ 5%) and large b-values (b >1.4), which could not be explained. Their developed formula (also given in EurOtop, 2018) is:

$$b = 0.73 + 55\left(\frac{q}{g \cdot H_{m0} \cdot T_{m-1,0}}\right)^{0.8} \tag{4.9}$$

For relative discharges of $q/(g \cdot H_{m0} \cdot T_{m-1,0}) < 10^{-4}$ the average value of b is around 0.75, in agreement with previous studies.

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Building on the work of Victor et al. (2012), Gallach-Sánchez (2018) studied overtopping characteristics at steep low-crested slopes, both for smooth and rough impermeable slopes. He also fitted the shape parameter to the highest 10 % overtopping volumes (as Hughes et al., 2012) and proposes the following formula for the shape factor:

$$b = (0.59 + 0.23 \cdot cot\alpha)\, exp\left(-2.2\frac{R_c}{H_{m0}}\right) + 0.83 \tag{4.10}$$

For SR, Gallach-Sánchez (2018) reports a slightly reduced shape factor, meaning that the overtopping consists of larger events. However for blocks and ribs, no changes in shape factors are observed compared to the smooth slope.

When the scale and shape factors are known, the maximum overtopping volume (V_{max}) can be calculated as follows:

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$$V_{max} = a \cdot [ln(N_{ow})]^{\frac{1}{b}} \tag{4.11}$$

where N_{ow} is determined with Eq. 4.8 in which the probability of overtopping (P_{ow}) is in turn estimated by:

$$P_{ow} = exp\left[-\left(\frac{R_c/H_{m0}}{c}\right)^2\right] \tag{4.12}$$

where c is an empirical coefficient. Various studies have proposed values/predictions for c. For slopes, Van der Meer and Janssen (1994) suggest that c is related to the 2 % run-up distribution, while both Victor et al. (2012) and Gallach-Sánchez (2018) propose predictions for c based on the slope angle. For vertical breakwaters Franco et al. (1994) found c=0.91.

4.2.5 Model and scale effects related to wave overtopping

Model effects occur when prototype conditions, such as boundary conditions or experimental setup, are not realistically reproduced. One major model effect in wave overtopping models is the negligence of wind (Lykke Andersen et al., 2011; EurOtop, 2018). Furthermore, the position and width of the overtopping chute have been shown to cause variability in overtopping rates. Differences in wave overtopping rates also occur between flumes (EurOtop, 2018).

Most physical models that investigate wave processes on coastal structures are scaled according to Froude's law as gravitational and inertial forces are dominant (Hughes, 1993). As a result, viscosity, elasticity and surface tension are not correctly represented in small scale. These incorrect representations lead to inaccuracies, known as scale effects. The quantification of scale effects related to wave overtopping is not yet fully resolved, although some insight is given by the CLASH project, as described in EurOtop (2018).

Negligible scale effects are expected for smoother slopes ($\gamma_f \geq 0.9$), provided that critical limits are satisfied (Reynolds number > 10^3; Weber number > 10). Overtopping rates measured at rougher slopes in small scale are often underestimated, especially when overtopping rates are low (EurOtop, 2018). A low overtopping rate is defined by EurOtop (2018) as $q < 1\ l/s$ per m (in prototype), while Capel (2015) mentions that tests with $q < 0.5$ l/s per m are potentially affected by scale effects. Consequently, the majority of small-scale tests at SR are likely affected by scale effects.

For low overtopping rates, EurOtop (2018) presents adjustment factors for model and scale effects as a function of γ_f and $cot\alpha$. These adjustment factors are unrelated to the applied scale. The quantification of scale effects remains challenging, since it is hard to recreate the same conditions at different scales in order to isolate the scale effects (Lykke Andersen et al., 2011).

Besides model and scale effects, the variability in wave overtopping measurements in physical models should be noted. Variability partly stems from random starting phases resulting in different wave time series generated from the same wave energy spectrum. Consequently, the distribution of high waves varies which leads to variability in wave overtopping. This variability increases with relative freeboard (Williams et al., 2019), especially when the relative freeboard exceeds 1.4 (Romano et al., 2015; Daemrich et al., 2012). Lower overtopping rates and lower probability of overtopping similarly show greater variability in overtopping (Williams et al., 2019).

4.3 Physical model tests

4.3.1 Test facility

Full-scale physical model tests were conducted in the Large Wave Flume (GWK), situated in Hannover, Germany. The flume is 307 m long, 5 m wide and 7 m deep with a maximum water depth of 5 m (Führböter et al., 1989; Schulz, 1992). With its piston type wave maker, equipped with active wave absorption, JONSWAP spectra were generated to study wave overtopping.

4.3.2 Experimental setup

Stepped revetments are particularly suitable as coastal protection in urban areas, where the availability of space for constructions is often limited. Along the coast of Germany, seaward dike slopes typically range between 1:3 and 1:7 ($cot\alpha$=3-7) (Schüttrumpf, 2008). A slope of $cot\alpha$=3 was selected to allow for a smaller footprint and simultaneously be representative of a German dike cross-section. Examples of built SR with similar geometries are at HafenCity in Hamburg, Germany (Kerpen et al., 2014) and Den Oever, the Netherlands (Steendam et al., 2018). Moreover, the selected slope was previously tested in small-scale (Kerpen et al., 2019; Van Steeg et al., 2018).

Previous research (Van Steeg et al., 2018; Kerpen et al., 2019) has shown that the relation between the wave height (H_{m0}) and the step height (S_h), called the step ratio, is of key influence to the performance of SR. To consider a wider range of step ratios, two model configurations were tested with uniform step heights across the structure. The first configuration had a step height of 0.50 m, selected to add the recreational benefit of offering a place to sit, i.e. serve as a bench. The second configuration had a step height of 0.166 m (= 0.50/3), rounded to 0.17 m, that is considered to represent a typical height for walking up a staircase. Fig. 4.2 shows the model setup and instrumentation for both configurations.

The models consisted of a sand-filled core, an under layer of gravel and precast concrete elements which were stacked on one another to form the stepped geometry.

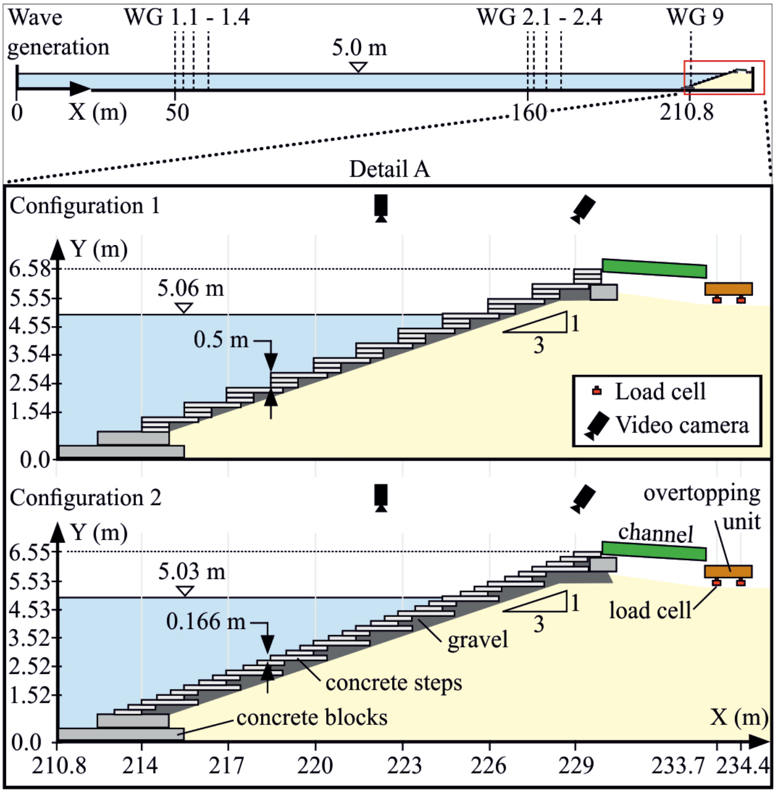

Figure 4.2: Side view of model configurations and instrumentation

Step elements were attached to one another by a rod and anchor system. The steps of the first configuration (S_h=0.50 m) comprised three stacked elements, while the second (S_h=0.17 m) had single stacked elements. Both models stretched across the flume width (5 m) and were placed at a distance of 210.8 m from the wave maker (X=210.8 m). The construction time for the first and second configuration was 4 and 3 weeks, respectively.

The surface elevation along the flume was measured by 9 wave gauges. They were placed in two arrays: the first (WG 1.1 to WG 1.4) between X=50 m and X=60 m, and the second (WG 2.1 to WG 2.4) between X=160 m and X=170 m. A reflection analysis on each array allowed for the determination of incident and reflected wave conditions. An additional wave gauge (WG 9) was placed in line with the toe of the structure, at X=210.8 m. Further instrumentation included a 2D laser scanner, pressure sensors, velocity propeller probes and a void fraction probe for measuring aeration properties. These are not shown in the schematic setup as their measurements are outside the scope of the present study.

A container (bin), consisting of an inner and outer shell, was installed as the overtopping unit (Fig. 4.2) behind the structure's crest to collect and measure the wave overtopping. The water entered the inner shell of the container through a 3.5 m long

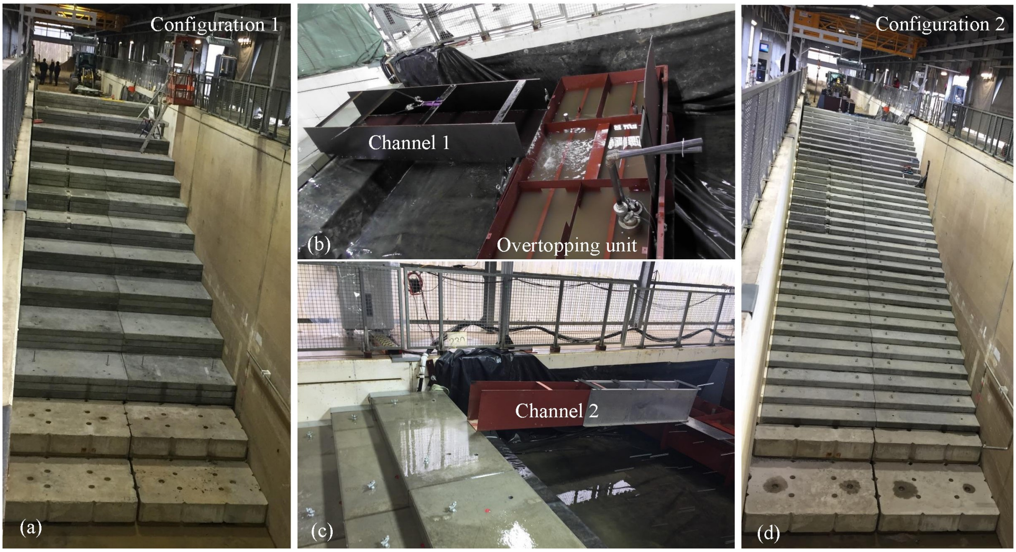

Figure 4.3: Stepped revetment model setup ($cot\alpha = 3$) with overtopping unit. **(a)** Configuration 1 with a step height of 0.5 m during construction, **(b)** corresponding overtopping unit and 1.0 m wide overtopping channel. **(c)** Reduced overtopping channel width of 0.34 m for **(d)** Configuration 2 with a step height of 0.17 m.

chute that stretched from the structure's crest. The weight of the water was measured by four load cells that were placed underneath the inner shell of the container. Two pumps were installed that were switched on in the event that the container reached its capacity. Initially a channel with an inner width of 1.0 m was installed (Channel 1 in Fig. 4.3). After 4 tests in Configuration 2, it was noted that the container filled too quickly and a smaller channel, with a width of 0.34 m was installed instead (Channel 2 in Fig. 4.3). The middle of both overtopping channels was placed at a distance of 1.25 m from the flume wall.

4.3.3 Test programme and procedure

The model tests were conducted over a period of 16 days. A total of 20 overtopping tests were performed with irregular waves (JONSWAP spectra; $\gamma = 3.3$). The duration of each test was based on 1000 times the peak wave period (T_p), resulting in test durations between 75 and 120 minutes. The first waves in a test are affected by the start up of the wave maker and the initial motionless water mass in the flume. To disregard these effects, the first minute of waves are repeated at the end of the test. In this way it was ensured that the first waves were disregarded, while the correct spectrum and time frame ($1000 \cdot T_p$) could be used for the analysis. WaveLab3 Software (Aalborg University, 2013) was employed to obtain the wave conditions from the surface elevation measurements. The incident wave parameters at the toe of the structure were determined by a reflection analysis at the second wave gauge array (WG 2.1 - 2.4) adhering to recommendations by Klopman and Van der Meer (1999) that waves should be measured at a distance greater than $0.4 \cdot L_p$. Where L_p is defined as the peak wavelength based on the local water depth.

The test programme was designed to include tests with a range of wave steepness and step ratios by systematically changing the wave heights and periods. Wave overtopping measurements were performed with mainly one water level (h=5 m) and all tests were classified as non-breaking ($\xi_{m-1,0} \geq 1.8$). The test conditions are given in Tab. 4.2.

4.4 Analysis and results

4.4.1 Average overtopping and roughness factors

The overtopping rates were determined from the weight measurements of the four load cells underneath the overtopping container. The time series of the four weights were added together and a median filter was applied to remove unrealistic spikes. A trigger signal was recorded to determine the time intervals for which the pumps were switched on. With these intervals and the pump discharge curve (determined from two calibration tests) the total volume of water pumped from the container was determined.

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The average overtopping rate q for each test was calculated by dividing the change in water volume in the overtopping container by the test duration and the width of the overtopping channel. Thereafter, γ_f was calculated with Eq. 4.2, by substituting the measured q, the freeboard R_c and the wave parameters obtained from the reflection analysis. Since the tests were preformed with surging wave conditions, the roughness factor was corrected by applying Eq. 4.3. Note that these corrected roughness factors are shown in all figures with γ_f on their axis.

The average overtopping results for the large (S_h = 0.50 m) and small steps (S_h = 0.17 m) are shown in Fig. 4.4(a). The results show a substantial reduction in overtopping compared to the EurOtop (2018) prediction for overtopping rates at smooth slopes (i.e. $\gamma_f = 1$). Resultant roughness factors range between 0.48 and 0.75.

The SR with large steps shows lower roughness factors and is thus more effective in reducing wave overtopping. At first sight, both SR series present a clear trend of decreasing relative wave overtopping for increasing relative freeboard. When Eq. 4.2 was fitted through each series, γ_f values of 0.55 and 0.67 were determined for the large and small steps respectively. Due to scatter, the coefficients of determination (r^2) were relatively low with 0.66 and 0.39 respectively.

The calculated roughness factors are compared to the prediction of Van Steeg et al.

Table 4.2: Stepped revetment geometry and range of test conditions (wave conditions given at the toe of the SR)

Parameter	**Symbol**	**Unit**	**$S_h = 0.50$ m**	**$S_h = 0.17$ m**
Wave height	H_{m0}	m	0.77 - 1.07	0.84 - 1.04
Wave period	$T_{m-1,0}$	s	4.58 - 6.52	4.14 - 5.57
Wave steepness	$s_{m-1,0}$	%	1.4 - 2.8	1.8 - 3.5
Breaker parameter	$\xi_{m-1,0}$	-	1.98 - 2.78	1.80 - 2.50
Dimensionless step height	$cos\alpha \cdot S_h / H_{m0}$	-	0.45 - 0.61	0.15 - 0.19
Freeboard	R_c	m	1.52 - 1.69	1.67
Relative freeboard	R_c / H_{m0}	-	1.42 - 1.96	1.46 - 1.80
Slope	$cot\alpha$	-	3	3
Water depth	h	m	4.89 - 5.06	5.03

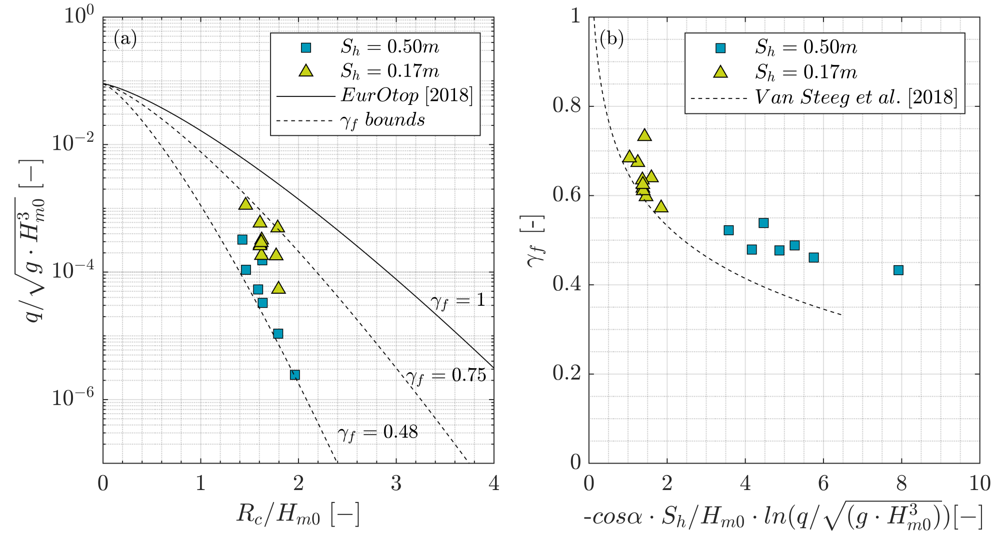

Figure 4.4: **(a)** Average overtopping rates and **(b)** roughness factors for the stepped revetments

(2018) (Eq. 4.4) in Fig. 4.4(b), who tested similar boundary conditions as in this study at a scale of 1:10 (Tab. 4.1 and 4.2). Although their prediction was derived based on the TAW (2002), the roughness factors calculated with the TAW (Eq. 4.19) and EurOtop (Eq. 4.2) approaches are very similar (Fig. 4.15). The shown γ_f prediction underestimates the roughness factors of the large steps (S_h = 0.50 m) by 15 to 31 % and of the small steps (S_h = 0.17 m) by 2 to 19 %. As a result, the full-scale results of this study show higher overtopping rates than predicted by Van Steeg et al. (2018). This observation highlights the need for a revised prediction to present the full-scale tests and to explore the results in greater detail.

4.4.2 Influences on roughness factors of stepped revetments

Influences of the wave conditions on the slope roughness (1-γ_f) are examined in Fig. 4.5. Slope roughness is shown to investigate which influences contribute to the overtopping reduction compared to a smooth slope. Since γ_f is calculated with Eq. 4.2, it is already a function of the measured wave overtopping, the freeboard and the wave height. Additionally, as γ_f is corrected by Eq. 4.3, it becomes a function of the breaker parameter.

For the small steps, a linear relation between the slope roughness and the wave period was found, as seen in Fig. 4.5(a). The slope roughness decreases with increasing wave period. In the case of the large steps, a similar linear reduction is found for $T_{m-1,0}$ > 5 s. For $T_{m-1,0} \approx 4.5$ s, the slope roughness is similar compared to $T_{m-1,0} \approx 5.5$ s. This may indicate that the large steps become less effective in reducing wave overtopping when wave periods exceed a certain threshold. However, it cannot be excluded that the

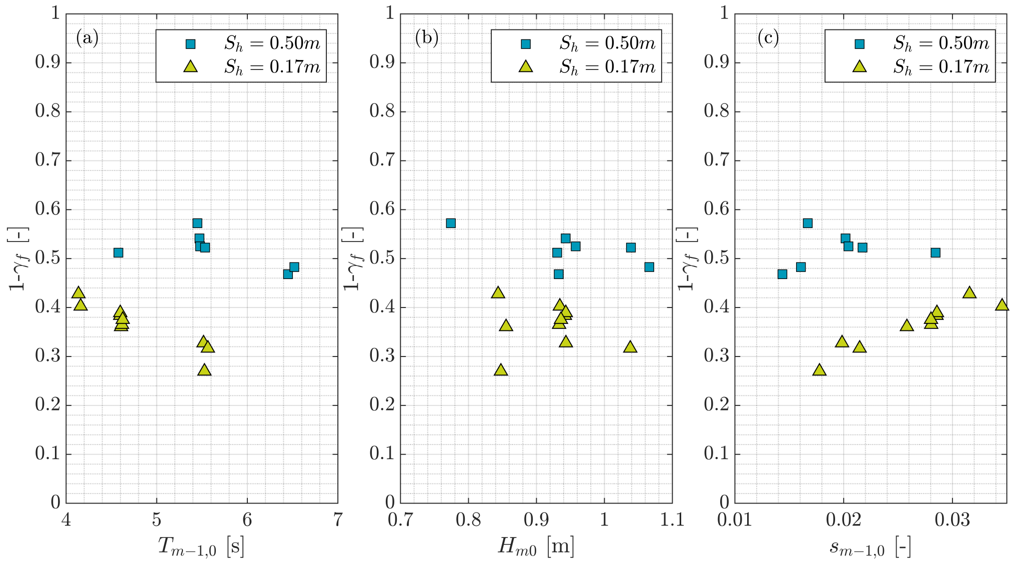

Figure 4.5: The influence of **(a)** wave period, **(b)** wave height and **(c)** wave steepness on the slope roughness

single data point at $T_{m-1,0} \approx 4.5$ s is an outlier. Nevertheless, the wave period notably influences the slope roughness.

More scatter in the slope roughness is present when plotted against the wave height, Fig. 4.5(b). A decrease in slope roughness may be recognized for increasing wave height H_{m0}, which indicates a slight reduction in the effectiveness of SR to reduce wave overtopping.

The combined influence of wave period and height is examined in Fig. 4.5(c) by means of the wave steepness ($s_{m-1,0}$). Overall, the slope roughness increases with wave steepness. This means that SR become more effective in reducing wave overtopping when waves are steeper. For the small steps, the trend is linear. Yet, for the large steps, the curve flattens with increasing wave steepness. It could indicate that the wave steepness is less influential for the slope roughness of the large steps.

Following Van Steeg et al. (2018), the influence of the dimensionless step height (Fig. 4.6(a)) and relative overtopping rate (Fig. 4.6(b)) are considered. Two data point clouds are observed in Fig. 4.6(a), indicating the order of magnitude of the slope roughness. As evident from Fig. 4.6(b), the relative overtopping rate influences the slope roughness of the SR. The larger the relative overtopping rate becomes, the less effective the SR is in reducing overtopping, Fig. 4.6(b). Overtopping events typically leave a residual water layer on the steps, making it easier for subsequent waves to run up on. Larger overtopping rates (i.e. more frequent overtopping events) thus make the structure less effective in reducing wave overtopping. For the small steps, the steeper trend shows that this influence is more prominent.

Additionally, the slope roughness as a function of characteristic step height to

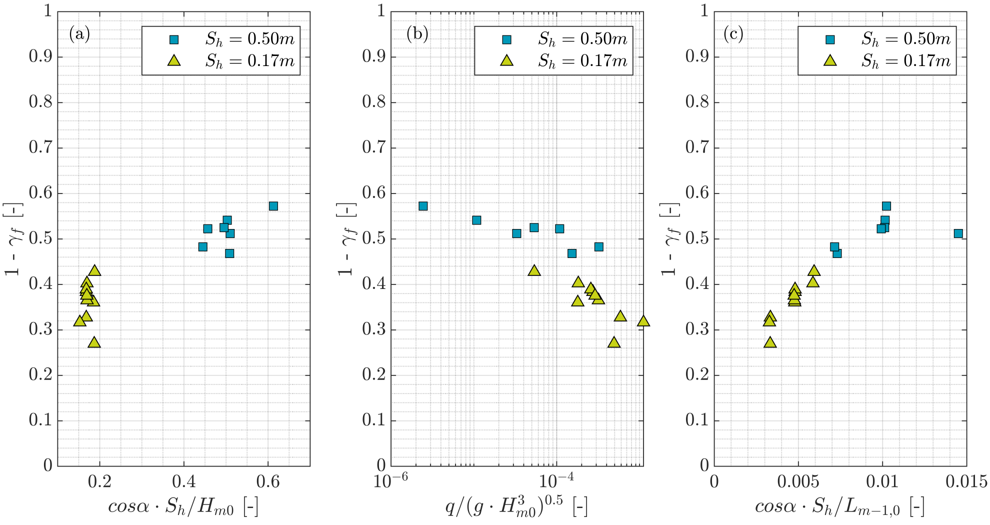

Figure 4.6: The influence of **(a)** dimensionless step height, **(b)** relative overtopping rate and **(c)** ratio of characteristic step height to wavelength on the slope roughness

wavelength is shown in Fig. 4.6(c). The slope roughness increases almost linearly with $cosa \cdot S_h/L_{m-1,0}$ and flattens off around $cosa \cdot S_h/L_{m-1,0} = 0.015$. Since this flattening of the curve is only based on one data point, additional tests would be required for verification.

4.4.3 Estimating roughness factors of stepped revetments

Based on the insights from Figures 4.5 and 4.6, a prediction for the roughness factor of SR was developed (Eq. 4.13 and Fig. 4.7). The first variable included in the prediction is the relative wave overtopping. However, to avoid iteration, the relative overtopping at a smooth slope ($q_{\gamma_f=1}/\sqrt{g \cdot H_{m0}^3}$) is incorporated instead of the relative overtopping at the SR. Hence, Eq. 4.2 is used to determine $q_{\gamma_f=1}$. This way the most important influencing parameters for wave overtopping (i.e the wave height and freeboard) are included in the prediction. Two additional variables, the characteristic step height ($S_h \cdot cosa$) and the wavelength ($L_{m-1,0}$), are included in Eq. 4.13. The slope roughness increases with characteristic step height, which enhances the structure's ability to reduce wave overtopping. The characteristic step height is made dimensionless by the wavelength, thereby including the observed influence of the wavelength in Fig. 4.6(b). The prediction yields a coefficient of determination of r^2=0.92 and a root mean square error (RMSE) of 0.03. The prediction is valid for the boundary conditions presented in Tab. 4.2.

$$\gamma_f = 1 - 0.55 \cdot tanh\left[-31.07 \cdot ln\left(q_{\gamma_f=1}/\sqrt{g \cdot H_{m0}^3}\right) \cdot cosa \cdot S_h/L_{m-1,0}\right] \tag{4.13}$$

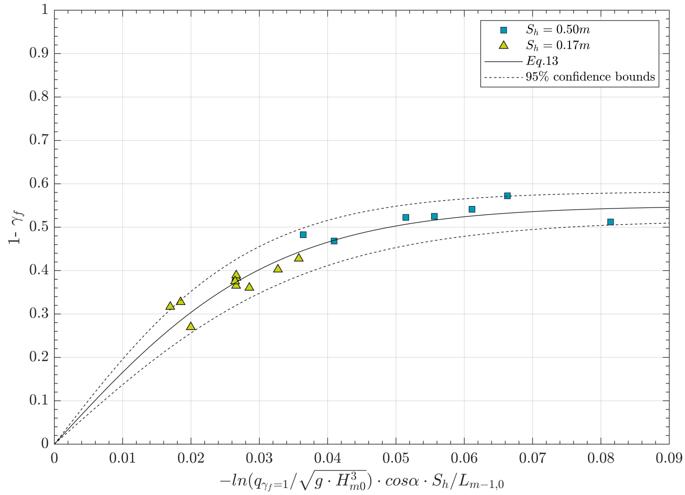

Figure 4.7: Empirical formula for estimating the roughness factor of stepped revetments

4.4.4 Individual wave overtopping volumes

The wave overtopping characteristics at SR were further investigated by examining the individual overtopping volumes. To determine the cumulative overtopping of each test, the volumes discharged by the pumps were added to the measurements of the load cells (see Section 4.4.1). An example of an obtained cumulative overtopping volume is shown in Fig. 4.8(a). From the cumulative overtopping volume, individual overtopping events were identified in three steps. Firstly, the start of an overtopping event was detected when the derivative of the signal exceeded 0.1 l/s. Secondly, unrealistically small ($< 5\ l$) or short events ($< 4\ s$) were discarded. Lastly, missed events were added based on visual inspection of each time series. Consecutive overtopping events could not always be distinguished from one another, since the water of two overtopping events could cause an uninterrupted discharge through the 3.5 m long overtopping channel.

The overtopping events were further analysed following Victor et al. (2012) and Gallach-Sánchez (2018). Based on Eq. 4.6, the following equation for the individual overtopping volume V is obtained:

$$V = a(-(lnP_v))^{\frac{1}{b}} \tag{4.14}$$

where P_v is the exceedance probability that can be approximated by ranking overtopping events in descending order (r):

$$P_v = \frac{r}{N_{ow}+1} \tag{4.15}$$

Applying the logarithm to both sides of Eq. 4.14, yields:

$$log(V) = log(a) + \frac{1}{b} log(-lnP_v) \tag{4.16}$$

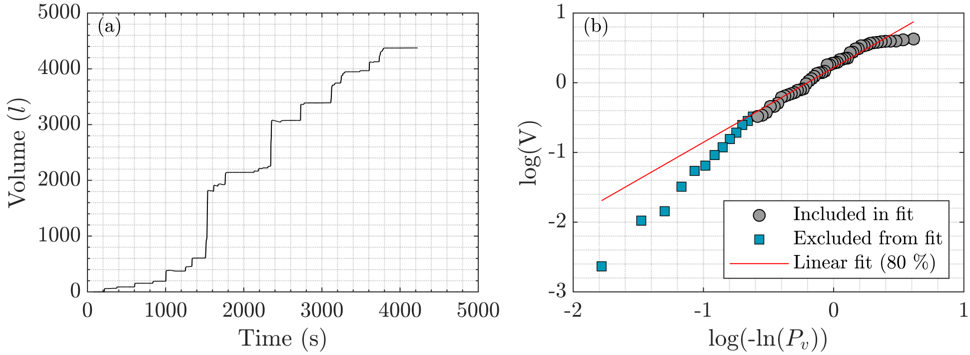

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Figure 4.8: Example of individual wave overtopping for test with H_{m0}=0.9 m T_p=6 s R_c=1.51 m. **(a)** Cumulative wave overtopping **(b)** Weibull plot of individual overtopping events

Table 4.3: Shape factors determined with linear fitting

	Shape factor (b)		
Points fitted [%]	**Median**	**Minimum**	**Maximum**
60	2.08	1.10	2.58
80	1.63	0.93	1.87
90	1.41	0.88	1.81
100	1.22	0.78	1.72

With Eq. 4.16 the Weibull distribution is presented as a line with a gradient of $\frac{1}{b}$ and a y-intercept of $log(a)$. An example is shown in Fig. 4.8(b), with $log(-ln(P_v))$ on the x-axis and $log(V)$ on the y-axis. A line was fitted through the data points of each test to determine the shape factor b and scale factor a. However, the determined factors are sensitive to the number of data points to which the fit is applied. This study applied the linear fitting to the 80 % largest volumes. The largest 15 % and the smallest 20 % of the volumes were generally lower than estimated by the Weibull distribution. Tab. 4.3 shows the calculated shape factor for a varying percentage of largest volumes used for the fit.

The scale factors a, determined through linear fitting, are compared to the theoretical scale factors a_t calculated with Eq. 4.7 in which b=1.63. A good agreement (r^2=0.99; $RMSE$=0.086) was found between these scale factors as shown in Fig. 4.9(a). This confirms that the overtopping volumes are well represented by a Weibull distribution.

Furthermore, the measured and calculated (Eq. 4.11) maximum overtopping volumes are compared in Fig. 4.9(b). Although the comparison shows a good agreement (r^2=0.94; $RMSE$=0.86), the theoretical maximum overtopping volumes exceed the measured volumes. This is in line with our finding that the highest overtopping values were overestimated by the Weibull distribution.

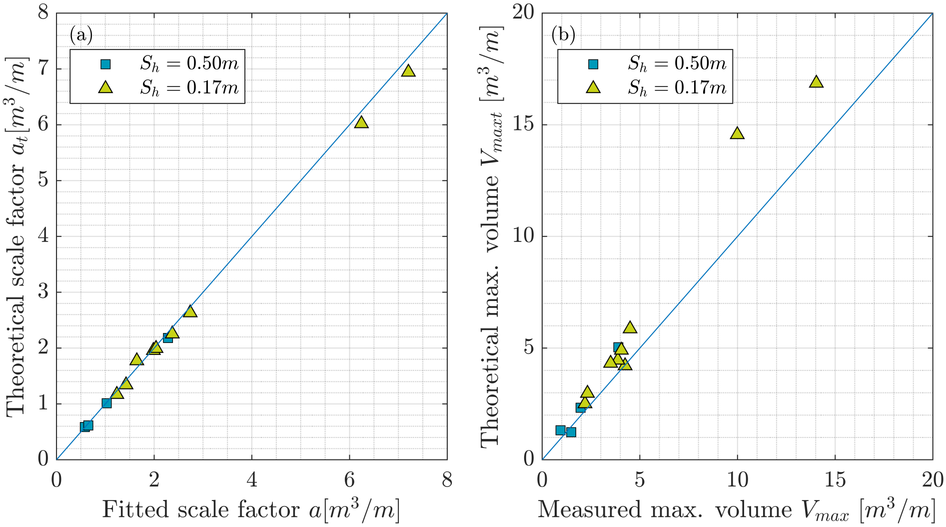

Figure 4.9: **(a)** Comparison of calculated and theoretical scale factors. **(b)** Comparison of measured and theoretical maximum overtopping volumes

Lastly, a linear relation was found between the maximum measured overtopping volume and the average overtopping rate of each test, Fig. 4.10. This linear relation is shown in Eq. 4.17 (r^2=0.91; $RMSE$=1) and can be applied to estimate the maximum overtopping volumes for known overtopping rates within the boundary conditions of Tab. 4.2.

$$V_{max} = 4111 \cdot q \qquad [m^3/m] \tag{4.17}$$

The linear relation in Eq. 4.17 is only applicable for the tested SR and conditions in Tab. 4.2 and cannot be applied for other structure geometries or wave conditions.

4.5 Discussion

As multi-functional coastal protections, SR are particularly suitable to be implemented in urban areas. Their coastal safety function to limit wave overtopping was proven by small-scale studies (Van Steeg et al., 2018; Kerpen et al., 2019; Gallach-Sánchez, 2018). These studies tested a wide range of boundary conditions that provide first insights in understanding the mechanisms causing overtopping reduction at SR compared to smooth slopes. However, the results of these small-scale studies alone are not sufficient to develop design recommendations. To build on existing evidence of wave overtopping reduction, full-scale physical model tests were conducted on two SR cross-sections.

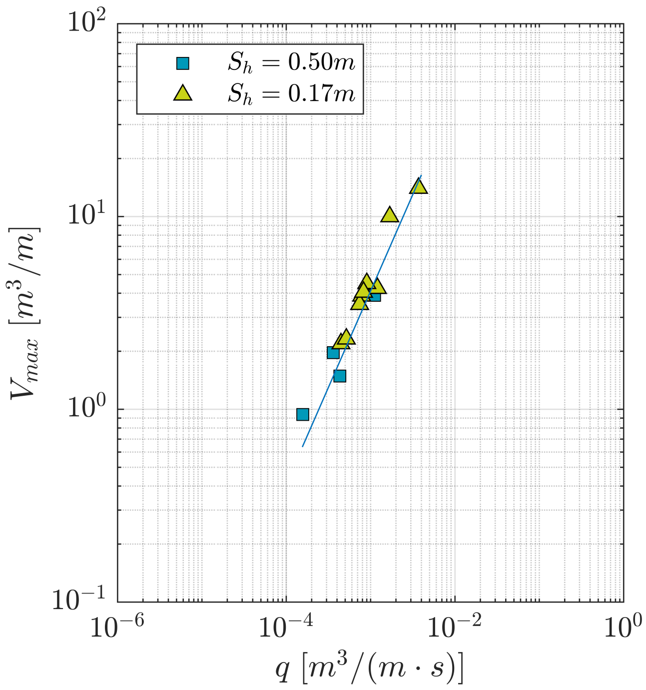

Figure 4.10: Relation between overtopping rate and maximum overtopping volume at stepped revetments

4.5.1 Reduction of average wave overtopping

Analysis of the average wave overtopping results focused on calculating the roughness factor (γ_f) for SR and investigating what parameters affect the slope roughness (1-γ_f). We found that the roughness factor is not constant, but rather a function of the hydraulic conditions and the geometry of SR. This finding is consistent to what was found at pattern placed revetments (Capel, 2015; Chen et al., 2020) and SR in small scale (Van Steeg et al., 2018; Kerpen et al., 2019; Gallach-Sánchez, 2018).

We identified that the relative overtopping rate, characteristic step height and wavelength are key parameters governing the overtopping reduction of SR. The important effect of the wave overtopping rate on overtopping reduction at slopes with roughness elements was previously highlighted (Capel, 2015; Van Steeg et al., 2018; Chen et al., 2020). When overtopping rates are large, preceding waves more frequently leave a layer of water on the SR as the next wave approaches. Accordingly, the residual water layer reduces friction between the incident wave and the steps. Examples of the water layers on the two tested SR are displayed in Fig. 4.11.

The characteristic step height influences the overtopping reduction in a similar way. With small characteristic step heights, the step openings are easier filled, creating a smoother ramp for consecutive waves to run up on. With the large steps (i.e. larger characteristic step heights) it was observed that waves more distinctly "jump" from step to step, while with the small steps a more continuous flow was noticed.

Furthermore, the model tests demonstrated that the wavelength significantly influences the overtopping reduction at SR, especially for the small steps (Fig. 4.5). When waves are longer, they contain more energy, and are accordingly less affected by the steps. It was also noticed that steeper waves dissipate more energy during the breaking process, thus reducing overtopping and enhancing the effects of the steps. The small-scale data was studied for similar influences. Inconsistent with our findings, the data of Van Steeg et al. (2018) display a slight decrease in slope roughness with wave steepness, while the data of Kerpen et al. (2019) showed no clear influence of the wave steepness.

Figure 4.11: Residual layer of water on stepped revetments as next wave approaches. **(a)** Configuration 1 with S_h=0.50m and **(b)** Configuration 2 with S_h=0.17 m

The small-scale prediction of Van Steeg et al. (2018) (Fig. 4.4(b)) underestimates the roughness factors of the tested SR by 2 to 19 % for the small steps and 15 to 31 % for the large steps. Presuming this underestimation is only due to scale effects, scale correction factors for roughness (f_{γ_f}) can be estimated relative to Eq. 4.5, where $f_{\gamma_f} = \gamma_{f(full\,scale)}/\gamma_{f(Eq.5)}$. For the large steps f_{γ_f} ranges between 1.18 and 1.45, while for the small steps f_{γ_f} ranges between 1.02 and 1.24. As a result, the small-scale study overestimates the ability of SR to reduce wave overtopping and it is thus critical to consider scale implications.

According to EurOtop (2018), substantial scale effects can be expected for low overtopping rates i.e. $q < 1 l/s$ per m (in prototype). In Van Steeg et al. (2018), 58 % of the tests had low overtopping rates. Moreover, in Kerpen et al. (2019), 88 %[1] of the tests with small steps (S_h=0.05 m), and 100 %[2] of the tests with large steps (S_h=0.30 m) had low overtopping rates. Consequently, the majority of the small-scale tests are likely affected by scale effects. In small scale, low overtopping events may be undetected as they are too small to measure.

Apart from potential scale effects, the overestimation of the overtopping reduction by small-scale predictions can partly be explained by the omission of some key parameters. Kerpen et al. (2019) only included the dimensionless step height to determine the

[1] when assuming a scale of 1:3.4 relative to S_h=0.17 m
[2] when assuming a scale of 1:1.7 relative to S_h=0.50 m

roughness factor. Their data revealed no clear influence of the wavelength on the roughness factor, while no significant improvement of their prediction was achieved by including the relative overtopping rate. Van Steeg et al. (2018) additionally included the wave overtopping rate. Practically only one wave period ($5.1 < T_{m-1,0} < 5.3$) was tested in their study. Since different wave periods were tested in our study, we could not validate the Van Steeg et al. (2018) prediction with our data.

We developed a prediction for the roughness factor of SR including the characteristic step height, relative overtopping rate (for smooth slopes) and wavelength, Fig. 4.7. This new prediction is compared to the roughness factors of previous small-scale studies in Fig. 4.12. Only slopes of $cot\alpha = 2; 3$ are presented. The prediction range was limited to values between $0 < -ln(q_{\gamma_f=1}/\sqrt{g \cdot H_{m0}^3}) \cdot cos\alpha \cdot S_h/L_{m-1,0} < 0.1$. The formula predicts roughness factors of both large-scale and small-scale studies over the full prediction range with the majority of data points within the 90 % confidence bounds. Due to variability in the small-scale data, the roughness factors can still be overestimated or underestimated. This is especially the case for the tests with low overtopping rates ($q < 1$ l/s per m), Fig. 4.12. A level of variability in overtopping results and between laboratories is to be expected as discussed in Section 4.2.5. However, it should be noted that in more cases Eq. 4.13 is conservative compared to the small-scale tests, as it predicts higher roughness factors and thus lower overtopping reduction for SR.

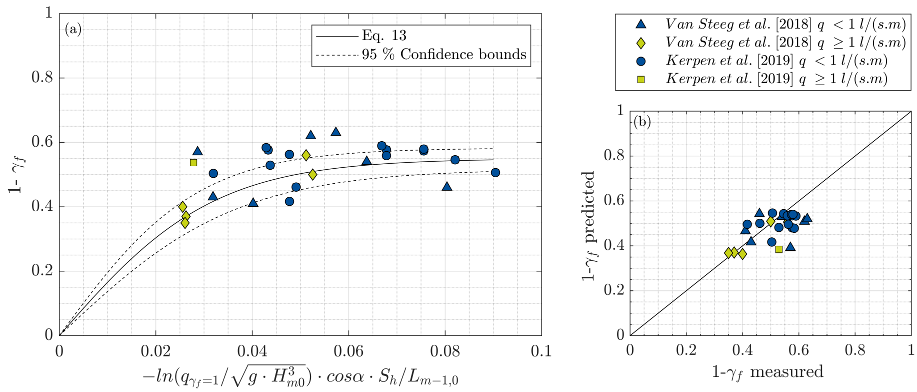

Figure 4.12: **(a)** Comparison of small-scale studies with Eq. 4.13. **(b)** Predicted and measured slope roughness of small-scale studies

Unlike in the small-scale studies (Van Steeg et al., 2018; Kerpen et al., 2019), reference tests with a smooth slope were not performed, due to financial and time restrictions. Instead Eq. 4.2 was used as smooth slope.

4.5.2 Individual wave overtopping at stepped revetments

Individual overtopping volumes were analysed to investigate the distribution of overtopping events and determine the maximum overtopping volumes at SR. Consistent with the literature, this study showed that the individual overtopping volumes at SR also

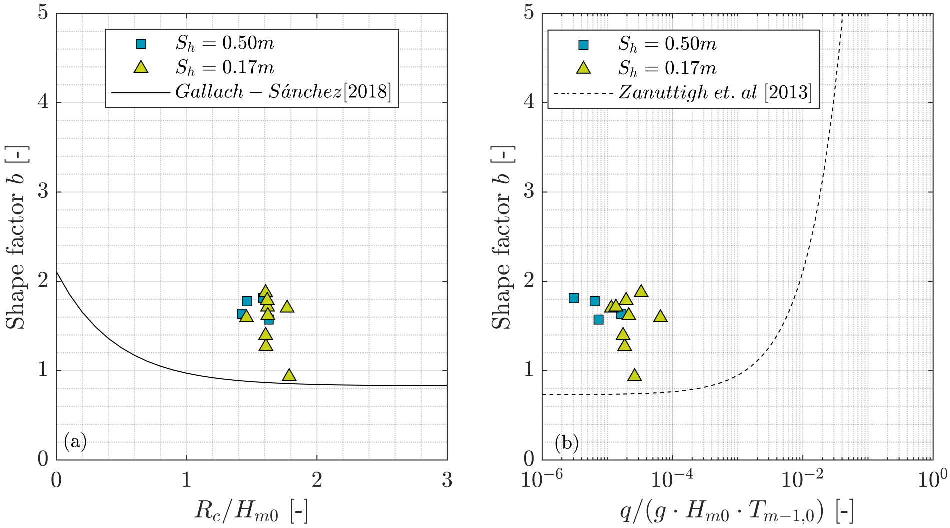

Figure 4.13: Shape factors for stepped revetments compared to previous studies

follow a two-parameter Weibull distribution with scale factor a and shape factor b.

Earlier studies (Bruce et al., 2009; Van der Meer and Janssen, 1994) found that the shape factor for most coastal structures is 0.75, while Victor et al. (2012), Gallach-Sánchez (2018) and Zanuttigh et al. (2013) developed formulae for determining the shape factor. The shape factors of the present study are compared to the predictions of Gallach-Sánchez (2018) and Zanuttigh et al. (2013) in Fig. 4.13. These predictions substantially underestimate the shape factors of the SR, meaning that the largest individual overtopping events at SR were more equal in volume. This finding is in contrast with that of Gallach-Sánchez (2018), who found slightly lower shape factors for steep low-crested SR compared to smooth slopes. However, for a small number of tests at rubble mound breakwaters with low percentages of overtopping waves (P_{ow} < 5%), Zanuttigh et al. (2013) found large shape factors (b >1.4). In the present study P_{ow} ranges between 2.6 % and 9.3 %, and as such, the higher shape factors could possibly be associated with the low probabilities and/or slope roughness. Further research is required to verify the shape factors of SR and to investigate the reasons for the underestimation by previous studies. We did not identify a clear influence of the wave steepness on the shape factor as found by Besley (1999) for other structure types.

The probability of overtopping at the tested SR (Eq. 4.8) is presented in Fig. 4.14. The number of incident waves was determined by the reflection analysis at the toe of the structure, while the number of overtopping waves was based on the number of overtopping events. It was not always possible to distinguish between consecutive overtopping waves. As a result, the actual number of overtopping waves could be underestimated, and the probability of overtopping could be slightly higher.

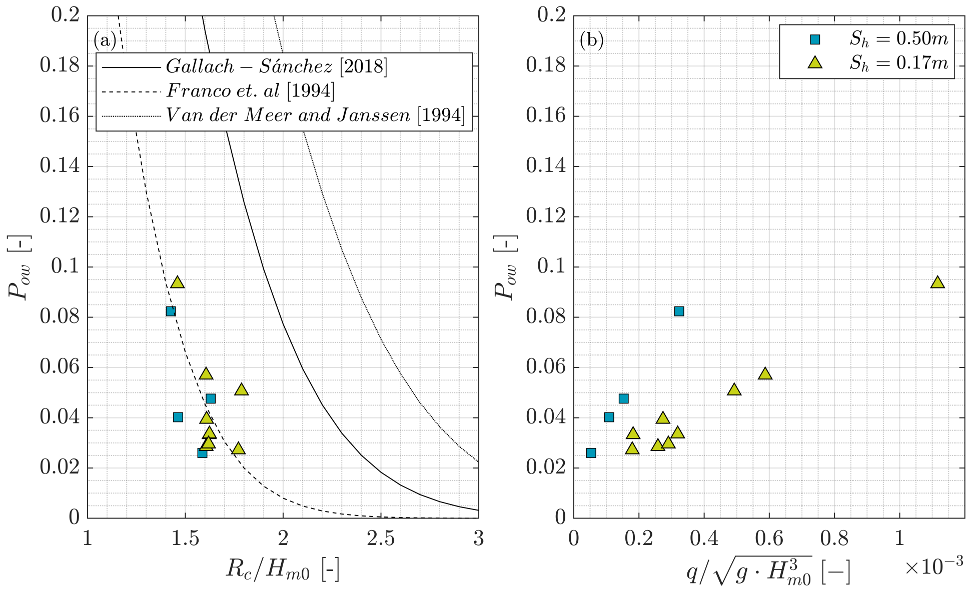

Figure 4.14: Measured probability of overtopping at stepped revetments **(a)** compared to previous studies and **(b)** as a function of relative overtopping

The probability of overtopping against relative freeboard is compared to equations from previous studies in Fig. 4.14(a). The relation is best described by the equation of Franco et al. (1994), which was developed for vertical walls. Gallach-Sánchez (2018) found that the probability of overtopping at steep low-crested SR is similar to that of a smooth slope shown in Fig. 4.14(a).

The probability of overtopping increases with relative overtopping as displayed in Fig. 4.14(b). For the same relative overtopping, a greater number of waves will cause overtopping at the SR with large steps (S_h=0.50 m) compared to the SR with the small steps (S_h=0.17 m). Within the tested range, the increase of probability of overtopping with relative overtopping is more rapid for the larger steps.

Furthermore, a linear relation between the maximum volume and the average overtopping rate of $V_{max}/q = 4111\ s^{-1}$ was identified for the SR. As this relation varies with structure geometry, wave conditions and storm duration (EurOtop, 2018; Franco et al., 1994), it is only applicable for the tested geometries and conditions in Tab. 4.2. For other structure types, including smooth slopes and vertical walls, the relation ranges between 100-1000 s^{-1} for a storm duration of 2 hours and $s_{m-1,0}$=0.04 (EurOtop, 2018).

4.5.3 Recommendations and limitations

The SR with large steps (S_h=0.50 m) proved more effective in reducing wave overtopping (0.43≤ γ_f ≤ 0.54). This supports the finding of Kerpen et al. (2019) that optimum wave overtopping reduction at SR occur at dimensionless step heights in the range of 0.5< $cos\alpha \cdot S_h/H_{m0}$ <2. Beyond this range γ_f increases (Kerpen et al., 2019) (i.e. lower

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reduction in overtopping rate). Nevertheless, the SR with small steps (S_h=0.17 m) still significantly reduce overtopping rates (0.57≤ γ_f ≤ 0.73).

This study developed an empirical prediction (Eq. 4.13) for γ_f of SR to be applied in the EurOtop formula (Eq. 4.2). In this way the overtopping at SR can be calculated and compared to various structure types in the EurOtop Manual. The downside of this approach is that the developed prediction for γ_f incorporates the uncertainties of the EurOtop formula.

Eq. 4.13 has the advantage that it is based on full-scale model tests and therefore not subjected to scale effects. However, as full-scale model tests are labour-intensive, expensive and time-consuming, Eq. 4.13 is based on only 20 tests with a somewhat narrow range of boundary conditions. The validity of Eq. 4.13 is restricted to the tested range of boundary conditions in Tab. 4.2 and further research is recommended to verify the equation.

As only one mild slope ($cot\alpha$=3) was tested in this study, it is uncertain how representative the results are for other slope angles. For steep SR ($cot\alpha$ ≤1), the data of Gallach-Sánchez (2018) suggests an influence of the slope angle on γ_f. For mild slopes ($cot\alpha$=2; 3), including the influence of slope angle on γ_f did not significantly improve the prediction of Van Steeg et al. (2018). This suggests a less significant influence of slope angle for the tested mild slopes.

Differences between roughness factors in small-scale and large-scale model tests were identified. Caution should thus be applied with adopting small-scale results for design purposes, as especially low overtopping rates (q < 1l/s per m) are underestimated. Still it should be noted that model effects may significantly influence model tests, both in small and large scales. In this study the width of the overtopping channel (see Section 3.2) may be a source of model effects. As the tested SR were homogeneous across the flume width and no influence of the channel was observed, we do not expect it to be a major source of model effects. In the small-scale studies (Van Steeg et al., 2018; Kerpen et al., 2019) the wave flumes were divided in parallel sections to test multiple cross-sections simultaneously. This could be a source of model effects when the reflection coefficients of the parallel sections are too variable. To quantify the role of scale effects in previous studies in greater detail, the replication of the full-scale model tests of this study on scale would be required.

The findings regarding individual overtopping events at SR are based on 12 tests. Apart from Gallach-Sánchez (2018), who studied steep low-crested SR, no other literature studied individual wave overtopping at SR. Further verification of the findings regarding individual wave overtopping at SR is therefore advised.

Furthermore, the effects of wind and oblique wave attack are not considered in this study. Future studies in a 3D wave basin are recommended to determine the influence of oblique waves on wave overtopping at SR. Based on full-scale model tests, further research on design aspects at SR is ongoing. The analysis of wave reflection and wave run-up will enable guidance on maintenance requirements and public safety at SR, while a study on wave impacts will provide insights in structural design of SR.

4.6 Conclusions

SR are effective in reducing wave overtopping and, as such, are a suitable alternative to protect coastal communities, especially in urban and touristic areas. SR with large steps (S_h=0.50 m) offer a significant reduction in wave overtopping rates compared to a smooth slope (0.43≤ γ_f ≤ 0.54). With smaller steps (S_h=0.17 m), the overtopping reduction is less, but still significant (0.57≤ γ_f ≤ 0.73). Previous predictions for wave overtopping rates at SR were based on small-scale model tests and overestimate the overtopping reduction. A revised prediction (Eq. 4.13), based on full-scale model tests, is proposed to determine the roughness factor of SR (r^2=0.92; $RMSE$=0.03). The prediction incorporates the relative wave overtopping rate, characteristic step height and wavelength.

Furthermore, individual wave overtopping events at SR were analysed and found to follow a Weibull distribution. A median shape factor of b=1.63 was found, which is larger than for other coastal structure types. The average overtopping rate at SR thus consists of events more similar in volume. A linear relation (Eq. 4.17) was identified between the average overtopping rate and the maximum individual overtopping volume at SR. The proposed prediction for the roughness of SR and insights gained in individual overtopping volumes improve existing design support of SR. Since the findings are based on 20 tests with a somewhat narrow range of boundary conditions, further experimental verification is recommended.

Chapter highlights

- Based on full-scale wave flume tests, a new formula (Eq. 4.13) is proposed for the roughness factor (γ_f) of stepped revetments to be applied in combination with EurOtop (2018) overtopping formulae.
- The tested stepped revetment with large steps (S_h=0.50 m) proved more effective in reducing wave overtopping (0.43≤ γ_f ≤ 0.54). Nevertheless, with small steps (S_h=0.17 m) a significantly reduction of overtopping is still achieved (0.57≤ γ_f ≤ 0.73).
- Previous small-scale studies underestimate the roughness factor (γ_f) of the tested stepped revetments by 2–31 %. As a result, basing the crest-level design of stepped revetments on small-scale overtopping measurements could thus lead to unsafe designs.
- The individual overtopping events were analysed and found to follow a Weibull distribution. A higher median shape factor (b=1.63) was found for stepped revetments compared to other structure types.

Acknowledgement

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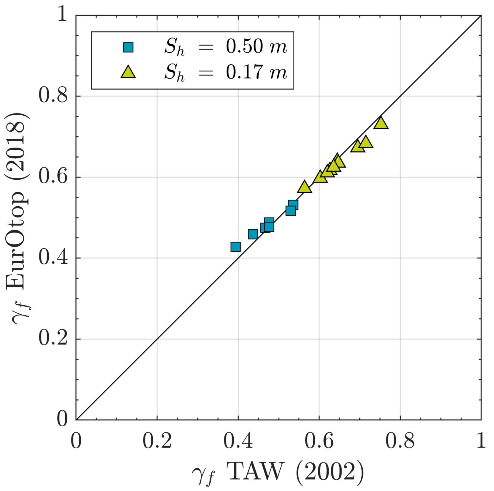

Figure 4.15: Comparison of roughness factors calculated with EurOtop (2018) and TAW (2002)

funding this research. We wish to thank Matthias Kudella and the technical team at FZK for the model construction, operational support and advice.

Additional information

Guidance for predicting wave overtopping at dikes is given by the Dutch Technical Advisory Committee on Flood Defence (TAW, 2002). Average wave overtopping rates at slopes can be determined by:

$$\frac{q}{\sqrt{g.H_{m0}^3}} = \frac{0.067}{\sqrt{tan\alpha}} \cdot \gamma_b \cdot \xi_{m-1,0} \cdot exp\left(-4.3\frac{R_c}{H_{m0} \cdot \xi_{m-1,0} \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \gamma_v}\right) \tag{4.18}$$

with a maximum of:

$$\frac{q}{\sqrt{g.H_{m0}^3}} = 0.2 \cdot exp\left(-2.3\frac{R_c}{H_{m0} \cdot \gamma_f \cdot \gamma_\beta}\right) \tag{4.19}$$

Fig 4.15 indicates that there are only slight differences between the roughness factors calculated with Eq . 4.2 and Eq. 4.18 ($r^2 = 0.94$).

Chapter 5

Wave Reflection and Wave Run-up at Stepped Revetments

This chapter is originally published in:

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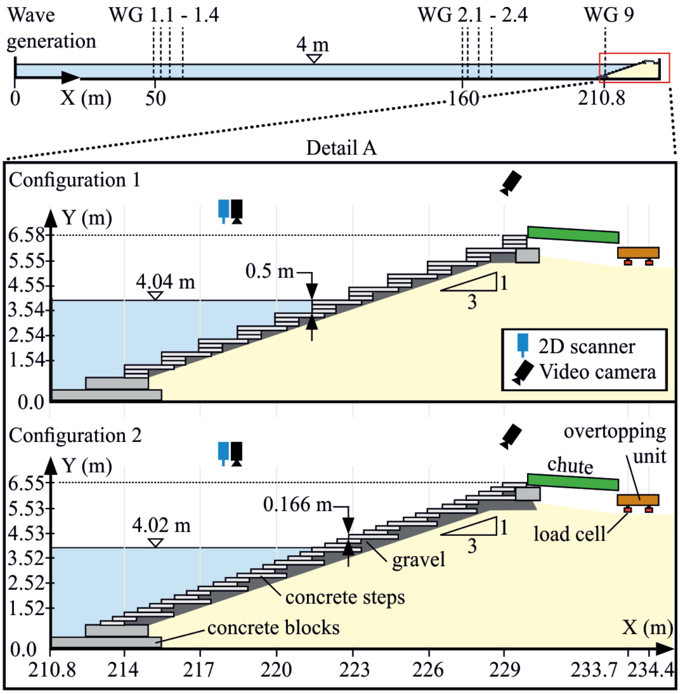

Figure 5.1: Side view of model configurations and instrumentation. Position of wave gauges (WG) and laser scanner indicated. Configuration 1 with large steps (S_h = 0.50 m) and Configuration 2 with small steps (S_h = 0.17 m) are shown.

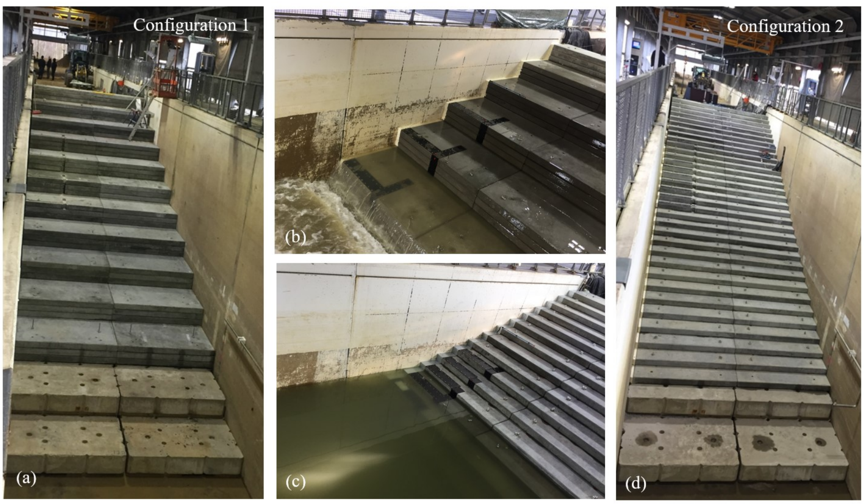

Figure 5.2: Stepped revetment model setup (cot α = 3). **(a)** Configuration 1 with step heights of 0.50 m during construction and **(b)** model testing. **(c)** The small steps of **(d)** Configuration 2 with step heights of 0.17 m.

5.1 Introduction

With rising sea-levels and higher waves (IPCC, 2019; Taherkhani et al., 2020; Young and Ribal, 2019), coastal communities and infrastructure are at elevated risk of flooding. To ensure coastal safety in the face of climate change, newly built, reinforced, adapted or higher coastal structures are necessary. Stepped revetments (SR) effectively reduce wave overtopping compared to smooth slopes due to the steps functioning as roughness elements (Kerpen et al., 2019; Schoonees et al., 2021). As a result, the required crest level of SR can be lower and the structure's footprint smaller, compared to a smooth dike. Their lower crest height, together with recreational benefits (e.g. providing access or seating), make SR especially suitable in urban or touristic settings.

A greater physical, process-based understanding of how waves interact with SR contributes to recommendations for optimised designs. The wave reflection process indirectly provides insight in how wave energy is dissipated on a SR as waves break, run up and run down the structure. Furthermore, knowledge on the wave reflection of structures is important as wave reflections may exacerbate scour (USACE, 2002), affect navigation (Seelig and Ahrens, 1981) and are required as input for numerical wave penetration models (Zanuttigh and Van der Meer, 2008). The wave run-up process is important for crest-level design and the wave run-up height distribution is valuable in planning structure maintenance and ensuring safety at SR accessible to the public. Both run-up heights and wave reflection coefficients could also be used for the calibration of numerical models that simulate wave-structure interactions.

Although some research has been carried out on wave reflection from SR (e.g. Kerpen, 2017), there is little published on reflection coefficients of SR. In contrast, several studies have focused on wave run-up on SR (see Section 5.2). These studies were often conducted with regular waves and at small scale. As it was established that wave overtopping measured in small-scale is affected by scale effects (Schoonees et al., 2021), it is anticipated that scaling also influences the run-up process.

With full-scale flume tests, we seek to broaden our current understanding of hydraulic processes affecting the functioning of SR as coastal structures. The objectives of our study are to (1) determine wave reflection coefficients of SR, (2) investigate wave run-up heights on SR and to (3) identify parameters influencing the hydraulic performance of SR. In this research two SR cross-sections are studied, both have a gentle slope ($\cot\alpha = 3$), while each has a different step height (S_h=0.17; 0.50 m). The present study is a complementing study to Schoonees et al. (2021), which focused on wave overtopping and the roughness of stepped revetments, and is based on the same model setup.

5.2 Previous studies

Since the 1930s stepped revetments have been investigated with physical model tests. In this section research related to wave reflection (Section 5.2.1) and wave run-up (Section 5.2.2) on SR is summarised. For a review on SR literature, see Kerpen and Schlurmann (2016) and Kerpen (2017).

5.2.1 Wave reflection

As waves interact with a coastal structure, their energy is dissipated, reflected and, in some cases, transmitted. The energy balance can be described by USACE (2002):

$$E_i = E_d + E_r + E_t \tag{5.1}$$

where E_i is the incident wave energy, E_d the dissipated energy, E_r the reflected energy and E_t the energy transmitted over the structure's crest.

In the case of rough impermeable structures, energy is dissipated due to wave breaking and the structure's roughness (Seelig and Ahrens, 1981). When wave overtopping occurs, energy is transmitted over the crest of the structure. The remaining wave energy is reflected from the structure. Considering the energy balance in Eq. 5.1, wave reflection provides indirect insight in the energy dissipation of a structure with known incident and transmitted wave energy. In turn, knowledge on energy dissipation of coastal structure types facilitates a greater understanding of their hydraulic performance in terms of coastal safety.

Apart from providing a better understanding of the energy dissipation, knowledge on the reflection of structures is important as wave reflection influences sea states and may hinder safe navigation (Seelig and Ahrens, 1981; Zanuttigh and Van der Meer, 2008) or manoeuvring in ports. Wave reflection may also affect the stability of coastal structures, as high reflections can exacerbate scour (USACE, 2002). Moreover, for numerical wave penetration models the reflection of a structure is required as input (Zanuttigh and Van der Meer, 2008). Typically wave reflection is quantified by the reflection coefficient C_r [-] (Goda, 2010):

$$C_r = \frac{H_{m0,r}}{H_{m0,i}} = \frac{E_r}{E_i} \tag{5.2}$$

where $H_{m0,i}$ [m] and $H_{m0,r}$ [m] are respectively the incident and reflected spectral significant wave height.

Empirical formulae for estimating the reflection coefficient of various structure types were derived by several previous studies. For the reflection coefficient of smooth impermeable slopes, Battjes (1974) developed the following formula:

$$C_r = 0.1 \cdot \xi_0^2 \tag{5.3}$$

in which the breaker parameter ξ_0 [-] is defined by:

$$\xi_0 = \frac{tan\alpha}{\sqrt{(H_{m0}/L_0)}} = \frac{tan\alpha}{\sqrt{(2\pi \cdot H_{m0}/g \cdot T_p^2)}} \tag{5.4}$$

where α is the slope angle [°] of the structure and L_0 [m] is the deep water wavelength based on the peak wave period T_p [s]. In more recent literature (e.g. EurOtop, 2018) the breaker parameter is often based on the spectral wave period $T_{m-1,0}$ [s], such that $\xi_{m-1.0}$ [-] is defined as:

$$\xi_{m-1.0} = \frac{tan\alpha}{\sqrt{(2\pi \cdot H_{m0}/g \cdot T_{m-1.0}^2)}} \tag{5.5}$$

Based on an extensive database of physical model tests with both monochromatic and irregular waves, Seelig and Ahrens (1981) derived a formula for estimating the reflection coefficients for beaches, revetments and breakwaters:

$$C_r = \frac{A \cdot \xi_0^2}{\xi_0^2 + B} \tag{5.6}$$

Where A [-] and B [-] are empirical coefficients based on the structure type. For impermeable smooth slopes, the coefficients are: A = 1.0 and B = 5.5.

For the reflection coefficient of rock slopes, Postma (1989) proposed the following equation:

$$C_r = 0.14 \cdot \xi_0^{0.73} \tag{5.7}$$

More recently, Zanuttigh and Van der Meer (2008) analysed reflection coefficients for various structure types in an extensive database and derived the following empirical equation:

$$C_r = tanh(a \cdot \xi_{m-1.0}^b) \tag{5.8}$$

in which a [-] and b [-] are empirical coefficients related to the roughness factor of the slope γ_f [-]:

$$\begin{aligned} a &= 0.167 \cdot [1 - exp(-3.2\gamma_f)] \\ b &= 1.49 \cdot (\gamma_f - 0.38)^2 + 0.86 \end{aligned} \tag{5.9}$$

For pattern-placed block revetments, Capel (2015) proposes the reflection coefficient can be estimated by:

$$C_r = (0.5 + c) + (0.4 - c) \cdot tanh(\xi_{m-1.0} - 2.2) \tag{5.10}$$

in which the empirical coefficient c [-] is related to the roughness factor of the slope:

$$c = 0.5 \cdot (1 - \gamma_f)^2 \tag{5.11}$$

Where γ_f is not a constant value, but a function of wave steepness, relative wave overtopping and the roughness density parameter.

Based on the mentioned studies, the reflection coefficients for sloped coastal structures are strongly related to the breaker parameter, but are also affected by the permeability and roughness of the structure. For permeable structures it was found that reflection reduces for rougher slopes (i.e. lower γ_f) (Zanuttigh and Van der Meer, 2008). In contrast, for impermeable slopes with roughness elements, reflection increases with roughness (Capel, 2015).

Wave reflection from SR is discussed in Kerpen (2017), who conducted small-scale model tests on SR with step heights (S_h) of 0.05 m and 0.3 m (model values). The study found that SR with larger step heights ($H_{m0}/S_h < 0.5$) are highly reflective as they experience reflections comparable to composite or vertical walls. Smaller step heights ($H_{m0}/S_h \geq 0.5$), classified as macro roughness, were found to be moderately reflective (but stronger dissipative) with reflection coefficients comparable to rock slopes with an impermeable core as measured by Postma (1989).

Full-scale model tests to determine wave overtopping at SR are described in Schoonees et al. (2021). As wave reflection is interrelated via the energy balance (Eq. 5.1) to energy dissipation and transmission, the findings regarding wave overtopping reduction at SR are of interest. The study highlighted that the governing parameters affecting wave overtopping reduction are the relative wave overtopping rate, the characteristic step height (the step height perpendicular to the SR slope denoted by $\cos\alpha \cdot S_h$) and the wave length. Also the wave steepness was shown to affect wave overtopping reduction. Based on these findings and the interrelation between wave overtopping and wave reflection in Eq. 5.1, expected influences of these parameters on wave reflection can also be anticipated.

To derive the influences of these parameters, two SR cases are considered with different crest-levels but otherwise identical geometry and hydraulic conditions. At the first SR case, wave overtopping occurs and $E_t \neq 0$. The crest of the second SR is sufficiently high such that no overtopping occur, i.e. $E_t = 0$. Since the incident wave conditions are identical, E_i is equal in both cases. Furthermore, similar energy dissipation will take place, although for the first case with overtopping it is expected that less energy is dissipated during wave run down. Considering the energy balance between the two described cases, the transmitted energy E_t of case 1 is thus related to E_r of case 2.

Where Schoonees et al. (2021) considered SR with overtopping ($E_t \neq 0$), the present paper considers SR where no wave overtopping occurs ($E_t = 0$). Although the water level was varied between the two studies and wave conditions were not identical (but similar), the findings regarding wave overtopping serve as basis for the expected findings of the present study. It is hypothesised that within the tested range of boundary conditions ($2.6<T_p<6.5$; $0.8<H_{m0}/S_h<5.2$) wave reflection is reduced for lower wave periods, steeper waves and larger characteristic step heights.

5.2.2 Wave run-up

As a wave reaches a sloped structure, it runs up the slope, reaches a highest point with maximum potential energy, reverses direction and runs down the slope again. The run-up height is defined as the vertical distance between the highest run-up point and the still water level. For irregular waves, the run-up height is commonly expressed as $R_{u2\%}$, defined as the run-up level exceeded by 2% of incoming waves. Although in recent designs the crest level is often determined based on allowable wave overtopping (EurOtop, 2018), $R_{u2\%}$ is also considered when determining the crest level of a dike or an embankment. Apart from the crest level design, the run-up height also provides insight in understanding wave-structure interaction. Moreover, the wave run-up height distribution can be valuable in planning structure maintenance and ensuring safety at SR accessible to the public. Surface elevations and run-up heights can also be used for the calibration of numerical models that simulate wave-structure interactions.

Research on wave run-up at dikes dates back to the 1930s (EurOtop, 2018; Wassing, 1957). Wassing (1957) summarises Dutch experimental research spanning 20 years and discusses factors governing wave run-up at dikes. According to the study the governing hydraulic factors are wave height, steepness and angle of wave attack. Structure-related factors influencing wave run-up include foreshore conditions as well as dike slope, shape and cover. Various dike covers were tested, including covers with roughness elements of steps, blocks and ribs.

Numerous studies on the influence of roughness elements on wave run-up followed (e.g. Battjes, 1974; Franzius, 1965; Führböter et al., 1989; Savage, 1958). The reduction in wave run-up by roughness elements is quantified by the roughness factor (γ_f), defined as the relation between the run-up on a rough and a smooth slope under otherwise identical conditions. For ribs wave run-up reduction increases with rib height in relation to run-up water thickness (TAW, 1972; Franzius, 1965). However, rib heights larger than the run-up water thickness, do not show additional benefits in run-up reduction. The slope angle, rib height in relation to wave height, distance between ribs, as well as wave steepness has been shown to influence roughness factors (TAW, 1972).

Van der Meer and Janssen (1994) described wave run-up as a function of breaker parameter and reduction factors, including the roughness factor (γ_f). Ranges of γ_f are presented for various block elements and ribs. Building on Van der Meer and Janssen (1994), Dutch guidelines (TAW, 2002) proposed updated equations for estimating wave run-up, including empirical coefficients for probabilistic calculations. An extensive list of roughness factors for various dike covers is presented based on new research. The roughness factors are given as single values, instead of ranges.

Based largely on European research and guidelines (including TAW (2002)), joint European guidelines on wave run-up and overtopping at coastal structures were compiled in a manual (EurOtop, 2018). The EurOtop manual proposes the following empirical equation for estimating the wave run-up height on a slope:

$$\frac{R_{u2\%}}{H_{m0}} = 1.65 \cdot \gamma_b \cdot \gamma_f \cdot \gamma_\beta \cdot \xi_{m-1.0} \tag{5.12}$$

with a maximum of:

$$\frac{R_{u2\%}}{H_{m0}} = 1.0 \cdot \gamma_f \cdot \gamma_\beta \cdot (4 - \frac{1.5}{\sqrt{\gamma_b \cdot \xi_{m-1.0}}}) \tag{5.13}$$

where $R_{u2\%}$ is the wave run-up height exceeded by 2% of the incoming waves [m], H_{m0} is the spectral significant wave height [m], γ_b depicts the influence factor for a berm [-], γ_f the influence factor for the roughness [-], γ_β the influence factor for oblique wave attack [-] and $\xi_{m-1.0}$ is the breaker parameter [-]. Only influence factors for roughness are considered in this study, such that $\gamma_\beta = \gamma_b = 1$.

In EurOtop (2018) roughness factors are proposed for various roughness elements. Mostly a constant value for γ_f is proposed, thus unrelated to wave conditions. These proposed roughness factors are valid for plunging waves ($\xi_{m-1.0} \leq 1.8$) and have to be corrected for surging waves ($\xi_{m-1.0} > 1.8$) by applying the following equation:

$$\gamma_{f,surging} = \gamma_f + (\xi_{m-1.0} - 1.8) \cdot (1 - \gamma_f)/8.2 \tag{5.14}$$

Similar to slopes with roughness elements, the stairs of SR reduce wave run-up and could also be considered as roughness elements. Although until now the values of γ_f for SR are not included in guidelines such as EurOtop (2018), several studies on wave run-up at SR were conducted since the 1930s. Some previous studies on wave run-up on SR are further discussed, see Tab. 5.1 for an overview. For reviews and visualization of data from previous studies in wave run-up on SR see Kerpen and Schlurmann (2016); Kerpen (2017).

In several experimental studies, physical model tests were conducted with regular waves to study wave run-up at SR, Tab. 5.1. Saville (1955, 1956) studied a steep SR (cot α = 1.5) with a foreshore slope of 1:10. The depth at the toe of the structure (d) were varied ($0 \leq d/H \leq 3$). Relative wave run-up (R/H) is presented as a function of wave steepness and relative water depth. Results show relative wave run-up decreases with wave steepness for all relative water depths. Jachowski (1964) conducted model tests for two SR slopes (cot α = 2; 3) with a flat foreshore with relative water depths (d/H) ranging between 4 and 15. In line with Saville (1955, 1956), Jachowski (1964) also found that relative depth and wave steepness influence relative wave run-up at SR.

Stoa (1978) analysed the data from Jachowski (1964) and Saville (1955, 1956) and calculated roughness factors from their results, Tab. 5.1. For the data of Saville (1955, 1956), an average roughness factor of 0.76 was found for $d/H = 3$, which is similar to the average roughness factor of 0.75 found for the data of Jachowski (1964) with a steeper slope (cot α = 2). The results of Jachowski (1964) indicate that the flatter slope (cot α = 3) is less efficient in reducing wave run-up. Stoa (1978) notes that a constant roughness factor is not applicable for all wave conditions for a certain roughness element.

5

Xiaomin et al. (2013) conducted model tests with 5 cross-sections of SR, all with a steep slope (cot α = 2.5). The SR are installed on a caisson and thus does not continue until the seabed. An average roughness factor of around 0.5 was found.

Kerpen et al. (2014) studied wave run-up and overtopping for two SR slopes (cot α = 2; 3). Similar to Xiaomin et al. (2013), the SR were also placed on a caisson. Although roughness factors for wave run-up are not specified, the following equations for estimating wave run-up are derived:

$$R_u/H = 0.6 \cdot \xi \qquad \xi < 3.75 \tag{5.15}$$

$$R_u/H = 2.36 \qquad \xi \geq 3.75 \tag{5.16}$$

Furthermore, Kerpen (2017) researched wave run-up on SR by means of physical model tests with irregular waves. Three SR slopes (cot α = 1; 2; 3), each with two step heights (S_h = 0.05 m; 0.30 m), were tested with a wide range of hydraulic conditions. The measurements showed that run-up on SR increases with wave height and wavelength as well as for steeper slopes. In contrast to run-up on smooth slopes, no distinct difference in relative run-up between plunging and surging waves was found. The study attributes this to highly turbulent run-up on the steps for all breaker types. The study proposes equations for estimating the wave run-up on SR by fitting a function following Schüttrumpf (2001):

$$\frac{R_{u2\%}}{H_{m0}} = a \cdot tanh(b \cdot \xi_{m-1.0}) \tag{5.17}$$

where:

$$a = 3.0; \quad b = 0.65 \quad \text{for smooth slopes } (\gamma_f = 1) \tag{5.18}$$

$$a = 2.6; \quad b = 0.38 \quad \text{for } H_{m0}/S_h \geq 1 \tag{5.19}$$

$$a = 2.0; \quad b = 0.28 \quad \text{for } H_{m0}/S_h < 1 \tag{5.20}$$

Moreover Kerpen (2017) determined a roughness factor for each test condition. The roughness factor was calculated relative to the run-up equation for smooth slopes according to Schüttrumpf (2001) (Eq. 5.17). Analysis of the calculated roughness factors revealed that the governing influences on roughness factor of SR are the ratio between wave height and step height as well as the breaker parameter. An optimum reduction in wave run-up (minimum roughness factor), regardless of wave steepness, was identified for a step ratio of $H_{m0}/S_h = 2$. Moreover, it was found that the position of the still water level on the step becomes important when $1 < H_{m0}/S_h < 2$.

Wave run-up measurements on SR exposed to irregular waves are even more challenging than those exposed to regular waves. In regular wave studies, Saville (1955) determined wave run-up heights by observation, while Xiaomin et al. (2013) determined run-up heights with camera and manual post processing. Run-up heights in Kerpen (2017), measured for irregular waves, were determined with a wave gauge consisting of two parallel steel rods. The wave gauge was positioned parallel to the structure slope, stretching across the step edges. The study highlights the importance of post processing of run-up measurements, due to the turbulent run-up flows and air entrainment. Moreover, the positioning of the gauge over the edges of the steps affects the measurements, since this is where frequent local splash-ups and air entrainment occur as water hits the vertical surface of the step. Since these challenges and potential inaccuracies are expected to be more severe at larger steps, where splash-ups occur more often, it is important to explore innovative measurement techniques for wave run-up at SR.

5.3 Materials and Methods

5.3.1 Test facility and experimental setup

Wave reflection and wave run-up on stepped revetments were studied by means of full-scale physical model tests. The model tests were conducted in the 307 m long Large Wave Flume (GWK) located in Hannover, Germany. The flume is 5 m wide, 7 m deep and has a maximum operational water depth of 5 m. Regular and spectral waves were generated with the GWK's piston type wave maker, equipped with active wave absorption.

With the available time and budget it was possible to test two SR cross-sections. Small-scale studies (Kerpen, 2017; Van Steeg et al., 2018) found that wave overtopping reduction at SR are largely influenced by the step height relative to incident wave heights. Based on this finding, it was decided to test two step heights, Fig. 5.1. The first configuration was tested with a large step height of S_h = 0.5 m. This height was selected based on the height of a bench, such that the structure could provide seating as additional function. The second configuration had smaller steps with a typical dimension for walking up a staircase ($S_h = 0.5/3 \approx 0.17$ m). Both configurations were constructed with a slope of 1:3 (cot α=3) to have a small structure footprint whilst representing a German dike slope which typically ranges between cotα=3 and cotα=7 (Schüttrumpf, 2008).

The models consisted of a sand-filled core and an underlayer of gravel on which precast concrete elements were stacked to form the revetments' steps. A rod and anchor system was used to attach the concrete elements to one another. For both configurations the models were placed at a distance of 210.8 m from the wave maker (X = 210.8 m) and extended across the entire flume width of 5 m. The construction time of Configurations 1 and 2 was 4 and 3 weeks, respectively.

A total of 9 wave gauges were employed to measure the surface elevation with a sampling frequency of 100 Hz. To separate incident and reflected waves, two arrays of wave gauges were installed, consisting of 4 gauges each. The first and second array were placed respectively between 50 to 60 m and 160 to 170 m from the wave maker. A ninth wave gauge was placed in line with the structure's toe (X = 210.8 m).

Even though the wave maker is equipped with active wave absorption, some wave energy could still be re-reflected from the wave board, especially infragravity waves. As a result, resonance may occur in the flume, causing variability in wave spectra and affecting the water level at the SR. In turn, this potential change of water level may affect wave run-up and reflection measurements. The wave power spectra at the second wave gauge array (WG 2.1-2.4) were checked for spectral energy with frequencies below 0.04 Hz, showing low levels of wave energy. It is thus expected that the results are not substantially affected by flume resonance.

The surface elevation on the structure was measured by a 2D LiDAR (Light Detection and Ranging) sensor placed at X = 217.80 m. The sensor, manufactured by SICK, recorded 631 points per profile with a sampling frequency of 25 Hz, accuracy of 12 mm and angle resolution of 0.167°. Additionally two video cameras, synchronized with other instrumentation, filmed the model tests. Further instruments included load cells to

Table 5.1: Overview of previous studies on wave run-up on stepped revetments.

Study	**Waves**	$\mathbf{H_{m0}/S_h}$	$\mathbf{S_h}$ **[m]**	**cot** α	γ_f	**Comment**
Saville (1955)	Regular	3-12	0.018	1.5	0.56-0.80	Scale 1:17; foreshore slope 1:10; $0 \leq d/H \leq 3$
Jachowski (1964)	Regular	1.3-5.0	0.019	2 3	0.64-0.68 0.74-0.76	Scale 1:10
Xiaomin et al. (2013)	Regular	0.3-8.0	0.01; 0.022; 0.044; 0.088; 0.176	2.5	0.35-0.77	Scale 1:10
Kerpen et al. (2014)	Regular	0.38-1.22	0.04; 0.08	2; 3	Not specified	Scale 1:5 Foreshore berm
Kerpen (2017)	JONSWAP spectrum	0.22-4	0.05; 0.30	1; 2; 3	0.35-0.95	γ_f calculated relative to Eq. 5.17 (Schüttrumpf, 2001)

measure overtopping (see Schoonees et al. (2021) for overtopping results) and pressure sensors to determine wave impacts. These measurements fall outside the scope of the current paper.

5.3.2 Test programme

The test programme was designed to include tests with a range of wave heights, periods and wave steepness. The model tests in the scope of this study were conducted over 12 days and included tests with both regular and irregular waves. With regular waves it is easier to study the effect on the wave run-up process systematically by changing individual parameters. For each regular wave test a total of 50 waves were generated. An overview of the regular wave conditions is given in Tab. 5.2.

The tests with irregular waves (JONSWAP spectra; γ = 3.3) were performed with approximately thousand waves, based on 1 000 times the peak wave period (T_p), resulting in test durations between 50 and 120 minutes. The first minute of waves was repeated at the end of the test. This allowed the first waves to be disregarded as they are affected by the start-up of the wave maker and the initial motionless water in the flume. This way the correct spectrum with time frame ($1000 \cdot T_p$) is considered in the analysis. The ranges of tested wave conditions are given in Tab. 5.3. The present study does not include tests which resulted in wave overtopping.

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Table 5.2: Regular waves: Stepped revetment geometry and test conditions (wave conditions given at toe of SR).

Parameter	**Symbol**	**Unit**	**S_h = 0.50 m**	**S_h = 0.17 m**
Wave height	H	m	0.43 - 1.64	0.34 - 1.01
Wave period	T	s	2.89 - 7.00	2.88 - 5.95
Wave steepness	s	%	1.8 - 3.5	1.8 - 3.9
Breaker parameter	ξ	-	1.78 - 2.51	1.68 - 2.49
Water depth	h	m	4.04	4.02
Slope	$\cot\alpha$	-	3	3

Table 5.3: Spectral waves: Stepped revetment geometry and test conditions (wave conditions given at toe of SR).

Parameter	**Symbol**	**Unit**	**S_h = 0.50 m**	**S_h = 0.17 m**
Wave height	H_{m0}	m	0.44 - 1.01	0.30 - 0.87
Wave period	$T_{m-1.0}$	s	2.69 - 6.43	2.62 - 5.53
Wave steepness	$s_{m-1.0}$	%	1.1 - 4.1	1.0 - 3.5
Breaker parameter	$\xi_{m-1.0}$	-	1.65 - 3.13	1.77 - 3.33
Dimensionless step height	$\cos\alpha \cdot S_h/H_{m0}$	-	0.47 - 1.07	0.18 - 0.53
Step ratio	H_{m0}/S_h	-	0.89 - 2.02	1.74 - 5.13
Slope	$\cot\alpha$	-	3	3
Water depth	h	m	4.04 - 5.06	4.02 - 5.03

5.3.3 Determining incident and reflected waves

The assessment of both wave reflection and run-up are based on the wave conditions at the toe of the structure. A reflection analysis was performed on the second wave gauge array (WG 2.1-2.4), thereby adhering to the recommendation of Klopman and Van der Meer (1999) for the distance between the structure's toe and wave gauges. The reflection analysis is based on a least square method that decomposes incident and reflected wave fields. Software was employed for this purpose. For tests with regular waves, incident wave heights and periods were determined by the L~Davis software (TU Braunschweig, 2016). For the reflection analysis of irregular waves, WaveLab3 software (Aalborg University, 2013), based on Zelt and Skjelbreia (1992), was used. The reflection coefficients and incident wave conditions obtained from the reflection analysis were analysed as described in Section 5.4.1. Moreover, incident wave heights and periods at the toe of the structure are utilised in the analysis of both wave reflection (Section 5.4.1) and run-up (Section 5.4.2).

5.3.4 Calculating wave run-up heights

The measurement of wave run-up on a SR poses challenges as maximum wave run-up heights are not as easily distinguished as on a smooth slope. As waves run up a stepped slope, frequent splash-ups and air entrainment occur as waves interact with the steps, thus making it difficult to determine accurate wave run-up heights.

Wave run-up heights were determined from the measurements of the 2D LiDAR sensor. For each of the model configurations a reference test of the structures themselves was recorded. With the sensor coordinates of the reference test and the known dimensions of the revetment steps, the measurements were converted from the sensor coordinate system to the real coordinate system. The coordinates were converted by applying routines developed by Forschungszentrum Küste.

After the coordinate conversion, the measurements were separated according to the revetment steps, based on their X-coordinates (Fig. 5.1). This way a separate time series for every revetment step is created. For each time step the water level on every revetment step was determined by taking the median of the measurement across the entire width of the step. By taking the median, unrealistic spikes in the data and small splashes were removed. Based on the wave gauge at the toe of the structure (WG 9) the start and end time stamps for each incident wave was determined by employing the L~Davis software (TU Braunschweig, 2016). With the known time stamps for each wave, the maximum run-up height was determined for each wave. In the analysis, water depths on steps lower than 3.5 cm were disregarded since it was noticed that LiDAR readings of the wet surfaces of the steps showed small inconsistencies with the reference tests. Furthermore, in some cases the median water depth on a step was higher than the step height itself, while no water was detected on the step above. In these cases, the run-up height was limited to the step height itself (S_h = 0.50 m for Configuration 1 and S_h = 0.17 m for Configuration 2). For selected tests, visual validation (between LiDAR measurements and video recordings) confirmed that the applied methodology gives realistic run-up results.

Subsequently, run-up heights for each test were arranged in descending order and the run-up height exceeded by 2% of waves was calculated. The number of waves was determined by the reflection analysis performed on the second wave gauge array (WG 2.1-2.4).

Wave-phase averaged surface elevations were also determined for selected regular wave experiments of the large steps (S_h = 0.50 m), see Section 5.3.2. Firstly, the median surface elevation on every revetment step was calculated for each time step (t). Secondly, the obtained surface elevations were wave-phase referenced (t/T) and the first and last 10 waves of each run were discarded. Thirdly, the median wave-phase referenced surface elevations (i.e., the wave shape) was determined. The surface elevations were then shifted such that the maximum surface elevation of each wave shape (i.e., the wave crest) corresponded to t/T = 0 at the first revetment step.

5.4 Results

5.4.1 Wave reflection from stepped revetments

Influences of wave conditions and geometry on wave reflection

Wave reflection coefficients at the stepped revetments were analysed and examined to study how they are influenced by wave and geometric conditions. The influence of wave breaking, quantified by the breaker parameter ($\xi_{m-1.0}$), is examined in Fig. 5.3. The waves in the majority of the tests were surging ($\xi_{m-1.0} > 1.8$), while in four cases the waves are classified as breaking ($\xi_{m-1.0} \leq 1.8$). Overall, the reflection coefficient (C_r) increases with breaker parameter, thus leading to higher reflections for surging waves ($\xi_{m-1.0} > 1.8$). This finding is in agreement with Eq. 5.8 for impermeable smooth slopes ($\gamma_f = 1$) (Zanuttigh and Van der Meer, 2008), i.e. the stronger the surging, the higher the wave reflection.

In comparison to smooth slopes, the tested SR have lower reflection coefficients with the exception of one test condition, which has a similar wave reflection coefficient. The lower reflections can be ascribed to the revetment steps functioning as roughness elements. Mainly two groups of reflection coefficients can be distinguished in Fig. 5.3. The reasons for the groups are further considered below.

According to Zanuttigh and Van der Meer (2008), reflection coefficients are additionally related to the roughness factor of the structure ($\gamma_f = 1$). This approach is shown as reference, as it is based on an extensive data base and different types of coastal structures (see Section 5.2.1). By fitting Eqs. 5.8 and 5.9 with γ_f as the only unknown variable, the two groups can be described with $\gamma_f = 0.72$ and $\gamma_f = 0.39$. The tests with large steps (S_h = 0.50 m) are mainly categorized in the group of $\gamma_f = 0.39$. This is in agreement with Schoonees et al. (2021) which found lower roughness factors for the large steps. The tests with small steps (S_h = 0.17 m) are included in both groups, although most have higher reflections.

To obtain a better understanding of the governing influences on wave reflection at SR, wave parameters were systematically varied as presented in Fig. 5.4. Three sets of tests were conducted with ranging wave height and constant wave period as shown in Fig. 5.4(a). For completeness, other conducted tests are displayed in grey. All three sets show lower reflections for higher waves, independent of step height. Additionally, the influence of varying wave period on reflection was considered in two sets of tests shown in Fig. 5.4(b). It was found that for a constant wave height, the wave reflection

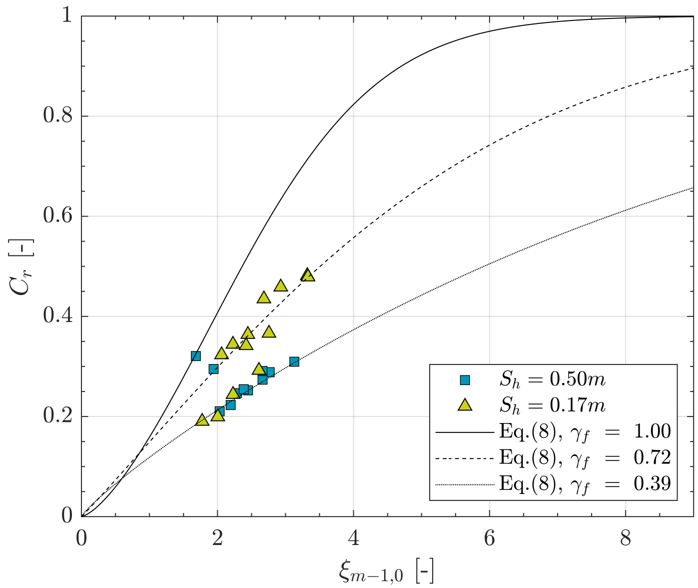

Figure 5.3: Wave reflection coefficients for stepped revetments with large (S_h = 0.50 m) and small steps (S_h = 0.17 m). Comparison to Eqs. 5.8 to 5.9 of Zanuttigh and Van der Meer (2008) for a smooth slope (γ_f = 1) and two rough slopes (γ_f = 0.72; 0.39).

increases with wave period. This increase is more dominant for the small steps (S_h = 0.17 m). Varying the wave height or wave period affects the steepness of a wave ($s_{m-1,0} = H_{m0}/L_{m-1,0}$), which in turn influences the wave breaking. The wave steepness for the corresponding sets in Fig. 5.4(a) and (b), is shown in Fig. 5.4(c). Lower reflections occurred for steeper waves as they overturn and break before interacting with the structure. With this type of breaking, more energy is dissipated and reflection decreases. In contrast, surging waves have a lower steepness, causing lower energy dissipation and higher reflections.

Previously, Postma (1989) found that reflection from rock slopes is influenced more by wave period than by wave height. Similarly, Muttray et al. (2006) found that wave period is the governing influence on reflection from rubble mound breakwaters, while the effect of wave height is negligible. For SR, an effect of both wave period and wave height is evident in Fig. 5.4(a) and (b). However, the wave period also has a larger influence than the wave height, as found by Postma (1989).

Generally, the wave reflection is lower at the SR with large steps (S_h = 0.50 m) compared to that of the small steps (S_h = 0.17 m), Fig. 5.4. To investigate the influence of the step height on wave reflection in more detail, three wave conditions were repeated for both step heights, Fig. 5.5(a). The wave conditions are not identical since the wave steepness varies slightly. These tests revealed that the reflection coefficient for the small

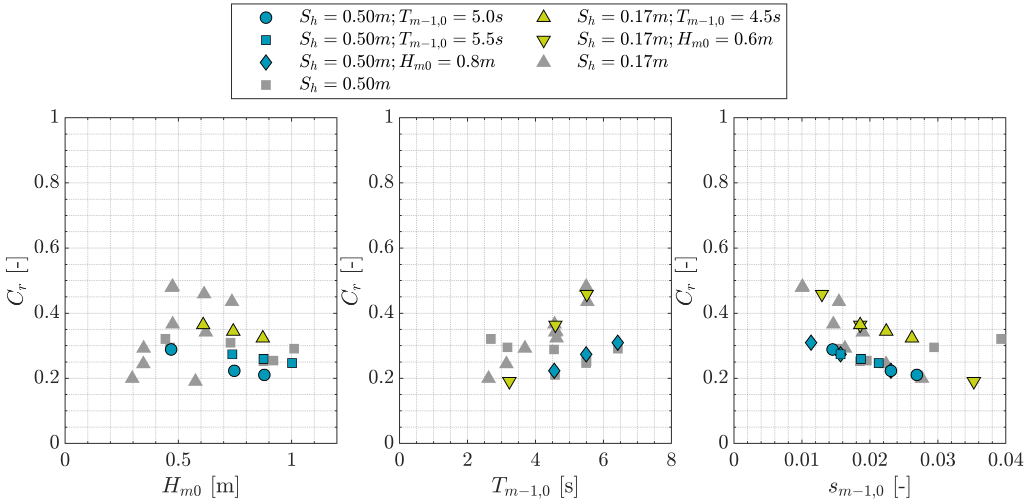

Figure 5.4: Influence of wave conditions on wave reflection from stepped revetments. **(a)** Wave reflection decreases for higher waves and constant wave period. **(b)** Wave reflection increases with wave period and constant wave height. **(c)** Wave reflection is lower for steeper waves.

steps is around 55% higher ($\Delta C_r = 0.13$) compared to the large steps. This observation can be explained by the higher roughness of the large steps, which cause more energy dissipation and consequently less energy is reflected.

However, it is not purely the step height that affects the wave reflection, but the step height in relation to the hydraulic conditions, as deduced from Fig. 5.5(b). Three wave conditions (and step heights) were scaled to a Froude scale of 1:3. Within these tests, negligible differences were found in wave reflection between the two scales, showing that scale effects were insignificant. Hence, the similar reflections for the two step heights in Fig. 5.4(b) highlight the importance of the relation between wave conditions and the SR geometry for wave-structure interactions and processes.

Influence of wave conditions relative to stepped revetment geometry

Two dimensionless parameters of SR and their relation to wave reflection were investigated, namely the dimensionless step height ($\cos\alpha \cdot S_h/H_{m0}$) and the step height to wavelength ratio ($\cos\alpha \cdot S_h/L_{m-1.0}$) (Fig. 5.5). The effect of dimensionless step height on wave reflection was found to vary within the tested range of wave conditions, especially for the small steps in Fig. 5.6(a). For the large steps, the reflection seems to increase with dimensionless step height. This observation is plausible, since an infinite dimensionless step height would imply a vertical wall, leading to the formation of (partial) standing waves in front of the structure and high wave reflection. The ratio of characteristic step height to wavelength, hereafter called wavelength ratio, has a more substantial effect on the wave reflection from SR, as seen in Fig. 5.6(b). Overall, the wave

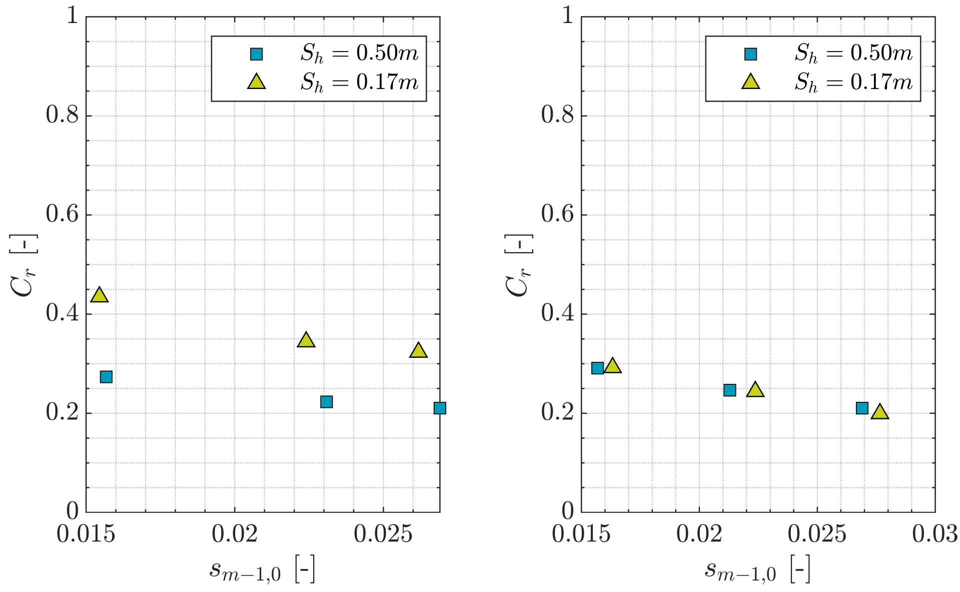

Figure 5.5: Influence of step height on wave reflection: **(a)** Three wave conditions were repeated for both stepped revetment configurations **(b)** Three wave conditions scaled to 1:3.

reflection decreases with increasing wavelength ratio up to $\cos\alpha \cdot S_h/L_{m-1.0} = 0.015$. The reflection from the large steps seems to be less influenced by the wavelength ratio and shows an increase beyond $\cos\alpha \cdot S_h/L_{m-1.0} = 0.015$.

Estimating wave reflection from stepped revetments

The wave reflections measured in this study show that wave reflection from SR is influenced by the breaker parameter, the wavelength ratio and the dimensionless step height. Firstly, like other structure types, the wave reflection coefficient at SR increases with breaker parameter, Fig. 5.3. Please note that only one slope was tested in this study. The ranging breaker parameter is therefore only due to the varying wave steepness. Although the slope angle will affect the physical processes on SR, this effect could not be assessed in the present study. Secondly, a smaller wavelength ratio at stepped revetments results in higher reflections, Fig. 5.6(b).

Considering the influences of both the breaker parameter and wavelength ratio, Eq. 5.21 is derived for estimating the reflection coefficient at SR (Fig. 5.7). The derived equation shows a good agreement ($r^2 = 0.98$ and RMSE = 0.012). However, when the wave height is smaller than the step height (i.e. $\cos\alpha \cdot S_h/H_{m0} \geq 1$), the wave reflection is higher than predicted by Eq. 5.21. It should be noted that Eq. 5.21 is only valid for the hydraulic conditions and geometry investigated in this study.

$$C_r = 0.021 \cdot [\xi_{m-1.0} \cdot (cos\alpha \cdot S_h/L_{m-1.0})^{-1}]^{0.45} \tag{5.21}$$

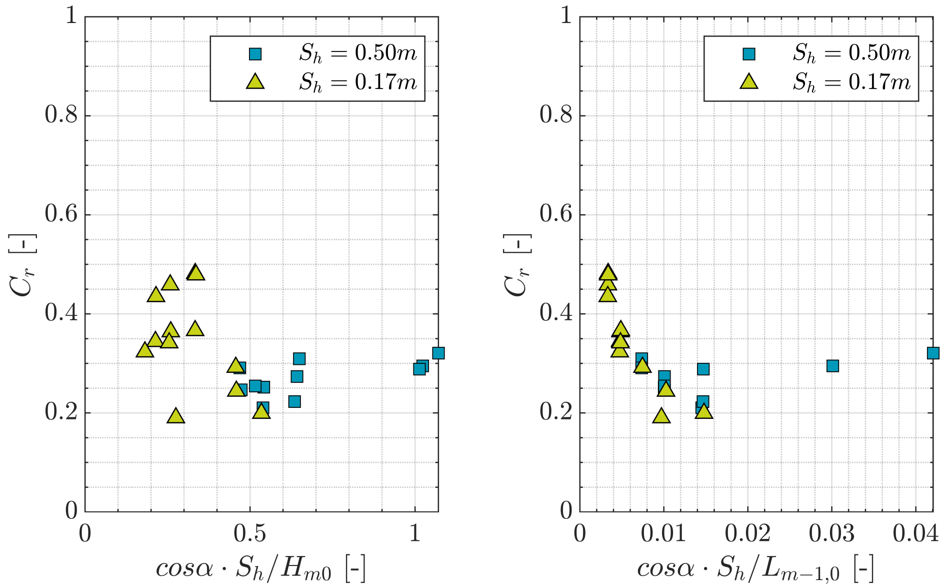

Figure 5.6: Wave reflection from stepped revetments: **(a)** Effect of dimensionless step height on wave reflection **(b)** Effect of wavelength ratio on wave reflection.

valid for:

$$0.18 < cos\alpha \frac{S_h}{H_{m0}} < 0.65$$

$$0.003 < cos\alpha \frac{S_h}{L_{m-1.0}} < 0.015$$

$$1.7 < \xi_{m-1.0} < 3.4$$

5.4.2 Wave run-up on stepped revetments

Wave run-up caused by regular waves

Wave run-up on SR is characterized by strong turbulent flows and frequent splash-ups as waves interact with the steps. With wave run-up caused by irregular waves, also the variable preceding waves that run down the revetment complicates the investigation of run-up heights. A total of 18 model tests were therefore performed with regular waves to systematically study run-up and its influencing parameters. Four sets of tests were performed with ranging wave period and constant wave height, all showing an increase in run-up height with wave period, Fig. 5.8(a). Two sets of tests with ranging wave height and constant wave period show that the wave run-up heights also increase with wave height, Fig. 5.8(b).

From literature on other structure types (Section 5.2.1) it is expected that relative run-up (R_u/H) is influenced by the breaker parameter ξ. This relation is investigated in Fig. 5.8(c) for the corresponding tests in Fig. 5.8 (a) and (b), showing variability in relative

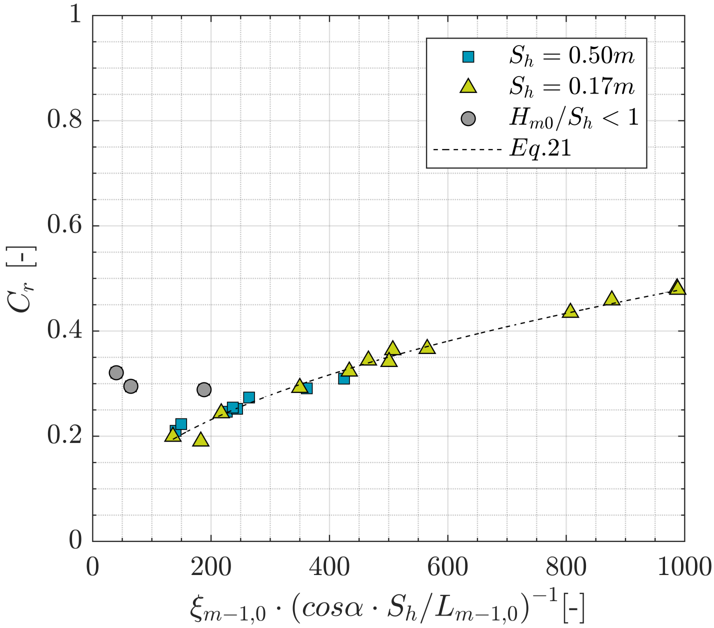

Figure 5.7: Estimating wave reflection at stepped revetments.

run-up. Considering each set with ranging wave period, an increase in relative run-up with breaker parameter is noticed. The two sets of tests with ranging wave heights show an initial increase with breaker parameter up to around $\xi = 2$, thereafter a decrease is noticed. As a result, the sets with ranging wave period and wave height seem to influence the relative run-up differently. However, it should be noted that this observation is based on a small number of tests with a limited range of ξ. Beyond the tested range of ξ there may be a clear influence of ξ on relative run-up.

The relation between wave height and run-up height for the 18 tests conducted with regular waves is shown in Fig. 5.9. The wave run-up height on SR increases with wave height. This increase can be described by:

$$R_u/H = 1.53 \tag{5.22}$$

The variability around this relation could be ascribed to the influence of wave period, since the run-up height increases with wave period as seen in Fig. 5.8(a). Nevertheless, Eq. 5.22 shows a good agreement with a coefficient of determination (r^2) of 0.96. The wave height thus has the largest influence on the wave run-up heights on the SR within the tested wave conditions.

Apart from the wave conditions, the step height also has an influence on the relative run-up height. Four test conditions were repeated for both large and small steps. Higher

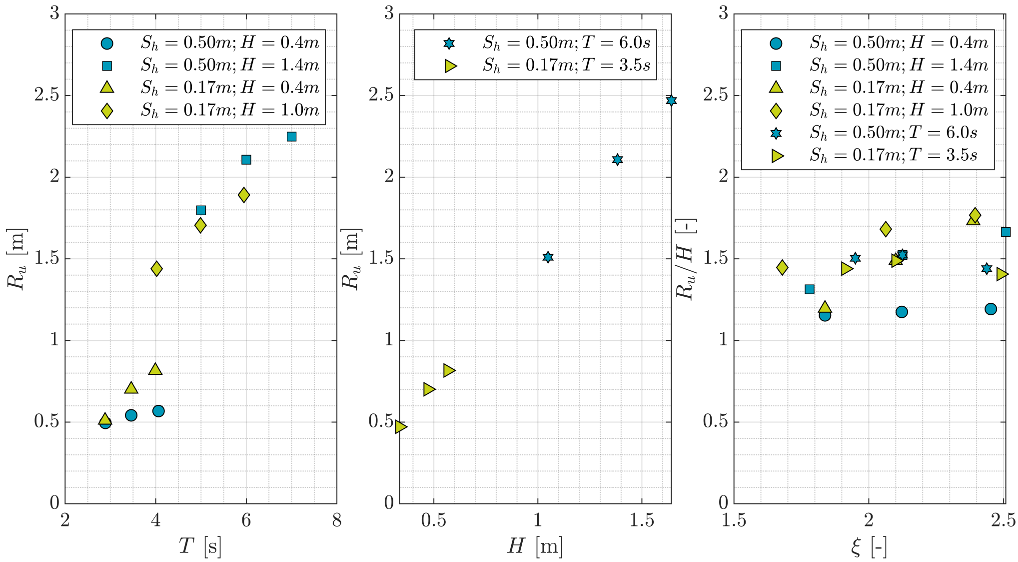

Figure 5.8: Wave run-up on stepped revetment exposed to regular waves. The effect of **(a)** wave period and **(b)** wave height on wave run-up is investigated. **(c)** Relative run-up as a function of breaker parameter.

relative run-up was measured for the small steps compared to the large steps, as seen in Fig. 5.10(a). Moreover, larger differences in relative run-up between the two step heights are identified for the higher and longer waves.

Similar to wave reflection, it is not only the step height that influences the relative run-up, but also the wave conditions in relation to the step height. Also for the regular wave tests, five wave conditions (and step height) were scaled down to a scale of 1:3. These tests displayed a lower relative run-up for the scaled tests for all cases except $\xi = 2.4$ (Fig. 5.10(b)). This could indicate that in smaller scale, wave run-up heights are underestimated, as was also found to be the case for wave overtopping discharge (Schoonees et al., 2021). Nevertheless, the differences between the scales could also be explained by the measurement uncertainties for wave run-up heights (Section 5.3.4).

By considering regular waves, the effect of individual parameters on wave run-up heights could be investigated. However, wave run-up heights do not only depend on the wave itself, but also on the characteristics of the preceding wave. Due to the stochastic nature of waves in reality, wave run-up on SR is further studied with irregular waves. The findings from regular waves form the basis for assessing the irregular wave run-up.

Irregular wave run-up on stepped revetments

Irregular wave run-up is quantified by $R_{u2\%}$ which is defined as the run-up height exceeded by 2% of the incident waves. For smooth slopes, relative run-up ($R_{u2\%}/H_{m0}$) increases with breaker parameter (EurOtop, 2018). This relation seems to be different for the tested SR as seen in Fig. 5.11. For the small steps it appears that the relative run-up decreases with breaker parameter. The relative run-up exposed to regular waves also

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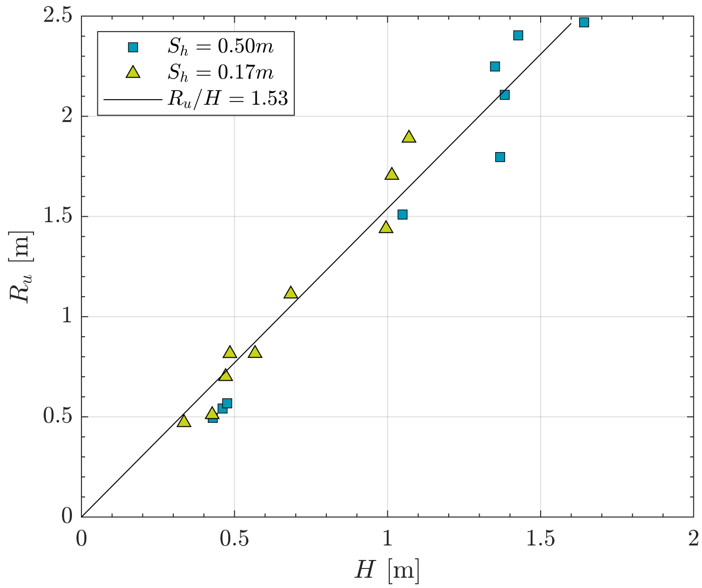

Figure 5.9: Wave run-up as a function of wave height.

presented variability in breaker parameter (Fig. 5.8(c)).

Although only three test conditions in the present study had step ratios (H_{m0}/S_h) below 1, the majority of relative run-up heights are higher compared to small-scale measurements of Kerpen (2017) for $H_{m0}/S_h < 1$ as shown in Fig. 5.11. For the tested large steps (S_h = 0.50 m) step ratios ranged between 0.9 and 2.0, whereas the small steps (S_h = 0.17 m) had step ratios between 1.7 and 5.1 (Tab. 5.3). As presented in Fig. 5.8, the small-scale run-up formulae underestimate wave run-up on SR between 31 and 51%. This underestimation can be due to scale effects, but since different measurement techniques were applied, model effects cannot be excluded. Previously, it was also found that small-scale studies underestimate wave overtopping rates at SR (Schoonees et al., 2021).

As is the case for regular waves, irregular wave run-up shows a linear relation with wave height, see Eqs. 5.23, 5.24 and Fig. 5.12. The run-up is higher for irregular waves than for regular waves. This increase was expected due to the influence of the variability in the wave run-up height of preceding waves. Higher run-up heights cause a residual layer of water on the structure, which reduces the roughness effect of the steps for the next approaching wave.

A linear fitting was applied to two ranges of step ratios (H_{m0}/S_h). The fit shows slightly higher relative run-up heights for higher step ratios ($H_{m0}/S_h \geq 3$). There is a small difference in performance between the step ratios, which can be ascribed to the type of run-up flow. For smaller step ratio ranges ($H_{m0}/S_h < 3$) frequent splash-ups

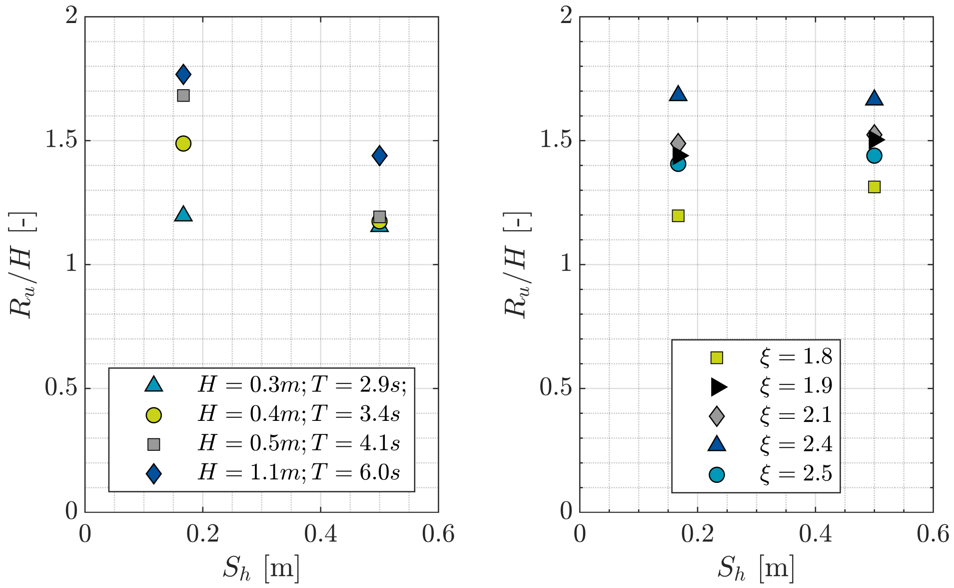

Figure 5.10: Influence of step height on relative wave run-up exposed to regular waves. **(a)** Four test conditions were repeated for both large and small steps **(b)** Comparison relative wave run-up of test conditions scaled to 1:3.

occur as waves “jump” from step to step. For the higher ratios ($H_{m0}/S_h \geq< 3$), a more continuous flow was observed. The derived relations between run-up height and wave heights are:

$$R_{u2\%}/H_{m0} = 2.09 \quad \text{for} \quad H_{m0}/S_h < 3 \quad (r^2 = 0.97 \text{ and RMSE} = 0.089) \tag{5.23}$$

$$R_{u2\%}/H_{m0} = 2.32 \quad \text{for} \quad H_{m0}/S_h \geq 3 \quad (r^2 = 0.97 \text{ and RMSE} = 0.044) \tag{5.24}$$

5.4.3 Reduction of wave run-up in comparison to smooth slopes

The steps of SR obstruct flow during wave run-up, generate turbulence and therefore reduce wave run-up heights compared to those on smooth slopes. This reduction in run-up height is quantified by the roughness factor (γ_f). Roughness factors were calculated with Eqs. 5.12 to 5.13 and for surging wave conditions corrected with Eq. 5.14. For the large steps γ_f ranges between 0.63 and 0.79, whereas for the small steps γ_f is between 0.59 and 0.83 (Fig. 5.13). These ranges are similar for both step heights and the SR reduce wave run-up effectively.

The influence of ranging wave conditions on slope roughness (1 - γ_f) is investigated in Fig. 5.13, where a higher slope roughness indicates a larger reduction in run-up

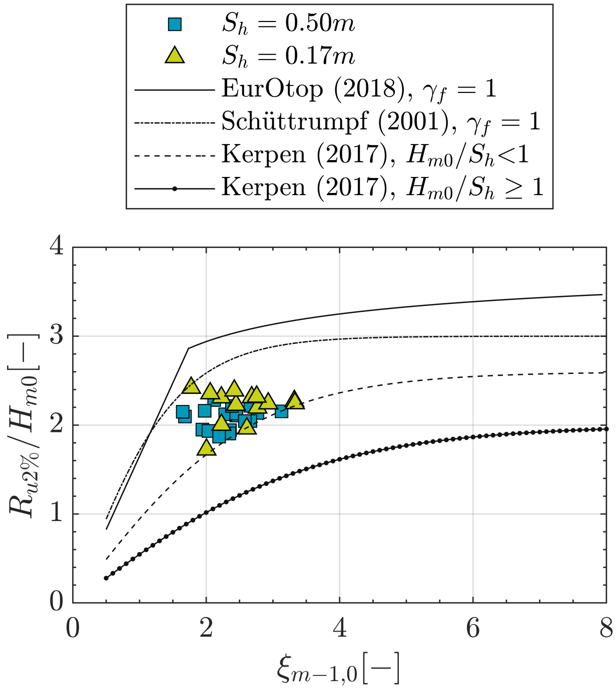

Figure 5.11: Relative wave run-up as a function of breaker parameter.

heights in comparison to a smooth slope. For the small steps, the slope roughness decreases with wave height, Fig. 5.13(a). This result implies that the small steps become less effective in reducing run-up for higher waves. For the large steps, the influence of wave height on slope roughness is rather unclear, due to considerable variability in the data. The influence of the wave period does also not show a distinct relation with slope roughness, for either step height Fig. 5.13(b). In contrast to what was found for wave overtopping, the slope roughness does reduce with wave steepness, Fig. 5.13(c). Hence, for steeper waves (plunging breakers) the steps are less effective in reducing wave run-up heights.

In the absence of wind in the laboratory, the splash-ups that occur as waves impact with vertical step surfaces lead to higher run-up heights, but do not necessarily lead to higher overtopping volumes, as they often collapse and fall vertically downwards. As such, this flow behaviour could explain the different findings between wave run-up and overtopping.

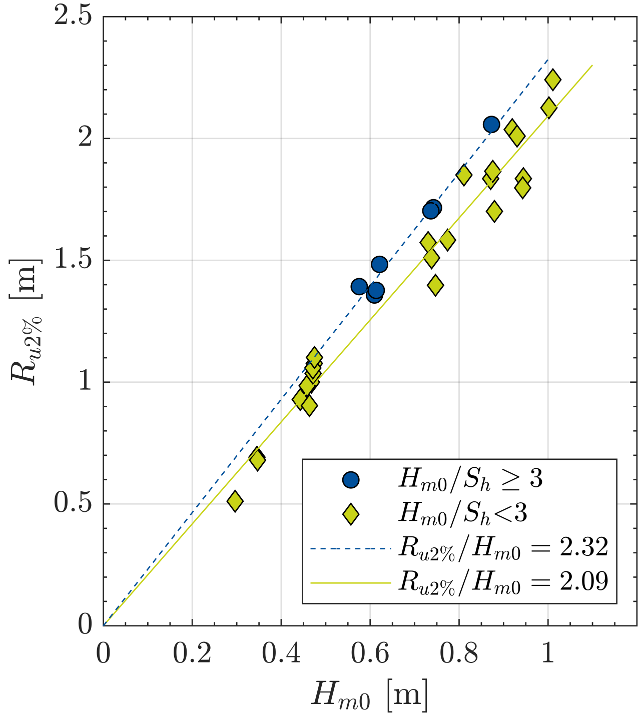

Figure 5.12: Irregular wave run-up as a function of wave height.

WAVE RUN-UP DISTRIBUTION AND PROCESSES ON STEPPED REVETMENTS FOR S_h = 0.5 M
To better understand the flow behaviour and to gain insight in the functioning of SR, an analysis beyond maximum wave run-up heights and reduction is required. The cumulative distribution (P) of wave run-up heights (R_u) on the SR with large steps (S_h = 0.5 m) is presented in Fig. 5.14 for two sets of tests. The first set shows three tests with wave periods of $T_{m-1,0}$ = 5.49 s and ranging wave heights (Fig. 5.14(a)). In the second set three tests with H_{m0} = 0.74 m and ranging wave period are shown (Fig. 5.14(b)). Although the presented range in breaker parameters is smaller for varying wave heights (Fig. 5.14(a)), the resulting range in $R_{u2\%}$ values is larger than for varying wave period. This confirms the finding that the incident wave height has a larger effect on the run-up height than the wave period.

Exceedance probabilities other than $R_{u2\%}$ may be of interest for planning public safety measures or maintenance. A run-up height of 1 m is exceeded by 15-47% of waves with varying wave height, while only 10-21% of waves with varying wave period will run up higher. In the presented figures, vertical lines occur around run-up heights with multiples of 0.5 m. These vertical lines are present since run-up heights that exceeded

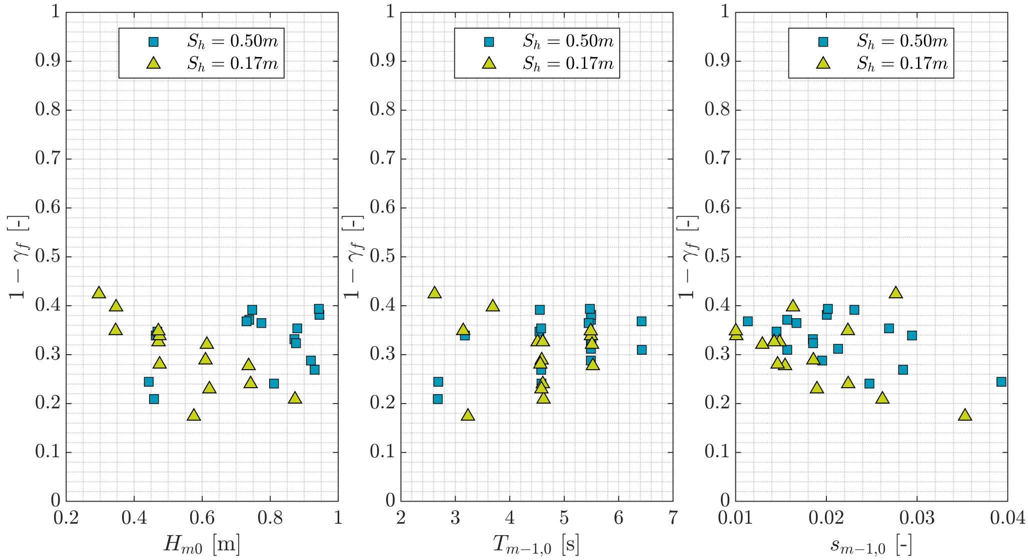

Figure 5.13: The roughness of SR steps dissipates wave energy and therefore reduces wave run-up heights in comparison to those on smooth slopes. This reduction is quantified by the roughness factor γ_f. with $\gamma_f = 1$ for a smooth slope. Influences of wave conditions on the slope roughness (1 - γ_f) of stepped revetments are shown. A higher slope roughness indicates a larger reduction in run-up heights in comparison to a smooth slope. Displaying how ranging **(a)** wave height, **(b)** wave period and **(c)** wave steepness influence the slope roughness of stepped revetments.

the step height and with no water layer on the step above, were limited to the step height (see Section 5.3.4). These lines thus indicate on which steps frequent splash-ups occurred.

The effect of the revetment steps on the wave shape as waves run up the SR is investigated in Fig. 5.15 for $S_h = 0.50$ m with regular waves. The figure shows a time series (t) of the water elevation on the revetment steps divided by the wave period (T). The bottom and top panel present two sets of tests: (a) to (k) show three tests with constant wave height and ranging wave period, whereas (l) to (v) show three tests with constant wave period and ranging wave height. It should be noted that the surface elevations shown include the effects of wave reflection.

While the wave run-up height can mostly be described by the wave height (Section 5.4.2), the results in Fig. 5.15 demonstrate that the wave period also affects the wave run-up process on stepped revetments. Considering (a) to (k), it is shown how waves with different shapes break and transform on the structure. The longest and flattest wave (ξ = 2.5; T = 7 s) is non-breaking and reduces systematically in height from (c) to (g), while in (h) a peak is seen as the wave splashes upwards upon impact with the step. The shortest and steepest wave (T = 5 s; ξ = 1.8) becomes steeper in (d) to (f), and overturns and plunges down as it reaches (g). Although the wave with T = 6 s (ξ = 2.1) is non-breaking (confirmed visually in video recordings), the wave splashes considerably

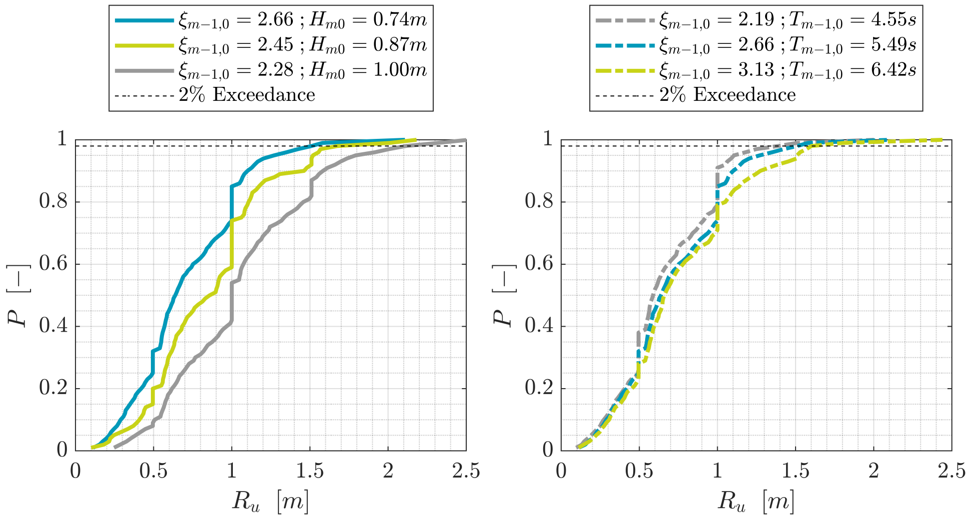

Figure 5.14: Distribution of wave run-up heights on stepped revetments for **(a)** ranging wave height with constant wave period ($T_{m-1,0}$ = 5.49 s) and **(b)** ranging wave period with constant wave height (H_{m0} = 0.74 m).

in (f), thus leading to the peak. The maximum wave run-up increases with wave period: for waves with T = 7 s and T = 6 s the maximum is reached at (j), while T = 5 s reaches its maximum at (i).

In (l) to (v), the transformation of waves with ranging wave heights is investigated. The wave condition with H = 1.4 m and T = 6 s (ξ = 2.1) is shown again as reference. The wave with H = 1.7 m (ξ = 1.9) surges up the structure and systematically decreases in height until reaching its maximum run-up height in (u). The wave is not affected by individual steps as the water runs up uniformly. For H = 1.0 m (ξ = 2.5) the wave surges up the revetment, but since H = 2 $\cdot S_h$ the wave “feels” individual steps as water splashes vertically upwards in (q). The maximum run-up heights evidently increase with wave height.

5.5 Discussion

Stepped revetments (SR) offer protection against wave action. As these structures effectively reduce wave overtopping compared to smooth slopes, their crest heights could be lower (Schoonees et al., 2021). Although a long history of investigations on SR exists, design recommendations for SR are not yet included in design manuals. A deeper understanding of how waves interact with SR would improve design recommendations of SR. Especially little is known regarding wave reflection from SR, while the wave run-up process was predominantly studied in small-scale and with regular waves. To address

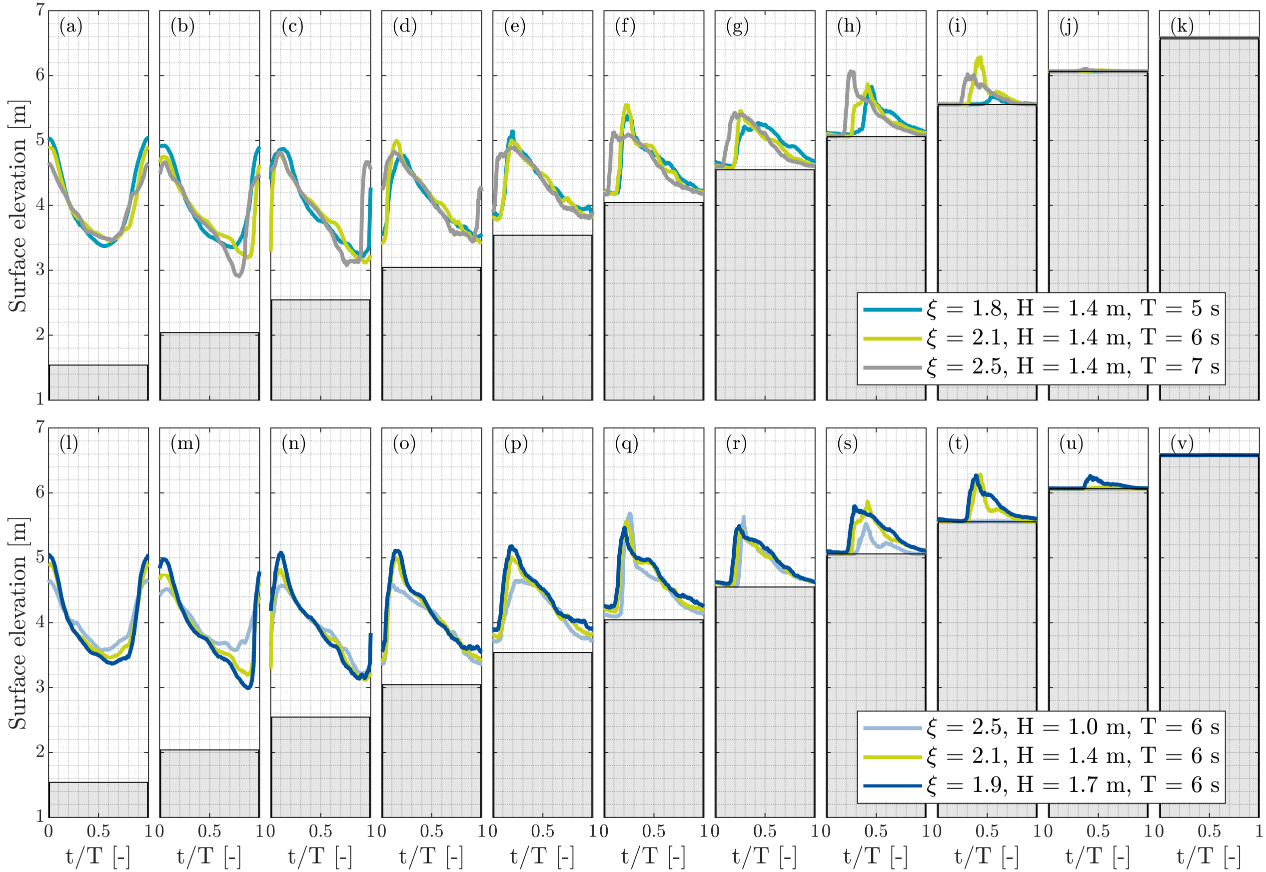

Figure 5.15: Wave-phase averaged surface elevations across stepped revetment: **(a)**-**(k)**: ranging wave period with constant wave height **(l)**-**(v)**: ranging wave height with constant wave period.

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these knowledge gaps and inform design recommendations, full-scale physical model tests were conducted on two gentle sloped (cot α=3) SR cross-sections, one with large steps (S_h = 0.50 m) and the other with small steps (S_h = 0.17 m).

5.5.1 Wave reflection

As waves reach a SR, a portion of the wave energy is dissipated due to wave breaking and the induced turbulence by the revetment steps, while the remaining wave energy is reflected. This wave reflection takes place gradually as a portion of wave energy is reflected from each vertical step surface with which the waves interacts. Since the steps of SR dissipate wave energy the wave reflection from SR is lower than from smooth slopes, as confirmed by the measured reflection coefficients presented in Fig. 5.3. Reflection coefficients between 0.19 and 0.48 were found for the tested SR (0.9 $\leq H_{m0}/S_h \leq$ 5.2). These reflection coefficients partly overlap with those found by Kerpen (2017), who measured reflection coefficients in small-scale model tests. Kerpen (2017) found that for SR with $H_{m0}/S_h \geq 0.5$, the reflection coefficients ranged between 0.35 and 0.60 and he thus classified these structures as moderately reflective.

Further analysis (Section 5.4.1) revealed that reflection coefficients of SR are not only influenced by the breaker parameter, but also by the wave conditions in relation to SR geometry. Repeated wave conditions for both the large (S_h = 0.50 m) and the small steps (S_h = 0.17 m) showed that reflection coefficients in the tested range were 55% higher for the small steps (Fig. 5.5(a)), indicating lower energy dissipation for the small steps. Furthermore, a negligible difference in reflection coefficients was found when waves and step heights were scaled 1:3, Fig. 5.5(b). Lastly, also the influence of wave conditions related to geometry was expressed by two dimensionless parameters (Fig. 5.6): dimensionless step height ($\cos\alpha \cdot S_h/H_{m0}$) and wavelength ratio ($\cos\alpha \cdot S_h/L_{m-1,0}$).

Based on the experiments, an empirical formula for estimating the wave reflection from SR was derived, Eq. 5.21 (r^2 = 0.98 and RMSE = 0.012). When the step height is larger than the wave height ($\cos\alpha \cdot S_h/H_{m0} > 1$), the reflection coefficients are higher than predicted and thus Eq. 5.21 is not valid. This can be explained by the influence of the number of steps the waves interact with. Wave energy is dissipated at each revetment step the waves interact with during wave run-up and run-down. Thus when $\cos\alpha \cdot S_h/H_{m0} > 1$, the waves interact with 1-2 revetment steps, resulting in lower energy dissipation and higher wave reflection. It was also found that the reflection coefficients decrease with wavelength. As the wavelength becomes larger in relation to the characteristic step height, higher reflections are experienced. It should be noted that Eq. 5.21 is derived for a limited range and is only applicable for the conditions as specified.

Wave reflection and wave overtopping are strongly related processes (Zanuttigh and Van der Meer, 2008) and findings on wave overtopping at SR can provide insights in wave reflections from SR. Based on the findings of Schoonees et al. (2021) for wave overtopping at SR, it was hypothesised (Section 5.2.2) that within the tested range of boundary conditions, lower reflection coefficients could be expected for shorter waves, steeper waves and larger characteristic step heights. The measured wave reflections of this study confirm this hypothesis. Furthermore, Zanuttigh and Van der Meer (2008) developed an equation for estimating the reflection coefficients of coastal structures based on the roughness factor γ_f, see Eqs. 5.8 and 5.9. The roughness factor for SR was measured in Schoonees et al. (2021) for wave overtopping and ranged from 0.43 to 0.73. By fitting Eqs. 5.8 and 5.9 to the reflection coefficients measured in this study, roughness factors of $\gamma_f = 0.39$ and $\gamma_f = 0.74$ were calculated (Fig. 5.3). As a result, the roughness factors measured in Schoonees et al. (2021) and determined with Eqs. 5.8 and 5.9 are within the same range, confirming the approximation of Zanuttigh and Van der Meer (2008). Note that Eqs. 5.8 and 5.9 were not developed for SR nor for deriving roughness factors of structures. As is the case for rough impermeable structures, lower reflections were experienced for rougher SR in this study. This finding is in contrast to what Capel (2015) found for impermeable block revetments, where wave reflection was found to increase with structure roughness.

5.5.2 Wave run-up

Wave run-up on SR caused by both regular and irregular waves were studied. The regular wave run-up showed that run-up heights increase with wave height and period for the large (S_h = 0.50 m) and small step (S_h = 0.17 m) configurations. However, within the tested range this relation is substantially stronger between run-up height and incident wave height. For regular waves a linear relation in Eq. 5.22 between run-up and wave heights was established, showing a good agreement ($r^2 = 0.96$). Under the same wave conditions, higher run-up heights occur on the smaller steps (S_h = 0.17 m), see Fig. 5.10(a). Overall, the wave run-up heights caused by regular waves are mainly influenced by wave heights and the dimensionless step height ($\cos\alpha \cdot S_h/H_{m0}$).

The relative run-up ($R_{u2\%}/H_{m0}$) on SR caused by irregular waves does not show a clear relation with the breaker parameter, unlike other sloped coastal structures (EurOtop, 2018). This could be due to vertical splash-ups as wave interact with the vertical step surfaces. Under irregular wave conditions, also a linear relation between run-up heights and wave heights was established for each of the step configurations in Eqs. 5.23 and 5.24. This relation could be used to estimate wave run-up at SR within the tested parameter range, see Tab. 5.3.

The physical model tests showed that wave run-up on SR is lower than at smooth slopes. This reduction is quantified by the roughness factor (γ_f), which shows high variability. As previously reported by Stoa (1978) and Kerpen (2017) the calculated roughness factors vary with wave conditions and structure geometry. Similar ranges of γ_f were calculated for both step heights. For S_h = 0.50 m, γ_f ranges between 0.63 and 0.79, while for S_h = 0.17 m, γ_f varies between 0.59 and 0.83. In small-scale, Kerpen (2017) found that roughness factors are predominantly influenced by the step ratio and breaker parameter. In the present study only the roughness factors for the small steps showed a clear relation with breaker parameter.

Furthermore, the roughness factors of the present study are higher than found by Kerpen (2017). Also for wave overtopping it was found that small-scale studies underestimate γ_f (Schoonees et al., 2021). Moreover, the roughness factor for run-up is higher than for wave overtopping found in Schoonees et al. (2021). This could be due to the vertical splash-ups, but also due to the wider range of dimensionless parameters measured in the present study (Tab. 5.3 and Schoonees et al. (2021)). Also in small scale, differences between roughness factors for wave run-up and wave overtopping were reported by Kerpen (2017).

5.5.3 Description of wave interaction with stepped revetments

Based on studying video material of regular waves, the interaction of waves with SR is described. Excerpts of video material can be viewed in the supplementary files (S.1 - S.4). As waves approach the SR, they either break on the structure (as in S.1) or surge up the structure (as in S.2), Fig. 5.16(a). The type of wave breaking, quantified by the breaker parameter, is based on the wave steepness and structure slope. The crest of the steeper wave overturns as the wave plunges on the structure before it runs up the structure. The flatter (and longer) wave has a wider crest and surges up the structure. Both breaker types show high levels of air entrainment. During wave breaking, more energy is dissipated for plunging waves, thus wave reflection increases with the breaker parameter (Fig. 5.3).

As the wave runs up the structure, a layer of water is present from the run-down of the previous wave. This residual layer of water reduces the roughness of the revetment steps for the subsequent wave running up the slope. The manner in which the wave runs up the slope is determined by the wave height in relation to the step height as schematized in Fig. 5.16(b). If the wave height is much larger than the step height, in the order of $H \geq 3 \cdot S_h$, the wave does not "feel" individual steps (see S.3). The step openings are easily filled with water and the top layer of the wave runs up seemingly unhindered. However, the steps still reduce the wave run-up compared to a smooth slope. In the case where the wave height is in the order of $H < 3 \cdot S_h$, the step openings are larger and not as easily filled during wave run-up. As a result, it is more visible how the waves "jump" from step to step (see S.4), thereby dissipating more wave energy compared to a smooth slope than the aforementioned run-up behaviour.

After the wave reaches its highest point with maximum potential energy, the flow direction reverses. The wave run-down flow regime can be classified analogous to stepped spillways (Chanson and Toombes, 2002, 2004; Schoonees et al., 2020). Initially the water runs down coherently and skims the steps, i.e. skimming flow. But as the wave descends, a nappe flow regime develops as jets of water flow down from step to step (Fig. 5.16(c)). The steps thus contribute to energy dissipation during the wave run-down. Since wave energy is dissipated at each revetment step the wave interacts with during wave run-up and run-down, the number of revetment steps the waves interact with is important. This energy dissipation owing to the steps reduces wave reflection in comparison to smooth slopes.

5.5.4 Recommendations and limitations

The tested stepped revetments in full scale proved to effectively dissipate wave energy since wave reflection coefficients (C_r) were relatively low, ranging between 0.19 and 0.48. The lowest values of C_r, i.e. highest energy dissipation, were found for wavelength ratios in the range of $0.01 \leq \cos\alpha \cdot S_h/L_{m-1,0} \leq 0.015$. An empirical formula, Eq. 5.21 was derived to estimate C_r, which is valid in the range of $0.18 \leq \cos\alpha \cdot S_h/H_{m0} \leq 0.65$. When the step height is higher than the wave height, reflections exceed the prediction of Eq. 5.21.

Additionally, the SR showed an effective reduction in wave run-up heights when

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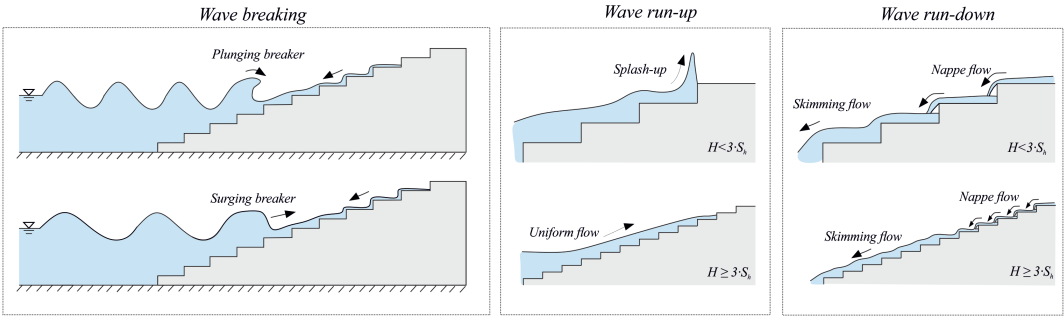

Figure 5.16: Wave interaction with stepped revetments: schematization of **(a)** wave breaking **(b)** wave run-up and **(c)** wave run down.

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compared to estimates for a smooth slope (Eqs. 5.12 and 5.13). A linear relation between wave height and run-up height was determined with which run-up heights can be estimated (Eqs. 5.23 and 5.24). Wave run-up heights are higher for SR with larger steps ($H < 3 \cdot S_h$). As waves hit against the vertical surfaces of steps (especially in cases where $H < 3 \cdot S_h$), the flow is directed upwards and high splashes occur. These splash-ups make it harder to measure exact wave run-up heights and also to translate these heights to crest levels. Therefore it is recommended to base crest-level designs on wave overtopping instead of wave run-up. However, it should be noted that wave overtopping was measured in laboratories in the absence of wind. It is expected that an onshore wind would have a significant effect on wave overtopping rates when blowing vertical splash-ups over the crest of the structure. A possible solution to counteract these vertical splash-ups by deflecting water seawards, is to have curved step faces, similar to wave return walls or recurve seawalls.

The findings and derived equations of this study are only valid for the tested hydraulic conditions and SR geometry (Tab. 5.3). Although the study has the advantage that it was tested in full scale, the number of tests and test conditions were limited. The influence of relative water depth and slope angle fall outside the scope of this study, while it was previously reported that these parameters substantially influence wave run-up on SR (Jachowski, 1964; Kerpen, 2017; Saville, 1955, 1956). Additionally, unlike Kerpen (2017), the position of still water level on the step was not varied and therefore not considered.

Comparing wave overtopping at small and full scale revealed that small-scale studies underestimated wave overtopping at SR (Schoonees et al., 2021). This was also found to be the case for wave run-up, where the estimation formulae derived in small-scale (Eqs. 5.18 and 5.19) underestimate the full-scale relative run-up heights by 31-51%. Therefore, small-scale results should be carefully interpreted when used in stepped revetment design.

To further develop design guidance for SR, the following research topics are recommended. For structural design guidance of SR research with extended ranges of geometric and hydraulic parameters is necessary, although Kerpen et al. (2018) and Steendam et al. (2018) provide insight in wave impacts on SR. Model tests in a wave basin are recommended to study the effect of oblique wave attack on wave-induced responses of SR. Additionally, the presented full-scale model tests could be replicated in different scales for an in-depth study on scale effects. The present study is based on a limited range of hydraulic and geometric parameters, and as such, studies with parameters in other parameter ranges would be beneficial. An alternative approach to extend the range of parameters is to explore wave-structure interactions with numerical models. Although the complexity of wave-structure interaction (e.g. air entrainment) poses challenges, the LIDAR sensor measurements of this study can be used for an extensive validation for a numerical model. Lastly, the role of model effects (such as salt water and wind) and the performance of SR under storm conditions could be studied with field measurements at existing SR (e.g. similar to the study of Oosterlo et al. (2021) at dikes).

5.6 CONCLUSIONS

As coastal structures, stepped revetments (SR) offer protection to infrastructure and coastal communities against wave action. The wave reflection and run-up processes provide insight in the effectiveness of SR as a coastal structure. Several studies have examined the wave run-up process on SR by means of small-scale physical model tests, often with regular waves. In contrast, relatively few studies investigated wave reflection at SR. Since it was previously reported that small-scale studies overestimate the effectiveness of SR to reduce wave overtopping in comparison to a smooth slope, the results of these small-scale studies on wave run-up and reflection may also be influenced by scale effects. In this study, we measured wave reflection and wave run-up at non-overtopped SR by means of 2D physical model tests at full scale. Two mild sloped SR ($\cot\alpha = 3$) were tested, one with large ($S_h = 0.50$ m) and the other with small steps ($S_h = 0.17$ m).

Measured wave reflection and wave run-up from SR are lower compared to that of smooth slopes reported in literature. The reduction is caused by the revetment steps that dissipate wave energy during the run-up and run-down process. The SR with small steps ($S_h = 0.17$ m) experienced 55% higher reflections than from the large steps ($S_h = 0.50$ m) under the same wave conditions. It was also established that the breaker parameter, together with the wave conditions in relation to step geometry, affect wave reflection from SR. Based on these findings, an empirical equation (Eq. 5.21) was derived to estimate wave reflection from SR ($r^2 = 0.98$ and RMSE = 0.012).

The results of this study showed that wave run-up on SR is predominantly influenced by wave height. As was found for wave reflection, the relation between wave conditions and step geometry is also important for wave run-up. Higher run-up heights were measured at the SR with smaller steps ($H_{m0} \geq 3 \cdot S_h$) compared to the larger steps ($H_{m0} < 3 \cdot S_h$). Unlike other sloped coastal structures, the relative run-up ($R_{u2\%}/H_{m0}$) on stepped revetments did not show a clear relation with the breaker parameter. A comparison between relative run-up heights measured in full scale and small scale revealed that small-scale predictions underestimate relative wave run-up heights by 31-51%. Small-scale wave run-up measurements should thus be used with caution in designs.

While the findings of this study are based on a relatively narrow range of boundary conditions for non-overtopped SR, the study contributes with full-scale measurements towards a better understanding of the hydrodynamic processes on SR and their functioning as coastal structures.

Chapter highlights

- Based on full-scale physical model tests, an empirical formula (Eq. 5.21) is proposed for estimating wave reflection coefficients of stepped revetments.
- The tested stepped revetment with small steps (S_h = 0.17 m) experienced 55% higher reflections than from the large steps (S_h = 0.50 m) under the same wave conditions. Consequently, the large steps more effectively dissipate energy during the wave run-up and run-down process compared to the small steps.
- A linear relation between wave run-up heights and wave heights was found. Higher run-up heights were measured at the SR with smaller steps ($H_{m0} \geq 3 \cdot S_h$) compared to the larger steps ($H_{m0} < 3 \cdot S_h$).
- Previous small-scale studies underestimate relative wave run-up heights by 31-51%. As a result, basing the crest-level design of stepped revetments on small-scale wave run-up measurements could thus lead to unsafe designs.

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Chapter 6

Discussion

6.1 Overview of research

A suite of coastal solutions is required to continue keeping coastal communities and infrastructure safe amidst climate change effects, growing coastal populations and concomitant urbanisation. Apart from offering coastal safety, coastal solutions need to be sustainable, i.e. consider the environmental and social challenges of the future. Hard coastal structures form physical barriers against waves and stabilise shorelines, thereby offering effective protection against coastal flooding and/or erosion. However, the benefit of effective coastal safety is accompanied with unintended environmental impacts. Since these structures form barriers, they bring changes to hydrodynamic and morphological processes, which in turn influence coastal ecosystems. Hard coastal structures also affect habitats and ecological connectivity as new hard substrata are introduced. The resulting environmental impacts may stretch across temporal and spatial scales. This dissertation explored how hard coastal structures can be implemented in a more responsible way with regard to environmental impacts, while ensuring coastal safety.

Chapter 2 critically discusses the use of hard coastal structures on the foreshore (groynes, breakwaters and jetties) and onshore (seawalls, revetments and dikes). Groynes and offshore breakwaters are built on the foreshore to control or mitigate erosion, while jetties protect navigation channels. Seawalls and dikes are shore-parallel, onshore structures protecting the coastal hinterland from flooding and wave overtopping. Revetments are sloping structures covering shoreline profiles to prevent erosion. While these hard structures have intended (positive) effects of protecting against flooding and erosion, they also bring unintended (negative) effects to the environment. Unintended effects include erosion down-drift from structures, colonisation of non-native species and forming barriers to fauna and flora. Based on literature, Chapter 2 proposes nature-based adaptations for the structure types to minimise these unintended effects and create new ecosystem services. For instance, vegetated foreshores could complement dikes or micro-habitats can be incorporated in seawall designs. Overall, Chapter 2 reviews and provides guidance on environmental aspects of hard coastal structure design to support a transition towards sustainable and environmentally friendly coastal solutions.

The move towards sustainable coastal solutions requires the inclusion of environmental aspects in the standard practices and guidelines of coastal engineering. With the example of stepped revetments (SR) in Chapter 3, this dissertation explores environmental aspects to be considered in designs. These environmental aspects for SR are identified based on the reviewed literature in Chapter 2. The discussed aspects include environmental impacts as well as nature-based adaptations to minimise these impacts or to create new ecosystem services. The vertical and horizontal step surfaces of SR provide valuable areas where roughness and surface complexity can be maximised. Additionally, their steps could be adapted to include micro-habitats, e.g. rock pools. Considering environmental aspects from earliest design phases enables coastal engineers to optimise their designs for minimising environmental impacts and creating opportunities for ecological enhancement. Additionally, awareness and general understanding of environmental aspects facilitate interdisciplinary collaboration for coastal engineers.

In addition to environmental aspects, guidelines related to hydraulic responses of SR are also lacking in design manuals. Previous studies were investigated to gain insight in the current understanding of hydraulic responses of SR and to identify knowledge gaps. To expand design recommendations related to hydraulic responses of SR, Chapter 4 investigates wave overtopping and Chapter 5 studies wave run-up and reflection at SR by means of full-scale wave flume experiments. The experimental work presented in this dissertation provides one of the first investigations into the hydraulic responses of SR at full scale. On a horizontal flume bed, two SR cross-sections were tested with slopes of 1:3 and with uniform step heights. The first cross-section had larger step heights of 0.50 m while the second had smaller steps heights of 0.17 m. Wave overtopping, run-up and reflection were measured, analysed and studied to determine which parameters influence these hydraulic responses. Based on these insights, empirical formulae were developed for estimating wave overtopping, wave run-up and wave reflection. The flume tests thereby contribute towards a better understanding and quantification of hydraulic responses of SR, which is required to ensure coastal safety and the success of nature-based adaptations.

With the gained knowledge on environmental aspects and hydraulic responses of SR, the design recommendations for SR were improved in terms of coastal safety and sustainability. For the first time empirical formulae were derived based on full-scale data. Comparing the measured full-scale data with previous small-scale data revealed that previous studies underestimate wave run-up and overtopping at SR. The findings on hydraulic responses thus improve the reliability of design recommendations in terms of coastal safety. With the review of environmental aspects to be included in SR designs, this dissertation also expand design recommendations for SR, supporting a transition towards sustainable SR designs.

6.2 Hydraulic responses of stepped revetments

Stepped revetments protect the coastal hinterland from wave overtopping and flooding. The stairs of SR obstruct the flow as waves run up and down the structure, thereby inducing turbulence and dissipating wave energy. Owing to the energy dissipation by the stairs, wave run-up heights and wave overtopping volumes at SR are lower compared to those at a smooth slope or dike. Wave run-up heights and overtopping volumes are used as criteria to design the crest height of coastal structures. The crest height not only affects coastal safety, but also has social impacts. Especially in touristic and recreational areas, a high crest height has negative impacts on the landscape (Capel, 2015). Unobstructed views to the beach and coastal areas have high cultural and emotional importance (Cox and Pearce, 2016). SR are therefore especially suitable in urban and touristic settings as their required crest level could be lower than that of a smooth slope to obtain the same level of coastal safety.

Evidence from full-scale physical model tests confirms that SR effectively reduce wave overtopping and run-up compared to a smooth slope. This reduction is quantified by γ_f, the influence factor for roughness, or roughness factor. As such, γ_f indicates how effectively a rough slope reduces wave overtopping (or wave run-up) in comparison to a smooth sloped structure under the same wave conditions. In the case of SR, the reduction is caused by the roughness of their stairs. Roughness factors of the tested SR were calculated based on wave overtopping measurements. It was found that γ_f is not constant, but varies for different hydraulic conditions and SR geometries. For the tested hydraulic conditions and geometries, it was determined that γ_f for SR ranges between 0.43 and 0.73. Compared to γ_f of other roughness elements on impermeable slopes (see Tab. 6.1), such as ribs or blocks, SR show a more effective reduction of wave overtopping. As a result, SR are effective coastal structures to reduce wave overtopping while retaining lower crest levels.

Table 6.1: Roughness factors for revetment types according to EurOtop (2018)

Revetment type	γ_f
Concrete	1.0
Asphalt	1.0
Closed concrete blocks	1.0
Small blocks over 1/25 of surface, optimum height	0.85
Small blocks over 1/9 of surface, optimum height	0.80
Ribs, optimum dimensions	0.75
Stepped revetments S_h =0.50 m (this study)	0.43-0.54
Stepped revetments S_h =0.17m (this study)	0.57-0.73

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To optimise the wave overtopping reduction of SR, the factors influencing γ_f were identified and investigated. It was identified that the key parameters influencing the wave overtopping reduction are characteristic step height, relative wave overtopping rate and wavelength. As waves run up the SR slope, the stairs obstruct their flow. With smaller characteristic step heights ($\cos\alpha \cdot S_h$), the flow is less obstructed and the step openings are easier filled with water from previous waves, thus creating a smoother ramp for consecutive waves to run up on. Higher characteristic step heights form larger obstacles to the flow and as a result, waves more distinctly "jump" from one step to the next. In contrast, for smaller characteristic step heights, more continuous run-up flows are observed. The tested SR with large steps (S_h = 0.50 m) achieved higher wave overtopping reductions ($0.43 \le \gamma_f \le 0.54$) than the small steps ($S_h$ = 0.50 m; $0.57 \le \gamma_f \le 0.73$).

Furthermore, the relative overtopping rate also influences the wave overtopping reduction of SR. When wave overtopping rates are large, for example during storms, the effectiveness of SR to reduce overtopping decreases. In such conditions, preceding waves more frequently leave a layer of water on the SR as the next wave approaches, thereby reducing the roughness between the incident wave and the steps. The third key parameter affecting the wave overtopping reduction of SR is the wavelength. Longer waves contain more wave energy and are therefore less affected by the revetment steps. The wavelength also affects the wave steepness and wave breaking. Flatter (and longer) waves have wider crests and surge up the SR, resulting in less energy dissipation and lower wave overtopping reduction. With steeper (and shorter) waves, more energy is dissipated during wave breaking and the broken wave running up the SR is more affected by the revetment steps. Based on these insights gained on the key parameters affecting wave overtopping reduction of SR, an empirical formula was derived for estimating γ_f of SR. Within the tested range of boundary conditions, the highest wave overtopping reduction can be achieved for higher characteristic step heights, lower relative overtopping rates and shorter waves.

In addition to wave overtopping rates, the volume and distribution of individual overtopping events are important parameters for estimating risks of hazards e.g. to pedestrians or structural instabilities (EurOtop, 2018). In line with literature, the individual overtopping volumes at SR follow a two-parameter Weibull distribution with scale factor a and shape factor b. Comparing the derived shape factors for SR to empirical formulae developed by Zanuttigh et al. (2013) and Gallach-Sánchez (2018) for other structure types, it shows that the tested SR have substantially higher shape factors, with a median shape factor of $b = 1.64$. This finding implies that the largest individual overtopping events at SR are more equal in volume. In contrast, Gallach-Sánchez (2018) found slightly lower shape factors for steep low-crested SR compared to smooth slopes. With low-crested SR, more stairs are submerged, thereby making the stairs less effective in reducing wave overtopping, which could in turn affect individual overtopping volumes. It was found that the probability of overtopping at SR is best described by an estimation developed by Franco et al. (1994) for vertical walls. Moreover, the maximum individual overtopping volumes were assessed, revealing a linear relation between the maximum volume and the average overtopping rate (V_{max}/q= 4 111 s^{-1}). Note that this relation varies with structure geometry, wave conditions and storm duration. Besides Gallach-Sánchez (2018), who studied steep low-crested SR, no other studies focused on individual wave overtopping at SR.

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The energy dissipation by the SR steps during wave run-up and run-down also reduces wave reflection. The wave reflection coefficients at the tested SR are thus substantially lower than estimations for reflection coefficients from smooth slopes in literature. With lower reflections from SR, it is expected that scour may be less severe and sea-states will be less influenced. The measured reflection coefficients for the tested SR vary between 0.19 and 0.48 for $0.9 \leq H_{m0}/S_h \leq 5.2$. Further analysis showed that reflection coefficients are influenced by wave breaking (quantified by the breaker parameter) as well as the wave conditions in relation the SR geometry. Wave conditions were repeated for the large (S_h = 0.50 m) and small steps (S_h = 0.17 m) showing that reflection coefficients were 55 % higher at the small steps. In line with what was found for wave overtopping, the large steps more effectively dissipate energy within the tested range of boundary conditions. However, wave energy is dissipated at all revetment steps that interact with waves during wave run-up and run-down. Therefore, when the step height is larger than the wave height ($\cos\alpha \cdot S_h$>1), the incoming waves only interact with 1 or 2 revetment steps, resulting in lower energy dissipation and higher wave reflection. Consequently, the number of steps interacting with waves is a significant factor for the hydraulic responses of SR. Lastly, it was also found that the reflection coefficients decrease with wavelength. As the wavelength becomes larger in relation to the characteristic step height, higher reflections are experienced. Based on these findings, an empirical formula for estimating the wave reflection from SR was derived.

The processes of wave reflection and wave overtopping are strongly related (Zanuttigh and Van der Meer, 2008). Wave overtopping reduction (quantified with γ_f) occurs due to the energy dissipation by the revetment steps, while wave reflection represents the wave energy that was not dissipated due to wave breaking, transmission or interaction with the revetment steps during wave run-up or run-down. Both wave reflection and wave overtopping show similar influences for dimensionless step height ($\cos\alpha \cdot S_h / H_{m0}$) and wavelength ratio ($\cos\alpha \cdot S_h / L_{m-1,0}$), see Figures 6.1 and 6.2. Zanuttigh and Van der Meer (2008) developed an equation for estimating the reflection coefficients of coastal structures based on their roughness factors γ_f. The roughness factor for the tested SR varied from 0.43 to 0.73. By fitting the equation of Zanuttigh and Van der Meer (2008) to the measured reflection coefficients, roughness factors of γ_f = 0.39 and γ_f = 0.74 were calculated (Fig. 5.3). The roughness factors for the tested SR and those determined by the approach of Zanuttigh and Van der Meer (2008) are thus within the same range. Note that the approach of Zanuttigh and Van der Meer (2008) was not developed for SR nor for deriving roughness factors of structures. Nevertheless, these findings illustrate that the wave reflection and wave overtopping are strongly related and the findings regarding these processes are in agreement.

Wave run-up measurements provide further insights in the wave-structure interaction of SR. Unlike other structure types, the relative run-up ($R_{u2\%}/H_{m0}$) on the tested SR does not show a clear relation with the breaker parameter (within the tested range of boundary conditions). Instead, the wave run-up height increases with both incident wave height and wave period. However, the relation between run-up height and wave height is substantially stronger and can be described with a linear relation (r^2=0.97). This relation can be applied to estimate wave run-up heights on SR. Additionally, run-up is affected by the dimensionless step height ($\cos\alpha \cdot S_h / H_{m0}$). When waves are much larger than the step height ($H \geq 3{\cdot}S_h$), wave run-up takes places with a continuous flow where the wave does not "feel" individual steps and the step openings are easily filled with water. In contrast when $H < 3{\cdot}S_h$, the waves follow the shape of the steps as they "jump" from step to step, thereby dissipating more energy. With this run-up flow behaviour, as waves hit the vertical step surfaces, the flow is directed upwards, resulting in high vertical splash-ups. These splash-ups make it hard to measure wave run-up heights accurately enough to develop design recommendations for crest level design. Consequently, it is recommended to base crest-level designs of SR on wave overtopping instead. As was found for wave overtopping and reflection, SR substantially reduce wave run-up compared to smooth slopes with γ_f ranging between 0.59 and 0.83. Larger variability in γ_f was found for run-up than for overtopping, potentially due to the difficulties in measuring wave run-up with frequent vertical splash-ups. For SR accessible to public, wave run-up estimates and distributions are important for ensuring public safety. In addition, for nature-based adaptations and maintenance purposes (e.g. algae growth), wave run-up provide insight on how often parts of the SR are submerged.

Comparing full-scale wave run-up and wave overtopping measurements to those of previous studies measured in small-scale, reveal that hydraulic responses measured in small-scale are likely affected by scale. The empirical formula developed by Van Steeg et al. (2018) based on wave overtopping measurements at a scale of 1:10 underestimates the roughness factors (γ_f) of the tested SR by 2 to 19 % for the small steps and 15 to 31 % for the large steps. Also the empirical formulae developed by Kerpen (2017) for estimating wave run-up on SR, underestimate relative run-up heights by 31-51 %. Since the underlying hydraulic conditions and SR geometry are similar, but not identical, it cannot be said with certainty that differences between full-scale and

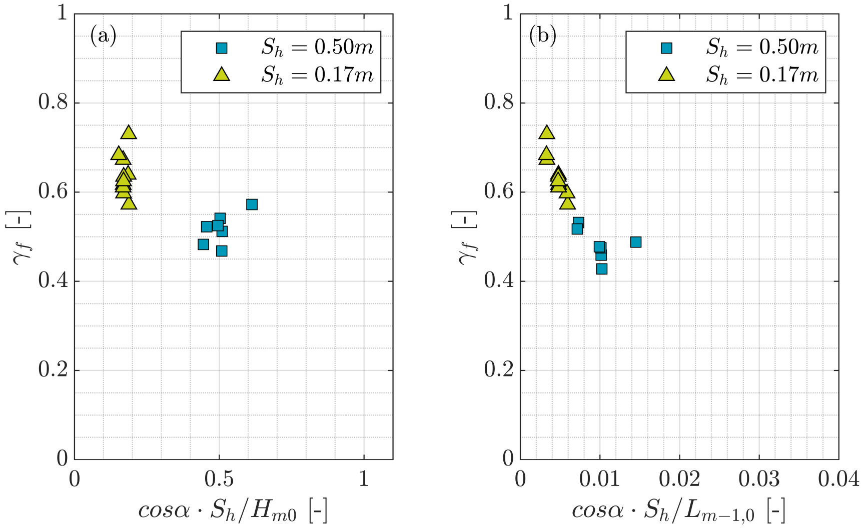

Figure 6.1: Influence of dimensionless step height and wavelength ratio on the roughness factor

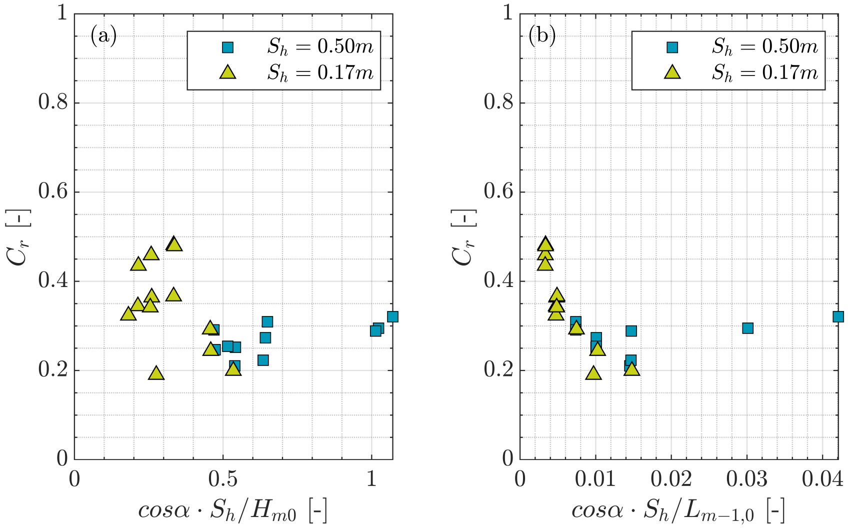

Figure 6.2: Influence of dimensionless step height and wavelength ratio on the reflection coefficient

small-scale measurements are (only) due to scale effects. Different calculation methods also influence the differences between full-scale and small-scale measurements. For instance, Van Steeg et al. (2018) derived roughness factors based on the overtopping formulae in TAW (2002) (Eq. 4.18-4.19), while roughness factors for the full-scale measurements were calculated with the formulae in EurOtop (2018) (Eq. 4.1 - 4.2). In Eq. 4.1 - 4.2 the roughness factor is applied to the power of 1.3, whereas in Eq. 4.18-4.19 the roughness factor is applied to the power of 1. However, the roughness factors are still very similar, see Fig. 4.15 for a comparison between roughness factors calculated with TAW (2002) (Eq. 4.19) and EurOtop (2018) (Eq .4.2). Nevertheless, the small-scale empirical formulae for estimating wave run-up and wave overtopping substantially underestimate the full-scale measurements. Basing SR designs on small-scale physical model tests could thus lead to unsafe designs and therefore caution should be applied in the interpretation of small-scale measurements.

6.3 Designing sustainable coastal structures: an example of stepped revetments

Sustainable coastal structures consider future social and environmental needs (including climate change effects). In addition to fulfilling the primary function of coastal safety, coastal structures can be multi-functional by offering other secondary benefits (Evans et al., 2017). At project inception it is necessary to formulate well-defined project goals (Firth et al., 2014). Secondary benefits or functions need to be identified and ranked by priority in consultation with stakeholders. Table 6.2 summarises examples of benefits and design options for sustainable SR. The table also lists typical design considerations and aspects that need to be included to ensure functionality.

For SR the primary function is coastal safety, i.e. to protect the coastal hinterland against flooding and wave overtopping. SR also fix the position of the shoreline and could therefore be applied to prevent shoreline retreat beyond the structure. For SR to fulfil their primary function, sufficient available space (vertical and horizontal) is a requirement. The structure's footprint and geometry in turn depends on the specified level of coastal safety, e.g. quantified by allowable overtopping limit. To ensure wave overtopping is below the allowable limit, a sufficient crest level and freeboard are required when exposed to the design wave conditions and water-level. In this dissertation wave heights (H_{m0}) up to 1 m and wave periods ($T_{m-1,0}$) up to 6.5 s were tested. At a scale of 1:10, Van Steeg et al. (2018) tested wave heights up to 1.6 m and wave periods up to 5.2 s. Therefore, currently the functionality of SR in literature is quantified for locations exposed to medium wave conditions.

The design of SR also need to consider morphological conditions and maintenance. Scour at the toe of the SR, shoreline retreat or erosion down-drift of the SR could lead to structure failure in the future. Also aeolian sand on, or in front of the SR, may affect its functioning. Seabed changes and ecosystem deterioration during the SR lifetime may result in more extreme wave conditions at the structure over time. In terms of structural design, SR need to account for wave impacts during storms (Kerpen et al., 2018; Steendam et al., 2018). Further aspects, such as construction, available building material, structure monitoring and maintenance need to be considered.

To reliably reduce coastal flood risk throughout the SR's design life, climate change effects on hydrodynamics need to be included. However, large uncertainties persist in climate change projections which, in turn, bring large uncertainties in calculating hydrodynamics for future scenarios. Overestimating future storm conditions and designs based on worst-case scenarios lead to costly over-designs of coastal solutions (Van Gent, 2019). Instead of basing designs on long design lives, SR could be designed and built in such a way, that they can be adapted (e.g. increasing crest level or combining other coastal solutions) for future scenarios when hydrodynamics and sea-level rise can be better estimated (David, 2021).

Enhancing the ecological value of SR can be done by minimising the environmental impacts and creating new opportunities for the environment with nature-based adaptations (Tab. 6.2). Both of these options require a sound understanding of the local biotic and abiotic factors. Mimicking local environmental conditions as closely as possible increases the ecological value. As SR are hard structures, they are generally more intrusive in sedimentary environments than along rocky shores.

In case a SR is required along sedimentary coasts, it is recommended to position the SR as high as possible in the beach profile (Dugan et al., 2011), to reduce their physical environmental impacts. Building SR from partly permeable blocks could be an option to reduce habitat loss. Erosion can be compensated with sediment nourishment or sand bypassing, although these interventions bring other environmental impacts. Employing numerical models to estimate sediment transport aid design optimisation regarding morphological impacts.

Along rocky coasts where rock pools are common, the ecological value of SR can be increased by including micro-habitats in their design that retain water (Firth et al., 2014). The horizontal surfaces of SR can be adapted for this purpose. To improve habitat variety, it is recommended that SR along rocky coasts are placed as low as possible in the intertidal (Firth et al., 2014). Maximising roughness and surface complexity, including crevices, holes and casting concrete with rough finishes, improves revetment habitats to more closely resemble natural ones. Alternatively, local rocks can be incorporated in the design.

Milder SR slopes are beneficial as they provide more intertidal space and habitats. Additionally, coastal ecosystems could be applied in the foreshore, not only for ecological enhancement, but also for reducing wave heights and offering erosion protection. However, sufficient space between built environments is a prerequisite for coastal ecosystems to flourish. The designs of all mentioned adaptation options need to carefully consider local hydrodynamics and morphological conditions. The hydraulic design aspects investigated in this dissertation provide insight for a design basis.

A sustainable SR should not only provide coastal safety and consider environmental aspects, but also requires social and cultural acceptance. Adaptations to SR can also enhance their social and cultural value. The stairs of SR provide safe access to the beach or water areas. Alternatively, SR designs can be adapted to provide seating or include walkways. Including micro-habitats and coastal ecosystems not only create opportunities for the environment, but also for recreation and education purposes. For increased cultural value, art installations can be incorporated in SR designs. An example of this is the sea organ in Zadar, Croatia (Fig. 1.1(b)). When SR are accessible, public safety needs to be ensured, e.g. by restricting access during storm conditions. Organism growth on the revetment steps need to be considered and monitored as it may make step surfaces slippery, creating a slip hazard. Moreover, aesthetics play an important role in attracting residents and tourists. SR designs should thus pay attention to aesthetics and fit in with the surrounding landscapes. As far as possible, the crest level should not obstruct views to the beach or sea. The hydrodynamics and morphological conditions also largely influence the success and safety of social benefits. As a result, coastal engineers have an important role to play in creating sustainable coastal solutions. Ultimately for successful sustainable coastal solutions, close collaboration between various disciplines (e.g. architects, social scientists, ecologists and engineers) is needed.

6.4 LIMITATIONS AND FUTURE AVENUES FOR RESEARCH

The full-scale flume tests described in this dissertation enabled the development of empirical formulae unaffected by scale effects. As these tests are expensive, labour-intensive and time-consuming, a limited range of boundary conditions could be tested. The developed empirical formulae for wave overtopping, reflection and run-up are only valid for the boundary conditions tested in this study. Apart from budgetary and time constraints, the wave maker's capacity also influenced the range of tested boundary conditions. Spectral significant wave heights (H_{m0}) of up to 1 m and spectral wave periods ($T_{m-1,0}$) up to 6.5 s were generated. Physical model tests are recommended for designing SR in locations exposed to higher and/or longer waves. The effect of foreshore slopes also need to be regarded, since this research tested a horizontal foreshore. The water depths tested in this dissertation (h = 4 - 5 m) are classified as transitional water ($0.5 > h/L > 0.04$). Wave overtopping and run-up measurements in previous small-scale studies were lower than in the full-scale measurements presented here. This indicates that small-scale measurements are likely affected by scale. However, boundary conditions and calculation methods were not identical in small scale which could also cause differences in measurements between scales. It is thus recommended to replicate the presented full-scale tests in small scale to quantify the role of scale effects more precisely. Since only one revetment slope ($\cot\alpha$=3) was tested in full scale, more research is required to quantify the influence of slope angle on scale effects. For rubble mound breakwaters it was found that gentler slopes lead to higher scale effects (EurOtop, 2018), which could also be the case for SR. As full-scale tests are expensive, the quantification of scale effects is important to develop reliable designs of SR based on small-scale model tests.

Although the full-scale model tests could eliminate scale effects, it is important to note that model effects may still substantially affect the measurements. Model effects can never be eliminated completely, as the reality cannot be fully replicated in a laboratory. The model tests were conducted with fresh water and without wind. Due to the differences in densities between fresh and salt water, aeration properties are different in fresh water, which could affect the hydraulic responses. However, it is expected that this effect on wave overtopping, reflection and run-up is small. The absence of wind in the laboratory is an important source of model effects (EurOtop, 2018). For instance, onshore winds may increase wave overtopping at SR when blowing up-rushing water over the structure crest. Although the wave maker is equipped with active wave absorption, some wave energy may still be re-reflected, possibly leading to resonance in the wave flume. Resonance may in turn influence wave spectra and the water level at the SR, thus affecting the measured hydraulic responses of the tested

Table 6.2: Benefits and design considerations for sustainable stepped revetments

Function	Design options	Design considerations and aspects
Coastal safety	Protection against flooding and overtopping Shoreline stabilisation	Sufficient available space Wave conditions and water level (incl. climate change effects) Wave forces for structural design Scour, sediment and aeolian transport Foreshore conditions and changes Construction, monitoring and maintenance
Enhanced ecological value	Minimising environmental impacts Nature-based solutions in the foreshore Combine with sediment nourishment Incorporating micro-habitats Maximising roughness and surface complexity Using local building material	Mimic local ecological conditions Morphological changes and resulting habitat loss Tidal range, SR geometry and position in intertidal Hydrodynamics (incl. climate change effects) Sufficient available space Construction, monitoring and maintenance
Enhanced social and cultural value	Provide access to beach/water areas Provide seating or walkways Art installations Including ecosystems and micro-habitats	Ensure public safety at accessible SR Restrict access during storms Crest level affecting line of sight Aesthetics and fit in with landscape Hydrodynamics and morphological conditions

SR. Wave gauge measurements were checked for infragravity waves and the effect is expected to be negligible. Since experiments were conducted in a wave flume, the effect of oblique wave attack on the hydraulic responses of SR was not considered. Field measurements at existing SR are recommended to determine the role of model effects and study the performance of SR under storm conditions. In reality many shorelines are already armoured with coastal structures. To keep protecting coastal communities and infrastructure in the face of climate change, a suite of coastal solutions is required. Therefore it will become increasingly important to investigate the combination of different coastal solution types (Van Gent, 2019; Chen et al., 2020). Further research is thus recommended to study the use of SR in combination with other coastal solution types.

The environmental aspects discussed in this dissertation are solely based on general guidelines, recommendations and examples from literature. Environmental impacts and the success of nature-based adaptations are highly dependent on local conditions. Therefore environmental aspects require site-specific assessments, in terms of hydrodynamics, morphology, social and environmental aspects. Replicating projects and examples in literature for different settings should be approached with great caution. Environmental and social aspects of designs always need to be developed and reviewed by experts in relevant fields, e.g. ecologists, biologists and social scientists. Greener designs of hard coastal structures may mitigate or minimise environmental impacts, but the ecological value of designs could easily be overestimated and thus need to be assessed critically. Including nature-based adaptations in coastal engineering designs may create the misconception that adverse effects may easily be mitigated (Airoldi et al., 2021) or may create the opportunity for green-washing to obtain approvals for developments (Firth et al., 2020). Simultaneously, there are still many unknowns to nature-based adaptations of hard coastal structures (Firth et al., 2020), for instance experimental evidence is often very limited in temporal and spatial scales (Airoldi et al., 2021). Future research should focus on applications on larger scales and monitor the functionality of nature-based adaptations for longer time frames to account for temporal variations in environmental conditions (Bouma et al., 2014; Airoldi et al., 2021).

Chapter highlights

- Considering environmental aspects from earliest design phases enables coastal engineers to optimise their designs for minimising environmental impacts and creating opportunities for ecological enhancement. Additionally, awareness and general understanding of environmental aspects facilitate interdisciplinary collaboration for coastal engineers.
- With the gained knowledge on environmental aspects and hydraulic responses of stepped revetments, the design recommendations for stepped revetments were improved in terms of coastal safety and sustainability.
- For the first time empirical formulae based on full-scale data were derived for estimating wave run-up, reflection and overtopping of stepped revetments. Comparing the measured full-scale data with previous small-scale data revealed that previous studies underestimate wave run-up and overtopping at stepped revetments. The findings on hydraulic responses thus improve the reliability of design recommendations in terms of coastal safety.
- With the review of environmental design aspects, this dissertation expand design recommendations for stepped revetments, supporting a transition towards sustainable designs.

Chapter 7

Conclusions

Coastal communities, infrastructure and ecosystems are increasingly at risk from flooding due to the effects of climate change (Taherkhani et al., 2020; Vitousek et al., 2017). At the same time the world's growing coastal population drives urbanisation and infrastructure development in coastal areas (Neumann et al., 2015). Coastal ecosystems provide numerous benefits to human welfare, known as ecosystem services (Barbier et al., 2011). These services include wave attenuation, erosion control, carbon sequestration, food production and supporting recreational activities. To safeguard coastal communities, infrastructure and ecosystems, sustainable coastal solutions are required that balance the economic, social and environmental needs of current and future generations.

A suite of coastal solutions is required in response to the effects of climate change. Soft and nature-based solutions for coastal protection are dynamic as they change over time, for instance due to hydrodynamics or seasonal influences. In comparison, hard structures are rigid with generally limited changes in their functionality during their design life (provided that required maintenance is ensured). This rigidness of hard structures arguably makes it easier to estimate their coastal flood risk reduction, opposed to that of soft and nature-based solutions affected by temporal changes. Maintaining hard structures comes at high costs, whereas well-designed nature-based solutions could be self-maintaining (Temmerman et al., 2013). Generally soft and nature-based solutions require more available space than hard coastal structures to achieve the same level of coastal safety. Further local conditions, such as hydrodynamics, ecologic and morphological conditions, affect the feasibility of coastal solution types.

On the one hand, the use of hard coastal structures is widely criticised for their environmental impacts, for instance, inducing changes to habitats, ecosystem degradation and erosion. On the other hand, they fulfill important functions: by keeping high-value infrastructure safe, facilitating safe navigation and in some cases, supporting recreational activities. In some instances these functions can be fulfilled in a more environmentally friendly way: by soft or nature-based solutions or alternatively, a combination of nature-based solutions with hard coastal structures, i.e. hybrid coastal solutions (Sutton-Grier et al., 2015). Due to their environmental impacts, the use of hard coastal structures should in general terms be avoided when possible. However, in some cases hard coastal structures are the most suitable and effective solution. For instance, in cases where little space is available (e.g. coastal urban areas), where high reliability needs to be ensured (e.g. high-risk buildings like hospitals) or locations exposed to high wave conditions. In these cases, where the use of hard coastal structures are most suitable, environmental aspects can be adopted in designs to minimise their environmental impacts or create new ecosystem services.

The successful inclusion of environmental aspects in designs commands a good understanding of local environmental conditions, both biotic and abiotic. As such, close collaborations between experts from various disciplines are required. With well-designed coastal structures the effects on hydrodynamics, scour, erosion and their associated ecological changes are minimised. Literature shows examples of successful nature-based adaptations to hard structures, where design adaptations mimic local habitats as closely as possible. Considerations for adaptation are the position, geometry and building material of the hard structure as well as the artificial creation of a variety of habitats for native marine organisms. Although the inclusion of environmental aspects in designs increases the ecological value of hard coastal structures, these will not fully compensate their environmental impacts.

Coastal engineers have an important role to play in the development of sustainable hard coastal structures that balance environmental, economic and social aspects, while retaining their primary function of ensuring coastal safety. Apart from influencing coastal safety, the hydraulic responses of coastal structures also affect the success of secondary benefits such as enhancing their ecological and social value. This dissertation investigated stepped revetments as an example of a sustainable and multi-functional hard coastal structure type.

Stepped revetments reduce wave overtopping effectively in comparison to smooth dikes, as their steps dissipate energy as waves interact with the structure. With full-scale wave flume tests this reduction in overtopping was quantified under medium wave conditions ($0.8 \leq H_{m0} \leq 1.0$; $4.6 \leq T_{m-1,0} \leq 6.5$; $1.4\ \% \leq s_{m-1,0} \leq 3.5\%$) for a mild sloped stepped revetment ($\cot\alpha = 3$). This overtopping reduction was greater for SR with large steps of $S_h = 0.50$ m with γ_f ranging between 0.43 and 0.54 ($0.45 \leq \cos\alpha \cdot S_h / H_{m0} \leq 0.61$). Nevertheless, for stepped revetments with small steps of $S_h = 0.17$ m, γ_f ranges between 0.57 and 0.73, which still show a substantial reduction in wave overtopping ($0.15 \leq \cos\alpha \cdot S_h / H_{m0} \leq 0.19$). The large steps form higher obstacles to waves running up the structure, resulting in higher energy dissipation. This was also found for wave reflection, where the same wave conditions resulted in 55 % higher reflection coefficients for the small steps. Stepped revetments also reduce both wave reflection and wave run-up in relation to smooth slopes. Compared to small-scale wave run-up and overtopping measurements, this dissertation reveals that hydraulic responses measured in small scale are likely affected by scale effects. Small-scale studies overestimate the wave overtopping reduction (γ_f) by 2–31 % and underestimate relative wave run-up heights ($R_{u2\%}/H_{m0}$) 31-51 %. As a result, basing the design of stepped revetments on small-scale measurements could lead to unsafe designs. Empirical formulae were developed for quantifying wave overtopping, wave reflection and wave run-up at SR. The quantification of these responses also influences the designs of adaptations with ecological and social benefits.

In addition to their primary function of ensuring coastal safety, stepped revetments offer opportunities for ecological enhancement (e.g. incorporating micro-habitats) and social benefits (e.g. offer seating or provide access to the beach). The multi-functionality of stepped revetments make these structures especially suitable in urban and touristic settings. This dissertation improves design recommendations for stepped revetments by identifying environmental design aspects and improving design formulae for their hydraulic responses (wave reflection, wave run-up and wave overtopping).

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Author Statement

As the author of this dissertation I declare that:

1. I read the regulations for doctoral candidates at the Faculty of Civil Engineering and Geodetic Science, Leibniz University Hannover
2. this dissertation has been composed by me and is based on my own work unless specified by references or acknowledgements
3. that I have no competing financial interests or gains that could have appeared to influence the work reported in this dissertation
4. that the dissertation has not been submitted at another academic institution before
5. I have not previously applied for an exam as a doctoral candidate at another academic institution

This dissertation takes the form of a cumulative dissertation consisting of three peer reviewed first-authored articles in international scientific journals. The articles and my contributions to these articles are outlined below:

- **Hard structures for coastal protection, towards greener designs**

  Estuaries and coasts 42, 1709-1729, 2019 doi.org/10.1007/s12237-019-00551-z

  T. Schoonees, A. Gijón Mancheño, B. Scheres, T. J. Bouma, R. Silva, T. Schlurmann, H. Schüttrumpf

  I conceptualised and prepared the layout of the paper together with the co-authors. As first author, I wrote the first draft of the introduction, discussion and conclusions of the paper. I performed the literature review on the functions, environmental impacts and nature-based adaptations of seawalls and revetments. The literature review on foreshore structures and dikes was performed by A. Gijón Mancheño and B. Scheres.

- **Full-scale experimental study on wave overtopping at stepped revetments**

  Coastal Engineering 167, 103887, 2021 doi.org/10.1016/j.coastaleng.2021.103887

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  I drafted the test programme, conducted the physical model tests and analysed the test results. I conceptualised, wrote and revised the manuscript as well as prepared the figures and graphs.

- **Full-scale experimental study in wave reflection and run-up at stepped revetments**

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