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STEPPED REVETMENTS – REVISITED

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ABSTRACT

This paper summarizes and critically reflects present knowledge on stepped revetments implemented in coastal protection structures for more than 60 years. Published and unpublished results of experimental investigations on wave run-up, wave overtopping and wave loads on stepped revetments are being re-analyzed and persistent knowledge gaps addressed. Due to an increasing surface roughness stepped revetments reduce the wave run-up height. Magnitudes of reduction are primarily driven by the slope angle, the presence of a shore face and the steepness of the incoming waves. Less influential is the geometry/shape of the single step, e.g. with inclined step faces or round edges. Energy dissipation rates are inclined to be dependent on the relation of step height to wave height but the specific influence is unknown up to date in accordance with a lack of knowledge in the fully physical understanding about drivers and inherent processes on wave run-up, reflection and dissipation. Present experimental investigations analysed from literature undergo scale effects that have a significant influence on the results due to dominant air intrusion in the run-up process on stepped revetments.

KEWORDS: stepped revetment, wave run-up, overtopping, surface roughness, friction.

1 INTRODUCTION

In the course of time, new demands regarding environmental and touristic compatibility of coastal protection structures play a more pronounced role to be tackled by any coastal authority worldwide. This trend can be inevitably witnessed in the design processes of present coastal protection systems. Stepped revetments combine needs in current coastal protection by increasing the surface roughness of the coastal protection structure to reduce wave run-up and wave overtopping, enable a quick, accurate and aesthetical fabrication with pre-fabricated components coincidentally allowing an easy access to the water for residents and tourist in storm seasons. Up to date a significant number of structures with stepped face or stepped revetment are constructed and in operation in typical costal protection systems as of today (e.g. Cleveleys beach front, Burnham-On-Sea defense, New Sea Wall Blackpool and Margate, UK; Bonny Dam, Colorado, US; Sea Organ, Zadar, Croatia; Polder Neumuehlen Westkai, Baumwall and Marco Polo Terraces, Hamburg, Germany). In order to assure a distinct definition of relevant geometries as well as step shapes related and hydraulic parameters involved in the interaction of waves and stepped revetments, the most important ones are depicted in Figure 1.

- B revetment width
- d depth at revetment toe
- h_s deep water depth
- i shoreface slope
- n revetment slope
- n_s slope of a single step
- R_c freeboard height
- R_u run-up height
- S_h step height
- S_w step width
- p pressure load

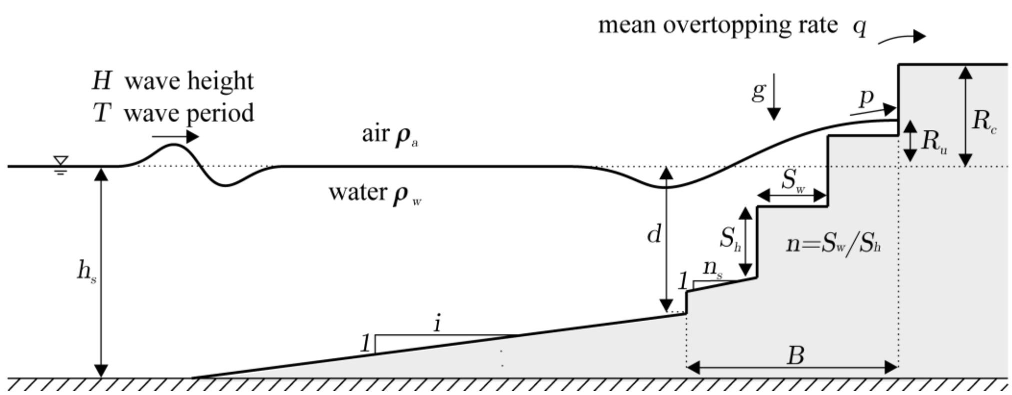

Figure 1. Definition of geometry related and hydraulic parameters.

2 LITERATURE REVIEW

A comprehensive overview of literature providing data and discussions with respect to wave run-up, wave overtopping, scouring and wave loads on stepped revetments is visualized in Figure 2 including information about references and knowledge improvements. Pivotal interest is the determination of the step-geometry-related surface roughness leading to an increased friction and dissipation rates on the slope. These processes finally lead to a reduced wave run-up in comparison to a smooth slope and can be taken into account within design formulae by an empirical derived friction coefficient γ_f.

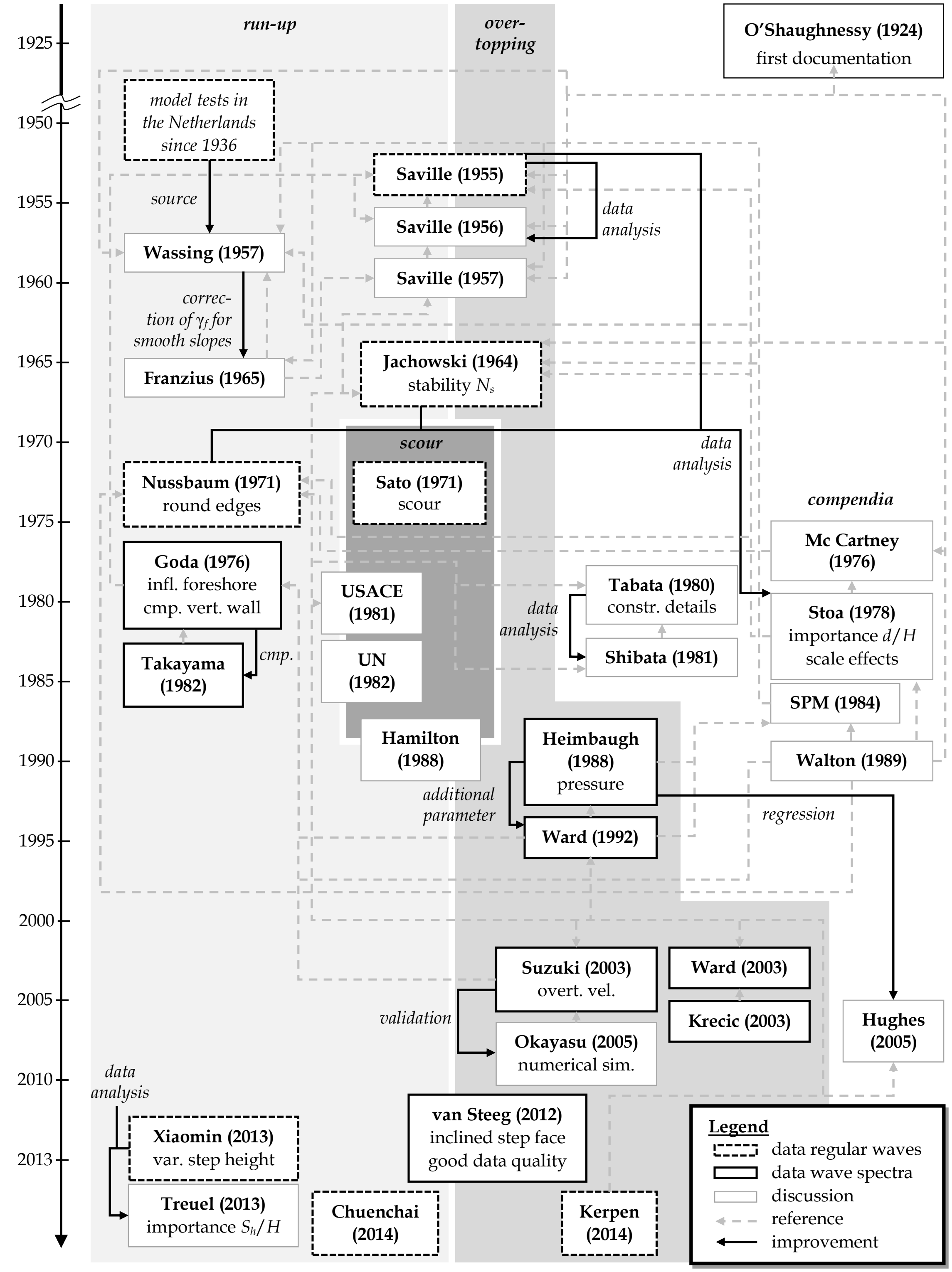

Figure 2. Flow chart of literature providing data and discussions with respect to wave run-up, wave overtopping, scouring and wave loads on stepped revetments

First historic documentation of a stepped revetment has been reported by **O'Shaughnessy *et al*. (1924)** in order to discuss the design and construction of the ocean beach esplanade sea-wall along the great highway on San Francisco's ocean beach. The seawall was intentionally installed to mitigate beach erosion. It consist of a stepped section founded on concrete pilings topped by a recurved reinforced concrete section. After construction of the seawall the beach in front of it remained relatively stable over its lifetime and may led practitioners to look more thoroughly into stepped revetments intended for beach front conservational purposes.

First scientific investigations on stepped revetments have been carried out in hydraulic model tests (Froude scale 1:17) by **Saville (1955)** and **Saville (1956)** analyzing the wave run-up on composite slopes. Within revetments of smooth surfaces, riprap and vertical walls although a slope (n = 1.5) with stepped revetment – here called 'Step - Faced Wall' – was analyzed regarding wave run-up (25 tests) and wave overtopping (88 tests). Saville (1955) concluded that the wave run-up R_u and wave overtopping volume q at stepped revetments increases with increasing wave height, increasing wave period or increasing Iribarren number and decreasing wave steepness. The reduction of wave run-up and overtopping over a stepped revetment in comparison to a smooth surface was derived to $0.65 < \gamma_f < 0.85$ for $\xi < 3.3$ and $0.35 < \gamma_f < 0.7$ for $\xi < 3.3$, with γ_f depicting the reduction factor. An own re-analysis of data shows that this reduction factor increases disproportional to an increasing overtopping volume q. The range of reduction factors derived in the tests spreads significant for wave run-up with $0.33 < \gamma_f < 0.85$ and for wave overtopping with $0.06 < \gamma_f < 0.99$. Data shows higher relative run-up heights at stepped revetments in comparison to e.g. riprap covers and smaller values as in tests for vertical walls. **Saville (1957)** intensifies the analysis of these data sets with focus on composite structures.

Wassing (1957) summarized results of physical model tests on wave run-up carried out on the influence of 'steps' on a 1:3.5 inclined slope. Small steps (S_h = 0.14m) reveal a reduction factor of γ_f = 0.78 and large steps (S_h = 0.35m) a reduction factor of γ_f = 0.77. Hydraulic parameters are not given in the paper, thus the step height 'small' and 'large' cannot be seen in proportion to other parameters like e.g. the wave height. According to Wassing (1957) the wave run-up can be calculated by

$$\frac{R_u}{H} = A \cdot \gamma_f \cdot sin2\alpha \tag{1}$$

with A as the probability of exceedance of the run-up value and γ_f is a factor for the characteristic of the dike facing and represents in this case a reduction factor of the run-up in comparison to a revetment of neatly-set stones (large steps: γ_f = 0.89, small steps: γ_f = 0.90). Hence, the factor will be corrected directly for a smooth protective layer (large steps: γ_f = 0.77, small steps: γ_f = 0.78) as mentioned in Franzius (1965), too.

Jachowski (1964) tested a stepped revetment build up by interlocking concrete blocks in order to benefit from wave energy absorption and reduced wave run-up known from rubble mound revetments. Therefore, Jachowski (1964) performed stability (Froude scale 1:10), run-up and overtopping (Froude scale 1:16) tests in a physical model with focus on the interlocking of a precast concrete block seawall with a stepped slope (n = 2 and n = 3). The water depth was kept constant to h = 0.38, whereas the water depth in front of the stepped revetment was varied between $0.0 < d < 0.38$m. A step height of S_h = 0.19m was tested. He concluded that a stepped-face seawall is most effective in reducing the wave run-up. If a benefit exists at all, inclined-face steps reduce the wave run-up slightly in comparison to vertical-faced steps. The run-up height is influenced by the Iribarren number ('run-up and overtopping were greater for surging waves, as differentiated from plunging waves, at the time of breaking') and reduced with increasing relative water depth at the toe of the structure (d/H_{m0}). Reduction factors γ_f are not given in the paper. Tests on the stability of the vertical-face stepped revetment revealed a stability number $N_s > 12.8$ (derived by Hudson (1959)). This large value is addressed to a great extent of the mechanical interlock of the ship-lap joints. For large freeboard heights R_c the revetment failed at or slightly above the SWL due to single blocks that are forced up the slope by incident breaking waves followed by a gradual dislodging of the blocks in the area of SWL. At set-ups with low freeboard height wave overtopping occurs and the stepped revetment fails due to lift-up and displacement of the top row (an adequate anchorage of the top row is needed but feasible). The prime importance of a properly designed underlying filter was mentioned in order to avoid sand losses through joints between the blocks.

Nussbaum and Colley (1971) conducted physical model tests regarding the wave run-up on a smooth and stepped revetment constructed by soil cement. They concluded that the wave run-up will be reduced by a stepped configuration and an increasing slope steepness $1/n$ results in higher run-up heights. Larger steps are not as efficient in reducing the wave run-up as smaller steps. A 1 on 2 slope with rounded steps results in higher relative run-up heights than a smooth slope.

Sato *et al*. (1971) investigated on scour prevention works at the toe foot of sea walls. In physical model tests (Froude scaling 1:10) in a 31.65 m long and 0.17 m width wave flume they compared the scour development at the toe of a 1:3 slope with smooth and stepped surface among others. As sediment a fine sand (D_{50} = 0.2 mm) was used to set up a 1:10 inclined fore slope. Result is a very similar scouring process in front of a plain and a stepped slope. The scour depth in front of the toe increases with increasing number of attacking waves. For both revetment geometries the maximum scour depth is in a distance to the toe of $L/3$ and the maximum distance of influence on the slope of one wave length L.

Goda and Kishira (1976) published results of experiments on irregular wave overtopping characteristics of five seawalls of low crest type (n = 2). The physical model tests (9 tests) were conducted in a wave flume at the Port and Harbour Research

Institute in Japan with Froude scaling (1:33). They concluded a requirement of lower crest heights for ’stepped slopes’ in comparison to smooth slopes. Furthermore, the stepped slope requires a 10-20% higher freeboard height in comparison with a vertical wall. Goda and Kishira (1976) analyzed the influence of an inclined foreshore in front of a stepped revetment and found that overtopping volumes are about 30 times smaller for a 1:30 foreshore in comparison to a 1:10 foreshore whereas a overtopping volume reduction for a plain slope adjacent to a inclined foreshore is only 20 times smaller.

McCartney (1976) published a survey of 25 coastal revetment types. Within this report a ’gabion-stacked’ revetment and a ’soil cement’ revetment are presented. These two represent geometries similar to a stepped revetment. For ’gabion-stacked’ revetments McCartney (1976) estimated a reduced wave run-up in comparison to a smooth slope of $0.5 < \gamma_f < 0.6$ and a low reflection coefficient. The author remarks that gabions should be stacked to form a stair step face and the suggestion of a gabion mat as toe protection. For ’soil cement’ revetments the author cites small-scale tests by Nussbaum and Colley (1971) and gives a reduction factor of $0.7 < \gamma_f < 0.8$ with estimated moderate wave reflection coefficient.

Stoa (1978) reanalyzed among others investigations on wave run-up at ’stepped-slope’ configurations by Saville (1955), Jachowski (1964) and Nussbaum and Colley (1971) and summarized *r*-values, representing a reduction factor for the wave run-up at the same structure with smooth slope and equivalent to the reduction factor γ_f used in this paper. Stoa (1978) also concluded that the relative water depth d/H has significant influence on the wave run-up even with a zero toe depth. The reduction factor γ_f is not significantly influenced for varying wave steepness values if the foreshore is flat. Due to the high variation of γ_f for nearly any kind of revetment type, the factor has to be highly dependent on the several wave and structure conditions so as ’any one value of γ_f does not seem applicable for all wave conditions for a given armor unit’. Furthermore, scale effects in model tests on the wave run-up at smooth and rough slopes are discussed. Stoa (1978) calculated the wave run-up with the correction factor including a correction regarding scale effects. The scale effect correction factor has to be applicable to the data from which γ_f is derived.

Tabata *et al*. (1980) provided a then first state-of-the-art summary based on existing design data of 107 so called ’stepped face seawalls’ in Japan including principal design conditions, design drawings, construction details and photographs to establish a comprehensive design method. Hydraulic data are not presented. Shibata *et al*. (1981) discussed the data set introduced by Tabata *et al*. (1980) regarding general design methods and frequency distributions of appropriate geometry parameter configurations. Thereby, the overall revetment width accounts between 1.5m and 6.0m, most revetments have a width of 3.0m ($N = 36$). The slope of the stepped revetments lies between $1.5 < n < 5$ with a dominant slope of $n = 3$ ($N = 27$). The steps have a height of 0.2m, 0.25m and 0.3m with 9 outliers between $0.15\text{m} < S_h < 0.5\text{m}$. The step width varies between $0.25\text{m} < S_w < 2.1\text{m}$; most of the steps have a width of 1.5m ($N = 33$) followed by 1.0m ($N = 21$). Interesting is, that only 17 structures show a horizontal orientation of the horizontal step face, whereas all others are slightly inclined offshore between $2.5 < n_s < 20$; most steps are inclined by $n_s = 10$ ($N = 27$). In most cases ($N = 18$) the low water level is 0.5m under the toe-level of the stepped revetment. The position of the low water level varies from site to site between 2m under toe-level up to 3m above low water level.

USACE (1981) discussed the functional applications, limitations and the general design concepts of seawalls with a subsection for ’Stepped Face Seawalls’. Within this section the potential of step-faced seawalls for dissipating wave forces, reducing wave reflection, run-up and scour effects (toe armoring) and parallel providing an easy access to a wide fronting beach exposed to moderate wave action is itemized. In addition, wave overtopping may be decreased by a wave-absorbing slope face of concrete steps.

Takayama *et al*. (1982) conducted physical model tests with irregular waves to determine the amount of wave overtopping discharge at low crest type sea walls including 135 test runs to run-up at stepped revetments. For two different step heights and three varying slope angles ($n = 2, 3, 4$) the wave overtopping was studied for a range of hydraulic boundary conditions. In front of the stepped revetment a 1 in 30 inclined foreshore berm was installed. Their data shows a decreasing reduction factor γ_f with decreasing slope steepness and for several H/S_h. Compared to results of overtopping volumes at a vertical wall, the volumes increase slightly for stepped revetments, but less dominant as in data by Goda and Kishira (1976). With increasing dimensionless overtopping volume $q/\sqrt{2gH^3}$ the reduction coefficient γ_f becomes larger and therefore the influence of the surface roughness of the stepped slope less important for the energy dissipation. Takayama *et al*. (1982) defined the reduction factor for wave overtopping under given boundary conditions to $0.68 < \gamma_f < 0.9$.

The **SPM (1984)** gives a comprehensive summary of shore protection methods including some comments regarding ’stepped structures’. Stepped-face sea walls are designed for stability against moderate waves. A combination of stepped- and curved face sea wall is built to resist high wave action and reduce scour (energy dissipation during the wave run-down process). SPM (1984) lists and refers findings by Saville (1955) and Saville (1956). Among others, the wave overtopping rate depends on the slope face weather it is smooth, stepped, or riprapped.

Hamilton (1988) confirmed a reduction of run-up heights and overtopping of the crest of a prototype structure in storm case within a communication on the design and construction of sea defenses at Sheerness. Furthermore, Hamilton (1988) approves beach retention at the toe of a stepped face slope due to a reduced down-wash and indicates good places for sitting at the sea. Hence, findings by United Nations (1982) are confirmed, findings by Sato *et al.* (1971) are contradictory.

Heimbaugh (1988) conducted hydraulic model tests for a 'stepped seawall' in irregular waves (TMA spectra, Froude scale 1:19) with focus on wave overtopping limitation at stepped seawalls, wave-induced pressures on these steps to aid in final design and toe stability. A sloping structure (n = 1.5, n = 2) with step height of S_h = 0.024 m was analyzed. At the top of the revetment a curved seawall was added. Overtopping volumes were less for more gentle slopes (n = 2). A design formula for wave overtopping prediction could be deployed to

$$q = 2.371 exp\left(-7.476 \frac{R_c}{\sqrt[3]{H_{m0}^2 L_p}}\right) \quad (R^2 = 0.948). \tag{2}$$

Due to seiche effects in the flume overtopping data scatter. Heimbaugh (1988) tried to correct present effects of seiches in the wave flume by an adaptation of the prediction formula to

$$\frac{q}{\sqrt{gH_{m0}^3}} = exp\left(-7.476 \frac{R_c}{\sqrt[3]{H_{m0}^2 L_p}} + \frac{\left(\sqrt{2}\,\frac{H_{m0,total}^2 - H_{m0,wind}^2}{4}\right)}{R_c}\left(C_3 \frac{R_c}{\sqrt[3]{H_{m0}^2 L_p}} - C_2\right)\right). \tag{3}$$

The regression coefficients C_3 and C_2 are not given in the paper whereas this correction is not applicable. Additionally, the pressure on the vertical step faces were measured. Short-duration (< 0.02 s) pressure shocks followed by about 90% smaller secondary pressure magnitudes (2−3 s) were measured on the steps. The position of max. pressure is dependent on the *SWL*.

Walton *et al*. (1989) provided a tabular summary with run-up reduction coefficients for various surfaces including stepped surfaces and stated that the results base on high end laboratory measurements as against Stoa (1978) or SPM (1984) for instance. Due to labor-intensive on-site forming of cast-in-place concrete, precast components like e.g. stepped-face seawalls are nowadays commonly used.

Ward and Ahrens (1992) re-analyzed tests by Heimbaugh (1988) and extended the data by additional test series for more wave heights and wave periods. Ward and Ahrens (1992) summarized that a stepped seawall is able to dissipate energy by evoking more turbulence and thereby reduces the wave overtopping volume. But in comparison to a large revetment without steps it seems to be less effective in reducing overtopping volumes. Nevertheless, observations during the tests indicated, that the steps might have been too small for an effectively flow disruption ($9 < H/S_h < 10.5$). They improved Heimbaugh's formula to

$$\frac{q}{\sqrt{gH_{m0}^3}} = exp\left(-11.174 \frac{R_c}{\sqrt[3]{H_{m0}^2 L_0}} - 10.667 \sqrt{\frac{H_{m0}}{L_0}}\right) \quad (R^2 = 0.948). \tag{4}$$

Ward (2003) conducted wave overtopping studies of a stepped revetment for the city of Chicago, Illinois at Lake Michigan shoreline in a model scale of 1:35 with irregular waves. Key parameters included the number and size of steps, the crest elevation R_c and the width of a promenade. They stated that the incident wave energy is increased by re-reflected waves at the wave board. They observed reasonably low overtopping rates for model configurations with high freeboard heights R_c, a large parapet on the promenade and an additional installed offshore breakwater. A prediction model was developed following the form

$$\frac{75q+1}{\sqrt{gH_{m0}^3}} = exp(1.1215)\, exp\left(-7.743 \frac{R_p}{\sqrt[3]{H_{m0}^2 L_0}} - 10.501 \frac{R_c}{\sqrt[3]{H_{m0}^2 L_0}} - 14.222 \frac{d}{L_0}\right). \tag{5}$$

Krecic and Sayao (2003) conducted a re-analysis of the results by Ward (2003). According to the authors Ward (2003) did not measure the incident wave height at the toe of the structure, they estimated H_s by the shoaling equation defined by Goda (2010) (2nd edition). Main part of the reanalysis of the data was a dimension analysis that yields to an adapted design formula for wave overtopping prediction at stepped revetments

$$\frac{q}{\sqrt{gH_s^3}} = 2.24 \cdot 10^{-5} exp\left(-2.5 \frac{R_c + R_p}{H_s}\right)\left(\frac{d}{L_0}\right)^{-2.28}\left(\frac{B}{H_s}\right)^{-1.23}\left(\frac{B_r}{H_s}\right)^{-2.28} \quad (R^2 = 0.92) \tag{6}$$

valid for $0.442 < R_c/H_s < 1.919$, $-0.182 < R_p/H_s < 0.417$, $0.009 < d/L_0 < 0.07$, $2.545 < B/H_s < 8.655$, $1.004 < B_r/H_s < 8.701$.

Suzuki *et al*. (2003) conducted physical model tests with the focus on wave overtopping volume and velocity of overtopping water over the crest for smooth and stepped slopes (n = 3). The step height S_h was 0.01 m. Two free board heights R_c and two water depth at the toe of the seawall d (0, 0.05 m) were tested for four different sets of incident waves. The authors conclude a relatively overtopping rate reduction and a smaller reflection coefficient at relatively large water depth in comparison with smooth seawalls.

Okayasu *et al*. (2005) compared wave overtopping volumes resulting from physical model tests conducted by Suzuki *et al*. (2003) over stepped and plain slope (n = 3) with results of numerical model tests (2D and 3D large eddy simulation). The 2D simulation model far overestimated the overtopping volume whereas the 3D simulation model fits the overtopping volume for a smooth surface but underestimates the overtopping by 50% for the stepped surface. They mention that the step height in the simulation was equal to one grid size and therefore the model could not account for additional wave dissipation due to small eddies generated by the steps. The importance of an appropriate evaluation of shear stress and energy dissipation by the sea wall steps is mentioned.

Hughes (2005) estimated the influence of rough, impermeable slopes on the wave run-up identifying only slightly differences of the run-up between waves that broke on the slope and non-breaking waves. Therefore, the formula is valid for breaking and non-breaking incident wave conditions. The implemented reduction factor considering the effect of an impermeable rough surface on the wave run-up of $\gamma_f = 0.505$ was derived from a best fit regression of relative wave run-up based on data by Heimbaugh (1988) and Waal and van der Meer (1992) resulting in

$$\frac{R_{u,2\%}}{h} = 4.4(tan\alpha)^{0.7}\sqrt{0.639\left(\frac{H}{h}\right)^{2.026}\left(\frac{h}{gT^2}\right)^{-\left[0.18\left(\frac{H}{h}\right)^{-0.391}\right]}}\,\gamma_f \tag{7}$$

valid for $2.0 \leq cot\alpha \leq 4.0$.

van Steeg (2012) conducted hydraulic model tests (Froude scale 1:10) in irregular waves for various geometries of stepped revetments and list reduction factors that can be used for overtopping prediction according to Eurotop (2007).

Xiaomin *et al*. (2013) conducted physical model tests (Froude scale 1:10). Regular waves were generated by a piston-type wave maker. Five different step heights and as reference a plain slope (all geometries with n = 2.5) were analyzed in order to evaluate the influence of the step height in relation to the wave height on the wave run-up (54 tests). They attest a good wave absorbing effect. Their data shows a reduction factor for the wave run-up over stepped slopes of $0.35 < \gamma_f < 0.77$. **Treuel (2013)** reveals a minimum wave run-up height in the data set of Xiaomin *et al*. (2013) for a certain number of steps (not the minimum, not the maximum). According to the authors the run-up reduction coefficient can be derived by

$$\gamma_f = 1 - \frac{S_w}{S_w + 6H_s}. \tag{8}$$

Physical model tests (Froude scale 1:10) by **Chuenchai *et al*. (2014)** in a 16.0 m long and 0.6 m width wave flume with flap type wave maker result in a design formula for wave run-up prediction for regular waves on 'stepped slopes'. The formula bases on 840 test runs. Four different step heights (S_h = 0.02m, 0.03m, 0.04m, 0.05m) were tested with varying slope angles ($2.1 < n < 3.7$). The authors conclude a decreasing wave run-up height with increasing riser heights due to increasing friction. Furthermore, higher relative run-up heights for relative step heights S_h = 0.15 in comparison to S_h = 1.5. They defined the reduction factor for wave run-up under given boundary conditions to $\gamma_f = 0.64$.

$$\frac{R_u}{H} = 0.98\xi^{0.94}\left[1 - 0.46\left(\frac{S_h}{H}\right)^{0.12}\right] \quad (R^2 = 0.81, STD = 0.23). \tag{9}$$

Kerpen *et al*. (2014) refined an approach to predict the mean wave overtopping in regular waves by Schüttrumpf (2001) with adaptation for stepped revetments. The adaptation is based on physical model tests (Froude scale 1:5) conducted in a laboratory wave flume with standard slope angles (n = 2 and n = 3). A formula for prediction of the mean overtopping discharge over stepped revetments was derived to

$$\frac{q}{\sqrt{2gH^3}} = \left(0.163 - \frac{0.336}{\xi_d^3}\right)\left(1 - \frac{R_c}{2.25\tanh(0.5\xi_b)H}\right)^{3.2} \quad (R^2 = 0.98, STD = 0.0005). \tag{10}$$

3 DISCUSSION

If one goes into detail, Saville (1955), Goda and Kishira (1976), Takayama *et al*. (1982), Heimbaugh (1988), Ward and Ahrens (1992), van Steeg (2012), Xiaomin *et al*. (2013) and Kerpen *et al*. (2014) provide data regarding wave run-up, wave overtopping, wave reflection and wave forces on stepped revetments. These data are displayed in Figure 4 to show up limits of comparability. For test in regular waves (black labeling) the mean wave height H and the mean wave run-up R_u is underlying the displayed data whereas for tests with wave spectra (colored labeling) the spectral wave height H_{m0} and the wave run-up height $R_{u,2\%}$ exceeded by 2% of all run-ups are shown.

First of all, the hydraulic boundary conditions are compared regarding the range of data availability for different wave steepness H/L in comparison to the relative water depth h/L in Figure 4 (a). Direct or indirect information regarding the water depth h was given only by Saville (1955), Heimbaugh (1988), van Steeg (2012) and Xiaomin *et al*. (2013). Most tests were conducted for intermediate water depth, a relatively large data set (178 test runs) by Heimbaugh (1988) is close to shallow water conditions. Deep water conditions were tested in some cases by Saville (1955) and Xiaomin *et al*. (2013). Data by Takayama *et al*. (1982) seem to underlay extreme shallow water conditions with $0.012 < h/L < 0.03$. Therefore these data are not given in the Figure 4 (a). To evaluate the hydraulic boundary conditions of the impacting waves, the data sets are displayed in Figure 4 (b) by wave steepness H/L over slope n. Data sets in regular waves are displayed in monochrome scheme, data sets with wave spectra are displayed in color. For reasons of visual clarity data by Goda and Kishira (1976) and Ward and Ahrens (1992) are slightly shifted horizontally next to $n = 2$, whereas a slope of $n = 2$ was tested. Saville (1955) provides data with a broad spectra of wave steepness for a relatively steep slope angle. Data by Goda and Kishira (1976) are available for a moderate narrow banded variation of wave steepness and a particular slope angle. Data sets by Takayama *et al*. (1982) underlie tests with a single wave steepness but support a broad range of slope angles. Heimbaugh (1988) tested two relatively steep slope angles for average wave steepness. Data by Ward and Ahrens (1992) and Xiaomin *et al*. (2013) cover a wide range of wave steepness for more gentle slope angles in comparison to Saville (1955). van Steeg (2012) tested an average range of wave steepness for two gentle slope angles. The ratio between slope angle of a structure and wave steepness can be expressed by the Iribarren number $\xi = (1/n)/(H/L)$ which is used hereinafter.

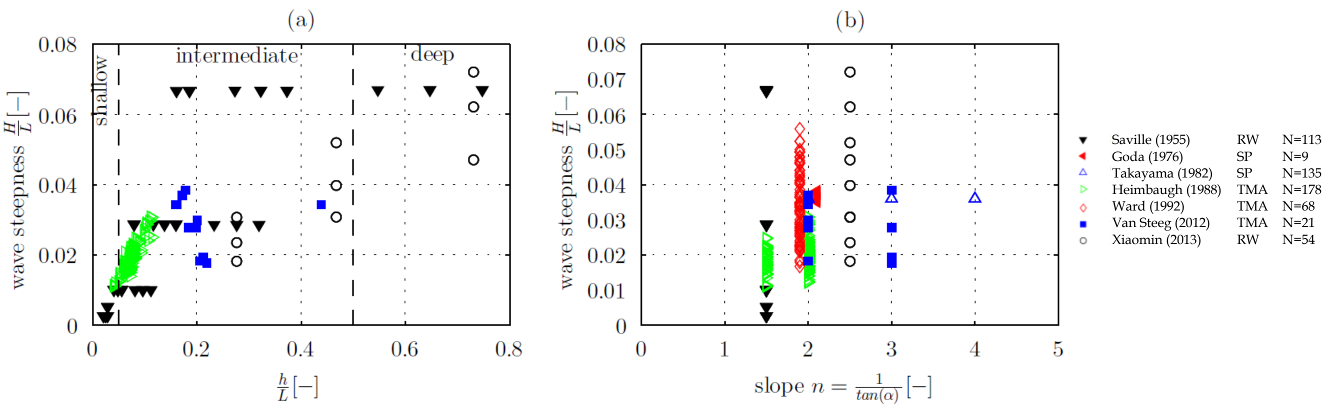

Figure 3. Overview to data sets obtained from literature with respect to (a) relative water depth and (b) wave steepness

Figure 4 (c) gives an overview to data sets evidenced by named authors with respect to the relative step height S_h/H over the Iribarren number ξ. The Iribarren number is covert for $0.88 < \xi < 6.3$ and three particular tests up to $\xi = 13$ (not shown in the figure to assure visual clarity). Therefore, except spilling wave breaking ($\xi < 0.2$), a broad spectra of wave breaking is covered by literature with plunging ($0.2 < \xi < 2 - 3$), collapsing ($2 < \xi < 3$) and surging ($\xi > 2 - 3$) wave breaking. Relative step heights S_h/H are covered for a range of $0.025 < S_h/H < 3.6$. Most data are available for $2 < \xi < 6$ and $0.2 < S_h/H < 0.45$. Data by Goda and Kishira (1976) and Takayama *et al*. (1982) represent small relative step heights in comparison to the other test series. Xiaomin *et al*. (2013) provide the only data set with step heights S_h larger than the wave height H. Nevertheless, all data sets cover a different range of parameter combination, causing differences in the wave breaking over the revetment. This leads to problems in the comparison of the results concluded out of these data.

The incorrect reproduction of turbulence in small scale models is well known (Frostick *et al*., 2011) and has a strong influence to the output of physical models. Therefore, Figure 4 (d) allows a closer discussion of the range of Reynolds numbers (according to Eq. 3.5) in the different data sets. The kinematic viscosity was assumed to $\upsilon = 1.004 \cdot 10^{-6}$ in all cases. Even the data sets with very small step height in the model (Goda and Kishira (1976) and Takayama *et al*. (1982)) have Reynolds numbers $Re > 3 \cdot 10^4$ and can therefore be evaluated as valid.

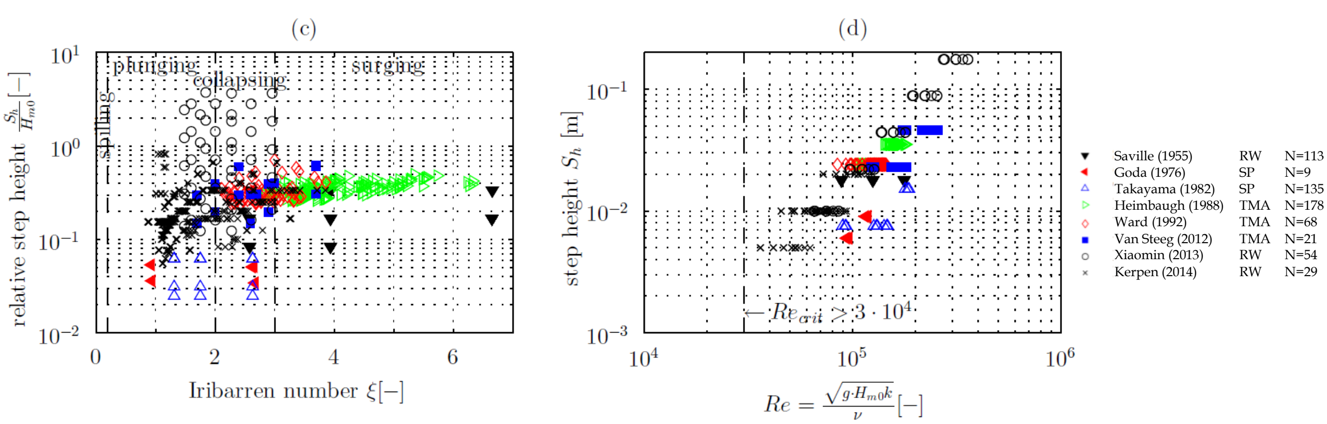

Figure 4. Overview to data sets obtained from literature with respect to (c) relative step height and (d) Reynolds number

Figure 5 (e) gives an overview to data sets evidenced by named authors with respect to dimensionless wave run-up $0.5 < R_u/H < 3.5$ over the Iribarren number $0.9 < \xi < 3.6$ (with additional four particular tests up to $\xi = 7.2$). The dimensionless wave run-up over Iribarren number has similar ranges all compared data sets independent for test in regular waves (black) or wave spectra (colored). The dimensionless wave run-up over stepped revetments tend to increase with increasing Iribarren number, although the different data scatter significantly.

The dimensionless overtopping discharges in the rage of $2 \cdot 10{-}5 < q/(gH^3)^{0.5} < 0.1$ are displayed in Figure 5 (f) in a semi-logarithmic scale over the dimensionless freeboard height $0.1 < R_u/H < 3$ for data provided by Saville (1955), Goda and Kishira (1976), Takayama *et al*. (1982), Heimbaugh (1988), Ward and Ahrens (1992) and van Steeg (2012). The measured overtopping rates for the same relative freeboard height differ in order to the power of two. But, a grouping of data regarding the different authors can clearly be seen. Including the insights of Figure 4 (c) the reason of scatter might be seen beside geometry related differences in different wave breaking conditions within the model tests and thereby differences in the energy dissipation, too.

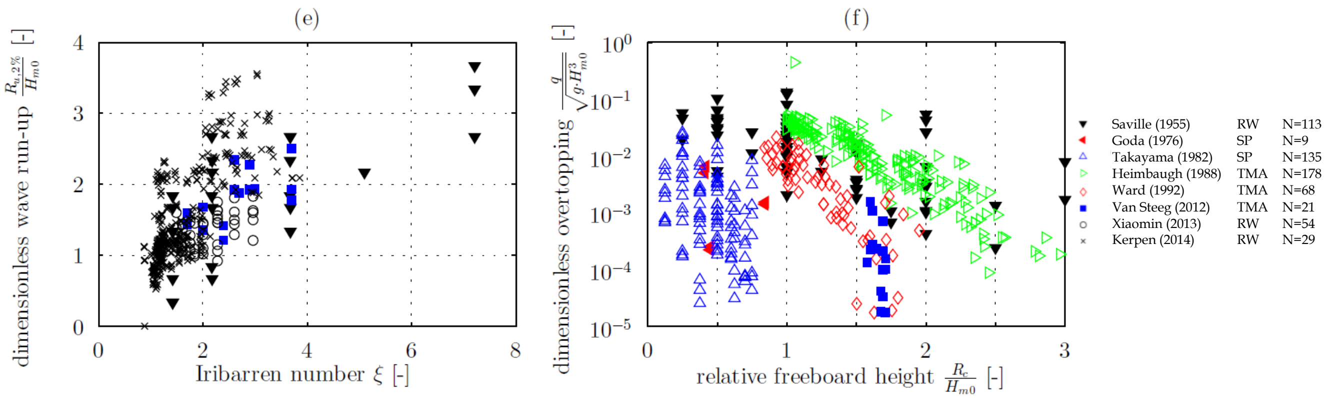

Figure 5. Overview to data sets obtained from literature with respect to (e) dim. wave run-up and (f) dim. Overtopping

In Figure 6 (g) the compared data regarding dimensionless overtopping over relative freeboard height are normalized by the wave steepness. All data sets except the ones by Takayama *et al*. (1982) and Goda and Kishira (1976) show roughly the same exponential decrease of the dimensionless overtopping rate with increasing dimensionless freeboard height. Clarification for the still visible significant differences in the overtopping issues a closer discussion of the range of step heights in the different test series. Goda and Kishira (1976) as well as Takayama *et al*. (1982) tested very small step heights in model scale ($S_{h,Takayama}\{0.008m, 0.015m\}$, $S_{h,Goda}\{0.006m, 0.009m\}$). This can be a first hint of the importance in division of boundary conditions with micro and macro roughness. Xiaomin *et al*. (2013) tested a wide range of dimensionless step heights with $0 < S_h/H_{m0} < 3.8$ in regular waves. Treuel (2013) concluded that the data tend to constitute an optimum step height $S_h = 0.5H_{m0}$ leading to a lowest wave run-up.

Figure 6 (h) shows a similar approach according to van Steeg (2012) considering the slope angle and the wave steepness additionally. The data by Treuel (2013) shows a minimum at a relative step height of 0.25. Still their statement cannot be proven yet, because all other data have boundary conditions of $S_h/H < 0.5$. But, the most valuable data set regarding the quality of the hydraulic model tests by van Steeg (2012) prove this finding for wave spectra in the range of $S_h/H < 0.5$. Data by Saville (1955) suit this approach. Data by Goda and Kishira (1976) shows lower reduction coefficients. Obviously, additional tests with $S_h/H > 0.5$ are outstanding.

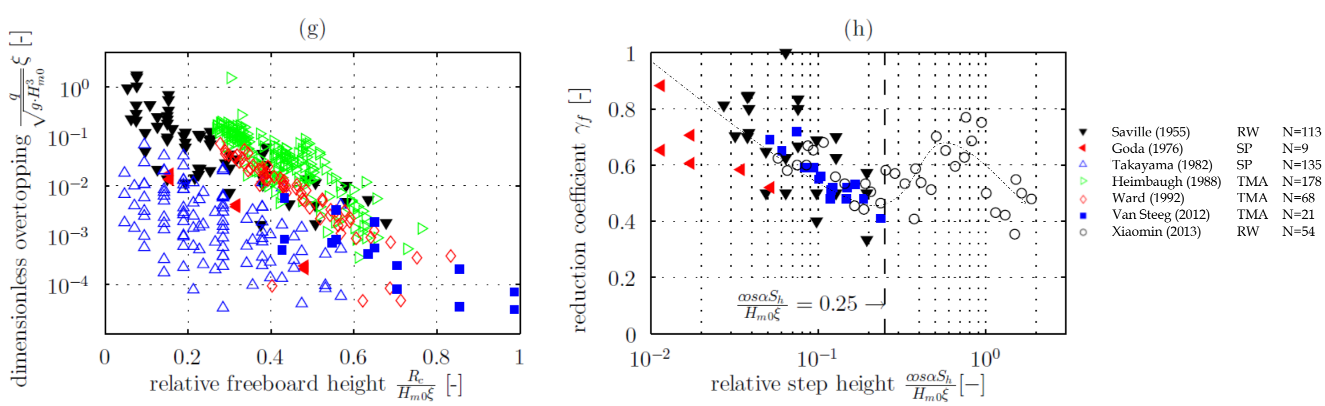

Figure 6. Overview to data sets obtained from literature with respect to (g) dim. overtopping and (h) reduction coefficient γ_f

4 CONCLUSION

Within the literature review almost 30 publications could be identified, describing the interaction of sea states with stepped revetments. If one compares all findings against each other, the authors agree in following issues:

- As for each sloping structure, also for stepped revetments the wave run-up and as a consequence thereof the wave overtopping volume decreases with increasing wave steepness, decreasing slope angle of the revetment, the presence of a shore face and an increasing freeboard height.
- An increasing surface roughness reduces the wave run-up. The surface roughness of stepped revetments is dependent on the step geometry in proportion to the hydraulic boundary conditions. Takayama *et al*. (1982) and Ward and Ahrens (1992) concluded that the tested step height was too small for an effective flow disruption and in consequence limited influence on the run-up process. Since Ward and Ahrens (1992) extended model tests by Heimbaugh (1988) this finding is applicable although for this data set. In these tests the wave height is at least nine time larger than the step height. Hence, Goda and Kishira (1976) overrun this value, too.
- The relative run-up height R/H decreases with decreasing relative step height $0 < S_h/H < 0.5$.
- The effect of an inclined vertical step-face as well as the geometry of the step's edge on the wave run-up-reduction can be neglected.

Outcome of single studies without proof by associated literature:

- The reduction factor γ_f decreases exponential for increasing wave overtopping rates q. (Saville, 1955)
- A vertical block face is more stable than an inclined one. (Jachowski, 1964)
- For vertical bock faces a stability number N_s >12.8 could be derived. Stepped revetments fail at or slightly above *SWL* by forced up blocks, at wave overtopping due to a lifted top row. (Jachowski, 1964)
- The major scour influence was observed in a distance of $L/3$ from the foot, general influence measured up to 1 L distance from the structure foot. (Sato *et al.*, 1971)
- A stepped slope requires a higher freeboard height R_c than a vertical wall. (Goda and Kishira, 1976)
- The wave run-up has a minimum for $S_h/H = 0.5$. (re-analysis from data measured by Treuel (2013))

Furthermore, some opposing findings turned out that have to be proven in prospective investigations:

- Jachowski (1964) spotted no difference in the scouring in front of stepped revetments and plain slopes, Sato et al. (1971) measured a slightly increasing scouring, USACE (1981) states a potential for scour protection due to toe armouring and Hamilton (1988) emphasizes an improved beach retention.
- Nussbaum and Colley (1971) observed a more effective reduction of the run-up for small step heights whereas van Steeg (2012) and Chuanchai *et al*. (2014) concluded the opposite system behavior.

In consequence that one is rarely able to compare the existing data and no data set is available that combines a wide range of geometry-related and hydraulic boundary conditions enabling the identification and description of universal processes there is still a need for additional research regarding the identification of the process-relevant input parameter and their optimization for the energy dissipation. This can be obtained by a closer discussion of the reflection and transformation of the incident waves with respect to the geometry-related parameters. A comprehensive analysis of the dimensionless step height S_h/H is promising in this context, too, to state the findings by Treuel (2013).

APPENDIX – LIST OF SYMBOLS

α	slope angle
γ_f	friction reduction factor
υ	kinematic viscosity
ξ	Iribarren Number
B	width of revetment
B_r	width of a toe berm
d	*depth at revetment toe*
g	gravitational acceleration
h_s	deep water depth
H	wave height
H_s	significant wave height
H_{m0}	spectral wave height $4\sqrt{m_0}$
i	slope of a shore face
L	wave length
n	slope ($n = 1/tan\alpha$)
n_s	slope of the horizontal step face
N	quantity of test runs
N_s	stability number (Hudson)
q	overtopping volume per second per meter crest length
r	reduction factor
R^2	coefficient of determination
R_c	vertical elevation of the crest over SWL
R_p	vertical elevation of a promenade over SWL
R_u	vertical wave run-up over *SWL*
RW	regular wave
S_h	step height
S_w	step width
SP	spectrum
STD	standard deviation
SWL	still water level
T	wave period
TMA	shallow water spectrum

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