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Peters, Karsten; Hunnius, Jani; Gerdes, Heiner

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Strategies for Elevating Existing Flood Protection Lines in Urban Environments or in Limited Space

K. Peters & J. Hunnius
Ramboll GmbH, Hamburg, Germany

H. Gerdes
Staatliches Baumanagement Ems-Weser, Wilhelmshaven, Germany

Abstract: Worldwide climate change cause an increase in sea levels along the world's coastlines, which result in different consequences regarding existing and planned coastal protection measures. In Germany, the responsible authorities for coastal protection along the North Sea and Baltic Sea coasts adapted their boundary conditions for the design of coastal protection measures as well as their mitigation strategy. Especially in urban or industrially used areas with limited space for coastal protection measures special solutions are needed. This mitigation process is discussed referring to a project in Wilhelmshaven, where a sheet pile wall was chosen as protection measure. Within the design process different calculation methods for the wave overtopping were applied and compared. In order to gain a uniform design height of the protection line it was agreed with the coastal authority to approve a lower design height and allow a higher wave overtopping rate up to 2 l/(s m) compared to normal requirements for protection measures with 0.5 l/(s m). Therefore additional measures were required on the landside of the vertical sheet pile wall like paving or increasing the capacity of the drainage system to conduct the overtopping water. This paper shows how to mitigate extreme boundary conditions.

Keywords: coastal protection, wave overtopping rate, discharge, sheet pile wall, wave overtopping formula, design height, discharge tolerances, approval design

1 Introduction

Coastal protection has a long history over hundreds of years in Germany. Today more than 2.5 million people live and work in low lying areas (size: ~ 12,000 km²) along the North Sea coast (length: 1,590 km) and Baltic Sea coast (length: 2,100 km), which are endangered by storm flood events (Figure 1).

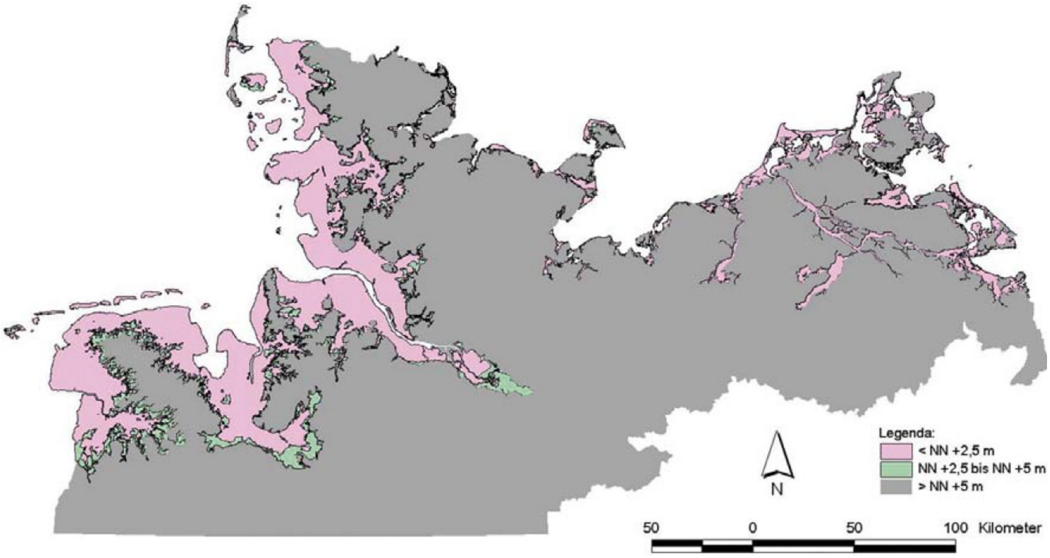

Fig. 1. Low lying areas in northern Germany endangered by storm flood events (Hofstede et al. 2009).

In Germany the coastal protection line consists mainly of four different types of protection systems, like dikes, open natural sandy coastlines with dunes or cliffs, artificial slopes and revetments and vertical and aligned protection measures like sheet pile walls (see following Tab. 1). Geological, topographical and hydraulic boundary conditions have always influenced the construction and development of coastal protection systems. At the North Sea coast dikes are the dominating protection measure, while at the Baltic Coast open sandy coasts and artificial slopes and revetments are the main coastal protection systems. In urban and industrial areas vertical and aligned protection measures are often realized. For more details concerning the German coastal protection measures and corresponding boundary conditions it is referred to Die Küste (2008).

Tab. 1. Coastal protection systems in Germany (Peters et al., 2012)

Coastal protection system and corresponding elements		Examples
Dike	main dike foreland wadden sea	
Open natural sandy coast	cliff dunes beach profile underwater profile	

Coastal protection system and corresponding elements		Examples
Artificial slopes	revetments breakwaters groins	
Vertical and aligned protection measures	high water protection walls locks barriers	

Worldwide climate change cause an increase in sea levels along the world's coastlines and also has an impact on storm events based on dynamic changes in the atmosphere. These measurable and ongoing changes result in different consequences regarding existing and planned coastal protection measures. In Germany, the responsible authorities for coastal protection adapted their boundary conditions for the design of coastal protection measures as well as their mitigation strategy. These mitigation strategies have to be found especially in urban or industrially used areas with limited space for coastal protection measures.

The objective of this paper is the documentation of the planning and realization of a mitigation measure for flood protection in Wilhelmshaven/Germany in order to show how existing calculation methods for wave overtopping can be used in the approval process. Normally the design height of a flood protection measure is specified by the responsible coastal authority. In this respect this paper can be seen as a practical example to convince clients and coastal authorities how to deal with the design parameters in case of overtopping rates, which are larger than normal overtopping rates provided by coastal authorities. These explanations will be done using the project case in Wilhelmshaven.

2 Case of application: Naval Base Wilhelmshaven

2.1 Project area and boundary conditions

The latest assessment of sea level rise and flood protection heights by the responsible coastal authority NLWKN led to an adjustment of the design water levels and waves for the area of Wilhelmshaven. It was found that the existing flood protection constructions (dike with varying seaward slope) within the perimeter of the naval base Wilhelmshaven (see Figure 2) were too low for future storm surge events. The existing dike height varies between NHN +7.0 m and NHN +7.6 m.

The new design water level was calculated by NLWKN with NHN +6.55 m, which is around 2 m higher than most of the quays in the naval base, and wave heights of up to 1.85 m. Caused by the length of the protection line of around 4 km and its orientation the wave impact varies significantly.

Fig. 2. Navy Base Wilhelmshaven (red dotted line) (source: www.wilhelmshaven.de).

The protection line was divided into different sections according to the intended improvement of the construction. The main attention lies on the protection around the existing locks, rebuilt after the second world war in 1964, with two chambers of around 390 x 60 m each. These locks not only serve the Navy but also all other maritime traffic coming and going towards the inland harbor. The sea side of the lock consists of a large bunker which serves as a garage for the lock gates, which has the disadvantage that the gates cannot be heightened unlike the rest of the flood protection line. This is a critical factor, as the section around the locks is one of the most stressed in case of a surge.

2.2 General risk assessment

As in many cities, densely populated and developed areas, flood protection in the naval base of Wilhelmshaven comes along with challenges. First, the flood protection line inside the base is also part of the city's and administrative district's protection line respectively. This is mainly a management issue, meaning that within the perimeter of the base the military and its service center have to maintain the constructions and operate mechanical parts like flood gates.

Unlike rural or housing only areas the base and some facilities nearby are also home to critical infrastructure and potential hazards for the environment. In addition to primarily protecting lives, risk assessment must also take into account that for instance fuel depots must not be harmed and no leakages can occur.

2.3 Engineering challenge in the lock area

The engineering challenge in the lock area for the design and realization of the protection line (see Figure 3) consists of the consideration of the following design criteria:

- limited space for the heightening of the protection line
- locally varying topographical and constructional conditions
- hydraulic design conditions
- existing traffic roads (flood protection gates are needed)
- lock gates with limited height
- harbor and lock operations (no restrictions of the harbor allowed in normal operation for persons and vehicles)
- large number of necessary cables, media lines and pipes crossing the proposed flood protection line (cable crossings are needed)

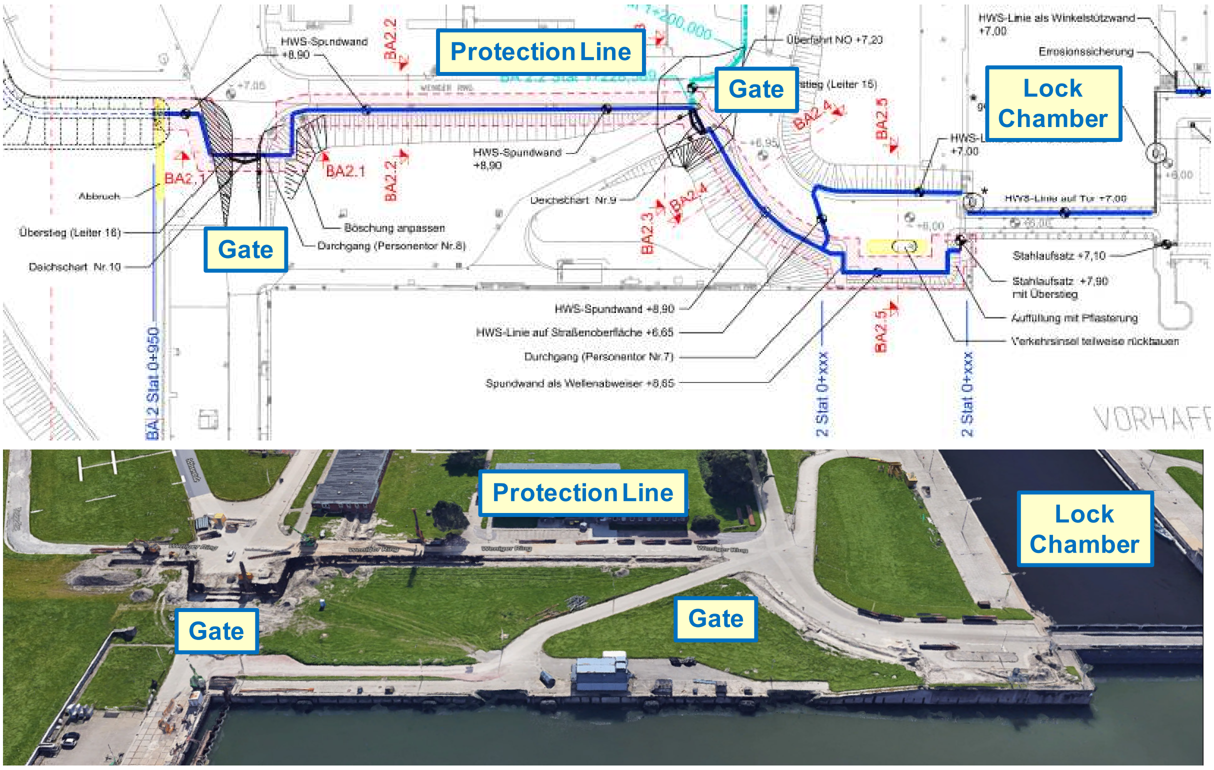

Fig. 3. Eastern part of the lock area with the designed protection line and gates (above) and under construction as aerial view (Source: Google, below).

Based on the mentioned design criteria only a sheet pile wall with flood gates could be considered as an adequate protection measure in the lock area. But different boundary conditions normally result in different design heights of the protection line. Therefore the unification of the protection line in position and height became the most important factor, maintaining an adequate level of safety.

2.4 Requirements for approval

High water protection measures in Germany need to be approved by the responsible coastal authority and fulfill specific requirements.

In case of the protection measure in Wilhelmshaven the requirements of the coastal authority in the first step contained only a limit of the overtopping rate with 0.5 l/(s m), which is also valid for other protection measures like dikes.

2.5 Design approach

The protection line in the lock section, also around 1 km long, could only be realized with strictly vertical constructions with limited horizontal extension because of the limited space available. Because of the inhomogeneous soil conditions a sheet pile wall with regularly varying bond length was chosen. The immediate connection to the lock incorporates smaller T- and L-shaped metal and concrete walls respectively. Furthermore, the lock chambers were surrounded by a second protection line, in case one or even both exterior lock gates would fail to close.

Within the planning process first calculations on the necessary protection height and the corresponding overtopping rate have been conducted. Within this project phase it became evident that based on first requirements the resulting design level of the protection line would be too high and would also vary extremely according to the topographical conditions.

Based on these first results an intensive communication and technical discussion with the coastal authority NLWKN was conducted with regard to the following issues:

- Understanding of the specific hydraulic processes and conditions within a representative cross-section (see Figure 4)
- Shoaling effects and wave transformation processes in the surf zone under hydraulic design conditions in order abbreviate realistic input parameter for the calculation of overtopping rates

- Search for adequate methods for the calculation of overtopping rates and abbreviation of the design level of the protection

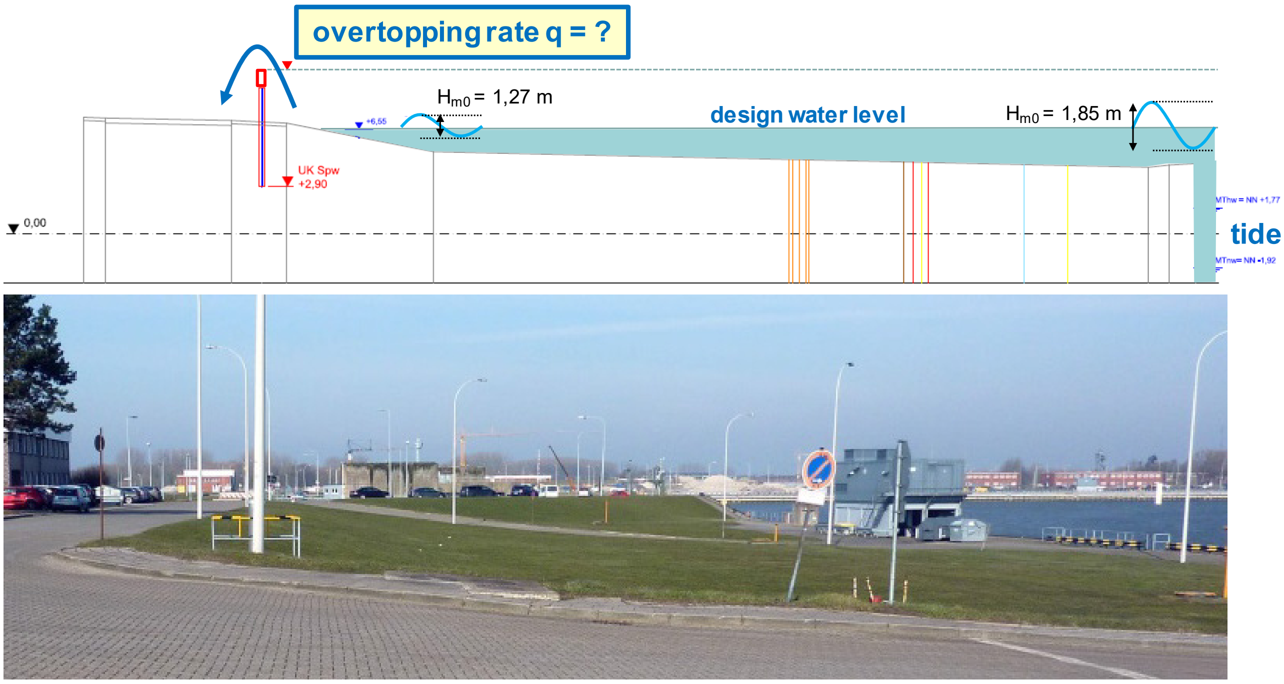

Fig. 4. Representative cross-section east of the lock with extensive green and paved foreland under design conditions (above) and as photo with normal conditions (below).

2.6 *Calculation models and methods*

Today engineers can resort to all kinds of calculation models and methods. When it comes to coastal engineering the Overtopping Manual EurOtop (EurOtop, 2016) nowadays provides a wide range of approaches and even online calculation tools for overtopping rates for different kinds of cross-sections.

Yet, as stated above, construction types cannot always be chosen from available standards or selected primarily in respect to efficiency and economic efficiency. This is due to the local boundaries. In those cases, assumptions and simplifications must be made to create a calculation model or applying existing methods. Another option, as utilized here, is to determine models or methods that best describe the current situation and compare results, considering known differences between model and reality. This will be exemplarily shown in detail for the construction section around the lock.

The coast line within the base and lock perimeter respectively, mostly consist of quays roughly 2.5 m above mean high tide but around 2.3 m below design water level. The quay areas are usually sealed with concrete pavement and 6 m to 10 m wide. After another 10m to 35 m of flat green the area shallowly rises to around 6.8 m NHN in most parts of the lock section. This is just 0.25 m above design water level. The new flood protection line is located on top of those elevations/ slopes but in front of the roads, which run along most of the elevated shore line. Therefore, model cross-sections with added flood protection walls look similar to Figure 4.

For the assessment of overtopping rates three different approaches have been used which are presented and compared in the following. Tab.1 includes the main parameters for the calculations. Among those are the total height of the construction as well as the freeboard R_C (difference between total height and design water level) and the wall height above ground. In this case the wall can be assumed to be placed on top of a green slope as schematically depicted in Figure 5. Wave conditions are expected to be impulsive due to the height and angle of the slope in correspondence to the design water level.

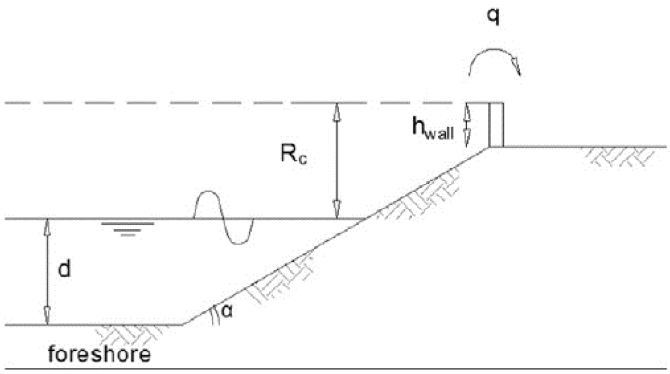

Fig. 5. Definition of design parameters (van Doorslaer et al., 2015).

The relative freeboard equals freeboard divided by wall height. It is usually used to define boundaries for the application of the formulas. In van Doorslaer et al. (2015) scale model tests were performed for $0{,}6 < R_C/H_{m0} < 2{,}6$ for this kind of geometry.

Tab. 1. Calculation Parameters east of lock

Input Parameters	value	[unit]
wave height	1,0	[m]
wave period	4,3	[s]
wave length	28,3	[m]
(angle of attack)	~90	[°]
total height	8,9	[m NHN]
wall height	1,3	[m]
freeboard	1,5	[m]
relative freeboard	1,4	[-]
impulsive conditions		

The following formulae were used in this cross-section. (7.10) and (7.11) are taken from the EurOtop Manual (2016) and are applicable to plain vertical walls in impulsive conditions with an emergent toe of the wall. (7.10) represents the probabilistic and (7.11) the deterministic approach. For each calculation both types were used. (8) is presented by van Doorslaer et al. (2015) and applicable to multiple cross-sections around the lock but with certain limitations. Geometries were only tested with slopes of 1:2 to 1:3 and for non-breaking waves. In the case of Wilhelmshaven waves would break before or at the toe of the structure and slopes are usually around 1:6. Because of that the results from calculations utilizing the findings of van Doorslaer et al. (2015) are rather used for general comparison than detailed planning.

The probabilistic approach results in the lowest overtopping volume of just 0.7 l/(s m) while van Doorslaer et al. (2015) returns the highest volume, at 3.4 l/(s m). This is in correspondence with calculated q for other cross-sections. During the design process it was established to use the higher results from deterministic formulae to be on the safe side. The van Doorslaer et al. (2015) results always turned out to be the highest, for some sections also topping the set goal of 2 l/(s m) while still staying within range of the values proposed by EurOtop Manual (2016). As stated above, the formula of van Doorslaer et al. (2015) was actually derived for steeper slopes and non-breaking waves, ultimately overestimating overtopping volumes for the cross-sections considered here.

In general, many calculations had to be tested because most of the available approaches are validated only for a rather specific geometry and other boundaries. It cannot be concluded that exceeding those boundaries does not have a significant influence on the results.

Tab. 2. Applied formulas by EurOtop Manual (2016) and van Doorslaer et al. (2015) and calculated overtopping rates

Applied formulas by EurOtop Manual (2016) and van Doorslaer et al. (2015)		Calculated overtopping rate [l/(s m)]
$\frac{q}{\sqrt{gH_{m0,deep}^{3}}} \cdot \sqrt{ms_{m-1,0}} = 0.043 \exp\left(-2.16 ms_{m-1,0}^{0.33} \frac{R_c}{H_{m0,deep}}\right)$ valid for $2.0 < ms_{m-1,0}^{0.33} \frac{R_c}{H_{m0,deep}} < 5.0$; $0.55 \le R_c/H_{m0,deep}$ $s_{m-1,0} \ge 0.02$ 5; Note – data only available for $m = 10$ (*i.e.* 1:10 foreshore slope)	7.10	0,7
$\frac{q}{\sqrt{gH_{m0,deep}^{3}}} \cdot \sqrt{ms_{m-1,0}} = 0.043 \exp\left(-1.95 ms_{m-1,0}^{0.33} \frac{R_c}{H_{m0,deep}}\right)$ valid for $2.0 < ms_{m-1,0}^{0.33} \frac{R_c}{H_{m0,deep}} < 5.0$; $0.55 \le R_c/H_{m0,deep} \le 1.6$; $s_{m-1,0} \ge 0.02$ 5; NB – data only available for $m = 10$ (*i.e.* 1:10 foreshore slope)	7.11	1,2
$\frac{q}{\sqrt{g \cdot H_{m0}^{3}}} = 0.2 \cdot \exp\left(-2.28 \cdot \frac{R_c}{H_{m0}} \cdot \frac{1}{\gamma_v}\right)$	(8)	3,4

2.7 *Calculation Results*

The results of the applied calculation methods showed reasonable variations of the overtopping rate between 0.7 l/(s m) and 1.2 l/(s m) for the given cross-section in case of a fixed design level of NHN + 8.9 m. Figure 6 shows a comparison to the protection height in case of 0.5 l/(s m) overtopping rate.

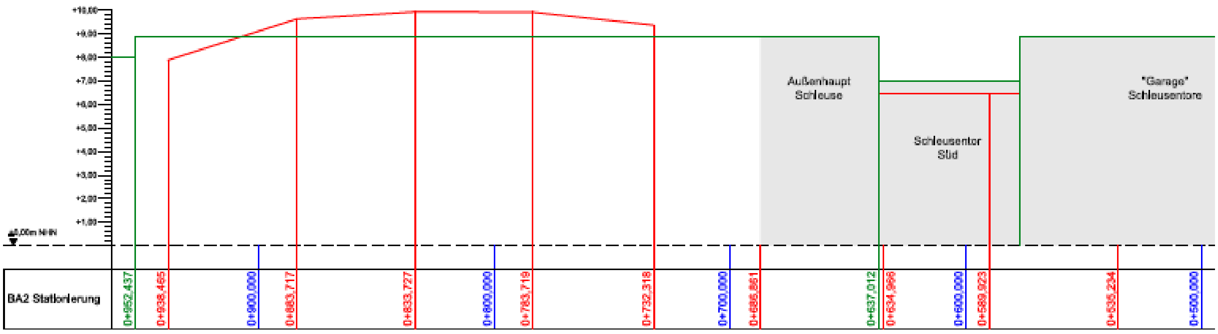

Fig. 6. Comparison of necessary design height in case of 0.5 l/(s m) (red line) and finally abbreviated design height (green line).

The coastal authority NLWKN specified necessary heights for a sheet pile wall of almost NHN + 9.5m west of the lock, to limit wave overtopping to a maximum of 0,5 l/(s m). This equals maximum wall heights above ground of 2.7 m. At the same time necessary heights differ around nearly 1 m within short distances. East of the lock, due to the direction of wave attack heights would easily reach NHN + 10 m or higher. For comparison, in the north of the naval base maximum dike heights would have to be around NHN + 8.5 m or sheet pile wall NHN + 11.3 m NHN under given boundary conditions.

According to the given limitations for overtopping for pedestrians and damage (see Figure 7) to the defense crest the calculated overtopping rate is feasible.

Within the planning process additional measures were considered on the landward side in order to guarantee that the overtopping water cannot cause damages at the surface. But along almost 3 out of 4 km of flood protection line inside the naval base, the main roads run directly behind or in case of the slopes and dikes on top of the current protection line. Therefore, drainage turned out to be just as urgent as structural safety of the surfaces.

Table 3.2: Limits for overtopping for pedestrians

Hazard type and reason	Mean discharge	Max volume[(1)]
	q (l/s/m)	V_{max} (l/m)
Trained staff, well shod and protected, expecting to get wet, overtopping flows at lower levels only, no falling jet, low danger of fall from walkway	1–10	500 at low level
Aware pedestrian, clear view of the sea, not easily upset or frightened, able to tolerate getting wet, wider walkway[(2)].	0.1	20–50 at high level or velocity

(1) *Note: These limits relate to overtopping velocities well below $v_c \approx 10$ m/s. Lower volumes may be required if the overtopping process is violent and/or overtopping velocities are higher.*
(2) *Note: Not all of these conditions are required, nor should failure of one condition on its own require the use of a more severe limit.*

Table 3.5: Limits for overtopping for damage to the defence crest or rear slope

Hazard type and reason	Mean discharge
	q (l/s/m)
Embankment seawalls/sea dikes	
No damage if crest and rear slope are well protected	50–200
No damage to crest and rear face of grass covered embankment of clay	1–10
No damage to crest and rear face of embankment if not protected	0.1
Promenade or revetment seawalls	
Damage to paved or armoured promenade behind seawall	200
Damage to grassed or lightly protected promenade or reclamation cover	50

Fig. 7. Hazard types and mean discharge (EurOtop Manual, 2016).

In conclusion, it was decided that adjusting the flood protection wall height, in most of the section around the lock, to NHN + 8.9 m, thereby reducing the total amount of sheet pile tonnage needed and creating a more uniform appearance of the construction. But that also meant a local increase in the acceptable overtopping volume to around 2 l/(s m) while reducing it in some other parts. This is four times the initial value set by the NLWKN but still within limits suggested by the EurOtop Manual (2016). In order to protect building structure elements, from incoming wave heights of 1 m to 3 m the EurOtop Manual (2016) suggests 1 l/(s m). And 2 l/(s m) were still determined to be safe and economical because of already heavily sealed surfaces behind the new to be build flood protection wall and the large distances to buildings. In addition, an anyhow necessary drainage adjustment makes sure that overtopping water is quickly distributed in between the sections with more or less overtopping and draining it all into the inland harbor.

3 Conclusions and recommendations for practical use

Based on the experiences in the Wilhelmshaven project regarding the design of high water protection measures the following can be concluded:

- The definition and abbreviation of design heights of protection lines is an interactive process between different parties, like client, designer, coastal authority and approval authority. Open communication between these parties is recommended at an early stage.
- Complex topographical conditions need detailed investigations on hydraulic boundary conditions in order to abbreviate reliable design parameters.
- Calculation methods for overtopping rates, which resemble the topographical conditions, have to be selected carefully and compared to ensure the applicability.

- For the abbreviation of the design height of the protection measure all necessary requirements have to be taken into account. The design height might be reduced if the discharge of overtopping water at the landward side can be guaranteed. A detailed check is needed.

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