HYDRODYNAMIC LOADING OF WAVE RETURN WALLS ON TOP OF SEASIDE PROMENADES

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To reduce coastal flooding risks in several coastal towns in Belgium wave return walls on top of the existing seaside promenades are designed. The structural strength and foundation of the wave return walls have to be designed taking into account the hydrodynamic loading due to overtopping waves. Based on existing relations for layer thickness and layer speed of overtopping waves a semi-empirical formula is developed to deliver a design value for the hydrodynamic loading on a wave return wall for given geometric and hydraulic boundary conditions. Using experimental results of scale models in wave flumes the empirical parameters of the semi-empirical formula are to be calibrated and validated for the range of applicability representative for the configurations occurring along the Belgian coastline.

PROBLEM DEFINITION

Introduction

The region of Flanders in Belgium borders the southern part of the North Sea. In winter time (September until March) storm surges occur in this area caused by depressions traveling over the North Sea. If very strong northwesterly winds last for days and are combined with high spring tides, very high surge levels are reached. Such superstorms are a natural threat from the sea for the inhabitants of the Belgian coastal zone. The coastal land is low-lying, with a ground level several meters below the surge level. If coastal defenses fail, flooding of the land occurs for many kilometers inland, causing property damage, human casualties and widespread devastation. The design of coastal defenses along the coastline, such as sea dikes, is based on both the characteristics of possible superstorms as the devastating effects of coastal flooding. The coastal zone of Flanders is low-lying and densely populated. So, it is an area with a high risk of damage and casualties by coastal flooding. On the one hand there are risks associated with large scale flooding of the coastal plain in case of breaches in the coastal defenses line. On the other hand there are risks for property and people situated close to the coastline especially in the coastal towns where part of the dikes are built up with apartment houses and people live in rooms with a sea view along the seaside promenade. During a storm surge overtopping occurs and waves can reach the apartment houses and in worst case scenarios serious damage and casualties may result, especially when the structural stability of the buildings on top of the sea dike is threatened. See Fig. 1 for a

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typical Belgian sea dike during modest storm conditions, with a little bit of
overtopping occurring.

Figure 1. Picture of a typical Belgian sea dike during modest storm conditions, with a
little bit of overtopping occurring.

The sea dikes in Belgian coastal towns function as parts of the chain of the
coastal defense line, but most of the time they are a recreational promenade with
high importance for the touristic sector. In superstorm conditions however surge
levels can reach 5 m above mean sea level, freeboard becomes limited to a few
meters and wind waves with a maximum individual wave height of ca. 10 m
and a wave length of ca. 100 m impact on the coastal defenses. Although a large
part of the incoming wave energy can be dissipated by a high and wide beach,
hence the execution of beach nourishments is an important measure to
strengthen the coastal defenses in the Belgian coastal towns, the sea dikes are an
essential part of the coastal defenses system. Fig. 2 shows a sketch of the typical
superstorm conditions in a Belgian coastal town.

Figure 2. Sketch of a typical Belgian coastal defense during superstorm conditions.
Wave return walls on wide-crested dikes

A horizontal distance of several tens of meters between the seaward revetment and the apartment houses on top of the dike exists in all Belgian coastal towns. These are called wide-crested sea dikes, in contrast with the typical grass dikes in rural areas that have a crest width of only a few meters. These wide-crested dikes in coastal towns are often built on former dune belts. Previous research (Verwaest et al, 2010) resulted in a semi-empirical formula to estimate the effect of the wide crest in reducing overtopping in Belgian coastal towns. Due to the crest width kinetic energy is dissipated on the crest and water on the crest can flow back towards the seaside. The reduction factor is given by Eq. 1.

\[
\frac{q}{q_0} \equiv \alpha = \sqrt{\exp(-22 \cdot \kappa \cdot \beta) - 0.21 \cdot \frac{t}{\kappa} \cdot (1 - \exp(-22 \cdot \kappa \cdot \beta))} \quad (1)
\]

with \( \alpha = 0 \) if the expression under the root is negative,
and with Eq. 2 and Eq. 3 defining the dimensionless parameters \( \kappa \) and \( \beta \) :

\[
\kappa = \frac{g \cdot n^2}{(R_u - R_r)^{1/3}} \quad (2)
\]
\[
\beta = \frac{B}{(R_u - R_r)} \quad (3)
\]

Relevant parameters are listed below.

- Crest width \( B \);
- Seaward slope of crest \( t \);
- Freeboard \( R_r \);
- Manning roughness of the promenade surface \( n \), for which a typical value is \( n = 0.02 \) s m\(^{-1/3}\);
- Run up height \( R_u \), with 2 % exceedance probability for a wave in the wave train, which can be estimated with state of the art empirical overtopping formulas in function of primarily the incoming wave characteristics wave height \( H_m \) and wave period \( T_{m-1,0} \) and the slope of the revetment (EurOtop, 2007);
- Gravity \( g = 9.81 \) m s\(^{-2}\);
- \( \frac{q}{q_0} \equiv \alpha \) is the reduction factor, defined as the ratio of the overtopping discharge \( q \), and the overtopping discharge if crest width were zero \( q_0 \).

These wide-crested dikes in Belgian coastal towns have a width of several tens of meters, which gives plenty of space to locate wave return walls without to
much hampering the daily use of the promenade. Wave return walls are an effective and efficient measure to reduce coastal flooding risks. In several coastal towns in Belgium wave return walls on top of the existing seaside promenades are designed. For the technical design consideration is given to the reduction of overtopping by the wave return wall and to the structural stability of the wave return wall impacted by overtopping waves. The structural strength and foundation of the wave return walls have to be designed taking into account the hydrodynamic loading due to the overtopping waves. In this study a high stiffness of the wave return walls is assumed, as is certainly the case for wave return walls made of concrete. Fig. 3 shows a schematized problem description.

![Figure 3. Schematized problem description.](image)

For reducing the overtopping, it is most effective to locate the wave return wall at a distance away from the seaward revetment, and to include a seaward recurve, called parapet wave wall (Van Doorslaer et al, 2010). Hydrodynamic loading on the wave return wall is expected to reduce if this distance $D$ becomes larger. A seaward recurve however might result in an increased hydrodynamic loading on the wave return wall. Apart from technical considerations it is very important also that the wave return wall is integrated in the coastal town’s environment. One aims not only to reduce the coastal flooding risks, but also to increase the attractiveness of the coastal town resulting in touristic-recreative benefits. Different alternative engineering solutions offering the prescribed level of safety are developed, but the design concept selected is based also on the requirements of the local stakeholders and investigated as part of an architectural study. An important aspect in Belgian coastal towns is the visual disturbance of a wall if its height...
exceeds 1m or so. For this reason a parapet wave wall is generally preferred, because due to the recurve a smaller wall height is needed to give the necessary overtopping reduction.

**SEMI-EMPIRICAL FORMULA**

Based on existing relations for layer thickness $h_0$ and layer speed $v_0$ of overtopping waves a semi-empirical formula is developed to deliver a design value for the hydrodynamic loading on a wave return wall for given geometric and hydraulic boundary conditions. A mathematical form of the formula is established using the relations proposed in literature for narrow-crested dikes (Schüttrumpf et al, 2005), see Eq. 4 and Eq. 5:

\[ h_0 = a \cdot (R_u - R_c) \]
\[ v_0 = b \cdot \sqrt{g \cdot \sqrt{(R_u - R_c)}} \]

in which $a$ and $b$ are constants for a given exceedance probability of waves in the wave train. Note that we have low exceedance probability values for $h_0$ and $v_0$ in mind because design load on the wall is determined by the highest waves in the wave train. One however has to bear in mind that empirical evidence is accumulating and will be more evident in future when additional wave flume research experiments measuring velocities and layer thicknesses of overtopping waves deliver results, that “constants” $a$ and $b$ are no constant values when considering widely varying geometries of dikes and/or incoming wave characteristics. For example, some recent experimental results have shown that $b$ has a noticeable variability in function of the slope of the dike (van der Meer et al, 2010). Also, it is to be expected that “constants” $a$ and $b$ will have some dependency on the shape of the incoming wave spectrum.

The momentum rate of the flowing water layer on top of the dike crest is forced to change direction and speed by the wave return wall. The proposed empirical prediction formula for the force $F_{\text{design}}$ on the wall states that the hydrodynamic loading on the wall is proportional to this momentum rate, see Eq. 6.

Substitution of the relations Eq. 4 and Eq. 5 in Eq. 6 results in the proposed semi-empirical formula Eq. 7.

\[ F_{\text{design}} = cte \cdot \rho \cdot h_0 \cdot v_0^2 \]
\[ F_{\text{design}} = \beta \cdot \rho \cdot g \cdot (R_u - R_c)^2 \]

in which $\rho$ is density and $\beta$ is a proportionality factor to be determined by empirical investigations. The proportionality factor $\beta$ is supposed to be
primarily dependent on the ratio between the height of the wave return wall $H$ and the layer thickness $h_0$, so the wall height is scaled with $(R_u - R_c)$.

Secondary influence factors on $\beta$ are the angle of the seaward recurve $\vartheta$, the seaward slope of the crest $t$ and the distance between the wave return wall and the seaward revetment $D$, which is assumed to also scale with $(R_u - R_c)$.

Although it is not a variable in practical design for Belgian sea dikes, another influence factor from theoretical point of view is the roughness of the surface of the promenade, characterized by its Manning roughness $n$. In analogy of Eq. (1) the dimensionless parameter $\kappa$ as defined by Eq. 2 is introduced. In summary, the dimensionless factor $\beta$ is proposed to be a function of five dimensionless parameters as written in Eq. 8:

$$\beta = f \{ H / (R_u - R_c), \vartheta, t, D / (R_u - R_c), g \cdot n^2 / (R_u - R_c)^{1/3} \}$$  \hspace{1cm} (8)

The effect of the last three of these parameters combined $\{ t, D / (R_u - R_c), g \cdot n^2 / (R_u - R_c)^{1/3} \}$ could possibly be estimated by using Eq. 1 which originates from a concept of a gradual decrease of velocity of the overtopping water mass when propagating over the wide crest. Because momentum rate is proportional to the square of the velocity, one then proposes Eq. 9.

$$\beta = f \{ H / (R_u - R_c), \vartheta \} \cdot \alpha^2$$  \hspace{1cm} (9)

WAVE FLUME EXPERIMENTS

Using experimental results of scale models in wave flumes the empirical parameters of the semi-empirical formula are to be calibrated and validated for ranges of applicability. By convention the “design” load is defined as the extreme value for which the probability of exceedance during a storm surge peak with duration of 3000 waves is 10%.

A small series of laboratory experiments with varying values of $H / (R_u - R_c)$ was carried out in WL Delft Hydraulics for some relevant Dutch configurations (Den Heijer, 1998). See Fig. 4 for the set-up.
In these experiments the dimensionless wall distance $D/(R_u - R_c)$ was varied in the range 0.8 to 1.5 and crest slope and recurve angle were zero. This small set of experiments reveals an interesting dependency of $\beta$ on the dimensionless wall height $H/(R_u - R_c)$ as shown on the Fig. 5.

Figure 5. Experimental results of Den Heijer (1998) showing a dependency of the proportionality factor $\beta$ on the dimensionless wall height $H/(R_u - R_c)$ (with $\vartheta = 0^\circ; t = 0\%; D/(R_u - R_c) = 1$).
One observes from Fig. 5 increasing values of $\beta$ for increasing values of dimensionless wave height $H/(R_u - R_c)$ until a maximum value is attained. This maximum $\beta \approx 0.3$ is a constant for $H/(R_u - R_c) > 0.6$. A physical explanation can be given for this behavior: when the wall height is smaller than the overtopping water layer the hydrodynamic loading is only a fraction of the total momentum rate namely proportional to this wall height, but when the wall height is larger than the water layer the total momentum rate is impacting the wall so there is no dependency anymore on the wall height.

One can think of the effect of the recurve as a way to increase the “effective height” of the wall. A recurve makes the wave wall more effective to reduce overtopping, but at the same time one expects the loading will increase. To estimate the increased loading due to a recurve one can reason as if the wall height were higher.

From these results and considerations the mathematical form for the proportionality factor $\beta$ is proposed to be as in Eq. 10:

$$\beta = \min \left[ c_1, c_2 \cdot \left( \frac{H \cdot f(\vartheta)}{R_u - R_c} \right)^{c_3} \right] \cdot \alpha^2$$

in which $c_1$, $c_2$ and $c_3$ are dimensionless constants, and $f(\vartheta) = 1$ for $\vartheta = 0^\circ$, and $\alpha$ from Eq. 1 with $B = D$.

**CONCLUSION AND OUTLOOK**

A semi-empirical formula is proposed to determine a design value for hydrodynamic loading of a wave return wall on top of a sea dike. The formula describes the influence of the hydraulic boundary conditions with only one parameter $(R_u - R_c)$, and the influence of the geometry of the crest with a set of five parameters $\{f, D, n, H, \vartheta\}$. A set of three calibration constants needs to be determined experimentally.

Execution of an extensive program of wave flume experiments is needed to calibrate and validate the proposed semi-empirical formula. The approach to follow for reaching practical applicability of the semi-empirical formulae is to limit variability of hydraulic boundary conditions and geometrical parameters focusing on values within ranges typically occurring for a given coastal area.

Typical characteristics for Belgian coastal towns are a smooth dike with a relatively steep slope of 1:2, a very shallow foreshore with a water depth at the toe of the dike of less than 2 m, incoming wave characteristics in superstorm conditions very much related to this water depth (with an important part of wave energy inside long waves generated by breaking of waves on the beach), a freeboard of 0.5 to 3 m, a smooth and wide crest of several tens of meters, a
seaward slope of the crest of 1 to 2 %, a wave return wall with a height of 0.6 to 1.2 m, with or without a recurve.
Future experiments are envisaged in the 4 m wide wave flume at Flanders
Hydraulics Research in which measurements of run-up and hydrodynamic
loading can be executed simultaneously by separating the wide flume into two
test sections. Typical configurations for Belgian coastal towns will be scaled
down 1/20. Each overtopping experiment with irregular waves will consists of
a series of at least thousand waves. The loading on the wall caused by the
impact of the overtopping waves will be determined by load cells as well as
pressure sensors, distributed over the surface of the wall. Load and pressure
time series will be measured with a high sample frequency (~1 kHz), to be able
to investigate peak values of very short duration.

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Coastal defenses
Coastal safety
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