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Distributions for wave overtopping parameters for stress strength analyses on flood embankments

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ABSTRACT

A process based assessment of the probability of failure of a flood embankment, as well as an assessment of the consequences of failure of an embankment require insights into the stresses on the landside slope of an embankment. These assessments are hindered by the empirical nature of the wave overtopping parameters. Failure initiation is often linked to an allowable mean overtopping discharge which forms the input for the overtopping volumes distribution. The high level of uncertainty associated with predicting the mean overtopping discharge therefore leads to high levels of uncertainty in predicting wave overtopping volumes. The mean overtopping discharge is thereby not directly related to run-up parameters. This paper addresses these issues by presenting new distributions for the velocity, discharge, depth, volume, and shear stresses at the crest for those waves that overtop which have been derived from the wave run-up parameters. The proposed distributions are independent on the mean wave overtopping discharge and the large inaccuracies associated with predicting this. The proposed method has the added benefit of being able to express overtopping parameters in terms of each other. The paper also provides a method for determining the change in these random overtopping values along the landside slope, thereby facilitating a direct comparison between wave overtopping events and overflow events.

1. Introduction

Decisions on investments in flood protection are based on flood risk analysis (Agency, 2009, Gouldby et al., 2010, Hall et al., 2003, Kuijken, 2015). The risk of an embankment failing is determined by the probability of the embankment failing multiplied by the consequences of this failure. The probability of failure usually follows from a stress strength analysis. Failure is thereby defined as the moment whereby the embankment is no longer able to fulfill its design function. The consequences of failure are amongst others a function of the rate at which an embankment breaches. Embankment breach models have been developed to assess damage formation, and hence the residual strength of embankments as a function of loading due to overflow (Macchione, 2008, Singh et al., 1988, Wu, 2013, Zhu, 2006), but few methods have yet been developed to assess the damage formation of embankments due to wave overtopping (D'Eliso, 2007).

An accurate assessment of the probability of failure due to overflow or wave-overtopping, starts with a detailed assessment of the stresses exerted on the embankment in relation to the strength of the embankment. The probability of failure of an embankment under overflow has been studied in detail. For example, for a grass covered embankment, the initiation of failure has been given by relating the stresses on the grass cover to the change in strength of a grass cover with time (Dean et al., Jan, 2010, Hughes, May, 2011). However in the case of wave overtopping a more empirical approach is used whereby a maximum allowable mean overtopping discharge is given which has been determined experimentally. The scale parameter of the Weibull distribution used for describing the overtopping volumes is thereby a function of the mean overtopping discharge.

On the other side of the spectrum research has been performed to the wave run-up heights, velocity, depth and discharge. Van Gent (2002) attempted to relate the wave overtopping volume to wave run-up parameters but did not compare the results with the overtopping volumes that follow from the Weibull distributions. This paper relates the overtopping volume to wave run-up parameters like the run-up depth, height, and velocity. The outcome could be used to relate the overtopping volumes to stress parameters, making a process based stress strength analysis possible.

The analysis presented in this paper works from the assumption that the wave overtopping volume and mean overtopping discharge are directly related to wave run-up parameters like the run-up depth, height, and velocity. It thereby assumes that the incoming waves are Rayleigh distributed. Where possible the newly derived distributions

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have been validated against data, among which the CLASH database (Steendam et al., 2004) which consists of a collection of outcomes of wave overtopping experiments. Probability distributions for the peak velocity, depth, discharge, overtopping volume and shear stress are derived in Section 3. The means by which each of these parameters has been determined are discussed in Section 2 whereby in Section 2.6 methods are discussed for converting the peak shear stress at the crest to peak shear stresses at any point along the landside slope.

2. Wave run-up parameters

The run-up discharge [m^3/s/m], velocities [m/s], depths [m], and hence volume [m^3/m], and shear stresses [N/m^2] are related to the wave run-up height (Schüttrumpf and Oumeraci, 2005). The run-up height is here defined as the vertical distance above the still water line over which a wave travels up the assumed infinitely long waterside slope. Hunt (1959) related the run-up height of an overtopping regular wave to the wave height according to

$$R = \frac{\tan\alpha H}{\sqrt{\frac{2\pi H}{gT^2}}} \tag{1}$$

where R is the wave run-up height measured in the $\hat{z}$ direction and α is the waterside slope angle of the embankment (see Fig. 1). Furthermore H [m] is the deterministic wave height of a regular wave, g [m/s^2] the gravitational constant, and T [s] the deterministic wave period. The wave run-up height, velocity, and discharge are given for a specific location (x^*, z) on the waterside slope, where x^* and z are respectively the distance in the horizontal $\hat{x}$ direction, and the vertical $\hat{z}$ direction relative to the interface of the Still Water Level (*SWL*). As a wave runs up the waterside slope it may reach the point (x_R, R), measured relative to the intersection of the waterside slope with the still water level.

Battjes (1974) showed that for deep foreshores, the wave run-up height for relatively smooth slopes is Rayleigh distributed. Battjes (1974) adapted the Hunt run-up formula to irregular waves, to arrive at a new methodology to be used in the Netherlands for calculating the 2% wave run-up heights for plunging breakers. This equation was later incorporated in the current standard in respectively the Netherlands and Europe, the TAW2002 (der Meer. W. J., 2002) and the EurOtop manual (EurOtop, 2007), as

$$\frac{R_{2\%}}{H_s} = \min\left\{1.65\gamma_b\gamma_f\gamma_\beta\xi_{m-1,0}, \gamma_f\gamma_\beta\left(4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}\right)\right\} \tag{2}$$

Here $R_{2\%}$ refers to the run-up height exceeded by 2% of Rayleigh distributed incoming waves, H_s [m] is the significant wave height, $\xi_{m-1,0}$ is the breaker parameter $\frac{\tan\alpha}{\sqrt{H_s/L_0}}$ where α is the waterside slope angle, and L_0 [m] is the deep water wave length given by $\frac{gT_{m-1,0}^2}{2\pi}$. Here $T_{m-1,0}$ [s] refers to the spectral period. Furthermore γ_b is a factor to account for the effects of a berm, γ_f is a factor to account for the roughness on the slope, and γ_β is a factor to account for the angle of incipient wave attack.

Relationships have been developed for the wave run-up velocities, depths, discharges, and volumes based on the run-up height R exceeded by $n\%$ of the waves, whereby n is often set at 2. The 2% run-up level alone however does not necessarily provide sufficient information on whether the scale parameter of the Rayleigh distribution is a function of normalized parameters, or a constant. Hence for the purpose of the analysis outlined in this paper the Rayleigh distribution has been fitted against wave run-up data for several exceedance probabilities to determine how the scale parameter changes for different exceedance probabilities (see Fig. 2). The data was made available by Van Steeg (2015). The normalized run-up height was found to approximate a Rayleigh distribution with scale parameter 0.658, which results in the generic expression for the wave run-up height given by

$$\frac{R_{n\%}}{\epsilon H_s} = 0.93(-\ln(P))^{\frac{1}{2}} \tag{3}$$

for

$$\epsilon = \min\left\{\xi_{m-1,0}\gamma_b\gamma_\beta\gamma_f, \frac{\gamma_f\gamma_\beta}{1.65}\left(4.0 - \frac{1.5}{\sqrt{\xi_{m-1,0}}}\right)\right\} \tag{4}$$

Here subscript n refers to the percentage of waves exceeding the run-up height R [m], and P refers to the probability that run-up exceeds the level represented by $R_{n\%}$ [m]. Based on the Rayleigh distribution for the normalized wave run-up height, relationships have been developed for the normalized wave run-up velocity, run-up depth, run-up and overtopping discharge, and run-up volume. A relationship for the run-up shear stress has also been developed using the run-up velocity and run-up depth. In the following sections each of these parameters will be discussed starting with the normalized wave run-up velocity.

2.1. Wave run-up velocity

The wave run-up velocity is defined as the peak velocity on the waterside slope measured at an arbitrary height z above the still water level during a single run-up event. The run-up velocity denoted by $u_{n\%}$ [m/s] refers to the peak run-up velocity exceeded by $n\%$ of the incoming waves. As a wave runs up a slope, kinetic energy is transferred into potential energy and dissipated due to friction and turbulence. The run-up height is therefore related to the

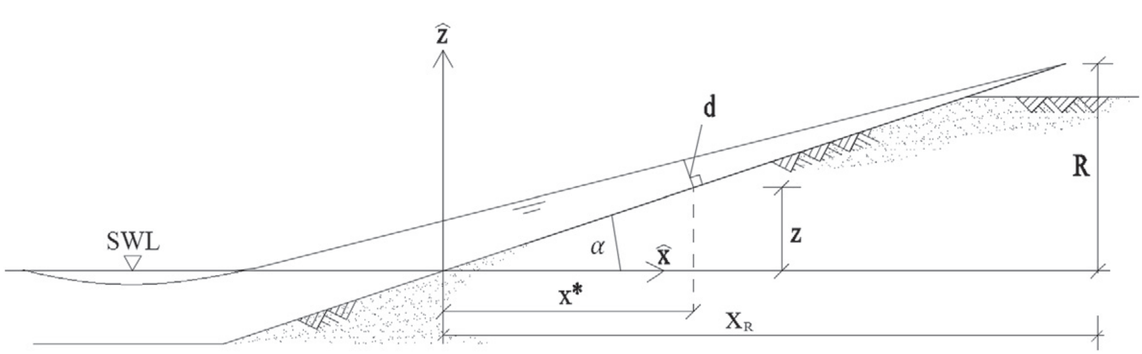

Fig. 1. Definitions used in describing wave run-up (for explanations parameters see text).