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Investigation of uplift impact forces on a vertical wall with an overhanging horizontal cantilever slab

Dogan Kisacik [a,*], Peter Troch [b], Philippe Van Bogaert [b], Robby Caspeele [c]

[a] Institute of Marine Sciences and Technology, Dokuz Eylül University, Haydar Aliyev Boulevard 100, 35430 Inciralti, Izmir, Turkey
[b] Department of Civil Engineering, Ghent University, Technologiepark 904, B-9052 Zwijnaarde, Belgium
[c] Department of Structural Engineering, Ghent University, Technologiepark 904, B-9052 Zwijnaarde, Belgium

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ABSTRACT

A vertical wall with an overhanging horizontal cantilever slab is tested in a small scale test set-up under wave impact (impulsive) loads. Tested waves are regular and irregular. A single approaching wave creates two individual impacts which occur sequentially on the wall and slab of the structure. Results from wave impact tests are used to analyze the uplift impact forces under breaking waves. Then, a set of parameters which is responsible for the uplift impact forces is investigated under regular waves and results are compared with the results of irregular wave tests. Incident wave height at the toe of the foreshore (H_1), water depth at the structure toe (hs) and incident wave period (T) are the main parameters governing the uplift impact forces. In addition, the influence of different geometric properties, described by dimensionless ratios like c'/h_s (c' is the clearance between still water level and slab of the structure) should be taken into account. Based on the experimental investigations on breaking wave kinematics and impact loads, a new prediction model for uplift impact forces, occurring on the overhanging horizontal cantilever slab, has been derived as a function of the rise time t_r. t_r is the time duration between points t_1 and t_2, see Fig. 2. The formula is tested within the range $0.45 \leq H_1/h_s \leq 1.2$, $2.0\ \mathrm{s} \leq T \leq 2.8\ \mathrm{s}$ and $0.75\mathrm{m} \leq h_s \leq 1.65\mathrm{m}$. The formula is developed based on the scaled test results. Therefore, a correction of the formula is required for the prototype dimension. In addition, the formula considers the maximum uplift impact forces which are more intense but shorter lasting. Thus, the structural response of the cantilever should be taken into account in design process. This paper is a completion of previously published works (Kisacik et al., 2012a and Kisacik et al., 2012b) on this type of structure.

1. Introduction

Vertical breakwaters and sea walls are frequently used as coastal protection structures against the storm waves and high water levels. In view of this, controlling the overtopping of the waves for the vertical breakwaters is a critical issue. This is why engineers/designers tend to provide the vertical breakwaters with a return crown wall or a completely horizontal cantilever slab to reduce the overtopping. However, upward impact beneath the horizontal cantilever slab results in a significant uplift force. These forces are impact loads and they should not be substituted by a static equivalent. Therefore, a detailed description of the space and time distribution of the wave impacts becomes imperative.

On vertical structures, the front shape of the breaking wave has a significant consequence on the wave pressure and force. According to the breaker shape, four main breaker types are suggested. These are (a) slightly breaking waves (SBW), (b) breaking waves with small air trap (BWSAT), (c) breaking waves with large air trap (BWLAT) and (d) broken waves (BW) (Kisacik et al., 2012a). Wave loads on vertical structures may vary between slowly-acting 'quasi-static' (or 'pulsating') loads, and more intense but shorter lasting 'impact' (or 'impulsive') loads. Mainly SBW or BW result in quasi-static loads and breaking waves (BWSAT and BWLAT) result in impact loads which generally introduce localized damages. Coastal structures are bulk structures and most researchers did not consider these short-duration impact loads in their design formulas. However, Oumeraci and Kortenhaus (1994) emphasize the importance of impact loads in the design of vertical structures. In the literature, several formulas from design codes allow calculating impact loads on vertical structures. Minikin (1963) suggests a parabolic pressure distribution for the breaking waves on vertical walls. He used field measurements obtained by De Rouville et al. (1938) to calibrate his relationship for wave loads. However, there are some incompatibilities found between different versions of Minikin formula which are mainly due to a unit mistake converting from British units to metric units. Therefore, Minikin formula is out of fashion in the recent years (Bullock et al., 2004). Goda (1974) proposes his own formula for the wave loads on the vertical walls based on theoretical

* Corresponding author. Tel.: +90 232 2785565/108; fax: +90 232 2785082.
E-mail addresses: dogan.kisacik@deu.edu.tr (D. Kisacik), Peter.Troch@UGent.be (P. Troch), Philippe.VanBogaert@UGent.be (P. Van Bogaert), Robby.Caspeele@UGent.be (R. Caspeele).

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and laboratory works. For impact loads, his formula was subsequently extended with the incident wave direction, modification factors applicable to other types of vertical walls and the impulsive pressure coefficient (Takahashi et al., 1994). Blackmore and Hewson (1984) suggest another formula based on full-scale field measurements. They consider the effect of entrained air which results in a reduction in the impact pressure of field tests compared to laboratory tests. Allsop and Vicinanza (1996) recommend a prediction formula for horizontal wave impact force on the vertical walls according to the model tests at HR Wallingford. Within PROVERBS (an EU project to develop and implement probability based tools for an integrated design of vertical breakwaters) a prediction method for horizontal impact force has been developed based on large data sets that include small and large scale physical tests and field measurements (Oumeraci at al., 2001). Within the framework of the PROVERBS, Oumeraci et al. (2001) gave guidelines for assessment of horizontal impact forces on seawalls under breaking waves. The application of the PROVERBS methodology is nevertheless complex and may still lead to significant scatter in predictions of wave impact loads even under relatively similar design conditions. Recently, Cuomo et al. (2010a) present prediction formulas for both quasi-static and impact loads on vertical face coastal structures (such as seawalls and caissons breakwaters) based on experimental work carried out in the CIEM/LIM large flume at Barcelona within the framework of the VOWS (Violent Overtopping by Waves at Seawalls) project.

In addition, many researches have been conducted to evaluate wave loads on horizontal platforms due to slamming of approaching wave crests. Kaplan and Silbert (1976), Kaplan (1979, 1992) and Kaplan et al. (1995) investigated wave forces on flat decks and horizontal beams on offshore platforms. They developed a semi-analytical model for the evaluation of time history wave loads on horizontal members. Shih and Anastasiou (1992) and Toumazis et al. (1989) analyzed wave-induced forces and pressures on horizontal platform decks at small scales. Bea et al. (2001) have proposed a semiempirical method which is accounting for the dynamic amplification of slamming due to dynamic response of structural elements. Tirindelli et al. (2002) and McConnell et al. (2003, 2004) measured wave loads on deck and beam elements in a series of 2-dimensional physical model tests. These measurements were analyzed to explore the process of wave loading, with the objective of developing improved predictions. Then, Cuomo et al. (2003) and Cuomo (2005) developed a new method for the analysis of non-stationary time-history loads which is based on wavelet transform.

Contrary to the previous vertical structures or horizontal decks, structures consisting of both vertical parapets and horizontal cantilevering slabs have scarcely been considered. Recently, Kisacik et al. (2012a) described the loading conditions due to violent wave impacts on a vertical wall with an overhanging horizontal cantilever slab, where tested in a small scale test set-up (with a scale factor of 1:20) under loading conditions of breaking and non-breaking waves. Furthermore, Kisacik et al. (2012b) analyzed the pressure distribution on the same cross-section of the vertical structure with an overhanging horizontal cantilever slab. They proposed an expression for the location of maximum pressures p_{max} on the vertical part as a function of the wave steepness.

However, the magnitudes of uplift (vertical) impact forces due to breaking waves on a vertical wall with an overhanging horizontal cantilever slab are still missing a prediction formula. In this paper, based on structure geometry and wave conditions, a set of basic parameters (and certain combination of these), governing the prediction of the wave loading on a vertical wall with an overhanging horizontal cantilever slab, is presented. The parametric investigation is carried out as a series of small-scale model tests which is described in Kisacik et al. (2012a). The results from the regular and irregular waves are compared and discussed. Finally, an empirical model is suggested to predict uplift impact forces on the slab as a function of the rise time (t_r) under breaking conditions of regular waves.

2. Experimental set-up and data acquisition

Physical model tests have been carried out in the wave flume (30 m × 1 m × 1.2 m) of Ghent University in Belgium (see Fig. 1). The structure is a combination of a vertical wall and an overhanging horizontal cantilever slab. For the sake of simplicity, we use terms wall and slab instead of vertical wall and overhanging horizontal cantilever slab, respectively, in the entire text. All the events are recorded by a high speed camera and pressures are measured with 10 Quartz pressure sensors using a 20 kHz sampling frequency. The main parameters used in the parameter map for the determination of the wave loading on a structure are the incident wave height (H), water depth at the structure toe (h_s), incident wave period (T), rise time (t_r) and the geometrical parameters of the structure. The incident wave heights are experimentally measured at wave gauges from 1 to 8 as H_1 to H_8, while L is determined by linear wave theory for any depth. The foreshore slope is 1/20. The geometric parameters l_m, h_m and c' (structure width and height and clearance between SWL and slab, respectively) are shown in Fig. 1, where, $h_m = 0.3$m and $l_m = 0.6$m.

Tests are conducted using regular and irregular waves for four different values of h_s and five different values of T (see parameter matrix in Table 1). For each combination of h_s and T, the wave height (H) has been increased in successive tests to achieve the range from non-breaking to broken waves. For the comparison of parameters such as T and h_s, wave induced force results include impact loadings since breaking waves (BWSAT and BWLAT breaker types) are used.

The horizontal and uplift forces on the structure are calculated by integrating the pressure results from sensors located on the wall and slab, respectively (see Kisacik et al., 2012a). The shape of the force signal at the wall per impact is defined with a nick name called "church roof". Fig. 2 shows a definition sketch, which is similar to Bullock et al. (2007), to determine the rise time t_r, maximum impact and quasi-static forces (F_{max_imp} and F_{max_qs}). F_{max_imp} is the maximum dynamic force caused by the impact of a single breaking wave on the wall. However, F_{max_qs} is the maximum quasi-static force due to the downward acceleration of the water body for a single wave which first rises on the wall after the impact and then free falls. F_{max_imp} and F_{max_qs} are considered as relative forces to the calm condition. The impact forces are defined at a threshold value corresponding to $F_{max\_imp}/F_{max\_qs} \geq 2.5$. This criterion for defining an impact force region is suggested by Kortenhaus and Oumeraci (1998).

In general, the maximum force F_{max} for breaking wave is equal to maximum impact force F_{max_imp} and F_{max_qs} follows it (see Fig. 2). Rise time t_r is the time duration between points t_1 and t_2 which respectively show the instant of impact start and of maximum dynamic force. The t_1 is defined as the start point where the wave-induced force rose above the noise level. Time from t_1 to t_3 shows the duration of impact force, while the time from t_3 to t_5 shows the duration of quasi-static force. The value of F_{max_qs} is measured after the dynamic portion is eliminated by a low pass filter. Data editing is carefully considered for the extreme values in the data clouds. In this study, F_{max_imp} and F_{max_qs} and t_r values are determined per impact.

The use of 20 kHz sampling frequency with space resolution varying in the range of 1.5–3 cm will provide a false impression regarding the ability to catch the spatial details of the process. To check the above condition, Lamberti et al. (2011) suggested a practical rule which says that let the acquisition time interval be equal to or greater than transit time among sensors. The transit times among sensors are around 0.0001 s which are close values to the acquisition time interval with sufficient analog low pass filter of 2.7 kHz.

More details about experimental set-up, scaled model and instrumentation are given in Kisacik et al. (2012a, 2012b).

3. Parametric analysis of the impact forces

The incident waves create two individual impacts which occur sequentially on the wall and slab. Due to these impacts, the structure