ADAPTATION WORKS TO AVOID THE FLOODING OF PIAZZA SAN MARCO (VENICE): PHYSICAL MODEL TESTS TO EVALUATE OVERTOPPING DISCHARGE

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According to the management of the Mo.S.E. system, the water level in the Venetian lagoon is maintained below a certain threshold, that however does not guarantee the complete defense of the main Piazza. Flooding of the Piazza is presently tolerated, although limitedly to a minor extent, and can/will be avoided only thanks to additional adaptation works. One of the possible flooding mechanisms is the wave overtopping, and this note investigates the efficiency, as possible mitigation option, of a small temporary barrier placed along the S. Marco quay.

Keywords: Venetian lagoon; wave overtopping; experimental investigation

INTRODUCTION

The city of Venice is recurrently affected by flooding caused by high-tide events. A monotonic increase of the frequency of these events has been recently observed (Lionello, 2012), caused by sea level rise and by subsidence, that severely affects the Venetian coast (Tosi et al. 2013). In order to cope with these phenomena, the well-known Mose (or Mo.S.E., MOdulo Sperimentale Elettromeccanico, www.mosevenezia.eu) system is designed aiming to keep the water level below a certain value, thus preserving the Venetian lagoon, its towns, the cultural heritage, the unique environment, the people mobility and the economic activities.

The design water level was chosen as a compromise to achieve safety with a limited number of closures per year, thus allowing for a sufficient water circulation in the lagoon. To anticipate the predicted wind set-up and hydrologic runoff (Rinaldo et al. 2008), the Mose gates close when the mean water level at the inlets is much below the design value. However, the system cannot avoid the flooding of Piazza S. Marco, since its floor has a very low level and additional works are necessary.

Currently, inundation mainly occurs due to backflow through sewer drains or manholes. The lowest parts of Piazza S. Marco have an elevation of 87 cm CD, and during high tide, the square is flooded. The flooding will be completely avoided by a series of adaptation works aiming at mitigating the return flow through the existing drainage system, filtration (Ceccato et al. 2021), overflow trough the boundaries and wave overtopping from S. Marco Basin. An option for the mitigation of latter mechanism is the topic of this study.

Favaretto et al. (2020) performed a numerical analysis to evaluate the wind set-ups and the wind waves in front of the S. Marco square, in a scenario that considers the Mose system fully operational. The authors found that a wind with velocity of 25 m/s, occurring approximately once every 10 years, blowing from South-East, generates waves up to 50 cm. Such waves would be able to overtop the quay (named "Riva San Marco", Fig. 1) and cause a severe flooding of Piazza S. Marco.

In the literature, the wave overtopping is widely investigated since it is an essential design constraint for coastal and maritime structures. Some simple predictive tools are the Goda's design charts (Goda, 2010), the "Coastal Engineering Manual" (USACE, 2002), the EurOtop 2018 manual (van der Meer et al. 2018) and the Neural Networks developed by Van Gent et al. (2004) and by Formentin et al. (2017). However, the evaluation of the overtopping discharge at the S. Marco quay with such tools is not appropriate, since the complex topography of the site lays outside their range of applicability: in particular, the Riva San Marco is characterized by a natural stone pavement mildly sloping toward the vertical wall or, in some parts, a wall with descending steps in front, i.e. with a geometry that has never been investigated.

The expected overtopping discharge was studied in the laboratory by the Authors (Ruol et al., 2020) in the absence of any mitigation measure. The experiments showed that, under waves higher than 50 cm (an event that is expected to occur a few times per year), the overtopping is significant for all the tested water levels. Even for the more frequent case of waves of the order of 40 cm, the overtopping is critical and some mitigation measure is required since the discharge alone exceeds the limits of the drainage system envisaged for the rainfall.

Note that adaptation works must cope with historic and architectural constraints that will not be presented or discussed. A possible solution is the building of a small (temporary) barrier, vertical and

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0.4 m high, placed along the critical stretches of the quay. The purpose of the work is to investigate the feasibility of this solution with respect to the efficiency and the wave loads applied to the barrier through new laboratory tests that integrate the previous study.

This paper is organized as follows: next section describes the investigated area, section 3 describes the experimental investigation set-up and section 4 presents the results, discussing the effect of the small temporary barrier. Eventually, the conclusions are drawn.

INVESTIGATED AREA

The Venetian lagoon is a shallow water body of approximately 550 km^2, characterized by several small islands, tidal flats, marshes and channels. The lagoon is connected to the Northern Adriatic Sea through three inlets, from North to South: Lido, Malamocco and Chioggia, where the well-known Mose system is present.

Bora, from North-East, and Scirocco from South-East are the two prevailing wind regimes (Ruol et al. 2018). The local tidal regime is about 1.0 m and storm surges are a critical additional contribution. Recently, two extreme events occurred in Venice: on October 29th, 2018, the maximum level was 1.56 m CD; on November 12th, 2019, the water level reached +1.87 m CD, the second-highest recorded level since 1872, due to an unlucky combination of an astronomical tidal peak and a severe storm surge (~ 1.31 m).

The Piazza S. Marco (Figure 1) is the main square of Venice, built in the ninth century and it is part of an island surrounded by channels. The quay connected to the basin, where wind-generated waves can induce overtopping, is named Riva S. Marco. The quay is characterized by a pavement, formed by masonry blocks named "masegni", mildly sloping toward the lagoon. In some parts, the quay has 5 descending steps in front, and in some other parts, it is just a vertical wall. In front of the "Marciana" Library, an area with very low elevation is present.
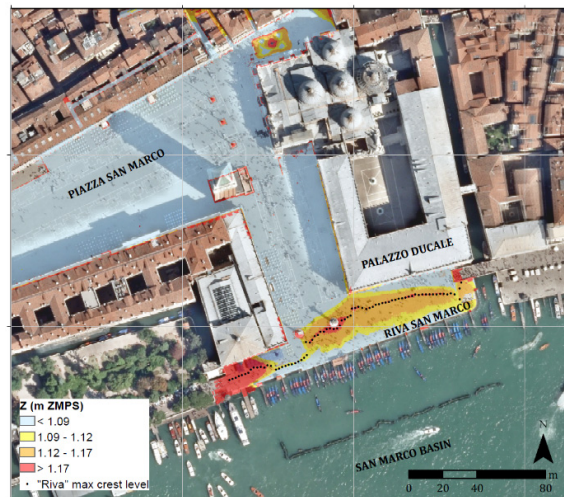

Figure 1. "Piazza San Marco" Venice, plan view: the area in blue is below 1.10 m, i.e. the design (maximum expected) water level when Mose. is closed

METHODS

Physical model tests are carried out in the wave flume (2D) of Padova University (www.dicea.unipd.it/en/ services/laboratories/maritime-laboratory). The flume is 36.00 m long, 1.00 m wide, the maximum depth is 1.30 m, and it is equipped with a dual piston-flap type wavemaker, capable of generating regular and irregular waves, with active wave absorption.

Tests are carried out in geometrical scale 1:5, according to Froude similarity. In the following, prototype dimensions are given. The tested wave states are characterized by significant wave heights H_s variable between 0.2 m and 0.5 m and two wave steepness, namely H_s/L 0.048 and 0.063 (i.e. peak period Tp variable between 1.5 s and 3.5 s). Three different water levels (WL) are considered: +0.90 m, +1.00 m and +1.10 m, i.e. the maximum expected water level when the Mose. is closed, with reference to the local chart datum (0.23 m below m.s.l.).

Ruol et al. (2020), in similar tests carried out in the absence of any mitigation measure, showed that the critical stretch of the quay is placed in proximity of the "Marciana" library. In this area, with the highest water level (1.1 m CD), almost all the tested wave states caused overtopping. This stretch is characterized by a vertical wall with a crest height of 1.10 m CD, distant 5.0 m from the edge of the quay. The representative cross-section (configuration A) tested during the experimental investigation is presented in Figure 2.

For the new tests, a vertical wall, 0.4 m high, was placed as barrier against overtopping in two locations:

- Configuration A1: barrier placed at the edge of the quay, with crest freeboard at 1.40 m (Figure 3 left);
- Configuration A2: barrier placed 2.5 m inward the edge of the quay, with crest freeboard that is slightly higher, at 1.45 m (Figure 3 right), due to the slight slope of the Piazza pavement.

The bottom of the model is fixed and non-erodible and reproduces a simplified bathymetry. The bed is horizontal, except for the zone in front of the structure that has a slope of 1:10.

Incident and reflected waves are measured by means of 8 wave gauges using the method described in Zelt et al. (1993). The measurement system includes also a flowmeter to measure overtopping discharge, and 3 load cells to measure the force (at 600 Hz) applied to the barrier for configuration A2, which cannot be predicted through simple analytical formulas.
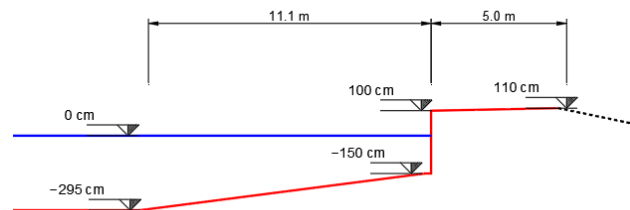

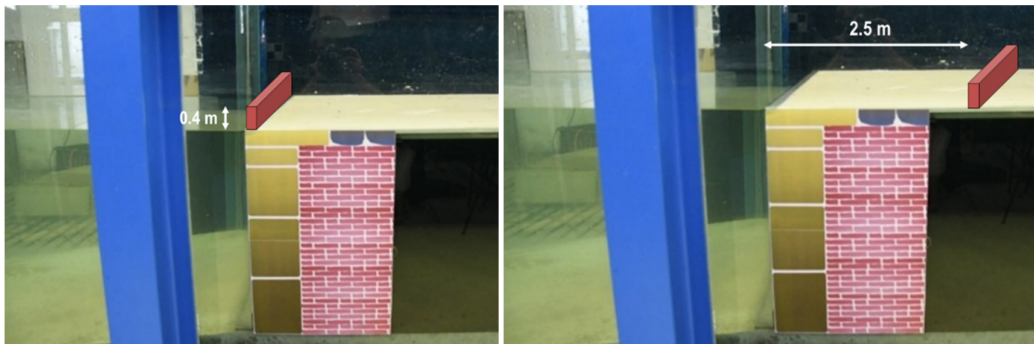**Figure 2. Geometry of the tested cross-section (Configuration A)**

Figure 3. Side view of Configuration A1 (left), Side view of Configuration A2 (right)

RESULTS

The measured overtopping discharge for Configurations A1 and A2, relative to a cross section with vertical wall and a temporary defense barrier located at different positions, will be hereafter compared to the results presented in Ruol et al. (2020) for Configuration A, that is the same cross section, in the absence of any barrier.

For Configuration A, the discharge was found to exceed the limits of the drainage system envisaged for the adaptation works, and a significant reduction of the overtopping discharge was considered necessary.

The pictures in Figures 4 and 5 show the qualitative behavior of the small temporary barrier under Hs= 0.5 m for the highest water level, WL=1.1 m. The pictures show that Configuration A2 seems more effective, since the overtopping volume in the rightmost panel appears much lower in Figure 5 rather than in Figure 4. Furthermore, a large impact behind the wall for Configuration A1 can be observed

(strong jets are visible in Figure 4), that may be critical both for the wall and for the historical pavement (named "Masegni").

It can also be predicted that, for configuration A1, the reflection coefficient k_R (ratio between reflected and incident Hs) is high, since for all water levels the waves are reflected by a vertical wall, formed by the quay and the barrier. For configuration A2, k_R was found slightly lower for the highest water levels, for which the quay is partially submerged.
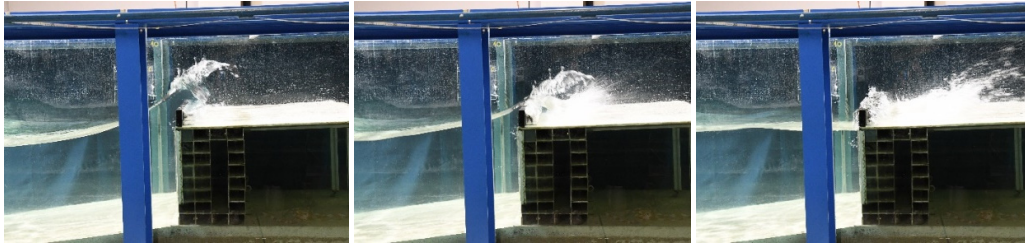

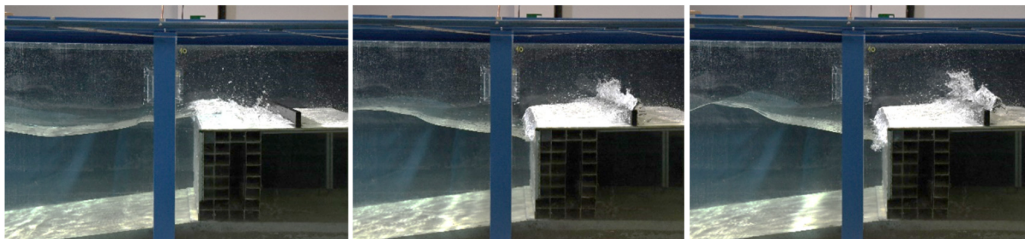**Figure 4. Snapshot of a wave approaching Conf. A1 (Hs = 0.5 m and WL = 1.10 m)**

Figure 5. Snapshot of a wave approaching Conf. A2 (Hs = 0.5 m and WL = 1.10 m)

Results are given in non-dimensional form, in terms of wave overtopping (Figures 6 and 7) and wave loads (Figure 8). The R_c/H_s is a commonly used abscissa in overtopping design graphs, where R_c is the crest freeboard and the main design parameter. However, in the following, H_s/R_c is used, being in our opinion more suited to describe experimental results (avoiding spurious correlation due to the division of the data by a common factor), and it is also appropriate for typical graphs describing the wave loads.

Figure 7 shows the comparison between the mitigated and non-mitigated scenarios. As expected, the barrier is effective in reducing the wave overtopping compared to the present condition (configuration A). The most attenuated discharge is relative to configuration A2 (it is of order 1/6), but also configuration A1 performs well (with a reduction of order 1/3).

Figure 8 shows, for the configuration A2, the dimensionless load applied to the wall $F/(\rho g H_s d)$ where d is the wall height. Some influence of the wave steepness is present (points with similar target H_s/Rc have different steepness) not shown for brevity. Wave steepness is proportional to the orbital acceleration and hence some dependence has to be expected.

In Figure 8 (right), the dimensionless approach proposed by Van Doorslaer et al. (2017) are used. The authors analyze the non-breaking wave load F divided by $(\rho g Rc^2)$, applied to a storm wall placed at the end of a promenade over a smooth sloping dike. In the presence of a promenade and a barrier (as for Configuration A2), the overtopped waves progress over the promenade with a certain flow depth h and flow velocity U, eventually impacting on the wall and overtopping it. The authors suggest that a first 'guestimate' of the impact forces may be found by the correlation between the individual dimensionless load and the individual dimensionless discharge $(U{\cdot}h / \sqrt{gRc^3})$. Although in Conf. A2 we do not have the direct measurements of U and h before the barrier, but rather of the discharge overtopping it, we could derive the necessary discharge through the tests in Conf. A. Figure 8 (right) plots such data and it can be seen that the correlation depends on the water level over the promenade, that characterizes the approaching velocity. Obviously, the correlation will necessarily depend also on other geometrical parameters, constant in these tests, such as the height of the barrier itself.
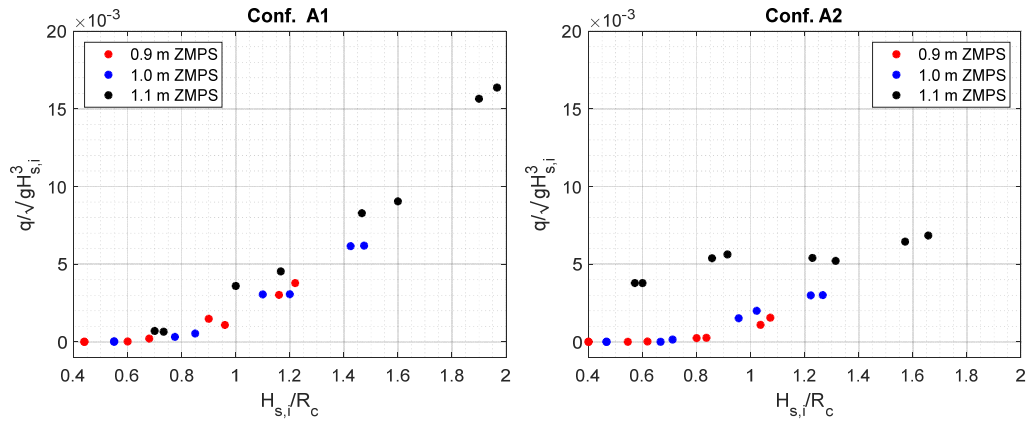
Figure 6. Wave overtopping results for Configuration A1 (left) and for Configuration A2 (right)
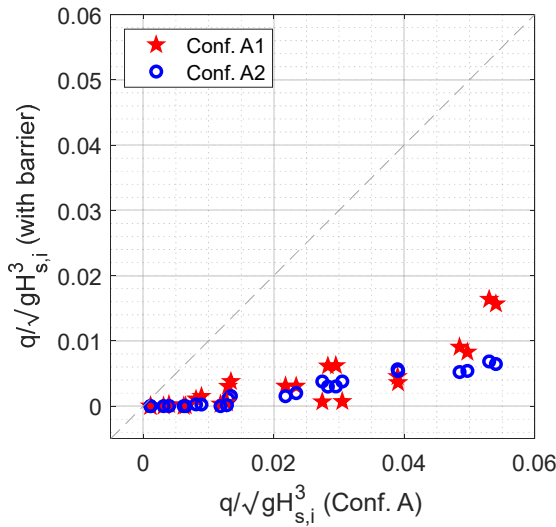

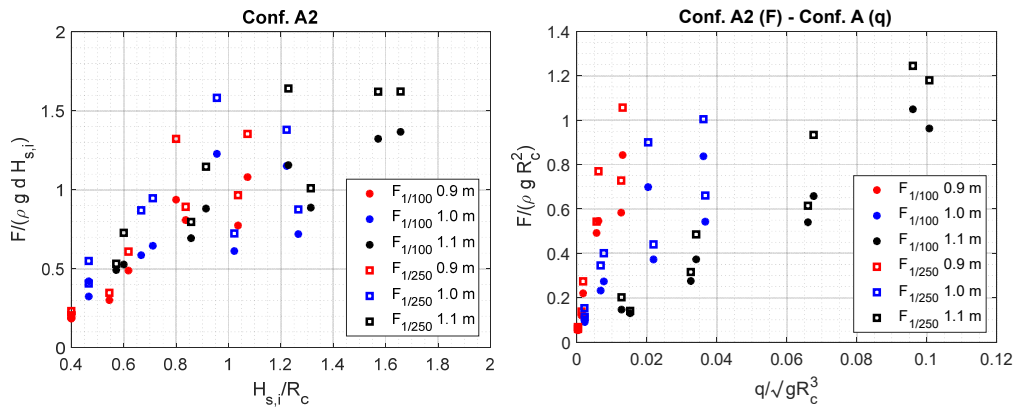**Figure 7. Wave overtopping attenuation (Conf A1, A2) Vs present conditions (Conf. A)**

Figure 8 – Wave load ($F_{1/100}$ and $F_{1/250}$) applied to the barrier for different water levels (0.9 m, 1.0 m and 1.1 m) in Configuration A2, presented through different adimensional approaches. In order to evaluate the discharge approaching the barrier in the right panel, the data measured in Conf. A are used.

CONCLUSIONS

Physical model tests were carried out in the Maritime laboratory of the Padova University in order to investigate the wave overtopping discharge expected to flood Piazza S.Marco when the Mose will be operational. In absence of some mitigation measures, the expected overtopping discharge is excessive and therefore a temporary barrier to reduce this phenomenon is foreseen.

A new set of tests were discussed, analyzing a 40 cm high barrier, in two configurations: with barrier placed on the edge of the quay (configuration A1) and 2.5 m distant (configuration A2). For both configurations, the discharge is reduced to acceptable levels. However, the experimental analysis shows that configuration A2 is more effective than configuration A1 in reducing overtopping discharge. Moreover, during A1 tests, large jets were observed, that may have some consequences on the pavement stability, in absence of additional protection.

Finally, the measured loads applied to the barrier in Conf. A2 (the preferred solution) were presented, therefore providing useful design information.

This research is carried out in view of the hydraulic management of the Piazza. During high tide, the rain, the filtration and the overtopping discharge will be drained by a new pumping system connected to the existing net of channels ('*gatoli*') placed under the pavement.

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