RENOVATION AND REDESIGN OF THE MALAMOCCO LOCK GATES IN THE VENETIAN LAGOON

by

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ABSTRACT

As a part of the major storm surge barrier project to protect the Venetian lagoon from high tides, a large navigation lock was built at Malamocco, Italy. Only months after commissioning, the sea side lock gate was severely damaged during a storm in 2015. First, a thorough investigation of the damage phenomenon has been carried out to determine the physical phenomena causing damage to the lock gate. Afterwards design improvements regarding the layout of the lock gates have been developed. The efficiency of these design improvements was quantified by means of model scale testing. This paper focuses on the design improvements that can be made to avoid future damage to the lock gates. Finally, based on an extended risk and reliability analysis and in view of retaining the most cost-effective solution for the problem, the lock gates have been reviewed and if needed redesigned. A different approach for renovation and redesign is proposed for either gate of the Malamocco lock.

1 INTRODUCTION

To protect the Venetian lagoon as well as the iconic historic heritage of the city of Venice from flooding, the Italian government initiated the MOSE project (MOdulo Sperimentale Elettromeccanico; E: Experimental Electromechanical Module). Together with other measures, such as coastal reinforcement, raising of quaysides, etc., the MOSE project basically consists of storm surge barriers at the three large inlets to the Venetian lagoon: Lido (north and south), Malamocco and Chioggia, as shown in Figure 1a. Each barrier consists of an integrated system of rows of mobile flap gates as illustrated in Figure 1b. In case of forecasted *acqua alta* high tides, the mobile flap gates are raised to isolate the lagoon temporarily from the high storm surges on the Adriatic Sea.

To allow vessels to enter or leave the Venetian lagoon when the MOSE system is operational, navigation locks are constructed at each of the inlets. At Malamocco, the main access lock was constructed to guarantee the accessibility of the Port of Venice in Mestre for large ships during *acqua alta*, see Figure 1a. The ships intended to use the lock are tankers (up to approx. 70.000 DWT), bulk carriers (up to approx. 80.000 DWT), container ships (up to approx. 60.000 DWT) and RORO vessels (up to approx. 50.000 DWT). At present no passenger ships use the access channel to the port of Venice via the Malamocco inlet. At the Lido and Chioggia inlets, small locks are constructed inside service harbours to allow emergency vessels, fishing boats, etc. to shelter and transit.

The entire MOSE project was initiated by law (L.171/73) in 1981. The Consorzio Venezia Nuova (CVN) is responsible for the execution of the works and the subsequent maintenance and acts on behalf of the Italian Ministry of Infrastructure and Transport (Venice Water Authority). Construction, coordinated by CVN and

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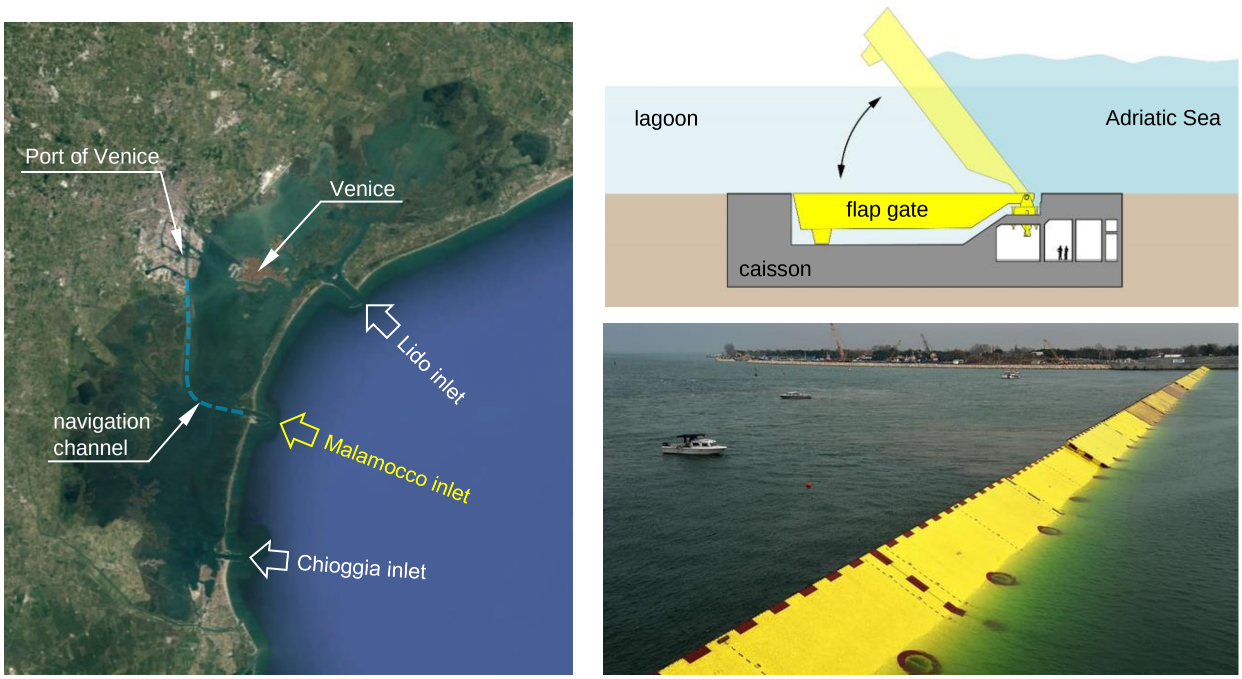
executed by COMAR, began simultaneously in 2003 at all three lagoon inlets. At present, the construction works are still underway and the primary line of defense of the MOSE system is expected to be finished by the end of 2019.

The Malamocco lock was finished and operational in the course of 2014 as the lock was intended to be used as access channel to the lagoon during the installation works of the barrier flap gates at Malamocco. Only months after commissioning, however, the sea side lock gate was severely damaged during a storm. This paper focuses on the damage phenomenon and how the lock and the gates have been adapted to avoid future damage.



(a) (b)

Figure 1: (a) Overview of the Venice lagoon and the MOSE system locations. (b) Design principle and picture of the mobile flap gates of the MOSE storm surge barrier.

2 THE MALAMOCCO LOCK AND THE DAMAGE

2.1 Original design of the lock

With a length of 380m, a width of 50.5m and a sill level at -14mIGM1942, the navigation lock at the Malamocco inlet is the largest lock in the MOSE system. As shown in Figure 2, the lock is equipped with two identical sliding gates, one at the lagoon side and one at the sea side. No redundant lock gates are foreseen.

The original lock gates were designed as sliding gates supported on two hydrofeet. As shown in Figure 3, a hydrofoot consists of a removable supporting tube in the lock gate with a horizontal bearing at the bottom. The hydrofeet are supported on a UHMWPE sliding track on the bottom of the lock. Whenever the lock gate is moving, pressurized water is pumped through small nozzles in the bearing plate of the hydrofoot, creating a thin water film beneath the hydrofoot and reducing the friction between the hydrofeet and the sliding track. A couple of pictures of the sliding track and the hydrofeet of the existing lock gate are shown in Figure 4.

The lock gates are 54.3m long and 16.5m high. The total weight of the lock gate with all technical equipment is around 1300 tonnes. The lock gate is designed with a ballast chamber that allows adjustment of the operational net weight of the lock gate and manipulation of the lock gate in floating position.

Figure 2: (a) Overview of the Malamocco inlet & lock. (b) Aerial photo of the Malamocco lock. (c) Overview of the existing sea side lock gate during installation.

The choice for the net operational weight of the existing lock gate is defined by the nominal bearing capacity of the hydrofoot system. The bearing capacity of one hydrofoot varies between a minimum of 100kN and a maximum of 1350kN in the existing design. The net operational weight of the lock gate varies between these limits with changing water levels because the technical rooms located at the top of the lock gate act as supplementary buoyancy chambers with varying water levels, as shown in Figure 3. As a result, the static reaction force of the ballasted lock gate on the hydrofeet may not exceed 1250kN per hydrofoot at the lowest water level (-1.0mIGM1942) and 180kN at the highest design water level (+2.7mIGM1942). Furthermore, as a result of variations in hydrofeet forces during motion of the lock gate, the maximum bearing capacity is reduced further. When the gate is moving, the traction force causes an increase of the vertical reaction on one hydrofoot and a decrease of the vertical reaction on the other hydrofoot.

In the existing lock gates, skin plating is only foreseen on one side of the lock gate. This is because the lock gates at Malamocco will only be operational at exceptional *acqua alta* high tides. Therefore, only skin plating is foreseen at the lagoon side of both lock gates, leaving the internal structure of the lock gates open on the sea side.

Filling and emptying of the lock chamber occurs through the lock gates, via 10 levelling tubes located at the bottom of the gate structure. The opening and closing of the horizontal levelling tubes is operated hydraulically with vertically operated sliding valves, located at the sea side of the lock gate. On top of the lock gate, a road platform is foreseen to allow traffic and pedestrians to cross the lock chamber. At both ends of the road platform, movable platforms are foreseen that are operated hydraulically as well.

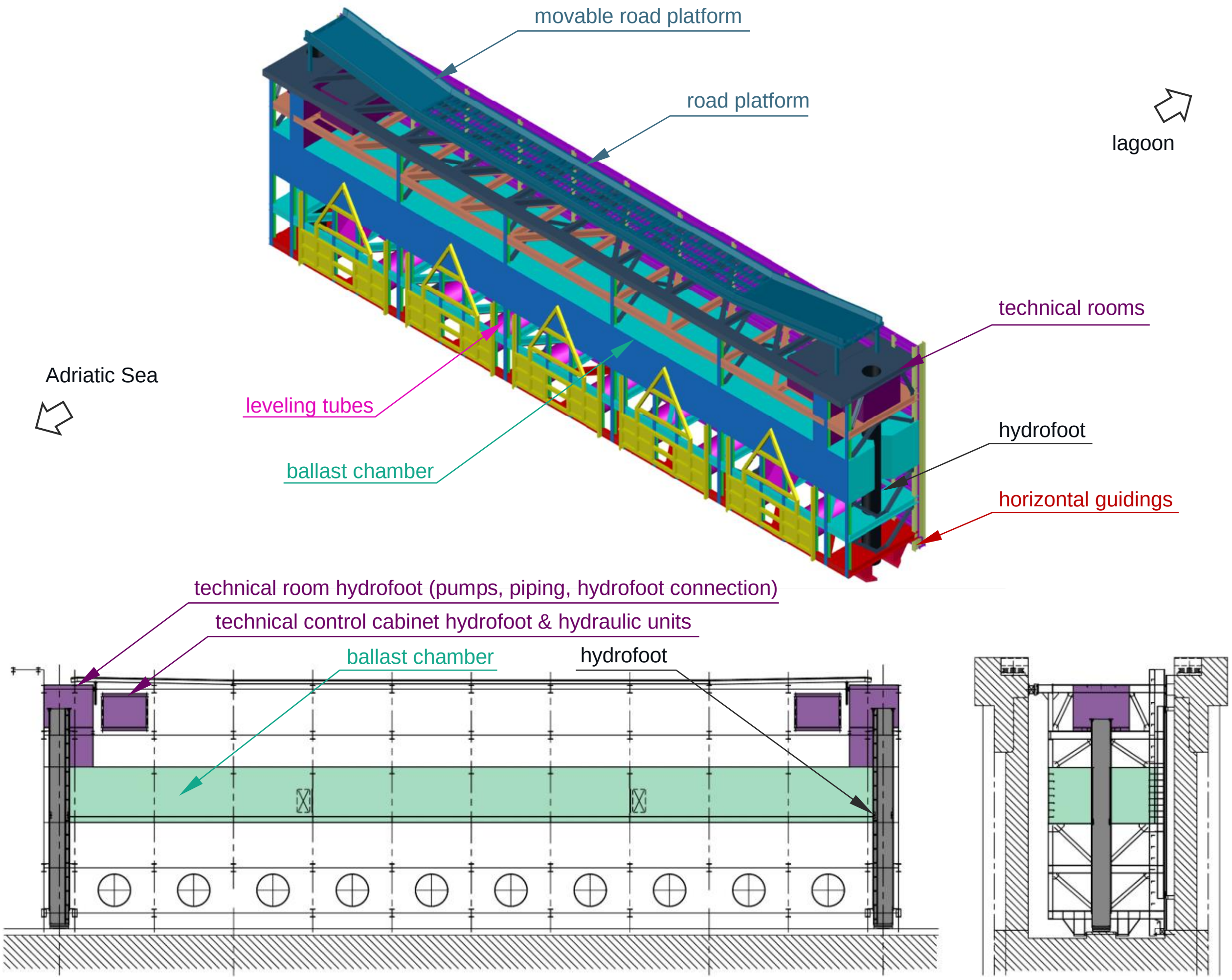
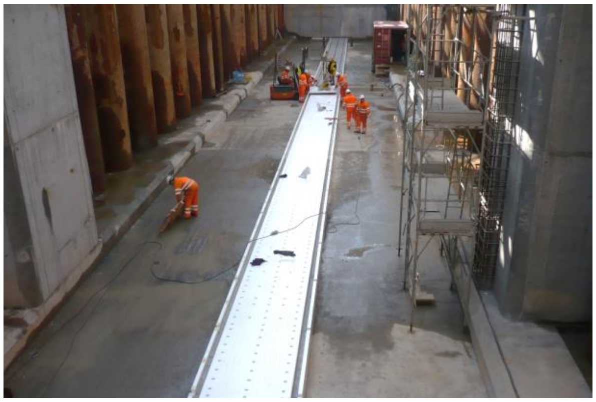


Figure 3: Schematical 3D model & cross sections of the existing lock gates.



(a)

(b)

Figure 4: (a) Sliding track during construction. (b) Hydrofoot extracted from the existing lock gate.

2.2 Damage and problem description

The existing sea side gate suffered severe damage during a nocturnal storm on the Adriatic Sea on the 5-6th February 2015. This damage was caused by vertical uplift forces on the lock gate. Subsequent lifting and falling of the lock gate on the sliding track caused damage at several points. Damage was inflicted on the guiding structures of the lock gate but more importantly, the lifting and falling of the hydrofoot bearings on the sliding track led to failure of bolted connections between the inner and the fixed outer hydrofoot tubes. These bolted connections were located in the technical spaces at the top of the lock gate, see Figure 3, where the pumps, filters and control equipment of the hydrofoot system were located. The bolted connection is an integral part of the water tightness system of the hydrofeet, the failure of the bolted connections led to flooding of these technical spaces and put the lock gate out of service.

The cause of the damage being unknown at that time, precautionary measures were taken by CVN and COMAR to avoid any further damage to the lock gates. Both gates were pulled into the gate recesses to avoid direct attack of waves on the lock gates. Additionally, the gates were ballasted with water filled bags to increase the net weight of the lock gates.

3 INVESTIGATION OF THE DAMAGE PHENOMENON

After a full inventory of the suffered damage and some preliminary studies, a.o. Grasso et al. (2015) and Volpato et al. (2015), CVN appointed SBE to carry out a full investigation of the damage phenomenon and to propose design solutions for both the sea side and lagoon side lock gates. SBE is an engineering consultant situated in Belgium with more than 25 years of experience. It is internationally renowned for the engineering design of some of the largest locks in the world, a.o. in the Port of Antwerp in Belgium, in the Netherlands, in Panama, etc.

One of the primary tasks in this investigation was to determine the wave climate in front of the lock gate at the time of the incident, since no wave monitoring devices were present in the Adriatic approach harbour at that time. A numeric study was therefore carried out, cf. Reijmerink et al. (2016), to reverse engineer the incident wave conditions based on available offshore data. Additional goals of this study were to confirm the results of the original wave study on which the design of the existing lock gates was based and to study wave penetration in the lock chamber when the sea side gate is in the gate recess. Additionally, wave amplifications originating from reflections off the temporary island were studied in detail, because this temporary situation was not considered in the original wave study and hence not taken into account for the design of the existing lock gates.

Based on all available studies and based on the inspection of the damaged lock gate, several critical aspects were identified to be at the origin of the failure of the sea side lock gate at Malamocco. A full account of the investigation leading to this conclusion is not within the scope of this paper. Only the main conclusions with respect to the design of the lock gates are therefore summarized in the following.

The first critical cause is related to the specific properties of the incident waves attacking the lock gates. This issue is particularly acute for the Malamocco lock gate given the **long wave periods** found at this location. The wave study by Reijmerink et al. (2016) reconfirmed that the wave period may be as high as approx. 10 to 12s under normative storm conditions at the lock location. For wave periods of this magnitude, the wave length could be in the order of 100-150m. Given that this is much longer than the width of the entire gate, the gate will feel either a peak or a trough over its whole width. Furthermore, the dynamic wave pressures resulting from long wave periods penetrate deep enough to cause significant alternating uplift and downlift forces on the ballast tank of the lock gate. For shorter waves with a wave period of e.g. 4s and corresponding wave lengths of the order of meters, there will usually be a peak and a trough across the width of the gate, averaging to smaller net vertical forces. Additionally, the dynamic wave pressures decrease much faster with depth for shorter wave periods and the pressures may not penetrate deep enough to cause fluctuating vertical forces.

A second critical factor is the **ballast and open volumes** in the gate. The positioning of the technical rooms that house the hydrofoot technical equipment at the top of the lock gates, see Figure 3, and to a lesser extent the hollow box girder beam for lateral guiding of the gate are very unfortunate. These components are not below the water level at all times and therefore unintentionally act as supplementary buoyancy chambers for the lock gate. As a result, with changing water levels, e.g. under long wave impact, the net operational weight of the gate will fluctuate, even if the ballasting condition of the lock gate is kept fixed.

Third, it is unsure what the actual **ballasting condition of the lock gate** was at the time of the incident. It is not unlikely that the net weight of the lock gate was insufficient at the time of the incident. Because there is no weight monitoring system in the existing lock gates, it is impossible to tell at any specific time what the net operational weight of the lock gate is.

Finally, it should be mentioned that there was **no hydraulic head** acting on the lock gate during the storm. Since the MOSE system is not yet operational, the water levels in the Adriatic Sea and the lagoon were the same at the time of the incident. The lock gate was hence not pressed against its sealings and no resisting frictional forces were present.

4 DESIGN IMPROVEMENTS

It is believed that the combination of the aspects listed in the previous section have led to the failure of the sea side lock gate at Malamocco. Based on the understanding of these physical phenomena, structural adaptations and design improvements have been proposed to alleviate the effect of fluctuating vertical forces, as listed below. Some of these improvements are obvious and need no further confirmation while others require elaboration to quantify the efficiency of the proposed measures.

(I) <u>Elimination of the technical spaces between the lowest and highest design water levels</u>

It needs no further clarification that the net operational weight of the lock gate should at all times be under control and not subject to water level variations and by extension wave action. Therefore, in the new design, all ballast compartments and technical spaces should be moved below the lowest design water level, taking into account wave action.

(II) <u>Perforation of the ballast tank</u>

To minimize pressure differences above and below the ballast tank, perforation tubes may be foreseen in the ballast tank so that excess pressures may be alleviated by flow through these tubes. The size of these perforation tubes should be verified to quantify the efficiency of this design adaptation.

(III) <u>Reduction of wave penetration into the lock gate (sea side skin plate, etc.)</u>

Closing off the seaside of the lock gate should reduce the penetration of pressures into the gate structure. As the penetration is reduced, the pressure differences below and above the ballast tank as well as the resulting uplift forces should decrease as well. It should be noted however that even a small gap is sufficient to allow pressure penetration of pressure into the gate. The smaller the gaps however, the larger the pressure losses through these gaps, and the smaller the resulting uplift forces.

To quantify design improvements (II) and (III), both a numerical study (Pancham et al., 2016) and a physical model test campaign (Lena et al., 2016) have been performed in the framework of this study. In these studies multiple different configurations have been considered. In this report, the following configurations are reported, see also Figure 9:

(1) The basic lock gate configuration similar to the existing design but with lowered ballast compartment according to improvement (I);
(2) Similar to configuration 1, with added skin plate at the sea side;
(3) Similar to configuration 2, with perforation of the ballast tank;
(4) Similar to configuration 3, with smaller gap at the lock floor;

(5) Similar to configuration 4, with smaller gaps at the lateral sides of the lock gate. Since the gate recess is much wider than the lock gate itself, see also Figure 2c, the gap between the gate and the civil works is considerable and the effect of closing it off should be studied. This alternative is only relevant in a 3D testing environment.

4.1 Numeric sensitivity analysis

The numerical study consisted of 2D CFD simulations for configurations 1 to 4. Because of the 2D character of these simulations, configuration 5 could not be considered due to its inherent 3D character. In fact, configuration 5 could therefore be considered identical to configuration 4 in a 2D schematization. The numerical model used for the simulations was the open-source, general-purpose CFD-toolbox OpenFoam using the wave generation and absorption toolbox waves2Foam (Jacobsen et al., 2012).

For each configuration, the mean peak vertical forces were determined and compared for several sets of regular wave conditions, as discussed in detail in Pancham et al. (2016). The wave conditions considered were artificially determined in the framework of a sensitivity analysis. This way, the effect of wave conditions and the efficiency of the proposed design optimizations could be assessed.

From the simulations, the vertical net force on the lock gate is found to decrease from configuration 1 to 4, with additional design improvements being made. For normative wave conditions (wave period in the order of 10.0s), the peak vertical forces in configuration 4 are up to 70% lower than in configuration 1. The simulations also show that the vertical forces increase with increasing wave period, increasing wave height and decreasing water level. This is all the result of larger pressure variations at the gap between the bottom of the skin plate and the lock floor.

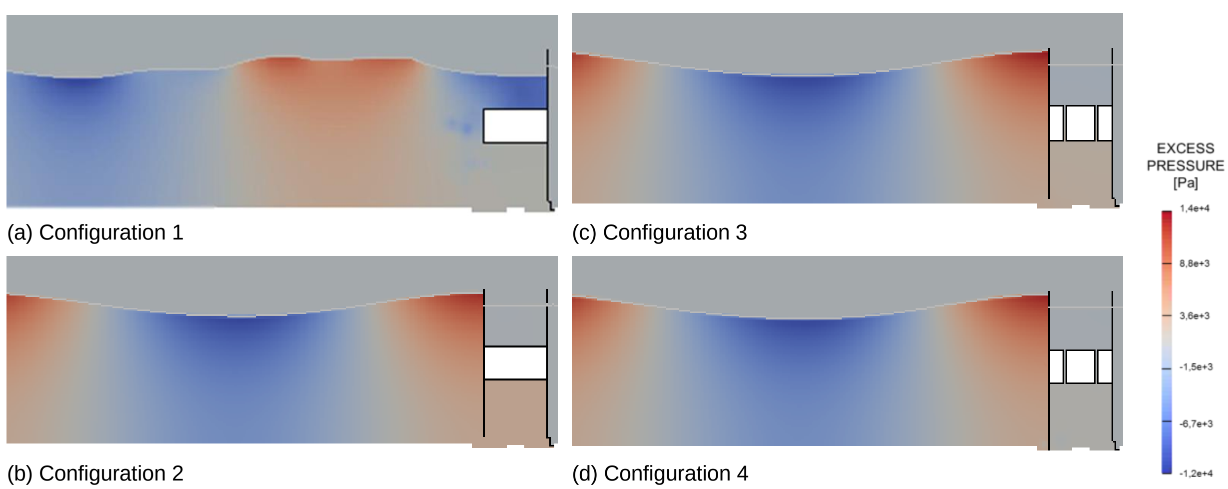


Figure 5: Excess pressure plots for configurations 1 to 4, determined by means of 2D CFD simulations (Pancham et al., 2016)

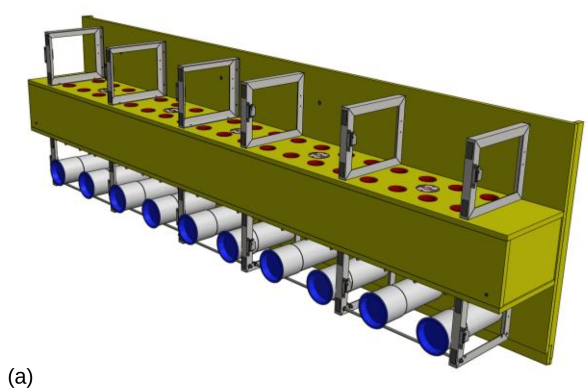
The simulations allow to assess and compare the pressure fields in the different configurations as well. In Figure 5, the excess pressure field is shown for configurations 1 to 4 at one time frame with resulting uplift forces for illustration. The excess pressure shown in these plots is defined as the pressure in excess of the hydrostatic pressure, i.e. the wave-induced pressure field. It can be observed for configuration 1 that large wave-induced pressure differences occur above and below the ballast tank as the wave impacts and penetrates the gate structure. This results in large alternating vertical forces on the lock gate. The addition of a skin plate in configuration 2 decouples the pressure on top of the buoyancy chamber from the wave-induced pressure. However, this is not sufficient to significantly reduce the vertical force acting on the gate due to wave action underneath the buoyancy chamber. The subsequent addition of vertical tubes in the buoyancy chamber in configuration 3 significantly reduces the vertical force on the gate. Finally, the vertical forces are reduced even further by making the gap between the sea side skin plate and the lock floor smaller. The smaller the cross sectional area ratio (smaller height of the gap), the smaller the vertical force will be on the lock gate, because the pressure difference on the ballast tank and through the vertical tubes will decrease along with a decrease in the mass flux through the gap at the bottom, in and out of the gate.

In conclusion, the 2D CFD simulations confirm the physical phenomena taking place as presented in section 3 and qualitatively establish confidence in the proposed design optimizations. However, there are a couple of important limitations:

- No 3D effects are taken into account. This leads to simplifications on many points: the importance of the lateral gaps (cf. configuration 5) cannot be quantified; the perforation of the ballast tank is schematized as an equivalent 2D area ratio; the effect of the lock gate in opened position cannot be quantified, the net forces are based on extrapolations from values 'per running meter', etc.
- No irregular waves have been considered in the 2D CFD simulations.
- At the highest water levels, wave overtopping of the gate occurs. Because of the 2D character of the simulations, this leads to inaccurate results for configuration 2 and to a lesser extent configurations 3 and 4 as well, as the water that is captured in the gate structure above the ballast tank cannot drain towards the recesses, etc. Additionally, the overtopping waves may influence the pressure distribution as well.

4.2 Model scale tests

To study the effects that could not be studied in the 2D CFD simulations and to determine actual design loads, physical scale model tests have been performed in the shallow water basin of the MARIN research facilities in the Netherlands (Lena et al., 2016). The scale model tests were performed on a simplified wooden model of the lock gate at a geometrical scale of 1:20. Based on previous investigations, the model was simplified to the main components that influence the wave induced vertical forces, i.e. the ballast compartment, skin plating, filling/emptying tubes, etc. The complex inner structural supports and stiffeners in the lock gate were not modelled to scale and to six square frames above and below the ballast tank to give the requested rigidity to the model.



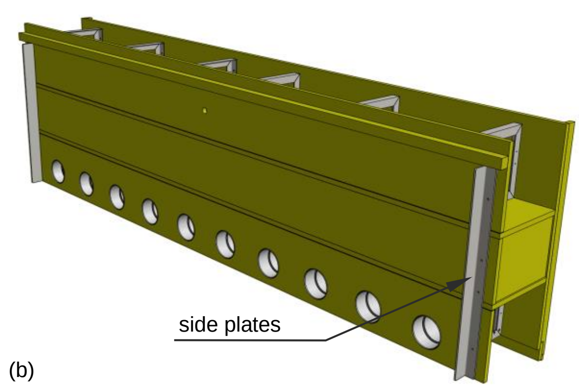
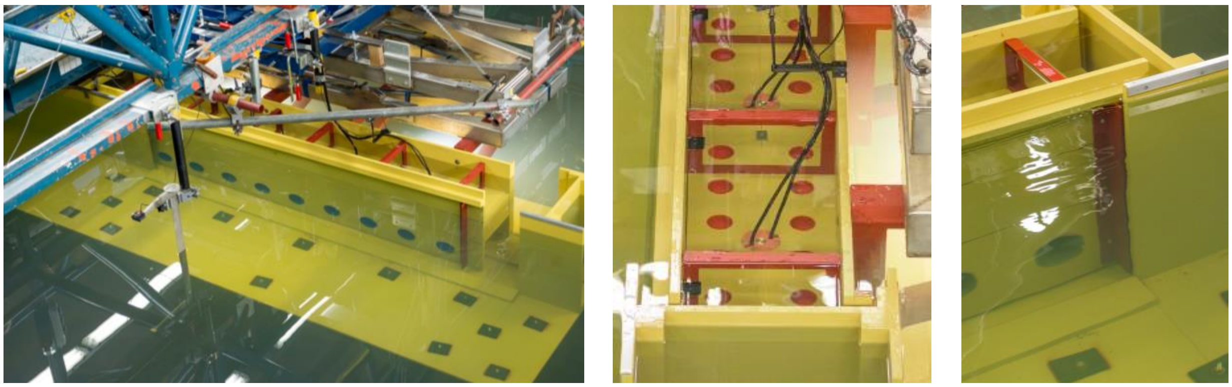


Figure 6: 3D rendering of the modular scale model in configuration 1 (a) and configuration 5 (b).

As illustrated in Figure 8, the scale model was rigidly connected to a six component frame in order to directly measure the wave induced forces and moments in six degrees of freedom. The recesses of the gate and the geometry of the lock floor were modelled as well, in order to properly reproduce the water flow around the gate.

Based on the 2D CFD study, it was concluded that the size of the gaps between the gate structure and the recesses has a major influence on the reduction of the vertical forces. However, in the model scale setup, special attention had to be paid to avoid any connection or accidental contact between the gate structure and the recesses that could affect or invalidate the force measurements. The setup was designed so that a minimum distance of 5mm would be respected in all the gaps. This distance was considered safe in order to compensate for any deformations of the setup when subjected to the highest wave loads and to avoid any contacts. At the same time, this introduces some inaccuracies in the model set-up as well, since the gaps will be significantly smaller in reality. Based on the results of the 2D CFD simulations, this simplification is deemed conservative as forces are expected to decrease with smaller gaps.



(a) (b) (c)

Figure 7: (a) Overal view of the test setup. (b) Side top view of the scale model in the basin; the plastic caps to close off the vertical tubes are visible; pressure sensors were installed on the top and botttom of the ballast tank as well to validate the force measurements. (c) Side plating to close of the gate recesses.

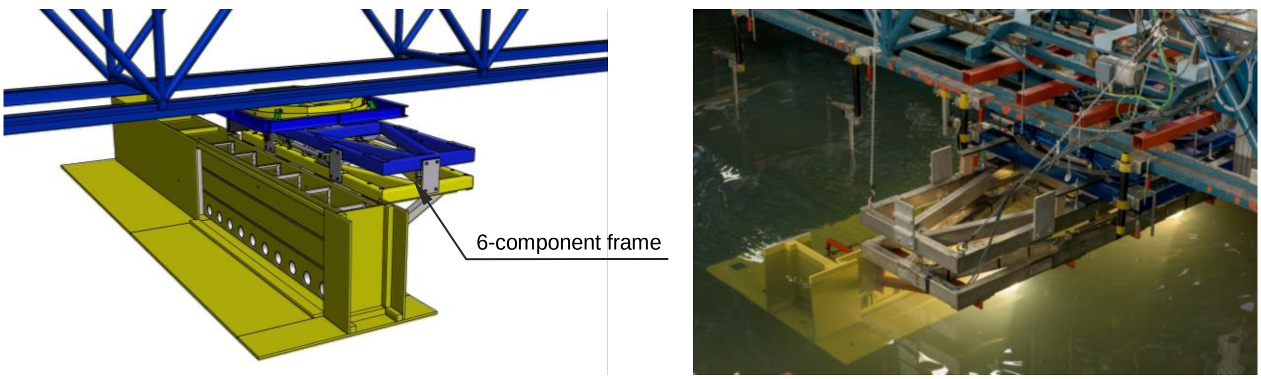


Figure 8: 3D rendering of the six component frame attached to the scale model & picture of the setup in the test basin.

In the scale model tests, only normative wave conditions have been considered. Based on the meteomarine wave studies, a.o. Reijmerink et al. (2016), the normative wave conditions are related to the two principal wind directions in the area: Bora conditions (60°N) with very intense winds but shorter fetch lengths and Scirocco conditions (140°N) with very long fetch lengths across the deep waters of the central Adriatic Sea and limited effects of shoaling. Both conditions result in reasonably large wave periods (in the order of 10–12s).

In the wave study of Reijmerink et al. (2016), it was concluded that the wave climate in front of the lock gate was very disperse as a result of reflections in the approach harbours and no preferential wave direction could be determined. Therefore, the most conservative situation with irregular waves approaching the lock gate perpendicularly have been tested in the scale model tests.

In Figure 9, the results of the scale model tests for four normative wave conditions (Bora/Scirocco at high/low water level) are shown and compared with the numerical results of the 2D CFD simulations (Pancham et al., 2016).

During the first series of tests at high water levels, it was observed that the uplift forces increased in configuration 2 before decreasing again in configurations 3 to 5. The reason that configuration 2 yields worse results than configuration 1 is attributed to 3D effects in the scale model tests. Also, the effect of wave overtopping may play a part in this behaviour. In any case, configuration 2 was omitted in the second series of tests at lower water level because it was clear that the perforation of the ballast tank has a major impact on the vertical forces on the lock gate.

For configuration 1 a large variability of the vertical forces is observed depending on the incident wave conditions. For the geometries where significant structural changes have been undertaken (e.g. configuration 5), the vertical forces are much less dependent on external factors and the vertical forces are in a much smaller range. The smaller the range, the more predictable the vertical forces become, independently of the applied wave conditions.

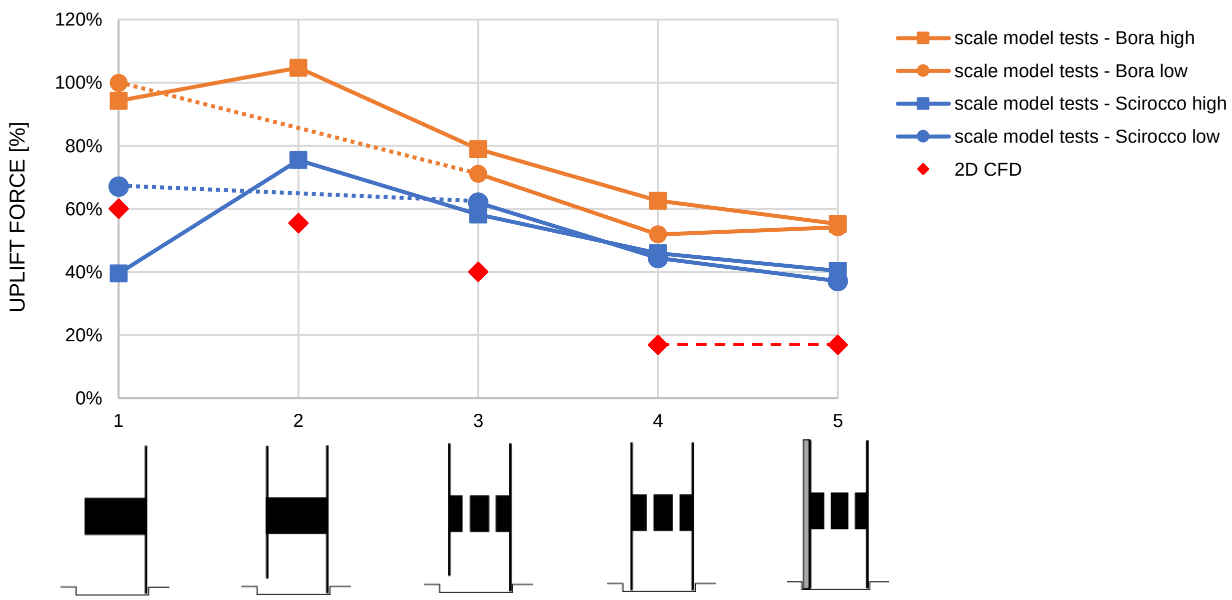


Figure 9: Comparison of peak uplift forces from scale model tests (Lena et al., 2016) and from 2D simulations (Pancham et al., 2016); values are scaled w.r.t. the largest force in configuration 1.

The comparison between the scale model tests and the results of the numerical sensitivity analysis should not be expected to yield identical results. The results cannot match for several reasons: different wave conditions were considered (wave height, wave period, regular vs. irregular), 2D vs. 3D effects are taken into account, etc. In general, the numerically predicted forces are much smaller than the forces determined through scale model testing and the efficiency of the design optimizations is also larger in the numerical simulations. The latter is most likely related to the fact that the gap between the lock gate and the lock chamber is much smaller in the numerical simulations (configuration 4) than in the scale model tests. As discussed this is one of the deficiencies of the scale model tests to avoid inaccurate results.

Nevertheless, it is interesting to observe that a similar trend can be observed for both the numerical and the scale model test approaches. The subsequent measures (skin plate, perforation, closing the gaps) result in a gradual optimization of the original design. The scale model tests also illustrate that closing off the lateral gaps towards the gate recesses (cf. Figure 7c) results in a further decrease of the vertical forces.

Concluding, it is clear that configuration 5 yields the best improvement of the layout of the gate in view of vertical force reduction. The vertical uplift forces are reduced by up to 40%. Also, the vertical force becomes more predictable in the sense that the variability of the peak vertical forces in different wave conditions is much smaller for configuration 5 when compared to configuration 1.

5 RENOVATION AND REDESIGN OF THE MALAMOCCO LOCK GATES

Aiming at a safe design for the Malamocco lock gates for the future, both the existing sea side gate and the lagoon side gate have to be adapted. However, a substantially different solution is considered for both gates based on different operational conditions of the gates as outlined in the following.

5.1 Renovation principles

Resulting from an extensive risk analysis and aiming at a safe design solutions for both lock gates, a substantially different solution has been selected for either gate. The current differentiated approach is aimed at providing reliable solutions for both gates in the most cost-effective way.

Since the **sea side lock gate** is part of the primary line of defense of the MOSE system, a safe and reliable solution is required under all conditions. In this light, the sea side lock gate had to be entirely redesigned from scratch. The redesign of the sea side lock gate consisted of a dual solution where the design optimizations discussed in section 4 are implemented on the one hand and the net operational weight of the lock gate is increased on the other hand. The redesign of the new sea side lock gate is described in detail in the next section 5.2.

From a reliability-safety point of view, the operational conditions of the **lagoon side lock gate** are very different from the sea side gate. Indeed, by imposing that the sea side lock gate must always be in the closed position during operation of the lagoon side gate, the direct influence of the wave climate from the Adriatic Sea on the lagoon side lock gate can be eliminated as illustrated in Figure 10. Wave actions on the lagoon side lock gate while moving are therefore only wind-driven and determined by a relatively short fetch length; in this case the length of the lock chamber. A smaller wave period (2.0 to 3.5 s) and a smaller wave height may be therefore be expected to act on the lagoon side gate during operation.

(a) If the sea side gate is closed, the lagoon side gate is protected and can be operated

(b) When the sea side gate is opened or absent during *acqua alta*, the lagoon side gate must be closed and anchored.

Figure 10: Operational principle for the lagoon side gate.

Because the lagoon side gate was not damaged during the storm and in view of cost-effectiveness, it was decided to retain the existing design of the lagoon side gate with some important design changes.

The first major design change comprises the addition of a mechanical anchorage system on the gate. This anchorage system should provide the necessary resistance against uplift forces from wave action in the situations that waves originating from the Adriatic Sea can reach the lagoon side gate, i.e. when the sea side gate is in open position or away for maintenance. In these situations, the lagoon side gate must be closed and in anchored position. The restrictions on the operation of the lagoon side lock gate are incorporated in the newly designed control system of the entire lock complex.

In the light of reliability and safety, this approach is acceptable because the likelihood of an extreme event in combination with one of the above scenarios is much smaller and measures can be taken to avoid or limit the occurrence of such events. The sea side gate should e.g. not be taken out for maintenance in a season when high waters and storms are likely to occur. In the unlikely event that such a situation would nevertheless occur, additional safety measures are available, e.g. the complete filling of the ballast chambers of the lagoon side gate to increase the net weight of the gate as much as possible. This is an emergency measure that takes time and costs money and should therefore not be considered in a primary line of defense.

In the existing lock gate, however, there is no weight monitoring system available. This is the second design change that was made to the lagoon side gate. At the top of the hydrofoot tubes, several structural changes are made to install a direct force measurement load cell. By incorporating this weight monitoring system in the control system of the lock complex, a warning alert can be programmed whenever the net weight of the gate becomes too low for the predicted sea conditions. Addtionally, the connection of the inner and outer tubes of the hydrofoot system is relocated outside the technical rooms.

5.2 Redesign of the sea side lock gate

While the lagoon side gate is in the secondary line of defense and sufficient safety measures can be installed to allow small design modifications, this is not the case for the sea side lock gate. The sea side lock gate is part of the primary line of defense of the MOSE system and a safe and reliable solution is required, under all conditions, at all times. To achieve this, it was decided to design a completely new sea side lock gate, based on the design optimizations as discussed in the previous sections.

Besides the qualitative approach in the discussion of the paper until now, the design value of the uplift forces can be determined based on the scale model test results. The design value is found by applying a safety factor on the maximal peak value of the uplift forces from the tests. The safety factor was determined through a statistical safety analysis allowing for a 1% exceeding probability and accounts for all measuring uncertainties (repeatability of the measurements, corrections for static force measurements, etc.) and modeling uncertainties (wave condition variability, modeling simplifications, scaling deficiencies, etc.).

Although the design optimizations result in a significant reduction, the design value of the vertical uplift force was found to remain fairly high. In this light, a dual solution was developed for the design of the new lock gate. On the one hand, all design optimizations (I, II and III) as discussed in section 4 were implemented to reduce the vertical uplift forces due to wave action. An overview of the eventual design of the lock gate with these structural changes is shown in Figure 11. On the other hand, the resistance of the lock gate to vertical uplift forces had to be increased. The latter can only be achieved by increasing the net operational weight of the lock gate.

As discussed in section 2.1, however, the support system of the existing lock gates is based on two hydrofeet, sliding along a sliding track on the lock floor. Hydrofeet have a limited bearing capacity however, which would largely be exceeded by increasing the weight of the lock gate. Therefore doubled or tripled hydrofoot support would be required to allow motion of the lock gate but there are many practical and technical objections to such a proposal. Above all, the supporting technical equipment that should also be tripled in such a scenario must be relocated into the ballast chamber of the lock gate to comply with design improvement (I). Doing so, the net operational weight of the gate is decreased to the point that the weight becomes insufficient to resist the vertical uplift forces, which is unacceptable.

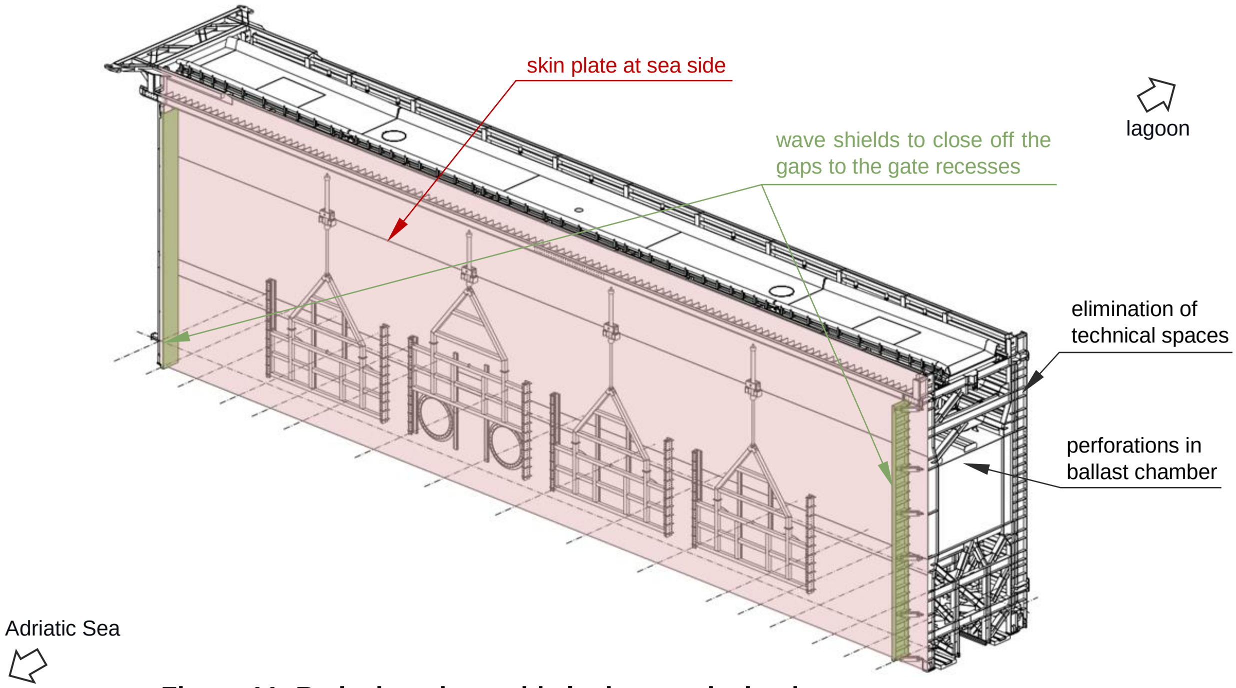


Figure 11: Redesigned sea side lock gate: design improvements.

Taking into account all technical and practical objections and given that a robust and reliable solution is required for the sea side lock gate, an alternative supporting system has been proposed: the redesigned lock gate will be supported on two lower rolling wagons instead of hydrofeet. The rolling wagons can be designed for much higher bearing capacities and by using two lower rolling wagons, it is possible to fit the new design within the existing civil structures. There are some important changes in the design of the lock gate itself as well as changes that have to be made to the existing civil works of the lock complex to achieve such a design.

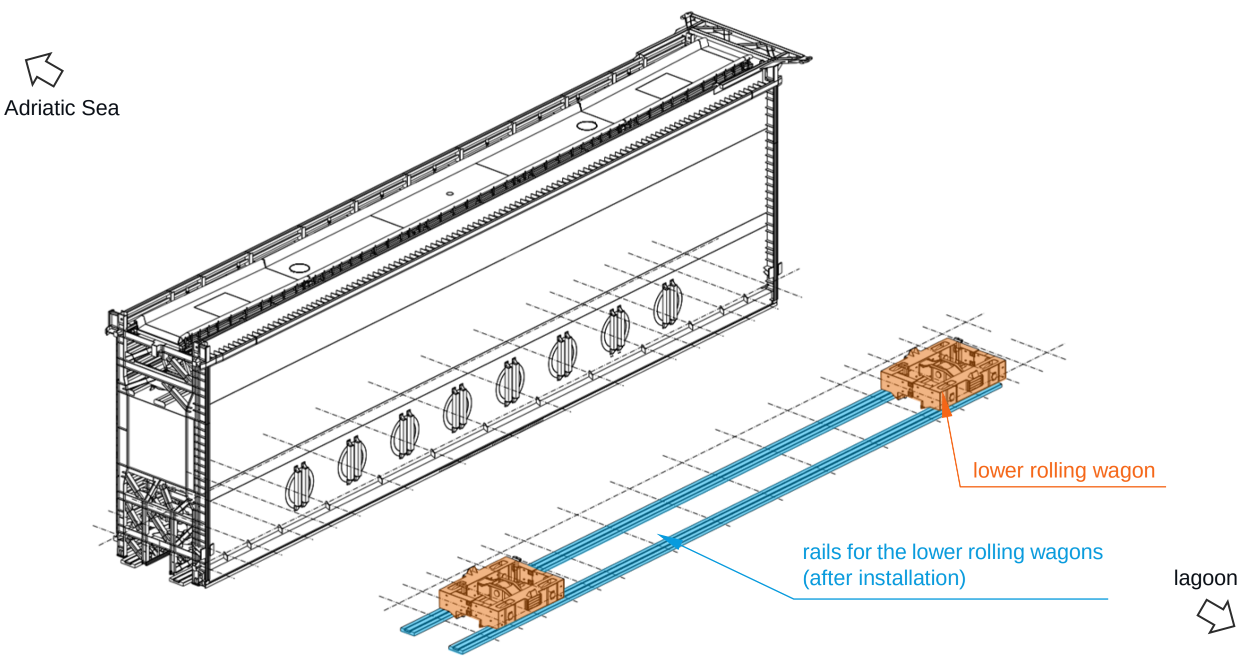


Figure 12: Redesigned sea side lock gate: lower rolling wagons & rail system.

First of all, a rail system needs to be installed at the bottom of the gate recess and the lock chamber. The overall geometry of the rails and the rolling wagons is shown in Figure 12. The presence of the sliding track (see Figure 4a) is both an added value as it is used as lateral guiding structure for the rolling wagons, as well as an important constraint for the design of the rolling wagons. The rails are constructed on either side of the sliding track. Since the lock chamber and gate recesses cannot be put in dry conditions for maintenance a habitat structure has been designed for the construction of the rail structures and the installation of the rails. The habitat structure is purposely designed for this application and allows to construct the rail structure in segments of 12m on either side of the sliding track in dry conditions. The habitat is equipped with a moving mechanism that allows it to move on the lock floor, without the need to be lifted and lowered again by cranes at the surface. An overview of the habitat structure is shown in Figure 13.

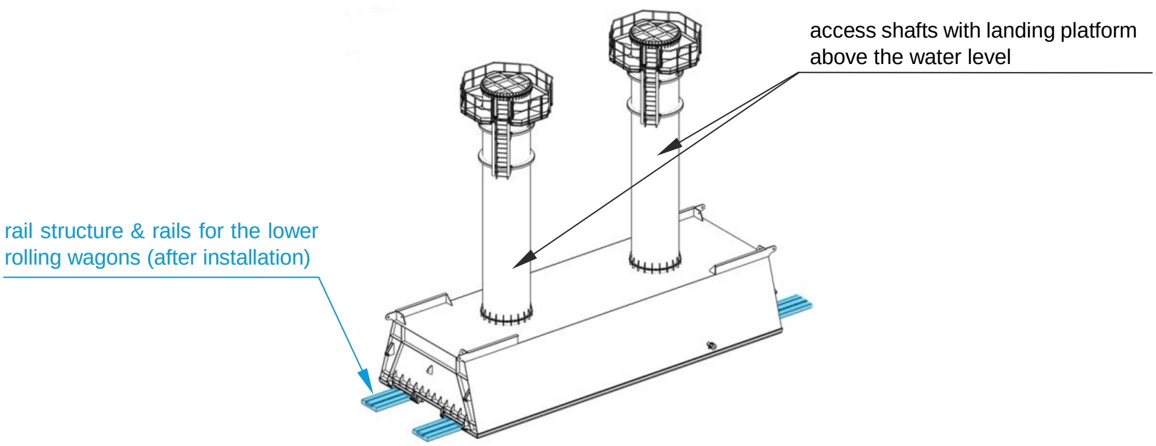


Figure 13: Habitat structure that will be used to install the rail system on the lock floor.

Similarly as for the lagoon side gate, a weight monitoring system is foreseen in the design of the new sea side lock gate. By monitoring the weight of the gate at all times, warning alerts can be incorporated in the control system of the lock complex if the net weight of the gate is too low for the predicted wave conditions so that precautionary measures (e.g. increasing the ballasting condition) can be taken. Not only the water level in the ballast compartments is monitored, but the reaction forces on the lower rolling wagons are also directly measured by means of load cells that are installed in the reaction tubes of the lock gate, as shown in the cross section of the new sea side lock gate in Figure 14.

It can be observed in the figures (e.g. Figure 14) that only 8 levelling tubes are present in the new sea side lock gate instead of 10 levelling tubes in the original design. This is due to the presence of the lower rolling wagons and the shifted location of the reaction tubes. To make sure that the levelling time is not decreased as a result of this adaptation, several remedial measures can be proposed. Either an increased diameter of the levelling tubes could be used or the lifting speed of the levelling valves may be increased, as long as the hydraulic forces on the ships in the lock remain acceptable during levelling. In the new design of the sea side lock gate, the original tube diameter was retained but a dual lifting speed was proposed for small and high head differences.

The increased weight of the gate requires an increase of the traction force for the driving mechanism of the lock gate to ensure that the lock gate can still be operated in a reasonable time frame. The new driving mechanism is designed in such a way that the original power requirements are retained and only minimal adaptations are required to the civil works. All equipment is installed in the purposely built technical room at the end of the gate recess.

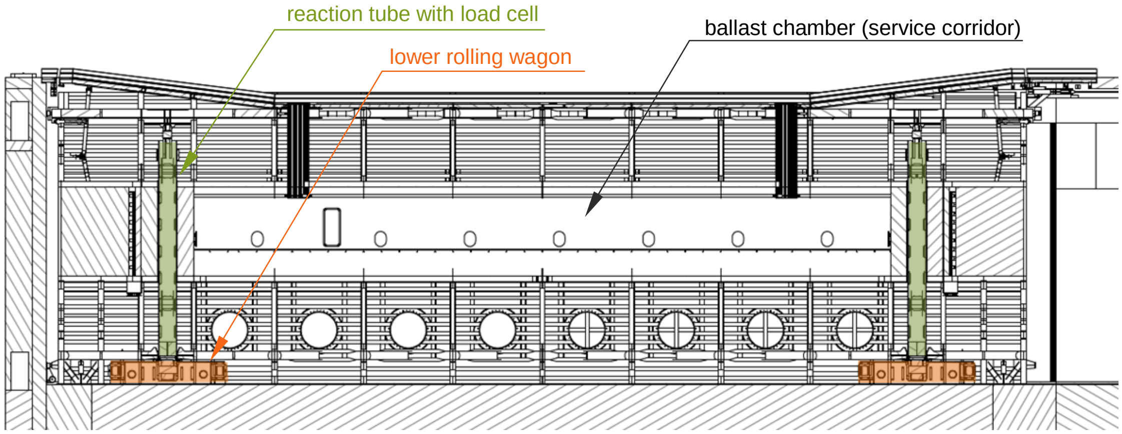


Figure 14: Redesigned sea side lock gate: cross section.

Given the urgency to finish the MOSE system, the design studies for the lock gates had to be performed in the shortest possible time frame. The exploratory and preliminary design phase, including the investigation of physical causes and design optimizations took SBE about six months to complete. This period included the hydraulic research and tests performed at MARIN. The final design of the lock gate and the preparation of tender specifications was finished in about one year. At present, the construction works for both the new sea side gate and the improvements of the lagoon side lock gate are underway and the primary line of defense of the MOSE system is expected to be finished by the end of 2019. The Malamocco lock is expected to be fully operational by March 2020.

6 CONCLUSIONS

After a thorough investigation of the damage phenomenon through terrain inspections and numerical simulations, the physical phenomenon causing the damage was discovered. The combination of the specific wave climate with long wave periods at this location and the open structure of the existing lock gate with technical rooms acting as buoyancy chambers above the lowest design water level has led to unacceptable vertical uplift forces.

Based on the understanding of the underlying physics, design improvements have subsequently been developed. The efficiency of these design improvements was quantified by means of scale model tests and the uplift forces were reduced by up to 40%.

Finally, based on an extended risk and reliability analysis and in view of retaining the most cost-effective solution for the problem, a different approach for renovation and redesign was proposed for both gates of the Malamocco lock. The engineering design was performed in approx. For the lagoon side lock gate, minor design adaptations were proposed in combination with an updated control system regulating the operation of the lagoon side lock gate under firm restrictions. For the sea side lock gate, an entirely new design was made. All proposed design optimizations were taken into account and a new support system had to be designed for the lock gate.

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