WAVE LOADS ON A MULTI-PURPOSE PLATFORM IN VADO LIGURE (ITALY)

Antonio Lizzadro, Giovanni Cuomo, Maria di Leo, Giulia Bragantini, and William Allsop

A new multi-purpose platform will be built in Vado Ligure to replace existing terminal facilities. The proposed “open” structure is formed as an elevated deck on cross beams, suspended on vertical piles, themselves supported by submerged caissons. A pseudo 3D physical model was carried out to support the design, aimed at measuring the wave loads acting on the structure during design conditions. The paper describes the outcomes of the physical model study; it discusses differences between results obtained measuring wave loads by means of pressures and force transducers and on how the loadings relate to the wave transformations over the structure.

INTRODUCTION

Project background

In 2007 Technital was appointed by Port Authority of Savona to undertake the design of a Multi-Purpose Platform at Vado Ligure (Italy) to create docking and operating space for container ships, oil and coal bulk carriers, replacing existing bulk and oil terminal facilities. The platform is 350 m long and 250 m wide.

Figure 1. Present situation and proposed platform for Port of Vado Ligure (Italy).

The estimated foreseen capacity of the container terminal (on the right hand side in the picture above) is of 720,000 TEUs per year, providing berthing for last generation container ships like Emma Maersk. Opposite to the container terminal, an oil quay for 30,000 t ships will relocate the 2 operators (ESSO-AGIP) currently using the two existing jetties (see Fig. 1, left), to be demolished. The coal bulk quay, on the shorter side of the platform, will allow to operate 80,000 t ships, providing a conveyor link to the local Tirreno Power Plant.

1 Technital S.p.A.– Via Cattaneo 20, 37121 Verona, Italy
2 HR Wallingford – Howbery Park, Wallingford, Oxfordshire OX10 8BA, UK
Platform structure typologies
Approximately 100,000 m² of the platform will be formed as reclamation bounded by 25 concrete caissons and revetments. The outer (deeper water) half of the structure will be formed as an elevated deck on cross beams, suspended on 936 vertical piles, themselves supported by 104 submerged caissons (see Fig. 2). This typology was selected to comply with requirements of the Environmental Authority, imposing that the seaward half of the platform had to remain “open”, in order to allow water circulation and therefore guarantee water quality within the bay of Vado Ligure. The submerged reinforced concrete caissons have dimensions of 22 m x 27 m and height varying between 6 m and 8.4 m, each supporting 9 square columns (3 m wide). Rock protection will be placed between caissons to prevent toe scour. The elevation of the platform was fixed at +4.5 m m.s.l.. The deck elevation above still water level was set based on both landscape and operational constraints, as a compromise between the need to reduce visual impact (by keeping the deck elevation low) and that to reduce wave-in-deck loads (by increasing the deck clearance and therefore the elevation).

Figure 2. Plan view of platform and close up of the structure platform.

Scope of the physical model
Due to the relatively low clearance of the deck above the still water level, large wave-in-deck loads are expected to act on the platform during most severe wave conditions. A preliminary estimate of wave loads was undertaken based on guidance available in literature (McConnell et al. 2004, Cuomo et al. 2007 and 2009). Those methods refer to traditional structures (deck on cross beams supported by piles) and can therefore be regarded as reliable for the seaward edge of the deck. None of these methods nevertheless provides information on how the interaction between waves and caissons can affect wave loading on the structure. Therefore physical model tests were used to assess both pulsating and impulsive wave forces acting on the platform deck and beams. This allowed to monitor the interaction between waves and structure as well as wave transformation along the structure.
PHYSICAL MODEL DESCRIPTION

Model setup
A physical model study was carried out at a Froude scale of 1:60 in one of the wave flumes of HR Wallingford. The wave flume is 45 m long, 1.2 m wide and 1.6 m deep; random waves can be generated at the offshore end of the flume by an absorbing wave paddle. Measurements included wave loads acting on the platform deck and beams, piles and submerged caissons, using both pressure sensors and force transducers; wave overtopping onto the top deck and scour at the toe of submerged caissons were also monitored during testing.

The model structure deck elements, beams and supporting elements (piles and caissons) were designed to provide the required stiffness while allowing installation of pressure and force transducers at agreed positions. Rock protection at the toe of the caissons was reproduced using appropriately scaled rock armour.

Two longitudinal sections of the structure were tested, each representing the “open” portion of the structure, from the seaward end up to the edge of the reclamation.

The two sections have different geometrical characteristics, as described in Fig. 3. For completeness, their location within the structure is also shown in Fig. 4.

Figure 3. Longitudinal sections tested- Section A (up) and Section B (down).

Figure 4. Plan view (left) and prospective view of model structure (right).
Test programme
Two test series A and B (referred to section A and B) were carried out on the model. Wave conditions (1:5, 1:25, 1:100 and 1:500 years, see Table 1) were calibrated using an offshore wave probe located on the -42.5 m (test series A) and -48 m (test series B) contour. Test series A0 and B0 looked at wave transformation over the platform area without deck and beams. Test series A1, B1 looked at: force and pressure measurement, overtopping visual observation, scour protection, wave reflection at the toe of the structure.
Tested wave conditions include both frequent and extreme conditions, as summarised in Table 1. Each test was run for a duration of 1000 waves.

<table>
<thead>
<tr>
<th>Table 1. Test conditions – waves in front of the structure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Return period (years)</td>
</tr>
<tr>
<td>-----------------------</td>
</tr>
<tr>
<td>5</td>
</tr>
<tr>
<td>25</td>
</tr>
<tr>
<td>100</td>
</tr>
<tr>
<td>500</td>
</tr>
</tbody>
</table>

Methodology and measuring system
The supporting elements (caissons and piles) of the model structure were installed in the flume and 32 twin wire wave gauges were deployed at key locations along the model structure (see Fig. 4) to identify the most critical locations along the platform in terms of both wave agitation and maximum crest elevation and to inform the selection of key elements to be instrumented for wave loads evaluation.
Two adjacent portions of the deck to be instrumented were identified at the seaward end of the structure, namely UNIT1 and UNIT2 (see Fig. 5), on basis of the observation of wave transformations along the structure.

Figure 5. Plan view of instrumented deck units: UNIT 1 (left), UNIT 2 (right).
UNIT1 was instrumented using 23 pressure sensors and 5 force transducers (3 on deck panels and 2 on beam elements), while UNIT2 was instrumented using 14 pressure sensors and 5 force transducers. Fig. 5 shows a plan view of the deck portion corresponding to the aforementioned units, with location of pressure sensors. Pressures were logged at a frequency of 5000Hz (~650Hz prototype scale) in order to capture both impulsive and pulsating loads. Pressure time histories for each sensor were analysed to extract statistically significant parameters, such as the pressure maxima at 1/1000 and 1/250 significance levels, both for impulsive and quasi-static loads.

Reflected wave heights were quantified using an array of 3 in-line wave gauges placed in front of the structure.

Wave overtopping was observed visually and recorded using two cameras pointing at the structure head from the top and the side. Overhead pictures were taken before and after each test to record movement of the rock protection at the toe of the caissons. These images were then overlaid and compared to identify any armor movement or extraction.

TEST RESULTS

Waves propagation along the structure

Wave measurements along the structure were undertaken during tests carried out without the superstructure (A0 and B0), in order to get insight on wave transformations due to interaction with the complex under-structure and to identify the areas within the platform that would most likely be affected by the heaviest loadings; this information was used to inform the selection of the most appropriate locations to be instrumented with pressure and force transducers during subsequent testing phases.

Wave transformation over the model structure exhibited a very similar pattern for all wave conditions tested. Fig. 6 shows the variation of wave height and maximum crest level along the structure, for the 1/100 return period (moving average was used, in order to filter local oscillations). Wave height and crest level reduce dramatically within the first 50 m (which corresponds to about 2 submerged caissons) due to depth-limited breaking and destructive interference over the top face of the caissons.

For section A the energy loss along the platform is more pronounced due to the different crest level of the caissons adopted for section A and B: the crests of the seaward caissons row lie both at -8.2 m, but moving from the first to the second row, for section A waves encounter a “step” of about 5 m whereas for section B there is only a difference of 50 cm in height. Moving shoreward, in the following 50 m, the freeboard decreases with similar trend for both sections, but the water depth over the caissons top is considerably lower for section A, which enhances breaking and energy dissipation, thereby reducing considerably the wave height and its maximum crest level.
Figure 6. Variation along platform of significant wave height (top) and maximum crest level (bottom) for the 1/100 return period.
Wave loads on deck and beams
Wave loads were measured using both force transducers and pressure sensors. Force transducers provided a direct estimate of overall wave loads on UNIT 1 and UNIT 2. Simultaneous pressures were integrated over the instrumented structural elements to derive overall forces to be compared with directly measured ones. An influence area was therefore assigned to each pressure sensor, as shown in Fig. 5. Vertical forces (both measured and derived) were analysed with respect to UNIT 1 and 2 in terms of impulsive and quasi static components; horizontal loads were directly measured on seaward beam (in UNIT 1) and internal beam (first seaward beam in UNIT 2); for comparison horizontal pressures were integrated over exposed areas.

Both measured and derived forces were filtered to remove any corruption of signal due to noise and instrument drift (Cuomo et al. 2007). Subsequently a dynamic filter was applied to extract the quasi-static loads, based on the dynamic characteristics of the structure. The filtering retained though part of the impulsive component, thereby obtaining a “static-equivalent” force. In consideration of the preliminary stage of design, preliminary structural calculations were carried out using a static approach based on the static-equivalent forces, whilst retaining the original time histories for use in a more detailed and comprehensive dynamic analysis at a later stage of design. An example of vertical force time history, showing the difference between filtered and raw signal, is presented in Fig 7.

Figure 7. Time history of measured uplift force on deck (section A, UNIT 1) for the 1/100 years wave condition.

Comparison of measured and derived impulsive and “static-equivalent” forces (1/250 values) related to section A and B is shown in the following Fig. 8, 9, 10 and 11.
Figure 8. Measured/derived vertical impulsive forces versus $H_s$, on UNIT 1 and UNIT 2 deck, for section A (left) and section B (right).

Figure 9. Measured/derived vertical “static-equivalent” forces versus $H_s$, on UNIT 1 and UNIT 2 deck, for section A (left) and section B (right).
Figure 10. Measured/derived horizontal impulsive forces on beams versus $H_s$, on UNIT 1 and UNIT 2, for section A (left) and section B (right).

Figure 11. Measured/derived horizontal “static-equivalent” forces on beams versus $H_s$ on UNIT 1 and UNIT 2, for section A (left) and section B (right).
When measuring wave loads, it is generally expected that forces derived by integrating pressure signals will give higher impulsive loads and shorter duration than those measured by using force transducers. This has been observed in previous studies (particularly when measuring high frequency impulsive loads) but comparative studies are relatively published (see Alderson and Allsop, 2007 for one of the few exceptions). The differences between these two sets of results will increase with increasing impulsiveness of the loading. This is confirmed in Fig. 8, 9, 10 and 11, with wave-induced forces measured using force transducers being generally less intense than those derived by integrating pressure over their corresponding area of influence.

The difference between measured and derived forces is greater for impulsive than for “static-equivalent” loads, both for wave loads on decks (Fig. 8, 9) and beams (Fig. 10, 11). In these cases differences in impulsive forces can reach up to one order of magnitude, whereby “static-equivalent” loads can more than double, but still remain within the same order of magnitude. This is due to the fact that the spatial coherency of loading reduces as their impulsiveness increases (i.e. the rise time decreases). The difference between measured and derived forces observed in UNIT 2 is larger than for UNIT 1 (see Fig. 8, 10), both for vertical and horizontal loads, confirming that internal elements might trap wave energy underneath the deck and experience larger impulsive loads than the external ones. The uplift wave loads on deck measured during test series B (related to cross section B) were generally higher than those measured during test series A (related to cross section A), due to the differences in wave propagation along the structure for the two sections. The difference is moderate (20% average) in terms of measured forces but becomes significant when comparing peak pressures, especially for deck elements in UNIT 2.

**Toe scour protection**

The armour movement stability tests were carried out for section A and B for the 1:5 through to the 1:500 year wave conditions. The scour protection armour rock (2-300 kg) around selected caissons was photographed from fixed positions before and after each scour test part. The armour was not repaired between test parts so that all damage recorded by the end of testing was cumulative.

During the armour stability tests for both sections A and B, no obvious movement of scour protection material occurred up to testing of overload condition. The armour movement in the overload condition was concentrated at the corner of the foundation caissons and in particular at the top of the slope. This is confirmed in Fig. 12 (corresponding to testing of section A), showing differential damage at the end of the 1/5, 1/25, 1/100 and 1/500 years conditions. No noticeable displacement of the armour units was observed in any of the tests up to 100 years conditions, with the scour protection material showing only moderate (<5%) cumulative damage at the end of the test performed with the 1:100 years condition. The damage after the 1:500 years condition (overload) was nevertheless serious with 11.3% cumulative damage observed.
Figure 12. Rock stability analysis for section A: differential damage after testing for 1:5, 1:25, 1:100 and 1:500 years wave condition; numbers refer to damage during each test part.

Wave reflection at the toe of the structure
Cross spectral analyses were carried out in order to determine the incident and reflected significant wave heights in front of the open platform structure. Reflection coefficients ($C_r = H_{sr}/H_{si}$) were quantified using an array of 3 in-line gauges placed in front of the structure: results are summarized in Table 2.

<table>
<thead>
<tr>
<th>$H_s$ (m)</th>
<th>$C_r$ (section A)</th>
<th>$C_r$ (section B)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.1</td>
<td>0.66</td>
<td>0.69</td>
</tr>
<tr>
<td>4.1</td>
<td>0.64</td>
<td>0.66</td>
</tr>
<tr>
<td>5.0</td>
<td>0.63</td>
<td>0.68</td>
</tr>
<tr>
<td>6.1</td>
<td>0.66</td>
<td>0.69</td>
</tr>
</tbody>
</table>
CONCLUSIONS

A new multi purpose platform will be built in Vado Ligure (Italy) to create docking and operating space for container ships, oil and coal bulk carriers. The outer seaward surface will be formed as an elevated deck on cross beams, suspended on vertical piles, themselves supported by submerged caissons. As the clearance of the deck above the still water level is limited due to landscape constraints, large wave-in-deck loads are expected during storms. Due to the expected complexity of the wave-structure interaction, a pseudo 3D physical model was carried out to assess both pulsating and impulsive wave forces acting on the superstructure and its foundations.

Test results showed that wave height decays dramatically within the first 50 m as the wave travels shoreward, (first two rows of seaward caissons) due to depth-limited breaking and destructive interference over the top face of the caissons. Wave loads from direct force measurements and integrated pressures were analysed at 1/250 significance levels, for two selected typical sections of the structure having different geometrical features. Wave load time-histories recorded on the deck and beam elements exhibited the characteristic pattern with short duration peaks followed by a quasi-static oscillation. Forces measured using transducers appeared less intense but longer lasting than those derived by integrating pressure over their corresponding area of influence, due to a combination of dynamic response of the physical model setup and to the necessarily discrete distribution of pressure sensors over the monitored area. As for wave decay along the structure, uplift wave loads on deck for cross section B were generally higher than those measured for cross section A.

No noticeable displacement of the rock protection at the toe of the submerged caissons was observed in any of the tests up to the 1:100 year conditions; the damage after the overload 1:500 years condition was nevertheless serious. Reflection coefficients of the structure were quantified in the range of 0.6 - 0.7.

ACKNOWLEDGMENTS

Support from Technital SpA and HR Wallingford Ltd is gratefully acknowledged. The authors wish to thank Guido Fiorini (Technital), John Alderson and David Robinson (HR Wallingford) for their help and the many interesting discussions during this project.

REFERENCES


