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Why do suspended deck coastal structures keep failing

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WHY DO SUSPENDED DECK COASTAL STRUCTURES KEEP FAILING

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Abstract

When complex structures are attacked by severe waves and/or surge, particularly severe loading patterns may result. This paper provides a brief introduction to wave-in deck loads, including a listing of the latest and most significant literature relating to wave-in deck loads on pier, jetties and suspended bridge decks. A pier in the inertial zone subjected to problematic impulsive wave loads is presented along with investigation into remedial measures to limit further or future damage to the exposed structure. For such dynamic wave load cases, physical model testing represents the most appropriate tool for the analysis of the problem and optimisation of design solutions and where therefore used in the investigation.

1. Introduction

A surprising number of suspended-deck coastal structures, including highway bridges, piers, jetties and wharves, have been damaged by wave action in recent years. This damage is despite wide availability of design guidance on "wave-in-deck loads" from offshore and coastal engineering practice, and good understanding amongst specialists on the severity of such wave loads. This prompts three questions: a) what caused the failure loads; b) could those loads have been anticipated; and are owners / engineers ignoring the issue?

2. Wave-in deck loads on exposed jetties and piers

Wave loads acting on a deck, beam or other projecting elements (including fenders) can be defined "wave-in-deck loads", summarised as in Figure 1:

- uplift loads on decks;
- uplift loads on beams or other projecting elements;
- downward loads on decks (inundation and suction);
- horizontal loads (both seaward and shoreward) on piles or other projecting elements.

Wave loadings can vary significantly for different structures and wave conditions. Structures mostly formed by vertical or slender horizontal elements (piles, beams and fenders) may be susceptible to horizontal loads (F_h), but structures with decks or larger horizontal elements (such as jetties, piers, or highway bridges) are generally more exposed to vertical wave loads (F_v) are generally relatively slender. These vertical wave loads can be substantial when structures are placed close to the still water level.

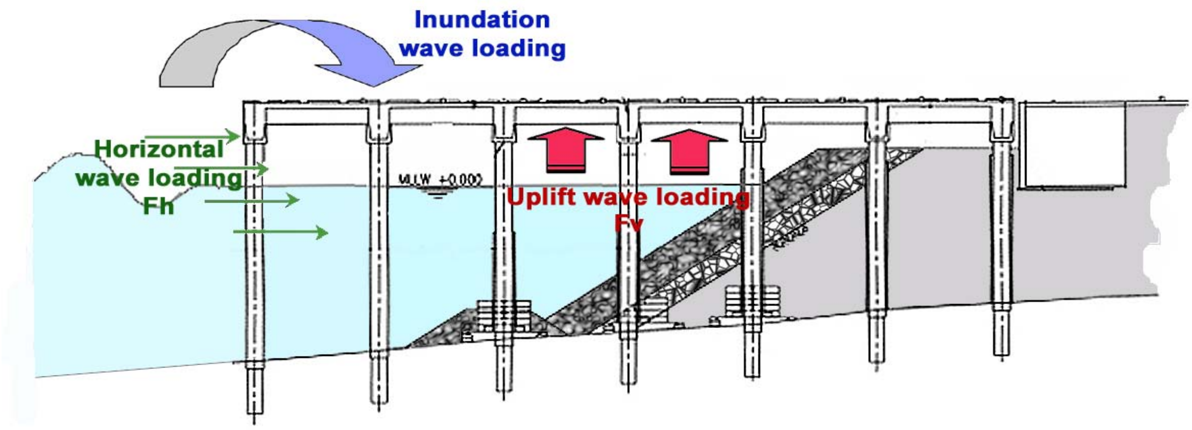

Figure 1. Wave-in-deck loads on a platform supported by piles.

3. Katrina, lessons learned ?

Hurricane Katrina hit the coasts of Alabama, Mississippi and Louisiana at the end of August 2005, causing extensive damage to infrastructure along the coast. A number of highway bridges were severely damaged, many of them had several spans lifted and displaced, see examples in

Figure 2. Wave damage to bridge decks, caused by Hurricane Katrina.

Figures 2. Previous Hurricanes, Camille (August 1969) and Ivan (Sept. 2004) caused similar damage to highway bridges on I-10 and US-90 along the Gulf coast.

During these storms, large surges led to normally sheltered areas being exposed to increased wave action, which with elevated water levels caused failure of bridge decks. The numerous failures during Hurricane Katrina again highlighted the following:

Wave-in-deck loads on coastal structures such as jetties, piers and coastal bridges suspended slightly above the still water level can be critical;

- Extreme design conditions must take account of both high surge and "strong"-wind generated waves):
- Extreme surges:
  - reduce the "gap" between the still water level and the suspended deck;
  - increase any depth-limited wave height limits, thus allowing higher local incident waves;
- Design methods for bridges did not appear to take account of wave forces at time of original design;
- Bridge designers appeared to be unaware of guidance on wave-in-deck loads.

4. Prediction methods for wave-in deck loads

Wave-in-deck loads

Over the last fifty years, many prediction methods have been developed to evaluate wave-in-deck loads on maritime structures, and useful reviews are given by McConnell et al (2004) and Cuomo et al (2007). Significant works started as far back as 1950, with most studies using physical models to explain key processes and derive prediction methods. The most widely used prediction methods by the offshore community are described by Kaplan et al. (1992, 1995) and Bea et al. (2002) who developed semi-analytical models to evaluate wave-in-deck time-history loads on both vertical and horizontal members, based on both theoretical considerations and physical model tests of wave forces on flat decks and horizontal beams.

Whilst most research into wave-in deck loads has concentrated on elements of offshore structures, prediction methods are also required for near-shore structures such as piers or jetties. For these structures, wave transformations over local bathymetry (shoaling, refraction, depth-limited breaking) and other non-linearities can have significant effects on wave loads. This is particularly true for coastal piers and bridges, where the bathymetry will have varying water depths, and where approach lengths often dictate sections that are built very low over the water level. In these situations, methods presented by Kaplan et al. (1992, 1995) can lead to under prediction of effective wave loadings (see Trindelli et al., 2002, Cuomo et al., 2003).

Bearing this in mind, Cuomo et al. (2007a) refined the prediction methods for wave-in-deck loads on exposed jetties and piers previously suggested by McConnell et al (2004), then extended by Cuomo et al. (2007b) to the case of coastal bridges with vented or solid decks. The method has been successfully applied to explain failures of highway bridges (Allsop et al., 2006) where calculated wave up-lift loads successfully predicted failure for a low level section and survival of a high-level section. Similarly, Tirindelli et al. (2007) analysed failure of jetty pier terminals.

Pulsating or Impulsive loads

Predicting the effect of wave loading on structures can be a difficult process. In the design of a structure, slowly-varying wave loads, ‘pulsating’, should usually be treated as static loads without the need to consider dynamic effects. Typically pulsating loads can be associated with structures placed seaward of the surf-zone, but dynamic loads may need to be considered in some offshore conditions.

Short duration or ‘impulsive’ wave loads are generally of greater intensity than pulsating loads, but are much shorter in duration. Impulsive loads are generally associated with breaking waves, with greater magnitude loads being generated by plunging waves (particularly on impermeable vertical wall or slope structures). Such loads should not be treated as static, and their effects should be analysed taking account of the dynamic characteristics of the structure, as well as the load magnitude and duration. Load durations shorter than the natural response period of the structure or structure element in question will be damped, and reduction should be taken into consideration when selecting design loads.

Predicting wave loads

The greatest wave load problems for these classes of structure arise where the designer did not expect the waves to reach the deck. In those instances, the structure does not just receive a load slightly larger than anticipated, but it is subjected to a substantial load of position / magnitude / orientation that were not expected at all. As most design loads for piers jetties are dead loads downward, wave induced loads sideways or upward can easily cause dramatic failures, see Figure 2. So, not accounting for wave-in deck loads is very likely to cause problems to the structure stability and safety. On the other hand, predicting wave-in deck loads can be complicated, especially if such loads can be impulsive. Some of these complexities are summarised below:

- impulsive loads can vary spatially over the structure, often acting locally, but in some cases affecting the whole structure.
- It can be complicated to predict the maximum crest elevation in shallow and intermediate water depth region due to wave profile non-linearity, and breaking.
- Local changes to bathymetry (even during a storm) and/or to water level will alter wave breaking processes, so uncertainty within this information lead directly to uncertainties on loading. Tide levels are generally predictable, but the magnitude and phasing of surges are much more difficult to predict with certainty. These uncertainties therefore influence the prediction of maximum waves and increase difficulties in predicating wave-in-deck loads.
- Wave reflections from adjoining coastal structures or elements: i.e. bridge / pier support or natural features such as a cliff, can influence wave crests and hence wave-in deck loads. The structure itself can also alter local wave processes: e.g. downward facing beams or protruding element as seen at the Pier at Blankenberge (see below).
- Locally extreme ("freak") waves may be particularly important for offshore installation, although they may be unpredictable, they are luckily rare.
- Simple buoyancy effects can cause problems if not anticipated. Extreme surgees can submerge (or part submerge) a structure, reducing its resistance to sideways loads, although buoyancy alone is unlikely to cause failure without other loads.
- Influences of tide and estuary currents on waves can be difficult to predict, but may be important where wave breaking can be changed from pulsating to impulsive when driven against a strong current.
- The influence of trapped air can affect the wave-in deck loads. Air entrapment can cause the structure to be loaded differently to when no air is trapped.

5. The intriguing case of the Blankenberge Pier

Construction of a pier at Blankenberge started in February 1894 and in August 1894 the first visitors were allowed onto the pier. In October 1914 the first pier was set alight by Germans because of its strategic position, nevertheless, the skeleton of the pier remained in place until after the First World War. In 1930, the city decided to build a new pier, which was opened to the public in July 1933. It remained intact during the Second World War and was well used after the war, see Figure 3.

Pier's history

Figure 3 Blankenberge Pier photographed in the 1950s

Figure 4 Pier before (left) and after its most recent modernisation

Modernisation was needed in 1999 when the pier's head began to show signs of wear and tear. Renovation work was completed between 1999 and 2002 with the construction of a lower floor and central concrete cylinder that reaching below sea bed level. As well as the expansion downwards a new finish was given to the building. The pier can be seen before and after the renovation work in Figure 4.

During the winter of 2002 / 2003 the structure was damaged. Non-structural damage was caused to internal partition walls as a 10.7t reinforced concrete floor slab, with a stiffness of 3×10^8 N/m was lifted up in position. The location of the floor slab can be seen in plan in Figure 5. It was suspected that the damage was caused by wave impact loads as depth limited wave are rarely seen above $H_s \approx 3$m and therefore pulsating loads were unlikely to move the slab. The wave loading was therefore investigated in both prototype, see Verhaeghe et al (2006) and in physical modelling at Wallingford described below.

Following 2-dimensional flume tests (regular waves) in 2003, and field measurements by academics in Belgium in 2003-2004, Hydratec (Paris) suggested a number of possible solutions to reduce wave impacts on the underside of the pier. HR Wallingford was then asked to carry out 3-dimensional physical model tests to assist in the optimisation of protective wave screens to ensure the stability of the pier slab against wave up-lift.

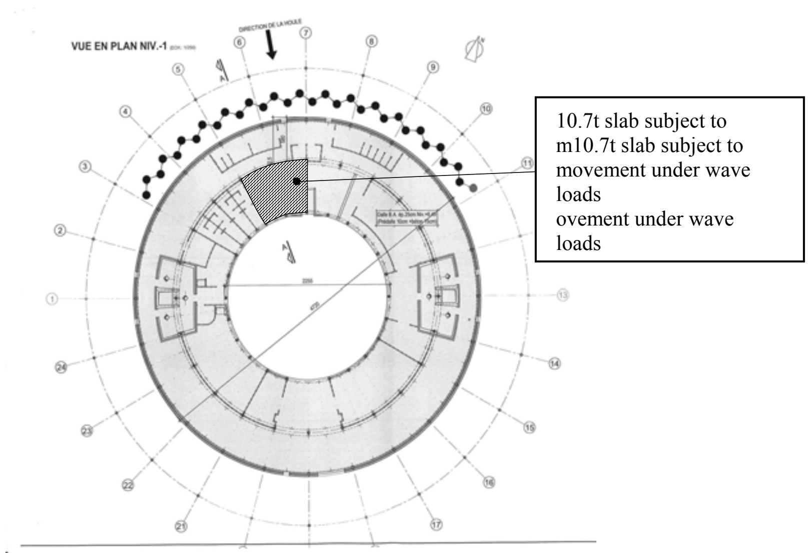

Figure 5 lower level subjected to wave impacts

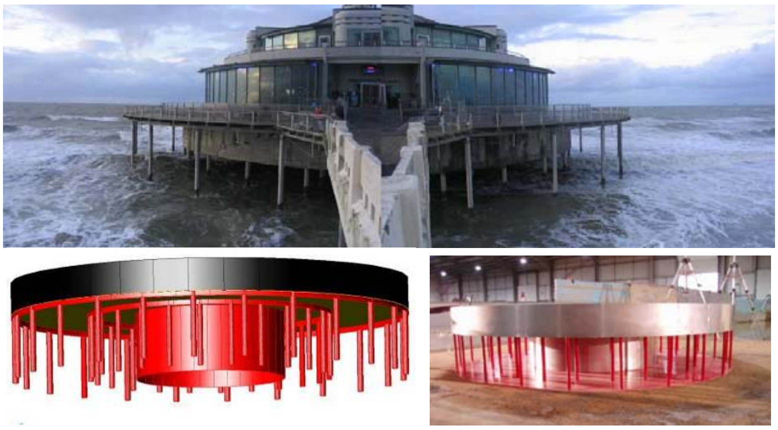

Figure 6. Structural elements (bottom, left) and 3D physical model test set-up

Physical model set-up

Physical model tests at Wallingford measured wave-in-deck loads on the Pier head, and screen elements (Figure 6). 3-dimensional physical model studies, see Figure 7, with a mobile random wave generator (able to absorb reflected waves) were carried out to assess:

- Wave disturbance around and underneath the pier;
- Effectiveness of modifications to the pile-supported protection screen;
- Wave induced up-lift loads on the deck of the Pier building;
- Wave loads on the protection screen.

Being in the inter-tidal zone in relatively shallow water, the Pier is subject to fully refracted conditions giving long-crested waves running normal to the shoreline (approximately 330ºN). Correct modelling of the key wave processes required a 3-dimensional model of the pier head and local sea bed, reproduced using the surveyed bathymetry data over a distance equivalent to 330m from the wave screen. At 1:20, the model scale selected was relatively large, ensuring correct reproduction of all wave processes governed by gravity / momentum forces, and minimisation of scale effects caused by viscosity and surface tension effects. A number of structural modifications were investigated, tested and analysed in an attempt to better understand (and reduce) problematic wave loading to the underside of the jetty head.

Wave condition reproduced in the model corresponded to those measured by University of Gent at the site, and in particular, to offshore H_{so} = 3.6 m, giving H_{si} = 2.8m at the pier head, wave period T_m= 7.2s and a total water level = +5.3m TAW. Additional series of wave conditions were used to investigate the influence of minor changes in wave period T_p, wave height H_s, water level and wave return period. Measurements of waves, pressures and screen forces were recorded for 1000 waves, corresponding to approximately 2 hours (prototype). In reality, the period of aggressive attack can occur for only short durations due to changing tidal levels. The longer duration used in the model allowed better confidence in the extreme statistics of the measured loads.

The model structure represented the inner cylinder of the pier, the floor at -1 level, the external wall rising from the -1 level to the walkway at +9.9mTAW, two radial beams that support the slabs at level -1 and the elevator shaft, see Figure 8. Piles to hold the protection wall screens seaward of the pier were installed allowing modified screens to be installed between the piles, see Figure 9.

Figure 7. Physical model layout

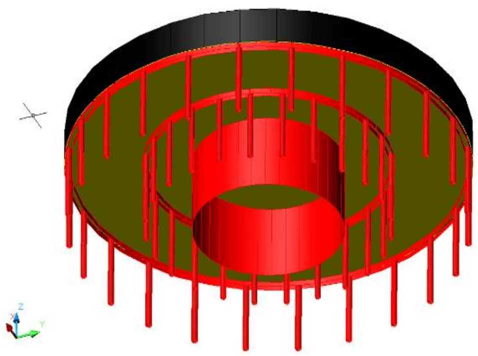

Figure 8 Views of the pier head as constructed in the model

Figure 9a Without wave screen

Figure 9b With present wave screen

Figure 9d 16% porosity to the lower section

Figure 9c 16% porosity to the upper section

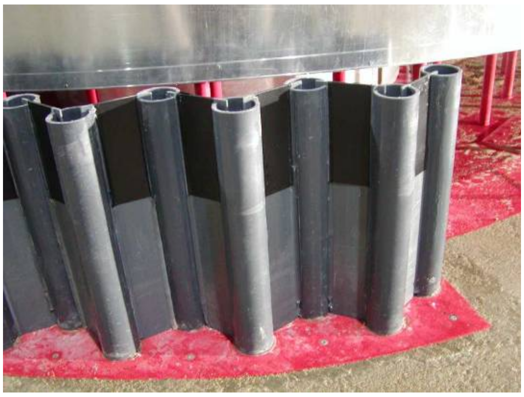

Figure 9e Fully closed

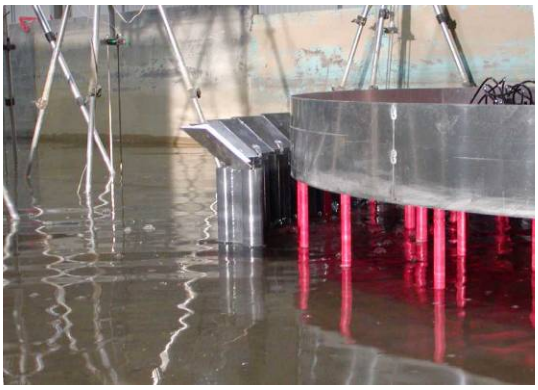

Figure 9f Return wall mounted on 16% porous lower section

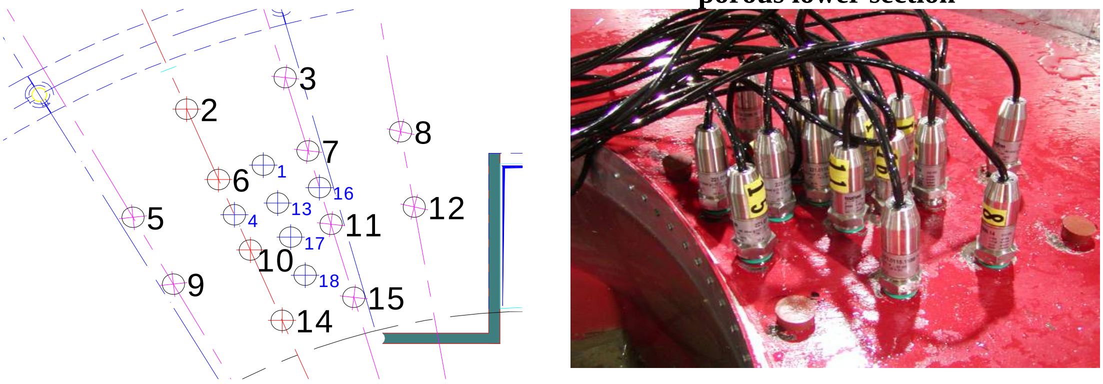

Figure 10 Array of pressure transducers used to measure wave-in-deck loads

HRW used two approaches to measure wave forces / pressures at Blankenberge Pier. The pier deck was instrumented using 18 downward-facing pressure transducers (Figure 10). In order to facilitate comparison with existing field measurements, one of the arrays (transducers 2, 6, 10 & 14 in Figure 10) was placed in the same location as the field measurement pressures transducers by Gent University. Loads on the piled wave screens were measured over a representative section mounted on 3 load cells giving the net horizontal load, and moment about a given point (-0.12mTAW). The measured load acted over four piles.

Outcome from physical model tests

Observations during the model tests supported the contention that energy was focusing on the problem slab. The processes of wave loading on the structure started with wave run up against the centre of the structure hitting the basement (-1 level) floor at the vertical wall intersection. As the water level increased, larger impacts occurred as waves were focussed and trapped around the lift shaft. Wave loads became less impulsive as the basement level was almost drowned by the rising tide. An example time history of an impulsive event is given in Figure 11 along with its spatial variation.

The protection wall did reduce waves at the surface. General water levels behind the protection wall remained around mean water elevation being relatively unaffected by incoming wave troughs. Water levels were seen to increase behind the protection wall as reflected waves rebounded from the central cylinder back to the wall. Following these initial investigative tests, further testing explored the potential to optimise the defence, summarised here in the Appendix to this paper.

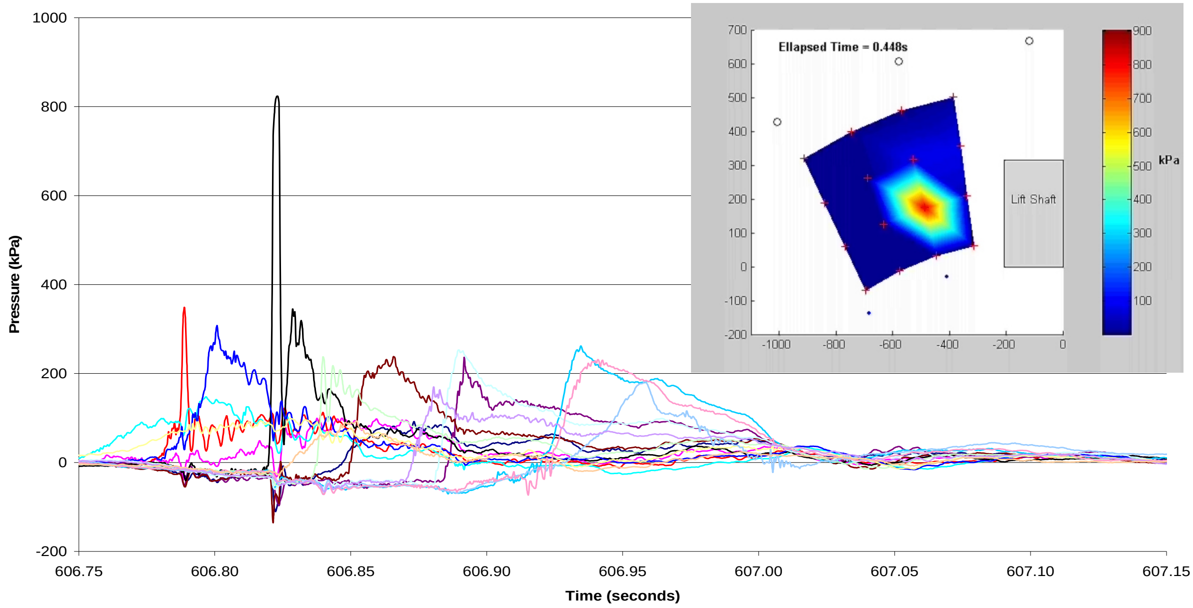

Figure 11 Time series & spatial extent of wave-in deck load on pier without protection

Footnote

On 9 November 2007, a North Sea storm forced Rotterdam port authorities to close the giant Maeslant surge barrier for the first time as high surge levels were experienced along the Belgium and Dutch coastlines. At Blankenberge, the higher than normal water levels allowed large waves to reach the structure. It is likely that the protection wall in front of the Pier head was almost submerged as waves crashed over it with significant overtopping projected upwards, as had been previously predicted in the model tests at Wallingford. Staff from HRW at the site shortly after the peak of the storm photographed up-lift damage to the walkway which projects from the pier at +9.9mTAW, see Figure 12.

6. Concluding remarks

A brief introduction to wave-in deck loads has been given, including a listing of the latest and most significant literature relating to wave-in deck loads on pier, jetties and suspended bridge decks. General tools are now available that allow the assessment of wave-in-deck loads for analysis / design of many suspended deck structures above static water.

When complex structures are attacked by severe waves and/or surge, particularly severe loading patterns may result. For such cases, physical model testing still represents the most appropriate tool for the analysis of the problem and optimisation of design solutions.

The key lesson learnt is that building any structure with a deck close to the water requires sufficient understanding in the hydrodynamic effects of both impulsive and pulsating wave loads, as well as an understanding of the structures dynamics as impulsive wave-in deck loads are likely to be significant.

Of continuing concern is the frequency with which owners / engineers place a deck above water without correctly assessing the risk (probability x consequence) of being hit by wave action. For the highway bridges

Figure 12 Wave up-lift damage to outer walkway, 9 November 2007

along the Florida, Alabama, and Mississippi coastlines, the consequences of such storms have generally been complete destruction as each re-build has required new piles even if it was only the deck lifted. The excuse given at a Federal Highways workshop that there was no guidance available for "highway bridges" is less than convincing given that Hurricanes Camille and Ivan had both previously destroyed highway bridges along these shorelines.

The reasons for the original problem at Blankenberge are perplexing as there surely cannot have been any doubt as to its exposure to waves? So why was it imagined that the addition of a large central core acting as a vertical seawall would not adversely affect the structure?

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Appendix – results of optimisation model tests

Wave conditions corresponded offshore H_{so} = 3.6 m, H_{si} = 2.8m at the pier, wave period T_m = 7.2s and total water level = +5.3m TAW. Measured waves, pressures and screen forces were recorded for 1000 waves, corresponding to approximately 2 hours (prototype).

The model represented the inner cylinder of the pier, the -1 floor, the external wall from -1 to the walkway at +9.9mTAW, two radial beams that support the slabs at level -1 and the elevator shaft. Piles to hold the protection screens were installed allowing modified screens to be installed between the piles. Observations clearly indicated that the wave screen was reducing both the frequency and severity of waves hitting the underside of the structure. Measured pressures gave a significant reduction in impact load, see Table 1

Table 1 Reduction factor for the representative up-lift force

	Without protection	Present protection (Coarse array)	Present protection (Fine array)	Reduction Factor
max (0.1%)	1700 kN	910 kN	890 kN	1.9
1/250	1560 kN	850 kN	800 kN	1.8 – 2.0

A 16% porosity plate replaced the solid section over the upper part of the protection wall and the 4.1m solid screen was moved to the base of the protection wall. Wave reflection from the protection wall increased. Measured wave heights in front of the wave screen saw values of H_s grow by approximately 20%.

Table 2 Reduction factor for the representative up-lift force

	Present protection	16% porosity to the upper section	Reduction Factor
max (0.1%)	910 kN	390 kN	2.4
1/250	850 kN	360 kN	2.4

Introducing the 16% porosity at the top of the pile system, reduced the representative force by approximately 2x, see Table 2.

The protection screen was changed with the 16% porosity plate forming the lower section of the protection wall and the 4.1m solid section was placed at the top. This wave screen configuration gave reduction of approximately 2x, see Table 3.

Table 3 Reduction factor for the representative up-lift force

	Present protection	16% porosity to the lower section	Reduction Factor
max (0.1%)	910 kN	450 kN	2.0
1/250	850 kN	400 kN	2.1

The configuration with fully closed screens over the whole of the protection wall increased measured value of H_s in front of the structure by approximately 30%, when compared to the present configuration. Closing the gaps between the piles completely, gave a reduction of x2, see Table 4.

Table 4 Reduction factor for the representative up-lift force

	Present protection	Fully closed wave screen	Reduction Factor
max (0.1%)	910 kN	470 kN	1.9
1/250	850 kN	390 kN	2.2

To reduce overtopping and further deduce the impact wave loads a design incorporating a 45° tilted return wall mounted on the top of the protection wall (configured with 16% porosity at the lower section of the screen) was tested.

The return wall was observed to perform well even though waves still overtopped the crest. Approximately 1 – 5% of waves overtopping reached above the +9.9m TAW whereas previous modifications approximately 20 – 50% of the waves overtopped. The effectiveness of the return wall reduced wave impacts significantly, as shown in Table 5.

Table 5 Reduction factor for the representative up-lift force

	Present protection	Return wall	Reduction Factor
Max (0.1%)	910 kN	140 kN	6.7
1/250	850 kN	90 kN	9.5

A rapid comparison of the Gent University field data with pressures extracted from this study suggests that any reduction of (model-derived) impact pressures would be small.

Table 6 Peak pressures measured in the field

Sensor	Max. pressure (kPa)	
	8 feb. 2004	12-13 nov. 2004
4	-	469
5	350	347
6	235	350
7	103	186

It is not possible to give direct comparisons, but peak pressures measured by Gent university, and measured over about 50-100 waves (10 minutes at $T_p \approx 8$s, $N_z \approx 75$ waves) are shown here in Table 6. Impact pressures measured at equivalent locations in the model are shown at 1/250 level (mean of top 4 pressures in 1000 waves) may be compared in Table 7.

Table 7 Peak pressures measured physical model

Transducer	Pressure (kPa) at 1/250 level (1000 waves)	
	Present protection (Coarse array)	Present protection (Fine array)
14 (4)	360	520
10 (5)	390	390
6 (6)	340	480
2 (7)	210	240

Given the high level of stochastic variation to be expected in comparing peak values of impulsive loads from very small data sets, there is a significant degree of agreement between the data in Table 6 and 7, both in absolute values, and in spatial comparisons. It would be logical to expect that 1/250 values from the field (if that were possible) would exceed the values shown in Table 6. Consideration of some of the extreme pressures measured in the model however confirm the general expectation that individual impact pressures will be variable, some exceeding the values shown in Table 7. Comparisons between similar tests in the model have also shown occasionally higher peak values.

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