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STRUCTURAL RESPONSE DUE TO WAVE IMPACTS ON A COASTAL PROTECTION

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Because of a new Master Plan for Coastal Protection, most Belgian coastal cities are planning new protective measures for their beaches and coastal promenades. One of the first locations for these new measures is the coastal town of Wenduine. Three separate new constructions will be designed to protect the town as well as the coastal region for the following 50 years. As opposed to purely vertical or horizontal surface structures, structures consisting of both vertical parapets and horizontal slabs have rarely been considered. The behavior of these structures is more similar to bridge conduct, since the incident loads vary with time and space, than to classic coastal structures. The effect of wave impact on these structures results in local patch loading on a swaying system, introducing local deformations, stresses, and accelerations. This patch loading may cause local deformations, but does not necessarily cause collapse or endanger neither structural equilibrium nor stability. This article studies the structural response of monolithic concrete coastal protection structures. The focus is on the structural response of the structure, comfort conditions for use, and structural safety. This is analyzed by numerical modeling of the structural response based on recently proposed design values for such wave loading. These models include the actual structure as well as the influence of foundations and ground layers.

Keywords: Dynamic calculation, FEM, Horizontal and vertical wave loads, Civil infrastructure.

1 INTRODUCTION

Because of a new and integrated Master Plan for Coastal Protection, most Belgian coastal cities are planning new protective measures for their beaches and coastal promenades. One of the first locations where these new measures are implemented is the coastal town of Wenduine, part of the larger town of De Haan. Based on the principles of the Master Plan, solutions have to be combined: strand suppletion as a low-impact measure, and construction of a storm wall as a high-impact solution. It was decided to widen existing seawall by about 10 m in the western part and 3 m in the eastern part. Three separate new constructions need designs with the perspective of protecting the town as well as coastal region for the following 50 years. They are linked to the three separate parts of the Wenduine coast line: the “Rotunda”, which also acts as a windshield; the western part of the seawall, which is used intensively by shops, hotels and restaurants; and the eastern part, which is at the quieter end of the village.

All hard impact measures are designed to withstand a theoretical storm with a return period of 1000 years and an overtopping level, which is 8m above the normal still water level. This return period is far stricter than relevant codes, but was demanded by the

Flemish Government (Schotte 2013a, Schotte 2013b, Schotte 2013c). This resulted in the following specific measures that had to be undertaken, also shown in Figure 1:

- At the "Rotunda", the existing wind shield was strengthened and unified so that it formed a linear structure acting as a storm wall;
- Based on physical model testing, it was decided that the protection of the western and eastern part of the sea wall would consist of a double wall system, placed 10 m apart, to reduce the overtopping flow. Both walls actually work as a stilling wave basin. Since this necessitates a widening of the sea wall, the frontal structure will also have to withstand horizontal pressures. The second wall will be disguised as a tourist bench. Both walls are circled in Figure 1.

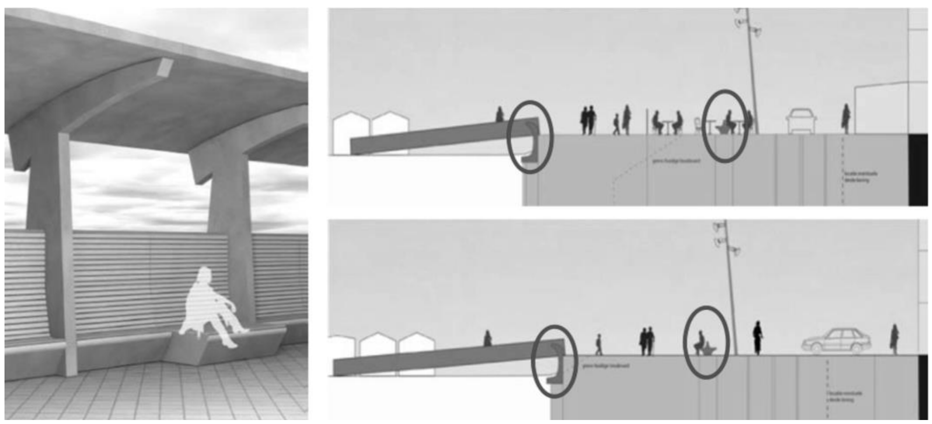

Figure 1. Artist impression of the new infrastructure necessary for the Wenduine storm wall: "Rotunda" (left), 10 m widening in the western part (above) and 3 m widening in the eastern part (below).

2 GEOMETRY OF THE STORM WALLS

Three separate structures were studied in detail. The "Rotunda" structure offered some design problems, since the inclusion of a deeper foundation, which was necessary based on preliminary calculations, was difficult to construct because of the roof structure that had to be preserved. After numerous variation with respectively very wide foundation plates, foundation plates with a toe structure, foundations with secant piles, etc. A solution with a deeper and heavier foundation block was finally chosen.

The frontal wall at the rest of the sea wall was designed using deep foundation, i.e., a secant pile wall, going deep in the clay layer, which starts at about 7.5 m under the sea wall structure. The secondary piles were strengthened using steel HEA300 profiles. The soil pressures will be represented in the finite element model using springs. The second part of the double wall structure consisted of a bench structure supported by an identical secant pile wall.

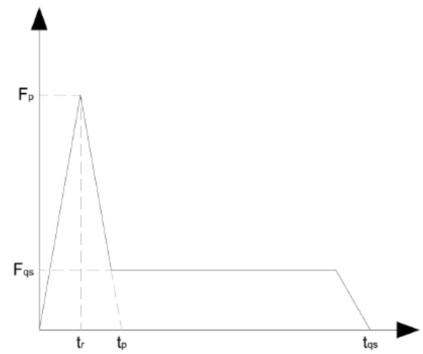

Figure 2. Church roof diagram of the typical horizontal wave loads acting on coastal structures.

3 CONSIDERED LOADS

The wave load, which forms the starting point for these calculations, was determined based on physical laboratory testing. The measured wave forces are dynamic impact loads characterized by a very abrupt peak load, followed by a much lower quasi-static load. This load pattern can described as a church roof diagram as per Figure 2. A number of different wave profiles were filtered out of the laboratory testing and used as input loads for the finite element calculations. One of the most relevant ones for the seawards storm wall is shown in Figure 3(a):

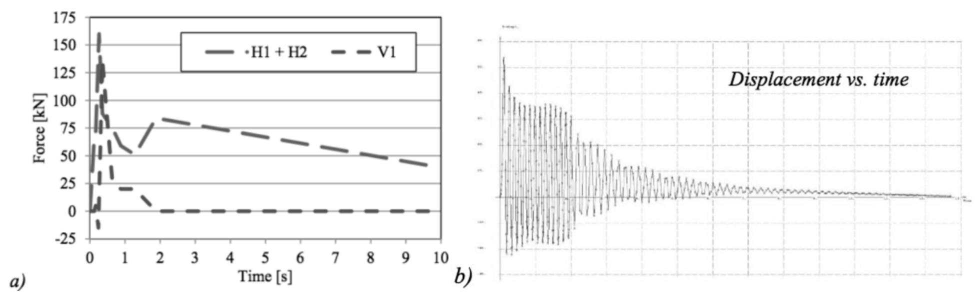

Figure 3. Forces on the seawards wall: (a) Wave pattern 1; (b) Dynamic response because of wave pattern 1.

4 FINITE ELEMENT MODELLING

Based on the geometries discussed above, a detailed finite element model was developed for all three structures in the multipurpose finite element software package SAMCEF. The soil pressures are introduced using spring and based on the assumption that the spring constants vary linearly over the thickness of the considered layer. In order to assure vertical stability of the structure, friction between soil and structure was also modeled. It was based on an internal friction angle of $\varphi' = \varphi/3$ for the foundation plates, which have a smooth surface, and $\varphi' = 2\varphi/3$ for the secant piles, which have a much rougher surface. The sand suppletion layer was actually modeled instead of being considered as an additional load. This was important in order to include all dynamic properties of the structure (i.e., inertia, ea.) and the surrounding soil, which is forced to follow the movement of the storm wall structure. The secant pile wall was modeled as a concrete wall, but the stiffness was changed based on the actual concrete strength and the inclusion of steel-beam profiles. It was assumed that the horizontal forces were acting on the entire front of the considered structure. For the frontal sea wall, equipped with a parapet structure, it can be assumed that additional vertical components of the wave forces will act in an upwards direction.

Table 1. Influence of a variation of soil characteristic; stresses in MPa, displacements in mm.

	k_{min}	$k_{average}$	k_{max}
Wave profile 1			
Max. displ.	**53.82**	35.65	27.03
Max. stress	15.11	15.06	14.65
Wave profile 2			
Max. displ.	46.69	31.63	24.65
Max. stress	**20.99**	20.6	19.86
Wave profile 3			
Max. displ.	48.22	34.04	-
Max. stress	11.96	12.64	-

Table 2. Dynamic vs. static calculations; stresses in MPa and displacements in mm.

	Dynamic	**Static**	**Ratio**
Wave profile 1			
Max. displ.	53.82	80.23	67%
Max. stress	15.11	25.72	59%
Wave profile 2			
Max. displ.	46.69	61.04	76%
Max. stress	20.99	22.01	95%

5 SENSITIVITY ANALYSIS

This section will focus on the result of the seaward storm wall. Results of the other structures are quite similar. Much uncertainty exists about the actual values of the spring constants representing soil pressures. Since they could not be determined by *in situ* testing, some assumptions had to be made based on literature. The sensitivity of the structure because of this parameter was studied for different wave profiles by repeating the calculation using the lowest, highest, and average spring constant given by literature. Weaker spring constants apparently result in larger displacements, combined with higher stress values within the concrete structure, as per Table 1. The most important objective of these detailed finite element calculations was to study the difference between a static and a dynamic analysis. It was assumed that a dynamic analysis would result in an optimal situation, since inertia would help to reduce the influence of the peaks in the peak of the church roof diagram. The results of this comparison are given in Table 2.

It appears as if the dynamic calculations offer considerably lower stresses and displacements. When looking at the first wave profile being considered, maximal displacements are about 33% lower in the dynamic calculation when compared with the static one. For the resulting stresses, the reduction because of considering dynamics is 41%. In addition, it is quite clear that the difference is quite marked when considering wave pattern 1, shown in Figure 3(a).

The dynamic response in terms of horizontal displacements of the top of the parapet of the seawards storm wall is shown in Figure 3(b). It is quite clear that the largest displacements occurred during the initial impact, after which the displacements slowly disappeared in about 13 seconds. The results for the two other constructions were quite similar. The landwards wall was exposed to much lower forces, but since the foundation structure was lighter, the resulting displacements had the same order of magnitude. Because of the more compact structure, displacement reduction due to dynamic calculations was less obvious. Stresses, however, were a lot lower and much more influenced by the dynamic calculations.

For the "Rotunda" structure as well, maximal displacement values were reached for wave pattern 1 and when using the minimal stiffness for the spring constants representing soil pressures. However, because of the fundamentally-different foundation system, not using secant piles (clamped within a clay layer deep below the surface) but using less deep block foundation), the static and dynamic results were comparatively different. Because of the boundary conditions, the structure could not offer enough resistance or inertia, resulting in displacements and stresses being augmented in dynamic calculations. When studying the dynamic response of this structure, the reduction of the displacements in time was extremely small.

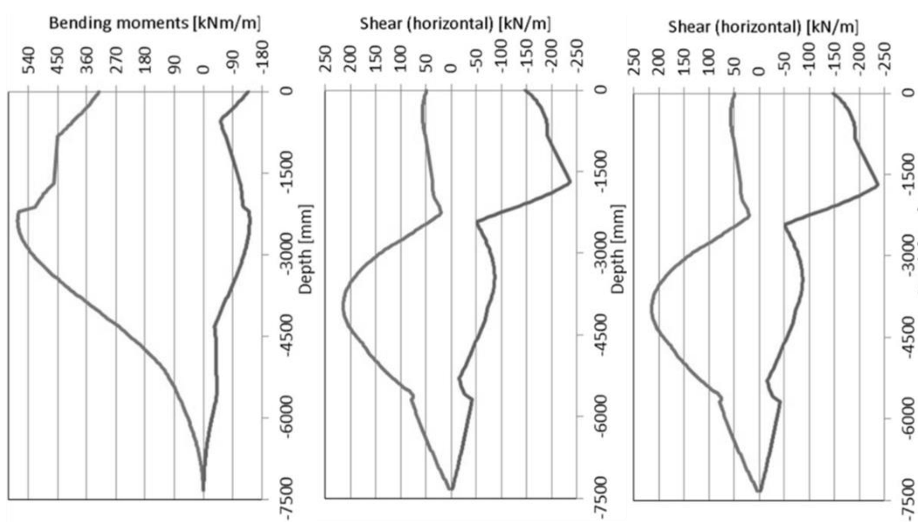

Figure 3. Internal forces within the seawards storm wall: bending moments (left), shear forces (middle) and normal forces (right).

6 INTERNAL FORCES

For design situations, working with internal forces can be a lot easier. Because of that, the resulting bending moments, shear forces, and normal forces within the parapet structure, as well as the the secant piles of the seawards storm wall, were determined for all considered wave patterns and variations of the boundary conditions. Envelope lines for all of these calculations are shown in Figure 4.

It is quite clear in these figures how the stiffness of the soil springs resulted in a total clamping of the secant piles at the bottom of the structure, where the internal bending moment is reduced to 0 kNm. In addition, the shift when going from secant piles to parapet structure, both having a totally different geometry and material strength, was clearly noticeable in bending moment and shear-force variations.

7 CONCLUSIONS

As opposed to purely vertical or horizontal surface type structures, structures consisting of both vertical parapets and horizontal slabs have rarely been considered in detailed research. The behavior of these structures is more similar to bridge conduct, since the incident loads vary with time and space, then to classic coastal structures.

The effect of wave impact on these structures results in extreme local patch loading on a swaying system, introducing local deformations, stresses, and accelerations whose peak values are not necessarily relevant for the entire structural behavior. This patch loading, moving rapidly with time as an upward pressure front, while acting on the lower side of the structure, may cause local deformations. But the loading does not necessarily cause collapse or endanger structural equilibrium or stability. The pressure wave is a frontally-moving load, limited in time and space. Fundamental insights in the structural response of these coastal protection structures necessitates a detailed calculation of the dynamic and static structural responses because of the wave impact. This research has shown that the influence of boundary conditions and soil characteristics is extremely important in deciding whether the dynamic calculations will have a positive or negative effect on the design values to be considered.

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