PHYSICAL MODEL TESTING OF AN INNOVATIVE COBBLE SHORE,
PART I: VERIFICATION OF CROSS-SHORE PROFILE DEFORMATION

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In early 2008, the Port of Rotterdam Authority awarded the first DCM contract of the Maasvlakte 2 project to PUMA, a joint venture of Boskalis and Van Oord. In this mega port extension an innovative sea defense has been designed and is being constructed: a 3.5-kilometer dynamic cobble shore with a foreshore reef of re-used blocks. The role of physical model testing in this design optimization and verification has been described in two Coastlab2010 papers. Part I deals with the cross-shore design aspects of a thick layer of cobbles, covering a sand core with a beach and dune profile.

Keywords: Maasvlakte 2, sea defense, cobble shore, foreshore reef, integral design, verification, cross-shore profile and physical model.

1. Introduction

In early 2008, after more than a decade of all-embracing project preparation and after having invited tender in 2005, the Port of Rotterdam Authority (PRA) awarded the first phase of the seaward expansion of the existing port area to PUMA, a joint venture of the Dutch dredging contractors Boskalis and Van Oord. This port extension project is known as Maasvlakte 2, a mega reclamation, deepening and construction work, which will create around 2,000 hectares of harbor area in virgin marine waters up to 18 meters deep. Expanding the port by about 20% (see Figure 1) will in particular meet the deep-draft needs of the container handling sector, accommodating the planned 12,500 TEU vessels in harbor basins with a nautical depth of 20 meters.

The scope of this DCM (design, construct and maintenance) contract includes an approximately 11-kilometer sea defense. The DCM contract stipulates completion of the design and construct work in 2013 and 5 years’ subsequent maintenance of this sea defense.

Unlike for traditional construction contracts, this DCM bid requires PUMA to come up with its own design, based on PRA’s functional requirements. In line with the current trends in DCM contracts, PRA decided in 2005 to itemize the Program of Requirements (PoR) in a functional and system-oriented manner, without basing it on a reference design. The specification of approximately 700 PoR items includes the operational phase (i.e. functionality, durability and object interfaces), the construction phase (i.e. environment, safety and completion delivery) and the maintenance phase (i.e. monitoring and control).

During the tender phase (2005-2007) PUMA elaborated this PoR in frequent dialogue with PRA into approximately 1,700 items of the Requirements Tree and Verification Matrix (RTVM). This working method, being innovative in DCM tendering, resulted in the RTVM as a bid document as well as a contract document. Each of these RTVM items had to be described as sufficiently SMART (specific, measurable, accountable, realistic and time-constrained). The RTVM indicates per elaborated requirement item - in a traceable and quality assured way - the verification method, the specific criterion and the document in which the verification assessment is described. During the contractual design phase up to 2010 the design of the stone part of the sea defense was further optimized and verified.
The resulting final design of the 3.5-kilometer northern part of the sea defense consists of an innovative dynamic cobble shore with a foreshore reef of re-used concrete blocks, see Figure 1. A key design feature of this cobble shore is the use of a thick layer of quarried cobbles over a sand core. This Part I paper casts light onto the development, optimization and verification of the cross-shore design aspects. The associated alongshore design aspects are outlined in the Part II paper (Loman et al., 2010). During the tender phase as well as the contractual design phase, extensive use was made of physical model testing as prescribed by PRA for the design verification of a stony sea defense. Here, wave flumes and wave basins of qualified and independent testing institutes had to be used.

This innovative sea defense consists of a dynamic shore revetment, being a thick layer of crushed cobbles over a sand core with a typical beach-dune profile. This shore design reflects the building-with-nature principle. Namely, natural cobble shores exhibit a remarkable degree of dynamic stability in the face of sustained wave attack. The typical S-shaped cross-shore profile absorbs the wave energy effectively because of the deformability, porosity and thickness of the cobble layer. Consequently, the design concept covering a sand core by a thick layer of gravel (natural pebbles or crushed cobbles) had already been considered in the past as a man-made dynamic shore protection (e.g. WL, 1973-1977; DHL, 1975; Allan et al., 2004, and Allan et al., 2005).

The development, optimization and verification of PUMA’s cobble shore design of Maasvlakte 2 are described in the following sections:

Section 2: Elaboration of Program of Requirements (PoR)
Section 3: Design development, optimization and verification
Section 4: Discussion of results of physical 2D model tests
Section 5: Conclusions

2. Elaboration of Program of Requirements (PoR)

According to the PoR the following four main operational requirements have to be fulfilled by the 11-kilometer sea defense.

i) Geometrical requirements, restricting the location and the foot width of the sea defense

In the PoR the spatial boundaries of each Maasvlakte 2 object are specified, see grey lines in Figure 5, left. This is the outcome of a comprehensive horizontal layout study carried out by PRA. Because of the selected port expansion layout (see Figure 1) the northern section of the sea defense is located relatively close to the Maasgeul access channel of the port of Rotterdam. As such the original PoR specifies that the sea defense has to consist of a northern static hard-shore section and a southern dynamic sand-shore section with a transitional section in between. A paramount aspect of the design freedom here is that the bidder/contractor is allowed to adjust the position of the given boundary in between the hard-shore and the soft-shore, as long as all the other requirements are fulfilled.

ii) Nautical requirements, restricting the plan-form of the sea defense, in particular the transition section

The PoR stipulates that during construction and after completion of the sea defense the present high level of nautical accessibility has to be secured by meeting tight cross-current criteria for the adjacent Maasgeul access channel. PUMA has to demonstrate that the cross-currents here do not exceed specific PoR limits for the considered phased construction geometry by calculating the flow pattern of a prescribed spring tide with the prescribed detailed numerical DELFT3D model FLOW+HLES. This calibrated numerical model accurately simulates the large tidal eddy, generated by the northwards floodtide current along the concave Maasvlakte 2 sea defense, thereby affecting the cross-currents along the Maasgeul access channel.

iii) Shore protection requirements, specifying the sea defense behavior at a design storm of 1 in 10,000 years

The PoR stipulates that the harbor area just behind the sea defense may not be eroded nor washed over during an extreme North Sea storm surge, occurring once every 10,000 years. This very stringent safety requirement reflects the outcome of PMV2’s integral risk assessment of port area inundation due to storm surges on the North Sea. The PoR of the first bid (July 2006) stipulated hydraulic design conditions at a single location near the sea defense. In the PoR of the final bid (October 2007), a 1 in 10,000 years’ storm level at the Hoek van Holland tide gauge (HvH) and both a 1 in 10,000 years’ significant wave height and wave period at the Europlatform wave gauge (EPL) were given. Furthermore, a numerical wave transformation study was prescribed to be executed by the bidder following strict quality standards in order to translate the given extreme wave conditions from EPL (approximately 30 kilometer offshore from Maasvlakte 2) to the various toe positions along the sea defense shown in Figure 2. Here, the prescribed numerical SWAN model was used with a triple-nested grid of the southern North Sea. For each of these given super storm conditions the joint hydraulic parameters were established from a statistical correlation analysis of the measuring time series (1979-2005) of the spectral wave parameters at EPL and the storm surge setup at HvH. This comprehensive study revealed that the extreme water level condition causes the most severe storm peak design condition at the transition section in water depths of about 18 meter below MSL (Mean Sea Level). Figure 2 depicts the considered time development of the wave height, wave period and water level during the design storm of about 39
hours, empirically set for the Dutch North Sea coast as 10 times the maximum surge setup in meters. The storm peak design conditions (1 in 10,000 years), determined at Kp2.600 seaward of the transition section (Kp2.150 - Kp3.500) shown in Figure 3 (left), are represented by a significant wave height of 7.9 meters, a spectral peak wave period of 13.5 seconds, a main wave direction fan of 300-330 °N, a short-crested directional spreading of 25° (equal to standard deviation around main wave direction) and a storm surge level of 5.1 m above MSL including 15 years of sea level rise. Figure 2 (right) shows the variation of these storm peak conditions along the northern part of the sea defense. The JONSWAP type of wave spectrum is shown in Figure 6.

Figure 2: Time development of 1/10.000 years design storm at Kp2.600 (left) and variation of storm peak conditions along sea defense (right)

iv) Sustainability requirement, prescribing the design selection according to the lowest Total Cost of Ownership

As regards the design of the transition sea defense section the PoR stipulates a financial sustainability requirement. The bidder/contractor has to demonstrate, by developing and comparing at least four alternatives, that the selected final design of this transition section has the lowest Total Cost of Ownership (TCO), being the sum of present value of the construction costs and the present value of 50 years of maintenance costs, thereby taking account of the uncertainties.

3. Design development, optimization and verification

In retrospect, the process of development, optimization and verification, leading to the final design of the 3.5-kilometer innovative cobble shore with reef, has the following three main design phases:

i) Design development and verification during the Competitive Dialogue Phase

During this first tender phase, PUMA discussed with PRA the top-down elaboration of the PoR as well as the bottom-up development of the design. For the development of the design of the transition section of the sea defense (see Figure 5, left) at least four design alternatives had to be considered in this desk design phase. Hence due attention was given to recent research findings published about quasi-dynamic shore protections. One of the considered alternatives consists of covering a dune and beach of coarse sand with a layer of gravel (pebbles or cobbles according to the commonly used Wentworth grain-size classification).

Figure 3: Aerial view of Maasvlakte 2 under construction (2010 Q2, left) with existing gavel shore section of Maasvlakte 1 (close up, right)

Here, the design research findings on a dynamic gravel shore as sea defense section of Maasvlakte 1 (WL, 1973-1977) formed one of the triggers of a feasible development of this cobble shore design concept. This 2.3-
kilometer gravel shore, called Zuidwal, consisted of a thick 1 in 8 sloped sea gravel layer, having an average median grain-size of around 18 mm (pebbles) and a thickness up to 3.5 meters, placed on a sand core. Because of the oblique waves this constructed gravel shore needed to be recharged regularly. At present, only two wooden groin-compartmented sections exist (see Figure 3), as the gravel layer of the remaining reach was subsequently covered with pitched concrete blocks because of the owner’s preference for zero-maintenance.

The knowledge gained from this dynamic gravel shore has led to further applied modeling research, as funded by Rijkswaterstaat of the Dutch Ministry of Transport, Public Works and Water Management, and as published by Delft Hydraulics (e.g. Van der Meer, 1988). The basic design method, as derived from the Maasvlakte I findings (DHL, 1975) and as applied by the Boskalis engineers in designing a gravel protected man-made island in the Beaufort Sea, has been used here in the desk design phase.

Based on the criteria of the four main requirements specified in the RTVM, the cobble shore design alternative was selected for PUMA’s first bid. Figure 5 shows a typical cross-section design of this 1.4-kilometer cobble shore. The cobble layer is up to 9 meters thick and consists of widely graded crushed rock with a grain-size distribution, resembling the lower grading curve of Figure 4 (with a D50 of 60 mm). A 1 in 10 year-round slope has been estimated for the cobble beach following the empirical rule of Powell (1993) for dissimilar sediments. A part of the thickness of the beach slope and the thickness of the dune slope have been determined by the maximum erosion depth, predicted by the set of empirical formulae of Van der Meer (1988) for the cross-shore profile deformation, thereby including 80% extra thickness for covering the various uncertainties in this estimate. The part of the beach thickness required to accommodate the alongshore drift losses has been described in the Part II paper (Loman et al., 2010). The crest level has been determined by verifying the wave overtopping with the numerical model PC-OVERSLAG at a maximum criterion of 10 liter/second/meter.

**Design optimization and verification during the Competitive Negotiation Phase**

Having received PRA’s invitation for the final bid, the updated PoR stipulated that PUMA had to verify in this bid phase that the cobble shore design meets the shore protection requirements through physical model testing by qualified, independent and ISO-certified institutes. Hence an extensive scale analysis was conducted by the experts of the consulted physical model testing institutes Deltares, HR Wallingford (HRW) and Danish Hydraulic Institute (DHI) in close cooperation with the PUMA experts. It was concluded that the cross-shore profile deformation, the alongshore cobble drift as well as the turbulent wave-damping percolation in the cobble layer have to be tested at a undistorted geometrical scale of at least 1 in 20 for the cobble grading given in Figure 4.

As a 3D wave basin does not yet exist to model the whole 1.4-kilometer cobble shore section at a scale of 1 in 20 for the given storm conditions, the 3D verification was split up into two process-separated verifications and one qualitative full-processes verification, being confirmed by the consulted experts as best state-of-art:

1) Verify quantitatively the cross-shore deformation capacity as well as the wave overtopping by physical 2D model testing at a scale of at least 1 in 6, as outlined hereafter in this Part I paper; in order to demonstrate during this large wave flume testing that the sand layer underneath the cobble layer will not disappear by turbulent intrusion in the upper layer, the turbulent suspension scaling of this sand has been considered too.

2) Verify quantitatively the alongshore buffer capacity by physical 3D model testing at a 1 in 20 scale combined with numerical 3D modeling of the alongshore wave- and tide-driven cobble drift, thereby calibrating the model to the outcome of the physical model testing, as outlined in the Part II paper (Loman et al., 2010).

3) Demonstrate qualitatively the integral functioning of the cobble shore section in between the adjacent static hard roundhead to the north and the dynamic widely-footed sand shore to the south by physical 3D modeling in a current and wave basin at scale of about 1 in 60.
Because of the required minimum scale, determined by the suspension scaling of the sand with an average prototype D50 of 370 µm, PUMA selected Deltares (previously called Delft Hydraulics) to carry out the physical 2D model tests in the large wave flume Deltagoot at scale 1 in 5.5. This wave flume at Marknesse, the Netherlands, is 240 meter long, 5 meter wide and 7 meter deep, and is able to generate irregular waves with a maximum wave height up to 2.5 meter, see Figure 7. The physical 2D testing of the design profile consists of measuring the deformed cross-shore profile as well as the wave overtopping volume after each wave condition test of three hours. Here, the storm condition curve of Figure 2 has been schematized into five constant wave conditions steps (each of three hours), lasting from 10.5 hours before to 4.5 hours after the storm peak. Figure 6 shows the wave spectrum of the some tested storm conditions. The verification criteria are that during this testing period the sand underneath the cobble layer does not disappear and that the wave overtopping does not exceed a specific discharge limit.

In these tests, the alongshore loss buffer has been omitted by conservatively assuming that this buffer was already fully eroded at the start of a test series. During the final bid phase (2007), a total of six design profile tests were conducted because of the time-consuming construction and testing. Due attention was given here to quantifying the effect of intruded sand (from deposits above) on the cross-shore deformation and the wave overtopping. Furthermore, the cross-shore deformation of the cobble shore design had also been tested in the Deltagoot for the sensitivity to bi-modal waves. According to a thorough wave analysis, this bi-modal wind wave and swell spectrum could occur as most extreme event 1 in 100 years, see Figure 6. These optimization and verification tests resulted in the design profile of the final bid, as shown in Figure 8.

Beyond 4.5 hours after the storm peak the verification of the remaining dune deformation capacity had to be demonstrated with the numerical model DUROSTA. During the large scale model tests, the maximum overtopping criteria from the previous phase had been reduced to 1 liter/second/meter by considering the significant crest-profile deformation occurring on account of large wave overtopping. Based on the discharge measurements of wave
overtopping, the numerical model PC-OVERSLAG was calibrated by means of a single profile-roughness parameter. Using this calibrated numerical tool, the crest height of the different cobble shore reaches was verified.

**iii) Design optimization and verification during the Contractual Design Phase**

After the contract award in early 2008, further physical 2D model tests were carried out in the Delta flume regarding the optimization and verification of the design profile of 2007. In total, six additional tests were conducted leading to the optimized and verified design profile of the transient section Kp2.150 - Kp3.500, see Figure 9. For the verification of the layer thickness and the wave overtopping, the safety factors from the previous phase were revisited in detail, dealing with the uncertainties of cobble grading, cobble density, sand mixing, 3D effects like beach cusps and accuracy and repeatability of physical model testing measurements.

In addition to the above, further optimization has been investigated in partnership with PRA during 2008 - 2009. This concerns the feasibility of shortening of the northern static hard sea defense section and lengthening of the dynamic cobble shore section. In order to minimize the remaining uncertainties of annual maintenance to an acceptable level, comprehensive additional physical testing of the alongshore cobble drift was carried out by HRW, as outlined in the Part II paper (Loman et al., 2010).

As a result of the considered alternatives, the innovative design of a dynamic cobble shore with a static foreshore reef of re-used concrete blocks (see Figures 10 through 12) appeared to yield the lowest TCO, thereby taking into account the belonging risks.

Figure 9: Birds eye view of sea defense with transition section (left) and a typical cross-sectional design profile (2008) of the cobble shore (right)

Before conducting the verification tests of the final design of the cobble shore with foreshore reef, a number of tests were carried out in the small wave flume Scheldegoot of Deltares at scale 1 in 54, see Figure 10. This scale had already been used for testing the design alternatives of the static hard sea defense in 2007 and 2008, including the static roundhead with the adjacent dynamic cobble shore. Based on the findings of the static stability tests of the foreshore reef and the reconnaissance tests of the cobble shore, the final verification conditions could be established in a sufficiently reliable and cost-effective way. Namely, the costs and time needed for conducting a test series in the Scheldegoot at scale 1 in 54 were considerably less than for conducting similar tests in the Deltagoot at scale 1 in 5.5.

At the dynamic plunging wave zone, a boiling cobble pit with prototype depths of more than one meter was observed in the Scheldegoot tests during the plunging of the first design storm waves on the cobble shore without reef (see Figure 10, mid and right) through the side-windows of the flume. As such window observation was not yet possible in the Deltagoot, marked wooden floaters were placed in the cobble layer at half and full depth of the layer thickness. During these tests, the half-layer floaters came upwards after the plunging of large waves. Remarkably after these tests, the cobble layer and the sand underneath were still in place at this location. This boiling pit phenomenon was not observed for the cobble shore with reef.

Figure 10: Side view from testing in Scheldegoot (left) thereby visualizing the dynamic impact zone of plunging waves (mid and right)

Figure 11: Birds eye view of 3.5-kilometer cobble shore with foreshore cube reef (left) and a typical cross-sectional design profile (2010) (right)
The final verification testing of the cross-shore deformation and wave overtopping of the cobble beach and dune with a static reef at the foreshore was conducted by means of three tests in the Deltagoot. This finally resulted in the verified design as shown in Figure 11, which is now being constructed. For the verification of the layer thickness and the wave overtopping, the conservative safety factors from the final testing without foreshore reef were applied.

4. Discussion of results of physical 2D model tests

The described process of design development, optimization and verification revealed interesting results for physical 2D model testing. In the following, some of these results are illustrated and discussed, grouped into:

i) comparison of test results from different scales;
ii) comparison of test results with numerical prediction;
iii) effect of intruded sand on the profile deformation and wave overtopping;
iv) effect of storm water level on the profile deformation with a foreshore reef.

i) Comparison of test results from different scales

During the final tender phase, the roundhead of the static hard sea defense was tested by DHI in their multidirectional wave basin at scale 1 in 54. Here the adjacent dynamic cobble shore was qualitatively modeled by broken sand with a D50 of 1.9 mm (model) and a D10 of about 1.4 mm. At this scale, the median grain-size represents the upper grading curve value of 100 mm well (see Figure 4). However, the D10 value needed to be raised in the model in order to assure the turbulent percolation regime. As the profile deformation measurements of these tests showed similar patterns as measured in the Deltagoot, this broken sand was also used in the reconnaissance tests regarding the cobble shore with reef design in the Scheldegoot. In order to quantify this scale effect comparison tests were conducted in the Scheldegoot at scale 1 in 54 with the cobble shore profile. Figures 13 en 14 show that this profile deformation at the design storm does not differ much from the results of the 1 in 5.5 tests.

Figure 13: Profile development of cobble shore without reef after design storm, Scheldegoot test (1 in 54) versus Deltagoot test (1 in 5.5)
The above minimum D10 of 1.4 mm corresponds reasonably well with Figure 5.139 of the Rock Manual (CIRIA et al., 2007) as used by the PUMA experts for estimating the minimum scale of 1 in 20 for the cobble grading shown in Figure 4.

**ii) Comparison of test results with numerical prediction**

During the first tender phase the cross-shore profile development after the design storm was determined by the empirical formulae of Van der Meer (1988). In order to show the optimization revenue here of physical 2D testing, this profile was compared in Figure 15 with the blue Deltagoot tested profile from Figure 13. The figure shows that the measured maximum erosion depth, as well as the crest berm height, was considerably less than predicted by the formulae of Van der Meer (VdM).

**iii) Effect of intruded sand on the profile development and wave overtopping**

During the final tender phase as well as the contractual design phase due attention was given to quantifying the effect of the intrusion of deposited sand onto profile erosion and wave overtopping. Because of the adjacent southern sand shore the temporary deposition of sand in the nearest cobble shore is possible. As such, a number of Deltagoot tests were conducted with a cobble layer, washed-in with sand. Based on an expert study of HRW concerning field data, the upper boundary of this washed-in sand has been taken to equal average high water at springtide (1.3 m above MSL). Figure 16 shows two measured Deltagoot test profiles after the design storm peak. The red line is the measured deformation of the cobble layer not washed-in with sand. The blue line represents the profile deformation of the same cobble layer washed-in with sand. To identify the porosity of the cobble layer before and after testing, cylinders with 0.25 meter diameter have been pushed into the cobble layer. The volume and the weight of the sediment within the cylinder have been determined. Based on these measurements, the average porosity of the cobble layer has been determined at about 41% without sand and 36% with sand. Figure 16 demonstrates that a high sand content in the cobble layer leads to less erosion in the lower slope and less deposition near the crest. The latter causes more wave overtopping.
iv) Effect of storm water level on profile deformation with a foreshore reef

Guided by the outcome of the reconnaissance tests in the Scheldegoot, the final verification tests of the cobble shore behind the foreshore reef were carried out in the Deltagoot for the 1 in 10,000 years design storm occurring at high water, as well as low water. Figure 17 shows that the low water storm yields the maximum erosion depth (cobble layer without washed-in sand).

6. Conclusions

This paper demonstrates that man-made cobble shores are one of the most effective means of sea defenses, thereby taking into account the required layer thickness for covering a sand core and the local cobble recharge or re-cycling needed because of varying alongshore wave power conditions leading to a year-round cobble deficit. This green (building-with-nature) shore concept has been developed, optimized and verified into an innovative tailor-made final cobble shore with reef design of 3.5 kilometer of the Maasvlakte 2 sea defense. From the overview given in this Part 1 paper about the requirements verification regarding the cross-shore design aspects, the following conclusions can be drawn:

- The success of this cost-effective innovation is the outcome of an integral design approach, being design development, optimization and verification hand-in-hand with constructability, life cycle costs and risks assessment.
- The success of this cost-effective innovation is the outcome a suitable system-oriented formulation of requirements without basing it on a reference design, and enabling sufficient design freedom.
• The success of this cost-effective innovation is the outcome of the joint incentive of contractually agreed partnering for the design optimization.
• The sea defense concept of a cobble shore with a reef of re-used construction material appears to be the best solution in meeting the four main requirements of the Maasvlakte 2 sea defense, having the lowest total cost of ownership. The innovative dynamic cobble shore with reef, constructed with re-used concrete cubes and quarry stones from the Maasvlakte 1 sea defense to be dismantled, appears to be cost-effective when considering the present value of construction costs and costs of 50 years of maintenance. For other sea defense projects, a cobble shore without reef may be more cost-effective because of the higher costs of the reef construction material.
• The verification of the cross-shore profile design aspect was only possible with physical 2D model testing, because of lack of sufficient reliable empirical formulae and thereby taking into account the effects of sand intrusion from deposition as well as the effects of sand intrusion from underneath the cobble layer.
• The use of combined physical 2D models at different scales, enabling a small wave flume testing in addition to large wave flume testing, enhances the effectiveness of physical testing because of the costly and time-consuming large wave flume tests and because the reconnaissance tests in a small wave flume may contribute in a tailored route of design optimization and verification.
• The development of this innovative sea defense concept demonstrates the paramount value of published data available from applied research and field observations on this building-with-nature topic.
• Further applied research would be welcomed to quantify the effect of different wide-graded grain-sizes onto the cross-shore deformation and to clarify more fundamentally the dynamic stability of the sand underneath the boiling cobble layer on account of wave plunging.

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