Determining Flow Velocities at Damaged Weir of Grave Using CFD

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Determining Flow Velocities at Damaged Weir of Grave Using CFD

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Abstract: On the 29th of December 2016, a ship carrying benzene collided with the weir near the city of Grave, The Netherlands. The impact caused significant damage to the weir and resulted in partial drainage of the upper reach. After an initial analysis of high water levels during river floods, Rijkswaterstaat decided to build a temporary dam downstream to partially block the flow through the weir. Thus, the upstream water levels could be restored and room was created to repair the damage. However, the partial closing meant that the entire river discharge was forced through the remaining section, less than half of the total width of the weir. This paper discusses the numerical simulations which were performed to approximate the flow conditions in that remaining section and to roughly predict the flow velocities which could occur during critical conditions. As it became apparent that simple weir discharge formula would not be sufficient, the choice was made to use a three-dimensional Computational Fluid Dynamics (CFD) Model. Because of time constraints, the model setup was relatively simple and a sensitivity analysis could not be performed. Nevertheless, the model gave an insight into the expected flow patterns up- and downstream of the weir and a more detailed estimation of the flow velocities near the bed than the models used in the river flood calculations (performed with WAQUA (WAQUA 2018)). From the results, it was concluded that extra bed protection was necessary. At the same time, a question arose about how well the numerical model could simulate supercritical and subcritical flows as well as transitions between the two flow regimes.

Keywords: Weir, CFD, supercritical flow, bed protection.

1. Introduction

On the 29th of December 2016, a ship carrying benzene collided with the weir near the city of Grave in the river Meuse, The Netherlands. The collision took place around half past seven in the evening in severe fog and darkness. The ship, Maria Valentine, missed the exit towards the shipping lock, sailed through the floating marking line, and broke through several of the support beams. The speed of the ship was such that it went through the weir completely and ended up about 3 m lower downstream of the weir. Fortunately, the collision and the drop down did not cause a leak and the liquid benzene was kept contained in the ship’s cargo hold. Despite the significant damage, (the wheel house was demolished and piping and railings on the deck were damaged) the crew was able to anchor the ship about 500 m downstream of the weir.

The collision damaged five of the support beams, which were also pushed over their iron buttresses at the weir sill. This caused an uncontrollable flow through the weir, emptying the upper reach of Grave. To make matters worse, this upper reach is connected to the channel between the rivers Meuse and Waal. Due to problems with closing the lock at Heumen in time, the collision also led to a partial drainage of this canal. Over a distance of about 30 km, between Grave and Sambeek, the water level in the Meuse was decreasing with a rate of 50 cm/hr, which later decreased to about 20 cm/hr. The water level decreased about 3 m in total. Thus, this part of the Meuse River could not be used for navigation. Instead, ships had to take a different route using inland channels (Zuid-Willemsvaart for smaller ships and the Albert Canal for larger ships). Furthermore, the low water level meant a significant increase in head difference over the upstream weir of Sambeek and led to a reduced stability of the river banks as the phreatic levels do not follow the water level on the river directly.

To restore the water levels in the river reach of Grave as soon as possible and to make room for repair, Rijkswaterstaat decided to construct a quarry stone dam downstream of the damaged section of the weir between the bridge piers. The construction of the dam was started on the 10th of January and finished on the 23rd of January. During construction, the water level upstream of Grave was gradually increased to allow shipping again. The dam was made impermeable by applying a foil on the upstream slope. The dam was made in such a way that the upper part could be removed within 48 hours in the event of a river flood due to rainfall and/or melt water. The closing of one of the weir sections allowed for the repairs. Five of the support beams were damaged, three could be repaired and two had to be replaced.
Furthermore, the technical condition of the iron buttresses anchored in the concrete sill had to be determined. On the 4th of July, the repairs to the weir had been completed and the removal of the temporary dam was started.

This paper addresses the situation where one section of the weir was blocked by the temporary dam. The remaining cross-section was less than half of the total cross-section of the original weir. This resulted in a higher head difference over the weir and higher flow velocities in the operational part of the weir compared to normal operating situations. The weir and bed protection downstream of the weir were never designed to cope with these conditions. Simultaneously with the design of the temporary dam, Rijkswaterstaat decided to extend the existing bed protection further downstream. However, a study into the possible local flow velocities had not yet been performed. This study combined handbook formula and three-dimensional Computational Fluid Dynamics (CFD) simulations. This paper focusses on the CFD modelling part of the study and the resulting flow conditions. The flow through the weir was calculated using the commercially available CFD package Star-CCM+ using the Volume of Fluid (VoF) method for free surface tracking. This kind of model has been applied before for subcritical to critical flow as shown by Feurich and Olsen (2014) and Bayón-Barrachina et al. (2015).

The study this paper discusses has been commissioned by Rijkswaterstaat-GPO (Grote Projecten en Onderhoud) and reported in de Loor and Weiler (2017). Supervision from Rijkswaterstaat was performed by Wim Kortlever. At the side of Deltares, the one-dimensional analysis was performed by Otto Weiler and supported by Rob de Jong. The CFD simulations were performed by Alexander de Loor and Tom O’Mahoney.

2. Domain of Interest

2.1. Meuse Canalization

The weir near Grave, completed in 1929, is one of the seven weirs in the Dutch stretch of the river Meuse. These weirs are located, from upstream to downstream, near the villages of Borgharen, Linne, Roermond, Belfeld, Sambeek, Grave and Lith. These weirs, as part of the canalization of the Meuse, enable year-round navigation down the river. The Meuse river is fed by rainwater and melt water from the Ardennes. This nature explains the choice of canalization and the use of weirs to enable shipping, especially in drier periods. The dependence on rainwater means that the river discharge fluctuates greatly and can increase in a relatively short period of time. Due to this fact, it was very important that the upper part of the temporary rock dam could be removed at short notice. Thus, in the event of a high river flood when the river also flows through the floodplains, the extra increase of the water level related to the blocking by the dam will be limited. Besides, the flow conditions at the operational part of the weir can change rapidly. As the winter and spring show a high chance of high river discharges, it was very likely that this would occur while repairs were still ongoing.

2.2. Weir at Grave

Four of the seven weirs in the Dutch part of the Meuse consist of Stoney gates for water level control and directly next to it a Piorée weir which also enables the passing of ships at high river discharges. The Piorée weir in this case has vertical beams and rectangular steel panels and, in principle, may be compared to the older needle weirs. Two of the other weirs, near Borgharen and Lith, consist of large lifting gates with flap gates mounted on top for water level control. At present, the gates of the Lith Weir allow for a combination of underflow and overflow simultaneously at medium river discharges.

The weir near Grave, which is located below the John S. Thompson Bridge in the main river channel, is again different because it has two flow passages with a width of 50 m (west) and a width of 61 m (east) and a bridge pier of 7 m in between. Those passages have been split up in 9 and 11 sections, each of which consists of a set of three panels supported by a vertical beam. At low river discharges, the water level in the upper reach of Grave is controlled by lifting or adding the right number of panels. At high discharges, the vertical beams including the panels can be lifted out of the water and turned under the bridge deck to allow ships to pass the weir. In short, the Grave weir can be seen as an upside-down Piorée weir. To enable navigation at low discharges, ships can pass by the shipping lock on the eastside of the weir. On the westside of the weir there is a fish passage. On both sides of the main river channel there are flood plains which will flood at high river discharges.

The weir has a concrete sill. Directly downstream of the support beams there is a shallow and short stilling basin followed by a bed protection of concrete blocks. The latter with added roughness intended for energy dissipation. Further downstream, the existing bed protection consist of quarry run of which the first 20 m has been grouted. Just
before the temporary dam was constructed in the eastern passage, an additional layer of stones had been placed and the grouting extended to about 50 m. Before the collision had taken place, there was already a scour hole behind the weir with a depth of about 10 m (visible in Figure 8 later).

Water level data is available from two measuring points near the weir. One is located upstream (‘Grave Boven’) at the upstream entrance to the lock approach. The other is located downstream (‘Grave Beneden’) around 500 m downstream of the downstream entrance of the lock approach. This data is used in the one-dimensional calculations to estimate the water levels at the weir.

**Figure 1.** Construction of the weir in the 1920’s showing the support beams and panels.  
**Figure 2.** Flow through the weir during operation showing the water level control using the panels by partial closing.  
**Figure 3.** View of the weir showing the damaged eastern section near the right bank.
2.3. Temporary Dam

The temporary dam, shown in Figure 5 and Figure 8, which had been built out of quarry stone using different rock gradings, was located in the eastern passage of the weir between the bridge piers. The center pier was lengthened in the downstream direction by stacking a number of shipping containers. This wall of containers retains the dam so that the slopes of the dam do not reduce the flow area of the remaining passage. To secure these containers, these are ballasted with rock and also anchored to the dam. The dam was made impermeable by applying a foil on the upstream slope. By closing the eastern passage, the maximum width of the flow at the weir reduced from 111 m to 50 m, not taking into account the obstruction by the support beams. In crosswise direction, the dam had a length of approximately 30 m and a height of 5.40 m. The crest level was NAP+8.00 m, equal to the target water level in the upper reach of Grave. The cross-section of the dam is given in Figure 8.

Figure 5. Aerial view of the weir with the temporary dam in place (river flows from top to bottom).
3. Model Setup

3.1. Software

The simulations were performed using version 11.04 of the Star-CCM+® software, a proprietary and commercial CFD package. Star-CCM+ is able to solve the Reynolds Averaged Navier-Stokes (RANS) equations from mass and momentum in a three-dimensional domain, both steady-state or transient, as used in this study. The Volume of Fluid method (VoF) is available to model free surface flows. The package also includes its own meshing algorithm, which is also used in this study. A number of different types of boundary conditions, numerical schemes, and turbulence models are available. Star-CCM+ has been used successfully by Deltares in recent years for the simulation of free surface flows in and around hydraulic structures, e.g. shipping locks and sluices, sewage systems, and tidal energy applications.

3.2. General Model Settings

The RANS-equations are solved in their transient form with first order temporal discretization and a segregated pressure-based solver. The equations are discretized on a fully unstructured three-dimensional mesh. The Finite Volume Method is used with second order upwind for advection. A two equation turbulence model is used, more specifically the k-ω/SST model. To model air and water as immiscible phases, the Volume of Fluid (VoF) method is used.

3.3. Geometry Construction

The numerical domain is basically built up out of two parts. The first part consists of the bathymetry and the river reach downstream of the weir. This part of the domain is based on bed level measurements available in Baseline (Baseline 2018) at the time of modelling. The bathymetry is exported in STL-format and imported in Star-CCM+ before selecting the relevant section of the river (shown in Figure 6). The embankments are not well represented in the Baseline Model; therefore, these are estimated. Due to time constraints, it was not possible to get a smooth transition all along the geometry between the embankment and the bathymetry. This was, however, limited to locations of less interest. Due to limited data, the curvature of the river and the rotation of the weir with respect of the downstream reach had to be estimated from aerial photographs. However, the use of Baseline data had the advantage of relatively detailed (5 m resolution) bathymetry data containing the most important features in the river bed, most importantly the extent of the bed protection and the scour hole downstream of the weir. The modelled part of the river extends towards the end of the dam that divides the river and the approach harbor of the shipping locks. It was estimated that this was enough to capture flow velocities near the weir but might be too short to capture a recirculation behind the weir properly. The river reach upstream of the weir is approximated by a constant bed level and not directly based on bed level measurements. The constant level is also visible in measured bed levels. The upstream boundary lies 100 m from the weir. The downstream boundary is located some 300 m from the weir.

Figure 6. The Baseline bathymetry that was used in the CFD geometry; the yellow box shows the approximate extent of the numerical domain, and the red arrow shows the approximate location of the weir.

Figure 7. Overview of the resulting numerical domain and the bed levels. Flow from top right to bottom left.
The second part of the geometry consists of the weir itself, including the temporary dam, support beams and control panels. This geometry was based on drawings of the weir (see Figure 4). The support beams and panels are simplified, so instead of H-profiles, the beams are modelled as solid beams and the panels are modelled as rectangular plates. The geometry of the dam was based on the design drawings available at the time where the containers are simplified as solid boxes. The shape of the weir, as used in the simulations, is shown in Figure 9. A cross-section of the dam is shown in Figure 8. Unfortunately, the geometry of the weir doesn’t connect smoothly to the river bathymetry along the entire width of the river. This resulted in a few sharp transitions between the bed protection and the river bed downstream of the weir. The resulting geometry combining the river bathymetry and the weir is given in Figure 7.

![Figure 8. Cross-section of the temporary dam (as built). Measures are in meters. Height levels in NAP+m. The colors indicate stone grading.](image)

![Figure 9. Overview of the geometry of the weir as used in the simulations. The sill of the weir is shown in yellow, the temporary dam in grey, the containers in brown, the bridge pier in green, the beams and control panels in orange and green, and the side walls in blue.](image)

### 3.4. Mesh Settings

To generate the mesh, the mesher of Star-CCM+ is used, specifically the Trimmed Cell Mesher. The Trimmed Cell Mesher creates a mesh by generating structured background mesh and refines this where necessary by splitting each hexagonal into eight pieces. Near boundaries, the cells are split to conform to the shape of the surface. The advantage of this type of mesh is that it is very suitable for calculations using the VoF method. The base size of the mesh is set to 1 m. Around the weir this is refined to 0.25 m. At the water surface, the cells have horizontal dimensions of 0.5 m and vertical dimensions of 0.2 m to better model the free surface. The prism layer is set to consist of two cells with a total thickness of 0.33 m. It must be noted that this resolution ended up being too coarse to properly resolve the wall function along the bottom. The lack of available time meant that it was not possible to perform a mesh dependence study or refine the prism layer. Therefore, the mesh dimensions have been chosen based on experience. However, this
was not deemed necessary for the purpose of this study to give a reasonable estimation of flow velocities. It is, however, advised for future calculations to perform a mesh dependence study.

3.5. Boundary Conditions

The numerical domain consists of an inlet at the upstream side of the domain and an outlet at the downstream side. From the one-dimensional analysis followed estimations for the water levels at the in- and outlet. At the in- and outlet of the numerical domain, these water levels were used to construct a hydrostatic pressure boundary. The difference in hydrostatic pressure over the domain resulted in a flow over the weir. The river bed and the surfaces of the weir were modelled as a rough surface with a modified wall function that is dependent on the chosen roughness height. The top boundary is also defined as a pressure boundary in order to let the air phase flow freely through this boundary.

3.6. Performed Calculations

Two simulations have been performed for two different conditions. The head difference over the weir for these conditions was based on the one-dimensional analysis. The first calculation was defined as a maximum discharge case in which the river was still completely discharging through the summer bed. This situation was chosen as it was expected to be the normative condition with the highest flow velocities. At the moment of setting up the calculation, it was anticipated that the support beams would be retracted from the water; however, during the calculation, it became clear this would not be the case in practice. However, the calculation was still continued to get an order of magnitude of flow velocities and discharge.

Furthermore, the one-dimensional analysis showed that the relatively high water levels both up- and downstream in the first calculation didn’t result in a normative condition. The analysis showed that energy dissipation is worse at lower discharges because of the higher head difference over the weir and the lower water depth downstream which is not sufficient to result in energy dissipation through a hydraulic jump close to the control panels. Therefore, an extra calculation was defined where the bottom row of flow control panels would still be in place. The selected water levels at the in- and outflow boundaries in the performed calculations are given in Table 1. To calculate the discharges and water levels in the one-dimensional analysis, previous investigations into the weir at Grave, which studied the possibilities for higher water levels in the upper reach, were used (Jongeling 2012). Furthermore, the analysis into the supercritical flow and hydraulic jump relations was based on those from Kolkman (1989), Nortier and de Koning (1996) and Chow (1959). Although not performed in this study, it should be mentioned that these kinds of CFD calculations can also be used to calculate discharge coefficients for use in one-dimensional models. However, this should also involve calibration using actual data of river discharges and water levels for correct results.

<table>
<thead>
<tr>
<th></th>
<th>Water level upstream [NAP+m]</th>
<th>Water level downstream [NAP+m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Open weir</td>
<td>7.90</td>
<td>6.20</td>
</tr>
<tr>
<td>Weir with lower panels</td>
<td>7.85</td>
<td>5.35</td>
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4. Results of CFD Calculations

4.1. Open Weir Case

As stated before, the open weir case simulates more or less the maximum discharge condition at which only the summer bed of the river is used. Also, the support beams are retracted against the bridge. The chosen water levels at the up- and downstream boundaries result in a head difference over the weir of 1.70 m. During the calculated time span, the calculated flow rate varied around 840 m³/s. The one-dimensional analysis predicted a higher flow rate of approximately 900 m³/s at this head difference, indicating a higher resistance from the partially closed weir than predicted. However, because of the relatively short calculation time, it was not possible to get to a situation with a truly constant flow rate.
This non-stationary behavior was also visible in the water levels immediately downstream of the weir, which varied visibly, as was the separation from the bridge pile. At the moment, it could not be determined if the fluctuating flow separation from the bridge pile led to the variations in the flow rate (and thus to changes in the downstream water levels) or if the calculation needed more time to obtain a more steady condition. However, as the goal from the work was to obtain a reasonable approximation of the flow velocities near the weir, the current state of the simulation was deemed sufficient to draw some first conclusions.

The results show a strong contraction of the flow in the weir as the flow separates from the bridge pile. This causes the river to discharge through an even smaller opening, further reducing the capacity of the weir. This also increases the local flow velocities and leads to supercritical flow (Fr>1) with a maximum Froude-number of about 1.5. Just behind the weir, above the bed protection where the depth increases, this leads to what seems to be a combination of a cross wave from the steep embankment of the weir and an undular jump as visible in Figure 11. The reported maximum depth average flow velocity is almost 7 m/s as shown in Figure 10. More importantly, the maximum flow velocity does not occur in the concrete structure of the weir but downstream at the bed protection.

Furthermore, as the flow enters the downstream reach from just one side of the river, this leads to a recirculation on the eastern side of the river. This recirculation forces the flow through the weir towards the western embankment and behind the recirculation, the flow is directed to the eastern bank. However, as this is close to the downstream end of the domain, the calculated extent of the recirculation is not certain.
4.2. Weir with Lower Panels Case

The second calculation included the support beams and lower row of control panels. The one-dimensional analysis showed that this condition would result in higher flow velocities even though the river discharge was lower. An important consideration at this point is that this condition, whereby the weir is still operational, occurs more frequently than the previous case. The one-dimensional analysis predicted a flow rate of approximately 600 m$^3$/s; the calculated flow rate was around 575 m$^3$/s, showing good agreement. These kind of discharge peaks are not rare during the winter period. The calculated flow rate in this simulation was more or less constant due to the influence of the control panels as these function as a short weir with critical flow.

Downstream of the control panels, the flow continues to be supercritical with Froude-numbers locally up to 2.25 and depth averaged flow velocities up to 8.5 m/s as shown in Figure 12. The flow results in cross waves downstream of the weir, again from the western embankment, but now also from the bridge pile, and waves on the water surface as visible in Figure 13. It is important to note that the supercritical flow is again not confined to the weir with the concrete reinforced bottom but extends over the bed protection.

Again, the flow results in a recirculation on the eastern side of the lower reach, which is smaller than in the previous calculation, but still forces the flow through the weir towards the embankment. During the actual repairs, this resulted in damage to the western embankment whereby a part of the embankment collapsed due to erosion behind the bed protection. However, this was far enough from the weir to be of any immediate concern.

Comparison to the real world conditions is difficult to make as no measurements have taken place. Focus of the work around the weir was on safety and to get the weir operational again as soon as possible. Also, the sudden nature of the incident did not give a chance to prepare measurements. However, video footage of flow through the weir during the repairs is available. From these videos, it seems that the flow through the weir is indeed supercritical at times as simulated (Werkzaamheden Brug Grave… [10-03-2017] 2018). Also, the calculated recirculation can be witnessed due to a construction barge that is being pushed against the eastern embankment downstream of the weir (Werkzaamheden Brug Grave… [13-03-2017] 2018).
Figure 12. Depth averaged water velocity at the weir for the weir with lower panels case.

Figure 13. View of the free surface elevations downstream of the weir when the lower panels are in place

5. Aftermath and Conclusions

The reported study, of which only the numerical part is discussed in this paper, led to measures to further strengthen the bed protection behind the weir on top of the measures already taken. A difficulty in the possible measures was that they would not lead to an increase of backwater in the upper reach of the river and not block access to the temporary dam. Eventually Rijkswaterstaat settled on strengthening the bed protection with large stones in a compact distribution and increased monitoring. Several discharge peaks occurred during the repairs; however, the bed protection near the weir stayed mostly intact. Yet, severe changes in the bed were visible behind the bed protection, locally increasing the depth of the scour hole and locally decreasing the depth due to the recirculating flow. Also, some damage occurred on the western embankment leading to a local collapse of the embankment. However, these were not located near the weir itself and caused no immediate concern. Thanks to video footage from Rijkswaterstaat and a local citizen, it was possible to witness the strength of the flow in reality. At the end of July 2017, the repairs to the weir were finished and the temporary dam was removed, bringing the weir back to full operation.

The results from the CFD calculations show supercritical flow and very high velocities outside of the weir itself at the bed protection. The reported velocities already pose a risk even for a grouted bed protection. The supercritical flow and especially the change from supercritical to subcritical will involve pressure variations. Fortunately, the Froude-
numbers are not so high that a strong hydraulic jump is to be expected, and thus the pressure variations will be limited. Furthermore, the CFD calculation shows a strong recirculation in the lower reach that directs the flow from the weir to the western embankment possibly leading to damage there.

This paper shows that CFD can play a role in cases where a simpler, one-dimensional approach cannot give enough information even when the available time is limited. It is important for such a study that the boundary conditions are clear and that every party understands that the results give approximate values. The extra information from a CFD-model, outside of the actual flow velocities, gives a wealth of information. The flow contraction from the bridge pile at high discharges was larger than expected and also the effect of the recirculation downstream became visible. Because of the complex shape of the bed downstream, it was very difficult to predict the extent of this recirculation beforehand and this shows the added value of the CFD-study.

Because of the time constraints of the project, some shortcuts had to be taken. It was not possible to perform a proper grid sensitivity study, study the influence of the time-step, or to run the calculations for a longer period to see if the flow rates would become more constant. Furthermore, the study focused on obtaining flow patterns and depth averaged velocities to decide if measures were necessary. The translation from flow velocities in CFD to bed protection is not made in this study. As CFD becomes an increasingly popular tool, it is recommended to look into this translation as this is not straightforward.

6. References