May 16th, 12:10 PM

A Deterministic Method for Evaluating Block Stability on Masonry Spillways

Owen John Chesterton  
*Mott MacDonald*, john.chesterton@mottmac.com

John G. Heald  
*Mott MacDonald*

John P. Wilson  
*Mott MacDonald Bently*

John R. Foster  
*Mott MacDonald Bently*

Charlie Shaw  
*Mott MacDonald*

*See next page for additional authors*

Follow this and additional works at: https://digitalcommons.usu.edu/ishs

**Recommended Citation**  
Author Information
Owen John Chesterton, John G. Heald, John P. Wilson, John R. Foster, Charlie Shaw, and David E. Rebollo
A Deterministic Method for Evaluating Block Stability on Masonry Spillways

O.J. Chesterton¹, J.G. Heald¹, J.P. Wilson², J.R. Foster², C. Shaw² & D.E Rebollo²
¹Mott MacDonald, Cambridge, United Kingdom
²Mott MacDonald Bentley, Leeds, United Kingdom
E-mail: john.chesterton@mottmac.com

Abstract: Many early spillways and weirs in the United Kingdom were constructed or faced with masonry. While some structures have deteriorated and require replacement with reinforced concrete, some remain in good condition. However, the evaluation of these structures presents a particular challenge to the engineer. This paper details the work undertaken to evaluate six masonry spillway structures using computational fluid dynamics, selected physical modelling and site testing. As a number of the spillways were stepped, the CFD models showed high sensitivity to the turbulence model selection and required the use of an LES model to adequately develop the transient turbulence structures that had been observed on the prototype and in physical modelling. A combination of CFD and on-site inspection and testing was employed to evaluate the masonry. The CFD model was used to provide velocity magnitudes close to the bed and joint and uplift pressures were estimated from onsite inspection and known joint pressure and velocity relationships. The uplift pressures were then applied to selected masonry blocks on site using a pull-out test rig. The assessment and testing showed that for masonry spillways in good condition, individual blocks would adequately resist the maximum predicted hydraulic forces and uplift pressures. Where masonry was in poor condition or where foundation conditions were less favourable, pull out tests resulted in early block failure indicating additional work or replacement would be required. As such, the modelling, inspection, and testing regime showed that the maximum potential for pull-out could be assessed with CFD and tested, providing a robust methodology for evaluation and safety assessment of masonry spillways.

Keywords: Spillways, weirs, masonry, stepped spillways, spillway joints, uplift, CFD, LES, physical modelling, on-site testing, inspection, hydrodynamic stone removal.

1. Introduction

In recent years there have been a number of notable masonry spillway failures in the UK, including Boltby in 2005 (Charles et al. 2011) (Figure 1b) and Ulley in 2007 (Hinks et al. 2008). While these incidents have sparked interest and research, minor incidents on masonry spillways occur with even more regularity, including issues with seepage, mortar loss, damage from vegetation growth, and block dislodgement during flood events, such as the damage to Butterfly Spillway in 2002 (Figure 1a). These incidents highlight many of the problems that will lead to failure of the lining if it is poorly constructed or not adequately maintained.

![Figure 1a](butterfly_spillway_damage.jpg)
![Figure 1b](boltby_spillway_failure.jpg)

**Figure 1.** (a) Butterfly spillway damage during flooding in 2002, (b) Boltby Spillway failure in 2005.

Even with sound construction and proper maintenance, there is a need to understand the limits of masonry linings and determine better methods to assess their capacity to safely convey floods.
1.1 Masonry Lining Failure

A masonry lining is considered to have failed on the dislodgement of one or more of the blocks. As removal of a single block can expose adjacent blocks to increased hydrodynamic force, the removal of one block may lead to the failure of many more or the “unravelling” of the lining. Depending on the location of the spillway and the underlying materials, failure of the lining may lead to spillway failure and could compromise dam safety.

Some of the factors leading to the removal of lining blocks can include poor construction with uneven masonry and wide jointing. Additionally, deterioration of the lining may occur over time leading to loss of mortar if regular maintenance is not carried out. Also, adequate drainage of the liner to prevent excess pore pressures from both ground water and hydrodynamic pressure may not be present or is difficult to verify on spillways that were constructed over a century ago. The mitigation of this failure mode is then dependent on understanding the risk of hydrodynamic block removal given the conditions and flood events the liner should be able to withstand.

1.2 Yorkshire Water Spillways

Yorkshire Water Services Ltd. own and operate a portfolio of 133 dams and reservoirs in the north of England, supplying water to over 5 million customers. Ten yearly independent reservoir inspections identified six masonry lined spillways that were in need of hydraulic assessment to evaluate their ability to convey the design floods. The spillways under consideration were at Lindley Wood Reservoir (1875), Swinsty Reservoir (1876), Fewston Reservoir (1879), Langsett Reservoir (1905), Underbank Reservoir (1907), and More Hall Reservoir (1929).

Given the vulnerability of masonry spillways to block removal, Yorkshire Water Services, in collaboration with their design and contracting partner Mott MacDonald Bently (MMB), has continued to improve their process for evaluating the various factors contributing to this failure mode. This paper discusses the application of numerical hydrodynamic modelling in the assessment of hydrodynamic block removal, and its specific application to the six spillways listed above.

2. Methodology

To evaluate the potential for lining failure, the following three-stage methodology was employed for the spillways under consideration: Review, Modelling, and Testing.

Review: Review of all historic drawings and records to determine as-build dimensions and drainage and to identify any modifications that may have been made since commissioning. The historic data was supplemented with new topographic survey and detailed terrestrial laser scanning.

Assessment and Modelling: Hydraulic modelling using computational fluid dynamics for all six spillways, complemented by selected physical modelling of one spillway (Langsett).

Testing: Onsite testing of the lining at each site included (a) Ground Penetrating Radar (GPR) to assess foundation conditions; (b) test pits to evaluate wall footings and support; (c) cores to verify masonry depth, founding material, and voiding; and (d) pull-out testing of selected blocks to assess their resistance to hydrodynamic block removal.

3. Review—Spillway Construction

All existing drawings and reports were reviewed, and the details of masonry blocks were assessed. A range of different construction techniques were used across the spillways to suit the terrain as shown in Figure 2.

The Langsett, Underbank, More Hall, and Lindley Wood spillways feature long steps with a deep riser course and intermediate tread courses of masonry. The intermediate tread course blocks were 305 mm deep and varied in length and width, with the exception of Lindley wood, where the depth of masonry was taken to be 300 mm.

Langsett and Underbank spillways have gentle stepped slopes, but both finish in a steep (1V:2.3H) stepped cascade. Each step on the steep cascade is formed of a single course of masonry and would not be considered as vulnerable as the intermediate courses on the longer steps in the upstream, gentler sloped section. An interesting feature of Langsett, Underbank, and More Hall spillways is shape of the steps which are curved in plan, presumably to focus flows towards the centreline of the chutes.
The majority of the Fewston spillway is gently sloped with pitched 305 mm masonry courses laid flush to the slope of the underlying concrete foundation. At its downstream end the spillway terminates with three stepped cascades separated by short landings. Each step in the cascade is formed by a single course of masonry whereas the landings are made up of several courses with exposed upstream joints making them more vulnerable to hydrodynamic block removal.

The Swinsty reservoir spillway has a gentle slope with courses of 305 mm deep masonry that are stepped individually by approximately 25 mm. Characteristic features and details of the spillways are given in Table 1.

3.1. Identification of Key Blocks

During the design flow (10,000 yr) and safety check flood (PMF), the stepped spillways are expected to operate in the skimming flow regime. This would result in flow separation over the risers and low velocities and recirculation behind downstream of each step. Higher pressures and velocities would also be expected where the flows impact on the downstream half of the step (Amador et al. 2004). The masonry blocks located on the downstream courses of the steps are therefore considered most vulnerable. Keystones, most evident on the Langsett and Underbank Spillways, were not considered vulnerable given their larger size and embedment below the tread courses. The steep cascades were also not considered vulnerable as each step is a single course with transverse joints exposed only to the low-pressure areas downstream of each step and held in place by the step above. On the pitched spillways (Fewston and Swinsty), all blocks were of similar size and considered equally vulnerable to hydrodynamic forces.

Table 1. Spillway and masonry dimensions and condition on the six spillways used in assessments.

<table>
<thead>
<tr>
<th>Liner details</th>
<th>Langsett</th>
<th>Underbank</th>
<th>More Hall</th>
<th>Fewston</th>
<th>Swinsty</th>
<th>Lindley Wd</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Curved steps with steep cascade at lower end</td>
<td>Curved steps single slope</td>
<td>Pitched with stepped cascade</td>
<td>Individually stepped</td>
<td>Straight steps single slope</td>
<td></td>
</tr>
<tr>
<td>Spillway slopes</td>
<td>1V : 7.44H</td>
<td>1V : 15.15H</td>
<td>1V : 3.3H</td>
<td>1V : 45H</td>
<td>1V : 13.75H</td>
<td>1V : 9H</td>
</tr>
</tbody>
</table>

Figure 2. Masonry invert linings: (a) Swinsty, (b) Lindley Wood, (c) More Hall, (d) Fewston, (e) Underbank, and (f) Langsett.
<table>
<thead>
<tr>
<th>Liner details</th>
<th>Langsett</th>
<th>Underbank</th>
<th>More Hall</th>
<th>Fewston</th>
<th>Swinsty</th>
<th>Lindley Wd</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1V : 2.3H</td>
<td>1V : 2.3H</td>
<td>1V : 3.69H</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Block Depth (m)</td>
<td>0.305 (12&quot;)</td>
<td>0.305 (12&quot;)</td>
<td>0.381 (15&quot;)</td>
<td>0.381 (15&quot;)</td>
<td>0.305 (12&quot;)</td>
<td></td>
</tr>
<tr>
<td>Condition</td>
<td>Excellent</td>
<td>Excellent</td>
<td>Excellent</td>
<td>Medium</td>
<td>Poor</td>
<td>Medium</td>
</tr>
<tr>
<td>Joint opening</td>
<td>≤ 5 mm</td>
<td>≤ 5 mm</td>
<td>≤ 5 mm</td>
<td>≤ 5 mm</td>
<td>≤ 5 mm</td>
<td>≤ 5 mm</td>
</tr>
</tbody>
</table>

3.2. Spillway Condition

To evaluate the lining condition, walk-over surveys, photographs, and high resolution terrestrial laser scans were completed. These site assessments showed that the workmanship was excellent for Langsett, Underbank, and More Hall, which were more recently built while the stone or workmanship on Lindley Wood, Fewston, and Swinsty was of a lesser quality. This was evidenced by the fact that Fewston, Lindley Wood, and Swinsty spillways have historically had a number of block failures with masonry being plucked out or damaged and requiring repair since 2000.

From site inspection and review of the photography and scans, a range of joint openings were found. MMB’s previous experience in repointing masonry linings using low pressure grouting showed that all open joints could be reliably filled for joints ≥ 5 mm. As such, it was assumed all masonry joints could be re-mortared as necessary and the joint opening for hydrodynamic analysis was taken to be 5 mm.

Vertical joint displacement or offset was assessed using the laser scans such as that shown below in Figure 3, verifying that the offsets were typically not significant.

![Figure 3. Laser scan of More Hall spillway invert with selected step contoured by elevation and showing chainage.](image)

4. Assessment and Modelling—Modelling Method and Software

To assess their capacity to convey the design floods, all six spillways were modelled numerically. This decision was taken due to the need to keep the models to hand during the evaluation and design and to avoid two phases of physical modelling and their associated costs. The high level of detail available from the numerical models and their reliability in previous assessments for Yorkshire Water Ltd. also influenced the decision to use numerical modelling. The software package, Flow3D, was selected to undertake modelling work due to its efficient meshing techniques, single phase flow, and sharp interface volume of fluid (VOF) model.

In addition to the numerical modelling, Langsett spillway was selected to be modelled physically in parallel at a scale of 1:20 at CRM’s hydraulic laboratories to provide validation of the numerical results, while Lindley Wood, Fewston, and Swinsty had been physically modelled in 2000 at a scale of 1:50 by HR Wallingford for previous work.

The main steps in the modelling setup were (a) 3D model build, (b) meshing and application of boundary conditions, and (c) model testing. Once the model was running with a satisfactory level of accuracy, production runs would then be carried out for the design flood and safety check floods of 1 in 10,000 annual probability of occurrence and the PMF, respectively. The main outputs required for each spillway were the spillway rating curve for flood routing, water levels to assess chute capacity and freeboard and velocity and pressure data to evaluate the masonry lining.
4.1. 3D Model Build

The 3D geometry was built around the topographical survey and laser scan data with additional detail added from record drawings as needed. While the laser scan data did capture individual masonry block roughness, the hydraulic surfaces were represented by smooth surfaces with an equivalent roughness (typically 30mm) added in Flow3D.

4.2. Model Testing

To maintain the accuracy of the modelling outputs and to minimise numerical modelling effects, a full range of sensitivity and validation testing was undertaken for all spillways modelled. Modelling parameters, including mesh resolution, boundary location and type, and turbulence scheme were subject to testing. The final geometry was also checked against the survey in key areas, such as the weir width and height, channel dimensions, and invert levels.

4.2.1. Boundary Conditions

Upstream boundaries extended into the reservoir for over 30 m in all cases so as to capture weir approach conditions for overflow depths up to 4 m. These were specified as hydrostatic pressure boundaries and their distance from the weir subjected to sensitivity testing. The downstream boundary was also a hydrostatic pressure boundary, typically informed by 1D modelling of the downstream conditions. All modelling was done in a single phase, only applying an atmospheric pressure boundary to the surface of the fluid. Air entrainment bulking was added to water levels in post.

4.2.2. Model Meshing

The computational mesh in Flow3D is structured and entirely orthogonal with mesh elements grouped together into mesh blocks. Cells are truncated to conform to the geometry where they intersect using Flow3D’s FAVOR™ meshing method. The use of Flow3D’s conformal meshing proved useful in limiting cell counts.

A range of mesh resolutions were tested, and time-averaged flows or energy loss were taken as the sensitivity parameters and the model refined until it was no longer mesh dependent (≤ 2% relative change). In some specific areas, additional refinements were required to capture specific interactions and careful judgment was needed to review and verify the models were appropriately meshed. The maximum mesh size used on the chutes was 100 mm. Cell counts across the six spillways at full resolution varied from around 25×10⁶ to 40×10⁶ depending on their size and hydraulics. Given the size of the models, the boundary layer was not refined and the wall functions used within Flow3D were relied on to predict near bed velocities.

4.2.3. Turbulence and Advection

The Large Eddy Simulation (LES) turbulence model was selected for this study over the more commonly used time averaged RANS models due to its ability to model transient turbulent structures, such as vortex shedding and flow recirculation. While the LES model typically requires significantly finer mesh resolution than the RANS model, the computational overhead was found to be acceptable and produced the expected transient performance in sufficient detail, including the transient perturbations of the free surface. While some of the turbulent energy cascade is undoubtedly lost given the mesh size selected, the total energy and hydraulic grade lines were subject to sensitivity testing and showed that at the selected resolutions the models were no longer sensitive (≤ 2% relative) to further increases (doubling) in resolution and compared favorably against the results taken from the physical modelling.

4.2.4. Flow Depth and Air Bulking

Open channel flows travelling at high speeds tend to entrain significant volumes of air which is known to damp out the turbulent fluctuations relevant to masonry block stability. However, air entrainment is problematic to estimate and validate in numerical modelling and is not yet implemented for the LES turbulence model within Flow3D. As such, air entrainment was not evaluated in the CFD model and the velocity and pressure results are conservative.

4.3. Model Outputs

Once tested, the final model runs showed excellent agreement to physical modelling results for the weir rating curves, flow depths on the chutes, and the reproduction of hydraulic conditions. The following figures showing the flow...
conditions on the More Hall spillway chute are typical of the performance and standard visual outputs across all six spillways.

![Figure 4. Plan and 3D view of More Hall chute model at safety check flood.](image)

The velocities and pressures are shown in longitudinal section on the centerline of the More Hall chute in Figure 25. This illustrates the resolution of the model and the skimming flow conditions created with the pseudo bottom at the leading edge of the steps. As expected, the pressures on each step are highest approximately 2/3rds along the step with velocities increasing toward the downstream edge, although this varied depending on the length of the steps investigated. Velocities and pressures also vary significantly with time, and turbulent structures occasionally disrupt the recirculation and high velocity zones on the steps and perturb the free surface.

![Figure 2. (a) Centerline velocities & vectors with mesh lines, (b) pressures on the More Hall chute (sample point for Figure 6: +).](image)

The instantaneous velocities and pressures highlighted the significant fluctuations and peaks that were captured using the LES turbulence model. For both water levels and velocities, the 95th percentile of the values sampled over 10 seconds was used in following assessments (see Figure 6).
To produce the information for assessment of block stability along the chute, data was extracted along the centerline of the chute at 100 mm above the stepped bed every 100 mm of chainage. The centerline was chosen as velocities tended to be higher, giving more conservative results.

4.4. Hydrodynamic Block Stability

To assess block stability, a number of assumptions were made in converting the values obtained from the model to forces on the masonry blocks. Firstly, the primary mechanism for block removal was assumed to be conversion of the local velocity head to joint pressure due to protruding blocks or impinging flow. Secondly, the modelled hydrodynamic pressures (Figure 25b) were not included in this assessment. Where high pressures exist, they will serve as a stabilizing force except where voids beneath the masonry may transfer these high pressures or where the high pressures are more transient. Also, low pressure zones generally coincided with low velocities, reducing the potential for removal in these locations.

The results from the model were used to estimate the forces applied to the blocks by sampling the near-bed velocities and assessing their potential to pressurize the masonry joints. The relationships derived in the USBR study (USBR 2007) on percentage of stagnation pressure mobilized as joint pressure were used to estimate the uplift forces that may result from these velocities. To allow for flows that were not parallel to the bed, it was conservatively assumed that the average grade of the spillway was the angle of incidence of water impinging on the gaps between the blocks on the horizontal spillway steps. The USBR method requires a vertical offset distance between adjacent blocks in its relationship between velocity and uplift forces. With all these assessments, the next largest offset from the USBR method was used in the calculations producing slightly conservative uplift forces. Finally, peak velocities were used but are associated with the edge of the steps rather than the joints that will be most vulnerable and will therefore be conservative.

4.4.1. Uplift Pressure Calculation

As the masonry for all of the spillways is reportedly bedded on concrete, undrained conditions were assumed. The formula (USBR 2007) for the uplift pressure for horizontal flow over the bed in undrained conditions is as follows:

\[ P = \left[ a(U \times 3.28083) b \right] / 3.28083 \]  

(1)

Where \( P \) = Joint pressure in m of water (note SI conversion), \( U \) = Bed flow velocity in m/s (0.1 m above the invert along the centreline), \( a \) = coefficient for joint offset and gap as detailed in USBR (2007), and \( b \) = exponent for joint offset and gap as detailed in USBR (2007).

Using the joint pressure derived from this method, the dynamic uplift force per unit area was calculated on the underside of the masonry blocks as follows. Dynamic uplift force (N/m²):

\[ F_u = \rho w \times g \times P \]  

(2)

where \( P \) = Joint Pressure (m), \( \rho w \) = Density of water (1000 kg/m³), and \( g \) = Gravitational acceleration.
The hydrostatic uplift force, or buoyant force, was also calculated to be added to the joint pressures during onsite testing in dry conditions where hydrostatic uplift force (N/m²) reads:

\[ F_h = \rho w \times g \times B_t \]  

(3)

where \( B_t = \) Masonry block thickness (m).

Finally, the weight force of the masonry blocks resisting movement was calculated. Block weight force (N/m²):

\[ F_w = \rho b \times g \times B_t \]  

(4)

where \( \rho b = \) Masonry unit weight assumed to be 2200 kg/m³.

Using the forces derived, the resultant force per unit area of lining was calculated as follows (N/m²):

\[ F_r = F_u + F_h - F_w \]  

(5)

However, during onsite testing the hydrostatic component of the force would not be present so it is added to the force required for onsite testing which is labeled the total destabilising force. Total destabilising force per unit area (N/m²) reads;

\[ F_d = F_u + F_h \]  

(6)

Forces, such as base adhesion, shear resistance through block interaction, and base adhesion, have not been included in the analysis presented in this paper. The total destabilising forces along the spillway centreline are presented in Figure 7 for the More Hall Spillway, showing an increase in destabilising force along the chute.

\[ \text{Figure 7. (top) More Hall centreline spillway profile; (bottom) block destabilizing forces and resultant forces for testing.} \]

\[ \text{Table 2. Maximum centerline bed velocities, stagnation pressures and destabilising forces for all six spillway chutes.} \]

<table>
<thead>
<tr>
<th>Flood</th>
<th>Langsett Bed Velocity, U (m/s)</th>
<th>Underbank Bed Velocity, U (m/s)</th>
<th>More Hall Bed Velocity, U (m/s)</th>
<th>Lindley Wood Bed Velocity, U (m/s)</th>
<th>Fewston Bed Velocity, U (m/s)</th>
<th>Swinsty Bed Velocity, U (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PMF</td>
<td>16.5</td>
<td>14.5</td>
<td>16.7</td>
<td>14.8</td>
<td>14.4</td>
<td>15.2</td>
</tr>
<tr>
<td></td>
<td>Stagnation Pressure (kN/m²)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>136.1</td>
<td>105.1</td>
<td>139.4</td>
<td>108.9</td>
<td>103.0</td>
<td>116.0</td>
</tr>
<tr>
<td></td>
<td>Destabilizing, Fd (kN/m²)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>63.3</td>
<td>49.9</td>
<td>65.0</td>
<td>51.4</td>
<td>48.7</td>
<td>44.6</td>
</tr>
</tbody>
</table>
5. Onsite Testing

Once the hydrodynamics and block assessment were finalized, they were used to inform the onsite testing of specific blocks. Testing of the blocks involved subjecting blocks to pullout force and coring through blocks to verify their depth and foundation conditions. Selected blocks were identified along the spillways in a range of conditions and locations.

5.1. Pullout Testing

With the total destabilizing forces provided, the pullout tests were carried out to apply uplift forces to the selected blocks within the invert while monitoring and recording any deflection experienced by the blocks, joints, and surrounding blocks. Tests were undertaken with a pile testing rig capable of jacking up to 110 kN shown in Fig. 7.

While the tests were not intended to lead to failure, should a failure occur between the block and the mortar below, then the block was removed to inspect the mortar before being reinstated. The failure load was also recorded. As part of the test regime a visual inspection of the block, mortar joints and adjacent blocks was carried out before the pull-out test, including photographs of the block. Once testing commenced, deflection of the block and the adjacent blocks was measured in millimeters to 2 decimal places. In several cases, the anchor bond or block itself failed through delamination. In these cases, the maximum force applied was recorded. Deflection versus load graphs were produced for each pull-out test followed by post testing visual inspection to confirm no permanent damage had occurred to the lining or the subject block during reinstatement.

Figure 3. Pull-out testing rig and monitoring equipment similar to that used in this study.

5.2. GPR and Coring

As well as pull-out testing, Ground Penetrating Radar (GPR) and coring of selected blocks were carried out to ascertain the foundation conditions beneath the blocks. While the GPR indicated the potential for voids beneath some of the spillway chutes, coring did not show this to be the case with generally good adhesion to the underlying mortar foundations.

5.3. Results

Table 3 summarises the pullout tests undertaken across the six sites. Only one pullout failure occurred on Swinsty spillway. However, in some conditions, testing resulted in delamination of the block or failure of the anchors, which are recorded below as “other.”

<table>
<thead>
<tr>
<th></th>
<th>Langsett</th>
<th>Underbank</th>
<th>More Hall</th>
<th>Lindley Wood</th>
<th>Fewston</th>
<th>Swinsty</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. force applied</td>
<td>110kN</td>
<td>110kN</td>
<td>110kN</td>
<td>110kN</td>
<td>110kN</td>
<td>110kN</td>
</tr>
<tr>
<td>Tests / Pullout / Other</td>
<td>7 / 0 / 3</td>
<td>4 / 0 / 0</td>
<td>4 / 0 / 0</td>
<td>4 / 0 / 0</td>
<td>6 / 0 / 3</td>
<td>7 / 1 / 6</td>
</tr>
</tbody>
</table>
As the block size varied, the force per unit area or pressure varied from 161 to 438 kN/m². Given most of the blocks were not pulled out, this showed that the block adhesion and shear would provide sufficient resistance to the hydrodynamic pressures predicted in the joints. The pullout pressures are plotted against the ratio of the pullout pressures to both the total available stagnation pressures and the pressures mobilized as predicted by the methodology (USBR 2007) to give factors of safety as shown in Figure 9. The increase in the factors of safety between these plots represents the geometry and conditions of the spillway joints where the good construction and grouting limit the potential for stagnation pressures to develop.

6. Conclusions and Outlook

The models developed as part of this study proved reliable in replicating the flow conditions on the spillways and agreed well with the physical modelling where validation data was available, passing independent review. Data extracted along the spillway bed was useful in setting the parameters for on-site testing and helped the designers to understand the magnitude of the velocities available to destabilize and remove blocks. The factors of safety developed from the testing show significant resistance to hydrodynamic block removal given adequate upkeep of the masonry.

As a result of the assessments and investigation done to date, all reservoirs included in this paper will be subject to either repointing or relining to improve the reliability of the masonry lining and minimize the risk of out-of-channel flows.

6.1. Further Work

The reliability of the actual values required for hydrodynamic block removal are difficult to validate given the remaining unknowns including base adhesion, side friction and the amount of drainage available. However, the modelling and testing brings us closer to understanding the locations and magnitudes of the hydrodynamic components within full scale modelling in regions of gradually varying transient flow without recourse to physical modelling.

In future studies, the modelling work could be built on and further detail added to refine boundary layers and to directly model the surface roughness of the masonry spillway. Higher frequency sampling and additional analysis of the outputs could also be beneficial. Air entrainment will have an effect on the velocities and pressures within the chute and modelling of this phenomena may develop to the degree it becomes possible to apply to such studies in the future.

Finally, the conditions for pull-out testing may be improved in the future to more closely replicate the hydraulic and drainage conditions during flood events possibly including monitoring of pressures and joint flow.
7. Acknowledgements
This work was conducted in collaboration with Yorkshire Water Ltd. and Mott MacDonalld Bentley Ltd.

8. References