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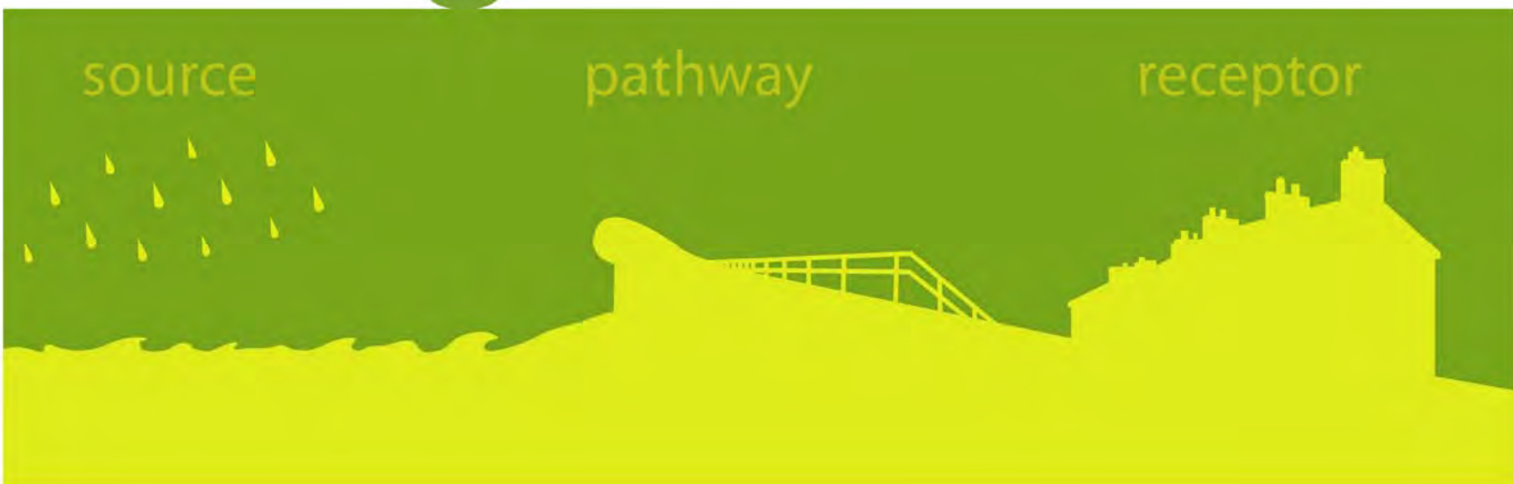

Guidance for the Design and Maintenance of Stepped Masonry Spillways

Project: SC080015

Flood and Coastal Erosion Risk Management Research and Development Programme

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Miranda Kavanagh

Director of Evidence

Executive Summary

Following the failures of stepped masonry spillways at Boltby Reservoir in 2005 and at Ulley Reservoir in 2007, the UK Environment Agency commissioned a research project to clarify the safety of such structures. The work was awarded to MWH UK Ltd in association with CRM Rainwater Consultancy Ltd. (CRM) and Building Research Establishment Ltd. (BRE) and took place between October 2008 and October 2009. This report summarises the results of that research.

The conclusions from this work with regard to hydraulic action can be considered applicable to stepped spillways formed of any material. Where such spillways are constructed of discrete elements or blocks set in a matrix of mortar, the results can be considered applicable to brickwork as well as masonry.

Previous work has been carried out studying the steep, stepped spillways that are typically used on the downstream faces of roller compacted concrete dams. In contrast, this report focuses on the much shallower stepped spillway slopes, typically in the order of 1v to 3h, associated with UK embankment dams.

Examples were collected of industry experience of operating such spillways and, in particular, of distress and/or remedial measures. This was supplemented by a programme of hydraulic model testing both to better clarify how flow depths can be calculated in such spillways and also to improve understanding of the localised hydrodynamic pressure fluctuations to which spillway walls and inverts may be subjected.

In terms of hydraulics, moderate flows in stepped spillways will tend to cascade from step to step losing energy progressively and with each step acting as a form of stilling basin. This is termed 'nappe flow'. Higher flows will begin to skip from step to step, with local vortices trapped within the steps. This is termed 'skimming flow'. In addition to these flow regimes, this research has identified that very high flows may ride on the top of enlarged vortices, hardly touching the steps. In terms of design criteria the highest flow velocities, depths and hydrodynamic pressure fluctuations will clearly be associated with skimming flow and so that is the regime on which the report focuses.

In the case of flow depths, recourse was made to a significant database of model testing on such chutes held by CRM. It was identified that equations generally proposed by Chanson give the most consistent results in the case of shallow, stepped chutes. These equations are presented in Chapters 3 and 5 of the report. However, it was also noted that the high levels of turbulence mean that absolute flow depths are difficult to quantify, instead there is simply a transition in the air/water mix to a progressively reduced percentage of water. The design flow depth in such cases is typically defined as one where the air concentration has reached 90%. It is common practice to allow some freeboard over and above this.

The pressure distributions that can occur on the inverts and walls of stepped spillways under high flow conditions are discussed in Chapter 4. Model testing revealed high pressure zones over the downstream surfaces of step inverts and on adjacent sections of side wall and low pressures on the vertical faces of steps, over the upstream zones of steps and, again, on adjacent sections of side wall.

If high pressures are injected into open textured masonry in high pressure zones such that they create back-pressures behind the masonry blocks in low pressure zones, then the blocks in these low pressure zones can be subject to removal. Moreover, model testing has shown that there can be considerable turbulence and pressure fluctuations during such flows, with the pressure differentials between transitory

maximum and minimum pressures often being considerably higher than between associated mean pressures. It was concluded that such potential pressure differentials on typical UK spillways could reach 5 to 10 metres of water head. The high levels of turbulence within high pressure zones can also be sufficient to dislodge blocks within those zones.

The exact zones of pressure distribution will vary depending on the geometry of the spillways in question and the flows being examined. Therefore, it is not possible to give generalised guidance but rather to draw attention to the potential and to the broad likely zoning. Readers are referred to the more detailed discussions and results in Chapters 4 and 5 of this report for further guidance.

Testing also indicated that significant pressure differentials could be produced by both locally protruding and locally recessed masonry blocks. Design charts to calculate such pressure differentials are given in Chapter 5.

Advice is given in the report on the inspection of masonry spillways, how to identify unacceptable forms of distress and also on appropriate means of repair and remediation. Photographic records of inspections are encouraged to record changes with time. The investigation of voids behind masonry facings is also encouraged either by tapping, in the case of walls, and/or by dragging chains, in the case of inverts. This can be supplemented by taking cores.

The integrity of mortar pointing is especially important in high pressure zones of masonry spillways and in particular, in the vertical joints, normal to the flow direction. Conversely, there may be some benefit in selectively omitting some of the vertical pointing on the vertical faces of steps to permit drainage relief. Mortar pointing in general should be finished either flush with the masonry blocks or finished with a "bucket handle" profile. The regular inspection of masonry spillways is important, as is the removal of any weeds and associated root growth in and around the spillways.

Remedial measures can include simply reinstating pieces of displaced masonry, local patching with concrete, pressure pointing to restore the integrity of the surrounding mortar matrix and/or, in extreme cases, demolition and rebuilding of the spillway or the replacement of masonry with specially profiled and textured concrete. These are described in more detail in Chapter 5 of the report.

For convenience, a more detailed summary of the principal conclusions of this research are drawn together in Chapter 6 of the report. However, practitioners are encouraged to read the remainder of the report to gain additional background and understanding of the various factors involved.

Most stepped masonry spillways in the UK have stood the test of time, with over 100 years of successful operation since such materials formed the standard method of construction. Their use will continue to be acceptable provided that maintenance works and inspections are undertaken on a regular basis by informed practitioners and are combined with careful remediation measures when required. These actions are of particular importance when the spillways are located along the mitre of the embankment where a collapse of the sidewalls could endanger the dam.

Acknowledgements

MWH would like to thank CRM Rainwater Drainage Consultancy Ltd. and Building Research Establishment Ltd. for their collaboration on this project. CRM undertook the physical modelling element of this research project and Ray Baker also conducted a literature review of hydraulic research undertaken on stepped masonry spillways to date. Alan Ferguson of BRE undertook a review of the use of masonry in reservoir spillways and provided guidance on its maintenance and inspection.

MWH would also like to thank United Utilities plc and Yorkshire Water Services for making available the results of model testing on their stepped masonry spillways that they were undertaking as part of routine and, in the case of United Utilities, extended reservoir safety work.

Thank also go to ARUP who provided MWH with information regarding computational fluid dynamics modelling and guidance on undertaking specific remedial measures on masonry spillways.

Finally, thanks to the following organisations that provided MWH with examples of issues and remedial measures at stepped masonry spillways in the UK: British Waterways Black & Veatch, Faber Maunsell/AECOM and Halcrow.

Contents

1 Introduction

On the 25 June 2007, heavy rains and flood runoff into Ulley reservoir, near Rotherham, caused the failure of a stepped masonry spillway. The channel was 1.83 m wide and the estimated flow at the time of failure was 6.1 m^3/s. The spillway slope was shallow at approximately 1 on 7. Although there were three spillways at the dam, the small, left bank, stepped, masonry spillway took flows preferentially.

Failure occurred over a length where the spillway ran along the toe of the main dam embankment. This led to erosion which, in turn threatened the security of the main dam, a major concern as an uncontrolled release of the 600,000 m^3 of water in the reservoir would have led to serious damage downstream. In the event, the situation was contained but as a precaution the M1 motorway was closed, several hundred people downstream were evacuated and the reservoir was lowered by pumping, all at great expense and inconvenience.

This failure was reminiscent of the failure of a similar stepped masonry spillway at Boltby dam near Thirsk on 19 June 2005. There are also thought to have been other, less well published, failures in the past including that at Toddbrook Dam in Whaley Bridge in 1985, and the more recent failure of the stepped masonry spillways at Klingenberg in Germany in August 2002. Furthermore, it became apparent that the floods in 2007 had affected a number of other masonry spillways in the UK, though less seriously.

Some authors, such as Chanson (2002) have discussed the various incidents and damages that have occurred to stepped spillways over the years and have also discussed the various flow regimes to which these spillways are prone; nappe flow, skimming flow and a transition stage between the two.

However, all research on these spillways seems to have focused simply on the hydraulic regime of the flow. Where boundary conditions, such as imposed hydrodynamic pressures, are concerned, research seems to have concentrated exclusively on the floor regions. It became apparent at both Ulley and Boltby dams that significant failure had occurred to the masonry walls. These walls were double skinned with dressed masonry internal water faces, set against rubble masonry backing walls. The post incident inspection review identified that wall failure had been initiated internally via the dressed masonry rather than externally. This implied, at least in the case of Ulley, that internal hydrodynamic effects had been a major contributor to the failures rather than, for example, overtopping.

In the absence of definitive guidance on such hydrodynamic forces, the post incident review, Hinks & Mason (2007), recommended that research work being undertaken to clarify the matter. The Environment Agency let a contract for such research in September 2008 to MWH Ltd, supported by CRM Rainwater Consultancy Ltd. and Building Research Establishment Ltd. In addition, United Utilities plc and Yorkshire Water Services Ltd. agreed to make available the results of model testing on stepped masonry spillways that they were undertaking as part of routine and, in the case of United Utilities, extended reservoir safety work.

This report summarises the results and conclusions from these research studies. Chapter 2 presents an overview of masonry spillway construction. This is not intended to be exhaustive as there are, inevitably, variations between structures, nevertheless, basic principles are summarised.

Chapter 3 summarises the industry's current understanding of the hydraulics of stepped masonry spillways and is presented as a prelude and basis for the physical modelling work.

Chapter 4 presents a summary of the results of the hydraulic model testing carried out as part of this research programme.

Chapter 5 focuses on practical recommendations for use by practitioners and is based on the results of this research. For convenience it has been divided into three sub-headings; hydraulic design, maintenance & inspections and remedial works. Examples are given of typical types of remedial works that have been undertaken on masonry spillways.

A summary of the main conclusions of the research is given in Chapter 6, however, practitioners are encouraged to familiarise themselves with the contents of the preceding chapters for background.

The report is concluded by three appendices covering industry responses, an inspection checklist and the comparison of standard flow depth formulae with actual values obtained during model tests on prototype spillways.

Ulley reservoir was completed in 1873 and the average age of British Dams is now about 110 years so there are many spillways which predate the introduction of concrete and where masonry was used. The focus of this report is to understand how these may be monitored and maintained so as to remain in effective and serviceable condition.

2 An Overview of Masonry at Spillways

2.1 Introduction

Masonry spillways are generally rectangular in cross-section and comprise three elements, an invert and two side walls (see Figure 2.1). Both inverts and walls tend to be planar, although slightly dished or concave inverts are not uncommon.

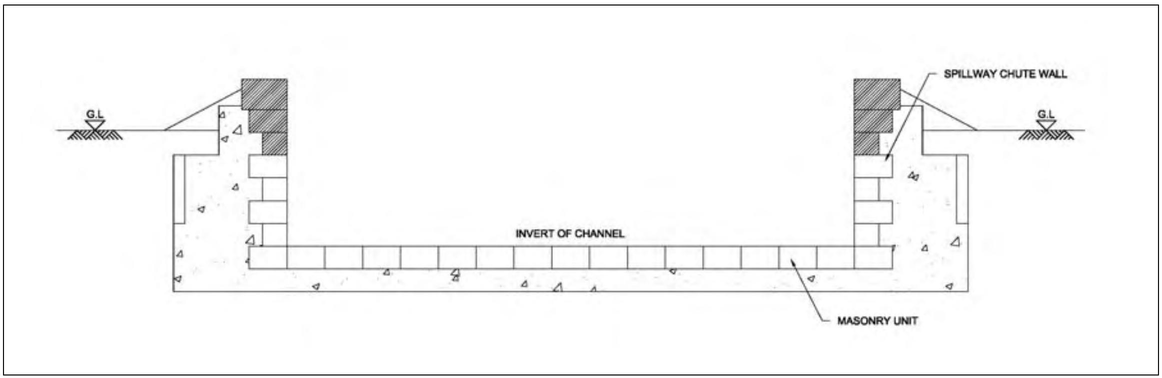

Figure 2.1 Cross-sectional View through a Spillway

Longitudinally, these spillways may follow the slope of the natural ground whereas others may comprise a series of horizontal and vertical steps. Stepped spillways are the primary focus of this report although some of the information and guidance provided may also prove useful for sloping spillways.

Sidewalls on both types of spillway can feature top surfaces which also slope with the line of the spillway (see Figure 2.2) or be formed in a series of steps (see Figure 2.3). Both types of upper wall profile can be found with both types of invert.

Figure 2.2 An example of a sloping bed in a masonry spillway, with the wall masonry laid parallel to the bed.

Figure 2.3 An example of a stepped bed in a masonry spillway. The wall masonry is laid horizontally.

Figure 2.5 illustrates the types of plans and details often associated with stepped masonry spillways. Further details of typical spillway construction can be seen in the remaining figures. One common feature in all cases is that the internal (water) faces of the spillways tend to be in dressed masonry. These may often be bedded into backing walls of random rubble masonry. Such an arrangement can be seen in Figure 2.4.

Figure 2.4 Examples of failures of masonry spillways. The spillway wall on the left has random rubble masonry backing, whilst the spillway wall on the right has mass concrete hearting between the random rubble backing and the dressed face.

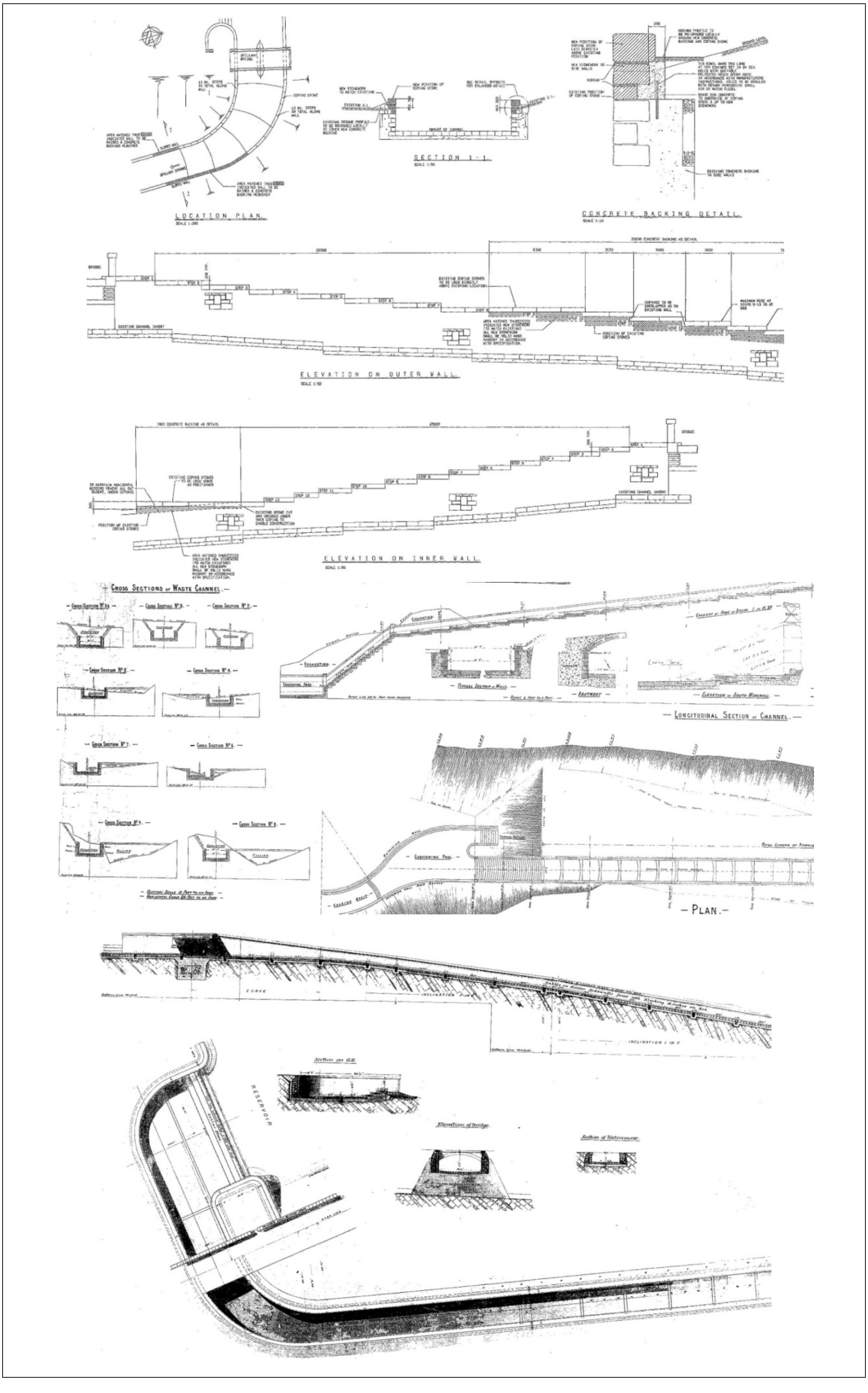

Figure 2.5 Typical original plan and cross-section drawings of masonry spillways

This report is generally focused on the hydrodynamic effects of high velocity flows on steps in masonry spillways and the forces and damage on such spillways that these flows can cause. However, this chapter is intended to give a particular understanding of masonry as a structural material and the other factors that might lead to its deterioration and susceptibility to damage.

Later chapters of the report will build on this and discuss specific aspects such as guidance for inspection, maintenance, repair and reconstruction of stepped masonry spillways.

2.2 Types of Masonry

Masonry comes in a variety of forms and can be broadly categorised as stone masonry or brickwork masonry. Stone masonry structures can be either random, brought to courses or coursed; brickwork masonry structures are coursed, although there are several bonding patterns that are typically used.

2.2.1 Stone Masonry

Stone masonry can be further divided into the following types:

- Non-mortared or “dry stone wall”;
- Random rubble;
- Stone that has been brought to courses (coursed stone with larger, irregular sized stones filling gaps);
- Coursed stone;
- Coursed dressed stone; and
- Ashlar.

Dry stone wall and random rubble masonry tend to be used in older constructions, while newer constructions tend to comprise cut blocks.

Figure 2.6 to Figure 2.11 show examples of the various types of stone masonry described above.

Figure 2.6 Example of dry stone wall masonry

Figure 2.7 Example of random rubble masonry backing to coursed dressed stone masonry wall

Figure 2.8 Example of stone that has been brought to courses

Figure 2.9 Example of coursed stone masonry

Figure 2.10 Example of coursed dressed stone masonry laid horizontally

Figure 2.11 Example of coursed dressed stone masonry laid horizontally with ashlar masonry copings

2.2.2 Brick Masonry

Brick masonry spillways generally comprise engineering bricks. The colours may vary to give a more sympathetic finish with the surrounding area. Figures 2.12 to 2.14 are examples of the more common bonding patterns to be found in brick masonry.

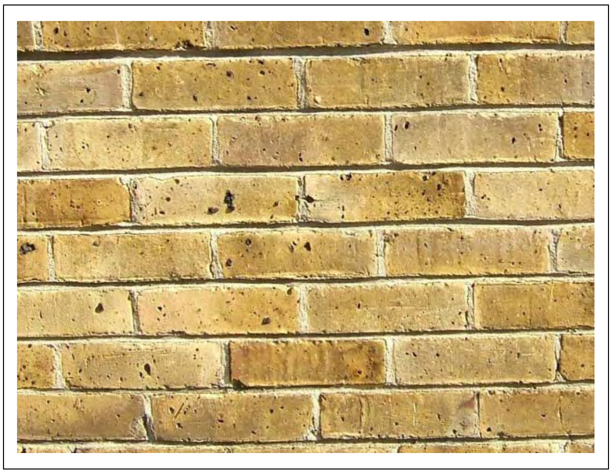

Figure 2.12 Example of stretcher bond masonry

Figure 2.13 Example of Flemish bond masonry

Figure 2.14 Example of English bond masonry

The type of bonding pattern used can given an indication of what lies in the bulk of the masonry behind the wall, for example, a stretcher bonded wall is a flat plate, half a brick thick, made up of bricks that run along the length of the wall. It does not include any bricks running through its thickness, so it is difficult to build it into walling behind it without using ties. As a result, it is likely that the masonry behind it is of a different type - such as random rubble.

At the same time, while English and Flemish bonded walls are typically at least one brick thick, they can be formed with what are called snapped headers - bricks that are cut to be shorter sometimes just half a brick thick.

2.3 Vulnerabilities

The sidewalls are, in masonry terms, retaining walls, in that spillways typically sit in the ground with the surrounding soil abutting their rear surfaces. However, the slope of the ground will generally ensure that water does not build up behind them causing the build up of hydraulic pressure from the rear. In fact some spillways will feature porous and/or drained backfill to ensure this.

However, this will not always be the case. Sections of spillway immediately upstream of clay cores or other waterproof elements on dams may be empty while having to resist external reservoir pressure. Similarly there may be instances where leakage paths within the masonry transmit water from one zone to another, external zone.

Similar arguments to the above can also apply to inverts. Ideally, the substratum of inverts should be free draining so that any seepage flows are transmitted away without being able to build up pressure. However, in zones upstream of clay cores, for example, the masonry may effectively form a forebay and be required to resist full reservoir pressure without failure, instability or distress.

All of the vulnerabilities mentioned above are intrinsic vulnerabilities; however, masonry spillways are also subject to deterioration. As can be seen from the nature of this report, masonry spillways can degrade due to internal hydrodynamic pressure fluctuations from the spillway flow. Such hydrodynamic pressures can dislodge both masonry elements and/or the mortar pointing between them. Furthermore, the loss of individual elements will increase local turbulence leading to further progressive failures to the zones immediately downstream. This effect can be further exacerbated where the sound “water face” of dressed masonry is backed by a much weaker zone of rubble masonry which becomes exposed to the same erosive forces once the dressed masonry has been removed.

Examples of localised loss are shown in Figure 2.15. Widespread external failure is illustrated on Figure 2.16. The same vulnerabilities apply to the invert as to the sidewalls, as can be seen in Figure 2.17. Details of examples of problems at stepped masonry spillways in the UK are given in Appendix A.

Figure 2.15 Examples of localised loss of masonry blocks

Figure 2.16 An example of a widespread failure of a stepped masonry spillway

Figure 2.17 Examples of invert failures of stepped masonry spillways

Such failures only serve to illustrate the need to maintain both the masonry blocks and the mortar pointing in good condition.

In a wider context the following problems can arise in masonry side walls and inverts:

- Cracking;
- Leaning;
- Loss of mortar pointing;
- Degradation of individual masonry blocks;
- Intrusive vegetation growth;
- Material solution by acidic and other aggressive waters;
- Inappropriate maintenance and repairs;
- Damage to foundations;
- Displacement due to frost heave.

These are explored in more detail in later sections of this chapter.

Failures of masonry spillways can be caused by a number of factors, acting either independently or in unison. Over and above any internal hydrodynamic forces from spillway discharge, the following are the three other principal generic causes:

- Foundation failure;
- External flow erosion;
- Masonry deterioration.

2.4 Foundation Failures

In the case of spillways, foundation failures are typically associated with the spillway foundations being undermined by water leaking through the bed, or invert of the spillway and washing material away as it does so.

Over time, this can result in the creation of voids beneath the invert leading in turn to the settlement and cracking of both the invert and the sidewalls. Examples of localised masonry invert failure are shown on Figure 2.2 and Figure 2.17. In the feedback received as part of the preparation for this report, examples were quoted of spillways where low flows disappeared into the invert masonry at the head of the spillways and reappeared at the toe. The passage of low flows was, therefore, travelling in voids below the main masonry surface.

A limited movement of the foundations is likely to be tolerated by the spillway, although this could well lead to the spillway creeping and bulging in places. When the movement becomes too great, the spillway is likely to crack.

There are a number of reasons why the foundations might move. These can include, ground heave, ground subsidence and slope stability, with a whole range of potential causes including, as discussed above, water flowing through the base of the spillway.

Ground heave can lead to the generation of cracks within a spillway. It occurs mainly in clay soils, with the take up of water being a principal cause, although the removal of a large tree in the area may result in the same symptoms. Such cracks are likely to vary in width depending on the season - they typically close up in winter and open in summer.

Subsidence can also lead to cracking within a spillway. It can occur in any soil and is often caused by water being removed from the ground, for example by trees, or material being lost. Again, the cracks are likely to vary in width depending on the season -they typically close up in winter and open up again in summer.

2.5 External Flow Erosion of the Spillway

External flow erosion is associated with rainfall runoff flowing down the area immediately behind the sidewalls, leading to the removal of soil from this location. Where the wall has been designed to assume such support, this can leave the sidewall vulnerable to collapse under high discharge flow.

Another possible reason for the loss of such support soil can be overtopping of the spillway walls during spillway discharge. Advice is given in Chapters 3 and 5 of this report regarding appropriate formulae for estimating flow depths, however, such depths are not absolute but rather, reducing percentages of water concentration. Furthermore, local features can give rise to cross waves and local overtopping.

In some cases flow erosion can also apply to the erosion of the soil or foundations at the downstream end of the spillway, resulting in regressive undermining.

2.6 Masonry Deterioration

Both the mortar and the masonry blocks are susceptible to damage. The deterioration of the mortar can be associated with both chemical and physical processes.

The ability of mortars to resist either chemical attack or simply the effect of water seeping through them, will depend on the binder used. Typically, older spillways will have been built using lime mortars, and the binder element, the lime, is essentially leached from the mortar over time, leaving just sand. In more modern mortars, cement is used as the binder. This is less susceptible to being leached from the mortar by water, although its strength can be reduced to almost zero by the presence of certain chemicals, such as sulfates.

Damage to masonry blocks can also arise from chemical attack, although it tends not to lead directly to the failure of the masonry. Typically, the mortar is weaker than the blocks and degrades more quickly. It is, therefore, the primary cause of the failure, although the masonry blocks may also be in a poor state when this happens.

Another factor which acts to degrade masonry is dampness. Without inherent dampness, masonry is much less susceptible to either frost or chemical attack. Therefore, if a wall remains dry and excludes water, it is likely to remain in good condition. The reasons for this are discussed later in this chapter. Indeed it may seem strange to suggest that a spillway should be kept dry. However, when the spillway is not in use, there are a number of reasons why the life of the masonry will be enhanced if it can be protected against inherent damp.

Dampness can result from poor design or maintenance, for example where the tops of walls are not adequately waterproofed or from external factors such as cracks in a spillway caused by ground movement creating a conduit for water to pass into the spillway wall. The likelihood of dampness occurring is increased in areas of high groundwater levels.

Masonry needs to be able both to resist water ingress from any source and be constructed from materials that make it as resistant to degradation as is reasonably possible. In any event, the external percolation of water should, as much as possible, be excluded from the masonry in a spillway. Where the water is acidic, the presence of water within the spillway can be particularly problematic as the acids will have a dissolving effect on the cement mortar and any aggregates containing carbonate.

The potential for water to cause deterioration to concrete can be assessed using the Langelier Index (ICOLD, 1989). This saturation index considers hardness, alkalinity, pH, temperature and total solid content in assessing the aggressivity of water. The index is given by:

$$LI = pH + \log C + \log A - 0.025\,T - 0.011\,S^{0.5} - 12.30$$

Where:

LI = Langelier Index

pH = pH value

C = Calcium hardness or calcium ion content expresses as $CaCO_3$ (mg/l)

A = alkalinity expressesd as e.$CaCO_3$ (mg/l)

T = temperature in ^{o}C where $0 < T < 25$ ^{o}C

S = total dissolved solids in mg/l where $S < 1{,}000$ mg/l

A negative value of LI indicates that the water is aggressive, with values more negative that -1.5 showing the water to be very aggressive. In these circumstances, concrete will be corroded. Conversely, a positive LI value indicates that the deposition of calcium is likely.

2.7 Specific Inherent or Internal Problems

This section discusses specific causes of masonry degradation associated with the design and selection of materials.

2.7.1 Coping Blocks

The top of a wall is one of the main routes that allow rainwater water into masonry. As a result, it is important that wall tops have appropriate damp proofing in place to stop water ingress and suitable cappings or copings to throw water falling onto it clear of the wall. In practice, construction of most masonry spillway walls focuses more on substantial and sound copings to shed rainwater from the wall rather than on using damp proofing materials.

2.7.2 Foundation Problems

As already mentioned in the previous section, if the foundations are inadequate, then it is likely that parts of the spillway will subside and the structure will crack. This may be an issue stemming from inherent inadequacies in the original design or problems may have occurred through inadequate preparation during construction. In addition, long-term deterioration of the foundations may occur over time.

2.7.3 Materials

The use of appropriate materials is vital. In most cases, a masonry spillway is likely to have been built using masonry blocks that are frost resistant, so it is unlikely that the blocks will degrade or erode. Similarly, bricks used in masonry spillways were historically of high quality and frost resistant, so are also unlikely to degrade or erode.

However, the same cannot be assumed of the mortars used. Early spillways are likely to have been constructed using lime mortars.

Without specific information to the contrary, it is always safest to assume that the materials present in a spillway could be susceptible to damage and to assess them with suitable care and attention.

2.7.4 Natural shrinkage

A number of masonry materials, including mortar, are made using cements and cementitious materials that shrink after they have been produced. As a result, there is a tendency for cracking to occur within a wall built with these materials. Stone will also shrink, to an extent, but its shrinkage coefficient is much smaller than that of concrete blocks.

Bricks expand after manufacture as they absorb water and so shrinkage cracking tends to occur in brickwork at those times of year when the bricks are drying out, such as in the summer, after having previously absorbed water.

These factors need to be taken into account when designing either spillways or remedial works to them.

2.7.5 Poor workmanship

Throughout their lives, spillways are susceptible to poor workmanship. The main area where workmanship issues can very easily affect the quality of the spillway is the mortar and its application. Missing, or poorly filled, horizontal bed joints and vertical joints will affect the integrity of the structure and reduce its ability to resist erosion by water flow. The use of a poorly mixed and/or a poorly gauged mortar will affect its longevity, as it will be more susceptible to both frost attack and erosion.

Other examples of poor workmanship include blocks that are not cut square (see Figure 2.15) and masonry blocks that have been laid in the wrong orientation. Wall blocks should be laid on their natural bedding plane and invert blocks should be laid either on their natural bedding plane, or, preferably, on edge in order to avoid delamination.

In addition, poor workmanship can result in a lack of integrity between the various sections of a spillway. For example, if the surface layer of masonry is poorly fixed to the materials behind it, then there is an increased likelihood that sections will come away from the body of the spillway over time.

2.7.6 Low quality maintenance

A particular area of concern can be the quality of re-pointing work. It is not uncommon to find repairs where the re-pointing mortar consists of little more than very thin smears, typically up to 5mm thick, of an inappropriately hard mortar that has been spread over the outer surface of the existing mortar. In these circumstances it is not uncommon for large areas of the repointing mortar to fall out of the bed joints. An example of missing mortar beds can be seen in Figure 2.18.

Another common failing is the use of inappropriate mortar finishes. Fully recessed or proud mortar finishes are more likely to allow water into the wall than a struck flush or recessed, bucket handle, finish.

A maintenance programme should be implemented to ensure that the condition of the spillway does not degrade over time. Maintenance should be undertaken regularly and frequently, with any required repair works executed in a timely manner.

Figure 2.18 An example of poor pointing and missing mortar beds in a spillway wall

2.8 Specific External Factors

There are a number of external factors that can affect a spillway and these are discussed below.

2.8.1 Frost

Frost damage occurs when masonry materials are wet or damp. As the external faces of a spillway are in contact with soil, it is likely that they will remain relatively damp for an appreciable proportion of the year. As a result, in areas prone to frosts, this is likely to lead, over time, to the degradation of one or more aspects of the masonry.

This degradation is likely to take the form of the mortar crumbling away and/or coming loose and individual bricks or stones on the surface of the side walls either spalling, crumbling, delaminating or also coming loose.

It is quite common to see the top 450 mm of masonry wall displaced inwards due to frost heave behind the wall, or the coping pushed inwards.

2.8.2 Chemical Attack by Sulfates

The main form of chemical attack associated with masonry is sulfate attack, also known as sulfate expansion. This is caused by a chemical reaction that occurs when the cement in the mortar and waterborne sulfates, having been dissolved out of either the soil or the masonry blocks, come into contact with each other.

Sulfate attack leads to the production of an expansive salt which forms within the mortar and causes horizontal cracks to form in the mortar beds. Depending on where in the bed joints the reaction takes place, sulfate attack can cause walls to lean.

Typically, sulfate attack occurs in the parts of a wall that are the dampest. This is normally at the tops of walls, although the fact that all the rear faces of spillways are in contact with the ground and that the inner surfaces will also be occasionally wetted, suggests that any part of a spillway could be susceptible to this form of reaction.

There are no cures for sulfate attack - the affected parts need to be taken down and rebuilt using materials that are sulfate resistant. In new works, protective water barriers can be incorporated and use can be made of special cements such as sulfate resistant cement.

A much less common form of sulfate attack on masonry - or concrete - is called thausmasite sulphate attack (TSA). TSA occurs when calcium silicate hydrate, carbonate and sulphate ions react to produce thaumasite. For this to occur, several conditions must be coincident:

- Presence of sulfates. This source is generally provided by sulfates or sulfides in the ground;
- Presence of mobile groundwater;
- Presence of calcium silicate hydrate. Generally derived from cementitious calcium silicate phases present in Portland cements;
- Presence of carbonate. Generally this is found in coarse and fine concrete aggregates, as bicarbonate in groundwater or as a constituent of the cement;
- Low temperatures (thaumasite formation is most active below 15 °C); and,
- pH of 10.5 or greater, such as that found in the cement paste matrix of non-carbonated concrete.

The calcium silicate hydrates provide the main binding agent in Portland cement, so this form of attack leads to the complete loss of structural integrity and strength in affected areas, as well as causing some expansion and, in advanced cases, the mortar is eventually reduced to a mushy,incohesive mass. As with standard sulphate attack, when TSA occurs, the whole section of affected masonry needs to be taken down and rebuilt. BRE (2005) discuss the causes and consequences of TSA in greater detail.

2.8.3 Other Forms of Chemical Attack

In some areas, such as peat catchments, the water flowing into the dam and down or around a spillway will be acidic and this acid can attack the mortar present in the masonry, causing it to erode over time.

Salt crystallisation occurs when salts in solution within masonry crystallise out within the body of the block, typically a short distance in from the block’s outer surface. When this occurs, sections of the front surface of the masonry either shear or pop off.

Typically, areas of salt crystals will form on the surface of these sections of the blocks, including on the outer surface. It is difficult to stop this process.

2.8.4 Vegetation Growth

A number of shrubs, trees and grasses have been known to grow within masonry. These include buddleia, sycamores and ivy.

Typically, ivy tends to cover and fix itself to the surface of walls. In this position it is only slightly invasive. However, its root system is much more invasive. Larger bushes and trees have much larger roots systems and, consequently, have the potential to be much more damaging to spillways.

Examples of vegetation growing on or in masonry spillway channels can be seen in Figure 2.19.

Figure 2.19 Examples of vegetation growing in or on stepped masonry spillways

There are three principal mechanisms by which vegetation assists in the degradation of masonry:

i. Their roots break up the structure of the spillway, loosening the surface blocks and causing increasing amounts of damage over time;

ii. Vegetation retains moisture in the masonry, allowing chemical reactions to take place;

iii. Vegetation acts to physically slow the passage of water in the spillway, reducing the spillway's effective capacity and increasing the likelihood of it overflowing.

Good housekeeping dictates that vegetation be removed from spillways as soon as possible.

Except for the smallest plants, it is best to use a systemic weed killer to kill the vegetation before attempting to remove its root system from a wall. This will make the process easier and will ensure that any roots that remain will be dead, leaving them unable to cause further damage.

Care should be taken when removing vegetation growth from a spillway, even when it is dead, because unsympathetic removal can cause whole areas of masonry to come loose and in need of repair.

It is possible that, in some circumstances, it would be more appropriate to remove specific masonry blocks from a face to allow the removal of a plant's root system and then replace them rather than simply pulling at the trunk of a plant.

2.8.5 Tree Growth

One cause of ground movement and cracking in spillways can be tree growth. This can have the knock-on effect of allowing water to pass through the masonry leading to localised voids or more general soil erosion from beneath the spillway. Further deterioration of the spillway is likely in both cases.

The species of tree, the tree height and its distance from the closest point of the spillway are the most important factors associated with tree damage to spillways.

Depending on the species, as a general rule of thumb, if a tree is more than 10 m from a spillway then it is unlikely that it will have an impact on the spillway foundations unless it is a very large example of the species. However, at a time when there is a water shortage, a spillway that is already cracked and is seeping water will attract the roots of large trees in the area, and this can lead to an increased risk of damage.

The following box summarises tree types and the maximum distances from which they have been shown to cause damage to houses. Whilst acknowledging that the foundations of spillways may be more substantial than those of houses, it is considered, nevertheless, that a similar level of potential threat can be assumed.

Box: Information on damage caused to houses by trees

As an indicator of the tree types most active in causing damage generally, the top five trees causing damage to houses are, in order of decreasing damage:

- Oak (13m)
- Poplar (15m)
- Lime (8m)
- Common ash (19m)
- Plane (7.5m)

Figures in brackets are the maximum distances that the trees were away from the damaged houses for 75% of the instances of damage (while there were a number that were further away, this represents the greater majority).

While the foundations of a spillway are likely to be considerably more solid that those of a typical house, there is no reason to believe that a spillway would be invulnerable to damage from trees or that the trees causing the damage are likely to be any different.

Direct root damage to spillways is also possible in this situation. Figure 2.4 shows an area next to a failed spillway that contains heavy undergrowth; the roots associated with the vegetation can clearly be seen in the soil behind the failed sidewall.

3 Current Hydraulic Understanding

3.1 Introduction

Stepped spillways using masonry construction or timber cribs are not a new idea; Chanson (2001) describes the development of the technology from ancient Greek and Roman origins through to more recent highly engineered structures. He points out that current interest in the design process is not a new finding but an awakening to a technique that has been tried and tested on some structures for decades or even centuries. Chanson's paper contains two tables listing stepped spillway sites from around the world from antiquity to the first part of the 20^{th} century.

Various researchers have looked at flow over stepped and smooth channels subjected to high velocity flow. They have measured pressures, velocities, air content, forces and turbulence to give a greater understanding of factors affecting the stability of the lining. Others have conducted tests on loose blocks of rock, concrete or masonry to determine failure conditions. Most of the work conducted to date has looked at the channel invert in a hydraulically wide channel with little emphasis on failure of the walls. The work of these researchers and a summary of current understanding are presented in this chapter.

In reviewing the capacities of stepped masonry channels it may also be necessary to make allowances for bends, curvature, super-elevation, obstructions or any other changes in geometry or cross- section, just as one would for any open channel flow calculations. However, the effect of these features on highly aerated stepped chute flow was not studied as part of this research. Physical modelling, mathematical modelling and engineering judgement may all be appropriate means of assessment in these cases.

Laboratory work has been carried out in parallel with this study to look at the effects of the flow on the stability of wall elements and a summary of the findings are presented in the following chapter.

3.2 Theoretical Considerations of Forces on a Masonry Block

Investigation on rock elements by Smith (1986) showed that their stability was a probabilistic event occurring when the turbulent force fluctuations occurred in the most unfavourable manner. These fluctuations were found to be random and normally distributed, and for short periods of time to be considerably in excess of the mean. The failure of masonry blocks in high velocity turbulent flow will be a similar probabilistic event and hence force equations need to be expressed in a probabilistic form.

The implication of this situation is that the stability of a masonry block cannot be evaluated as a finite value but must be expressed in terms of a risk of failure. For example, a block will lift if the force underneath is larger than the weight plus any down force. Failure is thus most likely to occur when the pressure below is experiencing a large positive fluctuation which coincides with a low negative fluctuation above.

The probability effect can be expressed in terms of the distribution parameters of fluctuating variables:

σ_X is the standard deviation

c_X is a multiplying constant to give failure condition

x represents the variable under consideration.

The flow conditions on a dam spillway normally result in the development of a two phase flow with an air water mixture. The air content dampens the turbulent fluctuations which slightly increases the block stability because the standard deviation of the pressure, shear stress and velocity fluctuations, σ_x , is reduced and hence a higher mean value can occur before critical conditions are reached.

In the equations below the following notation applies, Figure 3.1:

l = Block length

b = Block width

t = Block thickness

p = Pressure (fluctuating) suffix indicate location

τ = Shear stress (fluctuating) suffix indicate location

ρ = Density of water

ρ_B = Density of block

d = Water depth

α = Angle of inclination of the panel of blocks

μ = Coefficient of friction between block and underlayer

v = Velocity

C_L = Coefficient of lift

f = Force

c = Fluctuation variable

σ = Fluctuation variable

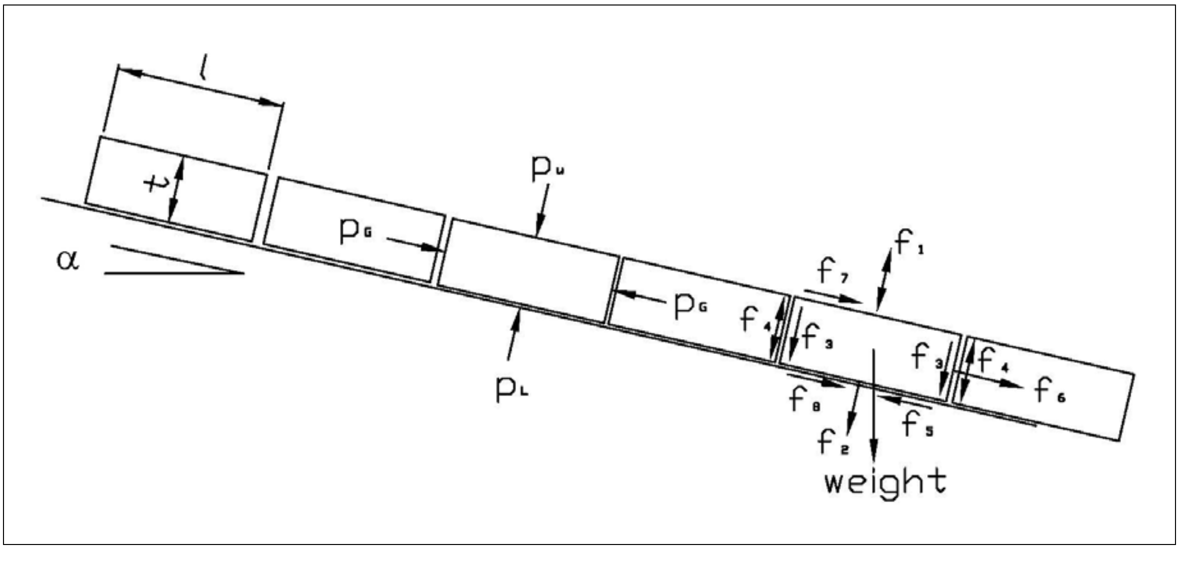

Figure 3.1 Forces and pressures acting on a masonry block in the invert

3.3 Forces Causing a Block on the Invert to Lift Normal to the Channel Bed

The difference in pressure between the upper and lower surfaces of the block is a fluctuating variable and can result in either an upwards or a downwards force. This is expressed by:

$$f_1 = (p + c_p\ \sigma_p)\ l\ b$$

Where:

$$p = p_L - p_U$$

For the blocks to be stable this must be exceeded by the normal component of water and block weight:

$$f_2 = (\rho d + (\rho_B - \rho)\ t)\ g\ l\ b\ \cos\alpha$$

plus the frictional resistance between blocks, f_3.

Baker (1990), working with concrete blocks in high velocity flow, found force f_3 to be significant but it is an ill-defined variable being dependant upon factors such as joint width, type of material in the joint and relative coefficients of friction. In a masonry spillway the joint will often have been pointed and on an old spillway the state of the pointing could be very variable adding further problems to the accuracy of any analysis.

There may be an additional force f_4 representing the shear force generated by water flowing in the gaps between blocks, this could be upwards or downwards dependent upon the direction of the flow of water:

$$f_4 = (\tau_G + c_{\tau G}\ \sigma_{\tau G})\ b\ t$$

.

For equilibrium of the block normal to the slope, these forces combine to give:

$$f_1 = f_2 + f_3 \pm f_4$$

A larger uplift force, f_1, would result in block movement, but it may not result in failure, since as soon as a block lifts, pressure relief will occur in the void created underneath resulting in a reduction of p_L and hence p. At the same time the block would project into the flow setting up different hydraulic conditions as described below.

The design graphs presented in Section 5.1.4 could be used to assess the velocity head that would be mobilised by an upstand in the invert.

Further complications occur when the vertical alignment of the spillway changes. A steepening of the gradient over a vertical curve, or edge, can set up separation pockets with associated zones of low pressure, whilst a flattening of the gradient will cause impact forces on the bed.

3.4 Forces Causing a Block to Slide

Assuming that the block system is stable, then the reaction force from the blocks upstream, r_U, must be transmitted through the block, and after combination with the forces acting on the block the resultant must be balanced by the force from blocks downstream, r_D. Ultimately this force must be resisted either by anchorage at the toe of the slope or by frictional resistance between the blocks and underlying material.

The frictional resistance between a block and the underlayer is given by:

$$f_5 = (\rho\ d + (\rho_B - \rho)\ t)\ \ g\ l\ b\ \mu \cos \alpha$$

The blocks will fail by sliding if the reaction $r_D + f_5$ is not able to resist the total sliding forces. These are made up of:

The parallel component of water and block weight;

$$f_6 = (\rho\ d + (\rho_B - \rho)\ t)\ \ g\ l\ b \sin \alpha$$

The fluctuating shear force between block and main flow;

$$f_7 = (\tau_U + c_{\tau U}\ \sigma_{\tau U})\ l\ b$$

The fluctuating shear force between block and seepage.

$$f_8 = (\tau_L + c_{\tau L}\ \sigma_{\tau L})\ l\ b$$

For equilibrium parallel to the slope without additional shear restraint:

$$r_D + f_5 = r_U + f_6 + f_7 + f_8$$

3.5 Failure by Rotation about Downstream Edge

Baker (1990) presented the failure of a thin block made from concrete as a rotational failure about the downstream edge. This consideration leads to a further force, p_G, from the pressure in the gap between blocks. In normal circumstances this would balance across the block and hence not be significant. However, if a block stands slightly proud of its upstream neighbour the resulting lip causes a stagnation pressure to occur and this pressure will be transferred down the joint increasing the upstream value of p_G and significantly increasing the sliding force. The pressure will also be transferred under the block increasing the value of p_L and hence the lifting force.

In addition, the curvature of streamlines over the lip introduces a lift force that adds to the other forces and can be defined by:

$$f_{lift} = \tfrac{1}{2}\ C_L\ \rho\ (v + c_V\ \sigma_V)^2\ l\ b$$

This type of rotational failure may not be possible in parallel sided masonry blocks with a significant depth, although it would be possible in surfaces made from thin sheets of masonry surfacing or from blocks that have been tapered by the stone mason so that the back face is smaller than the front face (see Figure 2.15).

3.6 Types of Masonry Spillway and Flow Regime

A masonry spillway can follow two structural designs, a flat spillway or a stepped spillway; although in practice on many existing spillways in the UK, the designers have adopted a combination of flat spillway channels separated by stepped cascades.

On the flat spillway, flow will be supercritical with a very high velocity head component to the specific energy available. Any obstruction to the flow will convert this velocity component into a stagnation pressure that could influence the stability of the masonry lining. As flow accelerates down the slope, it will go through zones of developing boundary layer and developing air entrainment until on a long straight slope, fully air entrained flow at normal depth occurs. In this flow regime, the water depth is ill-defined because the flow passes from water with air content on the bed to air containing water droplets many metres above the perceived water surface. Flow conditions can be further complicated by the geometry of the spillway which will often set up cross-waves that form zones of deep and shallow water throughout the spillway channel with resulting areas of high velocity flow jets.

On stepped cascades, low flow will pass over the edge of each step in a free trajectory and land on the step below with an air pocket in the lee of the step; this is referred to as nappe flow, Figure 3.2. As the quantity of water increases, the air can be drawn out of the nappe so that it fills with water, the main flow then skims over the surface of the steps in a regime called skimming flow, leaving a rotating core of water in the lee of the step, the centre of which is at a reduced pressure, Figure 3.3. Apart from a small amount of additional roughness added by the step, and the generation of the low pressure zones in the lee of the step, the flow conditions in skimming flow are very similar to those of a flat spillway. Chanson (1994) reports that at the transition between the two flow regimes, considerable flow instability and pressure fluctuations can occur which can generate a more severe design condition than either of the two flow states themselves.

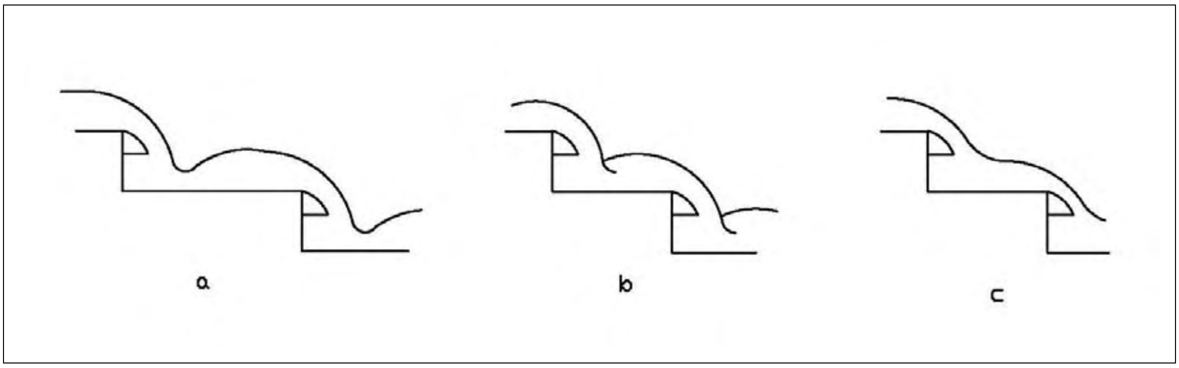

Figure 3.2 Nappe flow

a) Fully developed hydraulic jump b) Partial jump c) No jump

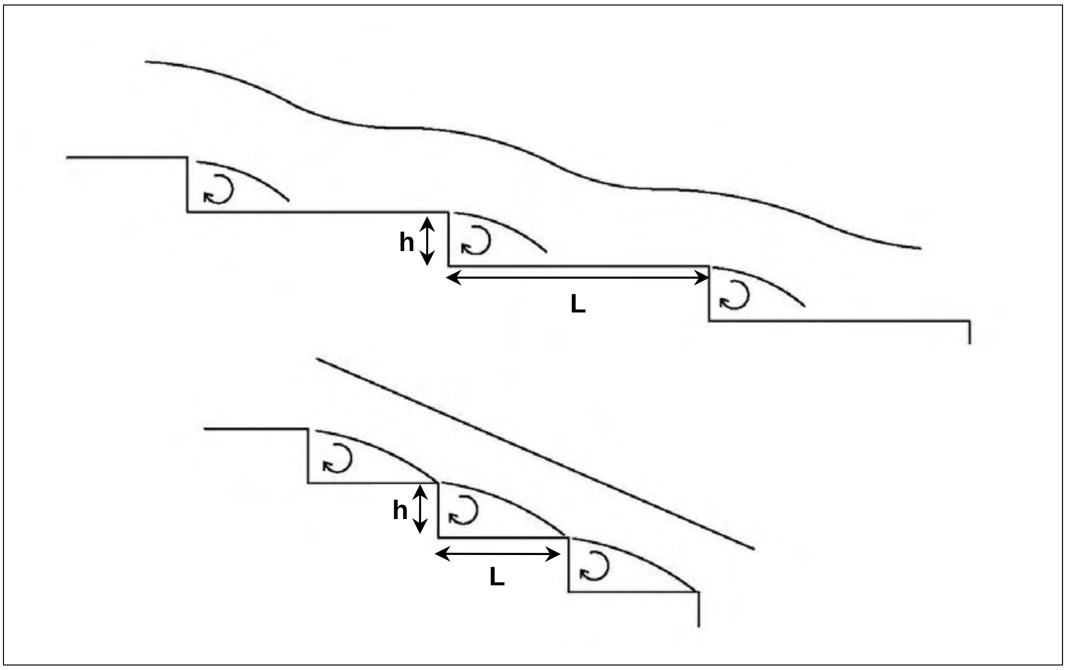

Figure 3.3 Skimming flow over steps

Ohtsu et al. (2004) report two different skimming flow regimes as shown on Figure 3.3. At low angle slopes the flow passes over the edge of the step and then re-attaches part way along the next step, the resulting water surface is not necessarily parallel to the line joining the tips of the steps. At higher slope angles the flow completely skims the tips of the steps and the water surface is parallel to the line joining the tips of the steps. For steps with a horizontal tread they suggest that the flow regime changes over when the slope angle is 19^o.

Sometimes on spillways made from a combination of spillways and step cascades, water will arrive at the top of the cascade with sufficient velocity for the trajectory to leap a number of steps or even the complete cascade. This can lead to problems of impact damage which are beyond the scope of this report, although equations in section 3.7 may be used to give an estimate of the pressures that could be generated.

A good overview of the hydraulics of stepped channels and of the various design equations proposed by different authors is presented in Chanson (1994) or the more recent update Chanson (2002). These books contain a large list of references to work conducted in this field which is beyond the scope of reproduction in this publication. A range of papers on the subject were also presented at the International Workshop on Hydraulics of Stepped Spillways at Zurich in Switzerland, this is documented in Minor & Hager (2000).

3.7 Nappe Flow on a Stepped Spillway

A spillway can be designed so that it only operates in nappe flow, although for most spillways this will be the flow regime at low flow only. The transfer from nappe flow to skimming flow is generally accepted to occur at discharges larger than a critical value where:

$$d_c = 1.057\,h - 0.465\,\frac{h^2}{L}$$

Where:

d_c = critical depth at the onset of skimming flow

h = step height

L = step length

Chanson (2001,2) actually suggests two limits for flat slopes ($3.4^o < \alpha < 60^o$), with a change from nappe flow to transition flow (see section 3.8) at:

$$d_c = 0.89\,h - 0.4\,\frac{h^2}{L}$$

and a second limit for the change from transition flow to skimming flow at:

$$d_c = 1.2\,h - 0.325\,\frac{h^2}{L}$$

If the spillway is to be designed for nappe flow then Stephenson (1991) suggests that a shallow slope is required with a step height to length ratio less than 1:5 (horizontal step) and flow conditions that generate dc<3h. This will be too shallow a slope for most UK applications.

As water drops over a step edge, it impacts on to the surface of the step downstream and this could lead to damage to a masonry surfacing. Chanson (1994) proposes a mean pressure (P_s) given by:

$$P_s = 1.253\, \rho\, h\, g \left(\frac{d_c}{h}\right)^{0.349}$$

This pressure will fluctuate and May & Willoughby (1991) suggest that the upper and lower limits of the fluctuations are given by:

$$P_s + 0.9\, \rho\, \frac{v^2}{2} \quad \text{and} \quad P_s - 0.6\, \rho\, \frac{v^2}{2}$$

Where:

v = impact velocity of the free falling flow

Chanson (2000) points out that this pressure could be as much as 10 times the hydrostatic pressure.

On the step, three different conditions can occur, Figure 3.2:

i. Formation of critical depth on the step edge with a full hydraulic jump on the step;

ii. Formation of a partially developed hydraulic jump on the step with a depth less than critical depth on the step edge;

iii. Super-critical flow on the step without a hydraulic jump.

In cases where a hydraulic jump forms, further damage could result to the masonry lining due to fluctuating forces under the jump.

Some designers have added end sills to the step to form a pool of water on the step, encouraging the formation of the hydraulic jump, increasing energy dissipation and extending the nappe flow regime to higher flows. According to Thorwarth & Köngeter (2007) the end sill causes the development of unsteady and periodic flow vibration with frequency 0.11-0.9 Hz which may affect the stability of masonry blocks.

Further details for designing for a nappe flow regime can be found in Pinheiro & Fael (2000), Toombe & Chanson (2008), Chanson (URL).

3.8 Transition to Skimming Flow

Once the limit of nappe flow has been exceeded then the flow will change to the skimming regime. Chanson (1994) draws attention to the fact that a number of recorded failures (e.g. Arizona Canal Dam (failed 1891); Minneapolis Mill dam (failed 1899); New Croton dam (failed 1955)) occurred at discharges well below the design flow. It is possible that the failures were partly influenced by extreme pressure fluctuations at the transition from nappe to skimming flow. This area has not been extensively researched.

The main concern is that at the transition, the air filled cavity below the nappe will periodically fill with water and then return to an air void. This hydrodynamic instability could cause large hydrodynamic pressure fluctuations on the steps and unacceptable vibration of the structure. Chanson (2000) directly attributes this to the failure of New Croton dam in USA which was extensively damaged in October 1955 under a 650 m^3/s flood - the design flow for the spillway was 1,550 m^3/s.

In most UK spillways, skimming flow will occur at the design flood. It is inevitable therefore that nappe flow conditions will occur at low flows and that at some point flow must pass through the transition region. If at all possible, prolonged operation at the transition should be avoided.

3.9 Skimming Flow in Stepped Channels

The wealth of different design equations presented in Chanson (1994) & (2002) or Minor & Hager (2000) for skimming flow can lead to confusion as to the correct approach for design. Most of the equations have been derived empirically by the authors from their experimental test data and it is important that the scope of the test programme is investigated before the equation is selected. The results will be influenced, for example, by parameters such as step height to length ratio and slope of the channel and utilising an equation that closely matches the conditions on the spillway under investigation is likely to yield the best results.

Most recent research work has concentrated on a stepped design process for spillways on the face of roller compacted concrete dams, these tend to be much steeper than the equivalent stepped masonry channel (Ditchley & Campbell (2000)) and thus are probably not suitable for the analysis of a typical masonry spillway making them beyond the scope of this report.

In the UK, the CIRIA design manual, Essery & Horner (1978), is a commonly used reference for the design of stepped channels. The publication presents various design charts to determine flow characteristics both on the steps and for the design of stilling basins; worked examples in the appendices to the report explain how to use the charts. The charts cover slopes from 0 – 20^o and step height / length from 0.1 to 1.0. The method tends to produce designs with large steps with d_c / step length lying in a range 0.03 – 0.14, as a result the charts will often not be very useful for the evaluation of an existing spillway because the existing parameters may lie outside the range of one or more of the variables.

3.9.1 Air Entrainment

A key feature of skimming flow is that the rough channel bed surface formed by the steps leads to a rapid development of the bottom turbulent boundary layer. This very quickly reaches the water surface at a location known as the inception point. Thereafter, atmospheric air will be entrained into the flow so that the fluid becomes a two phase air/water mixture that migrates from water with some air bubbles on the bed to air containing water droplets which can be many metres above the perceived water surface. The water surface itself becomes ill defined and depth needs to be expressed in terms of an air water mixture, for example, d_{50} would be the point where the fluid was half air, half water by volume.

Chanson (2001, 2) describes the rapidly varying flow conditions at the inception point. Immediately upstream the flow is extremely turbulent and the free surface appears to be subjected to a flapping mechanism. At irregular time intervals, a water jet impinges on the horizontal step face and air is trapped in the step cavity, an instant later a rapid unsteady flow bulking is observed downstream. Immediately downstream of the inception point there is a very rapid rise in the air content and resultant flow bulking.

Wood et al. (1983) suggest that the inception point occurs at a distance (L_i) from the crest where:

$$L_i = 13.6\, k_s\, (\sin \alpha)^{0.0796}\, F_*^{0.713}$$

Where:

k_s = Surface roughness of the spillway, which for a stepped channel is normally taken as the height of the step measured normal to the slope. Thus $k_s = h \cos \alpha$

α = Channel slope

$$F_* = \frac{q_w}{\sqrt{g \sin \alpha \, k_s^3}}$$

q_w = Flow of water per unit width of the channel

g = acceleration due to gravity

Whilst Chanson (1994) and Gonzalez & Chanson (2007) suggest:

$$L_i = 9.719 \, k_s \, (\sin \alpha)^{0.0796} \, F_*^{0.713}$$

and Chamani (2000) proposes:

$$L_i = 8.29 \, k_s \, F_*^{0.85}$$

Where:

$$F_* = \frac{q_w}{\sqrt{g \, \frac{h}{L} \, k_s^3}}$$

At the inception point, various authors have presented equations to estimate the depth (d_i). Wood et al. (1983) suggest:

$$d_i = \frac{0.223 \, k_s}{(\sin \alpha)^{0.04}} \, F_*^{0.643}$$

Whilst Chanson (2001,2) suggests:

$$d_i = \frac{0.4034 \, k_s}{(\sin \alpha)^{0.04}} \, F_*^{0.592}$$

Chanson has published extensively on issues of air entrainment on stepped spillways, most of the papers can be freely downloaded from Chanson (URL). Some of the equations could be used to estimate air content of the flow and hence depth bulking (see 1.1.8.2) although they would involve complex calculations.

3.9.2 Flow Depth

The design of wall heights in the air bulked flow is very subjective because water droplets will be travelling in the air well above the d_{50} depth and hence the surrounding ground surface will be subjected to significant wetting. Many designers use the d_{90} depth to define the wall height. Chanson (1994) suggests that the d_{90} in the air bulked fluid can be estimated from:

$$d_{90} = d_c \sqrt[3]{\frac{f_e}{8\,(1 - C_e)^3 \sin\alpha}}$$

Where:

d_{90} = depth where the fluid is 90% air

d_c = critical depth

α = channel slope

C_e = Air concentration, 0.9 sin α (for $\alpha < 50^0$)

f_e = Darcy friction factor for the air/water mixture

$$\frac{1}{\sqrt{f_e}} = 1.42 \ln\left(\frac{D_h}{k_s}\right) - 1.25$$

D_h = hydraulic mean depth, $D_h = \frac{4A}{P}$

A = cross-sectional area of flow

P = wetted perimeter

On a smooth masonry spillway it is possible to use gradually varied flow equations to predict the water depth and then apply a bulking factor from the air concentration. Various authors have attempted to apply this technique to a stepped spillway with no success unless a 'fudge factor' is applied. Chanson (2001 ,2) suggests that this is because two of the basic assumptions in a back water calculation using a Darcy-Weisbach friction factor (f) are invalid in skimming flow over steps. Firstly, the flow conditions must be gradually varying which is not the case due to the cavity recirculation and inter mixing with the main flow. Secondly the flow resistance must be the same as for uniform flow which is also not true due to the form drag and cavity recirculation conditions. The use of gradually varied flow techniques on stepped surfaces is thus not advised.

The design of spillways is often carried out with the aid of physical models. Boes (2000) discusses the issue of scale effects caused by the difference in air content between the model and the prototype and concludes that with scaling by Froude Number similarity the minimum scale needs to be in the range 1:10-1:15 for the scale effect to be ignored, otherwise the model will under estimate the water depth that will be achieved on the real spillway.

An alternative method of assessing depth is presented by Boes & Minor (2000):

$$d_{90} = 0.55 \left(\frac{q^2 h}{g \sin\alpha}\right)^{0.25} \tanh\left(\frac{\sqrt{g\, h \sin\alpha}}{3\, q}\,[x - L_i]\right) + 0.42 \left(\frac{q^{10}\, h^3}{[g \sin\alpha]^5}\right)^{\frac{1}{18}}$$

Where, from their experiments:

$L_i = 9.72\, k_s\, F_*^{0.86}$

x = distance along the spillway from the crest

However, see section 5.1.2 for the results of a review into the most appropriate formulae to use for masonry spillways typically associated with UK embankment dams.

3.9.3 Roughness

The roughness of the spillway surface affects the amount of energy that is dissipated on the slope. It, therefore, affects the design of stilling facilities needed at the base to return the flow to a low velocity sub-critical regime in the waterway downstream of the dam. Chanson (1994) recommends using an approach based around the Darcy friction factor (f) and physical surface roughness (k_s) rather than a value calculated from Manning's n because the implied $^1/_6$ power law velocity distribution in Manning's equation is invalid in the turbulent rough flows experienced on stepped spillways.

For flat slopes (α <12^o) and 0.02< $\frac{k_s}{D_h}$<0.3 data collected by Noori (1984) suggests:

$$\frac{1}{\sqrt{f}} = 1.42 \ln\left(\frac{D_h}{k_s}\right) - 1.25$$

Where:

D_h = hydraulic mean depth, $D_h = \frac{4 A_w}{P_w}$

A_w = cross-sectional area of the water

P_w = wetter perimeter of the water

Chanson (1994) analysed data on steeper slopes from various researchers and concluded that there was little correlation between the results and proposed that f = 1.0 should be used in the analysis to give the order of magnitude of the energy loss.

Ohtsu et al. (2004) present an equation for f:

$$f = f_{max} - A\left(0.5 - \frac{h}{d_c}\right)^2 \qquad \text{When } 0.1 \le \frac{h}{d_c} \le 0.5$$

and,

$$f = f_{max} \qquad \text{When } \frac{h}{d_c} > 0.5$$

If slope is in the range 5.7^o < α < 19^o then:

$$A = -0.0017\,\alpha^2 + 0.064\,\alpha - 0.15$$

$$f_{max} = -0.00042\,\alpha^2 + 0.016 + 0.032$$

If slope is in the range 19^o < α < 55^o then:

$$A = 0.452$$

$$f_{max} = -0.0000232\,\alpha^2 + 0.00275 + 0.231$$

Chanson (2001,2) suggests that the residual head at the end of the spillway is given by:

$$H = d\ \cos\alpha + \left(\frac{q_w^2}{2gd^2}\right)$$

Where:

H = Residual head

q_w = flow of water

d = unbulked flow depth

3.9.4 Pressure Distribution

Most of the work on pressure distribution on stepped channels has looked at pressures on the invert of a wide channel; very little work has concentrated on the pressures experienced by the walls in the vicinity of a step.

Baker (1990), Othsu et al. (2004) and Sanchez Juny et al. (2005) & (2007) describe the pressure distribution on a set of steps in skimming flow. The horizontal face of the step is split into two zones. The downstream half is characterised by the flow impacting onto the step with a pressure increase on the surface whilst the upstream part of the horizontal face and the vertical face are in the separation zone with reduced pressures. The larger the flow rate, the greater the pressure differential and the degree of fluctuations. Sanchez Juny et al. (2005) state that the pressure fluctuations in the flow are in the range 5 – 10Hz, pressure distributions across the step are presented in the paper.

Amador et al. (2004) used particle image velocimetry to measure velocities and flow distribution in skimming flow on a stepped spillway. They observed significant fluctuations in the instantaneous velocity values and also high velocity impact on the downstream end of the tread of the step, which they surmised would result in significant momentum transfer to the step. The velocity in the cavity in the lee of the step reduced towards the centre of the cavity. Their velocity variations mirror the pressure distributions found by others.

3.10 Laboratory Investigations

Reinius (1986) carried out model studies into the stability of rectangular blocks of rock in a flume at the Royal Institute of Technology, Stockholm, Sweden. One of the blocks was hollow and had 14 pressure tappings around its perimeter which were linked to a piezometer board via flexible tubing. The length of these tubes was such that the pressure fluctuations were damped and mean pressures only were recorded. The blocks were laid in various arrangements such as:

- Tilted with the leading edge exposed or protected;
- Test block laid with an up or down step.

He concluded that an upstand into the flow, as small as 0.1 times the flow depth, would result in high pressure in the upstream joint, whereas negative pressure coefficients could exist in the lee of a protrusion. He surmised that if these out of balance pressures became large enough then the block would be lifted, although he did not witness this in the laboratory. His tests with the block rotated show that with the leading edge exposed, high pressure coefficients occur in the joints and low pressure occurs in the separation zone on top of the block. Conversely with the leading edge protected, low pressure coefficients were observed in the joints.

Baker(1990) carried out model tests on rectangular concrete blocks in a 0.6 m wide spillway at the University of Salford, UK. He was able to influence the flow intensity that caused failure by altering block laying arrangements. Inter-block friction, which allowed unfavourable forces on an individual block to be transferred to the neighbouring blocks, was found to be the most significant. Blocks laid in rows and columns (stack bond) were found to be less stable than blocks laid in subsequent rows overlapping by half a block length(stretcher bond). It should be noted that in this context, masonry that makes a channel wall would present a 'rows and column' pattern to the flow. Baker observed failure with slow motion video and concluded that failure was always instigated by an individual block being removed from the matrix which would normally be followed by an instantaneous progressive failure of downstream blocks. Baker also showed that protecting the leading edge with a step significantly improved stability and this work led on to parallel Russian findings and the design of wedge shaped blocks that actually used the hydrodynamics of the flow to gain improved stability, Grinchuk & Pravdivets (1977), Baker et al. (1994), Hewlett et al. (1997).

Frizell (1997) carried out work on overlapping wedge shaped concrete blocks at the USBR Hydraulic Investigations and Laboratory Services Group at Denver, USA as well as in a large-scale facility at Colorado State University, Fort Collins, USA. The latter facility was a 1.5m wide, 15m high outdoor test facility capable of unit discharges as high as $3.2m^3/s/m$ making it possibly the largest laboratory test facility ever used for step spillway work. The work confirmed the immense stability of step blocks that have the leading edge protected from the flow.

Chamani (2000) carried out experiments in a steep stepped flume constructed in the T Blench Hydraulics Laboratory at the University of Alberta, Canada, mainly investigating the point of inception for air entrainment. Tests were conducted with L/h = 0.6 and L/h = 0.8. Some of his results are discussed in the section on air entrainment.

Hager & Boes (2000), Boes & Minor (2000), Boes (2000) and Boes & Hagar (2003) carried out tests in a 0.5m wide, 5.7m long stepped flume at the ETH-Zentrum in Zurich, Switzerland, mainly investigating the development of air entrainment in the two phase flow. The flume had a steep slope that was adjustable in the range 30^o- 50^o. Some of their results are discussed in the section on flow depth.

Various large scale hydraulic model investigations of stepped spillways have been carried out at CEDEX in Madrid, Spain, Iguáciel & García (2000). Measurement of aeration, pressure fluctuations and energy dissipation are reported for specific spillway projects.

Chanson has carried out extensive work into flow over stepped spillways in a 3.3m long 1m wide flume at University of Queensland, Australia. Various slopes in the range 3.4-26^o were investigated. The flume has also been used for work on turbulence and in air/water two phase flow. Most of the publications can be downloaded free from the University of Queensland web site, Chanson (URL).

Ohtsu et al. (2004) performed a systematic investigation of skimming flow on a 0.4m wide stepped spillway at the Nihon University, Japan. The spillway had variable slope angles from 5.7-55^o and a uniform slope with step heights between 6.25 and 50mm. Some of the results are discussed in the section on roughness. Their paper includes a flow chart that will lead a designer through the calculations needed to design a stepped channel.

Sanchez Juny et al. (2005) & (2007) carried out work on pressure distribution on a steep stepped spillway at Universidad Politécnica de Catalunya in Barcelona, Spain. The test facility was intended to simulate a RCC construction and had a height of 4.3m, width 0.6 m and L/h = 0.8. Some of their results are discussed in the section on pressure distribution.

Coleman et al. (2003) investigated the removal of a rectangular prismoidal block of rock from a panel of fractured rock in a flume at the University of Auckland, New Zealand. Blocks with different aspect ratio were mechanically raised into the flow until they were plucked away. An equation that allows the vulnerability of a block on a smooth slope to be predicted relative to its shape is presented, however, it is expressed in terms of the bed shear stress which makes it more difficult to use.

$$\theta_c - 0.002 = 0.0015\left(\frac{P}{L}\right)^{-1}$$

Where:

θ_c = critical dimensionless shear stress, $\frac{v_*^2}{g\,t\left(\frac{\rho_s}{\rho_w} - 1\right)}$

v_* = critical shear velocity

t = block thickness

P = amount the block protrudes above the surface

L = length of the block parallel to the flow

Peiquing & Aihua (2007) investigated the stability of blocks of rock in the stilling area at the toe of a dam under plunging jets formed downstream of ski-jump spillways. The equations presented may be helpful in assessing the stability of masonry blocks downstream of the main spillway if the flow is projected onto the bed of a stilling basin. There is a useful table in the paper that compares the empirical coefficients obtained from similar work by 17 previous authors.

3.11 Full Scale Test Facilities

In order to assess the risk of damage to a road embankment overtopped by flood water the United States Federal Highway Administration commissioned Simons Li and Associates (SLA) to construct a special test facility and perform full scale tests on commercial erosion protection products. Early phases of the testing only looked at geotextile type materials but later phases included hard surfacing made from concrete blocks, SLA (1988,9). The test embankment was only 1.8m high and thus fully air entrained supercritical flow did not have time to develop. Various failures were observed mainly due to poor crest detail or water getting under the blocks and inducing a slip of the embankment material.

One of the tests lined the surface with concrete building blocks, these had a much larger depth to length ratio than the commercial concrete erosion products tested and they performed much better. SLA attributed this to increased block weight but it is probably also due to the prevention of a rotational failure (section 3.5) and hence the need to lift the block a significant vertical distance before it could be withdrawn from the matrix. SLA conclusions include two recommendations relevant to masonry lined spillways:

- Occurrence of flow beneath the block system is undesirable because it can lead to failure either by erosion, or by uplift, if not relieved at the toe; and,
- Conversely, that the provision of a drainage medium beneath the blocks is important to allow pressure relief to occur. However, the drainage layer must have the capacity to conduct water beneath the system at a rate greater than that at which water is entering the sub-block environment.

Baker (1997) carried out tests on a full size concrete wedge block spillway at Brushes Clough, UK. Data collected showed that considerable energy was dissipated on the spillway and that a surface roughness (k_s) equal to the step height was appropriate for design. Air bulking between 1.5 and 2 times the calculated water only depth was measured. The blocks performed well with no sign of distress due to hydraulic considerations although the concrete did suffer damage from vandalism.

3.11.1 Observations from Spillway Failures

Chanson (1994) has a whole chapter related to analysis of the failure of masonry stepped spillways, the notable lessons appear to be that there is no evidence to suggest that stepped masonry spillways are any worse than a smooth channel. Factors affecting the failure can be attributed to hydrodynamic considerations like force fluctuations and flow impact but are just as likely to be attributed to other things like poor structural condition at the start of the flood, debris accumulation, sediment scour and abrasion or freeze-thaw action.

Chanson (2000) reviews 20 stepped spillway structures that have failed. Most of the structures are not masonry construction but the generalised findings would still be relevant to a masonry structure. Chanson attributes failure to either basic design errors such as underestimation of the design flood, poor foundation design or poor construction quality. Alternatively failure could be induced by issues specific to stepped surfaces such as lack of consideration of the impact forces in nappe flow or the pressure surging at the transition from nappe to skimming flow. In some cases poor maintenance was also seen to be an issue.

Walker (2008) discusses the discontinuance of Boltby reservoir following damage to the masonry spillway after a major flood event on 19 June 2005. The failure of the spillway channel led to some erosion damage to the embankment dam itself.

Mason and Hinks (2008) investigated the failure of the spillway on Ulley dam near Rotherham, UK which on the night of 25 June 2007, during a modest flood, suffered a catastrophic failure that lead to erosion of the dam. The spillway was only 1.83 m wide and failure may have been induced by a loss of masonry from the walls. The standard of maintenance at the site had been lower than might be desired with some loss of pointing and vegetation in the channel although the masonry lining was complete at the start of the incident. They conclude that regular inspections should ensure that the masonry is in good condition and that checks should be made on the hydraulic stability of the spillway using equations presented elsewhere in this report. Mason and Hinks (2009) outline a set of recommendations in the light of the incident.

Following the failures at Boltby and Ulley, the Environment Agency issued Bulletin 1 (Environment Agency, 2009), highlighting issues that should be checked by owners, inspecting engineers and supervising engineers when assessing the stability of similar masonry spillways.

4 Overview of Hydraulic Model Testing

4.1 Introduction

Following a review of the current understanding of the hydraulics of masonry stepped spillways, a laboratory investigation into the issues affecting block stability was undertaken. The findings of this investigation are summarised in this chapter.

4.2 Background Hydraulics

The review of current hydraulic understanding summarised in Chapter 3 found that the stability of a masonry block is a probabilistic event occurring when the turbulent force fluctuations coincide in the most unfavourable manner. These fluctuations are random and normally distributed, and for short periods of time they will be considerably in excess of the mean. As a result force equations need to be expressed in a probabilistic form and the risk of failure can not be evaluated as a finite value. For example, a block will lift if the force underneath is larger than the weight plus any down force. Failure is thus most likely to occur when the pressure below the invert, or at the back of a wall block, is experiencing a large positive fluctuation which coincides with a low negative fluctuation in the water above. When interpreting data in this chapter, it is thus important to consider the difference between maximum and minimum pressures and not simply average pressures.

4.3 Physical Model Testing

The hydraulic model testing followed two paths. Firstly a 1:20 scale model was constructed of the dam and spillway in question and used to assess the general hydraulic performance of the stepped spillway in terms of flow depth. Data collected from this was supplemented by data from the larger, second stage set of modeling. Comparisons were made using these data of various equations suggested to date for describing flow depth and these were also compared with results from similar model tests of other prototype spillways. The results from this are discussed in Chapter 5 of this report.

For the second series of tests, a larger 300 mm wide acceleration chute with a gradient of 1 in 3 and slope length of 7.32 m was constructed (see Figure 4.1 overleaf). Water was fed to a stilling tank on the roof of the laboratory building at flow rates up to 70 l/s a pump recirculation system and allowed to accelerate by gravity down the chute. The test area was about 1 m above ground level at the bottom of the chute. From there the water was stilled in a second tank and passed back into the laboratory sump. For some tests the chute was left smooth, in others it was roughened with strips of timber to increase turbulence and also to give lower net velocity.

With the smooth chute, the maximum discharge of 70 l/s achieved a peak velocity in the test facility of 6.2 m/s with a theoretical flow depth of 60 mm and associated Froude Number of 7.7; the measured bulked flow depth was 92 mm. For the rough chute the velocity dropped to 4.9 m/s with a theoretical flow depth of 84 mm and associated Froude Number of 5.4; the measured bulk flow depth was 92 mm.

Three test regimes were conducted. For Test 1 (stepped channel), the chute ended in a set of perspex steps on which pressures were measured both on the channel invert and on the side walls. For Test 2 (block stability), the chute continued as a smooth surface and a simulated masonry block was moved in and out of the wall to assess the effect that proud and recessed blocks would have on pressures in the mortar joints. For Test 3 (block failure) the pressure equipment was removed from the test facility and the void occupied by the test block was filled with a loose block.

The test rig was used to collect flow rate, velocity, depth and pressure data.

Figure 4.1 Acceleration chute

4.4 Test 1 – Stepped Channel

A set of perspex steps were mounted on the end of the acceleration chute as shown in Figure 4.2 and Figure 4.3. The steps were 170 mm wide and the bottom half of the chute was gradually narrowed from 300 mm to 170 mm with a long taper (see Figure 4.1).

Pressure tappings were set in the walls and invert of the channel and the locations of these are shown in Figure 4.4. The effective configuration of the pressure tappings when applied to a single step is shown in Figure 4.5. Initially tappings 1 – 5 were placed in the channel wall with tappings 6 – 8 in the invert of the channel on the centre line. However, it became apparent from the initial tests that more data was needed on the walls and, therefore, tappings 3, 6, 7 and 8 were sealed and additional tappings at locations 9 – 12 were provided.

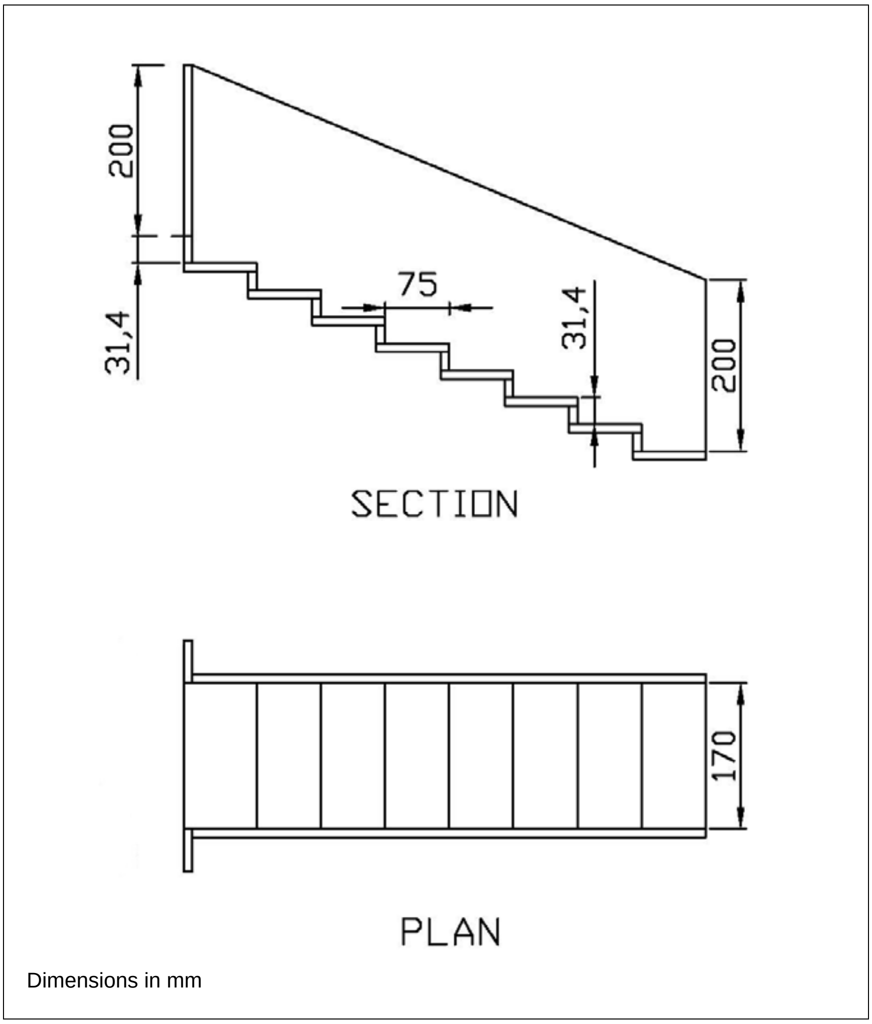

Figure 4.2 Step geometry

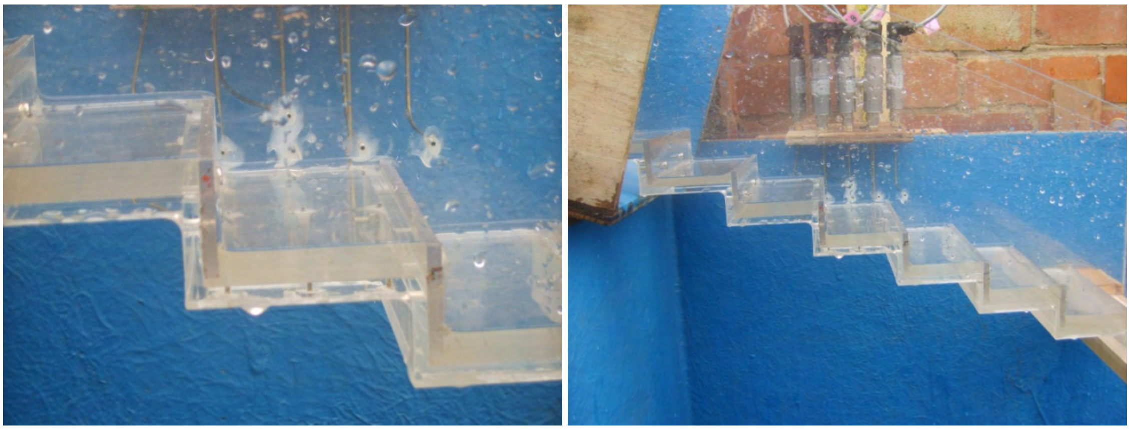

Figure 4.3 Pressure tappings on step

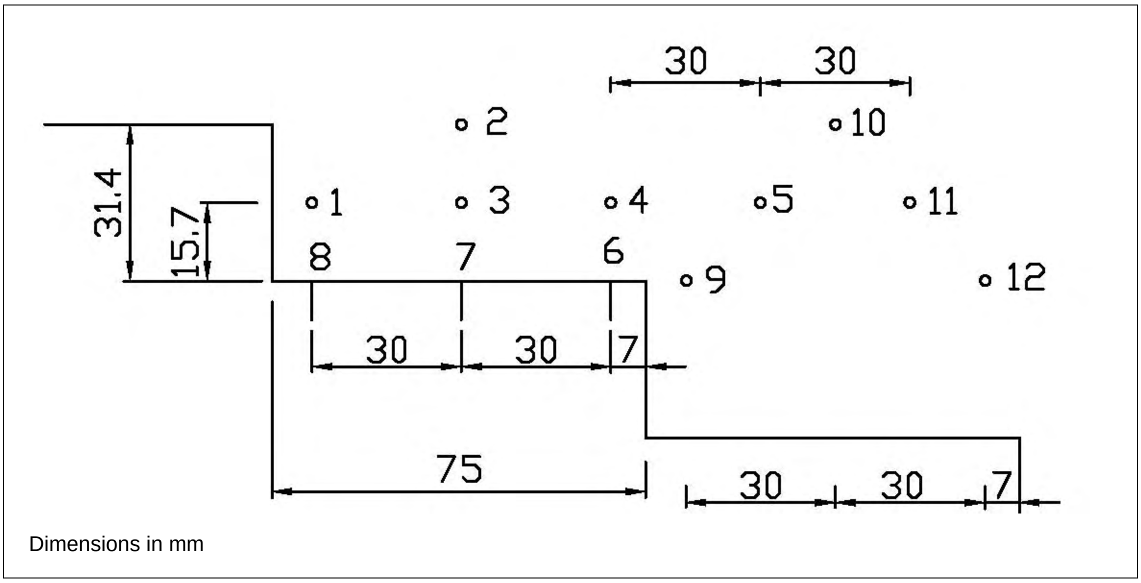

Figure 4.4 Actual location of pressure tappings

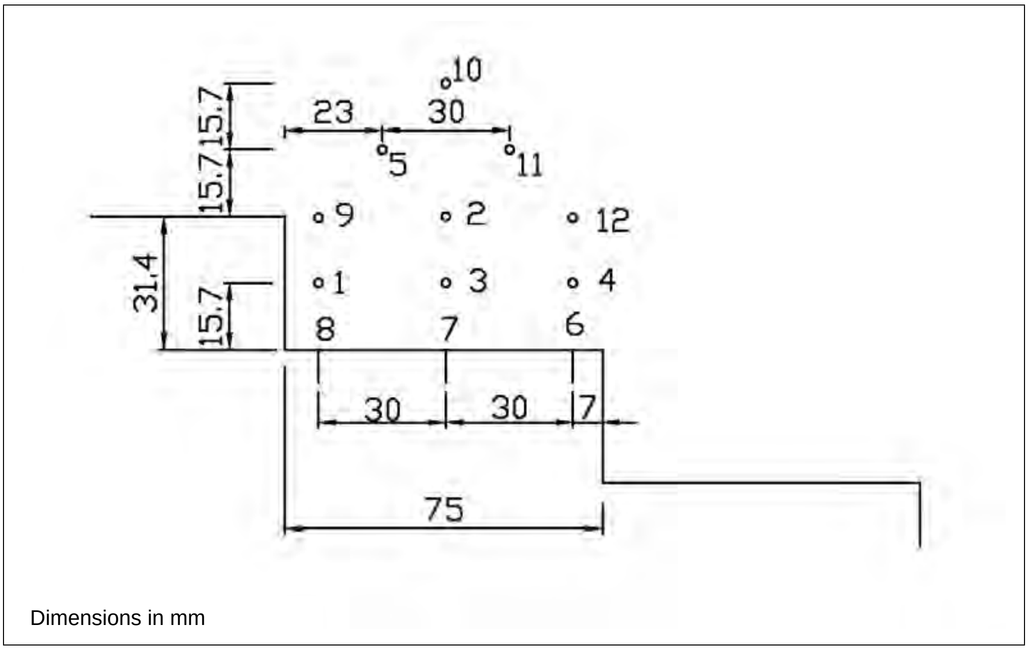

Figure 4.5 Effective location of pressure tappings on single step

CRM tested 9 configurations of the steps as detailed in Table 4-1.

Table 4-1 Step Configurations

Chute Roughness	Flow Projection	End Upstand
Smooth (0.5 mm)	Along Steps	✗
Rough (0.8 mm)	Along Steps	✗
Smooth	Along Steps	✓
Rough	Along Steps	✓
Smooth	Above Steps	✗
Rough	Above Steps	✗
Smooth	Above Steps	✓
Rough	Above Steps	✓
Smooth	Skipping Steps	✗

4.4.1 Flow Projected Along Steps – General Findings

Figure 4.6 shows 10 seconds of the pressure fluctuation data at 70 l/s for the case of the smooth chute with flow projected along the steps, which has fluctuation frequencies in the range of 0-10 Hz. The data for the other cases show a similar pattern. These charts are indicative and are not intended to be used for design purposes.

It should be noted in viewing these results that the potential back pressure on a masonry unit would be represented by the highest positive pressures shown feeding into the body of the masonry and exerting a back pressure in an area where the external load is represented by the lowest negative pressures.

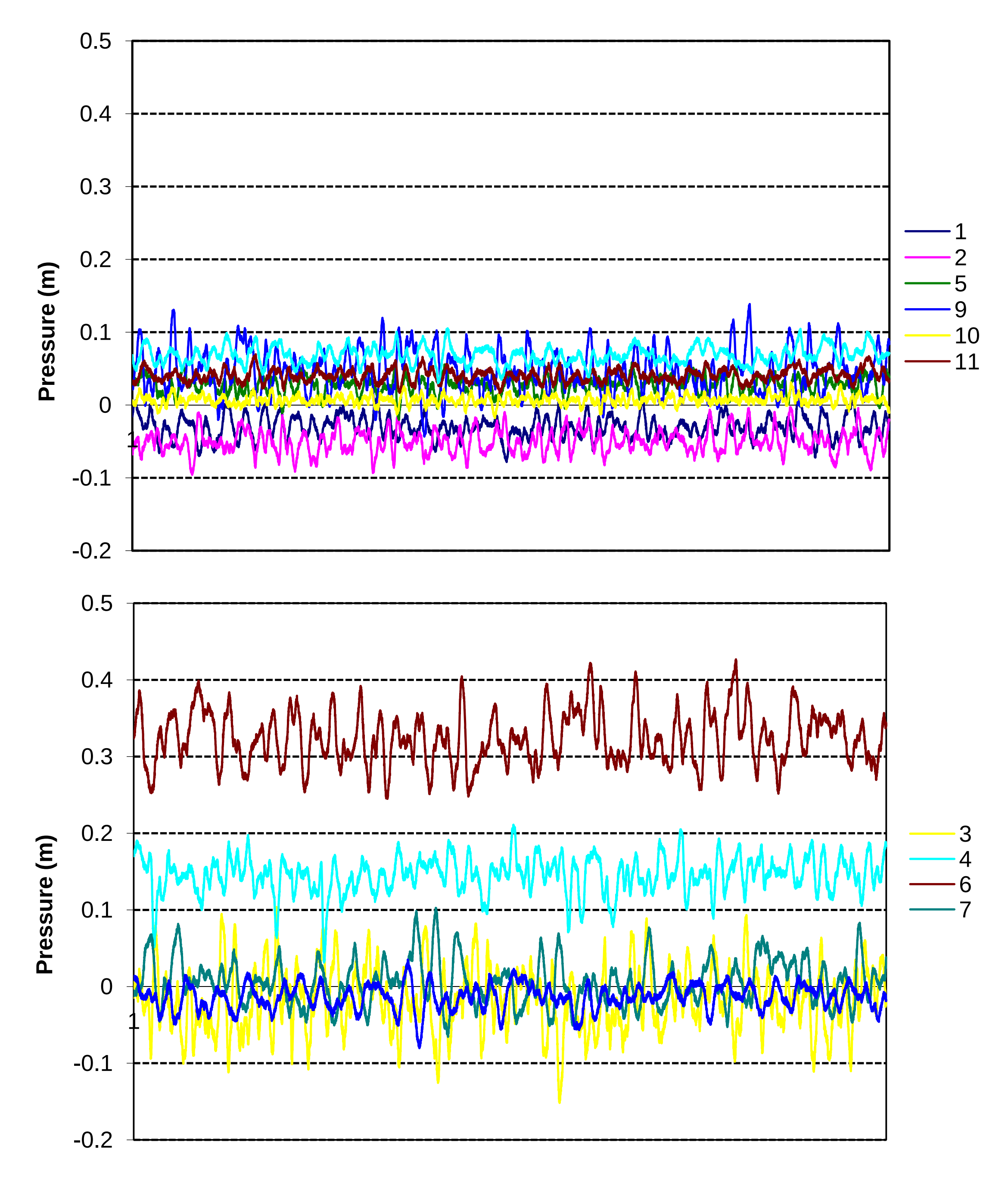

Figure 4.6 Pressure fluctuations at 70 l/s for a smooth chute

Figure 4.7 shows a contour plot of the mean pressure data recorded along the steps for a smooth approach chute.

A low pressure zone has formed around tappings 1, 2, 3 and 7 which are in the lee of the step, with a high pressure zone formed at the downstream end of the step around tappings 4 and 6. Tappings 5, 9, 10, 11 and 12 are in the general flow.

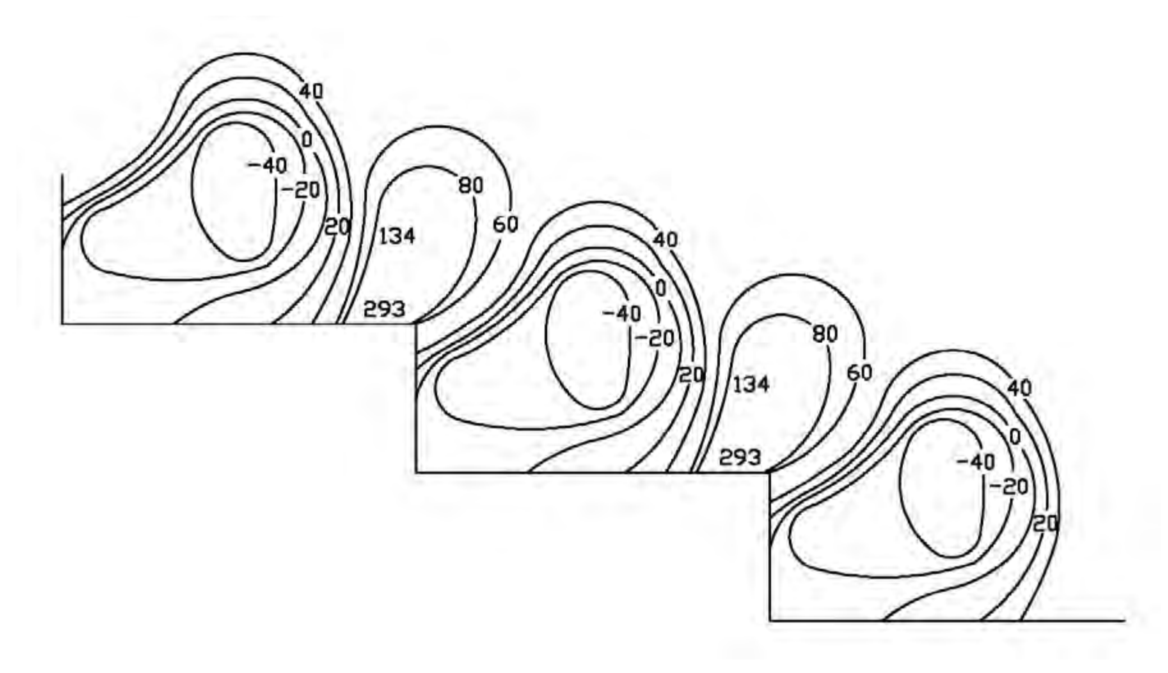

Figure 4.7 Mean pressure contour plot for flow over a set of steps

It can be seen from the contour plot that a very large pressure differential exists along the channel wall. If, due to poor quality pointing, the high pressure could be transferred to the back of the wall this could potentially track the short distance to the back of masonry blocks in the low pressure zone generating a large pressure differential across the block. From a design viewpoint, the mean pressures shown in Figure 4.7 do not represent a worst case because the pressures are fluctuating, as shown in Figure 4.6. For design purposes it would be more sensible to compare peaks of the pressure fluctuations.

Figure 4.8 shows the maximum pressure fluctuation on tapping 6 (located at the base of the wall at the downstream end of the step) relative to the minimum pressure fluctuation on tapping 3 (located in the centre of the wall) - the coincidence of these extremes represents the worst case scenario. The pressure differential on the model, where the steps are 31 mm high, is around 0.65 m at 70 l/s. If this represents a 1:10 scale model of a real 0.31 m high step, then the pressure differential on the real step would be 6.5 m of water pressure. This demonstrates the need to consider the peak negative and positive pressures and forces exerted by the flow when assessing the stability of a masonry block rather than simply considering the mean pressures and forces.

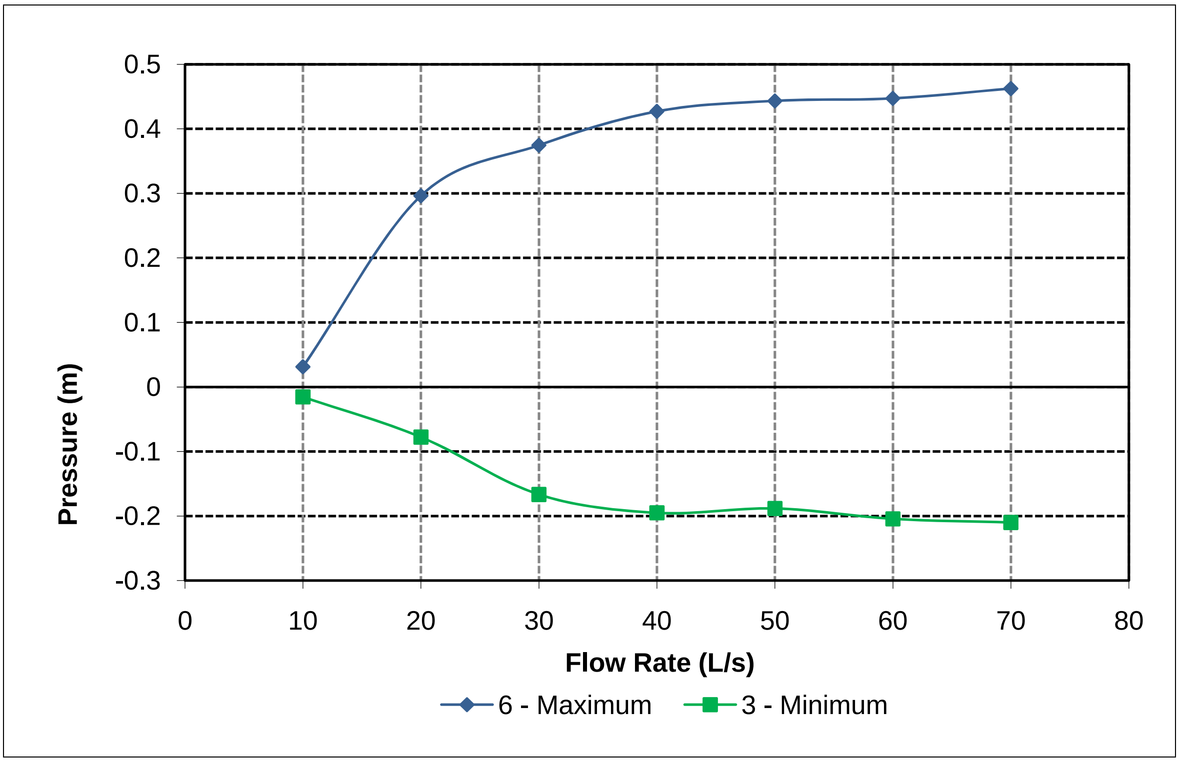

Figure 4.8 Possible pressure range across a block with poor pointing

Figure 4.9 shows the development of minimum pressure with increasing flow rate and Figure 4.10 shows the development of maximum pressure. As a generalised statement, pressures that start positive become higher with increasing flow, whilst those that start negative become lower. It therefore follows that the higher the flow rate, the more extreme will be the pressure differentials and hence the likelihood of a failure is increased.

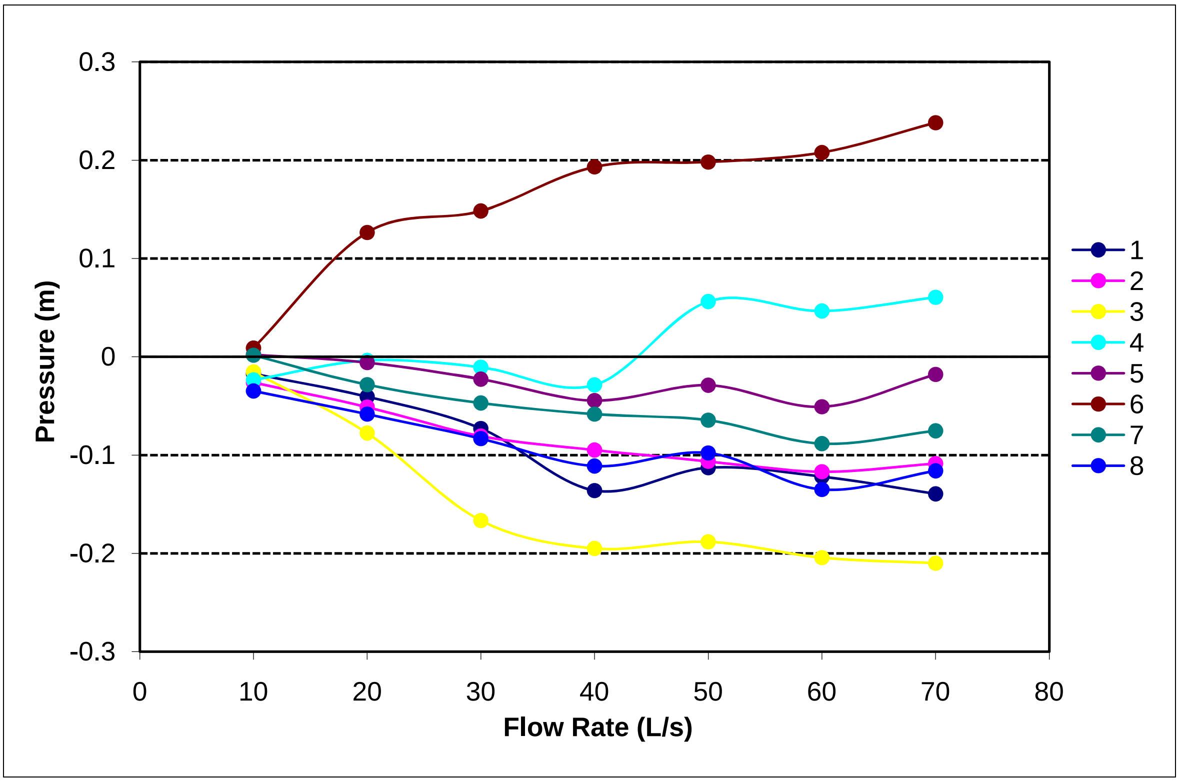

Figure 4.9 Development of minimum pressure with flow rate

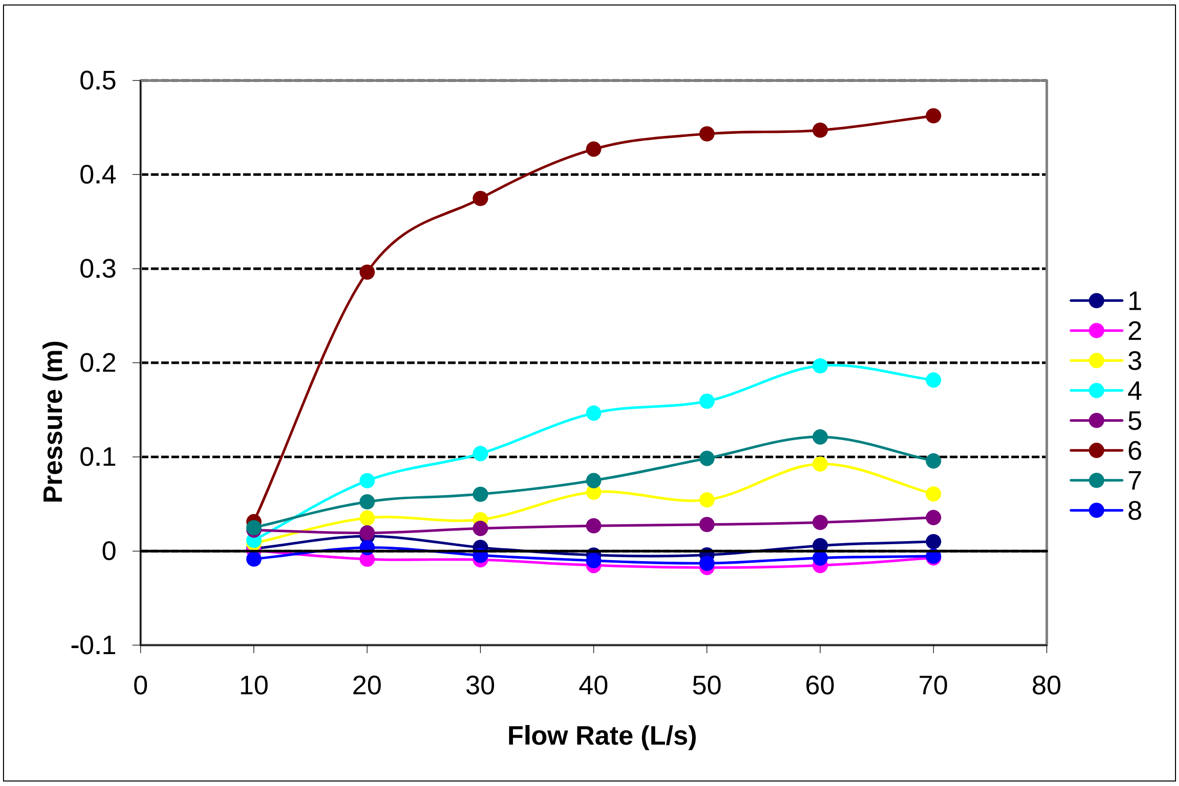

Figure 4.10 Development of maximum pressure with flow rate

4.4.2 Flow Projected Along Steps – Smooth vs. Rough Approach

Flow was introduced onto the steps via a transition curve from the acceleration ramp. The initial transition was designed to launch the flow parallel to the tips of the steps (see Figure 4.11).

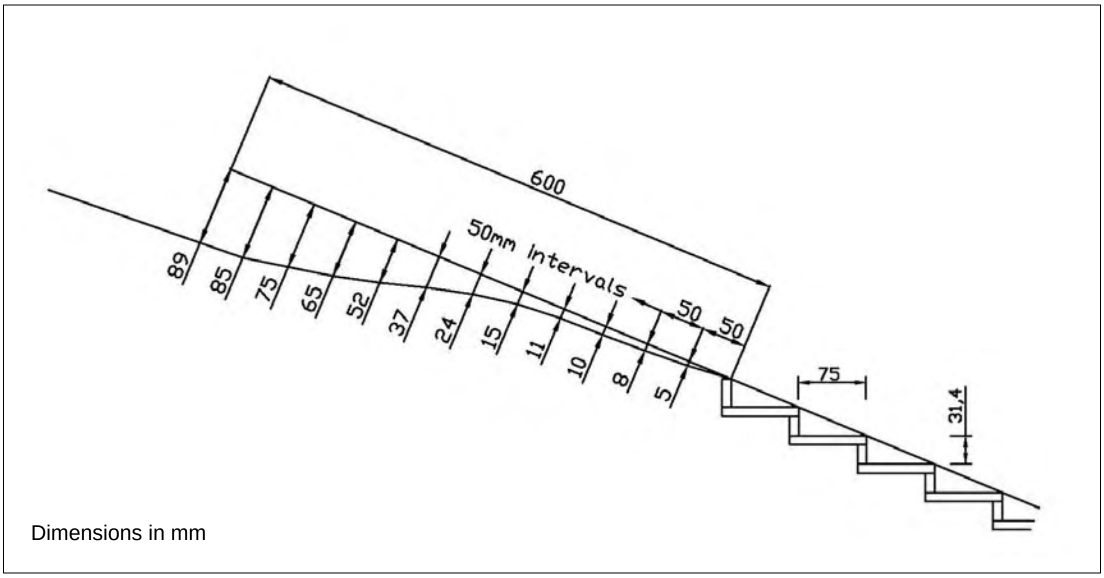

Figure 4.11 Approach transition to the steps

Figure 4.12 shows a contour plot of mean pressure data recorded along the steps with both a smooth approach and a rough approach. A large pressure differential across a small locality of the wall can be seen clearly, with the centre of the low pressure zone deeper into the lee of the step with the rough approach. The approach flow velocity was reduced by the roughness on the chute with a corresponding depth increase. This reduced the magnitude of the mean pressures but did not affect the fluctuations or the pressure distribution pattern.

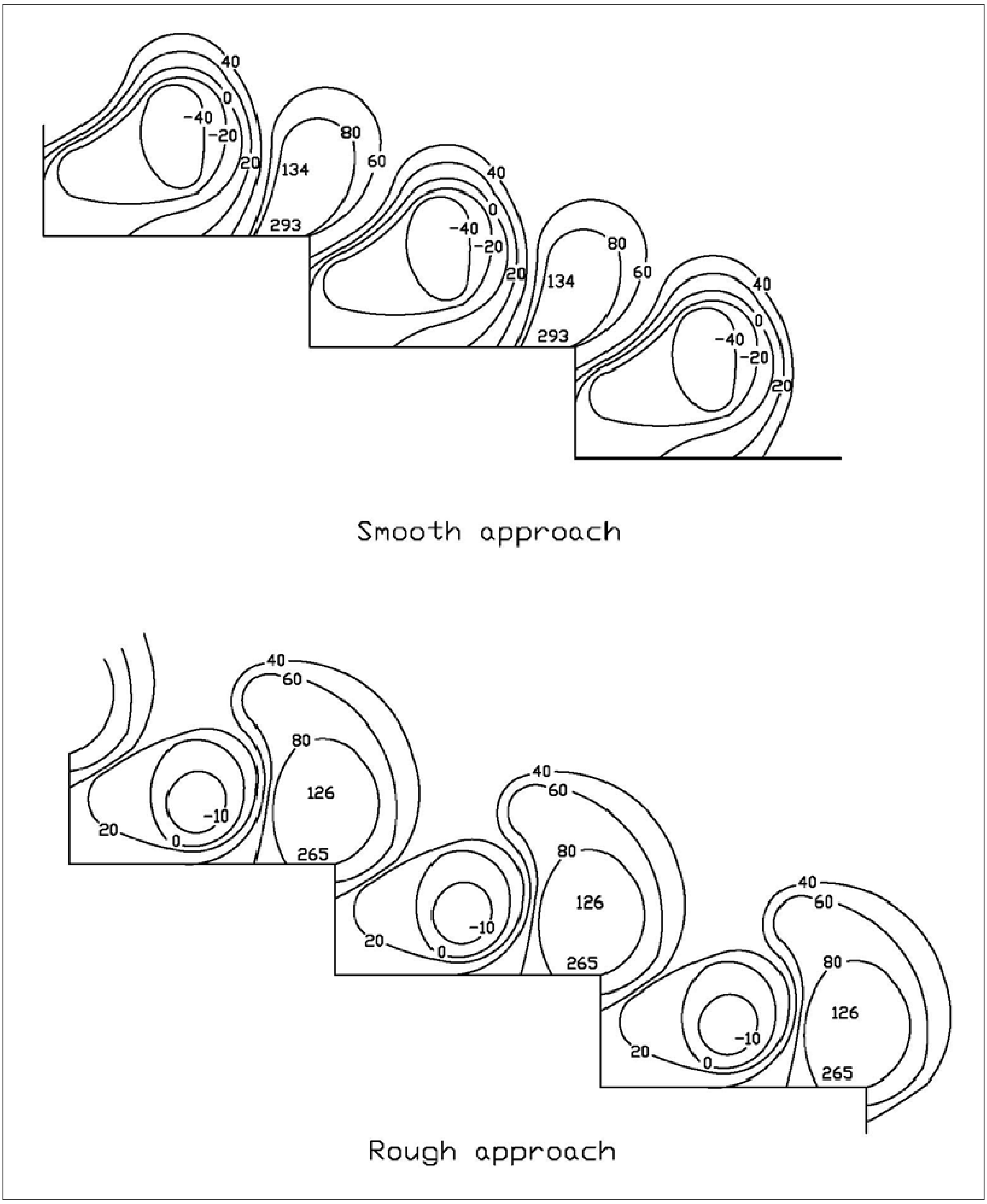

Figure 4.12 Mean pressure contour plot for flow projected along steps – smooth and rough approach

4.4.3 Flow Projected Along Steps – End Upstand

On some existing spillways, an end upstand has been provided on the downstream edge of every step to encourage a pool of water to form and assist with energy dissipation under nappe flow. Therefore, a 10mm high end upstand was added to the model steps to simulate this situation (see Figure 4.13).

Figure 4.13 Steps with end upstands

Modelling showed that the presence of the end upstand had the effect of increasing the pressure fluctuations, especially on tappings 3, 4, 6 and 12 and increasing the pressure on tapping 4 which is very close to the end of the upstand.

Figure 4.14 and Figure 4.15 show the pressure differentials along the steps for both a smooth and rough approach with the end upstand in place. It can be seen that for the rough approach, the pressure differential is always larger with the end upstand in place regardless of flow rate. However, for the smooth approach, the differential is smaller at low flow rates with the end upstand in place, but becomes much larger as the flow rate increases. At 70 l/s on the smooth approach, the potential pressure differential on the model is 0.79 m compared to 0.65 m for the normal step with no end upstand in place. The equivalent figures for the rough approach are 0.63 m with the end upstand in place and 0.52 m for the normal step. The implication of this is that, whilst the presence of an end upstand may aid energy dissipation under nappe flow conditions, it will most likely increase the risk of masonry failure at high flows.

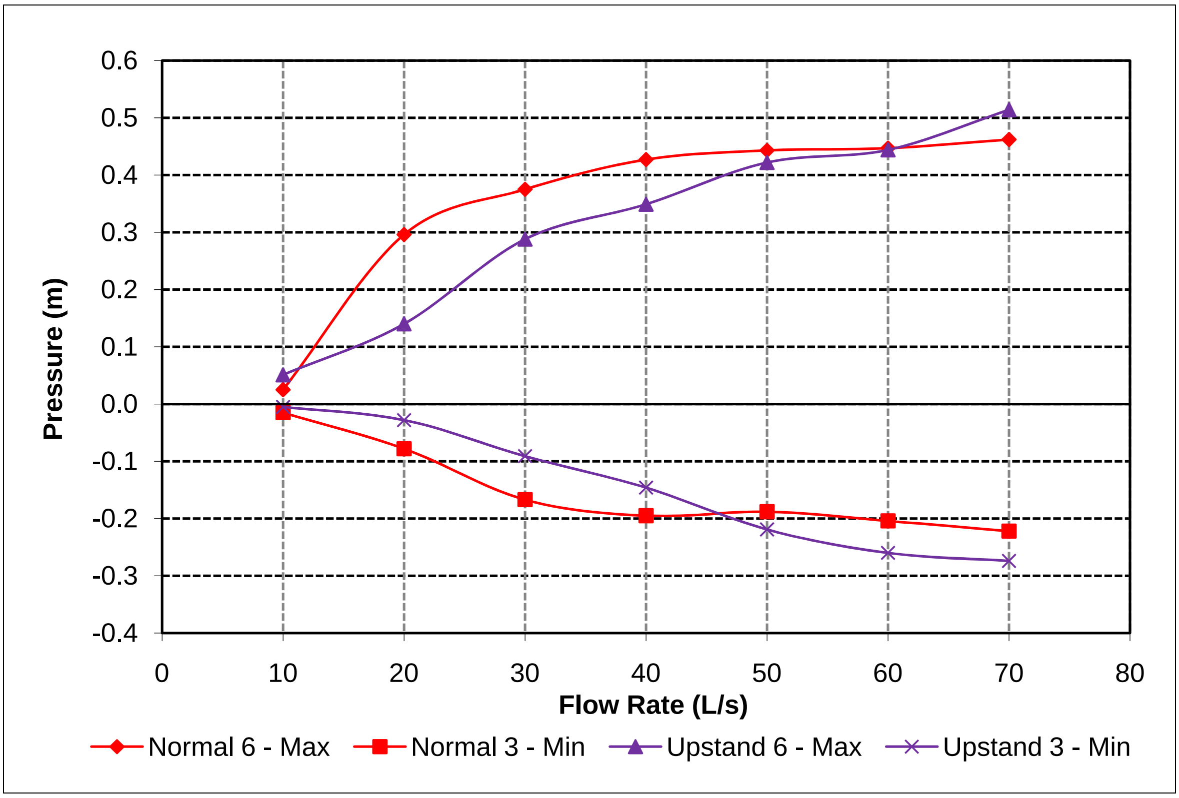

Figure 4.14 Pressure differential – smooth approach and end upstand

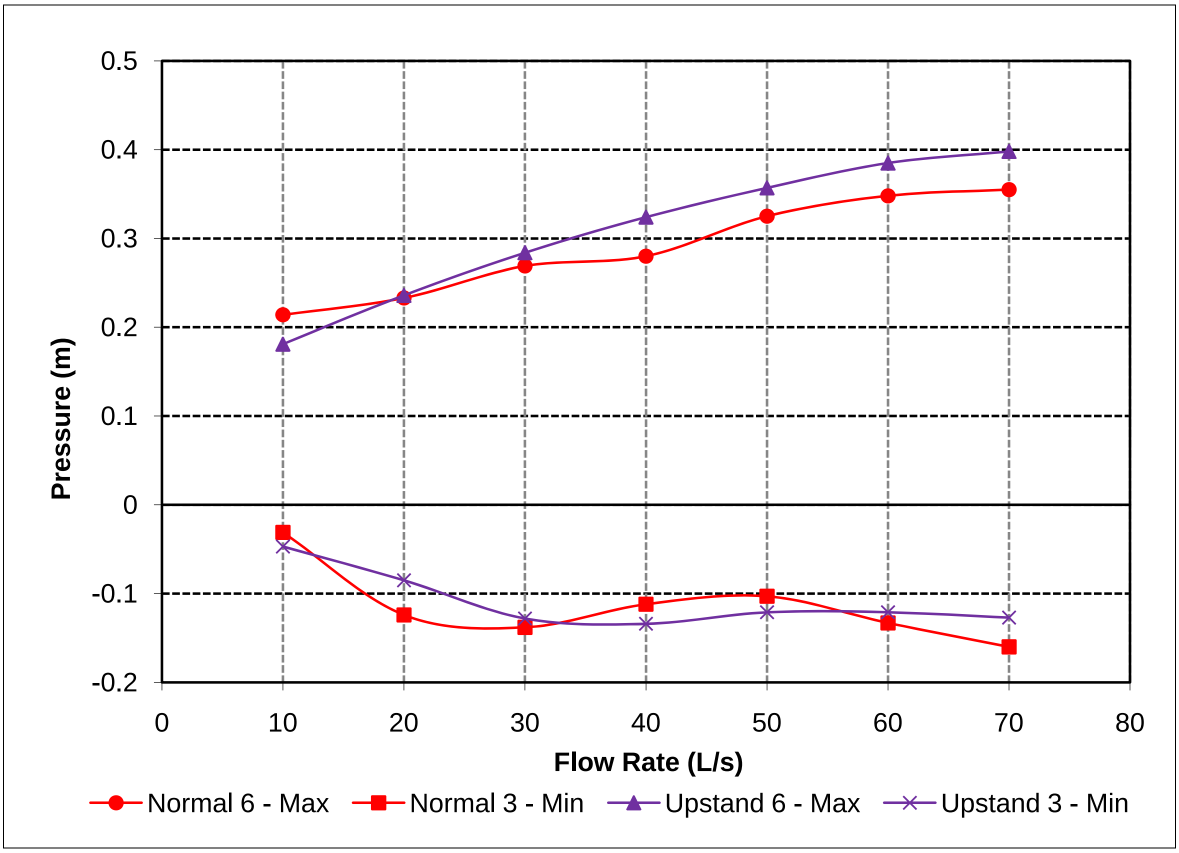

Figure 4.15 Pressure differential – rough approach and end upstand

Figure 4.16 shows a plot of mean pressure contours for the model data for the end upstand. When compared to Figure 4.12, it is clear that the end upstand does not have an effect on the pressure distribution, although the separation zone in the lee of the step is larger.

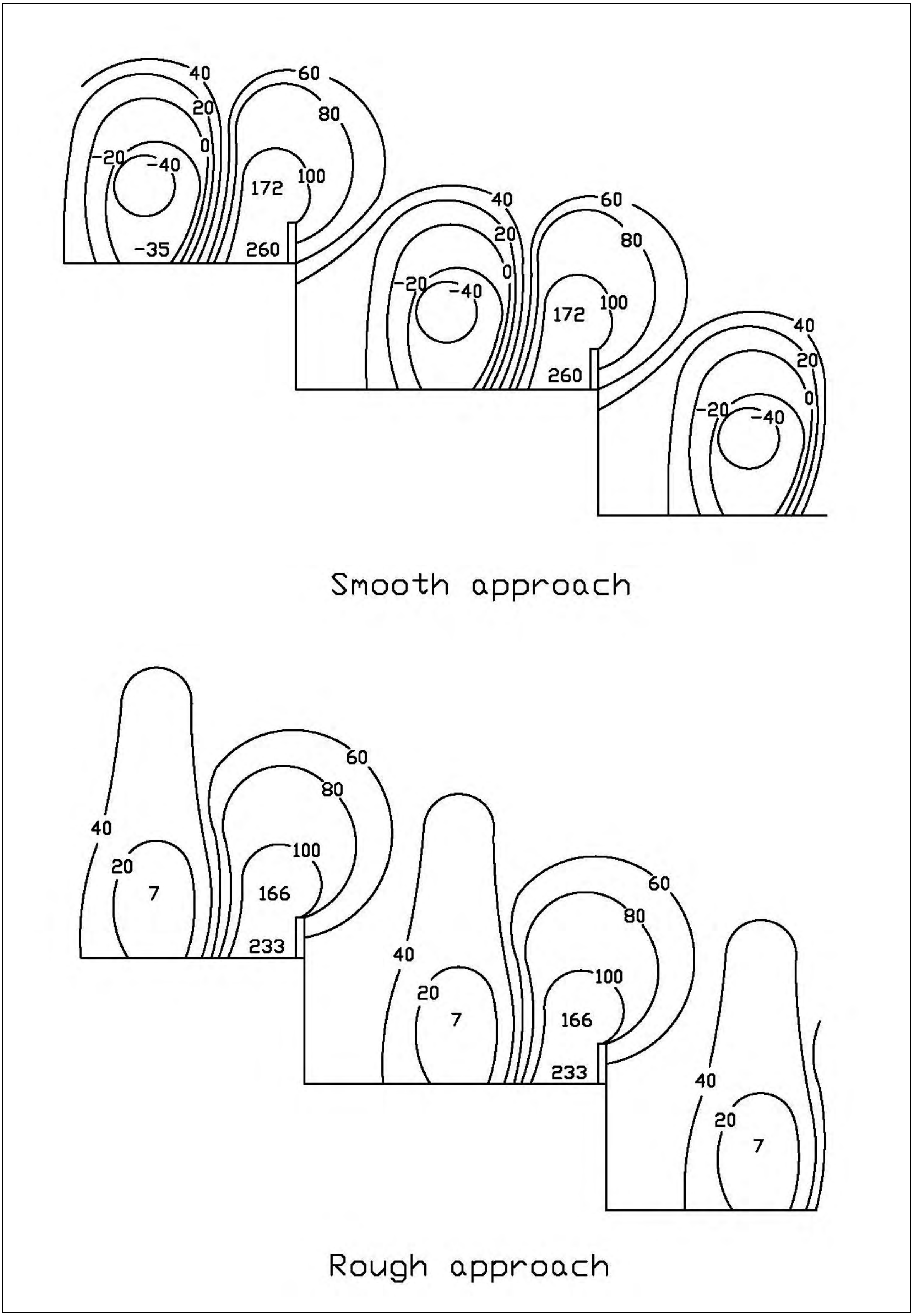

Figure 4.16 Pressure contour plots - end upstands

4.4.4 Flow Projected Above Steps

Many spillways in the UK have sections of flat or sloping chute interspersed with cascades of steps. This geometry will not normally present the flow parallel to the tips of the steps at the start of the cascade. Instead, flow may be projected above the steps and in some instances, if the change in angle is too great, the water may be projected free of the steps.

In order to investigate the case where the flow is projected above the step, whilst still retaining attachment to the surface of the channel, the geometry of the transition from the acceleration chute onto the steps was modified as shown in Figure 4.17.

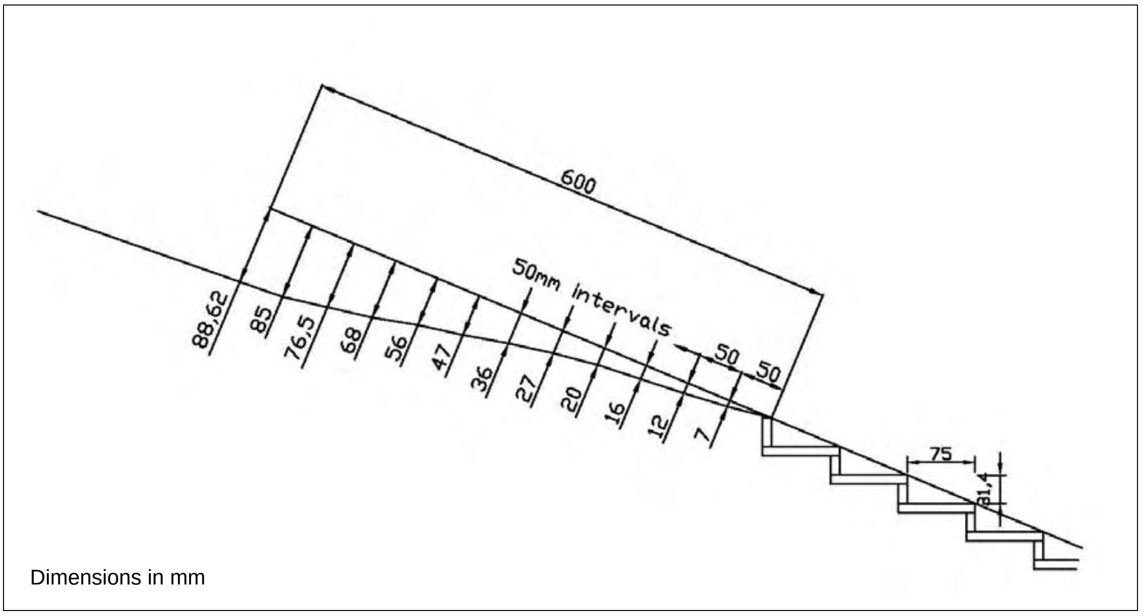

Figure 4.17 Modified transition geometry from acceleration chute onto steps

Modification of the transition was found to influence the flow pattern on the steps. In classical skimming flow, water separates from the edge of a step and then either re-attaches part way along the next downstream step or re-attaches at the very downstream tip of the step (see Figure 4.18). A roller incorporating a low pressure zone forms in the lee of the step and pressure in the roller is at reduced pressure.

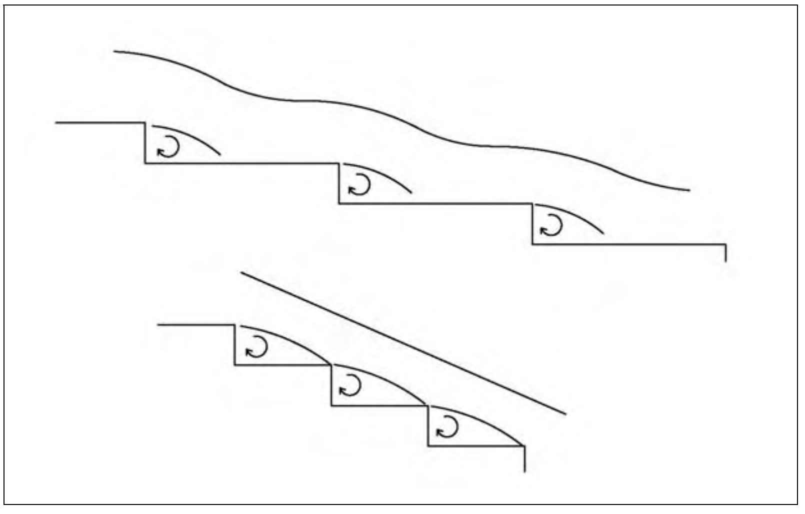

Figure 4.18 Classical skimming flow

With the modified transition in place, flow was projected high above the steps and skimmed across a line above the step tips with no re-attachment of the main flow to any of the steps. All of the steps were in zones of reduced pressure and the roller was observed to be above the level of the step edges (see Figure 4.19).

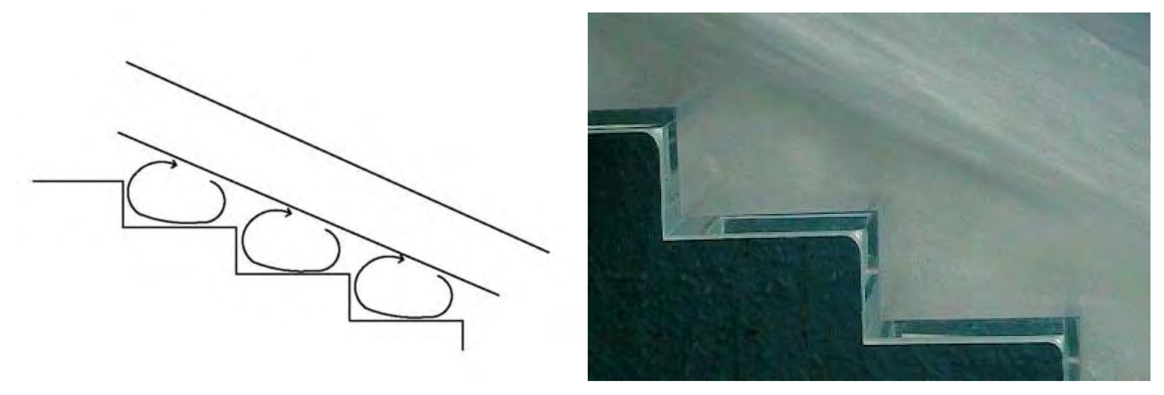

Figure 4.19 Roller above steps

Model testing of this transition arrangement with a smooth and rough chute with and without end upstands showed that under all options the modifications make minimal difference to the pressure differentials - only tappings 1 and 3 in the low pressure zone in the roller showed slightly more fluctuation.

4.4.5 Flow Skipping the Steps

The chute was adjusted so that the step approach was flat with an inclination of 1.8^{o} in order to determine under what conditions flow would skip the steps.

Under this arrangement, at velocities in excess of 2.6 m/s for an unvented nappe or 1.3 m/s for a vented nappe, the water completely separated from the steps, leaving a large air pocket between the steps and the water and reattached further down the cascade rather than skimming over each step. These findings indicate that if the steps are located downstream of a long channel section that is subjected to high velocities, the whole step section may be bypassed (see Figure 4.20). In sections that are subjected to lower velocities, reattachment will occur part way down the flight. Table 4.2 shows the reattachment points for various velocities at the top of the flight of steps with a flat 1.8^{o} approach; the flow depth was kept constant at 40 mm using an undershot gate.

Figure 4.20 Flow projected from a horizontal step approach

Table 4.2 Reattachment points for flow skipping the steps

Velocity at Head of Steps (m/s)	Distance to Re-attachment Point (mm) Horizontal	Vertical
Vented Nappe		
1.3	40	31
1.7	110	63
1.8	190	94
2.1	260	126
Unvented Nappe		
2.6	260	126
2.7	340	157
3.0	410	188
3.1	490	220
3.4	600	251

Figure 4.21 shows a plot of Froude Number against dimensionless reattachment distance (horizontal throw / step height) which would allow the impact point of separated flow to be determined. This chart is valid for a change in angle at the top of the flight of steps of 20.9°, Further research would be needed with varying changes of angle to obtain a family of design curves suitable for universal usage.

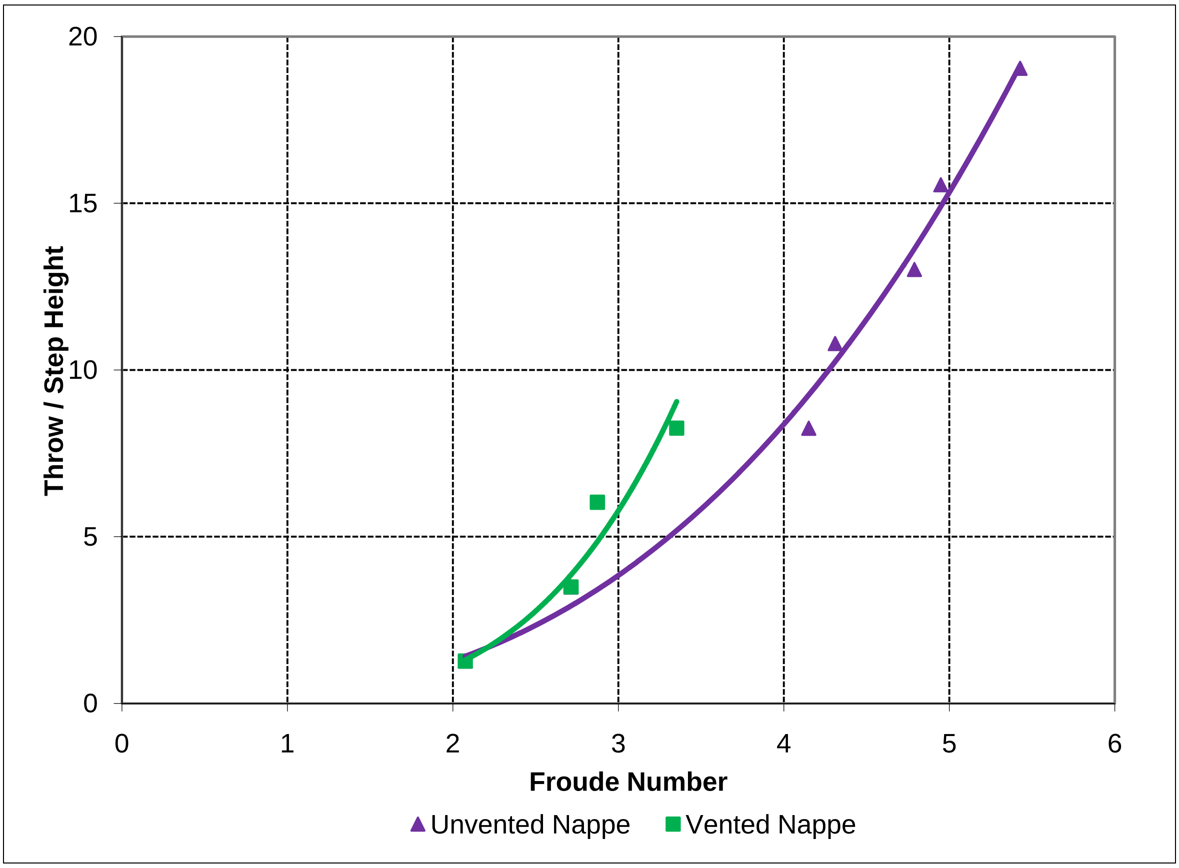

Figure 4.21 Horizontal location of the reattachment point for a change in angle of 20.9°

4.5 Test 2 – Block Stability

In the second phase of testing, the acceleration chute was modified so that it had a width of 60 mm along its entire length in order to increase the flow depth in the channel without requiring an increase in pump rates. The channel was projected horizontally at the bottom of the chute and a test 'masonry' panel was installed in the wall. This comprised a timber sheet with grooves cut into it to simulate mortar joints between the blocks and a 100 x 50 mm movable block in the centre of the panel (see Figure 4.22 and Figure 4.23).

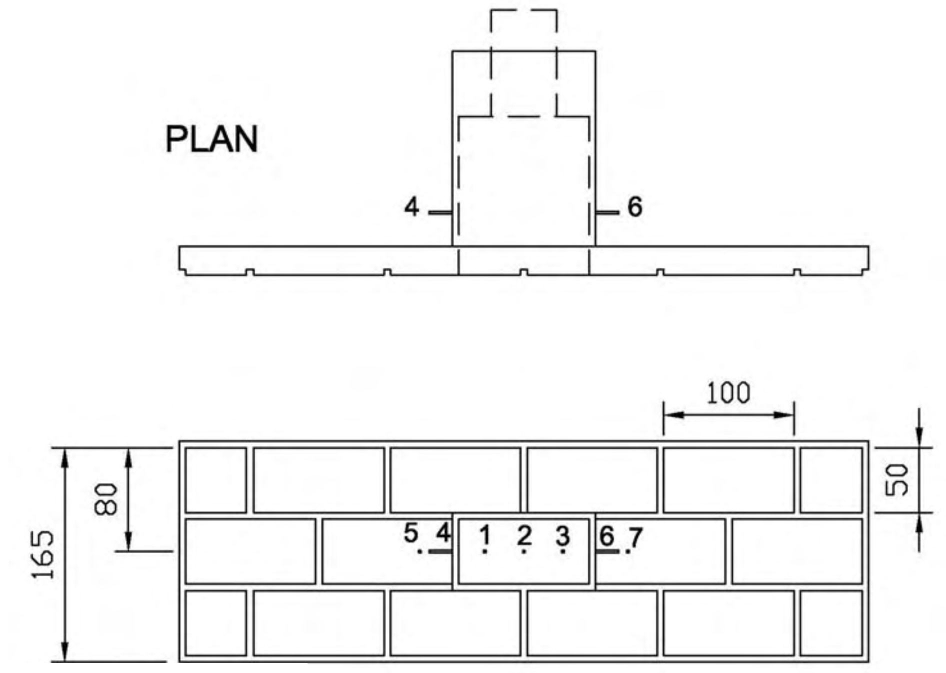

Figure 4.22 Masonry wall test arrangement

Seven pressure tappings were provided around the movable block as shown in Figure 4.22.

The block was tested flush with the other blocks and then set either proud or recessed from the line of the panel. In addition, the spaces around the block were left open to simulate missing pointing or sealed with a silicon sealant to simulate good pointing (see Table 4-3). Most of this testing was carried out with a fixed flow of 45 l/s, the maximum capacity of the narrower chute.

Table 4-3 Wall Configurations

Block Position	Joints	Section
Flush Proud Inset	Upstream and downstream joints open, top joints and void behind block sealed.	4.5.1 4.5.2 4.5.3
Flush Proud Inset	Upstream and downstream joints and void behind block open, top joints sealed.	4.5.4 4.5.5 4.5.6
Flush Proud Inset	All joints open	4.5.7 4.5.8 4.5.9

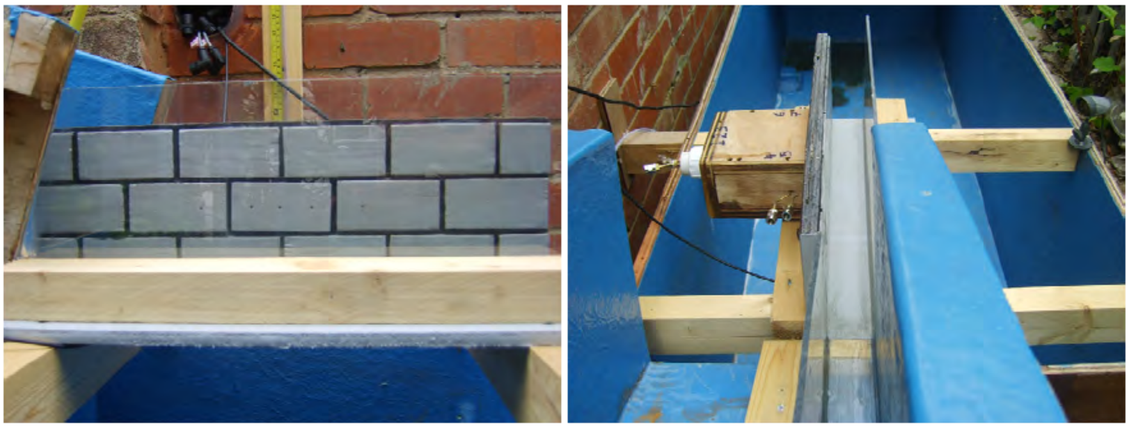

Figure 4.23 Photographs of the masonry wall test rig

4.5.1 Upstream and Downstream Joints Open – Block Flush

Under these test conditions, flow was uniform and parallel to the bed (see Figure 4.24) and there was very little pressure fluctuation against the wall, with only tappings 4 and 6 recording an offset from zero (the slight negative pressure is a result of the velocity of flow rushing across the joint setting up a suction effect).

Figure 4.24 Upstream and downstream joints open - block flush

In these tests, the flow was projected horizontally along the wall so that it ran parallel to the simulated mortar joints. It is recognised that in most spillways the mortar will be bedded horizontally and since the invert will normally be sloping, the water passes the vertical joints and horizontal bed at an angle. No tests were conducted to assess the affect of water not being parallel to the masonry jointing. However, it is not expected that this change in flow pattern would seriously influence the findings discussed below.

4.5.2 Upstream and Downstream Joints Open – Block Proud

The block was positioned proud of the panel by +5, +10 and +15 mm (see Figure 4.25).

The protrusion of the block into the channel even by only 5 mm was found to create a stagnation point that transferred the high pressure into the leading joint (tapping 4) and created extreme fluctuations in pressure. Tapping 5 upstream of the block was also found to be in a high pressure zone. On impact on the protruding block, the flow separated from the surface of the block, placing tappings 1, 2 and 3 in a low pressure zone. Further separation occurred at the downstream joint, putting tappings 6 and 7 in a low pressure zone as well. Separation of the flow on impacting on the block caused extreme splashing to occur on all tests where the block was standing proud of the wall by more than 6 mm (see Figure 4.25).

It should be noted that the difference in pressure rise between +10 mm and +15 mm is not that dramatic, most of the ‘damage’ is done by the block standing just a few millimeters proud of the surface.

Figure 4.25 Upstream and downstream joints open - block proud by 15 mm

4.5.3 Upstream and Downstream Joints Open – Block Inset

The block was positioned inset from the panel by -5, -10 and -15 mm (see Figure 4.26).

The effects of insetting the block into the wall are opposite to those where the block stands proud of the wall.

The indent in the wall acts like a step edge so that a low pressure zone forms in the lee of the step, drawing down the pressure at tapping 1 and tapping 4 in the upstream joint. Flow re-attaches to the surface of the indented block around tapping 2 generating an impact pressure slightly greater than that given at tapping 5 which is in the regular flow. The flow impacts onto the end of the downstream block generating a stagnation point with high pressure at tapping 3 and extreme pressure fluctuations at tapping 6 in the downstream joint. Tapping 7 sits in a small separation zone as the water rises over the edge of the downstream block.

Figure 4.26 Upstream and downstream joints open - block inset by 15 mm

4.5.4 Void Behind Back of Block Open – Block Flush

No significant differences were noted to the conditions witnessed for the block flush with the back edge sealed.

4.5.5 Void Behind Back of Block Open – Block Proud

The block was positioned proud of the panel by +15 mm.

The main differences noted between this case and the case where the back of the block was sealed were that whilst tapping 4 in the upstream joint still experienced large pressure fluctuations, the magnitude of the mean pressure had reduced because pressure relief had been possible around the back of the block. Conversely, the mean level of the negative pressure in downstream joint, tapping 6, had increased.

4.5.6 Void Behind Back of Block Open – Block Inset

The block was positioned inset from the panel by -15 mm.

Under this scenario, it was found that there were no mean pressure differences between tappings 4 and 6 in the upstream and downstream joints, although tapping 6 still had larger fluctuations as a result of the stagnation effect on the downstream face of the indent. In addition, the negative pressure downstream of the block at tapping 7 had reduced in magnitude.

4.5.7 All Joints Open – Block Flush

Under this scenario, it appeared that water entered the void around the block along the longitudinal joints at the top and bottom of the block. This flowing water stagnated against the downstream block causing high pressure fluctuations in the downstream joint, tapping 6, and raising the mean pressure in the downstream joint. Water exiting from the downstream joint caused a local flow disturbance that set up large pressure fluctuations in the flow downstream at tapping 7.

4.5.8 All Joints Open – Block Proud

The block was positioned proud of the panel by +15 mm.

When compared to the case where only the longitudinal joints were sealed, it was found that there were lower pressures in the upstream and downstream joints and considerably less pressure fluctuation in the upstream joint at tapping 4. It appeared as though some of the pressure around the block was relieved by the water that was able to flow around the block.

4.5.9 All Joints Open – Block Inset

The block was positioned inset from the panel by -15 mm.

Slightly higher mean pressures were recorded at tappings 1, 4, 5 and 7 than under the scenario where only the longitudinal joints were sealed, but otherwise the conditions were unchanged.

4.5.10 Summary of Pressure Comparisons

Figure 4.27 and Figure 4.28 show the development, and subsequent relief, of mean pressures and of pressure fluctuations for all of the modeled scenarios:

- Upstream and downstream joints open - block flush;
- Upstream and downstream joints open - block 15 mm proud;
- Void behind back of block open – block 15 mm proud;
- All joints open – block 15 mm proud;
- Upstream and downstream joints open - block 15 mm inset;
- Void behind back of block open – block 15 mm inset;
- All joints open – block 15 mm inset.

Both Figure 4.27 and Figure 4.28 demonstrate that pressure conditions surrounding the block are worst when top joints and void behind the block are sealed. The block being set proud of the wall provides far worse pressure conditions than if it is inset from the wall.

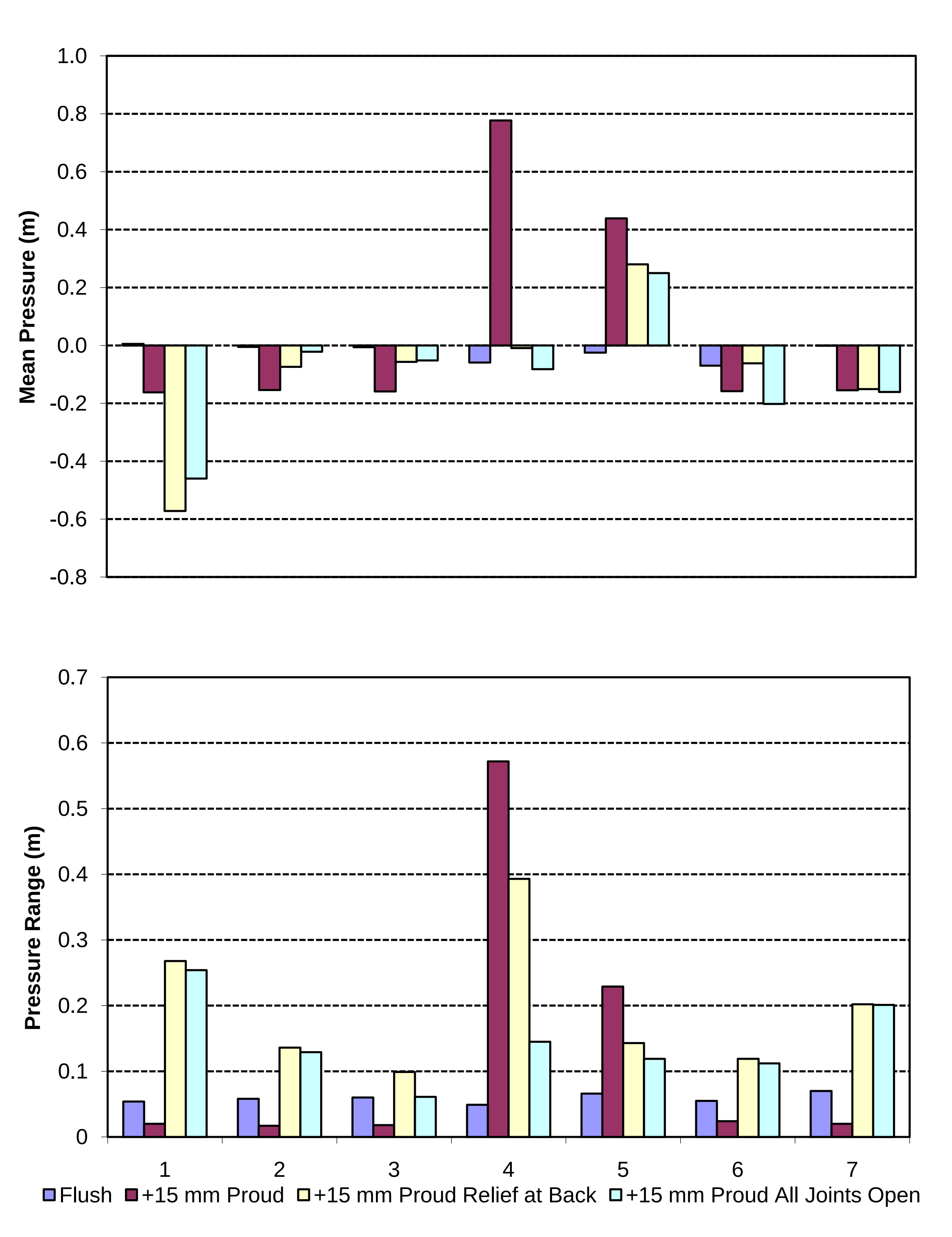

Figure 4.27 Pressure development for the block standing proud of the wall by +15 mm

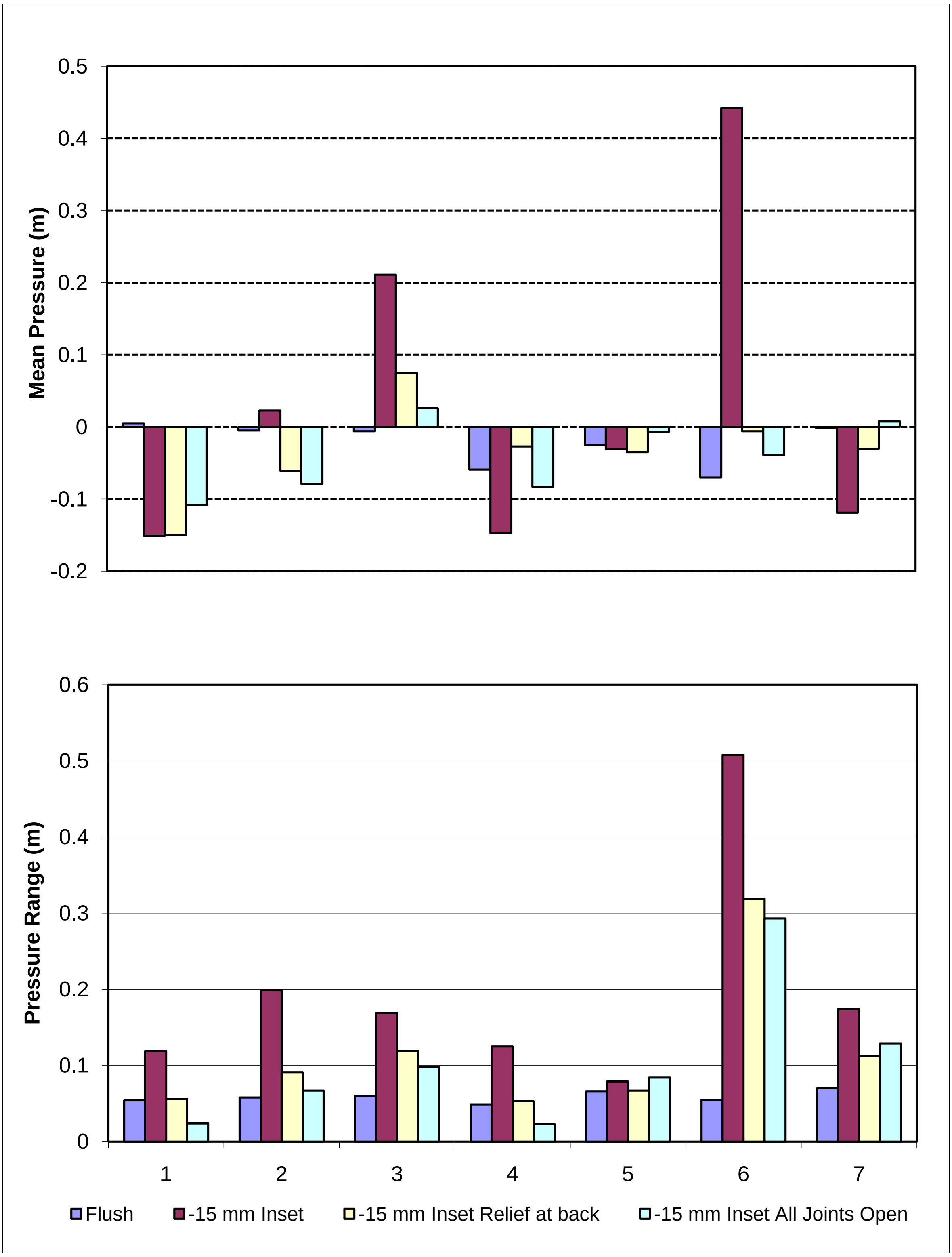

Figure 4.28 Pressure development for the block inset from the wall by -15 mm

4.6 Test 3 – Block Failure

The pressure equipment was removed from the test facility and the void occupied by the test block was filled with a loose block. Three different depths of block (25 mm, 50 mm and 100 mm) and two designs (parallel-sided and tapered so that the back face was smaller than the front face) were used. The loose blocks were fitted 'finger tight' into the void on spacers, so that they were able to move but would not be dislodged by floating or simple flow impact. Two tests were carried out: with all joints open and with just the upstream joint open.

It was found that the thin block (25mm) and the tapered block always failed by rotation about the downstream edge, the deeper blocks (50 mm and 100 mm) failed by sliding forward out of the recess, occasionally becoming wedged in the recess. A summary of the results is given in Table 4-4 below.

Table 4-4 Results of Block Failure Tests

Block Arrangement	25 mm Depth	50 mm Depth	100 mm Depth
Parallel-sided block			
Flush	No failure	No failure	No failure
2 mm projection, only upstream joint open	Failure by rotation	Moved but became stuck	Moved but became stuck
2 mm projection, all joints open	Failure by rotation	Failure by sliding forward	Failure by sliding forward
Tapered block			
Flush	No failure	No failure	No failure
2 mm projection, only upstream joint open	Failure by rotation	Failure by rotation	Failure by rotation
2 mm projection, all joints open	Failure by rotation	Failure by rotation	Failure by rotation

The results confirm that even a small protrusion into the flow can lead to a block failure by mobilizing the stagnation pressure generated against the edge of the block. They also confirm the failure mechanism.

5 Recommended Guidelines for Stepped Masonry Spillways

5.1 Hydraulic Design

5.1.1 Introduction

This section of the report provides draws on the work discussed in Chapters 3 and 4 to present recommendations for checking the hydraulic capacity and design of typical stepped spillways.

5.1.2 Design Equations

As part of their hydraulic model testing research, CRM compared the data collected on the test facility to that calculated by established design equations in order to test the validity of the equations. The process was assisted by reference to data collected on previous small-scale physical model studies conducted by CRM. This data is contained in Appendix B.

The findings of these comparisons are summarized below.

Onset of Skimming Flow

The normally accepted equation for the onset of skimming flow is:

$$d_c = 1.057\,h - 0.465\,\frac{h^2}{L}$$

Where:

d_c = critical depth

h = step height

L = step length

For the test facility this gives d_c = 0.073, which corresponds to a flow rate of 10.4 l/s in the 0.17 m wide chute. This flow rate is compatible with observations of flow in the test chute where skimming flow was observed to be forming at 10 l/s but was not fully developed.

It can be concluded therefore that the above equation is acceptable for estimating the onset of skimming flow and is valid for use in design reviews.

Length from Crest to Inception Point

The inception point is the position where the turbulent boundary layer reaches the surface and air is entrained into the water generating a two phase air/water mixture. A number of equations exist to calculate the length from the crest to the inception point, the following equation is the most commonly used:

$$L_i = 9.719\ k_s\ (\sin\alpha)^{0.0796}\ F_*^{0.713}$$

Where:

k_s = surface roughness of the spillway

α = channel slope

$$F_* = \frac{q_w}{\sqrt{g \sin\alpha\ k_s^3}}$$

q_w = flow of water per unit width of the channel

g = acceleration due to gravity

For a flow of 70 l/s in the test facility, this gives L_i = 4.0 m. This is compatible with observations in the test chute (7.32 m long) that air entrainment started about halfway along its length (3.7 m).

It can be concluded therefore that the above equation is acceptable for estimating the length from the crest to the inception point and is valid for use in design reviews

Roughness

The roughness of a stepped spillway is often quoted as the step height measured normal to the slope. In check calculations, this worked well for shallow slopes but over-estimated the depths on steep cascades of steps. A "rule of thumb" value of 100 mm gave a better fit to measured depths when used with Chanson's equation (Section 3.9.2) for predicting depth.

Depth

Depth is the most difficult parameter to measure and predict on a spillway.

Many designers use the d_{90} depth to define the wall height. When applied to Chanson's equation, this gives:

$$d_{90} = d_c \sqrt[3]{\frac{f_e}{8\,(1 - C_e)^3 \sin\alpha}}$$

Where:

d_{90} = depth where the fluid is 90% air

d_c = critical depth

α = channel slope

C_e = air concentration, 0.9 sin α (for $\alpha < 50^o$)

f_e = Darcy friction factor for the air/water mixture

$$\frac{1}{\sqrt{f_e}} = 1.42 \ln\left(\frac{D_h}{k_s}\right) - 1.25$$

D_h = hydraulic mean depth, $D_h = \frac{4A}{P}$

A = cross-sectional area of flow

P = wetted perimeter

One of the disadvantages of this equation is that in order to calculate f_e, the depth must be known. However, this can be solved by trial and error using computer iterations.

For a flow in the test facility of 70 l/s, this gives d_{90} = 0.077m, which is compatible with the observed depth in the test chute of 0.09 m. However, this estimate used k_s = 0.8 mm. Had k_s been taken as $k_s = h \cos \alpha$ = 0.03 m, then the resulting calculated d_{90} would be 0.214 m, which is considerably more than the observed value.

Appendix B contains tables of depth data collected on spillway models with typical scales of 1:20 or 1:25. At these scales, the air entrainment would not be fully developed and the model would underestimate the d_{90} value. CRM normally advise clients to apply a bulking factor to the data to account for this and recommend a 25% increase. The data suggest that for steeper step angles (around 30^{o}) where the skimming flow low pressure roller occupies the full tread of the step, values measured on the model and calculated by Chanson's equation give a reasonable comparison if a roughness of around 100mm is used. However, with lower slope angles where the roller occupies only part of the tread, relating the roughness to the step height using the equation $k_s = h \cos \alpha$ appears to work.

Chanson's equation appears to give satisfactory results if an appropriate value of the surface roughness of the channel can be determined.

Boes and Minor's equation, produced for steeply sloping roller compacted concrete spillways, gives unpredictable results, especially when used on spillways made from sections of plane chute inter-dispersed with cascades of steps.

5.1.3 Estimation of Pressure Differential on Walls

Figure 5.1 shows a design chart for the dimensionless pressure differential (pressure differential (p)/step height (h)) against Froude number. In deriving the dimensionless pressure differential, the step height was taken as 31.4 mm for both the normal steps and the step with the end upstand.

Figure 5.2 shows a design chart for maximum pressure differential presented as pressure differential/critical depth (d_c) against Froude number. The two design cases correspond to a relative roughness k_s/h of 0.015 and 0.025. The lines of best fit through the data would tend to converge on the origin 0 – 0 and it is probable that other lines of k_s/h could be drawn between these data lines, although more tests with different spillway roughnesses would be needed to confirm whether the spacing would be uniform in multiples of k_s/h.

Figure 5.1 and Figure 5.2 can be used to calculate the pressure differential that could exist on a channel wall adjacent to a step in skimming flow.

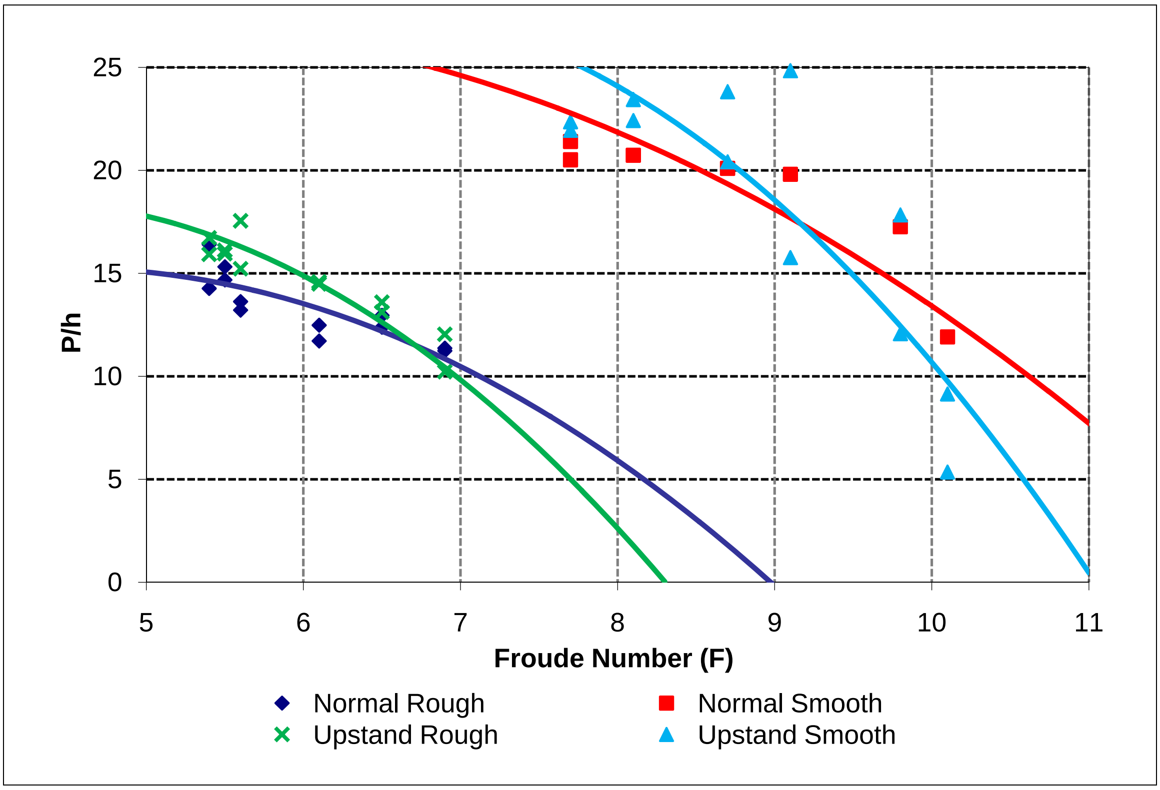

Figure 5.1 Pressure differential on wall design chart

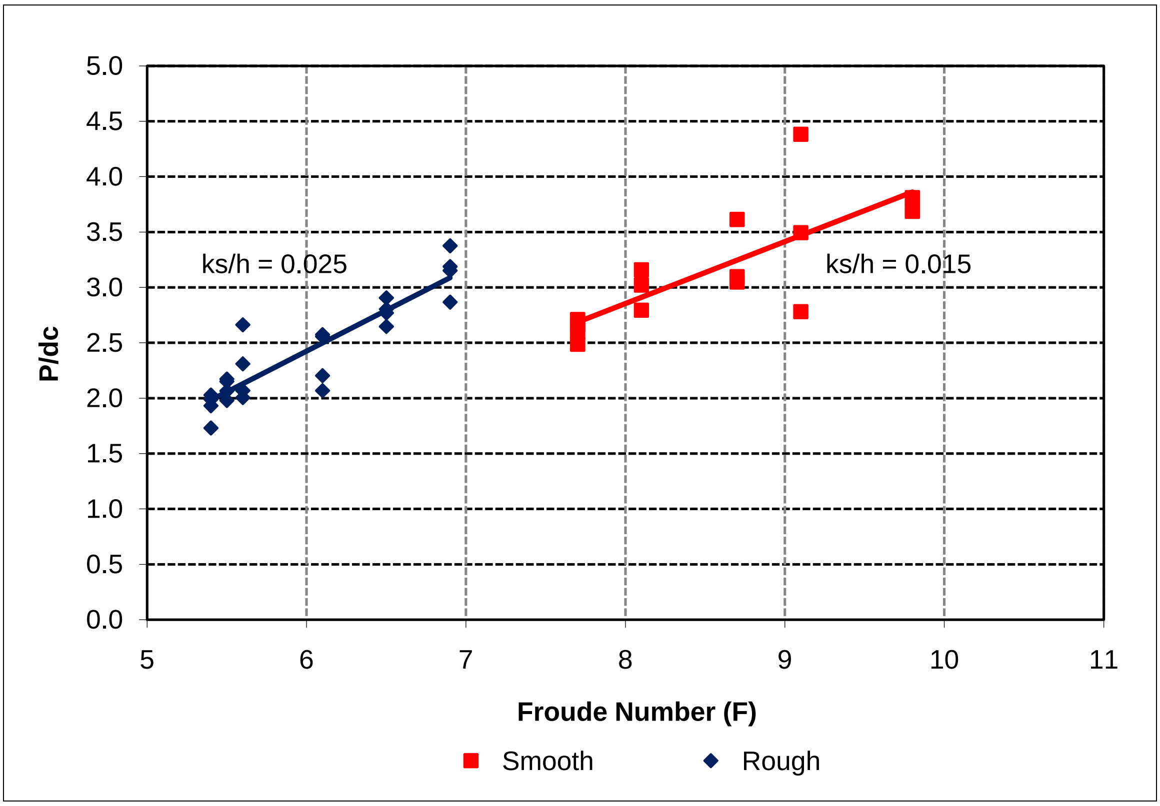

Figure 5.2 Maximum pressure differential on wall design chart

For both of these design charts, the Froude number has been derived from the depth using the continuity equation:

$$d = \frac{Q}{b\,v}$$

Where:

d = depth

Q = discharge

b = width

v = velocity

5.1.4 Estimation of Velocity Head

Sections 4.5.1, 0 and 4.5.3 discuss the situation where the upstream and downstream joints around a masonry block are left open. Under these conditions, if it is assumed that all of the pressure increase around the block is caused by the mobilization of the velocity head, then the results from these tests can be used to produce a design graph.

The depth averaged flow velocity in the test facility was 4.55 m/s (peak velocity is 6.3 m/s). This generated a velocity head ($v^2/2g$) of 1,055 mm. The mean and maximum pressures generated at the upstream joint for the case of the block standing proud (tapping 4) and in the downstream joint for the case of the block set into the panel (tapping 6) are plotted as a percentage of the mean velocity head against proportional intrusion of the block on Figure 5.3 and Figure 5.4. The proportional intrusion of the block is given by z/b where z is the amount the block is out of alignment (0, +5, +10, +15 and -5, -10, -15 mm) and b is the block vertical dimension (50 mm on the model).

These charts can be used to calculate the maximum and mean velocity heads on a channel wall adjacent to a step in skimming flow. Although the data were collected at joints in the channel wall, there is no logical reason why they could not be used to assess the risk of failure of blocks in the invert of a smooth masonry chute.

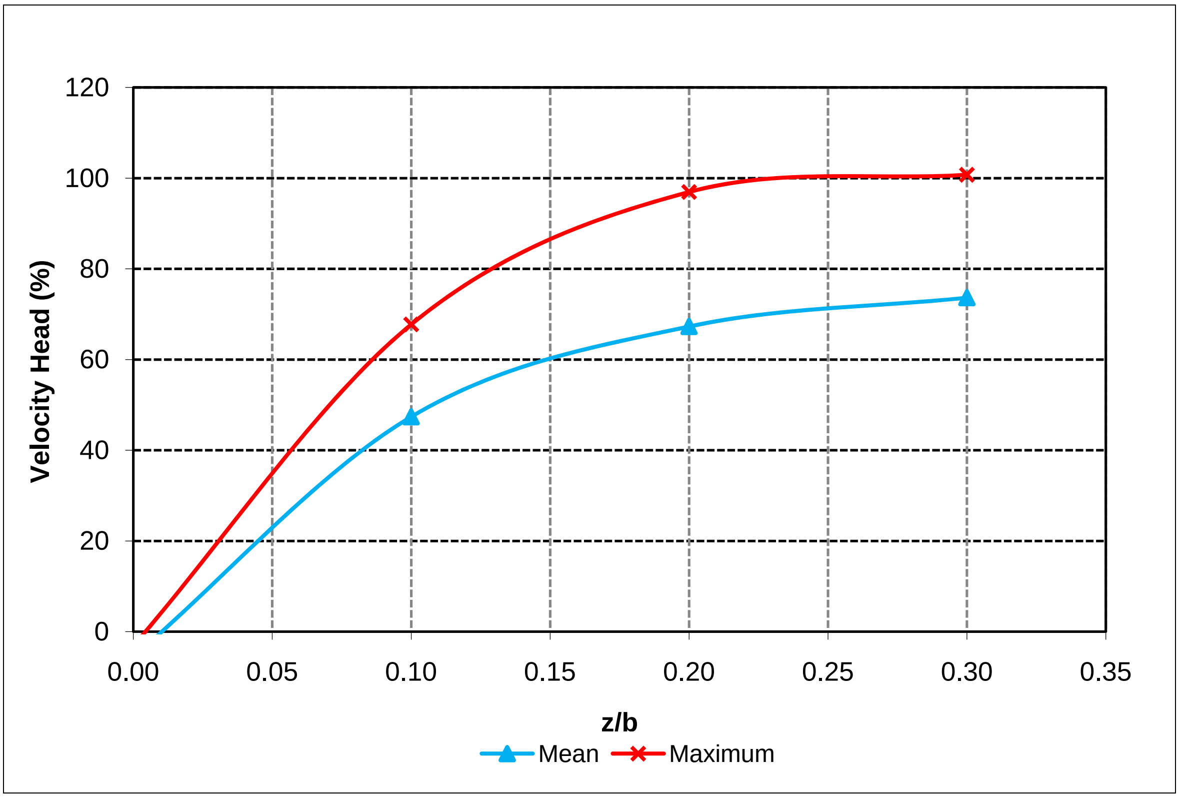

Figure 5.3 Percentage of velocity head mobilised in the upstream joint by a block protruding into the flow

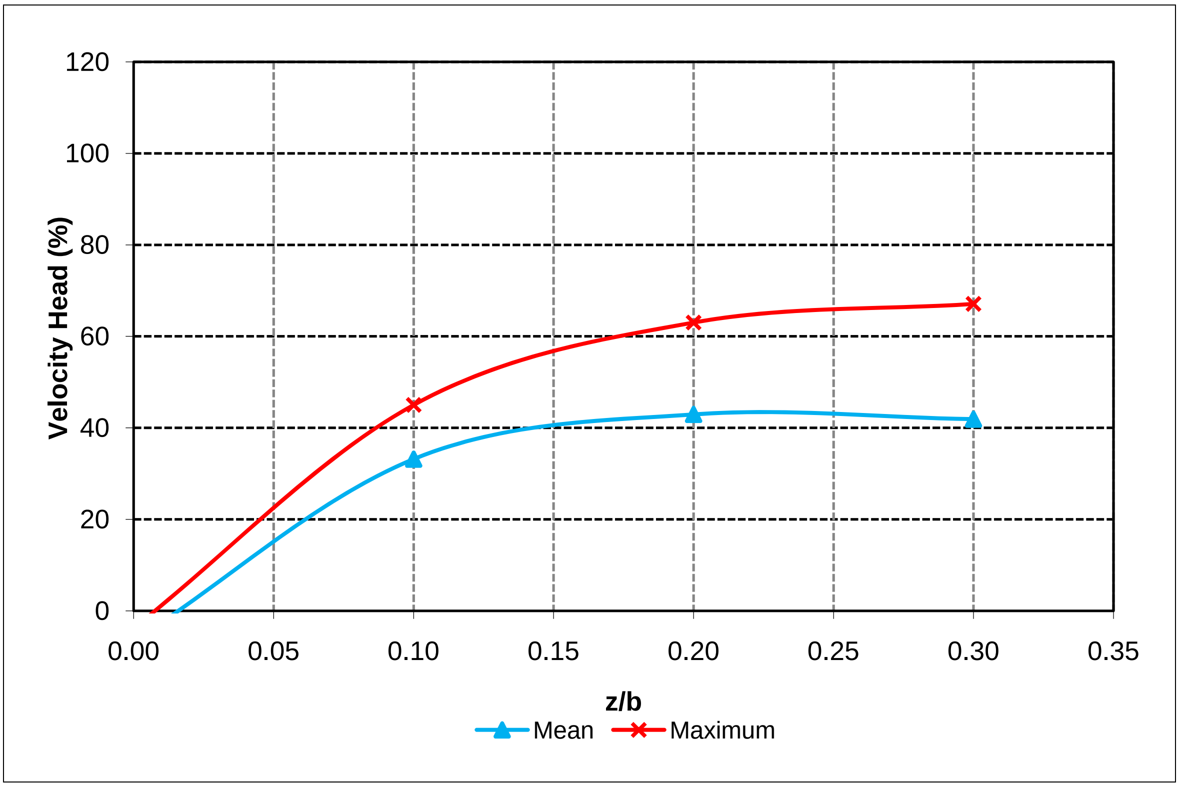

Figure 5.4 Percentage of velocity head mobilised in the downstream joint by a block inset into the wall

5.2 Computer Modelling

5.2.1 Introduction

Analysing the flow properties of spillways under extreme flow conditions is very difficult to do by direct observation with the use of scaled physical models traditionally being the only feasible investigation method. However, recent developments in high performance computers have meant that the use of numerical methods, such as Computational Fluid Dynamics (CFD), is becoming more attractive. This is due to the shorter preparation times and lower costs associated with numerical methods and the possibility of obtaining results through the whole computational domain rather than at a limited number of selected monitoring points.

The validation and verification of CFD predictions have been researched by many other industries, e.g. aerospace, and the relationships between the real physics, the mathematical model and the computer representation of the mathematical model have been studied widely. Many recommendations and guidelines that have arisen from these studies are also applicable to the CFD modelling of hydraulic structures. It should be noted that both physical models and CFD models are only mathematical representations of the real physics.

Hydraulic flows can be classified as "viscous" and "incompressible" flows with inertia effects. The relevant set of mathematical equations for these types of flows are known as Navier-Stokes equations. The flow variables such as velocities and pressures in the domain are solved in the computation of each time step.

Most engineering flow problems are associated with turbulence. By introducing a turbulence model to the equations, the computations become more complex. The most commonly used turbulence model is the Reynolds Averaged Navier-Stokes (RANS) model, which is an expanded form of the Navier-Stokes equations that involves Reynolds Stresses. An alternative method for modelling turbulence is Large Eddy Simulation (LES), which computes large eddies while a model represents small scale ones. LES is computationally very expensive compared to RANS. The details of different numerical methods for solving RANS or LES equations can be found in the standard CFD textbooks, e.g. Peric (2002) and Wilcox (1993).

5.2.2 Free surface modelling

The surface tracking method, moving mesh method, Eulerian multi-phase approach and thin film model are some of the approaches studied in the literature for modeling multi phase flows. Each of these approaches has strengths and weaknesses and depending on the application type, some will be more favourable than others.

The Volume-of-Fluid (VOF) method is one of the most commonly used approaches for modeling free-surface flow problems. This is a special case of the Eulerian multi-phase modeling approach in which the fluid travels through the cells of the fixed mesh. The interface between the water and the air is preserved by defining a new variable, volume fraction, and satisfying the continuity of it within the domain. Each cell is assumed to contain one of the phases or the interface between the phases. The water and air are considered as one continuum and parameters such as density and viscosity change sharply at the interface. The reader is referred to the Peric (2002) and Wilcox (1993) for detailed theoretical background.

Typical process flow of free surface modelling in CFD is itemized below.

Building the domain geometry

Channel walls are usually represented by simplified surfaces or a network of a connected surface mesh. Surface roughness details are not practical to include in the CFD model. Most CFD meshing software requires a clean geometry, i.e. air tight geometry with no gaps or overlapping surfaces.

Generating the computational mesh

Studies in the literature mostly recommend the hexahedral type of mesh for free surface flow type simulations. High mesh resolution is required near the free surface and on the channel walls. In stepped spillways, it is recommended to have at least three or four elements in the step height. Prism layers are also required on all the channel walls so that the boundary layer is modelled more accurately. Mesh resolution on the sky boundary can be kept lower than the mesh around the channel.

Setting up the CFD model

Boundary conditions:

- <u>Upstream boundary:</u> It is recommended to create the upstream boundary far from the spillway crest including some of the reservoir in the model. The reservoir water height will then be assigned as a boundary condition on the upstream boundary by defining the hydrostatic pressure distribution (for both air and water). With this approach, when the simulation starts the flow will develop and the water height and velocity profile at the spillway crest will be computed.

- Downstream boundary: The downstream boundary should be a pressure boundary which is built as far from the area of interest as possible to minimize the impact of the wave reflection at the boundary.
- Wall boundaries: An appropriate value of wall roughness should be defined to match the empirical Manning's or Chezy's coefficient.
- Sky boundary: This is ideally defined as a pressure boundary allowing air flow in and out of the domain. However, resulting pressure fluctuations should not interfere with the free surface. In some simulations, having such a large pressure boundary may cause convergence problems. In these cases, it is recommended to use a free-slip wall boundary, or a symmetry boundary to minimize this problem.

Turbulence models:

- RANS models can predict the mean flow reasonably well. However, they fail to predict the turbulent fluctuations. Although LES is more capable of capturing turbulent fluctuations, in most cases it is not practical to use LES in this type of application as it is computationally very expensive. There are many studies in the literature that uses RANS methods to model turbulence and many of them show reasonably good agreement with experimental studies in predicting the hydraulic profiles.

Simulation (solving the unknowns):

The simulations should run transiently and should run long enough for the flow to develop so that the initial conditions would not be affecting the flow field anymore. The time step size should be adequately selected to achieve convergence within the timestep. If a fully implicit solver is used, the timestep should be small enough to ensure the equations converge within each time step (meaning that the transient physics is well resolved). If an explicit solver is used, it should be ensured that the Courant number (the ratio of time step size to the cell residence time) is small enough for convergence. Limiting the Courant number to a maximum value of 0.3 is a commonly used criteria in the literature to determine the time step size.

Post-processing:

One of the biggest advantages of CFD compared to physical modelling is its capability to produce results through the whole computational domain rather than at selected locations of measuring sensors. It is possible to get a snapshot in time of the variable of interest, e.g. the pressure acting on the walls of the spillway.

Some sample images obtained from CFD are shown in Figures 5.5 to 5.7.

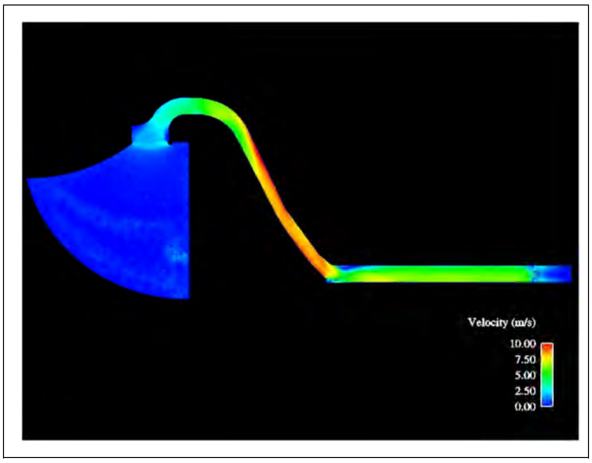

Figure 5.5 Reservoir spillway – velocity at the water surface

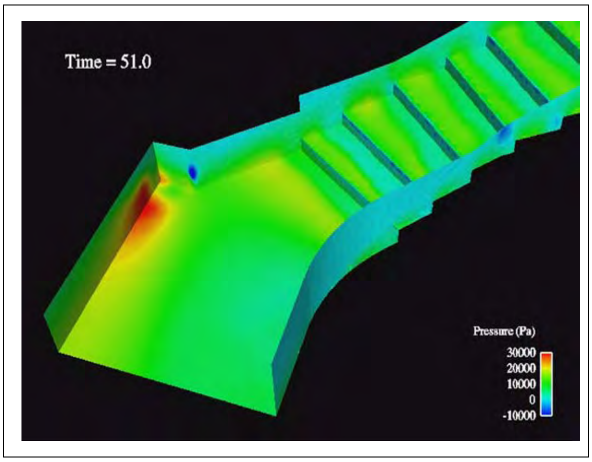

Figure 5.6 Reservoir spillway – pressure acting on the spillway walls

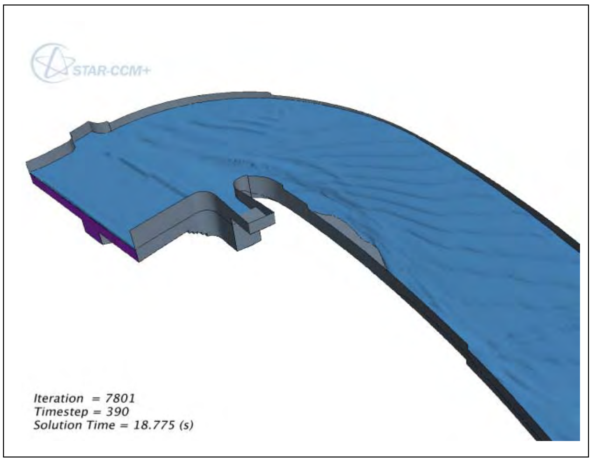

Figure 5.7 Reservoir spillway – water surface

5.3 Maintenance and Inspections

5.3.1 Introduction

One of the difficulties associated with assessing any masonry spillway is that it is unlikely that all aspects of its design will be immediately clear from simply inspecting its exterior. Therefore, wherever possible, and ideally prior to any site inspection, drawings should be obtained showing the design of the spillway, including cross-sectional views. This will help identify the type of material behind the masonry surface thus giving an indication of the location of vulnerable areas within the spillway.

The Undertaker of the reservoir is likely to be the most likely source for such information. An alternative would be to carry out research into the construction of the dam and the spillway. Organisations such as the Institution of Civil Engineers keep archives containing historical records of this type and it may be possible to get suitable information from them either on a given spillway or on similar spillways elsewhere.

Typical original drawings are likely to be similar to those shown in Figure 2.5. Coring at a number of locations in the spillway would allow the construction of the spillway behind the surface to be determined and the presence, size and distribution of any voids and unmortared joints to be identified. It is recommended that cores have a minimum diameter of 100 mm to allow for visual inspection.

In exceptional circumstances, and as an option of last resort, the possibility of carefully taking a section of the spillway apart could be considered in order to identify the form and materials of its construction. This is only likely to be appropriate as part of a larger scale assessment of the spillway involving the input of an appropriate specialist.

The following sections of the report (5.3.2 to 5.3.12) describe the aspects of masonry spillways that should be assessed during a routine inspection and identify some of the likely causes for damage. Recommendations for remedial works are made in section 5.4. A checklist for recording observations made during inspections is provided in Appendix B.

Following an inspection in which a spillway is found to be in a poor state of repair, then a programme of works to return it to a good condition should be prepared. If no detailed maintenance programme exists, then the frequency of future inspections should be determined, based on the anticipated likely rate of deterioration of the spillway. Each spillway is likely to prove to be different in this respect, based on the fact that the materials it is made from and/or the external factors affecting it will vary.

5.3.2 Assessing the General Condition of the Spillway

The condition of materials in a spillway, such as mortar pointing, provide a clear indication of the structure's condition. Degraded masonry may also have a range of telltale signs which can be read to provide evidence of their cause, and this can in turn provide information on other problems.

A note should be made of the construction of the spillway, including such items as:

- Whether or not the tops of the side walls are adequately waterproofed;
- The form of the cappings or copings present on the spillway sidewalls;
- The finish to the mortar pointing and its condition.

5.3.3 Surveying the Spillway for Problematic Areas

The specifics of the spillway's design will vary. However, the general maxim is that nothing must be either loose or hollow - everything must be solid.

The condition of masonry can be assessed in a number of ways, including the following:

- By dragging a chain along the invert of the spillway and noting any areas that sound hollow;
- By tapping a random selection of masonry blocks in both the invert and the side wall surfaces with a wooden mallet and noting any areas that sound hollow; or
- By undertaking sonic velocity testing to investigate apparent changes/variability in the character of the masonry.

There are also a number of additional tests that it might be appropriate in specific cases, including radar.

In all cases, any areas that either sound hollow or are identified as being potentially problematic will need to be examined in more detail.

5.3.4 Assessing the Condition of the Rear of the Spillway Sidewalls

The areas immediately behind the spillway's sidewalls are susceptible to soil erosion caused by overland flow and possibly by water-splashing over from the spillway. Therefore, these features should be inspected during the assessment and any erosion noted. The presence of any standing water in these areas should also be noted.

5.3.5 Identifying Areas of Dampness

Any sections of the spillway, in either the sidewalls or the invert, that are damp or have water trickling from them, even when the spillway is otherwise dry, should be noted. This is an indication of external water ingress and should be investigated further.

5.3.6 Assessing the Condition of the Masonry Blocks

As demonstrated by the physical model testing discussed in Chapter 4, the condition of the individual stones or bricks lining the spillway is very important as far as its overall performance is concerned. High flow rates and turbulence could dislodge loose stones or bricks from the wall, particularly if they are inset or protruding from the wall.

As part of the assessment, the blocks making up the surface of a spillway need to be assessed, with the following questions answered:

- What condition are the masonry blocks in; are they intact, are they partially eroded - e.g. are some or all of the corners missing, are they spalling and do they have any salts on their surfaces?
- Are any blocks loose or do they sound hollow when tapped?

All loose blocks or hollow sounding areas should be recorded.

5.3.7 Assessing the Condition of the Mortar Pointing

The following aspects of mortar pointing should be assessed for all areas of masonry:

- Are either whole or parts of mortar beds missing from any of the areas being assessed?
- Is the mortar cracked or crumbling and, if it is, can it be removed easily to a depth of 10mm or more with a flat metal blade?
- Has at least 10mm of the mortar been eroded away?
- Does the mortar contain a hard surface layer that can be picked out?
- Is the mortar joint flush with the block or does it have a bucket handle finish? Mortar that is flush with the block will provide better pressure and hydraulic conditions than mortar that has a bucket handle finish (see section 4.5).

5.3.8 Identifying Cracks in the Masonry

Both the location and size of each crack should be recorded, and an attempt should be made to determine their likely cause.

Cracking in masonry takes a number of specific forms and some knowledge of the cause(s) of the cracking can be determined from the widths, lengths and shapes of the cracks. Information linking the description of a crack, or a cracked area, to a possible cause for the cracking is contained in the following table:

Table 5.1 Possible Causes of Cracking in Masonry

Crack description	Likely cause
Short, hairline cracks; usually concentrated in areas that are damp, such as the top of the side walls	Frost
Cracks associated with the mortar; randomly dispersed within an area of the masonry, typically one that is damp	Sulfate expansion
Spalling of the surface of the masonry blocks into small plates roughly parallel to the unit's surface	Salt crystallisation within the masonry blocks
Parallel width cracks up the height of the side wall	Natural shrinkage of the masonry blocks
Parallel width cracks localised to the side of the mortar joints	Natural shrinkage of the mortar
Limited number of probably larger cracks in the side walls or bed: - In the side walls, they are wider at the top of the walls than at the bottom - In the bed, they are roughly parallel cracks	Soil movement - heave Tree roots
Limited number of larger cracks, which are wider at the bottom of the side walls than at the top; these cracks are likely to continue into the bed	Soil movement - subsidence Tree roots Inadequate foundations Dam movement and stability issues

5.3.9 Identifying Spillway Movement

The spillway should be checked to identify and record any areas that are not consistent with the rest of the spillway and an attempt should be made to identify the cause of all anomalies. For example, note should be made of any bulges or leaning parts on those sections that are otherwise planar.

All walls associated with a masonry spillway are likely to have been built to be plane and vertical. In many cases they will have also been built with horizontal upper surfaces. Any deviation from that original condition is therefore likely to be symptomatic of a problem.

More specifically, leaning, bowing or subsidence are likely to be associated with one, or more, problems with the foundations supporting the walls.

Leaning

If the wall leans by more than 25 mm in any 1 m height, then the cause of the lean should be investigated. In many cases, it will be associated with localised foundation failure, slope instability or frost heave. It is likely that the affected section of wall will need to be taken down and rebuilt.

Bowing

The cause(s) of bowing should be investigated as it can be symptomatic of:

- Localised debonding of the outer surface of the wall;
- Sulfate attack in the masonry;
- Localised failure of the foundations or of the fixity of that part of the wall to the foundations.
- Slope instability

Sinking

A sinking wall is a result of localised foundation failure. Where it occurs, further investigations should be undertaken to ascertain the precise cause. It is important though to establish whether that part of the wall is actually sinking rather than that sections of the spillway to either side are rising.

5.3.10 Identifying Vegetation Growth in the Spillway

Where vegetation is growing on or within the surface of the masonry, their presence should be recorded as should the species and the size of the vegetation.

5.3.11 Identifying Trees in the Vicinity of the Spillway

Large or mature trees that are within 10 m of the spillway should be recorded in terms of distance and species. Where their presence coincides with damage to the spillway this too should be noted.

5.3.12 Identifying Cracks in the Ground Surrounding the Spillway

The presence of cracks in the ground between the spillway and the dam or in the ground on the other side of the spillway should be reported immediately to the appropriate person(s) within the associated company responsible for reservoir safety and for maintenance.

5.4 Remedial Measures

5.4.1 Introduction

It is likely that most masonry spillways will be in a reasonable condition at the time of inspection and that the type of work identified as being needed will be repointing of the mortar and, perhaps, removing and refitting a number of loose masonry blocks. However, this might not be the case, and the inspection may have suggested that further investigations and remedial works will be needed, for example:

- Stitching masonry blocks together using stainless steel ties;
- Removal of loose mortar and pressure pointing to fill any voids in the construction
- Localised replacement of missing masonry blocks with concrete using “letter box” shutters to obtain a sympathetic finish;
- Taking down and reconstructing the spillway where movement or sulphate attack is found to be severe and/or widespread;
- Removal of vegetation within the spillway;
- Removal of large trees and vegetation in close proximity to the spillway;
- Remedial measures to drainage to reduce uplift instability or deterioration due to dampness.

The following sections of this report contain recommendations for remedial works that could be carried out to improve the condition of such spillways.

5.4.2 Replacing Masonry Blocks

Eroded Masonry Blocks

In cases where the masonry blocks have degraded and if fewer than 10% of the blocks need replacing, it is recommended that compatible blocks be stitched in using stainless steel ties. If more than 10% of the blocks in any given area have eroded and require replacement then it is likely to be more cost effective to rebuild the surface of that area of the wall.

Unless an entire area of blocks are eroded and it is believed that the blocks themselves were not appropriate for the location, the failed blocks should be replaced with new ones made from the same material.

Replacement masonry blocks should be stitched into place using stainless steel ties and should be set flush with the surrounding blocks, not proud or inset in order to reduce the pressures acting on them.

Loose Masonry Blocks

Where the masonry blocks are loose, they should be carefully removed from the surface of the masonry, set to one side and the condition of the exposed material should be assessed. Where a number of blocks in one area are loose, it is possible that their condition reflects a problem in the area behind them and the condition of this whole area should be assessed before the surface masonry is put back in place.

Where it is found that the material behind the masonry is in good condition, the masonry blocks should be stitched back into place using stainless steel ties. The blocks should be set flush with the surrounding blocks, not proud or inset in order to reduce the pressures acting on them.

Localised Replacement with Concrete

Discussions with Owners and Inspecting Engineers during the course of this research also revealed cases where the space left by missing single masonry blocks had been refilled using concrete. This was typically done using “letter-box” shutters.

Although this is not an entirely sympathetic option as far as appearance is concerned, it has the advantage of being relatively quick and inexpensive whilst at the same time achieving a good and thorough infill and bond within the gap. A good “hydraulic” profile can also be achieved with regards to the surrounding masonry. Coloured cements can be used for colour matching to the adjacent masonry, if desired.

5.4.3 Mortar repointing

For areas of masonry where the mortar is in poor condition or missing, repointing should be carried out. Poor condition is defined by:

- Either whole, or parts of, mortar beds are missing from any part of the area being assessed;
- The mortar is cracked or crumbling and can be removed easily to a depth of 10 mm or more with a flat metal blade;
- At least 10 mm of the mortar has eroded away;
- The mortar contains a hard surface layer that can be picked out.

Any deteriorated or loose mortar should be removed and replaced.

The aim should be to make the area impermeable to water ingress as the presence of water flowing through the spillway may lead to soil erosion either behind or under the spillway. This is contrary to common practice with retaining walls.

As demonstrated by the physical model testing discussed in Chapter 4, it is particularly important to ensure that sections of wall located along the downstream length of a step with missing mortar or mortar in poor condition are repointed as these sections will likely be subjected to high positive pressures during large flood events. Drain holes could be drilled in the lee of steps in order to relieve these positive pressures.

Repointing procedure

Masonry in the area to be repointed should be cleaned of all calcium or other deposits and any defective brickwork or masonry, fractures and poor mortar joints in the area to be treated should be repaired prior to the commencement of repointing. The mortar should be removed from the mortar bed to a depth of twice its width, i.e. if the mortar beds are 10 mm wide, then the existing bed should be removed to a depth of 20 mm from the surface of the wall and then re-pointed to the masonry surface.

The external surface of the mortar should either have a struck flush finish or a slightly recessed, bucket handle finish. These options provide a finish which reduces the likelihood of water passing into the masonry and reduce the pressures acting on the masonry when compared with other finishes.

Where large areas of the spillway require repointing, pressure pointing should be undertaken. Pressure pointing of all vertical and horizontal joints should be done by:

- Drilling through the masonry (not the joints);
- Installing adjustable packers at the ends of the drill holes at the grouting location and plugs at the other holes;
- Commencing at the lowest holes, systematically working up the wall injecting grout through the packers until it is expelled from neighbouring holes;
- The pressures used should be limited to avoid causing additional damage to the wall or invert.

Selection of repointing mortar

The repointing mortar needs to be of a similar strength to the mortar used in the existing bed joint - if the mortar is too strong, it risks both damaging the masonry blocks around it and coming loose. It can, however, be difficult to determine the original mortar mix used in the spillway's construction, in part because the mortar in the outer surface of the bed joint may, itself, be repointing mortar and, typically, existing repointing mortars are considerably stronger than the original.

As a result, the following are the best ways to determine the mix proportions of the original mortar:

- Look at the drawings for a specification;
- Make assumptions based on the age of the spillway. If it was built before 1900 it is likely to have been made using a lime mortar;
- Delve deeper into the masonry, beyond any repointing mortar, and remove a sample of mortar for testing.

In situations where a cement based mortar has been identified, a designation (ii) mortar (see Table 5.2) might be an appropriate mix to use to repoint with. Where a lime mortar was used, the use of a weaker mix - for example a designation (iii) or (iv) mortar might be considered. It should, however, be borne in mind that, while the selection of a weaker mix is likely to ensure that the masonry made with lime mortar remains intact and in good condition, it is also likely to mean that the mortar is more susceptible to frost damage and general erosion. This in turn means that the required frequency of inspection and repair of the spillway is likely to increase.

Table 5.2 Mortar Mixes

Types of mortar		Cement:lime:sand	Masonry - cement:sand		Cement:sand (plasticized)
Binder constituents		A Portland cement and lime, with or without an air entraining additive	Masonry cement containing a Portland cement and lime in the approximate ratio 1:1, and an air entraining additive	Masonry cement containing a Portland cement and inorganic materials other than lime and an air entraining additive	A Portland cement and an air entraining additive
Designation	**(i)**	1:0 to 0.25:3	-	-	1:3
	(ii)	1:0.5:4 to 4.5	1:3	1:2.5 to 3.5	1:3 to 4
	(iii)	1:1:5 to 6	1:3.5 to 4	1:4 to 5	1:5 to 6
	(iv)	1:2:8 to 9	1:4.5	1:5.5 to 6.5	1:7 to 8

Further information on mortar mixes can be found in BS 5628-3.

5.4.4 Spillway Movement

Where the spillway walls are found to be bowed, sinking or leaning by more than 25 mm in any 1 m height, then the causes of these movements should be investigated. It would also be prudent to monitor the movement of the spillway to identify whether the spillway is currently moving or has ceased.

The decision on whether to take down the wall and rebuild it will depend on what is causing the movement, whether it is continuing to move, and the extent to which the spillway is damaged. However, under most circumstances it is likely that the affected section of the spillway will need to be taken down and rebuilt.

5.4.5 Sulfate Attack

Where sections of the spillway have areas of mortar that are in poor condition or missing due to sulphate attack, then these sections should be taken down and rebuilt with sulfate resistant cement.

5.4.6 Removal of Vegetation

Where vegetation is growing on or within the surface of the masonry, they should be removed and any damage caused should be repaired. Except for the smallest plants, it is best to use a systemic weed killer to kill the vegetation before attempting to remove its root system from a wall. This will make the process easier and will ensure that any roots that remain will be dead, leaving them unable to cause further damage. The choice of weed killer should be compatible with the presence of any water supply in the vicinity of the spillway.

Care should be taken when removing vegetation from a spillway because unsympathetic removal could cause whole areas of masonry to come loose and these areas will need to be rebuilt after the removal is completed incurring further costs. In some circumstances, it may be more appropriate to remove specific masonry blocks in order to allow the removal of a root system and then replace them rather than simply pulling at the trunk of a plant.

Where trees are located less that 10 m from a spillway and damage to the spillway can be directly attributed to these trees, e.g. where tree roots have grown through the spillway walls, their removal should be considered in order to prevent the occurrence of further damage.

5.4.7 Examples of Remedial Works

Figure 5.8 Side wall with a definite lean – before and after local demolition. Wall will be rebuilt.

Figure 5.9 Plant for pressure pointing

Figure 5.10 Lines along a spillway channel to pressure point both sides of the wall on the right hand side of the spillway

Figure 5.11 Completed wall – spillway behind has been infilled

Figure 5.12 Damage to masonry wall repaired by replacement of missing blocks, repointing and fixing of leaky pipe

Figure 5.13 Channel invert before repairs

Figure 5.14 Repaired spillway channel

Figure 5.15 Cores being extracted from the side wall to determine thickness

Figure 5.16 Repointing of original brickwork with lime mortar

Figure 5.17 Examples of masonry replacement using textured concrete

Figure 5.18 Example of masonry replacement using textured concrete (courtesy of Reckli)

Figure 5.19 Replacement of masonry invert with concrete slab

Figure 5.20 Replacement of horizontal invert (impact) masonry slabs of steps with concrete slabs

Figure 5.21 Replacement of missing masonry invert blocks with concrete

Figure 5.22 Replacement of masonry stepped spillway with new stepped spillway constructed along same line in reinforced concrete

6 Summary and Conclusions

6.1 General

A review of the current understanding of masonry as a structural material and of the pressures and forces acting within stepped masonry spillways has been undertaken, followed by physical model testing of a prototype chute. From this, practical guidance for skilled professionals has been produced to assist with hydraulic design, maintenance, inspections and the design of any remedial measures. For all practical purposes the results of this work would also apply to any similar spillways constructed of brickwork. The main findings and conclusions from the research are summarised below.

6.2 Hydraulics

This research has identified the formula by Chanson (1994) as the most appropriate for calculating flow depths in the types of stepped spillway most likely to be associated with UK embankment dams. Furthermore it is noted that bulking due to aeration means that depths will be higher than those for the equivalent solid water flow. Often, the 90% water depth is calculated, that is the depth which corresponds to 90% of the water in the chute.

In reviewing the capacities of stepped masonry channels, it may also be necessary to make allowances for bends, curvature, super-elevation, obstructions or any other changes in geometry or cross- section, just as one would for any open channel flow calculations. However, the effect of these features on highly aerated stepped chute flow was not studied as part of this research. Physical modelling, mathematical modelling and engineering judgement may all be appropriate means of assessment in these cases.

Moderate flows in stepped spillways will tend to cascade down from step to step losing energy progressively and with each step acting as a form of stilling basin, this is termed ‘nappe flow’. As flows increase, it will begin to skip from step to step, with local vortices trapped within the steps. This is termed ‘skimming flow’. This research has identified that very high flows may ride on the top of enlarged vortices, hardly touching the steps.

The most appropriate equations for use when undertaking hydraulic design are given overleaf.

Onset of Skimming Flow	$d_c = 1.057\,h - 0.465\,\frac{h^2}{L}$	d_c = critical depth h = step height L = step length
Length from Crest to Inception Point[1,2]	$L_i = 9.719\,k_s\,(\sin\alpha)^{0.0796}\,F_*^{0.713}$	k_s = surface roughness of the spillway. α = channel slope $F_* = \frac{q_w}{\sqrt{g \sin\alpha\, k_s^3}}$ q_w = flow of water per unit width of the channel g = acceleration due to gravity
Depth[3,4]	$d_{90} = d_c \sqrt[3]{\frac{f_e}{8\,(1-C_e)^3 \sin\alpha}}$	d_{90} = depth where the fluid is 90% air d_c = critical depth α = channel slope C_e = air concentration, 0.9 sin α (for $\alpha < 50^o$) f_e = Darcy friction factor for the air/water mixture $\frac{1}{\sqrt{f_e}} = 1.42 \ln\left(\frac{D_h}{k_s}\right) - 1.25$ D_h = hydraulic mean depth, $D_h = \frac{4A}{P}$ A = cross-sectional area of the flow P = wetted perimeter

[1]The inception point is the position where the turbulent boundary layer reaches the surface and air is entrained into the water generating a two phase air/water mixture. [2]Chanson (1994) and Gonzalez and Chanson (2007). [3]Chanson (1994). [4] A "rule of thumb" roughness value of 100 mm gave a better fit to measured depths when used with Chanson's equation for predicting depth.

The pressure distribution that can occur on the inverts and walls of stepped spillways under high flow conditions is shown in Figure 4.12, which is reproduced here for convenience. It can be seen that high pressures can exist over the downstream regions of step inverts and on adjacent sections of side wall, whereas low pressures can exist on the vertical faces of step inverts, over the upstream zones of steps and, again, on adjacent sections of side wall.

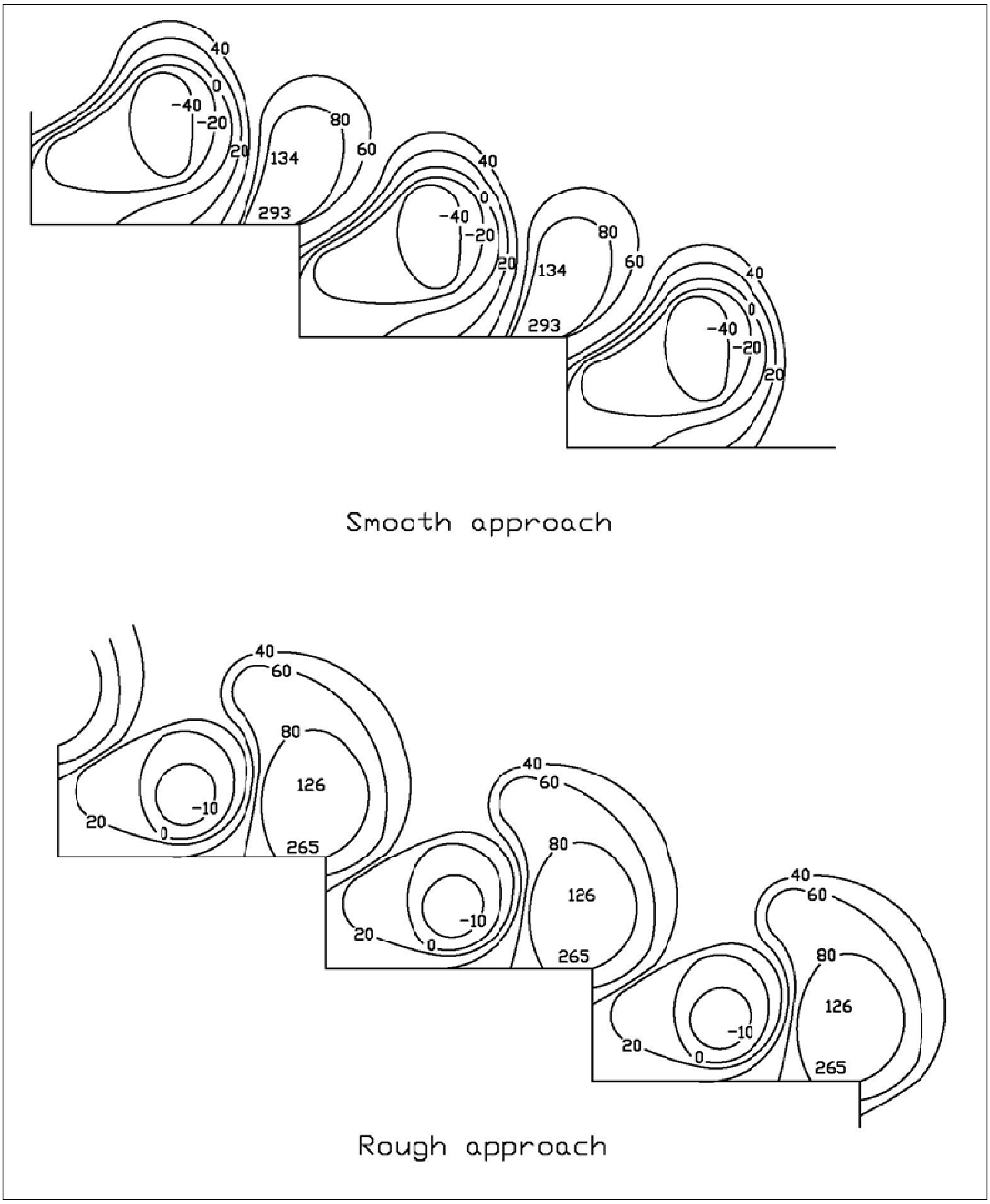

Figure 4.12 Mean pressure contour plot for flow projected along steps – smooth and rough approach

If high pressures are injected into open textured masonry in high pressure zones, such that they create a back pressure behind the masonry elements in low pressures zones, then the elements in the low pressure zones can be subject to removal. Moreover, testing has shown that there can be considerable turbulence and pressure fluctuations during such flows with the pressure differentials between transitory maximum and minimum pressures often being considerably higher than between associated mean pressures. Such potential pressure differentials on typical UK spillways can reach 5-10 m of water head. In addition, the high levels of turbulence within high pressure zones can be sufficient to dislodge blocks within these zones, as seen in the downstream zones of the steps on the Boltby spillway (see Figure 2.4, right photograph).

The exact zones of pressure distribution will vary depending on the geometry of the spillway in question and the flow being examined. Therefore, it is not possible to give generalised guidance but rather to draw attention to the potential and to the broad likely zoning. Readers are recommended to refer to the more detailed discussions and results in Chapters 4 and 5 of this report for further guidance.

In addition to the pressure differentials described above, testing indicated that significant pressure differentials could be produced by both locally protruding and locally recessed masonry blocks.

Design charts to calculate the pressure differentials that could exist on channel walls adjacent to steps in skimming flow are given in Figure 5.1 and Figure 5.2 of Chapter 5, and are reproduced here for convenience.

For both of these design charts, the Froude number has been derived from the depth using the continuity equation:

$$d = \frac{Q}{b\,v}$$

Where:

d = depth

Q = discharge

b = width

v = velocity

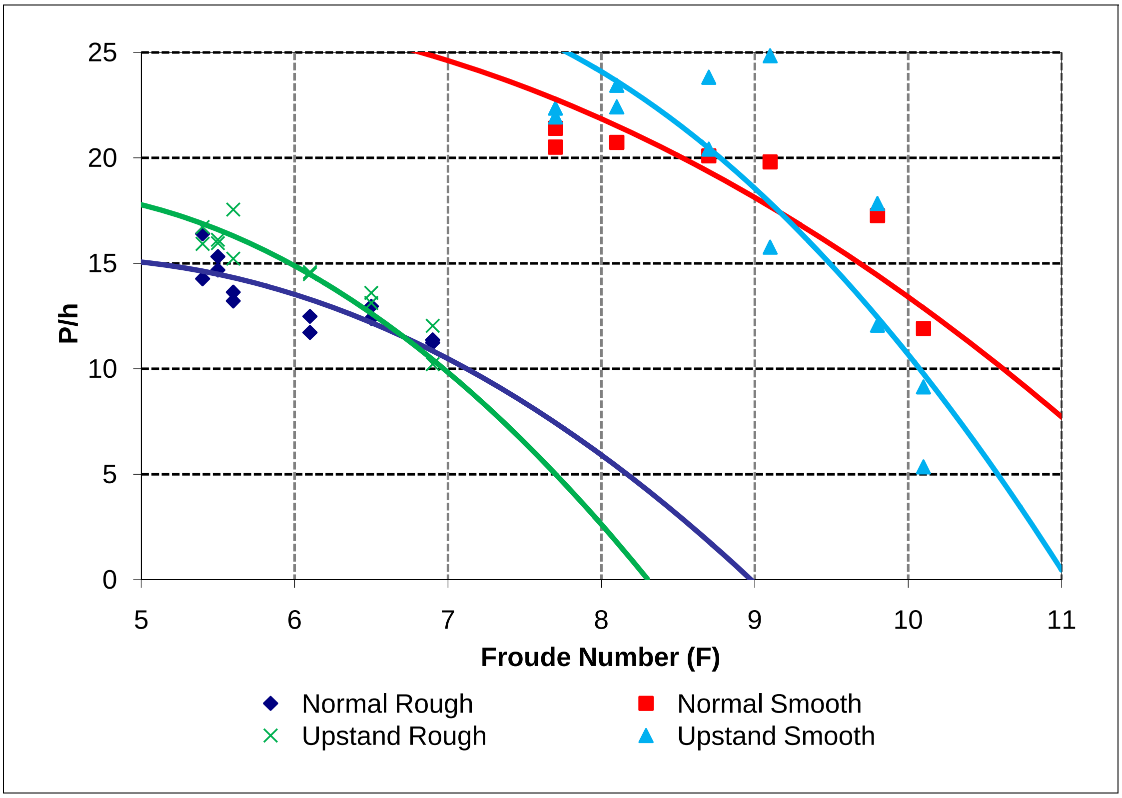

Figure 5.1 Pressure differential on wall design chart

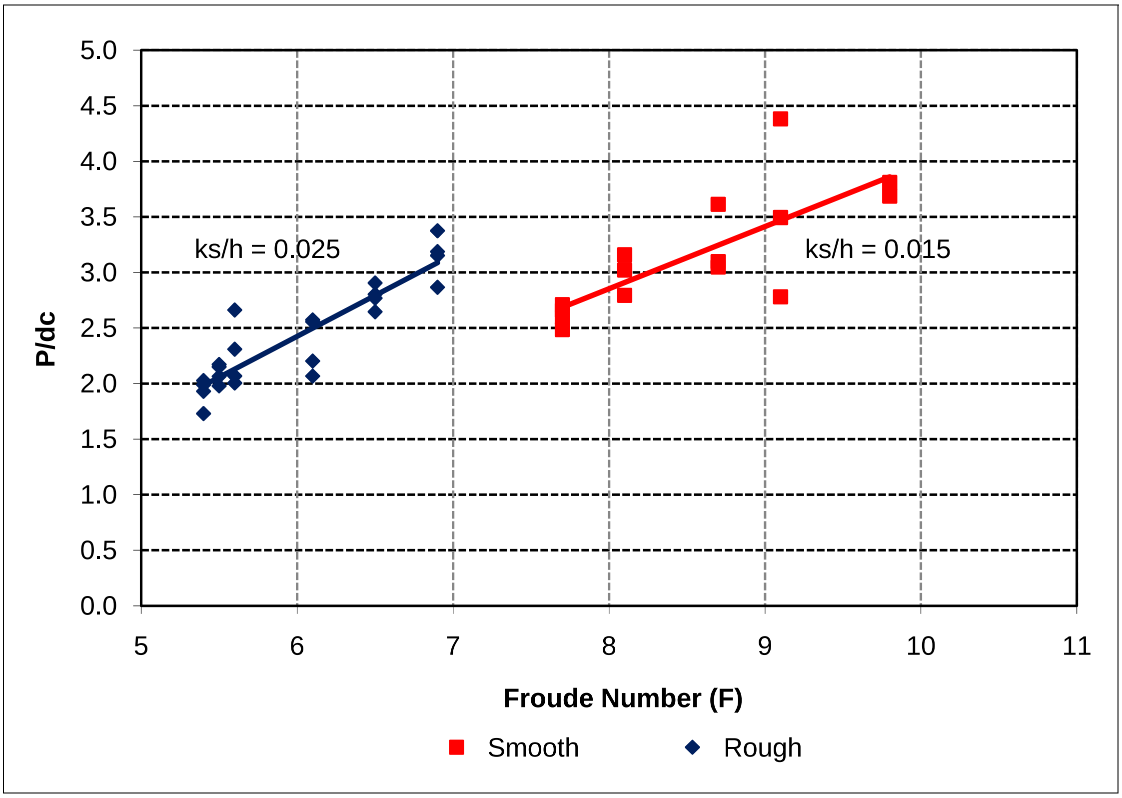

Figure 5.2 Maximum pressure differential on wall design chart

Design charts to calculate the local pressure heads that can be developed by both protruding and recessed blocks in a skimming flow regime are given in Figure 5.3 and Figure 5.4 of Chapter 5, and are reproduced here for convenience.

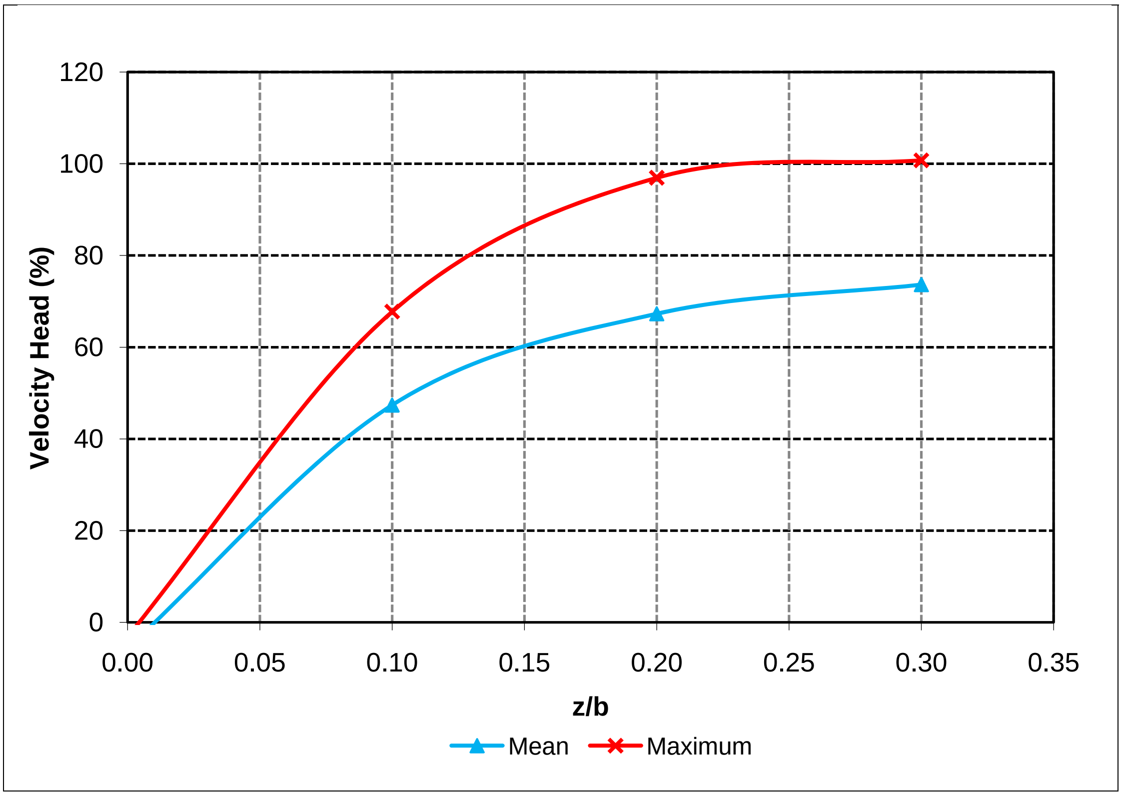

Figure 5.3 Percentage of velocity head mobilised in the upstream joint by a block protruding into the flow

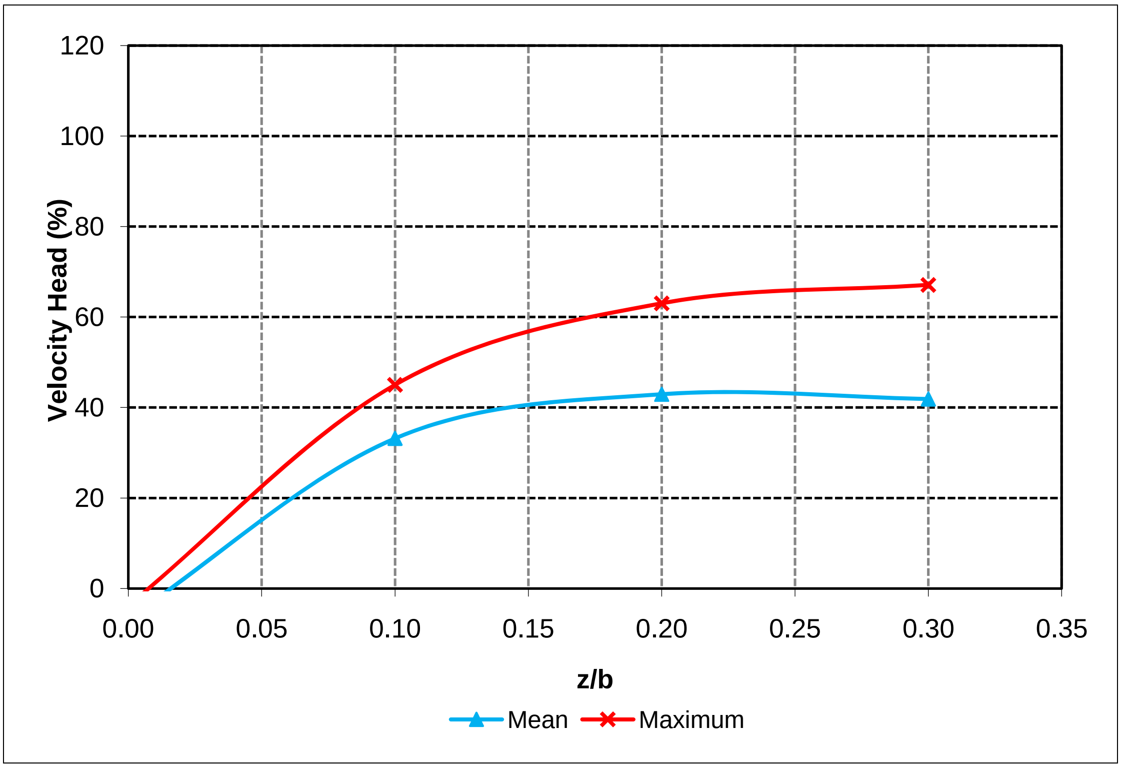

Figure 5.4 Percentage of velocity head mobilised in the downstream joint by a block inset into the wall

6.3 Inspections

Prior to inspection, attempts should be made to determine the construction of the stepped masonry spillway under consideration. Where there is limited information available from drawings, the spillway should be assessed based on its physical appearance. Coring could be undertaken to determine the construction of the spillway and, depending on the location of the cores, it could also ascertain whether the masonry face is tied to the backing skins and identify the presence of any voids behind the masonry skin. It is recommended that cores have a minimum diameter of 100 mm.

The presence of voids can also be indicated by tapping the masonry blocks for “hollow” sounding areas, while inverts may be surveyed by dragging chains, to achieve the same effect.

In addition to testing for voids, inspections should also identify the condition of the masonry blocks and mortar pointing, the presence of areas of dampness, cracks, vegetation and sulphate attack within the spillway, and signs of spillway movement. An inspection checklist similar to that given in Appendix B should be used to record the findings of an inspection.

Photographic records of spillways and records of vegetation in and around the spillways should be kept as these will provide useful information on changes in condition of the spillway over time.

6.4 Remedial Measures

Practitioners should be aware of areas within a spillway that may be subjected to extreme high or low pressures or that are highly turbulent (see the discussion above under “Hydraulics”). Particular attention should be paid to the condition of the masonry blocks and mortar pointing between the blocks in the high pressure zone locations. Drain holes could be formed in the lee of the steps in order to relieve the positive back pressures experienced here. One Owner suggested leaving some vertical masonry joints un-pointed in such low pressure areas to achieve the same effect.

Although it is difficult to apply the findings of the model testing generically, it could be expected that pressure differentials in the order of 5-10 m water head may exist along a step when a spillway is operating. However, provided that the masonry blocks and the mortar pointing between them are in good condition, then it should not be possible for the pressure to be transferred to the back of the blocks and the spillway should not suffer any significant damage. The fact that many such spillways have operated successfully for more than 100 years is adequate testimony to that.

As noted in the section on hydraulics, defective mortar pointing can result in the injection and development of significant pressure differentials across masonry blocks. Defective mortar pointing, especially in high pressure zones, should therefore be repaired so that it is sound and either finished flush with the blocks or finished with a bucket handle profile, as opposed to significantly recessed or protruding.

Blocks that protrude from the wall can cause extreme fluctuations in pressure around the block. In extreme cases, it may be appropriate to re-bed such blocks. In any event, it is important to ensure that the mortar pointing around such blocks is in good condition with a flush or bucket handle finish.

In high velocity and high pressure zones, the research indicated that it was more important to ensure that vertical mortar pointing (normal to the flow regime) was intact than horizontal mortar pointing (parallel to the flow regime).

Tapered blocks, such as those on the walls of the Ulley spillway, are less structurally stable than parallel-sided blocks. Therefore, it is particularly important where such blocks have been used in the construction of a spillway, that they are flush with each other and that the mortar pointing is in good condition with a flush or bucket handle finish.

Routine maintenance of stepped masonry spillways is crucial. This should include vegetation removal, block replacement, repairs to mortar pointing and the infilling of any voids by pressure pointing, as necessary. In the case of significant vegetation growth it may be appropriate to use a suitable weed killer prior to growth removal in order to minimise disruption. In addition, the presence of any trees within 10 m of the spillway should be noted, and if deemed to have the potential to cause or be causing structural damage, their removal should be considered.

It is likely that most masonry spillways will be in a reasonable condition at the time of inspection and that any remedial works required will be minor. However, further investigations and more involved remedial measures may be needed, such as:

- Stitching masonry blocks together using stainless steel ties;
- Replacing any loose mortar with a sound equivalent;
- Pressure pointing to fill any voids in the construction;
- Locally replacing missing masonry blocks with new blocks or with concrete using “letter box” shutters to obtain a sympathetic finish;
- Taking down and reconstructing the spillway where movement or sulphate attack is found to be severe and/or widespread; and,
- Removing vegetation within or in close proximity to the spillway.

Most stepped masonry spillways in the UK have stood the test of time, with over 100 years of successful operation since such materials formed the standard method of construction. Their use will continue to be acceptable provided that maintenance and inspection is undertaken on a regular basis by informed practitioners and is combined with careful remediation measures when required. These actions are of particular importance when the spillways are located along the mitre of the main embankment where a collapse of the side walls could endanger the dam (Environment Agency, 2009).

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Appendix A

Examples of Issues at Stepped Masonry Spillways Provided By Owners, Inspecting Engineers and Supervising Engineers

June 1999

Seepage emerging on valley side and pathway below masonry spillway, related to flow in spillway.

Personnel are photographed standing at locations of issue.

January 2000

Gap in masonry floor.

Routine repair.

June 2007

Damage to right sidewall of spillway at downstream end. Wall consists of dressed sandstone (550 mm long x 180 mm high). 3-4 blocks in length were removed over 3 courses vertically. Just downstream, 2 capping stones were lost. Backing of wall is made of random rubble which was eroded to a depth of 500-600mm.

Flood estimated to have 1 in 200 year return period.

June 2007

Loss of capping blocks on downstream stilling basin crosswall. All sandstone crest blocks were washed off into downstream channel and are resting against flow breaking blocks in channel below. There are 4 complete blocks with remainder being broken into 18 half blocks. Estimated that blocks were originally 1.2 m long.

Flood estimated to have 1 in 200 year return period.

June 2007

In worst area, a section 3 m wide by 6 m long was eroded to a maximum depth of 1 m. In masonry section on right-handside, 2-3 courses of sandstone block were removed from sidewall over 10 m length. On left-handside, damage was less with only 1 stone being removed. Root growth noted behind masonry.

Flood estimated to have 1 in 200 year return period.

June 2007

Damage to crosswall at downstream end of stilling basin. 10 sandstone blocks were removed and were laying in channel downstream; a further 3 blocks were displaced. Total length 26 m.

Flood estimated to have 1 in 200 year return period.

June 2007

Damage to lower section of main spillway. Channel is 69 m long, 3.65 m wide and 800 mm deep. Flood resulted in large proportion of blocks being removed (50% - 60% in upper section, 40% in lower section). Blocks were 400 mm x 200 mm and 200-250 mm thick and founded onto mortar and stone bedding, which remained in place. Bedding experienced significant erosion in only very few places.

Flood estimated to have 1 in 200 year return period.

Date unkown

Gaps in pointing of masonry at base of sidewalls allowing flow to disappear behind and re-emerge from base of sidewall.

General

1. S10 inspections highlighted lack of mortar in masonry spillways as a result of erosion or dissolving of cement matrix. Striking walls/floors revealed hollow spots where backing/underlying material had been drawn out by negative pressures.

   Deep re-pointing to replace mortar. Low pressure grout injection to fill voids.
2. Several cases of weathering of stones forming steps, sidewalls and copes of stepped masonry spillways.

   Routine repair.

Appendix B

Inspection Checklist

Reservoir: **Masonry Spillway Inspection Checklist**

Owner:

Spillway location:

Inspection by: **Date:** **Weather:**

Note that this inspection does not consider the impacts of failure of the spillway

DESK STUDY			
1	**Identifying the construction of the masonry spillway channel in detail**		
	Action:	**Source:**	
1.1	Obtain Drawings		
1.2	Dam and spillway chute construction (determined from drawings)		
1.3	Organise cores to be taken from sidewalls and invert if limited information available from drawings		
2	**Capacity of the masonry spillway channel**		
	Issue:	**Check for:**	**Record details**
2.1	Spillway geometry	Bends, confluence of channels, changes in channel width, junctions with stilling basins, pipe crossings, bridges	
2.2	Out of channel flow	Estimate the hydraulic capacity of the channel. Is there potential for out of channel flow?	

SITE INSPECTION			
3	**Condition of Materials:**		
	Issue:	**Check for:**	**Record in detail for sidewalls and invert**
3.1	Construction:	General description	-
		Finish to the mortar pointing	
		Presence of cappings or copings on the sidewalls	
3.2	Problematic Areas: (drag chain up the bed, tap selected unit with a rubber mallet or seismic velocity testing)	Loose masonry blocks	
		Voids behind dressed face	
3.3	Dampness:	Areas of damp or standing water	
		Running water	
3.4	Condition of the masonry blocks	Blocks that are missing, partially eroded, cracked, spalling, salts on surface, etc.	
3.5	Condition of the mortar pointing	Missing sections of mortar	
		Cracked or crumbling mortar	
		Locations where >10 mm of mortar has been eroded away	
		Hard surface layer that can be picked out	

	Issue:	Check for:	Record in detail for sidewalls and invert
3.6	Cracks in the spillway chute	Size and location of each crack Determine likely cause using attached sheet	
3.7	Signs of spillway movement	Lean (if more than 25 mm over 1 m height then attempt to identify the cause of the abnormality)	
		Bulge (attempt to identify the cause of the abnormality)	
		Sinking (further investigation required to identify the precise nature)	
3.8	Presence of plants	Species and size of plants growing within spillway	
4	**Condition of Surrounding Ground:**		
	Issue:	**Check for:**	**Record in detail for area at the rear of the spillway walls**
4.1	General Condition	Areas of erosion	
		Presence of standing water	
4.2	Cracks in the ground	Cracks between the spillway chute and the dam (immediately inform the owner if there are any issues with ground)	

	Issue:	Check for:	Record in detail for area at the rear of the spillway walls
4.3	Presence of plants	Large trees within 10 m of the chute (include distance, species)	
		Any associated damage to the chute	

Appendix C

Comparison of Depth Equations to Hydraulic Model Testing Data

Chanson (1994):

$$d_{90} = d_c \sqrt[3]{\frac{f_e}{8\,(1 - C_e)^3 \sin\alpha}}$$

Where: d_{90} = depth where the fluid is 90% air

d_c = critical depth

α = channel slope

C_e = air concentration (often estimated as 0.9 sin α)

f_e = Darcy friction factor for the air/water mixture

$$\frac{1}{\sqrt{f_e}} = 1.42 \ln\left(\frac{D_h}{k_s}\right) - 1.25$$

D_h = hydraulic mean depth, $D_h = \frac{4A}{P}$

A = cross-sectional area of the flow

P = wetted perimeter

Boes and Minor (2000):

$$d_{90} = 0.55\left(\frac{q^2 h}{g \sin\alpha}\right)^{0.25} \tanh\left(\frac{\sqrt{g\,h \sin\alpha}}{3\,q}\,[x - L_i]\right) + 0.42\left(\frac{q^{10}\,h^3}{[g \sin\alpha]^5}\right)^{\frac{1}{18}}$$

Where: $L_i = 9.72\,k_s\,F_*^{0.86}$

x = distance along the spillway from the crest

A. ANGLEZARKE

Converging approach channel leading to a series of small steps followed by a main cascade.

Approach Gradient = 2.9^{o}

Channel width = 12.1 m

Step height = not constant (0.9 m average)

Step length = not constant

Step slope angle = 34.4^{o}

Number of steps = 8

A length of L = 30 m has been used in Boes & Minor's equation, being the length from the first step to the top of the cascade.

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1 m	Boes L = 30 m	
Main cascade	40	0.5	1.0	Not recorded	0.597	1.133	
	60	0.75	0.75		0.740	1.381	
	81	0.95	1.0		0.869	1.453	

B. BELMONT

Converging approach channel leading to the cascade.

Approach Gradient = 1.5^{o}

Channel width = 7 m

Step height = 0.6 m

Step length = 0.883 m

Step slope angle = 34.2^{o}

Number of steps = 10

Length = 30 m is the length from the weir to the top of the cascade

<table>
<tr><th rowspan="2">Location</th><th rowspan="2">Flow rate (m^3/s)</th><th colspan="3">From model</th><th colspan="2">Calculated depth (m)</th><th rowspan="2">Notes</th></tr>
<tr><th>Depth min (m)</th><th>Depth max (m)</th><th>Velocity (m/s)</th><th>Chanson k_s = 0.1 m</th><th>Boes L = 30 m</th></tr>
<tr><td rowspan="4">Main cascade</td><td>9.5</td><td>0.23</td><td>0.25</td><td rowspan="4">Not recorded</td><td>0.383</td><td>0.652</td><td></td></tr>
<tr><td>19</td><td>0.25</td><td>0.6</td><td>0.545</td><td>0.938</td><td></td></tr>
<tr><td>38</td><td>0.3</td><td>0.8</td><td>0.788</td><td>1.230</td><td>1</td></tr>
<tr><td>57</td><td>0.55</td><td>1.25</td><td>0.984</td><td>0.957</td><td>1</td></tr>
</table>

Notes: 1 Out of channel flow

C. BEAVER DYKE

Stepped weir followed by 40 m of channel to the top of the stepped spillway.

Channel width = 3.58 m

Step height: top = 0.4 m, bottom = 0.37 m

Step length: top = 3.0 m (sloped), bottom = 5.0 m (sloped)

Step slope angle: top =9.3^{o}, bottom = 6.6^{o}

Number of steps: top = 10, bottom = 7

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson	Boes	
Top					k_s = 0.395 m	L = 40 m	
	4.25	0.3	0.4	Not recorded	0.61	0.774	
	8.5	0.9	0.9		0.795	1.094	
	17	0.6	1.2		1.129	0.792	
	25.5	0.8	1.4		1.479	0.401	
Bottom					k_s = 0.368 m	L = 70 m	
	4.25	0.3			0.6	0.832	
	8.5	0.6			0.81	1.197	
	17	1.2		7.7	1.15	1.529	
	25.5	1.2		9.3	1.51	1.151	
	34	1.2		9.5	1.82	0.787	

D. BLACKMOSS LOWER

Long approach channel with a stepped cascade on a bend in the channel.

Approach Gradient = 4.7^o

Channel width = 2.82 m

Step height = 0.18 m

Step length = 2.06 m

Step slope angle = 5^o

Number of steps = 11

If the full length of the approach channel is used with L = 118 m then the depths generated by Boes & Minor's equation are excessive (1.07 m @ 7 m^3/s rising to 3.37 m at 56 m^3/s), the issue, therefore, is what should be used as L? The calculations in the table below use half the approach length, L = 60 m.

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.179 m	Boes L = 60 m	
Main cascade	7	0.32		Not recorded	0.656	1.036	1
	14	0.8			0.941	0.640	
	28	0.85				0.384	
	42	0.9				0.384	
	56	0.9				0.423	

Notes: 1 Significant out of channel flow upstream of the cascade at flows above 7 m^3/s

E. HANGING LEES

Series of cascades of steps and sections of plane chute.

Channel width = 1.78 m

Step height = 0.305 m (12")

Step length = 0.765 m (30")

Step slope angle = 21.74^{o}

Location	Flow rate (m3/s)	From model			Calculated depth (m)		notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1 m	Boes	
Steps 1 17 steps	1.0	0.2		Not recorded	0.240	0.397	1
	1.3	0.2	0.4		0.271	0.455	
	2.2	0.2	0.4		0.351	0.548	
	3.0	0.2	0.4		0.410	0.448	
	3.6	0.35	0.45		0.451	0.334	
Steps 2 8 steps	1.3	0.2	0.4	Not recorded	As step 1	0.397	2
	2.2	0.2	0.4			0.455	
	3.0	0.2	0.4			0.600	
	3.6	0.35	0.45			0.704	
Steps 3 5 steps	1.3	0.2	0.4	Not recorded	As step 1	0.455	3
	2.2	0.2	0.4			0.600	
	3.0	0.2	0.4			0.706	
	3.6	0.35	0.45			0.777	
Step 4 8 steps	1.3	0.2	0.4	Not recorded	As step 1	As step 3	4
	2.2	0.2	0.4				
	3.0	0.2	0.4				
	3.6	0.35	0.45				
Step 5 3 steps	1.3	0.2	0.4	Not recorded	As step 1	As step 3	
	2.2	0.2	0.4				
	3.0	0.2	0.4				
	3.6	0.35	0.45				
Step 6 4 steps	1.3	0.2	0.4	Not recorded	As step 1	As step 3	
	2.2	0.2	0.4				
	3.0	0.2	0.4				
	3.6	0.35	0.45				
Step 7 7 steps	8	0.6	1.0	Not recorded	As step 1	As step 3	
	16	0.8	1.2				
	23	1.0	1.4				
	33	1.0	1.6				
Step 8 3 steps	8	0.6	1.0	Not recorded	As step 1	As step 3	
	16	0.8	1.2				
	23	1.0	1.4				
	33	1.0	1.6				

Notes: 1 L = 13.1m

2 Approach 5.97 m @ 0.9^{o}, L = 25.1 m

3 Approach 6.497 m @ 0.7^{o}, L = 35.4 m

4 Approach 5.308 m @ horizontal, L = 46.9 m

F. HURST

Concrete steps

Approach Gradient = 7^o

Channel width = variable for first 10 steps then 3.2 m

Step height = 0.55 m

Step length = 1.84 m

Step slope angle = 16.6^o

Number of steps = 23

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1 m	Boes L = 12.8 m	
Main cascade	10	0.45		Not recorded	0.529	0.103	
	20	0.75			0.769	-0.026	
	40	1.2			1.160	-0.039	
	60	1.4	1.8		1.520	-0.022	
	80	1.6	2.0		1.842	0.004	

Boes & Minor's equation has produced unexpected results since this is a stepped cascade similar to the one used in their test facility. The negative values occur because L_i becomes longer than the 12.8 m dimension to the step under consideration.

G. INGBIRCHWORTH

Continuous cascade of shallow steps on a long sweeping curve. The channel width converges down the spillway.

Maximum depths quoted are super-elevated.

Step height = 0.26 m

Step length = 5.06 m and 10.08 m (sloping tread)

Step slope angle = 6^{o}

Number of steps = 18

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.26 m	Boes	
Step 3		Width = 17.9 m				L = 15 m	
	10	0.125			0.351	0.522	
	20	0.25	0.4	4.4	0.458	0.320	
	30	0.3	0.5	5.9	0.545	0.131	
	40	0.45	0.63	7	0.619	0.072	
	50	0.63	0.7	7	0.686	0.049	
	60	0.7	0.7	7.7	0.747	0.040	
	70	0.7	0.8	7.7	0.803	0.036	
	80	0.7	1.0	7.7	0.857	0.036	
Step 11		Width = 14.4 m				L = 80 m	
	10	0.275				0.597	
	20	0	0.45	9.4	0.382	0.860	
	30	0.1	0.7	11.5	0.505	1.064	
	40	0.125	0.8	11.5	0.603	1.234	
	50	0.25	1.0	11.9	0.688	1.362	
	60	0.125	1.175	13.1	0.764	1.425	
	70	0.125	1.2	12.1	0.833	1.411	
	80	0.125	1.3	13.1	0.898	1.335	
Step 17		Width = 12.1 m				L = 140 m	
	10	0.175			0.41	0.655	
	20	0.25	0.35	8.6	0.547	0.952	
	30	0.4	0.4	9.1	0.656	1.166	
	40	0.45	0.45	9.4	0.751	1.357	
	50	0.55	0.6	10.4	0.835	1.526	
	60	0.625	0.55	11.3	0.913	1.675	
	70	0.8	0.7	11.3	0.985	1.802	
	80	0.8	0.7	13.7	1.071	1.899	

H. LONGWOOD COMPENSATION

Steeply sloping, converging approach channel leading to a cascade of irregular steps

Approach Gradient = 7.2^o

Channel width = 3.4 m

Step height = 0.87 m

Step length = 4.1 m

Step slope angle = 12^o

Number of steps = 7

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1 m	Boes	
Step 1				Not recorded		L = 40 m	
	8	0.4	0.8		0.460	1.223	
	16	0.6	1.0		0.664	1.591	
	24	0.8	1.6		0.829	1.062	
	33	1.0	1.6		0.991	0.525	
	40	1.3	2.0		1.125	0.328	
Step 2				Not recorded	As step 1	L = 44 m	
	8	0.6	0.7			1.223	
	16	0.8	1.1			1.677	
	24	0.9	1.1			1.348	
	33	1.2				0.736	
	40	1.5	1.6			0.473	
Step 3				Not recorded	As step 1	L = 50 m	
	8	0.6	1.0			1.223	
	16	1.0	1.4			1.732	
	24	0.8	2.0			1.699	
	33	1.0	2.0			1.098	
	40	1.4	2.2			0.740	
Step 7				Not recorded	As step 1	L = 70 m	
	8	0.6	1.0			1.223	
	16	0.8	1.2			1.759	
	23	1.0	1.4			2.130	
	33	1.0	1.6			2.146	
	40	1.4	1.8			1.838	

I. MIDHOPE

Plane approach chute with two cascades of steps in the chute

Approach Gradient = 4^{o}

Channel width = 8.88 m

Step height = 0.279 m (11”)

Step length = 0.762 m (30”)

Step slope angle = 20.1^{o}

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1 m	Boes	
Steps 1 7 steps						L = 93 m	1
	11.5	0.45		6.3	0.328	0.615	
	23	0.38	0.6	10.4	0.463	0.886	
	34.5	0.4	0.78	10.6	0.571	1.096	
	46	0.4	0.83	11.0	0.664	1.275	
Steps 2 12 steps					As Steps 1	L = 138 m	
	11.5	0.2	0.3	7.1		0.615	
	23	0.25	0.7	10.5		0.886	
	34.5	0.35	0.7	7.0/10.2		1.096	
	46	0.4	0.83	8.8/10.4		1.275	

Notes: 1 Curved approach channel

J. NEW YEARS BRIDGE

Continuous cascade of steps

Channel width = 2.36 m

Step height = 0.305 m (12”)

Step length: Top and step 5 = 1.22 m (48”), Main cascade = 0.813 m (22”)

Step slope angle: Top = 14^o, Main cascade = 20.6^o

Number of steps: Top = 23, Main cascade = 27

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1m	Boes	
Top				Not recorded	See step 5	L = 0 m	
	9	1.0				-0.056	
	18	1.9				-0.036	
	30	2.4	2.6			-0.001	
Step 5				Not recorded		L = 6.1m	
	9	0.7	0.9		0.605	-0.032	
	18	0.8	0.9		0.888	-0.016	
	30	1.4			1.223	0.017	
Mid cascade				Not recorded		L = 28.1m	
	9	0.55			0.61	0.728	
	18	0.85			0.896	0.266	
	30	1.2			1.236	0.167	
Bottom					As mid cascade	L = 50m	
	9	0.45		11.7		1.089	
	18	0.6		13.3		0.998	1
	30	0.9		13.8		0.543	1

Notes: 1 Some out of channel flow

Boes & Minor’s equation can not deal with x = 0 for the top of the cascade, other negative and depth decreasing results are unexpected since this is a continuous cascade of steps similar to that used in their test facility.

K. OGDEN (HASLINGDEN)

Sections of plane chute interspersed with cascades of steps.

Approach Gradient = 1.3^o

Channel width = 4.5 m

Step height = 0.406 m (16”)

Step length = 0.698 m

Step slope angle = 30.2^o

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1 m	Boes	
Steps 1 5 steps				Not recorded		L = 21 m	
	11.7	0.75	0.83		0.516	0.828	
	23.5	1.23	1.5		0.749	0.475	
	47	1.75	2.25		1.120	0.164	
Steps 2				Not recorded	As step 1	L = 44 m	
	11.7	0.45				0.869	
	23.7	0.75				1.226	
	47	0.88	1.13			0.969	
Steps 3				Not recorded	As step 1	L = 67.5 m	
	11.7	0.4				0.869	
	23.5	0.55	0.63			1.254	
	47	1.05				1.643	
Steps 4				Not recorded	As step 1	L = 91.5 m	
	11.7	0.4				0.869	
	23.5	0.75				1.254	
	47	1.0	1.25			1.786	
Steps 5				Not recorded	As step 1	L = 116 m	
	11.7	0.4				0.869	
	23.5	0.75				1.254	
	47	0.88	1.25			1.804	

L. OGDEN (HASLINGDEN) BY-WASH

Sections of plane chute interspersed with cascades of steps.

Approach Gradient = 3.1^{o}

Channel width = 5 m

Step height = 0.34 m

Step length = 0.875 m

Step slope angle = 21.2^{o}

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1m	Boes	
Steps 1 6 steps				Not recorded		L = 25m	1
	7.5	0.15	0.45		0.360	0.710	
	15	0.35	1.2		0.512	0.888	
	22.5	0.25	1.75		0.634	0.593	
	30	0.25	2.0		0.740	0.352	
Steps 2 7 steps				Not recorded	As step 1	L = 43m	
	7.5	0.1	0.95			0.710	
	15	0.25	1.5			1.021	
	22.5	0.25	2.5			1.213	
	30	0.25	2.5			1.182	
Steps 3 4 steps				Not recorded	As step 1	L = 62m	
	7.5	0.5	0.15			0.710	
	15	0.2	0.75			1.022	
	22.5	0.3	1.4			1.263	
	30	0.35	1.5			1.444	
Steps 4 4 steps				Not recorded	As step 1	L = 79m	
	7.5	0.25	0.6			0.710	
	15	0.25	1.0			1.022	
	22.5	0.5	1.3			1.265	
	30	0.5	1.5			1.469	
Steps 5 4 steps				Not recorded	As step 1	L = 96m	
	7.5	0.2	0.6			0.710	
	15	0.5	1.0			1.022	
	22.5	0.5	1.2			1.265	
	30	0.6	1.0			1.471	
Steps 6 4 steps				Not recorded	As step 1	L = 113m	
	7.5	0.5				0.710	
	15	1.13				1.022	
	22.5	1.2				1.265	
	30	1.5				1.471	

Notes: 1 Cascade on a bend

M. RIDING WOOD

Sections of plane chute interspersed with cascades of steps.

Approach Gradient = 0.98^o

Channel width varies

Step height = 0.3 m

Step length = 0.2 m

Step slope angle = 56.3^o

On other cascades at Riding Wood, flow jumped all of the steps.

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1 m	Boes	
Steps 1 Width 3.8m						L = 24.6 m	1
	15	0.72	1.0		1.096	0.869	
	25	1.2	1.4	8.4	1.539	0.923	
	35	1.3	1.65	8.6	1.925	0.808	
	41	1.2	1.8	9.5	2.140	0.743	
Steps 2 Width 3.3m						L = 33.5 m	
	15	0.5	1.0	5.2	1.220	0.952	
	25	0.9	1.5		1.715	1.119	
	35	1.1	1.7		2.146	1.066	
	41	1.2	1.8		2.385	1.001	
Steps 3 Width 2.7m						L = 42.4 m	
	15	0.8	0.9	6.3	1.428	1.060	
	25	1.1	1.3	9.3	2.007	1.272	
	35	1.6	1.7	9.8	2.511	1.247	
	41	1.6	2.2	10.3	2.791	1.186	
Steps 4 Width 2.4m						L = 51.6 m	
	15	0.8	0.9		1.568	1.130	
	25	1.3	1.4		2.205	1.396	
	35	1.3	1.4		2.788	1.425	
	41	1.8	2.0		3.066	1.379	
Steps 5 Width 2.4m					As step 4	L = 64.5 m	
	15	0.7	1.1			1.133	
	25	1.2	1.4	10.1		1.458	
	35	1.4	1.8	10.2		1.615	
	41	1.4	2.0	10.8		1.632	

Notes: 1 Cascade on a bend

Poor correlation to Chanson's equation with k_s = 0.1 m.

N. SCARGILL

Series of sections of plane channel interspersed with cascades of steps, channel snakes in plan, leading to a straight cascade of steps. Some super-elevation on the bends in the channel.

Channel width = 7.15 m

Step height = 0.29 m

Step length = 0.914 m (36")

Step slope angle = 17.6^{o}

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1 m	Boes	
Steps 1 3 steps						L = 40 m	
	4	0.2			0.222	0.413	
	8	0.3			0.306	0.594	
	16	0.55			0.429	0.854	
	25	0.7	0.85	5.9	0.540	1.013	
	32.5	0.85	0.95		0.619	0.957	
Step 3 3 steps					As step 1	L = 55 m	
	4	0.1	0.32			0.413	
	8	0.3				0.594	
	16	0.5				0.855	
	25	0.4	1.1	5.9		1.076	
	32.5	0.6	1.3			1.196	
Step 5 3 steps					As step 1	L = 70 m	
	4	Dry	0.35			0.413	
	8	0.25	0.5			0.594	
	16	0.3	0.9			0.855	
	25	0.35	1.2			1.081	
	32.5	0.6	1.3			1.235	
Step 8 5 steps					As step 1	L = 91 m	
	4	0.1	0.32			0.413	
	8	0.2	0.45			0.594	
	16	0.3	1.2			0.855	
	25	0.3	1.1			1.081	
	32.5	0.8	1.6			1.241	
Main cascade Top 38 steps	4	0.2	0.3		As step 1	L = 128 m	
	8	0.35	0.4			0.413	
	16	0.5	0.8	5.4/ 7.4		0.594	
	25	0.5	1.1	5.8/ 8.5		0.855	
	32.5	0.8	1.4	7.1/ 9.7		1.081	
	4	0.2	0.3			1.242	

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1m	Boes	
Main cascade Bottom					As step 1	L = 160 m	
	4	0.2				0.413	
	8	0.3				0.594	
	16	0.5		6.6		0.855	
	25	0.5		8		1.081	
	32.5	0.6		10.3		1.242	

O. UPPER RIVINGTON

Converging channel leading to a straight section containing sections of plane chute and individual steps followed by a cascade of irregular steps.

Channel width = 10.96 m

Step height = 0.31 m

Step length = 8.8 m

Step slope angle = 2^o

Number of steps = 4

Steps 5 is a set of 6 steps of variable height and tread.

Total fall = 6.39 m

Length = 10.7 m

Angle = 30.8^o

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1m	Boes	
Step 1				Not recorded		L = 40 m	
	7	0.2	0.4		0.353	0.791	
	15	0.6			0.518	1.011	
	30	0.8	0.9		0.746	0.437	
	45	1.2			0.930	0.258	
	60	1.36	1.44		1.110	0.211	
Step 2				Not recorded	As step 1	L = 44 m	
	7	0.2				0.791	
	15	0.3	0.4			1.085	
	30	0.58	0.65			0.560	
	45	0.9				0.321	
	60	0.96	1.0			0.252	
Step 3				Not recorded	As step 1	L = 50 m	
	7	0.2				0.792	
	15	0.3				1.143	
	30	0.56				0.764	
	45	0.8				0.432	
	60	0.96	1.0			0.323	
Step 4				Not recorded	As step 1	L = 70 m	
	7	0.1	0.2			0.792	
	15	0.25	0.35			1.180	
	30	0.3	0.6			1.371	
	45	0.6	0.8			0.937	
	60	0.8	1.0			0.654	

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson $k_s = 0.1m$	Boes	
Steps 5 6 steps				Not recorded		L = 70 m	1
	7	0.05	0.35		0.283	0.510	
	15	0.2	0.45		0.409	0.759	
	30	0.3	0.4		0.584	1.092	
	45	0.5	0.8		0.723	1.350	
	60	0.96	1.0		0.845	1.570	

Notes: 1 Parameters changed to the steep step data in Boes & Minor's equation.

P. UPPER RODDLESWORTH

Series of sections of plane channel interspersed with long cascades of steps, channel snakes in plan causing some super-elevation.

Approach Gradient = 0.5^o

Channel width = 15 m

Step height = 0.225 m

Step length = 0.455 m

Step slope angle = 26.3^o

Location	Flow rate (m^3/s)	From model			Calculated depth (m)		Notes
		Depth min (m)	Depth max (m)	Velocity (m/s)	Chanson k_s = 0.1 m	Boes	
Steps 1 9 steps				Not recorded		L = 50 m	
	16	0.25	0.55		0.309	0.497	
	33	0.55	0.8		0.440	0.727	
	42	0.7	1.0		0.497	0.824	
	66	0.7	1.6		0.628	1.019	
Steps 2 8 steps				Not recorded	As step 1	L = 63 m	
	16	0.1	0.4			0.497	
	33	0.4	0.7			0.727	
	42	0.4	0.8			0.825	
	66	0.55	0.9			1.042	
Steps 3 8 steps				Not recorded	As step 1	L = 78 m	
	16	0.05	0.5			0.497	
	33	0.1	0.85			0.727	
	42	0.15	1.1			0.825	
	66	0.3	1.0			1.046	
Steps 4 16 steps				Not recorded	As step 1	L = 94 m	1
	16	0.05	0.5			As step 3	
	33	0.1	0.55				
	42	0.1	1.2				
	66	0.2	1.2				
Steps 5 18 steps				Not recorded	As step 1	L = 115m	1
	16	0.2	0.35			As step 3	
	33	0.35	0.5				
	42	0.6	1.8				
	66	0.6	0.8				
Steps 6 44 steps				Not recorded	As step 1	L = 140m	1
	16	0.18	0.22			As step 3	
	33	1.0					
	42	1.2					
	66	1.8					

Notes: 1 Flow jumped steps

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