

Lenthall Dam Crest Gates – from agony to relief

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GHD

The automatic operating buoyancy type spillway gates at Lenthall Dam did not operate properly since installation. This paper discusses the problems encountered, the investigation conducted using computational fluid dynamics to quantify the problems and develop solutions. It describes the design of the modifications to the gate and flow regime and results after construction.

Keywords: *Spillway gates, CFD analysis, uplift forces, hydrodynamic forces, flow modification, gate locking mechanism.*

Introduction

Lenthall Dam was built in 1984 to augment water supply to the larger Hervey Bay area. It used an uncontrolled ogee spillway with full supply level at RL 24.0 to discharge floods. In 2008 the dam was raised by 2m with the addition of 5 crest gates on the spillway. The crest gates were an innovative design provided by a South African company with an existing installation on Tswase weir in that country. The gates were selected by the client because they operated automatically without the need for external power.

The four outer gates were each 15m wide and 2 m high with the centre gate only 10m wide. The gate is a reverse sector gate with its body forming a tank with hollow arms connected to an upstream trunnion. The hollow trunnion in turn connects to a hollow support structure. Each gate is individually connected through a series of 150mm pipes to an intake weir. When the water level in the dam overtops the intake weir, water starts flowing into the gate body via the pipe network and hollow support structure. Water however also continually discharges from the gate body through two orifice plates, one on each side of the gate. When the inflowing water exceeds the discharge, the gate body fills. As the gate fills it overcomes its buoyancy and starts to open (lower). This allows water to discharge over the spillway. When the upstream water level recedes below the intake weir, the discharge from the gate drains the gate until it becomes buoyant enough to close (raise). The intakes can be set at different levels to allow sequential opening of the gates. The gates could also be manually lowered by opening valves installed below the intake weirs. The gates could therefore be manually operated with the dam below full supply level. In case the intake weirs get blocked, emergency intakes to the gates have been provided on top of the piers adjacent to the gates. At a certain overflow level water would enter these intakes bypassing the inlet weirs.

At Lenthall dam the smaller centre gate will open first when the dam reaches a water level of RL 26.05m, or 50mm above full supply level. The intakes for the larger gates are set at 50mm increments from there with the last gate opening when the dam level reaches RL 26.25m. With slight variation, the gates have sufficient buoyancy to remain raised as the water in the dam drops below full supply level. Once the dam level reaches approximately RL 24.5m, the gates will start to lower with the water level. The side seals of the gates remain in contact over their full travel range. The sill seal plates extend 500mm below the bottom of the gates. The gates can therefore lower by 500mm with the sill seal still in full contact with the sealing plate. As the gates lower therefore with receding dam level between RL 24.5m and the crest level of RL 24.0m, water is never discharged allowing maximum retention of water in the dam during dry spells.

Initial problems with the gate operation were experienced due to the stiff sill seal and excessive amount of pre-compression provided during installation of the gates. The double stem sill seal design was not flexible enough under the high pre-compression and even though it had a Teflon inlay along the bulb, the friction force was too high to be overcome by the mass of the filled gate. This problem was resolved by cutting off the bottom stem and rounding the seal support. The gates were successfully commissioned by manual operation of the intake valves.

The gates however operated haphazardly during floods. They would open once successfully and then not again. This was eventually traced to a low point in the air intake and release pipe which is routed through the top arm of the gate and up the top support arm. When the gate opens fully and remains open long enough for the tank to fill fully, water also enters the air pipe up to the low point at the trunnion, effectively blocking the gate vent pipe. The gate would still open, as air would enter the tank through the emergency water intakes in the piers as it drains through the orifice plates. On subsequent filling though, the gates would only fill to a level where the air pressure in the gate exceeds the incoming water pressure. It cannot vent through the emergency fill pipes as when draining because the water fills through the same pipe. The air lock therefore prevents the gates filling and they remain in the raised position. It was initially thought that condensation was responsible for the blocked vent pipe and the operators would use a compressor to clear the vents. This could however only be done for the end gates due to access restrictions.

Since their installation the gates were overtopped without opening during a number of floods. When in December 2010 the raised gates were overtopped by 1.7m and shortly after by 2.2m, the dam safety regulator demanded immediate action. It also attracted negative publicity due to complaints from upstream landowners becoming inundated during smaller than expected floods.

Wide Bay Water Corporation (WBWC) who owned the dam then requested GHD to provide solutions. After careful review of the gate operating behaviour and review of the design, temporary air intakes were installed directly on the gates. This consisted of a 2m snorkel attached to a hole drilled in the gate tank access cover. The snorkels were vulnerable to damage by debris and were only a temporary fix while a more permanent solution could be prepared. An added advantage of the snorkels was that they could be used to monitor the gate movement because they protruded above the overflowing water.

The gates now operated consistently, but it was noted that although the gates initially open fully, they start to lift again as the flood height increased. For a flood height of RL 26.5m the gates would raise by about 500mm. This could only be ascribed to hydrodynamic effects caused by the overflowing water. It was expected that with higher flood levels the uplift effect would increase, and the probable maximum precipitation design flood (PMPDF) was determined as RL 34.57 m.

It became clear that the gate design provided by the South African company was inherently flawed and not suitable for this type of application. The problem with the vent pipe was insignificant compared to this much more basic design flaw. In September 2011 GHD undertook to quantify the problem and provide a solution. By March 2012 the initial investigation was complete and concept solutions presented. From these a preferred option was selected and the detail design commenced. The gates were finally re-commissioned in May 2016 after successful construction of the required modifications.

This paper describes the investigation and design methods and challenges encountered and preparation of the final solution. It also briefly describes the construction method and some of the problems encountered.

Investigation

Analysis of Forces

Three analyses were conducted; Flood performance, Gate opening & Gate closing analysis. Each of these analyses required different combinations of loads, as discussed in following sub sections.

Taking the level of the centroid of the trunnion as the origin, the relative angular displacement of the gate is described as follows:

- For the fully open position, the angular displacement is -1.2 degrees
 - For the fully closed position, the angular displacement is 14.8 degrees
- The full angular travel by the gate is therefore 16.0 degrees. For simplicity in hydrodynamic modelling and reporting, the angular positions have been defined as:
- Fully open is 0 degrees
 - Fully closed is 16 degrees
- Within the calculations the actual angular displacements have been used. This was necessary to capture any opening and closing effects generated by purely horizontal or vertical forces.

Flood Performance Analysis

This analysis was performed to determine the net force on the gate when it was in the fully open position for different reservoir flood levels. The gate was considered to be stationary and in the fully open position. Lintel seal friction, trunnion bearing friction and side seal friction were thus not applicable.

The only forces used in the flood performance analysis were the gate weight, weight of water in the variable buoyancy tank when completely full and hydrodynamic forces acting on the gate body. The masses of a 15 m gate and a 10 m gate including the pivot arms were calculated from the For Construction drawings as 114 kN and 79 kN respectively. The total volume of water contained in the variable buoyancy tanks were calculated using AutoCAD at 13.6 m³ for the 15 m gate and 8.6 m³ for the 10 m gate.

The range of reservoir levels relevant to the gates is from full supply level of RL 26.0 m to the probable maximum precipitation design flood (PMPDF) of RL 34.57 m.

Gate Opening and Closing Analyses

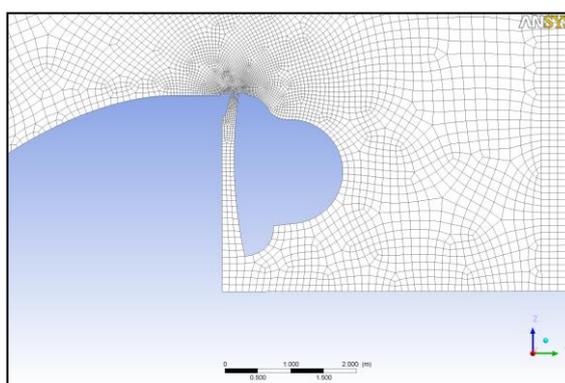


Figure 1 -Two-dimensional model.

The aim of these analyses was to track the behaviour of the gate through the opening and closing operations. Due to constraints of the hydrodynamic analysis gate angles above 12 degrees were not modelled. This was not considered to be a limiting issue because the gates behaved as expected at these upper angles.

The forces applicable in these analyses were the gate weight, the weight of water in the variable buoyancy tank, the hydrodynamic forces acting on the gate body and trunnion and side seal friction at various gate angles.

As the gate rotates the centroid of the gate mass and water volume changes. The centroid for these two cases was calculated at 200mm increments over the gate opening range

and the average determined. The average value was then adopted for the analysis

The hydrodynamic forces on the gates were estimated using computational fluid dynamics (CFD) modelling. The trunnion and side seal friction forces were calculated for each position of the gates taking into consideration the change in sealing length and pressure differential.

Trunnion friction occurs between the stainless steel shaft and the vesconite bearing as the trunnion rotates. A coefficient of friction of 0.2 was adopted. The trunnion friction forces act tangentially, and only during movement, when they act to oppose this movement.

The normal force applied to the side seal varies depending on the position of the gate. When fully closed, and the gate is retaining water, the pressure varies hydrostatically from the top to the bottom of the seal. When the gate is submerged, pressure is equalised between the upstream and downstream sides of the seal, so friction forces are negligible. Intermediate values will apply in intermediate gate positions, but as this is very difficult to quantify, a conservative assumption has been made and the full value of pressure applied for all gate positions. The side seal friction forces act tangentially, and only during movement, when they act to oppose this movement.

CFD Analysis

Two-Dimensional Modelling

The initial CFD modelling undertaken for the gates involved two-dimensional analysis of the existing gate geometry. The two-dimensional model comprised a typical 'slice' of the gate and crest cross section, as shown in [Figure 1](#) (the numerical mesh is also shown in this figure). The advantages of two-dimensional modelling are that the geometry requirements are small (meaning a simulation can be established quickly), and that computational requirements are low (allowing a large number of flow scenarios to be evaluated). Two-dimensional models do not include the effects of barriers to flow over the crest (such as piers or gate trunnions), but do allow the fundamental hydrodynamic process to be evaluated rapidly. Two dimensional modelling was not undertaken for the highest reservoir levels as three dimensional

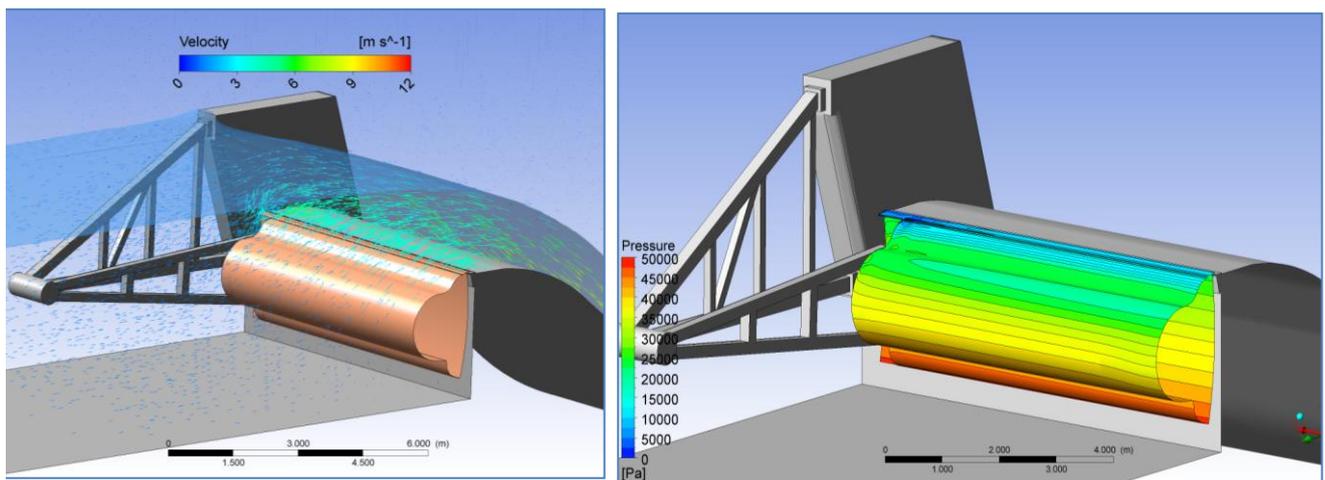


Figure 2 Three-dimensional velocity and gate pressure results.

effects such as flow curvature in the approach channel, superelevation and wake formation were expected to be significant and would not be captured in the two dimensional analysis.

The two-dimensional models were run by specifying the upstream water level and assuming free discharge from the spillway. The crest was modelled at a range of upstream water levels and at a range of gate closure angles (as well as fully open). The key results from the two-dimensional modelling were: the discharge coefficient, force on the gate and moment generated by the gate (about the trunnion pivot point).

The two-dimensional modelling demonstrated two important processes: firstly, that the gate encountered hydrodynamic lift when the crest was operating, and this lift increased with reservoir levels and flows. Secondly, this lift decreased as the gate closed. These results were compatible with observations that the gates close initially, then rise to an equilibrium position that is flow dependent.

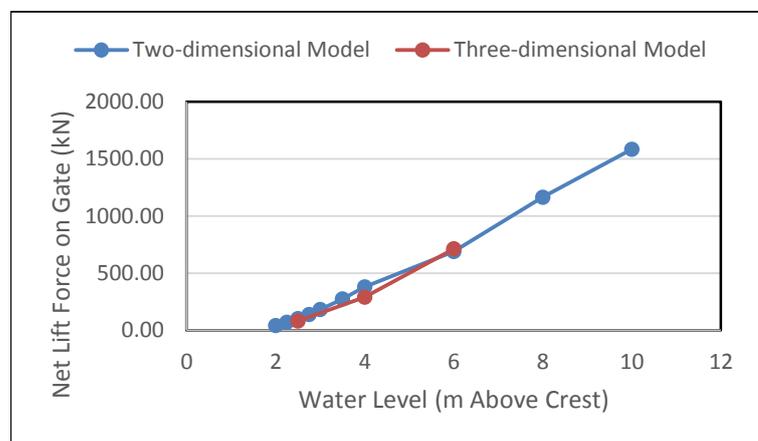


Figure 3 Lift force results for gate in fully open position.

Three-Dimensional Modelling

The two-dimensional modelling undertaken was effective in demonstrating the fundamental hydrodynamic cause of gate lifting. It did not, however, include the full and complex shape of the gates and therefore could not reliably be used in determining the stability of the gates. For the next stage of CFD modelling a three-dimensional model of a section of the gate and spillway including the gate, piers, and trunnions was created. In order to minimise the size of the model, symmetry boundary conditions were employed at the centre of the piers and gate, reflecting the observation that the gates can be approximated as a series of this repeating unit. The mesh in three dimensions (2 million cells) is much larger than that used in the two-dimensional cases. As with the two-dimensional modelling, a range of water levels and gate opening angles were evaluated using the three-dimensional model.

Figure 2 shows velocity vector and pressure results for a simulation where the upstream water level was at RL 26.5 m. The horizontal deflection of flow around the piers and trunnions can be seen, as well as the approximately two-dimensional flow over most of the length of the gate. The three-dimensional flow around the gate does create some variability in surface pressures on the gate. These factors account for the difference between the two-dimensional and three dimensional gate results.

Figure 3 shows example data output from the two and three dimensional modelling. The lift forces from the two-dimensional and three-dimensional modelling is standardised per 15 m gate, and is adjusted (using the calculated gate mass) to give net vertical (lift) force. Remarkably, when reservoir levels are 10 m above the crest, the lift force is more than four times the weight of the gate.

Validation of the Numerical Model

Data was provided by WBWC regarding the observed positions of gates 1, 2, 3 and 4 during a real flood event (the height of the gate snorkels were used as a gauge). These observed positions are summarised in **Error! Reference source not found, Table 1**.

This gate position data was compared to the equilibrium position data calculated using the CFD results (Figure 4). The CFD results indicate the gate will reach stability when closed around 5.8 degrees. This closure corresponds to a level of 690 mm above the crest. The result falls within the range observed.

Large Scale Simulation

In order to determine the significance of three-dimensional effects in the spillway approach channel, the entire spillway, approach channel, and a portion of the reservoir was simulated in a small set of extremely large simulations (Figure 5). The results of these simulations were used to evaluate the performance of the spillway at very high reservoir levels (where the two-dimensional or small scale three-dimensional results do not apply).

Modification Concept Design Process

The aim of the modification concept design process was to identify a method for securing the gates during floods. The hydrodynamic uplift forces were considered too large to be resisted by stand-alone solutions such as ballasting the gate tanks or the provision of lock-down devices. Changes to reduce the lift encountered by the gate were therefore required as part of the design, along with mechanical locking devices. CFD modelling was used to evaluate the performance of the design options.

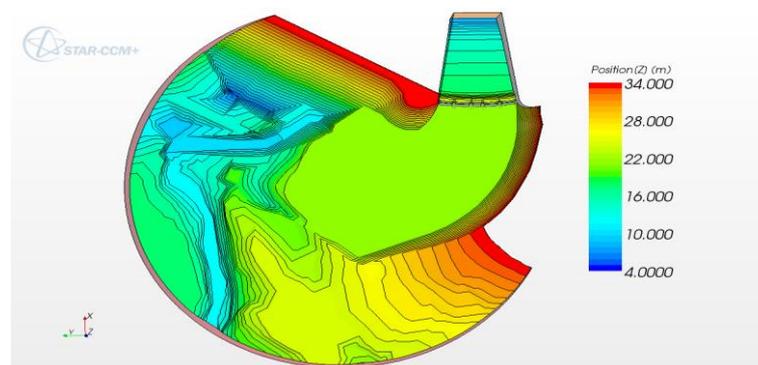


Figure 5 Full three dimensional model domain (plan view)

Table 1 Observed location of neutral buoyancy

	Stable level above crest, reservoir at FSL (mm)
1	550
2	800
3	300
4	500

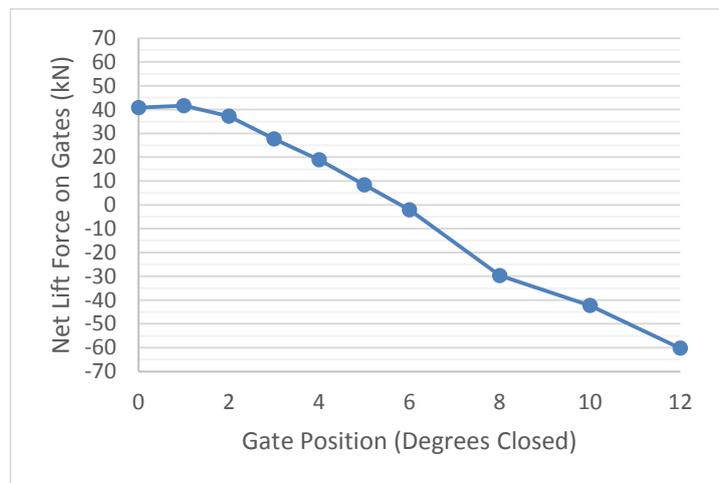


Figure 4 Lift force results at FSL for a range of gate angles.

Hydrodynamic Modifications

The first options investigated to reduce the lift force were spoilers (of various sizes) attached to the trailing edge of the gate. These had limited success and were becoming so large that they impacted on the spillway discharge coefficient. It would also be impossible to make them structurally strong enough to transmit the forces induced. Some spoiler configurations (on the upstream edge only) increased lift forces. Tests were also done

with lowering the gate below the spillway crest by up to 500 mm but without any significant success. More drastic measures were investigated and involved the use of a free standing deflector wall upstream of the gate. The aim of this wall was to create a flow shadow downstream of the wall, providing a relatively still body of water immediately above the gate. This principle was very successful. By experimenting with the height and location of the wall it was found that the optimum location was 2.5 m upstream and 500 mm below the spillway crest. Following the successful two-dimensional model results, three-dimensional modelling of the upstream wall solution was conducted to validate the previous analyses and include the impacts of any three-dimensional effects. The initial 3D results were unexpected and disappointing with the three dimensional modelling indicating a significant decrease in the benefits of the upstream wall. Investigation of the modelling results indicated that significant amounts of flow was being drawn around the sides of the deflector wall, negating its effect.

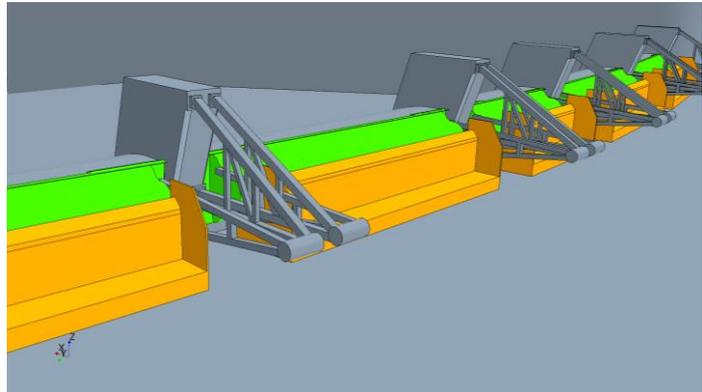


Figure 6 Concept modifications (deflectors in orange, gate bodies in green).

A number of options were evaluated to enhance the deflector walls. Ultimately, a wing (inspired by the front spoiler on race cars) on each end of the wall was found to prevent much of the flow short circuiting. These wings, as well as changes to the gate seal plates, were found to have a dramatic impact on lift forces. The lift forces for the modification concept were modelled using the full three-dimensional version of the model [Figure 6](#). The results ([Figure 7](#)) indicated that at moderate flow levels there was no net lift on the gates when in the open position. Lift is still generated under very high water levels however these are strongly mitigated compared to the original design. Lift was higher for the smaller gate 3 due to the increased relative significance of deflector edge effects.

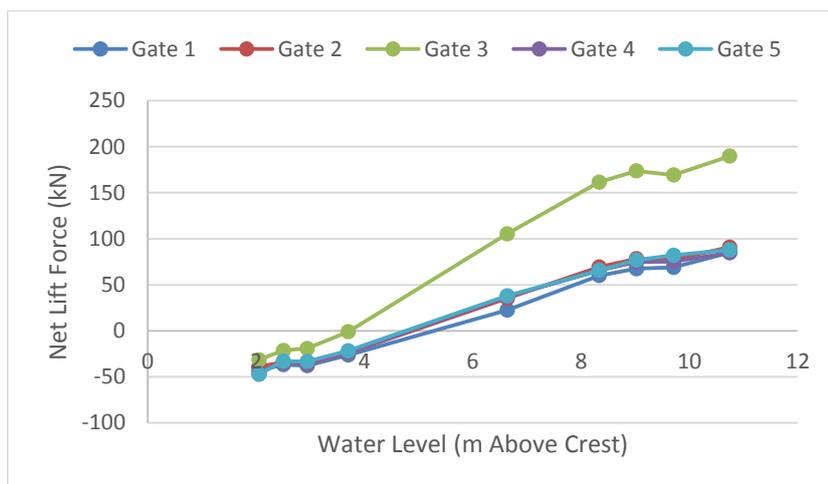


Figure 7 Concept modifications lift results.

Scale Modelling

CFD modelling has been used extensively in the analysis of the Lenthall Dam spillway gates under current conditions and in developing the preferred remedial design concept. Scale modelling of a section of the crest was undertaken for validation of the CFD modelling. A CFD study replicated the scale model geometry and modelled the laboratory configuration as closely as possible. The results of the CFD

modelling closely matched those of the scale modelling in all areas evaluated, including:

- The spillway discharge capacity,
- The water surface profile over the crest,
- Hydrodynamic lift on the gate generated by the flow ([Figure 8](#)), and
- Pressure on the spillway crest.

Although the total lift on the gate was modelled with reasonable accuracy, there were differences observed in pressures in the zone around the gate. It is likely that slight differences in the geometry of scale model and the CFD model were at least partially responsible for the discrepancies. In balance the CFD model was considered a valid design tool for this geometry where gate lift was the key design parameter.

Mechanical Locking Devices

The use of mechanical locking devices to keep the gate in the lowered position during all flood levels was also investigated. It soon became evident that attempting to physically lock the gate in all flood conditions would result in damage to the gate body or locking devices. The gate body had not been designed to resist this type of loading and extensive modifications would be required. The use of mechanical locking devices in isolation was thus not a viable solution.

Providing a mechanical locking device in combination with the flow modifiers discussed above seemed to provide the optimum solution. The aim for the final solution therefore was to limit the uplift forces enough to make a dogging device feasible.

Final Concept

The final concept for the modification consists of a suite of measures, including:

- Precast reinforced concrete upstream deflector wall, with the top of the 300 mm thick wall stem at RL 23.5 m
- Steel wingplates each end of the wall which extend upstream as well as downstream
- A 90 mm gap between the face plate of the gate and the concrete spillway crest, by removing the central lintel seal cover plates and cutting back the corresponding gate top lip. A 2.4 m length each end of the gate, corresponding to one cover plate unit, would remain in place
- A mechanical dogging device to positively lock down the gate during higher flood flows.

Detail Design

Deflector Walls

The preliminary design of the deflector wall comprised the following elements:

Concrete grade 50 MPa, required cover to reinforcement 50 mm

- Geometry as described in Figure 10
- Base reinforced with N28 bars at 200 mm centres on the upper face, stem reinforced with N28 bars at 200 mm centres on the upstream face, all other reinforcement N16 bars at 200 mm centres
- 40 mm diameter DSI double corrosion protected reinforcement grade rock bolts, stressed to 60% of UTS, installed 500 mm from upstream edge of base at 1 m centres. Length of rock bolt 12,200 mm into rock beneath base of wall

During detail design an unstressed alternative to the proposed stressed bar anchor was assessed. An unstressed anchor would have significant advantage from a maintenance perspective, as restressing operations would need to be conducted under water in potentially low visibility conditions. A concern however was the profile of the rock excavation from the original construction of the ogee seen in [Figure 11](#). The downstream toe of the deflector wall may be very close to a vertical cut face. This would significantly reduce the ability of the downstream toe foundation to resist high vertical stresses without deformation. Considering all of the above, the original stressed solution was recommended.

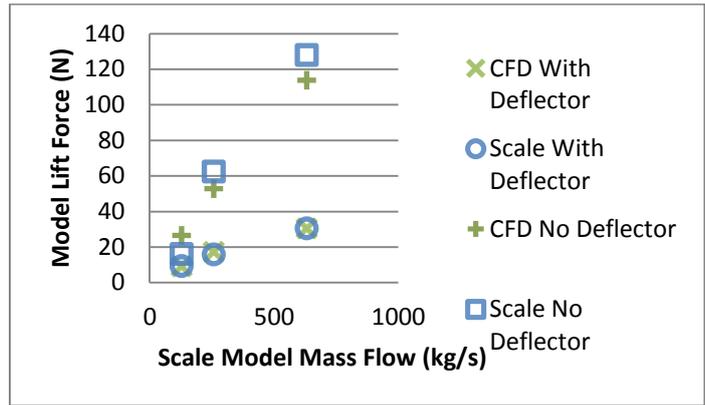


Figure 8 Scale model to CFD comparison.

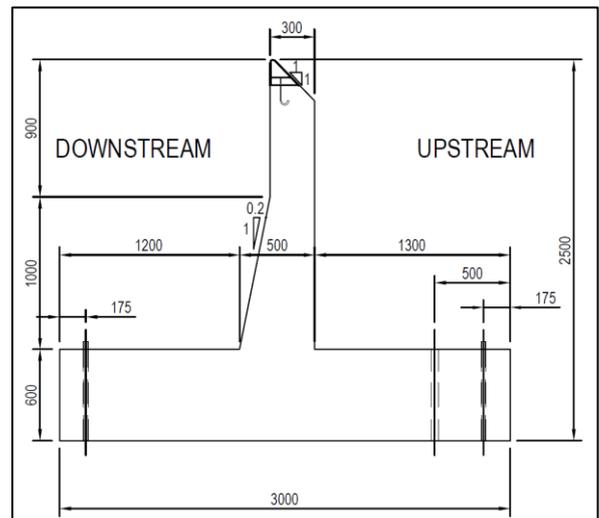


Figure 9 Geometry of deflector wall following preliminary design



Figure 10 Original ogee construction, showing rock excavation

Additionally, the existing ogee has been anchored down with 11 strand anchors. Checking the overall geometry, a significant potential existed for the proposed anchor bars to clash with the existing anchors. Therefore some iteration on the proposed base / rock bolt geometry was done during detailed design.

Input forces

Plots of vertical and horizontal forces were generated using the output of the CFD large spillway model for all five walls. The uplift force used in the design of the deflector wall was determined using graphical extrapolation of the force data, however sufficient lateral force data was available for a direct determination. As can be seen in [Figure 12](#) the upstream deflector wall behind Gate 5 appears to be experiencing the highest uplift (positive is upward, thus the smaller the force magnitude, the larger is the hydrodynamic uplift offsetting the hydrostatic component). The CFD modelling also predicted significant force fluctuations acting on the Gate 5 wall, which have

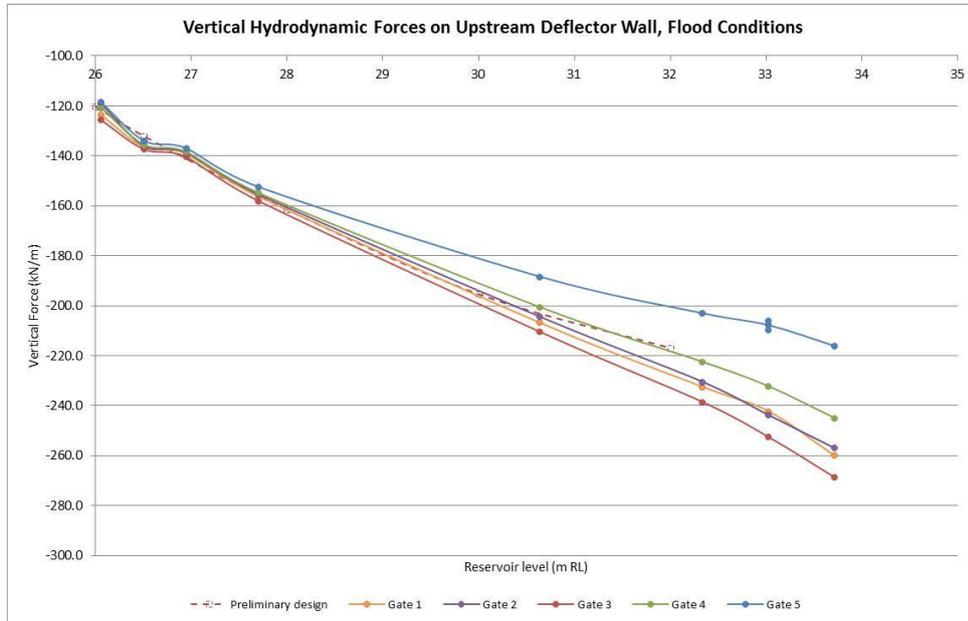


Figure 1142 Vertical hydrodynamic forces on upstream deflector walls

been shown on the plot. A value of 220kN/m was thus adopted for the vertical design load of the upstream deflector walls.

The fluctuations observed in [Figure 12](#) acting on the Gate 5 wall also impacted the horizontal hydrodynamic force. A conservative horizontal force of 140kN/m acting downstream was adopted for detailed design.

Uplift pressure was calculated as linearly varying hydrostatic

pressure between the upstream edge and the downstream edge of the wall base with water levels provided by the CFD modelling.

A value for hydrodynamic torque was also output from the CFD model, with centre of rotation about the upstream corner. This was performed for the final design iteration to remove any assumptions regarding the line of action of the horizontal and vertical hydrodynamic forces for the overturning assessment.

Analysis

Force and moment balance calculations were performed to determine the net force and the net moment in the system. The forces and lines of action used are as defined in Table 2.

Stressed system analysis

The analysis of the stressed system was based on a rigid body stability method similar to that used for gravity dam assessment. Output was focussed on the upstream and downstream contact stresses with the active prestress of the bar anchor supplying sufficient downward force to counter the net uplift and overturning of the deflector wall under high flood loading. A stainless steel system was used comprising Macalloy S1030 grade stressbar.

The anchor spacing was limited by the ability of the reinforced concrete wall base to laterally transfer the prestress loading across the section width. Sufficient structural depth was not available to construct a lateral stressing beam and a spacing increase to 1.33 m maximum centres was used. An L shaped cross section for the upstream wall was investigated to make allowance for the vertical cut face of the approach channel floor and the location of the ogee anchors. An acceptable solution was obtained with a new base slab length of

Table 2 Forces and lines of action, deflector walls

Force	Line of Action
Non buoyant self-weight	Mass centroid of wall
Hydrostatic and hydrodynamic vertical force along top surfaces of wall (combined into single value as output from ANSYS CFD modelling)	Taken as acting through centre of the top of stem prior to direct output of torque for final design iteration
Hydrostatic uplift beneath wall assuming base of wall is not sealed onto foundation	Centroid of trapezoidal uplift distribution
Hydrodynamic horizontal force	Taken as 250 mm below the top edge of the wall stem prior to direct output of torque for final design iteration for stability
Active rock bolt force (for stressed option)	500 mm from upstream edge

2500 mm and the anchors placed 500 mm from the upstream edge. Using N32 Macalloy bar spaced at 1.33 m maximum centres and stressed to 50% of 0.1% proof stress initially, a 20% **loss** of prestress in the anchors can be tolerated while still providing fully prestressed interface behaviour.

A total anchor length into the rock of 9,000 mm was calculated with a bond length of 4215mm using a rock density of 2150 kN/m³, a wedge angle of 90 degrees, 120 mm hole diameter and grout to rock bond of 1 MPa.

Proposed arrangement

The following final arrangement was implemented:

- N32 Macalloy S1030 stainless steel bar anchors, stressed to 355 kN, installed 500 mm from upstream edge of base at 1.3 m centres. Length of rock bolt 9,000 mm into rock beneath base of wall
- L shaped wall, stem as previously, with base same length but displaced upstream to avoid both the excavated area and the area of low hydrodynamic pressure behind the wall
- System does not need to be re-stressable. The design has been performed to provide full stressed system behaviour with up to 20% losses, but even if full loss of prestress occurred, which is considered unlikely, acceptable performance is still predicted

Steel Wing Plates

The CFD modelling has indicated that the attachment of end plates to the concrete deflector walls provided significant improvements to gate lift reduction. The end plates reduce the 3D flow effects around the ends of the deflector walls, similar to the winglets on the ends of race car front wings – thus term wing plates. To reduce the drag effect of the wing plates, it was decided to construct them from steel plate rather than from concrete. The wing plates were manufactured from grade 304 stainless steel to reduce maintenance requirements as they would be permanently submerged and difficult to inspect or replace. This was in line with the original reasoning used for the gate support structure.

Stiffening of the wing plates was done by bending them into a corrugated pattern refer to [Figure 13](#)~~Figure 13~~. The wing plates would be connected to the concrete T-wall using stainless steel adhesive anchors. The stress, deflection and anchor loads were determined using ABACUS FEA programme with the pressure distribution across the wing plates provided from the CFD analysis.

Locking Mechanism

The design criteria developed for the locking mechanism were:

1. Retain the present gate operating system
2. Assist to fully lower and lock the gate when gate lowering was initiated and gate stops moving without being fully lowered and water level is above gate intake.
3. Able to release the gate against buoyant gate force
4. Minimise use of electrical sensors at the gate
5. Have a low profile projecting into the water flow path
6. Is simple and robust and not susceptible to damage by floating debris or flood water
7. Can be installed with the least modifications to the gate
8. Does not require underwater work
9. Is accessible for maintenance or has low maintenance requirements

Options Considered

Five options were considered, refer Table 3, but only options 4 and 5 were suitable. Of these two options, the cylinder one is preferred as it does not have such a large frontal area as the winch and cable drum would have. A hydraulic cylinder is also easier to control as it is a linear motion with two distinct boundaries to help reset instruments if required.

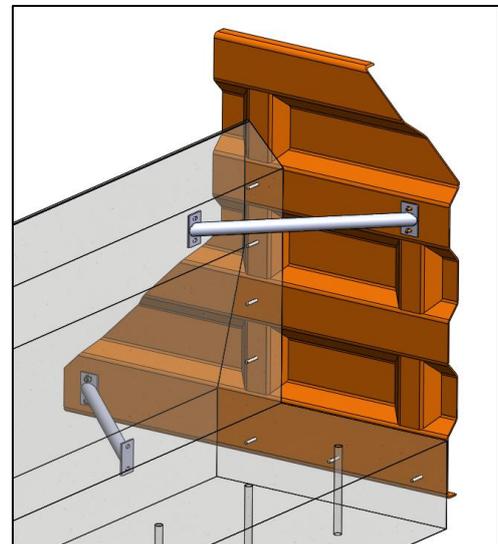


Figure ~~12~~¹³ Wing Plate Layout

Table 3 Options for Mechanical Hold Down

N o.	Option	Description	Advantage	Disadvantage
1	Cable Actuation upstream of gate	Installation of a cable and pulley system below the gates to open them using a winch system located on the embankment	1. Gates are positively lowered when required.	1. Complicated arrangement 2. Permanently submerged cables and pulleys - corrosion, maintenance issues 3. All underwater installation - expensive, poor quality control 4. Congestion of cables and winches
2	Rack and Pinion actuation	Mount a rack on the gate and use a hydraulic motor with a pinion to drive the gate up or down	1. Gates are positively lowered and raised when required.	1. Underwater installation 2. Permanently immersed 3. No visual position indication 4. Complicated anchoring 5. Significant modifications to gate 6. Exposed to debris impact
3	Hydraulic clamp (swivel)	Installation of a hydraulically operated swivel bar that engages the gate over the last 200-300mm movement and locks it down.	1. Close tolerances on gate position not required. 2. Gate can be driven down 3. All equipment above water 4. Easy to maintain 5. Small impact area 6. No or minor modification to gate	1. Exposed to debris impact - can be shielded 2. Reliant on gate position indication to operate 3. Vulnerable to gate position indication/sensors (can activate before gate is in reach, not activate at all) 4. Only in limited contact with gate (gate has to go down at least within reach of system before it is effective) 5. Double acting system, 2 hydraulic pipes required per gate side
4	Cable Actuation downstream of gate - winch system	Installation of a cable and pulley system on downstream side of gate with winch mounted on pier.	1. Gates remain in permanent contact with locking mechanism. Close tolerances on gate position not required. 2. Gate can be driven down if required 3. All equipment above water 4. Easy to maintain 5. Minor modifications to gate 6. Not dependent on gate position indication	1. Exposed to debris impact but can be shielded 2. Cable winds onto drum - large area exposed to flow
5	Cable Actuation downstream of gate - cylinder system	Installation of a cable and pulley system on downstream side of gate with cylinder mounted on pier.	1. Gates remain in permanent contact with locking mechanism. Close tolerances on gate position not required. 2. Gate can be driven down if required 3. All equipment above water 4. Easy to maintain 5. Cylinder in line with flow - small impact area 6. Hydraulic control system uncomplicated 7. Minor modifications to gate 8. Not dependent on gate position indication	1. Exposed to debris impact - can be shielded 2. Additional reeving (sheaves) 3. Higher friction component 4. Extended cylinder must be shielded from debris

Selected Option

The option selected to progress to detailed design consisted of a single acting hydraulic cylinder mounted lengthwise on top of the piers – refer [Figure 14](#). It is connected with a double reeving system to the gate. The gate operates under its own system as originally designed, when it fills with water it lowers and when it drains it rises. The hydraulic system will maintain a low pressure inside the cylinder sufficient to keep some tension on the cable. When the position indicator inside the cylinder confirms that the gate is fully open, a control valve will close and lock the cylinder in the extended position.

Once the reservoir level transducer determines that the water level is near each gate's intake weir, the control valve will open and allow the gate to retract the cylinder when it rises as water drains from it.

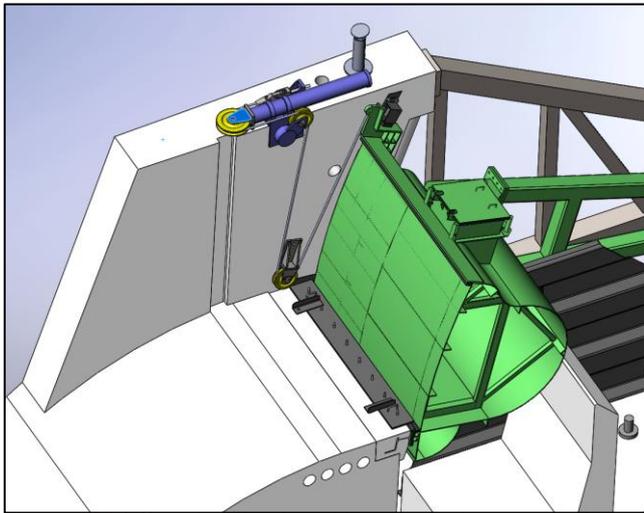


Figure 1344 Gate in raised position – cylinder retracted

cylinders is not required as this is provided by the buoyancy of the gate. The hydraulic pipes will be routed inside the gallery of the spillway and vertically to each cylinder through core drilled holes in the piers. The holes will be sealed against water ingress.

Design

All elements of the locking mechanism were designed to resist the maximum uplift force of 210kN calculated with the

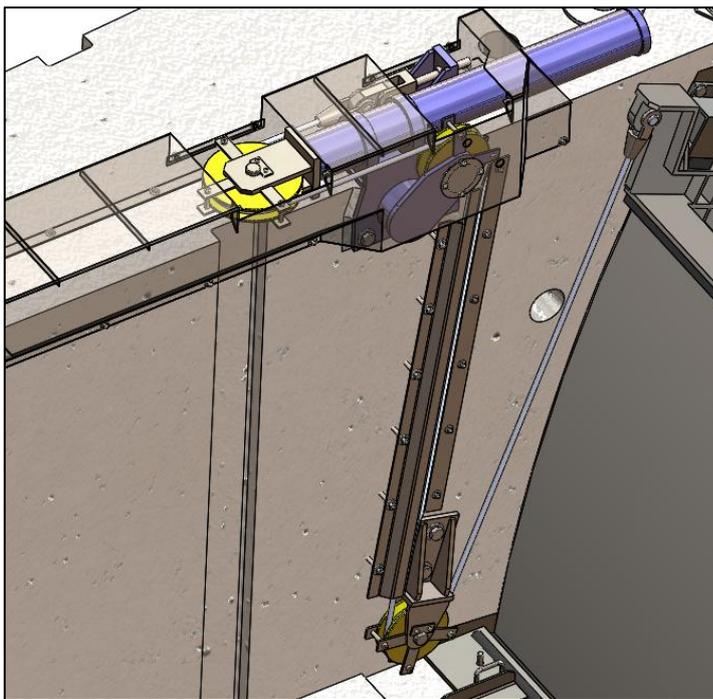


Figure 1445 Final arrangement of locking system and covers

CFD analysis. To reduce friction and maintenance requirements, the sheaves were manufactured from Nylatron GSM with Orkot ® bushes. All the pins and side plates are stainless steel. Although the cylinder rod is normally retracted it is made from hard chromed stainless steel but the cylinder is painted carbon steel. A powder coated aluminium cover is provided to shield all the moving parts on top of the pier and a stainless steel deflector plate covers the cable along the pier. The bottom sheave and cable to the gate have been left uncovered as the cable hanger on the gate will protect them when the gate is open.

Construction

Modifications had to be made to the gate to attach the wire ropes on each side. The gate structure was analysed using FEA software and localised reinforcements had to be provided.

The modification works were designed for installation in dry conditions behind an upstream coffer dam. The contractor however opted to do an underwater installation for the deflector wall and wing plates using a barge

Commissioning and Testing

During the construction works (which was done in the dry season), the lake level had receded to 150mm above the fixed crest level. On completion of the works, the control system was tested and all the gates were manually operated under no flow conditions. This was possible because of the low water level and sandbags were placed on the crest to minimise water loss. After successful “dry” commissioning the contractor cleared site.

Wet commissioning was postponed to wait for the dam to reach full supply level. During this time a storm event resulted in a flood passing through the dam in January 2016. The gate system however operated automatically as

The advantages of this system are that it is permanently connected and reacts to the movement of the gate. There are no sensors at the gate other than the position transducer which is inside the hydraulic cylinder. The only other sensors required are pressure switches and pressure relief valves at the hydraulic power unit and the centrally located water level transducer. The cable driven system cannot prevent the gate from opening, even if the hydraulic or control system fails. Should the control or hydraulic system fail when the gate is lowered, manual activation of the control valve will allow the gate to rise.

All components of the cable and actuator system are downstream of the gate and can be shielded against damage from debris or high velocity water. Using single acting cylinders reduces the complexity of the control and hydraulic system and hydraulic pipe leading to each cylinder. Synchronization of the

cylinders is not required as this is provided by the buoyancy of the gate. The hydraulic pipes will be routed inside the gallery of the spillway and vertically to each cylinder through core drilled holes in the piers. The holes will be sealed against water ingress.

Modifications had to be made to the gate to attach the wire ropes on each side. The gate structure was analysed using FEA software and localised reinforcements had to be provided.

Construction

The modification works were designed for installation in dry conditions behind an upstream coffer dam. The contractor however opted to do an underwater installation for the deflector wall and wing plates using a barge

mounted drill rig and divers. This decision resulted in a number of difficulties, including provision of clear records for quality control as a result of poor underwater visibility. The modification works were however successfully completed in October 2015.

Commissioning and Testing

During the construction works (which was done in the dry season), the lake level had receded to 150mm above the fixed crest level. On completion of the works, the control system was tested and all the gates were manually operated under no flow conditions. This was possible because of the low water level and sandbags were placed on the crest to minimise water loss. After successful “dry” commissioning the contractor cleared site.

Wet commissioning was postponed to wait for the dam to reach full supply level. During this time a storm event resulted in a flood passing through the dam in January 2016. The gate system however operated automatically as

designed to the great relief of all concerned. In March 2016 the gates were finally wet commissioned and some minor adjustments to the control system suggested.

Conclusion

The type of gates used at Lenthall dam have a very neat operating system and their appeal for this application is understood. They would have been ideal for many unpowered sites if they had worked as promised. Their design is however flawed as proven by the Lenthall experience and requires additional structures and modifications to ensure that they remain fully lowered during overflow conditions. The hydrodynamic modifications designed during this project show that the gates could still operate without the powered hydraulic cylinders and control system used, as long as the expected flood levels are moderate. Unless the same geometry of installation is used, careful analysis of the installation would be required.

This project has demonstrated the usefulness of CFD aided design and its capability to provide critical design information for which previously expensive and time consuming scale models were required. It is very efficient to inspect various layouts and options and assess detail elements. Where existing data can be used to calibrate the model the output has a high reliability.

Acknowledgement

We wish to thank Wide Bay Water Corporation and GHD for allowing this project to be shared with the wider dams' community.