

Analysis and Design Challenges Associated with the Catagunya Dam Restoration Project

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Following a number of years of investigation into the condition of the existing 1960's post tensioned anchors at Catagunya Dam Hydro Tasmania embarked on an options study to determine the best method to restore the dam stability to acceptable limits for the long term. The required solution was intended to not only resolve the issue of anchor deterioration but also to increase the flood capacity of the dam.

Based on preliminary design work a concrete buttress solution was recommended and approved for detailed design. The preliminary design utilised a simplified, 2-dimensional, rigid body model, including crack analysis. As part of the detailed design a finite element model was developed to refine the preliminary design. However, this model did not support the simplified analysis and further non-linear finite element analysis demonstrated that the proposed passive buttress design solution was not technically feasible. The options were reconsidered and the adopted solution was to replace the original anchors with new modern anchors with a high level of corrosion protection.

The new anchors adopted are the largest post tensioned anchor loading currently used for a dam in the world. This along with the existing post-tensioned anchors and the tight geometry of the dam, which has a central spillway with a cantilevered ogee crest, provided significant challenges with the design of this dam upgrade. Some of the key design challenges included:

- *Appropriate level of modeling and analysis to be able to make sound design decisions. (Hydraulic modeling and FEA).*
- *Congestion due to the tight geometry of the original design.*
- *Anchor head block detail to ensure the loads would be adequately secured and dispersed into the dam body*
- *Crest cantilever support to ensure that structural integrity was retained during construction and later in service. Innovative installation of carbon fibre reinforcement was used.*
- *Strain compatibility. It was important to ensure the structural contribution of new and old working together and that the consequences of application of new large stresses was manageable.*
- *Existing anchor degradation. The design needed to ensure that stability compliance was achieved for complete to zero effectiveness over time.*
- *Maintaining operability of dam and power station during construction.*
- *Achieving an effective long term maintainable solution.*

This paper will present the risk associated with committing to a solution too early and the design challenges and the solutions finally developed, providing the dam industry with a valuable reference for future similar projects.

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1. Background

Catagunya Dam was constructed from 1957-1961, is 49m high and has a crest length of approximately 280m (refer to Figure 1). Design and preliminary construction work were commenced early in 1957 for a mass concrete gravity dam. However, following a visit to Europe in 1957 by the Hydro Electric Commission's Chief Civil Engineer, the option of a post-tensioned concrete gravity dam was seriously considered. As a result it was estimated that a post-tensioned dam provided a saving at Catagunya in the order of 20 per cent of that for a conventional gravity dam (Wilkins & Fidler, 1959). As a result of the cost advantage, a decision was made to construct Catagunya Dam as a post-tensioned concrete dam, where removed concrete volume was replaced by 412 no. x 200 ton capacity, high tensile steel cables anchored to the dolerite foundation rock. At the time Catagunya was the highest post-tensioned concrete dam in the world.



Figure 1 – Catagunya Dam Prior to the Dam Restoration Project Works

The post-tensioned anchors used at Catagunya Dam were two-stage fully grouted and bonded anchors (ie. no free length), where the grout column was the only defense against corrosion. It was recognized at the time of construction that these anchors would have a finite life and at some point they would need to be replaced. Based on current standards, these types of anchors would be classified as temporary anchors, since modern day permanent anchors have at least two barriers against corrosion, and are load monitorable.

Following a Comprehensive Safety Review and the Portfolio Risk Assessment for Catagunya Dam in the late 1990s, the integrity of the post-tensioned anchors was highlighted as a key safety deficiency for the dam. As a result, a detailed investigation program commenced in early 2000 to try to ascertain the condition of the post-tensioned anchors. This involved in situ groundwater sampling and analysis, corrosion studies in the laboratory and in situ corrosion testing, which was summarised in an ANCOLD paper in 2002 (Herweynen & Hughes, 2002). The conclusion of this work was that

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some corrosion was occurring, and in 2004 a decision was made to make the existing anchors redundant in the 'medium term'.

As a result of this decision Hydro Tasmania Consulting embarked on an options study for the Catagunya Dam Restoration Project. The solution was intended to not only resolve the issue of anchor deterioration but also to increase the flood capacity of the dam. This option study, along with the final detailed design, provided the design team with significant analysis and design challenges, which is the topic of this paper.

2. Options Study

2.1 Initial Options Study

An options study was undertaken which looked at the following solutions:

- Gravity dam addition to the upstream face
- Roller compacted concrete continuous buttressing to the downstream face
- Discrete structural concrete buttresses supporting the downstream face
- New post tensioned anchors at the abutments and the spillway crest
- New post tensioned anchors at the abutments and the spillway flip bucket
- Rockfill buttresses on the abutments plus new post tensioned anchors on the spillway

These options were developed using preliminary analysis. Options were ranked in terms of estimated cost and identified risks. The key factors influencing acceptance / rejection of the options are listed below:

Gravity dam addition upstream: Expensive, required dewatering of the reservoir for an extensive period, therefore considerable lost revenue from Catagunya Power station. (rejected)

RCC: Expensive, some design risk in the interface detail between the existing dam and the RCC addition. (rejected).

Rockfill buttresses on the abutments plus new post tensioned anchors on the spillway: Expensive, difficult to construct due to the multitude of different activities in a restricted area - rockfill, structural concrete, post tensioned anchors. (rejected)

New post tensioned anchors at the abutments and spillway flip bucket: More expensive than crest anchor options as this location is less efficient, thus more anchors required. Long term maintenance of anchors would be required. (rejected).

New post tensioned anchors at the abutments and spillway crest: This offered a low cost solution but appeared to have significant design risk to ensure structural integrity at the spillway face in the vicinity of the anchor heads. Long term maintenance of anchors would be required. (short listed for final consideration).

Discrete structural concrete buttresses supporting the downstream face: This appeared to offer a viable solution (based on the outcomes of the 2D gravity analysis) at marginally lower cost. (short listed for final consideration).

Based on this option study the preferred solution was a Buttress Solution with a series of discrete concrete buttresses. This was seen to offer the lowest overall cost and it did not have any long term maintenance costs, as opposed to the anchor options.

Dam Decisions: Past Experiences, Future Challenges

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2.2 Concrete Buttress – The Initial Preferred Solution

As part of the initial options study the Buttress option was analysed as an equivalent block for a cantilever analysis, refer to Figure 2 below.

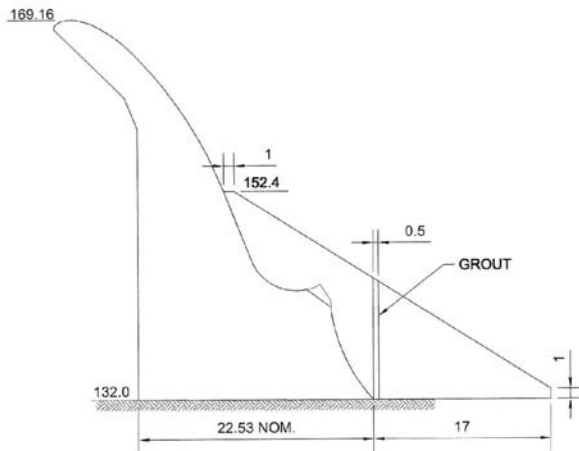


Figure 2 - The Buttress Solution with the Composite Block Section Analysed

COMPOSITE SECTION AFTER GROUT FILLED

The buttress concept was for two buttresses supporting each dam block. Each buttress was 2m wide, 20m maximum height and maximum base length of 17 m. In order for the original dam and buttresses to 'act as one', a procedure was devised to ensure that integration was achieved for all ambient and load conditions. In general terms, the procedure involved partially unloading the dam by lowering the reservoir prior to integrating the new buttress concrete with the existing dam. Either infill grout or concrete was required to achieve this integration while the storage was low and the dam at its optimal seasonal displacement during favourable ambient conditions.

Prior to the infill, with the storage lowered, the dam moves upstream and the dam and buttress acts as non composite separate structures. The gap is then filled with grout / concrete, cured, then the water raised to full supply level. Composite action now takes place. The stress distribution at the base will be on the basis of the combined moment of inertia. The total stress at the base will be the linear addition of composite and non composite action of the dam and buttress.

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In the simplified stability analysis, the relative stiffness difference between the dam and the buttress was ignored. Also, the effect of rock stiffness and consequent influence on foundation stress distribution, was not considered at this stage.

Following the initial options study, the design for the Buttress solution was further developed with detailed finite element analysis (FEA). Based on this analysis the preliminary details developed in the initial option study were proven to be inadequate as the tail of the buttress was shown to be ineffective (ie. the load as shown by the principal stresses going vertically into the foundation). This is shown in Figure 3 below, where the simplified, cantilever block analysis uses the full length of the buttress, while the finite element analysis indicated that the tail of the buttress was not being used resulting in a significantly reduced lever arm. With a significantly reduced lever arm, tensile stresses are generated at the upstream heel of the dam.

As a result the lever arm adopted in the preliminary calculations was too long and increasing the length of the buttresses did not help.

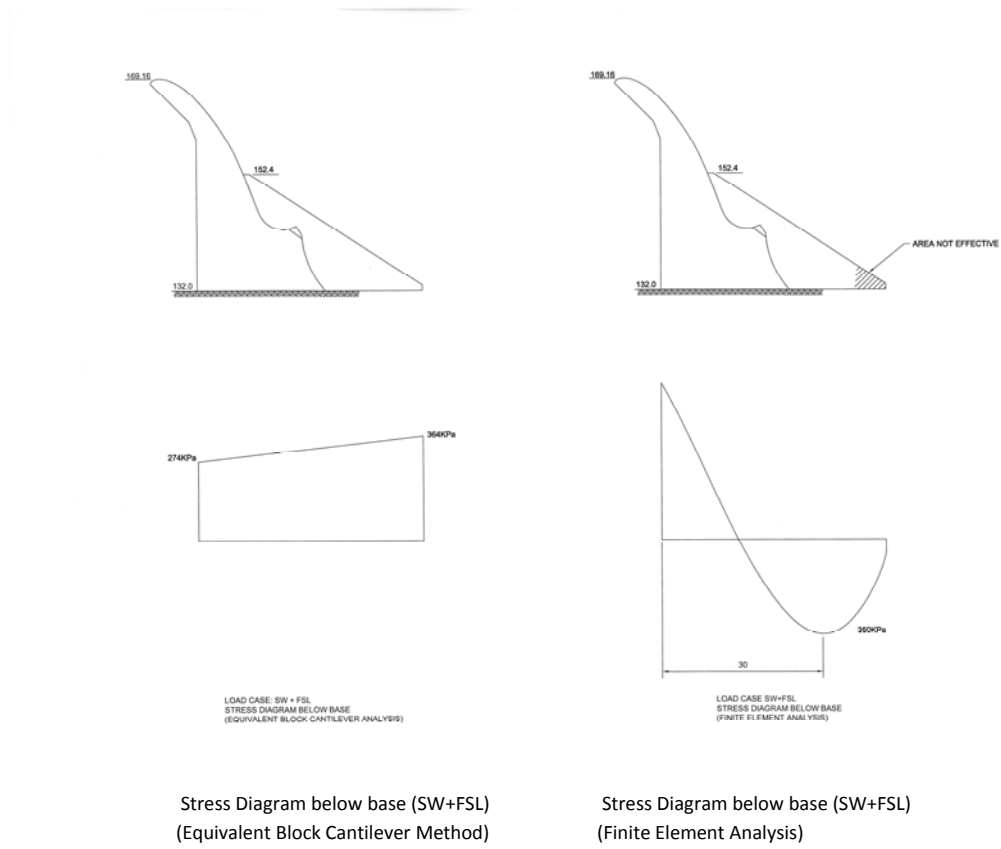


Figure 3 - Comparison between stress diagrams from simplified analysis and FEA

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The concerns regarding the successful integration and locking in the stress into the buttresses are elaborated as follows. Factors such as existing to new concrete interface, concrete shrinkage, reservoir level changes, dam thermal movement at the time of construction all influenced the efficiency of the buttress action. Dam foundation pressure distribution was sensitive to these effects. The initial solution was to have a second stage concrete between the dam and the buttress. The intention was for this second stage concrete to be placed following the reservoir level being dropped by 12m and during favourable thermal conditions. The ability to guarantee this and the cost of lowering the reservoir (due to loss in generation) were issues with this solution.

Following stability analysis by FEA, it was found that this 'passive' buttress solution could only meet the required stability criteria for floods up to the existing design flood. The design was found to be unstable under extreme flood conditions (PMPDF). Dam base cracking would initiate at the upstream foundation interface and propagate all the way through the section.

Different buttress geometries (width, length, height) were investigated to see if increased stiffness would solve the problem. It was found that improvements were only marginal and potential base cracking under extreme flood conditions was unacceptable.

The solution for solving these issues was to install jacks between the buttress and the dam to lock in a pre-loading to the buttress. This solution required some stress bar foundation anchors to stabilise the buttresses during this pre-loading. Pre-loading by jacking redistributes the original dam base pressure but does not increase the overall base reaction force. These additions to the buttress solution added significant cost and it was no longer a passive solution. The previous thinking of a 'simple passive' buttress solution had evolved into a more complex solution.

2.3 Redefining the Preferred Solution

As a result of the buttress studies, potential costs had escalated to the point where previously eliminated options required revisiting. It was clear that a primary issue with the buttress solution was that the original dam base pressure is not increased as there is no new applied stabilising vertical load. All new stabilising restraint forces are horizontal. Alternatively, options which increase the dam base pressure due to increased applied vertical load reduce the risk of base crack initiation and propagation. This is particularly relevant when the applied load is upstream of the dam centroid. The post tensioned anchor option is the most efficient in this respect.

The RCC option was refined further but was again rejected on cost.

Passive buttress solutions with reduced flood levels were considered. These options included spillway improvements such as a new auxiliary spillway on the right bank including a fuse plug embankment, right abutment crest level reduced and replaced with fuse gates. The potential for staged construction was considered, where interim design flood improvements could be achieved at reasonable cost, and then later works completed to achieve an acceptable maximum design flood.

Based on the re-visit of the various options it was clear that the post tensioned anchor option now offered a cheaper solution compared to the buttress options and had comparatively less design risk. Some simplification was achieved in the post tensioned anchor option with respect to maintaining

the structural integrity of the dam crest during construction. Brainstorming including design, construction and review personnel proved to be highly valuable in this respect.

Therefore from cost, design risk and construction risk points of view the final solution adopted for the Catagunya Dam restoration was the post-tensioned anchor solution.

2.4 Key Lesson Learnt from this process

Given the cost and time impacts of re-visiting the options, the key lessons learnt from this options study process were:

- That sufficient design and analysis should be undertaken on the options before they are selected or eliminated ('Sufficient design' should include both manual analysis and FEA where the geometry of the subject is unusual);
- When a solution is based on simplified analysis that a degree of caution be placed on this solution until detailed analyses has been completed; and
- Where the solution extends or challenges the boundaries of accepted practice, a higher level of technical oversight and caution is required.

Initial studies need to clearly identify and quantify the design risks and construction risks. Identification of these risks may drive the need for further analysis (in preference to educated judgement) before a final solution can be adopted. Our experience showed that although many design risks were apparent in the concept phase, there was insufficient definition to quantify them adequately. This was particularly significant in this case because the differences in estimated cost of the options were relatively small.

3. Final Adopted Solution

The post tensioned anchor option was adopted as the final solution. The solution involved:

- Reassessment of flood hydraulics based on revised hydrology. This included abutment overtopping and potential spillway trajectories for foundation erodibility studies.
- More extensive study and determination of foundation geology, particularly for potential stability issues on the left abutment where foundation joints dip toward the river and downstream. Deeper anchorage zones (compared to the original shallower anchors) required further scrutiny with respect to potential failure wedges and extent of identified chloritised zones. Joint structure and integrity of rock in spill impact zones was assessed.
- Installation of 17 anchors in the left abutment, 22 anchors in the right abutment, 53 anchors in the spillway section. (92 anchors in total). Maximum anchor design length 78m. Anchors are located within very tight dimensional constraints in the abutment sections in between existing anchors. In the spillway section, anchors are located immediately downstream from the original anchors. All anchors are 91 strand except for a single 36 strand anchor in one of the smaller blocks.
- Spillway anchor heads are recessed into the dam and covered with steel protective covers so that the spillway downstream face profile is maintained.

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- Abutment anchor heads are accommodated in raised reinforced concrete ‘beam’ sections on top of the original abutment crest.
- A shear key is included between previous gravity blocks L and M to provide the necessary stability to block M.
- Dam spillway crest integrity is achieved by carbon fibre reinforcement in between anchor head positions in the spillway section.
- Minimal rock erosion protection is included in identified local zones at the downstream spillway toe and abutment toe.

4. Design Challenges

In the development and detailed design of the final adopted solution a number of design challenges were experienced. Each of these design challenges will be discussed in detail below.

4.1 Appropriate Level of Modelling

Revised hydraulic modelling by CFD provided the essential hydraulic loads on the dam. The modelling showed the zones of negative pressure at the crest, the transition to neutral pressure down the spillway face, flip bucket pressures, tailwater pressures and spill trajectories. Understanding the pressure distribution on the downstream face was crucial for the stability analysis as both the crest pressure (destabilising) and flip bucket pressure (stabilising) played major roles in the overall stability during extreme floods. The modelling also defined the tailwater pressure variability according to the turbulence for various floods. Given the stability sensitivity of Catagunya Dam, all these loads needed to be clearly understood as part of the stability assessment. The modelling also showed the flow behaviour over and around the abutments and the extent of tailwater acting as a plunge pool. Areas above tailwater where potential spill impact on exposed rock would occur could be readily identified. Overall, the sophistication of the CFD studies was justified.

Stability and structural analysis by FEA proved crucial in the earlier buttress design and ultimately led to the elimination of this concept. The original 2D cantilever gravity analysis performed at the concept design stage proved insufficient to enable confidence in the option selection. We recommend that for future studies of unusual structures, that short listed options be studied by both manual stability calculations and by more detailed FEA prior to option selection.

FEA for structural analysis also proved to be crucial when considering matters of strain compatibility of adjacent blocks. This is discussed further below.

4.2 Congestion due to the tight geometry of the original design

A major challenge in modifying Catagunya Dam was the original tight geometry. The original designers achieved a very compact design with minimal concrete. The original dam anchors were relatively small in capacity and diameter but there were many of them, so available space for new anchors was very tight. Fortunately, original construction tolerances were also tight, therefore there was reasonable confidence that ‘surprises’ in interference of new / original details would be manageable.

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For the spillway section, new anchors were designed to be located immediately downstream from the original anchors. This enabled space for the new anchor heads to be recessed below the spillway face and just avoid interference with the original anchor heads.

For the abutments, the available crest width is very limited, so the new anchors had to be interposed between existing anchors. If new anchors had been positioned downstream of the existing anchors, excessive bending of the upstream dam wall would have occurred. In order to provide satisfactory structural integrity in the anchor head region, a head 'beam' was designed to accommodate suitable head block reinforcement and to distribute stresses at the contact with the dam crest. This detail meant that no oversize coring was necessary into the dam crest. Drilling below the dam crest was limited to 350 diameter for the anchor holes only.

Previous gravity blocks A, D, L, M, Ra and Rb required stabilisation in the new design. Achieving this stabilisation by direct anchoring proved to be feasible for all blocks except for block M due to congestion of the original diversion tunnel details. The solution for block M was to key the block to block L by means of a new shear key. Positioning anchors for block D was quite a challenge due to the location of the dewatering tunnel, gate chamber and training wall geometry. Satisfactory anchor location was achieved although ideally, an anchorage location further upstream would have been more efficient.

4.3 Anchor head block detail

4.3.1 Spillway

A typical spillway anchor head block (refer to Figure 4) is 1200 mm diameter by 1200 mm deep precast reinforced concrete encased in a steel cylinder. There are 7 anchors per block spaced at 2177mm apart. As the anchors are closely spaced, we needed to know the group load effect on each block due to a full set of anchors. FEA was performed in the head block region and surrounds, to gain a full understanding of the stress fields, in particular the bursting stresses and local stress effects at the termination of the anchor head bond. 3 cases were modelled including base bonded, half bonded and full bonded head blocks. Results showed that bursting stresses could be readily controlled within the head block and rapidly diminished below the head block. High local tensile stresses (> 4 MPa) surrounding the termination of the bond required special attention.

A solution was derived to cast the head block in a steel cylinder where the internal interface concrete / steel was bonded for the lower half but debonded for the upper half. The complete precast unit was designed to be installed in the cored hole in the dam so the cylinder exterior was fully bonded by epoxy grout. This detail would theoretically prevent local high tensile stresses occurring surrounding the base and walls of the cylinder but would transfer the vertical stress component over the height of the cylinder. The internal bond termination could not then initiate cracking in the dam concrete at this point as stresses are dispersed by the cylinder.

Anti burst reinforcement in the head block was determined by conventional theory and cross checked with the FEA results. Reinforcement was determined excluding the hoop steel effect of the cylinder. This is particularly relevant in the upper half, as this region is debonded. For the lower

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half, antiburst reinforcement is more than required due to the combined effect of the internal reinforcement and the cylinder which is integrated with the internal concrete by means of shear studs.

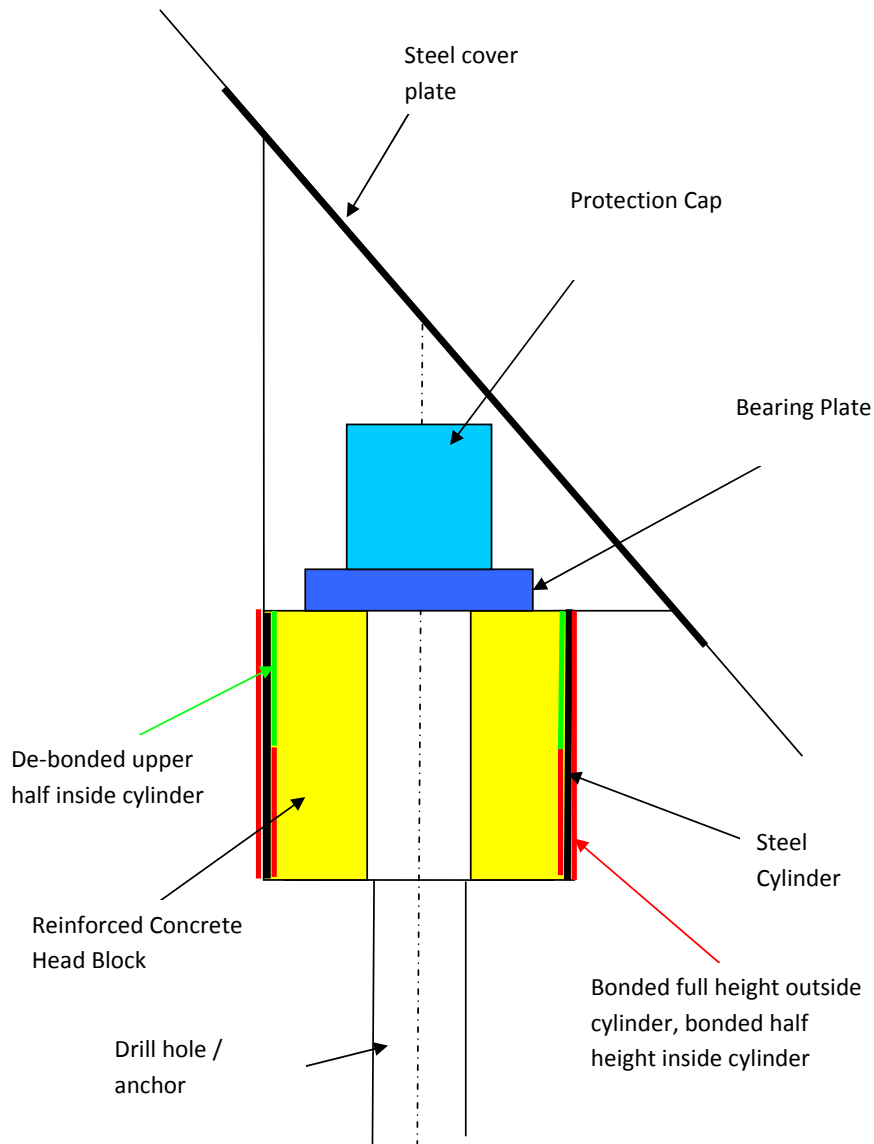


Figure 4 - Typical Spillway Head Block

4.3.2 Abutments

Abutment head 'beams' accommodate the head block anti burst reinforcement. These structural beams are 2820 wide by 1220mm deep for each dam block. Contraction joints at dam block interfaces were maintained. The beams serve the purpose of distributing the anchor loads evenly

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along each dam block crest. Anti burst reinforcement was determined by conventional theory. Stresses at the new head beam to original dam boundaries were checked by FEA.

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4.4 Crest Cantilever Support

A unique issue with the spillway section of Catagunya Dam is the cantilever design of the crest (refer to figure 2). This crest cantilever is a reinforced concrete element which had been designed for the self weight loads should the reservoir level drop. In order to install the large post-tensioned anchors in the spillway section a large portion of the tensile reinforcement for this cantilever needed to be cut to install the anchor block. Therefore the concrete that remained in place between the 1200mm diameter anchor block holes needed to be strengthened to ensure that the cantilever remained structurally adequate. The final solution that was adopted for this was to strengthen the spillway cantilever with carbon fibre bars prior to the holes being cut for the anchor blocks. The design detail required cutting 4 vertical grooves 14mm wide x 90mm deep into the dam spillway face extending equidistant upstream and downstream of the line of anchors. In each groove, 6 / 8mm diameter carbon fibre rods were to be installed by embedment in epoxy with sufficient cover to protect from exposure. In order to ensure effectiveness of this new reinforcement combined with the remaining existing reinforcement, installation was to be performed when the spillway face stress was close to zero. Given the importance of this reinforcement to the integrity of the dam crest and the fact that carbon fibres had never been used for this purpose the following was undertaken:

- A trial installation beam was constructed (see Figure 5). A single groove of dimensions 14mm x 90mm deep was cut and 6 / 8mm rods were installed in epoxy. Upon curing, the beam was cored through several locations to visually reveal the effectiveness of the epoxy encapsulation of the carbon fibre and the epoxy to concrete bond (see Figure 5). The trial results revealed success with the methodology so the detail was adopted as the final design.
- The behaviour of the dam crest cantilever was analysed to determine the reservoir level which would result in zero stress at the downstream face location of the carbon fibre reinforcement. This was determined to be close to the normal operating level of the reservoir, where the moment generated by the self weight is balanced by the opposite moment generated by the uplift under the overhang section.



Figure 5 - Test beam on the left & core sample on the right

- Analysis showed that when the reservoir is drawn down to EL 168 (ie. 1.16m below full supply level), the tensile stress in existing reinforcement is zero. If the storage is drawn down

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below the base of the cantilever EL 158.5m, the maximum tensile stress in the downstream reinforcement is 305kPa, taking into account both flexural loading and axial loading.

- The carbon fibres reinforcement is therefore installed when the storage is within its operating range so that original steel reinforcement and the new carbon fibre reinforcement have the same zero stress datum.
- After anchor installation, ignoring new reinforcement and with the reservoir drawn down, the strain in the existing reinforcement is 0.9 micron which corresponds to a stress of 186 MPa. The allowable stress in the existing reinforcement is 165 MPa. This necessitates the use of additional carbon fibre reinforcement to ensure that the existing reinforcement does not exceed permissible stresses.
- Considering new and existing reinforcement the strain in the combined reinforcement is 0.8 micron. This results in 157 MPa stress in both the existing & new reinforcement, which is less than the permissible stress.

4.5 Strain compatibility

It was important to ensure the structural contribution of new and old working together and that the consequences of applying new large stresses were managed. Therefore it was not only important to check the ultimate strength but also the strain compatibility. There was no use designing an element to be able to take a load if the load never eventuated because the displacements required to mobilise the load transfer never occurred. An example of where strain compatibility was an issue was in the stabilisation of the gate gravity block (Block D). Due to the limited space for installing anchors to stabilise this block, the initial solution was to transfer load to the neighbouring abutment block (Block C) using a retrofitted shear key arrangement. However, block D was a gravity block and therefore a very stiff block compared to the slender, post-tensioned block C. As a result it was determined that an unacceptable amount of displacement would need to occur for the load to be transferred from block D to block C. When subjected to increasing flood loading, unacceptable tension at the upstream foundation base of block D would initiate before significant support would be offered by block C. As a result this solution did not work due to strain incompatibility.

In analysing the behaviour, 2D FEA models were developed for block C/D, and similarly for block L/M to study the relative deformation of the adjacent blocks. Figure 6 compares the relative deflection of the adjacent blocks for C/D and L/M respectively when subjected to the same external reservoir loads.

4.5.1 Block C/D

It can be seen that when subjected to self weight, FSL load and prestress, block C deflects approximately 13mm upstream at the crest. Block D when subjected to self weight and FSL load showed minimal downstream deflection. (At this stage of design, no prestress was proposed for block D). The next step in the analysis was to apply equal and opposite loads at the interface nodes where a potential shear key would be located, over the height of the dam. The loads were adjusted until the deflection of each block was identical. The result showed the actual load transfer which would occur if the blocks were tied together. This exercise was repeated for the PMPDF load case

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and similarly the actual load transfer deduced. This analysis showed that Block C is insufficiently stiff to adequately support block D, even though there is considerable prestress force applied to block C. Transfer loads were too small to offer significant support. The base of block D would crack right through due to the inadequate support from Block C.

4.5.2 Block L/M

The same process was followed for Block L/M integration. In this case, Block L (the supporting block) is relatively stiff. The finding was that Block L could adequately support Block M by means of a shear key. Analysis showed that the load transfer at the key would approximate an inverted triangular load distribution. Given the deduced transfer load via a shear key, it was demonstrated that block M was stable under all load conditions

In summary, the strain incompatibility of Blocks C and D showed that a shear key would not work. Conversely, the strain compatibility of Blocks L and M showed that a shear key would work.

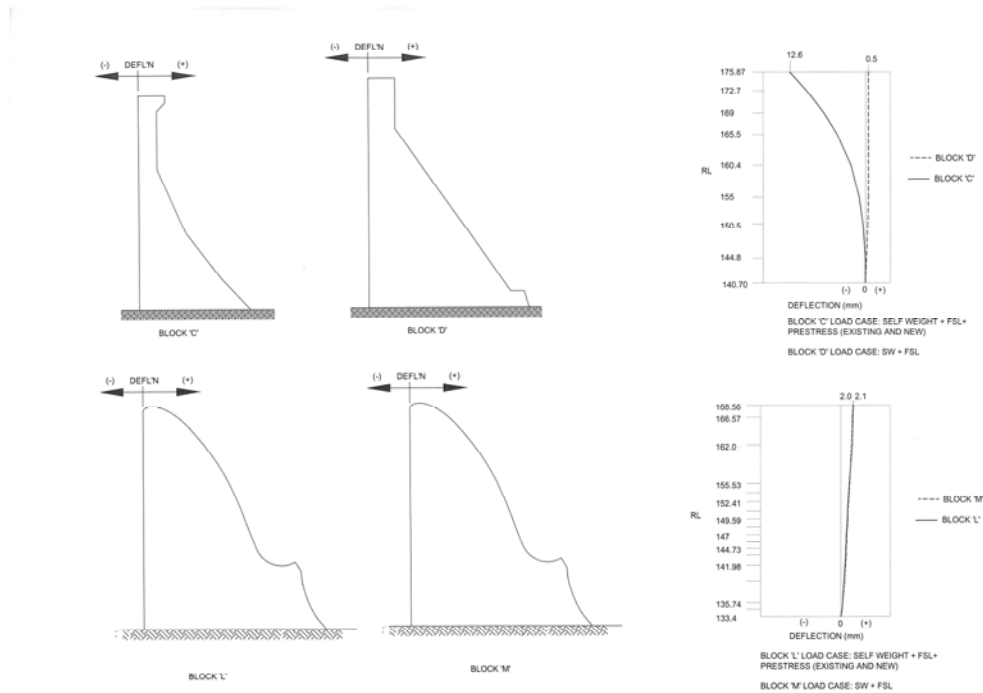


Figure 6 - Comparison of the Relative Deflection between Blocks C & D and Blocks L & M

4.6 Existing Anchor Degradation

Although the primary reason for the Catagunya Dam restoration project was to make the existing anchors redundant due to concerns regarding their long term integrity due to corrosion, the actual amount of degradation is an uncertainty. Therefore the design needed to ensure that stability and structural compliance was achieved for complete to zero effectiveness over time. If the new and the old anchors had 100% of their design loads then it was important that we checked the maximum

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compressive loads as well as the tensile stresses on the downstream face should the reservoir be lowered.

The loadings of primary interest were:

- Extreme flood load plus new prestress but with existing prestress reduced to zero (potential base crack initiation from upstream)
- Reverse loading where both the new prestress and the original prestress exist at 100% and the reservoir is lowered (potential base cracking from downstream).

The design for some blocks proved to be a balance between satisfying the two loadings. The aim was to ensure that theoretically, a portion of net uncracked base area would remain after being subjected to both loadings over time.

4.7 Maintaining operability of dam and power station during construction

Due to the value of generation (estimated to be in the order of \$10M per annum) there was a strong driver to maintain reservoir level during the restoration works. Therefore the design developed and the construction methodology adopted enabled the reservoir to be kept in the normal operating range, enabling the power station to operate and the dam to safely pass floods. This was possible with the adopted post-tensioning solution, which was a key advantage to this option.

Generally, by operating the reservoir closer to the Normal Minimum Operating Level, it was possible to reduce the risk of overtopping during construction to a manageable level. Following hydrological advice the construction team adopted a 1:50 AEP summer event as the basis for temporary works design on the dam.

4.8 Achieving an effective long term maintainable solution

In the early application of post tensioned anchor technology, satisfactory maintainability was not achievable when compared to modern practices. The original anchors have a single stage of grout encapsulation, are not restressable or load monitorable and cannot be inspected. The modern anchors have comprehensive corrosion protection throughout the length from the bond zone through the free length to a fully greased and protected head block enclosure. Modern anchors can have the protection cap removed and are load monitorable and restressable.

At Catagunya, the anchor head block outs and covers in the spillway are designed for maintenance access in accordance with the client's requirements. Sufficient clearance has been maintained in the detail to enable positioning of a jack for monitoring and restressing. Access is achieved via the same travelling gantry as used during construction.

For the abutment anchors, access can be readily achieved from the left and right abutment access roads respectively. Anchor caps are readily accessible at crest level. A permanent access bridge across the river to the right abutment, downstream from the power station, was constructed as part of this project.

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5. Construction

This paper will not go through the construction details of the Catagunya Dam restoration project as these are covered in a separate paper (Cubit et al, 2010).

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6. Conclusions

This paper has shown that there can be real risks associated with committing to a design solution too early. It is the authors' opinion that a reasonable amount of design and analysis be undertaken for the short listed options before a final decision is made on the preferred option to progress to detailed design. In the case of Catagunya Dam the completion of detailed finite element analysis was required before the technical feasibility of the final solutions was demonstrated, and that simplified analysis was proven to be inadequate.

This paper has also demonstrated the design challenges associated with working on an existing asset and the complications that this can often lead to. It has shown some innovative solutions to a number of these design challenges, including a carbon fibre solution to strengthening the cantilever spillway crest. A key benefit of the final solution that was developed for the restoration of Catagunya Dam was that the power station could remain operational.

7. Acknowledgements

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8. References

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