FILE
DAM/130/000/(

DOCUMENT RECEIVED BY NR&M
25/9/03
File No. Dam/130/000(1997)
File Location D. RYAN
Action By WIC000 D.RYAN
Registered Y/N Doc. Code RUC3/03484
MOVEMENT INSTRUCTIONS MUST BE SHOWN ON FRONT COVER.

Burnett River Dam – Stage 2 I

Review of SMEC Proposal.

By D Ryan, Dam Safety
August, 2003

INTRODUCTION

The proposed dam is located approximately 20km north-west of Biggenden and 80km south-west of Bundaberg. The dam is 13km upstream of the confluence of Degilbo Ck on the Burnett River.

Basic description of the dam:

Location	Burnett River, (AMTD 131.4km)
Dam Site Area	Lot 3 CK2702 Lot 2 SP 135369 Lot 4 BN 37379
Purpose	Irrigation Water Supply
Construction Type	Roller Compacted Concrete
Total length of embankment	1020m
Storage Capacity	300,000Ml
Length of river inundated	45km
Spillway (Primary)	
• Profile	Ogee Crest
• Crest Level	EL 67.6
• Length	315m
• Apron Slab Level	EL 30.9 (typically)
• Downstream Slope	0.64H:1V
• Capacity	23,000m3/s (from chart)
• Mono Width	45m
Spillway (Secondary)	
• Profile	Broad Crested Weir (Gravity Walls)
• Crest Level	EL78.0
• Length	500m
• Apron Slab Level	varies
• Downstream Slope	varies (0.64H:1V to 0.9H:1V)
• Capacity	50,000m3/s (from chart)
Abutment (left)	
• Profile	Gravity Walls
• Crest Level	EL83.0
• Height (maximum)	48m
Outlet Works	
• Construction Type	Reinforced Concrete
• Capacity (Irrigation)	$18m^3/s$
• Capacity (Environment)	245m3/s

Note:

1. Levels quoted are to Australian Height Datum (AHD)
2. Construction type is RCC unless otherwise stated
3. Primary spillway capacity is quoted to EL78.0 only (approx equivalent to 1:1,000AEP). At higher levels, the secondary spillway becomes operable.
4. Capacity quoted for the secondary spillway includes the primary spillway discharge and is equivalent to a reservoir level of EL83.0 (left abutment Crest EL, and approx equivalent to 1:10,000 AEP
5. Details described are as at September 2003.
6. Diversion and outlet works on the right abutment – the outlet works using the arrangement previously designed for the diversion works. Outlet works comprise a single 5.2m x 4m radial gate for environmental release (240m3/s), and 1.2m dia cone valve for irrigation release (17.8m3/s)

Features

- The decision was made not to provide a gallery – SMEC suggest there is little benefit in having the gallery, especially with the high tailwater levels during flood events
- The basalt layer overlying the alluvium on the right bank was left in place – SMEC suggest that this is the better option rather than removal of the basalt

Comments

The area downstream of the main dam appears open to erosion during a flood event. I have been unable to find any discussion on this – maybe, Burnett Water have not set any guidelines re downstream protection

HYDROLOGY (Sec 3)

General

A table presented in their Hydrology Section (Sec3 Pg9) lists the following as their adopted values: (this data has been combined with the results from the flood routing from Sec3 Pg11 of their report)

AEP (1:yr)	Peak Inflow (m^3/s)	Peak Discharge (m3/s)	Peak Level (m)
50	12808	12460	74.78
100	15326	14887	75.62
200	17799	17265	76.38
500	22033	21331	77.60
1000	25477	24821	78.47
2000	29202	28531	79.26
5000	37650	36828	80.74
10000	50328	49285	82.60
20000	71738	70584	85.15
30000	94861	93457	87.46
PMF	106863	104377	88.45

The hydrology section also describes the final adopted arrangement for spillways:

- A 315m long ogee shaped primary spillway at EL 67.6m
- A broad crested secondary spillway, over the whole of the right abutment, which commences overtopping at EL 78m. This corresponds to approximately the 1:1,000 AEP level
- Left abutment is set at EL 83m, which is the 1:10,000 AEP flood level

Comments

The Spillway Hydraulics section (Sec4, Pg1) describes the PMP Design Flood for the dam as 94,000m3/sec. Because SMEC have designed the structure to be overtopped, their claim is that the structure is "PMP immune".

SMECs claim in using the adopted design is that if the hydrology is reviewed at some future time, the dam can cope with some fluctuation in an estimate of the PMP.

However, they do not clearly state the value of the total Population at Risk used to determine the Incremental Flood Hazard Category (IFHC) as defined in ANCOLD "Guidelines on Selection of Acceptable Flood Capacity for Dams". It is possible that because the FIA was carried out for only a portion of the downstream reach, the PAR could well exceed the figure quoted by SunWater (PAR = 235). In fact it could well exceed a PAR of 1000, which would change the IFHC rating from High A to extreme. This needs clarification from Burnett Water – information request.

The use of the radial gate in accommodating the required releases has proved a reasonable solution. The details of this are given on Sec3 Pg 23 of their Hydrology report. (A full description of the flood routing and IQQM model is given in Appendix B of their report)

SMEC modelled the drawdown effects in relation to the critical environmental flow objective ie they were concerned that they may not be able to meet the criteria of environmental flows – especially for the small inflows within the bounds set in the brief. (It would be worth checking this as I do not have a full appreciation of what the criteria was as imposed by the BRD authority Refer Sec3 Pg23 – could do an independent flood routing check) their independent checks (ie using a program other than RORB) showed that the arrangement they proposed was OK.

The tailwater level for the PMF Design Flood is reportedly at EL 79m – however, there is no clarification as to what exactly the PMP Design Flood is – if it is the 1:30,000 or higher, then the afflux is at least 8.46m – based on data retrieved from the report

The ability to release the environmental flows is adequately described (Sec3 Pg 23) and achieved by the use of the radial gate (Capacity 240m^3/s which corresponds to 20,700Ml/d)

FOUNDATION (Sec 2)

General

SMEC are suggesting further site investigations to better define the geotechnical data at the site. This work includes 3 cored holes on the left abutment (25m deep), 3 x 20m holes in the river bed area, 4 x 15m holes in the right abutment - basaltic flow, 2 x 15m holes in the upper right abutment, 2 x 20m plus 2 airtrack holes in the approach channel, and 2 x 20m holes plus 6 air track holes in the exit channel.

A total of 24 holes in the dam area, along with an additional 23 boreholes and 50 backhoe test pits to assess the geotechnical conditions for the proposed Gahans Road upgrade (Ref sec2 Pg 23)

Left Bank

Excavation depth 4 to 5m into the highly weathered meta-siltstone
SMEC recommend that the wedge of alluvium overlying the goodnight beds be removed as part of the excavation process. Above the level of the 1:100AEP event, they recommend the alluvium be battered 2.5H : 1V and vegetated

River Bed – Main Dam

Remove alluvium and excavate surface irregularities in rock surface – estimate 0.5 to 1.0m

Right Bank

Vesicular basalt - Remove alluvium/overburden and excavate surface irregularities – estimate 0.5 to 1.0m)

Alluvium to east of basalt – excavate entirely to expose goodnight beds, max 12m reducing upslope. Excavate 0.5 to 1.0m into distinctly weathered meta-siltstone. Blocks of basalt up to 5m across are present in the talus below the basalt cliff – require breaking up prior to removal.

The basalt flow on the right bank was investigated (Ref Sec2Pg 6). SMEC opted to leave in place. Thickness of the basalt varies from 8.1 to 20m (av 12.6m) – thickness of alluvium underlying the basalt varies from 1.2 to 5m

Right bank ridge – excavate 4m to distinctly weathered meta-siltstone

Rationale for founding on basalt (Refer 3d view Fig2.5 Smec report)
Advantages – reduction in excavation for foundation and reduction of RCC material required
Disadvantages – potential for increased seepage (although seepage loss estimated at "significantly less than 10l/sec"), potential for sliding instability both during

construction and in service, and differential settlement of the sub-basaltic alluvium post construction.

Reference to Sec2 Pg11 – Rationale for founding on basalt layer - eastern side of channel. A stability analysis was undertaken where it was assumed there was a linear reduction in water pressure under the block to zero at the downstream toe. Similarly, for the western side of the channel, the linear reduction was also applied.

In both cases, it appears that the WL was set at EL72 (5m over the crest) – no reasoning was given for this. On the western side, anchors (free length 10m fixed 2 to 4m) were required to provide adequate safety factors.

The potential for settlement in the sub-basaltic layer was analysed. The results showed that tensile stresses were induced in the lower levels of the basalt, potentially allowing for seepage to develop. (ref Sec2 Pg 14) SEMC recommend that the preliminary design allow for curtain grouting through the basalt on completion of RCC placement

SMEC are also concerned about "squeezing" of the sub-basaltic layer (Sec2Pg15), and recommend underpinning the basalt to provide support for the outer eastern extremity.

It may be worth asking the question as to the capacity of the structure to handle settlement.- information request

Seepage Control

Sec2Pg7 suggests that the limits to closure they are using is less than 5 lugeons

Foundation Grouting is discussed in Sec2.3, but no grouting arrangement is described. Reference is made to a Method Statement in Sec14 of Vol 3 of their report

Foundation Drainage

No information found

Piping Control

No information found

Downstream Erosion Protection

Outlet Works Area

Downstream erosion protection is proposed in the Basalt by the use of 3m long dowels at 1.5crs with 5m dowels substituted in every third row. Support between the dowels should be provided by steel mesh pinned by the dowels. Mesh should be removed progressively as RCC is placed.

A similar method is suggested for areas in the goodnight beds

Approach Channel Area
In areas where basalt is found, a slope of 1:1 has been adopted, mainly because of the requirement for aggregate materials. From a constructability point of view, preference would be for 5m vertical rises with 5m wide berms to achieve the required slope

In the areas of the goodnight beds, support shall be provided by the use of 3m dowels in a 1.5 x 1.5 pattern and shotcreted.

TailRace Channel
A similar methodology is proposed for this area (Sec2 Pg 21)

Comments

The foundation levels and profile adopted seems reasonable but some concern does arise with the decision to found on the basalt which overlies an alluvium seam (Ref Sec 2 Pg 9) on the right abutment.

There appears to be reasonable concern regarding the sub-basaltic alluvium by SMEC, and the method of treatment including grouting and underpinning could prove expensive.

The suggestion to over-excavate the approach channel area to win basalt (for aggregate) conflicts somewhat with the decision to leave the right abutment basalt in place

The uncertainty associated with the basalt flow on the right abutment would suggest at least partial removal as compulsory – especially in areas of vesicular basalt. With incomplete geotechnical data in this area, it would be unwise to leave the material in place, and a risk analysis to prove otherwise would be based on incomplete data. The basalt that is removed could go to the crusher for aggregate production, so not all of the material is wasted.

A request for more detailed information on how and why the decision was made to leave the basalt flow in place would not be unreasonable. Also some further information on the proposal for a grout curtain, if any would seem appropriate.

No mention is made of the suggested landslide feature in the left abutment as reported in the URS report. Request for further information not warranted at this stage.

<u>A condition of issue of the development permit could be</u>:
Provide details as to how sub-basaltic alluvium on the right abutment is to be accommodated if any of it is to remain in place for the final design.

SPILLWAY & SPILLWAY HYDRAULICS (Sec 4)

General

The spillway hydraulics section describes the methodology used to determine the design head and hence crest profile of the ogee shaped spillway.

Design criteria are based on:

- PAR is 235 therefore SMEC assumed a High "A" category rating
- Dam is therefore required to be safe at the PMP design flood (94,000m^3/s)

The decision to construct the entire dam in RCC meant that it was possible to allow overtopping, especially if the crest levels were lowered and acted as secondary spillway sections.

With the spillway length set at 315m and Crest EL 67.6, the following are complied with Ref Sec1 Pg 4 General Arrangement):

- For the 1:100AEP, the storage level afflux remains below the control contour for land acquisitions resumed by Burnett Water
- The 1:1.5 yr environmental release flow is met even with the reduced spillway length

The ogee crest profile is described in detail on Sec4 Pg 3. A profile was determined based on the design head of 12.5m. The design head was based on 70% rather than the traditional 75% of max head (max head 17.9m) – this resulted in a design head of 12.5m, and a spillway length 315m. *(Claims that the SunWater approach in using an extrapolated value of head to design head gives erroneous results are unfounded. It should be remembered that this value is incorporated with other ratios of C/Co before a coefficient is determined, and this extrapolation will have minimal effect to the overall result. Additional checks are also carried out in the SunWater approach to ensure that the final value of C does not exceed an accepted upper limit)*

Apron & End Sill

Apron width of 20m adopted, with a 1m high end sill

Abutment Walls

Abutment walls are to be radius to improve the efficiency of the flow over the spillway

Secondary Spillway

Secondary spillway is located on the right bank, with the outlet works and fishlift located within the secondary spillway section.

Total length – including the outlet works and fishway is 505m, with crest EL78.0m. A stilling basin has also been provided downstream of the secondary spillway

Comments

Cavitation is discussed in the Design section of the SMEC report (Sec5 Pg17)

There are some areas that may need clarification:

- *Sec4 Pg 2 states there the flood routing calcs show that the PMP flood event could be reduced from EL92 to EL85.5 or EL87m depending on crest level and secondary spillway. Pg 10 of the Hydrology report suggests that the tabulated data is based on the adopted profile which gives a max reservoir level of EL87.46 for the 1:30,000 event and EL88.45 for the PMF – neither of these values resemble those stated in the Spillway hydraulics section.*
- *Sec4 Pg 1 states that the PMP design flood for the dam is 94,000m3/s. The hydrology section describes a 1:30,000 event as 94,861m^3/s with a PMF of 106863m^3/s – is it to be assumed that the 1:30,000 event the PMPDF event?*

There could be a case here for the request to clarify these "apparent inconsistencies". Information request

SEISMIC LOADING (No section dedicated to this topic)

General

Information was retrieved from the Seismology research centre (Sec 5 Pg 5), and the following values adopted

- Operating Basis earthquake – 500AEP – PGA 0.035g
- Maximum Design Earthquake – 1:10,000 AEP – 0.2g

Comments

The values adopted by URS are 0.17g and 0.5g respectively.

An explanation as to the derivation of the SMEC values or why there is such a variation would be desirable.- Information request

DAM DESIGN (Sec 5)

General

The proposed arrangement includes a membrane over the upstream face of the primary spillway section and extending nominally 1.8m (one precast panel height) above FSL on the secondary spillway and abutment sections. A drainage system will be installed behind the membrane, however, no gallery or foundation drainage system is proposed. (ref Sec5 Pg 5)

The membrane is protected by a layer of precast panels which are connected to the upstream face of the RCC

SMEC have also decided to eliminate the need for a gallery. Their argument is based on their findings that the dam is stable even without the gallery, and owing to the high tailwater levels, the drainage system that is incorporated into a gallery becomes relatively ineffective and hence uneconomical.

Load Cases

Load cases considered:

Load Type	Load Description
Normal	• Water at FSL (EL67.6m) • TW at EL32.0m • 50%uplift for membrane sections • 100% uplift for non-membrane sections
Unusual	• Water at FSL • TW at EL32.0m • U/S membrane failure – 100% uplift for all sections
Unusual	• Water at 1:1,000 AEP (EL78.0m) • TW EL 60.0 • 50% uplift for membrane sections • 100% uplift for non-membrane sections
Unusual	• WL at 1:10,000 (EL 83.0m) • TW EL70.0m • 50% uplift for membrane sections • 100% uplift for non-membrane sections
Extreme	• WL at PMP DF (EL 87.5m) • TW EL78.0m • 50% uplift for membrane sections • 100% uplift for non-membrane sections
Extreme	• MDE (0.2g) • WL at FSL (EL67.6m) • TW EL 32.0m • 50% uplift for membrane sections • 100% uplift for non-membrane sections

Sec5 Pg 4 describes the PMP Flood level as EL87.5m.. This agrees generally with info in the Hydrology Section.

Silt loading of a level equivalent to EL38.0m was used in the calculations (URS used EL 49) *No basis for the silt level was given in the SMEC* report

The allowable factor of safety values used in the analysis are as tabled below: (Ref Sec5 Pg 8)

Load Combination	**Shear Friction FOS**	**Overturning FOS**
Normal	3.0	1.5
Unusual	2.0	1.2
Extreme	1.3	1.0

The maximum allowable Design RCC Stress is outlined below:

Load Combination	**Compressive Stress (MPa)**	**Tensile Stress (MPa)**	**Cohesion (MPa)**
Usual/Normal	3.3	0	**
Unusual	5.0	0.4	**
Extreme	10.0	1.6	**

** Values not quoted in SMEC design report

Material Properties used as listed on Sec5 Pg 7 of SMEC Report
RCC stresses are based on a concrete compressive strength of 10MPa. Shear strength - friction angle of 45 deg and effective cohesion of 100kPa to 800kPa in concrete and 200kPa in the foundation

Analyses

The gravity method of analysis was used for the static load cases and the method developed by Fenves and Chopra for the earthquake analysis.

Stability analysis was undertaken on all sections for all load combinations (Sec5 Pg 11)

For the analysis as described (SEC5 Pg16) anchoring was required for the upper portion of the mono for the Unusual and extreme load cases.The anchoring described is a passive anchor, N36 bars at 0.75m above EL 64.0m. Anchors were extended from El 58 down to EL 57 for stress development.

Pg 17 of the SMEC report does report on the negative pressures that will occur on the crest of the spillway during flood events such as 1:10,000 and higher.

The approach adopted by SMEC was anchoring the crest using the N36 bars at 0.75m crs

Mono Joints – spacing at 45 metres – this is based primarily on thermal analysis results. (Ref Sec 6)

The downstream face
The downstream face was designed to be clad in conventional reinforced concrete. Sec5 Pg 35 describes the reasoning behind this – *there is no problem in adopting this approach except the constructability is time consuming.*

Apron Slabs
Apron slabs are proposed to be constructed of RCC. Sec5 Pg 39 provides the information as to why this option was taken. The slabs in the primary spillway are centrally reinforced in the top 300mm layer, and anchored in a 4.5m grid pattern, 3m into the rock foundation

Comments

The decision to eliminate the gallery is interesting in that it breaks with a traditional practice. However, the plan to remove the need for a gallery has already been agreed to by the Dam Safety Regulator (Ref Sec5 Pg 2).

It would be worth asking the question – how do they plan to monitor uplift in the foundation if they do not have a gallery.- Information request

From the URS report, their expert review panel suggest that while a gallery may not be required for stability (drains) in this case, other considerations dictate that a foundation gallery should be incorporated into any well designed large concrete gravity dam. They include:

- *Access across the dam to internal equipment*
- *Access to foundation grouting either during or post construction*
- *Safety inspections of the condition of the dam*

It could be argued that if there is no internal equipment, and safety inspections can be done externally, then the gallery is not required. Post construction grouting could be done from the top of the structure. Gallery grouting requires the gallery to be a reasonable cross section, and this is not always the case in some gravity dams.

Although that seems to reinforce the argument that a gallery can be eliminated, I have not cited in SMEC's analysis where they reduced the tailwater effect in their analysis owing to retrogression caused by the hydrodynamic effects of the plunging jet. Maybe this should be queried, and hence a more convincing argument on why the gallery should be eliminated. There was some discussion on sensitivity to the tailwater level, with reductions of up to 3m examined. (Sec5 Pg 18).

A request for information on allowance for the hydrodynamic effect is suggested – Information request

Use of the upstream membrane would be a significant change in the RCC design in Australia. By using the membrane, it is suspected the mix can be proportioned less in the area of cementitious material – this will make the structure slightly more flexible, and less heat will be generated during the curing process.

The membrane is placed behind some permanent formwork on the upstream face – this has advantages as well as disadvantages, but overall, the system appears fit for purpose

The method of seal into the conventional concrete has obviously been a point of prolonged discussion by the Alliance group. (Sec5 Pg 30) The methods described seem appropriate but possibly difficult to achieve (2nd level)

Have not read anywhere the method of transition from membrane to non-membrane sections – information request

Anchor bars were provided to counter uplift and cavitation – A concern would be the long-term effectiveness of these anchors, especially considering the protection of the anchor bar itself – is it galvanised/stainless – and if so, how does this protection method cope with the water in that area.

The apron slabs in the secondary spillway are unreinforced and unanchored – some nominal reinforcement would seem more appropriate.

It is interesting to consider the abrasion loss of RCC as opposed to conventional concrete and if their information is accurate, ask the question why this material cannot/has not been used in other areas of high abrasion such as the outlet works – there are obvious reasons why but maybe some consideration to improved mix designs?

RCC MIX DESIGN (Sec 6)

General

The RCC mix design has not been finalised for this design, but significant progress has been made towards identifying the optimum design.

There is no statement in their report re the criteria that has to be met. The current thinking though is that mixes with only 65-70kg of cement per cubic metre will suffice – one mix that appears to be suitable is 70-00-4.7 (ie 70kg/m3 cemetitious, 0% flyash, 4.7%moisture content

The main reason for avoiding the use of ash is that it causes RCC to become brittle in the long term. Other less significant reasons include:

- Concern of availability of consistent material
- Minimising truck traffic to the dam
- Simplicity in proportioning and mixing
- Eliminating the potential of putting flyash where cement should go.

A complete table of the RCC material properties is included in Appendix D of their report.

The modulus of elasticity is an important property for RCC design. A low modulus is desirable because the RCC can deform more than RCC with a high modulus. Lean mixes are far less brittle than the high cementitious content RCC

The thermal analysis thermal properties and schedule for construction were not reviewed as this time (Sec6 Pg 21)

Comments

From a regulators point of view, no comment, but the use of the membrane will obviously be grounds for future discussion.

OUTLET WORKS (Sec 7)

General

The outlet works are designed for three types of releases

- Irrigation releases – 17.8m3/s
- Environmental releases – 240m3/s
- Sediment removal releases

Two of the three 5.2m x 4.7m culverts used for river diversion will be used as the low level outlet conduit. Conduit # 1 will be used for environment purposes, while conduit #2 for irrigation releases.

Intake Tower

The intake tower is a 48m high structure, top EL 78which is the crest level of the secondary spillway.

The tower is a wet well, with horizontal cross beams at 5m crs for structural stiffness. Coarse and fine screens are fitted with a $20m^3/s$ capacity. The fine screens and shutters are removed for the environmental releases.

The intake structure includes shutters, the operation of which is by permanent winch arrangement. (Sec7 Pg 14). The shutters and fine screens will be operated by a fixed electric wire rope hoist (hoist is 12t capacity).

A sealing fixed wheel gate is located at the bottom of the tower (Ref Sec 7 Pg 1) – the fixed wheel gate acts as a guard gate for the radial gate, and allows the conduit to be drained for inspection. The guard gate is designed to shut under a differential head, but is not designed as a regulating structure. The guard gate is lifted by a hoist of estimated capacity 65tonnes. – this is required because of the significant pulldown forces on the guard gate during an emergency closure.

For design purposes, a maximum design pressure head difference of 2m has been adopted between the external and internal face of the walls (Ref Sec7 Pg9)

The critical design load combination cases considered for the ultimate limit state design of the tower are:

- Foundation bearing
  - 1.35 self weight (SW)
- Intake tower walls
  - 1.2SW + 1.2 external hydrostatic pressure
  - 1.0SW + 1.0 seismic
- Supporting elements in tower
  - 1.2SW + 1.5equipment live loads
- Connection to dam wall
  - 1.0SW + 1.0 seismic

The critical design load combination cases considered for the structure for serviceability limit states are:

- Intake tower walls
  - 1.0SW + 1.0 external hydrostatic pressure
- Supporting elements in tower
  - 1.0 SW + 1.0 equipment live loads

Separate stability limit state load combinations were not considered for the tower since its stability is controlled by the stability of the dam wall

Outlet Conduit

Two of the three 5.2m x 4.7m culverts required for river diversion during construction of the dam will be used as the low level outlet conduit

One conduit will be modified for irrigation releases. A 2.2m dia steel pipe will be retrofitted into the conduit for that purpose. To protect the pipeline, and act as a guard valve for the cone dispersion valves, a self closing 2.2m dia butterfly valve will be provided immediately downstream of the plug. This valve will have a design capacity to close against 11m/s flow.

A second conduit will contain the 5.2 x 4.0m radial gate

Comments

Dwgs Team1-dwg 303 show details of the irrigation outlet arrangement. It is not clear how maintenance will be provided to the 2 guard valves that are located toward the upstream end of the 2200dia pipe. Also, is there any provision for manholes in the pipework, or is access via other means. – information request

Structural design of the outlet works appears to be adequately catered for Ref Sec7 Pg 8. Loading combinations and factors appear standard practice and are as listed on Pgs 9 – 12 for the various components. However, the tower and components in the tower are designed for a max differential head of 2m – this is not considered adequate – Information Request

It is suspected that the shutters are raised to the invert level of the intake area. Water above that level is drawn off – the water at the invert level then being exposed to higher velocities and hence the tendency for water at or near that level to be drawn off in preference to water at say reservoir level

Did not see any comments on expected maximum flow rates in the 2.2m dia steel pipe, the thickness of the pipe, or its internal lining.

FISH LIFT DEVICE (Sec 8)

General

The passage of fish in the upstream direction is managed by the use of a fish-lift device – basically it is achieved by the use of a hopper, of 7500l capacity that runs up the back face, across the crest and down the front face of the dam. Attraction flow of 0.8m/s is used to attract the fish into the hopper.

The hopper travels at around 12m/min, and with the use of proximity switches and encoders, the transition through the various slopes and grades is expected to be smooth.

The downstream fish passage arrangement operates between EL67.9 and EL62.5. The attraction time is quoted as 10min, descent time 5min, sluicing 3min, and refilling 4min, giving a total cycle time of around 27min. *The cycle time as tabulated in Sec8Pg 23 is stated as 19min – obviously an overlap there?*

The method used to transport the fish downstream is similar to a standard "SunWater" fish lock mechanism with the exception that the arrangement uses smaller dia pipes in the exit area (750mm)

Comments

The fishway as proposed by SMEC is an interesting concept, and does not directly impact on areas of concern to the Dam Safety Group. Hence the comments here are purely that, comments from an engineering perspective rather than any regulatory role.

The claim that the upstream passage of fish via this method is less stressful to the fish, is easy to monitor and has moderate maintenance requirements is questionable (Sec8 Pg5). Maintenance issues associated with this device may have been severely underestimated. Cycle time was not identified, although the trapping time was quoted at approx 20min (Sec8Pg 23 states cycle time at 29min). An O&M person should look at this proposal

The reasons they described for not selecting a fish lock device for the upstream passage seem to have been ignored for the downstream passage arrangement – this may have been associated with the theory that upstream passage is usually for the smaller fish, and downstream for the larger sized specimen.

The vulnerability to structural damage of hoist structure and hopper and trolley may be underestimated

DIVERSION & CONTROL OF STREAM (Sec 7)

General

The diversion channel is located on the right abutment and is provided thru three 5.2 x 4.7m high culverts . The number and size has been selected to ensure adequate flood protection of the construction site. (Sec 7 Pg 3)

Comments

Although the diversion seems adequate, there is no detail as to what immunity the system provides. Also, as construction progresses, the number of culverts available for diversion changes as some culverts are fitted out for environmental and irrigation purposes.