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South East Queensland Water Corporation Limited

Report for Wivenhoe Dam Full Supply Level Review

Technical Assessment of Raising Potential

December 2009

INFRASTRUCTURE | MINING & INDUSTRY | DEFENCE | PROPERTY & BUILDINGS | ENVIRONMENT

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1. Introduction

1.1 Background

SEQWater has investigated options for the provision of additional storage in Wivenhoe and Somerset Dams as part of the ongoing investigations for the Regional Water Supply augmentation. An initial feasibility report prepared for NRW by SEQWater has identified the potential for raising the Full Supply Level of Wivenhoe Dam by 2m. As part of the feasibility study, SunWater investigated the impacts of this proposed raising on the flood passing capacity of the dam, which indicated that the 2 m raising would not significantly impact the flood passing capacity.

The Dam Safety Regulator, has requested that SEQWater carry out a structural review of the dam to assess the impacts of the raising on the main structural components of the dam. The structural review is to be a desktop review / analysis of previous investigations and design reports to assess the ability of the dam to safely manage the increase in the operating level.

1.2 Scope of works

A report on the impacts of the raised FSL on flood operations has been prepared by SunWater (2007) and will be used to obtain flood levels for the various AEP events in order to assess risks associated with piping for the dam embankment and loading of the dam components as required.

The scope of works as detailed in the brief included the review of the following components with no additional analyses.

- The central earth core main embankment including foundations, grout curtain, embankment zones (in particular filters and clay core), wave wall cut off, and the interface with the abutments at either end of the embankment. The review is to address the embankment stability (rapid draw down, flood, earthquake and normal load cases) and piping risk.
- The upstream sloping core left embankment including foundations, grout curtain, embankment zones (in particular filters and clay core), the wave wall cut off, and the interface with the existing spillway abutments at the right hand end of the embankment. The review is to address the embankment stability (rapid draw down, flood, earthquake and normal load cases) and piping risk.
- The two saddle dams located on the left hand side of the dam. The increased operating level will not apply additional load to the dams. The impact of the raised operating level on piping risk is to be evaluated.
- The secondary spillway located through the right hand side abutment of the dam. The review is to include the fuse plug embankment stability and piping risk, the concrete control crest and the concrete wall lining.
- The existing gated spillway located between the two embankments. :The review is to include:

- Inlet works penstock gate, trash racks, penstocks (3.6m and 1.9m), control valves and the hydro power station;
- Concrete gravity inlet training walls on the left and right hand side of the spillway;
- Concrete gravity crest units and piers supporting the road and service bridges;
- Radial gate components including the skin plate, cross girders, trunnion girders, trunnion pins and bearings, concrete corbel, and pier post tensioning;
- Mechanical and electrical equipment for operation of the radial gates including, the hydraulic winches, cables, hydraulic lines and controls.

2. Methodology

Our methodology comprised the following steps.

Start Up discussion with SEQWater, preliminary review of reports and site visit and discussion with Operators conducted over two days on 29 and 30 May by Malcolm Barker (Principal Engineer Dams), Jon Williams (Manager Dams), Barry Vivian Principal Mechanical Engineer) and Toby Loxton (Senior Hydrologist) of GHD.

Review of reports obtained during the site inspection, ones that GHD have in our library and additional reports including the Wivenhoe Upgrade Design report received 27 October 2008.

The reports and data were analysed to determine the following:

- Detailed description and design loads for the components including the safety factors or compliance with relevant guidelines or codes;
- Increased loads and assessment of the revised safety factors;
- Evaluation of the operational impacts of the raised FSL on the components

The above data was used to determine the acceptability of the proposed raising of the operation level by 2m from RL 67 to RL 69.

3. Description of Dam and Associated works

Wivenhoe Dam is a 56 m high, zoned earth and rock embankment separated into two parts by a concrete gravity spillway, controlled by 5 radial gates, each 12 m wide by 16.0m high. Two saddle dam embankments are located on the left side of the reservoir. The Brisbane Valley Highway passes over the dam.

The dam has four main functions by providing:

- A storage of 1.165 GL at full supply level (FSL EL 67.0m AHD) providing a safe water supply for Brisbane and surrounding areas;
- Flood mitigation in the Brisbane River with a dedicated flood storage volume of 1.45 GL up to EL 77.0m AHD (the Maximum Flood Level was increased to EL 80m AHD as part of the Wivenhoe Alliance Upgrade works in 2005, changing the flood storage volume to 2.0GL at EL 80m AHD);
- The lower pool for the Split Yard Pumped Hydro-Electric power station, which has a 500 MW generating capacity;
- A recreation area.

The dam was designed by the Queensland Water Resources Commission and a design report is available (DPI, 1995). It was constructed by a consortium of contractors between 1977 and 1985, supervised by the Commission.

The Wivenhoe main embankment is located on the right hand side of the centrally placed spillway. The 1.2 km embankment is a 56 m high central clay core embankment with both upstream and downstream filters supported by outer shells of compacted sandstone with river run gravel in the upper portion. The shoulder slopes are 2 horizontal to 1 vertical with a local steepening in the upper portion to 1.5 horizontal to 1 vertical. Riprap was provided on both upstream and downstream shoulders.

To the left of the spillway structure, the embankment has a sloping upstream core protected by both upstream and downstream filters and supported by a downstream shell of miscellaneous fill. Batter slopes are 3 horizontal to 1 vertical on the upstream face and 2 horizontal to 1 vertical on the downstream face. Riprap was provided on both upstream and downstream shoulders.

Two saddle dams close off low saddles on the left abutment of the dam. Saddle Dam 1 is a homogeneous embankment constructed from miscellaneous fill. Saddle Dam 2 is the higher of the two embankments and is constructed with a central clay core and random fill shoulders. Rip Rap is provided for both embankments on the upstream face for wave protection and the downstream slope is topsoiled and grassed, They have a crest level at EL 80m AHD and have a maximum height of 10 m. The saddle dams only retain water during flood operation with Saddle Dam 1 having an upstream foundation contact level of RL 73 and Saddle Dam 2 having an upstream foundation contact level of RL 72.

The spillway is located in a low saddle between the two embankments and is controlled by 5 radial gates supported on a mass concrete ogee crest. The radial gates are 12m wide by 16m high and discharge via a flip bucket spillway to an unlined rock discharge channel.

The five 12 metre wide by 16 metre high radial gates in the Wivenhoe spillway structure are operated by hydraulic motor driven wire rope winches, one on each side of each gate. The power units (2) for the spillway gates and penstock gate are located in a winch room in the left abutment of the dam. Also located in this winch room is an auxiliary diesel operated hydraulic unit capable of operating the gates.

The dam has an EXTREME hazard classification (according to current ANCOLD guidelines) because of the significant development downstream in the Brisbane and Ipswich metropolitan areas, with the population at risk (PAR) numbering in the hundreds of thousands.

The original spillway capacity, with an Annual Exceedance Probability (AEP) of 1 in 22,000, was well below current standards for an Extreme hazard dam. The Wivenhoe Alliance was formed by SEQWater to improve the flood security with a long-term goal of providing adequate spillway capacity to pass the Probable Maximum Flood (PMF). Investigation studies concluded that the two-stage upgrade program outlined below would provide a cost-effective risk reduction program.

- Construction of a new secondary spillway on the right abutment that would enable the dam to handle an inflow flood with an AEP of 1 in 100,000 at a Maximum Flood Level (MFL) of EL 80m AHD. The spillway is to be controlled by three fuse plug embankments in a 164m wide secondary spillway in an excavated chute that included concrete works for a 3m ogee crest to RL 67, apron slabs, chute lining and the divider walls;
- Upgrading of the embankment crest to retain a MFL of RL 80m AHD with zero freeboard by upgrading the existing concrete crash barrier to act as a water retaining structure;
- Upgrading of associated structures as appropriate, including protection of the gates and Spillway Bridge and strengthening of the spillway gravity structure with post tensioned anchors. In addition, provision of a steel deflection baffle upstream of the radial gates was provided to ensure the gates clear the flow profile for the raised MFL.

This Stage 1 upgrade changed the dam crest flood from a 1 in 22,000 AEP event to 1 in 100,000 AEP flood event. The initial trigger level for the first fuse plug embankment is at EL 75.7m AHD (approximately the 1 in 6 000 AEP flood event).

4. Data Review

4.1 Spillway Gate Operation and Flood Hydrology

This review is based on the following three documents:

- Wivenhoe Dam Spillway Augmentation Works, Volume 5, Design Discharges and Downstream Impacts of Wivenhoe Dam Upgrade Report, Report No: Q1091 WIV-RP-HD-004, prepared by Wivenhoe Alliance, September 2005.
- Assessment of Wivenhoe Dam Full Supply Level on Flood Impacts Report prepared by Sunwater, December, 2007.
- SEQWater Manual of Operational Procedures for Flood Mitigation for Wivenhoe Dam and Somerset Dam, prepared by South East Queensland Water Corporation, Version 7, 6 June, 2007

The Wivenhoe Alliance report documents and summarises the history of Wivenhoe Dam, and describes the assessment and design processes that were undertaken to upgrade the dam to pass the 1 in 100,000 Annual Exceedance Probability (AEP) primarily through the construction of a three-bay fuse plug spillway on the right abutment in 2005. This assessment led to a recent revision of SEQWater's Manual of Flood Operational Procedures (referred to as the Flood Operations Manual in this review) to incorporate the fuse plug spillway in their procedures. In late 2007, Sunwater investigated several options for raising the Full Supply Level (FSL) for Wivenhoe Dam to determine the likely impact on flood routing performance.

These three reports were the primary documents reviewed for this study. This hydrology review provides a summary of the Flood Operations Manual, an overview of the flood routing performance of the current dam, the flood routing impact of raising the full supply level, and the findings.

4.1.1 Flood Operations Manual Summary

The Manual of Operational Procedures for Flood Mitigation for Wivenhoe Dam and Somerset Dam (SEQWater, 2007) contains the management rules for both dams during flood events. Wivenhoe Dam and Somerset Dam are operated in conjunction so as to maximise the overall flood mitigation capabilities of the two dams and the procedures are based on the operation of the dams in tandem. The auxiliary spillway at Wivenhoe Dam works in conjunction with the existing gated spillway. The design intent of the auxiliary spillway is to try and ensure that the gates are fully opened by the time the first fuse plug is initiated. This is on the basis that the discharges through the existing spillway will result in less damage than allowing discharges through the auxiliary spillway.

While Wivenhoe Dam has the capacity to mitigate the flood effects of a Somerset Dam failure, in the absence of any other flooding, Wivenhoe Dam could be overtopped and destroyed by Somerset Failing during a major flood event. Current estimates of extreme floods indicated floods are possible that could overtop both dams. In the case of Wivenhoe Dam such an overtopping would most likely result in the destruction of the dam itself.

There are four basic flood procedures once the water level in Wivenhoe Dam exceeds 67.25m AHD (i.e. 0.25 metres above full supply level): Procedures 1, 2, 3, and 4. There are a number of sub-procedures for Procedure 1, which aims to minimise flooding of downstream bridge crossings. Procedures 2 or 3 are applied if the water level in Wivenhoe Dam reaches 68.5m AHD with the aims being not to submerge Fernvale Bridge and Mt Crosby Weir Bridge prematurely and to try and regulate the peak flow at Lowood to less than 3,500 m^3/s. In the case of Procedure 3, an additional aim is to regulate the release from Wivenhoe Dam so that the peak flow rate at the Bremer River junction does not exceed 4,000 m^3/s.

Procedure 4 normally comes into effect when the water level in Wivenhoe Dam reaches 74m AHD. However, the Senior Flood Operations Engineer may seek to invoke discretionary powers if earlier commencement is able to prevent triggering of a fuse plug. Under this procedure, the release rate is increased, as the safety of the dam becomes the priority. There are two sub-procedures for Procedure 4 known as 4a and 4b.

Sub-procedure 4a applies when all indications of the peak flood level in Wivenhoe Dam are that the flood event will be insufficient to trigger operation of the first bay of the fuse plug by reaching a water level of 75.5m AHD.

Sub-procedure 4b applies once indications are the peak flood level in Wivenhoe Dam will exceed 75.5m AHD using the minimum gate opening intervals for normal operation. In sub-procedure 4b the minimum time interval between gate openings can be reduced and successive gate openings of the same gate may be made. The gates are to be raised to ensure they are out of the water before the initiation of the first fuse plug (if possible). Where practicable, the gates are to be in the fully open position before the dam water level reaches 75.7m AHD.

The Wivenhoe Alliance spillway augmentation works report indicates the invert level of the first fuse plug is 75.5m AHD and a water level of around 75.8m AHD is required to initiate this fuse plug (Wivenhoe Alliance, 2005).

4.1.2 Flood Routing Behaviour of Existing Wivenhoe Dam

Wivenhoe Dam commands a catchment area of 7,048 km^2 and stores approximately 1,165 GL at a full supply level of 67m on the Australian Height Datum (AHD). In 2005 the Wivenhoe Alliance completed construction of a three-bay fuse plug on the right abutment of Wivenhoe Dam and raising the main embankment crest by one metre to 80.1m AHD. These works increased the spillway capacity of the dam to pass peak flows from an estimated 1 in 22,000 AEP event to a flood with an AEP of approximately 1 in 100,000. Further work is proposed in the future to install a single-bay fuse plug at Saddle Dam 2, which is anticipated to enable the dam to convey the Probable Maximum Flood (PMF) (Wivenhoe Alliance, 2005).

The Wivenhoe Alliance used three computer models to predict the flood behaviour of Wivenhoe Dam:

- WT42D a rainfall-runoff routing model for development of hydrographs into Somerset Dam, Wivenhoe Dam, and downstream tributaries;
- WIVOPS to derive outflow hydrographs from Somerset Dam and Wivenhoe Dam for floods that do not initiate a fuse plug; and,
- FLROUTE to determine the Wivenhoe Dam outflow hydrograph for events that do initiate a fuse plug.

Their hydrological assessment used the latest design rainfall estimate and temporal patterns for large, rare, and extreme floods which were applied to the calibrated hydrological models. IEAust (1999) defines floods with an AEP between 1 in 50 and 1 in 100 as large floods while those events with an AEP between 1 in 100 and 1 in 2000 are known as rare floods. Floods with a lower probability than 1 in 2,000 are considered extreme floods and these types of floods have a high degree of uncertainty in terms of their magnitude and probability. That is, data from very few, if any, extreme events (such as the Probable Maximum Precipitation (PMP) event) have been observed.

According to the flood frequency curve for outflows from Wivenhoe Dam, the commencement of Procedure 4 occurs for events with an AEP of around 1 in 500, while floods with an AEP of approximately 1 in 2,000 are likely to trigger Sub-Procedure 4b (Wivenhoe Alliance, 2005).

The initiation of the first fuse plug occurs when the water level in the dam reaches 75.80m during an event with an Annual Exceedance Probability (AEP) of approximately 1 in 6,000. The second and third fuse plugs are initiated at estimated AEPs of 1 in 11,500 and 1 in 22,500 respectively. The third fuse plug is initiated when the water level rises to 76.88m (Wivenhoe Alliance, 2005).

4.1.3 Flood Routing Impact of Raising the Full Supply Level

Sunwater investigated a number of options for raising the full supply level that included a one metre increase and a two metre increase together with the operating rules that applied at the time of the study and some modified rules that resemble those in the Version 7 of the Flood Operations Manual. For the purposes of this review, only the analyses using the "modified" rules were considered as these rules are very similar to those considered by Wivenhoe Alliance (Wivenhoe Alliance, 2005).

In order for Sunwater to investigate these options the WIVOPS program needed to be modified to accommodate other full supply levels as the current full supply level is "hard-coded" into the source code (Sunwater, 20007). Sunwater (2007) commented that there is little documentation available on WIVOPS and the code has been modified on several occasions to meet the changing dam configurations and to investigate changes in operational procedures.

Table 1 compares the AEPs derived by Wivenhoe Alliance and by Sunwater for the triggering of Procedure 4 (water level reaching 74m AHD) and for fuse plug initiation of the existing Wivenhoe Dam. This table shows there is good agreement for the AEP of those events that trigger Procedure 4. In the case of the fuse plug initiation, Sunwater predict a marginally higher probability than Wivenhoe Alliance. This difference is attributed to the modifications made to the WIVOPS program and that Sunwater did not use the FLROUTE program but chose to simulate the fuse plug performance through the modified WIVOPS program. The difference in results is not considered to be significant for the assessment of flooding behaviour for other full supply level options given the Sunwater result yields a slightly higher probability of occurrence.

Table 1 Comparison of Flood Routing Results, Existing Wivenhoe Dam

Full Supply Level (m)	Wivenhoe Alliance Estimate	Sunwater Estimate[a]
Triggering Procedure 4 (74m AHD)	1 in 500	1 in 430
Fuse Plug Initiation	1 in 6,000	1 in 4,500

[a]: Table 6-9 (Sunwater 2007)

Sunwater (2007) simulated the behaviour of a range of floods for a full supply level of 68m AHD, and 69m AHD with all gates working, plus the same scenarios where one gate was inoperable. The results of these analyses are summarised in Table 2. This table illustrates how the likelihood of triggering of Procedure 4 and of fuse plug initiation increases as the full supply level increases due to the loss of currently available flood storage.

Table 2 Flood Routing Results for Full Supply Level Options (Sunwater, 2007)

Full Supply Level Option	Triggering Procedure 4 (74m AHD)		Fuse Plug Initiation	
	All Gates Working[a]	One Gate Inoperable[b]	All Gates Working	One Gate Inoperable
67m AHD (existing)	1 in 430	1 in 550	1 in 4,500	1 in 3,500
68m AHD	1 in 330	1 in 400	1 in 3,500	1 in 2,500
69m AHD	1 in 100	1 in 200	1 in 2,300	1 in 1,800

[a]: Table 6-9 (Sunwater, 2007)

[b]: Table 6-10 (Sunwater, 2007)

Table 2 also illustrates the impact of one gate being inoperable during flood events and in the case of triggering Procedure 4, the probability is reported as marginally decreasing which is not considered a reliable result. If one gate is not working the flow rate past through the spillway is reduced (if other gate openings are not increased to compensate) and the water level should increase, and the flood event probability that this occurs should also increase, or be the same at best. The results for the first fuse plug initiation seem to be reasonable.

Sunwater (2007) commented that under the current operational procedures, with one gate inoperable, the opening of the remaining gates is adjusted to achieve the same discharge. This cannot be carried out in the exiting WIVOPS which has the number of available gates (five) hard-coded into the program. As a first pass, the input rating for one gate was reduced by 20% to account for one gate inoperable. It was recognised by Sunwater that this produces an overly conservative result up to a water level of 73m AHD at which level the inoperable gate would be overtopped. This is the probably reason for the anomalous result for the triggering of Procedure 4 AEPs.

The analyses demonstrate that only large to rare floods are still likely to trigger Procedure 4, and that only rare floods are still likely to initiate failure of the first fuse plug for either all gates working, or with one gate inoperable during the flood events. For a full supply level of 69m the analysis showed that the fuse plug is likely to be initiated at the upper end of the rare event category. By way of comparison, the retro-fitted auxiliary fuse-plug spillway for Warragamba Dam s reported to be initiated by a flood event with an AEP of approximately 1 in 750 (Wivenhoe Alliance, 2005).

Like the Wivenhoe Alliance, Sunwater did not consider a joint-probability analysis as part of their simulations. Both parties assumed the dams were full at the onset of the flood events which is considered to be suitably conservative approach for design works and for options investigations as the rainfall totals for a given probability tend to increase over time as more large and extreme rainfall events are recorded and subsequently used to refine rainfall distributions. This trait is highlighted when the Probable Maximum Precipitation (PMP) design estimates are considered for Wivenhoe Dam as noted by Wivenhoe Alliance (2005) that the 1977 estimate for the 48 hour storm was 480mm and in 2003 this estimate increased to about 1,050 mm.

A joint-probability analysis is most likely to result in lower AEPs than those reported here as that type of analysis takes into account the probability of the factors such as dam water level at the time of the event, the antecedent moisture condition of the catchment, the temporal pattern and duration of the storm, and the infiltration rate of rainfall into the soil. Defining the probability distributions for several of these factors (e.g. antecedent moisture, infiltration rates, and dam water level) is likely to be difficult, particularly for extreme flood events as very few of these events have occurred and data is limited to allow for an accurate definition of their domains.

4.1.4 Findings

Based on the modelling of a one metre and two metre raise of the full supply level undertaken by Sunwater, it appears that only large floods will trigger Procedure 4, and that only rare floods will still initiate failure of the first fuse plug for either all gates working, or with one gate inoperable during the flood events. That is, raising the full supply level of Wivenhoe Dam is not considered to result in an excessively higher probability of triggering Procedure 4 operation rules or initiation of the first fuse plug compared to the flood routing behaviour of the existing dam.

Sunwater (2007) noted a number of issues regarding the adopted methodology, assumptions, and the use of WIVOPS and made a number of recommendations. Should SEQWater proceed with detailed design for raising the full supply level of Wivenhoe Dam it is recommended that Sunwater's recommendations be addressed.

Additional reference:

IEAust, Australian Rainfall and Runoff (AR&R), A guide to flood estimation, Volume 1, The Institution of Engineers Australia, Barton ACT, 1999

4.2 Spillway Gates structural, mechanical and electrical

Key levels for the gates are summarised below:

- Fixed Concrete Spillway Crest: EL 57
- (Present) Existing FSL: EL 67 AHD
- Proposed FSL: EL 69 AHD
- Top of Gate: EL 73
- Max Flood Level: EL 77
- Nominal Crest Level: EL 80

Based on a requirement to attenuate a substantial flood volume, the top of the gates (EL 73) were designed much higher than the dam FSL (EL 67).

The gates were designed for the FSL of 68.5 AHD although the FSL for efficient operation of Wivenhoe Dam Hydro-Electric Power Station is EL 67m [Wivenhoe Dam Design Report, Volume 1 – Text, Sept 1995, Sect 23.2, p132]

According to the Loading Conditions and Design Stresses, Condition 1 – Water to EL 73 (top of gate), Gate Shut, even though this scenario prescribes a dam level that is 4.5 metres above the design FSL (EL 78.5) and will only be achieved during flood mitigation operations, it was considered to occur frequently enough to warrant being used as the normal design case [Wivenhoe Dam Design Report, Volume 1 – Text, Sept 1995, Sect 23.6, p134]

The load in this condition = 1.2 x (Dead Load + Live Load) and adopted allowable stresses of 0.50Fy as per US Corps of Engineers, as opposed to AS1250 0.60Fy.

For the purposes of this design the possible future long term design FSL was taken to be EL 68.5m AHD. The initial FSL will be EL 67.0m AHD. [Report on Wivenhoe Dam Design of Crest Control Structures, Apr 1995, Sect 1.0, p1]

A review of the seismic report revealed that only the trunnion pedestal hold down bolts reach yield under Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) loading. [Wivenhoe Dam, Report on Seismic Assessment of the Radial Gates, Piers and Bridges, August 2000, p88]. The use of Grade 4.6 hold down bolts for the trunion pedestals would not meet current standards and a revised analysis is recommended to evaluate the proposed raising.

The information provided in the above mentioned design reports indicates that raising Wivenhoe Dam's FSL (from EL 67m) by 1.5m to EL 68.5 can be undertaken as the dam (in this case specifically the gates) was originally designed to be operated at a FSL of 68.5m and that this was only reduced to EL 67m as a means of efficiently operating the hydro plant.

Furthermore, given the fact that Load Condition 1 of Water to EL 73.0m, Gate Shut (4.5m over EL 68.5) was adopted as the normal design case, this tends to indicate the gates can be operated as per normal at EL 69.0.

The information taken from the various Wivenhoe Dam design reports support the assessment that the Wivenhoe Dam radial gates can still be operated as per normal should the FSL be increased by 2m from EL 67.0m to EL 69.0m.

4.3 Spillway Crest Bulkhead Gate

The spillway crest bulkhead gate is used as a maintenance bulkhead in front of the radial gates. It has however also been designed as an emergency gate to close into full flow should one of the radial gates fail in the raised position.

The bulkhead gate has been designed for a water loading to EL 69.0 (0.5m over the design FSL of EL 68.5). This level is also the top of the gate in the closed position. The gate has not been designed to be overtopped, although the horizontal beams only reach their yield stress at a water level 1.65m above the top of the gate. This level represents a static head and does not include for impact from overtopping water on the beams. The maximum combined stresses in the girders with the water level at the top of the gate, are below the allowable 75% of yield stress for the material used. This indicates that it may be possible to modify the gate with a low wave board to prevent overtopping by wave action.

The bulkhead gate can however be used as is for the proposed new FSL at EL 69.0 if routine maintenance is scheduled for when the reservoir level is below EL 69.0.

The gate can also still be used as an emergency gate to safeguard reservoir storage in the event that one of the radial gates fail to close if the reservoir level is at EL 69 (Proposed new FSL)

It is, however, recommended that if the reservoir level is raised to EL 69.0, that a low wave board be added to the top of the gate. Review of the design calculations would have to be done to determine how high the wave board could be taken.

4.4 Selective Baulks

The selective baulks can be used to draw off water of selected quality through the outlet works. These baulks can be installed up to EL 71.0. The baulks have been designed for a differential head of 2m and can only be installed under balanced conditions. If sufficient water way has not been provided between the baulks, a control system prevents the downstream outlet valves from being operated. The bottom baulk is designed with a collapsible section which would open under a differential head of 1 m. With these safe guards in place, raising the FSL to EL 69.0 would thus not impact their function or operation.

4.5 Trash racks

The intake trash racks have been designed for a differential head of 1.5m. The differential head across the trash racks depends on blockage of the racks and not on the overall head. With the current operating and maintenance procedures in place, a raising of 2m would thus not present a problem.

4.6 Fixed Wheel Penstock Gate

The penstock gate has been designed for a FSL of EL 67.0m. This equates to a design head of 38m. The 2m increase in head would thus result in an additional 5% loading on the gate. It may be possible that the gate structure could accommodate this increase, but without a review of the design calculations this cannot be confirmed.

The higher head may also have an impact on the dynamic loading on the gate during emergency closure of the 3.6m penstock. The design was however done for a 30MW power station. The lower flows associated with the installed 4MW power station would therefore result in much lower dynamic effects. It is thus expected that the proposed 2m increase in head would not influence the dynamic loading on the gate.

It is recommended that the design calculations for the gate are reviewed to establish if the additional 2m raising would still be acceptable.

4.7 3.6m Diameter Penstock

The original design head for the penstock is not stated in the design report. Assuming though that it is the same as for the penstock gate, nl. EL 67.0m, the 2m increase in design head would result in approximately 5% increase in loading. The original design was however done for a 30MW power station with the associated surge pressures. This surge increased the normal design pressure to more than double the static head pressure. With the much smaller 4MW power station built in 2002, the expected surge pressure would be lower than the original design pressures. Taking into account also that the original design of the penstock disregarded the effect of the concrete surrounding the penstock, it is expected that the additional 2m head raising in FSL would not present a problem.

4.8 1.9m Diameter Outlet Pipe

The 1.9m diameter outlet pipe was designed for the same internal pressures as the 3.6m penstock. It makes allowance for a surge pressure resulting from a valve closure in 1.1 seconds. The only valve in the line is a hydraulically operated fixed cone dispersion valve. This type of valve cannot close in such a short time period. The original design pressures are therefore very conservative and an additional 2m in static head would thus be acceptable.

4.9 DN 1500 Fixed Cone Dispersion Valves

The fixed cone dispersion valves have been designed for a static head of 75m. The actual maximum operating head is only 37m assuming a FSL at EL 67.0m. The proposed 2m raising would thus be well within the capability of the valves.

4.10 Spillway Ogee, Piers and Retaining Walls

4.10.1 Spillway Stability

Factors of safety were determined and evaluated using FERC criteria (pg 24 of 65 q1091 WIV RP De 012 Spillway and Existing Section - Wivenhoe Alliance Report). This is consistent with the current draft ANCOLD guidelines as presented at the Draft ANCOLD Gravity Dam Design Guidelines Workshop held with the dams conference in November 2008.

- Sliding Usual 3, unusual 2, post eq 1.3 for cohesion
- Worst static 1.5, PMF 1.3 post eq 1.3 for no cohesion
- Overturning >1 with >1 for cracked analysis also.

The stability assessment under sliding was determined using foundation parameters of 100 kPa cohesion and 40 degrees.

The uplift distribution adopted is complex and reflects the unusual foundation geometry (refer pg 26 of 65 q1091 WIV RP De 012 Spillway and Existing Section - Wivenhoe Alliance Report) - zero drainage was assumed consistent with FERC guidelines.

Summary of factors of safety are repeated below. (refer pg 29 of 65 q1091 WIV RP De 012 Spillway and Existing Section - Wivenhoe Alliance Report)

- All Gates Open DCF level 1.55 or 1.15 (not stated but Sliding then Overturning FOS)
- 1 Gate Failed load transferred over 3 bays 1.25 and 0.96
- 1 gate failed load transferred over 5 bays 1.45 and 1.07
- Earthquake MDE with FSL 67 1.77 and 1.51

Note that there is no data on factors of safety for FSL conditions. These were requested of Richard Rodd (30 May 08) but no results have been received at December 2008. It is recommended that this data is obtained and reported. It should be noted that the Safety Review (GHD, Draft 1997, final 2002) reported satisfactory Sliding Factors and Shear Friction Factors for the FSL without post tensioning (albeit with tensile stresses which are not with current accepted design guidelines). Post tensioning will improve this stability.

4.10.2 Spillway Piers

The spillway piers were checked for additional loads from the baffle plate in the design of the spillway upgrade. As the baffle plates are symmetrical on either side of the pier the induced load is applied in the upstream and downstream direction. The induced loads were found to be well within the design capacity of the pier. The truss structure also provides significant transverse restraint to the pier for the critical load case of 1 gate down and MFL loading. (refer pg 17 of 65 q1091 WIV RP De 012 Spillway and Existing Section - Wivenhoe Alliance Report)

4.10.3 Fuse Plug Spillway

Filters are primarily designed to prevent the transition of soil between adjacent zones. In the trigger section, the coarse filter forms a large component of the embankment (downstream of the clay core) to facilitate rapid erosion of the embankment once the trigger section is breached (page 67-68 of 97 Options Selection and Concept Design Q1091 WIV RP De 009).

Filters have been designed in accordance with Sherard & Dunnigan (1984) and Fell et al (1992) with the clay core being a Type 2 soil (page 70 of 84 Right Abutment Auxiliary Spillway Design and Construction Report Q1091 Vol 7). The filters are therefore satisfactory for a water retaining structure, but it is recommended that some form of concrete protection is applied to the lower part of the fuse embankments to minimise seepage quantities with the water level raised by 2 m..

The embankment slope for both the upstream and downstream slope is 1V to 1.75H. This slope was selected based on stability assessment. A timber crib fence is placed in front of each trigger section to prevent wave action causing premature triggering and is not affected by the raised FSL.

4.10.4 Main Embankment (Left bank upstream sloping core and Right bank central core rockfill)

The embankment stability was checked in the upgrade design (page 68-70 of 97 Options Selection and Concept Design Q1091 WIV RP De 009)

Four cases were assessed:

- Downstream Stability with MFL of 78
- Upstream Stability with FSL of 67
- Upstream Stability with rapid draw down from MFL to FSL
- Downstream stability at FSL with Earthquake

Cross Sections were assessed at the Right Bank Drawing A1-50789 Ch 1335 to 1665, the Left Bank A1-50819 Ch 100-1016, the Saddle Dam 1 (GHD report 2001) and the Saddle Dam 2 (GHD report 2001).

Soil properties were adopted from the DPI Design Report (1995) and GHD 1997 “Design Review Report”. Lower Quartile properties were used (ref table 11 Options Selection and Concept Design Q1091 WIV RP De 009), these were also reported in the draft report on the Embankment Stability (Existing Embankment Design and Construction Report – Wivenhoe Dam Spillway Augmentation Works, Q1091, Volume not known, draft report date not known received electronically 27 October 2008). The calculated factors of safety reported in both references above are summarised below.

Table 3 Embankment Factors of Safety

Case (and crest level)	D/S at MFL 78	U/S at FSL 67	U/S RDD	D/S EQ
Right Bank RL 79	1.60	1.33	1.24	1.09
Left Bank RL 79	1.69	2.06	1.49	1.24
Saddle 1 RL 80	1.89	2.15	1.59	1.38
Saddle 2 RL 80	1.64	1.75	1.16	1.32
Min FOS required	1.5	1.5	1.25	1

Note on pg 70 of 97 Options Selection and Concept Design Q1091 WIV RP De 009 that phi 41 to 45 FOS is greater than 1.5 for Right Bank U/S RL 67. It was noted that shallow slip circle gives low confining stress, therefore higher phi, and therefore the factor of safety was considered OK.

Liquefaction was considered (ref pg 70-71 of 97 Options Selection and Concept Design Q1091 WIV RP De 009) because low density alluvial layers have potential to liquefy for the "Main Embankment". A recommendation was made that the Alliance perform geotechnical investigations and determine the potential for liquefaction. No results reported in this document. A comment from the Alliance Design Manager indicates that the alluvium was investigated and was not found to be liquefiable according to limits reported in Fell and Stapleton, "Geotechnical Engineering of Embankment Dams", 1992.

4.10.5 Saddle Dams 1 and 2

Saddle Dam 1

Dam Crest Level EL 80m

Downstream Toe Level EL 73m

Upstream foundation contact level of RL 73

This embankment comprises homogeneous clay fill with medium plasticity (Classification CL) compacted to near OMC with minor potential for shrinkage cracking. Upstream slope protection comprising rip rap and transition zone was provided.

Saddle Dam 2

Dam Crest Level EL 80m

Downstream Toe Level EL 71m

Upstream foundation contact level of RL 72.

This embankment comprises central core clay fill of high plasticity (Classification CH) with outer miscellaneous conglomerate fill compacted to near OMC with minor potential for shrinkage cracking. Upstream slope protection comprising rip rap and transition zone was also provided.

The Sunwater report Assessment of Piping Potential in the Wivenhoe Saddle Dams, Sunwater, November 2001, E00952-23 Material testing showed 2 of 7 samples had Emerson Class 2.

Note that the GHD report Factual Geotechnical Report Wivenhoe Saddle Dam Geotechnical Investigation, GHD, May 2001, showed 10 Emerson Class 2 tests in a total of 14 between both structures.

Conclusions from the Sunwater Report (Assessment of Piping Potential in the Wivenhoe Saddle Dams, Sunwater, November 2001, E00952-23) were as follows:

- Embankments comprise Miscellaneous clay fill, which is not dispersive
- The embankment materials are not prone to loss of moisture and cracking
- There is no need for a downstream filter zone.

Risk

The reservoir frequency data provided in the Sunwater (2007) report shows that the reservoir level is likely to reach Saddle Dam 2 when floods exceed the 1 in 10 AEP while the Saddle Dam 1 will be reached at about 1 in 50 AEP events.

The effect of raising the FSL by 2m results in increased frequency of flooding as follows:

- Saddle Dam Present AEP Raised FSL AEP
- SD 1 1in 50 1 in 40
- SD 2 Unknown as flood frequency data does not extend below the 1 in 50 event

The effect of the raised FSL on the likelihood of failure will require use of the SKM/Alliance Portfolio Risk Analysis.

Conclusions

There is a minor incremental impact resulting from the raised FSL.

References

Assessment of Piping Potential in the Wivenhoe Saddle Dams, Sunwater, November 2001, E00952-23

Factual Geotechnical Report Wivenhoe Saddle Dam Geotechnical Investigation, GHD, May 2001

Assessment of Wivenhoe Dam Full Supply Level on Flood Impacts, Sunwater, December 2007, P-AEXP-1802-AE-02

4.10.6 Coominya Saddle

It is noted that the Coominya Saddle located 7.5 m west of dam has level of RL 77.5 (note that new Western Corridor Pipeline passes through this saddle). Subsequent investigations of Qld Dept of Main Roads drawings show the Coominya Saddle on the Brisbane Valley Highway to be RL 83 m.

5. Discussions on Data Review (Fatal Flaw Analysis)

The spillway and embankment dams experience their highest loads at Maximum Flood Levels. The critical load case for the embankment dams is earthquake and rapid draw down from MFL 78 m (now 80 m) to FSL 67 and with a raised FSL the stability is improved. The embankment Saddle dams are not inundated until minor flood level and are largely unaffected by the raised FSL. Therefore changing the FSL does not present any change in stability to these structures.

The spillway gates were originally designed for a FSL of 68.5 and the controlling design case is flood loads to the top of the gate (RL 73) where the gates are being used to attenuate floods below the 1 in 100 AEP flood. The change in FSL to 69 presents very little change to the original design case and is within the flood attenuation loads.

The raising of the FSL does affect a number of areas contributing to the overall risk of failure including inundation of the saddle dams during more frequent floods, lowering of triggering procedure 4 (RL 74) from 1 in 430 AEP to 1 in 100 AEP, and increasing the fuse plug failure probability from a 1 in 4500 AEP event to a 1 in 2300 AEP event. These effects should be re-assessed using the SKM PRA, this is particularly important was recommended by the 2006 Comprehensive Inspection Report (SEQWater Dams, Wivenhoe Dam Comprehensive Inspection Report, July 2006, NSW Dept of Commerce Draft version 1.4 26 Sept 2007)

Key points to consider in raising the FSL from 67 to 69 are:

- The maintenance bulkhead for the spillway gates has a crest level of RL 69 which does not allow for wave action during maintenance when the reservoir is at FSL.
- The fuse plug ogee crest is at RL 67 and therefore loss of the fuse plugs will result in the reservoir being drawn down to the current FSL unless the ogee is raised. A raised FSL also presents a permanent water load on the fuse embankments.
- Although the fuse plug embankments have been fitted with filter zones complying with water retaining structures consideration should be given to providing a protective concrete capping to the new FSL to reduce seepage quantities.
- The stability conditions at the FSL are unknown for the spillway section with post tensioned anchors. It is recommended that stability results including earthquake are obtained from the designer for the current and raised FSL.
- The 2006 Comprehensive Inspection Report recommends re-running the SKM PRA with the current population and damage estimates and can be used to determine the effect of the more frequent fuse plug failure events on the overall risk of failure. This analysis can include assessments of the higher risk of piping failure of the saddle dams as well. This is the most important aspect of the viability of the raised FSL as the risk should remain below the ANCOLD tolerable limit line.

- The use of Grade 4.6 hold down bolts for the trunion pedestals would not meet current standards.

6. Conclusions

Raising the FSL from RL 67 to RL 69 is very feasible for the Wivenhoe Dam. The most significant conclusions that arose during this review of structural adequacy are:

- The radial gates were designed for a FSL of RL 68.5 and the FSL is not the critical design case. Flood drawdown and earthquake are the key design cases for the earthfill embankment.
- Stability analyses for the post tensioned spillway under FSL conditions are not available and should be run by the Wivenhoe Upgrade designer for both current and revised FSL cases.
- The maintenance bulkhead for the radial gates has a top level of RL 69 which will provide no protection to wave splash when it is used with the reservoir at the raised FSL.
- The penstock gate has been designed for a FSL at RL 67.0. Raising the FSL by 2m could thus overstress or reduce the allowable safety factors for some components.
- The use of Grade 4.6 hold down bolts for the trunion pedestals would not meet current standards and a revised analysis is recommended to evaluate the proposed raising.
- Although the fuse plug sections have filters that comply with water retaining structures, a concrete facing should be provided to the upstream face of the fuse plug sections to minimise seepage at FSL.
- The ogee section under the fuse plug sections is at RL 67 and in the event that the fuse plug sections breach the reservoir will be drawn down to this level.
- The fuse plug sections breach during more frequent floods with a raised FSL (although the first breach occurs at an AEP of 1 in 2300 (compared to 1 in 4500 for the current FSL).
- The SKM PRA should be re-run to assess the impact of the raised FSL and the more frequent breach of the fuse plugs.

7. Statement on Acceptability of Proposed Raising to Operating Level 69m

Subject to confirmation that the risk profile determined by re-running the SKM PRA is below current ANCOLD tolerability limits, the raising of the FSL from RL 67 to RL 69 is very feasible for the Wivenhoe Dam. Most structural elements have either been designed for a raised FSL (eg the crest gates where RL 68.5 was the design FSL) and other structures have critical loads at Maximum Flood Level or Drawdown conditions. Minor changes to the fuse plug embankment upstream protection is advised as these will be subject to permanent water loads which they were not designed for. The fixed wheel penstock gate design should be reviewed to assess its suitability at the higher FSL and an analysis is recommended to evaluate the use of Grade 4.6 hold down bolts for the trunion pedestals for the radial gates.

8. References

Data Review at SEQWATER Offices (May 2008)

- Wivenhoe Dam Spillway Augmentation Works, Volume 5, Design Discharges and Downstream Impacts of Wivenhoe Dam Upgrade Report, Report No: Q1091 WIV-RP-HD-004, prepared by Wivenhoe Alliance, September 2005.
- Assessment of Wivenhoe Dam Full Supply Level on Flood Impacts Report prepared by Sunwater, December, 2007 P-AEXP-1802-AE-02.
- SEQWater Manual of Operational Procedures for Flood Mitigation for Wivenhoe Dam and Somerset Dam, prepared by South East Queensland Water Corporation, Version 7, 6 June, 2007
- Wivenhoe Dam Design Report, Volume 1 – Text, DPI, Sept 1995
- Wivenhoe Dam Design Report, Volume 2 – Drawings, DPI, Sept 1995
- Report on Wivenhoe Dam Design of Crest Control Structures, DPI, Apr 1995
- Q1091 WIV RP De 012 Spillway and Existing Section - Wivenhoe Alliance Report
- Options Selection and Concept Design Q1091 WIV RP De 009
- Assessment of Piping Potential in the Wivenhoe Saddle Dams, Sunwater, November 2001, E00952-23
- Factual Geotechnical Report Wivenhoe Saddle Dam Geotechnical Investigation, GHD, May 2001

Additional References Reviewed

- IEAust, Australian Rainfall and Runoff (AR&R), A guide to flood estimation, Volume 1, The Institution of Engineers Australia, Barton ACT, 1999
- Wivenhoe Dam Report on Seismic Assessment of the Radial Gates, Piers and Bridges, GHD 2000
- Wivenhoe Dam Summary of Available Test Data, GHD, 2001
- Wivenhoe Dam Safety Review, GHD, Draft 1997, Final 2002 (same document)

Data Received on CD dated 27 October 2008

Original Design and Construction TIFF drawings

Wivenhoe Upgrade Alliance drawings

Wivenhoe Upgrade Alliance design reports including

- Auxiliary Spillway Design and Construction Report
- Dambreak Study

- Design Hydrology
- Existing Spillway Design Report
- Existing Embankment Design Report
- Phase 1 Geotechnical Report
- Phase 2 Borrow Materials Report
- Phase 2 Geotechnical Report
- Phase 3 Geotechnical Report

Crest Control Structures Civil Design Report, DPI, 1995

Wivenhoe Dam Design Report, DPI, 1995 (2 volumes)

SEQWC Manual of Operational Procedures for Flood Mitigation for Wivenhoe Dam and Somerset Dam, June 2007

Assessment of Wivenhoe Dam FSL on Flood Impacts, Sunwater, December 2007

Spreadsheets and annual inspection summaries

Comprehensive Dam Safety Inspection, NSW Dept of Commerce, Draft 1.4

SKM Portfolio Risk Analysis

Fuse Plug Spillway design calculations

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[redacted]

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