NSW Department of Commerce | NSW Water Solutions

Hunter Water Corporation

TILLEGRA DAM DESIGN - CONSULTANCY 361802

Concept Report - Final

VOLUME I

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DAMS & CIVIL

REPORT STATUS

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Executive Summary

General

Hunter Water is proposing to construct a 450,000 ML water supply dam on the Williams River at Tillegra, approximately 12 km upstream of Dungog (see Figure 1-1). The dam is approximately 76 m high and has a length of 800 m.

The additional source is now required to provide for high future population growth and to provide additional system capacity for drought management in the lower Hunter Region. It is intended that the main method of supplying water from Tillegra Dam would be by controlled release into the Williams River, extraction at Seaham Weir and pumping into Grahamstown Reservoir using existing infrastructure. A hydroelectric generator is to be installed to take advantage of flow maintenance releases and spillway discharges.

Option studies (Commerce February 2008) have indicated that a concrete face rockfill dam (CFRD) is the most suitable construction for the site. The layout is shown at Drawings C-102 and C-103 in Appendix F and consists of the following main elements:

- A 76 m high concrete faced rockfill embankment (CFRD);
- A chute spillway on the right abutment, controlled by an ungated ogee crest and terminating in a flip bucket;
- A diversion tunnel with inlet and outlet channels on the right abutment;
- An outlet works constructed within the diversion tunnel and discharging into the spillway plunge pool.

Embankment & River Diversion

The proposed CFRD design proposed is a conventional design in accordance with established international practice. The principal embankment parameters are:

- Embankment height of 76 m and rockfill volume of 2,100,000 m^3;
- Full Supply Level (FSL) at RL 152.3;
- Design Flood Level at RL 158.9 giving a maximum head of 6.6 m on the spillway crest;
- Embankment parapet level at RL 160.2 giving a dry freeboard of 1.3 m.

River diversion works during construction comprise:

- A 5.8m diameter concrete lined tunnel through the right abutment;
- An 850 mm bypass pipe located in the tunnel lining to provide environmental flows during outlet construction;

- Low height upstream and downstream cofferdams that divert normal river flows and small floods through the tunnel.
- A main cofferdam, referred to as the downstream stage, located within the downstream batter line of the main embankment and reinforced with a steel mesh to enable large floods to be passed over, and to some extent, through the cofferdam.

The river diversion procedure is a conservative design with critical construction activities tied to seasonal streamflow patterns. The design is expected to satisfy the current risk guidelines published by the Australian National Committee on Large dams (ANCOLD 2003). Compliance will be confirmed by a risk analysis which will be carried out once consequence studies have been completed.

Critical dimensions such as cofferdam heights and tunnel diameter are not fixed and may be adjusted during the risk assessment process to provide better outcomes or reduce cost.

Spillway & Outlet Works

A conventional chute spillway is located on the right abutment with:

- A 40 m ogee crest curved in plan;
- A fan shaped contraction to a 20 m chute;
- A 20 m wide flip bucket discharging into a pre-excavated plunge pool;
- A discharge channel to the river.

The outlet works consist of:

- A free standing wet intake tower at the upstream portal of the diversion tunnel, equipped with selective withdrawal facilities and bridge access to the dam abutment;
- A 2500 mm diameter steel liner within the tunnel from the grout curtain to the valve block at the downstream tunnel portal;
- An 850 mm bypass pipe located within the diversion tunnel lining to provide flow maintenance releases during outlet construction and a low discharge outlet during normal operation.
- A valve block at the downstream tunnel portal containing FDCV's and submerged valves for discharge control, butterfly valves as guard valves and associated interconnecting pipework and dissipator boxes.
- A 1350 mm connection from the main penstock to a link pipe to the Chichester Trunk Gravity Main (CTGM). This connection pipe extends across the spillway to connect with the HWA designed link pipeline.
- A mini hydro facility located within the valve block but separate from the main valve chamber.

Estimated Cost

The cost estimate is provided to HWC under separate cover.

Status of Design Activities

An options study report, Commerce (Feb 2008), recommended a CFRD design be adopted. A spillway optimisation report, Commerce (Apr 2008), recommended a 40m wide chute spillway be located on the right abutment,

The geological investigations for the rim of the storage have now been completed. This work has had priority and delayed finalisation of investigations for the dam and quarry. The latter investigations are in progress and reporting will not be available until early 2009. A synopsis of the geology of the dam site and proposed quarry alternatives based on the work completed to date is provided as part of this Report. The synopsis provides a general interpretation of the site conditions to provide parameters for the concept design of the dam. Investigation work for the low saddle areas, the left abutment of the dam and the quarries is in progress but little information is available for this Report.

The concept design stage investigations have found no geological impediment to the construction of the proposed dam. Conditions are considered favourable relative to many other dam sites which have been successfully developed. Further geological investigation will be undertaken as planned to confirm the geological model of the site and determine the foundation conditions for specific structures.

A draft hydrology report, Commerce (Nov 2008A), has been completed and is proceeding through the review process.

A hydraulic model study for the spillway has been recently commissioned and will be operable early in March 2009.

Consequence studies have commenced but will not be available until mid 2009.

Draft reports have also been completed for electrical works, Commerce (Dec 2008) and telemetry, Commerce (Nov 2008C). Truncated versions of these reports are included in this Report. An Information Memorandum, PB (2008), has been prepared providing technical information to parties interested in designing, installing and operating the mini hydro facility.

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1 Introduction

1.1 Tillegra Dam

Hunter Water is proposing to construct a 450,000 ML water supply dam on the Williams River at Tillegra, approximately 12 km upstream of Dungog (see Figure 1-1). The dam is approximately 76 m high and has a length of 800 m.

The additional source is now required to provide for high future population growth and to provide additional system capacity for drought management in the lower Hunter Region. It is intended that the main method of supplying water from Tillegra Dam would be by controlled release into the Williams River, extraction at Seaham Weir and pumping into Grahamstown Reservoir using existing infrastructure. A hydroelectric generator is to be installed to take advantage of flow maintenance releases and spillway discharges.

Option studies (Commerce February 2008) have indicated that a concrete face rockfill dam (CFRD) is the most suitable construction for the site. The layout is shown at Drawings C-102 and C-103 in Appendix F and consists of the following main elements:

- A 76 m high concrete faced rockfill embankment;
- A chute spillway on the right abutment, controlled by an ungated ogee crest and terminating in a flip bucket;
- A diversion tunnel with inlet and outlet channels on the right abutment;
- An outlet works constructed within the diversion tunnel and discharging into the spillway plunge pool.

This Report provides a concept design for the proposed Tillegra Dam.

1.2 The CFRD Proposal

1.2.1 CFRD Embankment

There is no recognised design manual for CFRD constructions, except for a brief ANCOLD Guideline (ANCOLD 1990) and an early ICOLD Bulletin (ICOLD 1993). There have been few changes in CFRD practice over the last 20 years for small CFRD dams such as Tillegra and current practice is still well summarised by Cooke and Sherard (1987). The changes that have occurred are probably best summarised by Nelson Pinto (2001).

The embankment design proposed for the concept design is a conventional design in accordance with the above references (possibly excluding the ICOLD Bulletin). The principal embankment parameters are:

- Embankment height of 76 m and rockfill volume of 2,100,000 m^3;
- FSL at RL 152.3;
- DFL at RL 158.9 giving a maximum head of 6.6 m on the spillway crest;

- Embankment parapet level at RL 160.2 giving a dry freeboard of 1.3 m.

The embankment is zoned for different rockfill quality and placement requirements. The main cofferdam, referred to as the downstream stage, is incorporated within the downstream shell of the dam with the downstream face strengthened with steel reinforcement to permit safe overtopping during construction.

A mix of fresh to slightly weathered rock is used in the upstream zones that carry the water load, in the river bed to provide drainage and in the downstream stage. A zone of mixed slightly and moderately weathered rock, as is likely to be obtained from the top 10 to 20 m of the quarries and upper spillway excavations, is located in the downstream shell.

1.2.2 River Diversion

The river diversion works developed comprise:

- A 5.8m diameter concrete lined tunnel through the right abutment;
- An 850 mm bypass pipe located in the tunnel lining to provide environmental flows during outlet construction;
- Low height upstream and downstream cofferdams that divert normal river flows and small floods through the tunnel. The upstream cofferdam would fail under overtopping and the height will be restricted to a safe level consistent with flood warning and evacuation procedures. An upstream cofferdam to RL 102 is proposed at this time.
- A main cofferdam, referred to as the downstream stage, located within the downstream batter line of the main embankment and reinforced with a steel mesh to enable large floods to be passed over, and to some extent, through the cofferdam. A crest level of RL 125 has been adopted at this time.

The river diversion procedure is a conservative design with critical construction activities tied to seasonal streamflow patterns. It depends on diversion of the river commencing on the 1st July and completion of the downstream stage by the following mid-November. The design is expected to satisfy the current ANCOLD risk guidelines (ANCOLD 2003). This will be confirmed by a risk analysis which will be carried out once consequence studies have been completed.

Critical dimensions such as cofferdam heights and tunnel diameter are not fixed and may be adjusted during the risk assessment process to provide better outcomes or reduce cost.

1.2.3 Spillway

A conventional chute spillway is located on the right abutment with:

- A 40 m ogee crest curved in plan;
- A fan shaped contraction to a 20 m chute;
- A 20 m wide flip bucket discharging into a pre-excavated plunge pool;
- A discharge channel to the river.

1.2.4 Outlet Works

The outlet works consist of:

- A free standing wet intake tower at the upstream portal of the diversion tunnel, equipped with selective withdrawal facilities and bridge access to the dam abutment;
- A 2500 mm diameter steel liner within the tunnel from the grout curtain to the valve block at the downstream tunnel portal;
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1.3 Current Status

An options study report, Commerce (Feb 2008), recommended a CFRD design be adopted. A spillway optimisation report, Commerce (Apr 2008), recommended a 40m wide chute spillway be located on the right abutment,

The geological investigations for the rim of the storage have now been completed. This work has had priority and delayed finalisation of investigations for the dam and quarry. The latter investigations are in progress and reporting will not be available until early 2009. A synopsis of the geology of the dam site and proposed quarry alternatives based on the work completed to date is provided as part of this Report. The synopsis provides a general interpretation of the site conditions to provide parameters for the concept design of the dam. Investigation work for the low saddle areas, the left abutment of the dam and the quarries is in progress but little information is available for this Report.

A draft hydrology report, Commerce (Nov 2008A), has been completed and is proceeding through the review process.

A hydraulic model study for the spillway has been recently commissioned and will be operable early in March 2009.

Consequence studies have commenced but will not be available until mid 2009.

Draft reports have also been completed for electrical works, Commerce (Dec 2008) and telemetry, Commerce (Nov 2008C). Truncated versions of these reports are included at Sections 10 and 11 respectively. An Information Memorandum, PB (2008), has been prepared providing technical information to parties interested in designing, installing and operating the mini hydro facility and this is reproduced at Appendix C.

The Drawings provided are final contract drawings rather than concept drawings. As a result they are a “work in progress” showing the current stage of design development. Some drawings are not complete and further detail will be added as it becomes available.

1.4 Design Criteria

The design of Tillegra Dam is to satisfy the requirements of Australian National Committee on Large dams (ANCOLD) and the NSW Dams Safety Committee for an Extreme Flood Consequence Category (FCC). Further studies may determine that the FCC is High A but design will still be based on an Extreme FCC.

Design criteria for the major elements are summarised in Table 1-1 with additional detail provided in the body of the report.

HWC requirements are for a conservation storage of 450,000 ML with storage of water to commence in 2013.

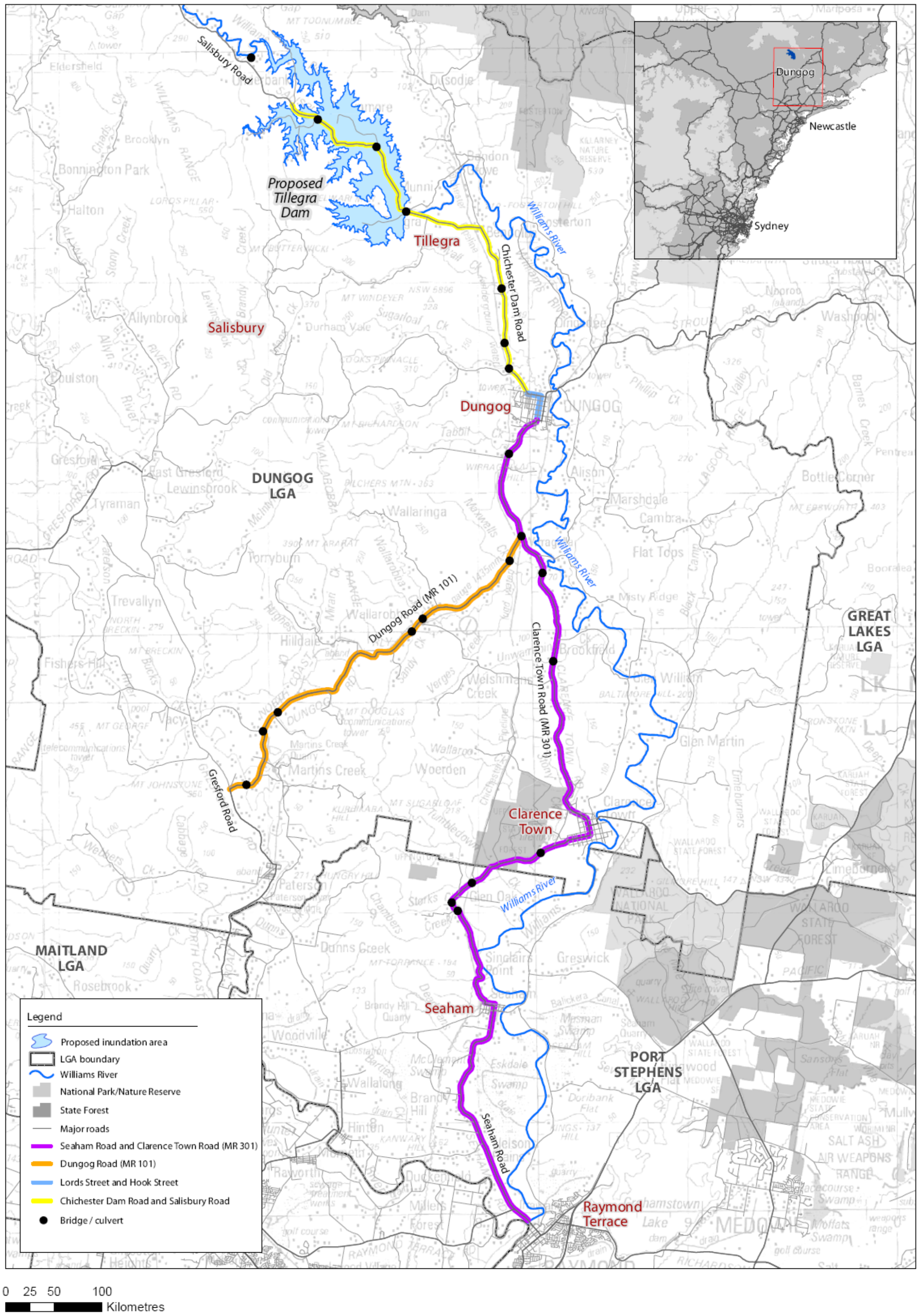
The design concepts and criteria will be further developed during the detail design phase using the results obtained from ongoing geotechnical investigations and physical hydraulic model studies.

Table 1-1 - Summary of Design Criteria

Issue	Design criteria
Methodology for flood hydrology	See Commerce (Nov 2008A)
Spillway Sizing	Designed for PMF on a full storage with the appropriate minimum freeboard
Tailwater Level	Based on MIKE 11 analysis of Williams River by HWA
Freeboard	USBR (1992)
Design Earthquake	Maximum Design Earthquake (MDE) is 1 in 10,000 AEP event Operational Base Earthquake (OBE) is 1 in 500 AEP event
River Diversion	In accordance with ANCOLD (2003). Guidance from DSC has been requested and discussion is in progress. ICOLD (1986) and ICOLD (1993) used as general references.
CFRD Embankment	ANCOLD (1990) and ICOLD (1989)

Issue	Design criteria
	See discussion at Section 7.1
Spillway	The spillway is controlled by an ungated ogee crest. The Maximum Design Flood is the maximum PMF outflow discharge. A spillway Design Basis Flood has not been used. The spillway is generally designed in accordance with USACE (1992) with exceptions noted. Chute wall lining is provided to the PMF “hard water” hydraulic profile with unlined rock walls containing aerated water and wave action. Flip bucket basin and discharge channel design will be based on hydraulic model studies and plunge pool erosion assessment based on recent and current unpublished work by George Annandale.
Outlet Works	HWC requirements for discharge capacity and accuracy are provided in detail at Section 2. Maximum outlet capacity is determined by emergency dewatering requirements. The recommendations of USBR (1990) are considered together with the capabilities of similar NSW dams. The intake tower is provided with selective withdrawal capabilities.
Structural reinforced concrete design for walls and slabs	Designed in accordance with AS 3600
Structural steelwork, ladders, platforms etc	Designed in accordance with AS 4100 and AS 1657
Bridge design	Designed in accordance with AS 5100 :2004

Figure 1-1 - Locality Plan (from Connell Wagner Environmental Assessment Report)



2 Operational Requirements

2.1 General

Hunter Water is in the process of preparing an Environmental Assessment Report for the project, which will present the proposed operating requirements for the storage, including environmental releases. These operating requirements will take into account the considerable environmental studies into ecology and river health, as well as consultation with various stakeholders. This section of the concept design report presents indicative operating requirements for the completeness of this document. The actual operating requirements will be an outcome of the Environmental Assessment process.

Storage operation for a 76 year period has been modelled by HWC and results are shown graphically at Appendix E.

Storage operation has been modelled by HWC using historical daily flow data for the site over the period 1931 to 2006 and the dam releases outlined at Section 2. Two scenarios have been modelled;

- A 90 GL/year demand, considered to be applicable to the year 2030:

- A 120 GL/year demand, considered to be applicable to the year 2050:

Flow maintenance releases are deliberate water releases delivered for the purpose of maintaining river health and ensuring that river water access by third parties will not be diminished by the presence of the new dam. They consist of:

- Transparent releases;
- Translucent releases;
- Simulated river freshes.

In addition, HWC will make bulk water releases in order to transfer water to Grahamstown Dam.

2.2 Storage Level

The Full Supply Level (FSL) is set at RL 152.3, providing a total storage volume of 450,566 ML and a surface area of 2,152 Ha. The effective dead storage level is RL99, giving a dead storage of approximately 2,700 ML.

2.3 Transparent & Translucent Releases

Transparent and translucent releases are delivered at all times when there is a positive flow into the dam except possibly when the dam is spilling;

- Transparent releases are required to the 90th percentile inflow which is 7.4 ML/d;

- Translucent releases of 60% are required to the 30th percentile inflow which is 100 ML/d;
  - An inflow of 10 ML/d requires a release of:

    $$Q_{out} = (7.4 + 0.6 * (10 – 7.4)) = 9.0 \text{ ML/d};$$

  - An inflow of 100 ML/d or larger requires a release of:

    $$Q_{out} = (7.4 + 0.6 * (100 – 7.4)) = 63.0 \text{ ML/d};$$

This required release will be provided by spills if available and mini-hydro releases with the outlet valves to make up any shortfall.

A leakage of 1 ML/d or 12 l/sec has been adopted for storage operation models as would be expected for a dam of this type and size. It is anticipated that the environmental release will normally be provided by the mini-hydro.

Table 2-1 Target Discharges for Simulated River Fresh

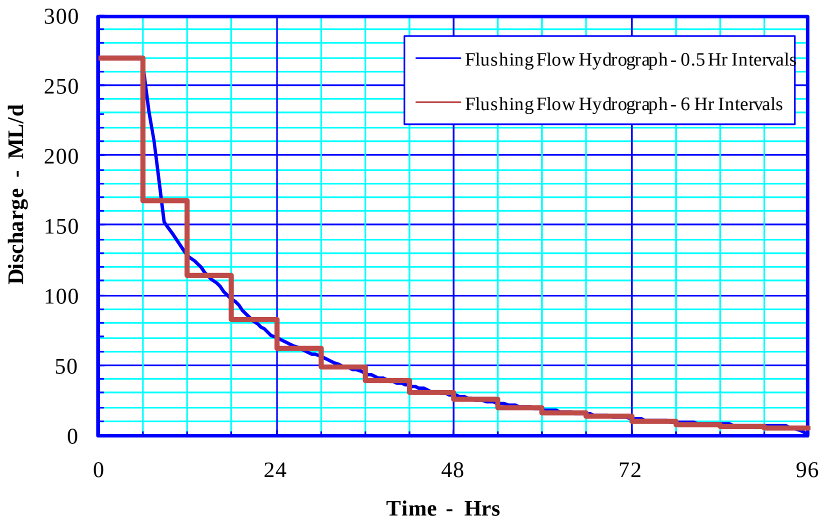
<table>
<tr><th>Time (days)</th><th>Time (hrs)</th><th>Target flow rate (ML/day)</th><th>Net flow in 6 hr (ML)</th><th>Net flow in day (ML)</th></tr>
<tr><td>0</td><td>0</td><td>270</td><td>67.50</td><td rowspan="4">160</td></tr>
<tr><td>0.25</td><td>6</td><td>168</td><td>42.00</td></tr>
<tr><td>0.50</td><td>12</td><td>115</td><td>28.75</td></tr>
<tr><td>0.75</td><td>18</td><td>83</td><td>20.75</td></tr>
<tr><td>1.0 0</td><td>24</td><td>63</td><td>15.75</td><td rowspan="4">46</td></tr>
<tr><td>1.25</td><td>30</td><td>50</td><td>12.50</td></tr>
<tr><td>1.50</td><td>36</td><td>40</td><td>10.00</td></tr>
<tr><td>1.75</td><td>42</td><td>32</td><td>8.00</td></tr>
<tr><td>2.00</td><td>48</td><td>26</td><td>6.50</td><td rowspan="4">20</td></tr>
<tr><td>2.25</td><td>54</td><td>21</td><td>5.25</td></tr>
<tr><td>2.50</td><td>60</td><td>17</td><td>4.25</td></tr>
<tr><td>2.75</td><td>66</td><td>14</td><td>3.50</td></tr>
<tr><td>3.00</td><td>72</td><td>11</td><td>2.75</td><td rowspan="4">8</td></tr>
<tr><td>3.25</td><td>78</td><td>9</td><td>2.25</td></tr>
<tr><td>3.50</td><td>84</td><td>7.5</td><td>1.875</td></tr>
<tr><td>3.75</td><td>90</td><td>6</td><td>1.50</td></tr>
</table>

2.4 Simulated River Freshes

A fresh is a deliberate release pattern that can be triggered under specific circumstances in addition to transparent and translucent releases. The target shape of the fresh release is a pattern of flow specified at 6 hourly intervals as detailed at Table 2-1. Hydrographs with discharges for 6 hour and 0.5 hour intervals are shown at Figure 2-1.

Storage operation data indicates that 101 freshes would be required in the 76 year period as shown at Figure 2-3. The volume of each fresh is 234 ML and the peak discharge is 270 ML/d (90 GL/year operation). While freshes are expected to be discharged at 40 week intervals on average, Figure 2-3 shows up to 5 can occur in any one year and there are long periods when river freshes are not required.

Figure 2-1 Hydrograph for Simulated River Fresh



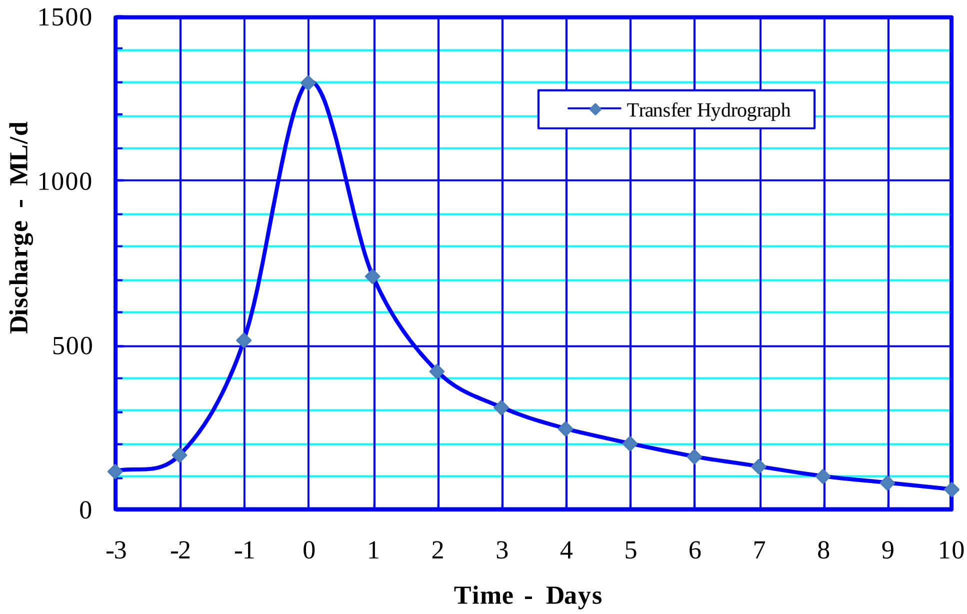
2.5 Controlled Transfer Releases to Grahamstown

Bulk water transfers are initiated under specified storage conditions in Tillegra Dam and Grahamstown Dam. In broad terms, bulk transfers are to be targeted when either of the following occurs:

- Grahamstown Dam has space to receive water and Tillegra Dam is relatively full;
- Tillegra Dam has available water and Grahamstown Dam is low.

A typical hydrograph for controlled transfer releases is shown at Figure 2-2.

Figure 2-2 Hydrograph for Bulk Transfer



2.6 Environmental Releases during Construction

For the short period during construction when the diversion tunnel is closed and dewatered to install the plug and pipework, environmental flows of around 40 ML/day will be acceptable. This assumes the duration of this work is limited.

The 850mm bypass can provide the maximum transparent release of 63 ML/d at a storage level of RL 100. The current program allows 6 months for outlet works construction. Releases during this period would be manually controlled using a temporary bypass extension past the valve chamber construction area and butterfly control valve.

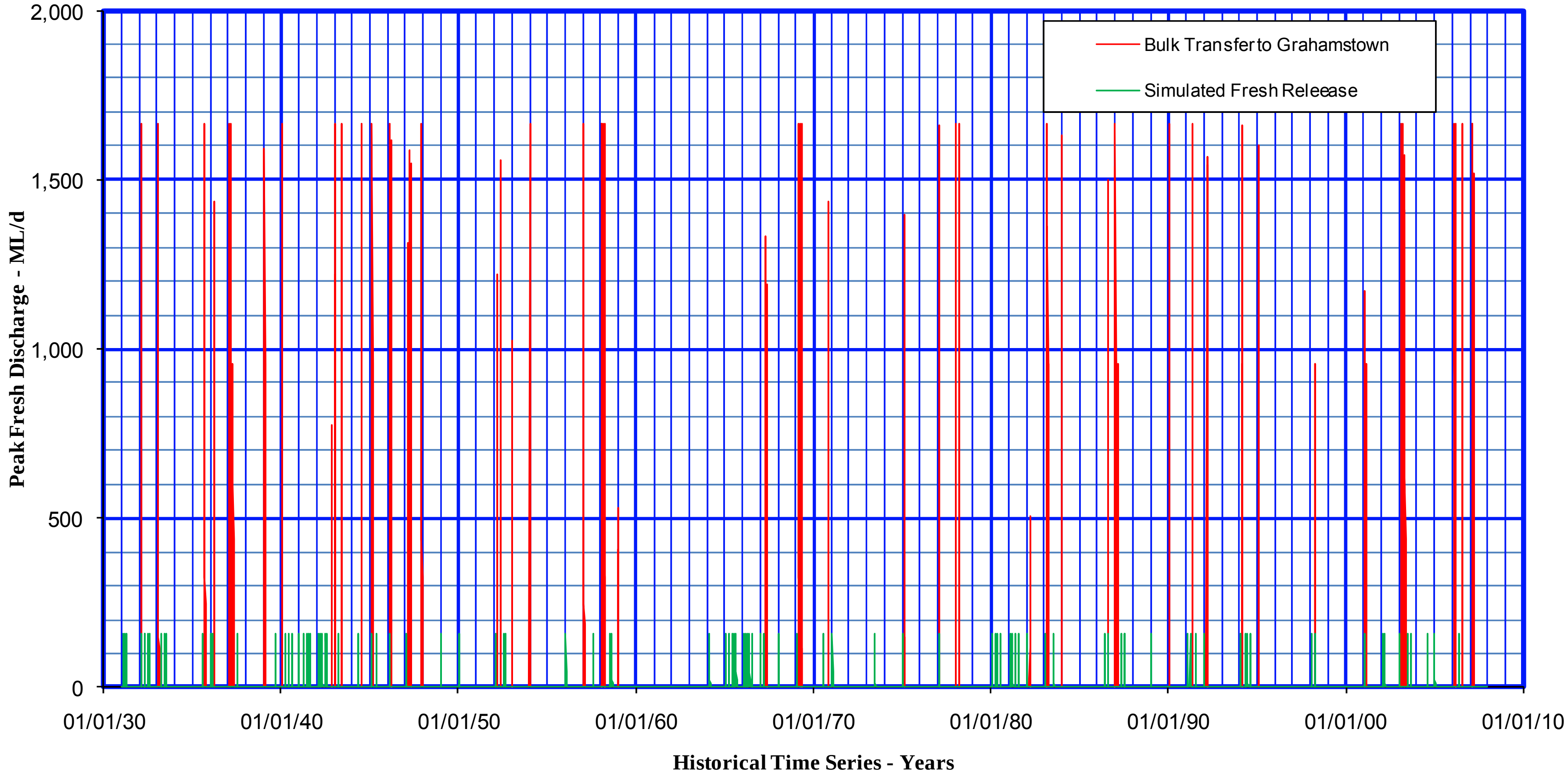
2.7 Transfer via Chichester Trunk Gravity Main (CTGM)

Releasing water via the CTGM to Dungog Water Treatment Works would only be required if there were a water quality problem at Chichester Dam which normally supplies the treatment works. Hunter Water Australia (HWA) will design the link to the CTGM. The outlet works will need to be able to provide 135 ML/day to the link pipe at the same time as any controlled transfer release to the river as described above.

2.8 Transfers from Chichester to Tillegra

Transfer from Chichester to Tillegra is no longer a requirement. It is however physically possible to transfer via the CTGM link pipe and through the Tillegra outlet works to the Tillegra storage.

Figure 2-3 Historical Occurrence of Simulated Freshes & Bulk Transfers (90 GL/year Demand)



2.9 Water Quality Requirements

An assessment of water quality issues has been undertaken by Connell Wagner, CW (2008) to support the Tillegra Dam Planning and Environmental Assessment process. Connell Wagner modelled the proposed Tillegra Storage to study the effects of stratification. Output consisted of data on storage temperature, dissolved oxygen and cyanobacteria for a 12 month period.

The Tillegra storage is similar in size, volume and depth to the nearby Glennies Creek storage (Lake St Clair). Lake St Clair has been subject to previous stratification studies by the Centre for Water Research (CWR) at the University of Western Australia.

Lake St Clair typically stratifies during spring and summer with cooling during autumn and winter when the storage becomes mixed from surface to bottom. The depth of the thermocline increases as surface heating progresses through spring and summer, reaching about 15 to 20 m at a maximum.

Frequent algal blooms are recorded at Lake St Clair requiring releases of water from deeper in the dam below the thermocline. Water temperatures measured 1 kilometre downstream of the dam are significantly lower than inflow temperatures in summer and significantly higher in winter. These thermal pollution effects largely dissipate within 20 kilometres. Controlled releases are relatively small with an average of 100 ML/d and a maximum of 626 ML/d.

Modelling shows the proposed Tillegra Dam will stratify in a similar manner to Lake St Clair. The depth of the thermocline increases as surface heating progresses to a maximum of 20 metres. Surface temperatures range from a 32 degrees Celsius in February to about 14 degrees in July. The bottom temperatures range from 12 to 14 degrees Celsius.

Stratification is expected to occur on an annual basis once the mean depth is greater than about 10 metres. This will occur very quickly after tunnel closure during construction.
Dissolved oxygen meets the ANZECC (2000) guidelines in the surface mixed layer to about 8 metres. The results for cyanobacteria indicate a succession from a diatom bloom in spring to a dominance of cyanobacteria in summer.

The key criteria are:

- Temperature;
- Dissolved oxygen;
- Blue-green algae.

CW (2008) notes that the requirement is to mimic the dam inflow temperatures and dissolved oxygen and to have blue green algae levels which meet the NH&MRC Guidelines for recreational use. These measures are expected to protect downstream aquatic life, including fish spawning and larval development.

Releases to meet downstream water quality objectives can be managed with a multi-level offtake that enables warmer, well oxygenated surface water to be released as summarised in Table 2-3.

Table 2-2 - Intake Levels to Meet main Criteria

Criteria	Model Results	Intake Level
Downstream water temperatures	***Summer:*** Warm surface layer extends from 5 to 10 m ***Winter:*** Storage mixed but temperature warmer in 5 to 10m range.	5 to 10m
Dissolved oxygen	Dissolved oxygen meets requirements in thermally mixed layer to 8m. but decreases with depth even in winter	5 to 8m
Blue green algae	Modelling too coarse to predict acceptable levels. A depth of 6m is estimated	6m plus
Overall Recommendation		***6 to 8m***

2.10 Emergency Dewatering

Australian authorities (including the DSC) generally give consideration to the United States Bureau of Reclamation (USBR 1990) criteria and guidelines for evacuating storage reservoirs and sizing low level outlet works. Strict compliance is not a usual requirement for large storages but an ability to provide reasonable control of the storage during initial filling and to provide substantial drawdown capability is considered advantageous.

The controlled release requirement is 1,670 ML/d plus 135 ML/d for the CTGM release giving a total of 1,805 ML/d. It is assumed that this discharge capacity should be provided down to 30 % storage, requiring a discharge capacity of around 2,200 ML/d at the FSL of RL 152.3.

At this stage, a maximum discharge capacity of at least 5,000 ML/d at FSL is considered necessary for emergency dewatering. On this basis, the emergency drawdown requirements dictate the maximum capacity of the outlet. These requirements are discussed in detail at Section 9.7.

2.11 Hydro-electric Power Station

A hydroelectric generator is to be installed to take advantage of the environmental and spillway releases. A preliminary Report on mini-hydro was provided by PB Australia (PB 2007) prior to finalisation of the environmental flow release requirements outlined at Sections 2.3 and 2.4. At this time a decision was taken that the mini hydro would be the subject of a separate "build-own-operate" (BOO) contract. PB have provided a second report, PB (2008) reproduced at Appendix C, which is an Information Memorandum (IM) providing technical information to parties interested in designing, installing and operating the mini hydro facility. This Report is based on the current environmental release requirements.

A mini-hydro designed to capture transparent and translucent flows (62 ML/d) can also capture a proportion of the simulated river freshes, although spillway discharges may not be available for some years, depending on the time taken for initial filling.

The hydroelectric power operation is discussed in more detail at Section 9.5.

2.12 Fish Passage

Fish passage is covered in the Environmental Assessment Report.

2.13 Controlled Releases

While the development of flow maintenance release strategies has been based on the characteristics of historic flow measured at the Tillegra Bridge streamflow gauge, this data will not be available once the dam is built. HWC has established the “Underbank Streamflow Gauging Station” located as shown at Figure 2-4.

Water level data is already being recorded at the site with the aim being that around 2 years of overlapping data will be available between the sites at the dam wall and at the new streamflow gauge. Water level data will be able to be converted to flow data as rating data become available over time. A good understanding of the overlapping flow series may well not be available for some time.

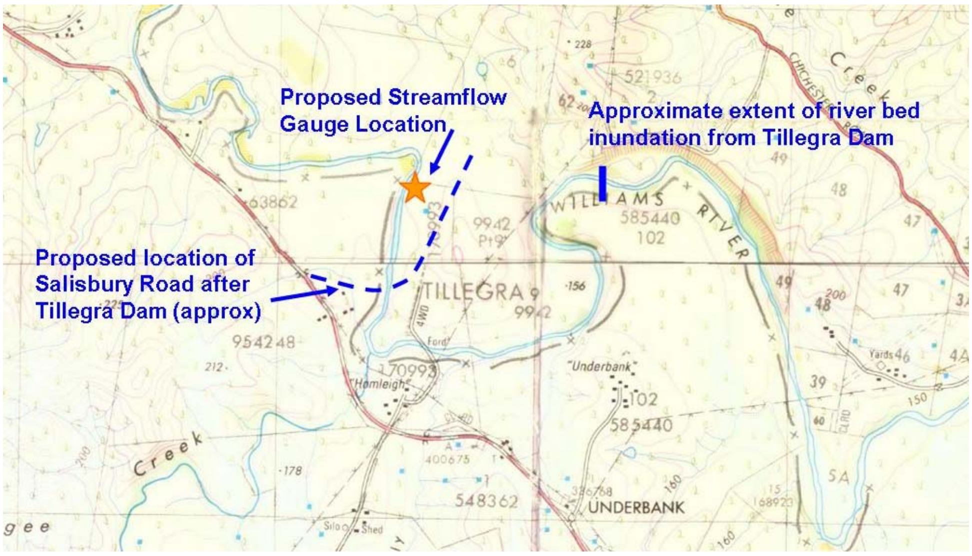
Transparent and translucent releases, and the trigger for freshes, will be controlled based on real time flow data that are collected at Underbank. Data will be transmitted via the Hunter Water telemetry system. Radio path has been checked at Underbank and a strong signal is available.

HWC requires the release from the dam to be automated such that releases from the dam can be controlled in real time remotely or by on-site control algorithms. At this stage the fresh and transfer flow patterns have been defined as 6 hourly and daily time steps respectively. It may be found that these time-steps are too coarse, and more continuous flow control may be required, possibly at half hourly intervals.

In order to be able to achieve the transparent and translucent releases that are being proposed, the control system and associated hydraulic components will need to be able to accurately deliver flow rates between zero and 1700ML/day, and in particular the transparent and translucent flows which are in the range zero to 63ML/day. The accuracy of flow control for flows above 63ML/day is probably less critical, though accurate measurement of any flows that are released is considered to be important.

The required level of accuracy will need to be developed in consultation with HWC. The HWC objective is to implement industry best practice. Preliminary suggestions are a dam outflow measurement accuracy of +/-5% or better for all controlled releases and +/-10% for all dam outflows including spills, Flow control resolution of transparent and translucent releases of +/-10% or better, with +/-20% control resolution for set points in the fresh and transfer release patterns.

Figure 2-4 Underbank Gauging Station



Measurement of controlled and uncontrolled releases is likely to involve a combination of some or all of the following:

- Measurement of discharges through the penstocks and bypass pipes;
- Measurement of discharge through the valves and mini-hydro;
- Assessment of discharge over the spillway from storage level. This is unlikely to provide sufficient accuracy for the long term small discharges over the spillway;
- A calibrated weir in the discharge channel downstream of the spillway plunge pool to provide total outlet discharge from valve block and spillway. This would provide more accurate assessment of small spillway discharges;
- A gauging station in the river downstream of the spillway.

Water measurement is discussed in more detail at Section 9.7.

3 Storage Operation

3.1 Long Term Operation

Various dam operating approaches have been tested by HWC with the aim of developing an operating strategy that maximises the utility of the dam for town water supply in conjunction with the other Hunter Water sources and at the same time minimises the environmental and other impacts of the dam.

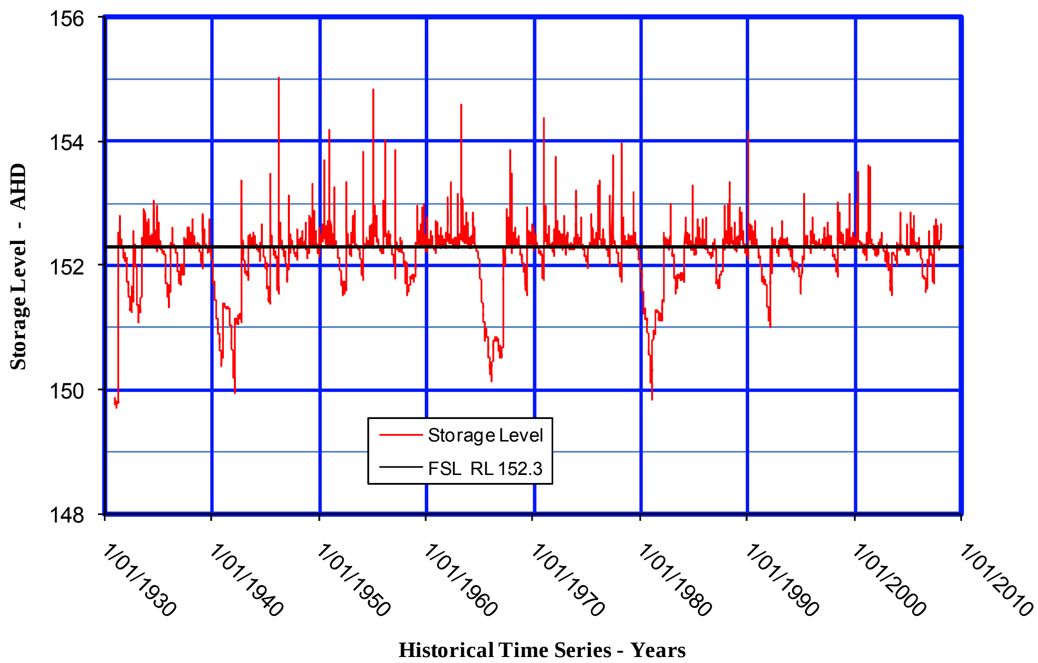
Storage operation has been modelled by HWC using historical daily flow data for the site over the period 1931 to 2006 and the dam releases outlined at Section 2. Two scenarios have been modelled;

- A 90 GL/year demand, considered to be applicable to the year 2030:
- A 120 GL/year demand, considered to be applicable to the year 2050:

It should be noted that the strategy that has been developed to date does not include any allowance for hydro electric power generation, and depending on how it is configured, hydro operation has the potential to impact on river flow patterns. If hydro is simply used as an alternative mechanism for delivering the controlled releases that are described in Section 2 it will make no difference, but if it is operated as an additional release, even during periods of spill, it will have an impact.

Detail results from the modelling are shown graphically at Appendix E.

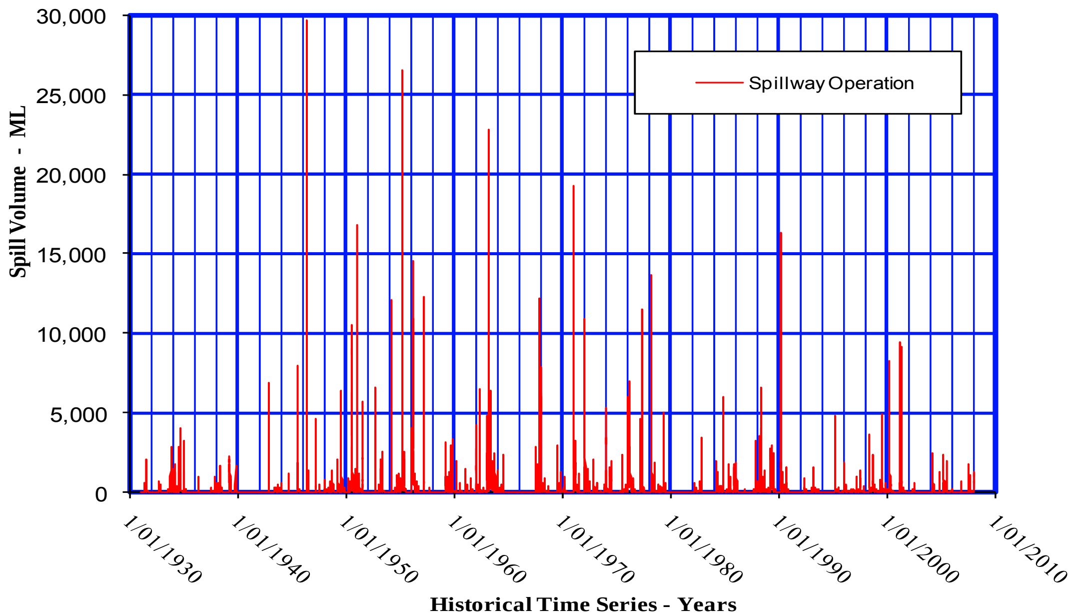
Figure 3-1 Storage Fluctuations Following Initial Filling – 90 GL/year



The modelled storage fluctuations following initial filling are shown at Figure 3-1. The storage normally fluctuates within the top 5% (equivalent to top 1.1 m or top 23,000

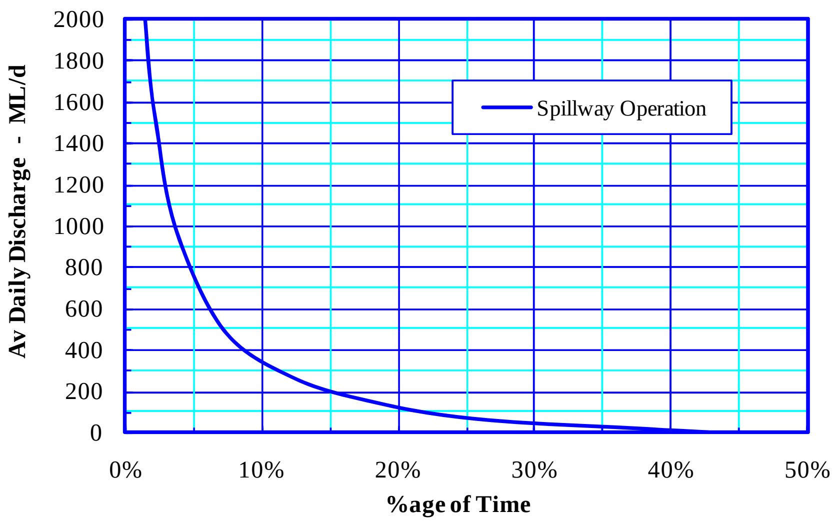
ML). It occasionally draws down to 88% (equivalent to top 2.7 m or top 54,000 ML). This emphasises the primary role for the storage being to provide an emergency supply and to permit the remainder of the system to be operated more effectively and minimise evaporation losses.

Figure 3-2 - Historical Spillway Operation



Spillway operation occurs for 43% of the time as indicated by Figure 3-2. However, most flows are relatively small as indicated by Figure 3-3. Mini hydro operation can reduce the small spills.

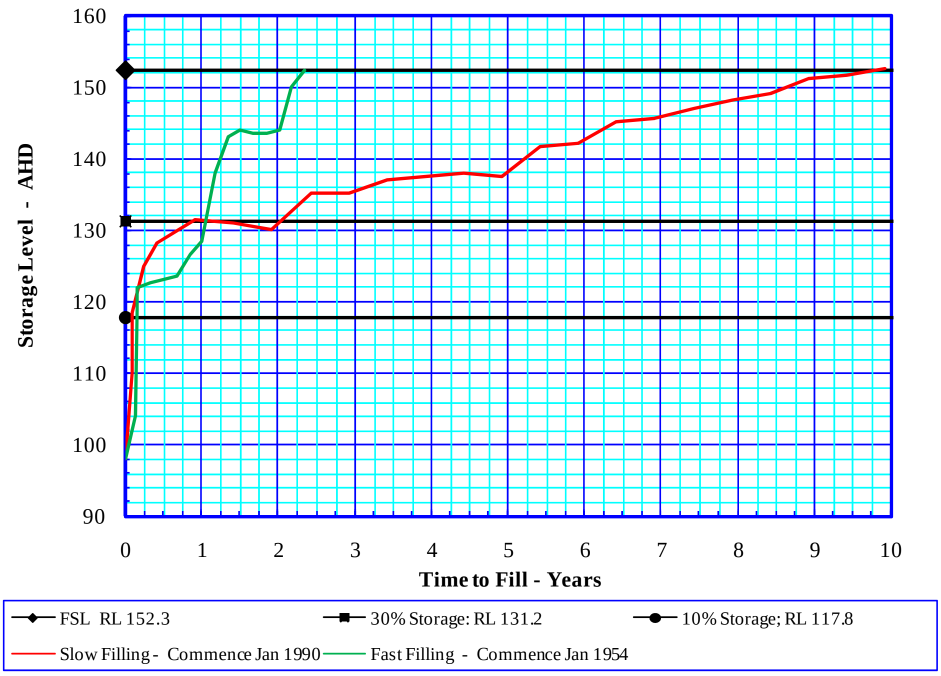
Figure 3-3 - Average Daily Spillway Discharges



3.2 Initial Reservoir Filling

Initial filling of the storage can take up to 10 years or as little as two. Examples of fast and slow filling are shown at Figure 3-4. While normal operation after first filling shown at Figure 3-1 shows storage levels confined to the top 3 m, the potential for long filling periods requires the outlet works to provide transparent and translucent flows above RL 100. It would appear reasonable to provide flushing flows above RL 120 and transfer flows above RL 131.4 (30% storage volume).

Figure 3-4 - Storage Filling Times



4 Site Geology & Geotechnical Investigations

4.1 Introduction

Option and concept phase geological and geotechnical investigations for Tillegra Dam are still in progress and detailed reporting will not be completed until early 2009. The main facets of the investigations are:

- Investigations for the dam elements, including embankment, spillway options and diversion;
- Investigation of the rim of the storage, including the various fault systems that may affect the storage area;
- Assessing potential sources of construction materials, in particular identifying quarry areas for the production of rockfill and concrete aggregate.

The investigation for the rim of the storage is complete and is detailed at Commerce (Nov 2008B). This work has had priority and delayed finalisation of investigations for the dam and quarry.

The following presents a synopsis of the geology of the dam site and proposed quarry alternatives based on the work completed to date. It provides a general interpretation of the site conditions to provide parameters for the concept design of the dam. Investigation work for the low saddle areas, the left abutment of the dam and the quarries is in progress but little information is available for this Report.

Detailed geological reporting is not available at this time. Investigations for the spillway and tunnel are summarised on drawings C-801 to C-803, C-806 and C-811 at Appendix D.

Current investigations for the concept design will be followed by final detailed investigations during the final design stage.

Accepted engineering geological terminology is used in this report. Appendix D, Tables D1 to D4, document the terminology relating to weathering, rock strength, discontinuity spacing, block size and aperture width of discontinuity spaces. Estimated rock strength refers to rock substance strength as opposed to rock mass strength.

4.2 Previous Work

Two (2) preliminary phases of investigation have been previously undertaken at the dam site. Initial site investigations were undertaken in 1952, followed by more specific work in 1970. Cost studies for the project concept were undertaken in 1985. Details are as follows:

- *HWB (1952):* Investigation included geological mapping and percussion boreholes, some extended with diamond coring. Drilling was mostly on the left alluvial terrace, upstream of the site, the right bank, extending upstream of the embankment footprint, and the lower left abutment. Drill core from the investigation is stored at Chichester Dam.

- *Hall (1952):* A regional survey of the area was undertaken by the Geological Survey of New South Wales.
- *SMEC (1970):* Investigation included additional mapping, a seismic traverse in the riverbed and inclined, diamond cored boreholes across the valley floor. Core from the investigation is currently stored at Chichester Dam.
- *WRC (1985):* Engineering and cost studies were undertaken for the development of the proposed dam. A concrete faced rockfill dam was adopted for the study.

In addition to the above work, Douglas (2007A) has recently undertaken an aerial photograph interpretation of the proposed site and surrounding area, together with a follow-up geotechnical inspection, Douglas (2007B).

4.3 Design Investigations

Concept Design Stage investigations at the dam were completed in early 2008 and included:

- Geological mapping of the dam site and storage perimeter;
- Test pit investigation, including the storage/storage perimeter and dam site. A total of one hundred and fifty one (151) test pits have been excavated. Test pits TP37 to TP109 relate specifically to the proposed dam site;
- Seismic refraction survey of the dam site by Coffey Geotechnics Pty. Ltd;
- Diamond drilling investigation of the dam site and potential Quarry Site B. A total of nine (9) boreholes have been drilled. Where appropriate, the boreholes were water pressure tested nominally at 3m stages;
- Petrographic analyses of rocktypes encountered at the site by Dr. B.J. Franklin;
- Aggregate testing of selected core from the proposed quarry source and river gravels from the Williams River by Boral Resources (NSW) Pty. Ltd.;
- Unconfined compressive strength testing by Australian Soil Testing Pty. Ltd.

Detailed design stage geotechnical investigations at the dam have commenced and work to date includes:

- Diamond boreholes in the spillway, right abutment toeslab, diversion tunnel, and Quarry Site A and Quarry Site B. A borehole has also been drilled in each of Saddle A and Saddle B. A total of twenty-four (24) boreholes have been drilled. Selected boreholes have been water pressure tested;
- Nine (9) trenches have been excavated along the alignments of the right and left abutment toe slabs, spillway, coffer dam, and also in Saddle A and Saddle B. The trenches have provided detail on rock types and defects in the upper portion of the rock mass;
- Seismic refraction along the spillway alignment, Saddle A, Saddle B and the extended left abutment ridge by Douglas Partners Pty Ltd.

The investigation is ongoing, with further investigation planned, including:

- Diamond boreholes under the river bed, left abutment toeslab, diversion tunnel portals, and extended left abutment;
- Trenches across the toeslab to indicate the range of lithologies present in an upstream/downstream orientation;
- Trial excavation of 4 – 5m depth along the toeslab to expose foundation conditions is selected areas.

4.4 Regional Geological Setting

The Dungog area falls within the major structural unit known as the Tamworth Synclinorial Zone, which forms part of the New England Fold Belt (Scheibner, 1976). The proposed dam site and storage are within the Gresford Block (Roberts, 1991).

Sedimentary rocks belonging to the Flagstaff Formation occur at the dam site and in the immediate environs. The formation is Early Carboniferous in age and includes thickly bedded lithic sandstone, with varying proportions of mudstone (shale) and conglomerate, with minor limestone.

Sandstones at the site comprise a high proportion of intermediate to felsic lithic fragments and have been termed 'tuffaceous sandstone' in the field investigations. It is recognised that the rocks have been subject to low grade regional metamorphism; however, the term 'tuffaceous sandstone' has been used to maintain consistency with previous investigations. Mudstones (or shales) are termed 'meta-shale'.
Structurally, a series of continuous faults trending north/south occur in the region, including the Brownmore Fault, to the west of the dam site, the Majors Creek Fault and Williams River Fault well to the east and the Tillegra Fault downstream of the dam.

4.5 Interpreted Geological Conditions

4.5.1 Soil

Soil cover associated with the meta-sedimentary rock sequence is generally very thinly developed, often less than 1m thick. Topsoil, comprising pale brown sandy silt, generally ranges from 0.15m to 0.25m in thickness. The underlying residual soils are generally in the order of 0.3m to less than 1m thick and comprise admixtures of gravel, sand, silt and clay. The soils generally classify as SC to CI/CL, ranging from clayey sand to sandy clay. Soils often include a gravel fraction, particularly with depth, comprising angular, weathered meta-sedimentary rock.

4.5.2 Lithology

An interbedded sequence of tuffaceous sandstone and meta-shale occurs at the dam site. Bedding strikes approximately north-south, across the orientation of the Williams River and dips moderately upstream (west). The ridge system forming the abutments is controlled by strike.

4.5.3 Weathering

Tuffaceous sandstone outcrops are generally moderately weathered to slightly weathered. Finer grained rocktypes are not expressed at the surface. Differential weathering is expected to occur between rocktypes.

The seismic refraction and drilling results have been used to divide the rock mass into three (3) characteristic zones. A shallowly developed upper Zone I comprising soil, grading to highly weathered rock, generally occurs in the upper 2m. The surficial layer grades into a Zone II of differential weathering. This zone is interpreted to represent moderately weathered to slightly weathered rock and generally extends to depths varying from 5m to 15m at the dam site.

Zone III fresh rock is interpreted to occur below Zone II.

4.5.4 Defects

At the site defects may be grouped into three (3) categories; bedding partings, joints and shears. No dykes have been found in the investigations to date; however, they are a common feature and may occur at the site. The characteristics of these defects are briefly discussed below.

Bedding Partings

The average strike of bedding is 155ºM to 175ºM, dipping upstream, to the west at 40º to 55º. Bedding thickness varies from thinly laminated/laminated in the meta-shale, to medium/thickly bedded in the tuffaceous sandstone. Partings are planar, generally rough, with common Fe/Mn staining in the weathered part of the rock mass. Clay coatings commonly occur in the highly weathered/moderately weathered rock.

Joints

Two (2) major joint sets occur at the site:

- A set striking 175ºM parallel to bedding, dipping 75º to the east, across bedding;
- A set striking 085ºM normal to bedding, dipping at 60º to 90º to the north, into the left abutment.

Joint spacing in the coarse-grained rock types ranges from very close/close to moderately wide. In the finer-grained rocktypes, joint spacing varies from extremely close /very close in weathered outcrop to moderately wide in fresh drill core. The combination of bedding partings and joints results in a prominently fractured rock mass.

Shears

A shear zone interpreted to be in the order of 1.4m wide (horizontally) has been identified in the valley floor (diamond drilling undertaken by SMEC in 1970). The zone comprises altered tuffaceous sandstone with an extremely close/very close defect spacing. Water losses in the order of 60UL were recorded in the water pressure tests across the shear zone. The zone is assumed to be sub parallel to the major joint set striking approximately east/west and is interpreted to be controlling the river orientation at the site. Narrow shears parallel to the joint set have also been observed in the gullies trending approximately north-south, draining off the abutments.

As a result of regional folding, shear zones parallel to bedding are also expected to occur in the finer grained rocktypes. The shears in meta-shale are characterised by extremely close defect spacing and a higher degree of weathering than the surrounding rock mass.

Faulting

The Tillegra Fault is a major north-south lineament (parallel to the strike of the bedding), located away from the dam, approximately 0.5km downstream of the dam site. The fault dips shallowly to the east at approximately 35º.

Rock Strength

Rock substance strength varies from very weak/weak in highly weathered rock, to medium strong/strong in fresh meta-shale and very strong in fresh tuffaceous sandstone.

Permeability

The rock substance at the dam site is considered to be impermeable. However, water pressure test results conducted in boreholes drilled to investigate the foundation conditions in the dam abutments, upstream portal area and the spillway crest, indicate that the rock mass is permeable due to leakage along defects.
A summary of the water pressure test results are presented in Table G5 at Appendix D. Test results are usually grouped for discussion as follows:

0 to 3 Lugeons	-	low leakage
3 to 20 Lugeons	-	moderate leakage
20 to 100 Lugeons	-	high leakage
>100 Lugeons	-	very high leakage

One (1) Lugeon (UL) is considered roughly equivalent to 1 x 10-5 cm/sec in terms of permeability measurement. However, this is only an approximate correlation.

Water losses were generally low to moderate, occasionally ranging to high. AS expected, very high losses were recorded in boreholes DDH3, located on the mid right abutment, and DDH8, located in the saddle immediately east of the left embankment. These more permeable areas are limited in extent and will be addressed with conventional grouting operations.

4.5.5 Water Table

Tight rock mass zones were encountered at depth in all the boreholes in the area of the proposed embankment. Standpipe piezometers were installed in the boreholes to allow monitoring of the water table level. Readings taken at the completion of the drilling program are presented in Table 4-1. On the embankment abutments, the water table depth ranges from approximately 23m to 29m below the natural surface. In the extension of the right abutment (DDH1), the water table occurs at approximately 43m depth. In the extension of the left abutment (DDH7 and DDH8), the water table ranges from approximately 25m to 32m depth.

Table 4-1 - Water Level Readings

Borehole No.	Location	Collar RL (m)	Reading (m)	Vertical Depth (m)	Water Table RL (m)
DDH1	RA spillway crest	173.7	49.09	42.70	131.0
DDH2	RL spillway channel	107.2	17.85	15.53	91.7
DDH3	Mid RA	118.7	27.03	23.51	95.2
DDH4	U/S portal diversion	122.4	26.30	22.88	99.5
DDH5	Lower LA	97.7			
DDH6	Middle LA	127.6	33.33	28.99	94.3
DDH7	Upper LA (saddle)	147.5	29.11	25.33	122.2
DDH8	LA spillway crest	165.6	36.92	32.16	133.4

In borehole DDH5, located on the lower left abutment, a gravel layer at the base of the soil profile continually introduces water into the borehole, artificially raising the water level.

4.6 Embankment

4.6.1 Left Abutment - General Stripping

Test pits were excavated to refusal in tuffaceous sandstone, or to a depth of difficult digging or refusal in the meta-shales. Moderately weathered/slightly weathered tuffaceous sandstone outcrop and concentrated surface float occurs along the embankment centreline and to a lesser extent in the upstream half of the embankment footprint. Test pits were located away from these areas, where there was little or no surface expression of rock.

Test pit results indicate a general stripping depth for the embankment foundation in the order of 1m. At that depth, generally highly weathered, ranging to highly weathered/moderately weathered and mixed minor slightly weathered rock will be exposed. Rock substance strength is expected to range from weak/medium strong in the tuffaceous sandstone, to generally medium strong in the meta-shale.

In areas of tuffaceous sandstone outcrop/bouldery suboutcrop, minimal foundation stripping is required.

Seismic refraction results interpret a low velocity surficial zone of 400 to 600m/sec, with an average thickness of 1m to 1.5m. The zone is interpreted to represent thin soil cover and highly weathered rock. The interpretation is consistent with the test pits.

Stripping depths are interpreted to increase in the flatter area south of Salisbury Road, in the upstream half of the embankment footprint (lower abutment/valley floor area). Stripping depths up to approximately 10m are expected in the alluvial terrace forming the left bank of the river. Gravel deposits in the river channel are estimated to be up to 1m to 2m in thickness (1.5m average), overlying slightly weathered/fresh rock.

4.6.2 Left Abutment - Toe Slab Alignment

The toe slab alignment will cross the strike of the bedding at an acute angle. Bedding dips upstream at moderate angles. Tuffaceous sandstone is interpreted to be the predominant foundation rock type, with interbeds of meta-shale; however, the final distribution of each rocktype will only be known after further investigation.

Test pits excavated in tuffaceous sandstone encountered refusal at depths of 0.9m and 1.4m on highly weathered/moderately weathered, medium strong rock. In meta-shales, difficult digging conditions were experienced at depths of 1.4m to 1.85m in moderately weathered, medium strong rock.

Borehole DDH6, on the middle abutment centreline, encountered highly weathered/moderately weathered, weak/medium strong tuffaceous sandstone at 1.3m (vertical) depth, grading to moderately weathered, medium strong rock at 2.4m. Borehole DDH5, on the lower abutment (upstream toe), encountered moderately weathered/slightly weathered, medium strong meta-shale at 1.45m depth.

Below the low velocity zone identified by seismic refraction, an intermediate seismic velocity zone extends to 5m to 10m depth below the natural surface. Seismic velocities range from 1900m/sec to 2,600m/sec. The zone is interpreted to represent moderately weathered/slightly weathered rock. Highly weathered/moderately weathered rock is expected to occur at the top of the zone, as exposed at the termination depths in the test pits.

A continuous trench has been partially completed in the upper abutment as part of the current design stage investigations. At this stage, the trench extends approximately 130m downslope from the survey point established at RL150m. Rocktypes predominantly comprise tuffaceous sandstone, with minor interbedded/laminated meta-shale. The rock at the surface is very bouldery. The trench varied in depth from 0.5m to 1m, with minor excavation to 2m. Clay seams and roots were encountered in the base of the trench in highly weathered/moderately weathered tuffaceous sandstone and in moderately weathered/slightly weathered meta-siltstone/meta-shale. Most defects were coated with sandy clay, with some grey clay infill.

Moderately weathered, generally medium strong rock occurring at the top of the intermediate velocity zone is interpreted to be a suitable foundation for the embankment toe slab. However over most of the slab, areas of clay fill and tree roots will require excavation depths of 3m to 4m to found the toe slab. It is anticipated that the toe slab could be excavated by a Caterpillar D9 bulldozer (single tyne) or equivalent, with cleanup/shaping by a 30 tonne hydraulic excavator, with rock hammer. A slightly irregular foundation will result, due to the differing stripping depths between the tuffaceous sandstone and meta-shale. Hammering may be required in thicker tuffaceous sandstone beds to achieve a regularly shaped foundation.

In the lower abutment (south of Salisbury Road), up to 10m of stripping will be required under the alluvial terrace adjacent to the river and up to 2m under the gravel in the river channel. It is anticipated that only shallow excavation into rock may be required, as rock close to the river level is likely to be slightly weathered or less weathered.

4.6.3 Right Abutment - General Stripping

On the right abutment conditions are similar to those on the left. Soils are thinly developed, generally being in the order of 0.4m to 0.6m thick, ranging up to 0.8m. Test pit excavator refusal occurred on highly weathered/moderately weathered tuffaceous sandstone at depths ranging from 0.8m to 1.4m. Rock substance strength was generally medium strong. In meta-shale, difficult digging conditions, ranging to refusal, were experienced at depths ranging from 1.4m to 2.5m in moderately weathered, ranging to moderately weathered/slightly weathered rock.

Seismic refraction results interpret a low velocity surficial zone of 400 to 550m/sec, with a thickness of 1m to approximately 2m. The zone is interpreted to represent thin soil cover underlain by highly weathered rock.

A general stripping depth for the embankment in the order of 1m will provide a generally highly weathered, ranging to highly weathered/moderately weathered rock foundation. Shallower stripping depths are likely over the steeply dipping lower abutment area, where rock outcrop is prominent. Rock substance strength is expected to range from weak/medium strong in the tuffaceous sandstone, to generally medium strong in the meta-shale.

4.6.4 Right Abutment-Toe Slab Alignment

The toe slab alignment on this abutment is again expected to cross the strike of bedding at an acute angle. Test pit refusal on highly weathered/moderately weathered, ranging to moderately weathered tuffaceous sandstone occurred at depths ranging from 0.8m to 1.35m. Rock substance strength ranges from medium strong to strong.

In meta-shale difficult digging conditions were experienced at depths up to 2.5m in moderately weathered, medium strong rock.

Below the low seismic velocity zone an intermediate seismic velocity zone generally extends to 7m to 15m depth below the natural surface. In the lower abutment, the zone is thinly developed, extending to 2.5m to 3m below the natural surface. The seismic intermediate velocity interpreted was 2,600m/sec. The zone is interpreted to represent moderately weathered/slightly weathered rock. Highly weathered/moderately weathered rock is expected to occur at the top of the zone, as encountered in the bases of the test pits.

A continuous trench along the toe slab alignment has recently been completed as part of the current design stage investigations. In the middle and upper abutment areas, the toe slab will be founded on predominantly tuffaceous sandstone, with intervals of interbedded laminated tuffaceous sandstone/meta-siltstone, and minor meta-shale. Bedding dips at 40º to 50º in direction 240º to 250ºM. There is some variation in bedding orientation that is interpreted to be largely from the measurement of undulating bedding surfaces. The toe slab footprint is sub-parallel to the strike of bedding. It will be possible for the foundation in any section of the toeslab to partially comprise tuffaceous sandstone, with meta-shale/meta-siltstone upstream and downstream.

The trench varied in depth from 0.5m to 2.5m, with the deepest excavation in the fine-grained and interbedded rocks and the shallowest in moderately to thickly bedded tuffaceous sandstone. Clay seams to 20mm thickness and roots were encountered in the base of the trench in highly weathered/moderately weathered tuffaceous sandstone and in moderately weathered/slightly weathered meta-siltstone/meta-shale. Excavation of 3m to 4m is likely to be required to found the toe slab below the zone of clay and tree roots.

Similar excavation characteristics to the left abutment are anticipated. A slightly irregular foundation will result, due to the differing stripping depths in the tuffaceous sandstone and meta-shale.

On the very steep slope immediately above the river channel, bouldery outcrop occurs. Shallower stripping is anticipated to remove the boulder scree to expose a foundation in sound rock free of tree roots. Excavation depths up to 1.5m to 2m may be required.

4.7 Extension of the Left Abutment

A well developed topographic saddle occurs to the east of the embankment left abutment. The surface of the saddle is approximately 13m above the proposed full supply level. Seismic refraction indicates a surficial, low velocity zone approximately 1.5m to 2m thick with a seismic velocity of 400m/sec. Test pit TP59, located in the saddle, encountered highly weathered, very weak/weak tuffaceous sandstone from 0.65m to 1m depth, near excavator refusal. The low velocity seismic zone is interpreted to represent soil and highly weathered rock.

Underlying the low velocity seismic zone, an intermediate zone extends to depths ranging from 4m to 14m. The seismic velocity ranges from 2,100 to 2,800m/sec. The intermediate seismic zone thins (to 4m depth) in the eastern or left half of the saddle. Borehole DDH8 indicates that the zone comprises moderately weathered rock, grading to moderately/slightly weathered rock with depth. Tuffaceous sandstone was the dominant rocktype across the saddle with interbeds of meta-shale. Minor narrow shear zones parallel to bedding occurred in the meta-shales.

Fresh, (stained) rock grading to fresh with depth is interpreted to occur below the intermediate seismic zone. Seismic velocities range from 3,200 to 3,700m/sec. Water pressure tests in borehole DDH8 indicated that the rock mass tightened at a depth of 38m (approximately 25m below the proposed spillway crest level). Iron-staining on the rock mass defects also ceased at that depth.

It is anticipated that the embankment grout curtain may have to be extended through the saddle to minimise potential leakage through this portion of the ridge.

4.8 Spillway

4.8.1 General

A spillway above the right abutment (to the south) has been adopted. Initially, the centreline of the alignment was located approximately 100m south of the embankment. The conceptual crest and channel width was 100m. The initial geotechnical investigations were based on this arrangement.

This alignment encountered deep deposits of slopewash deposits, overlying alluvial gravels. Following further design development, an alternative arrangement was then investigated which included a 40 m wide crest and a 20m wide channel. The outlet channel from the diversion tunnel was directed to the spillway dissipator pool under this arrangement. This alternative effectively moved the centreline of the spillway approximately 80m to the north, towards the embankment.

This second alternative alignment has been adopted for design.

4.8.2 Current Alignment

The new spillway alignment was initially investigated in March 2008. A series of test pits were excavated along the downstream portion of the proposed channel alignment. Consolidated slopewash occurs from the toe of the slope for a distance of approximately 120m downstream. The slopewash deposit varies in thickness from 5.1m to in excess of 6m. Alluvial gravels occur further downstream, varying in depth from greater than 6.4m to 1.6m towards the Williams River.

Additional trenching, diamond drilling and a seismic refraction survey have recently been completed as part of the current design stage investigation. Trenches have been excavated on the steep slope immediately downstream of the crest, in the downstream toe/flip bucket area and in the approach channel area, upstream of the crest. Boreholes TDDH10 to TDDH14 were drilled in the crest and along the channel alignment. Boreholes TDDH31 to TDDH37 were drilled in the flip bucket area to better define the thickening of the slopewash deposit at the toe of the slope, confirming the flip bucket location. Seismic Line 8 was shot along the alignment.

The approach channel upstream of the crest comprises predominantly tuffaceous sandstone with some interbedded meta-siltstone/meta-shale sequences. Rock is typically overlain by approximately 0.5m of residual soil, containing gravelly rock fragments with depth.

Downstream of the crest, meta-siltstone/meta-shale predominates for the first 50m of the slope with minor thickly bedded tuffaceous sandstone. The deeper anomalies in the seismic profile (to 10m depth) are interpreted to be sequences of predominantly meta-shale, which has a closer defect spacing than the tuffaceous sandstone. Tuffaceous sandstone then predominates for the next 50m in the steep portion of the slope, with one prominent meta-shale sequence. Tuffaceous sandstone is predominant at the base of the slope, with some intervals of interbedded meta-siltstone.

Slopewash deposits in the flip bucket area thicken to approximately 17m depth and are well defined by the 800m/sec seismic refractor in seismic line 8. A hidden layer may occur in the basal 1.5 to 2 m of this zone, representing predominantly highly weathered rock. The flip bucket location will encounter a rock foundation and predominantly moderately weathered and better quality rock for the concrete-lined portions of the wall.

As on the toeslab foundations, bedding dips to the west at moderate angles of 35º to 55ºin direction 230º to 260ºM. There is a variation in dip; however, this may be the result of irregularity in bedding surfaces. Some minor faults and shear zones associated with bedding planes and across bedding will be encountered and will have local influence on the slope stability of the spillway batters. These will be accommodated by local excavation and support. Batter slopes are some distance from the embankment constructions and embankment stability is not an issue.

The rock mass should be rippable to near the base of the second seismic refractor; however, rock with a seismic velocity of approximately 2,000m/sec to 2,500m/sec would be difficult to rip. Rock with a seismic velocity greater than 2,500m/sec is likely to require blasting.

The following batter slopes are envisaged:

- Soils/slopewash – 2 horizontal:1 vertical;
- Highly to moderately weathered (ripped) rock – 1 horizontal:1.5 vertical (especially considering the relative closeness of the defect spacing in rock of this quality);
- Slightly weathered to fresh (blasted) rock – 1 horizontal:2 vertical (due to the relative closeness of the defects in the rock mass).

Some small wedge and planar failures may occur in these batters. Local areas of slope stabilisation with combinations of rock bolts, mesh, anchors and shotcrete may be required.

4.9 Diversion

River diversion is proposed via a 5.8m diameter tunnel through the right abutment. The invert level of the diversion tunnel is approximately RL89m. At that level, the majority of the tunnel is well within the high seismic velocity zone, with velocities ranging from 2,800 to 4,500m/sec. The zone is interpreted to represent fresh (stained) rock above fresh rock. The alignment is approximately normal to the strike of bedding which dips moderately upstream. Boreholes encountered both tuffaceous sandstone and meta-shale/meta-siltstone. The relatively close defects and bedding orientation will require support by rockbolts and mesh in the tunnel crown. Minor faults and small shears were encountered in the boreholes and may require additional and/or longer support where locally occurring. A thin layer of shotcrete as temporary (primary) support may be required in the relatively closely jointed rock mass.

In the area of the diversion inlet channel and portal, the intermediate seismic velocity zone is thickly developed, with its base extending to depths of 15m to 20m below surface (to approximate RL90 to 95). The seismic velocity of the zone is 2,100m/sec. Borehole DDH4, drilled in the vicinity of the upstream diversion portal, encountered fresh (stained) rock at 5.22m depth. Water pressure test results indicated moderate leakage to 14.1m depth. The intermediate seismic zone is interpreted to comprise moderately weathered/slightly weathered rock, grading to fresh (stained) rock with depth. The relatively low intermediate seismic velocity is interpreted to be related to the open defects in the rock mass.

The end face of the proposed cut to establish the tunnel portal is approximately parallel to the strike of bedding, which is interpreted to dip at 45º into the excavation. The bulk of the excavation is expected to be in fresh rock, including the tunnel portal. The batter above the portal should have a slope less than the bedding angle to maintain stability, or be supported by a ground anchorage system. Support will be required in the local area around the portal (in fresh rock) to maintain a steeper batter angle.

Similarly, the stability of the left wall at the upstream end of the inlet channel will be controlled by the dip of the bedding. As the alignment of the channel turns to the east, towards the portal area, bedding will have less influence on batter stability.

Borehole TDDH15, drilled as part of the current design stage investigation, encountered uncharacteristically deeply weathered rock that is interpreted to be the result of local shearing. However, this borehole is approximately 35m distant from the actual portal in the inlet channel batter area. Actual portal conditions should be re-evaluated with an additional borehole. Based on borehole TDDH15, portal batters may need to be 1 horizontal :1 vertical, unless supported.

The downstream portal is expected to be excavated in rock beneath a thin soil cover. Bedding is interpreted to dip into the portal batter, with the bedding strike normal to the side batters. The stability of the batter will be influenced by a major joint set normal to bedding, which is interpreted to be dipping very steeply to the southeast, that is, into the excavation. The stability of the right wall will be controlled by a major joint set dipping moderately to the north.

Stability issues mentioned above are limited to the tunnel portal area excavation slopes. Embankment construction remains some distance from the excavations and these features have no effect on embankment stability. They do define the excavation batter slopes, and in some locations, the extent of rockbolting or other support.

Slopewash to depths of 6m or greater is expected to occur downstream of the portal, on the flatter area to the east. Seismic refraction results in the area indicate fresh (stained) and fresh rock occurs at approximately 10m depth.

4.10 Coffer Dam

The upstream coffer dam is located approximately 120m upstream of the main embankment. On the upper left abutment, test pits encountered highly weathered/moderately weathered tuffaceous sandstone at 1m depth. The rock graded to moderately weathered with depth, with very difficult digging conditions at 2.8m depth. Rock substance strength at pit termination was medium strong/strong.

On the lower left abutment, test pits encountered extremely weathered/highly weathered tuffaceous sandstone at 1.4m, grading to moderately weathered, medium strong rock at 2.9m depth.

Well rounded gravel in a clayey sand matrix occurred to a depth of 4.9m on the middle left abutment. The gravels overlie highly weathered/moderately weathered tuffaceous sandstone, with a medium strong/strong rock substance strength. The gravel may represent an abandoned, high level river channel.

The low seismic velocity zone along the left abutment centreline of the cofferdam embankment ranges from approximately 2m thick in the upper abutment, to 5m thick in the middle/lower abutment. Seismic velocity ranges from 400 to 600m/sec. The zone is interpreted to represent soil, including gravels, and highly weathered rock (grading to moderately weathered at the base).

Foundation stripping depths, ranging from 2m in the upper abutment, to 3m to 5m in the lower and middle abutments, respectively, are expected. At that depth, highly weathered/moderately weathered rock, grading to moderately weathered rock, is expected to form the foundation. Rock substance strength is interpreted to be generally medium strong. Depending on the results of further investigations, the dam could be founded on clayey gravel deposits, if suitable.

Up to 2m thickness of gravel will have to be excavated from the river channel, to expose the underlying slightly weathered fresh rock.

The right abutment has a very steep slope immediately above the river channel. Bouldery outcrop occurs over the slope. Minimal stripping depth is anticipated to remove the boulder scree.

4.11 Construction Materials

Two (2) quarry sites have initially been identified for investigation (refer to Figure C-102):

- Quarry Site A, comprising the right hand spillway excavation and possible extension along the ridge to the south;
- Quarry Site B, comprising the central ridge to the west of the dam that separates the two arms of the storage (known as Elwari Mountain). Surface mapping indicates that two (2) faces could be worked in this quarry, one in predominantly tuffaceous sandstone and one in an interbedded sequence of tuffaceous sandstone and meta-shale.

In addition, a third possible quarry site (Quarry C) has been identified. However, there are some rim stability issues with this site that require further consideration.

Seismic refraction indicates that the spillway may yield only small quantities of high quality rockfill for embankment construction and there is some doubt over the remainder of Quarry Site A.

Quarry site A is envisaged as an expansion of the (right) spillway crest area. Borehole DDH1 encountered highly weathered/moderately weathered, interbedded tuffaceous sandstone and meta-shale to 8m depth. Fresh, (stained) rock occurred below that depth. Further drilling investigation of the site to the south will be required in the area where the ridge rises away from the dam.

Borehole, DDH9 has been drilled in Quarry Site B. Moderately weathered rock was encountered to 4.4m depth. Below that depth, differentially weathered rock (moderately/slightly weathered and fresh, (stained)) was recovered to 25.64m depth. The interval contained approximately 50% fresh (stained) tuffaceous sandstone and may be suitable for use as high quality embankment rockfill. Below 25.64m (to the termination of the borehole at 60m depth) fresh, (stained) tuffaceous sandstone with a strong/very strong rock substance strength was encountered.

Due to access problems with steep topography, the drilling program at Quarry Site B was curtailed. Further work at the site is required, following significant upgrading of the access track.

Petrographic analyses indicate that the tuffaceous sandstone from Chichester Dam and Tillegra are similar in mineralogy. The Chichester sandstone provided good quality concrete aggregate for the construction of Chichester Dam.

Drill core from borehole DDH9 and gravel from the Williams River have been tested for their suitability as concrete aggregate. A summary of results is presented in Table 4-2.

Aggregate Crushing and Los Angeles Values are well within the recommended test limits of 25% and 30%, respectively. Negligible losses were recorded in the accelerated weathering test (Sodium Sulphate Soundness) In the Accelerated Mortar Bar Test, the drill core sample showed potential for mild/slow aggregate alkali reactivity. The river gravels were non reactive. Rock substance strength ranges from strong to very strong.

Fresh tuffaceous sandstone and the river gravels available from the Williams River are expected to be suitable for use as concrete aggregate.

Table 4-2 - Concrete Aggregate Test Results

Test	Core (DDH9)	River Gravels
Aggregate Crushing Value	11.0%	8.7%
Los Angeles Value	15%	10%
Sodium Sulphate Soundness	Nil to 0.2% loss	Nil to 0.2% loss
Unconfined Compressive Strength	66.9MPa to 144.6MPA	NA
Accelerated Mortar Bar Test	0.23% after 21 days	0.07% after 21 days

Quarry Site C comprising an extensive area of massive welded tuffaceous sandstone along the storage rim north of the dam is a third option that may be attractive to a contractor. The site has not been investigated and there are some rim stability issues that require further consideration.

4.12 Seismic Hazard

4.12.1 General

An earthquake hazard assessment for the Tillegra dam site (ES&S 2008) has been prepared by the Seismology Research Centre, a division of Environmental Systems and Services (ES&S).

4.12.2 Seismicity of the Tillegra Site

The site is in Carboniferous sediments which were faulted and folded during the Permian age, some 250 million years ago. The faults in the region experienced most of their relative motion at that time. There is no evidence of any geologically recent (within last 1 million years) movement of the faults in the Tillegra region.

The region immediately around Tillegra Dam has a very low level of historical seismicity, much lower than about Newcastle to the south. As at most locations, a large majority of the earthquake hazard at Tillegra Dam is from earthquakes occurring within tens of kilometres.

No active faults have been identified in the region surrounding the Tillegra Dam site.

There are a number of small faults within and or nearby the immediate Tillegra area. Their lengths are typically only up to a few kilometres; limiting the maximum magnitude event that would occur should they reactivate. ES&S have stated (ES&S 2008) that faults of about 3 by 3 km correspond to the rupture from an earthquake of moment magnitude of about ML 5.0 (Richter scale).

In summary, the ancient faults located within or quite close to the proposed Tillegra dam site and storage area are not active and if reactivated will not produce a large earthquake.

The peak ground acceleration for a return period of 475 years (10% probability of exceedance in 50 years) estimated at the Tillegra Dam site is about 0.04g - using the AUS5 model with local faults and area sources, and a minimum considered magnitude of 5.0. The corresponding peak ground acceleration for a return period of 10,000 years is estimated to be 0.24g.

The above seismic loading for the Tillegra Dam site is about average for Australian conditions (ES&S 2008).

There are no areas of the storage rim that may be susceptible to liquefaction during a major earthquake.

4.12.3 Reservoir Triggered Earthquakes

Reservoir triggered earthquakes have been reported from a small percentage of large reservoirs in many places including Australia. A recent world-wide listing suggested a figure of 2% of large reservoirs experience such events.

Reservoir triggered earthquakes would eventually have occurred naturally, but are triggered prematurely by changes related to large reservoirs.

The basic parameters of a reservoir that may contribute to triggered earthquakes include the water depth and the volume of water. A rule of thumb sometimes applied is that reservoirs with water depth less than 70 metres and water volumes less than 0.5 km^3 rarely trigger earthquakes.

Tillegra Dam, with a water depth of 67m (at full supply level) and 450 gigalitres storage volume is considered a borderline case in terms of reservoir induced activity. So it is possible that some relatively minor earthquake shaking ranging from micro-earthquakes (not felt) up to about ML 3.0 or 4.0 (resembling vibrations caused by heavy traffic) may be recorded.

Reservoir triggered earthquakes are thus considered marginally possible but would not threaten the integrity of the proposed Tillegra Dam and storage perimeter.

4.13 Conclusions

The concept design stage investigations have found no geological impediment to the construction of the proposed dam. Conditions are considered favourable relative to many other dam sites which have been successfully developed. Further geological investigation will be undertaken as planned to confirm the geological model of the site and determine the foundation conditions for specific structures.

5 Hydrology

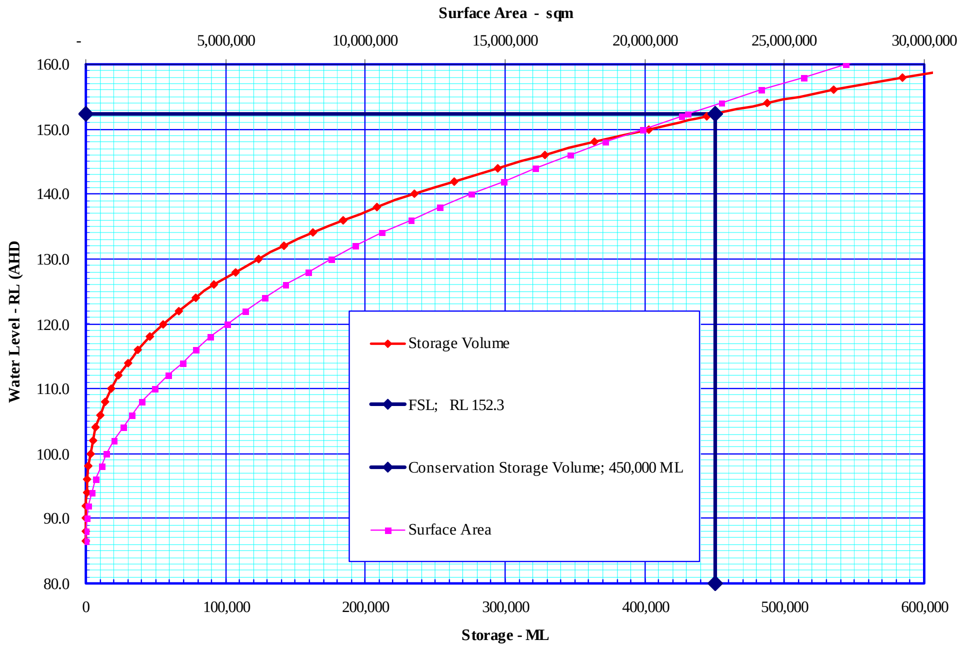
5.1 General

The final hydrology studies for Tillegra Dam, Commerce (Nov 2008), have been completed and are now proceeding through the review process. The notes in this Section summarise the critical aspects for dam design.

5.2 Storage Volume – Height Data

Storage volume and surface area data based on the latest lidar survey are shown at Figure 5-1.

Figure 5-1 - Tillegra Storage Volume & Surface Area



5.3 Rating Curves & Tailwater Levels

Discharge-stage relationships for the existing river have been developed by HWA using a MIKE11 analysis and these results have been accepted by HWC and the Department of Water & Environment (HWE) for the rating curve at the gauging station located on the downstream side of the road bridge. These river tailwater levels are shown at Figure 5-2. Backwater analyses provide the spillway discharge channel tailwater levels shown at Figure 5-3 upstream and downstream of a measuring weir in the discharge channel.

Figure 5-2 - Tillegra Tailwater Levels

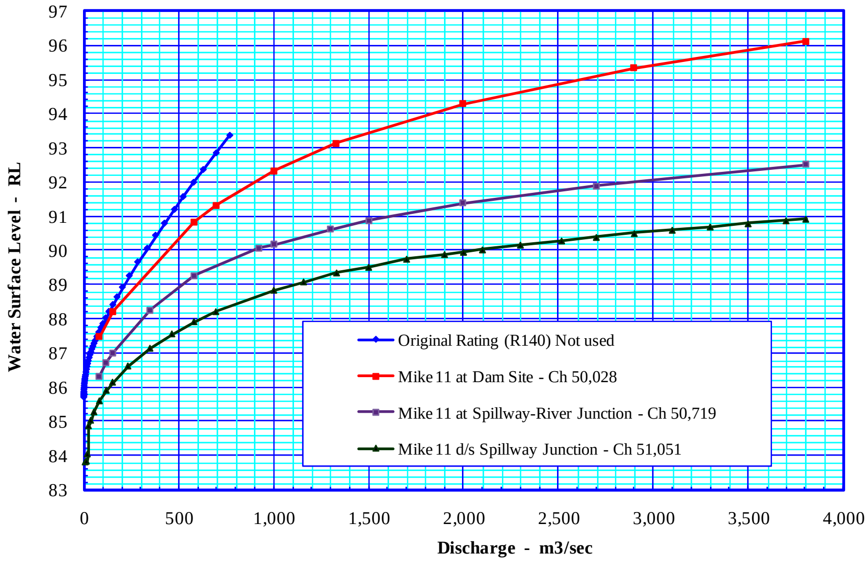
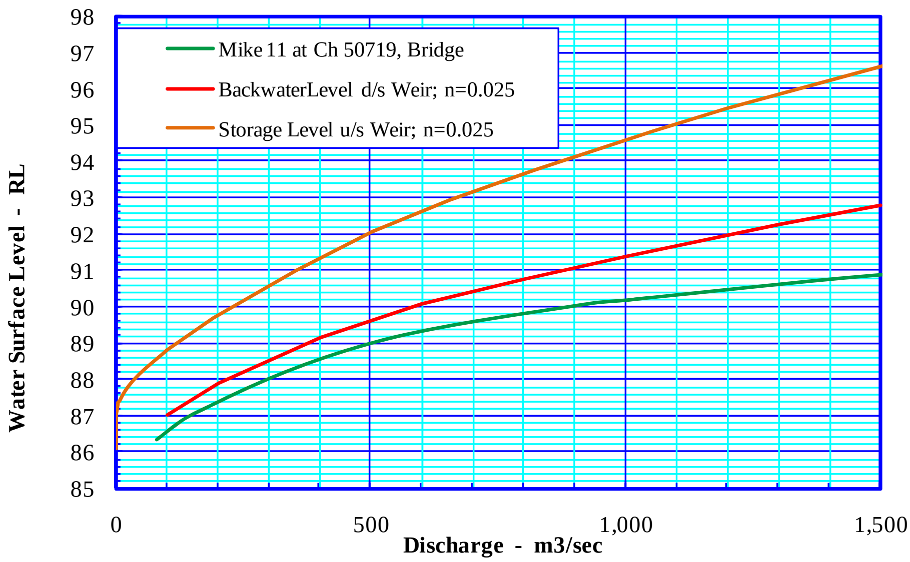


Figure 5-3 - Spillway Tailwater Levels

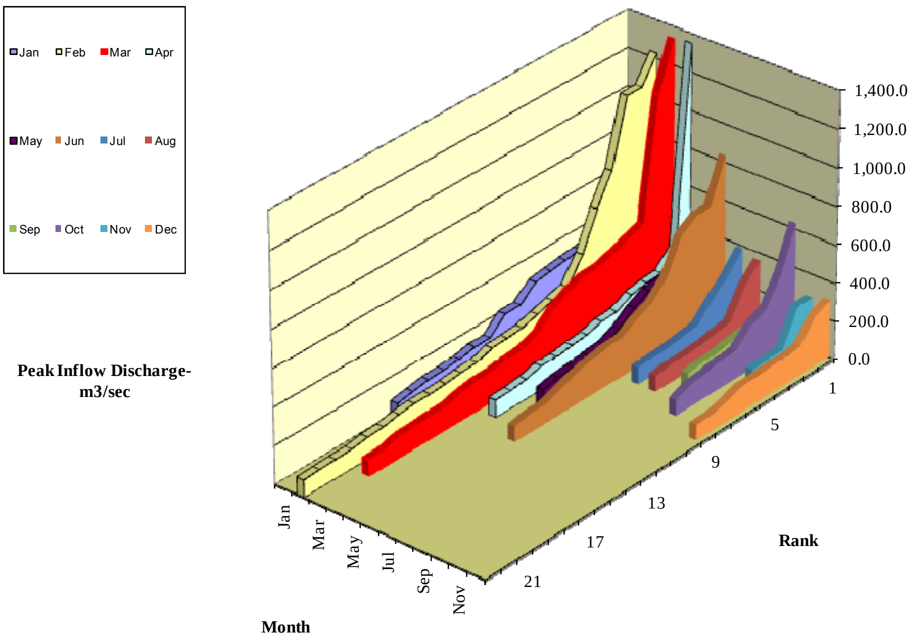


5.4 Recorded Streamflow Data

Streamflow data are available from 1931. Manual readings were taken from 1931 to 1960 and were stored on cards. While daily readings were taken, intermediate flood peak information was also recorded.

From 1960 to at least 1985 a Bristol Recorder operated at the site which recorded continuously the stream levels on charts. A new continuous flow recorder was installed in 1986.

Figure 5-4 - Peak Inflow Distribution; 1931 to 2007



The distribution of peak discharges from historical floods are shown at Figure 5-4. The large floods occurred in February to March with some high flows also in June and October. July, August, September and November-December had no history of high peak inflows.

A similar conclusion is obtained from average monthly inflow volumes shown at Figure 5-6. This seasonal distribution of peak inflow floods is the basis for the river diversion program.

5.5 Design Flood

The Design Flood adopted for spillway design is the Probable Maximum Flood (PMF). The Flood Consequence Category (FCC) for Tillegra has not been formally assessed but will be either Extreme or High A in accordance with ANCOLD (March 2000) and DSC 13 (March 2002).

The ANCOLD deterministic fallback requirement for Acceptable Flood Capacity (AFC) is the same as the DSC requirements for these categories:

- The Extreme category requires Probable Maximum Flood (PMF) capacity with a full storage prior to flood inflow together with an appropriate dry freeboard allowance;
- The High A category requires Probable Maximum Precipitation Design Flood (PMPDF) capacity with a full storage prior to flood inflow together with an appropriate dry freeboard allowance.

At Tillegra there is no difference between the PMF and PMPDF (see Section 5.7). Accordingly, the current dam layout assumes:

- PMF capacity with a full storage prior to flood inflow;
- A dry freeboard above Design Flood Level (DFL) of 1.3 m (See Section 7.7).

5.6 Flood Hydrology

5.6.1 General

The required design flood hydrographs are obtained by inputting design rainfalls into an appropriate rainfall runoff model to produce the required design flood hydrographs. A RORB model has been developed for this purpose.

The extreme design rainfalls were determined in accordance with ARR. The catchment is in a transition zone and PMP estimates have been determined using three available generalised methods:

- GSDM – Generalised Short Duration Method;
- GSAM – Generalised South Eastern Australia Method;
- GTMSR – Generalised Tropical Storm Method (Revised 2003).

The Generalised Short Duration Method (GSDM) was used for storm durations up to 6 hours. For durations 24 hours and longer, both the Generalised Southeast Australia Method (GSAM) and the Generalised Tropical Storm Method (Revised) (GTSMR) were used. For storm durations between 6 and 24, hours estimates were interpolated between the two methods using the Bureau's recommended procedures.

5.6.2 Annual and Seasonal Series

Flood hydrographs (for a range of durations) were determined for the 1 in 20 to the 1 in 500,000 AEP floods and for the PMPDF. Floods with an AEP of 1 in 1,000 or less are available for each of the three methods listed above.

Seasonal flood data was developed for use with the river diversion program. The four seasons were based on the seasonal grouping that the Bureau of Meteorology was able to achieve for their GSAM development:

- December to March;
- April to May;
- June to September;
- October to November.

The PMPDF data are summarised at Table 5-1 and summary tables for all AEP's and durations are provided at Appendix A. Details of hydrographs are provided at Commerce (Nov 2008A). The seasonal variation in PMPDF peak discharges is shown at Figure 5-5. The June to September series provides the lowest peak discharges and the December to March series provides the highest peak discharges. There is a marked seasonal effect for the short duration storms and a relatively small effect for the long duration storms.

Table 5-1 – PMF Peak Inflow Estimates

Storm Event	Storm Duration hrs	Peak Inflow Disch m^3/sec	Inflow Volume ML	Peak Inflow Discharge m^3/sec			
		Annual Series		Dec-Mar	Apr-May	Jun-Sep	Oct-Nov
GSDM	1	9,512	50,474	9,590	7,577	5,598	7,577
	1.5	9,596	64,808				
	2	9,424	74,002				
	2.5	9,355	83,624				
	3	8,817	90,885	8,899	7,055	5,052	7,055
	4	8,114	102,254				
	5	7,592	110,464				
	6	6,985	117,558	6,949	5,728	4,028	5,606
GSAM	9	5,729	140,418				
	12	4,753	157,762	4,825	3,744	3,145	3,684
	24	3,788	221,776	3,707	2,783	2,783	2,783
	36	2,989	245,277	2911	2285	2285	2285
	48	3,116	259,912	3,131	2,535	2,488	2,488
	72	2,749	271,041	2,687	2,587	2,406	2,406
	96	1,940	297,863	1,796	1,770	1,642	1,642
GTSMR	9	5,647	140,569				
	12	4,672	145,939				
	24	3,810	200,309				
	36	3,507	242,989				
	48	3,670	281,206				
	72	3,658	350,306				
	96	3,387	391,739				

Figure 5-5 - PMPDF Seasonal Data

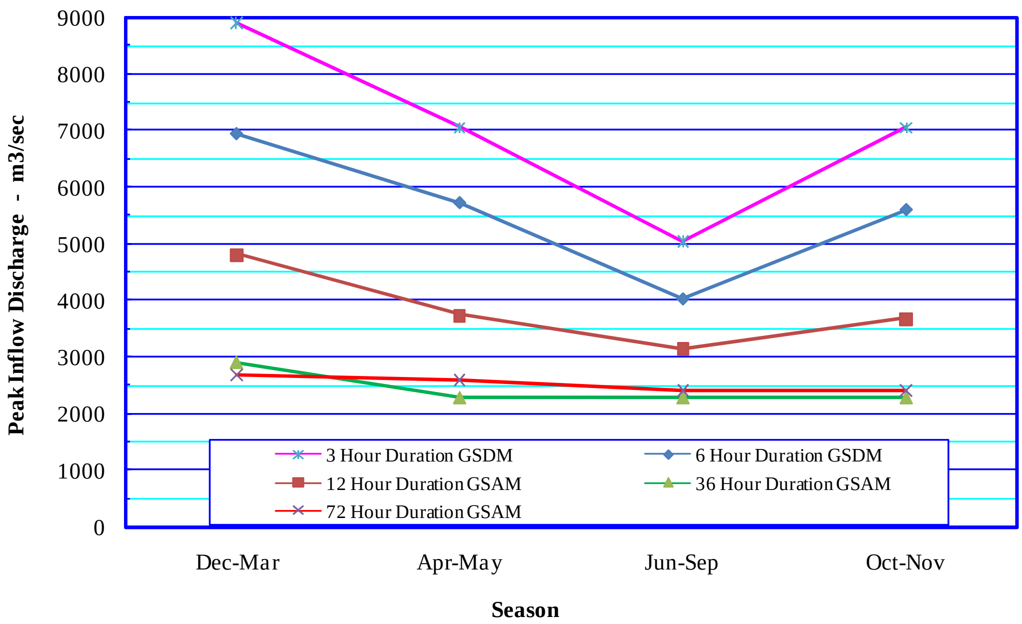
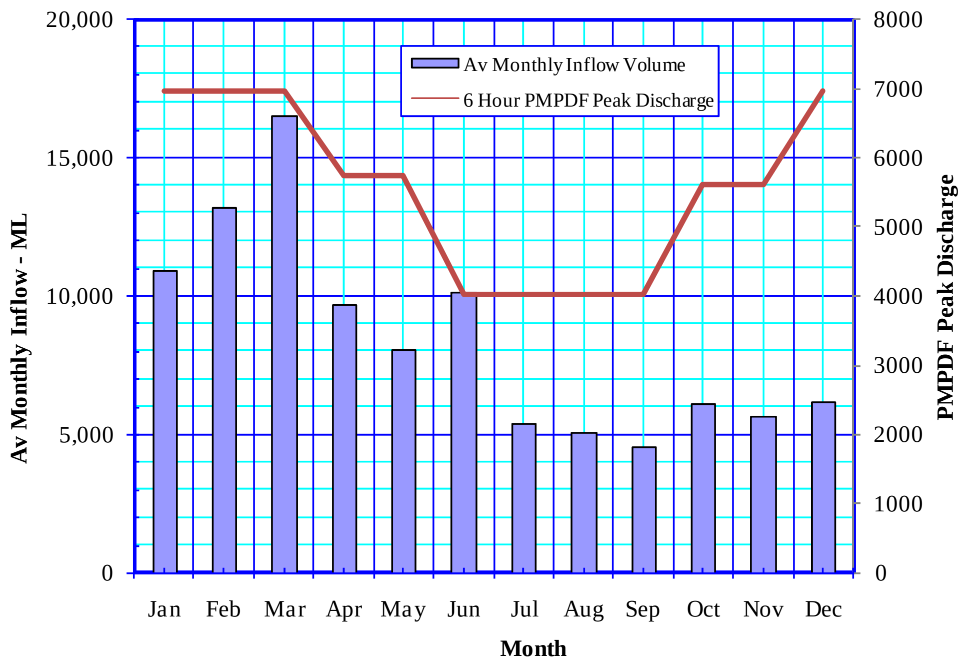


Figure 5-6 compares a typical PMPDF seasonal variation with average monthly inflow data used in the monthly storage operation analysis described in Section 3.1. The extreme PMPDF data has a very similar seasonal distribution to the historical average monthly inflow volumes and seasonal trends show little variation with the magnitude of the rainfall events.

Figure 5-6 - Comparison of Seasonal Distributions



5.7 The PMPDF and the PMF

The PMPDF (PMP Design Flood) is the design flood that results from the PMP assuming the AEP of the flood has the same AEP as the causative rainfall. To satisfy this requirement it is necessary for all other inputs used in transforming the PMP to a flood to be AEP-neutral. In practice this requires selecting representative values for all design inputs such as losses, temporal and spatial patterns.

The PMF is the upper limiting value of flood that could reasonably be expected to occur and is determined from the PMP but with other inputs selected to produce a limiting value that could be reasonably expected to occur. There is no requirement to maintain AEP neutrality and generally conservative values that are reasonably possible are selected.

Conceptually the PMF would be expected to be larger if not the same as the PMPDF and to have a smaller (but unknown AEP) than the PMPDF. The PMF and PMPDF inflows are often the same, especially for small ungauged catchments.

Until recently, the different inputs for PMPDF and PMF were the losses. Some practitioners are now suggesting worst case historical temporal patterns should be used rather than average variability patterns while others argue superimposing risks of very low probabilities is not reasonable. The current guidelines (ARR BOOK VI) suggest the use of different temporal patterns is optional for practitioners.

For Tillegra Dam, due to the selection of input values, the PMPDF is considered to be the same as the PMF. Typically for the PMF, initial loss is taken as 0 mm and continuing loss as 1 mm/hour. For the PMPDF, the values are selected after consideration of available observed data.

This study found that a continuing loss of 1 mm/hour and low initial losses needed to be adopted in order to obtain reasonable agreement between RORB/design rainfall based design flood estimates and those from observed flood frequency analysis. An initial loss of 0 mm and a continuing loss of 1 mm/hour were adopted for the PMPDF.

Generally Commerce Hydrology holds the view that for small catchments it is not appropriate to consider the worst case historical temporal patterns in trying to maximise peak flows from the PMP. The same average variability temporal patterns were therefore used for the PMPDF and PMF. This is also tempered by judged other conservatism in the flood estimation.

In any case, changing the temporal pattern while using the same PMP and losses does not change the hydrograph volume, only the hydrograph shape and peak flow. Due to the large storage volume at Tillegra, the hydrograph volume is the more critical parameter and minor changes to the hydrograph shape and peak flow have little impact.

The PMF and PMPDF have been assigned an AEP of $1*10^{-7}$.

5.8 Flood Frequency

The flood frequency curve derived from observed floods is shown at Figure 5-7. The flood frequency for the derived AEP floods depends on the storm duration and the precipitation methodology. Flood frequency curves for the GSDM and GTSMR methods are shown at Figure 5-8.

Figure 5-7 - Observed Flood Frequency Curve (Annual Series)

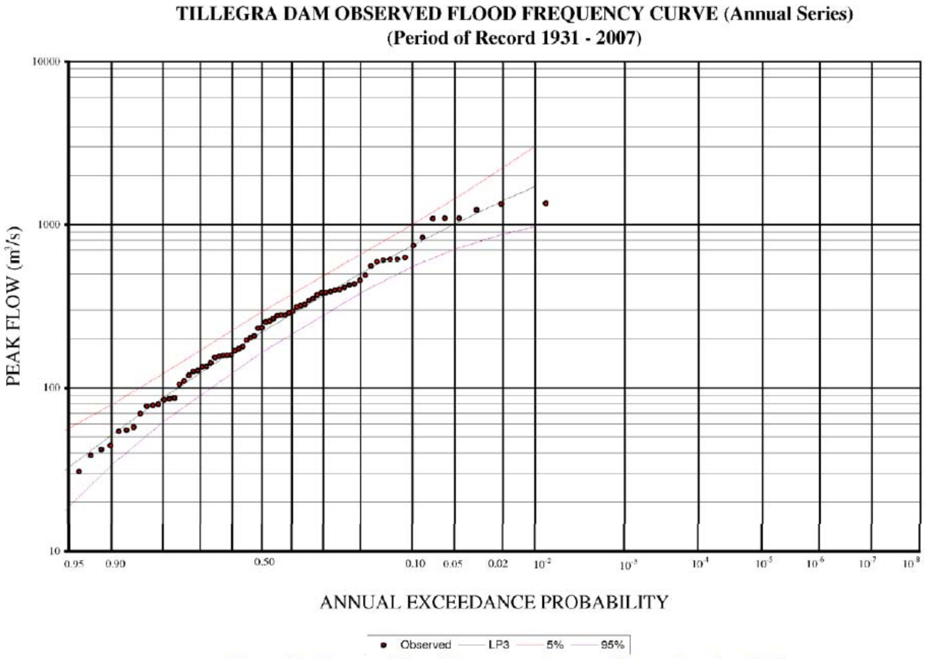
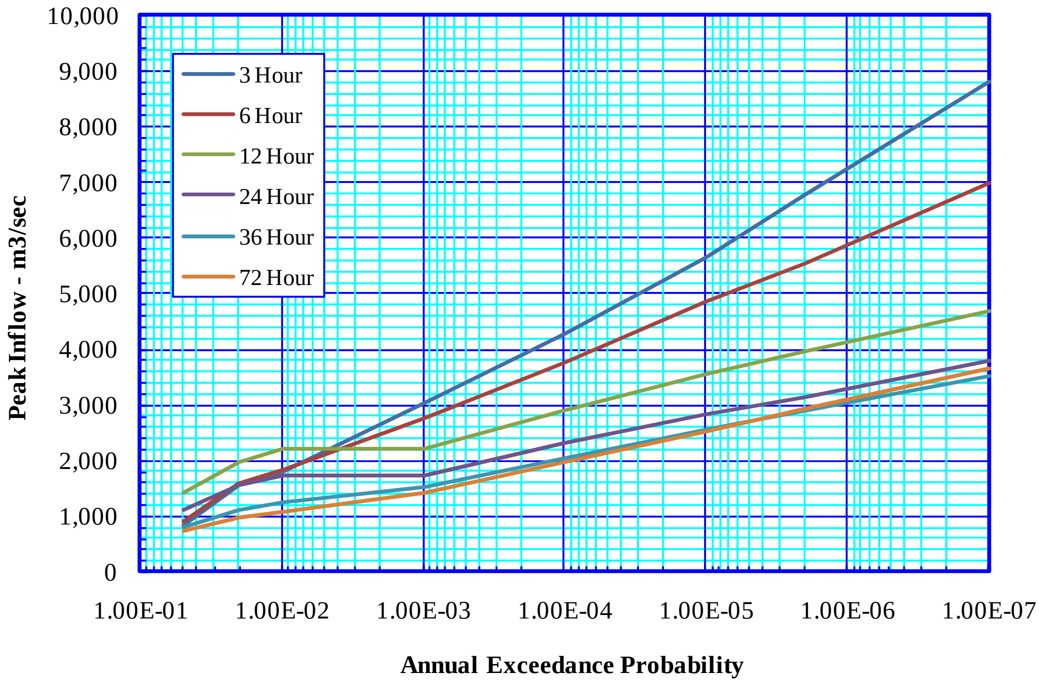


Figure 36: Observed Flood Frequency Curve – Tillegra Gauging Station

Figure 5-8 - Design Flood Inflow Frequency Curves (Annual Series)



6 River Diversion during Construction

6.1 General Comment

Attitudes to flooding and flood damage have changed considerably in recent years and while major advances have been made in risk assessment for dams in general, little work has been done on river diversion during construction. There have been few new on-river embankment dam constructions in recent years to provide precedents, or to require Regulators to develop policy. One recent study to address the problem is Hill et al (2008).

Risk assessment during river diversion can be a more complex issue than that for a completed dam. Complications include a rapidly varying probability of failure throughout the construction period and a rapidly varying consequence of that failure. Failure mechanisms during overtopping are difficult to assess. Guidelines are vague and Regulator requirements are still under development.

During the early stages of construction when cofferdam and embankment levels are low, embankment overtopping can occur for relatively small floods but consequences of failure may be low and largely limited to disruption of construction activities.

As embankment levels rise during construction, the probability of overtopping reduces but the consequences of an embankment failure may increase substantially. The loss of partly constructed embankments can have severe impacts on the downstream river, towns and infrastructure.

Specific construction activities, such as flood gap closures procedures, can involve increased risk of failure over very short construction periods.

Consequence studies will not be completed until mid 2009. A risk analysis will be carried out when the data becomes available. The river diversion procedure outlined below is based on experience with similar dam constructions.

6.2 Initial Diversion Proposal

The river diversion works proposed for the CFRD construction comprise:

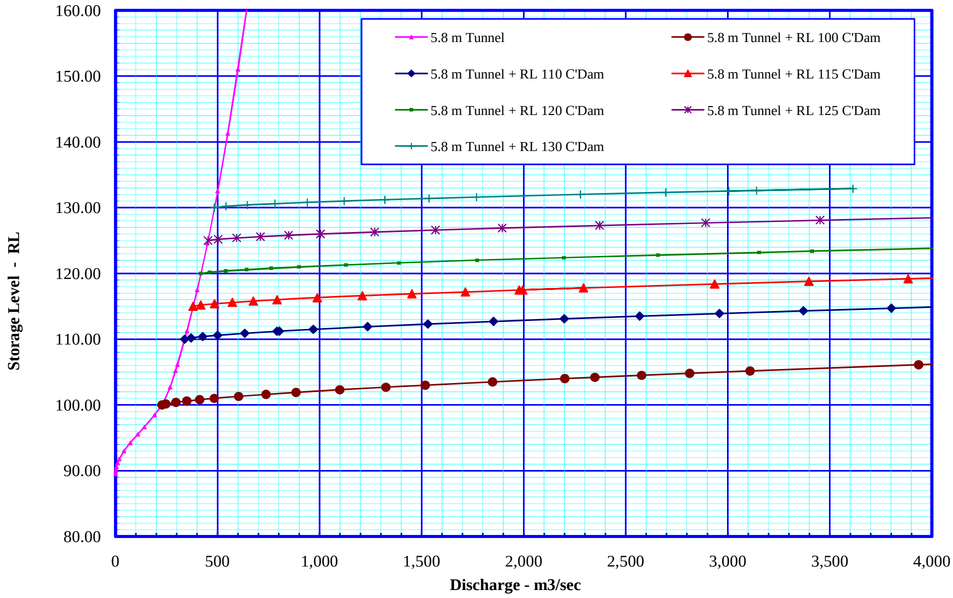
- A 5.8m diameter concrete lined tunnel through the right abutment;
- An 850 mm bypass pipe located in the tunnel lining to provide environmental flows during outlet construction;
- Low height upstream and downstream cofferdams that divert normal river flows and small floods through the tunnel. The upstream cofferdam would fail under overtopping and the height will be restricted to a safe level consistent with flood warning and evacuation procedures. An upstream cofferdam to RL 102 is proposed at this time.
- A main cofferdam, referred to as the downstream stage, located within the downstream batter line of the main embankment and reinforced with a steel mesh to enable large floods to be passed over, and to some extent, through the cofferdam. A crest level of RL 125 has been adopted at this time.

Critical dimensions such as cofferdam heights and tunnel diameter are not fixed and may be adjusted during the risk assessment process to provide better outcomes or reduce cost. The tunnel diameter and the area of steel meshing on the downstream face of the cofferdam are major cost items.

6.3 Tunnel and Cofferdam Hydraulics

The discharge ratings for the tunnel operating alone and for the tunnel operating together with various construction levels for the downstream stage (no failure) are shown at Figure 6-1.

Figure 6-1 - Discharge Rating for a 5.8 m dia Tunnel & Varying Cofferdam Levels



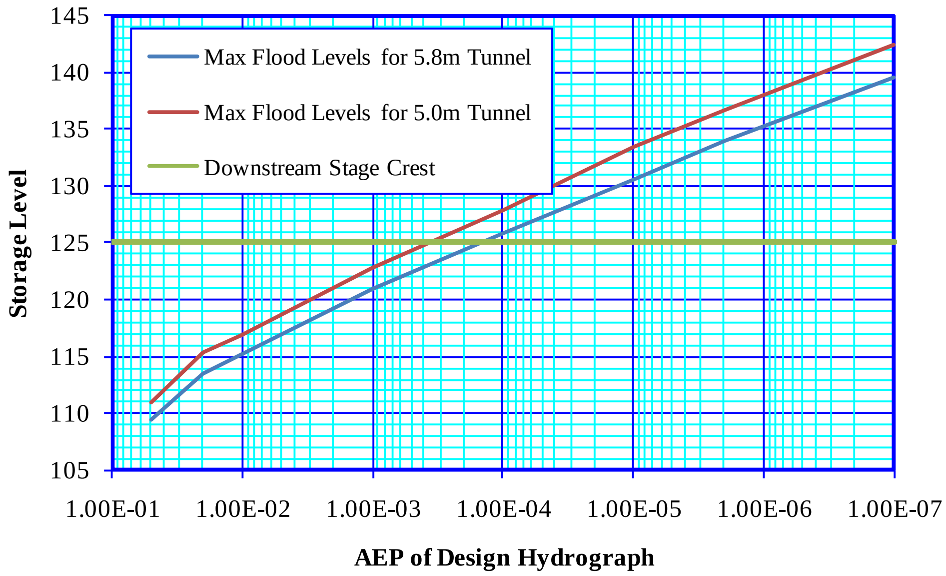
As noted at Section 5, inflow hydrographs are available for:

- 1 in 20 to 1 in 100 AEP events each for a range of storm durations from 1 hour to 96 hours;
- Three sets of AEP events for 1 in 1,000, 1 in 10,000, 1 in 100,000, 1 in 500,000 and PMP storms with:
    - GSDM , covering storm durations of 1 to 6 hours;
    - GSAM, covering storm durations of 9 to 96 hours;
    - GTSMR covering storm durations of 9 to 96 hours.

The above are available for annual flood series and additional hydrographs are available for four seasonal series. The construction programme is tied closely to seasonal variations in inflows with critical operations timed for the drier July to December period. Flood inflow estimates can be substantially lower in these months.

With the tunnel operating and a sufficiently high embankment to prevent overtopping, the hydraulics are controlled by the longer duration storms, typically the 36 hour storms for cofferdams levels up to RL 125 and 48 to 96 hour duration storms at higher levels. The variation in maximum storage level for longer duration is small. The maximum flood levels for AEP inflow hydrographs are shown at Figure 6-2. The tunnel can handle the 1 in 9,500 AEP inflow flood without overtopping the completed downstream stage. The smaller tunnel does not have a major impact and risk analysis may well demonstrate that a smaller tunnel is acceptable.

Figure 6-2 - Maximum Flood Levels for AEP Hydrographs



For situations where the cofferdam is overtopped, short duration storms give the maximum overtopping depth and outflow discharge for cofferdam levels up to RL 120. The overtopping duration is short, as is the warning time.

Longer duration storms provide a smaller overtopping depth and a lower outflow discharge. However the duration of overtopping is longer and the risk of failure for a given depth is higher. There is a significant warning time that should reduce consequence estimates.

It is noted though that downstream flood levels will follow a different pattern from natural floods. In the early stages river levels will be unusually low due to storage routing and liable to give the impression that flooding is limited. Once cofferdam overtopping commences, flood levels rise more rapidly than would occur for natural flooding.

At higher cofferdam levels, the longer duration storms become critical while the shorter duration storms do not have sufficient volume to cause overtopping.

Flood routing data for the diversion works are provided at Appendix B. The conditional probability of failure of the meshed downstream stage from overtopping is a function of both the depth of overtopping and the duration of overtopping.

The volume of the hydrograph is a major factor in downstream flooding and the short duration storms have much lower inflow volumes than the longer duration storms. Long duration storms will have the major impact on consequence studies.

Figure 6-3 - 5 Hour GSDM 1 in 100,000 AEP Flood Routed through the 5.8 m Tunnel & over RL 110 Cofferdam

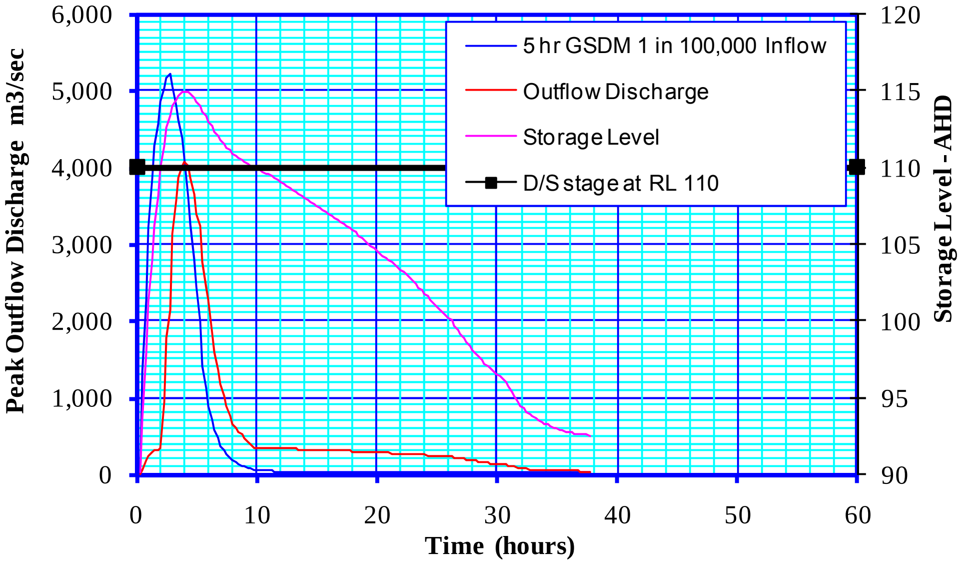
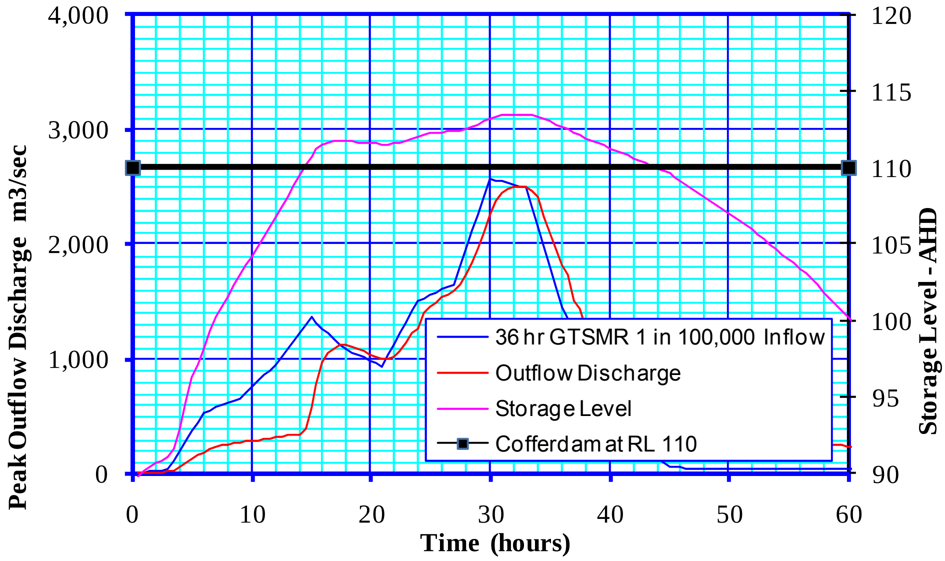


Figure 6-4 - 36 Hour GTSMR 1 in 100,000 AEP Flood Routed through the 5.8 m Tunnel & over RL 110 Cofferdam



It is not obvious which storm durations will be critical and a significant number of consequence estimates are required before a risk analysis can be undertaken.

Flood routing results for the 5 and 36 hour 1 in 10,000 AEP hydrographs are shown at Figure 6-3 and Figure 6-4 for a downstream stage level of RL 110 as an example.

The short duration flood has an overtopping depth of 3.3 m and an overtopping duration of 6 hours. The 36 hour storm produces an overtopping depth of 2.3 m and an overtopping duration of 29 hours.

If the time the water takes to rise from RL 100 to the peak outflow is taken as a somewhat arbitrary measure of warning time, then the 5 hour storm provides 2 hours compared with 26 hours for the 36 hour storm.

6.4 Construction Program and Sequencing

6.4.1 Construction Program

The construction program for river diversion operations is shown at Figure 6-5. The red points on this diagram represent the peak flood levels for historical floods routed through the tunnel. They provide a visual assessment of the diversion capabilities in terms of historical flood events.

For initial design purposes, the program assumes:

- An arbitrary date for award of contract of 3/6/2010;
- An average rate of fill placement in the downstream stage of 28,000 m^3/week, with lower rates in the early and final stages of construction. The peak rate is 35,000 $m^{3/}$/week;
- An average rate of fill placement in the main embankment of 60,000 m^3/week, with lower rates in the early and final stages of construction. The peak rate is 80,000 m^3/week.
- Mesh placement has been limited to 1,200 square metres per week and this controls the rate of rise of the downstream stage for part of the time.

The construction sequence is shown on drawings C-106 to C-108 and briefly described below. It ties key operations to the statistically dry periods of the year as far as possible:

- The upstream cofferdam is constructed in early July but controls the river diversion for only 8 weeks;
- The reinforced downstream stage is completed to RL 125 in mid November where overtopping has an annual probability of 1.6 * 10^{-4}.
- The main embankment construction is completed to RL 120 prior to the Christmas closedown period.

The embankment construction between RL 125 and RL 140 is a potentially critical period as the rockfill rises above the level of the mesh protection. Overtopping can erode rock at the downstream face, damaging the mesh protection at lower levels. This construction occurs in the wetter months of January to early February.

A risk reduction procedure shown on the programme at Figure 6-5 uses a three stage operation to raise the embankment rapidly (9 days) and safely to RL 135 where overtopping has an annual probability of $1*10^{-6}$.

If the risk analysis shows this operation contributes substantially to the overall risk of failure then the procedure can be modified.

The notes in the following Sections outline the requirements for construction works in each phase.

6.4.2 Phase 1 from Award of Contract to Commencement of Diversion:

The critical operations are construction of:

- The link road and bridge over the Williams River;
- The diversion tunnel and channels, concrete lining and intake tower base and installation of the bypass pipe;
- Overbreak and curtain grouting of tunnel;
- Excavation of the spillway discharge channel;

During this phase, the quarry is opened, crushing plant and concrete batch plant are operational and the spillway excavation is commenced. Embankment stripping, toe slab excavation and toe slab construction are commenced.

River flows are not affected in this Phase.

6.4.3 Phase 2 from Commencement of Diversion to Closure of Diversion Tunnel:

Phase 2A: Upstream Cofferdam Control

The upstream and downstream cofferdams are constructed and the river is diverted through the tunnel on the 1st July, 2011. The river bed foundations for the downstream Stage are completed and the downstream stage construction is commenced.

The river bed foundations for the main embankment and toe slab are excavated and the toe slabs constructed and grouted.

Spillway excavations are continued with suitable rock directed to embankment construction.

River Flows are passed through the tunnel. The tunnel and upstream cofferdam at RL 102 are capable of handling an inflow with an AEP of 1 in 4 on an annual basis and all recorded floods for the July, August, September period. Recorded floods in late June and early July would likely fail the cofferdam during construction.

During the latter stages of Phase 2A, the downstream stage provides a tailwater and the meshed protection provides additional security, progressively reducing risk of failure but not risk of flooding the works.

Phase 2B: Downstream Stage Control

The downstream stage with steel mesh protection is completed to RL 125 followed by the main embankment. The programme shows a Christmas closedown with the downstream stage at RL 125 and the main embankment at RL 120.

River Flows are passed through the tunnel while larger floods are passed over the meshed downstream stage. The Downstream Stage at RL 116.2 can handle all recorded floods without overtopping. At RL 125, the probability of overtopping is $1*10^{-4}$.

Phase 2C: Main Embankment Control

The embankment is raised to from RL 125 to RL 140 in a 4 stage process as shown at drawing 107:

- Construct a minimum width embankment to RL 135 on the left and right abutments leaving a 150 m wide central channel at RL 125.
- Close off the central channel to RL 135.
- Complete downstream embankment fill to RL 135, when the probability of overtopping is $3*10^{-6}$;
- Complete embankment to RL 140 as a full width raising. At this level it can handle the PMF without overtopping.

If the risk assessment shows this operation to be a significant contribution to total risk, then the initial stages may need to taken to a higher level.

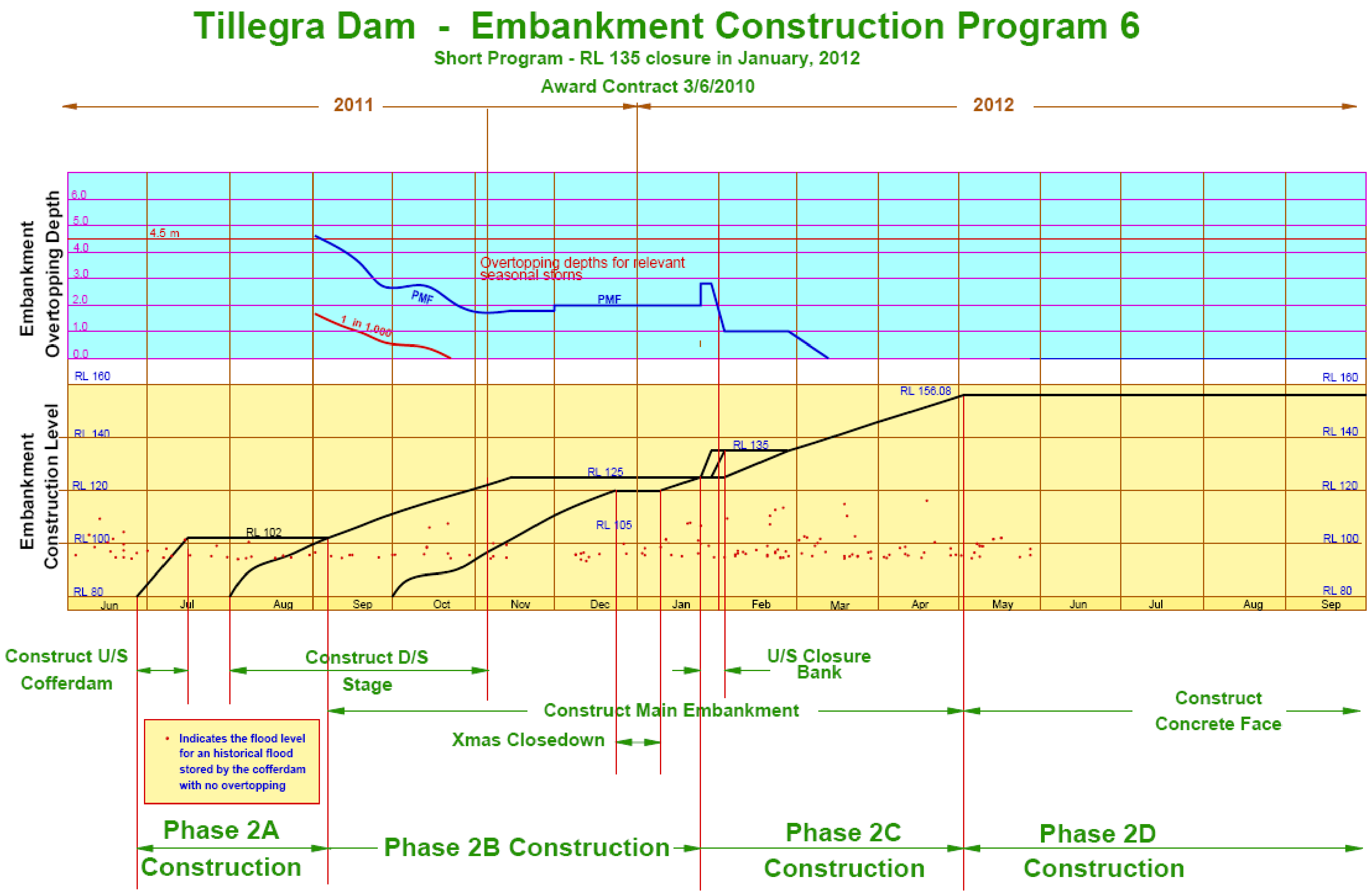
The remainder of the main embankment is then completed to the base of the parapet wall. At RL 135, the main embankment can still be overtopped by long duration PMF storms.

The spillway concrete would normally be completed by the end of this Phase. This is not critical at Tillegra, given the ability to pass the PMF at much lower storage levels.

6.4.4 Phase 2D: Face Slab and Parapet Wall Construction

The face slabs are slip formed to the base of the parapet wall, the parapet wall is constructed, and the crest roadworks completed.

Figure 6-5 - Construction Program for River Diversion



6.5 Phase 3 from Tunnel Closure to Completion of River Outlet

Following completion of the embankment, the tunnel is closed by installing stoplogs in the base of the tower and concreting the upstream diversion plug. The penstock and concrete surround are installed, followed by the valve block, pipework and valves. The mini hydro can be installed at this time if convenient as can the connection to the CTGM.

Minimum riparian releases are passed through the 850 mm bypass pipe during this Phase with a temporary bypass extension pipe permitting discharge past the valve block. A temporary downstream butterfly valve controls outflow discharges.

The dam commences to fill once the tunnel is closed.

6.6 Embankment Overtopping

6.6.1 Overtopping Depths

The overtopping depths for the downstream stage during extreme flood events are shown at Figure 6-6 and Figure 6-7. The vertical upturn at RL 125 indicates the increase in head due to constricting the cofferdam width to 150 m for a short period in Phase 2C.

Figure 6-6 - Overtopping Depths for PMF Inflow

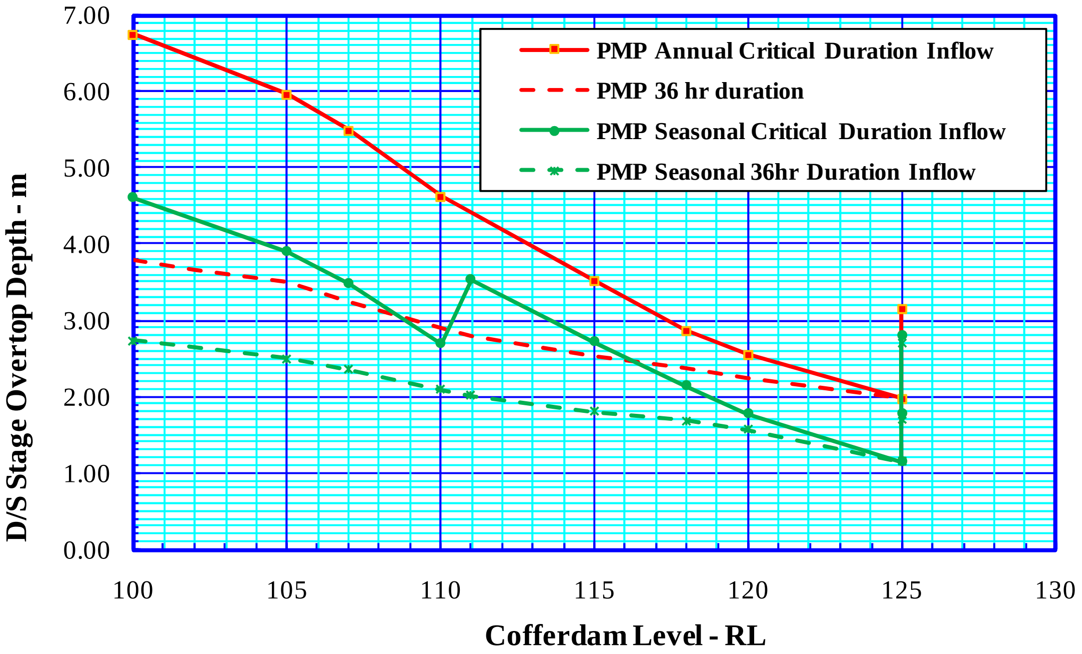
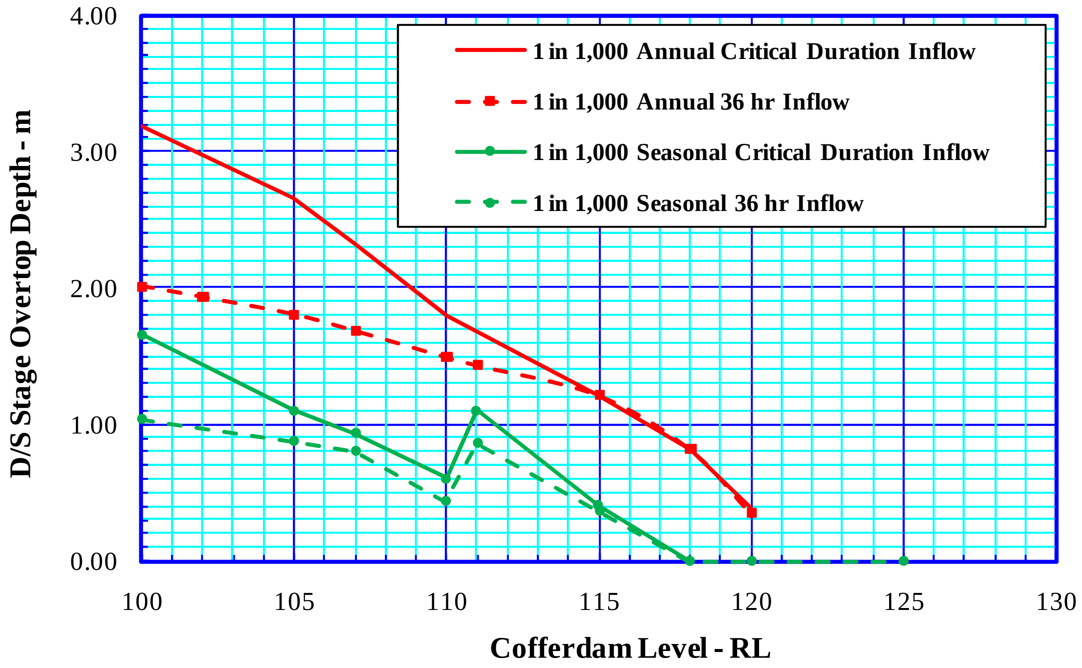


Figure 6-7 - Overtopping Depths for 1 in 10000 AEP Inflow



The following points are noted:

- The vertical discontinuities for seasonal flows in Figure 6-6 and Figure 6-7 are due to seasonal changes from the dry June-September season to the wetter October-November season and then again to the wetter December to March season;
- The vertical increase in Figure 6-6 at RL 125 also shows the effect of the staged construction reducing the effective width to 150 m;
- Seasonal PMF overtopping depths are less than 3.0 m once the cofferdam rises above RL 114. The larger depths are due to short duration storms with a shorter period of overtopping flow;
- Seasonal overtopping depths for the 1 in 1,000 AEP inflow are less than 2.0 m at any level and generally less than 1.0 m;
- The embankment construction is capable of handling the PMF by storage and tunnel discharge at RL 139.6;
- The 5.8 m diversion tunnel has limited effect on flood levels and the ability to handle extreme floods is primarily due to good storage characteristics. Use of a smaller tunnel is under consideration with the ability to control small floods at low embankment levels and the need to quickly drain the storage following a major flood event being important factors.

6.6.2 Reinforced Rockfill

Reinforced rockfill is widely used for cofferdam construction to protect the rockfill and any underlying materials from the erosive effects of flowing water. The design of the surface mesh is generally based on past performance and typical reinforcement arrangements are shown at ICOLD (1993). Attempts have been made to develop rational design methods with seepage flows estimated using flow net analyses for turbulent flow or scale model tests. A number of methods are available for stability analysis including computer programs that model embedded reinforcement elements.

Important considerations include:

- The mesh be designed such that its rate of installation is not a major restraint on rockfill placement. Mesh construction has been limited to 1,200 square metres per week. This controls downstream stage construction at the higher elevations but adds only 2 weeks to the program.
- Rockfill be sized to suit the mesh size and the mesh be constructed tightly against the rockfill to prevent movement during flow through conditions. Containment concrete is required to fill any large gaps between mesh and rockfill
- The extent of rockfill placed ahead of mesh protection be limited to ensure unexpected inflows do not overtop an uncompleted lift line;
- Particular care be taken to anchor mesh at abutments where discharges are concentrated.

The Tillegra proposal uses a full width reinforcement system extending from abutment to abutment. Consideration was given to a central channel system during construction as used at Babagon Dam, but the additional reinforcement required for channel side walls and the overall complexity showed little cost advantage.

ICOLD Bulletin 89 notes that performance data for reinforced rockfill is not readily available and no clear limitation on the depth of overtopping that reinforced rockfill can safely withstand has been established. It suggests that a flow in excess of 3 m critical depth (equivalent to a head of 4.5 m or a discharge of 15 m^3/sec per metre width) may be considered dubious, while noting that overflow depths of 10.5 m have been sustained without damage.

Overtopping depths of 3 m can occur only for low cofferdam levels, below RL 109 for the PMF and below RL 100 for the 1 in 1,000 AEP inflow.

6.7 Traditional Approach to River Diversion

6.7.1 International Practice

International practice is probably best summarised by P. Machado, the General Reporter for the 22nd ICOLD Congress Question 84 titled “Technical solutions to reduce time and costs in dam design and construction”.

> *The selected values of the construction flood may, and quite often are, different for different phases of the construction, reflecting the increased risk and/or the length of the period of exposure. In general for the main construction phases, the selected construction flood values correspond to recurrence periods (T_R) between 10 and 50 years (R. 1, R. 4) but values up to 100 years are sometimes considered (R. 29). Initial site works, covering limited periods of exposure, when construction work is still insipient and*

expected damages caused by overtopping is not too significant, may be designed for smaller T_R values, around 2 or 5 years. On the other hand, with construction activities well advanced and when embankment cofferdam height is large, storing significant volumes of water that could produce major damages at the construction site or downstream if released by cofferdam failure, diversion criteria can require a high T_R of the order of 500 years or more (R. 42, R. 45, R. 46, R. 48).

For rivers where there is a marked seasonality of the hydrological regime it is also usual to establish construction phases in such a way as to allow coping with the risk of overtopping in a progressive way. This means evaluating the probability of floods in a seasonal basis, as opposed to the annual basis, and defining different construction floods for wet and dry seasons.

As a rule, for the same return period, flood values computed for dry and wet seasons are very different, and allow designing diversion structures (cofferdams and diversion conduits) for different flow parameters in successive construction phases, assuming smaller risks as the height of dam increases and the construction enters into the wet season.

The ICOLD Bulletin 48 on River Control during Dam Construction, ICOLD (1986), also provides some guidance on river diversion, particularly on minimising river diversion costs. The annual cost of the diversion works is compared with the annual cost of damages where the latter typically include damage to:

- Loss of life.
- Property downstream and upstream;
- The construction site;

The ICOLD Bulletin is generally consistent with the practice reported in ICOLD Question 84. Where loss of life is specifically considered, it is treated as an item in the annual damages estimate. In contrast to current ANCOLD Guidelines no criteria are mentioned for loss of life, either in the ICOLD Bulletin or in other references.

6.7.2 NSW Practice

Dams & Civil have developed river diversion schemes for a number of embankment dams in NSW. Split Rock, Glennies Creek, Pindari (Stage 2), Windamere and Glenyon (Queensland) are of similar size to Tillegra while Copeton at 100 m height and 1,200,000 ML is a little larger. Smaller dams included Toonumbar, Lostock, Pindari (Stage 1) and Brogo.

All construction programs were tied to wet and dry seasons with strict time controls in the contract documents. A construction program similar to Figure 6-5 that could handle all floods of record once the cofferdam reached a substantial height was judged to provide adequate safety. A freeboard equivalent to a 10% increase in the hydrographs was a normal requirement. These procedures satisfied international review at the time.

ANCOLD practice at that time, ANCOLD (1986), proposed that a tolerable risk would be achieved by equating the probability of failure over the *at risk* construction period to the tolerable probability of failure of the dam over the *in service* life.

A more conservative approach was taken for Copeton Dam due to its storage volume and a requirement for staged filling of the storage prior to completion. Two diversion channels were excavated, one at a low level and one at a high level, with the latter designed to increase flood handing capability to 50% of the PMF. The PMF at that time (1960's) was a somewhat smaller flood magnitude that is obtained today. Both diversion channels operated during construction.

The program shown at Figure 6-5 is capable of handling all recorded floods for 70 years of records with a generous freeboard. The 50% PMF criteria is equivalent to a 1 in 3,000 year AEP and can be handled by the tunnel/downstream stage arrangement with a maximum storage level of RL 123.5. This is well below the crest of the meshed downstream stage.

This diversion program shown at Figure 6-5 would have been considered somewhat conservative by the above criteria and consideration would have been given to a lower level for the downstream stage (say RL 120) with some saving in face reinforcement.

6.8 Risk Analysis

6.8.1 Current ANCOLD Guidelines

The current ANCOLD Guidelines on Risk Assessment, ANCOLD (2003) takes the following approach.

> *In assessing flood provisions for dam works under construction, the traditional approach has been to relate the tolerable risk of failure to the duration of the "at risk" period (Institution of Civil Engineers, 1996 and Appendix A of ANCOLD, 1986), a practice that may satisfy the usual primary consideration of protecting the owner's assets. However, if lives could be at risk, owners need to be aware that taking account of duration, as in the two sources just cited, could be problematic. In such cases, the individual and societal risks to life should be estimated and the risks reduced ALARP.*
>
> *The time period over which a life safety risk applies, may not be a consideration in deciding whether the risks are ALARP*
>
> *The question of duration is a difficult one.*
>
> *Where persons are exposed to a risk for only a "once off short period", or intermittently on a regular basis, the application of exposure factors are allowed in these Guidelines, effectively permits a higher individual risk to life than in the Guidelines of G10-1 during exposure, on the basis that this higher risk is offset by periods of zero risk. There is an averaging process, which ensures that over any significant period of months or years, the average risk to the person complies with the Guidelines of G10-1.*

However, the situation could be different where there is a lengthy period (months or a year or so) of continuous exposure, but the exposure ceases at the end of that period. In such a situation it does not seem reasonable to speak of risk averaging". This aspect applies particularly to dams under construction, or being modified, where there is a population at risk. It could be argued that it is not acceptable to have a higher risk imposed on the population over the lengthy period of exposure – in particular to exceed the limits of tolerability in G10-1 – than would be tolerable for an indefinite period. In such a case, it is reasonable to recall that the life safety criteria at G10-1 are not qualified as to period of exposure.

Consequently, for dams under construction or being modified, the life safety risks should be estimated, and the risks reduced ALARP within the tolerability zones, by measures such as:

- *All reasonable practicable methods at the site, such as provisions for safely passing floods, that will reduce the life safety risks;*
- *Construction phase DSEP's, including flood warning and evacuation planning.*

6.8.2 Risk Assessment Process

A detailed risk assessment will be undertaken to finalise the diversion requirements including:

- The tunnel diameter;
- The crest level of the downstream stage;
- The construction program and critical contractual requirements;
- The need if any, for additional features such as staged construction, diversion channels etc;
- The impact of construction delays.

The following procedures are considered to be consistent with ANCOLD (2003):

- Inflow flood probabilities for initiating events are based on seasonal flood series;
  - Seasons are December-March (17 weeks), April-May (9 weeks), June-September (17 weeks) and October-November (9 weeks);
  - A separate risk analysis based on the annual flood series will be used for comparison purposes;
- One week time intervals are used;
- The conditional probability of a December-March flood occurring in any one week during this 17 week period is 1/17;
  - For the annual series, the conditional probability of a flood occurring in any one week during the 52 week year is 1/52.

- The conditional probability of failure of reinforced rockfill is a function of both overtopping depth and overtopping duration;
    - It is acknowledged that there is little data available on the failure of reinforced rockfill. As a result, failure is considered as a single mechanism, rather than a series of contributing mechanisms;
    - The conditional probabilities will be determined by a panel of experienced engineers.
    - The conditional probability of failure of non-reinforced rockfill subjected to an overtopping depth of 500 mm or more is 1.0.
- A number of storm durations are available for each inflow flood partition, each producing different overtopping depths and durations and different consequence estimates. The storm with critical consequence estimate is adopted;
- The total risk is compared to the ANCOLD (2003) F-N limit lines, plotting each week as a separate failure mechanism;
- For the risks to be reduced ALARP within the tolerability zones, it is assumed that:
    - All reasonable practicable methods at the site, for safely passing floods will be adopted;
    - Construction phase DSEP's, including flood warning and evacuation planning will be put in place.

The ANCOLD Guideline requirements may require a substantially more conservative river diversion procedure than previous Australia or current overseas practice. If the diversion procedure outlined above is not satisfactory, a number of risk reduction options are available:

- The use of staged construction to provide a rapid rise in embankment height during critical construction periods;
- The use of a high capacity diversion channel excavated around the downstream stage to reduce overtopping depths;

6.9 Tunnel & Channels

Tunnel and channel geology is described at Section 4.9.

The majority of the tunnel is well within the high seismic velocity zone, with velocities ranging from 2,800 to 4,500m/sec indicating fresh rock. The alignment is approximately normal to the strike of bedding which dips moderately upstream. It has been assumed that the relatively close defects and bedding orientation will require continuous concrete lining to handle the maximum diversion discharge of 500 m^3/sec with flow velocities of up to 19 m/sec. Primary support prior to lining would be by rockbolts and mesh in the tunnel crown. Minor faults and small shears may require additional and/or longer support where locally occurring. A thin layer of shotcrete may be required as primary support in the relatively closely jointed rock mass.

The upstream portal is located in a thickly developed intermediate seismic velocity zone, with a velocity of the zone of 2,100m/sec. This is interpreted to comprise moderately weathered/slightly weathered rock. The relatively low intermediate seismic velocity is interpreted to be related to defects in the rock mass.

Bedding is at 45^{o} and dips into the excavation and batter slopes of 1 on 1 are used. Steeper batters at the portal will require support by a ground anchorage system. These portal conditions need to be re-evaluated with an additional borehole.

Consideration is being given to moving the portal 25 m downstream, where it will be located in fresh (stained) and fresh rock with a seismic velocity of 3,600 m/sec. A new portal site will be investigated during the ongoing design stage investigations.

The downstream portal is expected to be excavated in rock beneath a thin soil cover. Bedding at the downstream portal dips into the portal batter, with the bedding strike normal to the side batters. The stability of the portal batters and the left channel wall will be influenced by the major joint set normal to bedding, which is interpreted to be dipping very steeply to the southeast, that is, into the excavation. The stability of the right wall will be controlled by a major joint set striking approximately east-west, dipping moderately to the north. Again, batters slopes of 1 on 1 have been provided.

6.10 Upstream Coffer Dam

The upstream coffer dam is a rockfill embankment with upstream clay core and filter zones. The foundation geology is described at Section 4.10.

The clay core and filter foundation would be excavated to moderately weathered rock and the rockfill zones to highly weathered rock. This would typically require stripping depths of 3 m and 1.5 m respectively. Greater excavation depths occur in places such as the middle left abutment where some 5 m of gravel in a clayey sand mix has been located.

7 CFRD Embankment

7.1 General Comment

7.1.1 Conventional CFRD

The modern CFRD using compacted rockfill was developed in the mid 1960's with Cethana Dam in Tasmania, the first stage of Pindari Dam in NSW and Kangaroo Creek in South Australia being among the earliest examples worldwide. CFRD design and construction has evolved with time as a series of incremental steps based on experience with the design widely promoted by a US consultant, J Barry Cooke.

There is no recognised design manual for CFRD constructions, except for a brief ANCOLD Guideline (ANCOLD 1990) and an early ICOLD Bulletin (ICOLD 1993). The development of the design has been documented in a series of international symposiums and ICOLD conferences, in particular:

- The first symposium held in Detroit in 1985 detailed a large number of recent CFRD constructions. From these, two papers were published in the ASCE Geotechnical Journal, Cooke and Sherard (1987) These are still considered the classic reference for this type of dam;
- The second symposium held in Florianopolis in 1999 reflected the progress made in the previous 15 years including the 180 m high dams, Aguamilpa and Tianshengqiao (TSQ-1);
- The third symposium held in Florianopolis in 2007, was specifically directed at the problems encountered with the recent high dams including Campos Novos (202 m), Barra Grande (185 m) and Mohale (145 m).

There has been little change in CFRD practice for small dams such as Tillegra since the above mentioned Cooke and Sherard (1987). Those changes that have occurred are probably best summarised by Pinto (2001).

Dams & Civil has worked closely with Cooke on 8 CFRD designs as well as on some 9 concrete and central core rockfill dams. The comments on CFRD construction in this document are in accordance with Cooke and Sherard (1987) and Pinto (2001) unless otherwise noted.

The design and development of CFRD construction has been primarily based on precedent and empiricism. The rockfill embankment batter slopes of 1.3H:1V are roughly the angle of repose of dumped rockfill. The compacted rockfill on a sound rock foundation has no water in the voids and is inherently stable. Stability analyses are not carried out unless the foundation has unfavourable joints or other planes of weakness. The concrete face thickness and reinforcement are based on precedent and for dams up to 100 m high a uniform thickness of 300 mm is generally used. Face slab widths are determined by slip form equipment and operations.

A large number of CFRD constructions have been completed in Australia, mostly in NSW and Tasmania, but also South Australia, Victoria and Queensland. The highest is the 122 m high Reece Dam in Tasmania. No serious problems have been encountered with these dams, nor with similar height dams constructed overseas. The design produces a high quality embankment with few problems for a project such as Tillegra.

Some dams have suffered from leakage through the concrete face, generally due to poor construction practice. Leakage is a business risk and not a dam safety issue as the design can safely handle flow through the rockfill. The 40 m high Brogo Dam filled and the spillway operated prior to construction of the concrete face. Although based on an older design with pervious Zone 1 material, the dam handled this situation without difficulty. Current designs provide a reasonably impervious Zone 1 material that limits leakage from any face slab deficiencies. The exposed concrete face lends itself to comparatively simple repair operations if leakage does occur.

7.1.2 High CFRD Constructions

The conventional assessment has been that CFRD construction could be used for dams up to 300 m in height. There have, however, been a number of problems with some recent larger CFRD designs over 150 m in height, mostly due to cracking of the concrete face.

The tendency of CFRD embankments to deflect downstream under water loading with a clear component towards the centre of the valley has been well known for decades. The compressive stresses in the central slabs was considered a positive feature while the tensile stresses produced adjacent the abutments were compensated for by increased reinforcement.

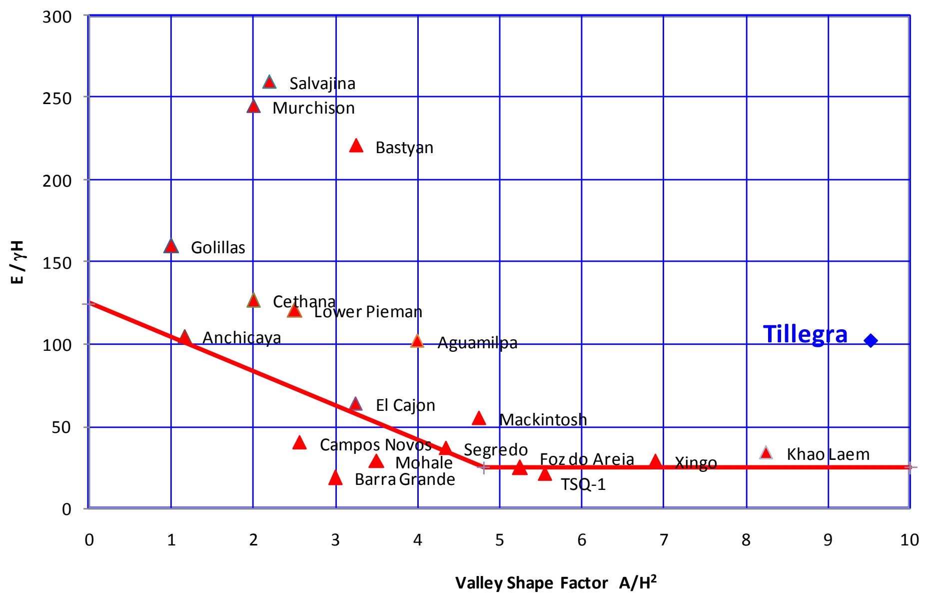
Extensive rupture of the concrete face however has now occurred on three dams, Compos Novus, Barra Grande and Mohale and more minor cracking on several others... A photograph of the damage at Campus Novus is shown at Figure 7-2.

The very high compressive strains imposed on some face slabs are considered to be the result a combination of dam height, low rockfill deformation modulus and unfavourable valley shape. Barra Grande, Compos Novos and Mohale were similar designs using poorly graded basalt that gave a low modulus fill, particularly in the downstream shell where there was less compaction and no water. All three were constructed in narrow gorges.

Figure 7-1 is a dimensionless plot of rock modulus ($E / \gamma_w H$) versus a valley shape factor (face slab area A / H^2) where E is the construction modulus and H the height of the dam. This graph is taken from Pinto (2007) with Tillegra added using a modulus of 75 MPa.

The dams below the line include the above three mentioned dams while other dams that have had some problems lie close to the line. Pinto recommends the graph “as a rough indicator of a potentially unfavourable situation in respect to excessive compressive stress in the face slab.”

Figure 7-1 - Rock Modulus versus Valley Shape Factor



The three dams that suffered major damage had significant leakage of up to 1,400 l/sec. Attempts to seal these leaks with silty clay material have been only partly successful for short periods of time. Campus Novus was drained due to quite separate problems with a diversion tunnel and the cracks repaired using vertical joints with compressible material to absorb the compressive strains. This appears to have been successful and the dam is now operating normally although leakage is around 1200 l/sec.

Barra Grande and Mohale have not been repaired and continue to operate with cracked face slabs and high ongoing leakage.

It is noted that while the face slab damage is severe, all three dams have continued in service. Leakage of up to 1,400 ML/d was at no time considered a danger to dam stability.

The process of developing CFRD designs progressively based on experience with previous projects is now seriously questioned. Considerable effort is being expended on finite element analyses (FEA) to develop appropriate analytical techniques. The results to date have not been impressive due to the overall complexity of the problem (Pinto 2007). Rockfill deformations have been reproduced reasonably well by FEA but face slab analyses have not reproduced compressive stresses high enough to crush concrete, Xavier et al (2007).

Other factors, apart from height, rock modulus and valley shape, that are typical of recent dams and may be relevant include:

- The widespread use of extruded kerb elements in recent dams (as discussed at Section 7.4.2) as the finishing surface for the upstream face of the rockfill. These may prevent the concrete face from floating on the rockfill and contribute to high compressive stresses. In particular, they may inhibit the opening of tensile joints. A number of projects have attempted to provide a membrane between the kerbs and the concrete face. Nelson Pinto has defended the use of kerbs (Pinto, 2007) and argued that they are incapable of transferring compressive strains to the concrete face. They continue to be used, with or without the membrane.

- The use of low modulus rock in the downstream half of the dam. There is some evidence that settlement may have been increased by leakage wetting the low strength material in the downstream half of the dam;
- The elimination of anti spalling reinforcement along compression joints;
- Reduced thickness of the concrete slab.

Figure 7-2 - Compression Failure in Face Slab at Campos Novos Dam

7.1.3 Relevance to Tillegra

The three critical factors causing problems with the concrete face are:

- Dam Heights over 150 m:
    - Tillegra at 76 m is a relatively low dam and there is no record of problems at this height;
- Low rockfill modulus due to poorly graded rockfill:

- Tillegra will use fresh to slightly weathered rock in the upstream Zone 3B, with a UCS of 65 MPa or higher, and is expected to produce a rock construction modulus of 75 MPa or more.

➢ A narrow valley shape with a shape factor of less than 3.5:

- Tillegra is a relatively wide valley with a shape factor of 9.5.

Tillegra parameters are plotted on Figure 7-1 and lie well above the critical line proposed by Pinto (2007). Tillegra is typical of the large number of moderate height dams built in Australia that have operated successfully with low leakage. It is located in a wide valley and will be constructed from high quality fill.

7.2 Layout

The typical cross-section for the initial layout is shown at Figure 7-3. The embankment is zoned for different rockfill quality and placement requirements. A mix of fresh to moderately weathered rock is used in the upstream zones that carry the water load, in the river bed to provide drainage and in the downstream stage. A zone of moderately weathered rock, as is likely to be obtained from the upper quarry and spillway excavations, is located in the downstream shell.

7.3 Rockfill Sources

Three quarry sites were identified for initial investigations:

➢ **Quarry Site A** comprising the right hand spillway excavation for the CFRD and a possible extension along the ridge.

➢ **Quarry Site B** comprising the ridge to the west of the dam. Surface mapping indicates that this quarry should be able to work two faces, one in predominantly sandstone and one in predominantly meta-shales.

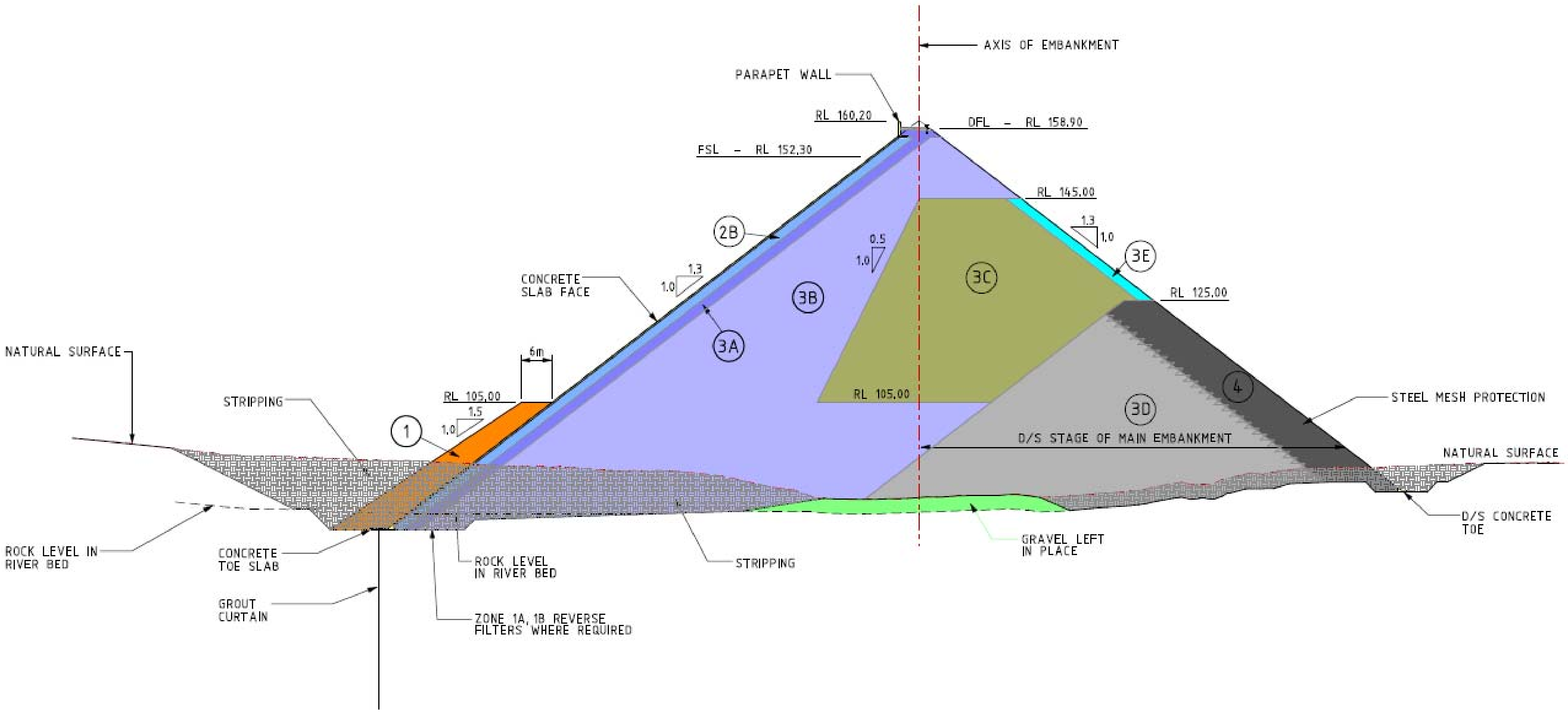
➢ **Quarry Site C** comprising an extensive area of massive welded tuffaceous sandstone along the storage rim north of the dam.

Quarry investigations have been limited to date by higher priority work. The current status is outlined at Section 4.11.

The current cost estimate assumes all quarried rockfill is obtained from Quarry B. A small quantity of sound rock will be available from the spillway. Mixtures of slightly and moderately weathered rock from the spillway and upper quarry benches would be routed to Zone 3C as discussed in Section 7.4.3.

Surface mapping indicates that separate faces could be worked in Quarry B, in predominantly tuffaceous sandstone and in an interbedded sequence of tuffaceous sandstone and meta-shale.

Figure 7-3 - CFRD Zoning



Quarry A remains an option and the short haulage distance to the dam from Quarry A would provide a lower cost rockfill. Quarry C also remains an option available to the Contractor although there are some rim stability issues that require further consideration. At this stage, no geological investigation has been carried out in Quarry C. While having a longer haulage distance than Quarry B, the high elevation and the ability to deliver rock to the dam without crossing the river has advantages. These assumptions are slightly different to those used in the Options Study at Commerce (Feb 2008).

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Petrographic analyses indicate that the tuffaceous sandstone from Chichester Dam and Tillegra are similar in mineralogy. The Chichester sandstone provided good quality concrete aggregate for the construction of Chichester Dam. Aggregate testing confirms that fresh tuffaceous sandstone and the river gravels available from the Williams River are suitable for use as concrete aggregate.

7.4 Embankment Zoning

7.4.1 General

Rockfill zones have been renamed from earlier reports and from the traditional Dams & Civil system. The current zone numbers now conform to general international systems with the processed rockfill referred to as Zones 2x, and the main rockfill zones as Zones 3x. Zone 4 is the meshed rockfill.

Rockfill material quantities within the embankment and placement specifications are listed in Table 7-2.

7.4.2 Transition Zone

The transition zone consists of a processed Zone 2B material backed by a selected Zone 3A rockfill. Both zones are generally placed in 400 to 500 mm layers and compacted to provide a high modulus support for the face slab. Zone 2A is a fine sand filter used behind the perimetral joint and over erodible foundation downstream of the toe slab.

For dams under 100 m high, the combined width of the two zones is generally 6 to 7 m. Dams & Civil practice has been to use a 4 m Zone 2 with Zone 3A a placement width (3m) of a half height layer of Zone 3B material. HEC used a 1.0 m width of Zone 2A to minimise the processing costs backed by a 5 m width of Zone 3A.

International practice (Pinto 2001) has been to reduce the width of Zone 2A and provide a specific Zone 3A to provide a more gradual transition to the main rockfill. The 178 m high TSQ-1 provided a 3 m width of Zone 2B backed by a 2m width of Zone 3A.

Widths of 3m and 500 mm layers are proposed for Zones 2B and 3A at Tillegra. At the perimetral joint, the transition zones are folded to the foundation in an L shape, and where appropriate are extended downstream over areas of weathered rock.

The “Sherard grading”, Sherard (1985) shown at Table 7-1 is preferred for Zone 2B. This is a minus 75 mm well graded material with sufficient sand sized particles to minimise segregation and provide an internally stable material.

Table 7-1 - "Sherard Grading" for Zone 2A

Sieve Size mm	Percentage Passing
75	90 – 100
37	70 – 95
19	55 – 80
4.76	35 – 55
0.6	8 – 30
0.075	2 - 12

Pinto (2001) notes that excess fines have produced cohesive materials at Xingo and TSQ-1 that have experienced cracks as a result of tensile strains and suggests the minus 0.075 mm material to less than 5 to 8%. However, the Sherard grading has been widely used and is proposed for Tillegra.

This ideal grading is not essential and a large number of dams have been successfully constructed with a coarser mix. Crushing for Babagon Dam in Malaysia could not produce the required fines for the Sherard grading and blending with imported material was not an option. Relatively coarse material was used successfully at many other projects including Pindari Stage 1, Cethana and Foz do Areia. The ideal grading may need to be adjusted when more data is available on the crushing characteristics of the meta-sedimentary rocks at Tillegra.

Zone 2B is placed in 0.5 m layers in a damp condition and compacted by 4 passes of a 10 tonne smooth drum vibratory roller. It is sensitive to excess water and the water content is not to be so high that the compaction equipment does not operate on a firm surface. Compaction to 98% of maximum density of the standard laboratory compaction test using minus 19 mm material is used to check the conventional method specification given in Table 7-2.

Face compaction using a backhoe-mounted plate vibrator is required if concrete kerbs are not used. The use of concrete kerbs and the potential downside is discussed briefly in Section 7.1.2. The type of face protection used prior to placement of the concrete face slab is generally left to the contractor. At this stage it is intended that the use of concrete kerbs be permitted if requested by the contractor. Requirements would include debonding of the face slab contact and joints at contraction joints in the face slab tension zones.

The kerbs are a lean concrete mix that are extruded along the face of the dam before the placement of Zone 2B.The height is the same as the Zone 2B layer thickness with the external face at the slope required for the face slab. An inclined internal face provides lateral support for the Zone 2B material during compaction. A 100 to 120 mm wide crest allows for some overlap of the kerb for successive layers.

Typical mixes have 75 kg/m^3 of cement, 1.9 mm maximum aggregate, 50% sand, 125 l/m^3 of water and are extruded at 40 to 60 m/hour, Pinto (2001). Lighter mixes using 60 kg/m^3 have been used recently, Pinto (2007). Compressive strengths are around 2 to 5 MPa and Zone 2B can be placed and compacted after 1 hour. The kerb contains the Zone 2B material, eliminating the need for face compaction.

7.4.3 Main Rockfill Zones

Zone 3A provides a narrow transition from Zone 2A to Zone 3B and preferably satisfies filter criteria between Zone 2A and Zone 3B. This may require some limited processing (grizzly) to remove larger material and achieve the required grading.

Zone 3B carries the water load from the concrete face to the foundation and requires a free draining high modulus fill. This is a quarry run material with a maximum size of 1.0 m obtained from fresh meta-sedimentary rocks from the nominated borrow areas and required excavations. Initial tests on Quarry B fresh rock gave unconfined compression strengths (UCS) of 67, 69 and 145 MPa. Current indications are that it will provide a reasonably well graded rockfill with a modulus of 75 MPa or higher. Further testing of rock will be carried out when samples become available.

The need for water during Zone 3A and 3B rockfill placement will depend on further testing. The one absorption test to date gave 4% indicating water may be advantageous.

The general specification for rockfill is:

- The maximum rock size is equal to the layer thickness;
- Not more than 20% should pass 4.76 mm;
- Not more than 10% finer than 0.075 mm.
- Minimum dry density of 2.0 tonnes/m^3.

Density control is generally not specified but test for density and particle size distribution are recorded for comparison with other projects.

Zone 3C comprises a mixture of fresh to moderately weathered rock, as is likely to be obtained from the top 10 to 25 m of the quarries and upper spillway excavations. The moderately weathered material is visually classed as strong but no test results are available at this time. Placement specifications for Zone 3C will be determined after further drilling and testing.

The final evaluation for Zone 3C is the ability of thoroughly wetted fill to support the travel of heavy trucks. An unstable construction surface with springing, rutting, and difficult truck traffic indicates the wheel loads are not carried by the rockfill skeleton and the rockfill will be relatively impervious.

Rockfill downstream from the axis is considered to have no influence on the face movement for small to moderate height dams and high modulus fill is often limited to the upstream one-third of the cross-section.

The 148 m Salvajina Dam in Columbia is an example with high modulus gravel in the upstream two thirds of the section and a low modulus rockfill 1/7th of gravel at 50 MPa) from spillway excavation in the downstream one-third. Marked differential settlement occurred at the interface during construction but caused no problems with the concrete face.

Table 7-2 - CFRD Embankment Zoning

Zone	Quantity m^3	Description	Material	Placement
2A,	5,300	Reverse filter material	Processed fine filter providing filter protection d/s of the toe slab where foundation conditions warrant.	Compacted to a min RD of 70%
2B	99,800	Semi-pervious u/s "cushion" zone under concrete face slab.	Crushed rockfill, max. size 100 mm, with sufficient sand sizes and fines to provide workability and low permeability. Grading as per Table 7-1.	Placed in 500 mm layer with 4 passes of a 10 tonne roller. U/S batter slope compacted with a vibrating plate or 10 tonne roller if kerbs not used.
3A	101,300	Transition rockfill	Free draining sound rockfill with max size of 0.5 m	Watered & compacted in 0.5 m layers with 4 passes of vibratory roller.
3B	871,965	U/S rockfill zone	Free draining sound rockfill with max size of 1.0 m	Watered & compacted in 1.0 m layers with 4 passes of vibratory roller.
3C	635,193	D/S rockfill zone	Moderately weathered rock.	Watered & compacted in layers as determined by trial embankment
3D	267,437	Rockfill in D/S Stage	Free draining sound rockfill with max size of 1.6 m	Watered & compacted in 1.6 m layers with 4 passes of vibratory roller.
3E		Facing rock on d/s batter	Select fresh large rock dozed to the d/s batter face and placed by excavator	Nominal compaction from excavator
4	123,194	Meshed rockfill in main cofferdam	Selected durable free draining rockfill.	Placement to suit detailed requirements of steel mesh protection.
5	50,000	U/S impervious zone	Fine silty material covering lower toe slab...	Placed in 0.5 m layers and compacted with construction equipment.

Recent large dams such as Campos Novos and Barra Grande have persisted with low modulus material in the central and downstream zones using 1.6 m thick layers of poorly graded basalt with no watering and moderate compaction. This design is now being questioned following the extensive cracking of the face slabs. Deformation of the upper zone on several dams has produced face slab deflections at the crest that are equal or greater than those at half height, the traditional location of maximum displacement.

The Zone 3C material proposed for Tillegra is not seen as a low modulus material as the modulus is more dependent on grading than rock strength. It will have substantially more fines and a lower permeability.

Zone 3D is the main rockfill zone for the downstream stage while Zone 4 is the steel reinforced zone on the downstream face.

7.4.4 Test Embankments

At this stage, quality rockfill appears to be available for all embankment zones. Test fills are not normally required prior to award of contract for these conditions. Test fills to fine tune embankment compaction requirements would be carried out after award of contract.

7.4.5 Foundation Excavation and Treatment

The foundation assessment is summarised at Section 4.6. The initial assumptions for foundation stripping are:

- The river bed consists of 1 to 2 m of gravel underlain by sound rock and this rock level extends under the river flats;
- Alluvial material on the river flats will be removed with excavation depths of up to 10 m;
- Gravel in the river bed will be generally left in place except under the toe slab and the area immediately downstream. Gravel will also be removed at the downstream toe of the main cofferdam to allow protective steel reinforcement to be anchored to rock;
- General stripping under rockfill zones will average 1 m to expose generally highly weathered, ranging to highly weathered/moderately weathered rock. The average depth of stripping may be shallower in the steeper sloping lower right abutment. Rock substance strength is expected to range from weak/medium strong in the tuffaceous sandstone, to generally medium strong in the meta-shale;
- Stripping on the abutments under the toe slab will require an additional 2 to 3 m of excavation to remove clay infill and tree roots;
- A “stepped” foundation surface is expected as the toe slab excavation crosses the various interbeds and additional trenching of 10 m^3/m length of toe slab is provided in the estimates.

General foundation excavation for rockfill zones requires dozer removal of soil like deposits to expose hard in-situ rock points. Ripping is not required and soil and surface material between hard rock points can be left in place.

Excavation for the upstream one third of the foundation involves backhoe removal of most soil and weathered rock between hard rock points. River gravels may be left in place but extensive sand deposits and any material considered to be subject to liquefaction is removed. In this area, overhangs and vertical faces higher than 2 m are trimmed to 1 vertical to 0.5 horizontal.

Cleanup of the embankment profile is only required under the toe slab and the adjacent transition zone as noted at Section 7.5.3.

As noted at Section 4.6.2, for the left abutment toe slab, moderately to slightly weathered, generally medium strong rock is considered to be a suitable foundation for the embankment toe slab. It is envisaged that stripping depths will range from 3 to 4 m to remove areas of clay fill and tree roots.

It is anticipated that the toe slab could be excavated by a Caterpillar D9 bulldozer (single tyne) or equivalent, with cleanup/shaping by a 30 tonne hydraulic excavator, with rock hammer. A slightly irregular foundation will result, due to the differing stripping depths between the tuffaceous sandstone and meta-shale. Hammering may be required in thicker tuffaceous sandstone beds to achieve a regularly shaped foundation.

Only shallow excavation into rock is anticipated in the river bed as the foundation rock is expected to be slightly weathered.

As noted in Section 4.6.4, the right abutment toe slab footprint is sub-parallel to the strike of bedding. It will be possible for the foundation in any section of the toe slab to partially comprise tuffaceous sandstone, with meta-shale/meta-siltstone upstream and downstream. Clay seams to 20mm thickness and roots were encountered in the base of the investigation trenches in highly weathered/moderately weathered tuffaceous sandstone and in moderately weathered/slightly weathered meta-siltstone/meta-shale. Excavation of 3 to 4 m depth is likely to be required to found the toe slab below the zone of clay and tree roots.

7.5 Face Slab, Toe Slab & Meshing Anchorage

7.5.1 Face Slab

Face slab thickness is generally based on height using the formula:

$$T = b + k*H$$

where T and H are the slab thickness and dam height in metres.

The k values have been gradually reduced with time as shown at Table 7-3, until the recent cracking problems in high dams. More conservative values are being promoted for dams over 150 m.

A face slab with a uniform minimum thickness of 300 mm is proposed for Tillegra. Practice has been to pay for a 400 mm thickness. Concrete strength is not critical and a 20 MPa 28 day compressive strength is adequate. Maximum size aggregate of 38 mm, air entrainment and use of flyash is standard practice.

Table 7-3 - Concrete Face Thickness Parameters

b	k	Notes
0.3	0.007	Used with dumped rockfill where concrete is cast over large rock
0.3	0.004	Typical for early designs where H < 100 m, such as Pindari Stage 1
0.3	0.000	Typical for heights less than 100 m in Australia - Dams & Civil, South Australian dams.
0.25	0.000	HEC practice for 6 dams under 90 m
0.3	0.003 to 0.0035	K = 0.003 widely used for large dams over 100 m, although some have
0.4	0.003	Currently proposed for the central slabs on the 205 m Bakun Dam

A single layer of centrally located reinforcement is provided. Typical reinforcement ratios for CFRD face slabs are 0.3% used horizontally and vertically over most of the slab with 0.4% used adjacent to the perimetric joint where some tension may be encountered. These are a little lower than the uniform 0.4% widely used on the earlier Australian dams. HEC used 0.5% but this was largely due to the thinner 250 mm slab that was HEC practice. Reinforcement percentages are used with the design thickness of 300 mm.

Reinforcement proposed for Tillegra is similar to that used at Babagon Dam:

- N20 @ 300 mm centres in the central compression slabs, equivalent to 0.34%;
- N24 @ 350 mm centres adjacent the perimetral joint equivalent to 0.43% (generally within 0.2H of the joint).

Anti spalling reinforcement has been widely used at the perimetral joint but this has been largely discontinued and is not proposed for Tillegra. Storage filling relieves the compression stresses on this joint. Double layer reinforcement is being considered at face slab joints for the very high dams subject to high compressive stresses but this is not relevant to Tillegra.

The face slab is slip formed in a continuous operation from toe slab (or face slab starter bay) to parapet wall level. Vertical contraction joints are typically 12 m to 18m apart to suit the slip form and this dimension is left to the Contractor. The representative dimension shown on the drawings is 12 m.

A 3.2 m high parapet wall minimises rockfill volumes while providing an adequate width of rockfill for slip form operations and minimises wave run-up. The design will provide for static water load to parapet crest level and for wave action. The parapet wall height is varied to incorporate 200 mm of camber in the central portion of the dam.

7.5.2 Water Seals

A typical perimetral joint detail used by Dams & Civil and by HEC (also shown in ANCOLD, 1991) is shown at drawing C-206 and comprises:

- A rear copper waterstop supported by a mortar joint pad;
- A central PVC centre bulb water stop;
- A compressible joint filler to prevent edge concentrations of compressive stress during construction and before first filling due to the rockfill settlement. After first filling the joint opens slightly as the rockfill moves downstream.

This has worked successfully on a large number of Australian projects. Common practice overseas has been to use a water face seal comprising a mastic secured by a PVC or Hypalon membrane. The water force on the membrane forces the mastic into any joint opening. This is used either as a third seal or as a replacement for the PVC centrebulb waterstop.

Dams & Civil used the mastic arrangement on the Pindari enlargement. The imported materials were expensive, the contractor had difficulty with installation and there have been long term durability issues where the mastic is above water level. The current proposal for Tillegra is to use the arrangement shown at drawing C-206, with further investigation of alternatives during detail design. This drawing also shows a typical contraction joint detail between face slabs proposed for both central joints under compression and abutment joints under tension.

7.5.3 Toe Slab

The toe slab foundation ideally consists of hard, non-erodible fresh rock which is groutable. A variety of treatments are then available to handle local imperfections to eliminate the potential of erosion or piping in the foundation. Toe slabs have been constructed successfully on faults, badly weathered seams, and on larger areas of soft rock with questionable erosion resistance.

The standard approach (ANCOLD, 1991) provides a hydraulic gradient across the toe slab that is appropriate to the foundation. ANCOLD (1991) provides the typical criteria shown in Table 7-4.

Table 7-4 - Hydraulic Gradient across Toe Slabs

Foundation Quality	Grade	Acceptable Hydraulic Gradient
Fresh	I	20
Slightly to moderately weathered	II – III	10
Moderately to highly weathered	III – IV	5
Highly weathered	IV	2

The acceptable hydraulic gradient can be achieved by a reinforced concrete toe slab plus a length of shotcrete or concrete on the final foundation surface downstream of the toe slab. A filter is then provided downstream of the protection, and sometimes on top of shotcrete in anticipation of cracking. Reinforced shotcrete is used in some cases.

The toe slab is expected to be founded on moderately to slightly weathered rock requiring a hydraulic gradient of 10. This requires a 7 m wide toe slab in the river bed areas, reducing to the minimum 3 m on the upper abutments. Where local foundation conditions require a longer slab, this can be provided as a shotcrete extension under the rockfill.

The toe slab has a minimum thickness of 300 mm (equal to face slab thickness) but an average thickness of 600 mm has been assumed for estimating purposes. A single layer of reinforcement is provided in the top face to prevent cracking but provide sufficient flexibility to adapt to minor foundation movement.

The slab is anchored to the foundation with 32 mm anchor bars at 3m longitudinal spacing to resist construction loads and pin the concrete to the foundation. Anchorage is based on precedent and foundation characteristics with no specific design requirements. Design for grout pressure is not a requirement at Tillegra as neither bedding planes nor joints are likely to parallel the toe slab base. A grouted bar length of 4 m is regarded as a general requirement with longer bars in areas of moderately to highly weathered rock.

The toe slab is subject to a variety of water loads, uplift, and rockfill loads. Conventional toe slabs of low height on sound rock have high frictional resistance to sliding and are stable. High toe slabs (particularly those exceeding 2 to 3 m) constructed across low points or overbreak and toe slabs over weak seams that daylight may be unstable. These require individual stability analyses.

The Tillegra slopes are relatively benign and sharp variations in foundation rock are not anticipated. Some excavation may be required to eliminate adverse combinations of bedding, joint planes and topography.

The toe slab foundation requires a thorough cleaning of the rock surface to obtain a good concrete-rock bond. The transition area downstream of the toe slab requires sufficient cleanup to facilitate inspection and determine the type and extent of foundation treatment. All faults, shear zones, infilled joints etc under the toe slab are cleaned out to a depth equal twice their width and backfilled with mortar or concrete. Closely-jointed or weak rock may require slush grouting or shotcrete to prevent deterioration during toe slab construction.

Once the required toe slab width has been achieved in areas of weathered rock, the foundation area immediately downstream may require a reverse filter to provide a non-erodible exit area to accommodate foundation leakage.

7.5.4 Protective Mesh Anchorage

The steel mesh securing the downstream face of the downstream stage needs to be anchored to sound rock. The vee shape created by the rockfill face and the foundation attracts a high volume of fast flowing water during overtopping and needs to be erosion resistant. A short concrete slab is anchored to the foundation and the mesh.

The foundation at the interface requires non-erodible rock to anchor the reinforcement.

7.6 Foundation Grouting

Orientation of bedding and joints suggests that a vertical grout curtain will intersect all defects in the rock mass.

Foundation grouting consists of both blanket and curtain grouting using the toe slab as the grout cap. Insufficient information is available at this time to develop a meaningful grouting layout and estimate. Current drilling indicates the foundation tightens at a depth of 20 m on the left abutment generally and 40 m in the saddle at the far left abutment. No information is available for the right abutment at this time and a grouting plan has yet to be developed.

7.7 Embankment Crest Freeboard

7.7.1 General

The *normal freeboard* for a dam is the vertical distance between crest of the dam and the normal reservoir full supply level. The *minimum freeboard* is the vertical distance between the crest of the dam and the design flood level (DFL). For CFRD designs such as Tillegra, the top of the parapet wall is taken as the embankment crest level. Camber is not included in the freeboard assessment.

Freeboard provides some assurance against overtopping due to:

- Wind setup and wave runup;
- Landslide and seismic effects;
- Settlement of the embankment and foundation;
- Malfunction of structures or operational failure, particularly for spillway gates;
- Other uncertainties in design construction and operation that could include:
  - Reliability of design flood estimates;
  - Changes in flood estimation techniques;
  - Changes in catchment conditions that increases inflow;

The relevance of these issues to Tillegra Dam is summarised at Table 7-5

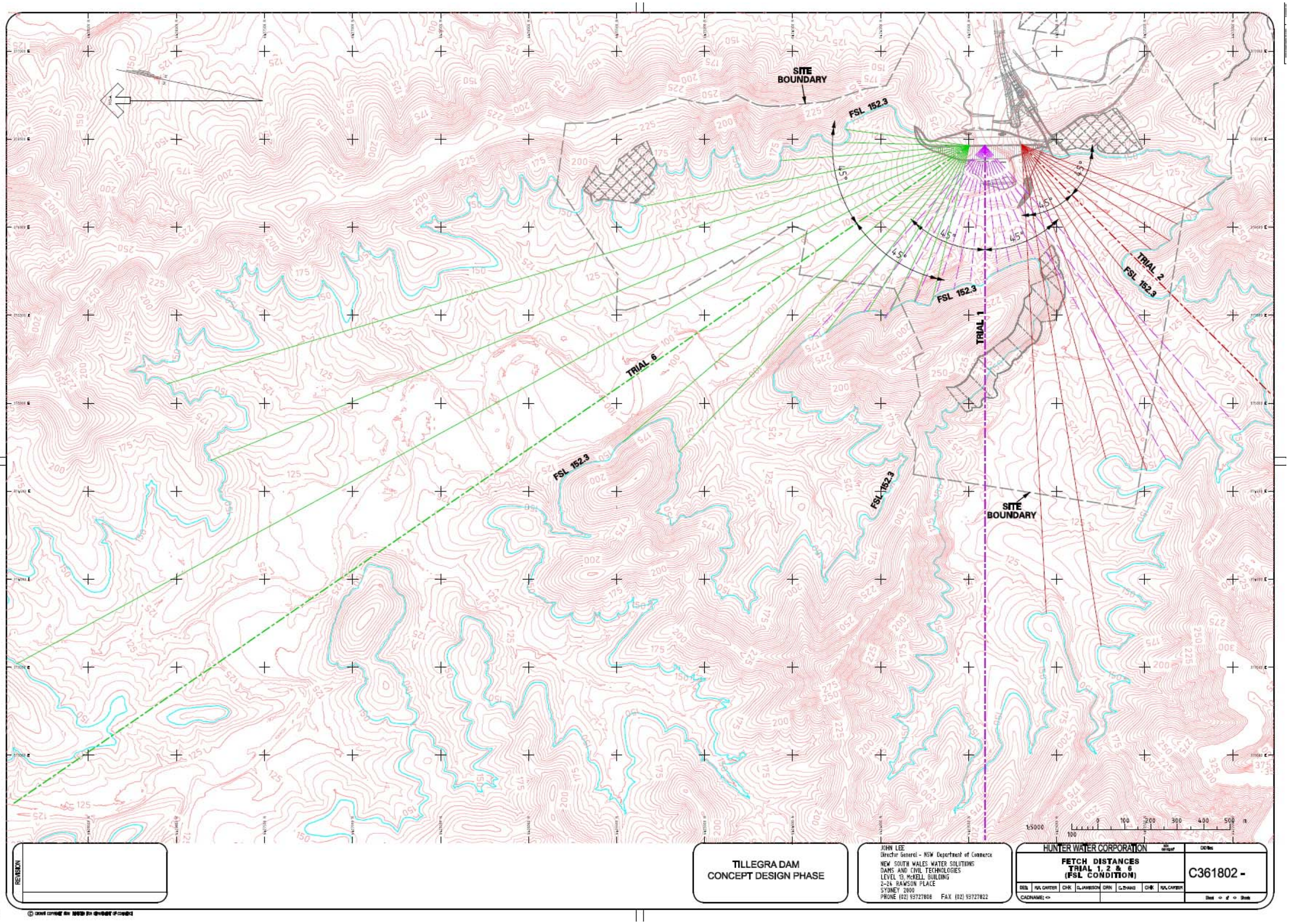
Table 7-5 - Freeboard Issues for Tillegra

Issue	Relevance to Tillegra CFRD
Wind setup	Relevant but has negligible effect for storage of this depth.
Wave runup	The major determinant of freeboard
Landslide and seismic effects	Relevant to Tillegra but a separate risk assessment study has been undertaken at Commerce (Nov 2008B).
Settlement	Settlement for a compacted rockfill dam is small and appropriate camber is provided. Dam foundations are hard rock, not subject to settlement.
Malfunction of structures	Not a factor in freeboard determination. The spillway is ungated & outlet works operation is not required during a major flood. The spillway control is located on a ridge and is not significantly affected by slope instability.
Type of dam and susceptibility to overflow	CFRD cannot sustain lengthy periods of significant overtopping, hence the conservative approach to the Design Flood and to wave runup.
Reliability of design flood estimates. Future changes in flood estimation techniques. Changes in catchment conditions.	The design flood assumes the PMF, estimated by experience personnel and subject to external peer review. Methodologies may change in the future and estimates may change due to environmental change or new research. Significant change in catchment conditions is unlikely, but possible. A freeboard of 1.3 m provides for a 25% increase in the PMF.

Three freeboard scenarios are examined for wind setup and wave runup in accordance with USBR, 1992:

- *Case 1: Normal water surface freeboard* considers the effects of the highest sustained wind velocity that could reasonably occur on a storage at Full Supply Level. A wind velocity in the range of 95 to 160 km/hr is suggested. A wind velocity of 160 km/hr has an AEP of around 10^{-5} for the Tillegra storage. The storage is at or close to FSL for long periods of time.
- *Case 2: Minimum freeboard at design flood level* considers the effects of average winds that would be expected to occur during large floods. USBR 1992 notes that for large storages and large catchments the wind may be independent of the storm event and suggest a wind with a 10% exceedance probability. This has been the approach adopted for Tillegra. It is noted that the combined probability of flood and wind is 10^{-8}.
- *Case 3: Intermediate freeboard cases* consider the effects of wind on intermediate storage levels. A combined probability of 10^{-4} is suggested by USBR 1992 but this was based on an understanding that the PMF would have a probability of this order. Australian practice assigns a probability of 10^{-7} to the PMF at Tillegra while the two limiting cases discussed above have combined probabilities of 10^{-5} (maximum) and 10^{-8}. A combined probability of 10^{-6} has therefore been adopted.

Figure 7-4 - Fetches at Tillegra



7.7.2 Fetch

Fetch is the distance over water a wave is assumed to travel from the point of origin to a point of impact. It determines the extent of exposure to wind a wave shall have in the reservoir. Wind is the principle factor for generating waves in inland reservoirs, when seismic or landslide situations are not encountered. Characteristics, such as wave height, are determined by the magnitude of the wind velocity, direction of the wind and amount of exposure to the wind (i.e. wind duration).

The fetches at Tillegra are limited by the shoreline directions as shown at Figure 7-4. The main fetch is almost parallel to the dam axis while the fetch perpendicular to the axis is broken by Elwari Mountain. Effective fetches are summarised at Table 7-6.

Table 7-6 - Fetch Summary for Tillegra

Fetch No.	Wind Direction	Effective Fetch at FSL km	Inclination to Dam Centreline
1	W	1.03	90^o
2	SW	1.21	45^o
6	NW	2.81	34^o

Regional wind speeds for Tillegra Dam were obtained for a range of probabilities from AS 1170. These 3 second gust speeds were converted to longer duration wind speeds using the data in Whitingham (1963). Minimum durations for Fetch 6 range from 22 to 25 minutes.

The significant wave height and wind setup data shown in Table 7-7 were obtained using the procedures and data in Saville et al (1962), including the following relationships:

$$g * H_s / U^2 = 0.0026 * (g * F_e / U^2)^{0.47} \quad (A)$$

$$g * T^2 / U = 0.46 * (g * F_e / U^2)^{0.28} \quad (B)$$

$$L = 1.56\ T^2 \quad (C)$$

where:

g = acceleration due to gravity in miles/hour/hour
Hs = significant wave height in feet
U = average wind speed for minimum duration in miles / hour
F_e = effective fetch in miles
T = wave period in seconds in (B) and (C)
L = wave length in metres

The wave runup on the concrete face was initially determined from the graph in Saville (1962) for a smooth slope and then increased by 17% as recommended in USBR (1992). Wave runup is also adjusted for the inclination of the fetch to the dam centreline.

Table 7-7 - Wave Height & Wind Setup for Fetch No 6

Probability 1 in	Wind Speed km/hr	Hs m	T sec	L m	R* With Adjust m	Wind Setup mm
5	93	1.01	3.4	18.3	1.55	1.8
10	99	1.09	3.5	19.5	1.67	2.1
20	106	1.17	3.6	20.6	1.78	2.4
50	113	1.24	3.7	21.7	1.90	2.7
100	117	1.30	3.8	22.5	1.98	2.9
200	124	1.37	3.9	23.6	2.10	3.2
500	130	1.45	3.9	24.7	2.22	3.6
1,000	135	1.50	4.0	25.4	2.30	3.8
2,000	139	1.56	4.1	26.1	2.38	4.1
10,000	148	1.66	4.2	27.6	2.54	4.6
100,000	159	1.78	4.3	29.9	2.74	5.3

* This is wave runup on concrete face slab
Wave runup for vertical wave wall is taken as $0.75*H_s$

For the 1 in 100,000 AEP and PMF floods, the wave hits the vertical parapet wall. In these cases, the wave runup was taken as 75% of the significant wave height to allow for the wave being above the stillwater level. In these cases, no adjustment was made for the inclination of the fetch to the dam centreline.

The wind setup was obtained from the conventional formula:

$$S = U^2 * F / (62000\ D)$$

Where:

S = wind setup in metres
F = fetch in km, generally taken as 2 * Fe in metres
D = average depth in metres

Wind setup is negligible for a deep storage such as Tillegra

The required parapet wall levels for the three design cases nominated in Section 7.7.1 are shown at Table 7-8. The minimum freeboard, Case 2, controls with a freeboard of 1.0 m. A freeboard of 0,.9 m is generally accepted as the minimum freeboard allowance.

The parapet level provided on the drawings of RL 160.2, gives a freeboard of 1.3 m which is generous. The parapet can handle a 25% increase in PMF with zero freeboard.

Table 7-8 - Parapet Levels for Freeboard

	Case 1 Normal F/B	Case 3 Intermediate F/B Examples			Case 2 Min F/B
Lake Level & Probability	RL 152.3 1	RL 154.7 1 in 100	RL 157.5 1 in 100,000	RL 157.5 1 in 100,000	RL 158.9 1 in 10,000,000
Wind Speed & Probability	159 1 in 100,000	149 1 in 10,000	100 1 in 10	142 1 in 100	100 1 in 10
Combined Probability	1 in 100,000	1 in 1,000,000	1 in 1,000,000	1 in 10,000,000	1 in 100,000,000
H_s	1.8	1.7	1.1	1.6	1.1
R	2.7	2.6	0.83*	1.20*	0.83*
S	0.005	0.005	0.002	0.003	0.002
Min Parapet Level	**RL 155.0**	**RL 157.2**	**RL 159.3**	**RL 159.6**	**RL 159.9**

*** Wave hits vertical parapet wall, Runup taken as 0.75*H_s**

7.8 Seismic Design Issues

Seismicity for the Tillegra site is outlined at Section 4.12 and dealt with in detail at ES&S (2008).

The CFRD is inherently safe against strong earthquake shaking and the designs studies are not normally undertaken for conventional CFRD constructions, even where sites are located in areas of strong seismic activity.

This capability was demonstrated by the survival of the 156 m Zipingpu CFRD in the May 2008 Sichuan earthquake (Xu Zeping, 2008). The dam crest settled about 730mm and there was extensive spalling of concrete at face slab joints but no danger of failure was apparent. Zipingpu Dam is located 17km from the epicentre of the M8.0 earthquake and was subjected to very severe shaking with an acceleration of 2.0g at the crest.

The expected settlement of Tillegra under extreme earthquake loadings would be less than 0.5 m. The dam has a freeboard above FSL of 7.9 m.

7.9 Instrumentation

Tillegra is a relatively small CFRD embankment constructed in a conventional manner with good quality rockfill and no need is seen for specialised instrumentation. The essential criteria for satisfactory operation are leakage and embankment settlement and the proposed instrumentation would consist of:

- Storage level recording system;
- Seepage measurement weirs for the main dam, for the left abutment ridge; and for the two rim saddles if grouting is required;
- Settlement points on the concrete face, parapet wall, the embankment crest and the downstream batter slope;
- Foundation piezometers, if considered necessary.

The instrumentation and the monitoring program will be reviewed using the USBR risk-based approach as presented by Smart (2006). This uses a decision tree model based on potential failure modes.

8 Spillway

8.1 Layout

The spillway location is shown at drawing C-301. It has a fan shaped contraction controlled by a 40 m long arc-crested ogee on a radius of 100 m. The crest contracts sharply to a 20 m wide chute, giving a contraction ratio of 50%. The chute terminates in a flip bucket located above tailwater level that discharges into a pre-excavated plunge pool.

The spillway geology is shown at drawings C-801 and C-806 in Appendix D. The approach channel, crest structure and chute are generally located on fresh rock but much of the excavation consists of moderately weathered rock. Concrete lining is provided from spillway crest to flip bucket.

Up to 20 m of alluvial and slopewash material is located in the lower chute and plunge pool area. This material is removed and the plunge pool is located in fresh rock. No concrete lining is provided downstream of the flip bucket apron slab.

As noted at Section 5.5, all spillway layouts have been designed for the critical PMF inflow (36 hour GTSMR storm) with a dry freeboard of 1.3 m to parapet crest level.

Alternative spillway locations were examined on both abutments (Commerce April 2008). Spillway crest widths were varied between 20 m and 60 m (with corresponding adjustments to embankment height) and both left and right bank locations were investigated. The variation in cost between left and right abutment locations and between spillway widths was small.

The left abutment spillways were a little more expensive, partly due to the need to relocate the diversion outlet works to the same abutment as the spillway. The right abutment options were preferred because of the flatter abutment slopes and the better alignment of the discharge channel with the river.

Brief consideration was given to other types of spillways, including labyrinth and tunnel spillways. No advantage was seen in these designs. An unlined spillway was considered but is not recommended. Considerations included the following:

- The spillway is expected to operate on a very frequent basis compared with typical Australian spillways. Storage operation data (Section 3.1) indicates operation for 24% to 49% of the time, depending on environmental flow requirements. This would make construction of remedial works difficult;
- Spillway discharges while frequent, will be relatively small. The outflow for the flood of record, assuming it occurs with storage at FSL, varies from 9 m^3/sec/m width at the crest to 18 m^3/sec/m width in the chute;
- Head-cutting through the storage rim is not considered to be a viable erosion mechanism. There would be significant erosion however and the extent of this erosion is difficult to quantify. It is expected that sufficient erosion would occur to cause some public concern;

- Spillway options are located reasonably close to the embankment and outlet works. Unlined spillways on other projects have been located further from areas of concern;
- Environmental regulator attitudes vary but there have been occasions when limited erosion has produced criticism;

A 30 m wide spillway on the right abutment located close to the dam abutment provided the minimum cost arrangement. Spillway locations further from the dam encountered more extensive deposits of slopewash and alluvial material.

As cost differences in the 20 to 40 m range were negligible, a 40 m ogee crest was adopted. This design is preferred by HWC as it provides somewhat less flood mitigation and is considered preferable from an environmental viewpoint.

8.2 Spillway Flood Routing

Flood data for the spillway design are summarised at Table 8-1. Flood routing data for the critical 36 hour GTSMR PMF are shown at Figure 8-1. The variation in maximum flood level with storm type and duration is shown at Figure 8-2.

Table 8-1 - Critical Flood Data for Spillway

Flood	Crit Storm Duration	Peak Inflow	Peak Outflow	Max Storage Level
	hrs	m^3/sec	m^3/sec	AHD
AEP Floods				
1 in 20	72	737.7	169.3	154.00
1 in 50	72	978.7	251.9	154.51
1 in 100	72	1096.2	286.1	154.70
1 in 1,000	36	1543.1	507.9	155.73
1 in 10,000	36	2051.5	748.6	156.65
1 in 100,000	36	2567.2	1,003.6	157.51
1 in 500,000	36	2890.4	1,166.0	158.01
1 in 10,000,000/PMF	36	3506.8	1,495.3	158.92
Historical Flood				
Apr-46		1,349	362	155.1

Figure 8-1 - 36 Hour GTSMR PMF

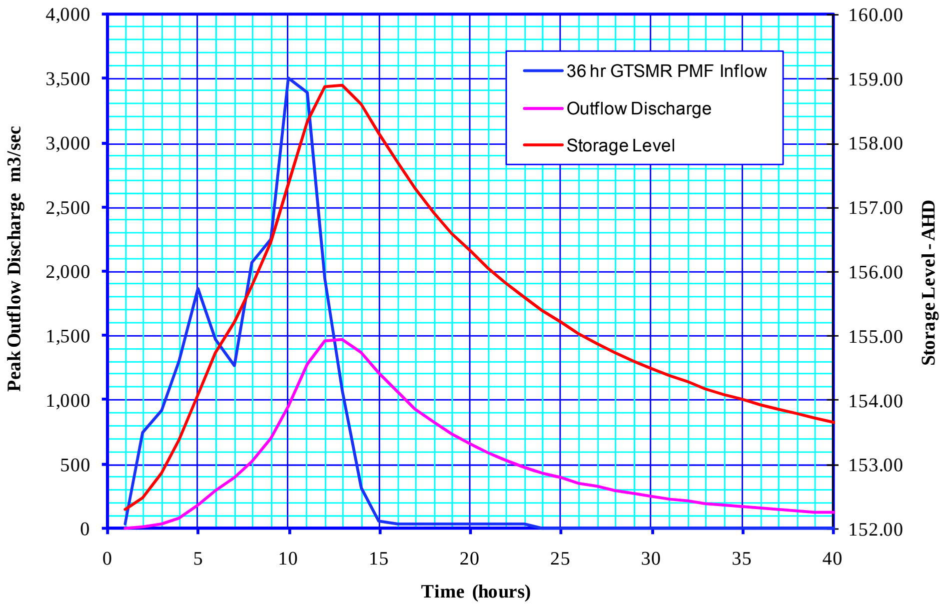
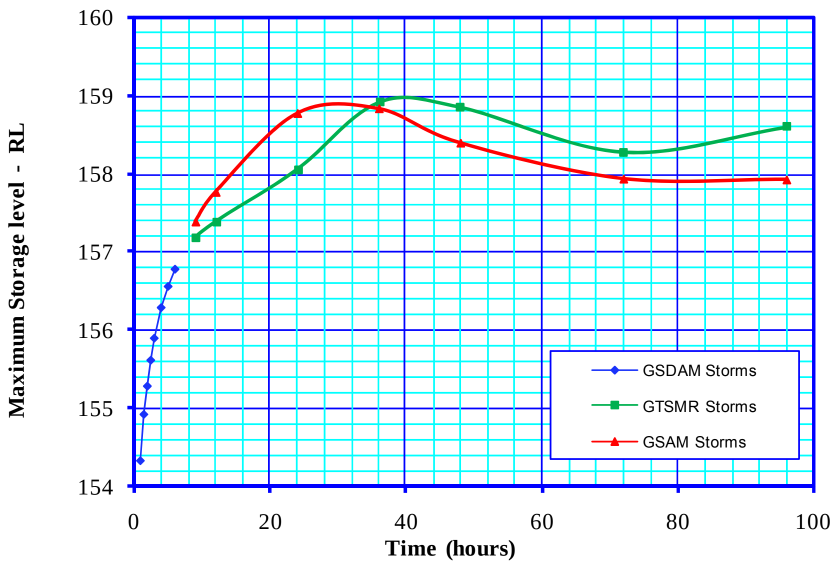


Figure 8-2 - Storm Types & Durations for PMF



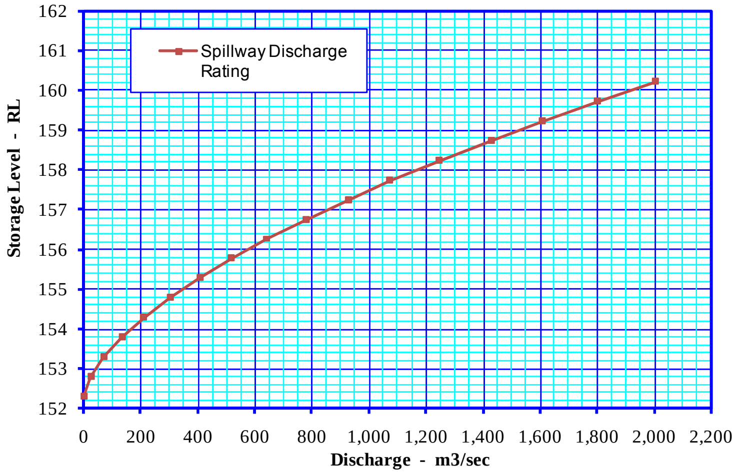
This outflow from the critical PMF is only slightly higher than the flood of record under natural conditions. At parapet level, the spillway can handle 1.35 times the PMF inflow.

The large storage produces considerable routing effect with outflow discharges ranging from 23% of the peak inflow discharge for the smaller floods to 43% for the larger floods.

8.3 Hydraulic Design

The ogee crest has been designed for a head of 5.0 m, approximately 75% of the head for the PMF, using standardised USBR data from USBR (1987). The coefficient of discharge obtained was 2.13 at the design head, increasing to 2.23 at DFL. The discharge rating is shown at Figure 8-3. Approach channel losses and approach flow irregularities have been ignored for design of the ogee crest.

Figure 8-3 - Spillway Rating



The crest design has been on an effective length of 40 m with no allowance for the trapezoidal section. The current arrangement, with a 1.0 m central bridge pier has a clear width at crest level (including curvature) of 41.8 m plus batter slopes of 1.0 horizontal to 1.5 vertical. This effective width is slightly greater than 40 m. The final hydraulic parameters will be determined by the model study.

8.4 Approach Channel

The approach channel foundation is excavated to RL 149.8 and the maximum depth of cut is around 20m. The lower 5m of the excavation is located in slightly weathered rock with a seismic velocity of 2,500 to 2,800 m/sec. Most of the excavation is located in the overlying mix of moderately and slightly weathered rock with the former predominating. Seismic velocities in the intermediate seismic zone range from 1,650 to 2,000 m/sec.

The flood of record produces a storage level of RL 155.1 giving a water depth of around 5 m. The channel velocities of 1.2 to 1.8 m/sec are contained within the lower band of slightly weathered rock. The PMF produces a water depth of around 9 m and velocities of 2.5 to 3.0 m/sec and the flow profile extends well into the weaker rock zones.

Batters of 1.5 vertical to 1 horizontal have been provided at this stage and the channel is unlined. Consideration is being given to flatter batters of 1 vertical to 1 horizontal as recommended at Section 4.8.2. It is anticipated that much of the excavation would be ripped with blasting in the fresh rock. Batters would be presplit over the full height to provide a clean face and minimise deterioration. Limited shotcrete (assumed 10% of the batter area) would be required to stabilise poor quality material. Weak seams would be excavated out to a depth of around 0.2m to 0.3m and infilled with shotcrete supported by dowel bars grouted into the excavated rock face.

8.5 Spillway Crest

The seismic results shown at drawing C-806 indicate a relatively low intermediate velocity area (1,900 m/sec) for the ogee crest and upstream chute foundation. It is likely that this indicates an area with more open joints rather than extensively weathered material. This is a relatively small ogee with a moderate head and foundation bearing capacity will be more than adequate.

If further investigations indicate more weathered rock in this area, consideration will be given to more conservative crest and chute anchorage.

Water testing in TDDH 10 indicates very high water losses (>100 lugeon) to RL 45 and very tight rock below this level. It is proposed to extend the toe slab grout line across the spillway crest despite the low head on the crest at FSL.

The ogee crest is anchored to the foundation with grouted anchor bars to provide stability against extreme flood loadings. No drainage is provided.

The batter excavations are lined commencing a short distance upstream of the ogee. The lining consists of 300 mm of reinforced structural shotcrete anchored to the batter slopes with reinforcing bars grouted 3 m into sound rock. Wall drainage is provided by an array of 75 mm holes drilled 3 m into the foundation. The upstream inclination and the detail at the shotcrete surface ensure the holes operate as eductor drains.

The height of the lining is based on the PMF outflow “hard” water profile, with no allowance for wave action.

Structural shotcrete for wall linings has been used on a number of recent spillways including Nepean Dam, Warragamba Dam auxiliary spillway and Wivenhoe auxiliary spillway and Shannon Creek Dam.

8.6 Spillway Chute

8.6.1 Chute Hydraulics

The fan shaped contraction with a contraction ratio of 50% is similar to the fan shaped spillways described in ICOLD (1992). The chute contraction angles are larger than those specified by USBR (1997) but the latter are considered more relevant to funnel shaped contractions.

The chute bed slope is varied to suit conditions:

- A short length of horizontal chute is provided immediately downstream of the ogee crest to enable a constant chute slope and horizontal transverse slabs downstream of the curved ogee.
- A length of relatively flat sloping chute follows to minimise velocities in the main contraction.
- This is followed by a steep chute on 1 vertical to 2.0 horizontal to locate the floor and flip bucket in slightly weathered rock.

Typical spillway parameters are summarised at Table 8-2 and Table 8-3. Chute velocities exceed 30 m/sec and the cavitation indices are low indicating that consideration should be given to chute aeration. An aerator in the lower chute will be considered during the model study phase.

The chute contraction is expected to produce standing waves with a pattern similar to that shown by the CFD model study for Blowering Dam shown at Figure 8-4.
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Table 8-2 - Spillway Parameters for Flood of Record – 362 m^3/sec

Location	Chainage	Unit Discharge m^3/sec/m	Water Depth m	Velocity m^3/sec	Froude No.	Cavitation Index
Approach Channel	960	6.37	4.8	1.26	0.19	18.3
U/S Chute	1013	10.25	1.00	10.07	3.4	2.13
U/S Change Slope	1042	16.27	1.24	11.7	3.59	1.60
D/S Change Slope	1070	18.08	1.00	17.5	6.28	0.70
Flip Bucket Entry	1148	18.08	0.64	27.5	12.26	0.28

Note: Discharge per unit width based on bed width although channel is trapezoidal.

Table 8-3 - Spillway Parameters for PMF Outflow – 1,495 m^3/sec

Location	Chainage	Unit Discharge m^3/sec/m	Depth m	Velocity m^3/sec	Froude No.	Cavitation Index
Approach Channel	960	25.3	9.0	2.5	0.28	60
U/S Chute	1013	42.4	3.24	12.3	2.36	1.71
Change of Slope	1042	62.3	4.13	13.5	2.35	1.51
D/S Channel	1070	74.8	3.46	19.4	3.88	0.68
Flip Bucket Entry	1148	74.8	2.2	31.7	7.82	0.24

Note: Discharge per unit width based on bed width although channel is trapezoidal.

Figure 8-4 - CFD Model results for Blowering Dam Spillway

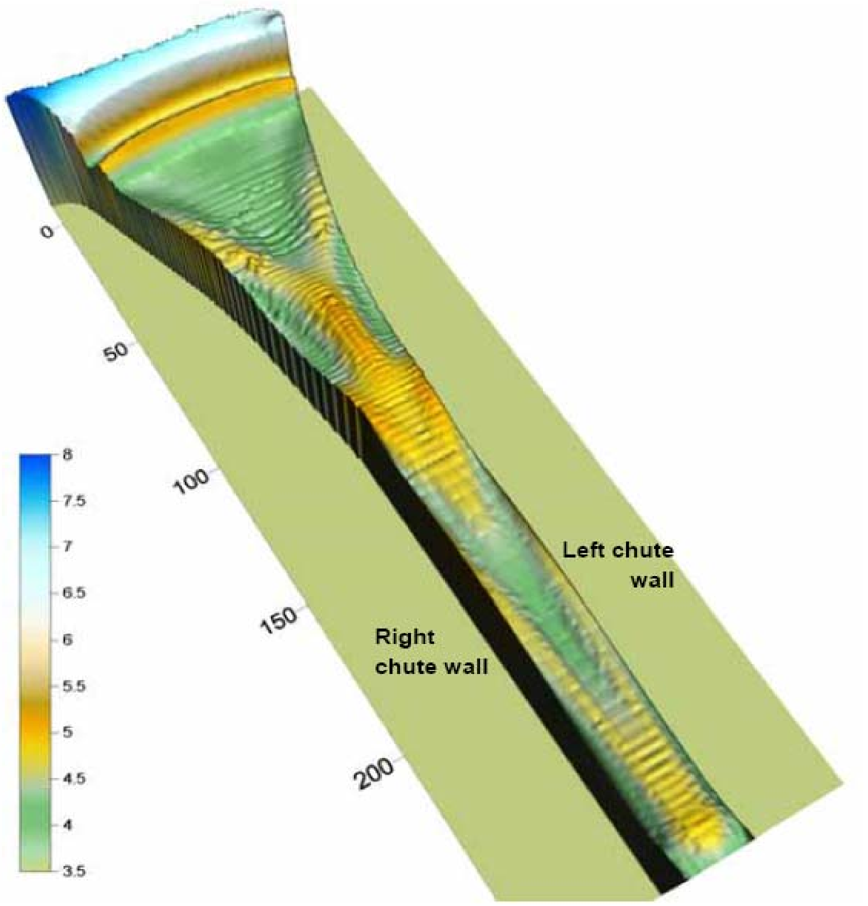


Figure 4.3: Case 1 (FL 386.281 AHD) – 3D plot showing CFD flow depth (exaggerated scale)

8.6.2 **Upper Chute to Ch 1140**

Immediately downstream of the ogee crest, the chute has a maximum batter height of 15 m. Intermediate seismic velocities at this depth are 1,900 m/sec. PMF water depths are 3.3 m and water velocities are relatively low at 12.3 m/sec.

Once the convergence is completed the upper chute bed slope steepens to follow the natural surface at an average excavated depth of 7m. PMF water depths reduce to 2.3 m and velocities increase to 30 m/sec. Seismic velocities vary from 3200 in the lower excavation to 2,000 m/sec in the upper levels. TDDH 11 has moderately weathered rock to below chute level.

Excavation batters of 1.5 vertical to 12 horizontal have been maintained at this stage and the full batter slope would be presplit. Again, consideration is being given to flatter batters of 1 vertical to 1 horizontal as recommended at Section 4.8.2. The chute walls will be tested in the spillway model to determine whether steeper slopes provide any significant improvement in hydraulic performance.

The chute floor has a minimum thickness of 300 mm and is anchored to the foundation with reinforcing bars grouted 3 m into sound rock. Chute drainage is provided by an array of 75 mm holes drilled 3 m into the foundation with eductor drains using a similar arrangement to that used for wall drains. It is expected that the chute will be slip formed in relatively long lengths with one longitudinal joint in the centre of the lower 20 m wide chute and additional joints in the upstream contraction.

No pipe drainage is provided at this stage although further consideration will be given to lateral drainage at slip form joints.

The walls consist of 300 mm of structural shotcrete as described above and again have been provided to cater for the “hard water” depths for PMF outflow. This makes no allowance for either wave action or aeration. The chute batters above the lining are mostly located in moderately weathered rock that has seismic velocities in excess of 1,900 m/sec. This material is considered to be capable of handling wave action and aerated flow from extreme floods with minor damage.

Some areas of more weathered and jointed material are expected and allowance has been made for shotcreting 10% of the wall area above the chute lining. The lining areas will be reviewed following spillway modelling and the final geotechnical investigations.

8.7 Flip Bucket & Plunge Pool

The natural surface flattens out in the lower chute area where a substantial quantity of alluvial and slopewash material is encountered. The flip bucket is located in moderately weathered rock, above maximum tailwater level. It has a radius of 20 m which is consistent with similar buckets at Toonumbar and Blowering Dams. A comparatively steep exit angle of 30 degrees has been provided to ensure the jet impacts the downstream pool at a steep angle to minimise wave action in the downstream channel.

Slopewash material in this area is around 20 m deep and is battered at 1 vertical to 2 horizontal with berms at 10 m intervals.

The plunge pool is pre-excavated for 8 to 12 m into sound rock to RL 78.0, 7 m below exit channel bed level. This material has a seismic velocity of 3,800 to 4,100 m/sec and TDDH 13 confirms that it is slightly weathered to fresh. It is overlain by 1 m of moderately weathered rock and 11 m of slopewash and alluvial material. Batters in sound rock are 2 vertical to 1 horizontal.

A range of jet profiles for three discharges are shown at Figure 8-5. The downstream profile for each discharge is based on the USACE theoretical profile while the upstream profile makes an allowance for air friction as given by Khatsuria (2005). The profiles are shown to extend to an estimated maximum scour depth in a granular material.

There are a large number of empirical formulas developed to compute scour depths and the results vary substantially. Five formulae, all based on case studies, have been used as summarised in Table 8-4. The USBR (USBR 1975) estimate is the original Veronese equation adapted for imperial units. This equation applies to scour from a vertical jet, as occurs with arch dams and gives very large scour depths. It is included for comparison purposes but not used for design. The equation was modified by Yildiz (Khatsuria, 2005) to allow for the angle of impact of a flip bucket jet, providing substantially smaller scour depths.

Experience gained with flip buckets on Turkish Dams (Yildez & Uzucek 1994) showed that scour formulae developed by Martins and by Chain gave the closest values observed on three large dams. The Martins formula also gave good results on a number of other spillways. The paper notes that the Martins equation “predicts the prototype scour depths quite well, although the coherence of the values used for obtaining the expression should be considered with some reserve”.

Mason 1989 provides a different formulation that includes the mean material size. Mason recommends 0.25 m if no other information is available but the formula is not highly sensitive to smaller material sizes. This method gives somewhat larger scour depths than the others.

Table 8-4 - Estimated Scour Depths

Event	PMF	1 in 100,000	Apr-46
Discharge	1495 m^3/sec	749 m^3/sec	362 m^3/sec
USBR (1975)	52.3	36.6	24.9
Yildez (2005)	26.3	18.8	13
Martins	29.5	19.8	12.9
Chain	27.7	19.7	13.7
Mason (1989)	41.0	28.0	18.6

Note: Scour depths given above are the depth of scour below tailwater level.

The Martins values have been adopted for initial design purposes. The excavated floor at Tillegra is in sound rock with a strike and dip that will tend to resist erosion.

The bed level at RL 78 should handle discharges of up to the flood of record with minimal scour. Larger discharges may cause some erosion in the plunge pool but this will be limited by the short duration of these discharges. The PMF profile (with air friction) impacts within the basin and scour during extreme flood events will not endanger the embankments or appurtenant works.

A more detailed assessment of erosion using the techniques developed by George Allendale will be developed once the final geotechnical assessment has been completed.

8.8 Discharge Channel

Downstream of the plunge pool, the fresh to slightly weathered rock level drops to the discharge channel bed level of RL 85. The alluvial and slopewash material is phased out and replaced with moderately weathered rock that forms the channel batter slopes.

The discharge channel has a bed width of 25 m and variable sided slopes, flattening to 1 vertical to 2 horizontal as the channel approaches the river junction. This section approximates the downstream river cross-section. A short dissipator basin is excavated into the channel bed, primarily to dissipate high velocity tunnel discharges during diversion. It will also serve to dissipate spillway channel wave action and improve flow conditions at the river junction.

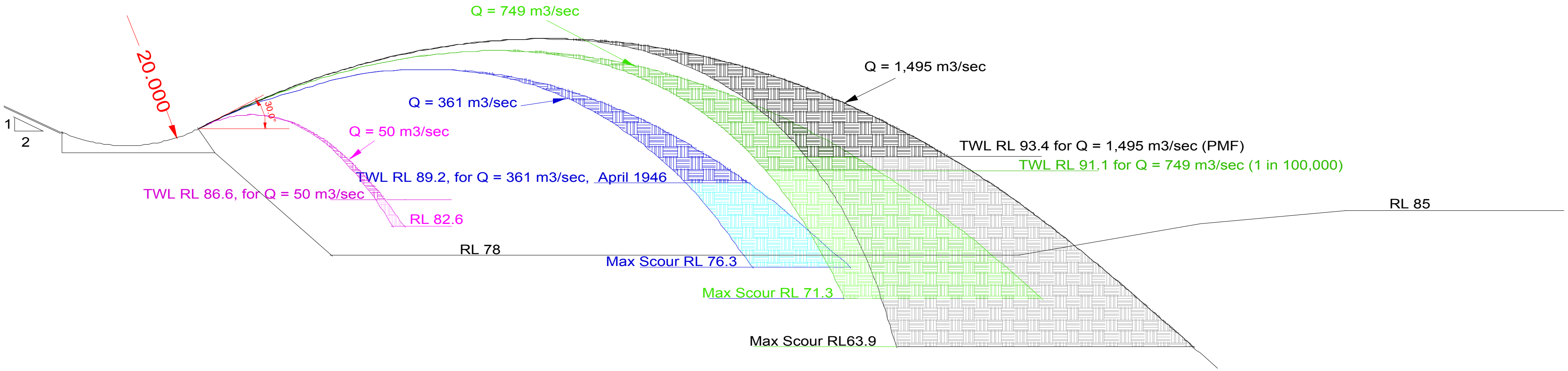
Rock quality improves as the channel approaches the river and TDDH 14 encounters slightly weathered rock at RL 90, providing 5m of fresh rock in the channel batters.

Batter slopes adopted are similar to those used further upstream with:

- 2 vertical to 1 horizontal in slightly weathered rock;
- 1.5 vertical to 1 horizontal in moderately weathered rock;
- 1 vertical to 2 horizontal in alluvial and slopewash material.

The existing river channel upstream of the junction will be used for waste disposal. Heavy riprap will be provided at the junction and for a short distance downstream.

Figure 8-5 - Flip Bucket Jet Profiles



8.9 Model Studies

8.9.1 Model Study Requirements

A model study is required to investigate the issues listed below. It is to be operated for three discharges unless otherwise noted below:

- The PMF outflow of 1,495 m^3/sec;
- The 1 in 10,000 AEP outflow of 749 m^3/sec
- The outflow from the April 1946 flood of record of 361 m^3/sec.

The main issues to be addressed by the model study include:

- The spillway rating curve and discharge coefficients over the full range of discharges up to embankment crest level (this exceeds PMF);
- The approach flow conditions, the impact on the discharge capacity and the effect if any on the embankment;
- The chute contraction, the cross wave action produced in the spillway chute and the impact on flip bucket performance;
- The impact of the sloping sidewalls on spillway performance and the improvement, if any, of using steeper batters, including vertical batters;
- The water profiles in the approach channel and chute;
- Air entrainment in the chute;
- Pressure distributions on the ogee crest, vertical curves and in the flip bucket;
- The cavitation index throughout the chute and the need for air entrainment slots;
- Flip bucket operation and trajectory, including effect of air friction, allowing for three bucket exit angles:
  - 30 degrees as shown on the drawings;
  - 20 degrees
  - 0 degrees
- The sweep out discharge for the three buckets;
- Operation of the pre-excavated plunge pool, including wave action, pressure distributions and fluctuations for the plunging jet;
- Downstream channel velocities, wave action and flow conditions in the downstream channel dissipator and at the river junction.

Electronic topographic data is available together with the proposed embankment construction. The spillway is not available in 3D at this time.

8.9.2 Physical Model versus CFD

Hydraulic models are required to assess the effects of approach flow conditions, chute contractions and dissipator action that are not well defined by conventional hydraulic analysis. Flip buckets in particular involve more uncertainty in design than the alternatives such as hydraulic jump dissipators or roller buckets. USACE recommend model studies of unit discharges over 250 ft2/sec/ft, (23 m^3/sec /m). The Tillegra chute has a unit discharge of 75 m^3/sec/m.

Historically, physical models have been constructed in hydraulic laboratories to study these behaviours. Physical models can be time consuming and expensive and can have problems with scaling effects. The development of efficient CFD codes and the ready availability of high performance computers have allowed the behaviour of hydraulic structures to be investigated numerically in reasonable time and at reasonable cost.

Research and development work in recent years has removed some of the difficulties and limitations of CFD models although further work is required to validate some issues. At the same time a large increase in physical modelling work in recent years has produced economies in model construction.

Worley Parsons have developed CFD models for a wide range of Australian spillways and the results have been validated against both published data and previous hydraulic model studies (Ho et al 2003 and Riddette et al 2006). The graphic shown at Figure 8-4 for Blowering Spillway shows results for a structure that is very similar to Tillegra, although this model did not extend to the flip bucket.

MHL have very extensive experience in hydraulic model construction. A physical model has some definite advantages if stakeholders have an interest in viewing the model and on some projects this has been a significant factor in gaining government and community acceptance for the project. Physical models allow fast experimentation with small changes but can be expensive if a major modification is required. CFD models have made significant improvements in visual presentation in recent years but physical model permit a “hands on” approach to modifications.

Little difference is seen in the value of physical and CFD models for spillway design purposes at Tillegra. While they are both acceptable, there is some preference for the physical model as it permits a “hands on” approach for designers.

The relatively low cost of CFD models has been a major factor in the rapid expansion of this work. However, the spillway arrangement at Tillegra suits a physical model in that the tailwater is relatively low and is contained within the river banks, even for the PMF outflow. The model required is a long narrow construction that is relatively cost efficient.

Prices for both a physical model and a CFD model have been obtained for Tillegra Spillway from Manly Hydraulic Laboratory (Department of Commerce) and Worley Parsons respectively. The physical model study price was 10% higher than the CFD model and was the preferred option. Construction is now in progress and the model should be operating by early March 2009.

8.9.3 Air Entrainment

The need for air entrainment in the lower spillway is borderline and will be assessed during the model studies. Neither model is particularly useful in this area and a decision will be taken on data from the model and conventional analyses. CFD models hold better promise in the longer term but considerable evaluation work is required.

Physical models can model the aerator operation but require a separate model to a scale of around 1:15 compared to 1:60 for the overall model.

Given that air entrainment is only borderline requirement at Tillegra, the expense of more detailed modelling is not warranted.

9 Outlet Works

9.1 General Arrangement

The outlet works arrangement outlined below has been developed to satisfy the discharge requirements detailed at Section 2 and the storage operation data summarised at Section 0.

The outlet works are located within the diversion tunnel and channel construction and the 5.8 m tunnel diameter is more than adequate for the outlet works requirements. Outlet works components include:

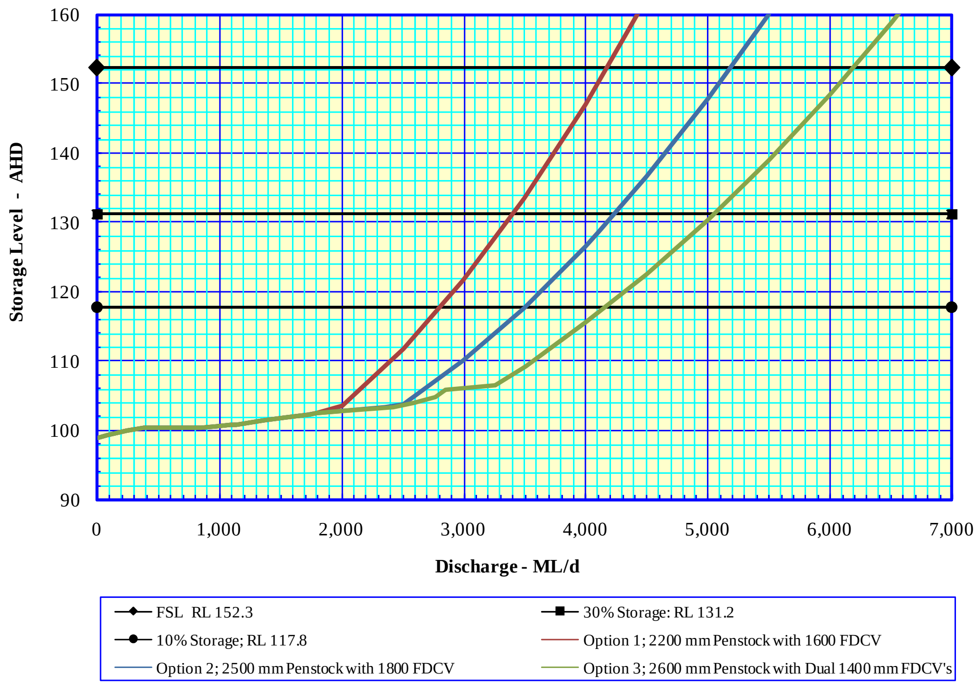
- A free standing wet intake tower at the upstream portal of the diversion tunnel equipped with selective withdrawal facilities. Trashracks and baulks have been provided for the full height of the tower.
- An access bridge from the tower to the adjacent abutment;
- A 2500 mm diameter steel liner within the tunnel from the grout curtain to the downstream portal. The annulus between liner and tunnel is backfilled with concrete;
- An 850 mm bypass pipe located within the diversion tunnel lining to provide flow maintenance releases during outlet construction and a low discharge outlet during normal operation. The bypass also provides a discharge facility that is independent of the main penstock system allowing the latter to be removed from service for maintenance.
- A valve chamber containing the pipework where it emerges from the concrete surround, the valves, interconnection and offtake pipes and the dissipator boxes for the fixed dispersion cone valves (FDCV) and submerged valves
- The mini hydro located within a separate room attached to the left hand side of the valve house. It is supplied by an 800 mm branch line from the main penstock. At this stage, no interconnection has been provided from the bypass line and there would be no supply to the mini hydro on those occasions when the 2500 mm penstock is dewatered for maintenance.

9.2 Discharge Capacities

The criteria for the main penstock discharge capacity include:

- The maximum required discharge capacity of 1,670 ML/d for bulk transfer to Grahamstown plus 135 ML/d for the link pipeline to the CTGM, giving a total discharge of 1,805 ML/d.
    - It is assumed that this discharge should be available from FSL to 30% storage (135,000 ML at RL 117.8).
- The ability to evacuate the storage rapidly, or control the rate of filling, under emergency conditions. This is discussed in some detail at Section 9.7;
- Scope to provide for future changed operating conditions.

Figure 9-1 - Main Penstock Discharges



Three basic outlet arrangements have been investigated with discharge capacities as shown at Figure 9-1:

- Option 1: a 2200 mm diameter penstock servicing a single 1600 mm FDCV;
- Option 2: a 2500 mm diameter penstock servicing a single 1800 mm FDCV;
- Option 3: a 2600 mm diameter penstock, bifurcating at the valve block to service two 1400 mm FDCV's.

All three provide more discharge capacity than is required for current operational requirements. Option 3 barely satisfies the USBR criteria for storage evacuation, Option 2 comes close while Option 1 operation lies well outside USBR criteria. The evacuation performance is compared at Figure 9-4. The performance of each option is compared with other similar sized dams in NSW in Table 9-2 and at Figure 9-5.

Detailed costing of the three options has not been undertaken at this time. Option 3 is the most expensive of the three due to the overall width and length of the structure to house the bifurcation and the additional valves. Option 1 would have a slightly lower cost than Option 2 due to the smaller valve sizes but the overall concrete dimensions are similar.

Option 2 has been adopted on the basis of:

- The small additional cost for substantial increase in discharge capacity over Option 1;
- A substantial decrease in evacuation time compared with Option 1;
- A capacity that is close to satisfying USBR criteria and is compatible with other major storages in the area.

The current outlet arrangement comprises:

- A 2500 mm main penstock leading to an 1800 mm FDCV;
- A 1400 mm branch line from the main penstock to a 1000 mm FDCV;
- An 850 mm bypass pipe leading to a 600 and a 250 mm submerged valve;
- A 1400 mm offtake from the branch line to the CTGM
- An 800 mm offtake from the main penstock to the mini hydro.

The operational ranges for all valves and the mini hydro are shown at Table 9-1. Discharge curves for the valves fully open are shown at Figure 9-2. The general layout is shown at Figure 9-3.

HWC specification for valve operation requires control over the full range of discharges from 5,200 ML/d down to 1 ML/d. The latter is an unusually low controlled flow for a high dam and poses some problems. The FDCV's are liable to cavitation damage at valve openings of less than 10% and are not suitable for small discharges. Two submerged valves have been included to cover low range discharges but there is some doubt that they can operate down to 1 ML/d.

The low flow problem could be handled by using larger intermittent discharges. This would have little impact on the river for low discharges but is not acceptable to the environmental regulator. Preliminary advice from valve manufacturers is that a 250 mm submerged valve can be ported such that it will operate down to 1 ML/d and also operate at higher discharges without cavitation. Design capabilities need to be confirmed.

If the 250 mm submerged valve cannot operate down to 1 ML/d, then gate control can be provided at the measuring weir in the spillway as outlined at Section 9.6.

Table 9-1 - Operational Ranges for Outlets

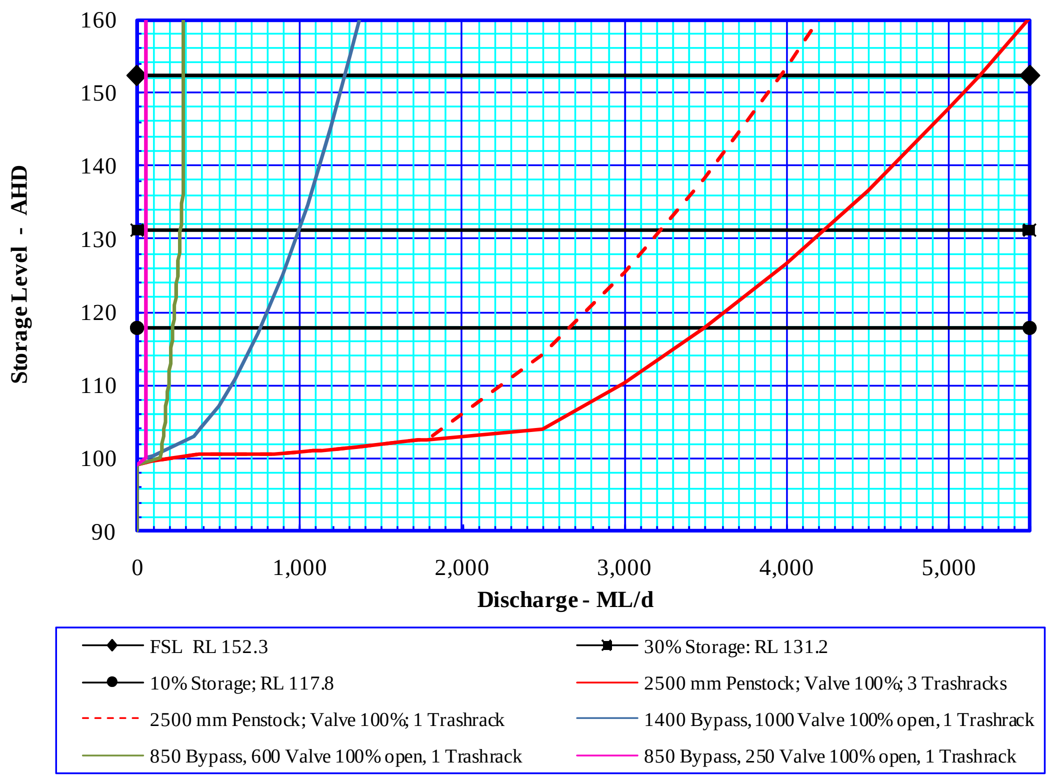
Outlet	Storage at FSL RL 152.3		Storage at 30% RL 131.37	
	Min Discharge	Max Discharge	Min Discharge	Max Discharge
2500 Main Penstock, 1800 FDCV	930	5,200	760	4,230
1400 Branch Line, 1000 FDCV	290	1,280	240	1,000
850 Bypass Line, 600 Submerged Valve	43	283	36	267
850 Bypass Line, 250 Submerged Valve	1	49	1	49
Mini Hydro Min likely	8	63	8	63

Discharges through the bypass line are restricted to 283 ML/d in order to limit velocities to 6 m/sec. Higher velocities are likely to erode the cement lining. Discharge through the 250 mm submerged valve is restricted to 49 ML/d to prevent cavitation damage.

Some limitations on this arrangement that may require further consideration include;

- The maximum discharge through the bypass system is 283 ML/d, sufficient to provide river freshes with the main outlet closed for maintenance but not large enough for bulk releases;
  - Bulk transfers occur less than once a year on average and maintenance operations could be planned for periods when bulk transfers are unlikely to be required
- The offtake to the CTGM is not available when the main penstock is closed for maintenance;
  - Discharge to the CTGM is understood to be a rare event.
- The mini hydro is connected to the main penstock only. It cannot be operated when the main penstock is closed for maintenance.

Figure 9-2 - Discharge Curves for Proposed Outlet Works



The valve operation for the specific operational requirements outlined at Section 2 are summarised at Section 9.7.

9.3 Intake Tower & Bridge

9.3.1 General Description

The conventional method of providing selective withdrawal involves a wet intake tower with a system of trashracks and baulks to control the level of the storage from which water is withdrawn. The baulks and trashracks are moved using a dedicated OHT electrical crane located on the deck of the intake tower.

The base of the tower can be closed off under balanced head using a "bathplug" gate. The gate has a diameter of 3.5 m and will weigh approximately 5.5 tonnes. A lifting frame (approximately 1.5 tonnes) allows the gate to be raised and lowered by the crane. A bulkhead for the 750 mm bypass pipe entrance is also located within the tower and handled by the crane.

A 210 m bridge is provided to allow intake tower components such as the bathplug gate, baulks and trashracks to be transported on a trolley. The bathplug gate at 7 tonnes is the heaviest item. The bridge will consist of:

- 4 * 42 m long steel box truss spans;
- 2 * 21 m span precast concrete spans using "Super T" bridge girders with a cast in situ concrete deck to cater for crowd loading. A gate would prevent crowd loading extending on to the steel girders.

The crane will be required to raise, lower, travel with and hold a maximum load of 8 tonnes. Control of the crane would be via a push button pendant operated from the deck of the intake structure. Conventional load limiting devices would be required, as would a full span walkway with handrails for maintenance with access from deck level.

Typical operation requirements are:

➢ Hoisting:	main speed	6 to 9.5 m/min
	Inching speed	1 to 1.5 m/min
➢ Longitudinal travel:	main speed	1 to 9.5m/min
	Inching speed	1 to 1.5 m/min
➢ Traverse travel:	main speed	1 m/min
➢ Hook path	vertical lift	67 m

The tower requires power to the crane and lightning protection.

Water quality probes will be attached to the tower at various depths to provide a profile at 2m intervals for:

- Water temperature
- Turbidity;
- Dissolved oxygen;

9.3.2 Operation for Water Quality

An assessment of water quality issues has been undertaken by Connell Wagner (CW, 2008) and these are outlined at Section 2.9. Consideration has been given to providing the best overall quality of water after considering likely variations in temperature, dissolved oxygen and cyanobacteria. CW (2008) recommends a multi-level offtake arrangement with water sourced from between 6 and 8 m below the water surface.

Long term storage operation obtained from HWC modelling for the 90GL demand is shown at Figure 3-1. The storage level is generally close to FSL and drops no more than 2.5 m below FSL. There are frequent spills, although the frequent small spills are likely to be channelled through the mini-hydro.

The conventional intake tower uses a combination of baulks and trashracks to control the level at which water enters the intake tower. A single 3 m high trashrack centred around 7.0 m below FSL would satisfy the above criteria for most of the time. Trashrack movement would only be required on the rare occasions the storage drops by more than a metre.

The trashracks and baulks would need to be regularly adjusted during the period of first filling. Figure 3-4 shows the storage will rapidly rise to around RL 122. The remaining 30 m to FSL could take 2 to 10 years to fill. Assuming trashracks are adjusted at 2 m increments in storage, the trashracks would need to be moved on 15 occasions.

9.4 Valve Block

The valve block layout is shown at Figure 9-3 and houses the following primary components:

- The 2,500 mm diameter penstock with;
  - A 2,500 mm diameter butterfly valve as the guard valve;
  - A 1,800 mm FDCV as the main outlet control valve, located in a reinforced concrete dissipator box;
- The 850 mm bypass pipe with:
  - An 850 mm diameter butterfly valve as guard valve;
  - A 600 mm submerged valve in a dissipator box as the main bypass control valve with a connection to a 250 mm submerged valve for small discharges;
- A 1400 mm branch pipe connecting the 2500 penstock to a 1,000 mm FDCV, located in a separate reinforced concrete dissipator box.
- A 1400 mm offtake pipe from the branch line that connects to the Chichester trunk Gravity Main (CTGM);
- An 800 mm branch pipe from the main penstock to the mini-hydro room;
- A filling line that commences upstream of the main intake tower bulkhead gate and travels through the tunnel lining to filling points downstream of the

main butterfly valve and the gate valves. The filling pipe is controlled by a 225 mm diameter gate valve;

- Ventilation system and Sump Pump.
- A control room;
- A separate room to house the mini hydro.

The dissipator boxes have been provided with steel liners. Recent projects have used unlined boxes with high strength silica fume concrete but there have been problems during construction. Unlined boxes will be investigated further during detail design. While there is some cost and maintenance advantage in unlined concrete, unsatisfactory concrete placement can severely limit operational capability during repair operations.

9.5 Mini-Hydro

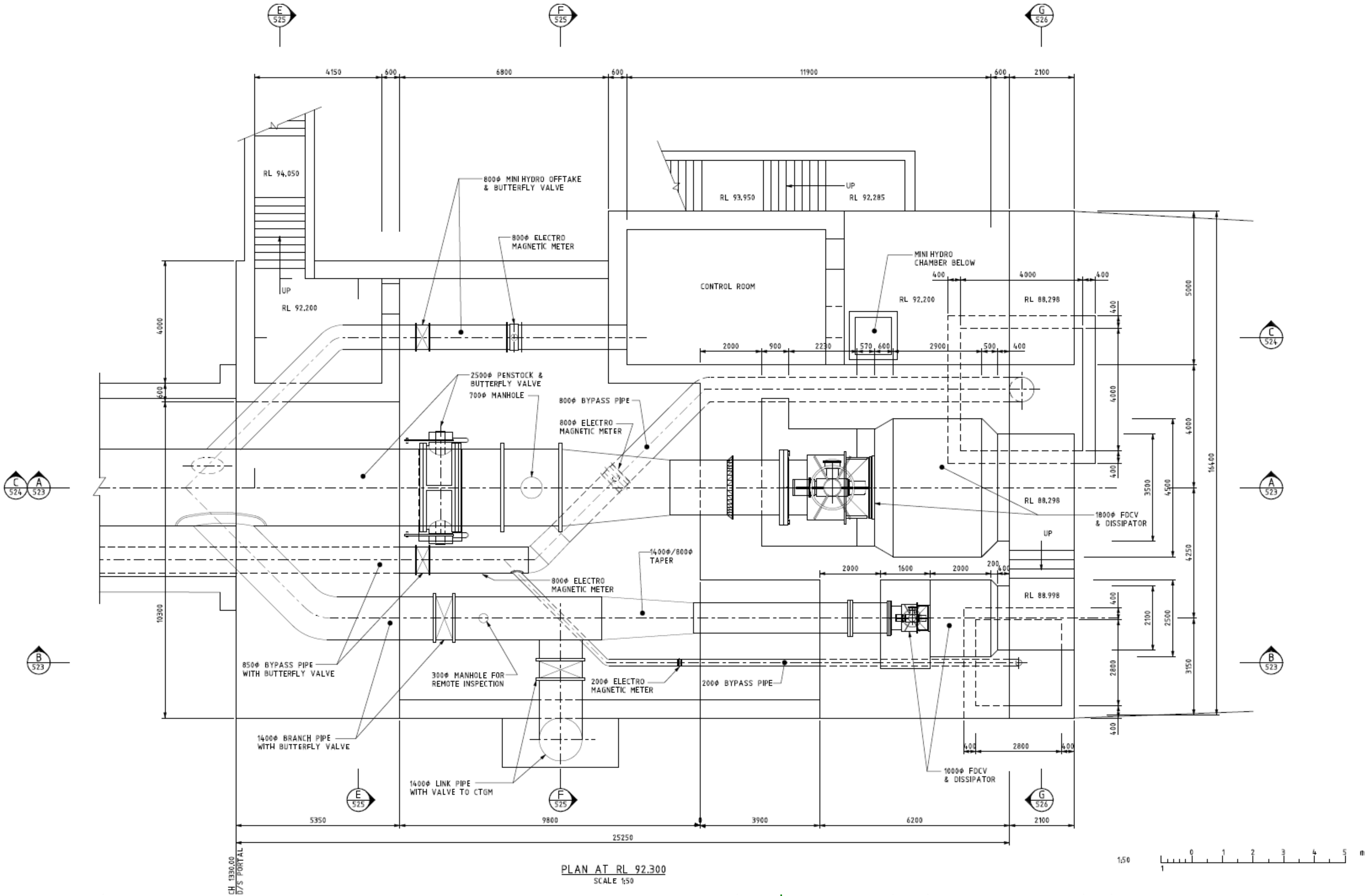
A preliminary Report on mini-hydro was provided by PB Australia (PB 2007) prior to finalisation of the environmental flow releases requirements outlined at Sections 2.3 and 2.4. At this time a decision was taken that the mini hydro would be the subject of a separate “build-own-operate” (BOO) contract. PB have provided a second report, PB (2008), which is an Information Memorandum (IM) providing technical information to parties interested in designing, installing and operating the mini hydro facility. This Report is based on the current environmental release requirements and is reproduced at Appendix C.

The capacity of the mini hydro will be determined by the BOO company and may not be known for some time. Given that the mini hydro will be owned and operated by others, it is preferred that it be located in a separate room beneath the control room in the valve house with a separate entrance. An 800 mm branch pipe to supply the mini hydro has been included but a larger pipe or a second tapping could be required if two turbines are to be installed.

The area required for the mini hydro could vary from 25 square metres for a small installation to 50 square metres for dual turbines. An area of 80 square metres has been provided at this stage which allows for a submerged outlet pit in case a reaction turbine is adopted.

The screening provided by trashracks on the intake tower is too coarse for a mini hydro installation. It is assumed that finer trash protection will be provided and maintained by the BOO company within the 800 mm offtake supplying the mini hydro.

Figure 9-3 - Valve Chamber Layout



9.6 Discharge Channel Measuring Weir

A measuring weir is located in the spillway discharge channel, to measure discharges up to 300 ML/d. This is the main discharge control for outlet valve operation during spillway discharges. A vertical lift gate may be required to control low discharges as outlined above.

There is some concern that the 200 mm submerged valve will not be able to operate accurately down to 1 ML/d. If this proves to be a problem, a hydraulically operated vertical lift gate will be installed in the spillway measuring weir. This gate will incorporate a vee shaped weir for accurate small flow control. The submerged valve or mini hydro will operate intermittently discharging water into the spillway dissipator pool. The vertical lift gate will be set to maintain the vee notch at the required distance below the dissipator pool level. This pool level will rise and fall with the intermittent valve discharges.

9.7 Water Measurement

9.7.1 Requirements

HWC requires accurate delivery of flow rates, particularly for the smaller transparent and translucent discharges. No final decision has been taken but the following has been suggested:

- +/- 5% for all controlled releases;
- +/- 10% for dam outflows including spills;
- +/- 20% for set points in the fresh and transfer release patterns.

9.7.2 River Maintenance & Simulated River Freshes

These releases are provided by the 800 and 300 mm lines from the bypass pipe, the 800 mm branch pipe to the mini hydro and small spillway discharges. Electromagnetic meters have been provided on all three lines in the valve house to allow flow measurement with an accuracy of around 0.5%. They need to be installed in a long length of straight pipe with a minimum of 3 pipe-diameters of straight pipe upstream and 2 diameters downstream. Longer straights are preferable and provide better accuracy.

Small spillway flows need to be measured accurately as these affect the release discharges. Spillway discharges at low flow flows cannot be accurately measured from storage levels. A measurement weir is located in the downstream discharge channel to measure total outflow from the mini hydro, outlet valves and spillway.

The measurement weir will provide the total outflow from the dam and valves and mini hydro will be adjusted to suit requirements.

The measurement weir will not function at large spillway discharges and these will be obtained from a model rating curve of the spillway. A downstream gauging station is also under consideration.

9.7.3 Bulk Transfers to Grahamstown

These require the main penstock to provide the higher discharge rate. Electromagnetic meters are not practical on the 2500 mm penstock and the release will be based on the downstream measuring weir. The manufacturer's rating for the valve opening can be used as a check.

9.7.4 Spillway Discharges Exceeding the Required Release

Small discharges up to 300 ML/d can be obtained accurately from the measuring weir using the vee shaped weirs. Discharges up to 100 m^3/sec can be obtained from weir overflow conditions before tailwater becomes a factor. This discharge equates to a storage level of RL 153.6, 1.3 m above the spillway crest. Larger discharges would be obtained from the spillway rating curve developed from model studies.

9.8 Emergency Dewatering

Australian authorities (including the DSC) tend to give consideration to the United States Bureau of Reclamation (USBR 1990) criteria and guidelines for evacuating storage reservoirs and sizing low level outlet works. It is not always accepted however, and a number of large dams here and in the US do not have this capacity. USBR in general, does not require additional capacity to be retrofitted.

This reference provides recommendations on reservoir evacuation rates based on the level of risk and hazard at the dam site. Risk, as defined by this USBR document, is the probability of occurrence of an adverse event. Hazard is the consequence of having an adverse event.

Three hazard categories are considered (low, significant and high) and Tillegra could be viewed as a borderline case between significant and high:

- **Significant** assumes "few" loss of life with no more than a small number of habitable structures are involved and economic loss expected to be appreciable with notable agriculture, industry or structures downstream;
- **High** assumes higher loss of life with urban development involving more than a small number of habitable structures and excessive economic loss due to extensive community, industry or agriculture downstream.

The definitions and categories used in these Sections are as defined by the USBR specifically for emergency dewatering. They should not be confused with the risk definitions and consequence categories used by ANCOLD and DSC.

The CFRD design for Tillegra is viewed as a low risk structure on the basis that the dam:

- Is not affected by active faults, liquefaction, rock solutioning or poor construction materials;
- Is not affected by hurricanes, has full PMF spillway capacity and flood inflows are small relative to the dam capacity;

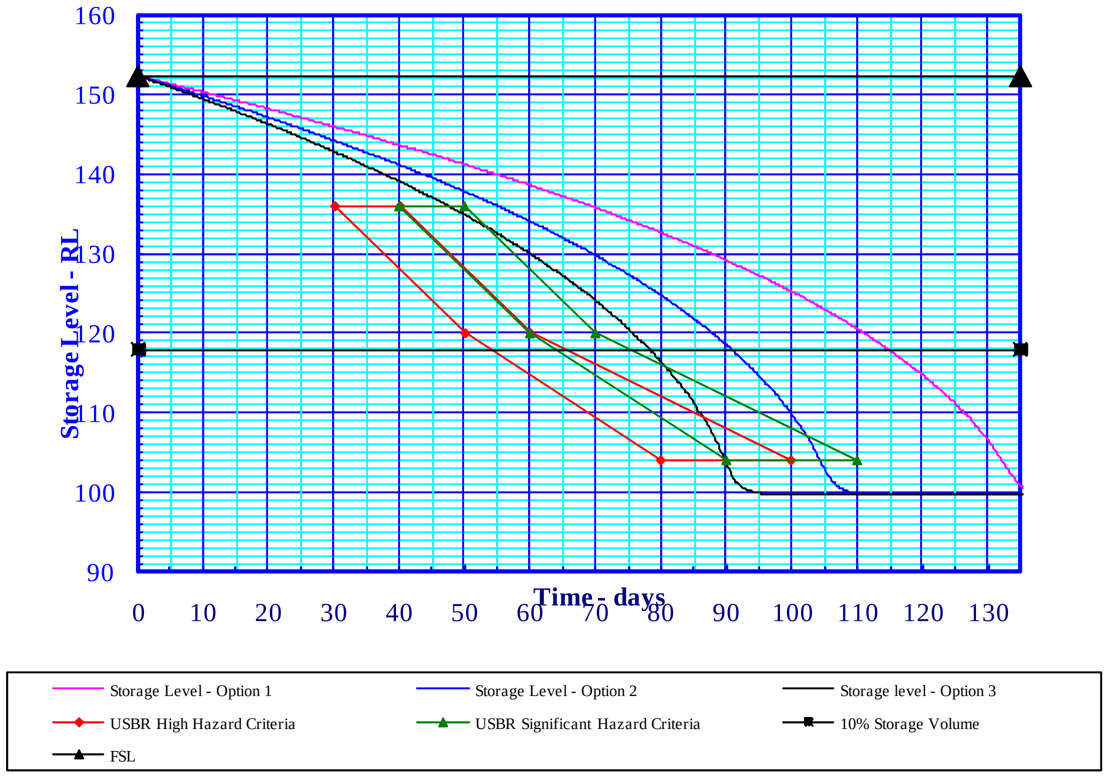
- The CFRD construction can handle leakage and is not sensitive to internal erosion or piping;
- The design involves a low level of complexity, and the dam is not large for this type of construction.

The time to empty the storage using the three outlet options described at Section 9.2 is shown at Figure 9-4 together with the USBR criteria. The inflow specified by the USBR is obtained by taking the mean monthly inflow for the 3 consecutive months of the year that produce the maximum total inflow.

The average monthly inflow volumes are shown at Figure 5-6 and the three critical months are January, February and March with mean daily discharges of 353, 467 and 533 ML/d respectively. These are small compared with the outlet works capacity and have little effect on the time to empty the storage.

Three options considered for the main discharge facility are described at Section 9.2. The larger Option 3 outlet arrangement barely satisfies the USBR criteria for a significant hazard with a low risk structure. Option 2 is outside but comes reasonably close to satisfying while Option 1 with the smallest valves lies well outside of the criteria. All three provide more capacity than is likely to be required for operational purposes.

Figure 9-4 - Time to Evacuate Storage



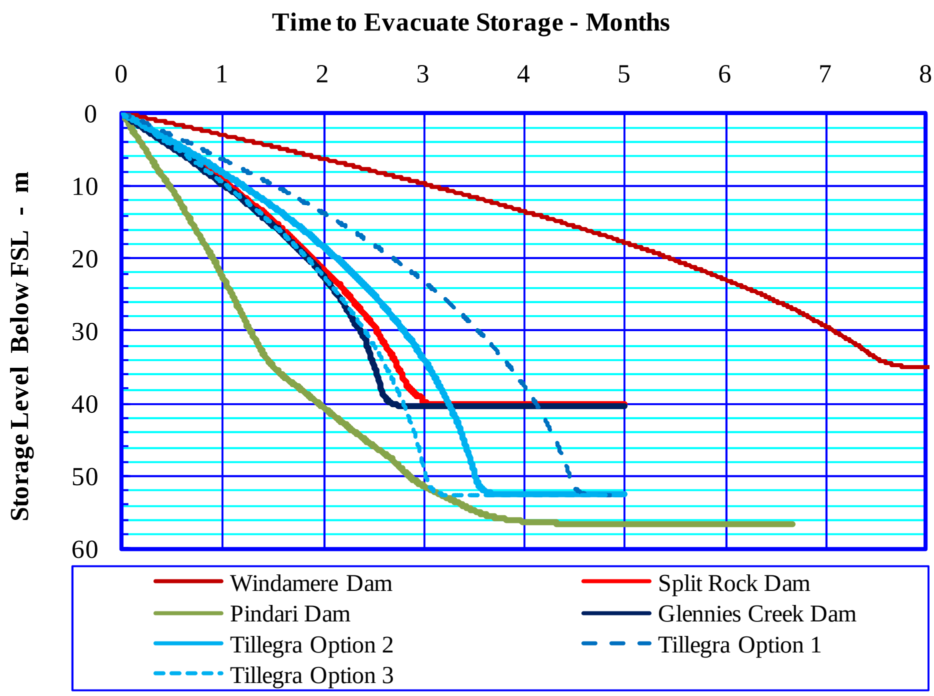
The discharge capacity and evacuation performance are compared with similar dams in north-east NSW as listed at Table 9-2 is shown at Figure 9-5.

Table 9-2 - Comparison of Outlet Works Capacities

Dam	Height of Dam m	Storage Volume ML	Main Penstock Discharge at FSL ML/d
Tillegra Option 1	75	450,000	4,180
Tillegra Option 2	75	450,000	5,200
Tillegra Option 3	75	450,000	6,200
Glennies Creek	67	283,000	4,450
Pindari	66	312,000	7,800
Split Rock	85	397,000	5,616
Windamere	70	368,000	2,074

The capacities for these dams were primarily determined by operational requirements rather than emergency evacuation. The high discharge capacity at Pindari is due to the two stage construction, with a second outlet provided for the Stage 2 enlargement works.

Figure 9-5 - Comparison of Storage Evacuation Times



There is little difference Tillegra Options 2 & 3 and the Split Rock and Glennies Creek Dams. Option 1 has a noticeably lower performance. Option 2 has been adopted.

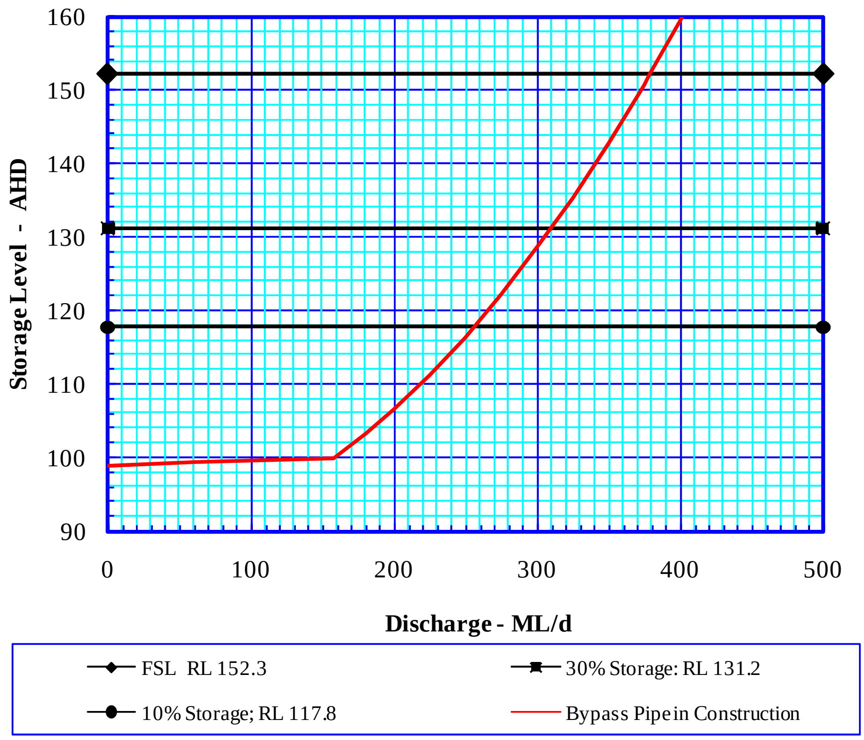
9.9 Operation During Construction

During embankment and spillway construction, river inflows are passed as uncontrolled flows through the diversion tunnels. Normal low inflows are equal to outflow. Large flood inflows are temporarily held in storage until the tunnel can discharge the inflow volume.

Once the embankment face slabs have been completed, the tunnel entrance is closed off and the bathplug gate in the intake tower is lowered to permit outlet works construction in the tunnel. River flows are maintained by water entering the bypass pipe from within the intake tower and discharged via a temporary elongation of the bypass pipe at the downstream tunnel portal. Control is provided by a temporary downstream butterfly valve.

The uncontrolled bypass discharge under these conditions is shown at Figure 9-6.

Figure 9-6 - Bypass Discharge Capacity during Outlet Works Construction



The storage level can rise rapidly once the tunnel is closed off and bypass discharges can far exceed the river release requirements.

10 Electrical Works

10.1 General

A detailed electrical works concept design is provided at Commerce (Dec 2008). A truncated version without Appendices is provided in this Section. Electrical drawings are provided in Appendix F.

The maximum demand for the installation has been calculated at about 85kVA with 30% allowance for contingency loads. It is considered at this stage that a supply authority power transformer of minimum rating 100kVA would be required for this installation. The maximum demand would need to be recalculated as part of the electrical detailed design. Maximum demand calculation details are included in Commerce (Dec 2008).

10.2 Overview of the Electrical Services

10.2.1 Electrical Site Locations/Switchboards

The electrical services for the electrical equipment items of the dam includes the provision of a Main Switchgear And Controlgear Assembly (MSCA) and a Outlet Tower Switchgear And Controlgear Assembly (OTSCA) as well as a Downstream Measuring Weir Electrical Enclosure (DMWEE). The OTSCA and the DMWEE will be connected to the MSCA via power sub-main cabling and by fibre-optic cabling for control. The MSCA will be located within a Control Room in the valve block.

10.2.2 Electrical Power Source

The electrical power will supply the MSCA via consumer's mains which are connected to a local 11 kV/433 V/250 V transformer supply authority substation. Power supply options from the MSCA to the OTSCA have been discussed and costed. Stand-by generator provisions will be incorporated into the MSCA. The Pre-Design Maximum Demand Calculation indicates that a 100kVA Transformer would suffice for the dam electrical services load.

10.2.3 Signals

The signals which are generated at the various locations of the dam site have been listed in Commerce (Dec 2008). It is proposed that all signals remote from the Control Room, such as from the DMWEE and the OTSCA, will be transmitted to the MSCA. Those signals which are neither sourced from nor directed to the DMWEE and the OTSCA are effectively generated within the MSCA.

10.2.4 Control

The control for all valves of the dam site and the downstream weir will be via a PLC of brand Schneider M340 (as per client's request), SCADA, located in the existing Operations Depot Building and local OMI (Operator machine Interface) on the MSCA.

As well, there will be a remote SCADA at the Hunter Water Corporation's Head Office. The SCADA systems will use Serck SCX6 software and, as requested by HWC, will be part of the telemetry works and compatible with the telemetry system. See also the "Control Hierarchy" Clause.

10.2.5 Flow Metering

The pipe flows for which higher accuracy is important and where the provision of pipe-diameters of straight upstream piping and pipe-diameters of straight downstream piping is not prohibitive will be implemented by magnetic flow meters. The measuring of the flows of higher value and for which a reduced accuracy is acceptable would be implemented by using the value of the extent to which the respective FDC Valve is Open and by measuring the pressure on the respective FDC Valve. An algorithm which calculates the flow values for different pressure values and different FDC valve positions be programmed into the PLC. The flow-metering is discussed further in this report.

10.3 MSCA (Main Switchgear and Controlgear Assembly)

10.3.1 Power Supply Arrangement

The power which would be generated by the on-site hydro-generator is expected to exceed the maximum demand value of the power consumed by the electrical equipment of the dam by a large amount. Further, the owner of the dam should be able to maintain an arms-length relationship with independent operators of a power station. The maximum demand value of the power consumed as well as the value of expected annual electrical energy consumption by the electrical equipment of the dam do not warrant a complicated administrative arrangement.

Additionally, the additional amount of electrical switching equipment which would be required for an installation to be powered by both the public electricity network as well as the hydro generated power would be prohibitively expensive. The minor gain in cheaper electricity for the dam load would not compensate for the extra capital outlay.

The initial equipment cost as well as the ongoing maintenance and operating costs should be considered here. Hence, it is proposed that the power supply for the electrical equipment should originate from an independent Point of Attachment to the local public electricity network.

The facility of providing power during protracted public electricity network failure should be implemented by a portable generator. The minimum rating could be 100 kVA, but this will be reviewed detailed design. This depends largely upon the equipment at the outlet tower.

10.3.2 MSCA Contents

This is a 415 Volt indoor SCA which contains starter and some control equipment associated with the valve block. It will be a segregated, compartmented Form 3b SCA, complying with Hunter Water Corporation STS 500 (General Requirements for

Electrical Installations), Clause 3.3.3 ("Free Standing Switchboards / Motor Control Centres). A Design Fault Level rating of 50 kA for 1 s, at 415 V is proposed.

Refer to drawings DC8117-01/02/03/04, the single line diagram for this MSCA and DC8117-05, the general arrangement for this MSCA.

The contents of this SCA would include:

- The power supply submain controlling circuit for the power supply to the Outlet Tower.
- The motor starters for the valve block auxiliary's drives for items such as sump pump, ventilation fan.
- A Distribution Board Compartment which houses all sub-circuit protection circuit breakers for all building services
- A generator changeover compartment. This would house two interlocked circuit breakers. These would be manually changed over when a portable generator is connected during extended power failure periods of the public electricity network.
- Interposing circuits for each valve actuator which allow for simple open/close control at the door of the respective compartment when *Local Board Control* is selected. These would also transfer control to the SCADA or off-site via Telemetry when *SCADA/Telemetry Control* is selected. This selection would apply for each valve or interlinked valve combination.
- OMI (Operator machine Interface) from which it is proposed to be able to set variables, as well as at the remote SCADA computer, located in the existing Operations Depot Building.
- A dedicated telemetry interface PLC (to suit the existing SERCK telemetry control system).
- A control PLC, which would be a Schneider " M3-40 " in order to suit HWC's current equipment selection practice.

10.3.3 MSCA Location

This would be located within the electrical control room.

10.3.4 Power Factor Correction

The loads are small and intermittent. The only single major load is the crane on the Outlet Tower. This crane is used to lift bulkheads into different positions so that the level of dam water from which water is discharged is selectable. This crane is used for short times and not frequently. Hence, the provision of power factor correction equipment is not recommended.

10.4 OTSCA (Outlet Tower SCA)

This is a 415 Volt outdoor SCA, located on the outlet tower. A Design Fault Level rating of 30 kA at 415 V is proposed. This switchboard is proposed to be built as a

Form 1 SCA, complying with Hunter Water Corporation STS 500 (General Requirements for Electrical Installations), Clause 3.3.4 ("Weatherproof Switchboards"). However, it will house all distribution circuit breakers in an integrated, proprietary distribution board. Refer to drawing DC8117-07, the general arrangement for this OTSCA.

It is proposed that this SCA will marshal the following signals sourced from the dam:

- Dam level
- Dam Turbidity – high level monitoring
- Dam Turbidity – low level monitoring
- Dam dissolved oxygen – high level monitoring
- Dam dissolved oxygen – low level monitoring
- Dam temperature – high level monitoring
- Dam temperature – low level monitoring

As well, the OTSCA will contain the crane power supply, single phase and three phase outlets and local lighting with remote control facility. To minimise cabling costs, restrictions will be introduced so that the three phase outlet cannot be simultaneously used with the crane. If both loads of crane and three phase outlet were to operate together, the cable to the OTSCA would need to be larger because of voltage drop considerations for this long run. The extra reticulation cost for a system with no diversity between the crane load and the three phase outlet load is considered to be too high to justify the inconvenience of not being able to use the three phase outlet and the crane simultaneously, as the need to simultaneously use both is considered to be infrequent.

The OTSCA will contain a remote Input/Output Unit(s) to interface with the control PLC in the Valve Block.

10.5 UPS (Uninterruptable Power Supply)

A UPS would be provided to power critical equipment within this installation including:

- PLC System
- Remote I/O at the OTSCA and the DMWEE
- Outlet Tower Instrumentation
- Weir Instrumentation
- Magnetic Flow meters near or in the valve block
- Telemetry radio system
- Pressure sensors for FDC valves

It is considered that a UPS of approximate rating 1kVA, 4 hours would be suitable for the loads envisaged. The UPS rating would be revised as part of the detailed design of the electrical services.

10.6 DMWEE (Downstream Measuring Weir Electrical Enclosure)

This is a 415 Volt outdoor SCA, located near the Downstream Measuring Weir. A Design Fault Level rating of 30 kA at 415 V is proposed. This switchboard is proposed to be built as a Form 1 SCA, complying with Hunter Water Corporation STS 500 (General Requirements for Electrical Installations), Clause 3.3.4 ("Weatherproof Switchboards"). However, it will house any distribution circuit breakers in an integrated, proprietary distribution board.

This assembly will contain the hydraulic power equipment for discharge gate operation.

It is proposed that this assembly will marshal the spill-way control signal and the weir discharge measurement.

10.7 Power Supply to Outlet Tower

10.7.1 Options for Power Supply

Four options for supplying power to equipment which is located on the tower deck of the outlet tower are being considered. Two of the options rely upon the 415 V/240 V MEN 50 Hz power supply of the 415 V Main Switchgear and Controlgear Assembly ("MSCA"); which would be located within the control room. The four options are:

Option 1 (Partial Power Supply Only):

A Solar Power Supply generated by solar panels which are located on the tower deck. Such a power supply would be entirely independent of the 415 V/240 V MEN 50 Hz power supply of the 415 V MSCA. This would be a main attraction of this power supply option. Another motivation for this power supply option would the aim of saving cable reticulation costs between the Outlet Tower deck and the 415 V MSCA. The current interest in environment – friendly forms of power generation was also taken into account. This installation would not be too prone to vandalism due to its remoteness from the general public.

Option 2

A submerged cable based 415 V/240 V MEN 50 Hz power supply. The submerged cable would be run underwater (on the floor of the dam) between the dam crest and the Outlet Tower Switchboard ("OTSCA"). The submerged cable would have to be rated for permanent underwater installation and thus would have an "EPR" sheath and "R – EP – 90" insulation. The respective 415 V/240 V MEN 50 Hz power supply would be supplied from the 415 V MSCA. Thus, a length of cable of PVC/PVC sheath/insulation construction, which is run enclosed by a PVC conduit underground in the land between the dam crest and the pumping station, (the main motivation for altering cable construction between land and underwater would be to save costs as a cable with "EPR" sheath and "R – EP – 90" insulation has a substantially higher per unit length cost than a cable with PVC sheath and PVC insulation) would also be part of this power supply option.

Option 3

A 415 V/240 V MEN 50 Hz power supply which would be carried by a cable which itself would be supported by a cable tray which is supported either below or on the side of a walk-on access bridge to the tower. The bridge supported cable would be run between the dam crest and the Outlet Tower Switchboard ("OTSCA"). The respective 415 V/240 V MEN 50 Hz power supply would also be supplied from the 415 V Dam Switchboard ("DSCA"). The entire cable would of PVC/PVC sheath/insulation construction as submergence in water would not apply to the entire cable length. Thus, a length of cable which is run enclosed by a PVC conduit underground in the land between the dam crest and the pumping station would also be part of this power supply option.

Option 4

Upon reflection on previous designs for electrical services for dams, a fourth option for supplying power to equipment which is located on the tower deck of the outlet tower was considered. This option also relies upon the 415 V/240 V MEN 50 Hz power supply of the 415 V Switchboard ("MSCA"). Here, the entire power cable would of PVC/PVC sheath/insulation construction and would run enclosed in PVC conduit which would be run parallel with the water outlet pipe which connects the Valve Block to the base of the Outlet Tower. The main attractions of this option are that the pits which would be required in the land-side of the dam as well as any bridge – supported cable tray systems for the other MSCA - sourced power supply reticulation options would be avoided. The conduit run would also be relatively direct between Valve Block and Outlet Tower. A large sweep bend would be part of the conduit run for the purpose of joining the horizontal in – ground run to the vertical conduit run within the concrete mass of the tower. In this method, there is a long cable pulling length. In previous installations this issue has been appropriately handled.

10.7.2 **Recommendation**

A Solar Power Supply generated by solar panels which are located on the tower deck only would only be adequate for providing the electricity supply to instrumentation equipment such as dam level sensors, dam water dissolved oxygen/ temperature sensors, dam water turbidity sensors as well as any possible radio telemetry receiver/transmitter which might be located on the tower deck. The typical electrical power yield of commercially available photo – voltaic cell panels is in the order of 80 W per square metre. Hence, larger electrical loads such as any lifting crane for bulk - head removal, electrical actuators, 20 A – 32 A 415/240 V three phase outlets, 240 V 10 A –15 A single phase outlets and reliable, high output flood lights could not be supplied from a local solar panel system.

It must be kept in mind that the spare area available for solar panel installation is limited if the solar panels are restricted to be located on the tower deck. Thus, larger electrical loads, as described, would have to be supplied by the 415 V/240 V MEN 50 Hz power supply of the 415 V MSCA. Hence, if the described larger electrical loads are part of the installation, then the cost of a solar power supply would be in addition to the cost of a 415 V/240 V reticulation from of the 415 V Dam Switchboard. The required load current supply rating of a 415 V/240 V reticulation from of the 415 V MSCA would only be reduced by a negligible amount by an auxiliary solar power supply. Cables and conduits are commercially only available in specific sizes.

Hence, the reticulation cost from the 415 V MSCA would not any lower even if the Solar Power supply supplies all the instrumentation and any radio telemetry loads.

In view of the statements of above, it is recommended that the most economical option (see Appendix E2 of Commerce (Dec 2008) for estimates and discussion) be accepted, viz:

- The outlet tower supply be supplied from land only, without any solar co – supply. The advantages of solar power at the outlet tower,
  - environmentally friendly supplementary green power usage
  - possible reduction of UPS load on the outlet tower as this could be provided by solar power,]

  are not considered to justify the provision of supplementary solar power.
- Use a conduit which follows the water outlet route.

10.8 Fibre Optic Reticulation

10.8.1 General

The number of signals which are generated by locations which remote from the Control Room's MSCA is of the order for which data-concentration is considered to be of merit. Refer to the list of signals in Commerce (Dec 2008).

Thus, these signals would be transmitted by a digital technology based data-link rather than by individual cores of a multi-core cable. This has the advantage that the transmission of additional signals is an easy process. The concept that a digital data link allows easy expansion of the numbers of signal which are transmitted is true for any electrical data link medium. However, fibre optic cabling has been chosen for the following reasons:

- Immunity against lightning strikes
- Immunity against electric noise

10.8.2 MSCA to OTSCA

The fibre-optic cabling to equipment which is located on the tower deck of the outlet tower should follow the same route as is recommended for the 415 V/240 V MEN 50 Hz power supply from the 415 V Main Switchgear and Controlgear Assembly ("MSCA"); which would be located within the control room to the OTSCA on the tower deck.

10.8.3 MSCA to DMWEE

It is proposed to link the weir pool weir control signals to the MSCA by fibre-optic cabling.

10.8.4 MSCA to existing Operations Building

It is proposed to link the MSCA to the SCADA computer which will be located in the existing Operations Depot Building.

10.9 Valving and Discharge Weir Control

10.9.1 Introduction

There will be four (4) modes of control for the valve operation. For one control mode, the control also involves position control of an actuated control weir gate in the discharge channel weir. All modes of controls will be automatic via PLC operation. The PLC will hold the control logic while the adjustment and display of the control logic will be via a SCADA screen. A SCADA screen or screens will be available for each of the modes of control, enabling the set-points etc. to be set via the SCADA system.

The modes of control are described below.

The normal (most common) mode of control is the *Transparent and Translucent Flows Mode*.

The *Simulated River Freshes Mode* is manually selected, but runs automatically. Once the desired, selected volume has been transferred, the control returns automatically to the "Transparent and Translucent Flows Mode".

The *Bulk Transfer to Grahamstown* mode of control is initiated manually via telemetry depending upon the level of the Grahamstown Storage Facility and the level of the Tillegra Dam. Adjustable set-points of the initiating levels within the Grahamstown Storage Facility and permissive levels of the Tillegra Dam will be entered via a SCADA screen. Automatic return to the "Transparent and Translucent Flows Mode" will occur when the required amount of volume has been transferred.

The *Discharge to the Chichester Trunk Gravity Main Mode* is initiated manually via the SCADA and then runs automatically. The finalisation of the *Discharge to the Chichester Trunk Gravity Main Mode* is automatic upon the finalisation of the transferral of the required transfer-volume.

As the *Transparent and Translucent Flows Mode* is the usual mode and is independent of the *Discharge to the Chichester Trunk Gravity Main Mode.* Thus, the *"Transparent and Translucent Flows Mode"* will continue to run in parallel with a required *"Discharge to the Chichester Trunk Gravity Main Mode"*.

The *Bulk Transfer to Grahamstown* mode of control overrides the *Transparent and Translucent Flows Mode* and the *Simulated River Freshes Mode*.

The *Discharge to the Chichester Trunk Gravity Main Mode* is independent of all other modes.

The *Simulated River Freshes Mode* overrides the *Transparent and Translucent Flows Mode*.

10.9.2 Description of the *Transparent and Translucent Flows Mode*

This mode is controlled by the inflow into the Tillegra Dam. There are four possible forms flow discharge:

- Spillway Flow,
- Release through the mini Hydro-Electric system's turbine,
- Release through the Low Flows Bypass Valve.

The following valves discharge into a weir-pool via the Outlet Works Channel:

- The 1800 mm FDC Valve,
- The 1000 mm FDC Valve,
- The 800 mm Mini-Hydro Offtake Butterfly Valve,
- The 600 mm Submerged Valve (a "Small Flows Bypass Valve"),
- The 250 mm Submerged Valve (a "Small Flows Bypass Valve").

These valves discharge into a weir pool. For the higher end of the range of inflows at which translucency is required, the weir-pool will not have control over the net outflow rate, the weir pool will overflow at the rate at which the flow in the outlet works fills weir pool. This expected to be for out-flows in the order of greater than 20 ML/d. The smaller valves will then control the net outflow rate.

However, for the lower end of the range of inflows at which full transparency and or translucency is required (less than 20 ML/d), the weir-pool will have control over the net outflow rate. For the lower end of the range of inflows, the weir pool is filled at the lowest controllable rate. Then, the weir of the weir pool will be used to produce an outflow which has a flow value which is as close as possible to the required flow value. Once the weir pool is empty, it will be refilled again. The reason for this weir pool operation is that transparency is required for even the lowest rates of inflow into the dam while even the smallest valve in the outlet works valve block can no longer control very low flows. This is a consequence of the valve selection. It is not considered practical or economical to provide additional, smaller valves other than the 600 and 250 mm submerged valves.

There is one inflow measurement only, from the upstream river gauging station.

For inflows of up to 7.4 ML/day, full transparency is required. This is achieved by opening the Small Flows Bypass Valves and/or the turbine valve to a degree which allows the difference between the sum of spillway flows (if any) and the inflow to be discharged, refer to item "9.2.2".

Preference is given to allowing the transparent flow to discharge through the Turbine Valve.

However, if the low through the Turbine Valve is too small for viable turbine operation, then the Low Flows Bypass Valves will achieve this transparency. The smallest Low Flows Bypass Valve will be used at the lower end of the inflow. For even lower inflow values, the weir-pool will come into effective operation.

For dam inflows over 7.4 ML/day, the operation will change from the transparent operation to the translucent operation. The component of inflow of 7.4 ML/day will then still be treated in the same manner as in the transparent mode.

However, only 60 % of the inflow component in excess of 7.4 ML/day is then discharged through the Turbine Valve and/or the Low Flows Bypass Valve and/or the Main Bypass Valve. Thus, 40 % of the inflow component in excess of 7.4 ML/day remains stored in the dam.

The above applies for inflows up to 100 ML/day. Thus, at an dam inflow of 100 ML/day –

(60 % of (100 ML/day - 7.4 ML/day)) + 7.4 ML/day ≈ 63 ML/day.

Thus, 63 ML/day is then discharged through the Turbine Valve and/or the Main Bypass Valve.

For dam inflows over 100 ML/day, the sum of all outflows is capped at 63 ML/day. The component of any dam inflow above 100 ML/day remains stored in the dam.

10.9.3 *Simulated River Freshes* Mode

A River Freshes flow profile (refer to the hydrograph of figure 2.1) will be programmed into the SCADA. This graph-derived flow profile will control the total discharge value of the sum of Main Discharge Valve flow + Turbine Valve flow + Seepage Weir flow + Spill Way flow into the river. This programmed flow profile will ensure that the total discharge flow reflects this hydrograph.

10.9.4 *Bulk Transfer to Grahamstown* Mode

This mode is initiated by Hunter Water Corporation control upon either one of two conditions.

Condition 1: This mode is initiated if Tillegra Dam is greater than 96% full, provided that Grahamstown has adequate storage capacity.

Condition 2: This mode is initiated if Grahamstown is less than 32 % full and Tillegra Dam is above 2 % full.

The control is similar to 10.9.3 (*Simulated River Freshes Mode*), except that the hydrograph of Figure 2-2 rather than the hydrograph of applies.

A SCADA alarm will be generated to alert operators that bulk transfer to Grahamstown is imminent.

10.9.5 Discharge to the Chichester Trunk Gravity Main Mode

A desired rate of discharge via a Chichester Trunk Gravity Main Off-Take Valve is manually set compared with a magnetic flow meter flow feedback at the selected rate.

10.10 Flow Metering

10.10.1 Key Issues

The metering of the flow through the hydro-generation turbine should as accurate as feasibly possible. The main amount of environmental discharge flow would usually pass through the hydro-generation turbine. The operator of the hydro-generation turbine as well as the dam owner should have an accurate measurement of the turbine water flow.

The metering of the flow through the bypass piping should also be as accurate as feasibly possible. The continuous environmental discharge would occur through this pipe whenever the hydro-generation turbine cannot accept water flow.
The metering of the flow through the larger FDC valve is not required to have the same degree of accuracy. The large pipe size of the piping to the larger FDC implies a problem with the required straight lengths of upstream and downstream piping. This problem of required straight lengths of upstream and downstream piping applies to any type of flow-meter and is discussed below.

10.10.2 Investigation

Magnetic flow meters can be installed with as little as three (3) pipe-diameters of straight upstream piping and as little as two (2) pipe-diameters of straight downstream piping. This is stated in the ABB data sheet at Appendix A3, Data 2 of Commerce (Dec 2008).

This opinion is also repeated in the ABB data sheet Appendix A3, Data 3 and Data 4 of Commerce (Dec 2008). It is to be noted however, that the data at Appendix A3, Data 2 states that the calibration reference installation conditions demand at least ten (10) pipe-diameters of straight upstream piping and at least five (5) pipe-diameters of straight downstream piping. This would indicate that at least three (3) pipe-diameters of straight upstream piping and at least two (2) pipe-diameters of straight downstream piping should be provided when magnetic flow meters are used. However, more upstream and downstream straight lengths than the minimum of three (3) pipe-diameters of straight upstream piping and two (2) pipe-diameters of straight downstream piping up to the reference value of ten (10) pipe-diameters of straight upstream piping and five (5) pipe-diameters of straight downstream piping would result in greater accuracy.

Other types of flow-meters were reviewed. Orifice Plates were considered as these once used to be marketed as a low-cost item which has the balancing negative aspect of high head-loss. The expectation was also that these could offer a lower accuracy but with the benefit of requiring lesser lengths of straight upstream and downstream piping. This would have been a possible solution for the larger FDC valve. This has not proven to be the case. The required lengths of straight upstream and downstream piping are in the order of twenty (20) pipe diameters for commercially offered orifice plates, albeit with measuring accuracies in the order of 2% full flow.

Another possible solution which was reviewed was ultrasonic clamp-on flow meters by GE-Infrastructure, the "Panametrics PT 878". However, even this equipment requires at least ten (10) pipe-diameters of straight upstream piping and at least five (5) pipe-diameters of straight downstream piping. This data is shown in Appendix A3 of Commerce (Dec 2008), Data 1, at its page 4. The additional stipulation is also made that the "lengths of straight upstream and downstream piping should be as large as possible". As GE-Infrastructure seems to be in the business of measuring flows in very large pipes, note that this data source mentions pipes up to seven (7) metres in diameter, it can be expected that this firm would have been faced with the problem of providing long, straight pipe lengths. Thus, this firm would be likely to have directed technical research effort in this direction. Even so, the commercially released ultrasonic equipment would not assist the cause of providing the feature of minimal required long, straight upstream and downstream pipe lengths.

10.10.3 **Recommendations**

Ideally, the metering of flows would be implemented by magnetic flow meters which are installed with at least three (3) pipe-diameters of straight upstream piping and at least two (2) pipe-diameters of straight downstream piping. However, it is preferable that more pipe-diameters of straight upstream piping and pipe-diameters of straight downstream piping be provided for any pipe flows for which higher accuracy is required and where the provision of pipe-diameters of straight upstream piping and pipe-diameters of straight downstream piping is not prohibitive. This is the case for the following piping:

- The 800 mm Mini-Hydro Offtake Butterfly Valve,
- The 600 mm Submerged Valve (a "Small Flows Bypass Valve"),
- The 250 mm Submerged Valve (a "Small Flows Bypass Valve").

The measuring of the flows of higher value and for which a reduced accuracy is acceptable, would be implemented by measuring the flow by other methods than by flowmeters. This is as per item "10.3.1" This is the case for the following piping:

- The 1800 mm FDC Valve,
- The 1000 mm FDC Valve.

The measuring of the flows of higher value and for which a reduced accuracy is acceptable (as per item " 10.2 "), would be implemented by using the value of the extent to which the respective FDC Valve is Open and by measuring the pressure on the respective FDC Valve. Coefficients of hydraulic head loss for different valve positions would have to be provided by the supplier of the FDC valves. These are commonly called " Cv values " An algorithm which calculates the flow values for different pressure values and different FDC valve positions would be developed. It would be specified that these algorithms be programmed into the PLC.

10.11 Electrical Building Provisions – Electrical Control Room

10.11.1 Introduction

This water storage installation has only the outlet valve block as a substantial building. However, it is thought that an open area within the valve block is not the optimum location for electrical switchgear and controlgear assemblies. These items require access floor space in addition to their own floor foot print. Additionally, a pipe leakage could cause damage to critical electrical equipment. The consequence of damage to a main switchboard for example, would be that electrical power to the outlet tower is lost. Hence, it is proposed that the MSCA be located within a separate Electrical Control Room within the valve block building. This room would be air-conditioned. Because the MSCA is located in a control room which is an integral part of the valve block structure, a separate distribution board for the control and protection of electric lighting and general purpose outlets within the valve block, located say, within the valve block room below the control room, is not considered warranted. Refer to drawing No. DC8117-06 for the layout of the electrical control room.

10.11.2 Location

The Electrical Control Room would be part of the Valve Block but be a separate, dedicated room. The electrical equipment such as control panel compartments and UPS should be out of the potentially wet piping space. Further, neither the lift-out access to the various valves should be compromised by the electrical equipment needs nor should the electrical equipment needs be compromised by the valve access needs.

10.11.3 Construction

Based on pre design assessment, the Electrical Control Room requires an approximate minimum internal floor space of 3.0 metres by 5.9 metres. The ceiling to internal floor height should be 3.0 metres. There should be two doors at opposite walls. This is an electrical safety practice. At least one of the doors should allow a floor to ceiling clearance of 2700 mm. This is required for MSCA installation. This floor to ceiling clearance of 2700 mm need not be provided for each entry event, a removable lift-out fill-in panel above the standard height door would be adequate. The removable lift-out fill-in panel above the standard height door would be removed only during switchboard installation and removal. It would be screw-fixed in place at all other times. There should be a cable basement space below the floor of the Electrical Control Room. This space should have a minimum height of 350 mm. The floor of the Electrical Control Room should be a removable access floor.

10.11.4 Standby Generator Connection Facility

This Electrical Control Room should have a connection box for the connection of a portable generator in the event of protracted public network failure. This connection box should be fixed to an external wall of the building.

10.11.5 Local SCADA Location

The Local SCADA Computer will be located in the existing Operations Depot Building, located approximately 500 metres from the valve block. This computer will be connected to the WAN (Wide Area Network).

10.12 12. Electrical Building Services

10.12.1 General

Electrical lighting of the internal rooms will be provided. There will also be external perimeter lighting for the valve block. This perimeter lighting will be provided by fluorescent luminaires attached to the building walls. However, road lighting is not proposed to be provided. Additionally, it is not proposed to provided power outlets and lighting for recreational facilities such as electrical barbecues. Further, it is not proposed to provide lighting for the access bridge to the Outlet Tower.

10.12.2 Internal Lighting Levels

Table 3.1 of Part 1 of AS 1680 has been reviewed. The lighting level which should be chosen for the Control Room is 400 Lux, as the tasks in this room would include reading manuals and other documentation at times. Additionally, the reading of information labels, internal as well as external to the various compartments of the MSCA during repair and fault finding is part of the activities within this room.

For the Valve Block area, a minimum lighting level of 200 to 240 Lux should be chosen. The tasks are considered to be Moderately Difficult. The size of the equipment would allow a lower minimum lighting level than 200 Lux, however, some of the equipment is items such as magnetic flow meters and electric valve actuators have small components. Some of these items also have small component legends which have to be able to be read during repair and fault-finding. It is also desirable to minimise shadows as these can cause loss of small removed equipment components.

10.12.3 Internal Luminaries

The lighting within the Control Room will be provided by commercial style ceiling mounted fluorescent luminaires. Some of these will be specified as having integral internal battery packs so that the loss of mains power allows safe exit movement and orientation.

The Valve Block will have mesh access platforms which create various floor levels for equipment access. There would be industrial style fluorescent luminaires fitted to the underside of the access platforms where these are of height above the floor level which provides safe walking head clearance . There would be industrial style fluorescent luminaires fitted to the ceilings above any access platforms also.

In addition, industrial style fluorescent luminaires would be fixed to the walls at a height of say 3000 mm above floor levels. These would be horizontal but angled towards the floor.

Some of the various industrial style fluorescent luminaires will also be specified as having integral internal battery packs so that the loss of mains power allows safe exit movement and orientation.

Where the ceiling is high above the floor, there will be high bay lighting. It is proposed that 400 W metal halide high bay luminaires will be provided. These high bay luminaires would also be specified as each having one instantaneous tungsten halogen lamp in addition to its metal halide main lamp. The tungsten halogen lamp would turn on instantaneously while the metal halide lamp is starting and be automatically turned off once the metal halide lamp is at its full light output state. This is a useful measure during network power dips. The control of this temporary stand-by lamp operation is part of the integral control gear of the high bay luminaires.

10.12.4 **Outlet Tower Luminaires**

There will be some lighting provided by outdoor style fluorescent luminaires which will be fixed to the columns of the portal frame.

10.12.5 **Building Power Outlets**

There will be commercial style 240 V single phase 10 A power outlets provided within the Control Room.

There will be industrial, IP 56 style 240 V single phase 10 A power outlets and 415 V/240 V three phase 32 A power with neutral outlets provided within the Valve Block.

There will be industrial, IP 56 style 240 V single phase 10 A power outlets and 415 V/240 V three phase 32 A power with neutral outlets provided on the external skin of the OTSCA.

There will be industrial, IP 56 style 240 V single phase 10 A power outlets and 415 V/240 V three phase 32 A power with neutral outlets provided on the external skin of the DMWEE

10.13 13 Signals

10.13.1 **General**

A list of signals which are proposed to be generated in this installation is included in Commerce (Dec 2008). All signals are available for telemetry transmission if desired by the client. However, those signals suggested for transmission are covered in Commerce (Nov 2008C)

It is proposed that all of the listed signals will displayed on the local SCADA screen.

In addition, all of the listed signals – except for the UPS generated signals - will be displayed on the MSCA.

10.13.2 **OMI (Operator Machine Interface)**

The OMI is located on the MSCA. Normally the OMI would only be used for monitoring purposes and not for control operation.

If the OMI is required to be used for control operation purposes, then a password entered on the OMI will enable this mode (OMI Active Mode) and the OMI will become active to control the electrical operation of the installation. In the OMI Active Mode, the local SCADA at the Operations Building as well as the remote SCADA at Hunter Water Chambers will become passive, only monitoring the installation.

10.13.3 **Local SCADA**

The local SCADA, located in the Operations Building would normally be used for monitoring purposes and not for control operation.

If the local SCADA is required to be used for control operation purposes, then a password entered on the Local SCADA will enable this mode (Local SCADA Active Mode) and the local SCADA will become active to control the electrical operation of the installation. In the Local SCADA Active Mode, the OMI as well as the remote SCADA at Hunter Water Chambers will become passive, only monitoring the installation.

10.13.4 **Remote SCADA**

The remote SCADA, located in the Hunter Water Chambers would normally be used for control operation.

In the event that password entry enables either the OMI to be in OMI Active Mode or the Local SCADA to be in Local SCADA Active Mode, the Remote SCADA will be rendered into passive, monitoring mode.

Note that the remote SCADA is not a duplication of the local SCADA, because the transmission of telemetry signals between the two SCADAs will be limited, to minimise signal transmission.

10.14 Dimensions of Switchboards

The dimensions of the MSCA and OTSCA for the purposes of this concept report are very approximate only.

If a detailed design approach is adopted for the electrical design of this dam, equipment selection and an equipment layout will be carried out as part of this detailed design and consequently MSCA and OTSCA dimensions will be refined, possibly resulting in revised dimensions for the valve block control room and the outlet tower platform.

Currently, our proposal covers only a design and construct (performance specification) approach for the electrical services for this dam. If this approach is retained, it is proposed that the switchboard manufacturer be required to submit the general arrangement drawings for the OTSCA and the MSCA in the preliminary stage of the contract and that the design of the valve block control room and the outlet tower platform be modified, if necessary, to suit the manufacturer's switchboard dimensions.

11 Telemetry

11.1 Scope of Works

A detailed Concept Report for telemetry is provided at Commerce (Nov 2008C) and is summarised in this Section. The telemetry concept includes:

- The telemetry Remote Terminal Unit (RTU)
- The telemetry Radio Repeater at Spotted Gum
- The telemetry Link
- Signal transmission path
- Telemetry monitoring and control signals
- Telemetry equipment proposed
- A cost estimate for the telemetry work.

The following drawings are provided at Appendix F.

WS080061-1 – Tillegra Dam – Communication Paths for Desktop Study
WS080061-2 – Tillegra Dam – Communication Paths – Option 1
WS080061-3 – Tillegra Dam – Communication Paths – Option 2
WS080061-4 – Tillegra Dam – Communication Paths – Option 3
WS080061-5 – Tillegra Dam – Dam Site Proposed Repeater Location
WS080061-6 – Tillegra Dam – Telemetry Repeater & Site Layout
WS080061-7 – Tillegra Dam – Typical Main SCADA Display
WS080061-8 – Tillegra Dam – Typical Main SCADA Display Notes

11.2 The Communication Path Profiles

There are two HWC Ultra High Frequency Radio Repeaters adjacent to Tillegra Dam namely:

- Lords Pillar
- Mount Richardson

The Lords Pillar UHF Radio Repeater is located approximately 9 km west of Tillegra Dam and has a licensed transmit frequency of 474.175 MHz and a licensed receive frequency of 479.375 MHz. The Mount Richardson UHF Radio Repeater is located approximately 10.5 km south of Tillegra Dam and has a licensed transmit frequency of 486.825 MHz and a licensed receive frequency of 481.625 MHz.

Desktop studies show an unsatisfactory communication path from Tillegra Dam to Lords Pillar (Path A). It was determined that the antenna height at Tillegra Dam had to be raised 85 m above the Outlet (Intake) Tower Structure to get a satisfactory radio communication path.

Studies also show an unsatisfactory communication path from Tillegra Dam to Mount Richardson (Path B). It was determined that the antenna height at Tillegra Dam had to be raised 115 m above the Outlet (Intake) Tower Structure to get a satisfactory radio communication path.

Desktop studies show a satisfactory communication path from Tillegra Dam to Lords Pillar using an additional Repeater at Spotted Gum Trig. Station (Path C).

The Communication Path Profiles for Paths A, B, C1 and C2 above have been included in Appendix B of Commerce (Nov 2008C).

In addition, site tests carried out with HWC on the 9-10-2008 indicated that:

- The Lords Pillar UHF Radio Repeater was a very difficult site to access and that the signal received at the test point at Tillegra Dam (RL 155) location was of marginal signal strength;
- The Mt. Richardson UHF Radio Repeater was relatively easy to access and that the signal received at the test point at Tillegra Dam (RL155) location was just satisfactory;
- An additional UHF Radio Repeater was required at Spotted Gum (RL 275);
- The additional UHF Radio Repeater at Spotted Gum would be 'cold standby' and mains powered to increase reliability and to reduce downtime for the site.
- A Communication Hut should be provided at the proposed Spotted Gum UHF Radio Repeater site, to increase security, keep spare parts and reduce possible damage to the sensitive telemetry equipment.
- The Telemetry RTU should be located in the Valve Block for increased security and because the Main Switchboard is located in the Valve Block.

The test results and photographs are included in Appendix C of Commerce (Nov 2008C). Based on the above, the Department recommends three options as follows:

- ***Option 1:*** The Telemetry Radio Antenna is located at the top of the Outlet (Intake) Tower or on the top of the Outlet (Intake) Tower Access Bridge, with 450 m coaxial cable connected from the antenna to the RTU in the Valve Block (see drawing WS080061-2).
- ***Option 2:*** The Telemetry Radio Antenna is located at the top of the Main Embankment with 200 m coaxial cable connected from the antenna to the RTU in the Valve Block (see drawing WS080061-3).
- ***Option 3:*** The Telemetry Radio Antenna is located at the top of the Valve Block with a 'Store and Forward' Facility located at the top of the Main Embankment and the normal Main UHF Radio Repeater System located at Spotted Gum (see drawing WS080061-4).

Details for the three options are provided at Commerce (Nov 2008C) together with and assessment of the individual advantages and disadvantages. These indicate that both Option 1 and Option 2, though feasible, will provide a major problem if the coaxial cable is critically damaged and needs replacement.

Option 1 is preferred to Option 2, because the latter requires a new cable path to the top of the Dam while Option 1 uses the same route as the existing cable.

Option 3 has two hops to the main repeater and requires the additional Store and Forward Facility, however the loss of the Store and Forward Facility is easier to resolve than the loss of the coaxial cable associated with Options 1 and 2. Based on the above the Department prefers Option 3, with short coaxial cable lengths and the additional ‘Store and forward’ facility.

It should be noted that, the Department of Commerce had discussions with radio communication people. The radio communication people showed a preference for Option 3, which allows for an additional ‘Store and Forward’ Repeater installed on the Main Embankment with very short coaxial cable lengths to the radios.

11.3 The Telemetry RTU

The proposed telemetry RTU is the Serck-Controls ‘PDS Compact Series 500’ with the ‘Motorola GM328’ Radio, which is compatible with the current existing RTUs in the existing Hunter Water Corporation Telemetry System.

It is proposed, that the additional RTU for Tillegra Dam will communicate with the existing HWC Telemetry Monitoring Facilities via one of the three options nominated in 3.2 above.

It is proposed for the telemetry RTU to be located in the Main Switchboard adjacent to the ‘Schneider Modicon M340’ Control PLC. The ‘PDS Compact Series 500’ RTU will be connected to the ‘Modicon M340 PLC’ via an RS 232 connection.

Details of the ‘PDS Compact Series 500’ RTU, the Motorola Radio and pricing have been included with the Supplementary Telemetry Information STI 6, 7 & 8 in Appendix D of Commerce (Nov 2008C).

11.4 The Telemetry Repeater

11.4.1 **Telemetry Equipment**

The proposed telemetry repeater is Serck-Controls ‘PDS Compact Series 500’ with Symax firmware which is compatible with the current existing Hunter Water Corporation Telemetry System.

It is proposed that the telemetry equipment to be located in a 2.7 m x 2.5 m brick building at the proposed Spotted Gum repeater site. The brick building will provide security from vandalism and possible damage to the sensitive telemetry equipment. The brick building also allows for future expansion to the radio equipment and will be a convenient place to store spare parts for the communication equipment.

It is proposed for the repeater equipment to be duplicated for increased reliability. It is proposed for the duplicated repeater equipment to be ‘cold standby’ to reduce the risk associated with lightning strikes. On the failure of the ‘duty’ repeater equipment, a telemetry alarm will be activated. The Operator will be able to visit the site and replace the failed equipment.

11.4.2 Mains Power

It is proposed, that access to the repeater site could be provided by a roadway or a track from the 'left abutment' of the Main Embankment, shown on drawing WS080061-5. It should be relatively easy to provide 240 V mains power from the left abutment of the Main Embankment to the proposed radio repeater site. An aerial electrical supply could be provided from the left abutment to the Site Boundary 300 m away. It is proposed to install underground mains along the Site Boundary to the Telemetry Repeater Building some 650 m away.

It is proposed, to provide a 240 V Mains Power Switchboard for light, power and a DC Battery Backup System in the Telemetry Radio Repeater Building.

11.4.3 Free Standing Tower

The Radio Survey used an omni-directional antenna mounted 6 m above ground level. Investigations indicated that a 13.7 free-standing two stage tilt over telescopic tower, is readily available from Nally Radio Towers Pty Ltd. It is proposed that the free-standing tower to be located on a 2000 x 2000 concrete slab adjacent to the Telemetry Repeater Building.

The hinged galvanised telescopic tower 'type A' is designed for the wind loading conditions in the Dungog area. Also the telescopic mast can be lowered via a hinge mechanism for connection of the antenna and cable. This will increase safety and make the telemetry installation easy. A typical plan layout of the communication hut and tower support slab is shown on drawing WS080061-6.

11.5 Telemetry Link

The proposed Spotted Gum Repeater will have a new ACMA licensed Point to Multipoint frequency. To complete the telemetry link with the existing Hunter Water Corporation Telemetry System, there needs to be another link radio on the same frequency as Spotted Gum setup at the Mount Richardson Repeater site, along with two PDS Compact 500 RTUs setup as a Net to Net link (or an audio bridge) to allow the routing to the Spotted Gum Repeater and the Dam.

11.6 Telemetry Signals

There are two locations for the telemetry signals:

- The Spotted Gum UHF Radio Repeater;
- The Electrical Control Room in the Valve Block.

Two types of telemetry signals are being considered in the concept:

- Monitoring Signals;
- Control Signals.

Details of the monitoring signals and control signals for each location are detailed at Commerce (Nov 2008C).

11.7 Telemetry Monitoring Facilities

The existing Head Office Telemetry Monitoring Facility is at the Hunter Water Corporation Head Office at 36 Honeysuckle Drive, Newcastle. The Tillegra Dam site telemetry will be configured for monitoring and control functions as detailed at Commerce (Nov 2008C).

The Tillegra Dam site telemetry will be configured for selection of analog signals either continuously or on demand to reduce the radio traffic from the Dam site. A typical Main SCADA display page is shown on drawing WS080061-7.

It is proposed that all the monitoring and control information is displayed on the Main SCADA display page.

It is proposed, that the telemetry contractor configure the Head Office telemetry computer to display all the monitoring and control functions as follows:

- Analog values nominated for radio traffic control in 3.7 are displayed in three columns marked ‘Continuous’ or ‘On request’. Double Clicking on either column selects the mode required with a “X” and the display is recorded in the third column as displayed below:
- The Repeater, Outlet (Inlet) Tower, Valve Block and Downstream Weir data will be displayed in separate areas on the Main SCADA display page. Refer to drawing WS080061-7.
- The valves will be displayed as per the valve arrangement, typical arrangement is shown on drawing WS080061-7. Clicking with the cursor on a ‘valve’ will display the valve monitoring status data shown in 3.7.2.4, as a ‘pop up menu’. Double clicking with the cursor on the ‘valve’ will display the control functions shown in 3.7.2.5, as a ‘pop up menu’. If the operator has entered the required password level, and if the Head Office Telemetry Monitoring Facility is the ‘master’, then, clicking with the cursor on a control function within the ‘pop up menu’ will activate the control function. The above mentioned ‘pop up menus’ will have an ‘X’ in the top right hand corner. Clicking with the cursor on the ‘X’ will ‘close’ the ‘pop up menu’.
- With reference to drawing WS080061-7, moving the cursor over a valve will generate an operator prompt as follows:
  - click on the valve for the Monitoring Pop-up
  - double click on the valve for the Control Pop-up

Table 11-1 - Telemetry Computer Display

Analogue Value	Continuous	On Request	Status
Dam Level	X		101.95 RL
Dissolved Oxygen from High Level Sensor		X	22.06 ppm at 10 am on the 12-01-2011
Temperature from High Level Sensor		X	22.1°C at 10.10 am on the 12-01-2011

The proposed Local Office Telemetry Monitoring Facility is to be located at the Dam Operations Building at Tillegra Dam. It is proposed, that the Local Office Telemetry Monitoring Facility has a desk top computer that is connected to the Modicon PLC in the Valve Block via a fibre optic cable connection. The database in the Local Office Telemetry Monitoring Facility will be continuously updated. The computer at the Local Office will operate on Serck Controls SCX Software.

The optical fibre cable connection from the Valve Block to the Operations Building will be part of the electrical work.

The Main SCADA display page on the Local Office Telemetry Monitoring Facility will be compatible with the Head Office Telemetry Monitoring Facility, accept that there is no provision for data ‘on request’.

The control function may be from either:

- Head Office
- Local office

It is proposed, that the Head Office is the ‘Master’ with the ability the transfer control to the Local Office via software. At all times, only one location will be the ‘master’ initiating the control functions.

12 Road Access

12.1 Road & Pedestrian Access

The existing road Salisbury Road bisects the dam site with the bridge over the Williams River located on the dam centreline and continues through the centre of the inundation area. A new public road is to be constructed to the east of the storage but is unlikely to be completed prior to the award of contract and in any case does not provide direct access to Dungog for landholders in the inundation area during construction.

It is proposed to construct a link road, referred to as the “Farley Link Road”, on the northern side of the river around the left abutment of the dam (drawing C-102). This road bridges the Williams River (single lane) adjacent the proposed Contractor’s Works Area and rejoins the existing Salisbury Road on the western side of the dam. This road would be part of the dam contract and would allow the dam work to proceed independently of the new Salisbury Road.

The bridge and downstream section of this road would be part of the permanent road access to the base of the dam, and to the left abutment embankment crest. The remainder would be ultimately inundated by the storage. Associated local roads consist of:

- *Outlet Access Road,* connecting the “Farley Road” with the spoil/ picnic area at the downstream toe of the dam. The route utilises an existing gravel farm access road running past the Lowery residence. From the picnic area, the road proceeds to the outlet works valve block, passes over the valve block and follows the outlet channel to its junction with the spillway plunge pool. A footbridge and pathway provide access to the measuring weir downstream of the plunge pool.
- *Embankment Access Road* connecting the “Farley Road” with the embankment crest road on the left abutment. A parking area adjacent this road provides a view of the construction area and the storage.

In addition to the above, road access is required from the existing Salisbury Road to the right abutment of the spillway. The final dimensions and treatment of this area will depend on the extent to which it is used as a quarry. If not used as a quarry, a level area will be provided that includes a car park and a public area with a view of the storage. If used as a quarry, extensive landscaping will be required.

Pedestrian access only is provided to the dam for the public. The crest can be accessed from a car park on the left abutment or from a footbridge over the spillway. A stairway located on the embankment will connect the embankment crest with the picnic area at the base of the dam.

12.2 Road Standard

The roadworks will conform with the Australian Road Design Guidelines for the design of rural roads, pavement design and bridge design, using a 40 km/hour speed limit and a T44 truck standard axle loading. Drainage strategies will be based on a 1 in 20 AEP storm.

The Contractor may provide a higher standard of road if appropriate for use during construction.

13 References

ANCOLD (1990)	Australian National Committee on Large Dams, 2000, *Guidelines on Concrete Face Rockfill Dams,* ANCOLD
ANCOLD (March 2000)	Australian National Committee on Large Dams, 2000, *Guidelines on Selection of Acceptable Flood Capacity for Dams,* ANCOLD March
ANCOLD (2003)	Australian National Committee on Large Dams, 2003, *Guidelines on Risk Assessment,* ANCOLD October
CBDB (2002)	Brazilian Committee on Large Dams, 2002, *"Large Brazilian Spillways"*
CW (2008)	Connell Wagner, 2008, *"Tillegra Dam, Planning & Environmental Assessment, Water Quality & Hydrology Draft"*, Connell Wagner, July.
Commerce (Feb 2008)	Department of Commerce, 2008, *Tillegra Dam Design - Options Study,* Dams & Civil Report DC07161, February
Commerce (Apr 2008)	Department of Commerce, 2008, *Tillegra Dam Design – Spillway Optimisation,* Dams & Civil Report DC07161, April
Commerce (Nov 2008A)	Department of Commerce, 2008, *Tillegra Dam Design – Design Flood Hydrology Report,* Report 08125, November
Commerce (Nov 2008B)	Department of Commerce, 2008, *Tillegra Dam Design – Storage Rim Stability and Seepage Potential,* Engineering Geotechnical Report, 08-GN31A-R2 draft V 3.0, Voles 1 to 5,November
Commerce (Nov 2008C)	Department of Commerce, 2008, *Tillegra Dam Design – Telemetry Investigation and Concept Development Report,* WS080061, November.
Commerce (Dec 2008)	Department of Commerce, 2008, *Tillegra Dam Design – Electrical Design Concept Report,* WS080068, December draft.
Cooke & Sherard (1987)	Cooke J.B, Sherard J.L, 1987; *Concrete Face Rockfill Dam: I. Assessment and II. and II. Design;* Journal of Geotechnical Engineering, Vol 113, No 10, October
DSC13 (March 2002)	NSW Dams Safety Committee, 2002, Consequence Categories for Dams, DSC13, March
DSC11 (June 2002)	NSW Dams Safety Committee, 2002, Acceptable Flood Capacity for Dams, DSC11, June
ES&S (2008)	Environmental Systems & Services, 2008. *Tillegra Dam Earthquake Hazard Assessment,* July

Geological Society (1977)	Geological Society Engineering Group Working Party, 1977. The description of rock masses for engineering purposes. Quarterly Jour. Egg. Geology, 10 (4), 355-388.
Hall (1952)	Hall, L.R. (1952):
Hill et al (2007)	Hill, P, McDonald, L and Payne, E, 2007, *Incremental Consequences of Dam Failure and the ANCOLD Hazard Classification System,* Proceedings of NZSOLD ANCOLD 2007 Conference, Queenstown, New Zealand, 19-20 November
Hill et al (2008)	Hill, P, Jordan, P, Nathan, R and Payne, E, 2008, *Estimating Construction Flood Risk,* Proceedings of ANCOLD 2008 Conference, Surfers Paradise, 13-14 November
Ho et al (2003)	Ho D. Boyes K. Donohoo S and Cooper B*. Numerical Flow Analysis for Spillways:* Proceedings of ANCOLD 2008 Conference, 2003.
ICOLD (1984)	International Committee on Large *Dams* 1984, *Question 84, Technical solutions to reduce time and cost in dam design and construction;* 22 nd ICOLD Congress
ICOLD (1986)	International Committee on Large *Dams* 1986, *Bulletin 48a, River Control During Dam Construction*
ICOLD (1989)	International Committee on Large *Dams* 1989 *Bulletin 70 Rockfill Dams with Concrete Facing*
ICOLD (1992)	International Committee on Large *Dams* 1992, *Bulletin 81 Spillways, Shockwaves and Air Entrainment*, *Review and Recommendations*
ICOLD (1993)	International Committee on Large *Dams* 1993, *Bulletin 89 Reinforced Rockfill and Reinforced Fill for Dams*,
ISRM (1978)	ISRM Commission on Standardisation of Laboratory and Field Tests, 1978. Suggested methods for the quantitative description of discontinuities in rock masses. J1. Rock Mechanics Min. Sci. and Geomech. Abstract., 15, 319-368.
Khatsuria (2005)	Khatsuria,RM, Hydraulics of Spillways and Energy Dissipators, 2005
Mason (1989)	"Effects of Air Entrainment on Plunge Pool Scour". Journal of Hydraulic Engineering ASCE, Vol 115, No 3, March 1989
McMahon et al (1975)	McMahon, B.K., Douglas, D.J., & Burgess, P.J., 1975. Engineering classification of sedimentary rocks in the Sydney area. Australian Geomechanics Journal, G5 (1), 51-53.
MHL (2008)	Manly Hydraulics Laboratory, Department of Commerce; *Proposal for Tillegra Dam Spillway Hydraulic Model Study*; August 2008.
PB (2007)	PB, 2007, *Tillegra Dam Mini Hydro;* PB Australia; October

PB (2008)	PB, 2008, *Tillegra Dam Mini Hydro Information Memorandum*; PB Australia; October
Pinto (1994)	Pinto N.L. 1994, *Conclusions to Q71* ICOLD 18th Congress on large Dams, Durban Q71
Pinto (2001)	Pinto N.L. 1994, *Questions to Ponder on Designing Very High CFRD's,* Hydropower & Dams, Issue 5, 2001.
Pinto (2007)	Pinto N.L. 2007, *Very High CFRD Dams, Behavior and Design Features,* Third Symposium on Concrete Face Rockfill Dams, Florianopolis, October.
Riddette et al, (2006)	Riddette K, Ho D, Edwards J. 2006, *Hydraulic Flow Simulation – Five year on: Lessons learned and Future Challenges*, Proceedings of ANCOLD 2006 Conference, Sydney.
Cooke et al (1987)	Cooke, J.B.' Sherard J.L. 1987, *Concrete face Rockfill Dams; Design, Construction and Performance.* American Society of Civil engineers Symposium October 21.
Smart (2006)	Smart, J.D. 2006, *Achieving Benefits from Instrumentation and Survey Measurements of Dam Behaviour.* Proceedings of ANCOLD 2006 Conference, Sydney.
USACE (1992)	United States Army Corps of Engineers, 1992*, Hydraulic Design of Spillways* EM1110-2-1603, , August.
USBR (1987)	United States Department of the Interior, Bureau of Reclamation 1987, *Design of Small Dams.*
USBR (1990)	United States Department of the Interior, Bureau of Reclamation, 1990, *Criteria and Guidance for Evacuating Storage Reservoir & Sizing Low Level Outlet Works*
USBR (1992)	United States Department of the Interior, Bureau of Reclamation, 1992, *Freeboard Criteria and Guidelines for Computing Freeboard allowances for Storage Dams*
Van Schalkwyk, (1994)	Commission Internationale des Grand Barrages, *Erosion of Rock in Unlined Spillways,* Commission Internationale des Grand Barrages, Van Schalkwyk, A. Jordaan, J.M. Dooge, N. 1994.
WRC (1985)	Water Resources Commission of New South Wales, 1985
Worley Parsons (2008)	"Tillegra Dam Spillway Design: *CFD Model Study Proposal,* Worley Parsons Proposal for a CFD model study of Tillegra Spillway; August 2008.

Xu Zeping (2008) Xu Zeping, 2008, Strong Earthquake on May 12 in China and its Impact on Dam Safety, Beijing, May.

Yildez & Uzucek (1994) Yildiz D. And Ezucel E.; *Experience gained in Turkey on Scours occurred Downstream of the Spillways of High Dams and Protective Measures*. ICOLD 18th Congress on large Dams, Durban 1994, Q71 R9.