GRAHAMSTOWN DAM STAGE 2 AUGMENTATION SELECTION AND DESIGN OF A LABYRINTH SPILLWAY AND BAFFLE CHUTE

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ABSTRACT. Grahamstown Dam is a major water supply source for the Newcastle area and it is proposed to raise the full supply level by 2.4m from RL 10.4m to RL 12.8m. The present spillway is inadequate to pass the PMF without overtopping of the existing embankments at the new FSL and part of the raising comprises construction of a new embankment of about 10m high with a right bank spillway upstream of the existing spillway capable of passing the PMF. The Pacific Highway is located some 600m downstream of the new spillway and a 60m wide culvert below the Pacific Highway is being constructed with capacity sufficient to pass the PMF. Significant changes were made to the feasibility design for the spillway and the Pacific Highway culvert using a labyrinth spillway and a baffle chute energy dissipator respectively. Both of these designs are uncommon and the process of finalising the designs as well as some of the problems in the use of a labyrinth spillway and the cost savings realised in the use of these designs are presented.

1 Introduction

Grahamstown Dam is a major water supply source for the Newcastle area and it is proposed to raise the present full supply by 2.4m from RL 10.4m to RL 12.8m. The dam, as shown on Figure 1, comprises the following embankments:

- The Pacific Highway to the north west of the reservoir
- The Main Dam to the West
- The Subsidiary/Saddle Dam to the South
- The new embankment adjacent to the proposed spillway to the north west

The raising is being completed in stages for which Stage 1 comprised raising the core level

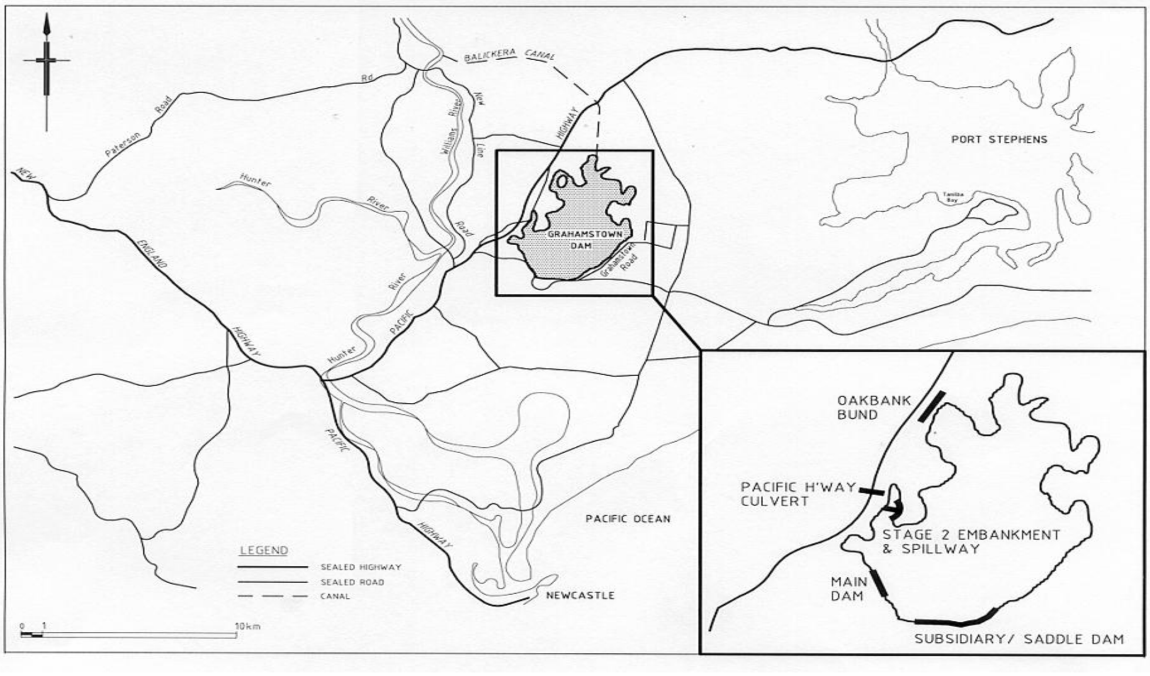

Figure 1- Locality Plan

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of the existing embankments to a level required for the Stage 2 full supply level and rip rap slope protection of the Main Dam. The stage 2 works comprises construction of a new spillway and associated embankment, a culvert below the Pacific Highway to pass the new spillway flows, a protection bund for the Newline Road which is downstream from the Pacific Highway culvert and pollution control works. Stages 3 and 4 have also been proposed for future water supply with a final increase in reservoir depth of 9m above the present level at RL 19.0m. Present design and construction work is required to accommodate future stages 3 and 4.

2 Feasibility Design

The feasibility study identified the preferred location for the new spillway and embankment and the size of the culvert required to convey the flood waters from the spillway below the Pacific Highway which is located some 600m downstream of the spillway. The general plan of the dam and the Pacific Highway culvert proposed at the feasibility level are shown on Figure 2.

The principal dimensions of the dam and Pacific Highway culvert are shown on Table 1. The feasibility level design for the spillway required the spillway channel to be excavated to RL 11.8m and RL 9m respectively upstream and downstream of the spillway. The excavated material was proposed as the borrow area used for construction of the new embankment, coffer dam and other works related to the Stage 2 construction. The excavation for the proposed spillway would result in excess spoil of between 160,000m^3 to 388,00m^3 in the worst case scenario.

Embankment	
Stage 2	
Type	Zoned Earthfill
Height	10m
Volume	120,000m^3
Coffer Dam Volume	60,000m^3
Stage 3	
Height	12m
Volume	60,000m^3
Stage 4	
Height	17m
Volume	270,000m^3
Spillway	
Type	Gravity Ogee
Length	180m
Capacity	PMF 635m^3/s
Pacific Highway Culvert	
Width	60m
Length	160m
Gradient	1:420
Capacity	635m^3/s
Peak Depth	4.5m

Table 1- Feasibility Study Principal Project

The feasibility study proposal for the Pacific

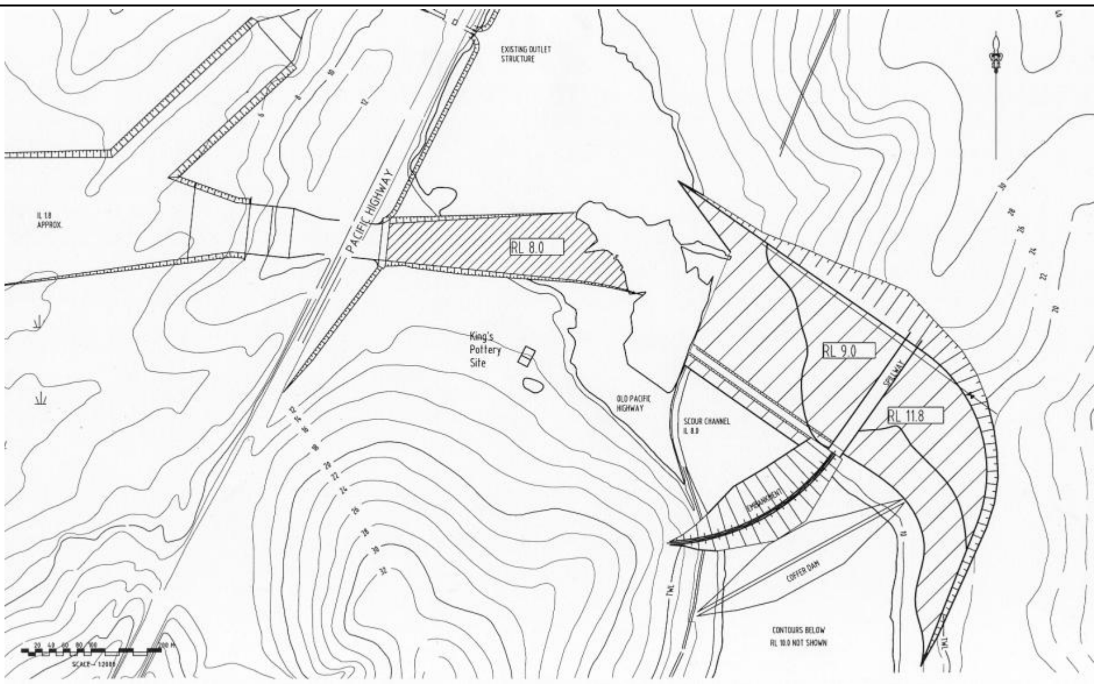

Figure 2 - Feasibility Study General Plan

Highway Culvert was a concrete lined channel of 60m width with a grade of 1 in 420 leading to a conventional USBR Type IV energy dissipator discharging into the Irrawang Swamp and down to the Pennington Drain outlets at the Newline Road which discharge into the Williams River.

3 Labyrinth Spillway

3.1 Risk Analysis

The use of an 85m wide labyrinth spillway was proposed which would considerably reduce the volume of excavation and cost of the spillway due to reduced concrete volume, both in the Stage 2 and future stages of construction. The reduced excavation volume for the labyrinth spillway also allowed more flexibility in selecting the location up the right bank slope from the embankment to produce the required volumes of soft and hard material for use in the construction. A risk analysis approach was used to determine the likely cost savings in the labyrinth spillway over the ogee spillway and the preferred location of the labyrinth spillway. The risk model was developed using @Risk developed by Palisade Tools (USA) with latin hypercube sampling. The model was developed for the Stage 2 and 3 works and included rock level variation, unit rates and labyrinth position on the slope. The rock profile was obtained from geological interpretation of borehole drilling for which a MOSS model was used to produce contours of rock, hard rippable and soft material for obtaining quantities of spillway excavation and embankment fill. At the time of design, road construction materials were being obtained from the spillway by the Roads and Traffic Authority (RTA) for the Pacific Highway upgrading and there was some uncertainty as to the volume and type of material to be taken by RTA. Consequently, a central core earthfill and central core rockfill embankment were considered in the analyses and the central core earthfill option selected once the RTA usage was confirmed.

The risk analysis showed that labyrinth spillway could result in a cost savings of $2.2M with an 80% likelihood of a $1.3M saving compared with the ogee spillway. The final position of the labyrinth spillway resulted in an estimated cost savings in the order of $1.4M for the labyrinth spillway compared with the gravity spillway.

The final layout and location of the labyrinth spillway is shown on Figure 3.

The visual impact of the labyrinth spillway was compared with the ogee spillway using three dimensional plots of the spillway and embankment viewed from the Pacific Highway

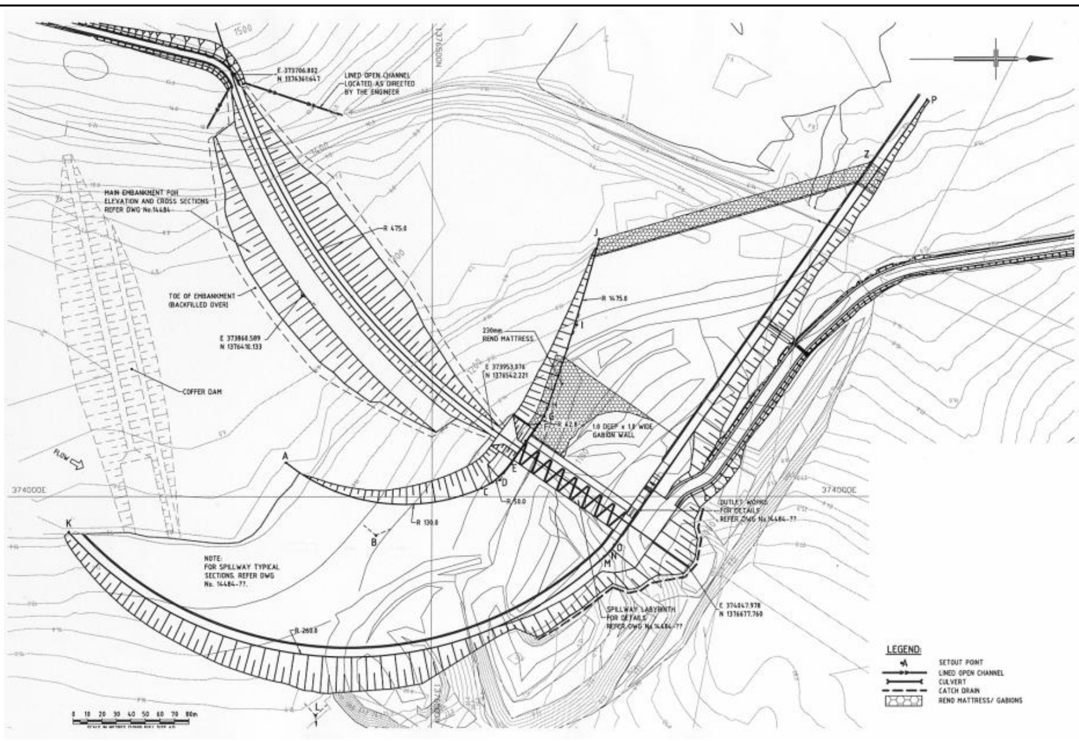

Figure 3- Plan of Embankment and Labyrinth Spillway

area. The plots were developed using the MOSS model and showed that both spillway options had minimal public impact and were similar in appearance and could not be set apart due to visual impact.

While the labyrinth provided a substantial reduction in construction cost, the labyrinth did require more detailed structural and hydraulic design compared with the ogee spillway. Furthermore, more attention was paid to the joint details and use of structural concrete in the labyrinth spillway than would be required for a mass gravity ogee spillway.

3.2 Spillway Rating

Labyrinth spillways are known to perform well under low head conditions when the depth of water is low in relation to the height of the labyrinth. The hydraulic efficiency of the spillway is a function of the crest shape, the head to labyrinth height ratio (h/p) and the length magnification of the weirs (l/w) (Figure 4)

The flow pattern in a labyrinth spillway passes through four phases as the upstream head increases:

- fully aerated - where the flow falls freely over the entire length of the weir.
- Partially aerated - there is flow restriction at the apexes, the tailwater rises and aeration of the nappe at the upstream apex is difficult. The flow at the upstream apexes is suppressed and air is drawn in from the downstream apexes forming a stable air pocket under the nappe.
- transitional - the nappe becomes suppressed at various locations and the stable air pocket breaks into smaller pockets causing instability in the nappe. This phase produces a discontinuity in the discharge rating curve.
- suppressed - the flow over the spillway forms a solid and non aerated nappe with no air being drawn under the nappe.

The discharge reduces rapidly in the transitional and suppressed zones, particularly once the weir is drowned and acts like a straight broad crested weir of the same length as the labyrinth width.

The labyrinth type spillway was first modeled in some detail by Hay and Taylor in the early 1970’s following which a number of physical and mathematical models have been used to determine the general characteristics of labyrinth spillways. When developing the rating curve for the Grahamstown Dam spillway, use was made of the data from the data from the following authors were used,

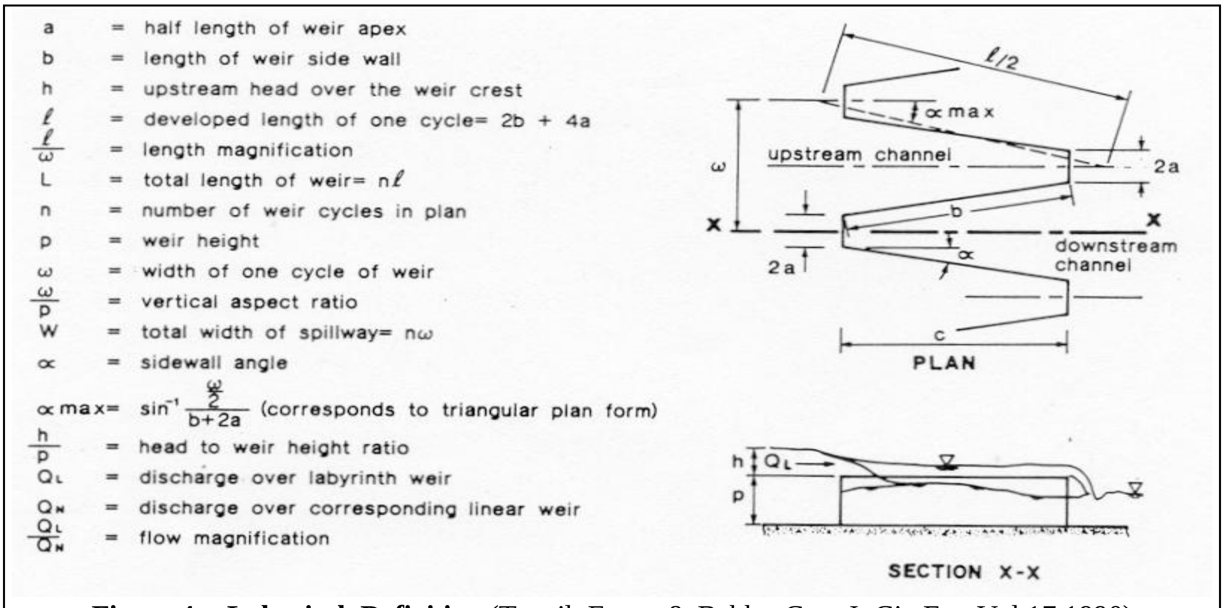

Figure 4 - Labyrinth Definition (Tacail, Evans & Babb - Can. J. Civ Eng Vol 17 1990)

grouped according to the calculation methodology or graphical presentation.

- Hay and Taylor (1970)
- Darvas (1970)
- Maghaeles P (1985); CIRIA
- Lux & Hinchliff; Houston & DeAngelis; Hinchliff & Houston (1984) (USBR); Afshar (1988)
- Tullis (1995)
- Yildiz & Uzucek (1996)

There are variations in the definition of head over the labyrinth and length magnification by some of the authors, for which due care is required when using the formula and design charts. In particular the definition of head where both the water depth at the weir (Hay & Taylor) and the energy head are used by various authors. When dealing with low head flow, as is the case with a number of labyrinthh spillways, this can have a significant affect on the discharge rating. Furthermore, the effective length of the weir must be based on the author's assumptions used to derive the discharge coefficients.

The model testing performed to date has generally been undertaken on quarter round upstream sections or sharp crested labyrinths due to the ease of construction. In the case of Grahamstown Dam, a composite curve has been used to improve the hydraulic efficiency at low heads and to assist in providing aeration of the nappe using splitters placed at the downstream section of the weir.

The paper by Tullis et al (1995) is one of the few that accounts for the width of the labyrinth weir crest when deriving the effective length of the weir and the discharge coefficeints. The definitions of the weir provided by Tullis are shown on Figure 5. Tullis' paper also provides a useful spreadsheet calculation for estimating the labyrinth dimensions as shown on Table 2 (it should be noted that an error was identified in the author's formula for the apron length B and has been corrected in table 2).

When developing the labyrinth design, the following recommendations made by various authors were used:

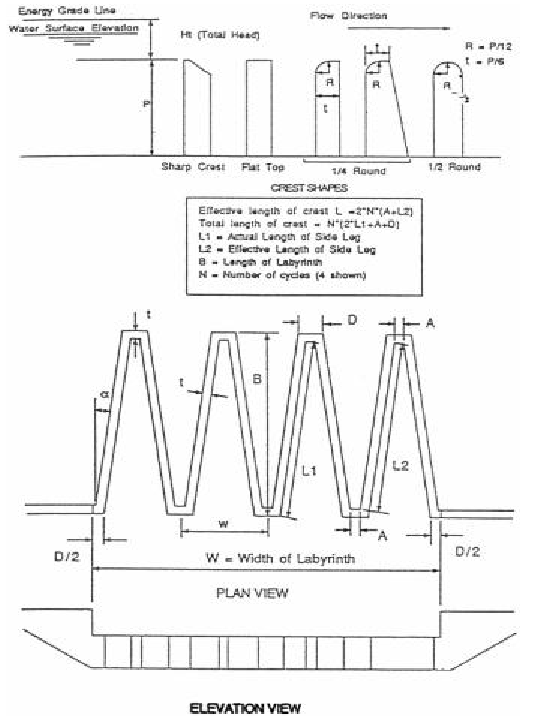

Figure 5 - Labyrinth Definition (Tullis 1995)

– Placement of the labyrinth weir as far

Parameter	Symbol	Value	Units	Source/Equation
Given Input Data				
Maximum flow	Qmax	628	m^3/s	
Maximum reservoir elevation	res	14.2	m	
Approach channel elevation			m	
Crest elevation	RL	12.8	m	
Total head	H_t	1.4	m	
Assumed Data				
Estimated inlet loss at Qmax	Loss	0.1	m	Estimated
Number of cycles	N	8		Select to keep w/p ~ 3 to 4
Crest height	P	3.8	m	Set P ~ $1.4H_t$
Angle of side legs	α	14.85	deg	Normally 8 to 12 deg
Calculated Data				
Thickness of wall	t	0.75	m	t=P/6
Inside width at apex	A	1	m	Select between t and 2t
Apex width at centre line		1.58	m	(A+D)/2
Outside width of apex	D	2.15	m	D= A+2tan(45-α)
Total head/crest height	H_t/P	0.368		
Crest coefficient	C_d	0.535		From Graphs
Effective crest length	L	240.1	m	$1.5Qmax/\{(C_dH_t^{1.5})*(2g)^{0.5}]$
L/W Ratio	L/w	3.04		L_3/W
Length of apron (parallel to flow)	B	14.85	m	B=[L/(2N)-A+t*tan(45-α/2)]*cos(α)+t
Actual length of side leg	L_1	14.58	m	L_1=(b-t)/cos(a)
Effective length of side leg	L_2	14.01	m	$L_2 = L_1$-t*tan(45-α/2)
Total length of walls	L_3	258.5	m	$L_3 = N(2L_1 + D + A)$
Distance between cycles	w	10.62	m	$w = 2L_1\sin(\alpha) + A + D$
Width of labyrinth (normal to flow)	W	85.00	m	W = Nw
Length of linear weir for same flow			m	$1.5*Qmax/[(C_d*Ht^{1.5})*(2g)^{0.5}]$ C_d for linear Weir
Distance between cycles/crest height	w/P	2.80		Normally between 3 and 4

Table 2 - Labyrinth Weir Design Spreadsheet (Tullis 1995)

upstream as possible with the sidewalls attached to the downstream apexes to reduce approach losses;

– Ho/p ratios between 0.45 to 0.55 to avoid operation in the suppressed zone;

– Minimum vertical aspect ratio w/p of 2 to minimise nappe interference and generally between 3 to 4;

- Angle of side legs 8° to 16°;
- Inside apex length between t and 2t where t is the labyrinth wall thickness.
- Length magnification ratios l/w up to 4 for Ho/p values up to 0.5;
- Minimum α/α_{max} of 0.75;

As shown on Figure 6, the various models provided an estimate of likely upper and lower discharge ratings for the labyrinth from which to assess the performance and possible range of error in the spillway rating. Three dimensional flow over the labyrinth spillway does not readily lend itself to mathematical modelling, however, it was concluded from the error curve that it was unnecessary to perform any physical modelling of the labyrinth to determine the rating curve which was based on the mean of the ratings determined using the available methods.

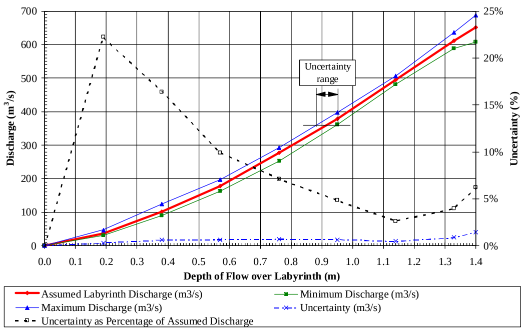

Figure 6 - Labyrinth Spillway rating Curve

3.3 Spillway Design

Some of the design considerations for the labyrinth spillway included aeration, noise suppression, joint treatment, anchoring and staging for future raisings.

Nappe aeration to prevent noise due to oscillating nappe against the downstream face was schieved using splitter piers placed on the crest of each section at between 0.08b to 0.1b from the downstream apex. As piers are only required for low flows, heights generally vary between 0.15Ho to 0.25Ho where Ho is the design head.

Design loads included hydrostatic, seismic OBE (1000 yrs ARI) and MDE (10,000yrs ARI) and thermal loading. The structure was designed using conventional uplift with full hydrostatic at the reservoir reducing to one third of the head differential between the upstream and tailwater levels plus the tailwater level at the line of drains to tailwater level at the toe. The following load cases were used for the stage 2, 3 and 4 Full Supply Levels (FSL) of 12.8m, 14.7m and 19.7m respectively.

Usual	Reservoir at FSL, No earthquake load.
Unusual	Reservoir at Design Flood Level, No earthquake load.
	Reservoir at FSL, Pseudo static earthquake loading with 0.16g acceleration (OBE).
Extreme	Reservoir at PMF level, No earthquake load.
	Reservoir at FSL, Pseudo static earthquake loading with 0.38g acceleration (MDE).

The factors of safety required and achieveed for each loading case were as follows:

Load Case	Factor of Safety	
	Sliding	Overturning
Usual	4.0	1.5
Unusual	2.7	1.3
Extreme	1.1	1.1

The stage 3 and 4 designs required additional weighting concrete to be placed between the labyrinth walls, as shown schematically on Figure 7.

4 Hydraulic Modelling

Due to the reduced width of the labyrinth spillway compared with the feasibility study ogee spillway, there is an increase in the specific discharge in the upstream and downstream spillway channels which are unlined apart from areas with an erosion potential. Furthermore, the operation of the Pacific Highway culvert and the upstream flow patterns and water levels against the Pacific Highway embankment were required to be checked to ensure that the level did not exceed RL 12.5m, as required by the Roads and Traffic Authority (RTA).

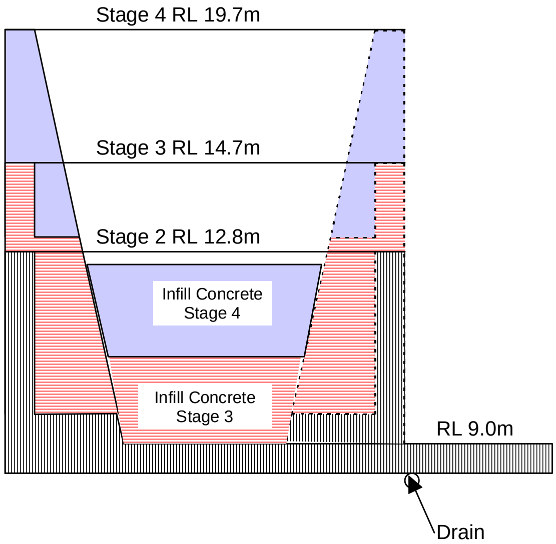

Figure 7 - Labyrinth Staging

Mathematical modelling was therefore carried out (a) for the entrance to the spillway and (b) for the downstream area between the spillway and the Pacific Highway culvert, as shown in Figure 2 in order to:

- Evaluate the flow patterns and velocities in the upstream entrance channel and the area downstream of the labyrinth to the Pacific Highway Culvert;
- Evaluate the potential for erosion problems resulting from high velocity flows;
- Determine the water levels in the vicinity of the Pacific Highway;
- Derive a tailwater rating curve for the labyrinth spillway design.

The SMS (Surface Water Modelling System: Brigham Young University, 1995) model was used for this investigation. The SMS model is a two-dimensional, in plan, finite element model that allows calculations of flow velocities and depths which have a two-dimensional flow area of varying topography.

In the SMS model the site is represented by a series of quadrilateral and/or triangular elements with nodes or calculation points located at the corner point of each element and also at the mid-point of each quadrilateral or triangle.

Flood depths in SMS are calculated using elements which can undergo wetting and drying. The model is 2-Dimensional and velocities are determined at each node, with X and Y components enabling determination of a specific flow direction at each node.

When the topography has been defined, the SMS model solves the vertically averaged time dependant Reynolds form of the Navier Stokes Equations which are established for each computational node.

Data entered into the model consisted of topographic information which was extracted from available ground contour mapping, upstream flow rates or discharges, downstream control water levels and Manning's "n" values for the ground surface.

Steady state analyses were undertaken using the assumption of a constant discharge at both the inlet and outlet to the model. Because of the relatively small plan areas considered in each of the two models it was not necessary to allow for dynamic effects within the modelling.

Two sets of analyses were undertaken:

- The upstream section of approach flow to the labyrinth weir under the probable maximum flood discharge condition.
- The downstream section, between the labyrinth spillway and the Pacific Highway culvert where the 10, 100, 200, 2000, 200,000 year ARI events and probable maximum flood (PMF) flow rates were used. The flow rates were extracted from the RORB design flows for Grahamstown Dam.

Sensitivity analyses were completed for the Mannings "n" using values of 0.016, 0.03 and 0.05 to determine the effect on the tailwater rating curve and the depths near the Pacific Highway.

The unscaled velocity vectors in the downstream area for the PMF are shown on Figure 7 which is also typical of the flow pattern for the lower flow conditions. Colour plots of velocity were also produced for each flow to evaluate the erosion potential in the entrance and downstream areas.

– the approach to the channel under the Pacific Highway;

– immediately downstream of the labyrinth in the spillway channel; and

– the end of the spillway channel.

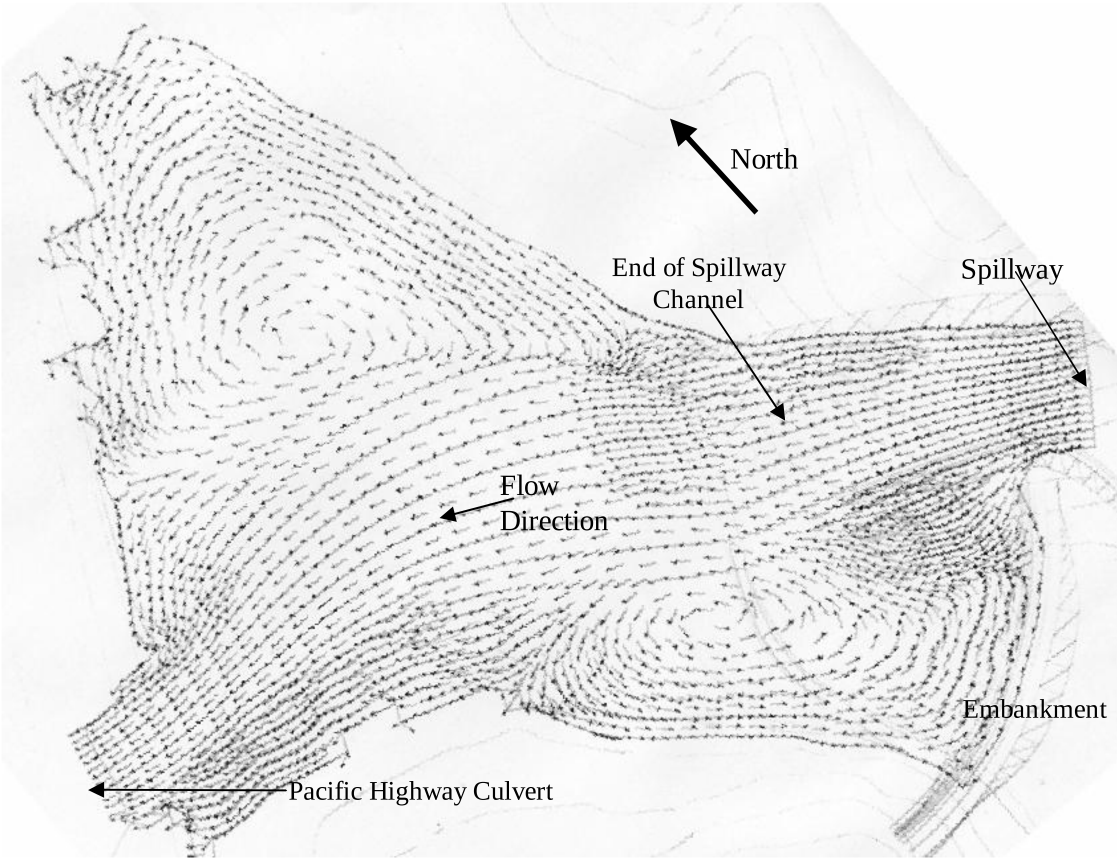

Figure 8 - PMF Velocity Vectors Obtained using SMS Model

The modelling indicated that approach flow conditions towards the labyrinth spillway will be relatively uniform and that the only areas of modest velocity will be in the immediate vicinity of the labyrinth spillway.

As shown on Figure 8, there will be a recirculation of water north of the predominant flow path from the labyrinth spillway to the Pacific Highway culvert and also to the south immediately downstream of the embankment. The strength of these recirculation patterns was modest and was not predicted to be of sufficient magnitude to cause any significant erosion risk in these areas.

The main areas of concern for erosion indicated by the model were:

Relatively high velocities up to 2.5m/s were indicated in these areas by the modelling for high frequency events (10 year to 200 year ARI) with model conditions simulating a clean spillway channel and approach channel to the Pacific Highway (n = 0.016). Sensitivity analysis results for higher Manning's "n" values up to 0.05 indicated a reduction in velocity to about 2m/s in these areas.

As theses velocities exceeded the erosion resistance potential for the excavated surfaces of the spillway channel and Pacific Highway entrance channel, it was recommended that suitable dense knitted grass be planted on the channel invert and sides up to the level of more frequent floods. It was also recommended that reno-mattress protection works be provided for

10m upstream of the entrance to the Pacific Highway culvert, and 10m at the end of the spillway channel and up to 50m immediately downstream of the labyrinth where areas other than rock would exposed in the excavation as these areas will be subject to increased velocities and turbulence due to rapidly changing flow conditions.

The model results also showed that the water level will be less than the maximum RTA level of RL12.5 upstream of the Pacific Highway for surface roughness values up to a likely maximum Manning's "n" value of 0.1in the main area of flow which relates to dense brush growth in the flow path.

5 Pacific Highway Culvert

The Pacific Highway culvert is designed to pass the PMF with a maximum flow of $680m^3/s$ which was the feasibility design flood. The passage of the PMF through a road culvert is unusual but has been adopted by Hunter Water Corporation for the management of risk associated with potential damage or failure of the Pacific Highway road bridge caused by operation of the Grahamstown Dam.

The concept design consisted of a conventional stilling basin downstream of the 60 m wide culvert which flared to 70 m wide upstream of the drop into the basin. The stilling basin, which was 32 m long USBR Type IV basin was likely to be founded on highly weathered basalt. The excavated channel entering the culvert was shown with a fall of 1:100, with the culvert chute having a fall of 1:363 prior to a 1:5 drop from RL 7.9 into the stilling basin with a level of RL 1.8.

Preliminary evaluation of the feasibility design indicated that:

- the flare from the chute into the drop was likely to result in standing waves forming due to flow separation upon entering the flare for flows greater than about $50m^3/s$ where the flow in the channel is supercritical.

- the flow in the steep entrance channel, excavated in soil, is subcritical for low flows but changes to supercritical as the flow invcreases. The flow never reaches high Froude numbers. This will mean a very unstable flow condition in the channel, with the potential for a weak hydraulic jump forming at low flows at the channel entry. This will be exacerbated by the presence of the bridge piers forming a contraction within the channel (which was not assessed). This may result in the formation of another weak hydraulic jump at the bridge piers under some flow conditions.

In order to maintain consistent flow conditions at the channel entry. The slope of the excavated entrance channel was reduced to 1:700 to ensure that the flow in the channel was subcritical for all flows.

The Pacific Highway Twin Bridges over the proposed channel had been designed and commenced construction subsequent to the feasibility design. The most significant change arising from the final design of the bridges was that the culvert slope had been revised to 1:420 and because the footings had been cast, this slope was fixed. The footings resulted in an entry level of RL 7.6 m and RL 7.3 m at the top of the drop into the stilling basin.

Because of the concerns about standing waves from the flare in the downstream channel, the design progressed to a conventional stilling basin with no flare.

Slightly weathered to fresh rock was not found in the foundations for the chute, so a thin concrete lining with anchors was not practical. Rather, a thick concrete stilling basin was designed to resist the pressure differentials expected with the formation of a hydraulic jump within the basin.

Conventional methods of assessing the effectiveness of the stilling basin were not applicable because there is insufficient drop into the structure to establish "normal" flow conditions. Rather a gradually varied flow analysis was undertaken for a number of flows to assess the distances required to establish normal flow conditions in each length of channel. It was assumed that the flow into the channel was at normal depth. The data from the analyses are shown on Table 3.

The Froude numbers obtained from text book analysis assume that there is sufficient distance to develop stable flow conditions. However, in the case of the Pacific Highway stilling basin, where the drop is small and the length down the drop is small (about 30 m) there will be insufficient distance to develop steady state conditions. The flow is still accelerating to the steady state condition meaning that the drop serves only as a slight perturbation to the flow, instead of developing a stable hydraulic jump condition.

Flow (m^3/s)	Froude Numbers				
	entry	chute	drop[1]	drop[2]	TW
25	0.7	0.9	4.1	19.8	0.4
50	0.7	0.7	1.8	14.1	0.8
72 1:5AEP	0.7	0.7	1.5	11.8	1.1
105 1:20AEP	0.8	0.8	1.3	9.9	0.8
680 PMF	0.9	1.1	1.2	4.3	0.3

[1] Gradually varied Flow; [2] Textbook analysis

Table 3 - Pacific Highway Culvert Hydraulics

Table 4 indicates the characteristic forms of hydraulic jumps for various Froude numbers.

Of particular concern is the flow regime for Froude numbers below 1.75 where damaging standing waves can form resulting in scour downstream of the stilling basin. The stilling basin in these cases has to be as long as practicable to ensure that scour does not progress back to the drop structure.

The hydraulic analyses shown on Table 3 for the gradually varied flow calcuations indicated that for flows greater than 70 m^3/s (1:5 AEP event) there is no jump formed in the stilling basin. For flows between about 35 m^3/s and 70 m^3/s an unstable pre-jump condition is developed and for flows less than about 35 m^3/s an oscillating jump forms.

The hydraulic analyses showed that the conventional USBR type stilling basin option was not ideally suited to the hydraulic conditions present in the channel requiring a considerable length of concrete lining of the stilling basin and heavy rip rap in the downstream channel.

Options for an alternative energy dissipators included a free overflow drop structure into a pool, an inpact basin or an impact type baffle chute.

There is insufficient vertical drop to ensure that a free overflow drop structure is not drowned during large flows. Drowning of the free overflow structure will result in loss of air to the underside of the nappe and energy losses will be significantly reduced. This may result in scour downstream of the drop structure.

Froude Number	**Jump Characteristic**
<1.75	No jump, surface waves are similar to standing waves and are difficult to predict
1.75 to 2.5	Pre jump, with smaller rolling waves on the surface
2.5 to 4.5	Poorly formed, turbulent and unstable with oscillating conditions between collapse of the jump and jump formation
4.5 to 9	Well balanced, stable jump
9 to 13	Transition from good to rough jump
>13	Rough and violent

Table 4 - Hydraulic Jump Characteristics

The impact type basin is a USBR type horizontal impact beam installed in the basin which allows flow under the beam. The flow depths vary from 0.1 m to 2.1 m in the drop structure which means a beam type impact basin is not likely to be effective for all flows.

The baffle chute energy dissipater is ideally suited to channel type drop structures and has been used with confidence for many years, as shown in the opening paragraphs of the relevant chapter in Monograph 25 "Hydraulic Design of Stilling Basins and Energy Dissipators" (USBR, 1984) repeated below:

"Baffled aprons or chutes have been in use on irrigation projects for many years. The fact

that many of these structures have been built and have performed satisfactorily indicates that they are practical and that in many cases they are an economical answer to the problem of dissipating energy. Baffled chutes are used to dissipate the energy in the flow at a drop and are most often used on canal wasteways or drops."

basin, however, the low Froude numbers indicate that minor standing waves will develop for which the concrete thickness of the shute slab is adequate to accommodate the differential heads across the jump.

Furthermore, the operation of the baffle chute dissipater is not dependent on the tailwater level within the downstream Irrawang Swamp, unlike

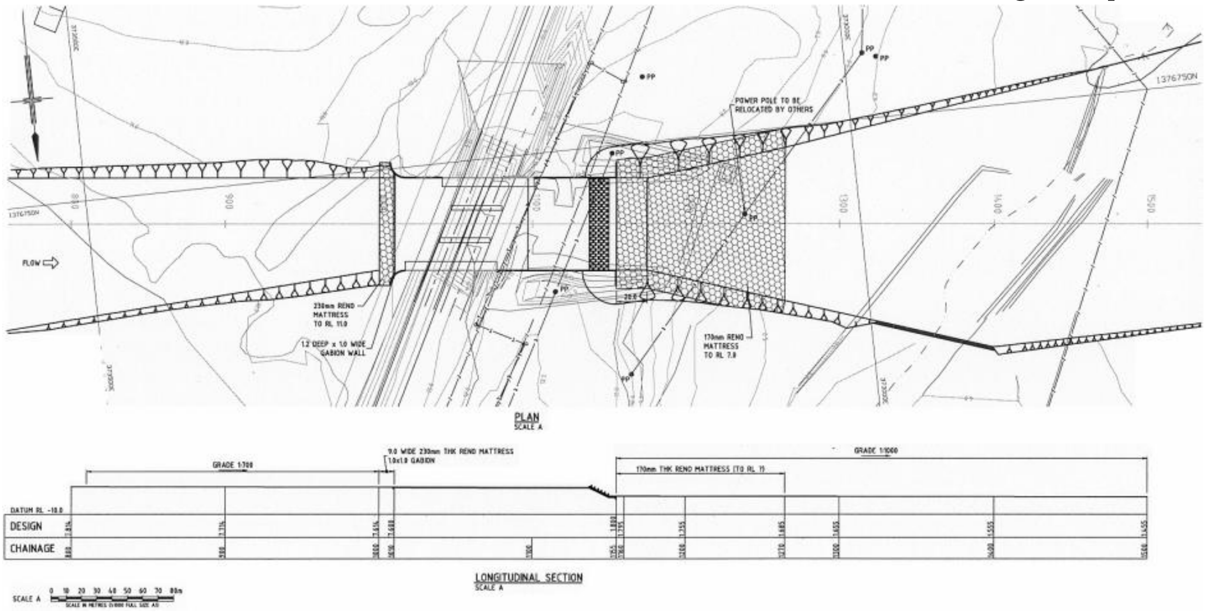

Figure 9 - Baffle Chute Spillway

a conventional stilling basin which relies on sufficient water level downstream of the structure to ensure the dissipater works effectively.

The baffle chute works by ensuring that the flow velocity does not increase down the chute. This is achieved by turbulent impact with the baffle blocks, and turbulent flow between adjacent blocks. The key factor in the design of a baffle chute is to ensure that the flow velocity entering the chute is less than a "critical" value. For flow velocities above this value, the impact with the first tooth is severe and causes the flow to "bounce" over a number of baffles (although some degree of "bounce" or standing wave is expected with higher flows). This may require a stilling pool upstream of the drop depending on the analysis of flow down the sluice.

To investigate the maximum permissible velocity and requirement for a stilling basin upstream of the drop, a number of basin depths were trialed. The analyses indicated that a stilling basin of 0.75m was required with an upstream flat section leading from the 1:420 channel chute. As shown in table 3, there is a transition from supercritical to subcritical flow in the chute section when entering the stilling

6 Conclusion

The use of laybrinth spillway, which is an uncommon spillway design, resulted in a $1.4M saving over the use of the conventional ogee spillway proposed in the feasibility study. While the labyrinth provided a substantial reduction in construction cost, the labyrinth did require more detailed structural and hydraulic design compared with the ogee spillway. Furthermore, more attention was paid to the joint details and use of structural concrete in the labyrinth spillway than would be required for a mass gravity ogee spillway.

The use of the baffle chute energy dissipator for the pacific Highway culvert discharge provided more control for energy dissipation at varying flows and allowed a shortened energy dissipator with consequent cost savings in the order of $900,000 compared with the USBR type stilling basin proposed for the feasibility level study.

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