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Uncontrolled Chute Spillway with Unlined Cascade, Dartmouth Dam

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SUMMARY The Dartmouth Dam spillway will have a design capacity of 2750 m^3/s, and will comprise: an unlined approach channel; an uncontrolled concrete weir 3 m high with an ogee crest 91.9 m long and a short upstream apron; and a steep concrete-lined parallel-sided chute extending 80 m downstream, and terminating with side deflectors and a horizontal flip step. The chute will discharge obliquely into the rock quarry, which was designed as an unlined stepped cascade about 500 m long and up to 350 m wide, consisting of nine 15-m drops and near-horizontal benches up to 85 m long. The paper presents the main design criteria used, describes the hydraulic features of the spillway and cascade, summarizes the results of hydraulic model tests, and mentions various practical factors affecting design.

1 INTRODUCTION

1.1 Flood Estimates

The Dartmouth Dam Project, currently under construction on the Mitta Mitta River in north-eastern Victoria, has been described in recent papers (Maver and Michels, 1975). Above the dam site, the catchment area is 3600 km^2 and the annual precipitation ranges from 640 to 1520 mm.

Estimates of the design flood, prepared by two different methods, agreed closely as regards hydrograph volume, and the design outflows were both about 2750 m^3/s; for an "embankment" (no-freeboard) flood, the peak outflow would be 4000 m^3/s.

1.2 Basic Arrangement

The flood provision comprises basically an uncontrolled weir and chute spillway discharging into an unlined cascade outfall formed by a stepped rock-quarry excavation (Fig. 1), a concept used earlier for the Bellfield Dam (Currey, Michels and Little, 1968).

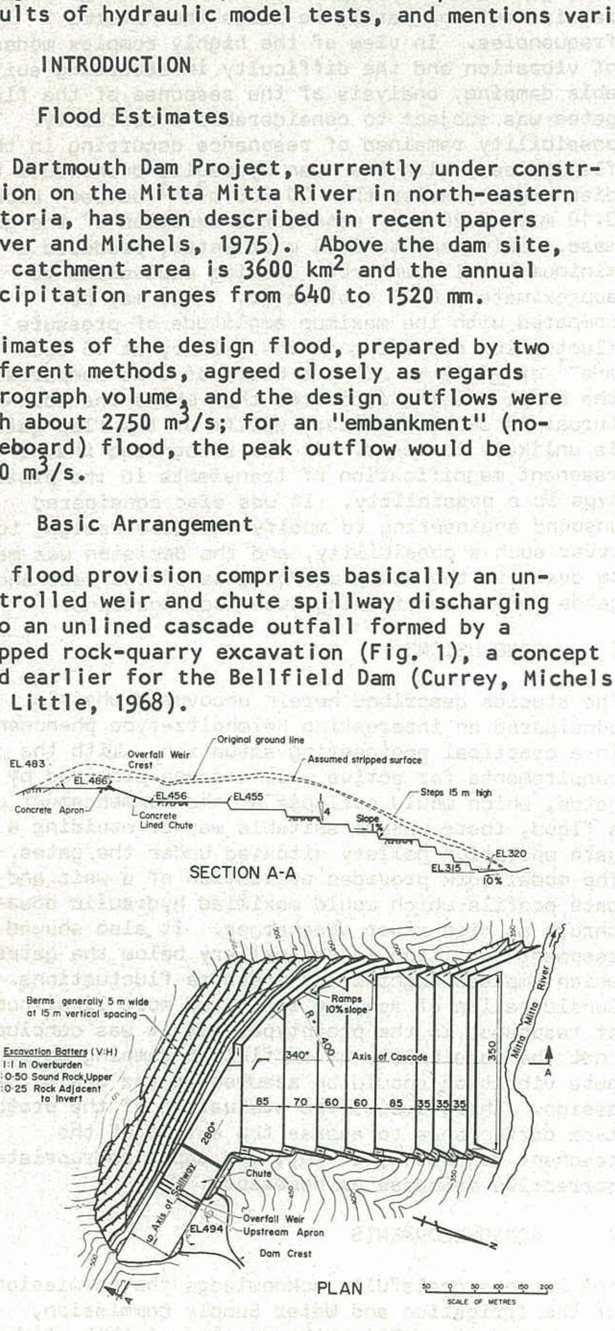

Figure 1 Spillway and cascade - general arrangement

Although gated alternatives were found to be marginally cheaper, an ungated design was adopted on operational grounds.

A chevron-shaped weir in plan followed by a convergent trough was also considered, e.g. the Nillahcootie and Rosslynne spillways (Michels, 1966, 1971; Johnson, 1970), but without a lined chute downstream.

Obviously, construction would be simplified (particularly for the chevron weir design) if the chute slope were connected tangentially to the ogee weir profile, but the discharge coefficient would diminish with the downstream slope.

The planned hydraulic model studies for rating purposes therefore included tests to evaluate this inter-relationship for economic comparisons, also for future designs of overfall-crest structures.

2 DESIGN CRITERIA

2.1 Quarry Requirements

The rock at the dam site is an extensively jointed granitic gneiss, intersected by several old faults and shear zones, which has weathered irregularly, particularly in joint troughs (Currey, 1976).

The quarry location, dimensions and alignment were fixed so as to: (i) avoid deeply weathered areas; (ii) yield the required volume of good rock for permanent construction, with an adequate margin; (iii) ensure that side batters would be predominantly in hard rock; (iv) keep the quarrying operations reasonably clear of dam construction work; (v) provide suitably graded access ramps to the cascade benches; and (vi) provide a reasonable face height for production blasts.

2.2 Hydraulic Requirements

The spillway was sited so that: (i) its right-hand excavation batters would be in competent rock; (ii) the cascade would clear the deeply weathered gully downstream of the dam, and (iii) high-velocity chute flows would pass well beyond the dam toe and the power station.

The spillway dimensions were fixed after a preliminary economic comparison of alternatives with different weir lengths and appropriately varied dam and weir heights, quantities, etc.

Cascade steps were limited to 15 m, with their faces sloped steeply enough to ensure a nonadhering jet at all but small flows, and theoretically a high efficiency of energy dissipation.

2.3 Maintenance Requirements

To ensure stability and access for maintenance, the 100-metre-high spillway cut was provided with sufficiently flat batters, suitably spaced berms, access ramps, catch and surface drains.

The lowest cascade bench forms a trap for eroded rock so as to mitigate sedimentation of the re-regulation pondage downstream and to limit rises in power-station tailbay level due to debris build-up. One low bench will provide an access crossing during periods of little or no spillway overflow.

2.4 Aesthetic Considerations

The weathered excavation faces will harmonize with natural features and the rockfill face of the dam. The cascade and the embankment will have overall dimensions of the same order and a similar "rough" texture; even major excavation irregularities will be dwarfed by these gigantic structures.

3 HYDRAULIC DESIGN

3.1 Main Features

The main features and dimensions of the spillway and cascade are shown in Figs. 1 and 2. Cut-off, drainage and anchorage details of the weir, chute and other components generally conform to Commission practice (McLellan, 1976).

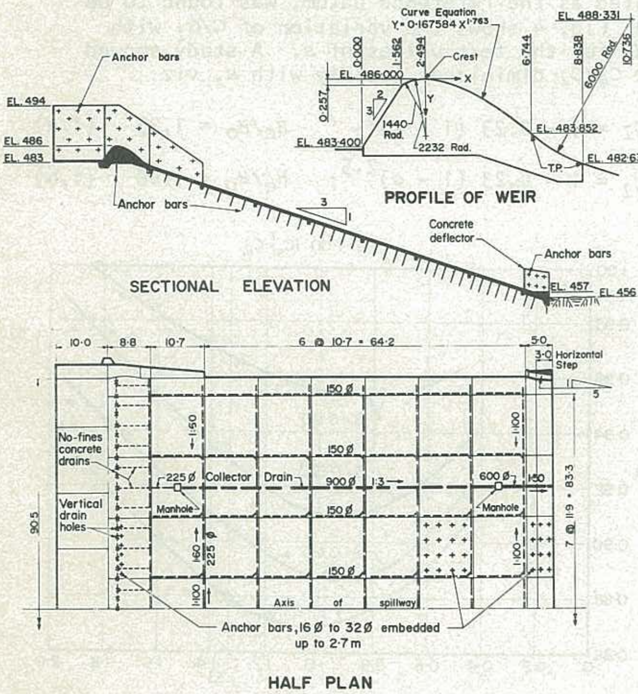

Figure 2 Spillway weir and chute - details

3.2 Approach Channel

The normal channel-to-weir approach ensured a reasonably symmetrical and uniform flow distribution upstream of and over the weir. The channel invert was set 3 m below weir crest to reduce velocities and friction losses to acceptable values.

3.3 Weir

The weir crest was profiled for a head of 75% of the design-flood head, in order to maximize the discharge coefficient under design flood conditions (Bureau of Reclamation, 1973). The upstream face was sloped to increase the discharge coefficient and improve structural stability, and a reverse curve was provided at the toe.

The upstream apron and batter lining were provided to minimize turbulence at the crest, prevent outflanking erosion, and increase the seepage path. The downstream batter lining affords protection where the rapidly accelerating overflow has a high downward velocity component.

3.4 Chute

The reinforced-concrete chute was designed for a nominal uplift pressure of only 10% of the velocity head, having regard to the sound rock foundation and the drainage being provided.

The chute was terminated with: (i) a horizontal step to deflect the high-velocity water away from the unlined rock immediately downstream; and (ii) lateral deflectors to aerate the lower nappe. It will discharge obliquely onto the uppermost bench of the unlined cascade.

The chute batters will have small nib walls along their base, but will be treated with backfill concrete, shotcrete or pneumatically applied mortar, as may be required. Some dental concrete and rock-bolting is likely at the zone of jet impact and expansion below the chute exit.

3.5 Cascade

The step faces were sloped 1:0.25* and the benches spaced 15 m vertically, to accord with the main design criteria. The side batters were also sloped 1:0.25, and access ramps positioned so as to guide flow around the curve and aerate the overflow jets.

The long left-hand half of the uppermost bench and the gradual increase in cascade width furnished by the curved boundary downstream, were aimed at providing some lateral expansion of the chute flow, with consequent reduction in the average unit discharge and energy dissipation.

The design required careful excavation of the step faces, with concreting and/or rock-bolting where necessary, to ensure stability in service. A typical quarrying scene is shown in Fig. 3.

Figure 3 Quarrying operations in cascade area, ELs 470-516

* All slopes are given as V:H ratios.

The actual trough-weathering slope normal to the direction of the steps, was estimated by the geologists to be about 1:0.7, and a slope of 1:1 was suggested for stability.

An analysis comparing the likely hydraulic performance of cascades with 1:0.25 and 1:1 step faces, indicated that a free-falling jet may not have its energy fully dissipated, and may merely reverse its curvature and turn smoothly into supercritical flow along the bench below (Chow, 1959). Also, the energy loss on irregular 1:1 batters could be of the same order as that for a free fall.

The formation of a hydraulic jump on a bench will depend on its slope, length, and effective rugosity (the roughness elements may be as high as the water depth). Hence, flow velocities and depths on the benches may differ considerably from theoretical values.

A prolonged free-falling jet overflow may undermine a steep step; therefore, a substantial basal fillet of sound rock should be left. The triangular rock mass between the final 1:0.25 face and the above-mentioned 1:0.7 slope should be in such a condition as to be stable under gravitational forces plus high-velocity flow; if not, immediate removal or stabilization may be desirable.

4 MODEL STUDIES

4.1 Aims of Model Tests

The principal aims of the hydraulic studies were:

(a) In preliminary tests, to determine the discharge coefficient of an ogee-profiled weir crest with: (i) a tangential exit therefrom directly onto the chute, for a practical range of slopes; (ii) the usual reverse curve, tangential to both the ogee profile and a series of downstream slopes.

(b) In comprehensive tests: (i) to study the hydraulic performance of the preliminary design and modify it as necessary to ensure a reasonable flow distribution onto the upper cascade benches, subject to construction economy and quarrying requirements; (ii) to check velocity profiles in the approach channel, rate the spillway, measure chute flow depths, develop acceptable chute exit conditions, and improve jet aeration and energy dissipation on the cascade.

4.2 Models, Scales, and Construction

Two models were constructed, their scales being governed mainly by space and discharge limitations: (a) Preliminary weir tests (1:27), and (b) comprehensive model tests (1:60). The model surfaces were either sheeted or so finished that their roughness would closely approximate the theoretical model values (Chow, 1959).

The weir model was built with suppressed end contractions, using a fairly long approach flume and a chute segment with an adjustable tangential slope. The comprehensive model reproduced part of the adjoining reservoir, the approach channel, spillway, and the upper four cascade benches.

4.3 Instrumentation and Accuracy

Model discharges were measured over V-notch weirs calibrated by volumetric tanks to an accuracy of ± 0.3 mm. Approach channel velocities were recorded by a transistorized flowmeter, and pressures measured with piezometer tubes to an accuracy of ± 0.2 mm.

4.4 Supplementary Tests

As quarrying proceeded, a shortfall of suitable rock appeared likely due to weathering depths and actual rockfill densities being greater than anticipated. The design allowed no changes to the top and bottom benches and to levels of intermediate benches. Hence, the necessary upstream (axial) shift of the intermediate bench batters involved either:

A. cutting back the already excaveted side batters so as to finish with the same final bench outline in plan; or

B. accepting the existing side batters and merely continuing them downward, thus shifting their basal toe intersections inward, and so narrowing the benches. Supplementary tests were therefore carried out to ascertain if alternative B, which had practical and economic advantages, could be adopted without adversely affecting hydraulic performance.

5 SUMMARY OF TEST RESULTS

5.1 General

All the figures for dimensions and other quantities given below are the full-scale (prototype) values.

5.2 Preliminary Weir Model

The weir was modelled for a length of 12.4 m and a height P of 2 m, with an upstream slope of 1:0.67, and a crest design head H_o of 4.5 m (Bureau of Reclamation, 1973).

Tests were run with the downstream slope a tangential to the crest curve and ranging from 0.20 to 1.00, under total heads H_e (static plus approach velocity head) of up to 1.95 H_o.

Designating by C_s the discharge coefficient for any downstream slope s the mean value of C_1 $(s = 1)$ adopted as the reference datum, was found to be 2.20; Fig. 4 shows the variation of C/C_1 with H_e/H_o for the test values of s. A study showed that C_s/C_1 diminishes rapidly with s, viz:

$$C_s/C_1 = 1 - 0.23\,(1 - s)^3; \qquad H_e/H_o = 1.33 \qquad (1.a)$$

$$C_s/C_1 = 1 - 0.23\,(1 - s)^{2.5}; \qquad H_e/H_o = 1.00 \qquad (1.b)$$

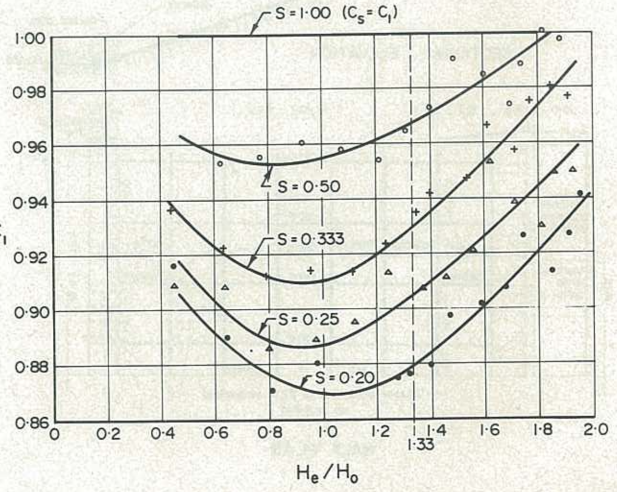

Figure 4 Variation of C_s/C_1 with H_e/H_o for different downstream slopes s - preliminary weir model (1:27)

For $s = 0$, and the particular upstream slope, edge rounding, and ratio P/H_o, Eq. (1) makes $C_o = 1.70$; this agrees well with the figure of 1.704 for critical-depth flow over a weir with a rounded upstream edge and a small downstream slope of the crest (King and Brater, 1963). Also, for an upstream slope of 1:0.67, C_o has a value of 2.12 at $H_e/H_o = 1$, (Bureau of Reclamation, 1973). The corresponding model value of C_s for $0.5 < s < 1.0$, ranged from 2.10 to 2.20 (Fig. 4).

Because for $s < 0.50$, C_s drops excessively ($\geq 3\%$), the normal design with a reverse curve was adopted. Also, for this and certain other reasons, the chevron weir alternative was abandoned.

5.3 Comprehensive Model

5.3.1 Approach channel

At a discharge of 500 m^3/s, separation from the right-hand batter transition near the channel entrance caused pronounced waves, eddies and vortices. These intensified with discharge, developing a sharp drawdown in the separation area and an irregular wave pattern down to the weir.

Separation from the left-hand batter transition developed at 2000 m^3/s. At the design flood discharge, channel velocity distribution was significantly disturbed by the batter transitions, causing uneven weir overflow.

5.3.2 Weir

A weir fitted inside a trapezoidal channel is equivalent to rectangular and triangular weirs, each carrying flow with suppressed and contractions. In the absence of appropriate experimental data the following "combined" formula was used:

$$Q = C_t\ (L + 0.8\ zHe)\ H_e^{1.5} \qquad (2)$$

where Q = total discharge, m^3/s; C_t = effective overall discharge coefficient for the particular trapezoidal weir; L = length of weir crest at its apex, m; z = end-wall batter slope to vertical (0.25 in this case).

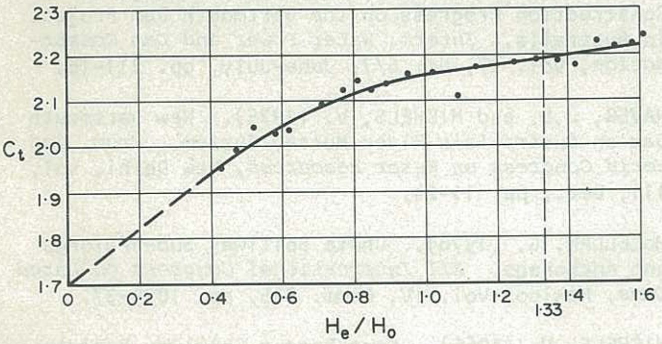

Figure 5 Variation of discharge coefficient C_t with H_e/H_o - comprehensive spillway model (1:60)

The variation of C_t with H_e/H_o is shown in Fig. 5. Presented in Fig. 6 are the experimental points and the theoretically derived rating curve. The latter was based on the practice of the Bureau of Reclamation (1973) excepting that an "equivalent" total length of a rectangular weir, $L_t = L + jzH_e$, was used, where j is a correction factor to allow the appropriate rectangular-weir discharge coefficient C to be used for the flows adjoining the sloping end walls, so that

$$Q = CL_tH_e^{1.5} \qquad . \qquad (3)$$

Assuming that each triangular area bounded by the batter wall and the vertical plane through its intersection with the crest, carries critical-depth flow at gradually varying head over the longitudinally sloping broad-crested weir formed by the wall, the theoretical total discharge Q_b for both sides would be

$$Q_b = 0.8\ C_b z H_e^{2.5} \qquad (4)$$

where C_b is the discharge coefficient for a broad-crested weir.

However, the flow between vertical planes through the crest end, and the intersection of the batter wall and the approach channel invert, respectively, changes gradually from broad-crested to ogee-profile conditions, while P increases from 0 to full height; the discharge coefficient was assumed to diminish linearly. Equating the sum of the component discharges to Eq. (3), it was found that

$$j = 0.8\ \frac{C_b}{C} - \frac{P}{H_e}\left\{ 1 - \frac{C_b}{C} \right\} \qquad . \qquad (5)$$

Over the range ($0.25 < H_e/H_o < 1.33$), j is almost constant (0.52 ± 0.02). Hence, in practice,

$$L_t = L + 0.2\ zH_e \qquad . \qquad (6)$$

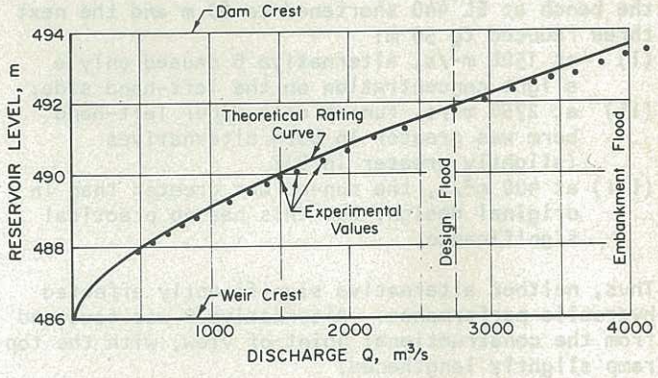

Figure 6 Spillway discharge rating - theoretical curve and experimental results

5.3.3 Chute and cascade

The original design and various modified layouts were tested over the full range of discharges. Observations showed that small flows approached the right-hand side of the cascade, but without any concentration at the side batters. At the maximum discharges, the flow concentrated at the left-hand batters, with run-up onto adjoining berms.

The modified arrangements tested, included:

a) Variation of axial lengths of quarry benches;
b) Sloping each bench at 1:10 towards the axis for a width of 40 m on both sides;
c) Sloping the chute 1:5.6 to a bench 20 m long and on a grade of 1:100 downstream, followed by a drop of 13.8 m to the top bench;
d) Sloping the chute 1:3 to EL 457, then a horizontal 3-m sill and a 1-m drop to the top bench.

Modifications (a) and (b) did not significantly improve flow distribution or reduce run-up. Alternative (c) caused an implosion of entrained air bubbles, likely to cause large pressure fluctuations and high dynamic loading. Arrangement (d) gave an acceptable flow distribution (Fig. 7).

In the final arrangement, the run-up onto the left-hand berms observed at high flows was inhibited by extending the bench access ramps upstream, and this also ensured end aeration of the overfall nappe.

The chute under-drains will discharge through two horizontal outlet pipes set in the end step. Negative pressures under the nappe beyond the end step of the chute ranged up to 20 kPa. Several aeration measures to prevent rock erosion by the water jet immediately below the chute exit were tried; side wall deflectors were finally adopted.

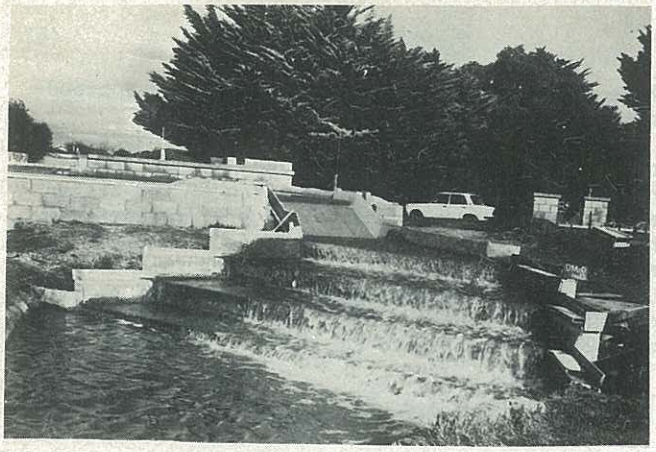

Figure 7 Comprehensive model (1:60) - design flood overflow (2750 m^2/s)

The supplementary tests (para 4.4) showed that with the bench at EL 440 shortened to 60 m and the next three reduced to 50 m:

(i) at 1500 m^3/s, alternative B caused only a slight concentration on the left-hand side;

(ii) at 2750 m^3/s, run-up onto upper left-hand berm was greater in both alternatives (slightly greater in B);

(iii) at 400 m^3/s, the run-up was greater than in original design, but this has no practical significance.

Thus, neither alternative significantly affected hydraulic performance. Alternative B was favoured from the constructional point of view, with the top ramp slightly lengthened.

6 CONCLUSIONS

a) The basic concept and hydraulic design were verified by model tests and supplementary calculations.

b) The model tests indicated several minor modifications for improved hydraulic performance:

   i) Smooth transitioning of change in batter slope at approach channel entrance.

   ii) Horizontal step and side deflectors at chute exit to throw jet away from unlined rock bench and batters and to aerate nappe.

   iii) Modifications to ramps and axial lengths of upper cascade benches to improve flow distribution and avoid excessive run-up.

c) A satisfactory agreement was obtained between the experimental and theoretical spillway discharge coefficients and rating curves.

d) The cascade layout met the requirements of dam safety, quarrying operations, hydraulic performance, maintenance access, batter stability.

e) The weir model tests established the correlation between discharge coefficient and downstream slope (tangential to crest profile).

7 ACKNOWLEDGEMENTS

The basic layout of the spillway and cascade was developed by the Snowy Mountains Engineering Corporation, design consultants to the Commission.

The Commission assumed direct responsibility for detailed spillway design and the hydraulic model studies.

Mr. G. McLellan, Designing Engineer, carried out the hydraulic and structural designs, and Mr. C. J. Barry, Resident Engineer at the Commission's Hydraulic Experimental Station, Werribee, supervised the model testing work.

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