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Hydraulic behaviour of the outlet of Lake Wakatipu, Central Otago, New Zealand

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Abstract

Outflows from Lake Wakatipu are normally controlled by the geometry of the gated weir across the outlet completed in 1926 and by the hydraulic characteristics of the Kawarau River, including the delta of the Shotover River, which joins the Kawarau River 3 km downstream of the outlet. On rare occasions, outflow from the lake is reversed when a large flood occurs in the Kawarau catchment and the peak response of Lake Wakatipu lags that of the Shotover River by several days. This paper describes a re-evaluation of the discharge rating for the modified lake outlet that takes account of the reverse flow phenomenon and the gradual drowning of the outlet weir leading up to the occurrence of reverse flow. The revised outlet rating utilises measurements of Lake Wakatipu level (NIWA site no. 75277) and the downstream Kawarau at Frankton river level (NIWA site no. 75263) and is based on standard hydraulic principles of weir behaviour.

Introduction

The natural outlet to Lake Wakatipu was modified in 1926 (Miller, 1949) with the completion of a gated weir structure across the Kawarau Falls (Figs. 1a and 1b). The structure was constructed for the Kawarau Gold Mining Company to dam the lake outlet to facilitate the mining of gold from the downstream bed of the Kawarau River. The cost of construction greatly exceeded the initial estimate, leaving only a small fraction of the original funds available to complete the rest of the scheme. The scheme promoters also failed to appreciate the role of the Shotover River delta in controlling downstream river levels. When

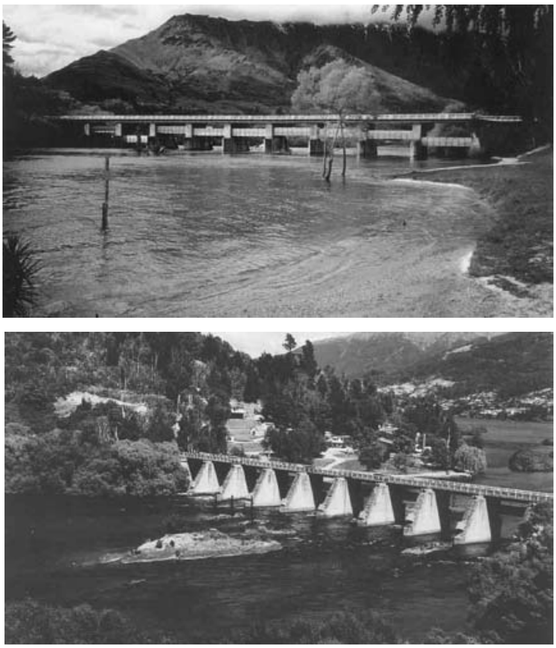

Figure 1 – Views of the Lake Wakatipu outlet weir at Kawarau Falls looking (a) downstream and (b) upstream towards the structure.

the outlet weir gates were closed for the first time, the water in the river did not drain away as expected but remained as a large still lake. Although a modest amount of gold was won by downstream claimholders during subsequent gate closures, these and other factors proved a fatal setback to attracting additional capital to continue the scheme. The company was wound up and ownership of the structure was eventually transferred to the Kawarau Dam Board. The structure continued to be used as a bridge on the Queenstown-Kingston Road.

The gated weir structure across the lake outlet remains today, with the gates occasionally operated for short periods (1-2 days) to maintain lake levels during low inflow conditions and preserve navigability in Queenstown Bay. Normally, however, the gates are kept fully open. Prior to 1972, there were periods when the gates were used to control the lake outflow for power development (Waugh *et al.*, 2000).

To reduce the flood risk to Queenstown and other lakeshore locations, the Queenstown Lakes District Council applied for resource consents in 2003 to lower part of the sill of the outlet weir, which would deepen the channel upstream and downstream. The Council and its technical advisors considered that the proposed sill lowering and channel deepening would allow larger outflows under extreme flood conditions, thereby reducing peak flood levels. As part of a review of these flood mitigation proposals, the discharge rating for the outlet weir was re-evaluated using flow-gauging data collected since 1971.

Description of Wakatipu and Shotover catchments

Lake Wakatipu is a natural lake occupying a deep glacial basin formed during the Pleistocene ice age. It drains a 3,133 km^2 catchment, which extends in the northwest to the South Island Main Divide (Fig. 2). The catchment includes sizeable areas of glacial ice and permanent snowfields in the catchment of the Dart River, which enters the northern end of the lake. The upper catchment area is subject to frequent northwesterly storms, which provide most of the rainfall input to the catchment and produce the majority of floods.

The Shotover River enters the Kawarau River approximately 3 km downstream of the outlet from Lake Wakatipu. It drains a 1,088 km^2 catchment immediately to the east and parallel to the northern half of the

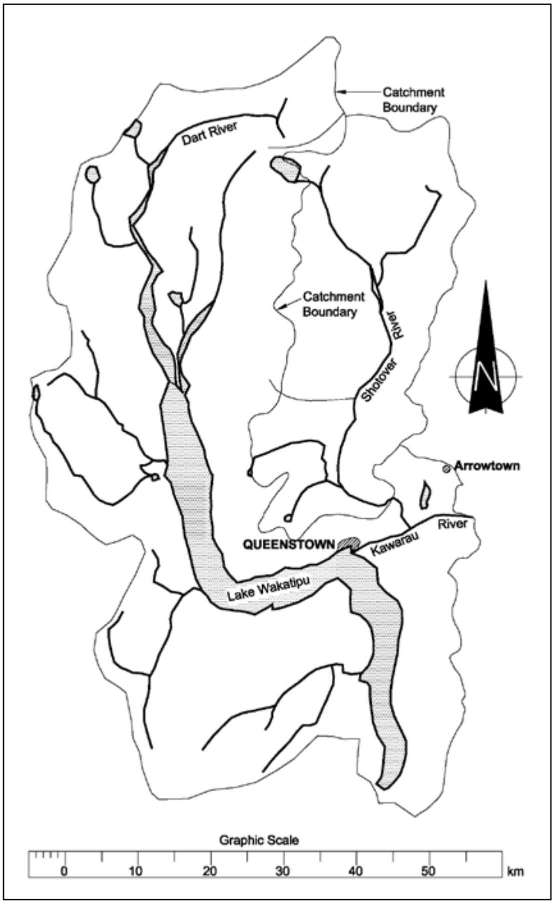

Figure 2 – Map showing the catchments of Lake Wakatipu and the Shotover River.

catchment for Lake Wakatipu. The Shotover catchment extends north to just short of the South Island Main Divide (Fig. 2). It too is affected by rainfall from northwesterly storms spilling over the divide. The river transports a heavy gravel bed load, which is principally due to erosion in the catchment related to '*the widespread distribution of pelitic schists and unconsolidated surficial deposits*' along with '*high rainfall, high altitudes and very steep slopes*' (National Water and Soil Conservation Organisation, 1977).

The bed load material transported by the Shotover River has formed a large delta at its confluence with the Kawarau River. The outer edge of the delta is eroded when there is a high differential gradient between the level of Lake Wakatipu and the level of the Kawarau River past the delta. During large floods in the Shotover River, the rapid deposition of gravel on the delta can partially block off the flow in the Kawarau River, with Shotover water flowing back into the lake.

Backflow into Lake Wakatipu from the Shotover River is not a recent phenomenon. Gilkison (1930) reported its occurrence during the large historic 1878 flood: *'the Shotover River, boiling down in flood to its junction with the Kawarau, stopped the outflow from Lake Wakatipu, and actually flowed into that lake'*. Similar occurrences, with 'dirty Shotover water' observed flowing back into the Frankton Arm of Lake Wakatipu, were reported for floods in 1957 and 1968 and also several times recently, including January 1994 and November 1999.

Hay's (1904) report on hydropower potential throughout New Zealand documented the very small gradient between the outlet of Lake Wakatipu and the Shotover delta, which helps create the conditions for backflow to occur.

Backflow events in major floods delay the outflow peak from Lake Wakatipu by around two days and cause more water to be stored in Lake Wakatipu. This is a natural phenomenon, with important ramifications for downstream flooding. The Kawarau River Water Conservation Order specifically recognized backflow as a special natural feature of the river and sought to protect it as part of the 'natural river system'.

Description of the gated weir structure

The existing gated weir structure across the lake outlet at the Kawarau Falls consists of a concrete sill with ten 11-metre-wide bays, giving an effective total width of 110 m. The concrete sill has an average elevation of 308.83 m (mean sea level datum). The water level in the lake is measured by a recorder (NIWA site no. 75277) about 500 m upstream of the outlet weir in the Frankton Arm, just beyond the boundary of the Kawarau Falls Motor Camp. The water level in the Kawarau River is measured by another recorder (NIWA site no. 75263) a similar distance downstream of the weir. From the water-level records for the two recorders, the upstream headwater depth relative to weir crest level h_1 and the downstream tailwater depth h_2 can be determined.

The structure incorporates ten vertical lift gates that can be operated to maintain lake levels during periods of low inflow, so that Queenstown Bay remains navigable for the tourist steamer *Earnslaw*. The gates are operated by lifting gear on the deck of the structure, which also functions as a bridge crossing over the Kawarau River for State Highway 6 between Frankton and Invercargill. Usually only some of the gates are closed, as jet boats carrying tourists regularly pass through the structure going both down river and up river on the return journey. Under flood conditions the gates are lifted well clear of the water in order to maximise the discharge capacity of the outlet structure.

The sill of the existing outlet structure consists of a horizontal concrete slab approximately 1 m wide founded on rock. The lake bathymetry upstream of the outlet sill and the river bathymetry downstream are both very irregular, with exposed and submerged rock islands. Flow depths below sill level generally exceed 1.8 m within about 20–40 m upstream of the outlet structure and 20–60 m downstream. The sill of the outlet structure approximates a broad-crested weir with the gates fully open (lifted clear of the water).

General hydraulic behaviour of the weirs

Figures 3(a-d) show a sequence of illustrations of a broad-crested weir with varying downstream conditions to demonstrate the general hydraulic behaviour of such a weir.

Figure 3(a) shows flow passing over a broad-crested weir, with the downstream tailwater level having no influence on the discharge over the weir (termed an undrowned condition). Under such conditions the discharge Q is related to the total upstream head relative to weir sill level H_1 by the equation

$$Q = (2/3)^{3/2} C_p b \sqrt{g} H_1^{3/2} \tag{1}$$

where C_p is a coefficient of discharge depending on the weir geometry (Ackers *et al.*, 1978), g is the gravitational acceleration and b is the width across the channel of the weir crest. If the approach flow depth is large enough for velocity head effects to be negligible, then the total upstream head H_1 in Equation (1) is equal to the upstream depth of water relative to weir sill level h_1. The discharge rating curve defined by Equation (1) is termed a free-discharge rating.

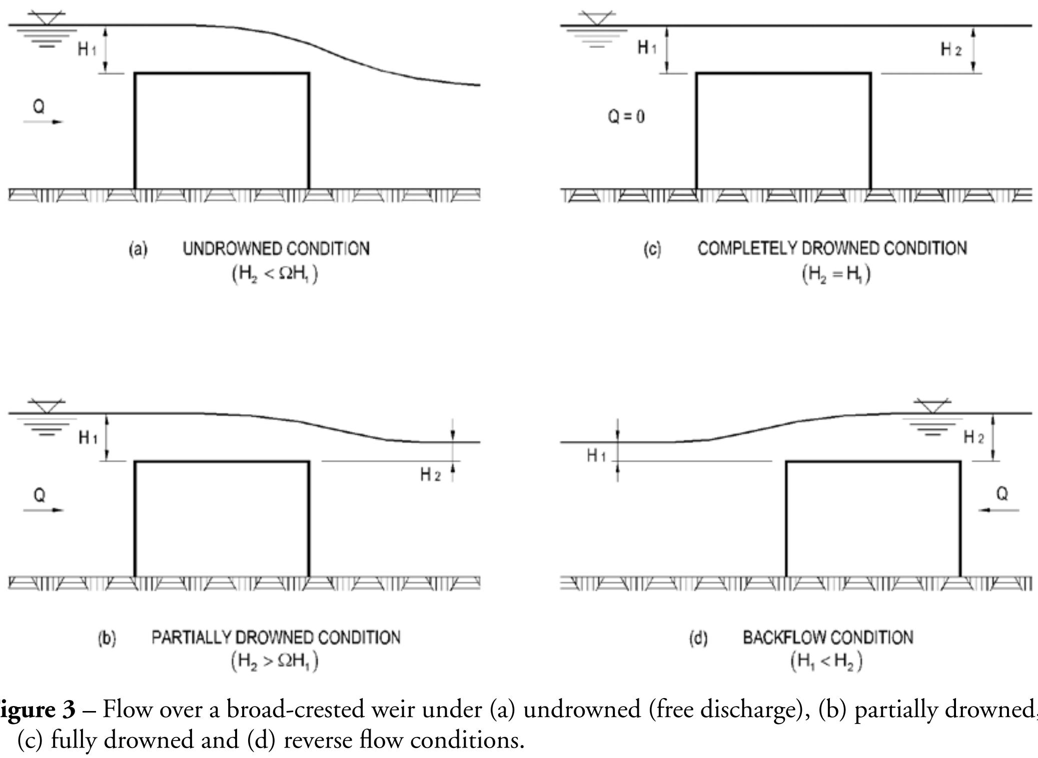

Figure 3 – Flow over a broad-crested weir under (a) undrowned (free discharge), (b) partially drowned, (c) fully drowned and (d) reverse flow conditions.

For low values of h_1 (or H_1) and large approach flow depths, the coefficient of discharge for the weir has a base value of C_p = 0.848 (Ackers *et al.*, 1978).

Figure 3(b) show the same weir with the downstream tailwater level relatively high due to a hydraulic control further downstream. The high tailwater level in this case reduces the discharge over the weir. This is termed a partially drowned condition, with the discharge Q given by a modified form of Equation (1) for undrowned flow conditions:

$$Q = (2/3)^{3/2} f\, C_p b \sqrt{g} H_1^{\,3/2} \qquad (2)$$

In Equation (2), f is a flow reduction factor, which is a function of the modular ratio H_2/H_1, H_2 being the downstream total head relative to the weir sill level (which also is equal to the downstream depth of water relative to the weir sill level h_2 if velocity head effects are negligible):

$$\begin{aligned} f &= function\left(H_2/H_1\right) \; for\; H_2/H_1 > \Omega \\ f &= 1 \; for\; H_2/H_1 \le \Omega \end{aligned} \qquad (3)$$

where Ω is the modular limit.

The modular limit Ω denotes the limiting value of the modular ratio H_2/H_1 for the onset of partial drowning of the weir. It is typically defined numerically as the modular ratio H_2/H_1 that causes a one per cent reduction in the value of the undrowned flow over the weir (Ackers *et al.*, 1978). Ackers *et al.* (1978) indicate that the modular limit Ω has a value of about 0.7–0.85 for a rectangular profile or broad-crested weir (this compares with a value of about 0.75 for a triangular profile weir). In other words, the tailwater depth H_2 over the weir sill has to be more than 70-85% of the headwater depth H_1 before the free discharge over the weir starts to be reduced. For tailwater depths of less than 70-85% of the headwater depth, the weir overflow is undrowned.

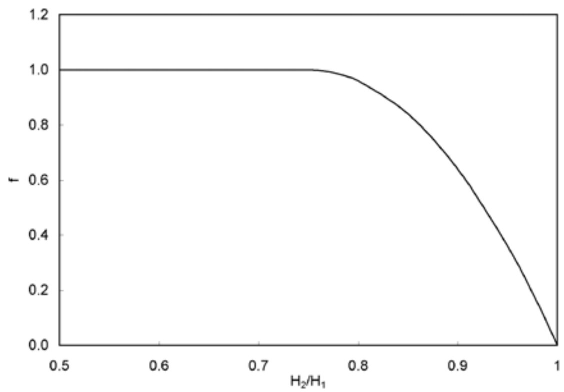

Figure 4 – Typical reduction factor curve for a partially drowned weir

The reduction factor f for partially drowned flow conditions typically has the form of the curve shown in Figure 4. This curve reduces asymptotically from a value of f = 1 as the modular ratio H_2/H_1 increases beyond the modular limit **Ω**, and then drops steeply towards zero as the modular ratio H_2/H_1 tends towards a value of 1.

At a modular ratio value of $H_2/H_1 = 1$, all flow over the weir ceases. This is illustrated in Figure 3(c). The downstream tailwater depth over the weir sill H_2 equals the upstream headwater depth H_1, so that the discharge Q is zero. This is termed a fully drowned condition.

If the downstream tailwater depth over the weir sill H_2 exceeds the upstream headwater depth H_1, so that the modular ratio H_2/H_1 is greater than 1, then back or reverse flow occurs. This is illustrated in Figure 3(d). In this case the discharge Q over the weir is defined as negative, as it occurs in the negative (upstream) direction.

Hydraulic behaviour of the Lake Wakatipu outlet

The Lake Wakatipu outlet weir exhibits all four types of flow behaviour shown in Figures 3(a)-(d). In this case, the combination of the downstream constriction formed by the Shotover delta and high Shotover River flows can act as a hydraulic control under flood conditions affecting Lake Wakatipu outflows over the outlet weir.

Figure 5(a) shows a diagram of the outlet weir in an undrowned condition, with the modular ratio of the tailwater depth to the headwater depth H_2/H_1 less than the modular limit for the onset of partial drowning. Figure 5(b) shows the outlet weir in a partially drowned condition. In this condition the constriction formed by the Shotover delta, combined with high Shotover River flows entering the Kawarau River, induce high tailwater depths H_2 below the weir, such that the modular ratio H_2/H_1 is greater than the modular limit defining the onset of partial drowning. Figure 5(c) shows the outlet weir in a fully drowned condition. This occurs when the Shotover River flows entering the Kawarau River peak during a major flood before Lake Wakatipu, with its very large surface area, reaches a peak level. In the same flood event, the lake outlet may also experience back or reverse flow, as shown in Figure 5(d). In this situation the water level in the Kawarau River at Frankton rises above the level of Lake Wakatipu so that water flows from the Kawarau River back into the lake.

Revised discharge rating for the Lake Wakatipu outlet

All flow gauging data measured since the 1960s and the corresponding lake level and dowstream river level measurements were assembled. Historically there has been some difficulty obtaining flow gauging data at high lake levels at the lake outlet, but this has improved in recent years, with one flow gauging of 766 m^3/s obtained during the very large November 1999 flood event.

The assembled flow gauging and level data were first examined by plotting the gauged flow Q values against the total upstream head $H_1^{3/2}$ to determine when the data departed from the straight-line relationship implied by Equation (1), and hence when the lake outflow was undrowned and when it was partially drowned. It was apparent that partially drowned outlet conditions occurred when the lake outflow exceeded

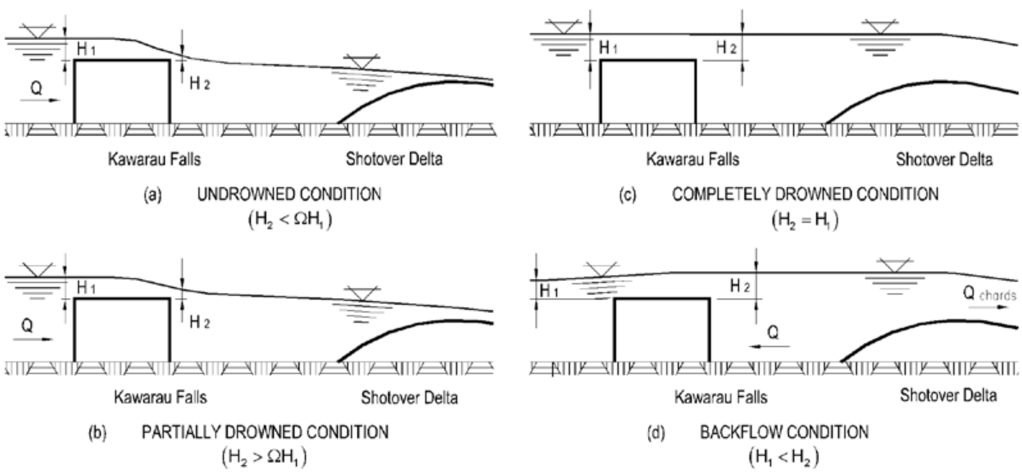

Figure 5 – Flow out of Lake Wakatipu under (a) undrowned (free discharge), (b) partially drowned, (c) fully drowned and (d) reverse flow conditions.

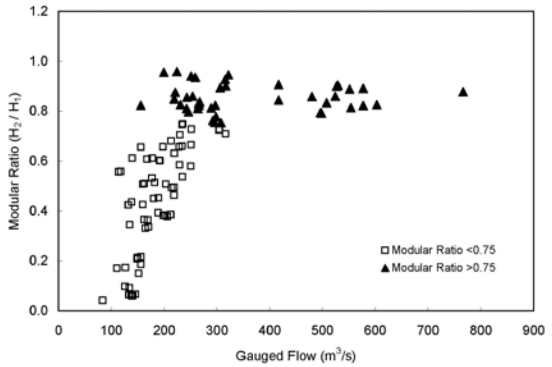

Figure 6 – Modular ratio as a function of gauged flow for the Lake Wakatipu outlet weir.

approximately 250 m^3/s. The modular limit Ω was then determined by plotting the modular ratio H_2/H_1 (as determined from the lake level and downstream river level measurements) as a function of the gauged flow Q values (Fig. 6). Plotting the data in this form confirmed there is a change in flow behaviour between lake outflows of less than and greater than about 250 m^3/s.

Although there is a fair amount of scatter in the data, Figure 6 indicates that, for lake outflows less than about 250 m^3/s, the modular ratio H_2/H_1 gradually increases with discharge up to a value of about 0.75. Under such conditions the lake outlet structure is undrowned. For lake outflows greater than about 250 m^3/s, the modular ratio H_2/H_1 varies in the range 0.8–1.0, irrespective of the outflow value. Under such conditions the lake outlet structure is partially drowned. The value of the modular limit defining the transition between undrowned and partially drowned conditions for the lake outlet structure is approximately Ω = 0.75, which is consistent with laboratory measurements for rectangular-profile or broad-crested weirs (Ackers *et al.*, 1978).

The plot of modular ratio H_2/H_1 against lake outflow Q (Fig. 6) does not imply any functional relationship between them. Although the total upstream head H_1 is a function of Q, the downstream total head H_2 is generally independent of Q since it is determined by the sum of Q and the value of the Shotover River flow (as well as the geometry of the Kawarau River down to the Shotover River delta). When the Shotover River flow is small relative to the lake outflow Q (i.e., the outlet weir is undrowned), H_2 is approximately a function of Q. The plot of modular ratio H_2/H_1 against lake outflow Q (Fig. 6) is therefore merely a device to roughly determine when two different weir flow regimes (undrowned and partially drowned flow) occur.

The gauging data with modular ratio values of $H_2/H_1 \leq 0.75$ (undrowned conditions) were used to define a weir rating of the form $Q = a\, H_1^{3/2}$ (this is consistent with the form of Equation (1) for a broad-crested weir). When the data were plotted in this form (Fig. 7) it was clear that there had been an apparent shift in the rating over time, possibly due to a change in the upstream bathymetry approaching the outlet weir or to some other factor that we were unaware of. For this reason a best-fit trend

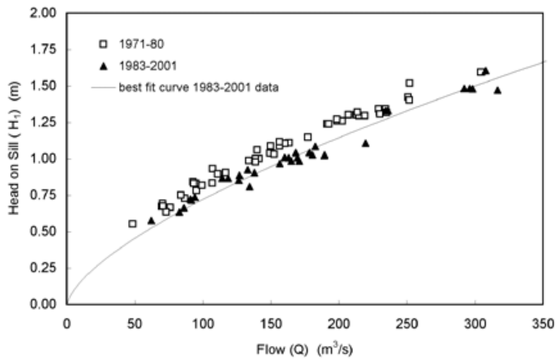

Figure 7 – Discharge rating for the Lake Wakatipu outlet weir under free discharge conditions.

line was fitted to the 1983–2001 data, resulting in the equation:

$$Q = 163.18 H_1^{3/2} \qquad (r^2 = 0.9682) \qquad (4)$$

The coefficient in Equation (4) is very close to the value of 159.0 obtained if the overall weir width b = 110 m and the base discharge coefficient value of C_p = 0.848 are substituted into Equation (1).

Equation (4) (the best-fit trend line, Figure 7) was used to calculate $Q_{undrowned}$ values for the second subset of gauging data points with modular ratio values of H_2/H_1 > 0.75. The $Q_{undrowned}$ values were then used to calculate drowned flow reduction factor f values where

$$f = Q_{gauged} / Q_{undrowned} \qquad (5)$$

These were plotted as a function of the modular ratio H_2/H_1 in Figure 8.

A best-fit curve of the form

$$f = \{a[1-(H_2/H_1)^2]\}^b \qquad (6)$$

was fitted to the 1983–2001 data in Figure 8. This curve has a value of f = 0 for a modular ratio value of H_2/H_1 =1 while, to ensure that it also has a value of f = 1 for the modular limit

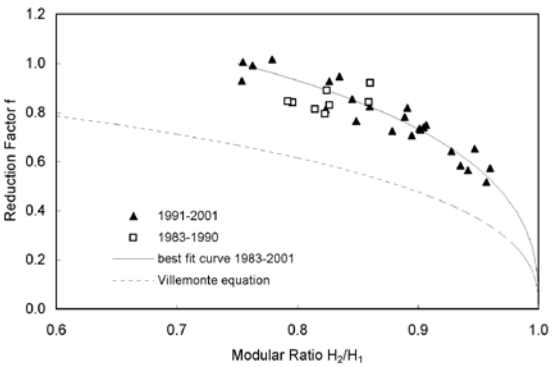

Figure 8 – Flow reduction relationship for the Lake Wakatipu outlet weir under partially drowned conditions.

of Ω = 0.75, it is necessary for the coefficient a to have the value 16/7. The value of the exponent b implied by the data in Figure 8 is 0.3725, so that Equation (6) then becomes:

$$f = \{(16/7)[1-(H_2/H_1)^2]\}^{0.3725} \qquad (7)$$

The best-fit curve defined by Equation (7) is tabulated as a function in Table 1.

Table 1 – Drowned flow reduction factor f as a function of the modular ratio H_2/H_1 for partially drowned conditions

H_2/H_1	f
0.00	1.0000
0.75	1.0000
0.78	0.9596
0.80	0.9299
0.82	0.8978
0.84	0.8628
0.86	0.8243
0.88	0.7814
0.90	0.7329
0.92	0.6771
0.94	0.6107
0.96	0.5271
0.98	0.4087
0.99	0.3163
0.999	0.1344
0.9999	0.0570
1.0000	0.0000

The curve defined by Equation (7) in Figure 8 and the tabulated data in Table 1 are similar in form to the empirical Villemonte equation (Ackers *et al.*, 1978) used for estimating the reduction in flow due to partial drowning of a thin-plate or sharp-crested weir:

$$Q_{partially\ drowned} / Q_{undrowned} = [1-(H_2/H_1)^n]^{0.385} \qquad (8)$$

where the value of the exponent n varies, depending on the shape of the weir. For a rectangular thin-plate weir, n has a value of 1.5. The curve defined by Equation (8) is also shown in Figure 8. IAHR (1994) gives a very similar curve (albeit defined by a different analytical equation) for a sharp-crested triangular block weir that compares favourably with the experimental data of Abou-Seida and Quaraishi (1976). The reduction factor curve defined by Equation (8) lies below that defined by Equation (7), indicating that a sharp-crested weir is more sensitive to the effects of partial drowning than a rectangular-profile or broad-crested weir.

Examination of bathymetric profiles through each bay of the outlet weir showed that the bathymetry upstream and downstream of the outlet 'weir' has a highly irregular form and cannot readily be approximated by any typical profile. Average bed levels upstream and downstream of the structure are similar. It was thus reasonable to assume that the same undrowned weir flow rating (Fig. 7) and the drowned flow reduction factor relationship (Fig. 8) would apply as a first approximation for reverse flow, when water flows from the Kawarau River back into Lake Wakatipu. The only differences are that the undrowned weir rating is expressed in terms of H_2 rather than H_1 ($Q = 163.18\ H_2^{3/2}$) and the drowned flow reduction factor relationship is expressed in terms of H_1/H_2 rather than H_2/H_1 ($f = \{(16/7)[1-(H_1/H_2)^2]\}^{0.3725}$).

Figure 9 shows the effect of partial drowning of the outlet weir on the undrowned weir rating (Equation (4)), based on an average downstream tailwater level rating for the Kawarau at the Frankton water level recorder. The amount of flow reduction due to partial drowning gradually increases as the lake outflow increases above 250 m^3/s. In practice the downstream rating varies due to changes in bed level past the Shotover delta. Figure 10 shows the effect of different

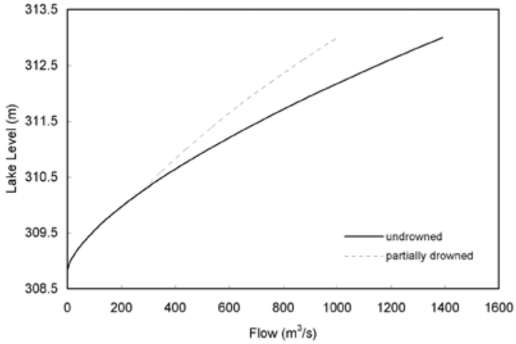

Figure 9 – Effect of partial drowning of the Lake Wakatipu outlet on the weir discharge rating, based on an assumed tailwater rating curve.

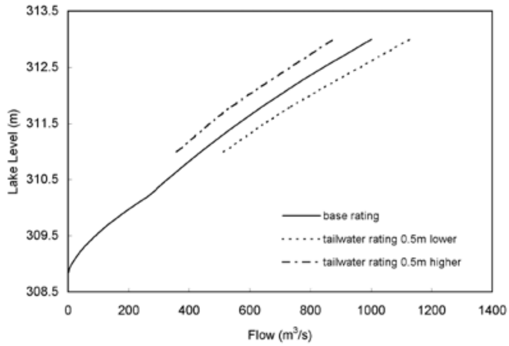

Figure 10 – Discharge rating for the Lake Wakatipu outlet weir for variable tailwater level conditions.

bed level conditions with assumed tailwater level variations of ±0.5 m compared to the average rating. Actual tailwater variations of up to +1.0 m and -0.4 m have been observed in the past from gauging data.

Application of the revised discharge rating

The revised discharge rating for the Lake Wakatipu outlet was applied to the very large November 1999 flood, when water from the Kawarau River backflowed into the lake. Figure 11 shows the resulting discharge hydrograph. Three-hourly average lake level and downstream river level data were used to

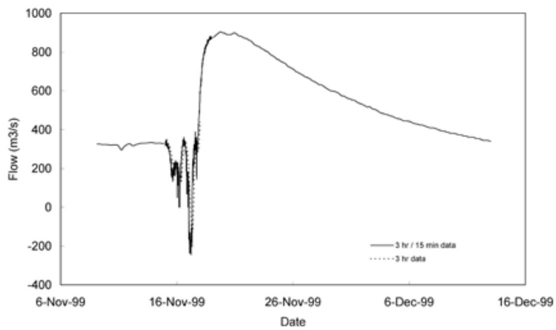

Figure 11 – Lake Wakatipu outflow hydrograph for the November 1999 flood, based on three-hourly average data prior to midnight on 15 November and from midnight on 19 November, and on both 15-minute instantaneous and three-hourly average data for the four-day period between 15 and 19 November.

compute this hydrograph from midnight on 9 November to midnight on 15 November prior to the flood, and from midnight on 19 November to midnight on 13 December over the peak of the event and during the recession. For the four-day period from midnight on 15 November to midnight on 19 November when the backflow into the lake occurred, 15-minute instantaneous lake level and river level data were used to compute the discharge hydrograph. Some isolated data points were smoothed manually to remove spikes from the outflow record over this four-day period. These spikes were probably due to the effects of either lake seiching or surging in the river downstream.

Prior to the flood, the lake outflow was steady at about 320 m^3/s. Over the four-day period during which backflow occurred, the lake outflow record was quite irregular, with several localised peaks and troughs. The first trough occurred about 3.30pm on 15 November when the outflow dipped to about 130 m^3/s. Thereafter the outflow peaked again just over 2.5 hours later at 6.00pm (at approximately 230 m^3/s). The outflow then dipped again to about zero at 5.00am the following morning, 16 November, before rising back to a maximum of about 350 m^3/s at 12.45 pm that same day. The outflow from the lake reversed just after midnight on 17 November and remained negative for about 7 hours. It reached a maximum negative value of about -240 m^3/s at 5.00am.

Figure 11 also shows the three-hourly average outflow record over the period of backflow into the lake from midnight on 15 November to midnight on 19 November plotted for comparison with the corresponding 15-minute instantaneous outflow record. The three-hourly average outflow record is smoothed significantly, with the maximum negative outflow reduced to -130 m^3/s.

The maximum positive lake outflow for the flood was about 905 m^3/s and occurred at 6.00pm on 19 November. Thereafter the lake outflow declined very gradually over time.

Figure 12 shows a complementary plot in which lake outflow is plotted against lake level from midnight on 15 November through to 12 December. Over the six days prior to 15 November, the lake level and outflow were relatively constant at about 310.55 m

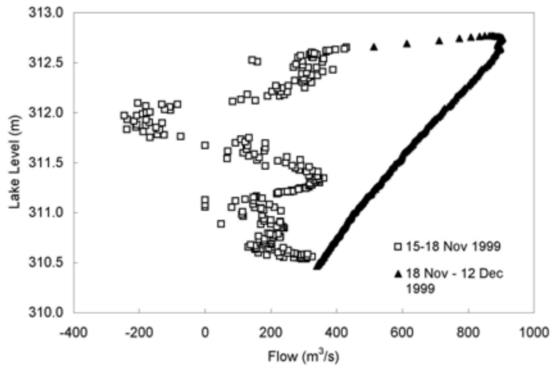

Figure 12 – Lake Wakatipu outflow as a function of lake level for the November 1999 flood.

and 325 m^3/s respectively, which marks the starting point for the data points plotted in Figure 12. The open square symbols indicate the fluctuating lake outflow (including the occurrence of reverse flow) over the period 15-18 November as the lake level gradually rose to about 312.65 m. The closed triangle symbols indicate the peak of the flood event and the subsequent recession over the period 18 November to 12 December.

The revised discharge rating for the Lake Wakatipu outlet was also applied to the long-term lake level and downstream river level records (three-hourly average data) for the period 1963–2004, even though the rating is strictly valid only since 1983 (see Fig. 7). The original long-term lake outflow record (1963–2003) in Figure 13(a) is compared with the revised lake outflow record shown in Figure 13(b). Comparison of these two records indicates that the original outflow record overestimates flood peaks larger than about 500 m^3/s and underestimates flood peaks less than 500 m^3/s. It also shows no backflows into the lake, although lake outflows are set to zero whenever the downstream river level exceeds the lake level. In contrast the revised outflow record (Fig. 13b) shows that backflow from the Kawarau River into Lake Wakatipu has occurred on 26 occasions since 1963. Table 2 summarises the dates of these backflows, their duration and the maximum

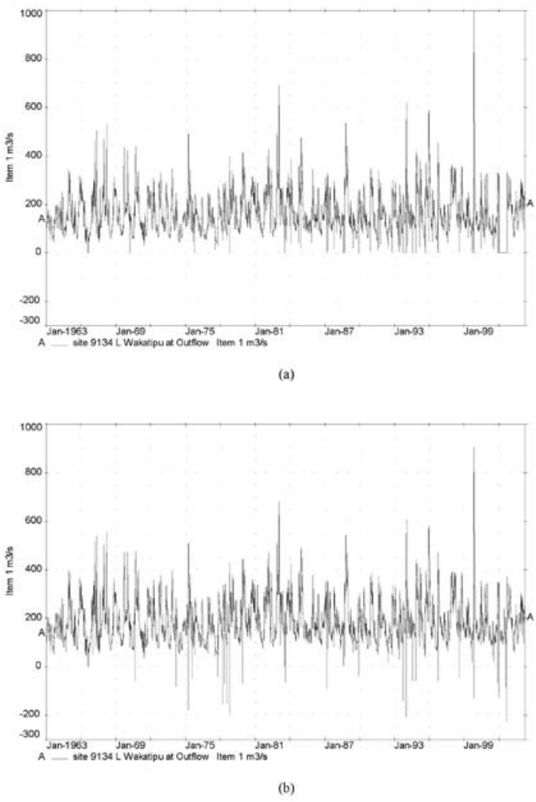

Figure 13 – Lake Wakatipu outflow record for the period 1963-2004: (a) original record and (b) revised record.

Table 2 – Summary of backflow events from the Kawarau River into Lake Wakatipu (1963-2004)

Date	**Duration (hrs)**	**Maximum Backflow (m3/s)**
28 August 1970	3	-59
15 March 1974	6	-83
30-31 March 1975	15	-178
5-6 August 1975	6	-48
23-24 September 1977	9	-41
27 March 1978	9	-153
12 May 1978	6	-70
10-11 August 1978	18	-154
13 August 1978	15	-196
2 December 1979	3	-66
31 July-1 August 1983	9	-64
10 March 1987	3	-90
15 December 1989	3	-36
5-6 October 1993	15	-140
9 January 1994	18	-207
14 August 1994	6	-60
7 November 1994	3	-60
7 October 1996	3	-59
22 July 1998	3	-45
17 November 1999	6	-130
19 November 2001	6	-67
4 January 2002	6	-126
13-14 August 2002	15	-84
18-19 September 2002	30	-225
29-30 December 2004	9	-56
31 December 2004	3	-32

negative flow into Lake Wakatipu from the Kawarau River. Backflow durations varied between 3 and 30 hours. The largest and longest backflow occurred on 19 September 2002, with a maximum three-hourly average discharge of -225 m^3/s and a duration of 30 hours.

Conclusions

Standard hydraulic principles have been used to develop a model of the behaviour of the weir controlling the outflow from Lake Wakatipu. The model has been calibrated against measured lake level, downstream river level and flow data. It satisfactorily describes undrowned, partially drowned, completely drowned and reverse flow behaviour of the outlet weir.

The model has been utilised to calculate a revised long-term record for Lake Wakatipu outflow. This shows that, in the period 1963-2004, backflow into the lake from the Kawarau River due to high Shotover River flows occurred on 26 occasions. The duration of backflow has typically been in the range of 3–30 hours. The maximum negative three-hourly average flow into the lake of -225 m^3/s occurred on 19 September 2002.

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