Report

Matahina Hydroelectric Power Scheme Reconsenting Project: River Hydrology, Hydraulics and Bank Erosion

Prepared for HOBEC on behalf of TrustPower Limited (Client)

By Beca Infrastructure Ltd (Beca)

May 2009

Revision History

Revision Nº	Prepared By	Description	Date
A	Peter Millais	Preliminary draft	11 September 2008
B	Peter Millais	Draft	24 October 2008
C	Peter Millais	Revised draft for consultation	7 November 2008
D	Peter Millais	Preliminary final draft	10 April 2009
E	Peter Millais	Final draft	20 April 2009
F	Peter Millais	Revised final draft	30 April 2009
G	Peter Millais	Final	13 May 2009

Document Acceptance

Action	Name	Signed	Date
Prepared by	Peter Millais	Peter Millais	13 May 2009
Reviewed and approved by	Graham Levy		13/5/2009
on behalf of	Beca Infrastructure Ltd		

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Executive summary

TrustPower Limited (TPL) owns and operates the Matahina Hydroelectric Power Scheme (Matahina HEPS) on the Rangitaiki River in the eastern Bay of Plenty region. The scheme consents expire in November 2009.

Beca Infrastructure Ltd has been commissioned by TPL to produce an assessment of the river hydrology, hydraulics, and bank erosion effects of the scheme. This technical report will form part of the Resource Consent Application and Assessment of Environmental Effects documentation.

Investigations undertaken

The scope of work outlined in this report covers the following matters:

- An overview of the hydrology in the Rangitaiki catchment;
- An assessment of the hydrology-related effects – both positive and adverse – of the Matahina HEPS;
- An assessment of the Matahina HEPS spillway capacity against design storm events;
- A preliminary assessment of the effects of a probable maximum flood (PMF) and of a dam break at the Aniwhenua HEPS on the Matahina reservoir and HEPS;
- A review of the July 2004 Rangitaiki flood event, including the operation of the Matahina HEPS and causes of flooding in Edgecumbe;
- An assessment of the causes of river bank erosion in the Rangitaiki River downstream of Matahina HEPS; and
- An assessment of the effects of a proposed revised operating regime.

The report assembles information from a number of previous studies that are listed in Section 11, in particular studies by Beca (2001 and 2005) concerning the effects of twin peak operation.

A separate hydrogeology and riverbank stability report is included as Appendix B.

Existing environment

The Matahina HEPS is located on the Rangitaiki River, which has a mean annual flow of 71 m^3/s, and an estimated 100-year return period flood of 780 m^3/s.

The Matahina dam is an 86 m high earth embankment completed in 1967. The dam forms the 8 km long Lake Matahina. The power station at the base of the dam generates up to 80 MW, using a daily peaking regime (single or twin peaks, depending on inflow). The existing consent limits the normal flow range downstream of the power station to between 22 MW (about 45 m^3/s) and 160 m^3/s. Flows outside this range are permitted during floods, and when the inflow is less than 40 m^3/s (see footnote[1]). No peaking is permitted when the river inflow is less than 40 m^3/s.

Downstream of the Matahina dam, the Rangitaiki River passes through a defined valley for 4 km then follows a meandering course across the Rangitaiki plains past Te Teko and Edgecumbe to the coast, 37 km downstream of the dam, see map. Stopbanks have been constructed downstream of Te Teko as part of a flood protection and drainage scheme managed by Environment Bay of Plenty (EBoP).

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[1] The existing consent assumes that the flow required to generate 22 MW is 40 m^3/s, which is inconsistent with operating experience, see Section 1.3.

A 'peaking regime' causes water level fluctuations of up to the order of one metre between the power station and Edgecumbe. Downstream of Edgecumbe the effect of peaking is less pronounced and tidal influences become more significant.

Rough running operating regime

The turbines have a "rough running" range at loads between 12 MW and 18 MW, when they are subject to cavitation damage. In order to avoid this rough running range, TPL reached an informal agreement with EBoP in June 2007 for a revised low flow operating regime under the following conditions:

- the revised regime only applies when reservoir inflows are below 22 MW (45 m^3/s);
- generation may drop below the rough running range at night, to allow generation above the rough running range during the day;
- the average generation outflow is to match the average reservoir inflow over 24-hours; and
- EBoP and Fonterra are to be advised at the start and finish of operation under the revised regime.

The minimum generation November 2007 to January 2008 was 10 MW, with a minimum flow of 25 m^3/s recorded at Te Teko.

Proposed operating regime

TrustPower is proposing a revised operating regime to provide for operational flexibility and to avoid the need to operate the turbines in their "rough running range" between 12 MW and 18 MW (29 to 41 m^3/s), which under the current consent conditions are required to operate at reservoir inflow rates. Features of the proposed operating regime are:

- the ability to respond more effectively to electricity demand;
- the ability to avoid the turbine rough running range;
- the ability to peak during low flow situations, although only minor peaks will be possible due to the limited lake storage;
- the ability to peak under low flow situations will in turn allow the lake level to rise under low flow situations;
- no limit on the number of peaks each day;
- peaks will be no greater in terms of magnitude, however the more peaks there are, the smaller each will likely be;
- reduced minimum flow;
- a self-regulating system, i.e. how the scheme is operated in the future is based on recent history;
- increased ramping rates.

Actual and potential environmental effects of current operation

River morphology effects that arise from the current operation of the scheme are listed below:

1. An increasing depth of sediment in the deep portion of Lake Matahina.
2. Trapping of sediment in Lake Matahina has halted or substantially reduced the historical rising river bed and coastal progradation of the Rangitaiki plains. This is a positive effect in that maintenance of the Rangitaiki-Tarawera Rivers Scheme would otherwise require regular dredging or stopbank raising to counter the effect of a rising river bed level. It should be noted, however, that the principal sediment capture that has reduced loads downstream has historically occurred at the upstream Aniwhenua dam.

3. Flood attenuation in Lake Matahina has caused significant reduction in the frequency and duration of flood peaks in the range 160-200 m^3/s downstream of the dam. This effect is also likely to apply to larger floods, though there are insufficient events with flow exceeding 300 m^3/s to draw statistically valid conclusions from the flow record. It is expected that this attenuation of larger floods results in reduced fluvial bank erosion compared to the natural river flow.
4. Bank erosion may be occurring during floods at the upstream end of the river reach below Matahina dam, as a result of the sediment deficit caused by sediment deposition in the lake.
5. Peaking operation creates a zone along the river banks where fluctuating river levels inhibit the establishment and maintenance of vegetative bank protection. This effect extends from the Matahina dam to Edgecumbe. The resulting lack of vegetation can allow bank erosion to occur during floods.

No mitigation is proposed for effects 1-3, which are positive.

Evidence for effect 4 is not conclusive. Any effect is minor and no mitigation is warranted, because if bank erosion is occurring as a result of the sediment deficit, then protecting the banks would merely transfer the problem elsewhere.

Effect 5 can be mitigated, as at present, by the contribution of TrustPower towards EBoP river bank protection costs.

Actual and potential environmental effects of proposed operation

Hydraulic effects of the proposed new operating regime are:

- The Matahina reservoir will generally be maintained at a higher level.
- Downstream of the dam there will be flow and river level variability at times of low reservoir inflow.
- The minimum output will be 10 MW, corresponding to a normal minimum flow of about 25 m^3/s. To allow for variations from the generation set point, TrustPower seeks a minimum consented flow of 20 m^3/s, except when reservoir inflow is less than 20 m^3/s. This compares to a normal minimum output of 22 MW (45 m^3/s) under the present consented operating regime, and a minimum of 10 MW (25 m^3/s) under the “rough running regime” agreed with EBoP for when reservoir inflow is less than 22 MW (45 m^3/s).

Approximately 25% of the time the low river flow will be less than under the present operating regime. The effect of lower flows on river level varies along the river. The greatest effect, with Matahina output reduced from 22 MW (45 m^3/s) to 20 m^3/s is a drop in water level of about 0.9 m at three locations. Downstream from the Edgecumbe earthquake fault the effect reduces gradually to zero at the river mouth.

At the shallowest points in the river (at Te Teko and at the rapids at the earthquake fault) the water depth is reduced from 1.5 to 0.9 and from 1.0 to 0.7 m respectively, for a flow reduction from 45 to 20 m^3/s. Elsewhere the water is generally much deeper.

The lower minimum flow in the river is expected to result in a longer and deeper saline wedge in the lower Rangitaiki River. This is unlikely to affect existing floating intakes (which draw from fresh water above the saline wedge), but will reduce the time during the tidal cycle that other takes downstream of Edgecumbe can abstract fresh water. This effect can be mitigated by the provision of floating intakes. Monitoring of salinity profiles during low flows is recommended.

Lower river levels and saline intrusion will adversely affect six of the 12 existing consented water intakes and an unknown number of permitted takes from the Rangitaiki River downstream of Matahina dam. These effects can be mitigated.

The lower water levels will have zero or negligible effect on adjacent wetland reserves as the only hydraulic connection with the river is close to the mouth, where water level changes due to the proposed regime are minimal.

There are no additional identified river morphology effects of the proposed new operating regime that differ from those of the existing operating regime. A single inspection by jet boat is proposed two years after the introduction of new operating conditions, to identify any significant change to the pattern or rate of erosion as a result of the changed operating conditions. The inspection should be from Matahina to Edgecumbe.

Conclusions

The river morphology effects of the current operation of Matahina HEPS are minor and can be mitigated. Observed bank erosion is predominantly the result of flood events, the frequency and duration of which have been reduced by attenuation in the Matahina reservoir. Operation is contributing to some extent to the erosion that occurs downstream, principally due to effects on riparian vegetation, and this can be addressed by TPL continuing to contribute to EBoP's river protection works programme.

The operation of the Matahina HEPS had no significant effect on the extent of flooding and bank erosion during the July 2004 flood.

There are no additional identified river morphology effects of the proposed new operating regime that differ from those of the existing operating regime. The effects on existing water intakes can be mitigated.

Given the minor effect of Matahina HEPS on river morphology downstream of the dam, there is no reason to continue the biennial monitoring required under the existing consent. A single inspection by jet boat is recommended two years after the introduction of new operating conditions, to identify any significant change to the pattern or rate of erosion as a result of the changed operating conditions. The inspection should be from Matahina to Edgecumbe.

Monitoring of salinity profiles in the tidal reach at low flows is recommended, to confirm the effects on existing water takes and the need for mitigation measures.

1 Introduction

1.1 Matahina HEPS

TrustPower Limited (TPL) owns and operates the Matahina Hydroelectric Power Scheme (Matahina HEPS) on the Rangitaiki River in the eastern Bay of Plenty region. The scheme consents expire on 30 November 2009.

The Matahina dam (completed in 1967) is located on the Matahina River 12 km upstream of Te Teko township, see Map 1. The power station has two turbines with a consented maximum generation discharge of 160 m^3/s. The scheme is operated as a 'peaking station', with up to two daily peaks and 40 m^3/s normal minimum flow, under an operating regime specified in Consent Nº 02 2195/1, refer Appendix A.

The Matahina dam is the largest earth embankment dam in the North Island, being 86 m high with a 76 m head behind the dam. Lake Matahina has approximately 8 km ponded length, including one kilometre in a narrow gorge downstream of the road bridge near the head of the lake. Its surface area is 2.3 km^2 at maximum normal operating level (76.2 m RL). The lake is an artificial gorge-type reservoir, 40-50 m deep near the dam, while the upper reaches of the lake are shallow (1 to 4 m).

Downstream of the Matahina dam, the Rangitaiki River follows a meandering course across the Rangitaiki plains past Te Teko and Edgecumbe to the coast, 37 km downstream.

1.2 This report

Beca Infrastructure Ltd has been commissioned by TPL to produce an assessment of the river hydrology, hydraulics, and bank erosion effects of the scheme. This technical report will form part of the Resource Consent Application and Assessment of Environmental Effects documentation.

The scope of work outlined in this report covers the following matters:

- An overview of the hydrology in the Rangitaiki catchment;
- An assessment of the hydrology-related effects – both positive and adverse – of the Matahina HEPS;
- An assessment of the Matahina HEPS spillway capacity against design storm events;
- A preliminary assessment of the effects of a probable maximum flood (PMF) and of a dam break at the Aniwhenua HEPS on the Matahina reservoir and HEPS;
- A review of the July 2004 Rangitaiki flood event, including the operation of the Matahina HEPS and causes of flooding in Edgecumbe;
- An assessment of the causes of river bank erosion in the Rangitaiki River downstream of Matahina HEPS; and
- An assessment of the effects of a proposed revised operating regime.

The report assembles information from a number of previous studies that are listed in Section 11, in particular studies by Beca (2001 and 2005) concerning the effects of twin peak operation.

A separate hydrogeology and riverbank stability report is included as Appendix B.

1.3 Conversion factors for flow and power output

Power output from the Matahina HEPS is measured in MW. The flow required for a given power output depends on the overall efficiency and the net head: it varies from about 1.75 m^3/s per MW at peak output with high lake levels to over 2.5 m^3/s per MW at low flows when the lake is near the bottom of the operating range, see Table 1.

Unless noted otherwise, equivalents quoted in this report (inside round brackets) are based on the mid-operating range (74.5 m RL lake level) values from Table 1. TrustPower has generally used an average value of 1.85 m^3/s per MW for compliance reporting. Other equivalents are used in the existing resource consent, and are quoted [inside square brackets].

Condition 5.1 of the existing resource consent (in Appendix A) specifies a minimum load of "22 MW [40 cubic metres per second] except when the river inflow is less than this". This conversion is inconsistent with actual Matahina HEPS performance: Table 1 shows that 22 MW output requires about 45 m^3/s (43 – 46 m^3/s range), and that 40 m^3/s will only produce about 17 MW at mid-operating range.

Table 1

Matahina HEPS Water to Wire Ratio (m^3/s per MW)

MW	Lake Level, m RL								Flow, m^3/s at 74.5 m RL lake level
	73	73.5	74	74.5	75	75.5	76	76.5	
10				**2.5**					25
12				**2.4**					29
18				**2.3**					41
20				**2.15**					43
21	2.127	2.109	2.090	**2.072**	2.053	2.035	2.016	1.998	43.5
22	2.096	2.077	2.058	**2.039**	2.020	2.001	1.982	1.963	44.9
25	2.045	2.025	2.005	**1.985**	1.965	1.945	1.925	1.905	49.6
35	1.963	1.941	1.920	**1.898**	1.876	1.855	1.833	1.811	66.4
50	1.930	1.908	1.885	**1.863**	1.840	1.817	1.795	1.772	93.1
60	1.921	1.898	1.876	**1.853**	1.830	1.807	1.784	1.762	111.2
72	1.917	1.895	1.872	**1.849**	1.826	1.803	1.780	1.757	133.0

Sources: Data for 10 to 20 MW output is based on recorded flows at Te Teko under the rough running regime November 2007 to April 2008. Other ratios supplied directly by TrustPower.

To allow for variations from the generation set point at the proposed 10 MW minimum generation output, TrustPower seeks a minimum consented flow of 20 m^3/s, except when reservoir inflow is less than 20 m^3/s.

2 Historical context of the Matahina HEPS

2.1 History

2.1.1 Construction and ownership

The Matahina HEPS was authorised under an Order in Council dated 14 January 1959. The Matahina Dam across the Rangitaiki River was completed in 1967 for the New Zealand Electricity Department (NZED).

After taking over the Matahina HEPS from NZED, the Electricity Corporation of New Zealand Ltd (ECNZ) obtained consents for the scheme with a 20-year term expiring on 30 November 2009.

TrustPower Limited purchased the Matahina HEPS from ECNZ in 1999.

2.1.2 Operating regime

The conditions of consent define (amongst other things) the maximum and minimum generating load, ramping rates (both increasing and decreasing) and the number of operating peaks per day. The conditions (refer Appendix A) have been varied a number of times.

The Matahina reservoir is relatively small and does not allow for seasonal storage, therefore the total daily inflow to the reservoir is approximately equal to the total daily outflow. At different times the scheme has been operated in three broadly distinct flow regimes:

Single peak	The scheme runs at maximum output for a single peak each day.
Twin peak	The scheme runs at maximum output for two peaks each day.
Run-of-river	The scheme runs with only minor daily fluctuations.

The early operating history is described in Works Consultancy, 1988:

- From 1967 to 1968 the scheme was operated about half the time as a single peak scheme, and half as a run-of-river scheme.
- From 1969 to 1979 the scheme was operated predominantly as a run-of-river scheme with limited single peaking.
- In 1980 there was a twin peak trial for 20% of the time. The trial ceased after concern was expressed regarding river bank stability, but no studies were undertaken at the time.
- From 1981 to 2002 the scheme was operated predominantly as a single peak scheme.

More recently:

- The maximum ramping down rate was increased to the current rates from 1990.
- Since 2002 the scheme has been operated with one or two peaks daily, depending on inflow, except during periods of low flow or floods.
- Since 2007, the scheme has been operating under a revised low flow operating regime, as described below.

2.1.3 Rough running operating regime

The turbines have a "rough running" range at loads between 12 MW and 18 MW, when they are subject to cavitation damage. In order to avoid this rough running range, TPL reached an informal agreement with EBoP in June 2007 for a revised low flow operating regime under the following conditions:

- the revised regime only applies when reservoir inflows are below 22 MW (45 m^3/s);
- generation may drop below the rough running range at night, to allow generation above the rough running range during the day;
- the average generation outflow is to match the average reservoir inflow over 24-hours; and
- EBoP and Fonterra are to be advised at the start and finish of operation under the revised regime.

3 Catchment hydrology

The objective of this section is to provide a brief overview of the hydrology and river hydraulics in the Rangitaiki catchment. This is intended to provide a background for the discussion and analysis in the following chapters.

The Rangitaiki catchment covers an area of 3005 km^2, of which 2844 km^2 (95%) is upstream of the Matahina HEPS, see Map 3. The scheme is located approximately 24 km upstream of the town of Edgecumbe and 12 km from the village of Te Teko. There are two other HEP schemes in the Rangitaiki catchment: Aniwhenua, approximately 20 km upstream of Matahina and Wheao, approximately 70 km upstream. Aniwhenua HEPS is owned by Bay of Plenty Energy Ltd, and Wheo HEPS by TPL.

3.1 Rainfall patterns

Rainfall in the Rangitaiki catchment is typical of the Bay of Plenty region, with significant high intensity storms being relatively frequent (e.g. rainfall studies by EBoP and Tauranga City Council, 2005). These storms are often cyclonic in nature with short durations (i.e. hours rather than days), although longer duration storm events are not unknown across the northeast of the north island.

A report by Opus (2006) identified two main seasons when major floods in the Rangitaiki catchment are likely, these being February and July – August. Events in February are typically due to tropical cyclones moving south from the tropics, producing short duration high intensity rainfall. In contrast winter storms can be of longer duration and are caused by the presence of low pressure systems over the Bay of Plenty, which can also produce short duration, high intensity events. Longer duration, heavy rainfall events can occur if these low pressure systems are blocked by a southerly front moving across the north island.

Table 2

Mean monthly rainfall depths 1990-2005

Month	Mean monthly rainfall (mm)	
	Te Teko	Tarapounamu
January	69	91
February	97	126
March	82	100
April	125	132
May	114	135
June	123	188
July	144	180
August	108	149
September	96	152
October	95	135
November	80	151
December	96	156
ANNUAL	1229	1695

Table 2 shows average monthly rainfall depths at the Tarapounamu gauge (Site 866801, in the upper reaches of the adjacent Whakatane catchment) and at Te Teko (Site 860710, downstream of Matahina Dam).

Table 3 shows the design storm depths at the Matahina dam for a range of return periods and durations, based on data from HIRDS[2] version 1.50b.

Table 3

Design rainfall depths at Matahina dam

ARI (years)	Rainfall depth (mm)				
	6 hour	12 hour	24 hour	48 hour	72 hour
2	84	108	139	173	191
5	112	144	185	230	255
10	131	168	216	268	297
20	148	191	245	304	337
50	171	220	283	351	389
100	189	242	311	386	428

Source: HIRDS, version 1.50b

3.2 Geology

There are two distinct geological types in the Rangitaiki catchment: greywacke and pumice ash. The greywacke is found in the east of the catchment along the Ikawhenua range, with the remainder of the catchment having pumice geology. The prevalence of pumice in the catchment is due to the volcanic nature of the area and the proximity to Mt Tarawera, which erupted in the 1800s.

The geology of the catchment has a significant effect on rainfall-runoff patterns, with pumice tributaries such as Pokairoa River having high infiltration, high base flows and small flood flows. In contrast the greywacke tributaries such as the Whirinaki and Horomanga Rivers have low infiltration, low base flow and large flood flows. The difference in runoff between the two geology types is illustrated by a comparison between recorded flows at the Galatea and Murupara flow gauges, with the Galatea gauge showing significantly higher volumes of runoff. The plains surrounding Edgecumbe have pumice geology but are very flat, so there are drainage problems in the area and stormwater flooding is not uncommon.

3.3 Land use and topography

The Rangitaiki catchment is predominantly rural, with a mix of bush and pasture. The major town in the catchment is Edgecumbe, located approximately 24 km downstream of the Matahina HEPS. Edgecumbe has a small population but significant infrastructure, principally a large Fonterra dairy factory, Transpower substation and State Highway 2. Other villages in the catchment include Te Teko, Waiohau, Galatea and Murupara, although none have a large population.

Forestry and dairy are the two most significant land uses in the catchment. There are extensive areas of pine plantation in the east of the catchment, particularly in the area around the Aniwhenua

[2] HIRDS (High Intensity Rainfall Design System) is a computer programme developed by NIWA. HIRDS version 1.50b is considered more accurate in the Bay of Plenty region than the later version 2.

HEPS, which are actively worked. Dairy farming is concentrated on the plains in the upper catchment and in the valleys surrounding Murupara and Waiohau. On the plains downstream of Matahina dairy farming is widespread.

Te Urewera National Park encroaches on the eastern part of the Rangitaiki catchment downstream of Murupara. Within the park boundary the land use is native bush. The estimated vegetative cover upstream of Matahina is given in Table 4.

Table 4 Estimated vegetative cover upstream of Matahina HEPS

Vegetation type	Coverage (estimated)
Native forest / scrub	29%
Exotic forest	50%
Pasture	21%

Source: Mitchell Partnerships, pers. comm.

The topography of the catchment is dominated by the Ikawhenua mountain range to the east of the catchment, within Te Urewera National Park and the volcanoes of Mt Tarawera and Mt Tauhara to the west. East of the Rangitaiki the topography is hilly with many small, steep streams. West of the river, particularly in the upper catchment, the topography is flat with extensive plains. Downstream of Matahina the topography is very flat. The highest elevation is 1240 m RL, in the upper reach of the catchment.

3.4 Natural flows

3.4.1 Flood flows

Estimated present climate flood flows are given in Table 5. The estimated flows entering Lake Matahina are based on flow records from four discharge gauges in the catchment: Rangitaiki @ Murapara, Rangitaiki @ Kopuriki, Waihua @ Gorge and Whirinaki @ Galatea. The flows at Te Teko are taken from the EBoP Hydrological Flow Summary for the period 1949 to 2005, based on the NIWA gauge record at that location.

Table 5 Estimated flood flows (present climate)

Return period, years	Flood flow entering Lake Matahina, m^3/s	Flood flow at Te Teko, m^3/s
Mean annual flood		260
5	370	320
10	470	410
20		505
25	600	
50	690	650
100	780	780
200	870	

The Te Teko flows are less than the flows entering Lake Matahina due to attenuation within the reservoir, as discussed in Section 4.3. Additional data on flood flows is presented in Table 12.

3.4.2 Normal flows

The median flow in the Rangitaiki River is 62 m^3/s (EBoP, 2007). The mean flow is 71 m^3/s. Over the period 1948 to 2008 the monthly mean flow has ranged from 33 to 184 m^3/s, see Table 6.

Table 6

Rangitaiki at Te Teko – Monthly mean flow (m^3/s)

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Mean
1947	?	?	?	?	?	?	?	?	?	?	?	?	?
1948	?	?	?	?	?	86.2	110.3	82.4	65.1	74.9	78.9	65.4	80.5?
1949	66.2	53.8	44.4	41.9	103.5	99.5	87.5	86.1	74.8	59.0	55.1	50.0	68.6
1950	40.1	51.8	41.8	43.5	56.2	61.2	71.9	62.7	61.2	53.5	62.9	47.5	54.5
1951	58.9	56.2	52.2	45.1	44.3	53.6	112.7	77.9	54.5	55.7	70.7	65.0	62.4
1952	58.8	57.4	47.8	45.8	48.3	84.1	86.1	76.4	61.6	75.4	129.3	118.0	74.1
1953	83.1	80.0	63.1	59.6	80.7	99.3	140.1	120.3	99.6	95.4	75.3	66.5	88.7
1954	54.3	48.5	62.0	57.4	64.5	63.1	67.6	87.0	80.5	56.6	49.0	56.5	62.4
1955	48.0	44.6	41.9	49.1	63.2	67.7	84.3	85.8	76.1	79.4	59.5	59.5	63.4
1956	60.2	50.0	43.8	69.9	110.4	156.1	146.7	120.8	96.5	101.6	95.6	81.1	94.5
1957	68.3	58.5	60.4	52.0	76.5	65.0	76.4	56.4	52.4	56.5	55.3	56.1	61.2
1958	46.7	77.2	63.2	47.0	46.6	47.8	67.7	74.8	67.2	59.9	97.2	126.8	68.5
1959	95.1	80.2	83.9	99.6	81.7	80.7	64.8	63.8	55.4	80.2	68.1	53.8	75.6
1960	46.6	89.1	70.3	52.1	51.1	92.7	86.2	76.1	77.6	82.0	74.1	59.2	71.3
1961	55.6	47.6	45.8	43.6	44.5	43.2	57.5	60.0	66.6	54.8	45.9	50.4	51.3
1962	47.6	56.7	85.0	80.6	149.0	150.4	107.7	105.7	110.1	142.8	133.2	132.0	108.7
1963	93.9	75.6	59.5	53.6	52.9	79.7	112.8	82.4	123.6	80.2	62.1	56.8	77.8
1964	55.3	49.1	77.1	47.1	47.6	49.2	127.2	98.0	97.7	109.1	81.1	72.5	76.1
1965	71.2	143.9	80.0	66.5	58.6	73.6	76.8	104.7	76.7	60.0	80.0	67.6	79.5
1966	86.9	71.5	88.8	68.1	83.6	82.6	116.1	110.8	107.8	92.0	92.4	75.8	89.9
1967	68.9	141.5	80.7	56.5	58.5	59.1	59.1	106.5	96.9	74.8	82.2	94.9	81.2
1968	72.4	59.8	55.5	61.5	79.1	95.0	103.2	90.7	89.4	76.0	66.9	69.1	76.6
1969	76.1	86.1	61.6	55.7	58.7	53.8	52.8	51.3	86.8	61.7	51.3	63.8	63.1
1970	54.4	46.6	47.5	50.6	50.4	92.8	76.1	157.1	136.4	153.5	121.7	78.5	89.0
1971	76.9	69.3	66.7	58.8	119.3	138.6	76.5	80.2	119.4	120.1	107.2	103.3	94.8
1972	82.7	64.1	112.3	64.9	59.7	56.2	81.6	73.7	74.1	69.6	55.9	49.7	70.5
1973	50.9	43.7	43.5	43.9	45.2	59.6	47.6	62.9	78.0	68.7	56.0	49.4	54.1
1974	44.9	44.9	41.8	77.1	57.8	82.6	108.5	109.4	90.4	92.4	74.6	83.7	75.9
1975	77.3	61.8	72.6	65.3	67.9	109.0	76.4	84.0	89.1	98.4	78.5	61.6	78.5
1976	85.9	97.9	62.6	66.2	72.7	71.7	80.4	85.8	81.0	80.8	69.3	62.6	76.3
1977	55.3	52.1	47.5	?	51.9	74.6	90.5	81.8	68.3	64.7	52.6	56.3	63.3?
1978	44.8	?	38.3	43.1	38.0	40.7	81.4	58.8	62.8	60.8	69.7	61.1	54.5?
1979	47.3	58.4	89.2	70.8	76.2	62.5	63.3	?	110.2	103.5	99.4	73.5	77.7?
1980	79.4	62.1	57.0	68.4	60.0	61.7	?	74.3	82.7	61.5	61.5	73.9	67.5?
1981	65.2	53.3	52.4	54.9	62.4	86.9	94.9	98.7	75.2	74.5	95.5	83.3	74.9
1982	71.5	62.8	62.5	56.6	61.0	74.8	62.4	61.9	55.8	50.3	45.2	45.3	59.1
1983	42.9	37.6	35.1	43.1	42.4	50.8	57.3	50.8	58.4	123.9	106.9	79.8	60.9
1984	53.8	57.5	72.3	57.9	46.7	46.6	73.6	63.6	64.8	56.6	49.5	76.4	60.0
1985	63.1	50.8	49.6	59.8	49.3	68.4	71.0	72.1	86.4	56.0	55.4	73.4	63.0
1986	105.0	66.0	67.6	52.9	63.8	70.7	70.1	96.2	91.7	68.3	60.0	52.5	72.2
1987	57.7	49.3	55.7	58.9	52.5	?	51.0	59.7	50.6	52.7	57.5	70.7	56.1?
1988	47.2	?	?	39.1	43.5	49.7	56.0	87.3	85.7	83.4	66.6	73.6	63.2?
1989	124.3	79.6	59.7	48.2	55.9	89.6	90.5	61.0	70.9	112.5	84.7	58.4	78.0
1990	57.7	54.9	59.3	50.6	62.8	58.6	60.7	124.6	78.1	93.6	104.5	68.1	72.9
1991	55.7	63.9	53.1	52.1	?	51.6	59.8	96.3	90.3	82.3	74.6	53.0	66.6?
1992	61.1	58.9	50.0	46.9	42.2	45.0	63.5	98.3	74.7	75.5	59.9	88.7	63.8
1993	55.2	48.1	43.7	43.1	45.9	85.0	58.3	48.2	43.2	39.4	43.3	41.5	49.5
1994	40.8	38.0	37.2	43.9	41.3	67.8	85.3	107.8	76.4	80.7	83.3	57.1	63.5
1995	52.3	55.0	52.3	91.3	73.3	78.3	120.5	100.3	97.9	102.3	82.3	104.1	84.4
1996	83.9	70.5	65.6	83.0	95.7	84.9	90.1	87.1	106.5	73.4	60.5	61.3	80.2
1997	60.7	51.8	61.3	56.8	51.4	101.3	80.5	59.7	59.0	74.0	58.1	49.4	63.7
1998	42.4	45.0	47.8	45.3	45.3	70.5	184.1	102.6	86.7	89.1	75.5	64.9	75.2
1999	55.7	48.0	50.3	53.0	57.6	81.0	73.9	79.1	85.0	62.5	105.6	70.0	68.5
2000	56.2	50.6	43.5	49.6	52.4	74.2	63.4	69.7	71.0	68.5	54.1	52.3	58.8
2001	50.5	63.4	51.2	61.3	76.4	58.4	51.6	63.6	61.5	68.3	78.3	112.4	66.4
2002	72.5	55.6	49.7	52.6	54.1	76.1	?	70.4	62.4	57.9	50.1	52.0	59.4?
2003	45.2	37.3	37.3	41.4	45.6	61.1	56.3	41.8	69.5	99.7	61.8	80.1	56.6
2004	69.1	69.8	67.2	45.2	58.5	98.3	174.2	106.1	79.0	89.0	78.7	71.8	84.1
2005	83.8	57.6	54.3	49.3	62.5	64.3	68.2	59.2	65.6	78.4	60.4	69.2	64.5
2006	67.0	79.3	52.0	58.1	95.7	100.3	83.2	108.4	64.7	57.6	66.0	54.3	73.8
2007	67.9	51.4	46.2	44.8	45.4	43.1	68.6	75.1	52.9	55.9	44.7	45.3	53.5
2008	37.6	32.8	33.5	55.7	57.8	48.6	?	?	?	?	?	?	44.4?
Min.	37.6	32.8	33.5	39.1	38.0	40.7	47.6	41.8	43.2	39.4	43.3	41.5	49.5
Mean	63.3	61.5	57.6	55.9	62.7	74.7	83.9	83.0	78.9	78.0	72.9	69.2	71.3
Max.	124.3	143.9	112.3	99.6	149.0	156.1	184.1	157.1	136.4	153.5	133.2	132.0	108.7

The Min Mean and Max of Annual values are for complete years only.

Source: NIWA gauge record.

3.4.3 Low flows

Low flows in the Rangitaiki River at Te Teko are given in Table 7. The 7-day low flows can be regarded as natural low flows, not affected by Matahina HEPS operation. The minimum flow recorded in 2008 is a result of the rough running regime at the time.

Table 7

Rangitaiki at Te Teko – Low flows

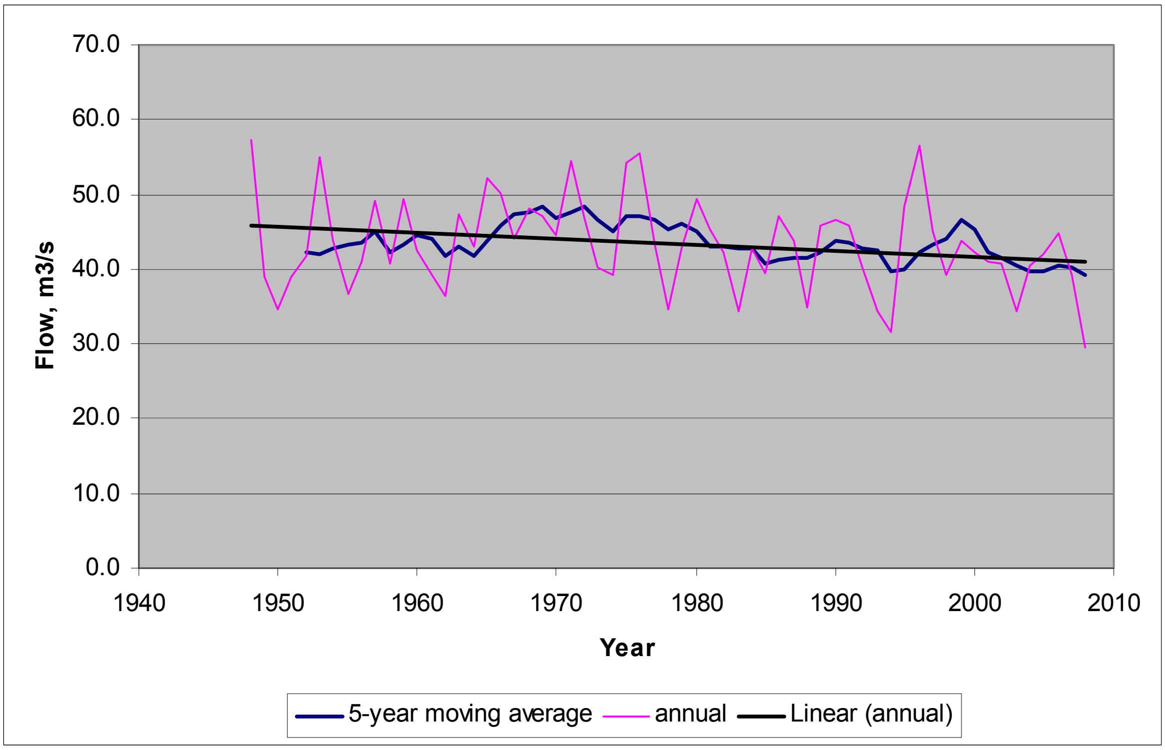
7-day low flow (mean annual)	42 m^3/s
7-day low flow (5-year return)	38 m^3/s
7-day low flow (10-year return)	36 m^3/s
7-day low flow (minimum, 2008)	30 m^3/s
Minimum recorded flow (2008)	25 m^3/s

Sources: EBoP Hydrological Flow Summary for the period 1949 to 2005, and NIWA gauge record.

The recorded low flows appear to be influenced by natural climate oscillations (the El Niño Southern Oscillation and the Interdecadal Pacific Oscillation), but also show a downward trend over time, see Figure 1: the trend line falls by 5 m^3/s over the 61 year period of record. The reasons for this are likely to be a combination of changes in catchment land use, increasing irrigation, and climate change.

Figure 1

Rangitaiki River at Te Teko: 7-day low flows 1948 to 2008



EBoP has not yet specified the Rangitaiki Instream Minimum Flow Requirement (IMFR) under Method 177 of the Bay of Plenty Regional Water and Land Plan (2008). The default instream minimum flow requirement (Method 179) is 90% of the Q5 7-day low flow, which is 34 m^3/s.

3.5 Takes and discharges

Takes from the Rangitaiki River include both consented takes and permitted takes under the Regional Water and Land Plan. Consented and permitted takes are described below, and their estimated total is 132,664 m^3/day. This is equivalent to an average 1.54 m^3/s, which is 4.0% of the 5-year return period 7-day low flow.

3.5.1 Consented takes and discharges

There are 40 consented takes from the Rangitaiki River and its tributaries totalling up to 120,875 m^3/day at a maximum rate of 2581 L/s, excluding non-consumptive hydropower takes and two minor takes from tributaries downstream of Matahina (EBoP, pers. comm.).

There are 28 consented takes upstream of the dam, totalling up to 77,057 m^3/day at a maximum rate of 1372 L/s, predominantly for irrigation.

There are 12 consented water takes below the dam, as listed in Table 8 and shown on Map 2. The most significant is the Fonterra take for the dairy factory at Edgecumbe. Others are for irrigation or frost protection. Seasonal limits on the higher takes for frost protection apply as noted. There are no seasonal limits on the irrigation takes.

Table 8

Consented water takes on the Rangitaiki River downstream of Matahina

Consent Nº	Date of grant	Date of expiry	Consent holder, Property address	Purpose	Maximum quantity, m^3/day	Maximum rate, L/s
20967	5.8.82	2026	Omataroa 7AC1A & 7AC1C2 Trust SH 30, Te Teko	Irrigation	749	18.3
21008	7.10.82	2026	KP & SE Lyell 124 East Bank Road, Thornton	Irrigation	50	1
21063	3.2.83	2026	IJ & JM Kinvig West Bank Road, Edgecumbe	Irrigation (Frost)	208 (2,000)	3 (60)
21224	6.10.83	2026	Gow Family Trust 396 West Bank Road, Edgecumbe	Irrigation	1,800	40
21703	1.8.85	2026	AB Schlepers Westbank Road, Edgecumbe	Irrigation	135	7.58
21852	7.8.86	2026	PS & MA Leaming, West Bank Road, Edgecumbe	Irrigation	82	3
21923	2.10.86	2026	AT & RR Harvey and JW Herbke Otakiri Road, Edgecumbe	Irrigation	164	11.4
60427	23.10.99	31.10.09	Ngati Awa Farms (Rangitaiki) Limited Hydro Road, Te Teko	Irrigation	6,500	75

Consent Nº	Date of grant	Date of expiry	Consent holder, Property address	Purpose	Maximum quantity, m^3/day	Maximum rate, L/s
62000	16.9.03	30.6.23	Fonterra Co-Operative Group Ltd Awakeri Road, Edgecumbe	Dairy processing	30,000	900
62566	29.6.04	30.6.14	The Trustees of Omataroa Trust 1856 and 1890 SH 30, Te Teko	Irrigation (Frost, 1 Apr – 30 Nov)	600 (800)	14 (14)
63177	29.6.05	30.6.20	CR Martin 1793 SH 30, Whakatane	Irrigation and frost protection	2,880	100
64967	8.2.08	31.1.28	RM & HR Hudson 2155 SH 30, Te Teko	Irrigation (Frost, 1 May – 15 Nov)	650 (1,800)	36.1 (100)
TOTAL (excluding frost protection)					**43,818**	**1,209**

Fonterra also has consents to discharge factory wastewater, cooling water and stormwater to the Rangitaiki River. The wastewater discharge consent (No 024211, granted 4 November 1997, expiry 30 June 2010) imposes limits on lactose concentration in the river, calculated on a daily basis.

3.5.2 Permitted takes

Rule 41 of the Regional Water and Land Plan permits the take of water for any purpose up to 15 m^3/day, generally for agricultural, horticultural or domestic use. EBoP has no information on the combined volume or seasonality of these permitted takes. We have estimated the permitted surface water use in the Rangitaiki catchment using a modelling approach, assumptions and methodology adapted from the Environment Waikato report *A Model for Assessing the Magnitude of Unconsented Surface Water Use in the Waikato Region (TR 2007/47)*. This includes the takes allowed under s14(3)(b) of the RMA for individual domestic and stock drinking water.

Geographic Information Systems (GIS) were employed to undertake this modelling. Inputs were:

- Rangitaiki River catchment as supplied by EBoP on 16 March 2009 as GIS files;
- The Agribase dataset (supplied by AssureQuality on 6th April 2009) was used to determine the extent of farm properties and stock numbers for dairy cows, beef cows, deer and sheep;
- The Land Information System (LINZ) Core Record System (CRS) land parcel database was used to estimate the population within each of the sub-catchments;
- Well consents as supplied by EBoP (parcels with bores are assumed to use groundwater); and
- The Drinking Water NZ website (http://www.drinkingwater.org.nz/supplies/supplies.asp) is used to identify urban or rural water schemes. Households using water supplied as part of a scheme are also removed from the calculations.

The Rangitaiki catchment was divided into three subcatchments for analysis, see Map 3, with results as shown in Table 9.

Table 9

Estimated permitted surface water use in the Rangitaiki catchment

Sub-catchment	Area, km^2	Stock water estimate m^3/day	People water estimate m^3/day	Estimated total m^3/day
Lower (sea to SH2 within catchment)	44	1715	522	2,237
Lower (sea to SH2 left bank)[a]	0[a]	572[a]	174[a]	746[a]
Middle (SH2 to Matahina dam)	98	365	379	744
Upper (upstream of Matahina dam)	2,808[b]	7,167	895	8,062
TOTAL	2,950[b]	9,819	1,970	11,789

[a] The lower subcatchment extends approximately 3 km on the right bank (east of the river) but is bounded by the left bank stopbank, see Map 2. However, properties on the left bank also abstract water from the Rangitaiki: their permitted abstractions are assumed to be one third of the right bank abstractions.

[b] There are minor variations between the areas above, based on the catchment as supplied by EBoP, and the areas quoted in NIWA publications and Section 3.

3.6 Hydropower schemes

The Rangitaiki River has three hydropower dams: Wheao, Aniwhenua and Matahina. Each of these dams has an effect on the flow regime in the Rangitaiki, although the effect of Wheao and Aniwhenua is not as noticeable as that of Matahina due to their relatively smaller scale of storage.

Storage in the Wheao and Aniwhenua reservoirs is small and during flood events (when the dam spill gates are open) they effectively act as run of river schemes with inflow approximately equal to outflow, so provide little or no attenuation. The hydrology and hydraulic effects of the Matahina HEPS are discussed in Section 4.

3.7 Rangitaiki River downstream of Matahina dam

Historically the Rangitaiki River carried a very high bed load of coarse pumice sands, which contributed to the formation of the plains (other sources are the Tarawera and Whakatane Rivers). Downstream of Te Teko the river is perched above the surrounding plains.

Since completion of the Matahina dam in 1967, most sediment entering Lake Matahina is trapped at the lake. As a result, the Rangitaiki River downstream of the dam has had to readjust its channel configuration by bank erosion and bottom scour to obtain equilibrium.

Callender & Duder (1979) estimated an annual sediment accumulation in Lake Matahina of 181,500 t of bed load and 144,500 t of suspended load. Phillips & Nelson (1981) report (TR Healy, University of Waikato, pers. comm.) that the pre-dam annual discharge figures for suspended and bed load sediment near the river mouth were about 201,000 and 188,000 t respectively, but post-dam values have dropped to 65,000 and 10,000 t.

Downstream of the Matahina dam, the Rangitaiki River passes through a defined valley for 4 km then follows a meandering course across the Rangitaiki plains past Te Teko and Edgecumbe to the coast, 37 km downstream of the dam.

The Rangitaiki plains were drained in the early 20th century, but subject to almost annual flooding. In the1970s the Rangitaiki-Tarawera Rivers Scheme was constructed. It included stopbanking of the main channel downstream of Te Teko, and provision of Reid's floodway from 3.5 km upstream of Edgecumbe to one kilometre above the river mouth (see Map 2). The design standard was for a 780 m^3/s flood (100 year return period), with Reid's floodway operating when flows exceed 610 m^3/s (40 year return period). In practice the design standard is lower, because in flood conditions dunes form in the river bed causing higher roughness than the original design allowed for, so that the river channel capacity is less than designed (EBoP, 2000). The design of the floodway is not hydraulically efficient as the channel width varies from 200 m upstream to an average of 44 m downstream (with a minimum width of only 25 m). The stopbank freeboard is 300 mm lower in rural areas than through Edgecumbe. This is to ensure that in super-design events farmland rather than the urban area would be inundated.

The bed gradient of the river has been affected by the artificial straightening of the channel downstream of Thornton in 1913, and by the Edgecumbe earthquake in March 1987, which resulted in a 1-2 metre discrepancy in stopbank levels at the fault scarp 5 km upstream of Edgecumbe. The earthquake lifted the river bed upstream of the fault exposing the stumps of a forest that had previously been covered by volcanic ashes and alluvial pumice gravels from the Mount Tarawera eruption in 1866.

The river reach from Edgecumbe to the coast is subject to tidal effects. The sea level at the mouth is expected to be very similar to the sea level at Whakatane, which has a normal range of 1.2 m for neap tides and 1.6 m for spring tides.

The consented water takes from the Rangitaiki River use various fixed, adjustable or pontoon-mounted suction pipes close to the river bank, as described in Appendix C.

4 Hydrology and hydraulic effects of Matahina HEPS operation

4.1 Lake Matahina level

4.1.1 Level fluctuation

The normal operation of Lake Matahina is within the limits defined by Conditions 3 and 6 of the consent, as summarised in Table 10. The maximum reservoir level rate of change is 0.25 m/h except under emergency conditions when the spillway is operating. Under flood conditions, the reservoir may be drawn down at 0.3 m/h, or 0.4 m/h in emergency.

Table 10

Lake Matahina normal operation

Condition	Maximum reservoir level, m RL	Minimum reservoir level, m RL	Operating range, m	Live storage, Mm^3
Normal	76.2	73.15	3.05	6.6
Flood < 200 m^3/s	76.4 (spillway gate crest level)	73.15	3.25	7.1
Flood, 200-500 m^3/s	76.8	73.15	3.65	8.0
Flood > 500 m^3/s	76.8	71.6 (draw down when flood expected)	5.20	11.2

The live storage data in Table 10 is derived from data supplied by TrustPower. The 6.6 Mm^3 live storage within the normal operating range for the reservoir represents just over one day's storage at the mean inflow of 71 m^3/s. Thus the reservoir has capacity for daily peaking, but not for seasonal storage.

4.1.2 Upstream flooding

The July 2004 flood caused minor flooding immediately upstream of Lake Matahina as a result of the accumulation of logs and other debris that was washed into the reservoir during the flood event (see Section 6).

The study on sedimentation in the lake by Phillips & Nelson (1981) identified three zones of sedimentation:

- Fluvial. The upper reaches of the lake are shallow (1-4 m) with a sinuous channel constricted by a narrow ignimbrite gorge. Fluvial sediments include sandy gravels and gravelly sands.
- Deltaic. As the gorge widens down the lake, the depth increases to 15 m along a delta front. Deltaic sediments are generally silty sands, sandy silts, or silts.
- Basinal. The lake depth is 40-50 m in the basin immediately behind the dam. Basinal sediments are silt and clay.

Phillips & Nelson reported that the delta front advanced at 40 m/year from 1975 to 1979, but anticipated slower progradation as the delta advanced into deeper water. The sediment bed load transported into Lake Matahina was greatly reduced after Lake Aniwhenua was formed in 1980 for the Aniwhenua Hydroelectric Scheme. Lake Aniwhenua is a 4.8 km long shallow reservoir on the Rangitaiki River upstream of Matahina, with significant sediment accretion at its upstream end: it is understood that Aniwhenua is now technically "full" of sediment, and that its operation during floods attempts to flush sediment downstream to maintain the intake to the canal. It is therefore probable that Matahina is once again receiving much of the natural sediment load of the Rangitaiki River.

It is expected that sediment inflow to Lake Matahina is transported to the delta front, and that there is no further sediment accretion happening at the upstream end of the lake that would cause bed level rise and consequently worse flooding in future. This assumption is supported by EBoP cross-sections of the Rangitaiki River upstream of Lake Matahina, which show no sediment deposition between 1993 and 2007.

It is worth noting that the first portion of the river channel upstream of Lake Matahina is within a narrow gorge with bush-clad sides. Therefore, any changes in bed level in this reach will have less than minor effects. The nearest farmland is well upstream of this area, and would not see any increase in normal flow or flood water levels as a result of the presence of the reservoir.

4.2 Peaking operation

The consented maximum discharge for generation is 160 m^3/s, and the minimum load is 22 MW (about 45 m^3/s, refer Section 1.3) except when the river inflow is less than 40 m^3/s. This means that flow variations of up to 115 m^3/s are permitted as a result of daily peaking.

Maximum ramping rates are specified in consent conditions 5.3 and 5.4:

- The maximum load increase shall not exceed 37 MW/h [70 m^3/s] …
- The maximum load decrease shall not exceed 16 MW/first hour [30 m^3/s], 12 MW/second hour [22 m^3/s] and 8 MW/hour [14 m^3/s] for every hour thereafter. …

Flow patterns down the river under various operating scenarios were modelled for the Matahina Twin Peaks AEE (Beca, 2001). The modelling assumed a maximum generation of 72 MW, or a discharge of approximately 133 m^3/s, which reflects normal practice: the normal maximum flow variation is therefore 72 - 22 = 50 MW, or 133 - 40 = 93 m^3/s. Three average inflows were selected for modelling, as shown in Table 11. At inflows greater than 60 MW and below 30 MW the station operates under partial or no peaking, so the effects of the HEPS are less.

Table 11

Matahina generation inflows

Average inflow range		Modelled inflow	% of time
m^3/s	MW	MW	
< 55	< 30		46
55 – 74	30 – 40	35	30
74 – 92	40 – 50	45	13
92 – 111	50 – 60	55	6
> 111	> 60		5

Conversions in this section are based on an average 1.85 m^3/s per MW, as used in the 2001 modelling.

The model inflow hydrographs are presented in Figures 2 and 3 for single peak and twin peak operation respectively.

Figure 2
Matahina Single Peak Model Inflow

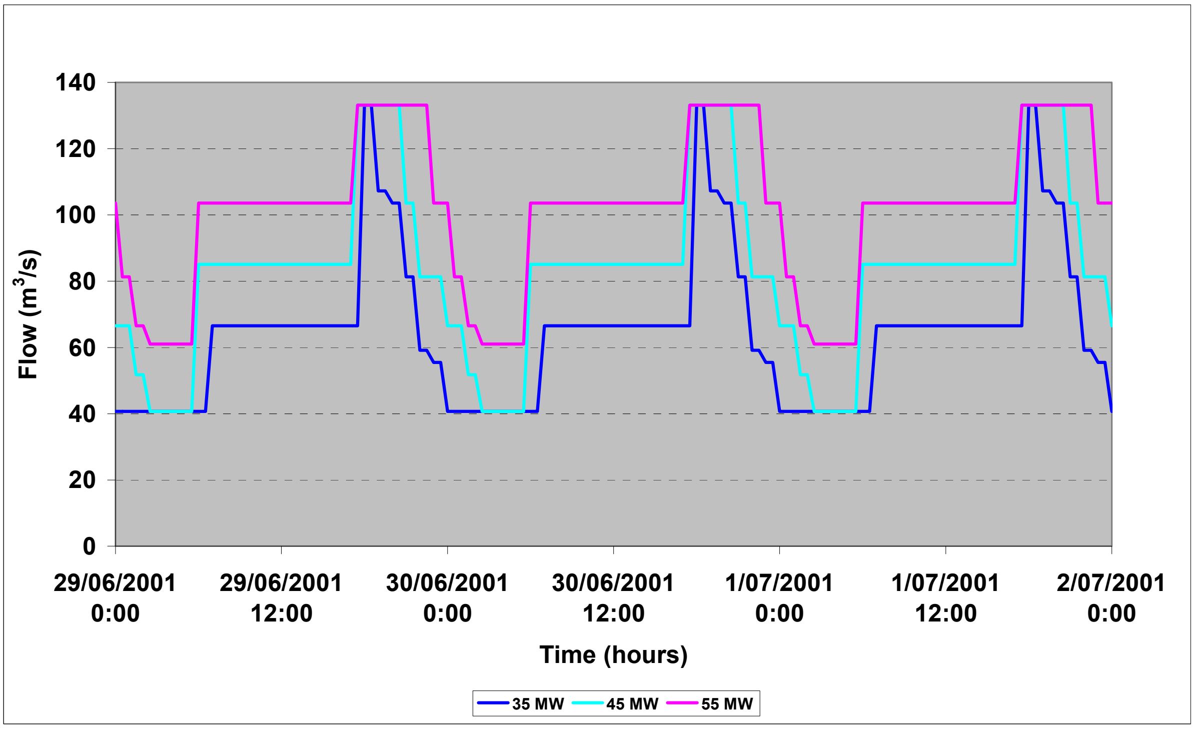
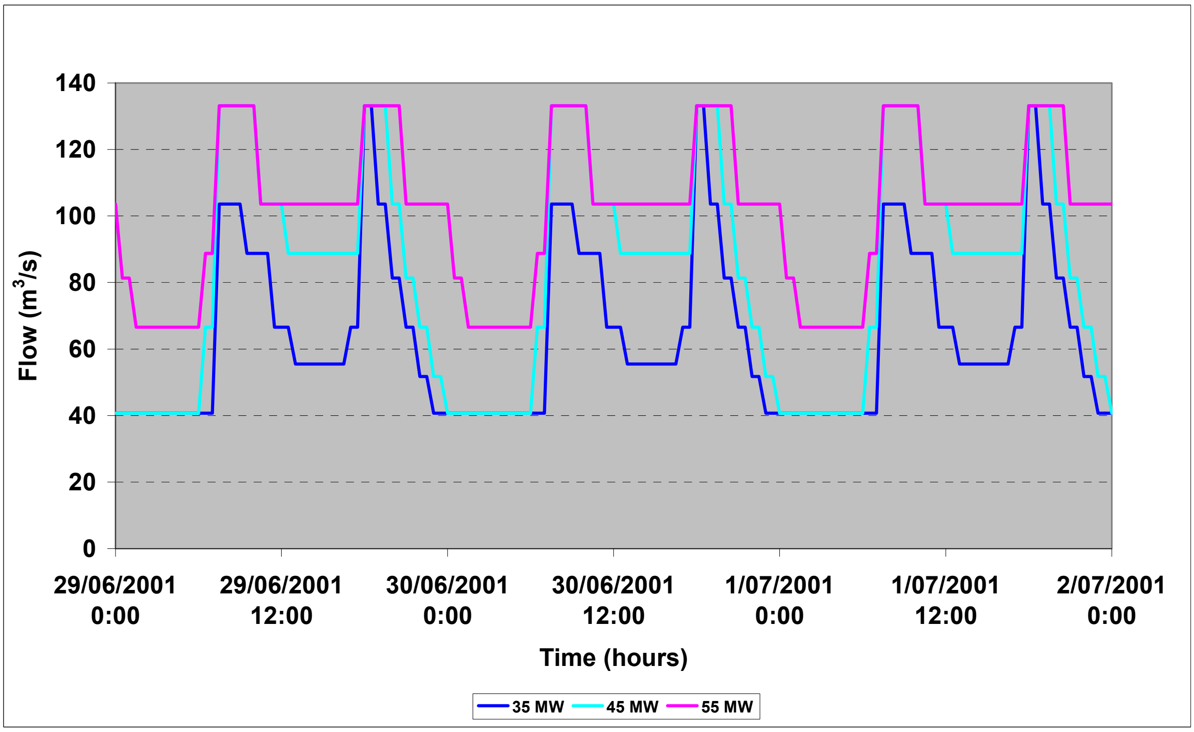


Figure 3
Matahina Twin Peak Model Inflow



Model outputs were presented for the following locations that were monitored as part of the twin peak erosion monitoring programme:

Matahina dam	
Erosion Site 10	8.0 km downstream from dam
Te Teko (Erosion Site 6)	12.0 km downstream from dam
Edgecumbe (Erosion Site 15)	24.2 km downstream from dam
Thornton (Erosion Site 1)	34.1 km downstream from dam

Site 14, used in the bank stability analyses (Appendix B) is 11.5 km downstream from the dam, which is close to the Te Teko gauge (Site 6), and would have similar flow patterns.

Figures 4 and 5 show the attenuation of flows down the river. The 35 MW flow pattern is shown, because this is the most common (see Table 11). Beca (2001) shows similar attenuation patterns for the other modelled flows.

Figure 4

Single Peak Flow Attenuation Down the River (35 MW inflow)

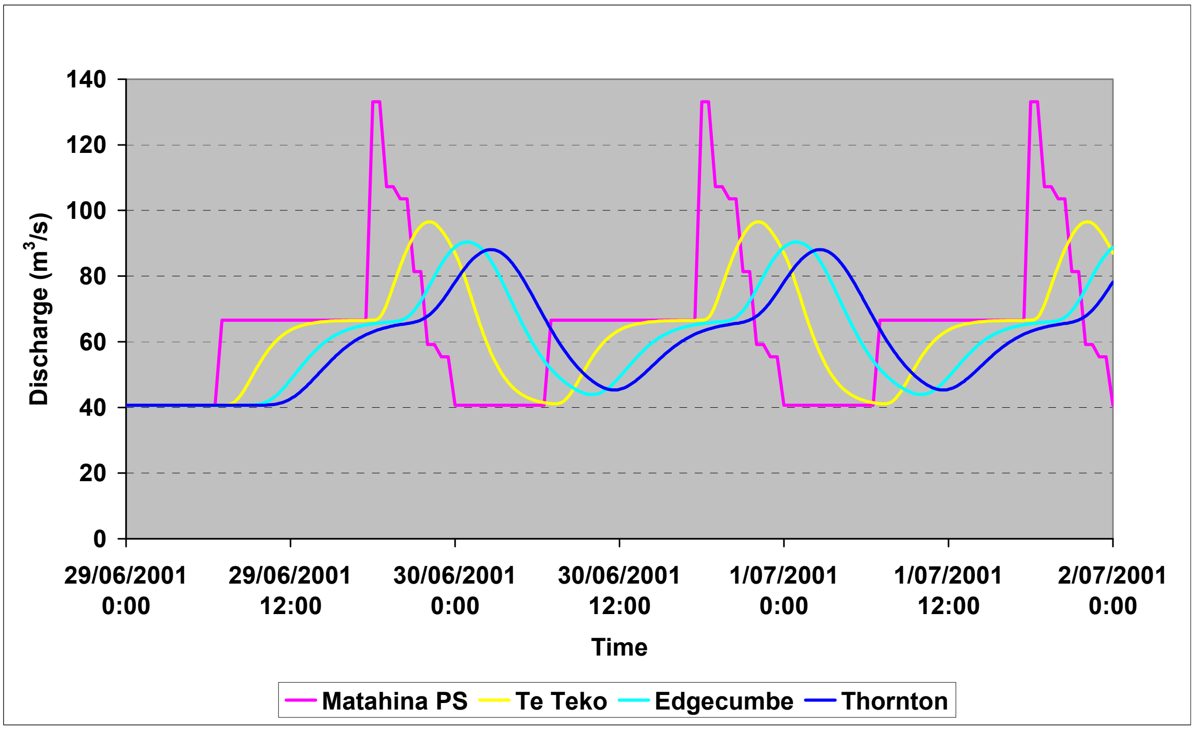


Figure 5
Twin Peak Flow Attenuation Down the River (35 MW inflow)

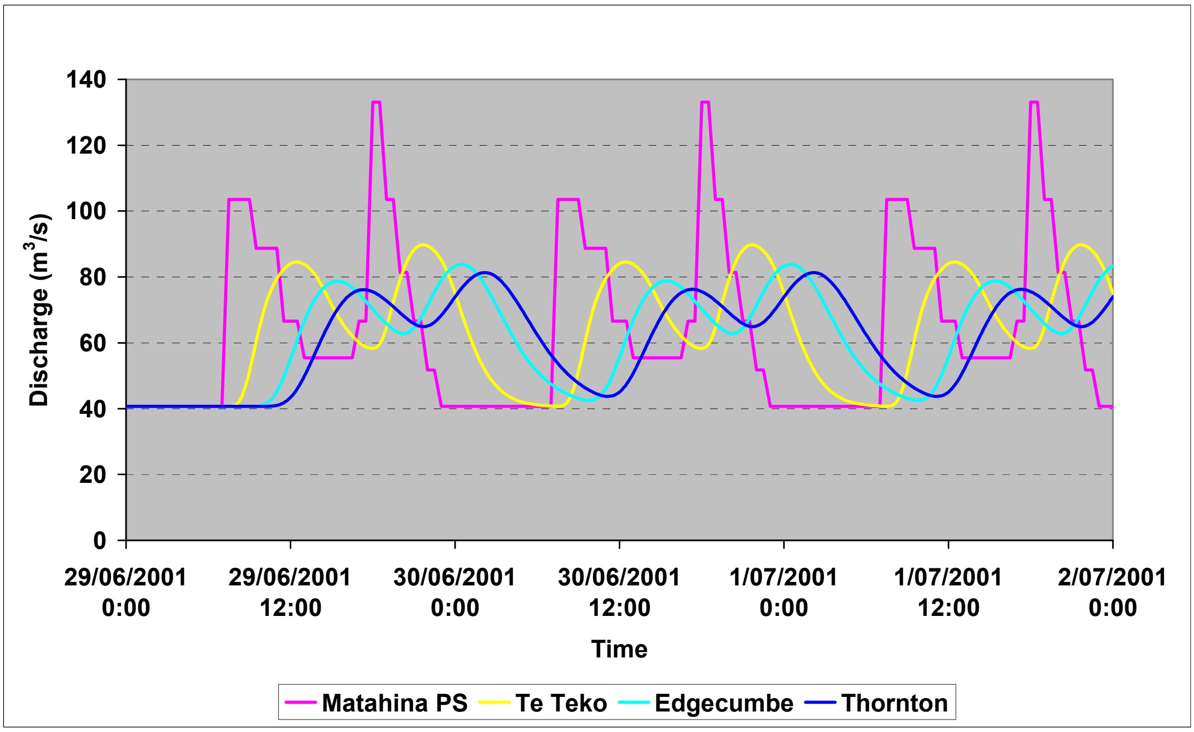


Figure 6 shows the flow profile down the river for the 35 MW single peak scenario. The profiles for other scenarios are very similar. More detailed flow profiles are shown on Figure D19 of Appendix D.

Figure 7 shows the level difference between peak and trough for each scenario. Downstream of Edgecumbe the effect of peaking is less pronounced and tidal influences become more significant.

Figure 6
35 MW Single Peak Flow Profile

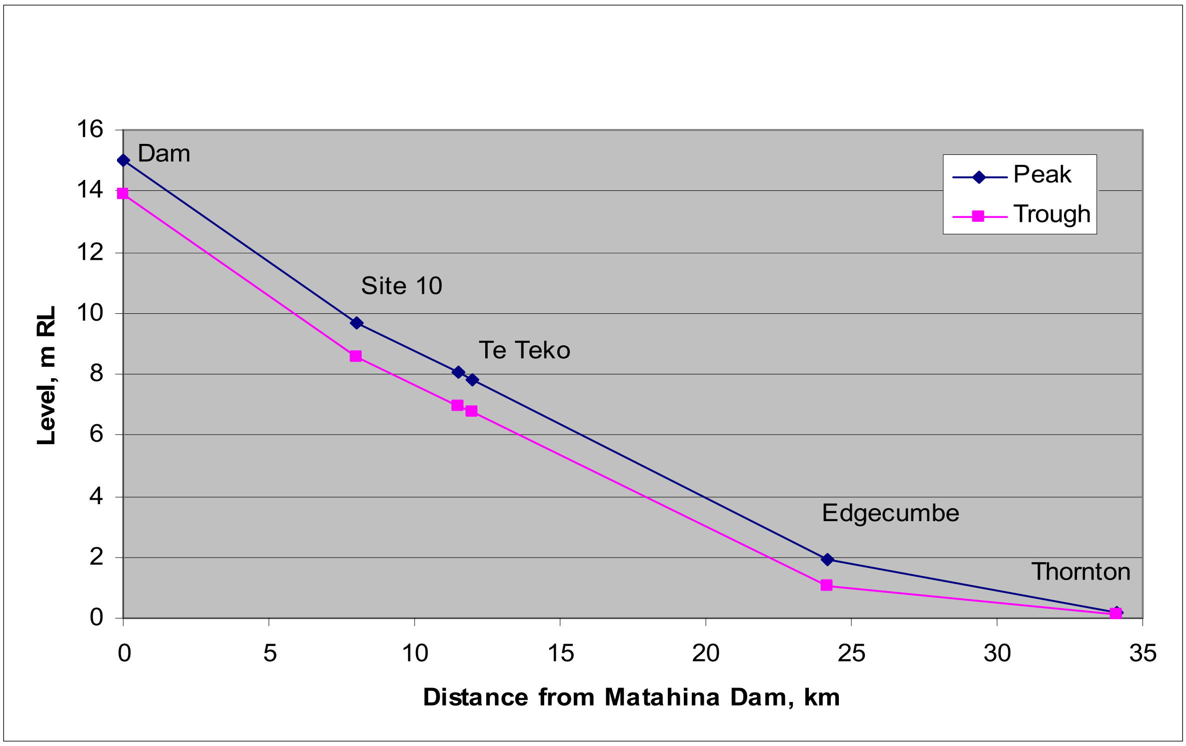


Figure 7
Level fluctuation due to peaking

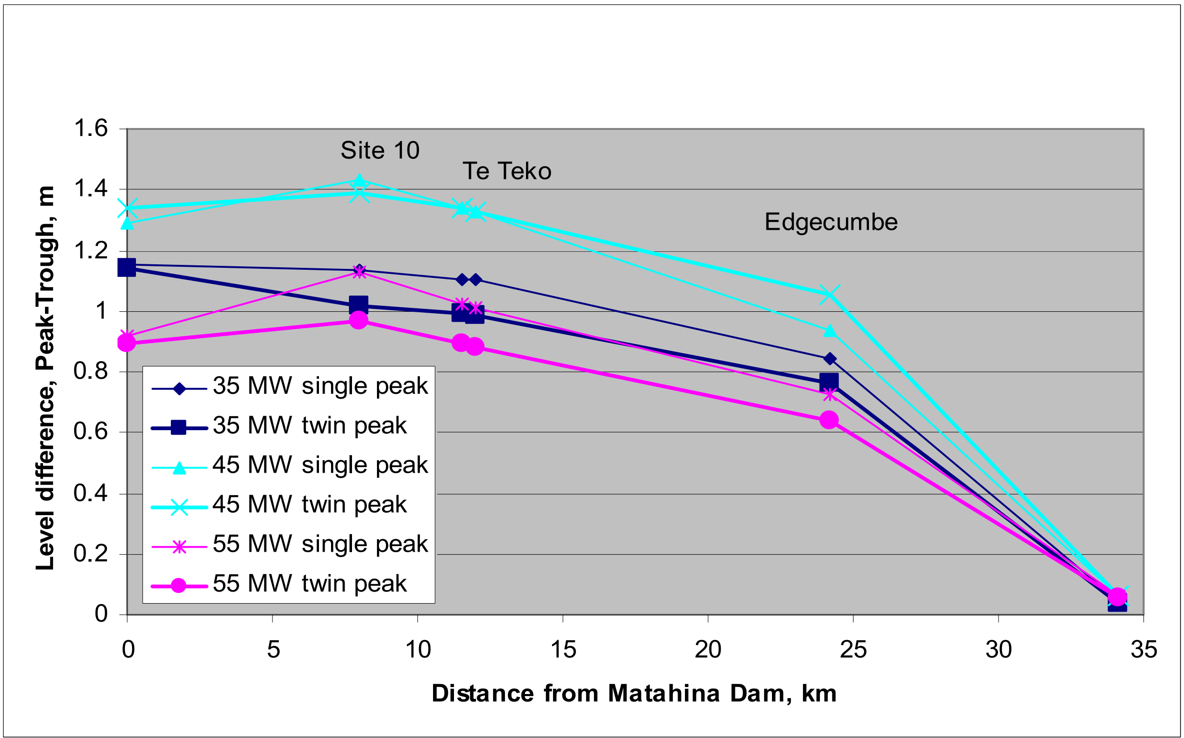
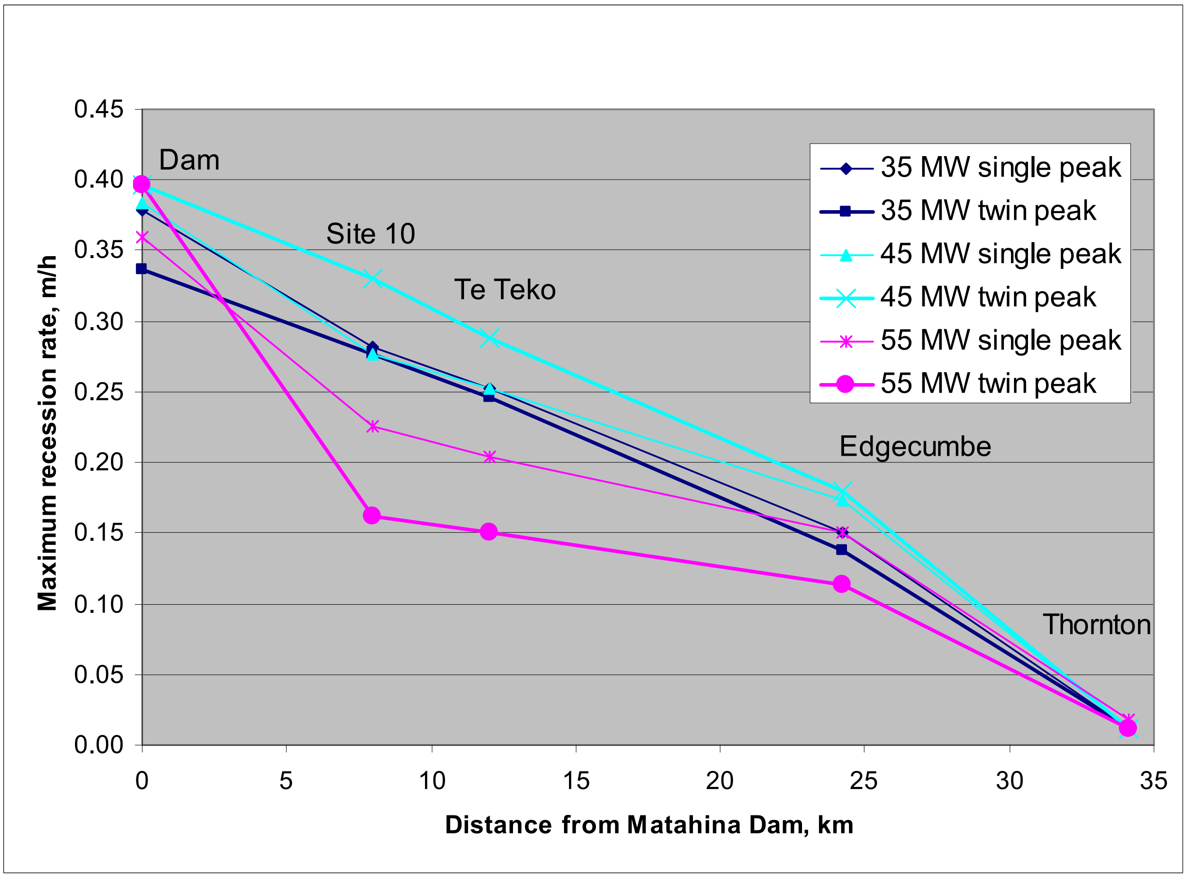


Figure 8 shows the maximum recession rate (the rate at which the water level drops after each peak) for each modelled scenario.

Figure 8
Maximum Recession Rates



4.3 Floods

The Matahina reservoir is not designed for flood attenuation, with little storage available in relation to the volume of large floods. Nevertheless, the scheme does fully attenuate minor floods and provide some peak reduction for larger floods.

The resource consent (Condition 3.4) sets a maximum reservoir level of 76.40 m RL (the spillway gate crest level) during floods of less than 200 m^3/s. In practice flood flows between the maximum generation flow (160 m^3/s) and 200 m^3/s are fully attenuated in the reservoir.

When a flood exceeding 500 m^3/s is expected, TrustPower is required (Condition 6.2.1), at the request of Environment Bay of Plenty (EBoP), to lower the reservoir level so as to provide some flood storage, and there are limits on the maximum rate of discharge: 600 m^3/s, or 755 m^3/s with EBoP agreement. These limits correspond to the design standard for the Rangitaiki-Tarawera Rivers Scheme without/with Reid's floodway in operation. In practice, total discharge from Matahina during a major flood event is decided in consultation with EBoP, who seek to manage peak discharge to avoid flooding in the lower Rangitaiki catchment. There is also co-ordination with BOPE concerning releases from Aniwhenua HEPS.

The Matahina scheme has a normal operating range of 3.05 m, from the minimum (73.15 m RL) to maximum (76.20 m RL) operating levels, see Table 10. Above this there is additional flood storage up to a maximum lake level of 76.80 m RL (design flood) or 77.40 m RL (probable maximum flood). The reservoir can be drawn down below normal minimum operating level to 71.6 m RL. Because the reservoir has this operational range, it is possible to draw the level down prior to a storm event in order to provide additional flood storage, although the level to which the reservoir is drawn down will depend on forecast inflows.

The effect of the Matahina HEPS on flood flows downstream of the dam is indicated in Table 12. Some caution must be used in drawing conclusions from this table, due to the variability of natural flows: for example, the mean annual flow is not affected by the presence of the dam, but has been 7% lower since the dam was completed in 1967.

The table shows that the number peaks exceeding 133 m^3/s has increased significantly, which is to be expected as the result of operation of the Matahina HEPS (133 m^3/s is the normal maximum generation flow during peaking operation, although generation flows up to 160 m^3/s are possible). However, the duration of flow exceeding 133 m^3/s has reduced as a result of Matahina HEPS operation.

The number of peaks and duration of flow exceeding 160 and 200 m^3/s are reduced significantly as a result of Matahina reservoir operation and attenuation. This effect also applies to larger floods, though there are insufficient events with flow exceeding 300 m^3/s to draw statistically valid conclusions from the flow record.

The effect of Matahina HEPS operation on floods downstream of the dam is discussed in Section 5.6. Section 6 reviews the July 2004 flood event, which had a peak reservoir inflow of 747 m^3/s, and a return period of approximately 100 years.

Table 12 Rangitaiki at Te Teko – Annual flood flow summary

Year	Mean flow (m³/s)	Peak flow (m³/s)	Number of peaks exceeding threshold					Duration of flow exceeding threshold (hours)				
			133	160	200	300	500	133	160	200	300	500
1949	68.6	278.9	1	1	1	0	0	170	124	84	0	0
1950	54.5	117.7	0	0	0	0	0	0	0	0	0	0
1951	62.4	309.1	2	2	1	1	0	132	88	36	12	0
1952	74.1	193.5	5	1	0	0	0	379	55	0	0	0
1953	88.7	331.1	4	3	2	1	0	455	243	114	18	0
1954	62.4	197.8	2	2	0	0	0	90	59	0	0	0
1955	63.4	128.1	0	0	0	0	0	0	0	0	0	0
1956	94.5	220.4	12	9	3	0	0	1383	608	89	0	0
1957	61.2	189.5	1	1	0	0	0	28	19	0	0	0
1958	68.5	409.1	6	6	4	2	0	577	375	211	68	0
1959	75.6	231.7	5	3	1	0	0	250	105	23	0	0
1960	71.3	228.7	3	2	1	0	0	140	90	31	0	0
1961	51.3	130.8	0	0	0	0	0	0	0	0	0	0
1962	108.7	405.9	16	9	6	2	0	1695	914	400	73	0
1963	77.8	208.7	6	3	1	0	0	392	128	25	0	0
1964	76.1	272.4	5	5	2	0	0	564	261	87	0	0
1965	79.5	595.5	4	3	2	1	1	360	194	121	55	23
1966	89.9	248.4	13	5	3	0	0	699	250	76	0	0
1967	81.2	566.9	7	3	2	1	1	461	286	135	46	12
1968	76.6	203.2	6	2	1	0	0	232	97	15	0	0
1969	63.1	210.0	3	2	1	0	0	91	40	8	0	0
1970	89.0	637.2	14	10	6	3	1	1284	764	389	109	18
1971	94.8	456.7	13	7	2	2	0	952	466	206	69	0
1972	70.5	226.8	4	2	1	0	0	190	151	68	0	0
1973	54.1	141.5	2	0	0	0	0	48	0	0	0	0
1974	75.9	316.1	8	3	2	1	0	311	148	78	14	0
1975	78.5	220.1	6	5	2	0	0	325	166	22	0	0
1976	76.3	306.3	4	1	1	1	0	142	70	42	7	0
1977	63.3	182.1	1	1	0	0	0	80	30	0	0	0
1978	54.5	227.6	2	2	1	0	0	122	60	32	0	0
1979	77.7	237.9	10	2	2	0	0	539	111	41	0	0
1980	67.5	130.9	0	0	0	0	0	0	0	0	0	0
1981	74.9	201.7	4	1	1	0	0	125	32	4	0	0
1982	59.1	138.9	2	0	0	0	0	42	0	0	0	0
1983	60.9	371.2	5	3	1	1	0	511	264	108	41	0
1984	60.0	214.4	1	1	1	0	0	73	57	28	0	0
1985	63.0	266.0	6	2	1	0	0	192	71	25	0	0
1986	72.2	231.4	9	5	1	0	0	478	203	32	0	0
1987	56.1	223.1	4	2	1	0	0	79	36	16	0	0
1988	63.2	186.8	6	2	0	0	0	162	30	0	0	0
1989	78.0	191.6	14	5	0	0	0	559	89	0	0	0
1990	72.9	213.3	16	3	2	0	0	578	87	23	0	0
1991	66.6	198.3	15	1	0	0	0	203	32	0	0	0
1992	63.8	145.8	8	0	0	0	0	226	0	0	0	0
1993	49.5	150.9	2	0	0	0	0	111	0	0	0	0
1994	63.5	212.8	14	2	1	0	0	445	58	15	0	0
1995	84.4	349.1	35	9	1	1	0	939	221	63	27	0
1996	80.2	204.7	7	2	1	0	0	263	55	5	0	0
1997	63.7	135.3	1	0	0	0	0	31	0	0	0	0
1998	75.2	463.8	5	2	1	1	0	694	293	175	68	0
1999	68.5	265.2	17	3	2	0	0	432	92	59	0	0
2000	58.8	162.3	6	1	0	0	0	118	3	0	0	0
2001	66.4	212.8	12	3	1	0	0	371	101	8	0	0
2002	59.4	164.9	4	2	0	0	0	211	64	0	0	0
2003	56.6	190.2	5	1	0	0	0	161	32	0	0	0
2004	84.1	770.0	22	4	2	1	1	818	343	180	99	46
2005	64.5	233.7	8	2	1	0	0	191	47	36	0	0
2006	73.8	248.1	15	4	2	0	0	597	221	69	0	0
2007	53.5	141.2	4	0	0	0	0	67	0	0	0	0
Means												
Full record, 1949-2007												
	70.2	258	6.98	2.63	1.15	0.32	0.07	352	141	53.9	12.0	1.7
Pre-dam, 1949-1966												
	73.8	261	4.72	3.06	1.50	0.39	0.06	406	195	72.1	12.6	1.3
Post-dam, 1968-2007												
	68.4	250	8.00	2.43	0.98	0.28	0.05	325	113	43.7	10.9	1.6
Change	-7%	-4%	+69%	-21%	-35%	-29%	-10%	-20%	-42%	-39%	-14%	+25%

5 Hydraulic modelling between the Aniwhenua and Matahina dams

5.1 Extent of modelling

Hydraulic modelling of the Rangitaiki catchment between the Aniwhenua and Matahina dams (including the Matahina reservoir, but not Aniwhenua or either of the dams) has been undertaken to consider the ability of the Matahina dam to safely pass high magnitude flood events. These safety assessments are based on peak water levels in the Matahina reservoir making use of the discharge relationships supplied by TrustPower.

The following events have been modelled:

- A Probable Maximum Flood discharge from the Aniwhenua reservoir, based on the 72-hour design event supplied by Bay of Plenty Energy (BOPE)
- A breach of Aniwhenua dam (i.e. spill gate failure)

A simple one-dimensional hydraulic model of the Rangitaiki River between the Aniwhenua and Matahina dams was developed using HEC-RAS[3], version 3.1.3.

The full model covers a reach length of twenty-seven kilometres and includes seventeen surveyed cross sections and elevation data extracted from topographic maps, where appropriate to extrapolate from the survey data. Additional cross sections included in the model were interpolated from the surveyed sections; this was particularly the case in the upper reaches (immediately downstream of Aniwhenua reservoir) where the topography made obtaining survey difficult.

5.2 Aniwhenua Probable Maximum Flood

The PMF event modelled in this scenario was supplied by BOPE, representing a 72-hour event discharging from the Aniwhenua HEPS. The supplied hydrograph is the modelled discharge from the structure, so was used as the upstream boundary condition with no representation of either the reservoir or the structure. For the design event the peak discharge from Aniwhenua HEPS is 2140 m^3/s.

The results of the Aniwhenua PMF modelling are shown in Table 13 for both model runs. The results show that with a peak inflow to the Matahina reservoir of 2110 m^3/s the use of the dewatering tunnel has no significant effect on either water levels in the reservoir or discharge from the Matahina HEPS.

The peak water level is slightly higher if the dewatering tunnel is not used, but this is to be expected. In both cases the peak reservoir level is approximately 77.00 m RL – this is at the upper reach of the reservoir's extreme flood range (76.80 – 77.00 m RL), but is considerably lower than the dam crest level (79.248 m RL).

It can therefore be concluded that the Matahina HEPS can safely pass the 72-hour design PMF event from the Aniwhenua HEPS.

[3] HEC-RAS is a hydraulic modelling program developed by the US Army Corps of Engineers.

Table 13

Aniwhenua PMF results at Matahina dam

Scenario	Initial WL (m RL)	Peak WL (m RL)	Peak inflow (m^3/s)	Peak outflow (m^3/s)
72-hour PMF, dewatering tunnel used	75.25	77.06	2110	2110
72-hour PMF, dewatering tunnel not used	75.25	77.00	2110	2108

Note that due to modelling approximations these values are indicative only.

5.3 Aniwhenua dam break

Following discussions with TrustPower and a visit to the Aniwhenua HEPS, two possible dam break scenarios were identified: a failure of the canal embankment and a failure of the spill gates.

An "instantaneous" failure of the spill gates was considered to be the worst-case scenario for an Aniwhenua dam break. During this scenario the two main gates would fail with the reservoir level at maximum. The scheme would then effectively become run of river, although no additional flood event was modelled (i.e. the inflow to the Aniwhenua reservoir was set to a constant value, for this purpose taken to be 35 m^3/s). Although this scenario would not release any greater volume of water into the lower sections of the Rangitaiki, the breach would occur quicker than a canal collapse and as such the resulting flood wave would be more severe.

The final modelled scenario was a gate failure. This was modelled in HEC-RAS with a failure time of ten minutes (0.166 hours) and a breach width of 19.30 metres (the combined width of the spill gates).

Of relevance to the Matahina HEPS, there would be about 2 hours warning of a dam break at Aniwhenua Dam, and the river level would be expected to rise approximately 0.5 m. Table 14 shows the flows, depths and velocities at key points along the Rangitaiki River

Table 14

Key river parameters before and after Aniwhenua dam break

Location	Velocity: Base flow / maximum flow (m/s)	Flow rate: Base flow / maximum flow (m^3/s)	Rise in water level (m)	Time for hydrograph rise to commence (hours)	Time for hydrograph to recede (hours)
Aniwhenua Dam	1.6/7.0	35/1123	1.91	0	1.42
Waiohua village	1.4/3.5	35/649	3.42	0.33	2.92
Matahina inlet	0.5/4.22	35/340	0.48	1.16	16.58
Matahina Dam	0.01/0.03	35/140	0.53	1.33	35.67

From the point of view of safety of Matahina dam, the following conclusions can be drawn:

- The peak flow rates from a dam break at Aniwhenua are well dissipated, initially in the river channel but also in the Matahina reservoir;

- The volume of the Aniwhenua Reservoir is not large enough to cause a problem with water level rise in the Matahina reservoir in the event of a failure;
- There are several hours of warning that would allow the operation of Matahina to be adjusted to accommodate the release from Aniwhenua, should that be considered necessary.

5.4 Matahina Probable Maximum Flood

The probable maximum flood at Matahina was estimated by Works Consultancy (1993). This predicted a PMF inflow to Matahina of 2510 m^3/s. This is greater than the Aniwhenua PMF, because the catchment area to Matahina is greater. As a result of flood routing through the Matahina reservoir, Works estimated a PMF outflow of approximately 2300 m^3/s. The Matahina PMF estimate is currently being updated as part of the dam safety review process, and is not part of the resource consent documentation.

The safety of the Matahina dam under PMF flows is discussed in Tonkin & Taylor (2009).

5.5 Matahina spillway capacity

The spillway capacity has been modelled in a steady state, with the HEPS discharging at the same rate as inflow to the reservoir. Under existing operating conditions this will not necessarily happen. The results (see Table 15) do however show that there is sufficient spillway capacity to pass each of these flood events without exceeding the maximum design flood level (76.80 m RL). The model was run without the use of the dewatering tunnel[4].

Table 15

Matahina spillway capacity

Return period, years	Flood flow, m^3/s	Lake Matahina peak water level, m RL
25	600	76.62
50	690	76.66
100	780	76.69
200	870	76.72

5.6 Flood attenuation modelling

As discussed in Section 4.3, there is limited available storage in the Matahina reservoir for attenuation of floods. At a meeting with EBoP on 18 March 2009, EBoP requested TrustPower to consider the practicability of using a lower extreme minimum reservoir level to provide additional flood attenuation.

Table 16 presents the results of flood attenuation modelling for three different flood magnitudes and reservoir drawdown levels. The modelling is based on the July 2004 inflow flood hydrograph, scaled for the specified peak inflow floods. The modelling assumes optimum reservoir operation within the consent rules on maximum rate of change of downstream flow and with 0.4 m/hour maximum reservoir drawdown rate. Optimum operation assumes foreknowledge of the inflow flood

[4] The dewatering tunnel allows drawdown of the reservoir when required for dam safety and repair work.

hydrograph, which is never available, and that the spillway gates will be continually adjusted to take account of changing reservoir inflow and level.

With present flood forecasting ability, it is likely that practicable flood attenuation is about half of the reductions shown in Table 16. This is consistent with TrustPower and Opus modelling of the July 2004 flood (see Section 6.4.3). The figures indicate that there would be only marginal flood attenuation benefit from adopting a lower minimum reservoir level. Against this must be set the risks and cost of greater drawdown: in the event that a forecast flood did not occur, there could be unnecessary downstream damage caused by a rapid draw down, and an inability to refill the reservoir during the flood would result in generation losses. Greater drawdown may also result in slumping of sediment deposits within the reservoir.

Table 16

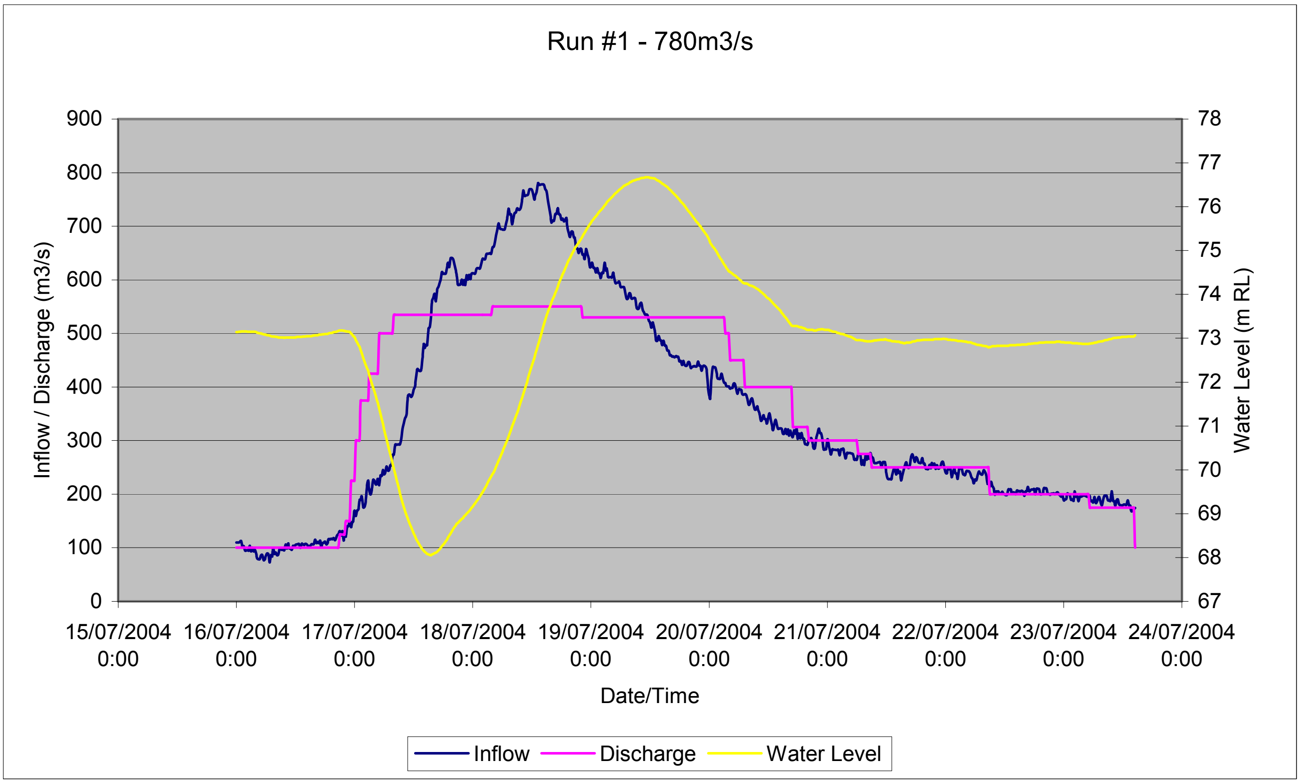
Optimum flood attenuation

Flood event return period, years	Peak inflow flood, m^3/s	Minimum reservoir level		
		71.6 m RL (existing consent limit)	70.0 m RL	68.0 m RL
Live storage volume below 76.8 m RL		11.2 Mm^3	14.4 Mm^3	18.2 Mm^3
100	780 m^3/s	600 m^3/s -23%	570 m^3/s -27%	550 m^3/s -29%
25	600 m^3/s	435 m^3/s -28%	415 m^3/s -31%	390 m^3/s -35%
14	500 m^3/s	350 m^3/s -30%	330 m^3/s -34%	310 m^3/s -38%

Figure 9 shows one example of the attenuation modelling. It illustrates that the reservoir drawdown would need to have commenced two days before the flood peak in order to achieve the optimum 29% peak attenuation. This indicates that better flood forecasting is a key element of flood management. Figure 9 also illustrates that the duration of high flows is extended by the attenuation, which may have implications for the stability of downstream stopbanks.

Figure 9

Optimum flood attenuation with reservoir drawdown to 68.0 m RL



6 July 2004 Rangitaiki flood

On the 16-19th July 2004 the Eastern Bay of Plenty Region experienced a significant storm event resulting in widespread flooding in the Rangitaiki and Whakatane catchments. The Rangitaiki River suffered severe flooding with the worst occurring at the town of Edgecumbe, where a breach of the stopbank during the event resulted in inundation of a significant area, including the Transpower substation and Fonterra dairy factory.

This section of the report analyses the July 2004 storm event and the effect of the Matahina HEPS on the resulting flood. This considers the storm hydrology, causes of flooding in Edgecumbe and the operation of Matahina HEPS during the storm event. The HEPS at Matahina, and to a lesser extent Aniwhenua, can have an effect on flood flows in the Rangitaiki catchment and this section will also consider whether any changes to the operation of Matahina could have served to limit the severity of flooding in Edgecumbe.

6.1 Event hydrology

The July 2004 storm event occurred due to the presence of a tropical low pressure system moving southwards from the sub-tropics and meeting a slow moving frontal rain system located over the north island. At the same time a blocking high system was present to the east of New Zealand, preventing the easterly movement of either the tropical low or the frontal rain system. This combination of weather systems resulted in three days of sustained heavy rain across the north island.

At 8:30 pm on Friday 16th July the Metservice issued their first heavy rain warning, indicating that rainfall of up to 120 mm could be expected through to midnight Saturday 17th. At 8:20 am on the Saturday morning this warning was revised with a further prediction of 100-140 mm of rain over the following 16-24 hours. At the time this revised warning was issued 100-150 mm of rainfall had already been recorded across the Rangitaiki catchment.

Rainfall intensity across the mid and lower catchment was high, with recorded intensities of between 5 and 10 mm/hr from 08:20 through to 15:00 on Saturday 17th. This subsequently eased to between 2 and 5 mm/hr through to end of the storm event on the afternoon of Sunday 18th. In total between 220 and 300 mm of rain was recorded over the 72 hour period in the mid to lower catchment – recorded rainfall in the upper catchment was approximately 120 mm.

Because of the catchment geology there was a difference in runoff between the greywacke sub-catchments in the east of the catchment and the pumice ash in the west, resulting in a flood event with both a rapid peak and a long duration. Extreme rates of runoff were recorded in the lower eastern sub-catchments of Whirinaki and Waihua, with the total runoff volumes approximately 85% of total rainfall. The slow response nature of the pumice deposits in the west is reflected in the total duration of the flood event; based on TrustPower's assessment of flood duration as being when inflow to Matahina exceeds generation capacity (160 m^3/s), the total duration was eight days.

The total flood inflow to the Matahina reservoir (covering only the period when reservoir inflow exceeded generation capacity) was 240 million cubic metres (note maximum live storage in the Matahina Reservoir is only 11.2 million cubic metres (refer Table 9). This cumulative volume equates to 50% of the total rainfall volume over the catchment.

Peak inflow to the Matahina reservoir was 747 m^3/s. Peak outflow from the HEPS was also 741 m^3/s, with discharge exceeding 500 m^3/s for 44 hours.

Table 17

Peak recorded flows for July 2004 flood

Gauge	Catchment area (km^2)	Peak flow (m^3/s)
Rangitaiki @ Murupara	1,184	57
Rangitaiki @ Aniwhenua	2,456	540
Rangitaiki @ Matahina	2,844	750
Whirinaki @ Galatea	534	250
Waihua	45	141

6.2 Catchment flooding

Some minor flooding occurred immediately upstream of the Matahina reservoir as a result of the accumulation of logs and other debris that was washed into the Matahina reservoir during the flood event. The accumulation of this debris caused a backwater effect and although flood levels were raised, no significant flooding resulted; there has been no mention of any significant flooding occurring between the Aniwhenua and Matahina HEPS. Most of this section of river is through gorges but there is a section of open, flat farmland immediately upstream of the Matahina reservoir. Given the reported discharges it would be expected that some inundation may have occurred in this area, but there are few properties.

During the flood event, the most significant flooding occurred at the township of Edgecumbe, located approximately 24 km downstream of the Matahina dam. Edgecumbe has a population of around 1700. Despite its small size there is significant infrastructure in town, namely a large Fonterra dairy factory, a Transpower sub-station and State Highway 2 (SH2). The town is located mainly on the west bank of the Rangitaiki River, with the Fonterra site and Transpower sub-station on the east bank.

Because of this infrastructure the township has extensive flood protection works, including stopbanking to prevent inundation from the Rangitaiki River and a 200-m wide emergency floodway (known as Reids floodway), which will activate during large flood events (610 m^3/s, equivalent to a 40-year flood standard). Despite these flood protection works, there is also an ongoing problem with stormwater flooding during heavy storm events due to the flat, low-lying topology. During the 2004 event there was extensive surface flooding in the area surrounding Edgecumbe.

At 10:40 on Sunday 18th the stopbank upstream of Edgecumbe breached on the right bank at Sullivans Bend, 400 metres upstream of the Transpower sub-station. The stopbanks were designed for a flow of 780 m^3/s with a 250 mm freeboard, but at the time of the breach the discharge in the Rangitaiki was an estimated 610 m^3/s as the Reids floodway had just come into operation.

The stopbank breach developed to a width of 150 m with a peak discharge of 250 m^3/s from the river into the floodway. This discharge is more than twice the design capacity of the floodway and resulted in a washout of the floodway bank downstream of McLean Road.

Following the stopbank breach the railway bridge and the western approach to SH2 crossing the floodway were both washed out, with flow constriction between the road and rail bridges raising water levels to 4.84 m RL. The Fonterra site was flooded with water up to 1 m deep in places. There was also extensive flooding of farmland to both the east and west of the floodway.

6.3 Causes of flooding

Although the plains downstream of the Matahina HEPS are prone to surface flooding because of the flat topography, the extent of local surface flooding in July 2004 was minor compared to the extent of flooding caused by the stopbank breach at Edgecumbe (based on a flood extent outline supplied by EBoP). Therefore the stopbank breach is considered to be the primary cause of flooding through Edgecumbe.

The stopbank breach occurred at a discharge of about 600 m^3/s, which is considerably lower than both the flood peak (750 m^3/s) and the design standard of the stopbank (780 m^3/s). Opus (2006) identify that the breach occurred as a result of a failure of the pumice sub-soil layer underlying the stopbank, rather than a failure of the stopbank itself. The Rangitaiki plains are underlain by a layer of pumice ash laid down following an eruption of Mount Tarawera and this had previously been identified as a potential weakness in the stopbank defences. During the 2004 event the hydraulic pressure of the floodwater caused a geotechnical failure of this pumice layer two metres below the stopbank at Sullivans Bend, causing the stopbank to be undermined and piping to occur under the stopbank. This in turn resulted in the stopbank breach and subsequent inundation of Edgecumbe.

6.4 Matahina HEPS operation

6.4.1 General operation during floods

When considering the effect of the operation of Matahina during the 2004 flood event, it must be remembered that the dam was designed and is operated as a hydroelectric rather than flood attenuation scheme. The penstock and spillway levels are designed to maximise the head of water over the turbines and the shape of the reservoir – long and narrow – provides a relatively low increase in storage volume as lake level increases. In contrast a flood detention reservoir would be maintained at a low operating level and would ideally have a large surface area reservoir such that the increase in storage with rising lake level would be large.

The operation of Matahina is also governed by rules laid out in the scheme's resource consent. These rules govern the maximum absolute change and rates of change in total discharge and reservoir level, see Section 4.3.

6.4.2 Operation during the July 2004 flood

Prior to the July 2004 flood event, the reservoir level was at 74.15 m RL, which is approximately one third of the normal operating range (see Table 10). Initial predictions for the storm event were based on a heavy rain warning issued by the Metservice at 8:30 pm on Friday 16th July, giving a rainfall warning of 100-120 mm over twenty-four hours. On this basis no pre-emptive drawdown of the reservoir was undertaken and the reservoir had a storage capacity of approximately 3% of the total flood volume. Inflow to the reservoir prior to the flood event was 80 – 100 m^3/s, which is around the average winter inflow.

On the morning of Saturday 17th July reservoir levels began to rise rapidly, with the reservoir rising by 200 mm/hr when an updated weather warning was received at 8:20 am. At 10:30 am the scheme began to spill (at 40 m^3/s) and the normal operating reservoir level (76.20 m RL) was exceeded at 11:30 am, when spill was 80 m^3/s. Because of continually increasing inflows the total discharge from the scheme was ramped up to 348 m^3/s at 12:45 pm with the agreement of EBoP staff. Following a further heavy rain forecast at 3 pm, the discharge was increased to 500 m^3/s and then 576 m^3/s at 5:30 pm due to continued rise in reservoir levels.

On Sunday 18th total discharge increased to 700 m^3/s at 9:30 am following continued rise in reservoir level. At 11:55 am the reservoir level peaked at 77.08 m RL. Because of concerns over the structural safety of the dam with water levels beyond 77.10 m RL, agreement was reached between TrustPower and EBoP that reservoir levels would not exceed 77.10 m RL and discharge would match reservoir inflow. On the Sunday afternoon reservoir inflow and scheme discharge peaked at 747 and 741m^3/s respectively.

Over the duration of the flood event, total discharge exceeded 500 m^3/s for a total of 44 hours. If more accurate forewarning – with regard to reservoir inflows as well as rainfall – had been available it may have been possible to drawdown the lake level to the minimum operating level of 71.6 m RL to provide some additional storage. Given the flood volume of approximately 155 Mm^3 (when inflow to Matahina exceeded 300 m^3/s only), this would likely have only given an additional 2% total storage. Even lowering the reservoir to 68.0 m RL, as discussed in Section 5.6, the additional storage volume below 71.6 m RL is only 7 Mm^3, or 4.5% of the flood volume. As spill would have had to start earlier the total duration of the downstream flood event would also have increased. Because the worst downstream flooding resulted from a stopbank breach at Edgecumbe that was caused by prolonged seepage, such an operating scenario may have been counterproductive, possibly initiating the stopbank breach earlier.

6.4.3 Optimal operation

Modelling of alternative operational scenarios by TrustPower (2004) indicates that with sufficient warning and reservoir drawdown the peak discharge could have been limited to around 690 m^3/s while maintaining a reservoir level below the maximum flood level of 76.80 m RL. Even under this optimal practicable operation, the peak discharge is still greater than the discharge at Edgecumbe at the time of the stopbank breach.

As the breach occurred prior to peak discharge in the Rangitaiki, there is no evidence that the operation of the Matahina scheme during the flood was responsible for the inundation through Edgecumbe. Following the flood event EBoP commissioned a study of flood hazard in Edgecumbe (Opus, 2006). This report considered possible alternative operating scenarios at Matahina during the flood event based on the operating rules specified in the Matahina discharge resource consent. This report concluded that, had sufficient prior warning been available, it may have been possible to draw the lake down sufficiently to attenuate the peak downstream discharge to around 600 m^3/s. Whether this would have had any impact on flooding is not clear, as this reduced discharge is still equal to the discharge at which the stopbank breach occurred. This scenario would also have required a sudden and significant increase in discharge from Matahina, which would have increased the total duration of the event downstream. Opus assessed that the resulting extended period of high flow would have increased the scour damage to banks and the risk of stopbank failure due to seepage.

The practicability of flood attenuation is also discussed in Section 5.6.

7 River bank erosion

7.1 Erosion mechanisms

River bank erosion along the Rangitaiki River downstream of the Matahina dam has been an issue for many years, and the subject of a number of studies. This section summarises the potential factors in bank erosion.

7.1.1 Fluvial entrainment

River bank erosion is the natural result of meander loop development for a meandering river on a wide floodplain, due to fluvial entrainment of bank material during floods. Fluvial entrainment is primarily related to channel velocity; it may also be the result of a sediment deficit in the flow downstream of Lake Matahina. As a result of the distribution of velocity in open channel flow, fluvial entrainment is most severe near the base of the banks. This can lead to the removal of toe support for river banks and increased pore pressures on the declining limb of flood hydrographs, resulting in bank failure and channel widening.

Increased channel velocity

Potential factors causing higher velocity are:

- The increased bed gradient caused by the artificial straightening of the channel downstream of Thornton in 1913.
- The increased velocity immediately upstream of the 1987 Edgecumbe earthquake fault.
- Increased channel velocity during floods, as a result of stopbanking the river downstream of Te Teko in the 1970s.
- The increased channel velocity during peaking compared to run-of river operation.
- Increased channel velocity occurs at the outside of bends; in conjunction with secondary currents this leads to erosion of material from the toe of the bank.

The artificial straightening downstream of Thornton, and the Edgecumbe earthquake, would both have had only minor effect on the overall river gradient (see Figure 6), and therefore on general river velocity. It can be concluded that, other than locally increased velocity at the earthquake fault, the ongoing impact from these changes is significantly less than (and indistinguishable from) the dominant contributing factors.

The effect of stopbanking downstream of Te Teko is considerable during floods, because the stopbanks are typically 2-3 m above floodplain level, and the channel velocity increases with increased water depth. For example, if the channel depth during flood increased from 4 m to 6 m as a result of the stopbanks, the velocity would increase by about 30%.

Sediment deficit downstream of Lake Matahina

Prior to completion of the Matahina dam in 1967, the high sediment load in the river resulted in a rising river bed and coastal progradation[5] of the Rangitaiki plains. These processes are likely to

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[5] Progradation is the seaward build up of a beach or delta by the nearshore deposition of sediment transported by a river and/or by longshore current.

have been halted or substantially reduced as a result of the trapping of sediment upstream of Matahina dam (since 1967) and the Aniwhenua dam (since 1980).

The Matahina dam repairs carried out from 1996 to 1998 would have resulted in significant sediment input to the river due to the lower reservoir level at that time.

By attenuating minor floods in the reservoir, the number of sediment transporting flood events downstream of the dam has been reduced, but when floods do occur they could result in bed degradation or bank erosion such that the sediment load equals the transport capacity of the river during flood. Such erosion would be expected during floods, primarily at the upstream end of the reach below the dam. It would be evident either as bed degradation or as bank failure and channel widening.

Comparison of river cross-section surveys between July 2001 and July 2008 (Beca, August 2008) shows both local degradation and aggradation, but no long term trend for degradation. It is therefore concluded that the river gradient is now stable following any degradation after the initial construction of the Matahina dam.

7.1.2 Bank weakening

Bank weakening processes that potentially cause bank instability are:

- Draw-down effects from river water level fluctuations (due to floods and HEPS peaking operation).
- Rainfall runoff over and through the banks.
- Bank instability resulting from removal of riverside vegetation.
- Stock damage to river banks.
- The apparent difficulty of establishing erosion protection vegetation on river banks within the zone of fluctuating water levels.
- Fluvial erosion at the toe of a bank can lead to over-steep banks and mass failure.

Draw-down effects refer to bank failure due to the excess pore pressure within the river banks on the declining limb of a flood hydrograph,

7.2 River bank erosion studies

Works Consultancy (1988) reported on the influence of the Matahina HEPS on river bank stability along the Rangitaiki River. The investigation included analysis of the flow regime, field investigation of all 29 identified sites with bank instability problems between Matahina dam and the river mouth, laboratory analysis of bank samples, and analysis of aerial photography. Key conclusions of the study were:

- Processes related to fluvial entrainment (i.e. river flow erosion) appear to be the primary cause of most of the bank instability problems.
- Draw-down effects and other processes likely to be aggravated by the daily operating regime were identified as the primary factor at only one of the 29 sites, and the rate of bank retreat at that site is relatively slow.
- Laboratory test results suggested the banks could be vulnerable to draw-down effects under some operating regimes. Further work would be required to quantify the operating limits.
- A historical analysis of bank retreat at site 6 (present Erosion Monitoring Site 12, see Map 1) using aerial photographs covering the period from 1945 to 1987 strongly suggested that significant bank retreat only occurs during major floods (probably greater than 5-year return period), regardless of the Matahina HEPS operating regime.

- The Matahina HEPS operating regime at that time was not a significant factor in bank instability problems downstream of the dam.
- The water level fluctuations associated with the 1980 twin peaking trial were larger than those of the current regime, being commonly 1.4 – 1.7 m. The concerns about bank stability from the trial may have arisen largely from a heightened awareness of existing problems, particularly as many of the sites then causing concern were identified in the1988 study as primarily related to river flow erosion.

EBoP (January 1992) reported on monitoring of a 2-year trial operating regime for Matahina HEPS with higher ramping down rates (the current ramping rates), stating: "*the trend in erosion has been reasonably stable indicating that a change in operating conditions has not resulted in a change to the pattern or rate of erosion ... The badly eroding sections downstream of the dam have been like this for some years … In general then the new operating conditions have not contributed to additional erosion. Erosion of the river below the dam has always been a problem with a certain amount being natural and some no doubt resulting from water level fluctuations from the dam.*

Resource Solutions (June 1998) compared aerial photographs of Sites 11 and 12 at various pre- and post-dam stages (1944, 1966 and 1985). Key conclusions were:

- Erosion at the sites is most severe in the pre-dam period 1944-1966, with a rate of 6.45 m/year measured at Site 12, compared with 0.57 m/year post-dam (1966-1985). This was consistent with experience elsewhere that indicates the ability of dams to provide a buffer against flood flow tends to slow down the rate of erosion.
- Land uses are contributing to localised erosion at the sites (Site 12 in particular), where runoff from pasture is contributing to piping and the formation of tomos at levels above the 'water line'.

Resource Solutions (July 1998) carried out an inspection of the Rangitaiki River immediately following a flood. Key conclusions were:

- The Edgecumbe earthquake the previous year had a profound impact on bank stability.
- Stock damage to the riverbank and stopbanks was evident at a number of places (particularly downstream of Te Teko).

Beca (2001) prepared an AEE for twin peaking operation of the Matahina HEPS. The report addresses in detail the proposed operating conditions, the existing environment, including erosion characteristics, and the effects of the proposed changes. The report identified that the 1980 twin peak trials most likely aggravated bank instability through the aggressive ramping rates applied at the time: the trial used ramping down rates of up to 62.2 m^3/s/hour, compared to the maximum rate of decrease of 30 m^3/s/hour for the first hour under existing consent conditions.

Beca (2005) reported on the 2002-2004 twin peak trial period monitoring. The objective of the trial was to compare the impact of twin peaking with the previous single peak regime. The trial period included continuous water level and turbidity monitoring, cross-sectional surveys at 15 erosion monitoring sites at 6-monthly intervals, visual inspections at 3-monthly intervals, and groundwater level monitoring and assessment of drawdown effects. The report concludes:

- The adoption of a double peak generation regime will not cause any significantly greater erosion problems than the single peak regime over most of the river banks.
- Observed erosion is primarily triggered by flood events or riparian activity, and not by the effects of twin peaking.

The erosion monitoring sites are shown on Map 1. Erosion at Site 12 was measured at 30.6 m over the period July 2001 to October 2004 (average 9.4 m/year). The erosion rate between individual surveys ranged from 1.0 to 21.8 m/year, with higher rates of erosion generally related to more frequent flood events above a 133 m^3/s threshold.

Beca (August 2008) reported on the biennial Rangitaiki River monitoring carried out in accordance with consent conditions 5.5 and 5.6. The report noted that extensive bank protection works had been carried out by EBoP since the July 2004 flooding, that 5 new areas of bank erosion did not involve major erosion and are unlikely to be due to scheme operation, and that the river bed level was generally within the range surveyed since 2001.

The **Rangitaiki hydrogeology and riverbank stability report** (Appendix B) analyses stability at Sites 10 and 14, using seepage and slope stability modelling. These sites were selected for analysis because of their potential susceptibility to draw-down effects. The report concludes that:

- It is the extreme high flows that have the greatest potential for erosion.
- Piping is not an issue at Site 10. Natural conditions at Site 14 indicate that the hydraulic gradient is steep enough to potentially cause piping within the pumiceous soils. Peaking operation and natural flood events slightly increase the hydraulic gradient, however hydraulic exit gradients are greatest during natural flood events.
- Both sites are marginally stable during natural conditions. Peaking lowers the factor of safety by approximately 10%; this may result in shallow surficial erosion which is common in meandering rivers. Natural flood events cause much larger draw-down effects and have a greater impact on the stability of the river bank than the daily peaking regime.
- The peaking regime is unlikely to cause significant increased erosion compared to that which occurs under natural conditions.

7.3 Discussion on bank erosion

The preceding sections have identified potential mechanisms by which the Matahina HEPS may be affecting bank erosion. These are listed and then discussed below:

1. The sediment deficit in the river flow downstream of the dam may result in fluvial erosion in the reach downstream of the dam.

2. Peaking operation may result in fluvial erosion as a result of the higher velocity during peaking compared to run-of river operation.

3. Peaking operation may cause bank instability due to draw-down effects.

4. Peaking operation may result in banks without vegetative cover being more vulnerable to fluvial erosion.

The effects identified at the time that Matahina HEPS was last consented are in Bay of Plenty Catchment Board, October 1989:

> *"Bay of Plenty Catchment Board and Electricorp have spent considerable sums of money in stabilising the river banks over the years. The causes of erosion are most likely a combination of mechanisms to which the river is subject. The contributing factors are, river flow stresses, <u>fluctuation due to dam operation</u> and tides, stock damage and rainfall. At a particular site one mechanism may be contributing more than others to erosion, but other mechanisms will accelerate the erosive process."*

The erosion sites noted in the 2008 biennial monitoring report (Beca, 2008) are predominantly located at the outside of bends, which is indicative of fluvial erosion.

7.3.1 Fluvial erosion due to sediment deficit downstream of Matahina dam

The cross-section evidence shows that any bed degradation resulting from the sediment deficit has now ceased, and that the bed profile of the river downstream of the dam is stable, with only local degradation and aggradation visible between surveys. The 2008 biennial monitoring (Beca 2008) noted bed degradation of up to 0.5 m in the reach from Matahina dam to Site 3 (21.6 km from the dam), but that the bed profile was generally within the range surveyed since 2001. The evidence concerning the effect of the sediment deficit on bank erosion is not conclusive. Resource Solutions (June 1998) concluded that the rate of erosion at Site 12 had decreased since construction of the dam, but Beca (2005) recorded a faster rate over the relatively short period 2001-2004, which included the major July 2004 flood. It seems most likely that fluvial bank erosion is primarily a function of flood flow, and that any effect due to the sediment deficit is minor, and restricted to the reach just downstream of the dam.

The absence of ongoing bed degradation is an indication that bank erosion due to the sediment deficit is likely to be minor. Any exacerbation of fluvial erosion by the sediment deficit is a consequence of the dam, and not of the operating regime.

This effect is minor and no mitigation is warranted. Of note is that if bank erosion is occurring as a result of the sediment deficit, then protecting the banks would merely transfer the problem elsewhere. Therefore any bank protection works should be limited to protection of stopbanks and infrastructure, and not of farmland.

7.3.2 Fluvial erosion due to increased velocity from peaking operation

The evidence from erosion monitoring inspections is that bank erosion occurs primarily during flood events, and that the velocities occurring during normal peaking do not result in significant erosion.

As the number and extent of flood events is a natural phenomenon, the scheme's contribution to this effect is minor and no mitigation is warranted.

7.3.3 Bank instability due to draw-down effects under peaking operation

The hydrogeology study (Appendix B) has concluded that draw-down effects under the present peaking regime are unlikely to cause significant increased erosion compared to that which occurs under natural flood conditions every year.

This effect is minor and no mitigation is warranted.

7.3.4 Fluvial erosion due to lack of vegetation in the range of peaking water level fluctuation

EBoP considers that the fluctuating river level due to peaking makes it difficult to establish vegetative protection on the banks. Other factors also hinder vegetation establishment (e.g. steep banks and stock damage).

Vegetation growth is inhibited below the normal peak generation flow of 133 m^3/s (this is flow at Matahina, which is attenuated downstream). Under natural flow conditions established vegetation would be expected on sloping banks above "normal" water level, say the mean flow of 71 m^3/s. Based on Figure 7, which models a minimum flow of 40 m^3/s, the band where vegetation growth is affected by peaking operation is up to about one metre high between the dam and Edgecumbe.

Fluvial erosion occurs primarily at bends, where secondary currents cause erosion at the toe of banks, which is below normal river level and therefore not affected by the lack of vegetation in the range of peaking water level fluctuation.

The area of water level fluctuation due to peaking is below stopbank level, therefore any fluvial erosion resulting from this effect only affects stopbank stability where there is no set back of the stopbank from the river bank.

EBoP says that as a consequence of being unable to establish vegetative protection it is usually forced to carry out erosion repairs using rock riprap. Riprap has the advantage that it provides protection against fluvial erosion below normal river level, which vegetation cannot.

Riprap is appropriate to protect assets such as stopbanks and roads threatened by erosion. In other areas, however, it might be argued that normal meander loop development should be allowed to happen without incurring the expense of riprap protection. The effect of extensive riprap protection could be to cause bed degradation or bank erosion elsewhere, because in erodible materials a river will erode until it carries sediment at its sediment transport capacity.

This effect applies only to sloping banks between Matahina and Edgecumbe. Further downstream, the water level fluctuations are dominated by tidal effects. The effect does not apply to over-steep banks where fluvial erosion at the toe of the bank leads to mass failure.

The effect can be mitigated, as at present, by the contribution of TrustPower towards EBoP river bank protection costs.

8 Proposed operating regime

8.1 Rangitaiki River – Flow constraints

The current operating regime (Condition 5.1) states that "the minimum load shall not be less than 22 MW [40 m^3/s] except when the river inflow is less than this. When the river inflow is less than 40 m^3/s no peaking is permitted." As described in Section 2.1.3, this condition causes operating problems in the turbine rough running range, and since 2007 the scheme has been operated under a revised low flow operating regime, in order to avoid operation in the turbines' rough running range. Consent is sought to formalise a revised low flow regime.

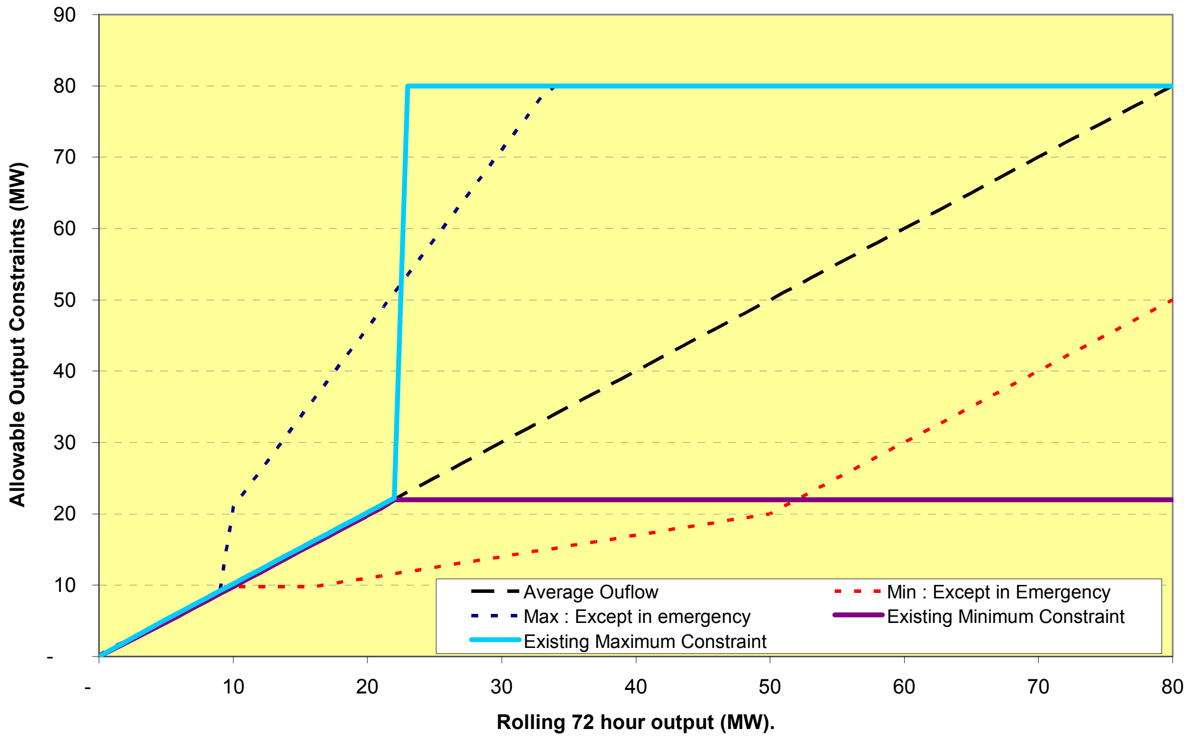
TPL also seeks greater flexibility in overall scheme operation, to allow more effective placement of the scheme into the market, and to better respond to local demand.

To provide for the desired operational flexibility, while retaining the general intent of the constraint on scheme operation, an envelope approach is being proposed and is described below.

8.1.1 Constraint Envelope

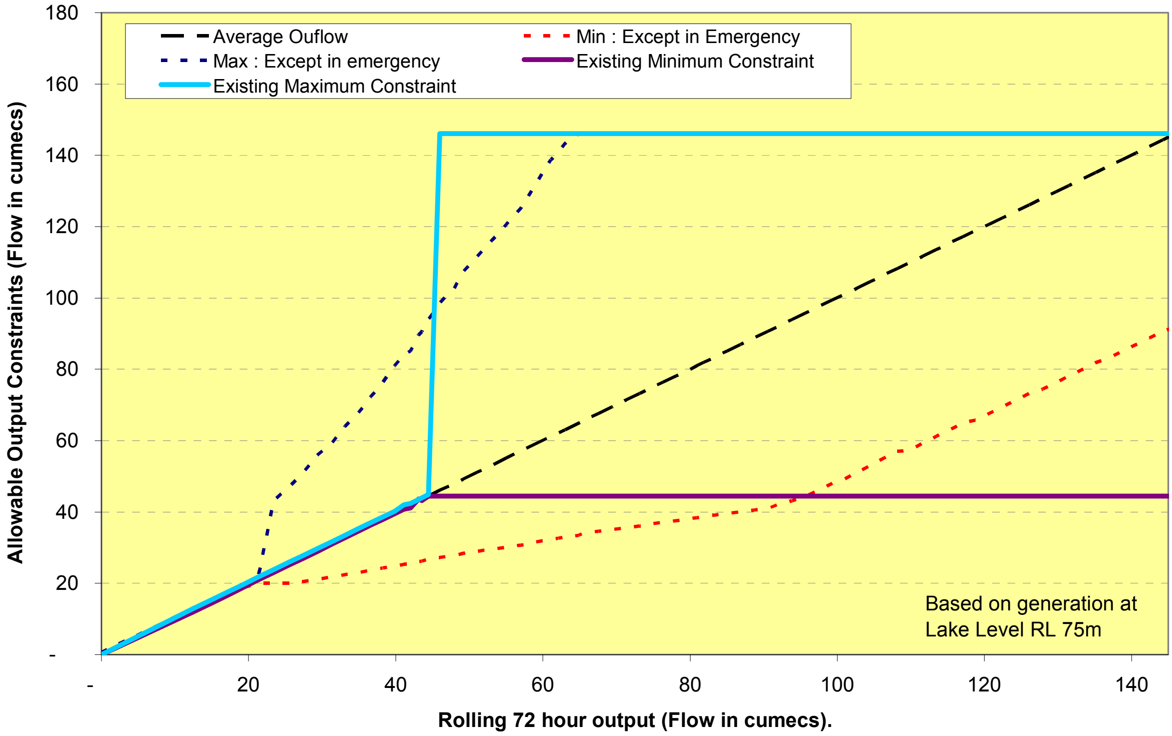
TPL proposes to operate the scheme within a defined maximum and minimum discharge which is based on average scheme outflow over the previous 72 hours. This 'Constraint Envelope' is depicted in Figure 10. The envelope has been developed around historical generation patterns and outflow. The existing constraints are also shown for comparison.

Figure 10
Proposed Constraint Envelope (MW)



The constraint envelope is defined in terms of MW output from the power station, since this is the measured parameter. Figure 10a shows the envelope in terms of flows, assuming that the reservoir is at mid-operating level (refer Section 1.3).

Figure 10a
Proposed Constraint Envelope (m^3/s)



By way of example, for a rolling average 72 hour output from the scheme of 26 MW (51 m^3/s), currently the scheme could operate a single peak of up to 80 MW (148 m^3/s), within the current ramping rates, with 22 MW (45 m^3/s) generated the remainder of the time. This is permitted within the existing constraints and gives a range of 58 MW (103 m^3/s). Under the suggested constraints the peak output would be limited to a maximum output of 61 MW (113 m^3/s) but the minimum would be reduced to 14 MW (29 m^3/s) (from 22 MW, 45 m^3/s) giving a range of 47 MW (84 m^3/s).

Likewise, two 80 MW peaks (148 m^3/s) is achieved, within existing ramping constraints, given an average output of 30 MW (58 m^3/s) and 22 MW (45 m^3/s) the remainder of the time. Again this is permitted within the existing constraints and gives a range of 58 MW (103 m^3/s) induced twice per day. Under the proposed constraints the peak output would be limited to a maximum output of 71 MW (131 m^3/s) but the minimum would be reduced to 16 MW (33 m^3/s) (from 22 MW, 45 m^3/s) giving a range of 55 MW (98 m^3/s).

For lower flows, below the current 22 MW (45 m^3/s) constraint, operational flexibility is retained to allow some peaking and provide for the rough running range on the machines. For example given an average output of 20 MW (43 m^3/s) currently no regulation is allowed. Under the proposed envelope this would change to a minimum of 11 MW (27 m^3/s) and a maximum of 46 MW (86 m^3/s).

There would be no constraint on the number of peaks each day: this will be governed by inflows, ramping rates and market demand.

8.1.2 How the Constraint Envelope is applied

The following graphs prepared by TPL provide some examples of how the constraint envelope would be applied based on a 72 hour rolling outflow (note all figures are in terms of MW). The first two are typical decay events. The other two are flood events. A "Large flood event" has not been modelled as "emergency" provisions will be included that allow the scheme to operate outside the "normal envelope".

These charts display how the "max and min" limits change (red and purple) based on the rolling average outflow (light green), the rolling average being based on the previous 72 hours of scheme output (dark green).

Inflow is irrelevant to the determination of the rolling average (and hence max and min) except for the fact that there is limited lake level to buffer differences between inflow and outflow (lake level shown as the dotted line).

Figure 11 Low flow decay example

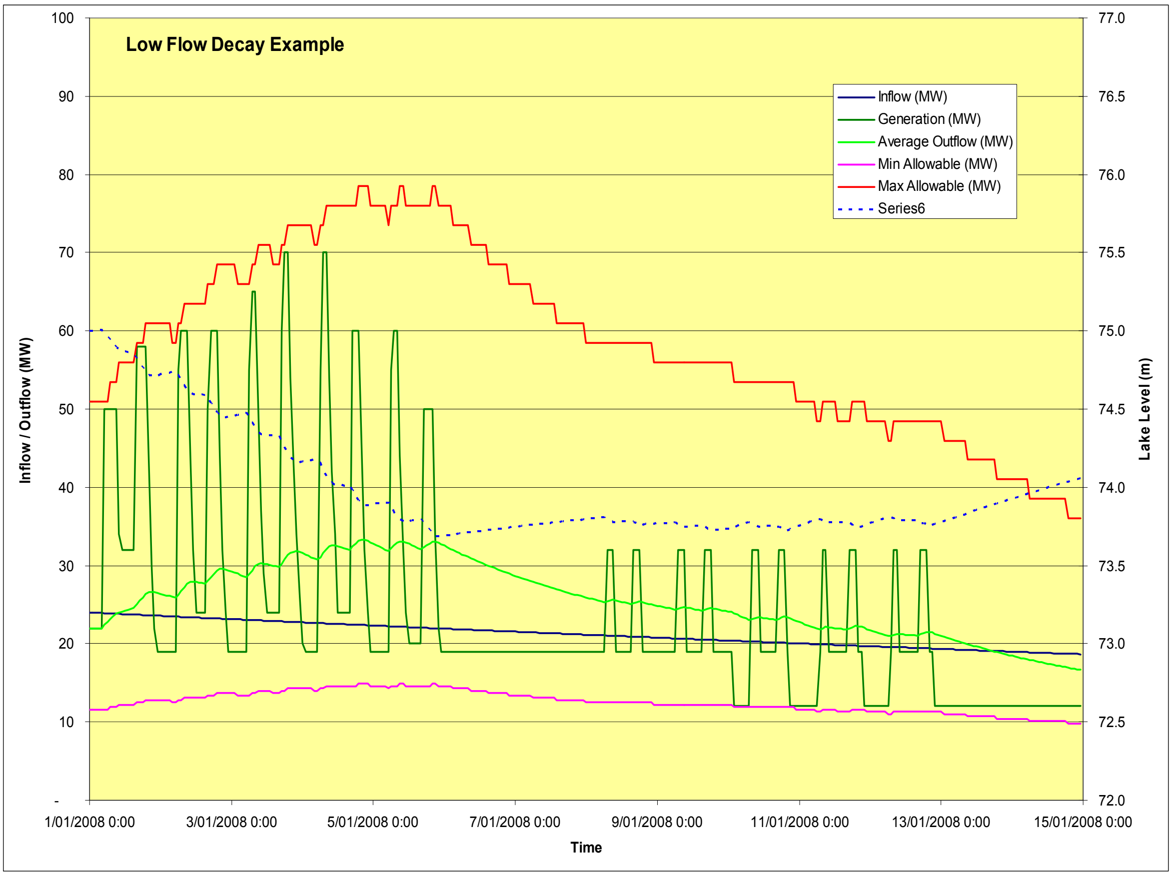


Figure 12 High flow decay example

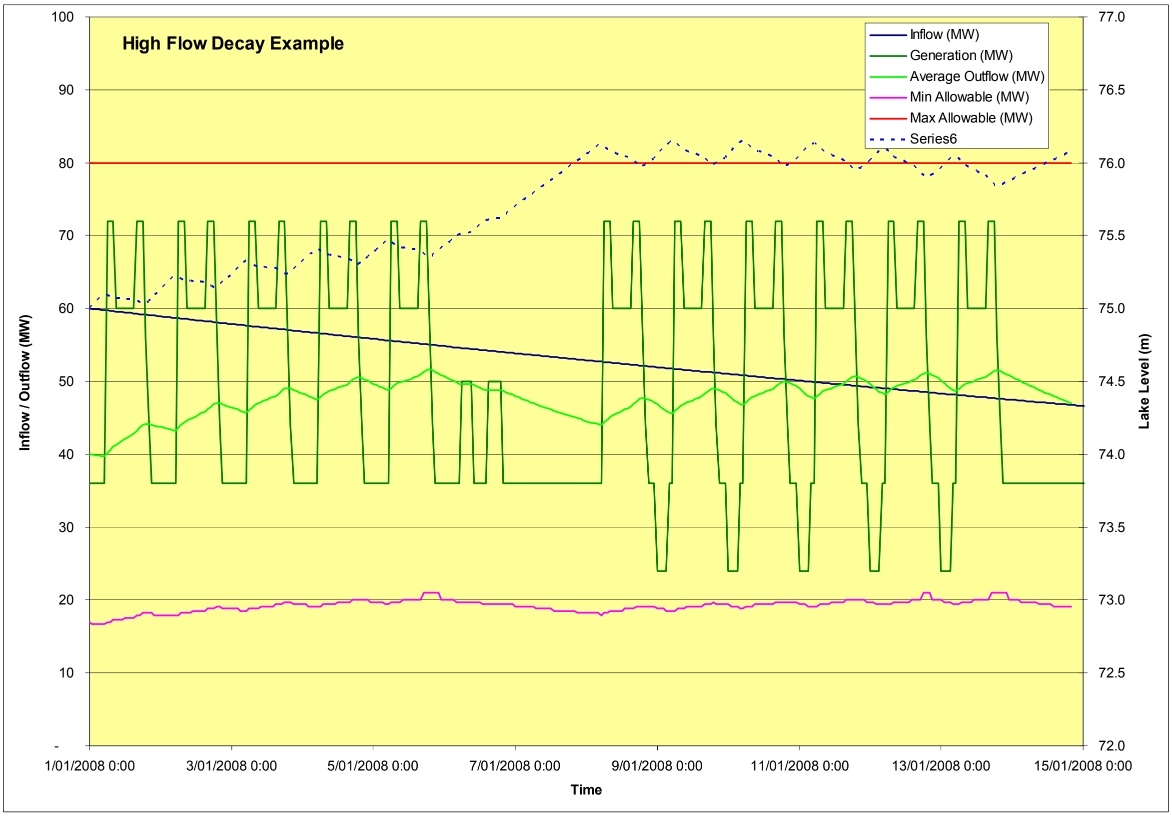


Figure 13 Small flood example

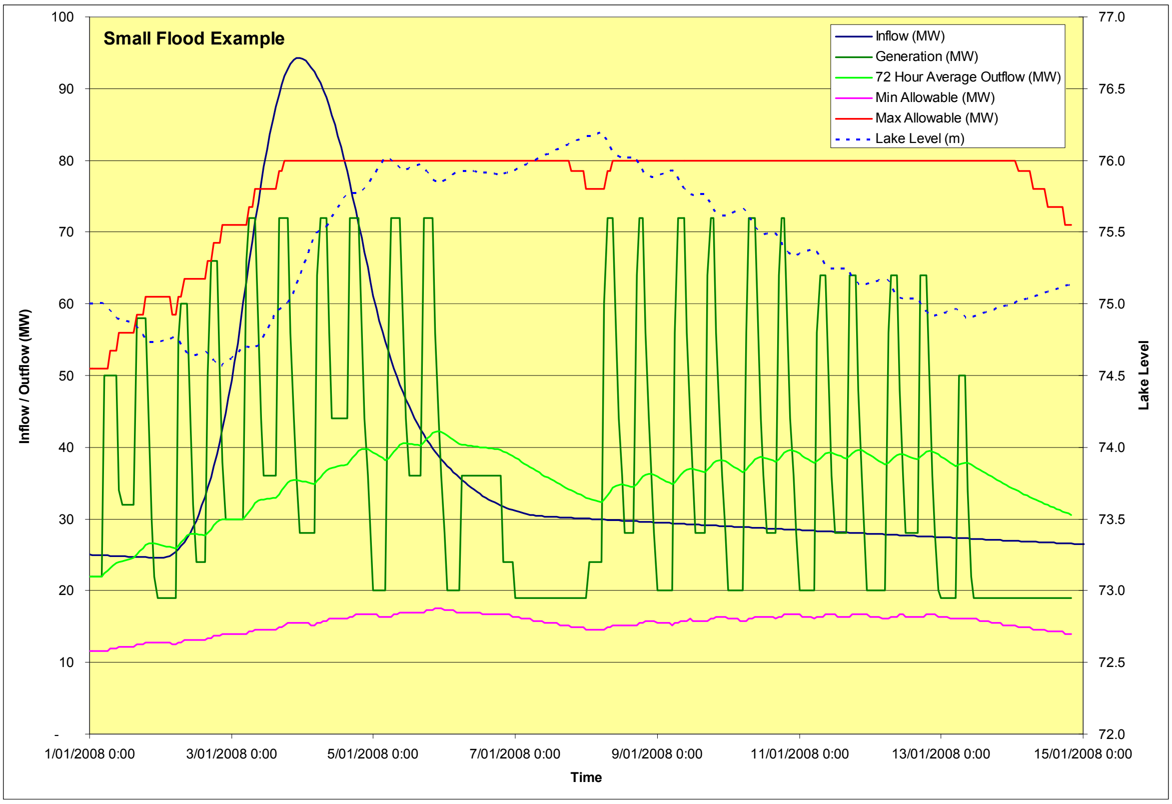
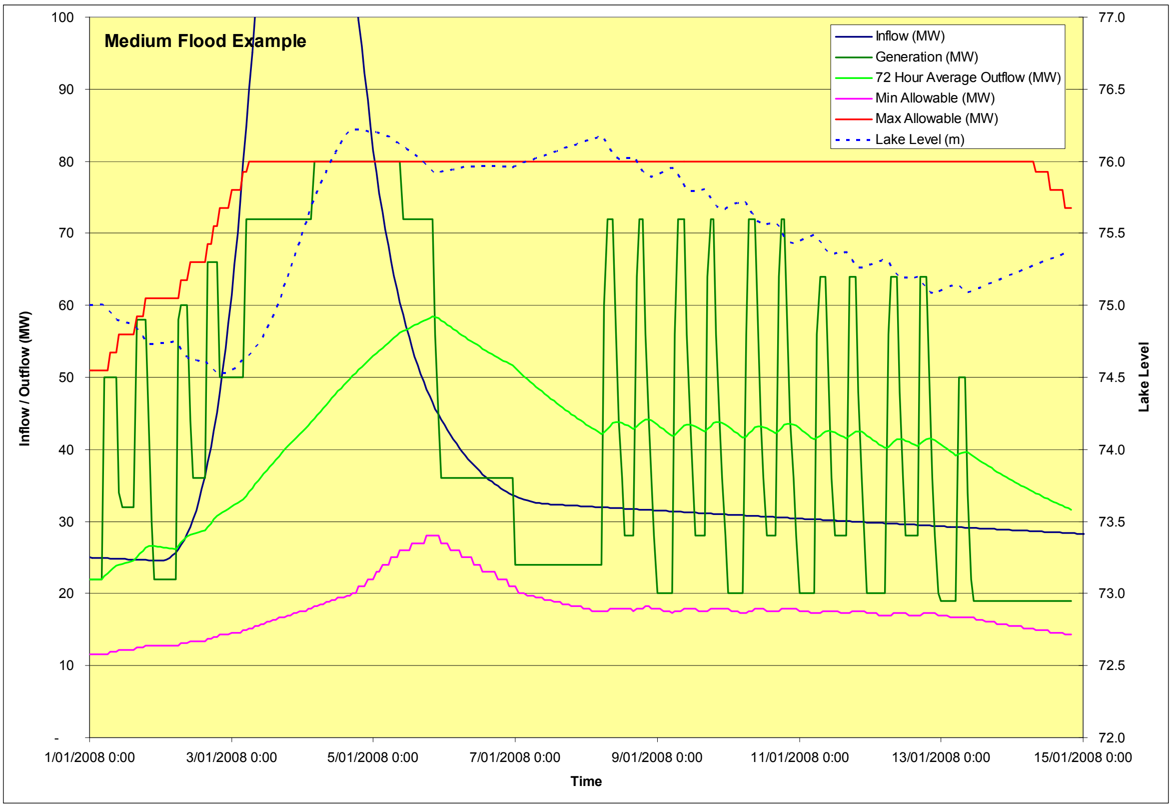


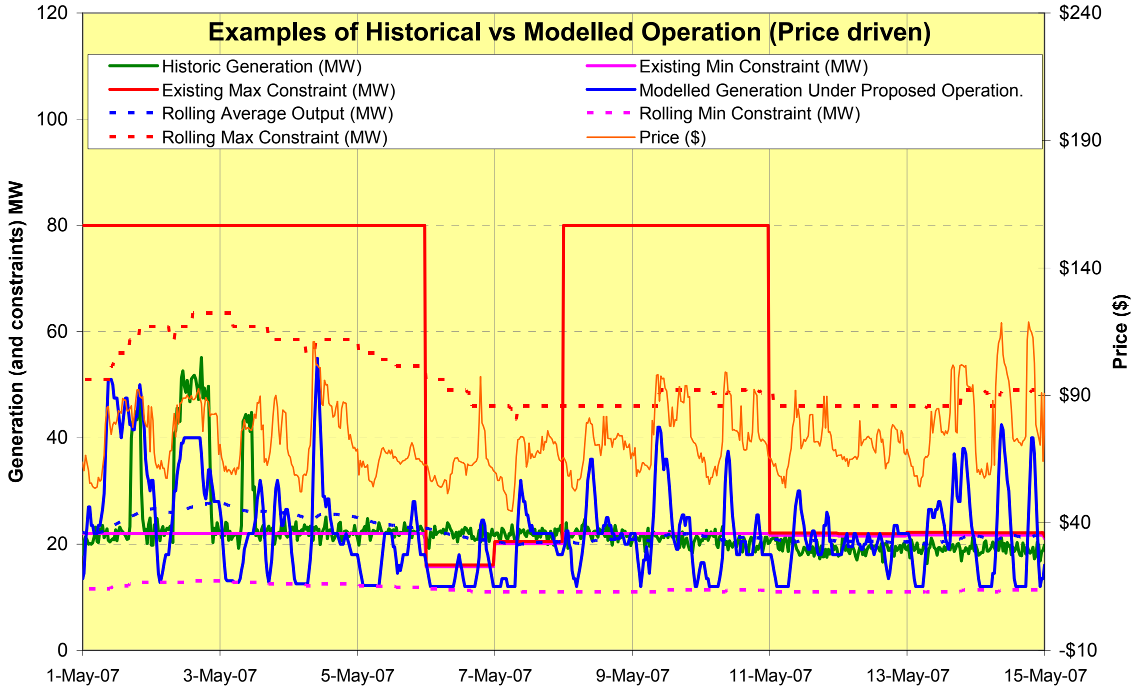
Figure 14   Medium flood example



TPL has carried out modelling of the proposed operating regime. The following charts provide some examples of the modelled output on a ½ hour basis. The proposed operation typically follows the price data profile compared to the historical operation which often shows peaking operation even during flat price data.

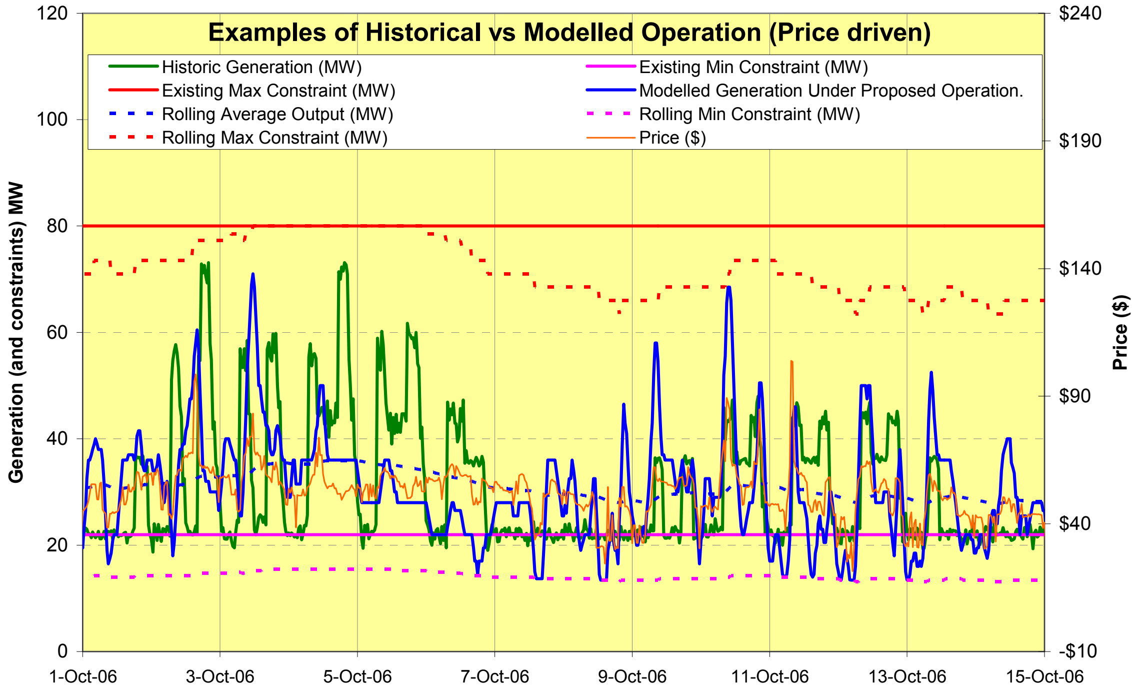
Figure 15 shows a period of low inflow where the existing operating regime required the below 22 MW of inflow constraint to be adhered to. Under the proposed regime the scheme would still be able to peak, typically in this example, between 12 and 40 MW compared to the periods of flat 22 MW operation under the historical regime.

Figure 15 Low inflow example



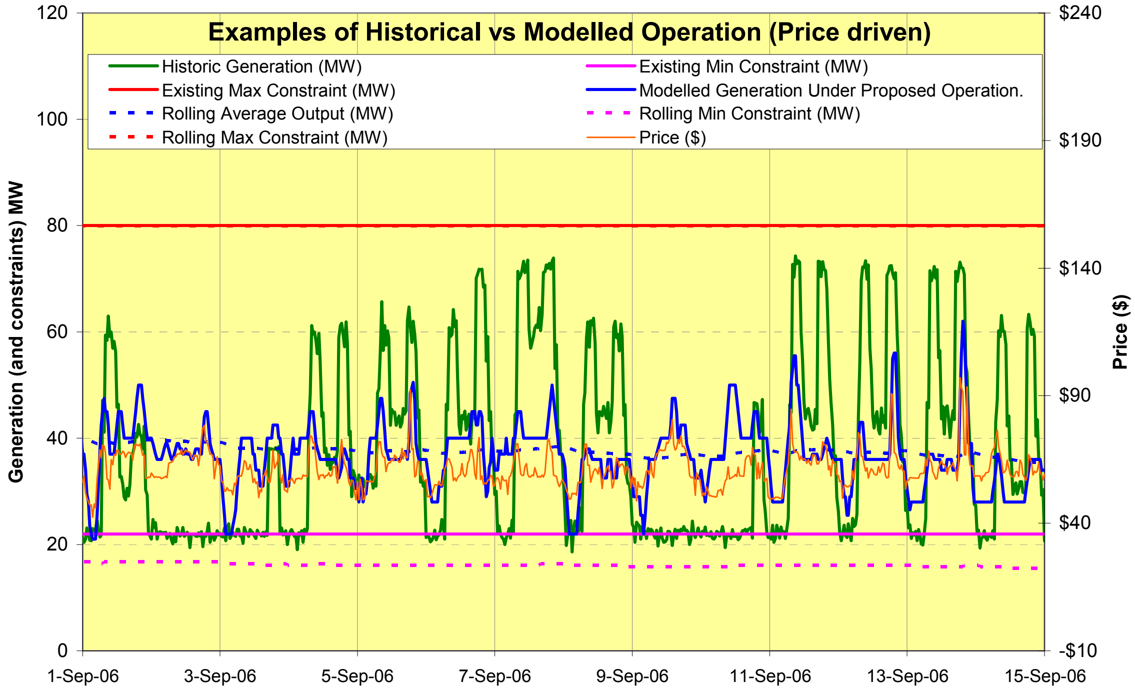
In the second example (Figure 16) flows are approximately average. Both the historical and modelled results show similar peaking however the timing and magnitude is often quite different as the modelled follows the price profile. The proposed is constrained throughout at the upper level and also shows some periods dropping below the existing 22 MW constraint.

Figure 16 Average inflow example



The third example (Figure 17) is during a period of slightly above average inflow. The historical operation shows significantly greater peaking than the modelled. This is because the peaks have to be committed well ahead for the historical situation due to the constraints under which the scheme operates. For the modelled (proposed) operation, peaks are reduced if the anticipated price rises do not eventuate. As the price is only showing minor variations, the modelled proposed generation also does not drop as low in comparison to the historical operation.

Figure 17 Above average inflow example



8.1.3 Operational Impact

The impact of the envelope from an operational point of view would be:

- the ability to respond more effectively to electricity demand;
- the ability to avoid the turbine rough running range;
- the ability to peak during low flow situations, although only minor peaks will be possible due to the limited lake storage;
- the ability to peak under low flow situations will in turn allow the lake level to rise under low flow situations;
- no limit on the number of peaks each day;
- peaks will be no greater in terms of magnitude, however the more peaks there are, the smaller each will likely be;
- lower minimum flow;
- a self-regulating system, i.e. how the scheme is operated in the future is based on recent history.

A critical consideration is the time frame over which the average scheme outflow is calculated. A time period of 72 hours has been selected due to the characteristics of the catchment (time for rainfall in the upper catchment to reach Matahina) and the way in which the scheme fits with these characteristics.

8.2 Effects of proposed Constraint Envelope on Rangitaiki River flows

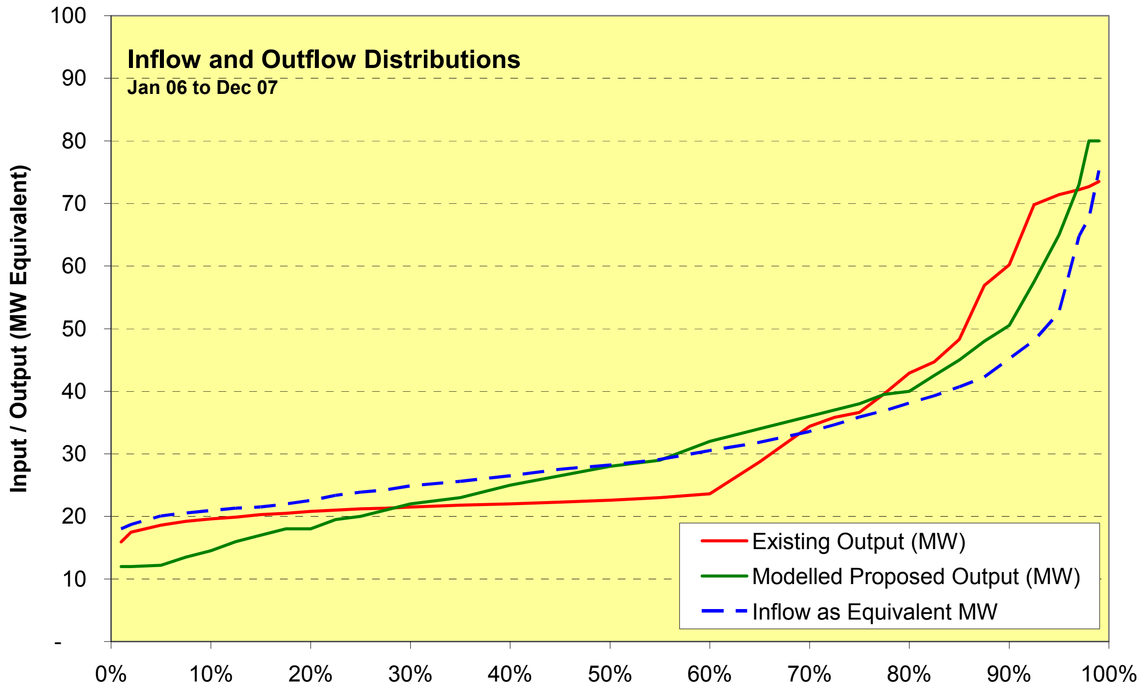
This section summarises output data from TPL modelling of the proposed operating regime.

8.2.1 Inflow and outflow distribution

See Figure 18. Approximately 25% of the time the low river flow will be less than the existing operation. Overall the distribution will follow a pattern closer to inflow given the removal of the artificial 22 MW constraint which induces the pronounced step in the historical distribution.

Historically the outflow has been below inflow approximately 70% of the time. Under the modelled regime the outflow is below inflow approximately 55% of the time.

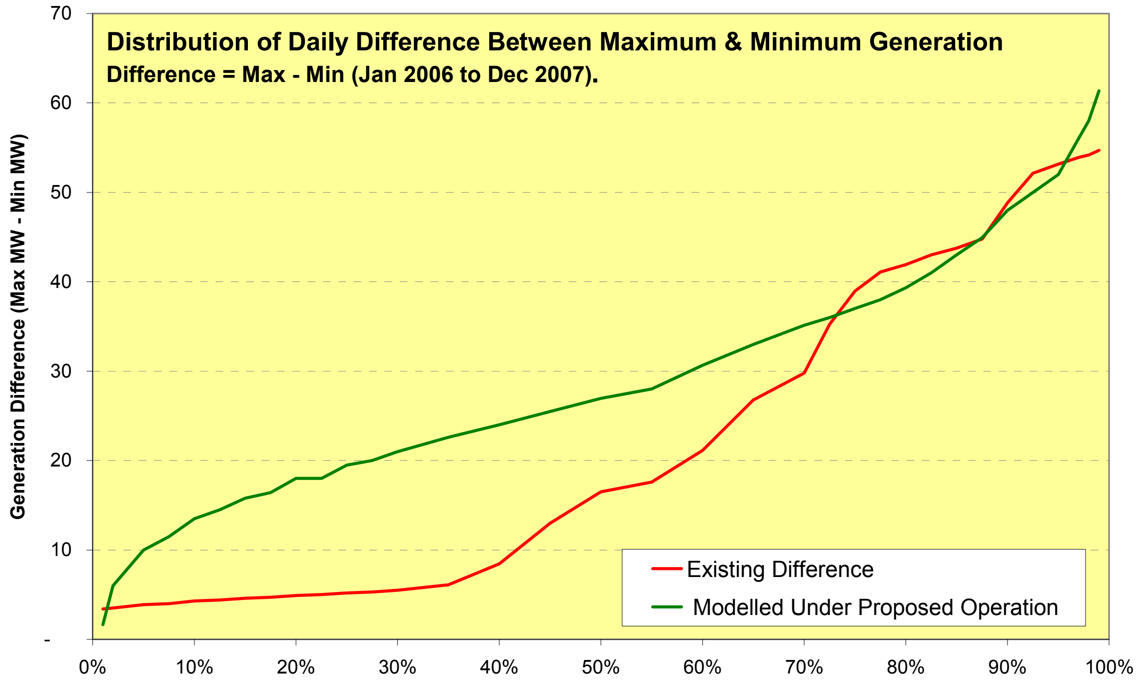
Figure 18 Inflow and outflow distributions for historical and modelled (proposed)



8.2.2 Difference between maximum and minimum generation on a daily basis

The proposed operation regime allows more flexibility at lower flows. Hence the difference (on a daily basis) between maximum and minimum generation is greater at lower flows but more or less the same at higher flows, see Figure 19 (TPL modelling). For example, under the historical generation 30% of the time the daily change was less than 6 MW. Under the proposed regime this would increase to approximately 21 MW.

Figure 19 Difference between maximum and minimum generation on a daily basis

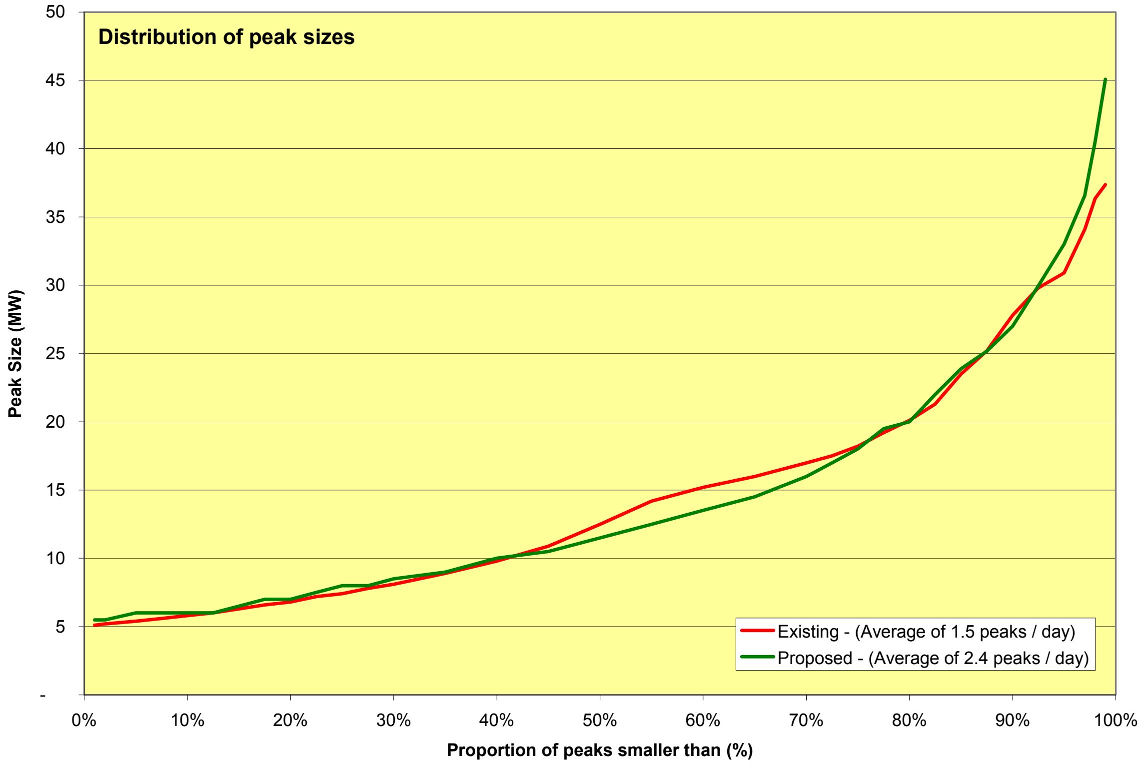


8.2.3 Distribution of peak size

Figure 20 shows the TPL-modelled distribution of peak size under the historical vs proposed regime (peaks less than 5 MW excluded). It shows that the peak sizes are equivalent but the average number of peaks per day would increase from 1.5 to 2.4

For example 40 % of peaks are less than 10 MW in both cases, 80% are less than 20 MW.

Figure 20 Distribution of peak size



8.2.4 Operational range

The proposed operational range is presented in Figures 21 and 22 as a percentage of the daily average (TPL modelling).

Figure 21 Comparison of existing and proposed discharge constraints, related to daily average generation

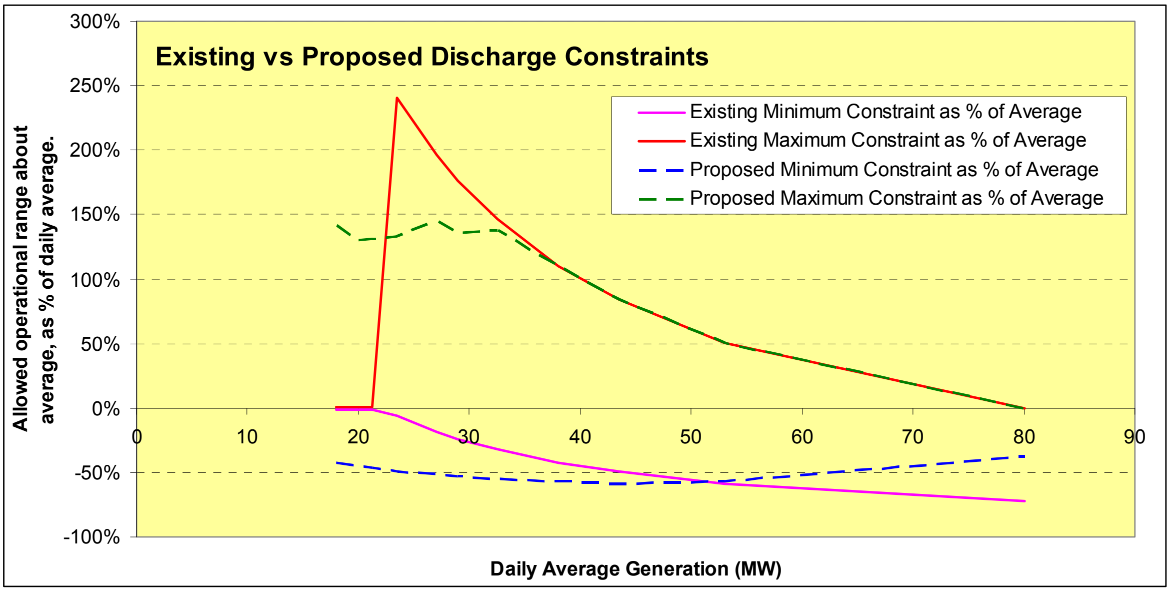
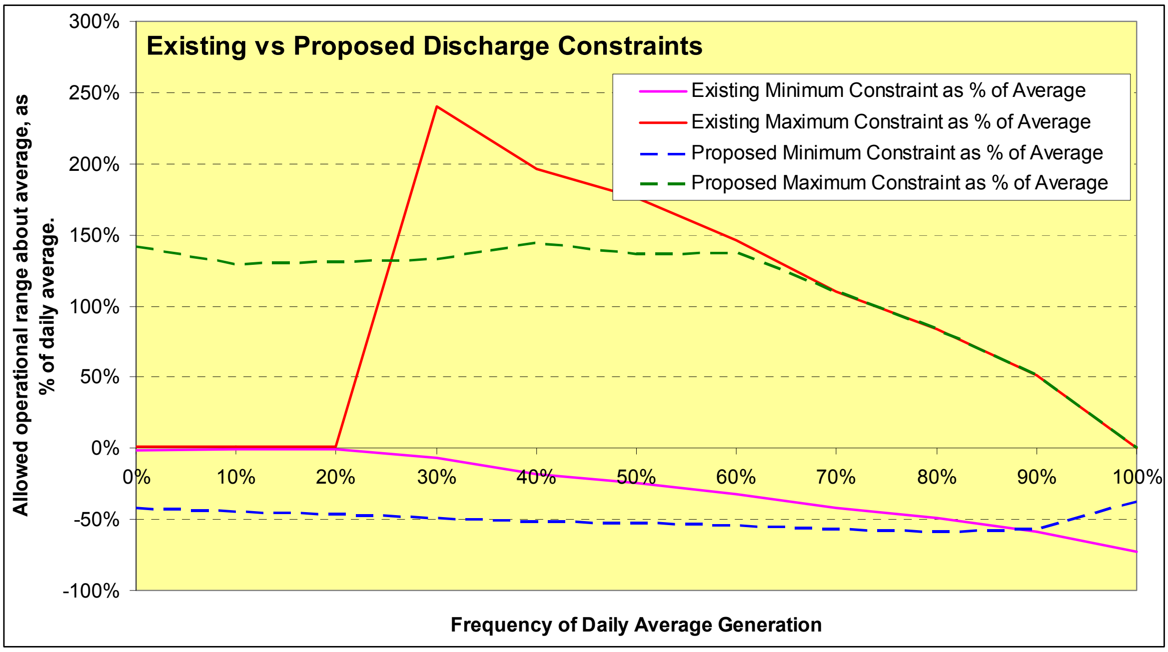


Figure 22 Comparison of existing and proposed discharge constraints, related to frequency of daily average generation



8.2.5 Hydraulic effects

The proposed operating regime allows for increased variability of flow, particularly when reservoir inflows are less than 45 m^3/s. The effect of this variability has been modelled for three 14-day periods with 50, 39 and 29 m^3/s average flows, see Appendix D. The modelling shows wider water level ranges than under existing operating practices as far downstream as Edgecumbe. At Edgecumbe the tidal water level fluctuations are just as pronounced as those due to the flow regime, and at Thornton the wide tidal range all but obliterates the effect of flow variations.

The proposed Constraint Envelope also allows for lower minimum flows. The effects of this lower minimum are:

- The river level can be lower than would be the case under the existing regime.
- The river level will be correspondingly higher after a low flow period, because the rolling average output will be no different.

The effect of lower flows on river level varies along the river, as shown on Figures D19 and D20 of Appendix D. The greatest effect, with Matahina output reduced from 22 MW (45 m^3/s) to 20 m^3/s is a drop in water level of about 0.9 m downstream of a constriction at km 4.5 and sills[6] at 12 (Te Teko) and 19.6 km from the dam. The latter is the site of the Edgecumbe earthquake fault: downstream from the fault, the effect reduces gradually to zero at the river mouth. Figure D21 shows water depths along the river: the least depth above thalweg[7] level, on the rapids at the earthquake fault, will be 0.7 m at 20 m^3/s flow; elsewhere the water depth is generally much deeper.

It is noted that 10 MW output has been used under the informal rough running regime since 2007, with a minimum flow of 25 m^3/s recorded at Te Teko.

Concern has been expressed about the effect of lower river levels on adjacent wetland reserves managed by Fish & Game. Review indicates that there will be zero or negligible effect on water levels in the wetlands:

- The Orini wetland is drained by the Reid's Canal, which connects to the Rangitaiki River 1.0 km upstream of the river mouth. At this location the maximum effect of the lower river flow is 0.04 m, which will have negligible effect on the water level in the wetland 4 km upstream.
- The Thornton wetland, immediately east of the Rangitaiki River mouth, is connected to the river via a culvert approximately 200 m upstream from the river mouth, where there will be negligible change in water level as a result of the proposed operating regime.
- The Awaiti wetland is 3 km northwest of the Rangitaiki River, and drains northwest to the Tarawera River mouth, so there will be no effect on the wetland from changes in the Rangitaiki River.

8.2.6 Saline intrusion

Federated Farmers has suggested (28 January 2009) that low river flows have resulted in the intrusion of sea water up the river, affecting water abstraction at Thornton.

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[6] A sill is a high point in the riverbed that acts to control upstream water levels at low flows. It is likely to be formed of locally harder material that is resistant to erosion.

[7] The thalweg is the lowest point on a river cross-section.

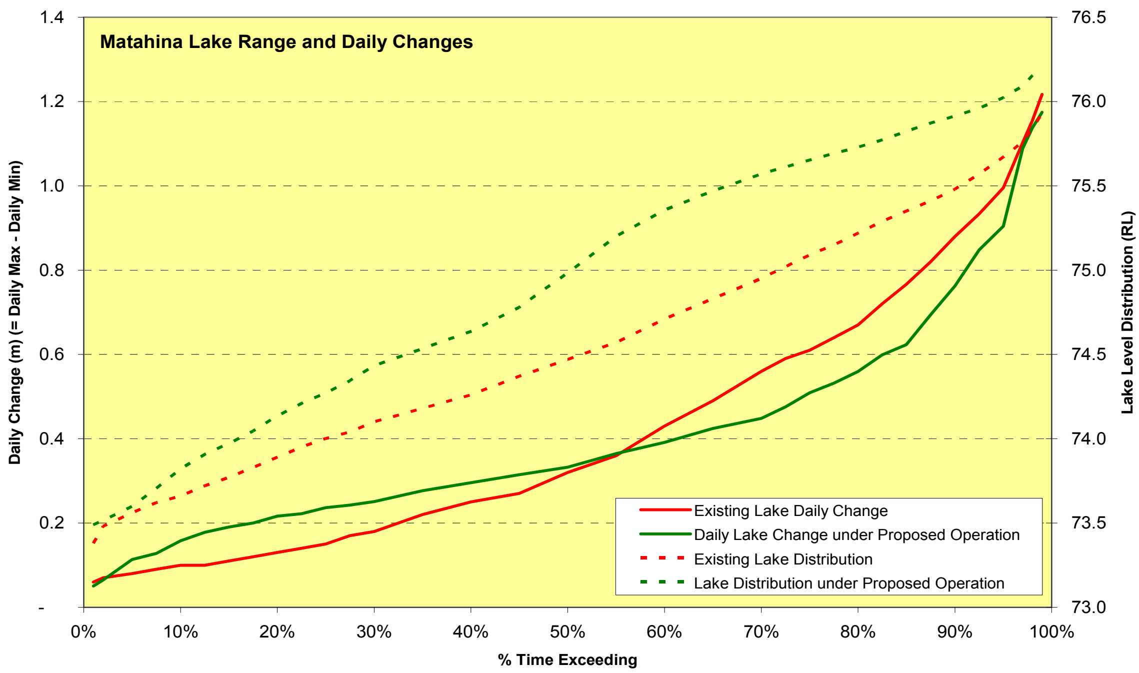
From a desktop analysis (Appendix D), it appears that some form of saline wedge will exist in the lower Rangitaiki River, or at least highly stratified conditions, for all river flows greater than 20 m^3/s. With a change down to the lower flow of 20 m^3/s, the wedge would extend further upstream, and the freshwater layer above the wedge would be shallower than at present. The wedge could extend to the upstream limit of tidal influence, the Edgecumbe earthquake fault 17 km from the river mouth, in the form of a saline near-bed layer.

8.3 Lake Matahina

No change is proposed to the current operating regime, as defined in Conditions 3.1 to 3.7 of the existing consent. The proposed operating regime will impact on lake levels, however.

TPL modelling of the proposed regime shows that the lake level will be significantly higher over the bulk (70% +) of the distribution. This is attributable to the increased operational flexibility of the proposed regime, and the ability to store water at low reservoir inflows, which is not allowed under the existing regime and results in the lake getting "stuck" at low levels. The more frequent smaller peaking under the proposed regime, results in a slightly less aggressive change in lake level on a daily basis. For example historically the lake is below RL 75 m around 72% of the time. The modelled regime indicates around 50% of the time.

Figure 23 Historical vs modelled (proposed) lake operation



8.4 Rangitaiki River – Ramping rates

There are opportunities to improve the operational flexibility of the scheme through changes to the ramping rates within the proposed constraint envelope.

8.4.1 Ramping up

The existing consent, Clause 5.3, defines a maximum load increase of 37 MW/h [70 $m^3/s/h$] under normal operation. This means that for a 45 MW twin peak scenario ramping up from 22 to 72 MW presently requires 1.35 hours; the 1.34 m water level rise at the power station is therefore at a rate of 1.0 m/h.

It is proposed to increase the maximum ramping up rate to 52 MW/h. This would allow the modelled 45 MW twin peak scenario ramp up time to be reduced to one hour, and increase the rate of water level rise at the power station to 1.4 m/h. The rate of rise will be attenuated downstream of the power station, in a similar manner to the attenuation shown on Figures 4 and 5.

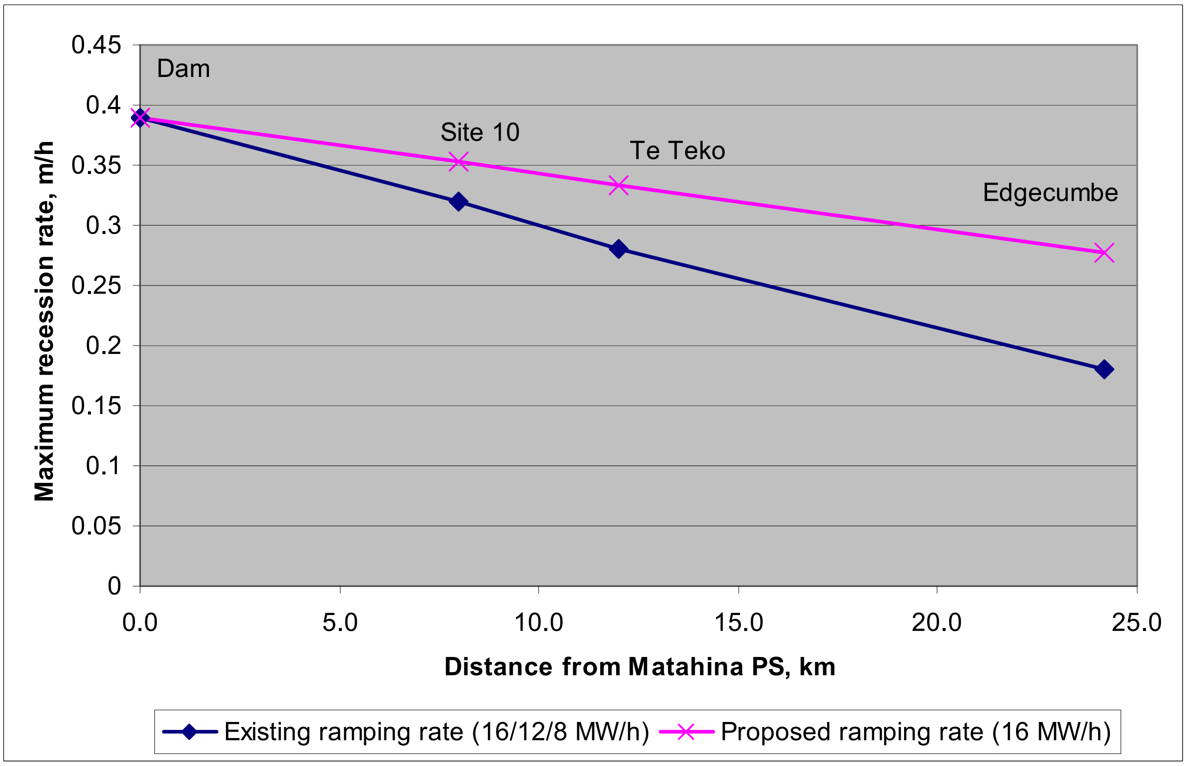
The more rapid increase in water level in the river downstream of the dam will have no effect on bank stability or other river users.

8.4.2 Ramping down

The existing consent, Clause 5.4, defines a maximum load decrease of 16 MW/h for the first hour, 12 MW/h for the second hour, and 8 MW/h thereafter. It is proposed to increase the maximum ramping down rate to a constant 16 MW/h.

Figure 24

Projected Recession Rates for 45 MW Twin Peak



The effect of the proposed ramping rate on peak recession rate has been estimated using results from previous modelling of peaking, see Figure 24. There will be no change in peak recession rate at the dam, which corresponds to the 16 MW/h maximum load decrease; it has been assumed that as a result of attenuation downstream (seen in Figures 4 and 5), the peak recession rate at Edgecumbe is a function of the average ramping down rate. For the critical 45 MW twin peak scenario, the existing ramping rates are 1 hour at 16 MW/h, 1 hour at 12 MW/h and 3 hours at 8 MW/h: average 10.4 MW/h. The peak recession rate at Edgecumbe under the proposed 16 MW/h ramping rate is therefore projected to be increased from 0.18 m/h to 0.28 m/h (0.18 times 16/10.4). A straight line projection is used for intermediate sites, as shown in Figure 24, with the peak recession rate at Site 10 increased from 0.32 to 0.35 m/h, and at Te Teko from 0.28 to 0.33 m/h.

The hydrogeology study (Appendix B) shows that the higher recession rate is not likely to cause piping or slope stability failure.

8.5 Flood conditions

TrustPower is discussing with EBoP possible changes to the current conditions, as defined in Clauses 6.1 and 6.2 of the existing consent. Flood attenuation modelling in Section 5.6 indicates that there would be only marginal benefit from adopting a lower minimum reservoir level.

8.6 Discussion on proposed new operating conditions

The preceding sections have identified the following potential hydraulic and bank erosion effects of the proposed new operating conditions.

8.6.1 Potential impact on Matahina lake level

As discussed in Section 8.3, the proposed new operating regime is expected to result in the lake being maintained at a higher water level.

8.6.2 Potential effect on downstream river level and depth

As discussed in Section 8.2.5, the proposed 20 m^3/s minimum flow will result in water levels up to 0.9 m lower than at 22 MW (45 m^3/s) at three locations, see Figures D19 and D20 in Appendix D. Elsewhere the reduction is less, reducing to zero at the river mouth.

Figure D21 shows the effect on water depth: at the shallowest points (km 12, Te Teko, and km 19.6, the earthquake fault) the depth is reduced from 1.5 to 0.9 and from 1.0 to 0.7 m respectively, for a flow reduction from 45 to 20 m^3/s. Elsewhere the water is generally much deeper.

8.6.3 Potential impact on salt water intrusion

As discussed in Section 8.2.6, the lower minimum flow in the river is expected to result in a longer and deeper saline wedge in the lower Rangitaiki River. This is unlikely to affect existing floating intakes, but may impact on other takes downstream of Edgecumbe.

8.6.4 Potential impacts on existing water takes

Lower river levels and saline intrusion will adversely affect six of the 12 existing consented water intakes and an unknown number of permitted takes from the Rangitaiki River downstream of Matahina dam.

Table 18 lists the consented takes in geographical order from the river mouth (refer Map 2), and outlines potential mitigation for intakes affected by low flows. Further details are given in Appendix C. The effects on existing abstractors can be mitigated, for example by modifying or replacing intakes (by a well at one site).

Table 18

Impacts and mitigation at existing consented takes

Consent No.	Intake description	Abstractor comments on effect of 2007/2008 low flows (25 m^3/s)	Comments and potential mitigation for flows down to 20 m^3/s
21008	Flexible pipe resting on river bed. Not in use.	Not in use	Will be affected by saline intrusion. Replace with floating intake if required.
21224	Floating	Daily cleaning of weed	None
21063	Suction pipe fixed to waratah and adjusted as required	Problems with vortex and air entrainment	21063 and 21852 are close to each other, at a location where the water depth close to the river bank is too shallow for an effective intake at low river levels (low flow with low tide, and likely to be affected by saline intrusion. Proposed mitigation is to provide a shared well for the two abstractors. An alternative could be to provide new floating intakes at another location where the river depth near the bank is greater, but this would need an easement to run pipes across an adjacent landholding.
21852	Suction pipe fixed to driven steel pipe and adjusted as required	Added extra length of pipe to extend into deeper water	
21703	Floating	No problems	None
62000 (Fonterra)	Floating	Screens grounded on riverbed. Used pump jetting to lower the bed level at the intake	Ongoing pump jetting of the bed is unlikely to be satisfactory. Proposed mitigation is to move the intake 50 m downstream, where the bed level adjacent to the bank is deeper because it is at the outside of a bend.
21923	Floating	No problems	None
60427	Adjustable suction pipe hangs off bank-mounted A-frame.	No problems	None
63177	Screen in frame on riverbed, connected to drum float: lifts off bed in flood	Cannot get screen low enough	Replace intake with an alternative design. For example as used at 60427.
62566	Pump in drum in river	No problems	None
20967	Fixed suction pipe	No problems	None
64967	Two fixed suction pipes (separate irrigation and frost protection)	Vortex at intake, air in pump	Extend the suction pipes to lower level.

Permitted takes are limited to 15 m^3/day. In areas not affected by saline intrusion, provision of, say, 20 m^3 storage tanks would allow water to be stored over periods when the river is too low to abstract from. In the tidal zone, it is expected that any permitted takes that do not use floating intakes will need replacement floating intakes or wells: monitoring of salinity during low flows is recommended to assess the need to mitigate for saline intrusion. The effect on abstractors can be mitigated as discussed above.

8.6.5 Potential impacts on wetlands

As discussed in Section 8.2.5, the lower water levels will have zero or negligible effect on adjacent wetland reserves as the only hydraulic connection with the river is close to the mouth, where water level changes due to the proposed regime are minimal.

8.6.6 Potential impacts on river bank stability

The hydrogeology study (Appendix B) has concluded that the current and proposed operation of the Matahina HEPS is not likely to cause piping or slope stability failure regardless of the operating regime adopted.

Monitoring is recommended two years after the introduction of new operating conditions, to identify any significant change to the pattern or rate of erosion as a result of the changed operating conditions. The monitoring should be a visual inspection by jet boat between Matahina and Edgecumbe.

9 Actual and potential environmental effects

9.1 Past effects of the scheme

River morphology effects that arose from the initial commissioning of the scheme, that are not now changing noticeably, are:

1. The creation of Lake Matahina behind the Matahina Dam.
2. Fluvial deposits at the upstream end of Lake Matahina, and a delta extending to the deep portion of the lake.
3. The trapping of most of the sediment transported into the lake by the Rangitaiki River.
4. The degradation of the river bed downstream of the Matahina dam, due to the trapping of upstream sediment in Lake Matahina.

No mitigation is proposed for these effects.

9.2 Effects of current operation of the scheme, where equilibrium has been reached

River morphology effects that arise from the current operation of the scheme, where equilibrium has been reached and there is no ongoing trend, are:

1. The creation of a zone along the river banks where fluctuating river levels inhibit the establishment and maintenance of vegetative bank protection. This effect extends from the Matahina dam to Edgecumbe (refer Figure 6: further downstream, the water level fluctuations are dominated by tidal effects). The effect does not apply to over-steep banks where fluvial erosion at the toe of the bank leads to mass failure.
2. Trapping of sediment in Lake Matahina has halted or substantially reduced the historical rising river bed and coastal progradation of the Rangitaiki plains. This is a positive effect in that maintenance of the Rangitaiki-Tarawera Rivers Scheme would otherwise require regular dredging or stopbank raising to counter the effect of a rising river bed level. It should be noted, however, that the principal sediment capture that has reduced loads downstream has historically occurred at the upstream Aniwhenua dam.
3. Flood attenuation in Lake Matahina has caused significant reduction in the frequency and duration of flood peaks in the range 160-200 m^3/s downstream of the dam. This effect is also likely to apply to larger floods, though the there are insufficient events with flow exceeding 300 m^3/s to draw statistically valid conclusions from the flow record. It is expected that the flood attenuation results in reduced fluvial bank erosion compared to the natural river flow.

Mitigation for the first effect above is discussed in the following section. No mitigation is proposed for the second and third effects.

9.3 Effects of current operation of the scheme, where there is an ongoing trend

River morphology effects of the scheme, where no equilibrium has been reached and there is an ongoing trend, are:

1. Increasing depth of sediment in the deep portion of Lake Matahina.
2. Bank erosion during floods at the upstream end of the river reach below Matahina dam, as a result of the sediment deficit caused by sediment deposition in the lake.
3. Bank erosion due to lack of vegetation in the range of peaking water level fluctuation.

No mitigation is proposed for the first effect above, because it will have no impact on the reservoir operation within the foreseeable future.

Evidence for the second effect is not conclusive. Any effect is minor and no mitigation is warranted, because if bank erosion is occurring as a result of the sediment deficit, then protecting the banks would merely transfer the problem elsewhere.

The third effect requires mitigation at some locations. Mitigation is recommended by the contribution of TrustPower towards EBoP river bank protection costs.

9.4 Effects of rough running operation regime

The informal rough running regime has been operated at low flows since 2007.

Hydraulic effects of the rough running regime are to introduce flow and river level variability at times of low reservoir inflow.

There are no identified river morphology effects of the rough running regime.

9.5 Effects of proposed new operation regime

Hydraulic effects of the proposed new operating regime are:

- The Matahina reservoir will generally be maintained at a higher level.
- Downstream of the dam there will be flow and river level variability at times of low reservoir inflow.
- The minimum output will be 10 MW, corresponding to a normal minimum flow of about 25 m^3/s. To allow for variations from the generation set point, TrustPower seeks a minimum consented flow of 20 m^3/s, except when reservoir inflow is less than 20 m^3/s. This compares to a normal minimum output of 22 MW (45 m^3/s) under the present consented operating regime, and a minimum of 10 MW (25 m^3/s) under the “rough running regime” agreed with EBoP for when reservoir inflow is less than 22 MW (45 m^3/s).

Approximately 25% of the time the low river flow will be less than under the present operating regime (see Figure 18). The effect of lower flows on river level varies along the river, as shown on Figures D19 and D20 of Appendix D. The greatest effect, with Matahina output reduced from 22 MW (45 m^3/s) to 10 MW (20 m^3/s) is a drop in water level of about 0.9 m downstream of a constriction at km 4.5 and sills at 12 (Te Teko) and 19.6 km from the dam. The latter is the site of the Edgecumbe earthquake fault: downstream from the fault, the effect reduces gradually to zero at the river mouth.

As discussed in Section 8.2.6, the lower minimum flow in the river is expected to result in a longer and deeper saline wedge in the lower Rangitaiki River. This is unlikely to affect existing floating intakes, but may impact on other takes downstream of Edgecumbe.

Lower river levels and saline intrusion will adversely affect six of the 12 existing consented water intakes and an unknown number of permitted takes from the Rangitaiki River downstream of Matahina dam. These effects can be mitigated: refer to Section 8.6.4 and Appendix C.

As discussed in Section 8.2.5, the lower water levels will have zero or negligible effect on adjacent wetland reserves as the only hydraulic connection with the river is close to the mouth, where water level changes due to the proposed regime are minimal.

There are no additional identified river morphology effects of the proposed new operating regime that differ from those of the existing operating regime. A single inspection by jet boat is recommended two years after the introduction of new operating conditions, to identify any significant change to the pattern or rate of erosion as a result of the changed operating conditions. The inspection should be from Matahina to Edgecumbe.

10 Conclusions

Existing operating regime

The river morphology effects of the current operation of Matahina HEPS are minor and can be mitigated. Observed bank erosion is predominantly the result of flood events, the frequency and duration of which has been reduced by attenuation in the Matahina reservoir. Operation is contributing to some extent to the erosion that occurs downstream, principally due to effects on riparian vegetation, and this can be addressed by TPL continuing to contribute to river protection works costs.

The operation of the Matahina HEPS had no significant effect on the extent of flooding and bank erosion during the July 2004 flood.

Proposed operating regime

Hydraulic effects of the proposed new operating regime are:

- The Matahina reservoir will generally be maintained at a higher level.
- Downstream of the dam there will be flow and river level variability at times of low reservoir inflow.
- The minimum output will be 10 MW, corresponding to a normal minimum flow of about 25 m^3/s. To allow for variations from the generation set point, TrustPower seeks a minimum consented flow of 20 m^3/s, except when reservoir inflow is less than 20 m^3/s. This compares to a normal minimum output of 22 MW (45 m^3/s) under the present consented operating regime, and a minimum of 10 MW (25 m^3/s) under the "rough running regime" agreed with EBoP for when reservoir inflow is less than 22 MW (45 m^3/s).

Approximately 25% of the time the low river flow will be less than under the present operating regime. The effect of lower flows on river level varies along the river. The greatest effect, with Matahina output reduced from 22 MW (45 m^3/s) to 10 MW (20 m^3/s) is a drop in water level of about 0.9 m at three locations. Downstream from the Edgecumbe earthquake fault the effect reduces gradually to zero at the river mouth.

The lower minimum flow in the river is expected to result in a longer and deeper saline wedge in the lower Rangitaiki River. This is unlikely to affect existing floating intakes, but may impact on other takes downstream of Edgecumbe.

Lower river levels and saline intrusion will adversely affect six of the 12 existing consented water intakes and an unknown number of permitted takes from the Rangitaiki River downstream of Matahina dam. These effects can be mitigated.

The lower water levels will have zero or negligible effect on adjacent wetland reserves as the only hydraulic connection with the river is close to the mouth, where water level changes due to the proposed regime are minimal.

There are no identified river morphology effects of the proposed new operating regime that differ from those of the existing operating regime.

Future monitoring

Given the minor effect of Matahina HEPS on river morphology downstream of the dam, there is no reason to continue the biennial monitoring required under the existing consent. A single inspection by jet boat is recommended two years after the introduction of new operating conditions, to identify any significant change to the pattern or rate of erosion as a result of the changed operating conditions. The inspection should be from Matahina to Edgecumbe.

Monitoring of salinity profiles in the tidal reach at low flows is recommended, to confirm the effects on existing water takes and the need for mitigation measures.

11 References

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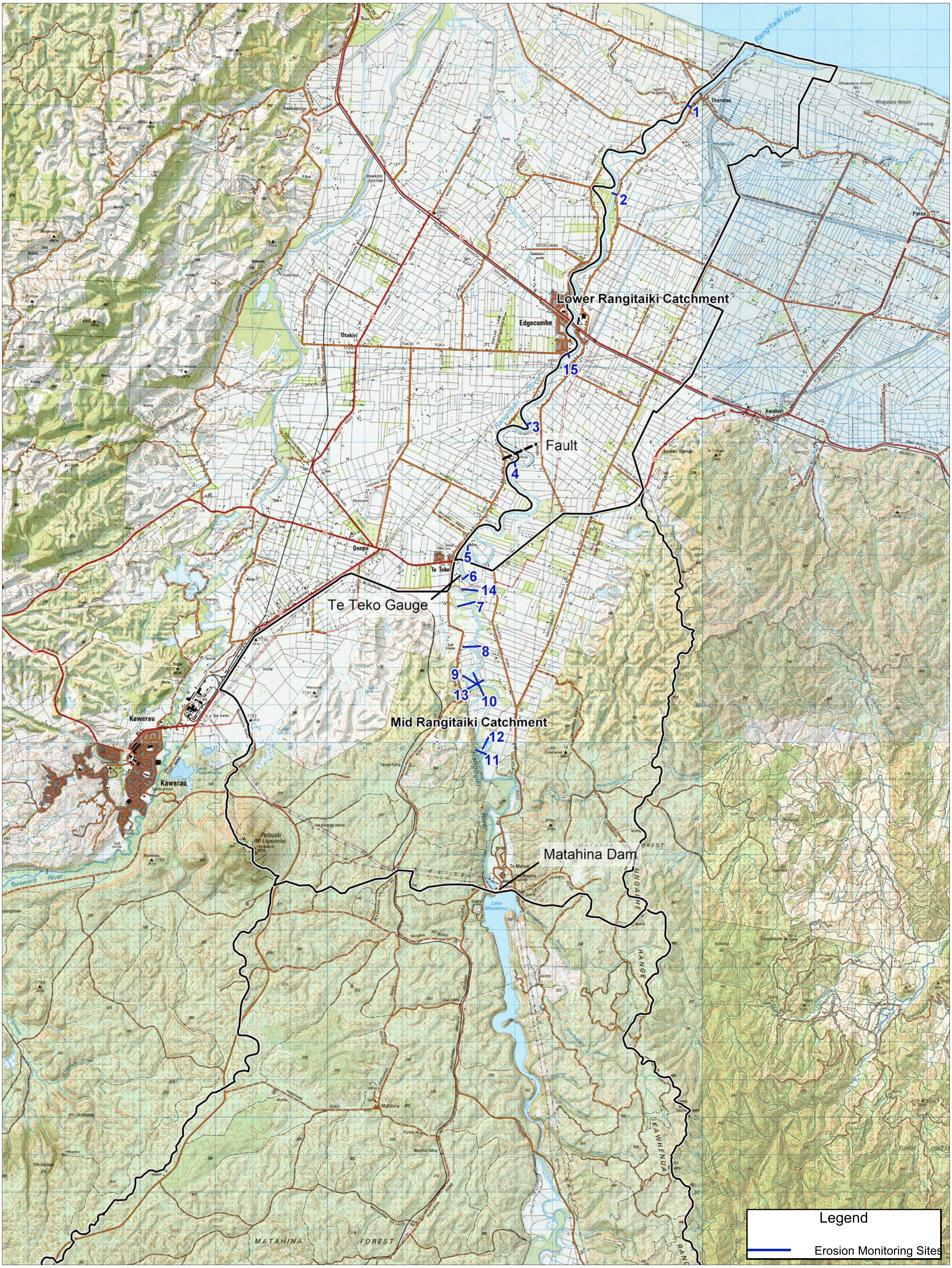
Maps



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Kilometres

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Lake Matahina and the Lower Rangitaiki River

Map 1

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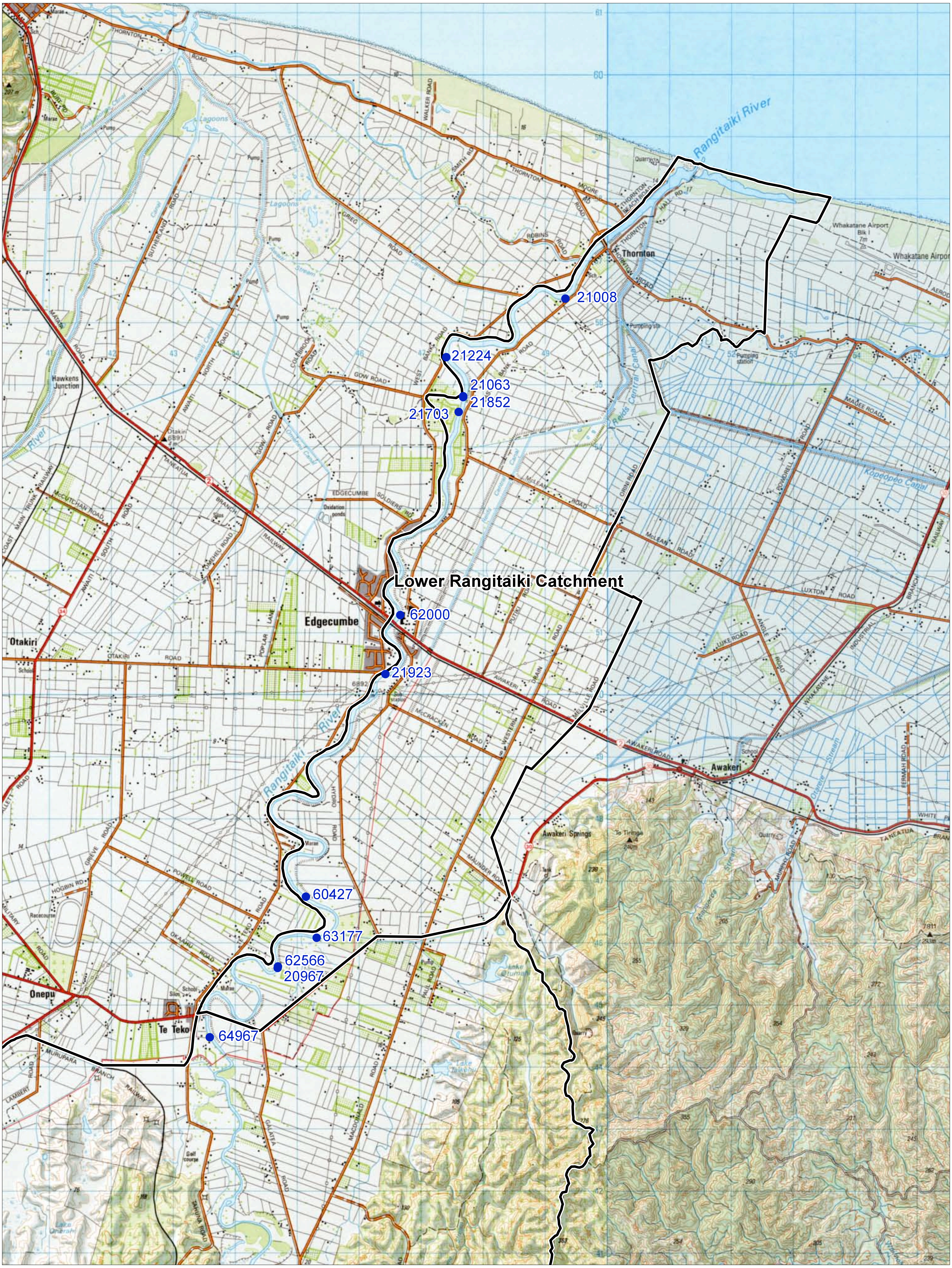
GIS@beca.com
Author: AH4
Printed Date: 20 April 2009
Created Date: 20 April 2009
Path: P:\325\3252280\TGI\55_Workspaces\03_geo\090420_Working.gws



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Consented Water Takes downstream of Matahina Dam

Map 2

Beca

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GIS@beca.com

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Rangitaiki Catchment

Overview Plan

Map 3

Beca

Appendix A

Existing Resource Consent 02 2195/1

THE BAY OF PLENTY REGIONAL COUNCIL

RIGHT IN RESPECT OF NATURAL WATER

Pursuant to Section 16(1)(d) of the Final Reorganisation Scheme for the Bay of Plenty Region and Section 21(3) of the Water and Soil Conservation Act 1967, **THE BAY OF PLENTY REGIONAL COUNCIL**, by a decision dated 23 November 1989 **HEREBY GRANTS** to:

TRUSTPOWER LIMITED
SEE TRANSFER ~~ELECTRICITY CORPORATION OF NEW ZEALAND LIMITED~~

~~P O Box 445~~ Trumans Lane
~~HAMILTON~~ Te Maunga
MOUNT MAUNGANUI

rights in respect of the Matahina Power Scheme to:

1 DAM THE RANGITAIKI RIVER AT OR ABOUT MAP REFERENCE V16 447 361 TO FORM A RESERVOIR KNOWN AS LAKE MATAHINA;

2 TAKE AND USE UP TO 160 CUBIC METRES OF WATER PER SECOND FROM LAKE MATAHINA AT OR ABOUT MAP REFERENCE V16 447 361 FOR THE PURPOSE OF GENERATING ELECTRICITY AT THE MATAHINA POWER STATION;

3 DISCHARGE UP TO 160 CUBIC METRES OF WATER PER SECOND TAKEN FROM LAKE MATAHINA BACK TO THE RANGITAIKI RIVER AT OR ABOUT MAP REFERENCE V16 447 362 AFTER USE FOR ELECTRIC POWER GENERATION;

4 DISCHARGE UP TO 1980 CUBIC METRES OF WATER PER SECOND FROM LAKE MATAHINA OVER THE SPILLWAY AT MATAHINA DAM TO THE RANGITAIKI RIVER AT OR ABOUT MAP REFERENCE V16 444 361; AND

5 DISCHARGE UP TO 140 CUBIC METRES OF WATER PER SECOND FROM THE MATAHINA DAM LEFT ABUTMENT DEWATERING TUNNEL TO THE RANGITAIKI RIVER AT OR ABOUT MAP REFERENCE V16 444 361; subject to the following conditions:

1 LOCATION

SEE CHANGE ~~The dam, penstock intake, tailrace, spillway and dewatering tunnel shall be sited as shown on the following Plans and Photographs submitted with the application:~~

~~NZED Plan Number FHN 3211~~
~~MWD Plan Number 8/1275/83/8804/12~~
~~MWD Plan Number 8/1275/89/8804/5~~

~~Photographs 1, 4-6 and 8.~~

1.1 The dam, penstock intake, tail race, spillway and dewatering tunnel shall be sited as shown in the following plans submitted with the application – AC 260016/B1 and B2.

1.2 The consent holder shall forward to the Bay of Plenty Regional Council "as built" plans for the Matahina Dam upon completion of the dam strengthening project.

2 DAM

2.1 The maximum height of the dam shall not exceed 80 m as measured from the bed of the river to dam crest.

2.2 The spillway width shall not exceed 30.5 metres.

2.3 The consent holder shall manage reservoir safety standards during strengthening of the Matahina Dam in accordance with the following reports:

SEE CHANGE
- Matahina Dam Strengthening Phase IIIC: Reservoir Safety During Construction of Strengthening Measures – Gillon, Anderson and Everett, January 1997.

SEE CHANGE
- Matahina Dam Strengthening: Second Report on Reservoir Safety During Construction of Strengthening Measures: Gillon, 10 April 1997.

SEE CHANGE
- Proposed plan to allow early strengthening of spillway gates attached to change if consent application dated 21 April 1998. The procedure specified in the proposed plan to allow early strengthening of spillway gates shall replace the procedures for spillway gate operation detailed in the Matahina Dam Strengthening Phase IIIC and the Matahina Dam Strengthening Reports.

2.4 The consent holder shall repair the dam to those standards set out in that part of the Executive Summary of the Matahina Dam Strengthening Project Phase 11B, Task 6.0B Final Design Report – Woodward Clyde Consultants, April 1997.

SEE CHANGE

2.5 The consent holder shall maintain and operate the emergency preparedness plan which contains measures to deal with the downstream effects of an uncontrolled release of the dam reservoir, and shall have a copy of this plan available for inspection by the Bay of Plenty Regional Council, at all reasonable times. "An uncontrolled release of the Dam Reservoir" shall mean an event which would have the flooding effects identified in the Matahina Dam Brake Report – Works Projects Services NZ Limited, January 1989".

SEE CHANGE

3 OPERATION OF LAKE MATAHINA

The normal operation of the Lake Matahina reservoir shall be restricted by the following conditions (all levels in metres above Moturiki Datum).

3.1 Extreme Minimum Reservoir Level (flood pending) = 71.6 m

3.2 Minimum Operational Reservoir Level = 73.15 m

3.3 Maximum Operational Reservoir Level = 76.20 m

3.4 Maximum Reservoir Level during floods or less than 200 cubic metres per second = 76.40 m

3.5 Design Flood Level = 76.80 m

3.6 Spillway Gate Crest Level = 76.40 m

3.7 Maximum reservoir level rate of change of 0.25 m/h except under emergency conditions when spillway operating.

3.8 Provided that for a period of 36 months from the granting of this change, Lake Matahina may be operated at a reservoir level of not less than 42.5 metres. This is to allow dewatering of the reservoir to facilitate dam embankment repairs. During the dewatering and repair period Lake Matahina shall not be considered to be under normal operation.

SEE CHANGE

4 SCREENS

The intake to the penstocks shall be fitted with a screen with a gap between bars no greater than 90 millimetres.

5 RANGITAIKI RIVER OPERATION

Under normal operation the following operating conditions apply:

5.1 The minimum load shall not be less than 22MW (40 cubic metres per second) except when the river inflow is less than this. When river inflow is less than 40 cubic metres per second no peaking is permitted.

5.2 ~~There shall be no more than one operating peak per day, except for the second year of exercise of this right when a peak operation once per day and an intermediate peak not exceeding the previous 24 hour mean plus 16 MW (30 cubic metres per second) is permitted on a trial basis.~~

SEE CHANGE

There shall be no more than two operating peaks per day. An operating peak is defined as an increase in river flow to a maximum and/or constant operating level followed by a subsequent decline in river flow from the power station or dam spillway. Two operating peaks are referred to as twin peaks in this consent.

5.3 The maximum load increase shall not exceed 37 MW/h (70 cubic metres per second per hour) except during emergency electrical – system low frequency conditions. Should it be necessary to exceed the rate of increase because of emergency electrical system low frequency conditions, the Grantee shall notify the Regional Council as soon as practicable.

5.4 ~~The maximum load decrease shall not exceed 8 MW/hour (14 cubic metres per second per hour) except for the first year of exercise of this Right when a maximum ramping download decrease of 16 MW/first hour (30 cubic metres per second), 12 MW/second hour (22 cubic metres per second) and 8 MW/hour (14 cubic metres per second) thereafter is permitted on a trial basis. Should it be necessary to exceed the rate of decrease because of emergency electrical system low frequency conditions, the Grantee shall notify the Regional Council as soon as practicable.~~

SEE CHANGE

load refers to machine discharge.

The maximum load decrease shall not exceed 16 MW/first hour (30 cubic metres per second), 12 MW/second (22 cubic metres per second) and 8 MW/hour (14 cubic metres per second) for every hour thereafter.

Should it be necessary to exceed the rate of decrease because of emergency electrical system low frequency conditions, the Grantee shall notify the Regional Council as soon as practicable.

5.5 ~~At the completion of trial operations of the Station as described in conditions 5.2 and 5.4 above the requirements of these conditions may be reviewed on application for a variation by the Grantee to the Regional Council.~~

SEE CHANGE

The Grantee shall continue to monitor river erosion trends at the 11 sites, as surveyed by the Regional Council for the trial period, on the Rangitaiki River between the Matahina Dam and Thornton.

(two yearly)

5.6 The Grantee shall carry out the monitoring, as specified under condition 5.5, on biennial basis throughout the term of this consent. The Grantee shall provide the General Manager of the Regional Council or delegate with the results of each biennial monitoring by 31 August of the same year that the monitoring was carried out. During the period of dewatering as specified in condition 3.8 the Grantee shall undertake the monitoring specified in conditions 5.5 and 5.6 on an annual basis.

SEE CHANGE

5.7 In the event that the biennial monitoring, as specified under condition 5.5, shows a significant change to the pattern or rate of river erosion as a result of the changed operating conditions relating to the maximum load decrease, the Grantee shall then change the ramping download decrease back to a maximum rate of 8 MW/hour (14 cubic metres per second). The Grantee shall complete the appropriate change to the ramping down rates, if necessary, within 6 weeks of being notified by the Regional Council or within a specified period agreed to between the Grantee and the General Manager of the Regional Council or delegate.

SEE CHANGE

5.8 (a) The Grantee shall monitor on a quarterly basis for a period of two years the matters identified below, except ecological monitoring which shall be undertaken twice per year. These matters shall be monitored from the commencement of twin peaking and shall be as surveyed by the Grantee during 2001/02 for the establishment of baseline river conditions on the Rangitaiki River between the Matahina Dam and the Rangitaiki River mouth. First monitoring shall be undertaken 12-14 weeks after the commencement of the first twin peak of the twin peak operating regime and at three monthly intervals thereafter, except ecological monitoring, which will be at six monthly intervals within the periods 1 March to 30 April and 1 August to 30 November.

SEE CHANGE

- The effects of the twin peak operating regime (excluding effects attributable to events beyond the control of the Grantee) on the cultural values of the river by comparison to the baseline established prior to the commencement of the twin peak operation.

- The effects of the twin peak operating regime (excluding effects attributable to events beyond the control of the Grantee) on bank erosion at the sites shown on figure 1 (attached) by comparison to the baseline established prior to the commencement of the twin peak operation. Bank erosion monitoring shall where appropriate include cross-section profiles and rate of water drawdown in banks. In addition aerial photographs of the Lower Rangitaiki River between the Matahina Dam and the river mouth shall be flown annually for comparison to the baseline aerial photographs flown in August 2001.

- The effects of the twin peak operating regime (excluding effects attributable to events beyond the control of the Grantee) on the hydrological characteristics of the river by comparison to the baseline established prior to the commencement of the twin peak operation. Hydrological characteristics to be monitored shall include passage of generation flow downriver, relative flows, velocities and water level profiles, turbidity and pore pressures at three sites by installation of piezometers. The location of piezometers to be approved by the Chief Executive of the Regional Council or delegate prior to installation.

- The effects of the twin peak operating regime (excluding effects attributable to events beyond the control of the Grantee) on the river mouth geomorphology by comparison to the baseline established prior to the commencement of the twin peak operating regime. Monitoring the river mouth geomorphological characteristics shall include differential global positioning system surveys to the spring high water mark, spring low water mark and accessible subtidal shoals, as well as spit cross section and channel dimensions as surveyed for the baseline.

- The effects of the twin peak operating regime (excluding effects attributable to events beyond the control of the Grantee) on the ecology of the river by comparison to the baseline established prior to the commencement of the twin peak operation. Ecological monitoring shall include effects on the bed substrate, macrophyte beds, standing crop and sewage fungus in the littoral margins and macro-invertebrate communities.

- The monitoring undertaken above shall be aimed at meeting the objectives for monitoring as detailed in monitoring briefs submitted with the application, entitles "Beca Planning Monitoring Brief (Final Version) No.1, No.2, No.3 and No.4", attached to this consent.

(b)The Grantee shall provide to the Regional Council and submitters to this consent change within four weeks of completing each monitoring event a report containing the data obtained during each monitoring. Distribution of reports, data or information obtained as a result of monitoring cultural values shall be provided to the consent holder and consent authority and subsequent distribution may be restricted pursuant to section 42 of the Resource Management Act 1991.

(c)At the conclusion of the two year trial period, the final quarterly monitoring report shall be accompanied by a summary report covering monitoring undertaken over the whole of the two year period including actions undertaken in mitigation of verified adverse effects attributable to twin peak operating regime (if any), during that period.

(d)After a period of two years from the commencement of the twin peak operating regime and following receipt of the final report required under condition 5.8 (c), conditions 5.2 and 5.8 may be reviewed by the Regional Council pursuant to Section 128 (1)(a)(i), (ii) or (iii) of the Resource Management Act 1991, with regard to the actual effects on the environment of two operating peaks per day taking into account the following matters:

- Effects on cultural values associated with the Rangitaiki River within the rohe of the Rangitaiki Hapu Coalition
- Effects on river bank erosion
- Effects on the hydrological characteristics of the river
- Effects on the geomorphology of the Rangitaiki River mouth
- Effects on the ecological characteristics of the river.

(e)Notwithstanding condition (d) above, if, at any time during the 2 year trial period, or at the conclusion of the trial period, it is found that a more than minor adverse effect is caused by the twin peak operating regime or monitoring information is not submitted as required by consent, the consent holder shall immediately revert to a single peak regime as the direction of the Chief Executive of the Regional Council or delegate.

In the event that monitoring as specified under Condition 5.8 shows greater than minor adverse effects directly attributable to the twin peak operating regime appropriate mitigation measures, including a programme for their implementation, shall be agreed with the Regional Council, and implemented by the consent holder.

(f) All reasonable costs incurred by the Regional Council in undertaking a review pursuant to Section 128(1)(a)(i), (ii) or (iii) as specified in condition 5.8 (d) above, shall be met by the consent holder as if the consent holder were the applicant for a resource consent.

(g)The twin peak operating regime shall only commence once the baseline monitoring for cultural values, hydrological characteristics, bank erosion effects and river mouth geomorphology and ecological monitoring has been completed and a schedule of the results of the monitoring is submitted to the Regional Council for its record. Sewage fungus baseline monitoring is undertaken at a site immediately downstream of the NZMP Limited discharge mixing zone at Edgecumbe during a spring period of maximum discharge by NZMP Limited. This monitoring must be undertaken before the commencement of the twin peaking regime.

5.9 (a)In the event that a site or sites of significance to Maori Cultural values are damaged, modified or destroyed by events that are directly attributable to the twin peak operating regime the Grantee shall advise the Regional Council as soon as practicable after the damage, modification or destruction is made known to it. In such an event actions and subsequent mitigation shall include:

SEE CHANGE

(b)As soon as practicable the Grantee shall advise the Resource Management Planner, Te Runanga O Ngati Awa, for the purpose of contacting the most appropriate person(s) from the affected hapu of the RHC to attend the scene.

(c) The Grantee shall commission the services of a suitably qualified archaeologist to provide a report on the site, including advice from the appropriate Pukenga of the RHC.

(d) As soon as practicable after receiving the archaeologist's report the Grantee shall forward the report to the Regional Council to determine whether the event may be attributable to the twin peak operating regime and, in the event that the effect is greater than minor, whether mitigation may be appropriate.

6 FLOOD CONDITIONS

6.1 Minor Floods

6.1.1 For floods up to about 500 cubic metres per second, the Grantee may decide on the conditions governing operation of Matahina Lake and the Rangitaiki River provided that Conditions 6.2.2, 6.2.3 and 6.2.4 are complied with.

6.1.2 The Grantee shall advise the Regional Council of lake levels and dam discharges throughout such floods to the satisfaction of the General Manager of the Regional Council or his delegate.

6.1.3 The Grantee shall advise the Regional Council in advance, if it is proposed to increase the level of Lake Matahina above 76.2 m (Moturiki Datum).

6.2 Major Floods

6.2.1 When a flood of about 500 cubic metres per second or greater is expected, the Grantee shall, at the request of the General Manager of the Regional Council or his delegate, provide storage in Matahina Lake in accordance with the procedure set out in the Design Engineer's Report attached. The minimum lake level requested will be 71.6 m (Moturiki Datum).

6.2.2 The maximum rate of discharge shall not exceed 600 cubic metres per second provided that the rate may be increased to 755 cubic metres per second with the agreement of the General Manager of the Regional Council or his delegate when the lake is rising so rapidly that it will exceed its maximum level.

6.2.3 Lake Lowering

The maximum drawdown rate shall not exceed 0.3 m per hour except in emergency situations when the General Manager of the Regional Council or his delegate may agree to a drawdown rate up to 0.4 m per hour. At all times when lake level is being drawn down, total discharge shall not exceed 550 cubic metres second.

The maximum increase in outflow shall not exceed 70 cubic metres per second per 15 minutes.

6.2.4 Lake Filling

The maximum rate of river level drawdown shall not exceed 1.2 m per 8 hours at the tail race of the dam. Drawdown to such a maximum may be made in a period of 30 minutes, provided the outflow is kept constant for the following 7 ½ hours.

6.2.5 For the purposed of Conditions 6.1.2, 6.2.1, 6.2.2 and 6.2.3 Regional Council staff with delegated authority shall be the:

SEE CHANGE

~~Director Implementation and Rural Services~~	Group Manager Operational Services
~~Design Engineer~~	Technical Services Manager, or
~~Hydrologist~~	Flood Manager

6.3 During the period of reservoir dewatering as specified in condition 3.8 all flood control procedures specified in 6.1 and 6.2 shall be replaced by the operating procedure defined in Works Consultancy memo from Grant Webby 23 May 1996 submitted with the application. The main objectives of control at this time are:

SEE CHANGE

- Aim to control lake level to RL 44.0 ± 1.5 m.
- Control of lake levels below RL 64.00 m by manipulation of sluice gate.
- Spillway gates to be kept fully open and lake levels above RL 64.00 will be controlled by natural flow over the spillway.

7 ELVER PASS

7.1 Within 6 months of the grant of this Right the Grantee shall report to the Regional Council as to the feasibility of installing and operating an elver pass or lift at the Matahina Dam.

7.2 If in the opinion of the Regional Council the said pass or lift in 7.1 is feasible the Grantee shall install an elver pass or lift at Matahina Dam within 12 months of the grant of this Right.

7.3 If in the opinion of the Regional Council the said pass or lift in 7.1 are not feasible options the Grantee shall continue a programme to trap, carry and release elvers into Lake Matahina to the satisfaction of the General Manager of the Regional Council or his delegate.

7.4 During the period of reservoir dewatering as specified in condition 3.8 elver transfers shall be undertaken as specified in consent number 02 4744 condition 6.

SEE CHANGE

8 **THE REGIONAL COUNCIL** is to be excluded from any liability for loss or damage to the Grantee's equipment or structures in the event of flood, or for damage caused to any land in the event of floodwater reaching such land due to any works or actions taken by the Grantee in respect of this Right.

9 TERM OF RIGHT

This right shall terminate on 30 November 2009.

10 **THE RIGHT** hereby authorised is granted under the Water and Soil Conservation Act 1967 and does not constitute an authority under any other Act, Regulation or By-Law.

~~11 TROUT MONITORING~~

SEE CHANGE

~~Within six months of the termination of the dewatering period as specified in condition 3.8 the Grantee shall report to the Eastern Region Fish and Game Council and the Bay of Plenty Regional Council setting out a programme of monitoring of trout and trout spawning, in the lake and its tributaries. Before implementation, this programme shall be approved by the General Manager of the Regional Council or delegate, in consultation with Eastern Region Fish and Game Council staff. The monitoring programme shall include a trout population density study in Lake Matahina and tributary spawning streams once the lake is returned to its normal level.~~

ADVICE NOTES:

SEE CHANGE

1 *The archaeologist's report referred to will include details of the location and extent of the site, the significance of any damage, modification or destruction and potential for mitigation.*

The provisions of the Historic Places Act may apply in circumstances where a site or sites of significance are damaged, modified or destroyed or artefacts or cultural objects discovered.

2 *The Bay of Plenty Regional Council may decide to review the conditions of resource consent as specified in condition 5.8(d). The need for a review will be based on the results of monitoring of effects on the Rangitaiki River.*

3 *In the event the review concludes that the actual effects attributable to the twin peaking are minor or less than minor and/or may be mitigated to the satisfaction of the Regional Council the operation of the twin peaking regime may continue for the duration of this consent.*

DATED at Whakatane this 16th day of January, 1990.

For and on behalf of
The Bay of Plenty Regional Council

for J A Jones
GENERAL MANAGER

CHANGE

This resource consent was changed in accordance with a decision of the Bay of Plenty Regional Council Environmental Monitoring Committee dated 8 October 1992, as follows:

Amend condition 5.2 by retaining only the first clause of this condition i.e. "There shall be no more than one operating peak per day", and deleting the remainder of this condition.

Amend condition 5.4 to read:

"The maximum load decrease shall not exceed 16MW/first hour (30 cubic metres per second), 12MW/second (22 cubic metres per second) and 8MW/hour (14 cubic metres per second) for every hour thereafter.

Should it be necessary to exceed the rate of decrease because of emergency electrical system low frequency conditions, the Grantee shall notify the Regional Council as soon as practicable".

Replace condition 5.5 with:

"The Grantee shall continue to monitor river erosion trends at the 11 sites, as surveyed by the Regional Council for the trial period, on the Rangitaiki River between the Matahina Dam and Thornton".

Create an additional condition 5.6:

"The Grantee shall carry out the monitoring, as specified under condition 5.5, on a biennial basis throughout the term of this consent. The Grantee shall provide the General Manager of the Regional Council or delegate with the results of each biennial monitoring by 31 August of the same year that the monitoring was carried out".

Create an additional condition 5.7:

"In the event that the biennial monitoring, as specified under condition 5.5, shows a significant change to the pattern or rate of river erosion as a result of the changed operating conditions relating to the maximum load decrease, the Grantee shall then change the ramping download decrease back to a maximum rate of 8MW/hour (14 cubic metres per second). The Grantee shall complete the appropriate change to the ramping down rates, if necessary, within 6 weeks of being notified by the Regional Council or within a specified period agreed to between the Grantee and the General Manager of the Regional Council or delegate".

R B Gardner
Manager Environmental Regulation and Monitoring

for J A Jones
General Manager

CHANGE

The change of this resource consent was approved under delegated authority of the Bay of Plenty Regional Council, dated 20 June 1996, as follows:

Add Condition 3.8

3.8 Provided that for a period of 36 months from the granting of this change, Lake Matahina may be operated at a reservoir level of not less than 42.5 metres. This is to allow dewatering of the reservoir to facilitate dam embankment repairs. During the dewatering and repair period Lake Matahina shall not be considered to be under normal operation.

Add to Condition 5.6

During the period of reservoir dewatering as specified in condition 3.8 the Grantee shall undertake the monitoring specified in conditions 5.5 and 5.6 on an annual basis.

Add Condition 6.3

During the period of reservoir dewatering as specified in condition 3.8 all flood control procedures specified in 6.1 and 6.2 shall be replaced by the operating procedure defined in Works Consultancy memo from Grant Webby dated 23 May 1996 submitted with the application. The main objectives of control at this time are:

- Aim to control lake level to RL 44.0 ± 1.5 m.
- Control of lake levels below RL 64.00 m by manipulation of sluice gate.
- Spillway gates to be kept fully open and lake levels above RL 64.00 will be controlled by natural flow over the spillway.

Add Condition 7.4

During the period of reservoir dewatering as specified in condition 3.8 elver transfers shall be undertaken as specified in consent number 02 4744, condition 6.

Add Condition 11 **TROUT MONITORING**

Within six months of the termination of the dewatering period as specified in condition 3.8 the Grantee shall report to the Eastern Region Fish and Game Council and the Bay of Plenty Regional Council setting out a programme of monitoring of trout and trout spawning, in the lake and its tributaries. Before implementation, this programme shall be approved by the General Manager of the Regional Council or delegate, in consultation with Eastern Region Fish and Game Council staff. The monitoring programme shall include a trout population density study in Lake Matahina and tributary spawning streams once the lake is returned to its normal level.

R B Gardner
Manager Environmental Regulation and Monitoring

for J A Jones
General Manager

CHANGE

The change of this resource consent was approved under delegated authority of the Bay of Plenty Regional Council, dated 5 June 1997, as follows:

Delete condition 1 Location and replace with a new condition 1 Location as follows:

1.1 The dam, penstock intake, tail race, spillway and dewatering tunnel shall be sited as shown in the following plans submitted with the application - AC 260016/B1 and B2.

1.2 The consent holder shall forward to the Bay of Plenty Regional Council "as built" plans for the Matahina Dam upon completion of the dam strengthening project.

Under condition 2 Dam add the following new conditions:

"2.3 The consent holder shall manage reservoir safety standards during strengthening of the Matahina Dam in accordance with the following reports:

- Matahina Dam Strengthening Phase IIIC: Reservoir Safety During Construction of Strengthening Measures – Gillon, Anderson and Everett, January 1997.
- Matahina Dam Strengthening: Second Report on Reservoir Safety During Construction of Strengthening Measures: Gillon, 10 April 1997.

"2.4 The consent holder shall repair the dam to those standards set out in that part of the Executive Summary of the Matahina Dam Strengthening Project Phase 11 B, Task 6.0 B Final Design Report - Woodward-Clyde Consultants, April 1997."

"2.5 The consent holder shall maintain and operate the emergency preparedness plan which contains measures to deal with the downstream effects of an uncontrolled release of the dam reservoir, and shall have a copy of this plan available for inspection by the Bay of Plenty Regional Council, at all reasonable times. "An uncontrolled release of the Dam Reservoir" shall mean an event which would have the flooding effects identified in the Matahina Dam Brake Report - Works Projects Services NZ Limited, January 1989."

R B Gardner
Manager Consents & Compliance

for J A Jones
General Manager

CHANGE

The resource consent was changed under delegated authority of the Bay of Plenty Regional Council, dated 13 May 1998, as follows:

Amend condition 2.3 by adding a third report entitled "Proposed Plan to Allow Early Strengthening of Spillway Gates" attached, to change of consent application dated 21 April 1998.

Also add the words "the procedure specified in the proposed plan to allow early strengthening of spillway gates shall replace the procedures for spillway gate operation detailed in the Matahina Dam Phase IIIC and the Matahina Dam Strengthening Reports".

R B Gardner
Manager Consents & Compliance

For J A Jones
General Manager

CHANGE

The change of this resource consent was approved under delegated authority of the Bay of Plenty Regional Council, dated 17 February 1999, as follows:

Delete all of condition 11 TROUT MONITORING

R B Gardner
Manager Consents & Compliance

For J A Jones
Chief Executive

TRANSFER

The transfer of the whole of this resource consent from ELECTRICITY CORPORATION OF NZ LIMITED to TRUSTPOWER LIMITED was approved under delegated authority of the Bay of Plenty Regional Council, dated 12 April 1999.

R B Gardner
Manager Consents & Compliance

For J A Jones
General Manager

CHANGE

The change of the consent was approved under delegated authority of the Bay of Plenty Regional Council, dated 31 July 2002, as follows:

Replace condition 5.2 with:

(a) There shall be no more than two operating peaks per day. An operating peak is defined as an increase in river flow to a maximum and/or constant operating level followed by a subsequent decline in river flow from the power station or dam spillway. Two operating peaks are referred to as twin peaks in this consent.

Insert a New condition 5.8 as follows:

(a) The Grantee shall monitor on a quarterly basis for a period of two years the matters identified below, except ecological monitoring which shall be undertaken twice per year. These matters shall be monitored from the commencement of twin peaking and shall be as surveyed by the Grantee during 2001/02 for the establishment of baseline river conditions on the Rangitaiki River between the Matahina Dam and the Rangitaiki River mouth. First monitoring shall be undertaken 12-14 weeks after the commencement of the first twin peak of the twin peak operating regime and at three monthly intervals thereafter, except ecological monitoring, which will be at six monthly intervals within the periods 1 March to 30 April and 1 August to 30 November.

- The effects of the twin peak operating regime (excluding effects attributable to events beyond the control of the Grantee) on the cultural values of the river by comparison to the baseline established prior to the commencement of the twin peak operation.

- The effects of the twin peak operating regime (excluding effects attributable to events beyond the control of the Grantee) on bank erosion at the sites shown on figure 1 (attached) by comparison to the baseline established prior to the commencement of the twin peak operation. Bank erosion monitoring shall where appropriate include cross-section profiles and rate of water drawdown in banks. In addition aerial photographs of the Lower Rangitaiki River between the Matahina Dam and the river mouth shall be flown annually for comparison to the baseline aerial photographs flown in August 2001.

- The effects of the twin peak operating regime (excluding effects attributable to events beyond the control of the Grantee) on the hydrological characteristics of the river by comparison to the baseline established prior to the commencement of the twin peak operation. Hydrological characteristics to be monitored shall include passage of generation flow downriver, relative flows, velocities and water level profiles, turbidity and pore pressures at three sites by installation of piezometers. The location of piezometers to be approved by the Chief Executive of the Regional Council or delegate prior to installation.

- The effects of the twin peak operating regime (excluding effects attributable to events beyond the control of the Grantee) on the river mouth geomorphology by comparison to the baseline established prior to the commencement of the twin peak operating regime. Monitoring the river mouth geomorphological characteristics shall include differential global positioning system surveys to the spring high water mark, spring low water mark and accessible subtidal shoals, as well as spit cross-section and channel dimensions as surveyed for the baseline.

- The effects of the twin peak operating regime (excluding effects attributable to events beyond the control of the Grantee) on the ecology of the river by comparison to the baseline established prior to the commencement of the twin peak operation.. Ecological monitoring shall include effects on the bed substrate, macrophyte beds, standing crop and sewage fungus in the littoral margins and macro-invertebrate communities.

- The monitoring undertaken above shall be aimed at meeting the objectives for monitoring as detailed in monitoring briefs submitted with the application, entitled "Beca Planning Monitoring Brief (Final Version) No.1, No.2, No. 3 and No.4", attached to this consent.

(b) The Grantee shall provide to the Regional Council and submitters to this consent change within four weeks of completing each monitoring event a report containing the data obtained during each monitoring. Distribution of reports, data or information obtained as a result of monitoring cultural values shall be provided to the consent holder and consent authority and subsequent distribution may be restricted pursuant to section 42 of the Resource Management Act 1991.

(c) At the conclusion of the two year trial period, the final quarterly monitoring report shall be accompanied by a summary report covering monitoring undertaken over the whole of the two year period including actions undertaken in mitigation of verified adverse effects attributable to twin peak operating regime (if any), during that period.

(d) After a period of two years from the commencement of the twin peak operating regime and following receipt of the final report required under Condition 5.8(c), conditions 5.2 and 5.8 may be reviewed by the Regional Council pursuant to Section 128 (1)(a)(i), (ii) or (iii) of the Resource Management Act 1991, with regard to the actual effects on the environment of two operating peaks per day taking into account the following matters:

- Effects on cultural values associated with the Rangitaiki River within the rohe of the Rangitaiki Hapu Coalition
- Effects on river bank erosion
- Effects on the hydrological characteristics of the river
- Effects on the geomorphology of the Rangitaiki River mouth
- Effects on the ecological characteristics of the river.

(e) Notwithstanding condition (d) above, if, at any time during the 2 year trial period, or at the conclusion of the trial period, it is found that a more than minor adverse environmental effect is caused by the twin peak operating regime or monitoring information is not submitted as required by consent, the consent holder shall immediately revert to a single peak regime as the direction of the Chief Executive of the Regional Council or delegate.

In the event that monitoring as specified under Condition 5.8 shows greater than minor adverse effects directly attributable to the twin peak operating regime appropriate mitigation measures, including a programme for their implementation, shall be agreed with the Regional Council, and implemented by the consent holder.

(f) All reasonable costs incurred by the Regional Council in undertaking a review pursuant to Section 128(1)(a)(i), (ii) or (iii) as specified in condition 5.8(d) above, shall be met by the consent holder as if the consent holder were the applicant for a resource consent.

(g) The twin peak operating regime shall only commence once the baseline monitoring for cultural values, hydrological characteristics, bank erosion effects and river mouth geomorphology and ecological monitoring has been completed and a schedule of the results of the monitoring is submitted to the Regional Council for its record. Sewage fungus baseline monitoring is undertaken at a site immediately downstream of the NZMP Limited discharge mixing zone at Edgecumbe during a spring period of maximum discharge by NZMP Limited. This monitoring must be undertaken before the commencement of the twin peaking regime.

Insert a new Condition 5.9 as follows:

(a) In the event that a site or sites of significance to Maori Cultural values are damaged, modified or destroyed by events that are directly attributable to the twin peak operating regime the Grantee shall advise the Regional Council as soon as practicable after the damage, modification or destruction is made known to it. In such an event actions and subsequent mitigation shall include:

(b) As soon as practicable the Grantee shall advise the Resource Management Planner, Te Runanga O Ngati Awa, for the purpose of contacting the most appropriate person(s) from the affected hapu of the RHC to attend the scene.

(c) The grantee shall commission the services of a suitably qualified archaeologist to provide a report on the site, including advice from the appropriate Pukenga of the RHC.

(d) As soon as practicable after receiving the archaeologist's report the Grantee shall forward the report to the Regional Council to determine whether the event may be attributable to the twin peak operating regime and, in the event that the effect is greater than minor, whether mitigation may be appropriate.

Insert new Advice Note 1 as follows:

Advice Note 1 to Condition 5.9(b).

The archaeologist's report referred to will include details of the location and extent of the site, the significance of any damage, modification or destruction and potential for mitigation.

The provisions of the Historic Places Act may apply in circumstances where a site or sites of significance are damaged, modified or destroyed or artefacts or cultural objects are discovered.

Insert new Advice Note 2 as follows:

Advice Note 2

The Bay of Plenty Regional Council may decide to review the conditions of resource consent as specified in condition 5.8(d). The need for a review will be based on the results of monitoring of effects on the Rangitaiki River.

Insert new Advice Note 3 as follows:

Advice Note 3

In the event the review concludes that the actual effects attributable to the twin peaking are minor or less than minor and/or may be mitigated to the satisfaction of the Regional Council the operation of the twin peaking regime may continue for the duration of this consent.

for R B Gardner
Manager Consents & Compliance

For J A Jones
Chief Executive

CHANGE

The change of the consent was approved under delegated authority of the Bay of Plenty Regional Council, dated 13 May 2005, as follows:

Amend condition 6.2.5 by deleting mention of the Director Implementation and Rural Services, Design Engineer and Hydrologist and replacing with Group Manager Operational Services, Technical Services Manager and Flood Manager.

6.2.5 For the purposed of Conditions 6.1.2, 6.2.1, 6.2.2 and 6.2.3 Regional Council staff with delegated authority shall be the:

~~*Director Implementation and Rural Services*~~	*Group Manager Operational Services*
~~*Design Engineer*~~	*Technical Services Manager, or*
~~*Hydrologist*~~	*Flood Manager*

A C Bruere
Manager Consent and Compliance

for J A Jones
Chief Executive

Appendix B

Rangitaiki hydrogeology and riverbank stability report

Report

Matahina HEPS Rangitaiki Hydrogeology and Riverbank Stability

Prepared for TrustPower Ltd

By Beca Infrastructure Ltd (Beca)

May 2009

Revision History

Revision N°	Prepared By	Description	Date
A	Rebecca Poole	Final for Issue	11-9-08
B	Paul Goff	Draft for Re-Issue	15-10-08
C	Rebecca Poole	Revised draft incorporating comments	27-10-08
D	Paul Goff	Final for Issue	13-05-09

Document Acceptance

Action	Name	Signed	Date
Prepared by	Rebecca Poole / Paul Goff		13.05.2009
Reviewed by	Sian France		13.05.2009
Approved by	Do Van Toan	DVToan	13/5/2009
on behalf of	Beca Infrastructure Ltd		

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Figures

Appendices

1 Introduction

TrustPower Ltd (TrustPower) is currently in the process of reconsenting the Matahina Hydroelectric Power Scheme (HEPS), located on the Rangitaiki River.

TrustPower has commissioned Beca Infrastructure Ltd (BIL) to undertake monitoring of groundwater and river levels, and further assess the effect of the scheme on river bank stability. TrustPower requires a change to the consent such that ramping up and ramping down options provides a greater degree of operational flexibility.

This report provides the results of groundwater and river level monitoring, including seepage and stability analyses undertaken at two locations on the Rangitaiki River and downstream of the Matahina HEPS.

This report has been prepared solely for the benefit of TrustPower Ltd as our client with respect to the particular brief given to us, and data or opinions in it may not be used in other contexts, by any other party or for any other purpose. To the maximum extent permitted by law, Beca Infrastructure Ltd disclaims all liability and responsibility (in contract or tort, including negligence, or otherwise) for any loss or damage whatsoever which may be suffered as a result of any reliance by any third party on this report, whether that loss is caused by any fault or negligence on the part of Beca Infrastructure Ltd or otherwise.

Notice to Reader/User of this Document:

Should you be in any doubt as to the applicability of this report and/or its recommendations for the proposed development as described herein, and/or encounter materials on site that differ from those described herein, it is essential that you discuss these issues with the authors before proceeding with any work based on this document.

2 Background

The Matahina HEPS is situated on the Rangitaiki River in Eastern Bay of Plenty (Figure 1). It is the largest earth dam in the North Island, with a 76 metre head of water behind an 86 metre high dam. The Matahina station has two 40 MW generators producing an average annual output of approximately 275 GWh.

The Matahina HEPS was commissioned in 1967. The power station at the base of the dam generates up to 80 MW, using a daily peaking regime (single or twin peaks, depending on inflow). The existing consent limits the normal flow range downstream of the power station to between 22 MW (about 45 m^3/s) and 160 m^3/s. Flows outside this range are permitted during floods, and when the inflow is less than 40 m^3/s (see footnote[1]). The existing consent assumes that the flow required to generate 22 MW is 40 m^3/s, which is inconsistent with operating experience. It can be expected that flood peaks are reduced due to the construction of the dam.

In 2003 TrustPower trialled a twin peaking operation and undertook groundwater monitoring at selected sites along the Rangitaiki River, along with associated seepage and stability analyses. These analyses concluded that the adoption of a twin peak generation regime would not cause any

[1] The existing consent assumes that the flow required to generate 22 MW is 40 m^3/s, which is inconsistent with operating experience.

significant increase in erosion problems above those that would otherwise occur with the single peak regime that was in operation at that time. The report notes however that there was increased potential for piping of fine grained soils and consequent erosion as a result of the revised operation, but that this effect would be minimal compared to that caused by periodic natural events such as floods.

TrustPower's existing resource consents authorise the take, use and discharge of water for hydro electricity generation purposes at the Matahina HEPS. The resource consents are due to expire in November 2009.

In addition to renewing their Resource Consents, TrustPower intend to provide an 'improved operating regime' to allow for flexibility in the placement of the scheme on the market while retaining the general intent of the constraint on the scheme operation.

3 Operating Regime

3.1 Current Operating Regime

The current constraints/operational requirements are summarised as below:

- Take and use up 160 m^3/s water from Lake Matahina and discharge to the Rangitaiki River.
- The minimum load shall not be less than 22 MW (about 45 m^3/s) except when river inflows are less than this.
- The maximum load increase shall not exceed 37 MW/h (70m^3/s/hr) except during emergency electrical system low frequency conditions.
- The maximum load decrease shall not exceed 16 MW/first hour (30m^3/s), 12 MW/second hour (22m^3/s) and 8 MW/hr (14m^3/s) for every hour thereafter.
- There shall be no more than two operating peaks per day.

3.2 Proposed Operating Scheme

The proposal is to provide for Scheme operation within a defined maximum and minimum discharge which is based on average scheme outflow over the previous 72 hours. This is defined as a "Constraint Envelope" and will potentially provide for more 'smaller peaks' through to a single 'larger peak'. In addition this will provide for some ability to peak in low flow conditions.

The proposed operating regime includes revised ramping rates to improve the operational flexibility of the scheme:

- Ramping up at 52MW/hr;
- Ramping down at 16 MW/hr.

4 Methodology

In 2003, three sites along the Rangitaiki River were selected for detailed soils investigations and monitoring of groundwater response to river level fluctuations; these were Sites 4, 10 and 14 as shown on Figure 1. These sites were chosen on the basis that they represent zones with the steepest river banks and therefore the greatest potential for slope failure. It is considered that an assessment of effects at these points demonstrates the worst effects along the river, all other areas having a lesser effect.

It was intended for the recent investigation that continuous groundwater monitoring would be undertaken using the piezometers installed at that time; however it was not possible to locate any of the piezometers and so in 2007, three new piezometers were installed at each of the sites noted above. The details of installations are provided in the Beca Matahina Piezometer Installation Report dated May 2007.

Pressure transducers were installed in each of the standpipe piezometers, and also in standpipes installed in the river at Sites 4 and 10. Due to access constraints at Site 14, installation of pressure transducers here was not possible. Monitoring of water levels was undertaken at each location from May 2007 through to December 2007, until the standpipes were finally damaged. Recordings in the transducers were set at approximately 10 minute intervals.

The river level at Site 14 is identified from NIWA monitoring data from a station (Te Teko) about 500 m downstream of Site 14. The river gradient between the two sites is approximately 0.23 m.

To assess the effects of the HEP operation on bank stability a series of 2D cross-sectional models were developed for each of the sites. The assessment identifies a number of hydraulic and operational regime scenarios:

- Bank erosion potential; and
- Slope stability issues.

The objective of the assessment is to determine the differences between current and proposed operating regimes and various flood events at critical sites, rather than to determine what percentage of the river is susceptible to bank erosion or slope failure.

Investigations undertaken in 2007 at Site 4 reconfirmed the status of the banks at this location, which are described as having gentle slope. Previous studies[2] indicate that there are no large scale stability issues here, and that the bank erosion that could be caused by piping at this site would be minor compared to that produced by other natural floods. Therefore no further modelling was therefore undertaken as an understanding of worst case only was sought.

As Sites 10 and 14 were chosen to represent the worst case sites, all other slopes are considered to have a lesser effect and are therefore less susceptible to slope stability and bank erosion issues.

5 Results of Investigations

The information collected during the 2003 and 2007 investigations were used to develop a conceptual understanding of soil and groundwater properties at each of the sites. The following summarises the findings of the investigations.

5.1 Soil Profile

The soil profile encountered in each of the boreholes at each of the sites typically comprised top soil overlying Holocene aged undifferentiated alluvial deposits. The alluvial deposits are composed of bedded very loose to loose pumiceous silts and silty sands, and sandy gravel. River timber and peat were also present.

[2] ICE Geo & Civil (Nov 1993), *Rangitaiki River Bank Drawdown Effects: Single Peak Versus Double Peak Generation Regimes*, Prepared for Beca Carter Hollings & Ferner Ltd.

For a detailed description of the soil profile at each site location, please refer to the Beca 2007 Matahina Piezometer Installation report.

5.2 Groundwater Monitoring

Results of the groundwater monitoring are presented in the Groundwater Monitoring Factual Report, prepared by Beca, 2008. Appendix A contains examples of typical groundwater monitoring data for each site.

In general the monitoring suggests a strong hydraulic connection between the river bank and the river levels with piezometers responding rapidly to changes in river level (i.e. there is no noticeable time lag).

6 Groundwater Modelling

6.1 Methodology

Steady state and transient 2D seepage modelling was undertaken using the computer software GeoStudio 2004 (and the application SEEP/W) to provide an assessment of:

- The background conditions at each site, with no ramping effects;
- The effects of the current operating regime;
- The effects of the proposed operating scheme, with various ramping rates; and
- The effects of natural flood events.

Computer simulations of natural conditions prior construction of the dam were not undertaken, as no data is available to develop a reliable model.

The completed assessments included modelling of seven distinct hydrogeologic conditions:

1. Natural hydrogeologic condition during low flow with power station not operating (May 2007) – steady state model
2. Natural hydrogeologic condition during low flow with power station operating (August 2007) – steady state model
3. Hydrogeologic conditions during twin peaking (up to 116 m^3/s) for existing ramping rates – transient model
4. Hydrogeologic conditions during twin peaking (up to 116 m^3/s) for proposed ramping rates – transient model
5. Hydrogeologic conditions during high rainfall (up to 136 m^3/s) – transient model
6. Hydrogeologic conditions during 4 year flood event (up to 275 m^3/s) – transient model
7. Hydrogeologic conditions during 100 year flood event (up to 738 m^3/s) – transient model

Once developed, the models were run to assess the ramping down effects of the operational regime only.

When power output is ramped up, flows and levels in the Rangitaiki River increase and by inference stabilise the banks by exerting pressure. The time lag between river level and groundwater level rise is such that water preferentially moves from the river bed to groundwater further reducing the potential for failure.

When power output is ramped down, a time lag occurs as groundwater levels recede later than river levels. The effect is in an increase in the seepage face along the river bank, which increases the potential for river bank failure.

6.2 Model Set Up

Two dimensional cross sections were set-up for Sites 10 and 14 (the sites with the steepest river banks).

The models were constructed using the latest survey data, soil profiles obtained from the 2007 and 2003 investigations, and the results of groundwater level monitoring.

6.2.1 Hydrogeologic Parameters

Initial hydraulic conductivity characteristics were obtained from correlations with gradings, soil descriptions and published data, as detailed in the 2003 report. The model was calibrated by varying these properties until the predicted water levels matched those observed during steady state low flow, with further calibration against transient flow conditions. Parameters used for each of the sites are detailed below.

Site 10

Unit	K (m/s)*	K_v/K_H	Seep K Function No.	Seep Water Content No.
Sandy Silt	7e-7	0.1	12 – Silt Loam	12 - Silt Loam
Sand	2e-5	1.0	1 – Uniform Fine Sand #1	1 - Uniform Fine Sand #1
Sandy Gravel	3e-3	0.5	8 – Well-graded #2	8 - Well-graded #2

*K = Hydraulic Conductivity

Site 14

Unit	K (m/s)*	K_v/K_H	Function No.	Water Content No.
Sandy Silt	7e-7	0.1	12 – Silt Loam	12 – Silt Loam
Sandy Gravel	3e-3	0.1	8 – Well-graded #2	8 – Well-graded #2
Clay	2e-8	0.1	15 – Uniform Silt	23 – Sandy Clayey Silt
Silt	1e-7	0.1	11 – Glacial Till	11 - Glacial Till
Silt/Sand/Gravel/	3e-5	0.2	6 – Silty Sand	6 – Silty Sand

*K = Hydraulic Conductivity

Rainfall Recharge

Average rainfall records from the Edgecumbe rainfall gauge (supplied by NIWA) suggest an annual rainfall of approximately 1420 mm/year. A recharge of 3% of the total rainfall was adopted for the sandy silt material (42.5 mm/year), whilst 7% of the total rainfall was adopted for the sandy gravel material (99.1 mm/year). These percentages are based on a review of similar permeability materials for previous assessments undertaken by Beca, and an understanding of their associated hydraulic behaviours. Inputs of these values into the model provided acceptable calibrated groundwater levels.

Boundary Conditions

The following boundaries were applied to the models.

Boundary Condition	Where	Reason
Constant Head Boundary	Two end boundaries of each model.	To represent the known regional groundwater level.
	A constant head of 8.38 m (site 10) and 6.9 m (site 14) was applied to the ground surface below the river.	Represents known river depth at low flow (steady state).
	A constant head of 9.6 m (site 10) and 8.2 m (site 14) was applied to the ground surface below the river.	Represents known river depth at high flow (steady state).
Hydraulic Boundary	Applied along the river channel.	To represent fluctuation of water levels over time (transient).
Seepage Face Review Boundary	Applied along the ground surface approximately 1 m above the river level along the river bank.	To allow the water table to rise above the ground surface (if required – steady state and transient).
Flux Boundary	Along open land at the ground surface of the model.	Represents rainfall infiltration into the model.

Hydraulic Boundary

The constant head river boundary conditions that were applied to the steady state and transient models at Site 10 were taken from recent monitoring undertaken in August 2007.

The constant head river boundary condition applied to the steady state model at Site 14 was taken from Te Teko monitoring data undertaken in 2007. Transient water levels at Site 14 were deduced based on NIWA data. The implications for adopting this methodology are discussed in the results section for Site 14.

As no real-time data was available to model the effects from the proposed regime changes, a synthesized hydrograph was produced. The synthesized hydrograph has been developed based on projected recession rates for the Rangitaiki River using a 45 MW twin peak scenario with a 16 MW/hour ramping down rate (See Figure 2).

The synthesized hydrographs are shown in Figures 3 and 4 and were initially established for the following rates:

- Site 10 – a recession rate of 0.35 m/h;
- Site 14 – a recession rate of 0.33 m/h.

Calibration

The models were calibrated to the recorded river and groundwater level obtained during the 2007 groundwater monitoring period. Data from the month of August was used, as this month recorded the greatest fluctuation in levels and the highest peaks in groundwater and river levels. Models were calibrated to within 0.4 m of the recorded field measurements.

Five year and 100 year flood events were also modelled. No field measurements were available for these time periods, therefore the change in groundwater levels are inferred from direct outputs from SEEP/W.

Sensitivity Test

With no real-time data to confirm the accuracy of synthesized hydrographs, projected recession rates (presented in Figure 2) were doubled, hydrographs re-synthesized, and the model re-run to check the sensitivity of river recession rates on piping and slope stability assessment. The decision to double recession rates was arbitrary, but allowed the opportunity to establish whether a more accurate method of hydrograph synthesis was required.

6.3 Results of Analyses

The hydraulic exit gradients for various operating regimes have been obtained from the SEEP/W model runs; the results of which are summarised below.

The hydraulic exit gradient represents the difference between the water level in the bank and that in the river. At its peak, it therefore represents the maximum exit gradient of water flowing out of the bank, which where critical could lead to bank erosion effects such as piping. It is considered that in pumiceous soils, hydraulic gradients above 0.2 could lead to effects of piping.

The results show the most critical exit gradient occurs during the recession curve. The critical point was selected where the greatest separation distance between groundwater and river hydrographs occurs. At all other times the effect is considered to be less.

Site 10

During current and proposed operating conditions a maximum exit gradient of 0.06 is predicted, with negligible change during extreme low conditions following peaking and normal flood conditions. This is consistent with the assessment undertaken in 2003.

The worst case exit gradient for water flowing out of the bank is 0.10, and occurs following a 100 year flood event.

Operating Regime	Hydraulic Exit Gradient
Steady State Low Flow (with power station not operating)	0.06
Steady State Low Flow (with power station operating)	0.05
Mid Low between current regime twin peaks (116 m^3/s to 74 m^3/s)	0.06
Low following current regime twin peaks (37.8 m^3/s)	0.06
Low following proposed regime peak (0.35 m/h recession)	0.06
Sensitivity Test (0.7 m/h recession)	0.06
Following Normal Flood event (136 m^3/s to 39 m^3/s)	0.06
Following 4 Year Flood Event (275 m^3/s)	0.08
Following 100 year Flood Event (728 m^3/s)	0.10

Site 14

During current and proposed operating conditions a maximum exit gradient of 0.15 is predicted, with variable change during extreme low conditions following peaking and normal flood conditions. Again this is consistent with the 2003 assessment.

As the range of river level effects (low to high) at Site 14 corresponds to the entire thickness of sand and gravel deposits (modelled as a single hydraulic unit), the transient results for piping remain unchanged regardless of the river stage considered.

Operating Regime	Hydraulic Exit Gradient
Steady State Low Flow (with power station not operating)	0.14
Steady State Low Flow (with power station operating)	0.14
Mid Low between current regime twin peaks (116 m^3/s to 74 m^3/s)	0.15
Low following current regime twin peaks (37.8 m^3/s)	0.14
Low following proposed regime peak (0.35 m/h recession)	0.13
Sensitivity Test (0.7 m/h recession)	0.13
Following Normal Flood event (136 m^3/s to 39 m^3/s)	0.14
Following 4 Year Flood Event (275 m^3/s)	0.14
Following 100 year Flood Event (728 m^3/s)	0.15

Discussion

During the scenarios modelled, the minor increases in the potential for piping due to the current operations and/or proposed changes do not appear to be an issue for either site. During large natural flood events, the risk of piping increases and the potential for bank erosion would also be higher.

From a review of flood hydrographs (Appendix A) and bank elevations (Appendix B), the potential increase in piping risk during flood events is likely to be the combined effect of over-topping of channel banks, breakout of river flows into the floodplain and saturation of sands and gravels at shallow depth within the floodplain. Due to the effects of saturation caused by flooding, the real time for groundwater to drain soils is longer and would lead to higher hydraulic exit gradients.

The difference in exit gradients between the two sites appears to be a result of river morphology and lithological variations (increased clay content). As these sites represent the worst sites along the length of the study area, exit gradients are expected to be less elsewhere.

Although changes in exit gradient for the current and proposed operational scenario runs and the sensitivity run do occur, variations are small. The limited change under a variety of regimes can be explained by good hydraulic connection between river and groundwater levels; which exhibit negligible time lag regardless of the recession rate applied (Appendix A).

7 Slope Stability Modelling

7.1 Methodology

Slope stability analyses using the computer software Geostudio 2004 SLOPE/W was undertaken to provide an assessment of:

1. Current slope stability during natural low flow conditions – steady state model
2. Current slope stability during current regime twin peaking flow conditions – steady state model
3. Slope stability during twin peaking (up to 116 m^3/s) for existing ramping rates – transient model
4. Slope stability during twin peaking (up to 116 m^3/s) for proposed ramping rates – transient model
5. Slope stability during high rainfall (up to 136 m^3/s) – transient model
6. Slope stability during 4 year flood event (up to 275 m^3/s) – transient model
7. Slope stability during 100 year flood event (up to 738 m^3/s) – transient model

7.2 Model Set Up

The slope, soil profile and groundwater level used for the slope stability analyses were obtained directly from output files created in SEEP/W.

Soil Parameters

The soil parameters used for SLOPE/W modelling were based on back analysis, augmented by Beca's experience with these pumiceous soils and correlation with in-situ test strengths from similar soils detailed below.

Site 10

Description	Unit Weight	Φ'	C' (kPa)
Sandy Silt	15.5	36	0.5
Sand	17	40	1.5
Sandy Gravel	18.5	36	2

Site 14

Description	Unit Weight	Φ'	C' (kPa)
Sandy Silt	15.5	36	0.5
Sandy Gravel	17	35	0.5
Clay	15.5	24	0
Silt	16	27	2
Silty Gravelly Sand	18.5	36	2

Results of Analyses

The results from the slope stability analyses are presented below and describe the factor of safety associated with slopes modelled at each of the sites. Slope/W outputs are presented in Appendix B.

If the forces available to resist movement are greater than the forces driving movement, the slope is considered stable. The factor of safety is calculated by dividing the forces resisting movement by the forces driving movement; therefore if a factor of safety with unity or above is present stable conditions are likely.

Site 10

In general the different operating schemes (including that proposed) do not significantly affect the stability of the river bank. Steady state conditions indicate that the bank is marginally stable at site 10 with a factor of safety of 1.1. A strong hydraulic connection exists between the river and groundwater flow, with no significant time lag between drawdown of the groundwater as the river level changes.

Operating Regime	Factors of Safety
Steady state low flow (with power station not operating)	1.1
Steady state high flow (with power station operating)	1.1
Mid low between current regime twin peak operation	1.0
Low following current regime twin peak operation	1.0
Low following proposed regime peak (0.35 m/h recession)	1.1
Sensitivity Test (0.70 m/h recession)	1.1
Following rainfall flood event	1.0
Following 4 year flood event	0.98
Following 100 year flood event	0.99

Discussion

The modelling suggests a 10% reduction of stability from background conditions during the existing operational regime, with marginal change during the proposed regime. Although a reduction in FOS would be expected with an increase in the recession rate (as seen for ‘low following current regime twin peak operating regime’ model scenario runs), the synthesized model shows a negligible change. The range of effects indicates that there is marginal effect on the FOS regardless of the regime imposed.

A slight reduction in stability is indicated to occur after larger water level changes following a ‘typical’ rainfall flood event; however the FOS does not drop much below 1.0.

Variations between FOS for current and proposed scenario runs do occur, but are negligible. This can be explained by good hydraulic connection between river and groundwater levels with negligible time lag regardless of the recession rate.

Site 14

In general the results are similar to Site 10 results, and indicate that the different operating schemes do not significantly affect the stability of the river bank. Steady state conditions indicate that the bank is marginally stable at site 14 with a factor of safety between 0.9 and 1.1. A strong hydraulic connection exists between the river and groundwater flow, with no significant time lag between drawdown of the groundwater as the river level changes. It is noted however that groundwater levels are held at a higher elevation for longer in the surrounding banks than at Site 10 and results in steeper exit gradients.

Operating Regime	Factors of Safety
Steady state low flow (with power station not operating)	1.1
Steady state high flow (with power station operating)	1.0
Mid low between current regime twin peak operation	1.1
Low following current regime twin peak operation	1.0
Low following proposed regime peak (0.35 m/h recession)	0.9
Sensitivity Test (0.70 m/h recession)	0.9
Following rainfall flood event	1.0
Following 4 year flood event	0.72
Following 100 year flood event	0.59

Discussion

The modelling suggests a 10% reduction of stability during the current regime with further reduction during the proposed regime. The reduction in factor of safety during the proposed regime could result in minor slumps that occur along the bank. For deeper slip circles, the factor of safety is higher at about 1.0, which is comparable to conditions during the existing regime with the power station operating at low flows in the river.

A more significant reduction in stability is indicated to occur after natural flood events (4 and 100 year events) with the FOS dropping below 1.0 (this is equivalent to a 20% annual risk of slope failure).

It can be seen that natural flood events have a much greater effect on bank stability than the proposed flow regime. Comparison between sites indicates that the presence of finer soils at the toe of the slope increases the likelihood of failure, due to higher pore water pressures.

Variations between FOS for current and proposed operational scenario runs do occur, but are very small. This can be explained by good hydraulic connection between river and groundwater levels with negligible time lag regardless of the recession rate.

8 Conclusions

In general it is the extreme high flows from natural flood events that have the greatest potential for bank erosion and/or slope stability failures; however it could be expected that some of these peaks are moderated by controls on flow imparted by the dam.

Under proposed operational conditions, piping should not have a detrimental effect on the bank stability for Site 10 or Site 14, with predicted hydraulic exit gradients being less than 0.2 (the typical threshold for piping). The risk increases during extremely high river level flows, when the floodplain of the river is breached and shallow soils become saturated.

Due to good hydraulic connection between groundwater and river levels, the significance of the twin peak operational regime (high frequency, low amplitude effects) is minor when compared to flood events (low frequency, high amplitude effects).

Slope stability modelling for both Site 10 and Site 14 confirm:

- The sites are marginally stable during current and proposed operating conditions;
- The current and proposed ramping regimes lower the factor of safety by up to10%, which may result in shallow surficial erosion common with meandering rivers; and
- The critical period for slope stability occurs during natural flood events when the groundwater / river level separation is at its greatest and a greater percentage of the bank is exposed to seepage.

Under normal low flow conditions the modelling suggests a factor of safety (FOS) against failure of approximately 1.1 for both sites. This is consistent with the riverbank naturally being in a marginal state of stability as could be expected for natural river banks in sandy soils (with a factor of safety close to unity). Typically river banks are eroded in the toe zone causing slumps following floods. This naturally creates slopes that just marginally stand under low flow conditions (as is currently seen in the banks). The extent of predicted slumping effects is shown in Appendix B (Slope Stability Outputs), which shows bank retreat of up to 20 m at Site 14 in some cases and less than 10 m at Site 10.

8.1 Summary

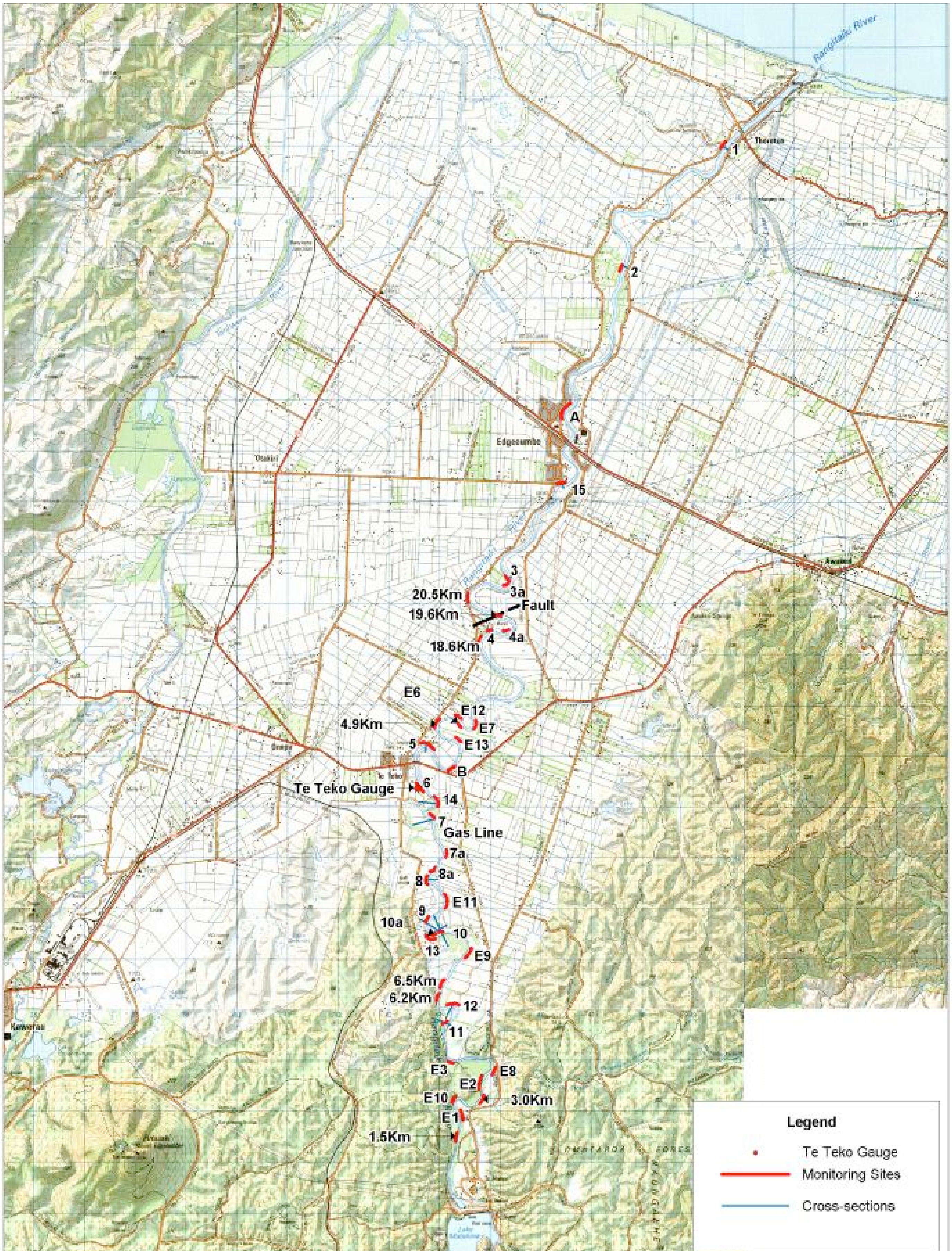
Information collected during investigation and modelling of the current and proposed HEP scheme has shown that:

- Good hydraulic connection exists between groundwater and river levels on the Rangitaki River;
- Current and proposed operation of the HEP scheme is not likely to cause increased potential for piping or slope failure considering the operation regime proposed; and
- Erosion from piping and slope stability issues that occur along the Rangitaki River appear to result from much larger flows from yearly or greater natural flood events.

References

1. The Bay of Plenty Regional Council; Right in Respect of Natural Water Consent Number 02 2195/1; Dated 23 November 1989.
2. Revising the Operating Regime at the Matahina HEPS.
3. Beca Carter Hollings and Ferner Ltd; Twin Peak Trial Period Monitoring – October/November 2003; Dated November 2003.
4. Beca Infrastructure Ltd; Matahina Piezometer Installation; May 2007.
5. Beca Infrastructure Ltd; Matahina HEPS – Biennial Rangitaiki River Monitoring; August 2008.
6. Beca Infrastructure Ltd; Factual Report Matahina Groundwater Monitoring; September 2008.

Figures



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Rangitaiki River
Erosion Monitoring Site Locations

Figure 1

Beca

Figure 2 Projected Recession Rates for 45 MW Twin Peak

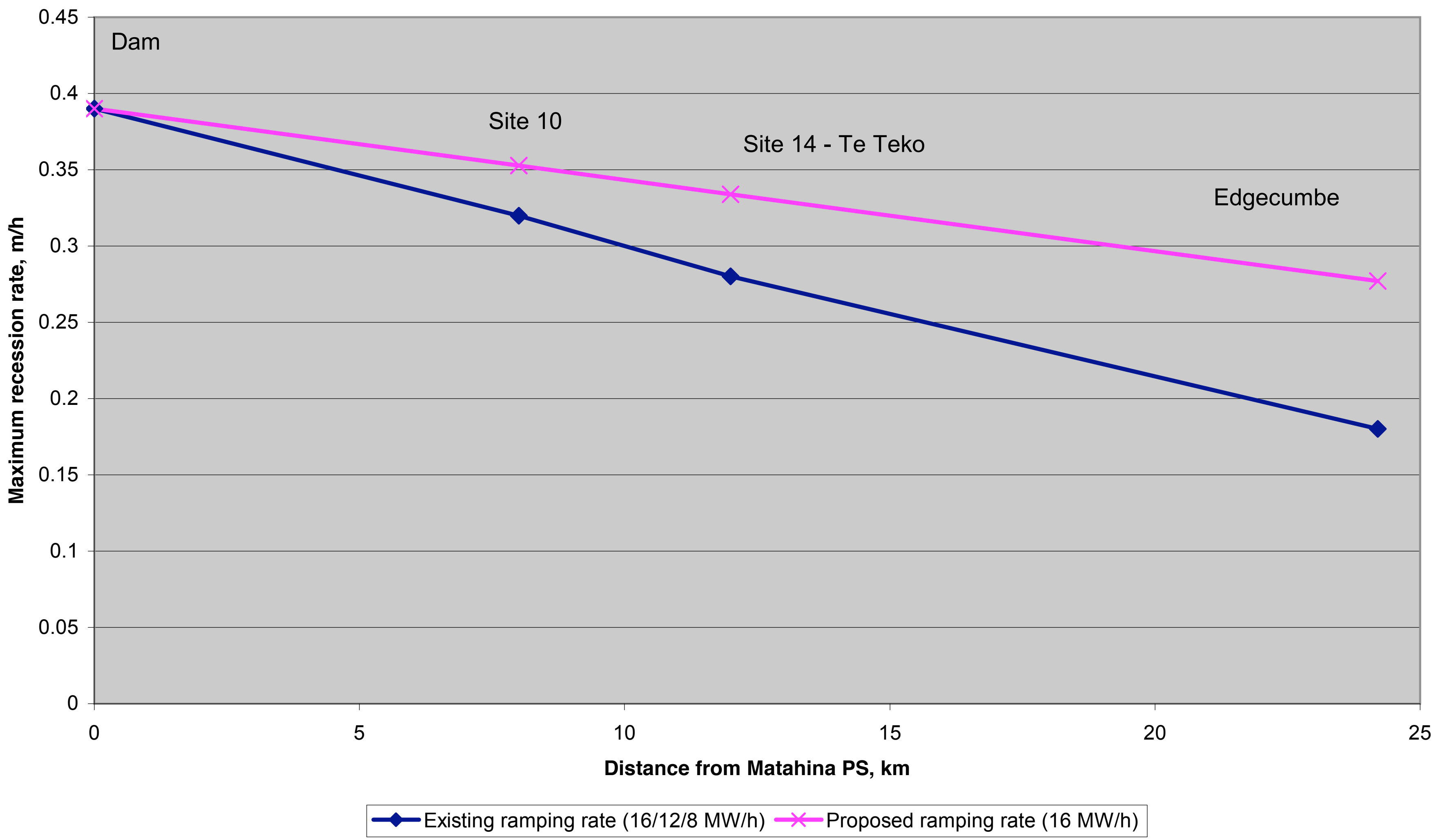


Figure 3 - Monitoring Site 10: Hydrograph Projections

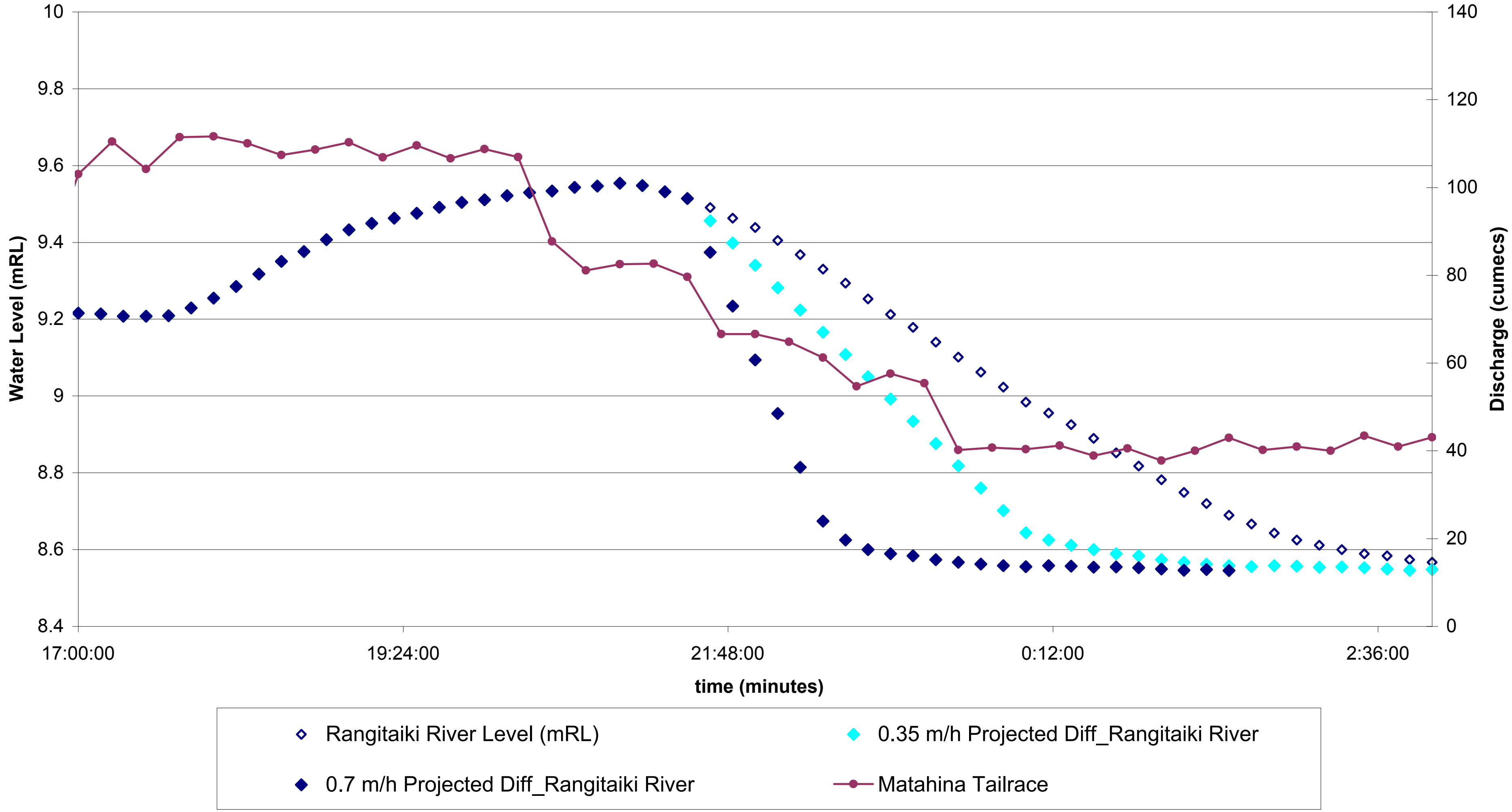
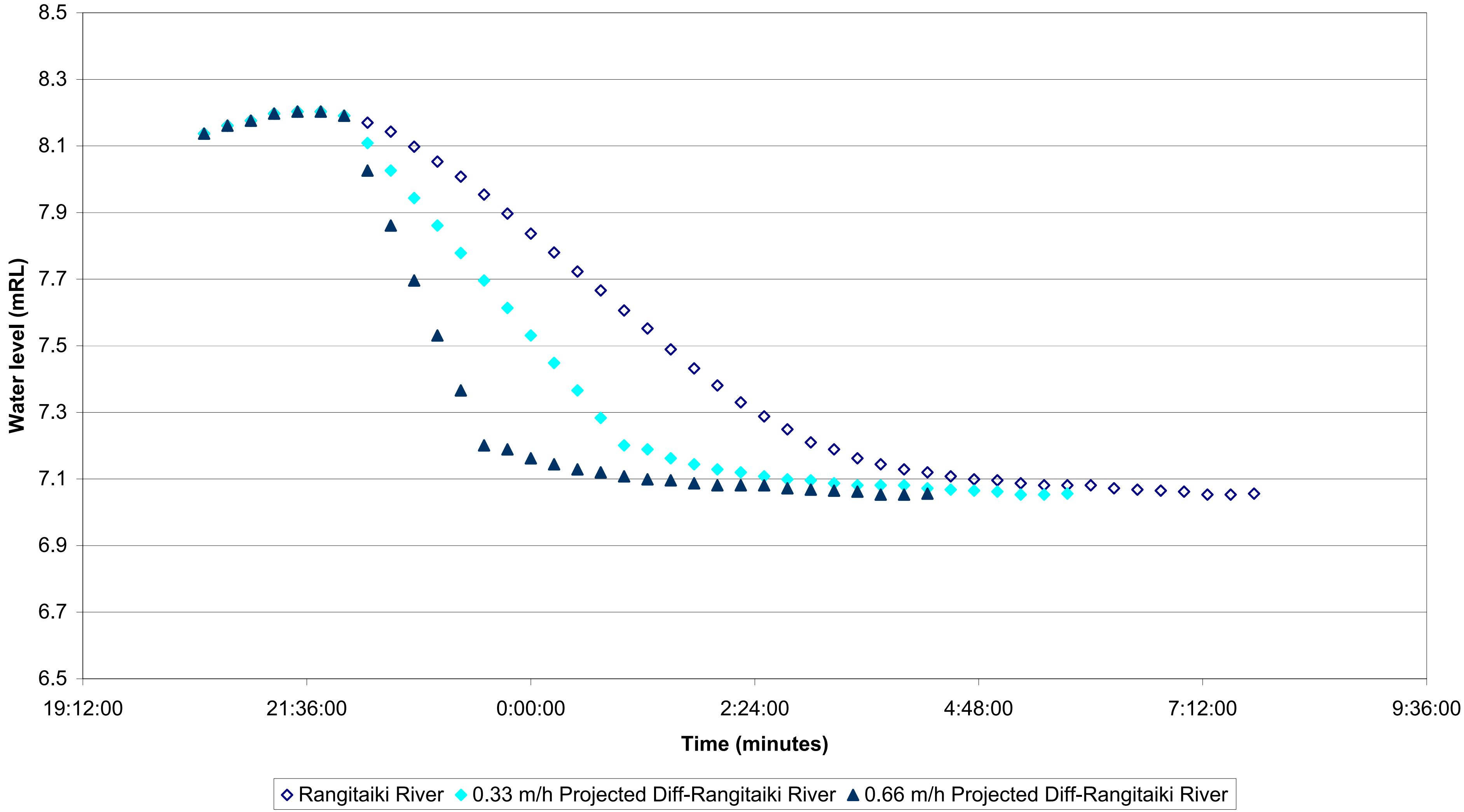
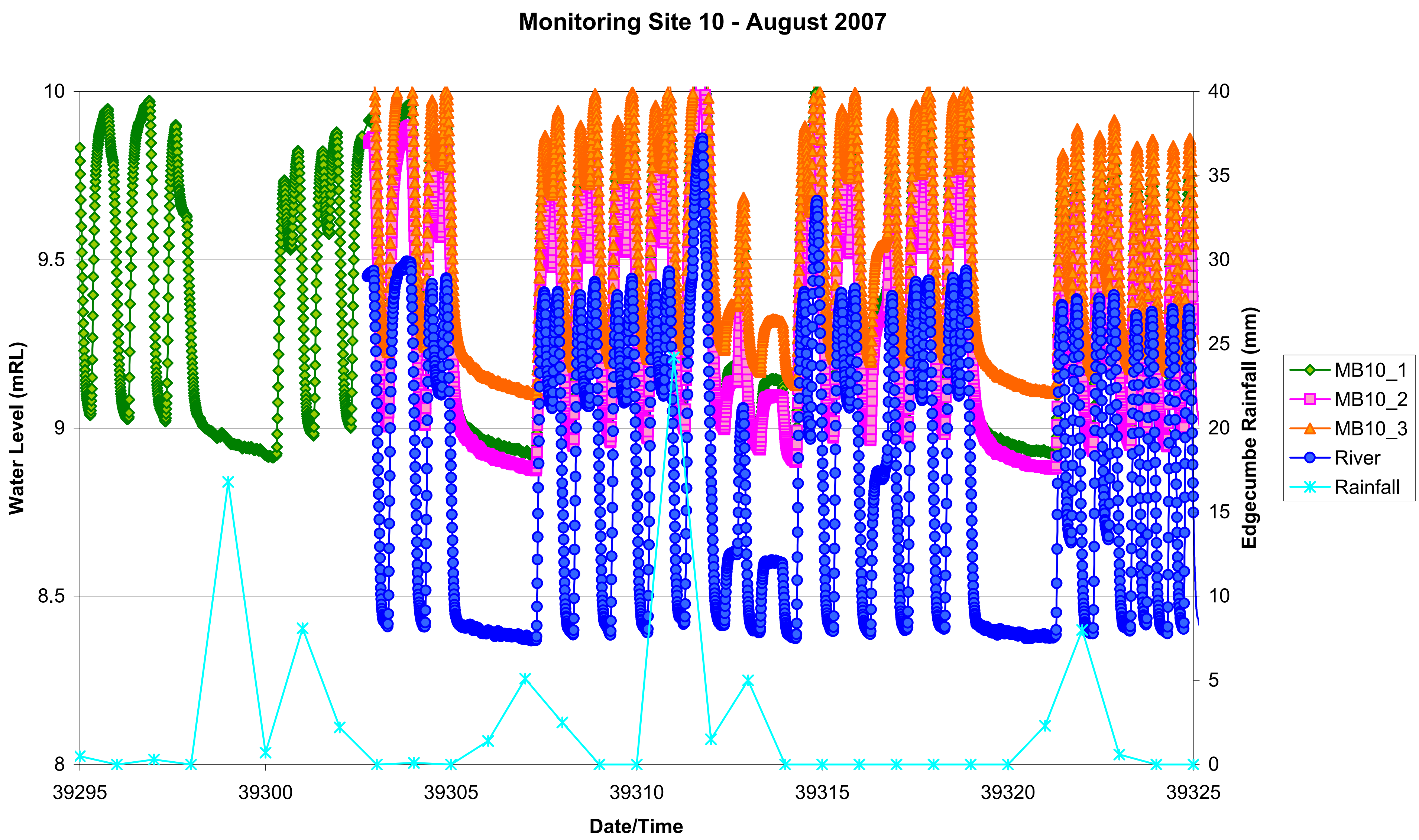


Figure 4 - Monitoring Site 14: Hydrograph Projections

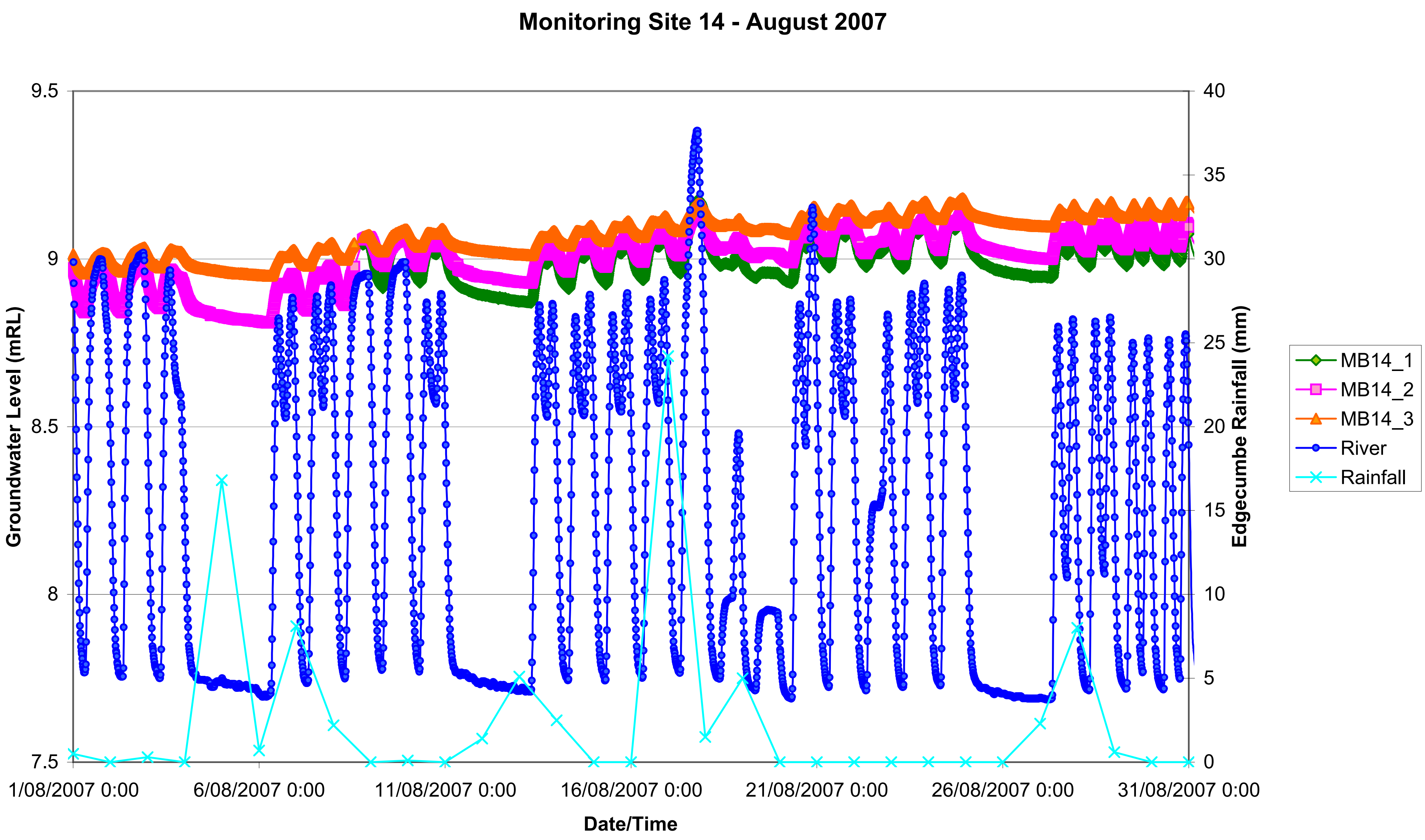


Appendix A

River and Groundwater Level Data



Monitoring Site 10 - August 2007
Water Level (mRL)
10
9.5
9
8.5
8
Edgecumbe Rainfall (mm)
40
35
30
25
20
15
10
5
0
39295
39300
39305
39310
39315
39320
39325
Date/Time
MB10_1
MB10_2
MB10_3
River
Rainfall



Monitoring Site 14 - August 2007
Groundwater Level (mRL)
9.5
9
8.5
8
7.5
Edgecumbe Rainfall (mm)
40
35
30
25
20
15
10
5
0
1/08/2007 0:00
6/08/2007 0:00
11/08/2007 0:00
16/08/2007 0:00
21/08/2007 0:00
26/08/2007 0:00
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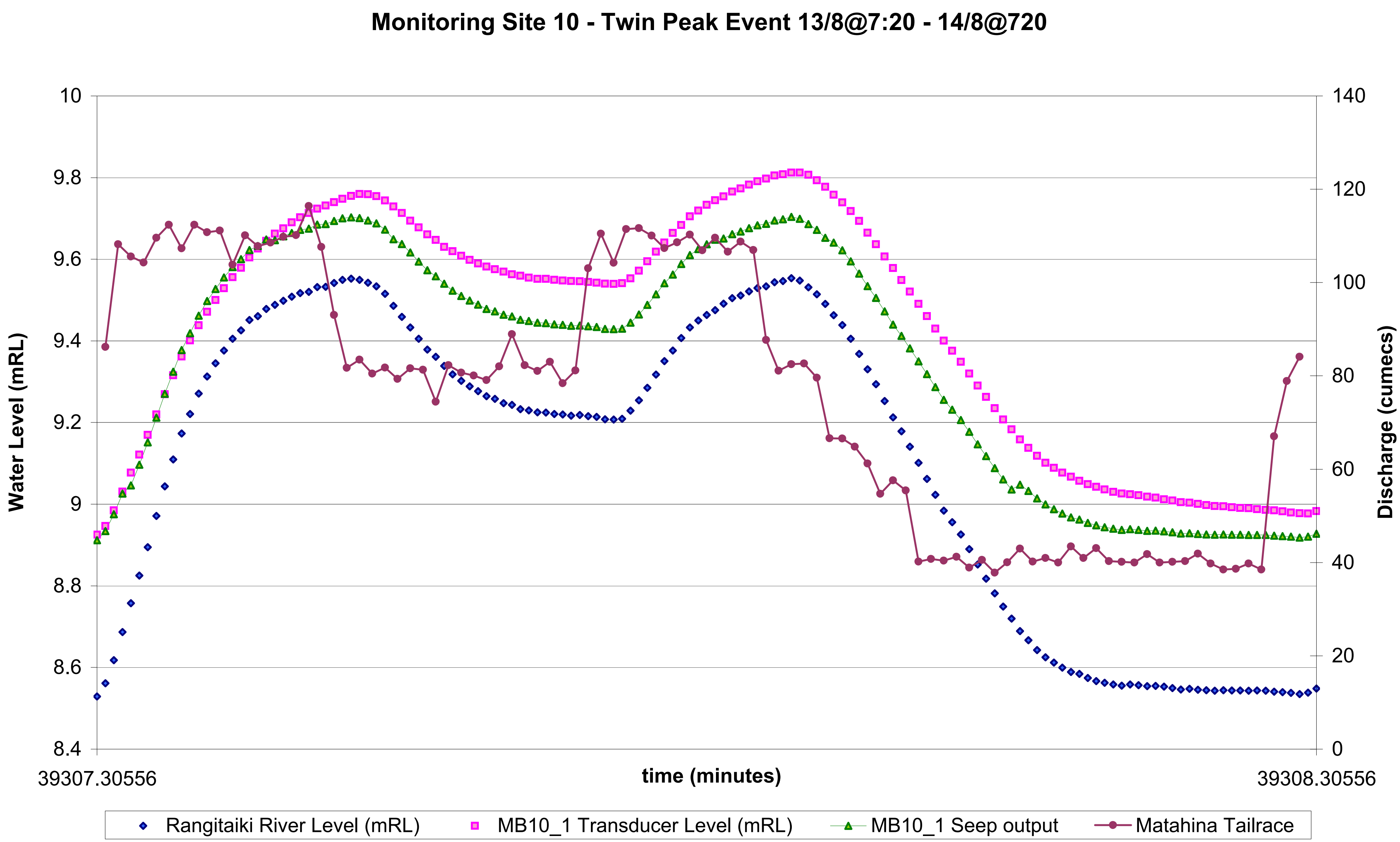
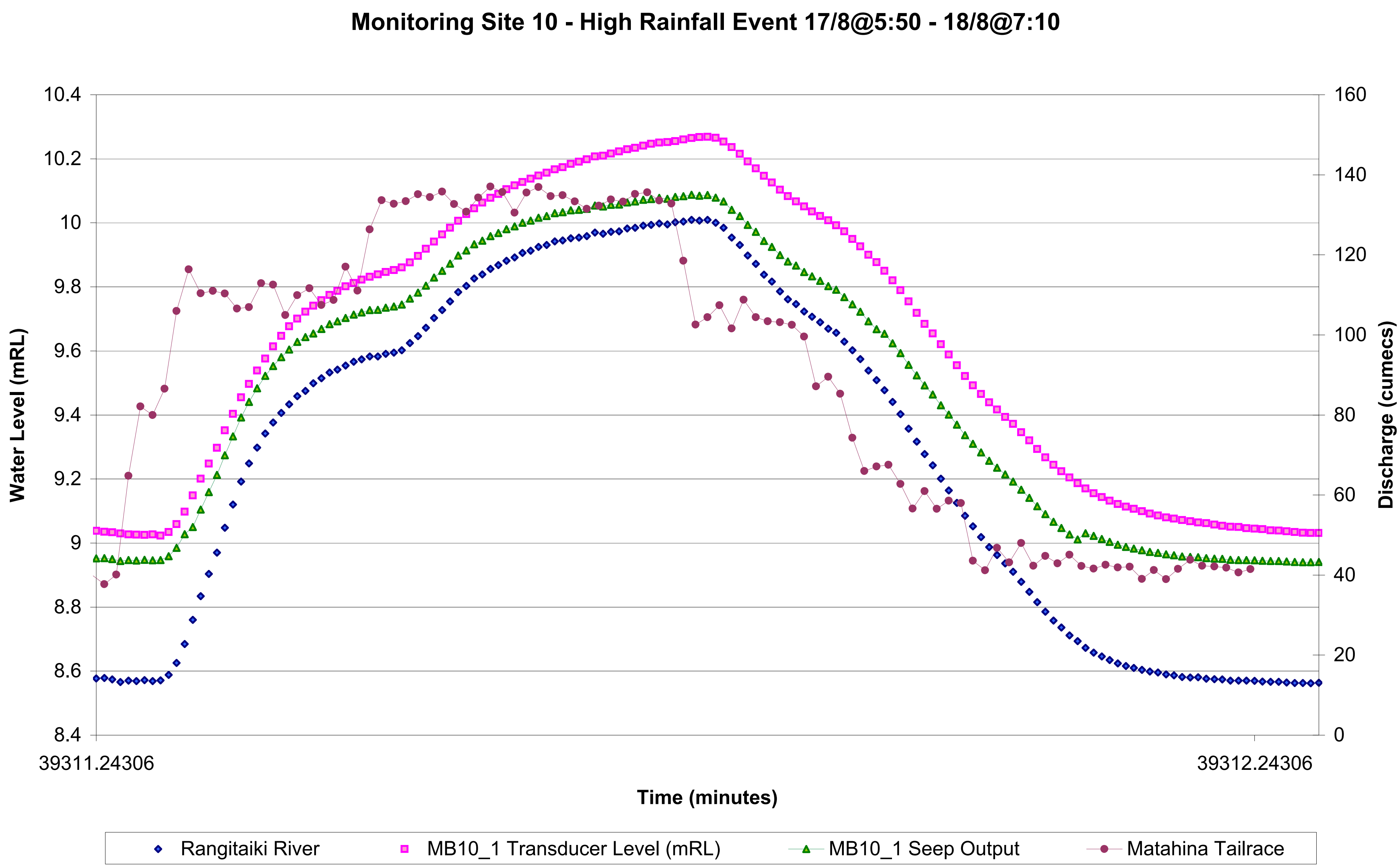
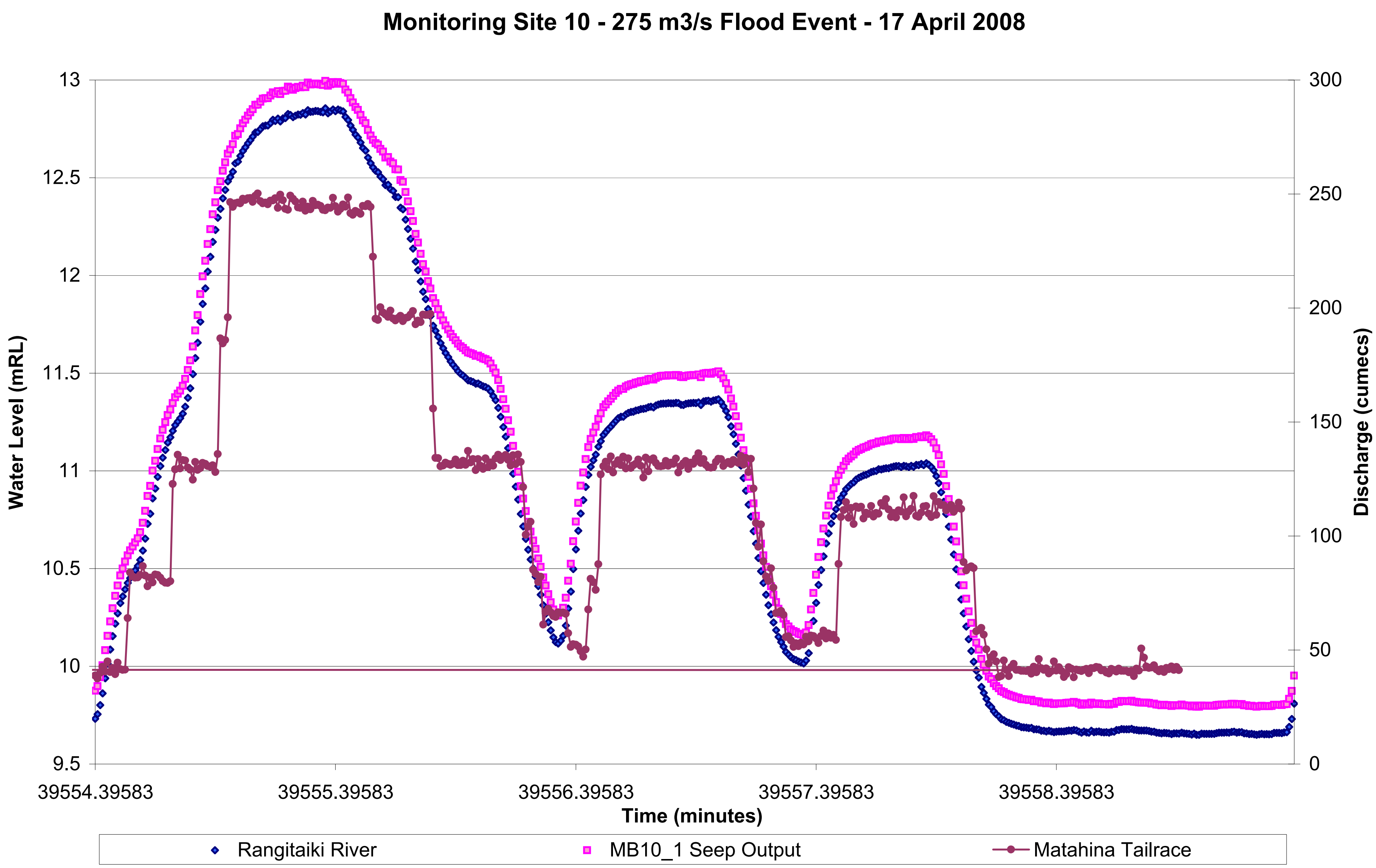
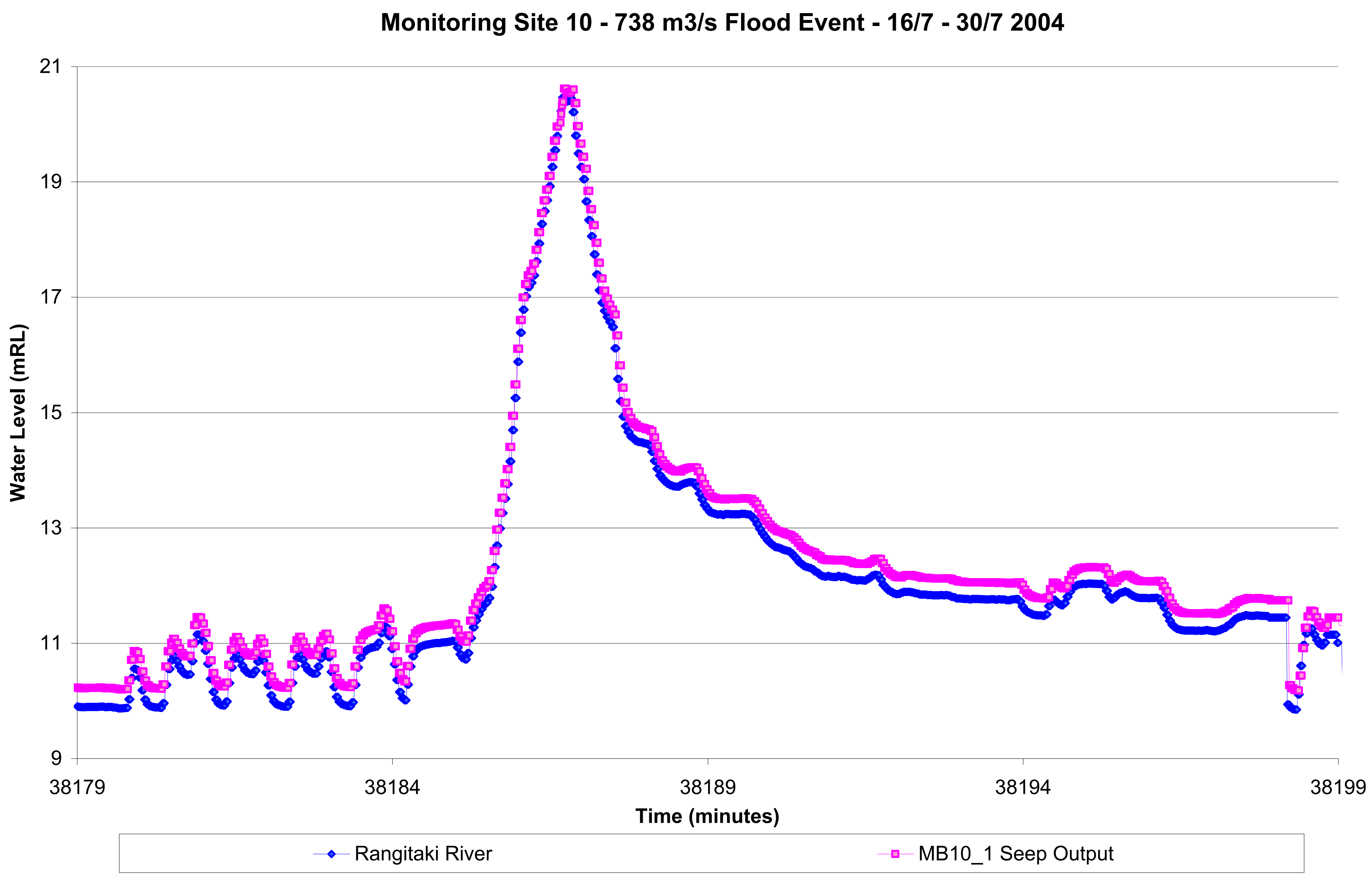
Date/Time
MB14_1
MB14_2
MB14_3
River
Rainfall



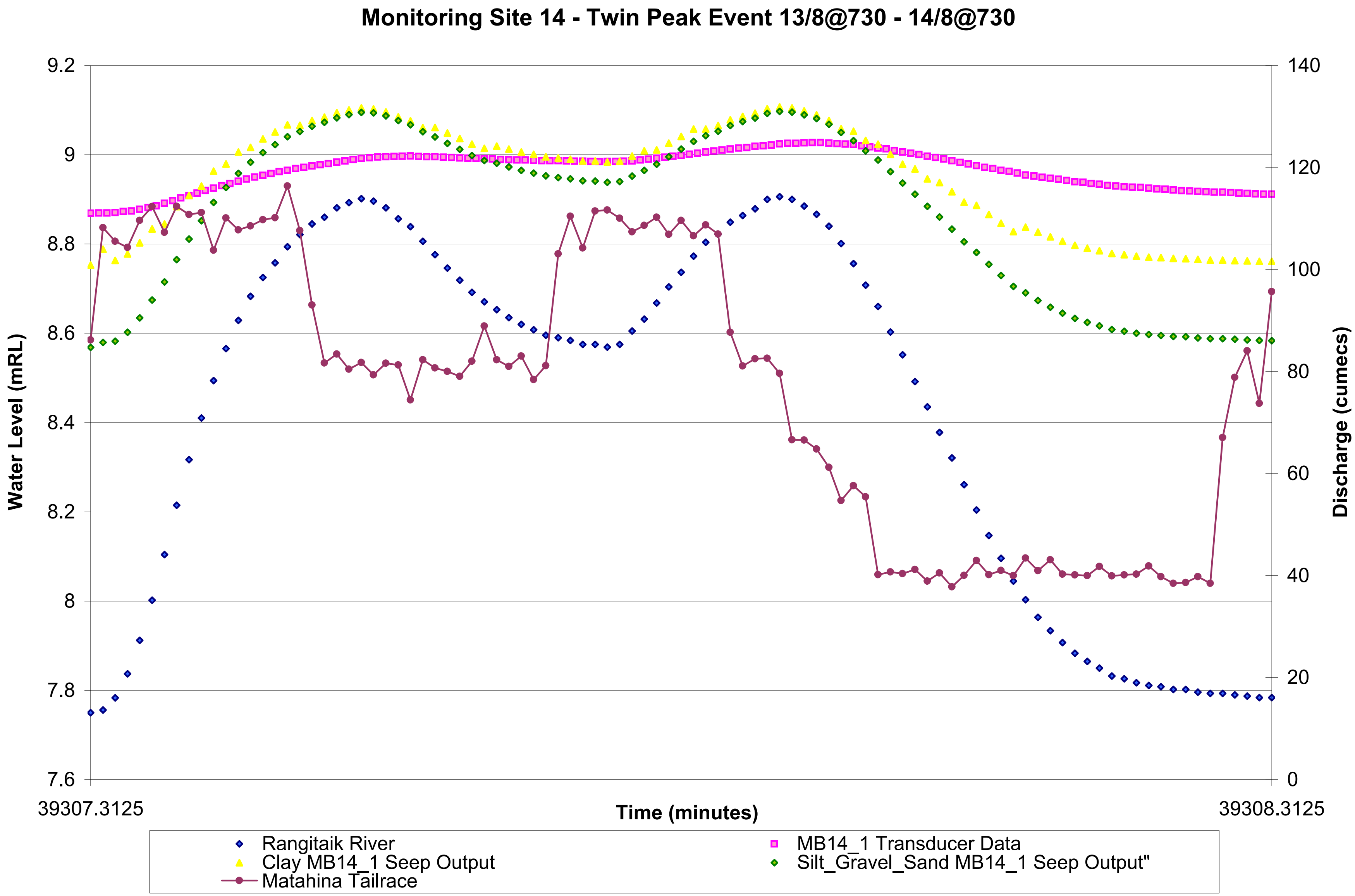
Monitoring Site 10 - Twin Peak Event 13/8@7:20 - 14/8@720
Water Level (mRL)
10
9.8
9.6
9.4
9.2
9
8.8
8.6
8.4
Discharge (cumecs)
140
120
100
80
60
40
20
0
39307.30556
time (minutes)
39308.30556
Rangitaiki River Level (mRL)
MB10_1 Transducer Level (mRL)
MB10_1 Seep output
Matahina Tailrace


Monitoring Site 10 - High Rainfall Event 17/8@5:50 - 18/8@7:10
Water Level (mRL)
10.4
10.2
10
9.8
9.6
9.4
9.2
9
8.8
8.6
8.4
Discharge (cumecs)
160
140
120
100
80
60
40
20
0
39311.24306
39312.24306
Time (minutes)
Rangitaiki River
MB10_1 Transducer Level (mRL)
MB10_1 Seep Output
Matahina Tailrace

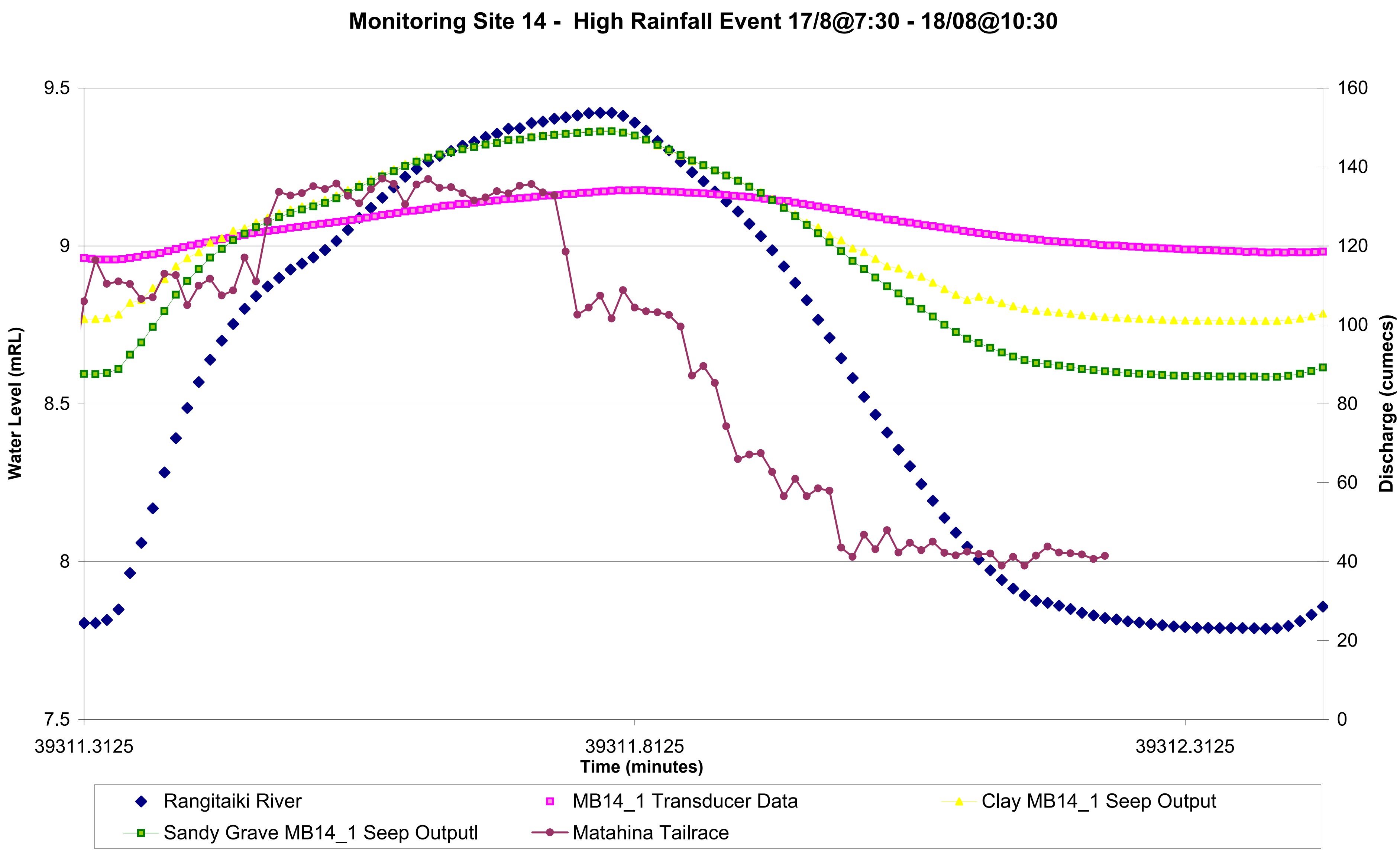

Monitoring Site 10 - 275 m3/s Flood Event - 17 April 2008
Water Level (mRL)
13
12.5
12
11.5
11
10.5
10
9.5
Discharge (cumecs)
300
250
200
150
100
50
0
39554.39583
39555.39583
39556.39583
39557.39583
39558.39583
Time (minutes)
Rangitaiki River
MB10_1 Seep Output
Matahina Tailrace

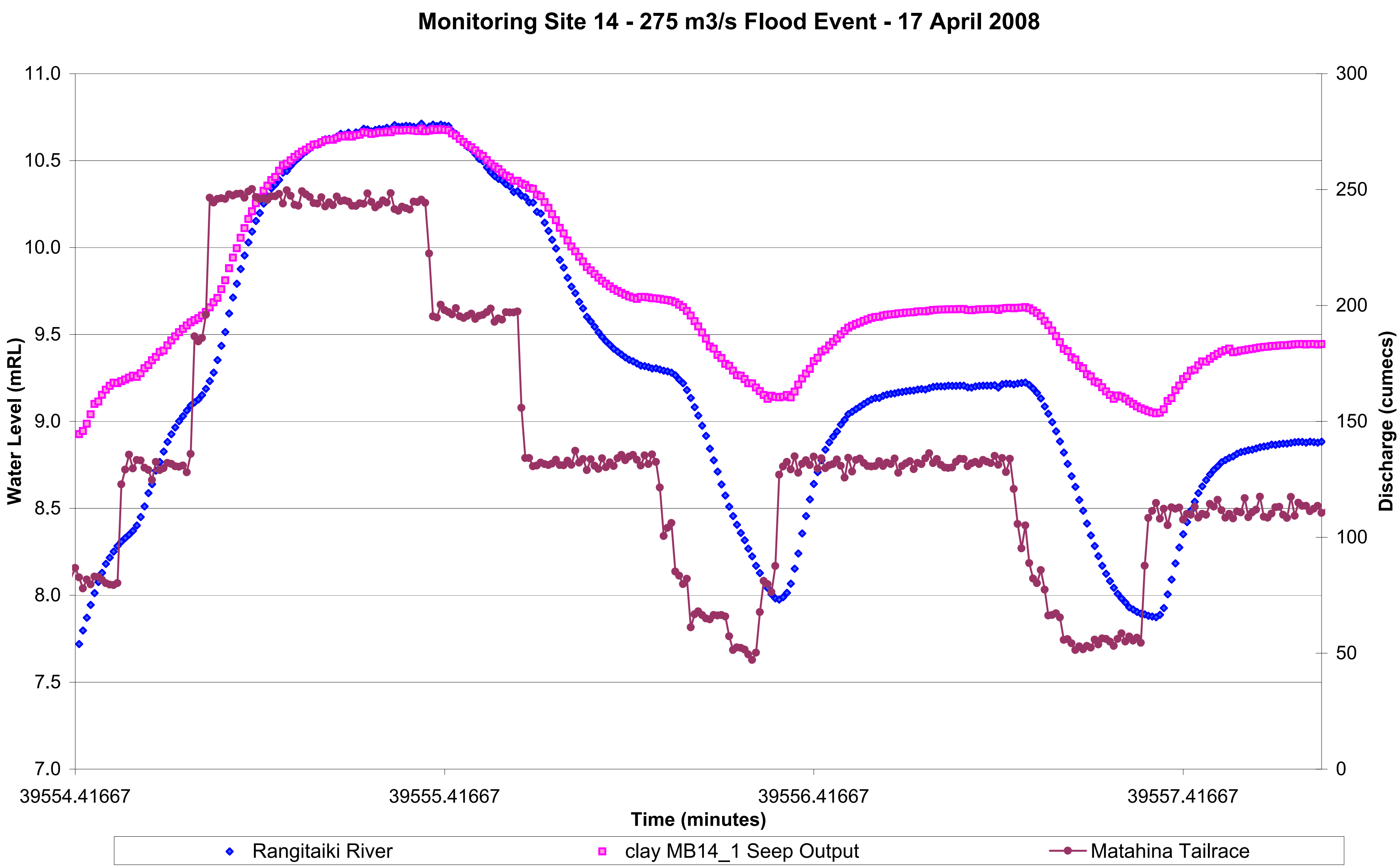


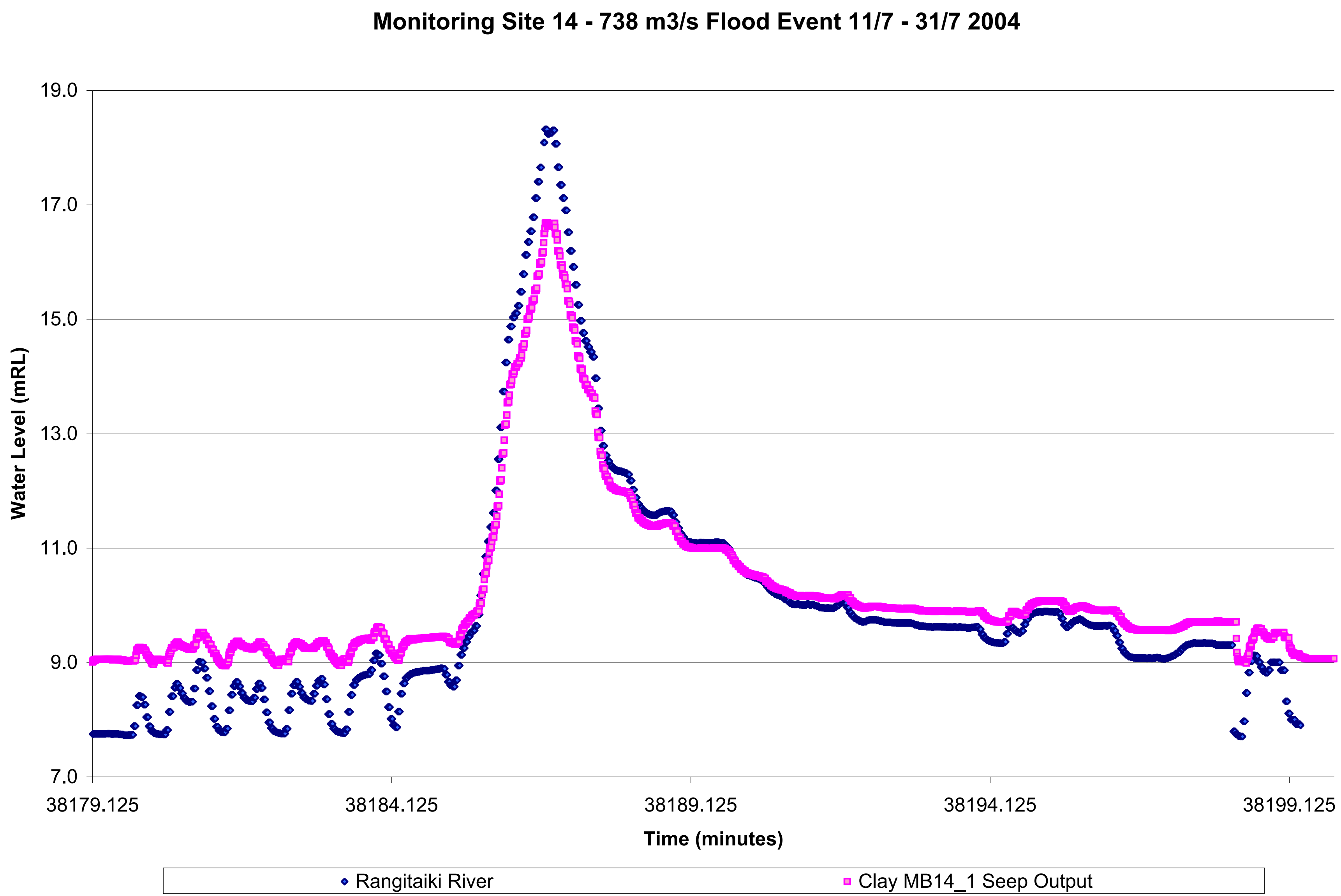
Monitoring Site 10 - 738 m3/s Flood Event - 16/7 - 30/7 2004
Water Level (mRL)
21
19
17
15
13
11
9
38179
38184
38189
38194
38199
Time (minutes)
Rangitaki River
MB10_1 Seep Output



Monitoring Site 14 - Twin Peak Event 13/8@730 - 14/8@730
Water Level (mRL)
9.2
9
8.8
8.6
8.4
8.2
8
7.8
7.6
Discharge (cumecs)
140
120
100
80
60
40
20
0
39307.3125
39308.3125
Time (minutes)
Rangitaik River
MB14_1 Transducer Data
Clay MB14_1 Seep Output
Silt_Gravel_Sand MB14_1 Seep Output"
Matahina Tailrace


Monitoring Site 14 - High Rainfall Event 17/8@7:30 - 18/08@10:30
9.5
9
8.5
8
7.5
160
140
120
100
80
60
40
20
0
Water Level (mRL)
Discharge (cumecs)
39311.3125
39311.8125
39312.3125
Time (minutes)
Rangitaiki River
MB14_1 Transducer Data
Clay MB14_1 Seep Output
Sandy Grave MB14_1 Seep Outputl
Matahina Tailrace

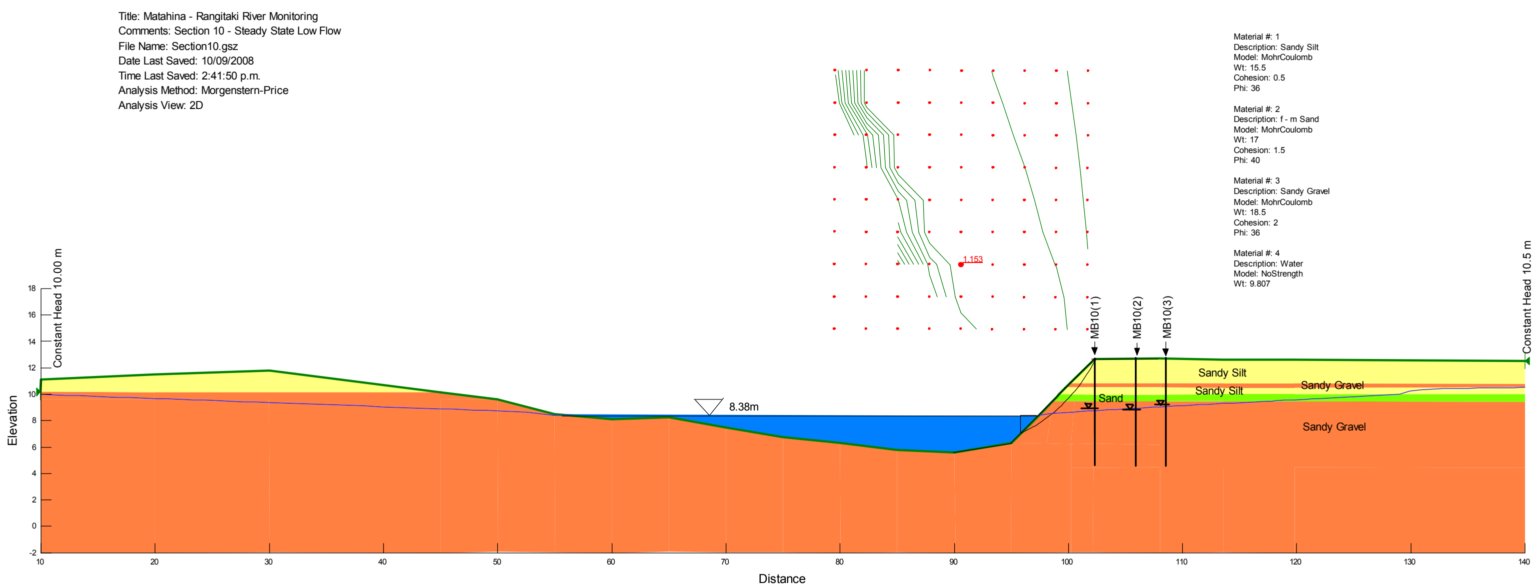

Monitoring Site 14 - 275 m3/s Flood Event - 17 April 2008
Water Level (mRL)
11.0
10.5
10.0
9.5
9.0
8.5
8.0
7.5
7.0
Discharge (cumecs)
300
250
200
150
100
50
0
39554.41667
39555.41667
39556.41667
39557.41667
Time (minutes)
Rangitaiki River
clay MB14_1 Seep Output
Matahina Tailrace

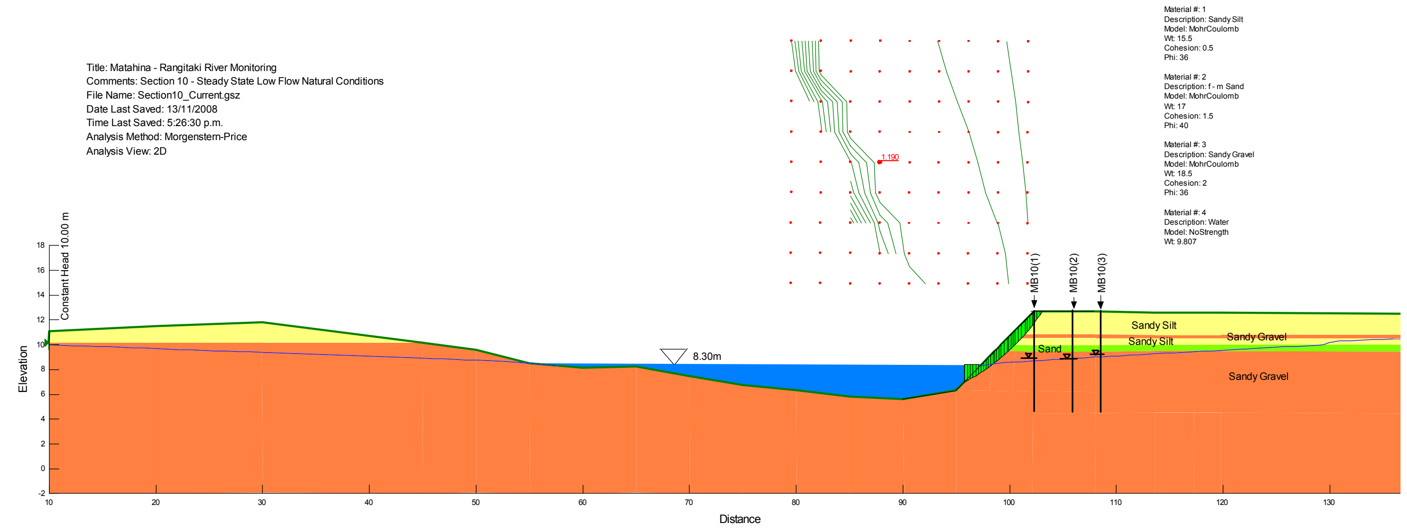


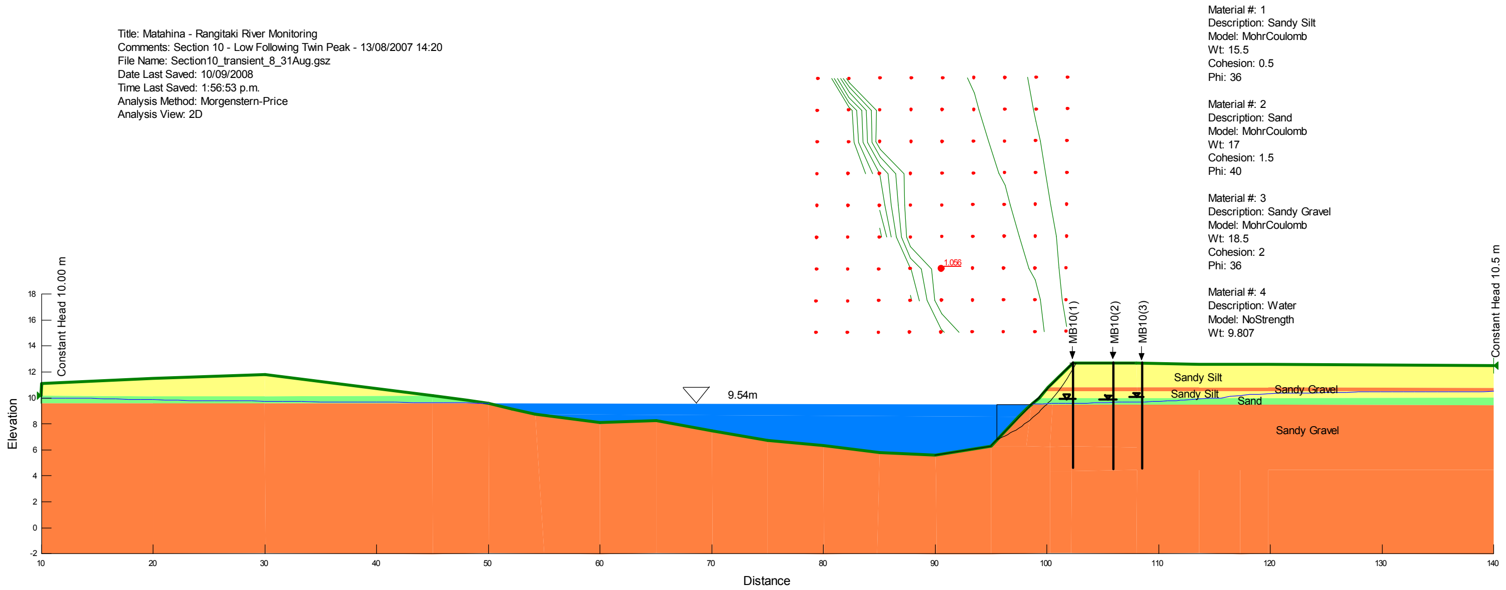
Monitoring Site 14 - 738 m3/s Flood Event 11/7 - 31/7 2004
19.0
17.0
15.0
13.0
11.0
9.0
7.0
Water Level (mRL)
38179.125
38184.125
38189.125
38194.125
38199.125
Time (minutes)
Rangitaiki River
Clay MB14_1 Seep Output

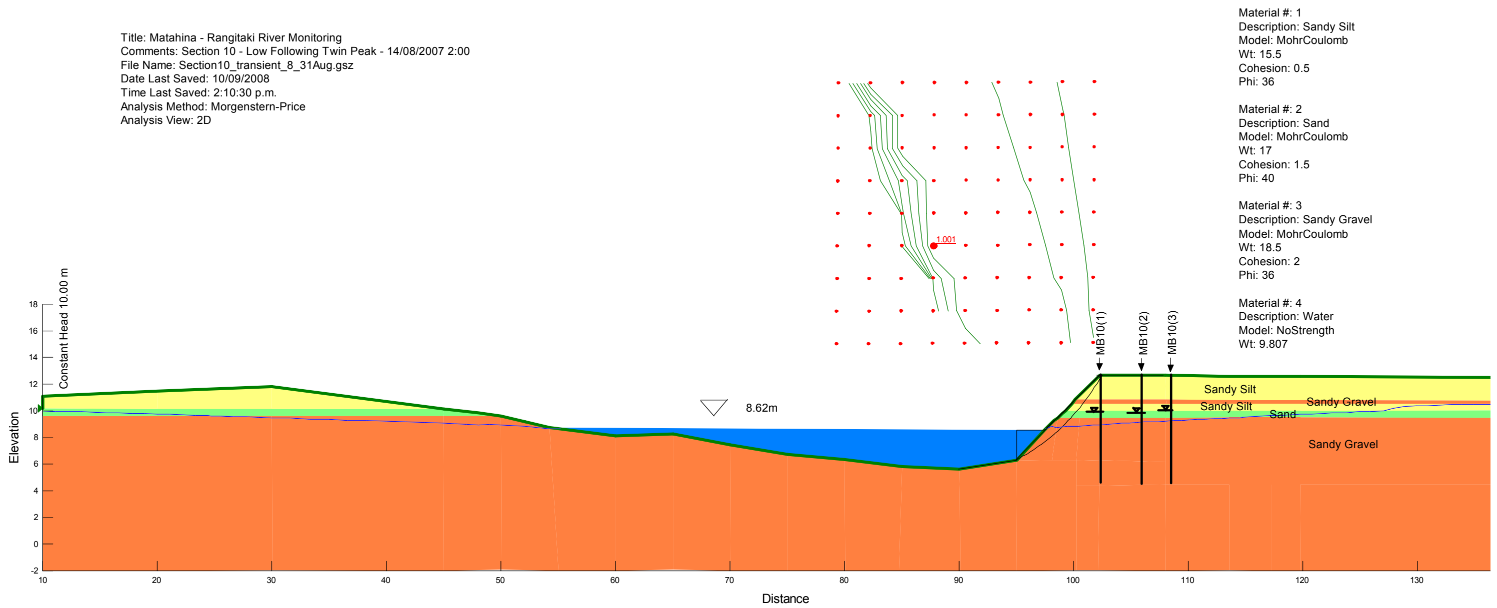
Appendix B

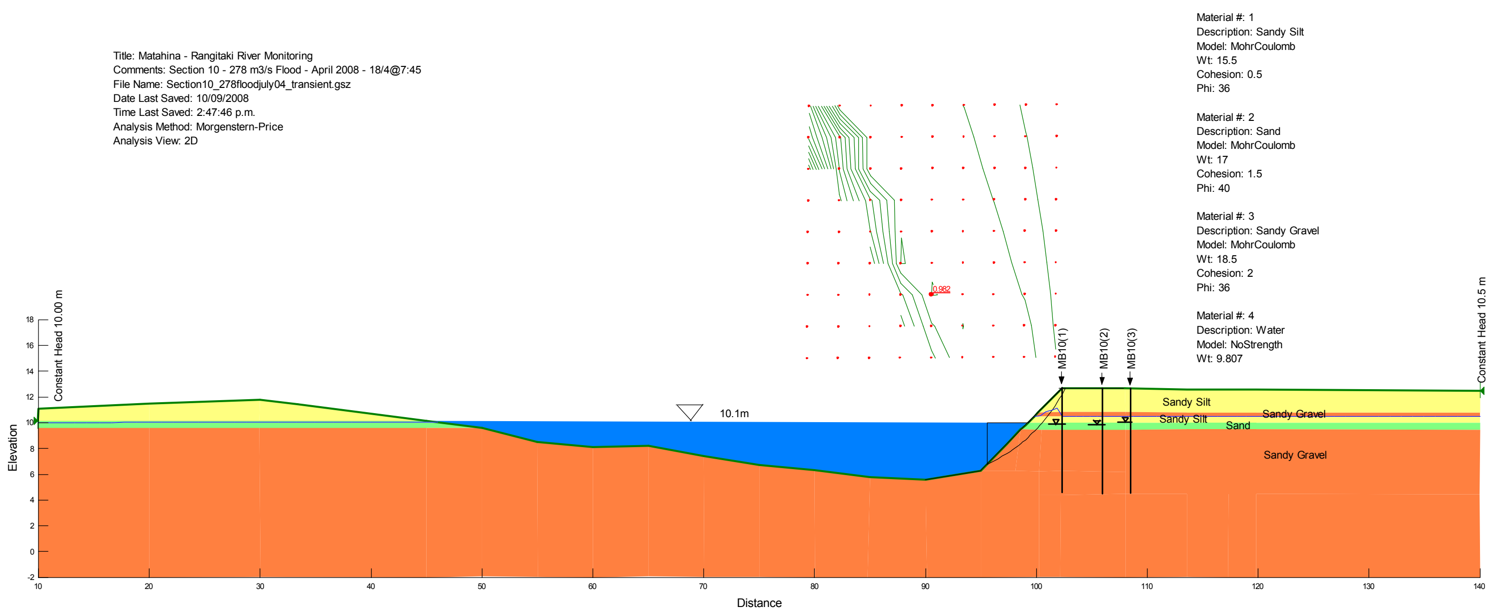
Slope/W Modelling Output

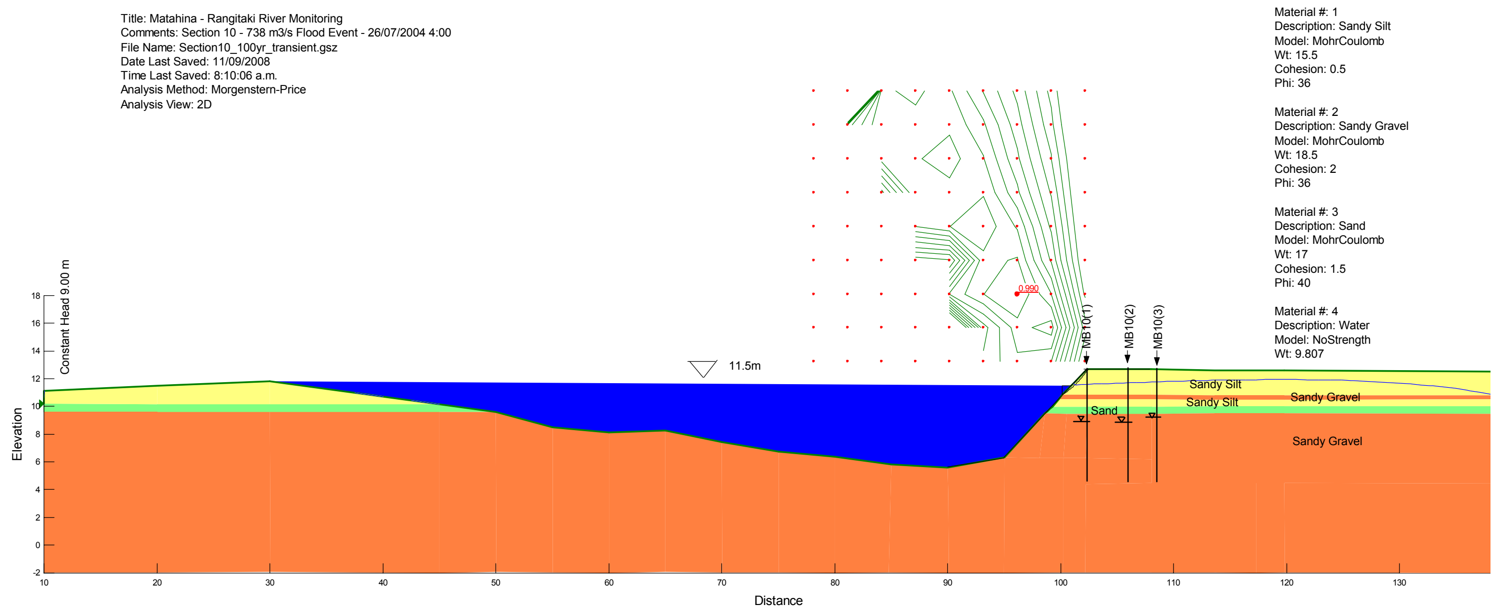




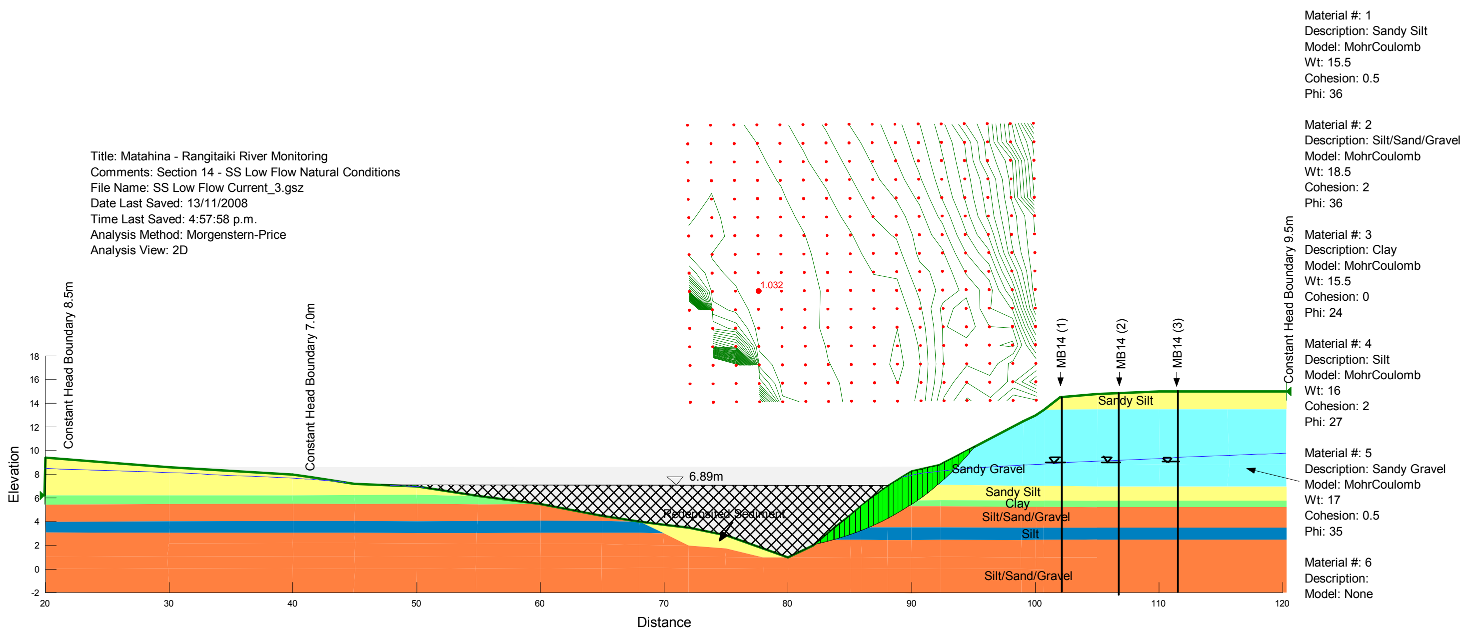


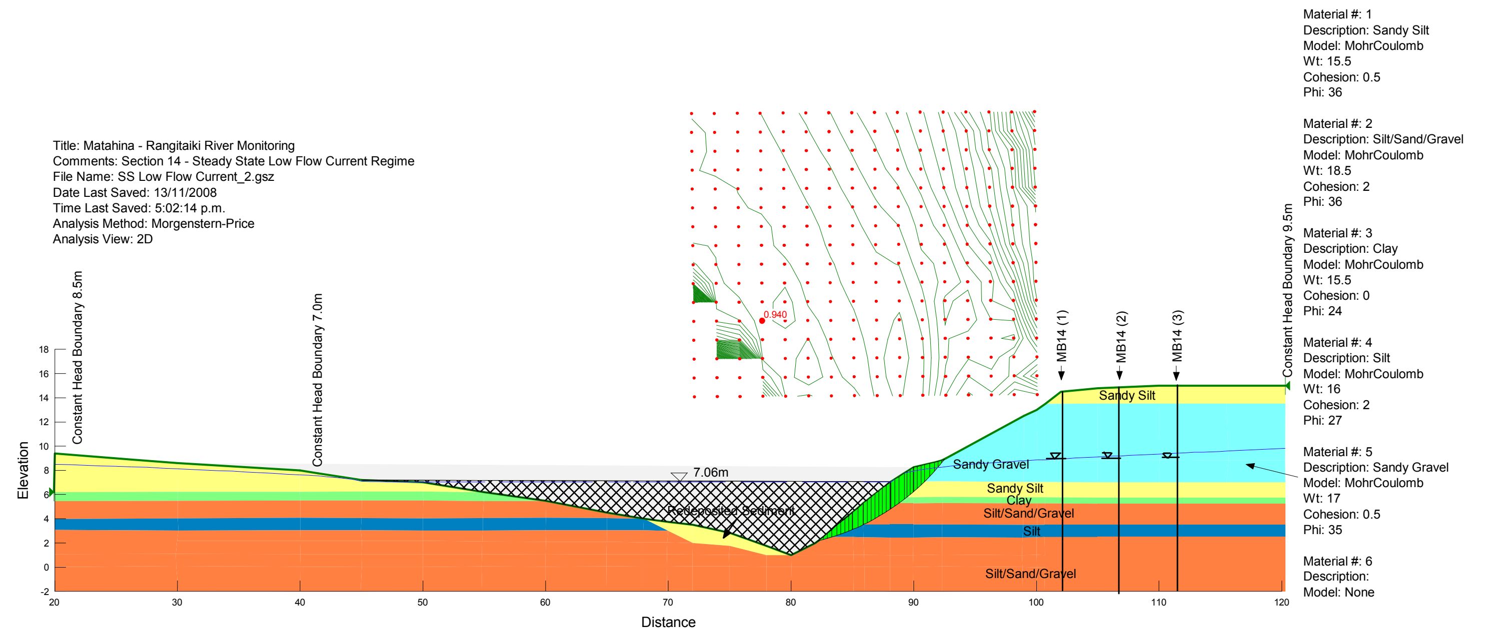


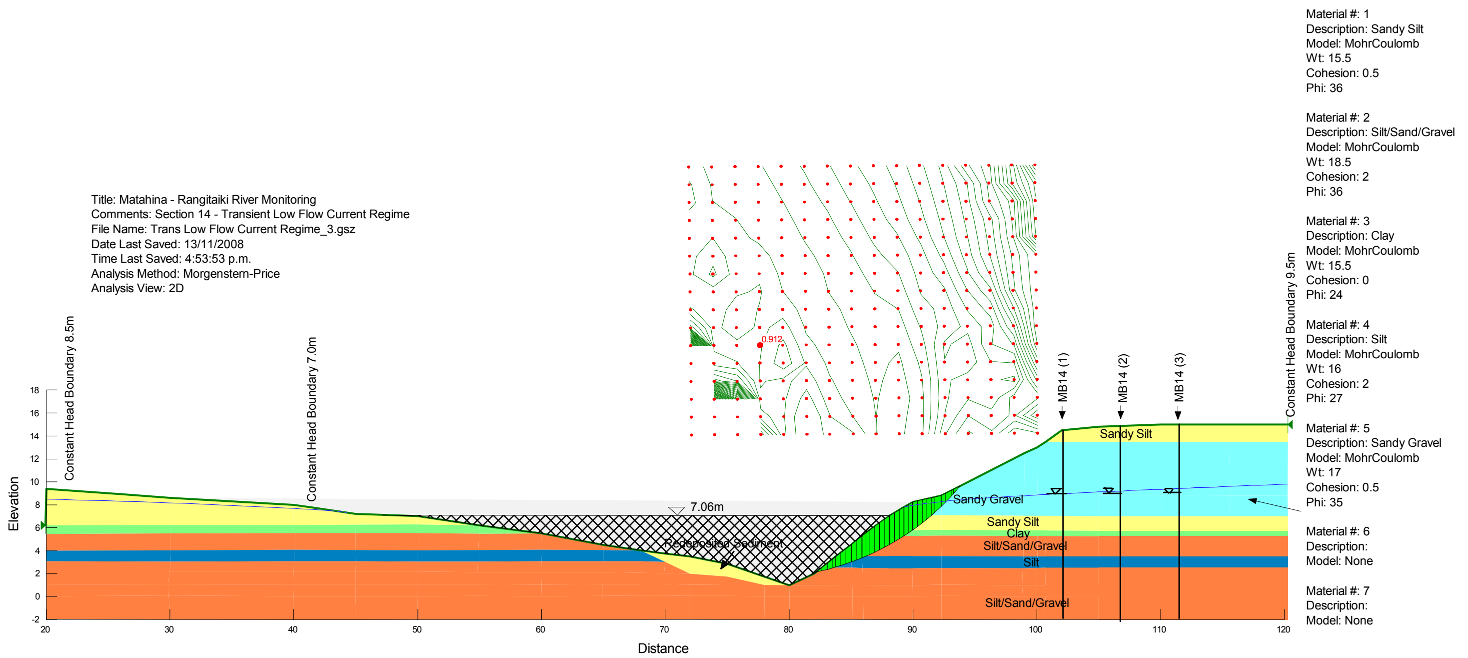


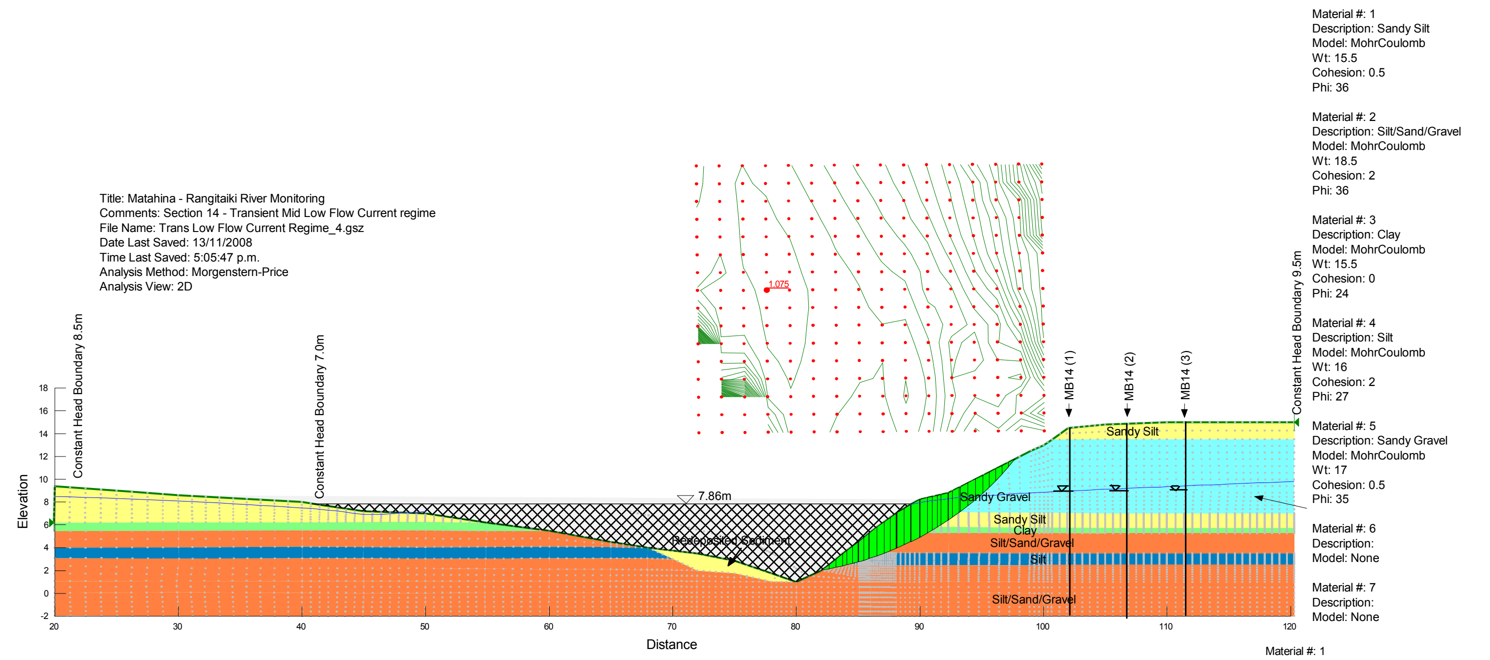


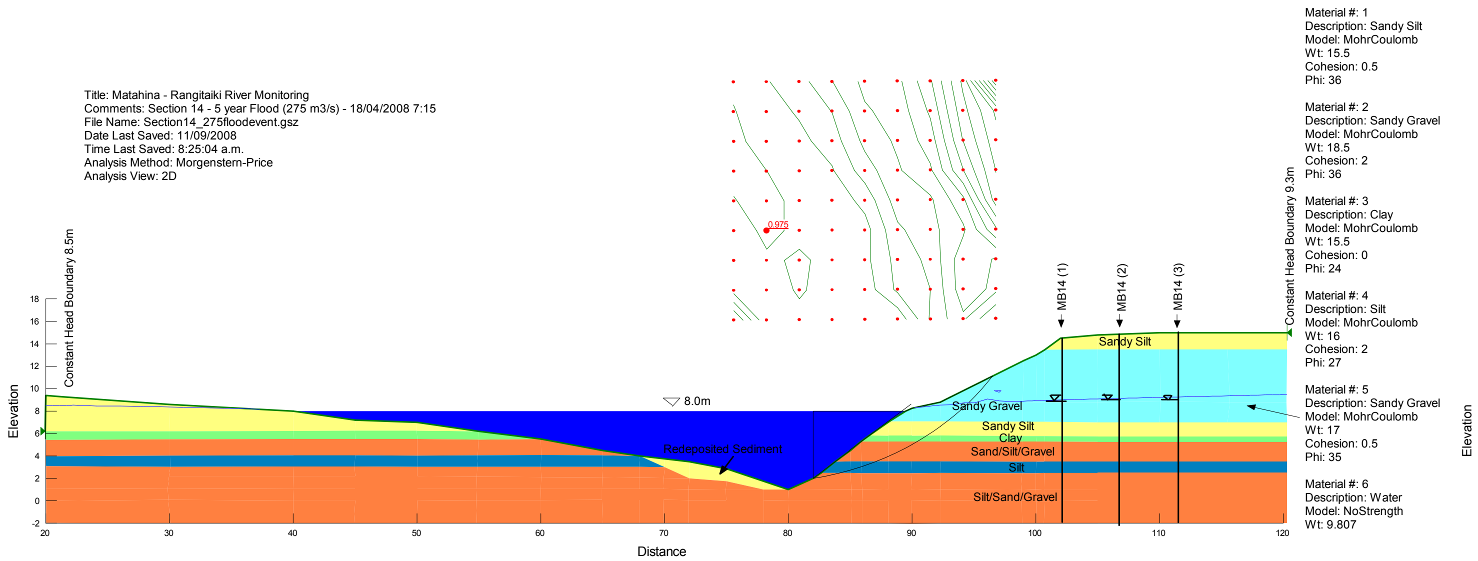
Site 10 - Slope Stability Outputs

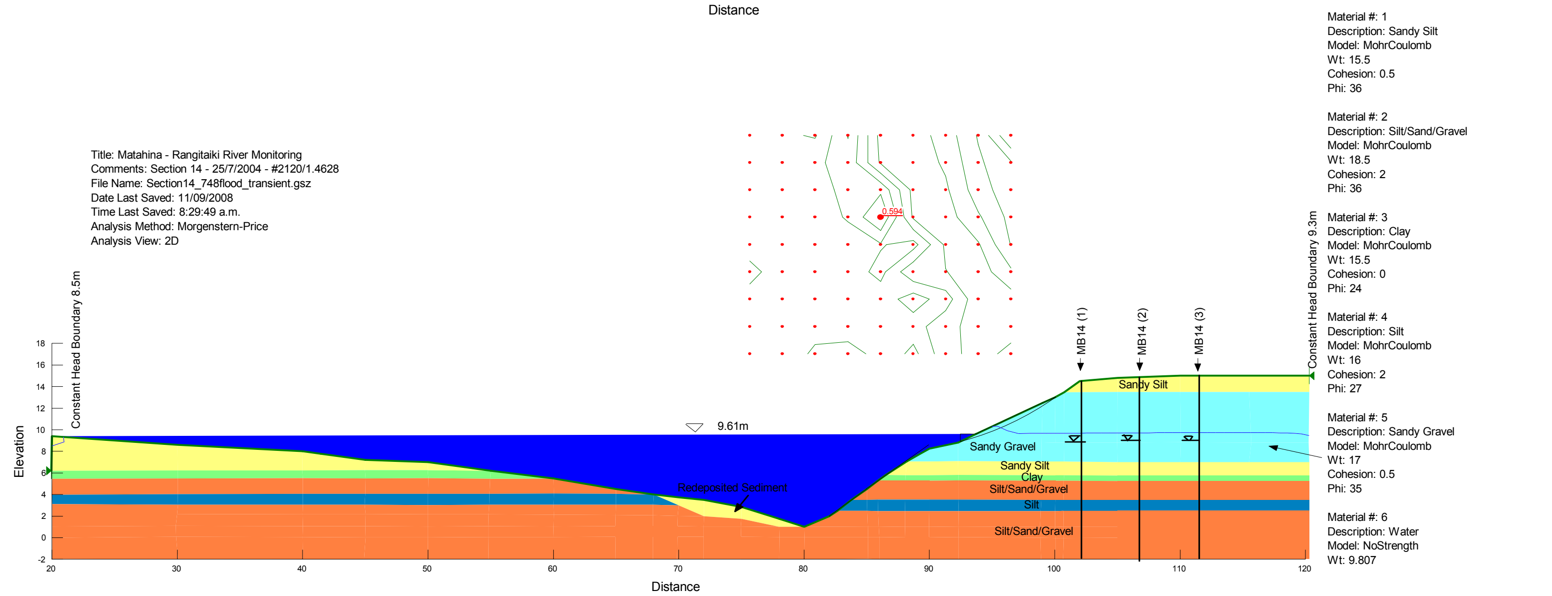




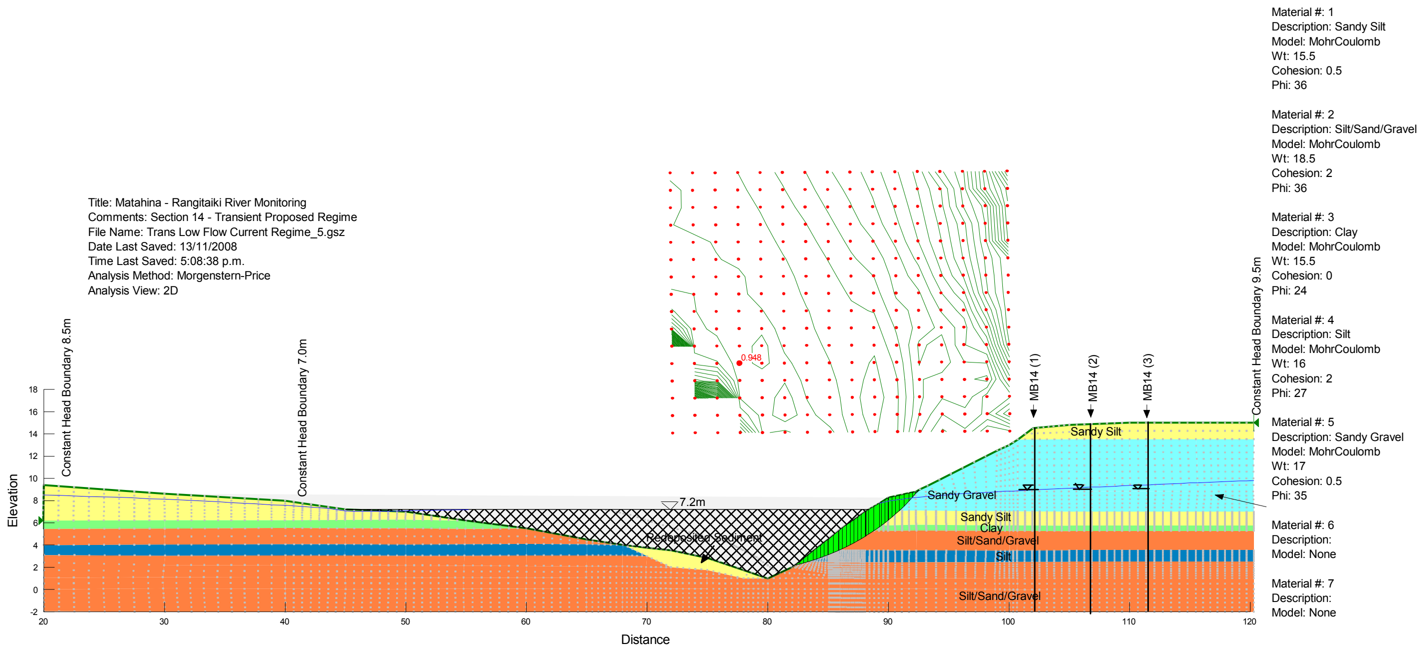


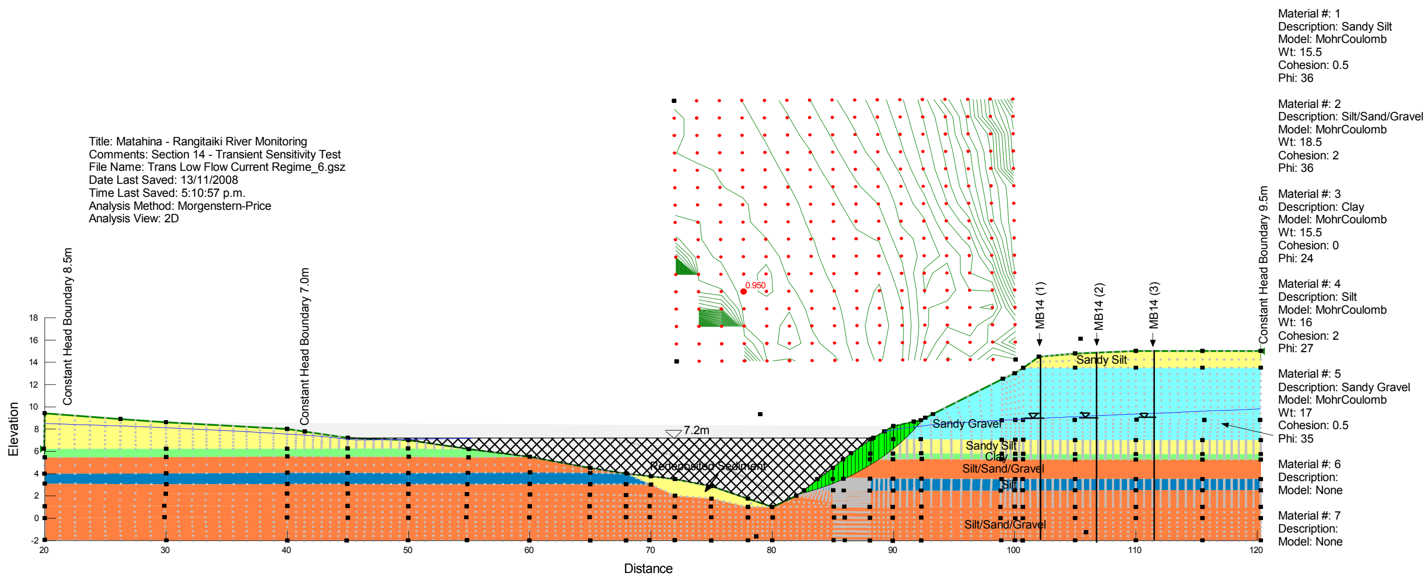




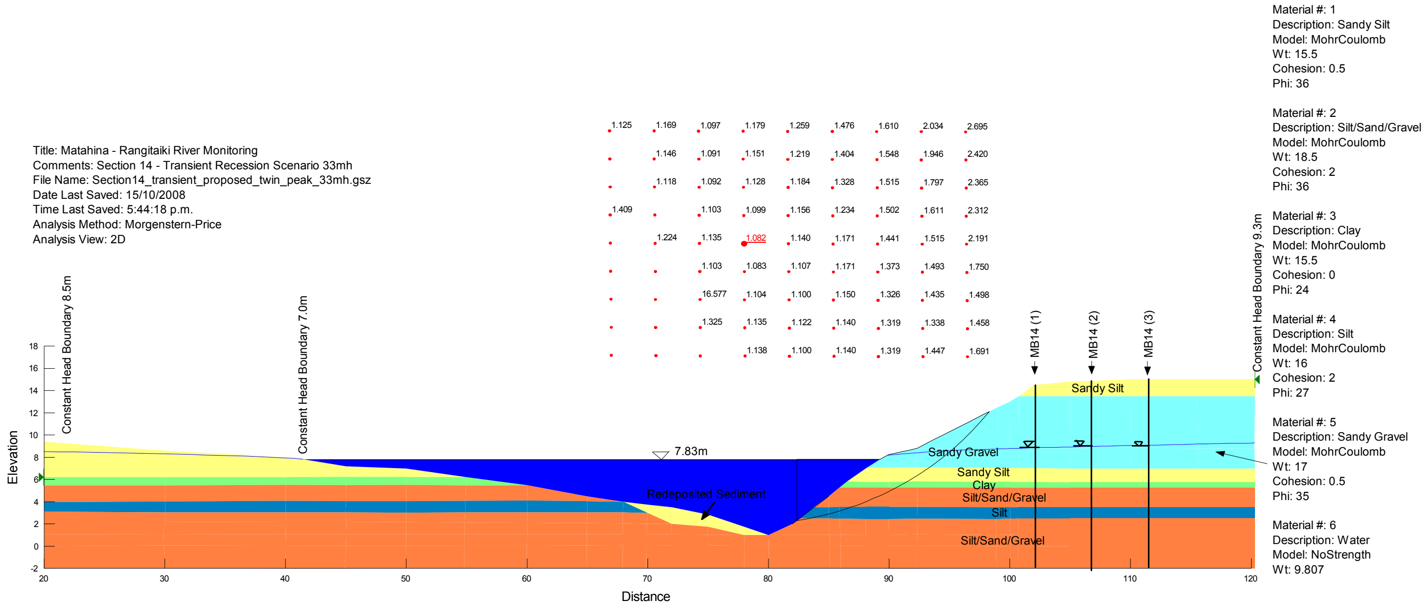


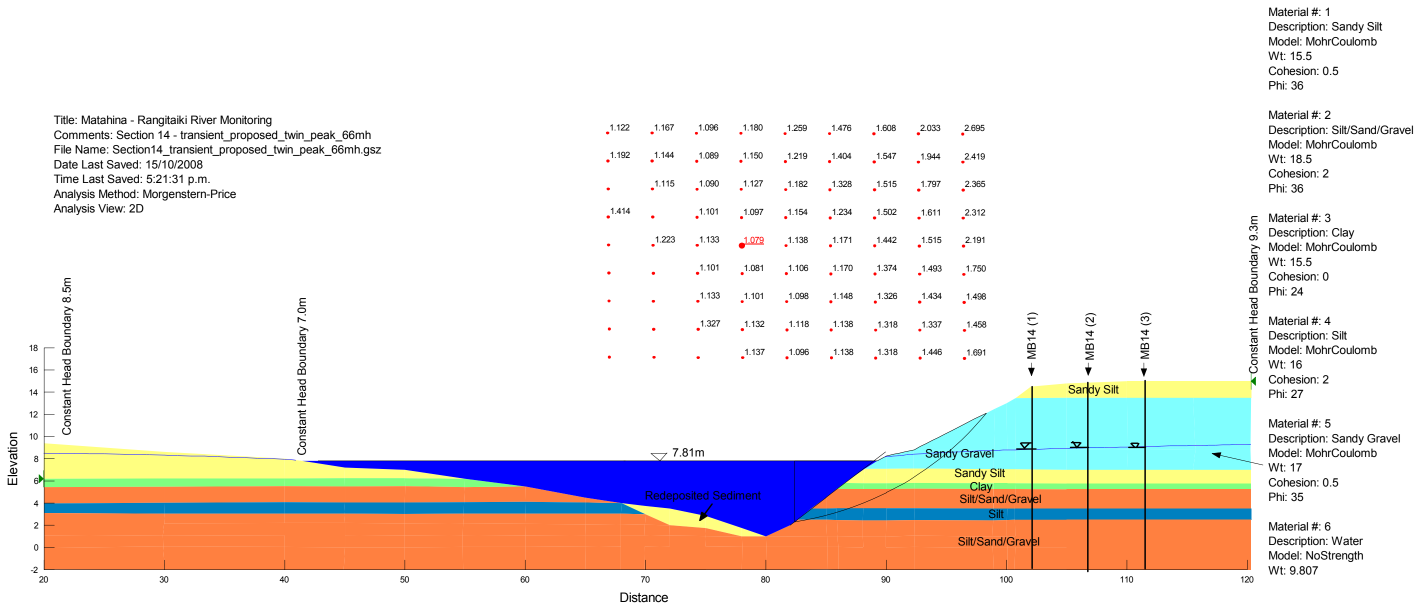
Site 14 - Slope Stability Outputs





Site 10 - Slope Stability Outputs





Site 14 - Slope Stability Outputs

Appendix C

Rangitaiki river intakes and mitigation

Water takes from Rangitaiki River downstream of Matahina dam

Consent number	Maximum quantity, m^3/day	Maximum rate, L/s	River distance from dam, km	Intake description	Pipe diameter, mm	River bed at intake m RL	Pipe IL at intake m RL	Modelled water level at intake, assuming low tide of -0.8 m RL (approximately MLWS tide), m RL					Abstractor comments on effect of 2007/2008 low flows ($25\ m^3/s$)	Indicative mitigation for $20\ m^3/s$ minimum flow
								$45\ m^3/s$	$40\ m^3/s$	$30\ m^3/s$	$25\ m^3/s$	$20\ m^3/s$		
River mouth			36.6											
Thornton Bridge			34.5											
21008	50	1	33.4 RB	Flexible pipe resting on river bed near bank	50	-1.0	on bed	-0.17	-0.22	-0.33	-0.38	-0.43	Not known (not in use)	Replace with floating intake (if intake required)
21224	1,800	40	30.6 LB	Foot valve 0.45m below water level in floating drum screen	150	-1.0	floating	0.17	0.09	-0.08	-0.17	-0.25	Daily clearing of weed.	None
21063	208	3	29.8 LB	Screen at end of pipe, anchored to waratah and adjusted as required	75	0.0	adjustable	0.29	0.20	0.01	-0.08	-0.18	Lots of problems with vortexing and air entrainment	See Note 3
	(2000)	(60)		On skids, installed as required for frost protection.	?								Not known	None
21852	82	3	29.8 LB	Screen at end of pipe, anchored to driven steel pipe and adjusted as required.	75	-0.4	adjustable	0.29	0.20	0.01	-0.08	-0.18	Add extra pipe length on end. Lots of weed.	See Note 3
21703	135	7.58	29.5 LB	2 pumps under pontoon, 1.5 m below water level.	75	-2.2	floating	0.33	0.24	0.05	-0.05	-0.15	No problems	None
62000	30,000	900	25.5 RB	Fire pump pontoon.	250	-0.8	floating	0.93	0.81	0.55	0.40	0.26	Screens grounded on riverbed. Dredged riverbed at intake.	Move intakes 50 m downstream, where bed level adjacent to bank is -1.8 m RL (on outside of bend in river)
				3 pontoon-mounted pumps. Screens extend to 1.25 m below water level.	3 x 300	-1.2	floating							
Edgecumbe Bridge			25.3											
21923	164	11.4	24.2 LB	Twin drum pontoon approx 3-4 m from bank; screem at end of pipe 0.5m below water level	75	-1.3	floating	1.16	1.03	0.74	0.58	0.41	No problems	None
Upstream limit of tidal effects			19.5											
Permitted (see Note 2)	15	?	?	Borehole pump installed in 2.4 m long concrete pipe with 25 mm holes.	50	?	?	?	?	?	?	?	Pump cut out about once per month due to low water level.	Provide storage or replace intake
60427	6,500	75	18.0 RB	Rotating screen on end of pipe. Backflushable. Hangs off shore-mounted A-frame.	300	2.3	adjustable	3.77	3.67	3.47	3.35	3.22	No problems	None
63177	2,880	100	16.8 RB	Screen in frame on bed, connected to drum float: lifts off bottom in flood.	120	3.4	3.90	4.69	4.60	4.40	4.30	4.19	Cannot get screen low enough. Screen further out creates withdrawing problem.	Replace intake
62566	600 (800)	14 (14)	15.7 RB	Pump in drum in river	75	3.7	4.13	5.02	4.91	4.51	4.52	4.38	No problems	None
20967	749	18.3	15.7 RB	Screen on end of steel pipe	90	3.0	4.26	5.02	4.91	4.51	4.52	4.38	No problems	None
Te Teko Bridge			12.5											
64967	650	36.1	12.1 LB	Rotating self-cleaning screen at end of steel pipe	220	4.5	5.61	6.44	6.32	6.09	5.95	5.81	Vortex at intake, air in pump.	Extend the intake pipe to lower level
	(1,800)	(100)		Rotating self-cleaning screen at end of steel pipe (Frost protection)	220	4.3	5.49						Not known	Extend the intake pipe to lower level

Notes:

1 Assumed criteria for pipe intakes (not including pontoons with screens, pump and/or footvalve below):
Minimum submergence = 0.1 m or one pipe diameter, whichever is greater.
Minimum clearance under pipe = 0.3 m

2 Federated Farmers advised at a meeting on 28 January 2009 that there are a number of permitted take abstractors downstream of Matahina, and agreed to provide contact details.
Malcolm Campbell is the only permitted take abstractor to have made contact following the meeting. His farm is between Edgecumbe and Te Teko.
Other permitted takes are also likely to need mitigation for lower river levels. These takes are limited to 15 m^3/day, therefore provision of, say, 20 m^3 storage tanks would allow water to be stored over periods when the river is too low to abstract water. Within the tidal zone, floating intakes are likely to be required in order to allow abstraction from the surface layer of freshwater.

3 The river bed level at km 29.8 LB is too high to allow for satisfactory intake operation at low river levels (low flow with low tide). Considering potential salinity problems at low flow, options here are:
(a) Construct new floating intake(s) at another location (e.g. at km 29.5 LB, where the bed level is 2 metres deeper). Requires property easement.
(b) Provide a shared well for the two abstractors.

Appendix D

Hydraulic modelling downstream of Matahina dam

1 Introduction

This appendix describes the modelling carried out to assess the hydraulic effects of the proposed operating regime. The modelling comprises example 14-day hydrographs, steady state low flows, and salt water intrusion.

The simulations were carried out using EBoP's MIKE-11 model, which extends from Te Teko gauging station to the sea at Thornton. The model was extended upstream to the Matahina dam, using additional cross-section data supplied by EBoP.

EBoP does not guarantee this MIKE-11 model for low flows. This caution is understandable, as gaugings at the Te Teko site show (for a chosen flow rate) water level variations over a year or two of about 0.2 m. These variations may be attributable to temporal variations in bed roughness, but are more likely to be due to cyclical deposition and erosion of bed sediment.

However, the model represents the best information available, particularly as there are no other gauged sites on the Rangitaiki River downstream of Matahina dam. The computed water levels should be regarded as somewhat approximate, but computed water level variations between scenarios are expected to be fairly robust.

River distances quoted in this appendix are the EBoP model distances plus 12.0 km. Those quoted elsewhere in this report are as quoted in the Twin Peak Trial Period Monitoring, resulting in minor differences due to the way that distances have been measured. The maximum difference is at the river mouth: 37.0 km from the dam using the EBoP model, and 36.6 km elsewhere in this report.

For consistency, all computations were carried out assuming a constant tidal range of 1.6 m, representing a spring tide. Mean sea level was kept at R.L. 0.0 m, i.e. neither "storm surge" nor permanent sea level rise due to climate change were modelled.

2 14-day Hydrographs

TrustPower provided historical Matahina 14-day output hydrographs for average flows of 50, 39 and 29 m^3/s, together with the corresponding hydrographs using the proposed constraint envelope. These hydrographs were input to the hydraulic model.

The historical hydrograph for 29 m^3/s average flow is for the period 3-15 March 2008, when Matahina was operating under the “rough running regime” agreed between TrustPower and EBoP. The 29 m^3/s hydrographs are based on flow ratings and conversions from MW to m^3/s available at the time. Subsequent auditing of the data suggests that actual low flows are slightly greater than indicated.

The constant tide is strictly speaking not realistic for the 14-day hydrographs; a full cycle of spring and neap tides would occur over that time. However, the constant spring tide ensures that spring tide and any worst part of the hydrograph are modelled in combination.

For the 29 m^3/s historical scenario, a second run (not presented) was carried out with the phase of the tide changed by 180 degrees (i.e. 6.2 hours). Although the details of computed water levels were different, both minimum levels for the 14 days and rates of drawdown were comparable. This gives some confidence that the full set of numerical experiments does not need to be repeated with various choice of the phase of the tide.

The model results for the three pairs of 14-day hydrographs are presented in two types of graph.

First, computed water level time series for the historical and proposed operating regimes are compared for the three different representative flow rates at four sites: Matahina Dam, the Te Teko flow gauge (12.0 km from the dam), Edgecumbe (25.2 km) and Thornton (34.3 km, 600 m upstream of the bridge). Comparable data are available for all model cross-sections between Matahina Dam and the sea, the cross-sections typically spaced 500 m apart.

These graphs (Figures D1 to D12) show that the wider flow fluctuations of the “proposed” scenarios are reflected in wider water level ranges as far downstream as Edgecumbe, along with drawdown rates that are sometimes more rapid than for the “historical” scenarios.

At Edgecumbe, though, tidal water level fluctuations are just as pronounced as those due to the flow regime, and at Thornton the wide tidal range all but obliterates the effect of flow variations.

Second, flow rate time series are presented for each scenario, with the four sites on the same graph (Figures D13 to D18). Changes in flow rate are progressively more gradual downstream from the dam, and by Edgecumbe the range of flows is reduced. Tidal fluctuations in flow rate at Edgecumbe are not that obvious, but at Thornton they dominate the record.

Table 1 shows the maximum hourly recession rates for the modelled historical and proposed regime 14-day hydrographs at four locations. The maximum rates occur at the Matahina dam, and then reduce downstream to Edgecumbe as a result of attenuation in the river. At Thornton the recession rate is dominated by tidal effects. The most significant change is that the 39 m^3/s historical hydrograph was relatively constant and peaking under the proposed regime results in increased recession rates from Matahina to Edgecumbe. The modelled rates are not directly comparable with those shown in Figures 8 and 24 of the main report, which are for higher average flows with particular inflow hydrographs.

Table 1

Modelled maximum hourly recession rates (m per hour)

Location	"50 m^3/s" hydrograph		"39 m^3/s" hydrograph		"29 m^3/s" hydrograph	
	Historical	Proposed	Historical	Proposed	Historical	Proposed
Matahina	0.43	0.30	0.19	0.43	0.39	0.41
Te Teko	0.15	0.19	0.04	0.20	0.13	0.16
Edgecumbe	0.11	0.13	0.07	0.14	0.12	0.12
Thornton	0.27	0.28	0.28	0.29	0.29	0.29

3 Steady Low Flows

Model runs for a range of steady flows were examined to get the water levels at existing intakes in Appendix C. The modelled steady flows are:

45 m^3/s	22 MW	The normal lower generating limit (mid-reservoir level)
40 m^3/s		Existing consent inflow limit below which no peaking is permitted
30 m^3/s		The 7-day low flow experienced in 2008 (the lowest on record)
25 m^3/s	10 MW	The lowest flow on record at Te Teko (2008, under the rough running regime)
20 m^3/s		The minimum flow under the proposed constraint envelope

The water level profiles are shown on Figure D19, and levels relative to the water level at 45 m^3/s are shown on Figure D20. For lower-river locations near Edgecumbe and Thornton, these water levels are low tide minima.

The uncertainties in the water level calculations are particularly relevant here, because the water levels are likely to be compared with water abstraction pipes at fixed locations. Nevertheless, the variation in water level with varying flow rate is expected to be modelled quite closely.

Figures D19 and D20 show that at low flows the water level is most affected immediately downstream of a constriction at km 4.5 and sills at km 12.0 (Te Teko gauge) and km 19.6 (the Edgecumbe earthquake fault). Immediately downstream of these sites the 20 m^3/s profile is approximately 0.9 m below the 45 m^3/s profile.

Figure D21 plots water depth along the river for different flows. The figure shows that a flow reduction from 45 to 20 m^3/s reduces the minimum depth at the thalweg (the lowest point on the river cross-section) from 1.5 to 0.9 m at Te Teko and from 1.0 to 0.7 m at the earthquake fault. Elsewhere the depths are greater (up to 4.8 m).

The tidal influence on water levels extends 17 km from the river mouth, up to the downstream end of the rapids at the Edgecumbe earthquake fault (km 20.0). Figure D22 shows the effect of different flows on water levels in this reach at high and low tide.

4 Salt Water Intrusion

Federated Farmers has suggested that low river flows have resulted in the intrusion of sea water up the river, affecting water abstraction at Thornton (i.e. 2-3 km from the sea).

In the absence of detailed salinity measurements, the exact shape of any saltwater wedge or layer cannot be determined. However, some generalisations can be made about salinity patterns:

4.1 Analysis

To examine saline intrusion, we first considered the tidal volume: for steady low flows, a standard approach is to express the computed tidal prism (where this is defined as the net inflow of seawater through the mouth on the flood tide), as a fraction of the river flow during the entire tidal cycle. The tidal volumes defined this way were computed from the MIKE-11 runs referred to in Section 2 above, by integrating discharge values.

Three river flow rates were analysed this way, with the mean spring tide range of 1.6 m assumed in all cases.

Table 1 - Ratio of tidal prism to river flow

River flow (m^3/s)		River flow volume over one tidal cycle (m^3)	Tidal prism (m^3)	Ratio
45	The normal lower generating limit (mid-reservoir level)	2.00×10^6	0.22×10^6	8.9
30	The 7-day low flow experienced in 2008 (the lowest on record)	1.33×10^6	0.52×10^6	2.6
20	The minimum flow under the proposed constraint envelope	0.89×10^6	0.78×10^6	1.1

Hansen & Rattray (1966) wrote the seminal early study of saline mixing in estuaries, and quoted a yet earlier study (Schultz & Simmons 1957) who observed, as a general rule, that a ratio of 1.0 or more indicated a highly stratified estuary with a well-defined salt wedge, whereas 0.25 indicated a partially mixed estuary and 0.1 or less indicated a well-mixed estuary with little discernable vertical salinity gradient.

Simmons (1966) provides a description of flow patterns in these three estuary types. Important features of a saltwater wedge are that (in the absence of any tide) near-bed saline water flows upriver, with a return flow of saline water in the upper half of the wedge. The river water flows seaward on top of the wedge.

A saltwater wedge is therefore expected to exist at river flows of 45 and 30 m^3/s. At 20 m^3/s, a saline wedge is also likely, but it would be safer to specify “highly stratified” rather than presume an actual salt wedge.

Keulegan (1966) sets out the form that the “arrested”, i.e. steady state, saltwater wedge takes, producing semi-empirical formulae for its length, the height at the mouth, and the profile. These formulae have now been applied to velocity and depth data from the MIKE-11 model of the

Rangitaiki. For each of the three flows, these calculations have been carried out for three states of the tide, and are presented in Table 2.

The calculations have assumed a uniform rectangular channel 130 m wide, with bed at RL -2.00 m. This cross-section is a good approximation of those within 2 km of the sea, but there are cross-section variations, particularly further upstream. For that reason, and given that the calculations are sensitive to the chosen cross-section, the calculated values in Table 2 should be regarded as approximate.

Table 2 - Calculated dimensions of saltwater wedge

River flow (m^3/s)	Tide level (m RL)	Wedge length (km)	Wedge height at mouth (m)	Freshwater layer at mouth (m)	Wedge height 2.7 km from mouth (m)	Freshwater layer 2.7 km from (m)
45	0.8	13.5	2.0	0.8	1.4	1.4
45	0.0	2.5	1.2	0.8	--	2.0
45	-0.8	0.18	0.4	0.8	--	1.2
30	0.8	Large	2.2	0.6	1.9	0.9
30	0.0	6.6	1.4	0.6	0.7	1.3
30	-0.8	0.48	0.6	0.6	--	1.2
20	0.8	Large	2.3	0.5	2.2	0.6
20	0.0	18	1.5	0.5	1.2	0.9
20	-0.8	1.3	0.7	0.5	--	1.2

(Those calculated wedge lengths designated “large” easily exceed the tidally influenced zone of the river, which is 17 km long.)

Theoretical considerations lead to the conclusion that the ebb and flow of the tide can, up to a point, be superimposed on an analysis of the fresh /saline interface. Ippen (1966) states that “Weak tidal action will … only result in a translatory motion of the salinity wedge to and fro”. In the present case, however, tidal action is not weak, and some change to the shape of the wedge through the tidal cycle can also be expected.

Table 2 gives the approximate saline wedge dimensions were sea level somehow held constant at spring high tide level, mean sea level, or spring low tide level. In practice, of course, the varying tide prohibits this, and it is unlikely that the “high tide” and “low tide” calculated wedge dimensions will be reached. So the various calculated wedge dimensions just provide an indication of the salt wedge and how it might vary with the tide. The very wide variation between the calculated wedge lengths for the three water levels indicates that, with tides, and with spring tides in particular, the saltwater wedge will be in a constant state of flux. However, the top layer of freshwater flowing to sea is always present with a saltwater wedge, and is somewhat less than a metre in depth in this example.

The last two columns in Table 2 show the calculated salt wedge height and freshwater depth 2.7 km from the sea (0.6 km upstream of the Thornton Bridge). These data show that in general the wedge here is significantly shallower than at the mouth, and further upstream its depth will be smaller still.

4.2 Implications for abstractors

Under the present operating regime, with a normal minimum flow of 45 m^3/s, the above analysis predicts a saline wedge extending a few kilometres upstream from the sea. The wedge is predicted to underlie a surface layer of freshwater flowing downstream, which will be less than a metre deep at the mouth, but deeper upstream near Thornton.

With the proposed lower flow of 20 m^3/s, the saltwater wedge is predicted to be deeper. The wedge is also predicted to extend further inland, and may extend to the end of the tidally influenced zone of the river, 17 km from the mouth. Any wedge might be more in the nature of a saline near-bed layer.

In principle, the risk of accidental abstraction of saline water will be increased. In practice, any increased risk may be small or negligible, particularly well upstream of Thornton. This is because the depth of the wedge may small enough there for salt water to be confined to the thalweg. In particular, those intakes that are constructed to float close to the water surface are likely to avoid salt water.

Four of the seven existing consented takes in the tidally affected reach use floating intakes (refer Appendix C), so are able to abstract water from the freshwater layer over the saline wedge. The three intakes that use fixed or adjustable level intakes will be affected both by the lower water levels and by the reduced depth of freshwater at the proposed lower flow: it is expected that these intakes will need to be replaced by floating intakes or wells.

4.3 Conclusions and recommendations

From this desktop analysis, it appears that some form of saline wedge will exist in the lower Rangitaiki River, or at least highly stratified conditions, regardless of the river flow provided this is at least 20 m^3/s. With a change down to the lower flow of 20 m^3/s, the wedge would extend further upstream, and the freshwater layer above the wedge would be shallower than at present. This could inconvenience abstractors of river water.

The above analysis takes a fairly simplified view of the physics, consistent with a first look at the issue at this river. Standard texts indicate that the problem cannot yet be reliably solved without site measurements. A more precise treatment would require measurements of salinity as it varies with distance upstream, with the phase of the tide, and with depth of sampling. Analyses of estuary hydraulics have always relied heavily on salinity measurements in particular. This is because it is very difficult to predict from first principles the amount of turbulent mixing at the salt/freshwater interface, and it is simpler to assess the mixing by measuring its most relevant and most obvious consequence.

In particular, we expect from the ratio of river flow to tidal prism that, at 20 m^3/s river flow, highly stratified conditions will remain. If this flow rate does in fact occur in the future, this assumption should ideally be checked against site observations.

We therefore recommend that, as part of ongoing monitoring, salinity profiles be measured during known, reasonably steady flow conditions, at a few selected locations. These would immediately allow a better assessment of any adverse effects of varying the flows, and would also provide valuable calibration data for any subsequent numerical modelling that might be considered necessary.

Should the monitoring indicate significant adverse effects at abstraction locations, simple remedies might prove cost-effective, such as installing floating intakes. If the problem were to prove more complex, a specialist numerical model would be extremely helpful in analysing the effect of different scenarios, if the cost could be justified. If the existing MIKE-11 model were to be used, MIKE-12 would be suitable; the salinity profiles mentioned above would still be needed.

4.4 References

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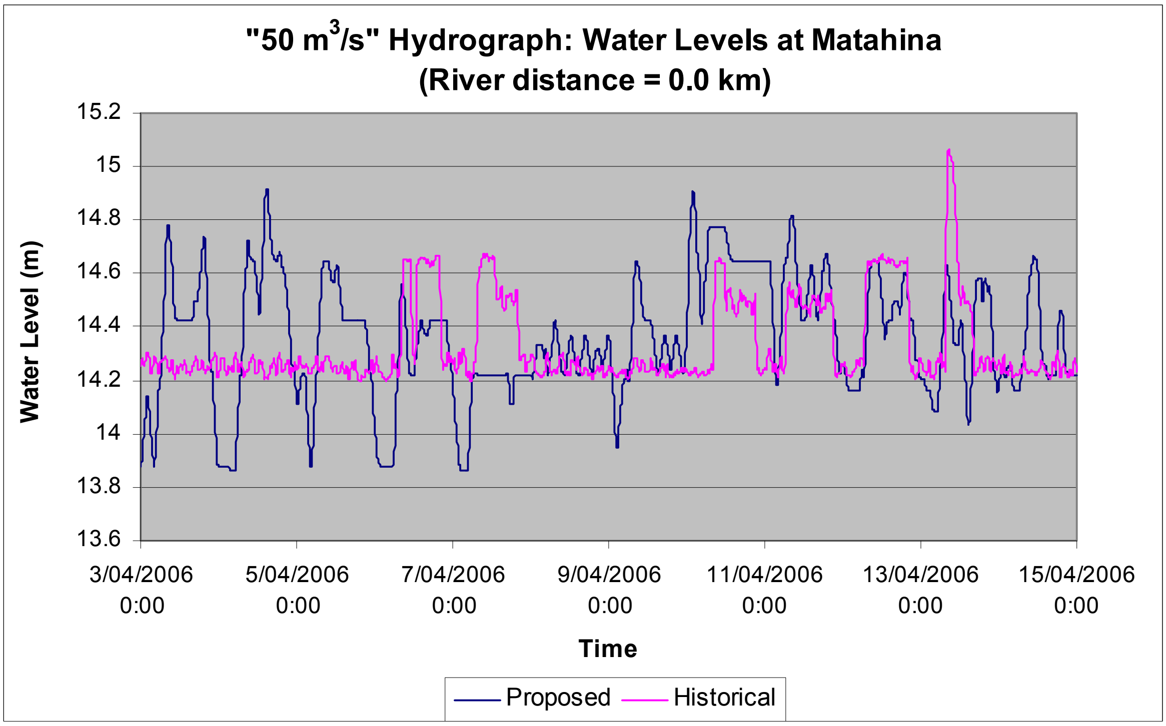

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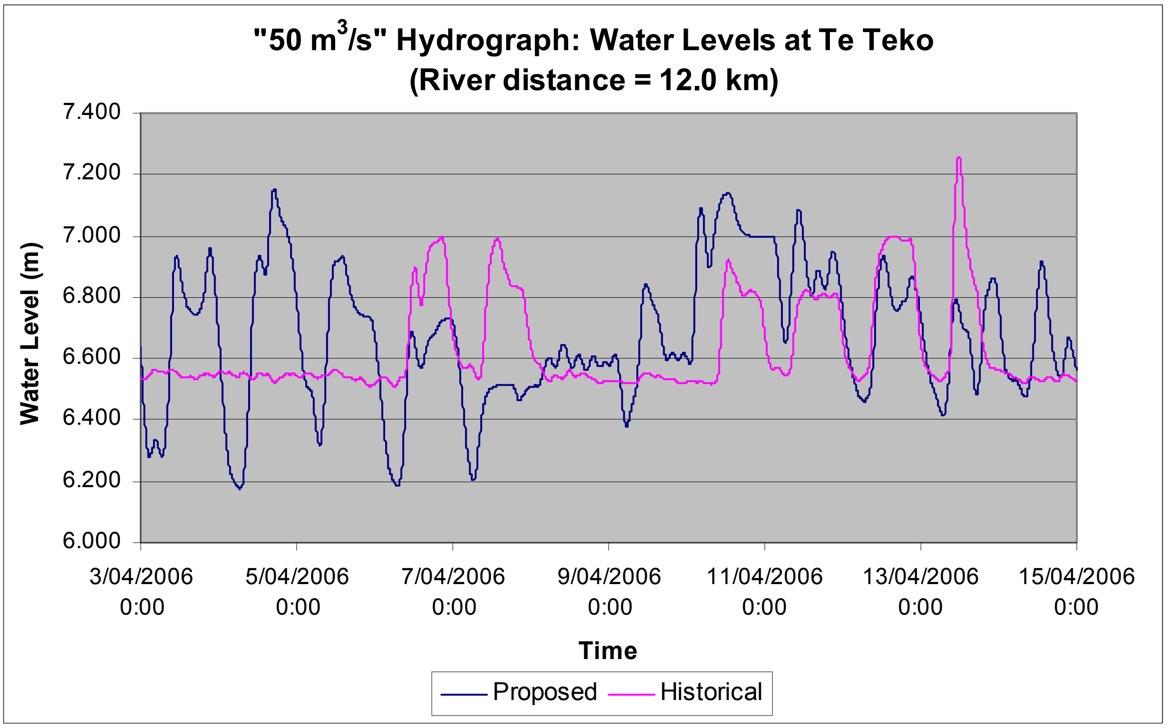

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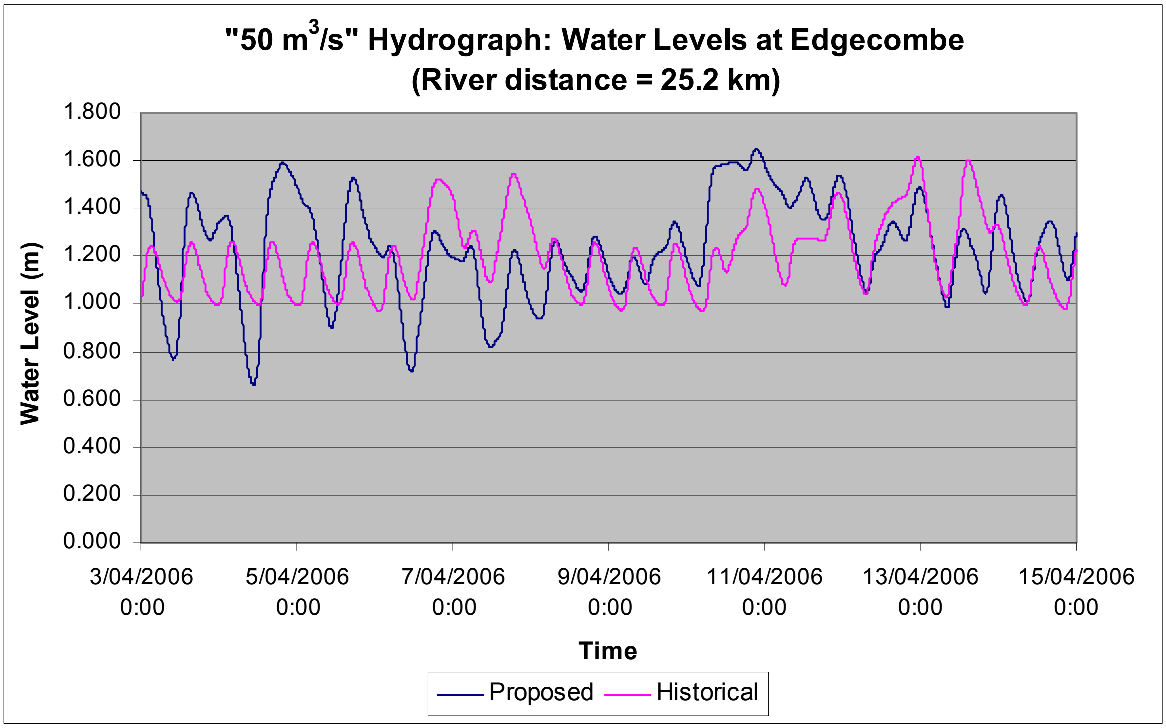

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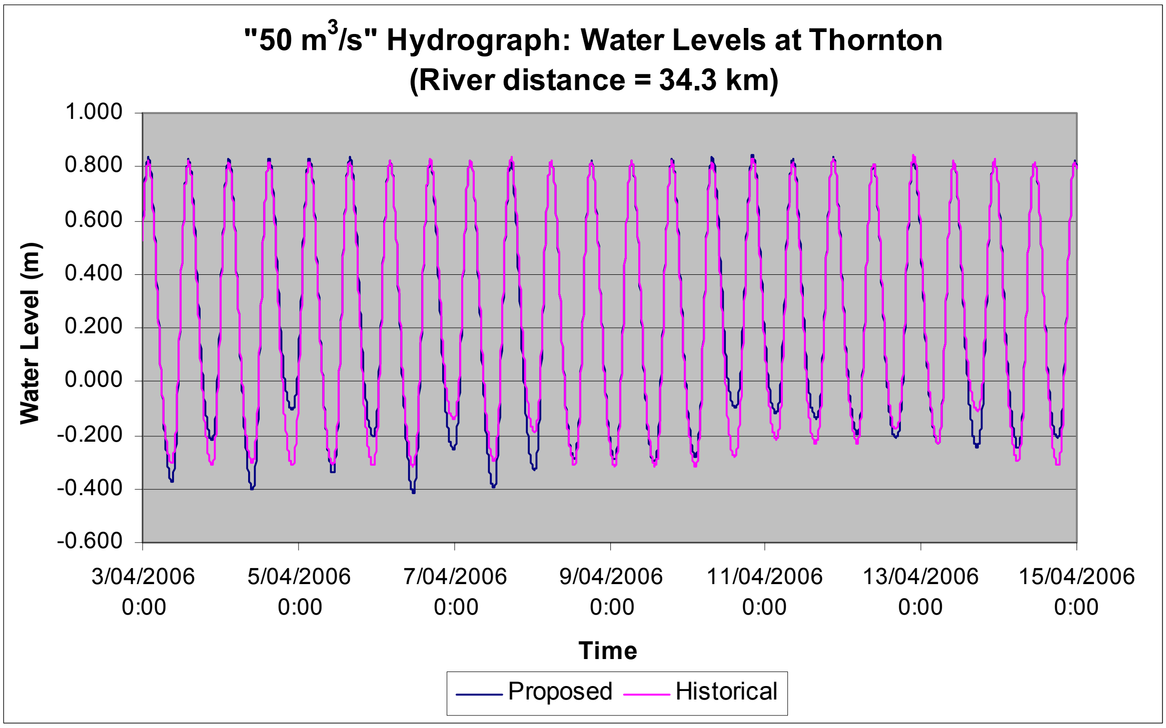

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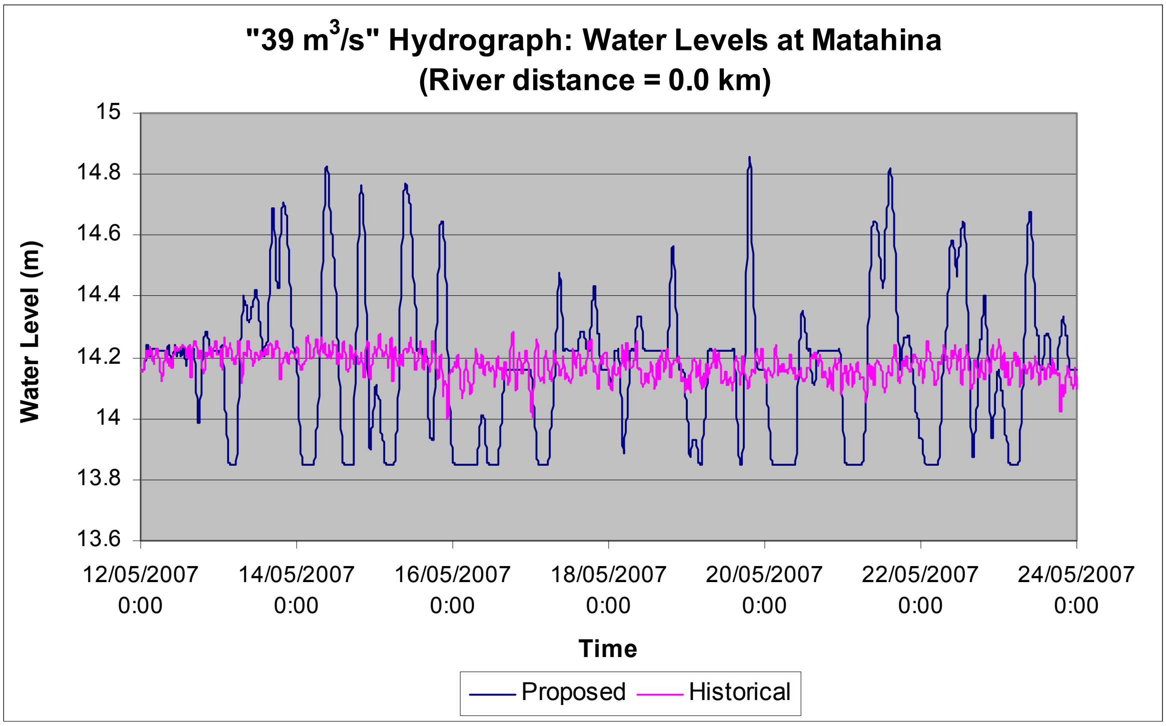

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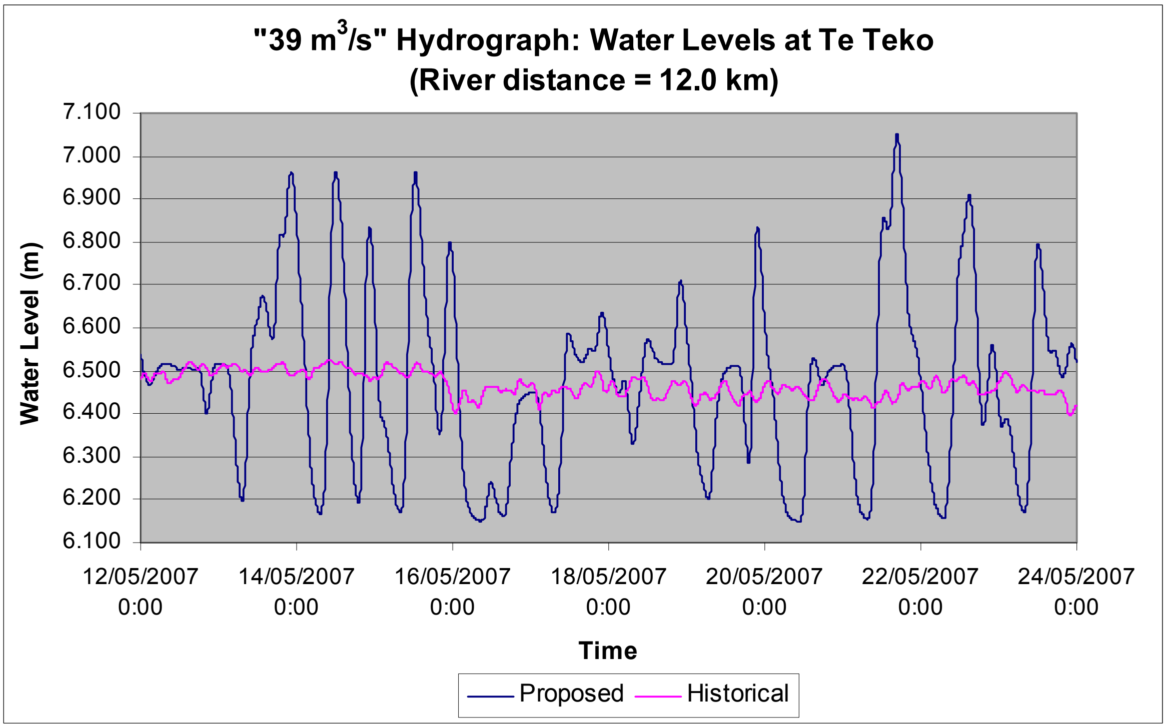

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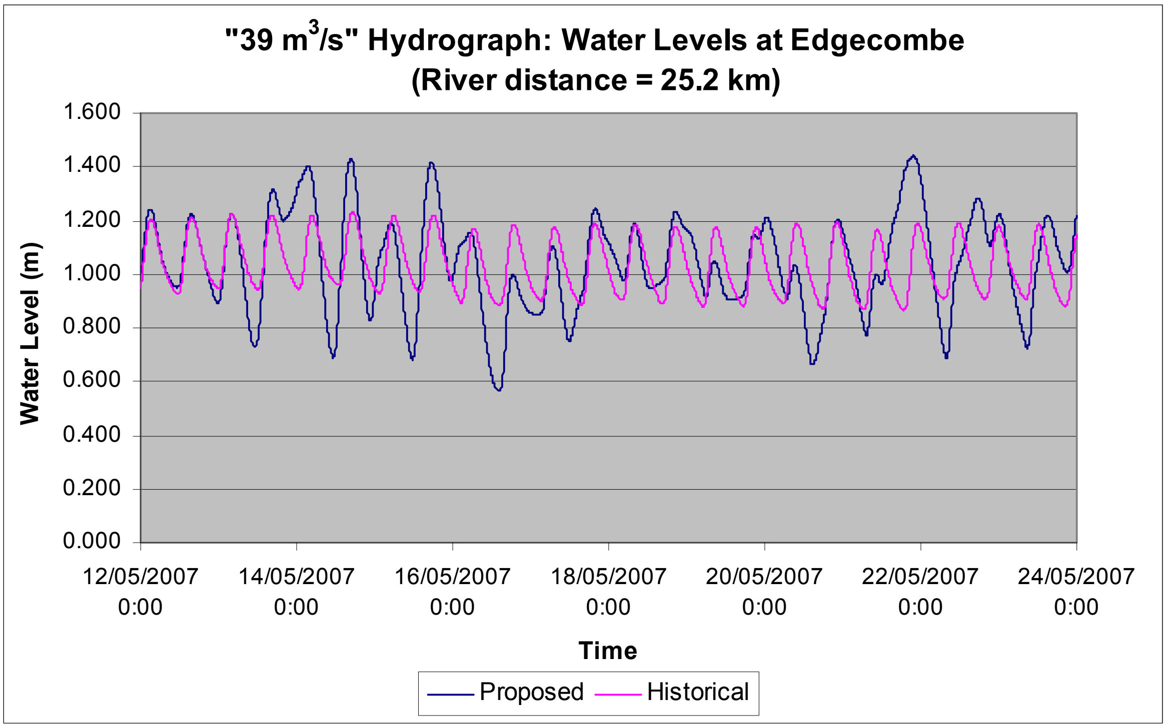

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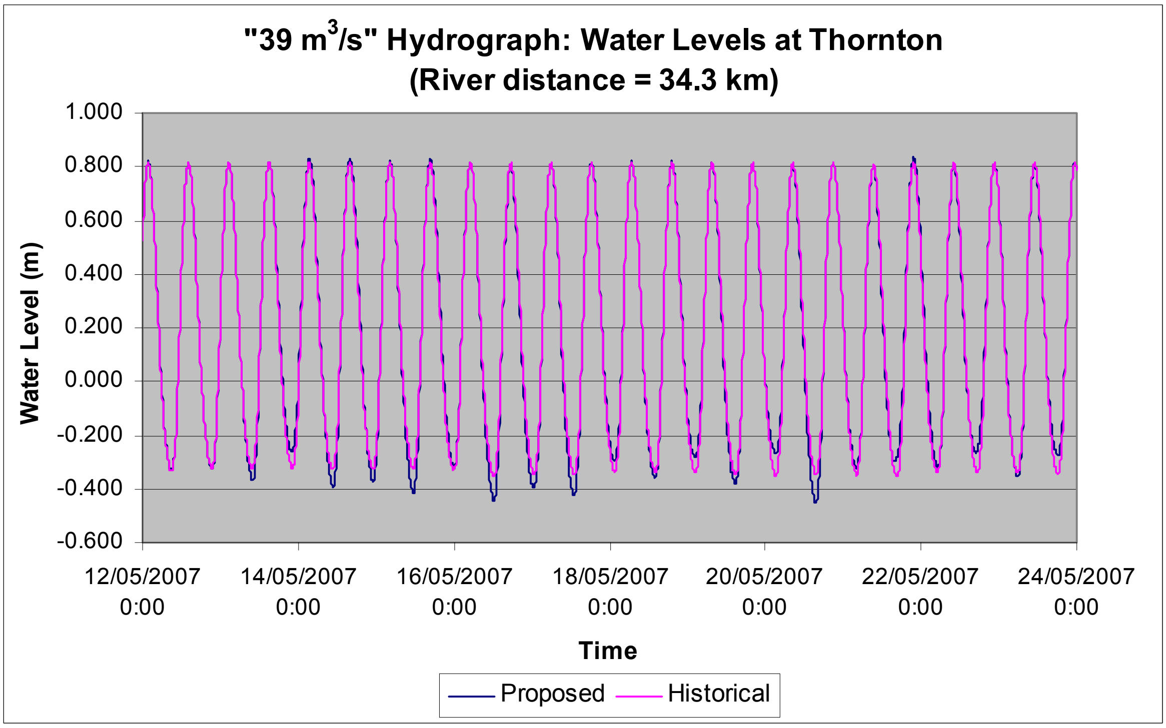

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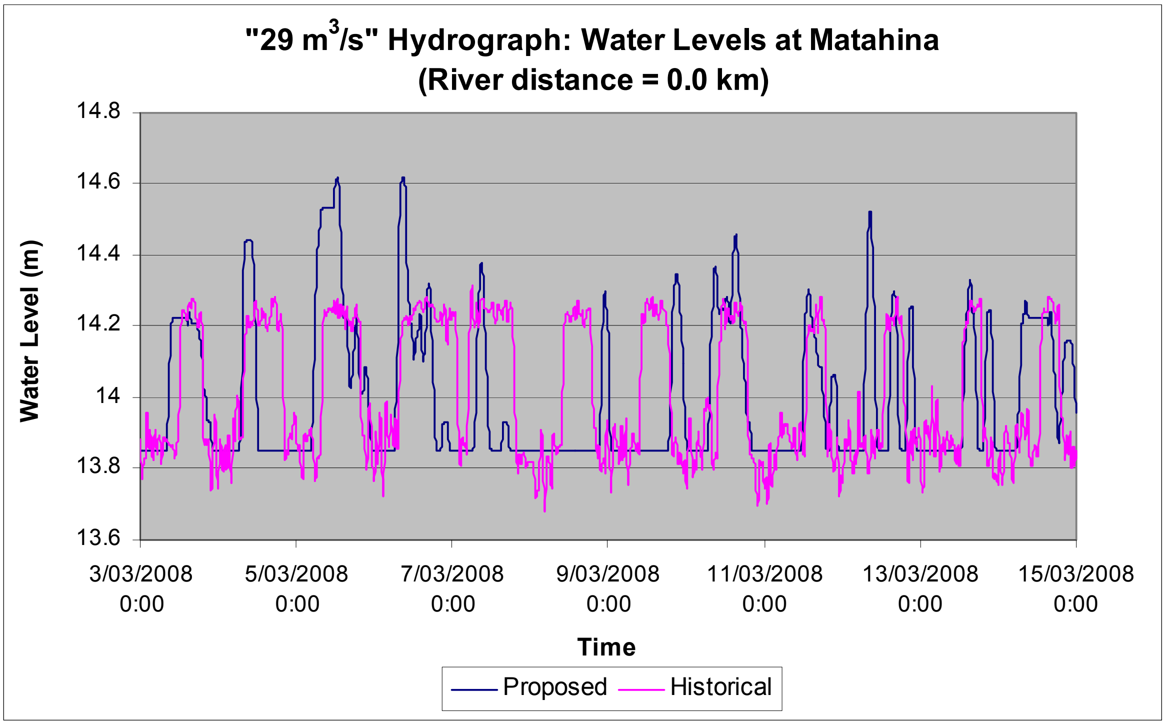

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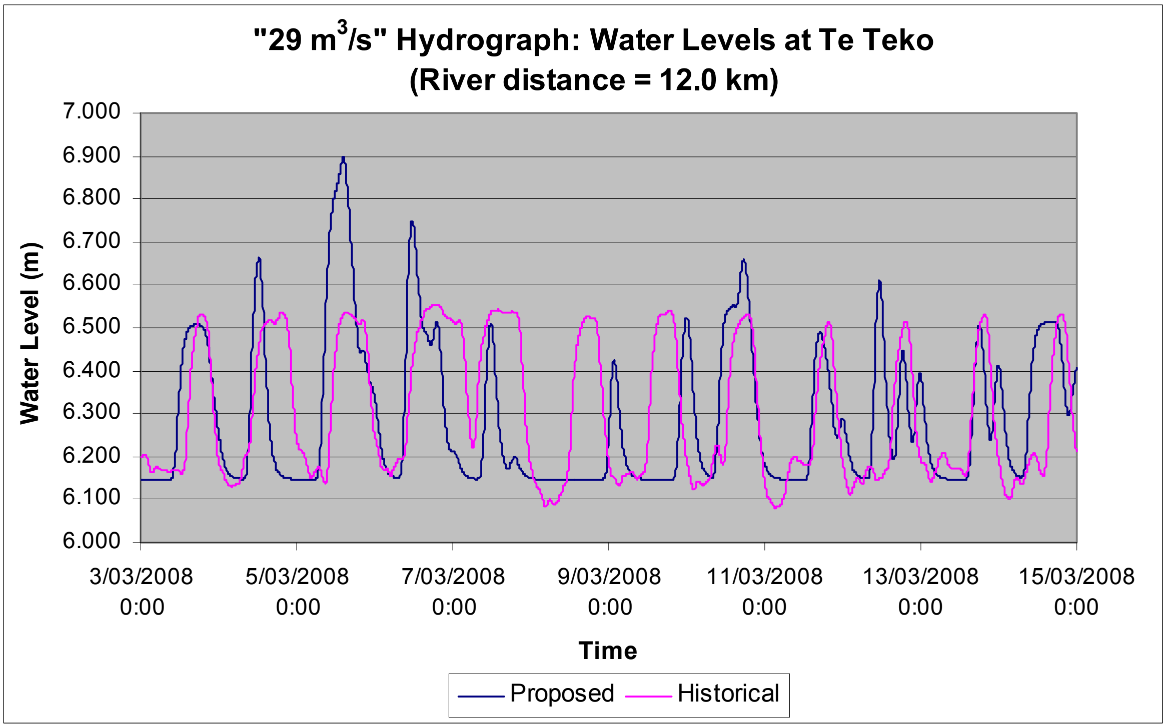

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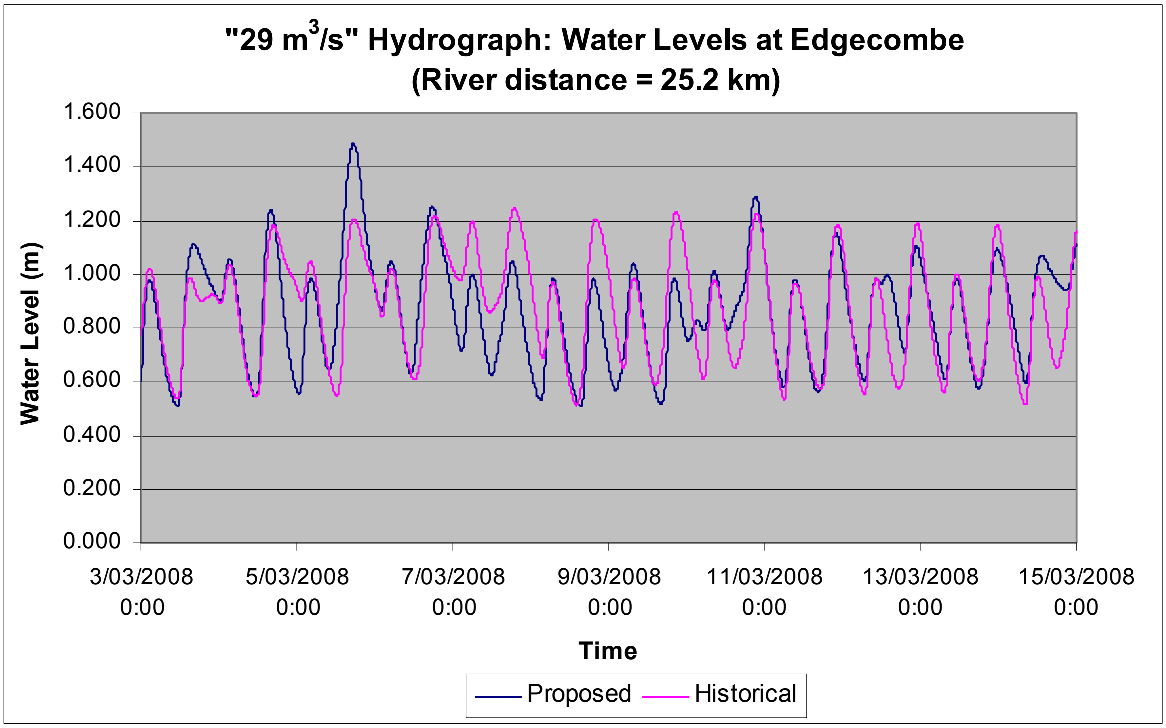

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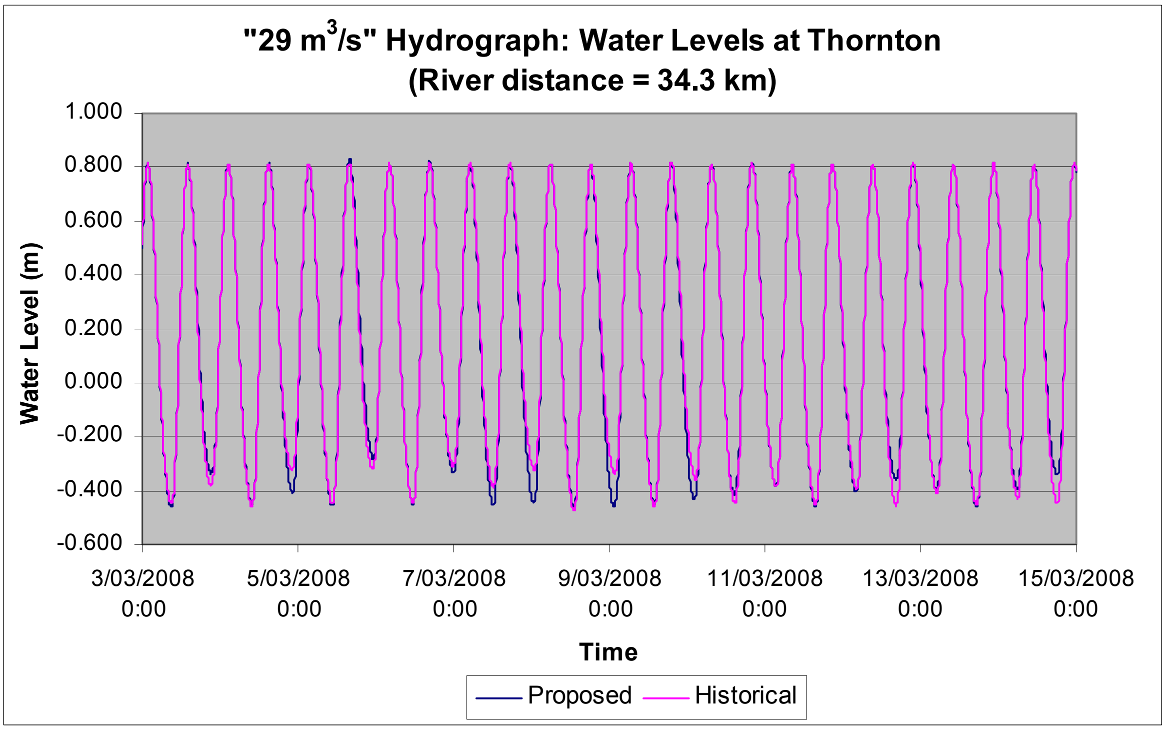

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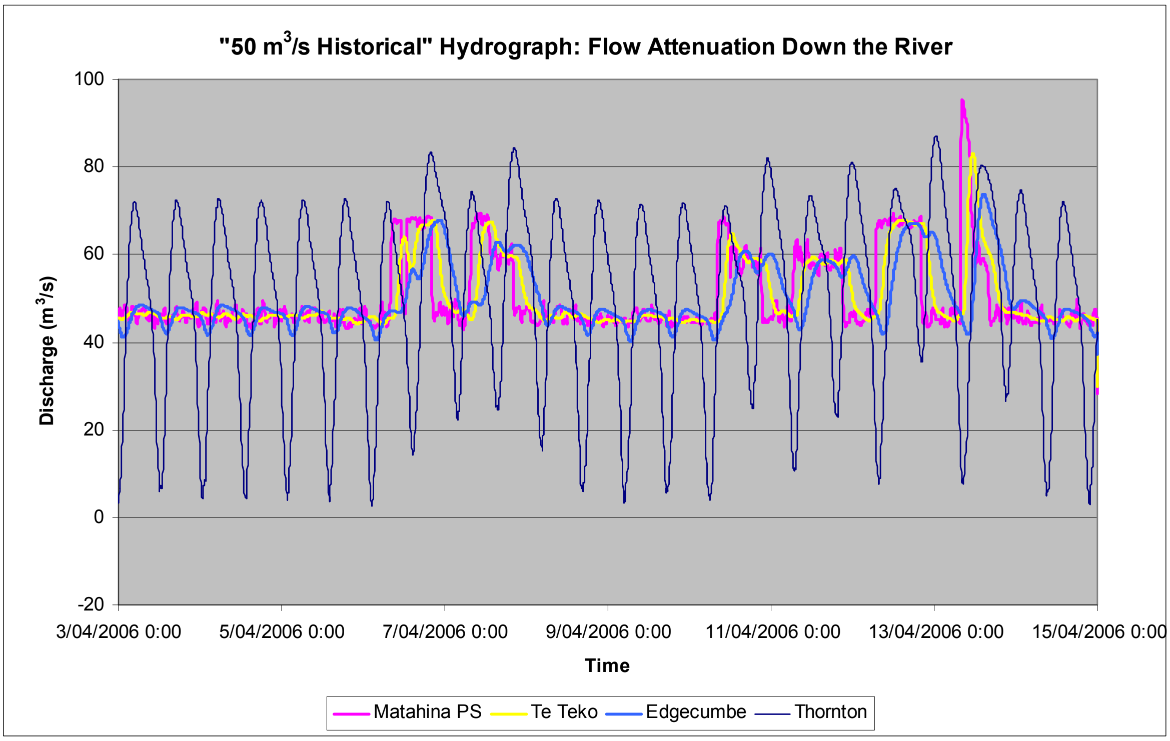

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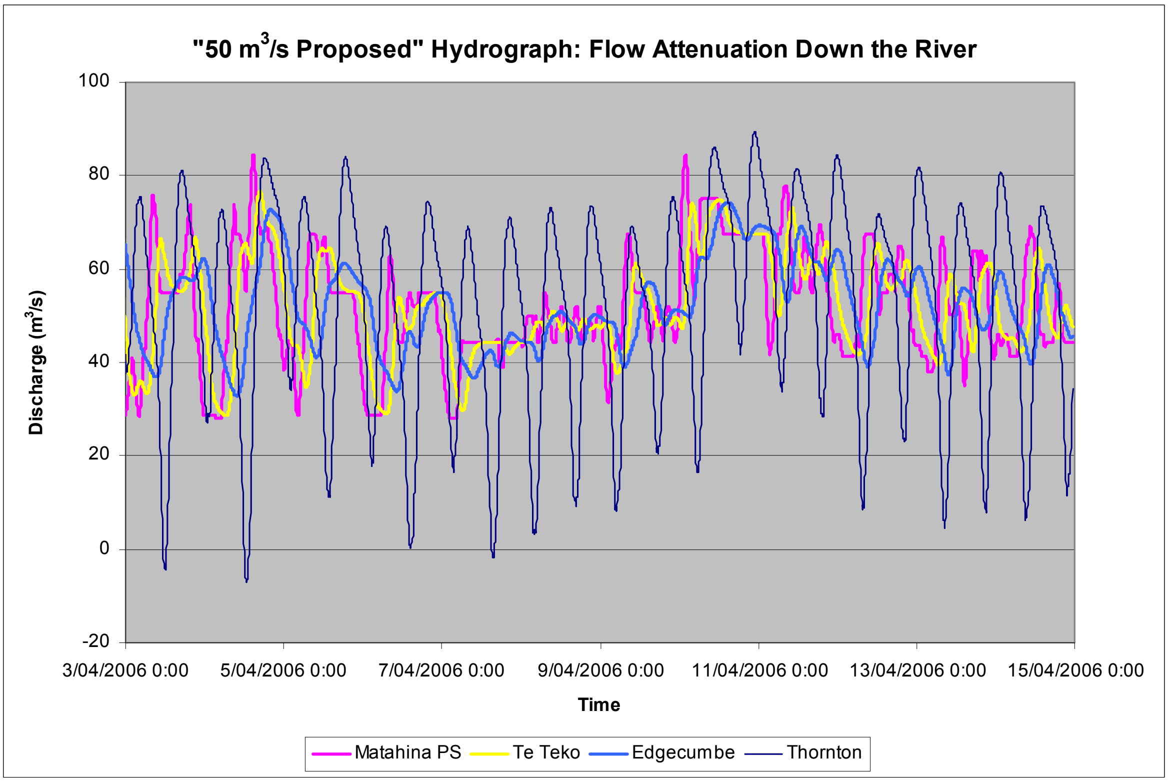

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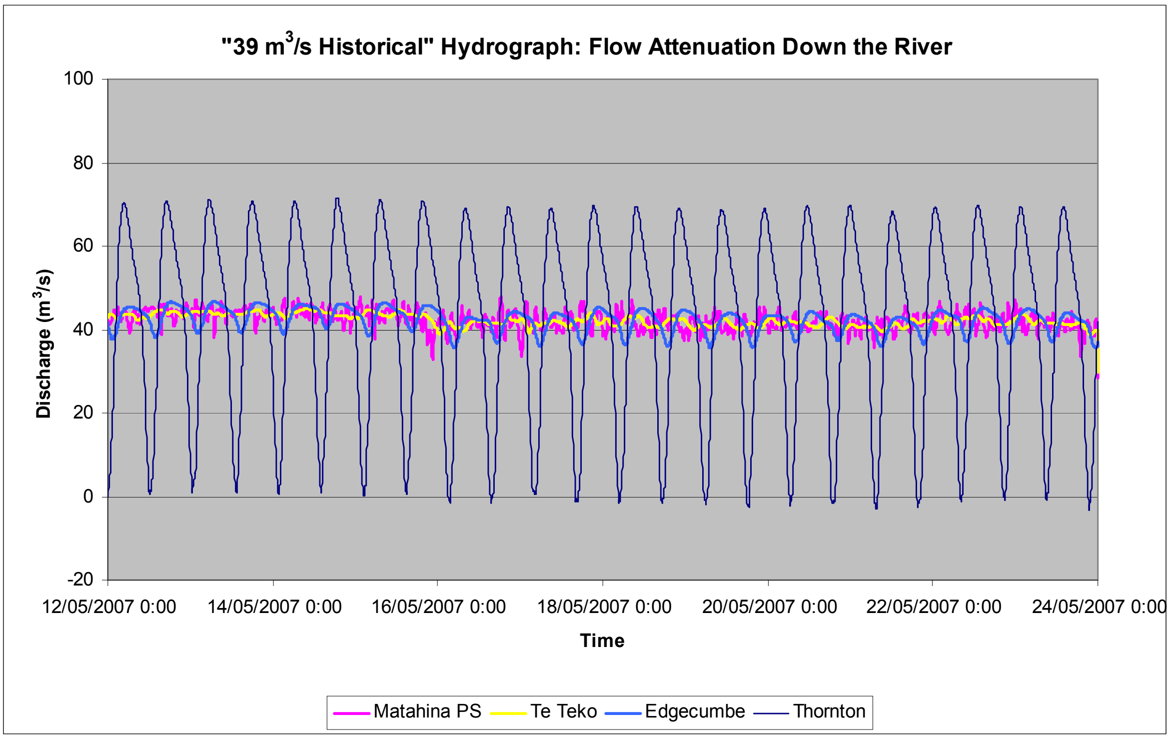

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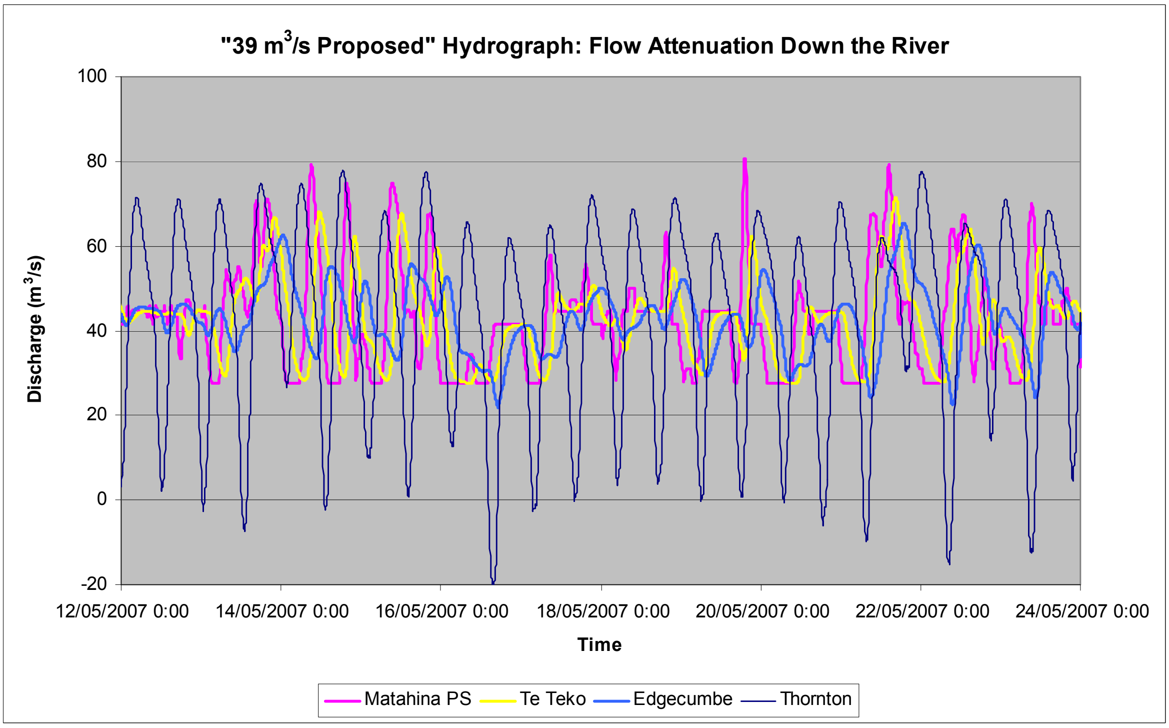

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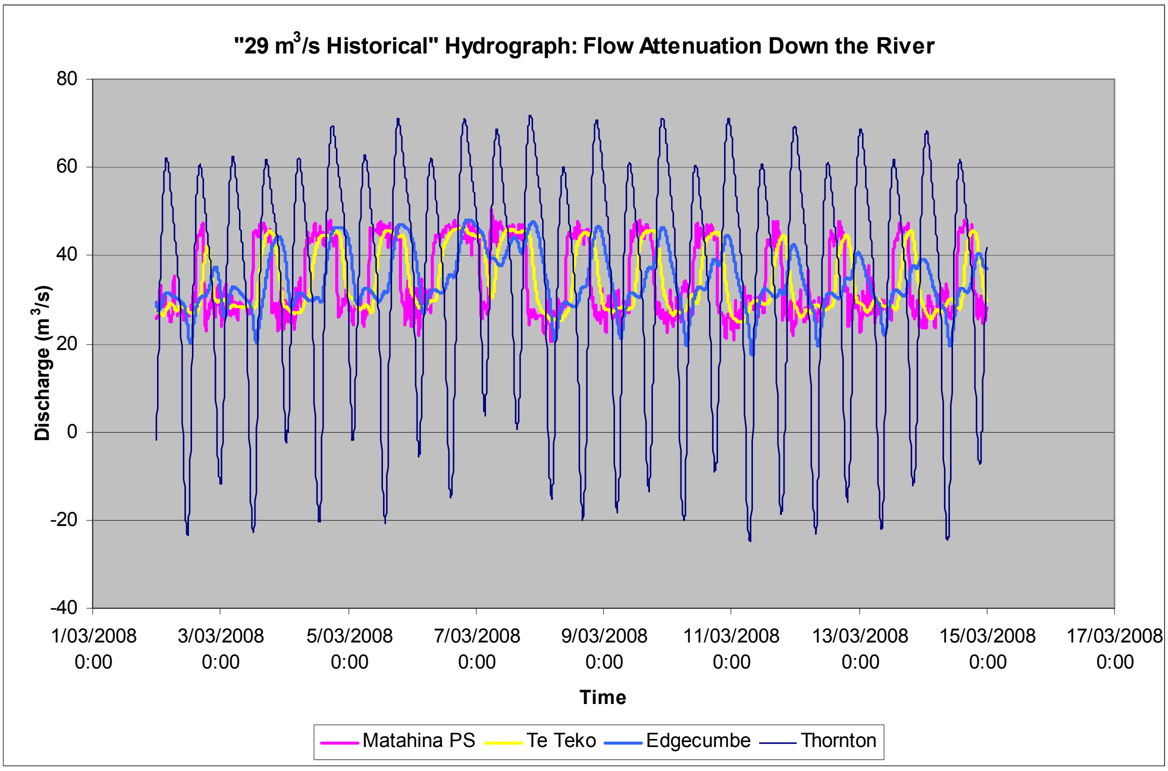

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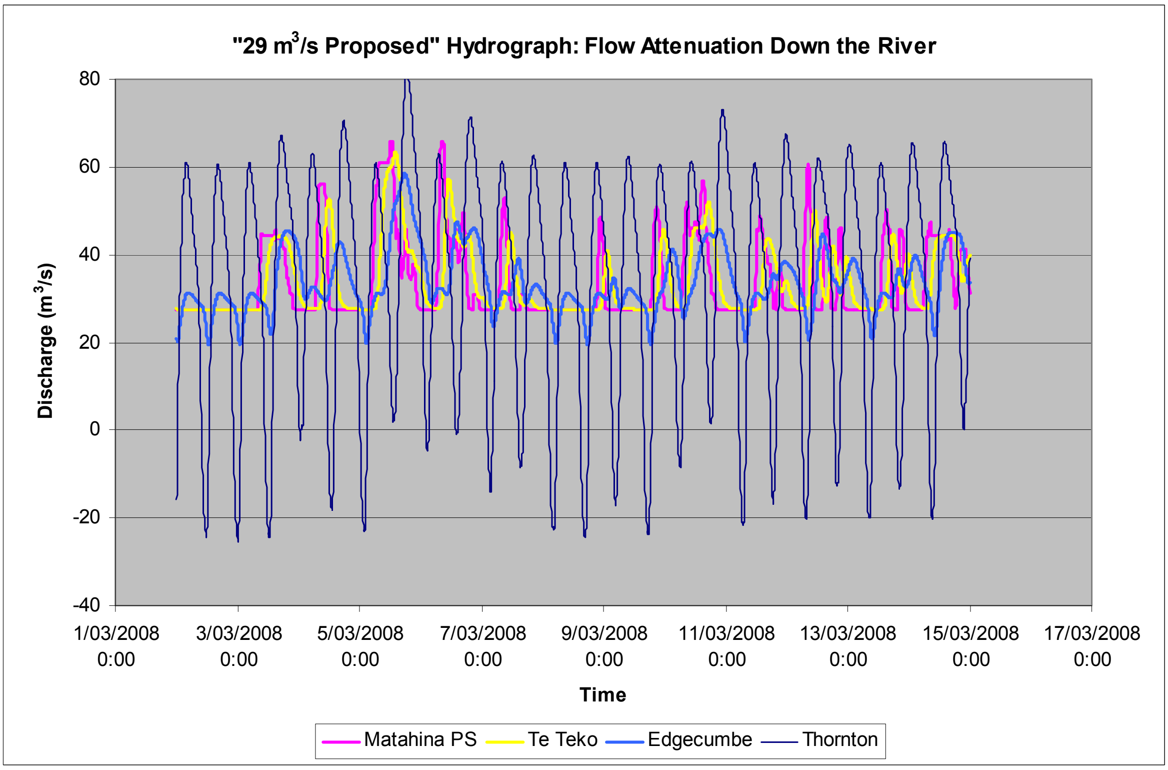

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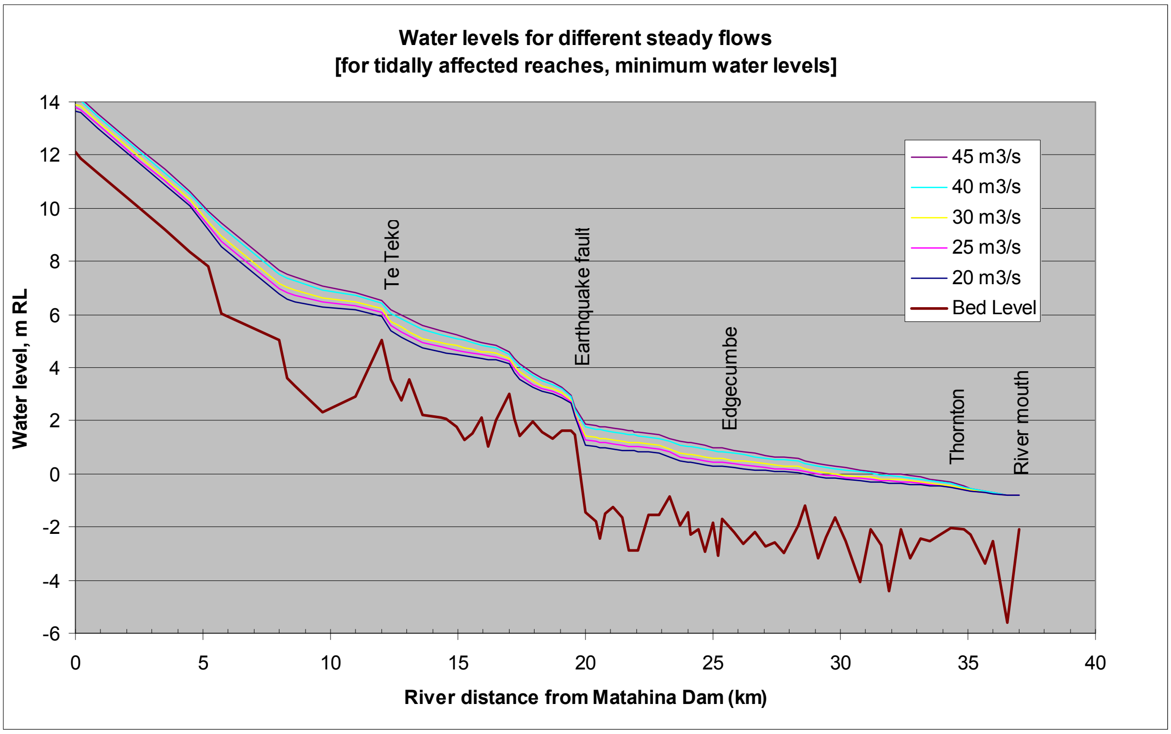

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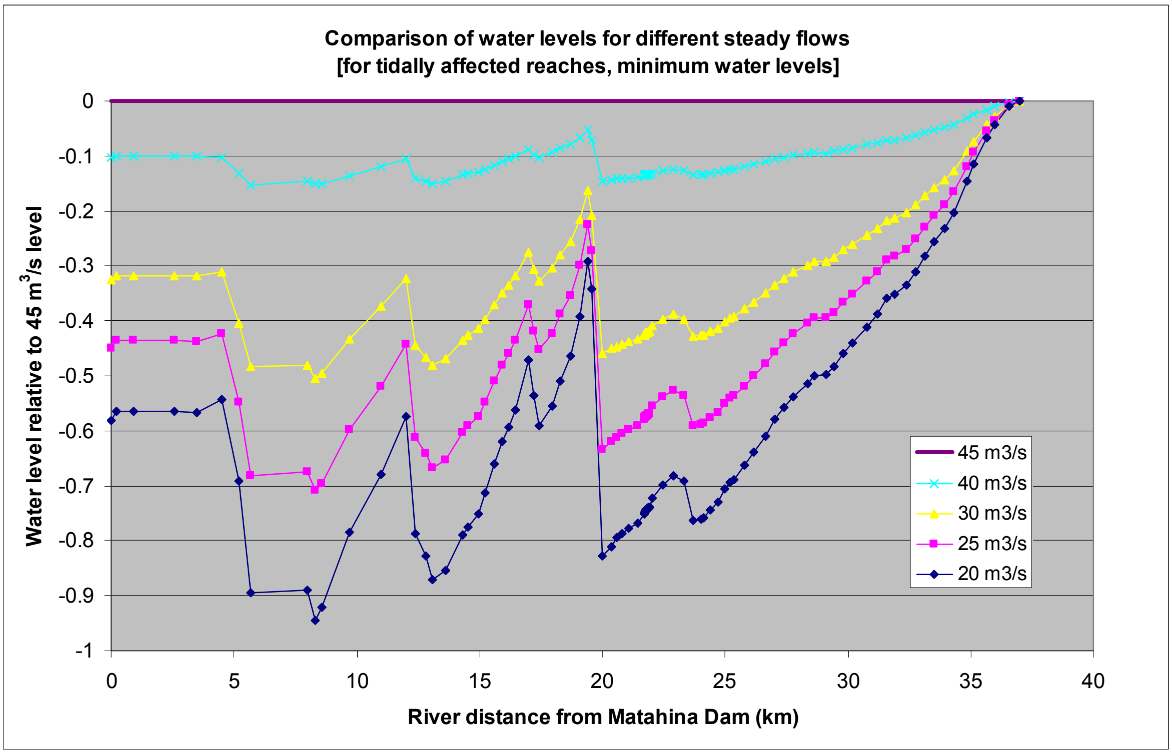

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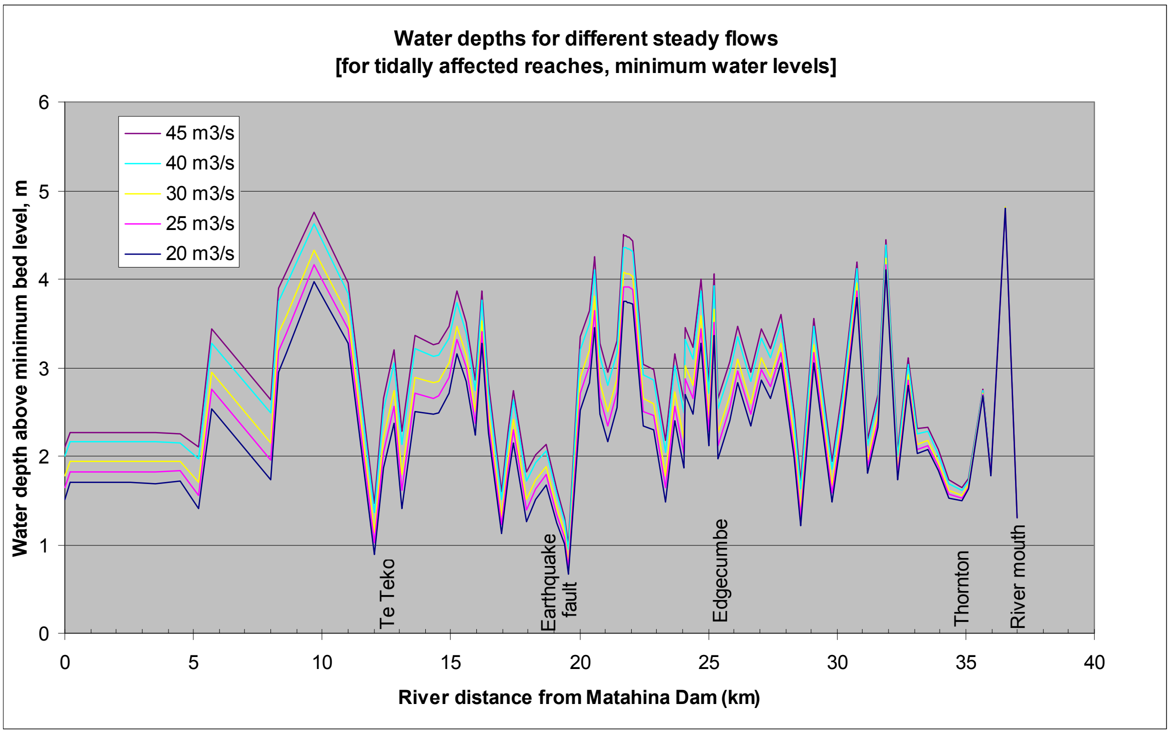

Figure D22