Lower Whanganui River
Flood Protection Investigations

Review of the June 2015 Flood and Update of Design Flood Level Estimates

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Front Cover Photograph
Severe flooding of Anzac Parade Houses, 20 June 2015 (7 ½ hours after flood peak)
Photo: Tane Humphrey

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FOREWORD

Introduction

Over the period 19 to 21 June 2015 the western area of the Manawatu-Wanganui Region experienced a very major rainfall event. This resulted in flood frequencies close to or exceeding 1% AEP (1 in 100 year) in several rivers. Very substantial flooding occurred through the City of Whanganui, due mainly to the flooding from the Whanganui River, particularly of the Anzac Parade-Kowhai Park locality, where water flooded numerous houses and reached depths of up to 2 m in some. Urban streams were also in very high flood, with for example both the Matarawa Diversion at No.3 Line and Awarua at Wikitoria Road experiencing floods of well over 1% AEP.

The Lower Whanganui River peaked at a stage of 21.975 m and flow of 4755 cumecs at the Te Rewa gauge at 0105 hours on 21 June 2015. This gauge is located some 50 km upstream of the river mouth. This flood flow equates to a 1.2% Annual Exceedance Probability (AEP) (1 in 85 year) flood at that site. The recorded flood flow of 4755 cumecs at Te Rewa is the highest flood flow recorded on the Whanganui River and furthermore, is understood to be the second highest flood flow ever recorded in the North Island – behind only the famous Mohaka Flood flow of 1938, estimated at 225,000 cusecs (6370 cumecs).

However, in the lower reaches this flood was characterised by well above normal tributary flows. These were due to high rainfalls on wet antecedent conditions, with the 48 hour rainfalls exceeding 1% AEP for almost the entire area downstream of Te Rewa.

Consequently flood levels were very high, reaching slightly above the 0.5% AEP (1 in 200 year) levels around Aramoho and upstream. There were several factors involved in reaching these high levels, though the additional tributary flow was the principal one.

Levels in the lower river were mitigated by scour occurring at the river mouth due to a favourable set of ambient conditions.

This report examines the size of the June 2015 flood and describes the consequent implications on flood frequency, flood levels and sedimentation through Whanganui City.

Revised Flood Frequency

The flood frequency analysis was updated to reflect the additional nine years of data since the previous formal assessment in 2007. Thus the analysis was based on a flood frequency analysis of the 59 annual maxima from 1957 to 2015 (inclusive), with a censored assessment including 12 historic peaks dating back to 1858. This analysis includes the top (2015) and 3rd highest (2013) floods recorded in the annual series, and indeed this has been a flood-rich period.

The estimate for the 1% AEP (100-year return period) flood of 4964 cumecs is marginally (1.7 percent) higher than the design estimates in the 2007 Horizons Regional Council report. Refer Table 2.4.

Table 2.4: Whanganui at Paetawa & Te Rewa Design Flood Frequency Estimates (Updated 2016)

RETURN PERIOD (YEARS)	PROBABILITY (%)	DISCHARGE (CUMECS)	Y VARIATE
1.5	67	1935	-0.0940
2	50	2232	0.3665
2.33	43	2369	0.5786
5	20	2963	1.4999
10	10	3448	2.2504
20	5	3912	2.9702
30	3.3	4179	3.3843
50	2	4513	3.9019
100	1	4964	4.6001
200	0.5	5413	5.2958
500	0.2	6005	6.2136

2015 Flood Size

Very clearly the peak flood flow through Whanganui City was significantly above the 4755 cumecs recorded at Te Rewa. With the 48 hour rainfalls exceeding 1% AEP in the reach below Te Rewa, on a very wet catchment, the tributary flows were significant. The final blow was a significant heavy burst of rain near the tail end of the storm.

The concluded flow that passed through the Town Bridge location is 5150 cumecs magnitude. We can expect a flood of this magnitude at an average annual probability of 0.77%, equivalent to a 1 in 130 year flood. (Note based on the flood frequency statistics available prior to this flood it would have been regarded as a flood of 0.67% AEP, equivalent to a 1 in 150 year flood).
Design Flood Levels
The MIKE11 model of the Whanganui River, from Paetawa to the Tasman Sea, built by Hydro Tasmania Consulting in 2007 as part of a wider MIKEFLOOD model, has been used by Horizons Regional Council (HRC) to inform flood predictions and flood defence design in the Lower Whanganui River.

The calibration process was conducted in several progressive stages as follows:

1. Application of the ‘original’ MIKE11 model with the cross sections surveyed in 1995.
2. Substitution of the model cross sections with those surveyed in December 2015.
3. Addition of the estimated tributary inflows in the June 2015 flood event.
4. Inclusion of the forecast mouth scour; and
5. Minor recalibration of the model to represent the observed flood levels.

The calibration results can be seen in Figure 3.5. Figure 3.6 shows how the modelled water levels correspond with the recorded flood levels with error bars of +/- 0.3 m. This is the magnitude of model imprecision commonly determined to assess a model's accuracy and is reflected as a component in the freeboard assigned of 0.5 m.

As the graphs show, the final calibrated model reproduced the flood levels recorded in the June 2015 flood to a good accuracy. Therefore, the model calibration is fine and the model formulation is an accurate basis for producing design flood estimates.

Design flood levels for the various flood sizes are presented in Table 4.3 and Figure 4.2.

For comparison design levels for the four design floods from the previous model are also presented in Figure 4.2. As can be clearly seen the design levels for each of the modelled flood events are higher than those predicted by the model developed in 2007. This increase is attributable mainly to the increases in the design flows.

The increases in the predicted flood levels at various points through the City are summarised in Table 1.1.

Table 1.1- Changes in design flood levels between 'original' and 'new'

	Change in modelled flood level (m)			
	2% AEP	**1% AEP**	**0.5% AEP**	**1% AEP + CC**
Railway Bridge	0.45	0.42	0.39	0.34
Dublin Street Bridge	0.41	0.42	0.40	0.37
City Bridge	0.36	0.37	0.37	0.35
Cobham Bridge	0.32	0.37	0.40	0.44
Yacht Club	0.10	0.12	0.15	0.17

Sedimentation

The Whanganui River was surveyed in 1995, and these cross sections were resurveyed in December 2015 to determine if there had been any significant morphological changes to the bed of the river that could affect its flood carrying capacity.

As Table 5.1 shows, although there has been a fluctuation of the mean bed level at the mouth of the river, there has generally been a raising of the mean bed level in the reach below Cobham Bridge. The mean bed level changes in this reach have been in the order of +/- 300 mm which is well within the range of natural fluctuation that one would expect to see.

In the reach above Cobham Bridge there has generally been a lowering of the mean bed level. This lowering, up to 1 m in places, is much more pronounced than the changes in bed level seen in the lower reach of the river.

This, however, is not backed up by the modelling results. In the 2007 model the river was modelled as a channel with steep banks that would contain the water. However, when the berms are included in the model, the wetted perimeter of the cross section dramatically increases once berm flow begins.

This increase in wetted perimeter means that the flood carrying capacity of the modelled channel is significantly reduced. This explains why both the observed flood levels and the recalibrated modelled levels, are higher than those predicted by the original (2007) model.

The floods against which the original model was calibrated and verified did not include any significant berm flows. It is for this reason that the modelling approach of 'glass walling' the channel was appropriate. However, if the updated 'new' model was to be calibrated against the June 2015 flood, using a 'glass walled' channel, then the model would exaggerate the design flood levels for the larger design flows.

It is concluded that there does not appear to be a significant sedimentation problem in the Lower Whanganui River, indeed in many locations the capacity of the channel has increased since it was last surveyed in 1995.

CONTENTS

1. Introduction

1.1 Objective

Over the period 19 to 21 June 2015 the western area of the Manawatu-Wanganui Region experienced a very major rainfall event. This resulted in flood frequencies close to or exceeding 1% AEP (1 in 100 year) in several rivers. Very substantial flooding occurred through the City of Whanganui, due mainly to the flooding from the Whanganui River, particularly of the Anzac Parade-Kowhai Park locality, where water flooded numerous houses and reached depths of up to 2 metres in some. Urban streams were also in very high flood, with for example both the Matarawa Diversion at No.3 Line and Awarua at Wikitoria Road experiencing floods of well over 1% AEP.

The Lower Whanganui River peaked at a stage of 21.975 metres and flow of 4755 cumecs at the Te Rewa gauge at 0105 hours on 21 June 2015. This gauge is located some 50 kilometres upstream of the river mouth. This flood flow equates to a 1.2% Annual Exceedance Probability, AEP (1 in 85 year) flood at that site. However, in the lower reaches this flood was characterised by well above normal tributary flows. These were due to high rainfalls on wet antecedent conditions, with the 48 hour rainfalls exceeding 1% AEP for almost the entire area downstream of Te Rewa.

Consequently flood levels were very high, reaching slightly above the 0.5% AEP (1 in 200 year) levels around Aramoho and upstream. There were several factors involved in reaching these high levels, though the additional tributary flow was the principal one. Levels in the lower river were mitigated by scour occurring at the river mouth due to a favourable set of ambient conditions.

At the Town Bridge gauge the peak stage reached 9.048 metres at 0255 hours on 21 June 2015. This is some 3.7 metres above average levels at that location. The stopbank at Kowhai Park commenced overtopping with flood waters at 2330 hours on 20 June 2015 at a level close to 8.7 metres at the Town Bridge gauge. Thus it overtopped by up to 350mm for a period of 12 hours, with no structural damage – just some light scouring, particular near tree trunks, reflecting the good condition of the stopbank.

This report examines the size of the June 2015 flood and describes the consequent implications on flood frequency, flood levels and sedimentation through Whanganui City.

1.2 Background

The Whanganui River and its tributaries rise on the central plateau volcanic mountains of Tongariro, Ngauruhoe and Ruapehu. The Whanganui River itself then follows a winding course through steep rugged hill country of siltstone, sandstone and limestone base rocks. The river is confined within very narrow and steep sided valleys nearly to the sea, with a small alluvial plain beside the lowest reach, where the City of Whanganui is located. This plain has been built up by volcanic deposits from the very large Taupo eruption that occurred about 2000 years ago, and by wind-blown sand dunes.

There is, then, only a small part of this plain that is floodable, even in large flood events.

The river is very flat graded for a long way inland and has a very large tidal exchange capacity, with large volumes of sea water flowing into and out of the river over the tidal cycle. Tidal flows are, thus, relatively strong, and there is a substantial tidal range. A large tidal cycle persists through small flood events, and even in large flood events a tidal cycle is superimposed on the flood hydrograph rise and fall along the lowest estuary reach of the river.

The river mouth has shifted naturally over time, and there was a long low sand spit separating the sea from the river estuary. This river outlet area is defined by the high terrace formation of Landguard bluff on the southern side, and a terrace remnant at Castlecliff on the northern side. Flood flows would then have washed over the spit and formed break-outs, and the position of the river mouth would have altered as floods and wave action affected the form and extent of the spit and adjacent coast. The river mouth has now been fixed by the moles, and the spit protected by works and built up by the planting of marram grass. All the river flow now goes between the moles, and this gives rise to a restricted mouth in large flood events, with a substantial (about 1 m) rise in water levels from the sea to the river inside the moles.

There are now three urban areas (residential, commercial and industrial) that are at risk from flooding. On the left (western) side from the Railway Bridge down to the sharp river bend at Shakespeare Cliff, around Kowhai Park, there is an area of residential land (including part of Whanganui Girls' College) at risk. This land was part of the river channel, as a wide flat beach area, and has been reclaimed and protected by stopbanks. The Matarawa Stream enters the Whanganui River in this area. These existing stopbanks provided protection to about the 5% AEP (1 in 20 year) up until 2014, when the stopbank was upgraded to the 2% AEP (1 in 50 year) with no freeboard standard – equivalent to a 3.3% AEP (1 in 30 year) with 300mm freeboard. This standard related to the level of protection provided by the bulk of the stopbank through the area and corrected some significant low spots and weaknesses in the stopbank.

On the right (eastern) side along Taupo Quay, around the City Bridge, there is a narrow area of lower commercial land at risk, with industrial land downstream to Cobham Bridge also at risk. This land is subject to flooding in events with peak flows greater than about a 3.3% AEP flood. The area includes the Town Wharf, and the old Railway yards, where a large stormwater channel flows into the river.

A similar level of flood risk exists to the Putiki Marae and some houses on the left bank at Putiki.

Previously there was also a similar level of flood risk to the industrial land between the Imlay freezing works and the harbour mouth on the right bank (including areas around Heads Road and Gilberd Street extending into the Balgownie swamp). However, in 2011 Horizons completed a major stopbank upgrade to provide protection to the 0.5% AEP with 500mm freeboard standard. These works comprised some 3.2km of predominantly stopbanks and floodwalls, together with two portable flood barriers. They provide protection to 60 ha of primarily industrial and commercial land at Balgownie.

In large well over-design floods there may well be a significant increase in depths and areas flooded. For example, during a flood in the early 1840s water was anecdotally recorded as 0.4 m (16 inches) deep at the intersection of Ridgway and St Hill Streets (it is possible this intersection was at a lower elevation in those days).

1.3 Scope of Work

This report documents the flood risks to the City of Whanganui from the Whanganui River. Whilst the MIKE11 computer model extends many kilometres upstream, the focus of the investigation is on flood risks in the City reach of the river. Urban stormwater is a separate matter and is not included in the report – although flooding at the Matarawa Stream confluence is of necessity included.

Tsunami risks and storm surge wave run-up are <u>not</u> included in the study scope.

1.4 Datums

The design levels presented are in terms of Wellington Vertical Datum 1953 (with 100m added to these levels to convert to Whanganui City Datum) – the only exception being the design sea levels in Table 2.7 are in terms of Moturiki Datum, which is close to Wellington Vertical Datum 1953.

2. Hydrology

2.1 Records of Annual Maxima

Flood flow records are available at the Paetawa recorder site in a continuous series for the period 1957 to early 2014. Prior to this, large floods occurred in the Whanganui River in (ascending size) 1935, 1936, 1926, 1883, 1897, 1939, 1891, 1864, 1875, 1858, 1904 and 1940. The peak discharges for these floods have been estimated from level information given by photographs of the floods and newspaper records, and the corresponding discharges as determined by hydraulic modelling. The National Institute of Water and Atmosphere (NIWA) has recently reviewed the size of these historical pre-1957 floods, resulting in a slight decrease in their estimated values (around 10 percent).

Horizons Regional Council has installed a new recorder site Te Rewa and this site has provided flow records since 2006. The site has essentially the same catchment area as Paetawa of 6643 km^2.

The largest flood in the continuous series prior to the recent extreme 2015 flood of 4755 cumecs is 4106 cumecs in 1990. The 1940 and 1904 floods of respectively 4689 and 4325 cumecs are the second and third largest in either dataset. Six of the historical floods are larger than the 1990 flood, illustrating the importance of researching the size of historic floods. Note the estimated size of the 1940 flood was 5200 cumecs, prior to the NIWA review.

The historic floods are presented in Table 2.1 and annual maxima in Table 2.2 and Figure 2.1.

Of possible relevance to future studies is that nine out of the top ten floods during the period 1957 to 2015 occurred in the negative phase of the Interdecadal Pacific Oscillation (IPO). This is a phenomenon that affects climate and flood sizes across the Pacific with shifts in phase in the mid-1940s, 1977/78 and around 1997/98. The 1940 flood is oppositely located in the positive phase. It has a pronounced effect on flood sizes in the Bay of Plenty, but there is to date little evidence of this in the Manawatu-Wanganui region.

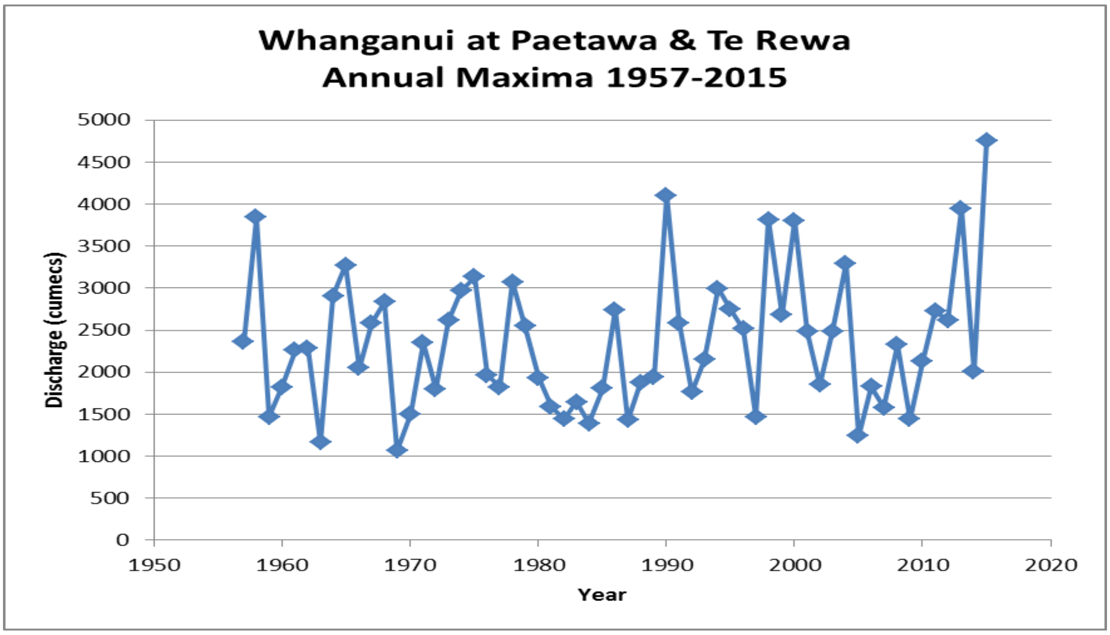
Table 2.1: Whanganui River at Paetawa Historic Floods

YEAR	DISCHARGE (CUMECS)	YEAR	DISCHARGE (CUMECS)
1858	4293	1904	4325
1864	4293	1926	3856
1875	4293	1935	3700
1883	3856	1936	3732
1891	4231	1939	4011
1897	3917	1940	4689

Table 2.2: Whanganui River at Paetawa & Te Rewa Annual Maxima 1957-2015

YEAR	DISCHARGE (CUMECS)	RANK	YEAR	DISCHARGE (CUMECS)	RANK
1957	2359	27	1987	1430	55
1958	3845	4	1988	1872	39
1959	1470	51	1989	1937	37
1960	1816	43	1990	4106	2
1961	2259	31	1991	2589	21
1962	2285	30	1992	1760	46
1963	1163	58	1993	2151	32
1964	2906	13	1994	2996	11
1965	3272	8	1995	2745	15
1966	2047	34	1996	2516	24
1967	2586	22	1997	1466	52
1968	2836	14	1998	3815	5
1969	1063	59	1999	2683	18
1970	1502	50	2000	3804	6
1971	2346	28	2001	2483	25
1972	1798	45	2002	1848	40
1973	2612	20	2003	2482	26
1974	2971	12	2004	3293	7
1975	3134	9	2005	1239	57
1976	1965	36	2006	1830	41
1977	1821	42	2007	1582	49
1978	3071	10	2008	2326	29
1979	2546	23	2009	1440	54
1980	1933	38	2010	2130	33
1981	1590	48	2011	2729	17
1982	1441	53	2012	2617	19
1983	1648	47	2013	3947	3
1984	1390	56	2014	2003	35
1985	1805	44	2015	4755	1
1986	2739	16			

Figure 2.1: Whanganui River at Paetawa & Te Rewa Annual Maxima 1957-2015



2.2 Flood Analysis Methodology

At-site flood frequency analysis was applied to:

a. The continuous series of annual maxima (1957-2015)
b. The continuous series of annual maxima plus the 12 historical peaks.

An L-Moments extreme value statistical fitting methodology was applied to the continuous series for both the Extreme Value Type One and General Extreme Value distributions – the latter resulted in an Extreme Value Type Three distribution – though close to Extreme Value Type One . Inspection of the frequency curve shows that the data conforms to an Extreme Value Type One distribution.

A linear trend-line was applied to the 'censored' dataset of the continuous series plus historical peaks plotted against the reduced 'y' variate. This produces an Extreme Value Type One distribution.

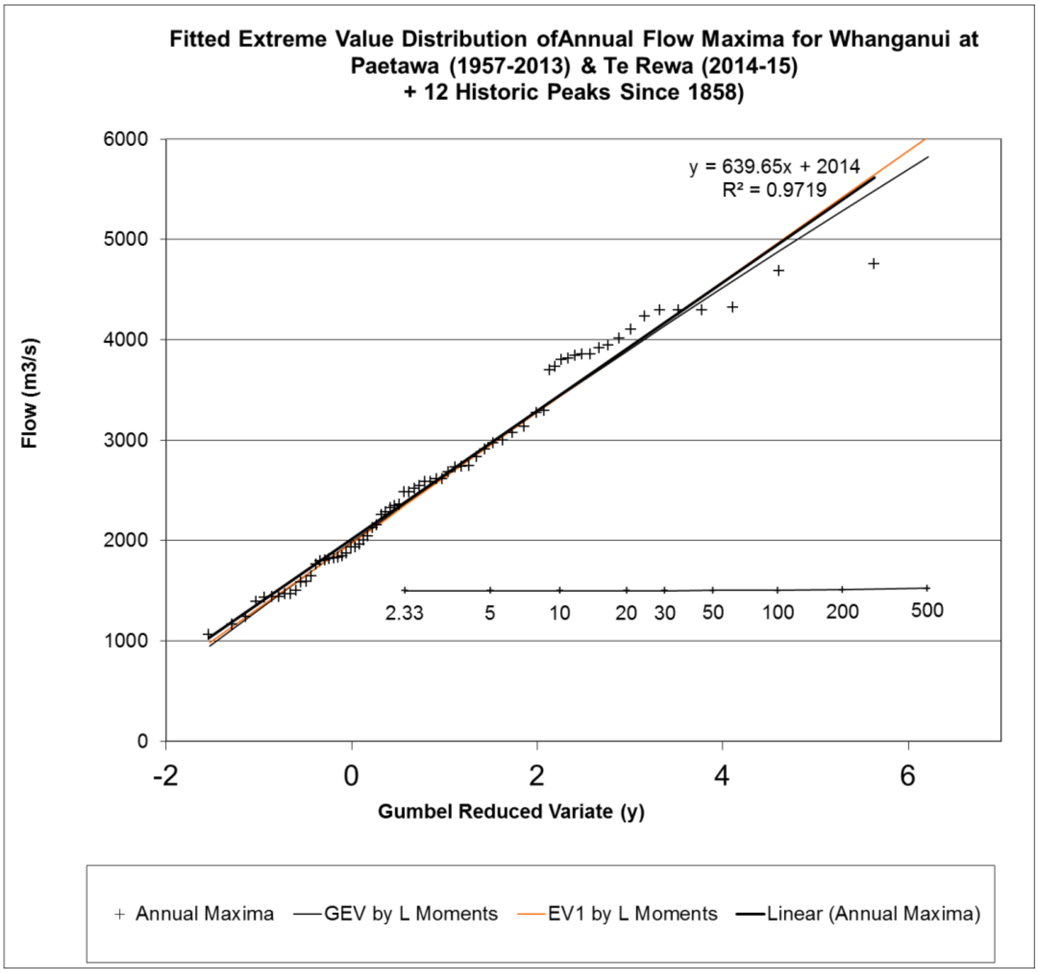
2.3 Flood Frequency Estimates

The design flood frequency estimates are presented in Table 2.3 and Figure 2.2.

Table 2.3: Whanganui at Paetawa & Te Rewa Flood Frequency Estimates (cumecs)

RETURN PERIOD (YEARS)		EV1	GEV	EV1 INCLUDING HISTORICAL
T	Y_T	Q_{T1}	Q_{T2}	
1.5	-0.0940	1915	1921	1954
2	0.3665	2215	2227	2248
2.33	0.5786	2353	2366	2384
5	1.4999	2953	2964	2973
10	2.2504	3442	3441	3453
20	2.9702	3910	3891	3914
30	3.3843	4180	4146	4179
50	3.9019	4517	4461	4510
100	4.6001	4971	4880	4956
200	5.2958	5424	5290	5401
500	6.2136	6022	5821	5989

Figure 2.2: Whanganui at Paetawa & Te Rewa Flood Frequency



2.4 Design Flood Frequency Estimates

It is clear that the historical flood peaks should be included in the analysis (with appropriate plotting position). They provide good information on the rarer flood sizes. Consequently, equal weighting has been applied to the two Extreme Value Type One analyses to produce the design flood frequency estimates in Table 2.4.

Table 2.4: Whanganui at Paetawa Design Flood Frequency Estimates

RETURN PERIOD (YEARS)	PROBABILITY (%)	DISCHARGE (CUMECS)	Y VARIATE
1.5	67	1935	-0.0940
2	50	2232	0.3665
2.33	43	2369	0.5786
5	20	2963	1.4999
10	10	3448	2.2504
20	5	3912	2.9702
30	3.3	4179	3.3843
50	2	4513	3.9019
100	1	4964	4.6001
200	0.5	5413	5.2958
500	0.2	6005	6.2136

The estimate for the 1% AEP (100-year return period) flood of 4964 cumecs is marginally (1.7 percent) higher than the design estimates in the 2007 Horizons Regional Council report and 4.1 percent below the previous (Whanganui District Council) figure of 5175 cumecs. That analysis was based on the continuous series of data 1957-1990 plus historical floods. The slightly lower figures for both the previous and current Horizons Regional estimate for the 1% AEP flood is entirely due to the NIWA review of the size of the historical floods; partially compensated by the additional period of record (1991-2015) containing several large floods.

2.5 Design Global Warming Flood

The continuation of global warming will cause an increase in flood sizes and frequency. An analysis for the Whanganui catchment, based on the Ministry for Environment Guidelines, has produced the following estimates for predicting flood sizes at a “bench-mark” date of 2090:

Increase in Temperature:	2.1 degrees Celsius
Increase in Rainfall Intensity:	18.6 percent
Increase in Flood Flow:	c. 20 percent (non-linear)
Current 1% AEP Flood Estimate:	4964 cumecs
2090 1% AEP Flood Estimate:	5957 cumecs

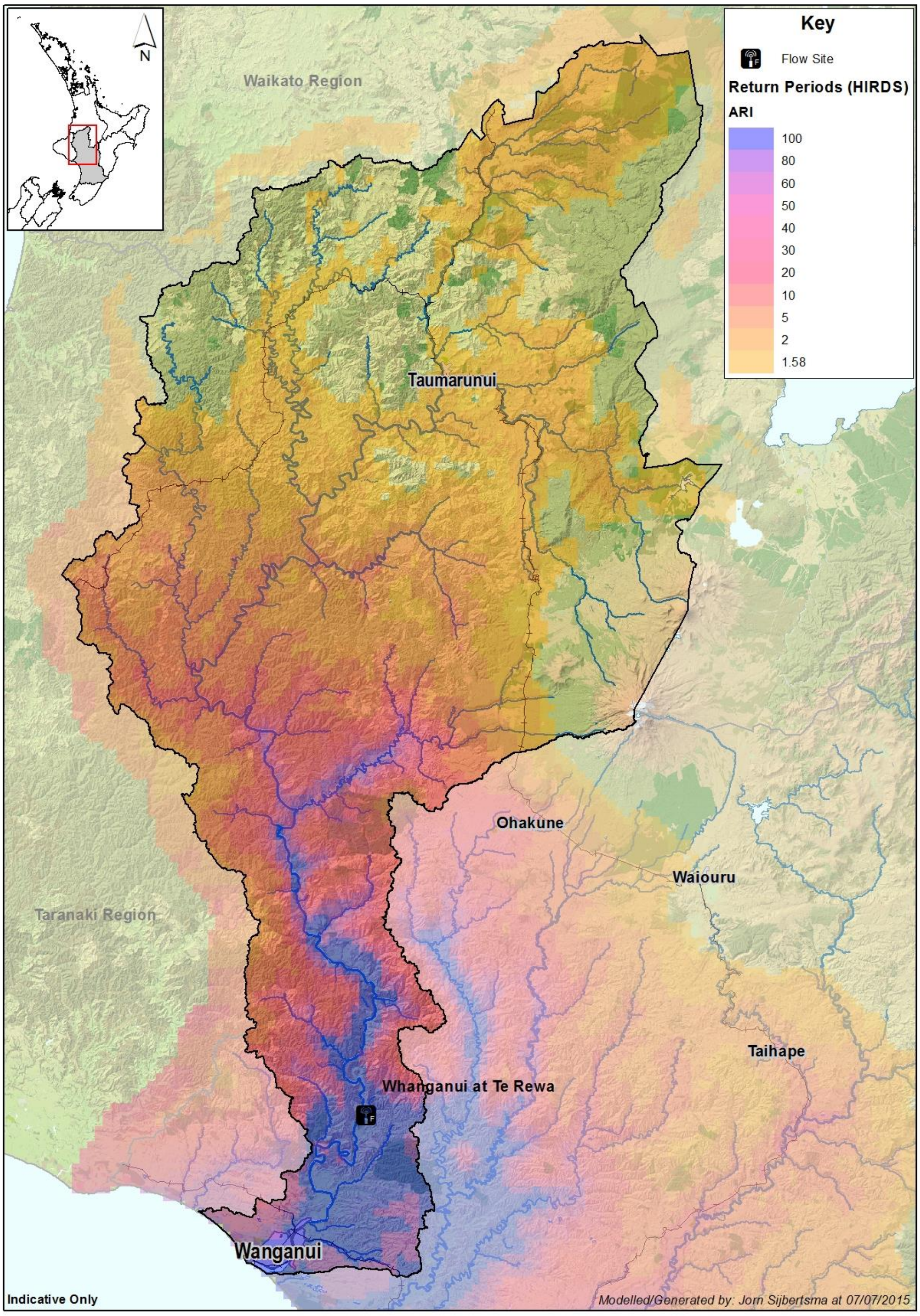
2.6 Tributary Flows in 19-21 June 2015 Flood

There are no current recorders on the tributaries so their flows in the 19-21 June 2015 flood event have been assessed through detailed slope area calculations on five catchments. The remaining catchments were all transposed from the slope area assessments, with adjustments for vegetation. The assessments generally cover the majority of the catchment areas.

The rainfall map in Figure 2.3 shows that the 48 hour rainfalls during the storm over the majority of the catchment downstream of Te Rewa exceeded 1% AEP (1 in 100 year) magnitude, and were slightly less in a small portion of the catchment. These rainfalls were on top of well above average rainfalls being experienced since early April, thus antecedent conditions were wet. Furthermore, a significant heavy burst of rain occurred at the tail of this storm. Assessment of other flow sites on streams in the western part of the region has shown flow return periods well above the rainfall return periods. Thus it would be expected that the majority of tributary flows in the Lower Whanganui catchment would be higher than 1% AEP magnitude.

The hydrograph shapes and their phasing were derived based on recordings of the storm event at the flow sites at Whanganui River at Te Rewa and Town Bridge and Kai Iwi at Handley Road and the rainfall recorded at the Matarawa at Matarawa Valley raingauge. The latter two sites are the only ones suitable for assessing the temporal distribution of rainfall and flow in the tributaries. The time of concentration was determined for the majority of the catchments, largely based on the Ramser-Kirpich formula plus 5-20 minutes, as this has proven reliable in this region. After inspecting bot the Kai Iwi and Whanganui hydrograph shapes the tributaries were set with a falling limb twice the length of the rising limb.

Figure 2.3: Whanganui Catchment 48 Hour Rainfall Frequencies June 2015 Storm



The assessed catchments cover 340.7 km^2 of the 462 km^2 between the Te Rewa gauge and Town Bridge (Awarua is downstream of Town Bridge). The balance area has been lumped into a single point inflow located midway down this reach, but with a short time of concentration of 1 hour, as all this unquantified area is close to the Whanganui River. Whilst this has a relatively high flow of 177 cumecs, the impact on flows at Town Bridge will be small, as the phasing of this flow will not coincide with the peak and attenuates rapidly. Refer Table 2.5.

Table 2.5: Estimated Tributary Flood Flow Peaks 19-21 June 2015

Site	Catchment Area (km^2)	Peak Flow 19-21/5/15 (Cumecs)	Time of Concentration (minutes)	Calculation Method
Upokongaro	94.8	145	330	Slope Area
Makirikiri	22.2	48	180 + 60 to Whanganui	Slope Area
Downstream of S/A sites Upokongaro & Makirikiri	12.27	30	60	Transposed Makirikiri
Kukuta	2.27	10.5	50	Slope Area
Matarawa at No.3 Line	70.2	58.2	300 + 60 to Whanganui	Slope Area
Mateongaonga excluding Matarawa at No.3 Line	28.73	31	240	Transposed Matarawa
Kauarapaoa	88.1	123	540	Transposed Upokongaro less 10%
Mangaiti	22.12	41	180	Transposed Upokongaro less 10%
Balance of catchment Te Rewa to Town Bridge	121.3	177	60	Transposed Upokongaro
Awarua	10.62	24.5	180	Slope Area

2.7 Design Tributary Flows

Previous flood assessments in the Lower Whanganui River have relied on the assumption that tributary flows downstream of the Te Rewa (or Paetawa) gauge are largely cancelled by the attenuation of the flood peak. For example applying the June 2015 flood of 4755 cumecs to the MIKE11 hydraulic model assuming no tributary inflows results in a peak flow of 4692 cumecs at Town Bridge; a drop of some 63 cumecs (a 1.3% drop). However, in the June 2015 flood the tributaries downstream of Te Rewa experienced very significant flows, generally around 1% AEP or greater. Whilst the peak flood flow at the Te Rewa gauge was a 1.2% AEP (1 in 85 year) flood, the levels through the City reach were substantially higher upstream of the City Bridge (downstream of this the low tidal conditions and mouth scour retarded flood levels). These levels reached above the 0.5% AEP level through Aramoho – though there were other factors involved including super-elevation and swash, that are accounted for by the scheme freeboard.

This is the second flood within the last 25 years that has exhibited these characteristics. Accordingly it is considered appropriate to make an allowance for tributary inflows to the design flood. Based on an assessment of the likely coincidence and timing of the Whanganui River and tributary flows the design tributary flows will be based on the 10% AEP tributary flood, coincident with the flood peak at Paetawa. Refer Table 2.6.

Table 2.6: Design Coincident Tributary Flood Flow Peaks 10% AEP

Site	Catchment Area (km^2)	10% AEP Peak Flow (Cumecs)	Time of Concentration (minutes)
Upokongaro	94.8	77	330
Makirikiri	22.2	31.4	180 + 60 to Whanganui
Downstream of S/A sites Upokongaro & Makirikiri	12.27	19.5	60
Kukuta	2.27	7.1	50
Matarawa at No.3 Line	70.2	23	300 + 60 to Whanganui
Mateongaonga excluding Matarawa at No.3 Line	28.73	13.7	240
Kauarapaoa	88.1	52	540
Mangaiti	22.12	23.7	180
Balance of catchment Te Rewa to Town Bridge	121.3	94	60
Awarua	10.62	14	180

2.8 Commentary on Design Flood Flows

A point that should not be missed is that this was essentially a storm focussed on the southern parts of the catchment. This is shown in Figure 2.3, where the 48 hour rainfall return periods exceed 100 year for most of the catchment downstream of Te Rewa, yet are generally in the range 2 to 10 year in the northern parts, where the proportion of catchment area is more. A storm centred on the mid catchment could potentially produce a significantly larger flood. However, the meteorological factors required for such a storm and its areal extent are unknown and the probabilities also unknown.

The flood estimates in this study are based on a robust period of almost 60 years of continuous data, augmented by appropriately included extreme floods dating back to 1858. Interestingly, the inclusion of the 1st and 3rd highest floods in the continuous series along with the other 7 years of additional data only increases the design 0.5% AEP flow by 1.7%.

Another important observation is that inclusion of the “Historic Floods” dating back to 1858 has not materially altered the flood frequency – in the 0.5% AEP flood the net impact is an estimate 0.4% less than the continuous series EV1 estimate. However, it has increased the confidence in the flood estimates.

The conclusion is that the design flood flows are reliable estimates.

2.9 Design ‘Stillwater’ Sea Levels

Design ‘Stillwater' sea levels (exclusive of wave run-up) are necessary to set the flood levels at the downstream end of the hydraulic model (being the Whanganui River Mouth). The sea levels were investigated in detail in the report entitled “*Storm Surge and Wave Run-up Design Levels for Foxton Beach: An Assessment of Flood Risks and Mitigation Options”*, May 2007, PL Blackwood. Sea levels at the Whanganui River Mouth are 0.1 m higher than those at Foxton. The design sea levels are presented in Table 2.7.

Table 2.7: Whanganui at River Mouth Design Sea Levels

RETURN PERIOD (YEARS)	PROBABILITY (%)	SEA LEVEL (M, MOTURIKI DATUM)
5	20	2.2
10	10	2.3
20	5	2.4
50	2	2.6
100	1	2.7
200	0.5	2.9

Note:
These values have been rounded to one decimal place to reflect the short length of data available and the transposition analysis applied. They are however, a sound basis for assessing design flood levels.

3. Hydraulic Modelling

A MIKE11 model of the Whanganui River, from Paetawa to the Tasman Sea, was built by Hydro Tasmania Consulting in 2007 as part of a wider MIKEFLOOD model. This model has been used by Horizons Regional Council (HRC) to inform flood predictions and flood defence design in the Lower Whanganui River.

The volume contained in flood hydrographs is large, consequently in floods of 2% AEP or greater, the levels predicted by the MIKE11 model will essentially match those in the MIKEFLOOD model.

3.1 Computer Software

MIKE11 uses an implicit finite-difference scheme for the computation of uni-directional unsteady flow in rivers. It also incorporates advanced computational models for the description of flow over hydraulic structures – including bridges, although bridges were not specifically incorporated into the model, because of minor impacts. The unsteady flow properties enable effective application of the temporal variation in flow.

The MIKE11 hydraulic model includes 50 cross-sections on the Whanganui River between Paetawa (river chainage 49330 m) and the river mouth (river chainage 96260 m).

3.2 Model Runs

The calibration process was conducted in several progressive stages as follows:

1. Application of the ‘original’ (2007) MIKE11 model with the cross sections surveyed in 1995.
2. Substitution of the model cross sections with those surveyed in December 2015.
3. Addition of the estimated tributary inflows in the June 2015 flood event.
4. Inclusion of the forecast mouth scour; and
5. Minor recalibration of the model to represent the observed flood levels.

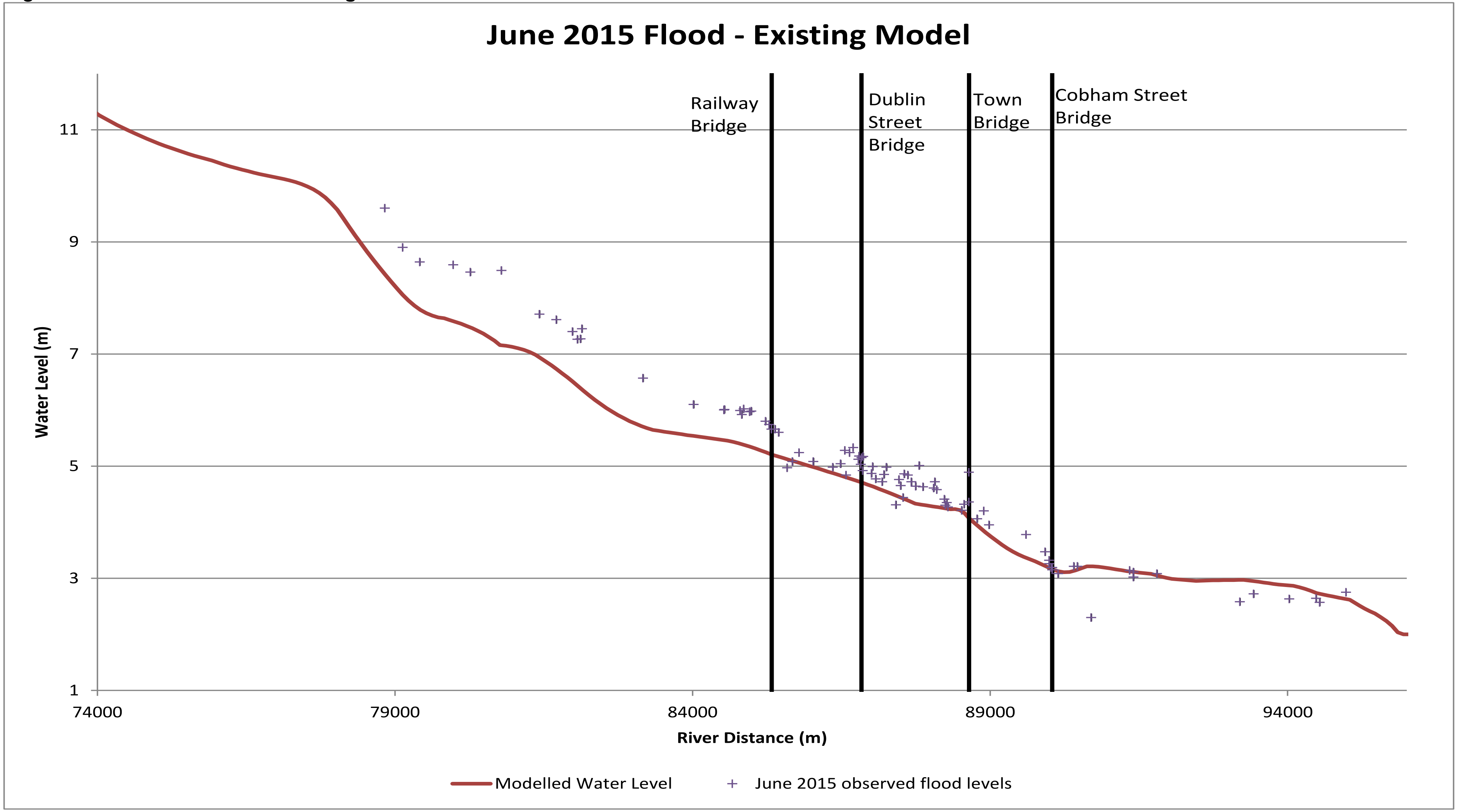
3.2.1 Existing Model Run of June 2015 Flood

The first model run was a simulation of the June 2015 flood using the ‘original’ MIKE11 model. The boundary conditions used for this model run were the recorded flows from the Whanganui at Te Rewa gauging site and the forecast Whanganui River at Mouth Sea Levels for the period between 19 and 23 June 2015.

The maximum water levels from this model run are shown on Figure 3.1 along with the debris levels that were surveyed following the flood event.

As Figure 3.1 clearly shows, the 'original' model significantly under predicts the maximum water levels seen through the City, above the Cobham Street Bridge.

Figure 3.1 – June 2015 Flood – 'original' Model



3.2.2 June 2015 Flood – Model Updated with 2015 Cross Sections

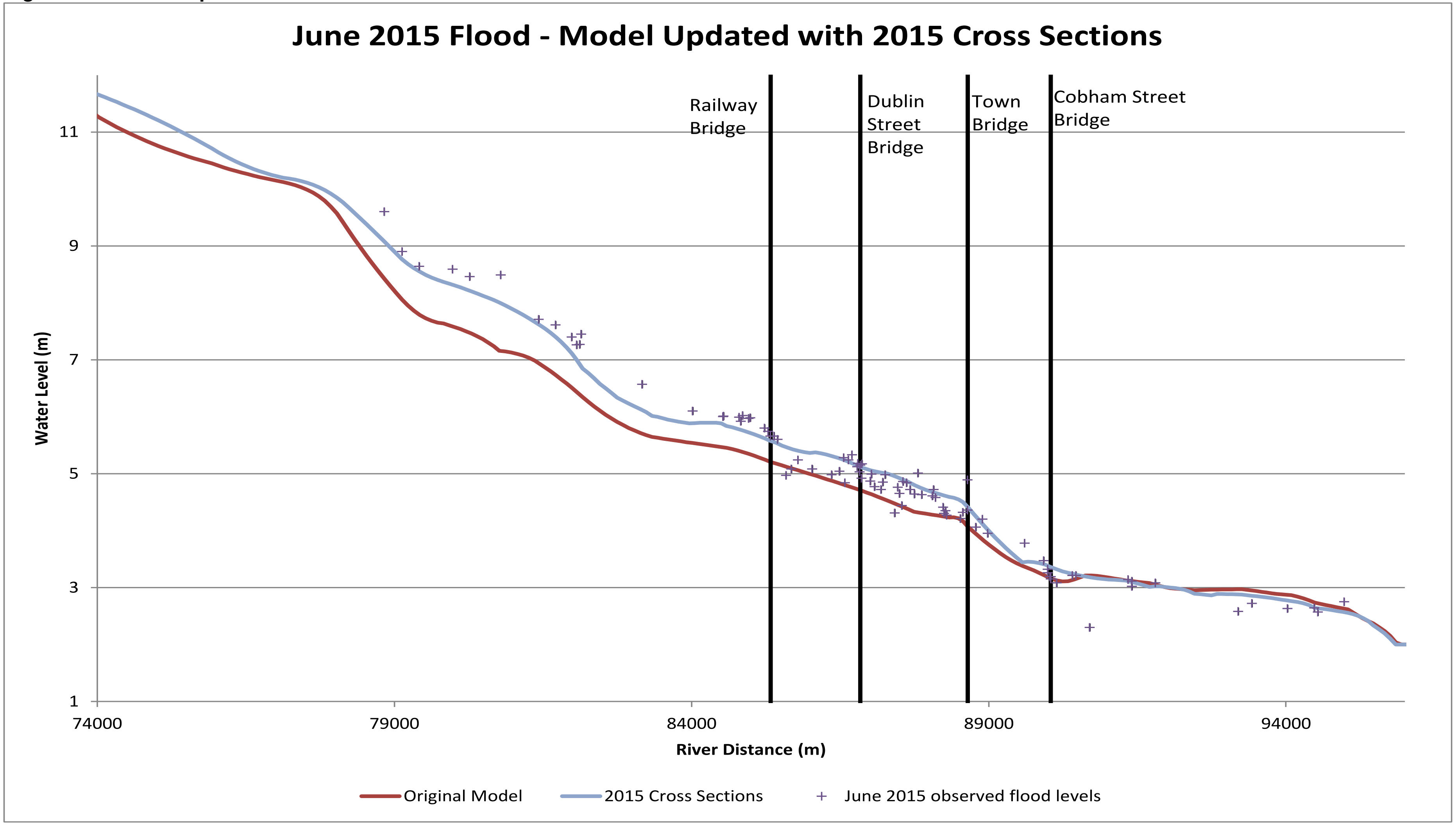
The MIKE11 model was updated with new cross sections of the Whanganui River that were surveyed in December of 2015, to see if changes in the morphology of the river could have affected the observed flood levels.

This updated model was run with the same boundary conditions discussed in Section 3.2.1.

The results from this model run are shown in Figure 3.2, along with the results of the previous model run for comparison.

Whilst the model has produced results that are close to the observed flood levels downstream of the Dublin Street Bridge, it is clear that there is a significant under prediction of river levels upstream of the Railway Bridge.

Figure 3.2 - Model updated with 2015 cross sections



3.2.3 June 2015 Flood – Model Updated with Tributary Inflows

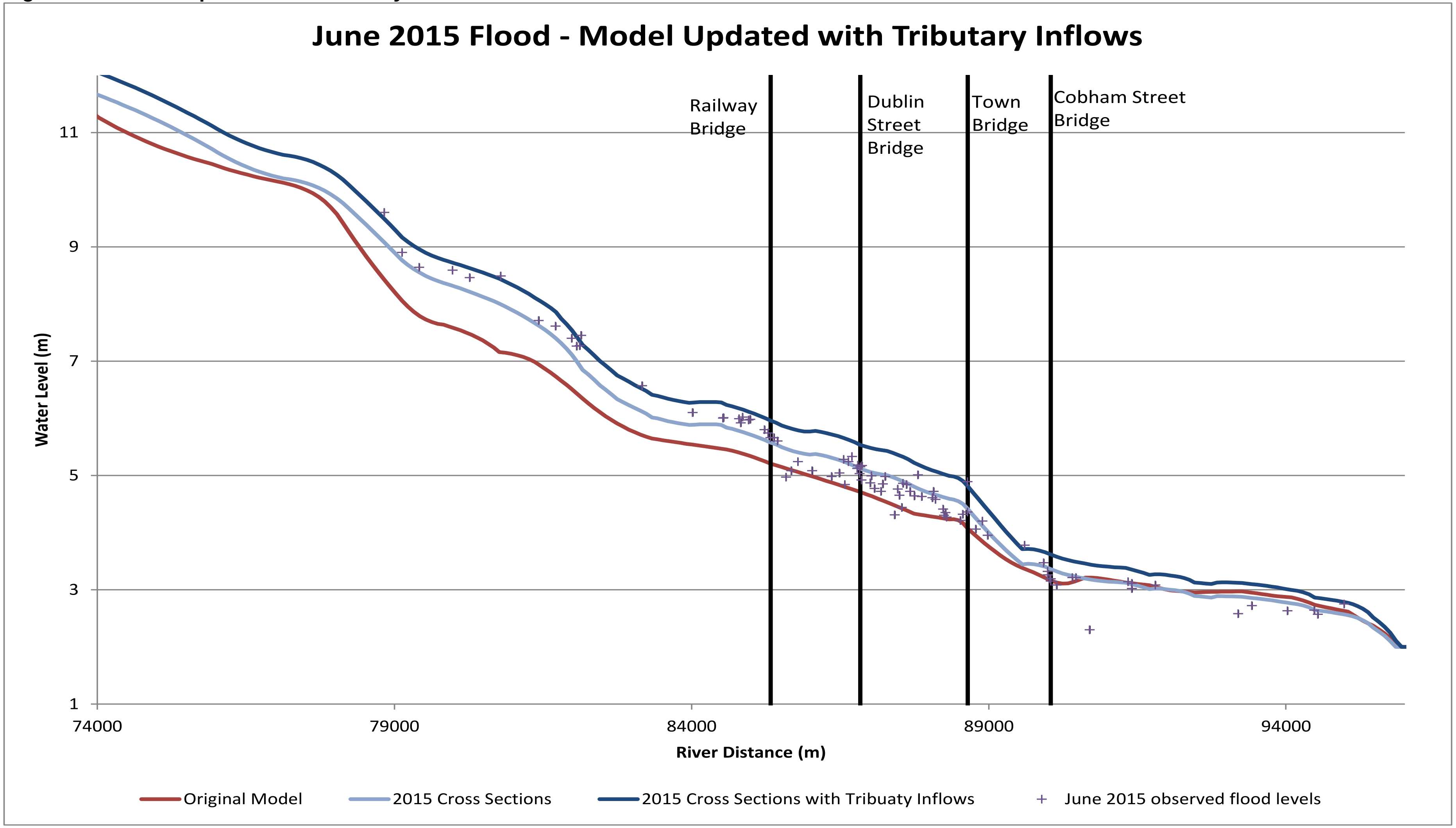
One of the significant features of the June 2015 flood event was the amount of rain that fell in the Lower Whanganui River Catchment and entered the River downstream of the gauging station at Te Rewa.

These inflows, and their estimations, are discussed in Section 2.

The model was run with these inflows as point sources into the Whanganui River along with the same boundary conditions as the previous model run. The results from this model run are shown in Figure 3.3

Whilst this updated model has accurately represented the observed flood levels upstream of the Railway Bridge, it is noticeable that the flood levels have been over predicted in the lower reaches of the river.

Figure 3.3 - Model updated with tributary inflows



3.2.4 June 2015 Flood – Scoured Mouth

To account for the bed scour that occurred near the Whanganui River mouth during the June 2015 flood, the model was run again with a number of cross section adjustments at the downstream end of the river. The scour was enhanced during this flood because the peak flow occurred on the ebb tide, thus velocities were increased and there was almost no storm surge. Therefore, the differential head and consequent velocities were well above normal, resulting in scour of the river mouth. This phenomenon is elevated when appropriate in major hydraulic models of river mouths.

The scour applied, through cross section lowering, was:

Cross Section 96260 m (Whanganui River mouth)	1.5 m
Cross Section 96040 m	1.5 m
Cross Section 95465 m	1.0 m
Cross Section 95045 m	0.5 m
Cross Section 94485 m	0.5 m

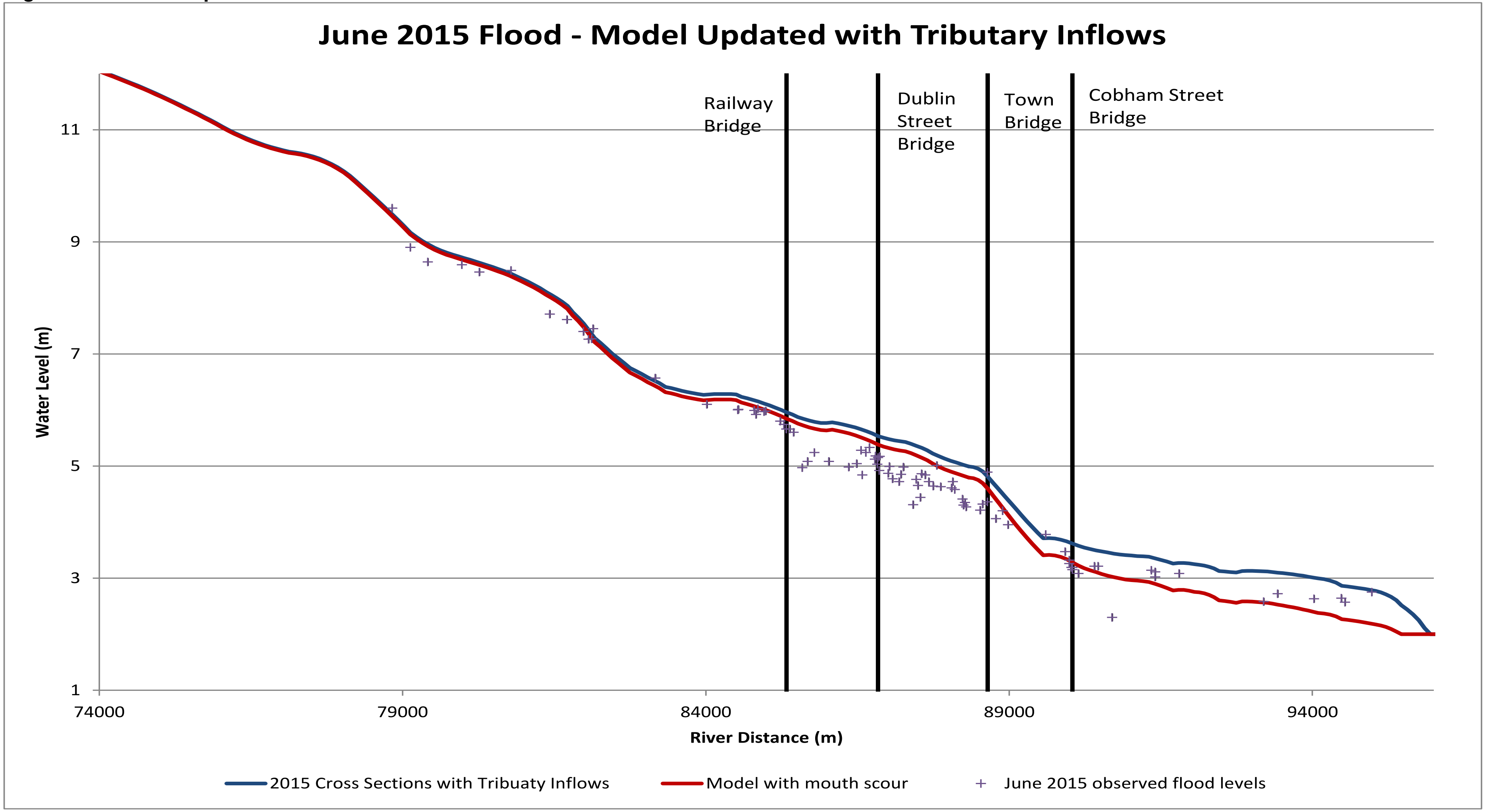
The results from this model run are shown on Figure 3.4 along with the results of the model run with no allowance for bed scour.

As the graph clearly shows, the model with mouth scour provides an accurate representation of the maximum water level seen in June 2015 above the railway bridge.

Between the railway bridge and the Cobham Street bridge the model appears to overestimate the flood levels. Whilst downstream of the Cobham Street Bridge the observed water levels appear to fall within the envelope of the models with and without mouth scour. This is in keeping with what one would expect to see as the scouring of the river mouth would be a gradual process throughout the flood.

Mouth scour will not be applied to design runs as there is no expectation of it occurring during every flood.

Figure 3.4 - Model updated with mouth scour



3.3 Calibration

3.3.1 Calibration Flood Levels and Boundary Conditions

The ‘original’ MIKE11 model had been calibrated against the 1990 and verified against the flood of 29 October 1998.

The ‘new’ (2016) calibrated model reproduced the recorded flood levels in these events to a good accuracy. The accuracy of this model explains why the ‘new’ model with mouth scour reproduced levels seen in June 2015 accurately for much of the River reach through Whanganui.

However, it is noticed that the model generally over predicted flood levels through Kowhai Park.

An analysis of the ‘original’ model parameters revealed that the cross section at City Bridge had been assigned a Manning’s n of 0.04. This value was noticeable higher than the upstream and downstream cross sections which had roughness values of 0.02 and 0.025 respectively. The higher value of Manning’s n may well have been assigned to model the effects of the channel constriction at the bridge.

The effects of varying this single roughness parameter were investigated. It was found that if the Manning’s n at City Bridge was reduced to 0.032, the model was able to reproduce the flood of June 2015 to a good accuracy.

The results of from this ‘new’ calibrated model (incorporating 2015 cross sections, tributary inflows and minor roughness adjustments) can be seen in Figure 3.5 and Figure 3.6.

Figure 3.5 - Calibrated Model Results

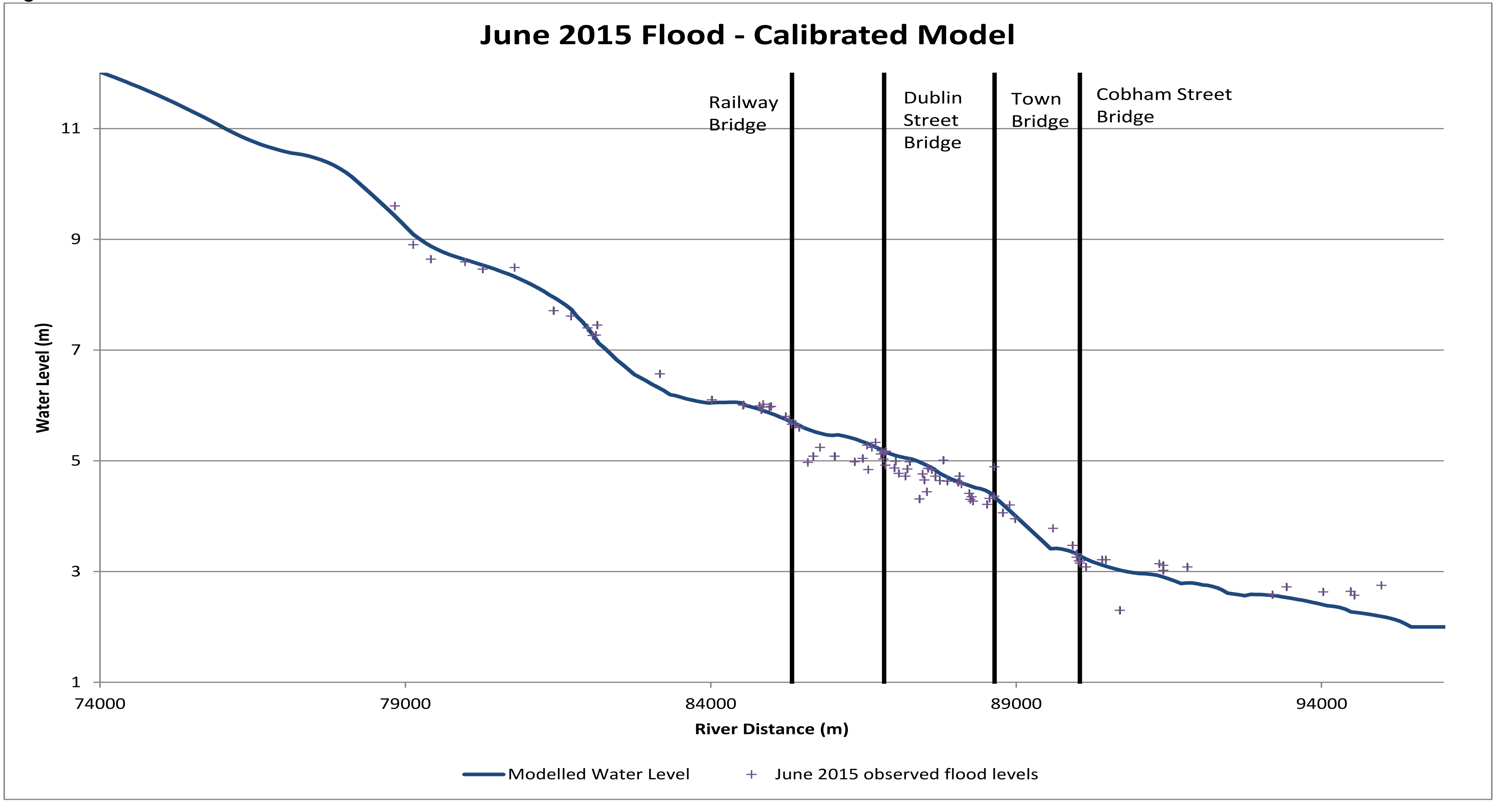
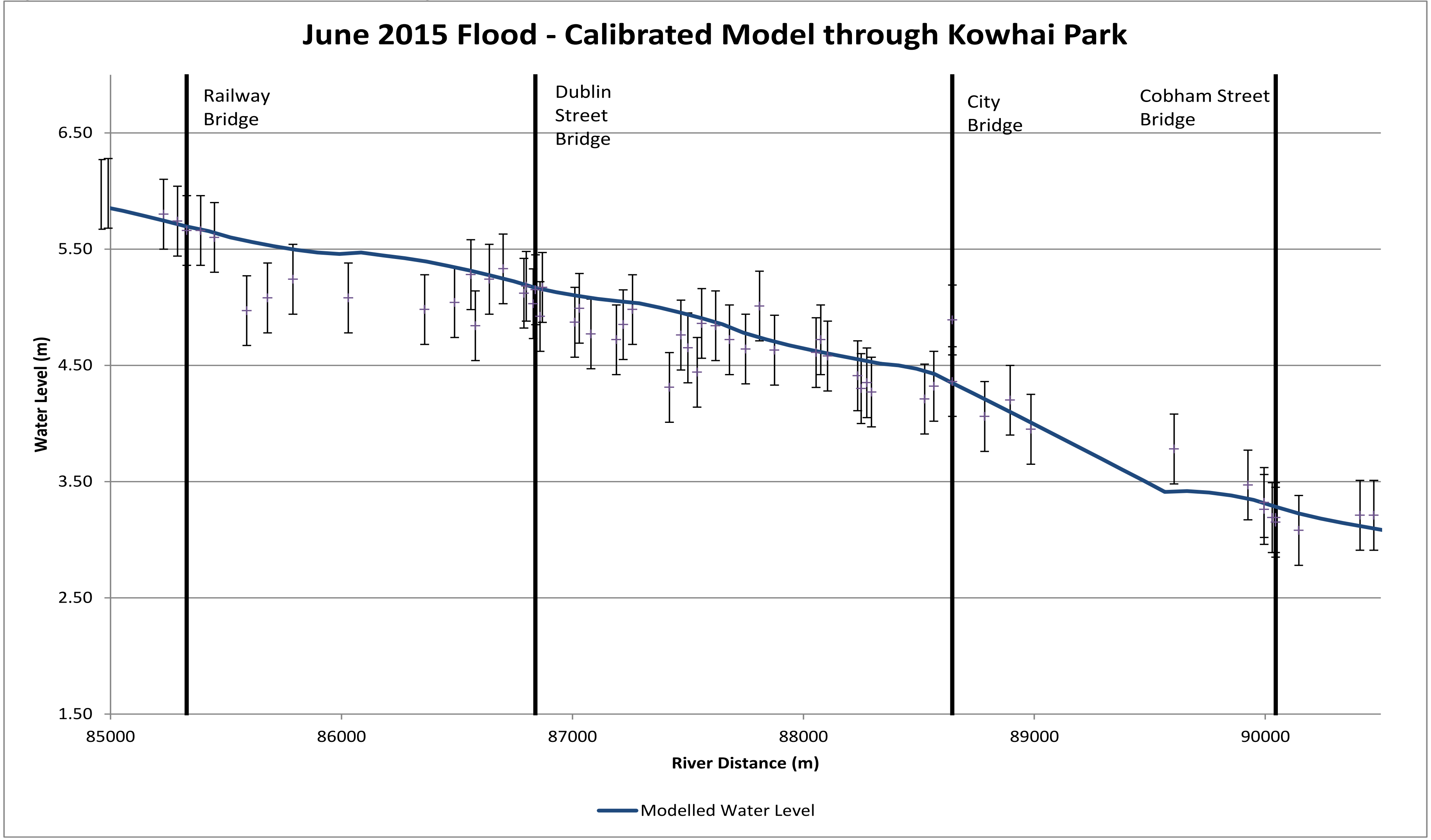


Figure 3.6 - Calibrated Model Results through Kowhai Park Reach



3.3.2 Calibration Results

The calibration results can be seen in Figure 3.5. Figure 3.6 shows how the modelled water levels correspond with the recorded flood levels with error bars of +/- 0.3 m. This is the magnitude of model imprecision commonly determined to assess a models accuracy and is reflected as a component in the freeboard assigned of 0.5 m.

As the graphs show, the final calibrated model reproduced the flood levels recorded in the June 2015 flood to a good accuracy.

There is a high water level recorded at City Bridge which can be explained by superelevation at this location. The application of the freeboard will ensure that design maximum flow levels are adequate and appropriate here. Furthermore, there are a few recorded flood levels in the Kowhai Park reach that are lower than the modelled levels. These are likely due to the debris marks being collected from 'sheltered locations' such as the downstream side of buildings.

Therefore, the model calibration is fine and the model formulation is an accurate basis for producing design flood estimates

4. Results

4.1 June 2015 Flood

The calibrated 'new' model has been used to replicate the June 2015 flood event. The modelled flood levels and flows from this event are presented in Table 4.1 and Figure 4.1. For comparison the flood levels predicted by the original (2007) model, with an allowance for mouth scour, are also presented.

The recorded flood flow of 4755 cumecs at Te Rewa is the highest flood flow recorded on the Whanganui River and furthermore, is understood to be the second highest flood flow ever recorded in the North Island – behind only the famous Mohaka Flood flow of 1938, estimated at 225,000 cusecs (6370 cumecs). The peak level of 21.975m at Te Rewa was some 17 metres above normal low flow levels, an incredible rise. Near the Kauarapaoa Stream mouth there was a similar rise and debris is still sitting near the tops of trees (Wayne Spencer pers comm).

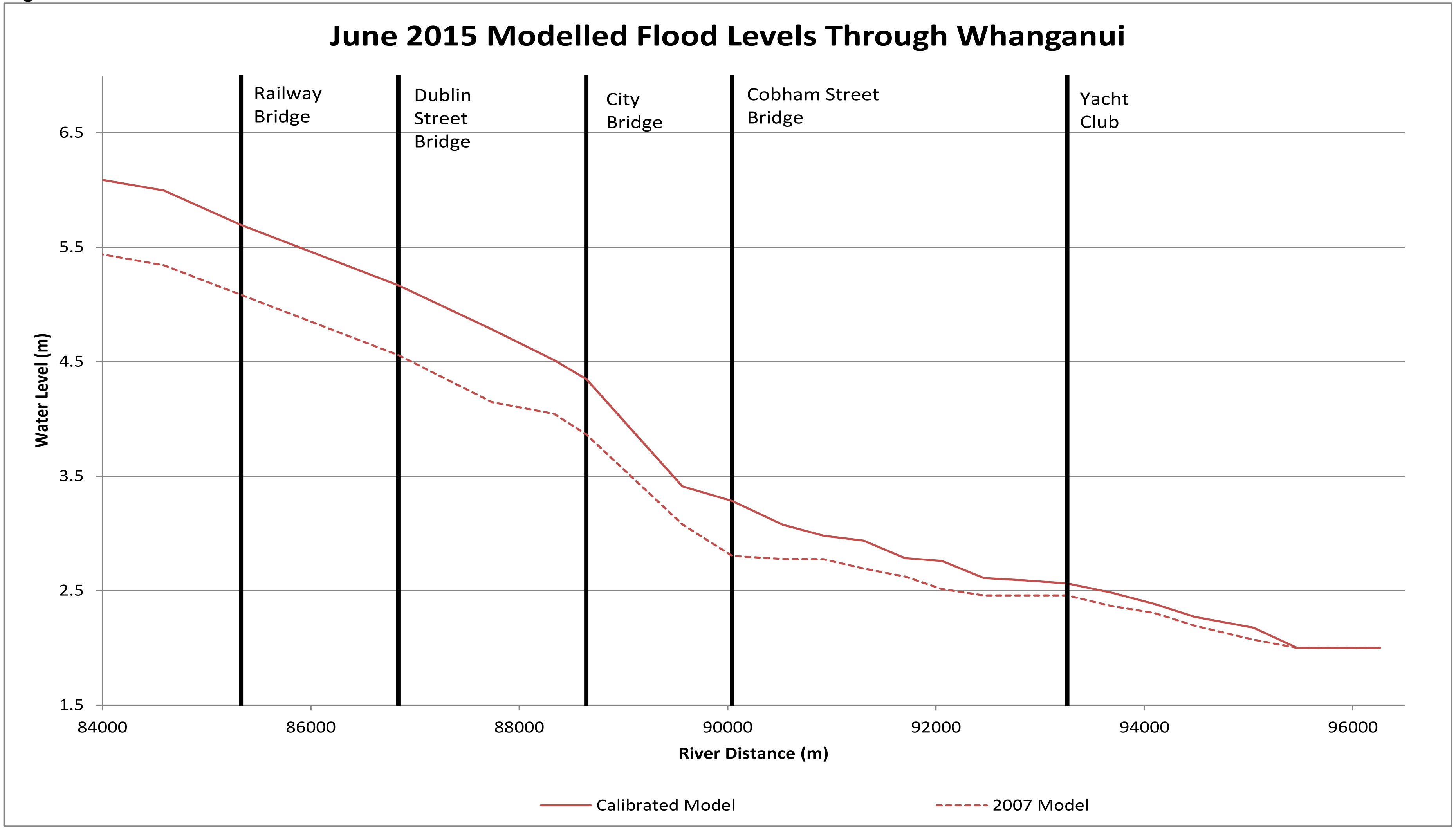
Very clearly the peak flood flow through Whanganui City was significantly above the 4755 cumecs recorded at Te Rewa. Past studies have assumed that the additional tributary flows are cancelled by the attenuation of the flood wave as it travelled to Whanganui City (and indeed as this approach was applied to both calibration and design estimates, there is some degree of self cancelling). This attenuation has been modelled at 65 cumecs in the June 2015 flood. However, with the 48 hour rainfalls exceeding 1% AEP in the reach below Te Rewa, on a very wet catchment, the tributary flows were significant. The final blow was a significant heavy burst of rain near the tail end of the storm.

The concluded flow that passed through the Town Bridge location is 5150 cumecs (refer Table 4.1) magnitude. We can expect a flood of this magnitude at an average annual probability of 0.77%, equivalent to a 1 in 130 year flood. (Note based on the flood frequency statistics available prior to this flood it would have been regarded as a flood of 0.67% AEP, equivalent to a 1 in 150 year flood).

Table 4.1 - June 2015 modelled flood levels

MIKE11 Chainage (m)	2007 Model		Calibrated Model	
	Max Water Level (m)	Peak Discharge (m^3s^{-1})	Max Water Level (m)	Peak Discharge (m^3s^{-1})
49330	20.69	4754	21.14	4754
51299.5	19.98	4737	20.48	4731
53269	19.37	4726	19.94	4720
55136.4	18.78	4720	19.41	4715
57102	18.05	4716	18.77	4710
57741	17.87	4715	18.61	4709
59231.5	17.24	4712	18.07	4706
60722	16.51	4710	17.45	4704
61574	16.20	4709	17.19	4703
62319	16.11	4707	17.12	4702
63384	15.94	4705	16.99	4701
64764.3	15.69	4703	16.80	4701
66046	15.23	4701	16.34	4940
67640.6	14.50	4700	15.54	4936
69235.1	13.74	4698	14.61	4933
70730	12.99	4696	13.60	4931
72230	12.23	4694	12.72	4928
74030	11.25	4694	12.01	4924
75930	10.43	4693	11.08	4922
77130	10.10	4692	10.56	4922
78130	9.39	4692	10.12	4921
79130	8.02	4692	9.09	4921
79827	7.58	4692	8.69	4922
80767	7.10	4692	8.34	4923
81362	6.92	4692	7.99	4924
82165	6.27	4692	7.12	5071
83335	5.54	4693	6.19	5093
84590	5.34	4694	6.00	5094
85330	5.08	4695	5.69	5095
86840	4.56	4697	5.17	5098
87740	4.15	4699	4.78	5147
88335	4.04	4700	4.51	5149
88645	3.86	4700	4.35	5150
89565	3.08	4703	3.41	5155
90045	2.80	4706	3.28	5159
90530	2.78	4711	3.08	5166
90920	2.77	4717	2.98	5184
91305	2.69	4724	2.94	5192
91705	2.62	4731	2.78	5199
92055	2.51	4736	2.76	5205
92460	2.46	4743	2.61	5212
92840	2.46	4752	2.59	5219
93260	2.46	4764	2.56	5231
93680	2.37	4778	2.48	5244
94095	2.30	4792	2.38	5258
94485	2.19	4803	2.27	5269
95045	2.07	4821	2.18	5286
95465	2.00	4834	2.00	5298
96040	2.00	4846	2.00	5310
96260	2.00	4849	2.00	5313

Figure 4.1 - June 2015 Modelled Flood Levels



4.2 Design Flood Levels

Design flood levels for the various flood sizes are presented in Table 4.3 and Figure 4.2.

For comparison design levels for the four design floods from the previous model are also presented in Figure 4.2. As can be clearly seen the design levels for each of the modelled flood events are higher than those predicted by the model developed in 2007. This increase is mainly attributable to the increases in the design flows.

The increases in the predicted flood levels at various points through the City are summarised in Table 4.2.

Table 4.2- Changes in design flood levels between 'original' and 'new' models

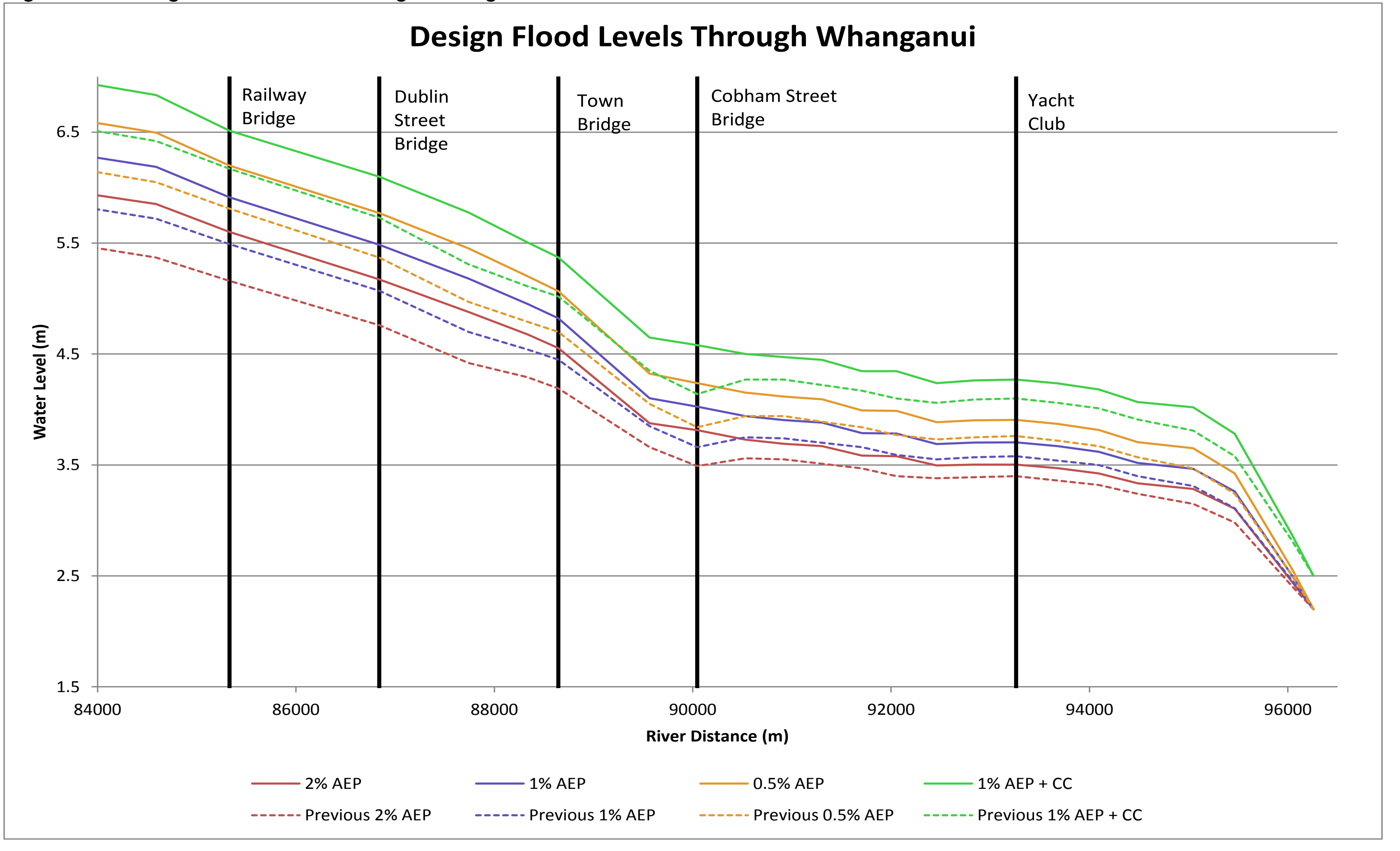
	Change in modelled flood level (m)			
	2% AEP	1% AEP	0.5% AEP	1% AEP + CC
Railway Bridge	0.45	0.42	0.39	0.34
Dublin Street Bridge	0.41	0.42	0.40	0.37
City Bridge	0.36	0.37	0.37	0.35
Cobham Bridge	0.32	0.37	0.40	0.44
Yacht Club	0.10	0.12	0.15	0.17

A freeboard component of 0.5 m should be added to all design levels. This is an allowance for the estimate precision (nominally ± 0.3 m) and phenomenon not explicitly included in the estimates including wind and wave effects, aggradation, bridge efflux and other hydraulic factors such as cross-section transitions and super elevation on bends.

Table 4.3 - Whanganui River Design Flood Levels

MIKE11 Chainage	Peak Flood Levels (m)				Peak Discharges (m^3s^{-1})			
(m)	2% AEP	1% AEP	0.5% AEP	1% AEP + CC	2% AEP	1% AEP	0.5% AEP	1% AEP + CC
49330	20.50	21.42	22.29	23.15	4512	4963	5412	5886
51299.5	19.83	20.74	21.60	22.44	4494	4944	5393	5867
53269	19.28	20.17	21.01	21.83	4481	4931	5379	5854
55234.6	18.71	19.59	20.41	21.21	4476	4925	5373	5847
57102	18.08	18.94	19.74	20.52	4472	4921	5369	5844
57741	17.92	18.78	19.57	20.34	4470	4920	5368	5844
59231.5	17.37	18.20	18.97	19.70	4467	4917	5365	5841
60722	16.74	17.53	18.26	18.95	4464	4914	5363	5840
61574	16.48	17.25	17.97	18.64	4463	4913	5361	5839
62319	16.40	17.18	17.91	18.59	4461	4911	5360	5838
63384	16.26	17.03	17.75	18.41	4459	4909	5358	5836
64764.3	16.07	16.83	17.54	18.20	4457	4907	5355	5834
66046	15.64	16.40	17.09	17.74	4518	4969	5418	5898
67640.6	14.85	15.59	16.26	16.89	4515	4966	5415	5896
69135.4	14.02	14.73	15.37	15.98	4513	4964	5413	5894
70730	12.98	13.65	14.25	14.82	4510	4961	5411	5892
72330	12.08	12.72	13.28	13.83	4508	4959	5408	5891
74030	11.45	12.06	12.59	13.12	4506	4957	5407	5888
75930	10.53	11.12	11.63	12.13	4503	4955	5405	5886
77130	10.03	10.61	11.09	11.56	4502	4954	5405	5885
78130	9.62	10.17	10.64	11.10	4502	4953	5404	5885
79130	8.67	9.14	9.57	10.00	4502	4953	5404	5885
79827	8.29	8.74	9.16	9.60	4502	4953	5404	5885
80767	7.95	8.40	8.82	9.27	4503	4953	5404	5885
81362	7.62	8.05	8.46	8.89	4503	4953	5404	5885
82165	6.82	7.23	7.61	8.02	4581	5034	5486	5969
83335	6.02	6.36	6.68	7.03	4591	5044	5496	5979
84590	5.85	6.19	6.50	6.84	4590	5043	5496	5979
85330	5.60	5.91	6.20	6.51	4590	5043	5496	5978
86840	5.17	5.49	5.77	6.10	4589	5042	5495	5978
87740	4.88	5.18	5.45	5.78	4606	5059	5512	5996
88335	4.68	4.95	5.20	5.51	4606	5059	5512	5995
88645	4.55	4.82	5.07	5.37	4606	5059	5512	5995
89565	3.88	4.10	4.32	4.65	4605	5058	5511	5995
90045	3.81	4.03	4.24	4.58	4605	5058	5511	5994
90530	3.73	3.94	4.15	4.50	4604	5057	5510	5993
90920	3.69	3.90	4.12	4.47	4614	5067	5520	6003
91305	3.67	3.88	4.09	4.45	4614	5067	5519	6003
91705	3.59	3.79	3.99	4.35	4613	5066	5519	6003
92055	3.58	3.78	3.99	4.35	4613	5066	5519	6002
92460	3.50	3.69	3.89	4.24	4613	5066	5518	6002
92840	3.50	3.70	3.90	4.26	4613	5066	5518	6002
93260	3.50	3.70	3.91	4.27	4613	5065	5518	6001
93680	3.47	3.67	3.87	4.24	4613	5065	5518	6001
94095	3.42	3.62	3.82	4.18	4613	5065	5518	6001
94485	3.33	3.52	3.71	4.07	4613	5065	5518	6001
95045	3.28	3.47	3.65	4.02	4613	5065	5518	6001
95465	3.11	3.26	3.42	3.78	4613	5065	5518	6001
96040	2.45	2.51	2.56	2.87	4613	5065	5518	6001
96260	2.20	2.20	2.20	2.50	4613	5065	5518	6001

Figure 4.2 - Design Flood Levels Through Whanganui



5. Sedimentation

The Whanganui River was surveyed in 1995, and the surveyed cross sections were used to build the original MIKEFLOOD model of the river as previously discussed. These cross sections were resurveyed in December 2015 to determine if there had been any significant morphological changes to the bed of the river that could affect its flood carrying capacity. The physical locations of these cross sections can be seen in Appendix A along with over plots of the 1995 and 2015 surveys.

The mean bed level and area beneath each cross section has been extracted from the data using HILLTOP Hydro software, and these are summarised in Table 5.1.

Table 5.1 - Whanganui River Cross Sectional Changes

	Mean bed level (m)			Area Under Section (m^2)		
Section	1995	2015	change	1995	2015	change
1	-3.465	-3.387	0.078	2173	2199	26
2	-2.06	-2.333	-0.273	4160.6	4017.4	-143.2
3	-2.075	-2.187	-0.112	3851.5	3797.1	-54.4
4	-1.825	-1.775	0.05	5044.2	5074.8	30.6
4a	-1.09	-1.124	-0.034	6602.5	6576.8	-25.7
5	-1.464	-1.221	0.243	6325	6505.1	180.1
6	-1.551	-1.366	0.185	4841.5	4947.3	105.8
7	-1.681	-1.362	0.319	4018.3	4172.1	153.8
8	-0.522	-0.423	0.099	5708.1	5767.8	59.7
9	-1.222	-1.434	-0.212	4973.4	4853	-120.4
9a	0.32	-0.683	-1.003	7801.6	7044	-757.6
11	-0.909	-0.951	-0.042	4952.8	4930	-22.8
12	-2.071	-3.069	-0.998	2196.2	1919.8	-276.4
13	-2.193	-2.412	-0.219	2022.1	1965.4	-56.7
14	-2.789	-3.502	-0.713	1564.7	1410	-154.7
15	-6.59	-6.896	-0.306	504.7	459.4	-45.3
16	-2.865	-3.099	-0.234	1548.4	1497.5	-50.9
17	-1.822	-1.899	-0.077	2068.3	2048.8	-19.5
18	-2.419	-2.601	-0.182	1474.7	1439.4	-35.3
19	-1.703	-1.826	-0.123	1828.7	1801.6	-27.1
21	-2.413	-2.13	0.283	1312.5	1361.5	49
22	-3.901	-2.761	1.14	896.6	1064.2	167.6

As Table 5.1 shows, although there has been a fluctuation of the mean bed level at the mouth of the river, there has generally been a raising of the mean bed level in the reach below Cobham Bridge. The mean bed level changes in this reach have been in the order of +/- 300 mm which is well within the range of natural fluctuation that one would expect to see. These changes in mean bed level are not considered to be significant in terms of the conveyance of flood flows, but could have other implications.

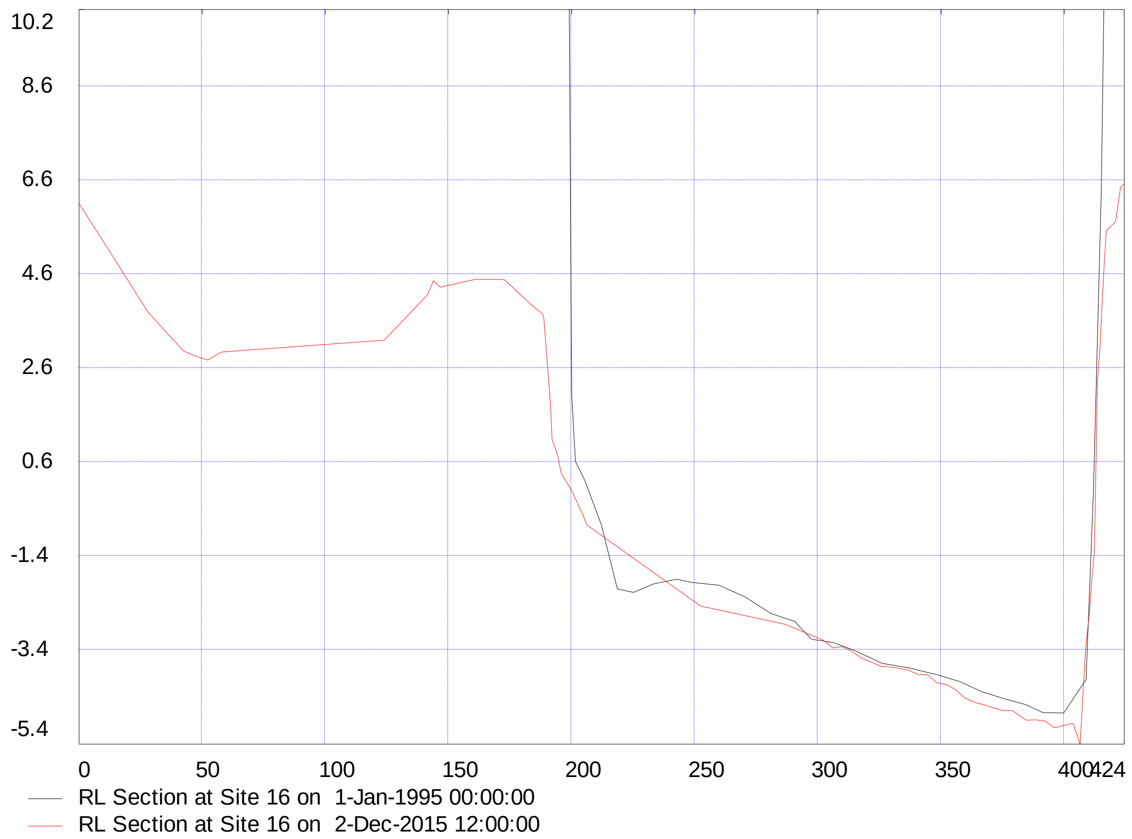
In the reach above Cobham Bridge there has generally been a lowering of the mean bed level. This lowering, up to 1 m in places, is much more pronounced than the changes in bed level seen in the lower reach of the river.

The changes in mean bed level are echoed in the changes in the area under each of the cross sections. That is, upstream of Cobham Bridge the area beneath each cross section has generally reduced since 1995. This is an interesting result as it means that the area of the river channel has increased over time suggesting that the flood carrying capacity of the river should also have increased.

This, however, is not backed up by the modelling results. As shown previously, in Figure 3.2, when the model was run with updated cross-sections the modelled water level was increased. The difference in the modelled flood levels is clearly explained by an examination of the modelled cross sections.

In the 2007 model the river was modelled as a channel with steep banks that would contain the water. The 'new' model, with 2015 surveyed cross sections, covered the full width of the river including the berms. This is illustrated by the overplot of the modelled cross sections at chainage 87740 in Figure 5.1. This cross section is found in the Kowhai Park reach of the river, between the Dublin Street and City Bridges, and is typical of the modelled cross sections through the City.

Figure 5.1- Cross Sections at MIKE11 Chainage 87740



As Figure 5.1 shows, at low water levels there is very little difference between the cross sections as previously discussed. However, when the berms are included in the model, the wetted perimeter of the cross section dramatically increases once berm flow begins. This increase in wetted perimeter means that the flood carrying capacity of the modelled channel is significantly reduced. This explains why the both the observed flood levels and the recalibrated modelled levels, are higher than those predicted by the original (2007) model.

The floods against which the original model was calibrated and verified did not include any significant berm flows. It is for this reason that the modelling approach of 'glass walling' the channel was appropriate. However, if the 'new' model was to be calibrated against the June 2015 flood, using a 'glass walled' channel, then the model would exaggerate the design flood levels for the larger design flows.

It is concluded that there does not appear to be a significant sedimentation problem in the Lower Whanganui River, indeed in many locations the capacity of the channel has increased since it was last surveyed in 1995.

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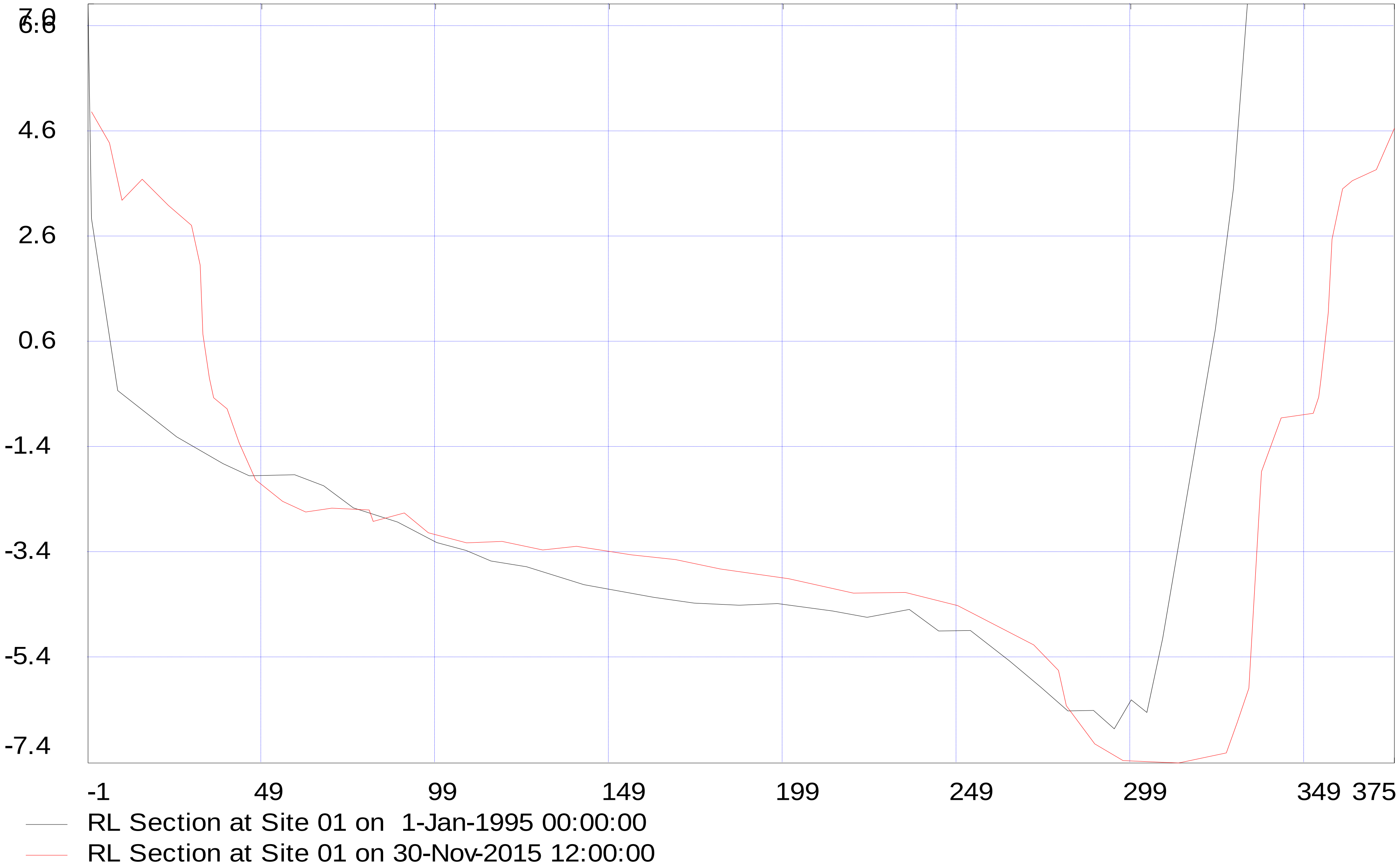
APPENDIX A

Whanganui River Cross Sectional Surveys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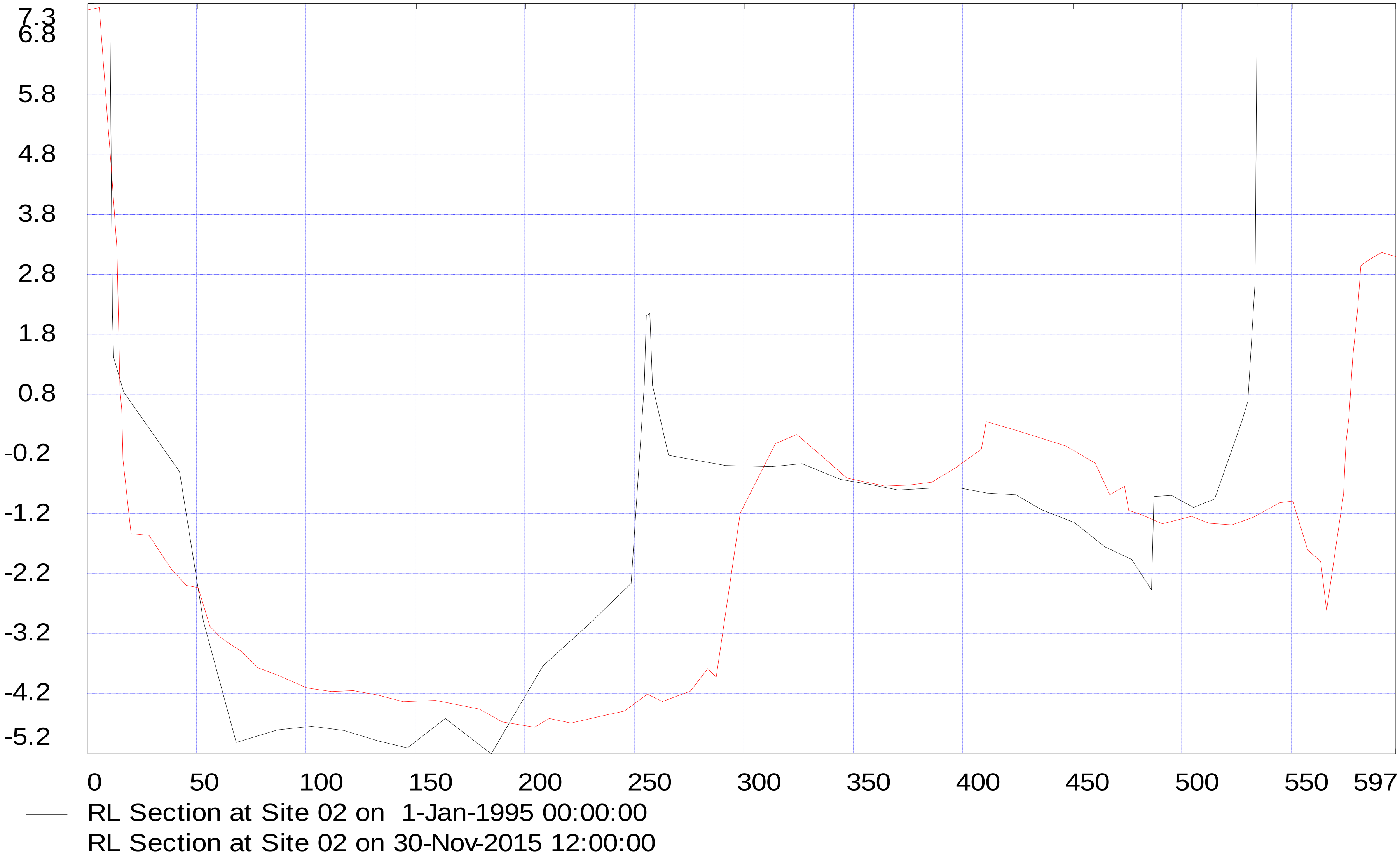
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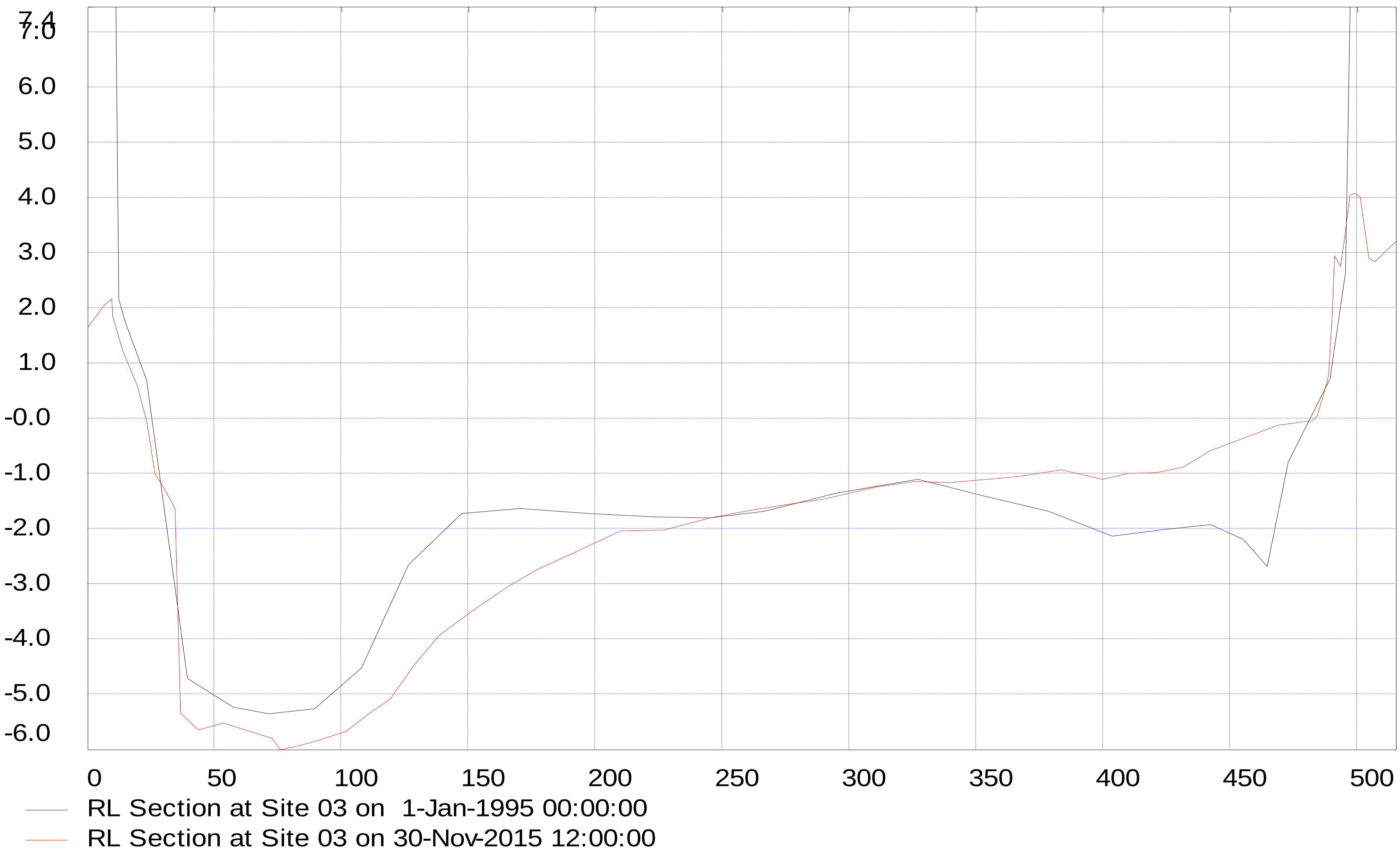
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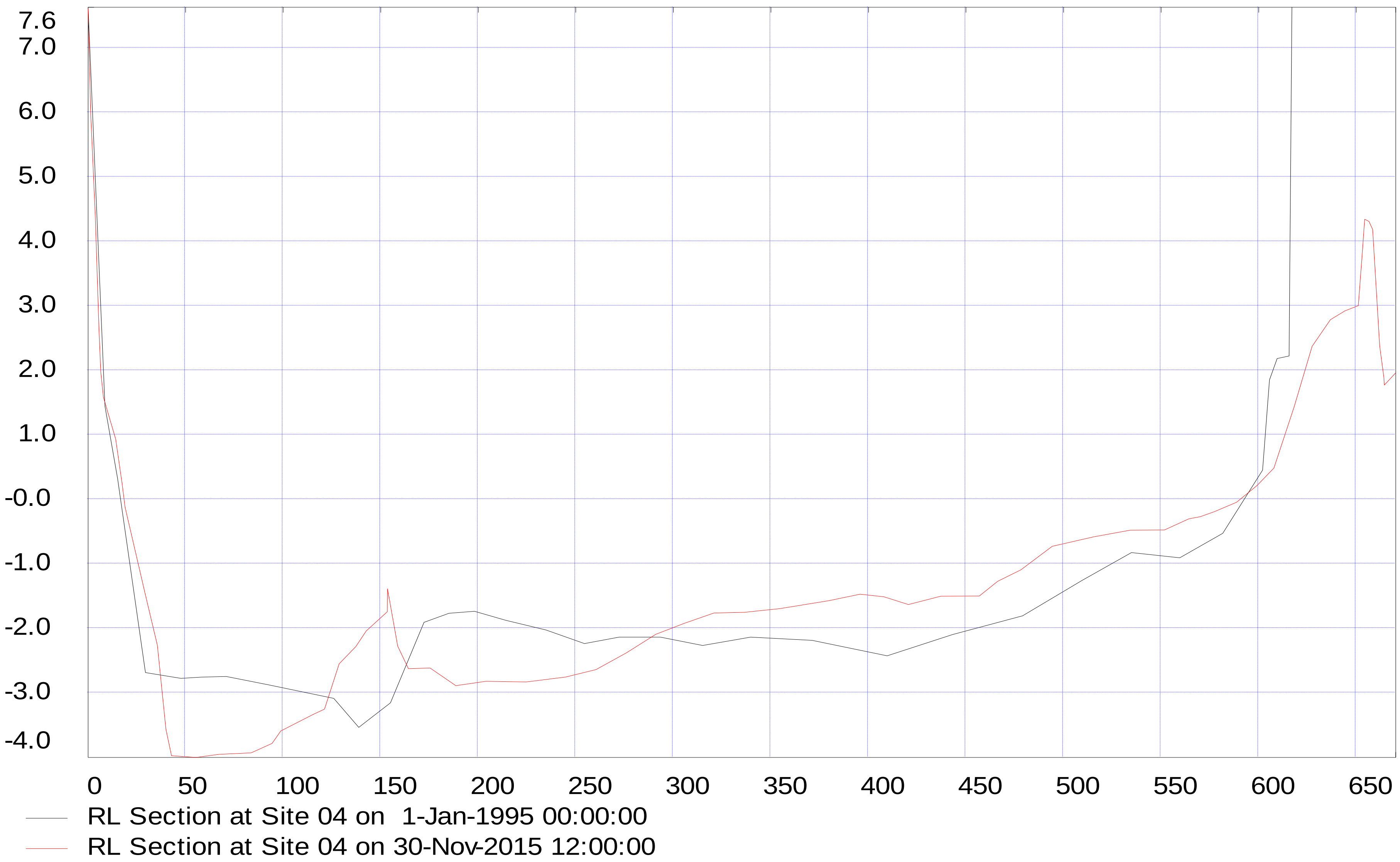

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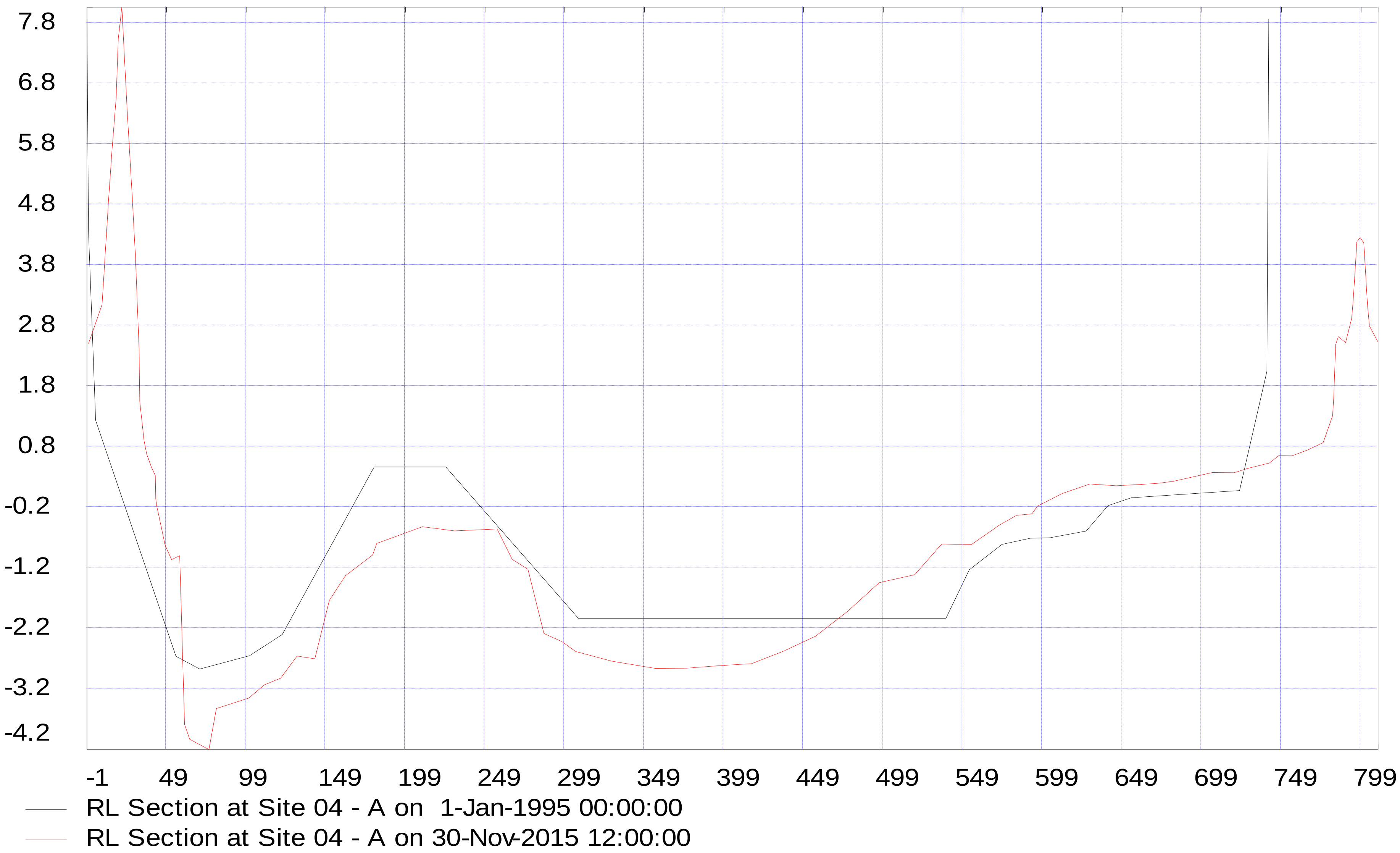
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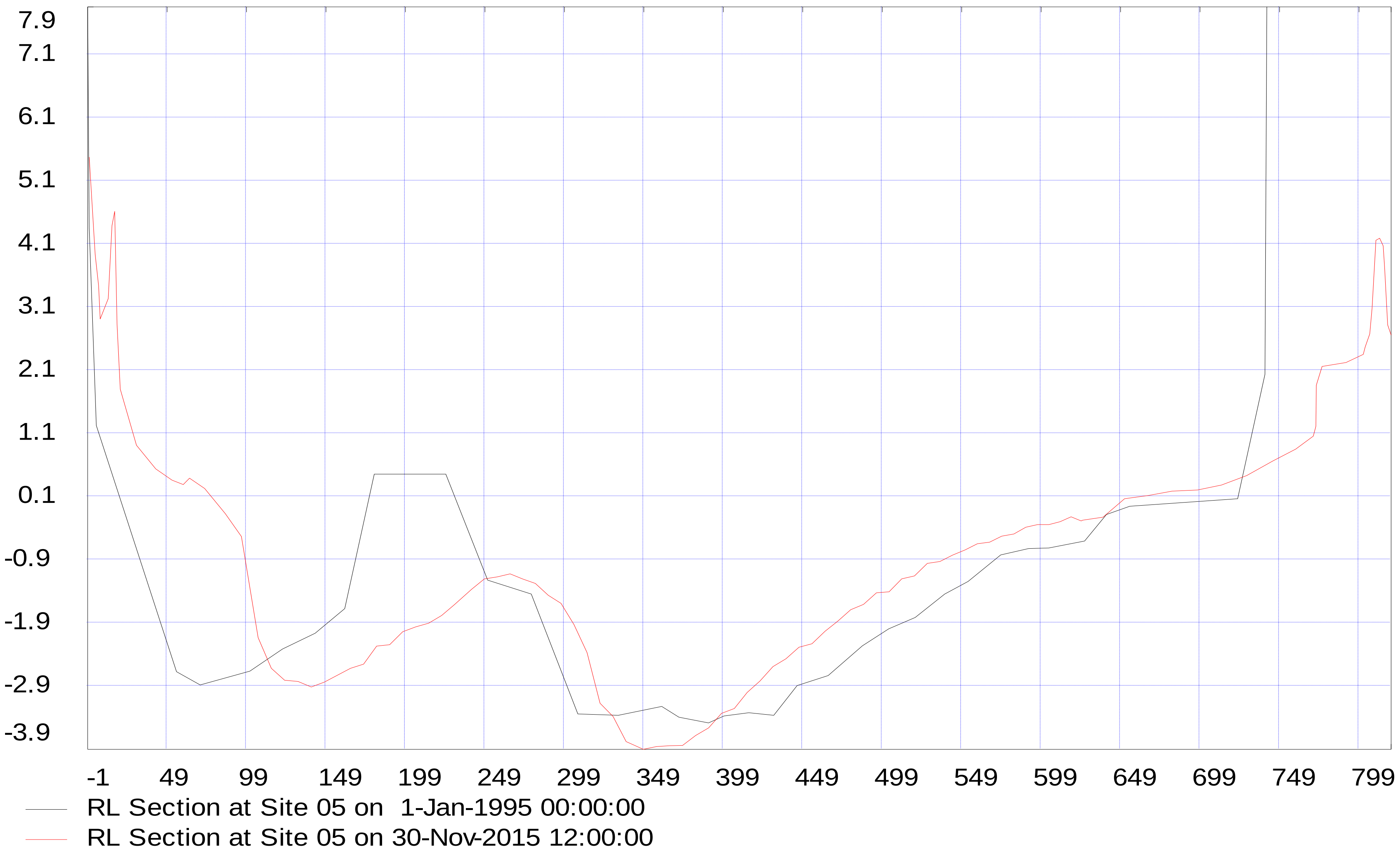
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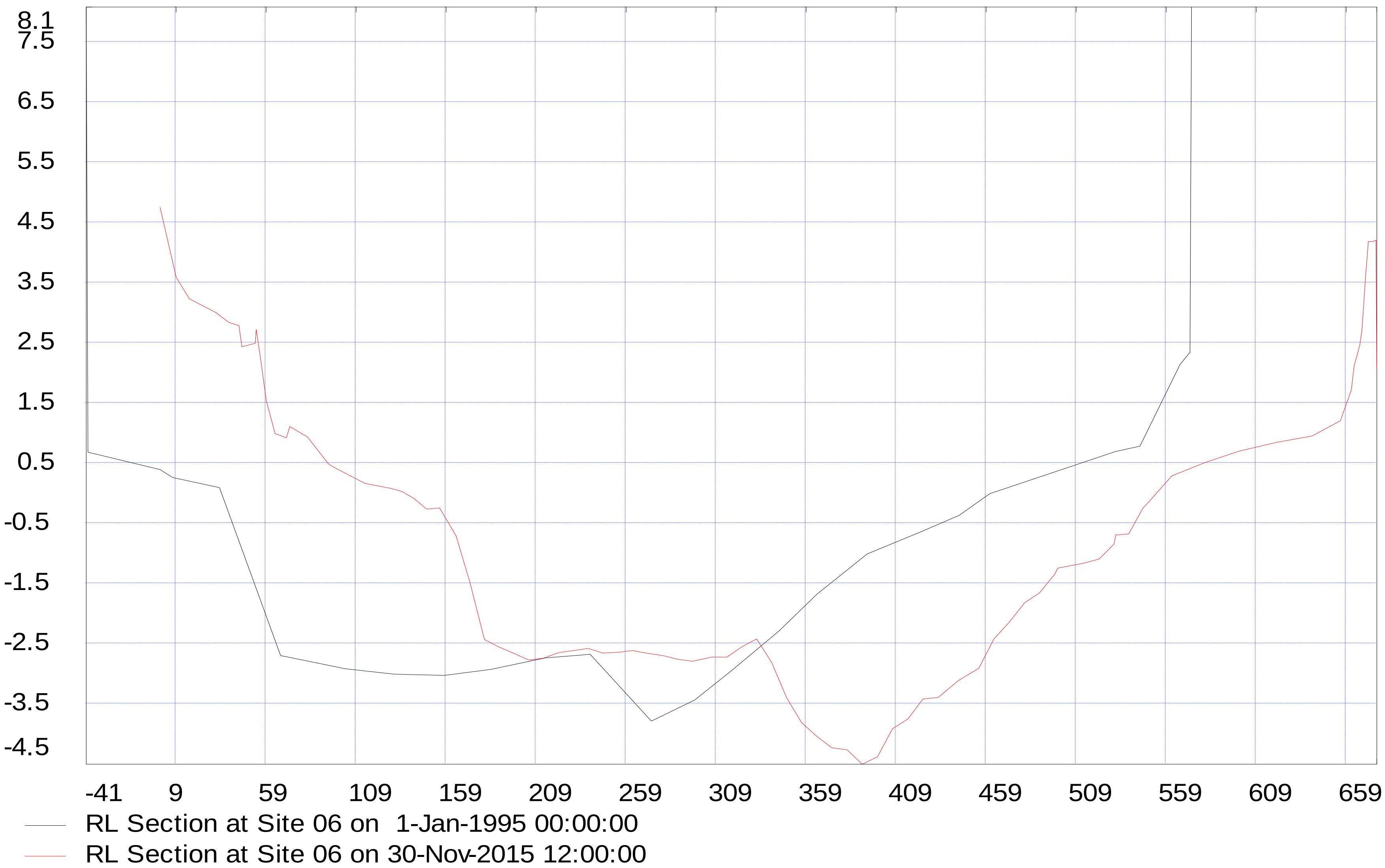
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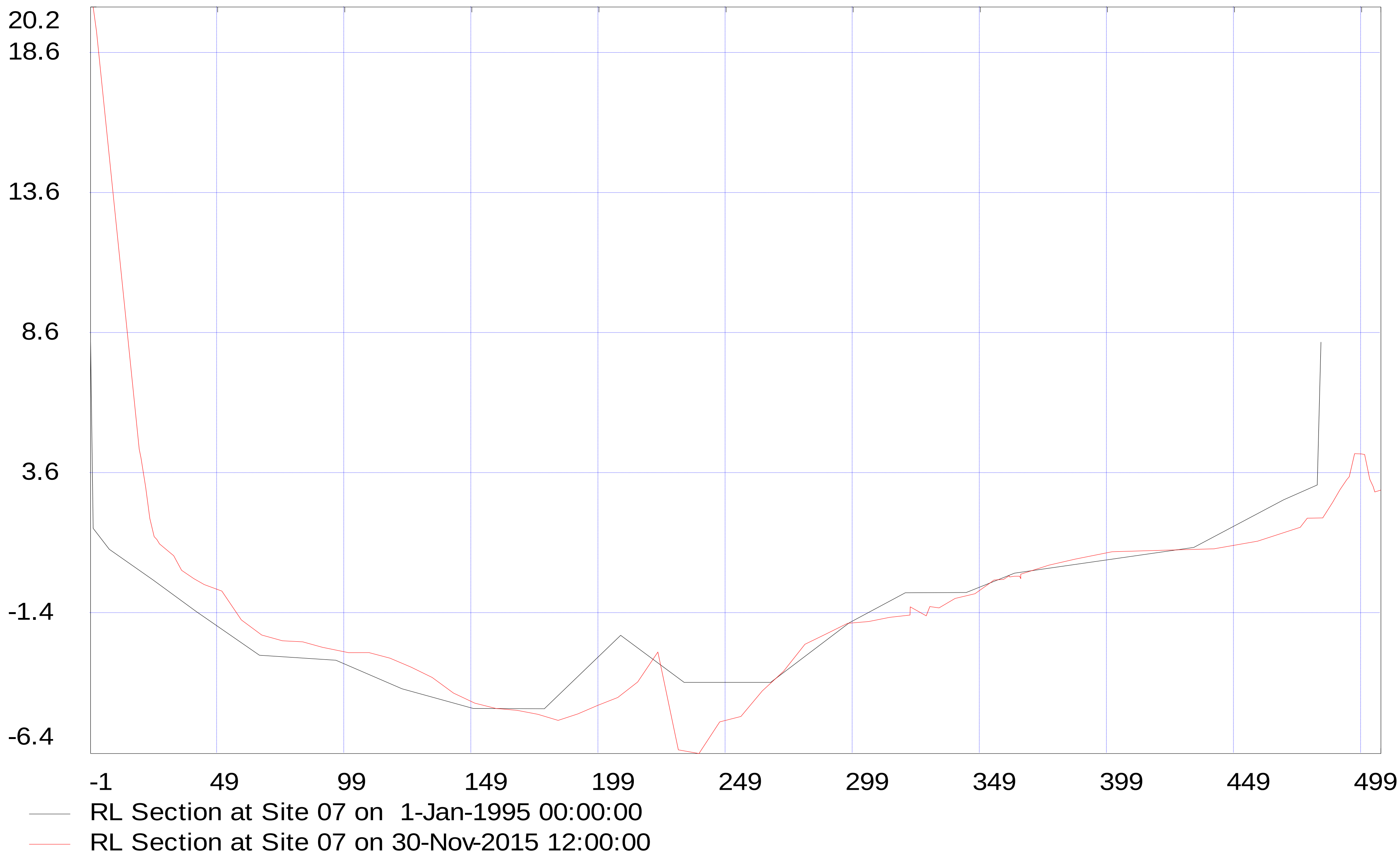
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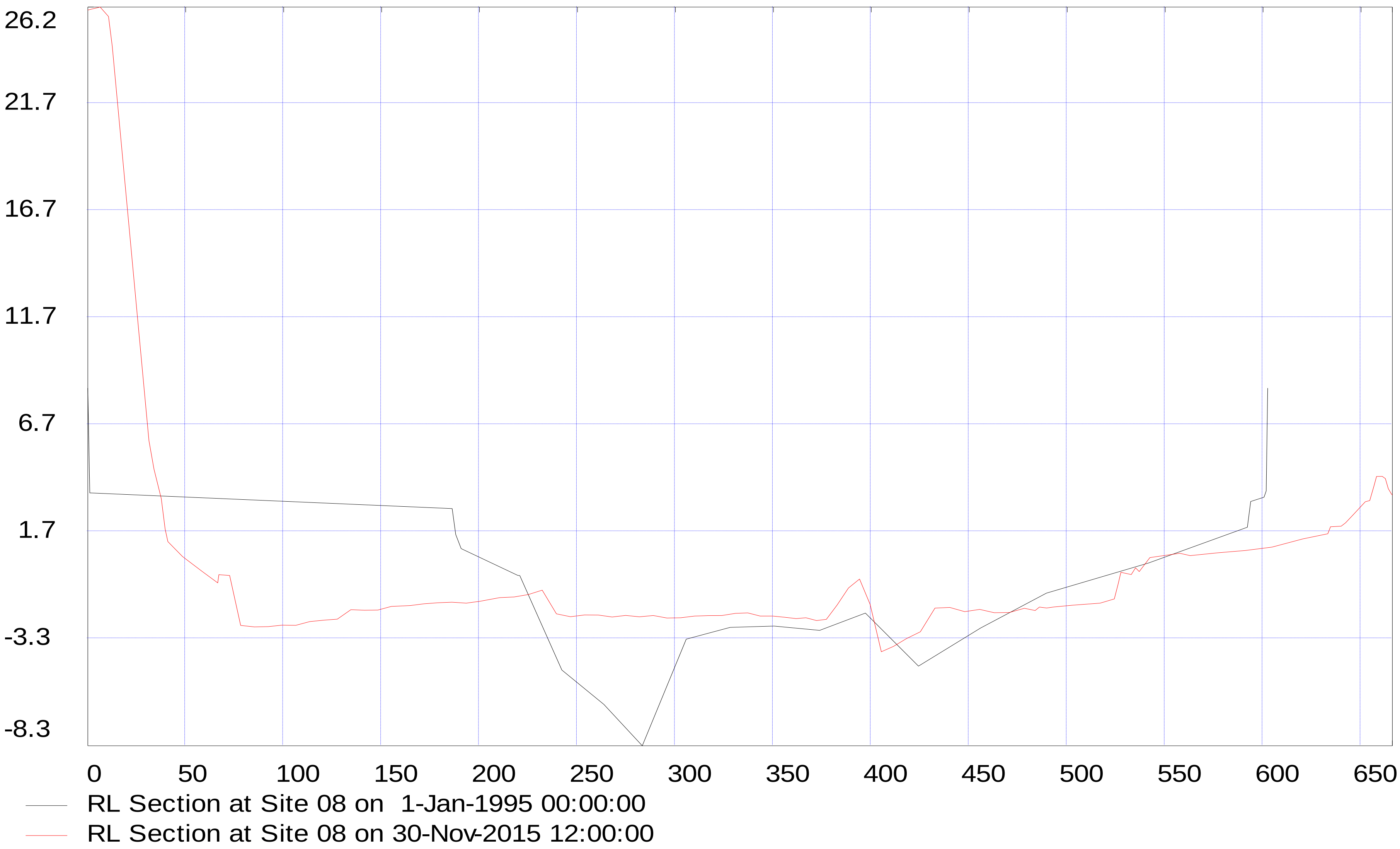
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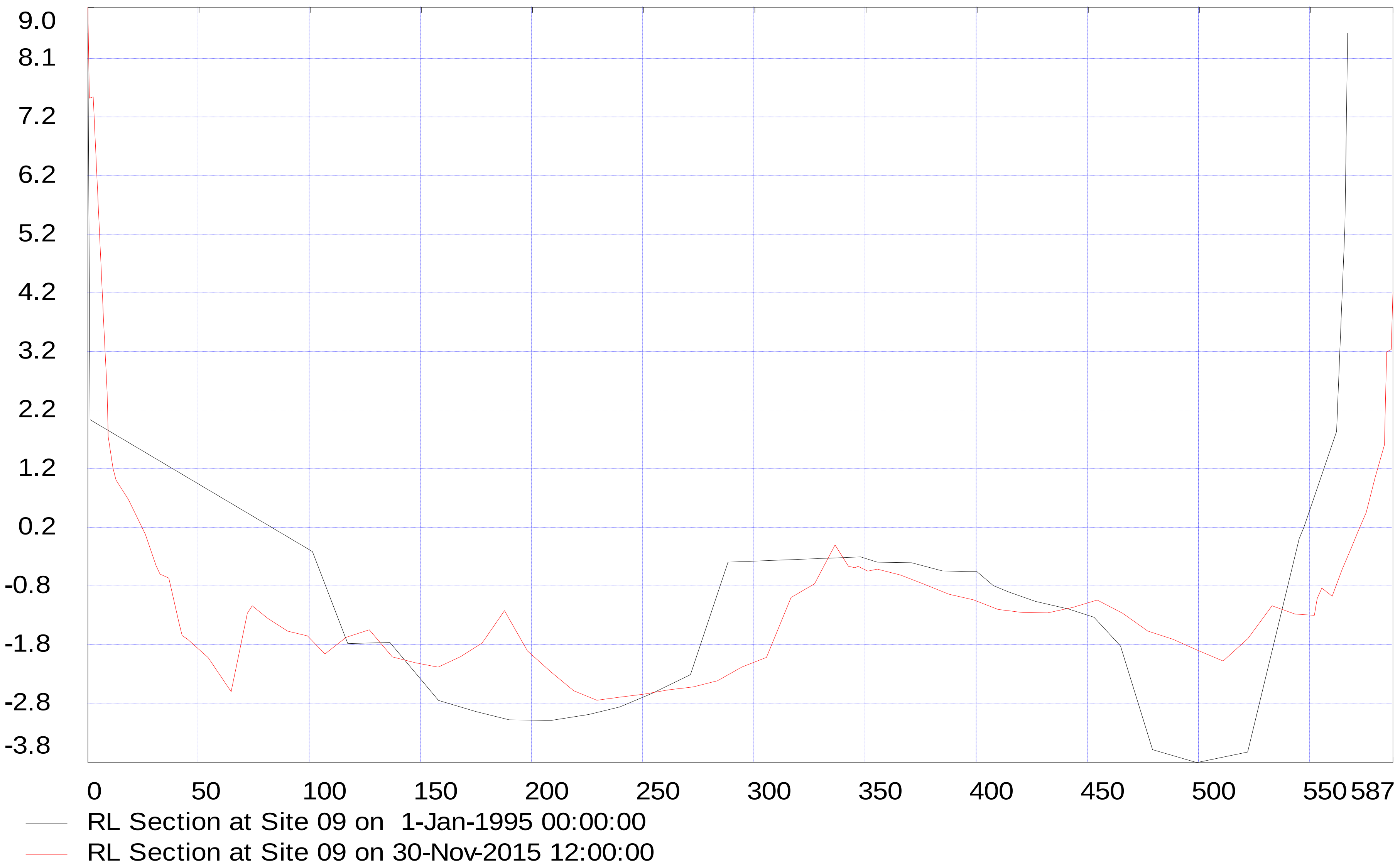
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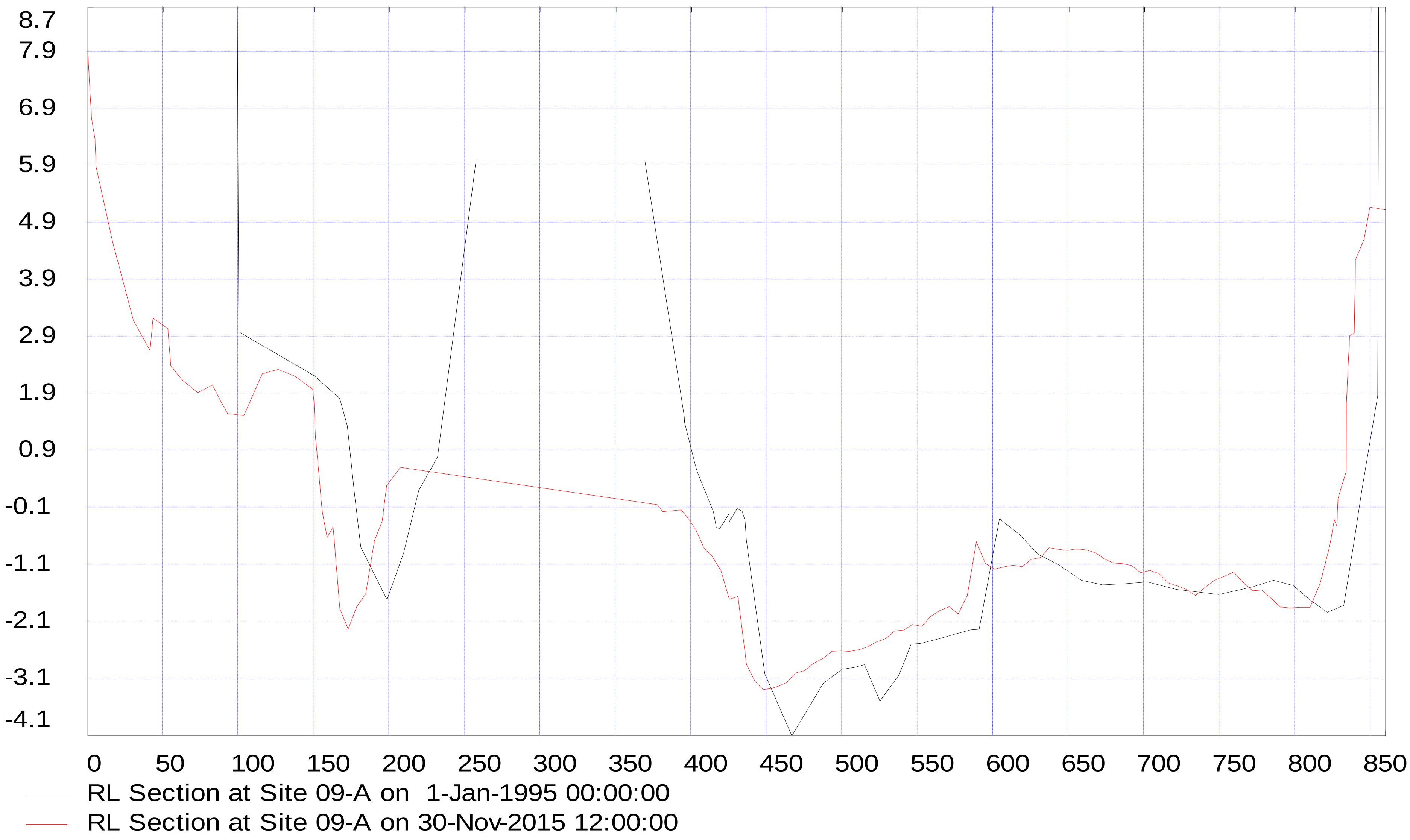
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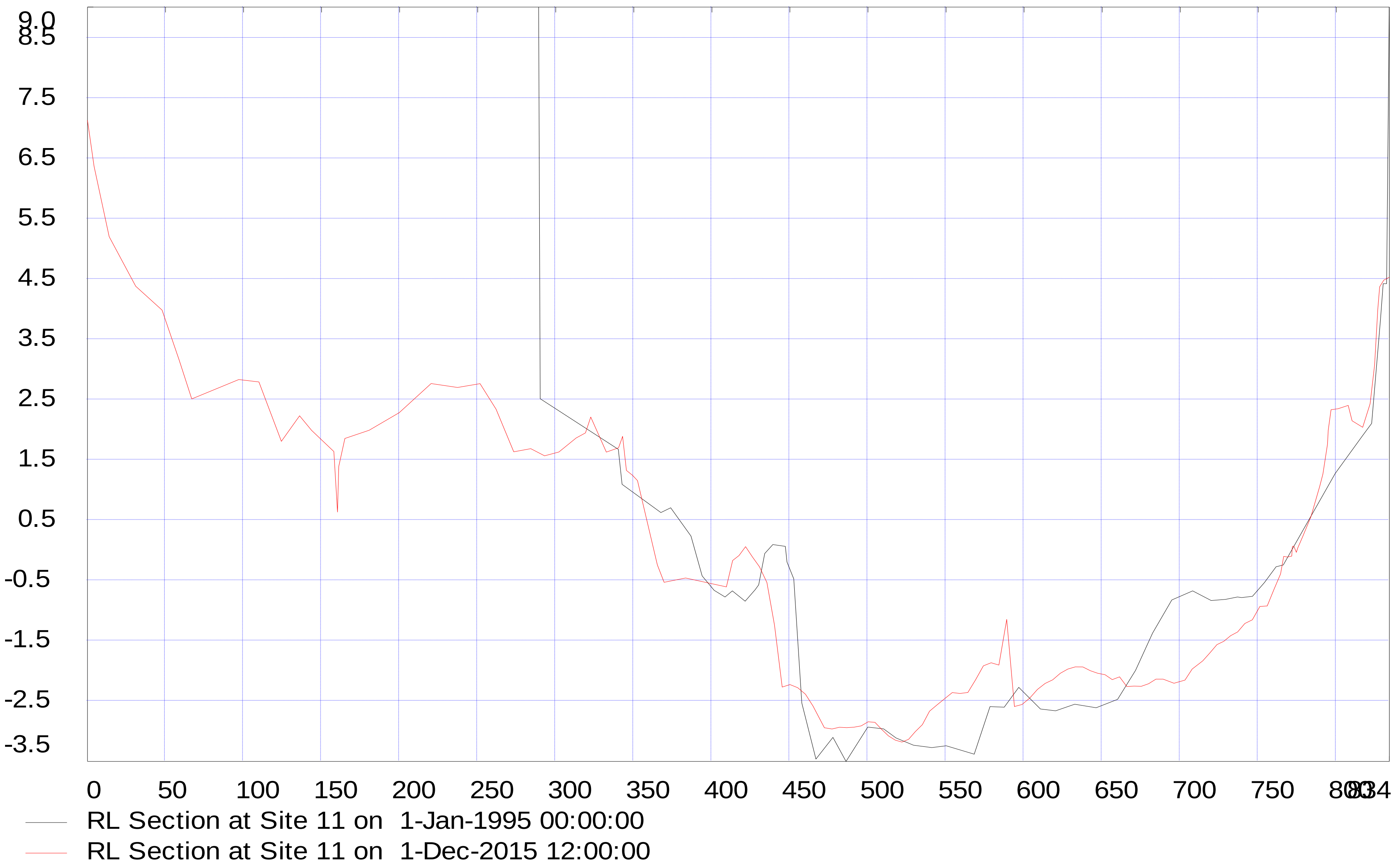
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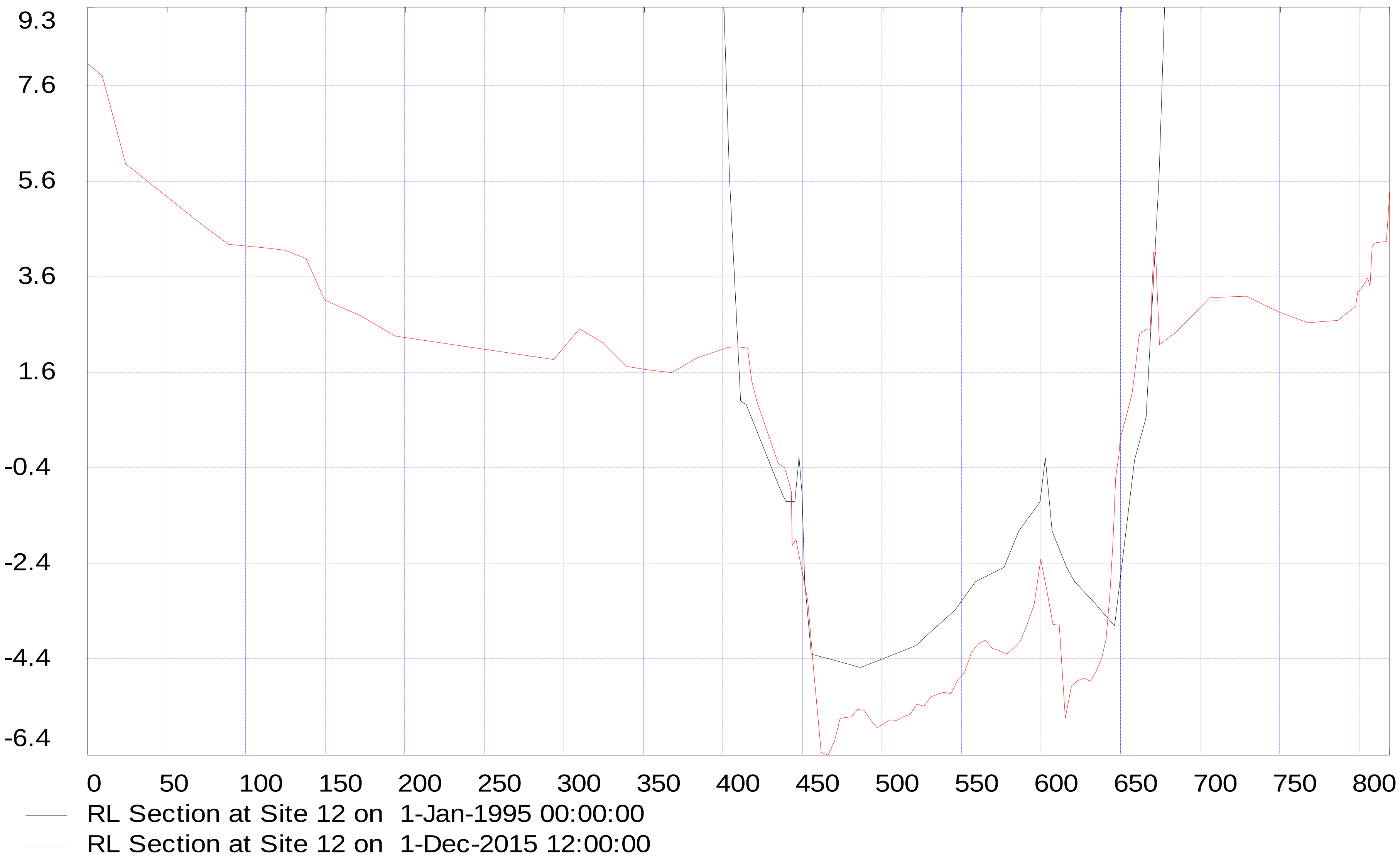
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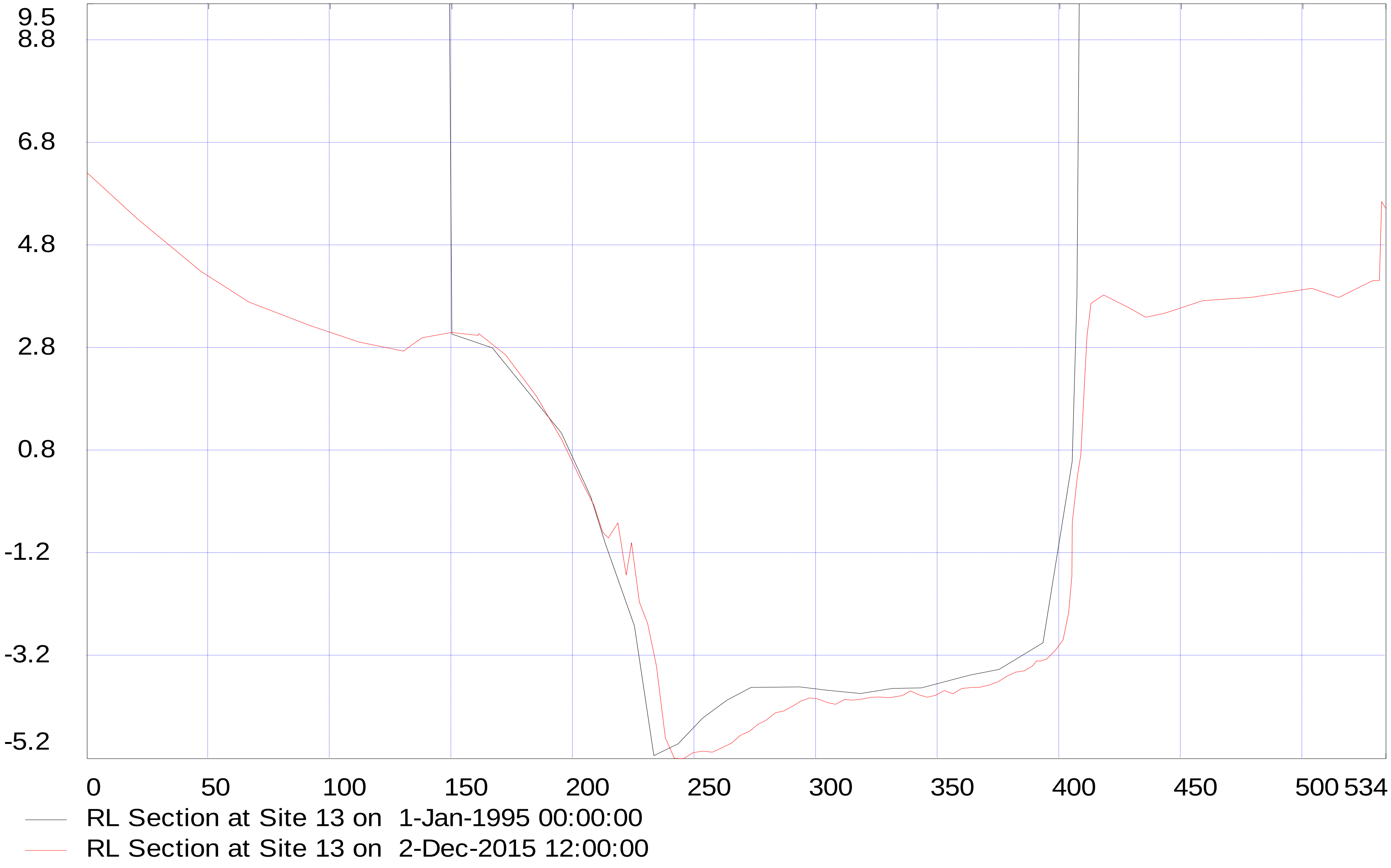

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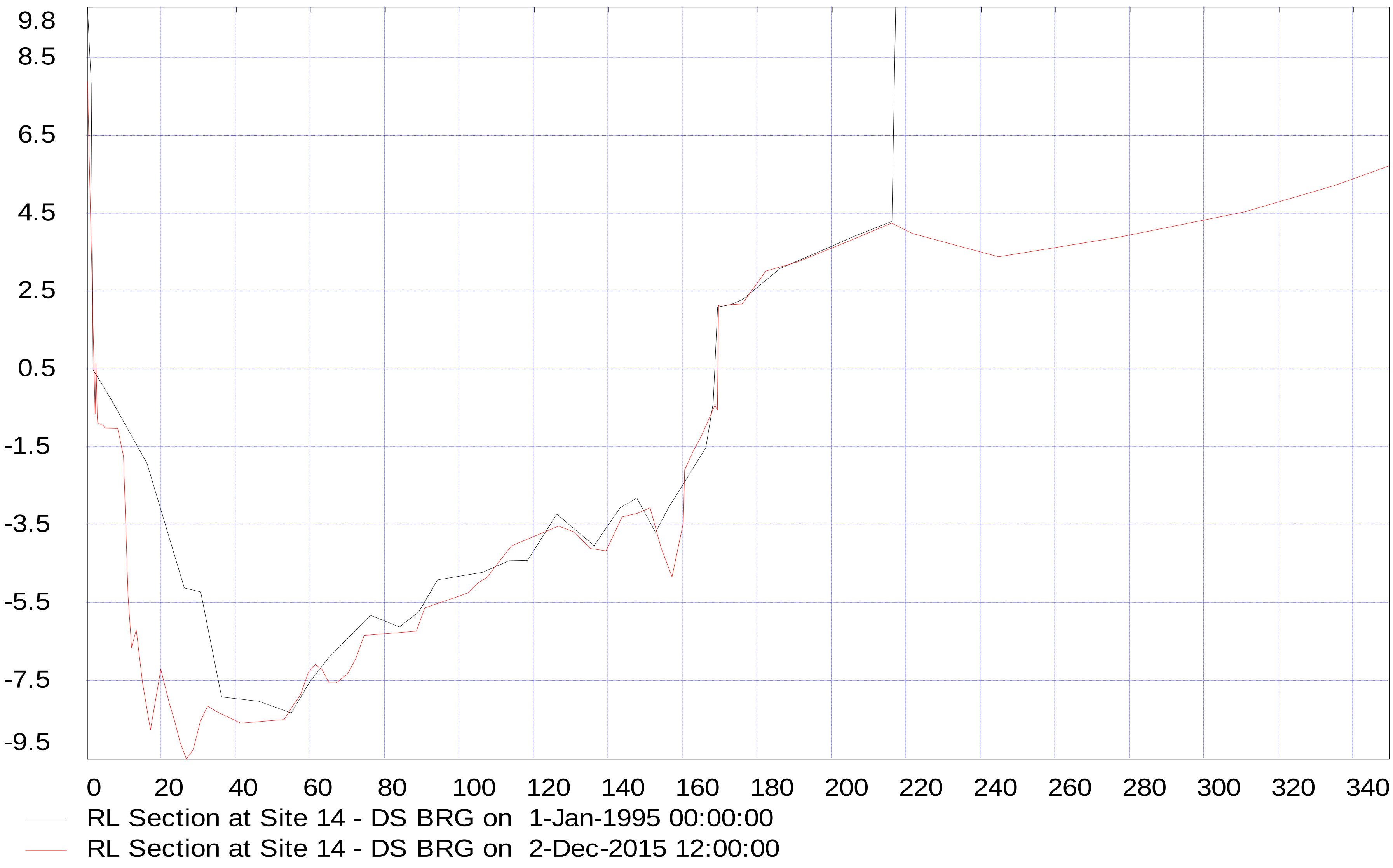
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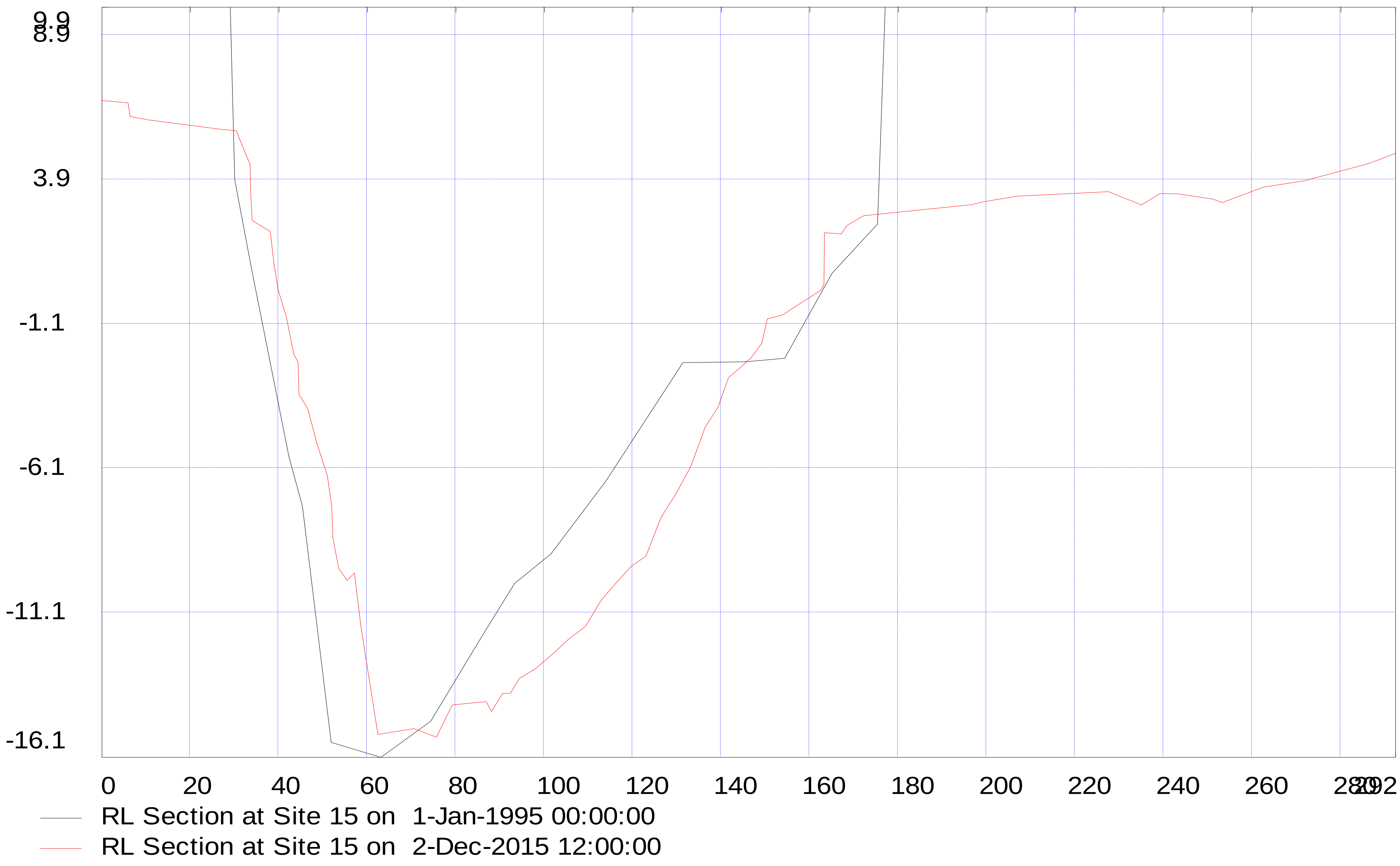
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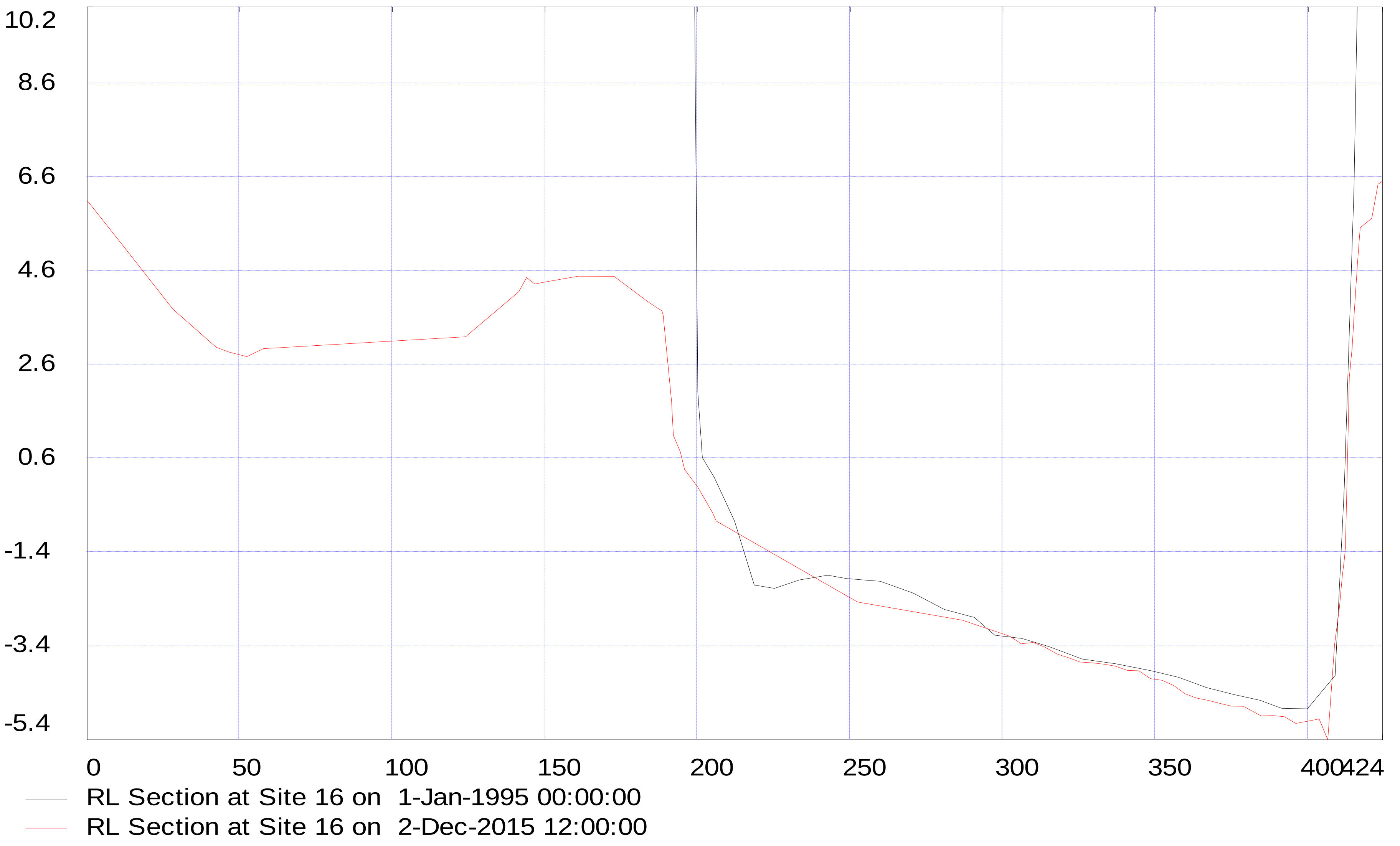
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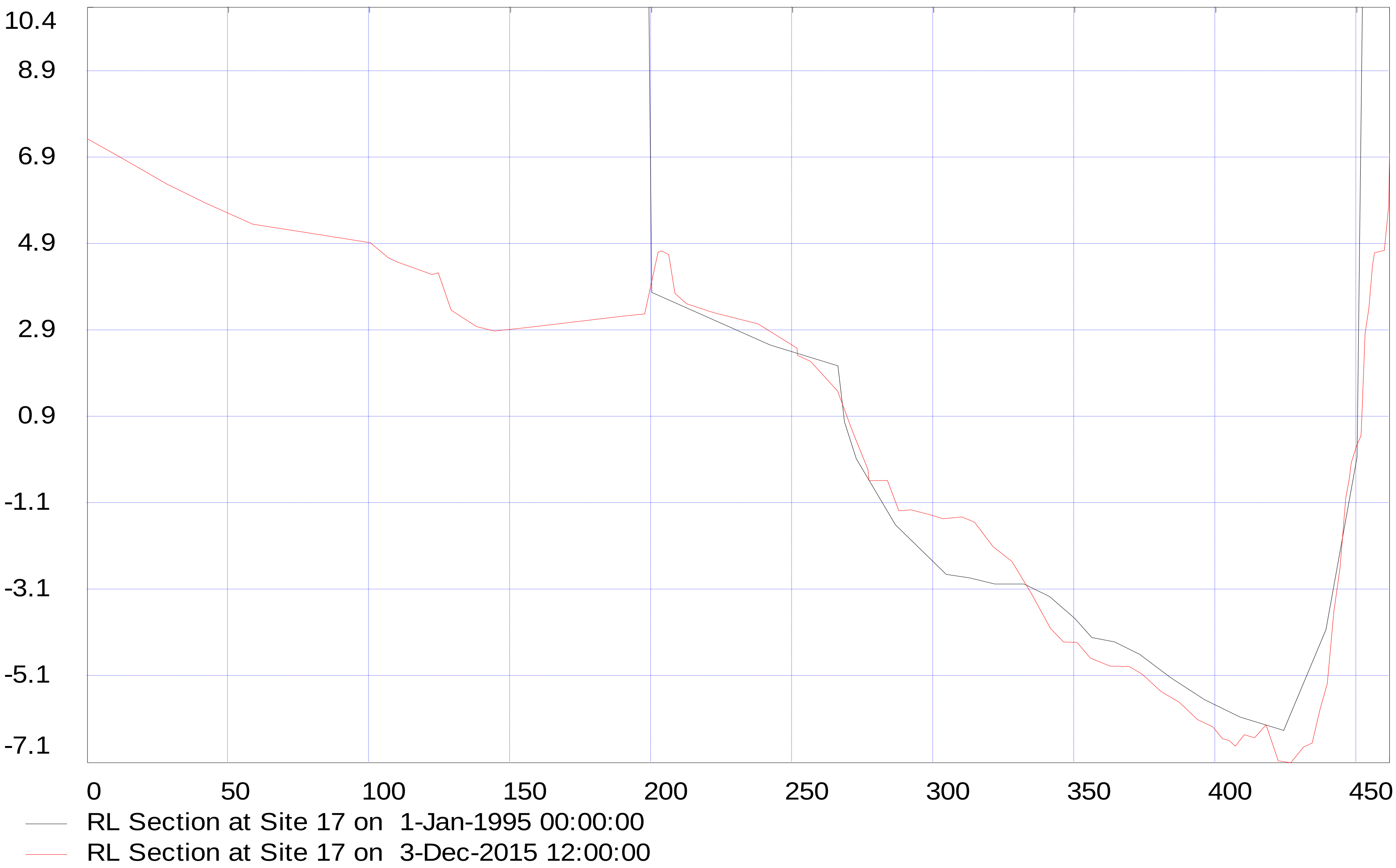

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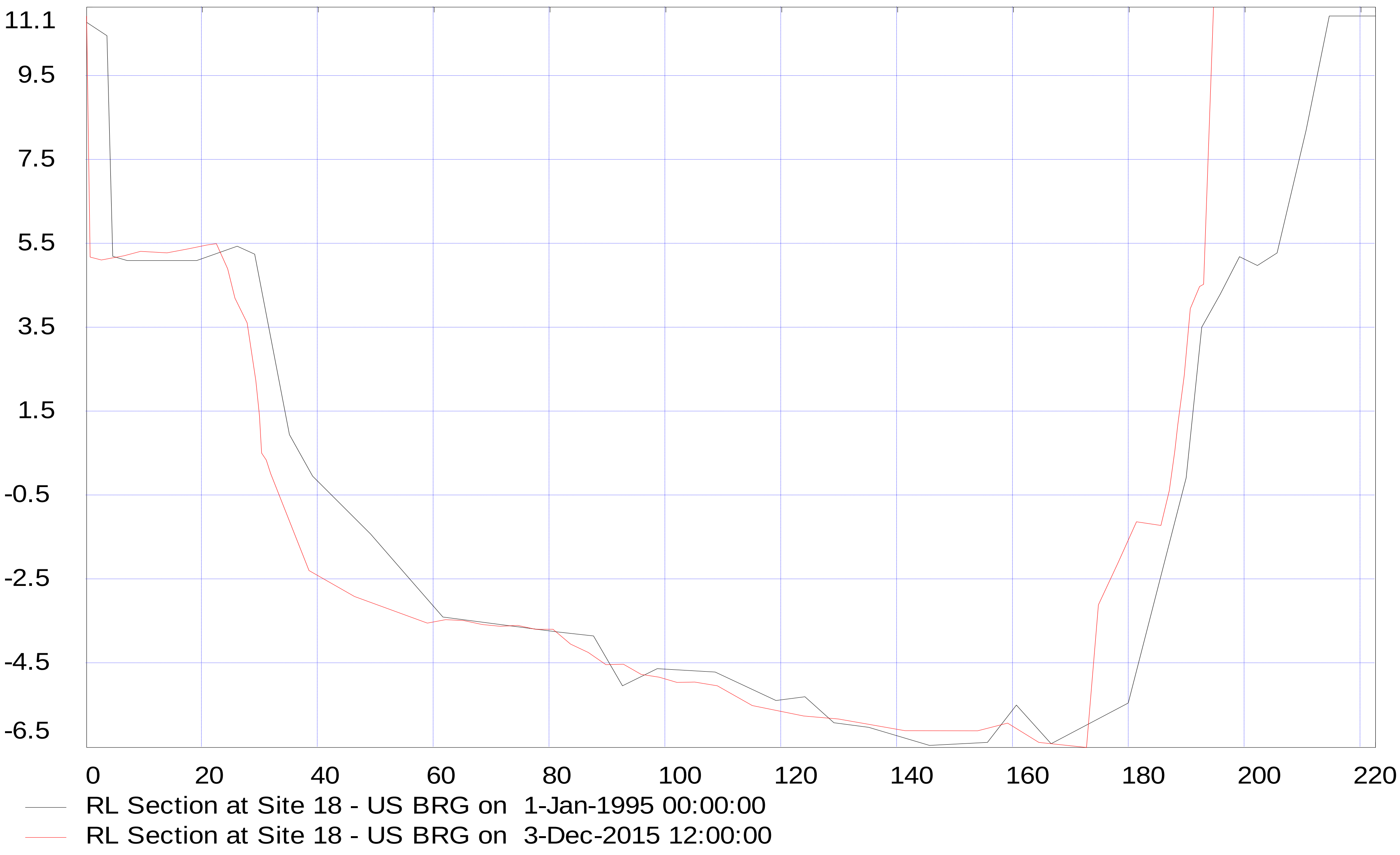
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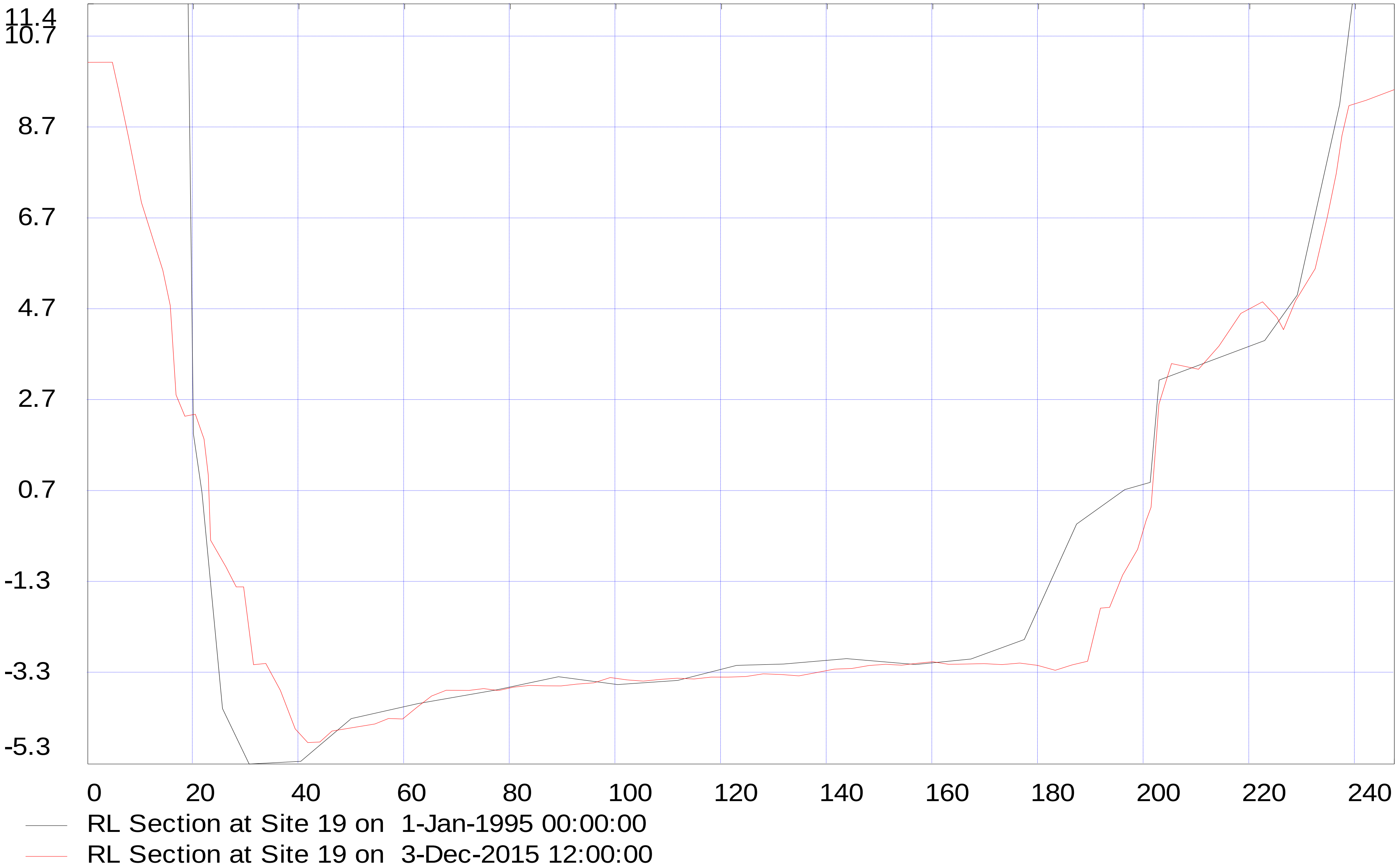
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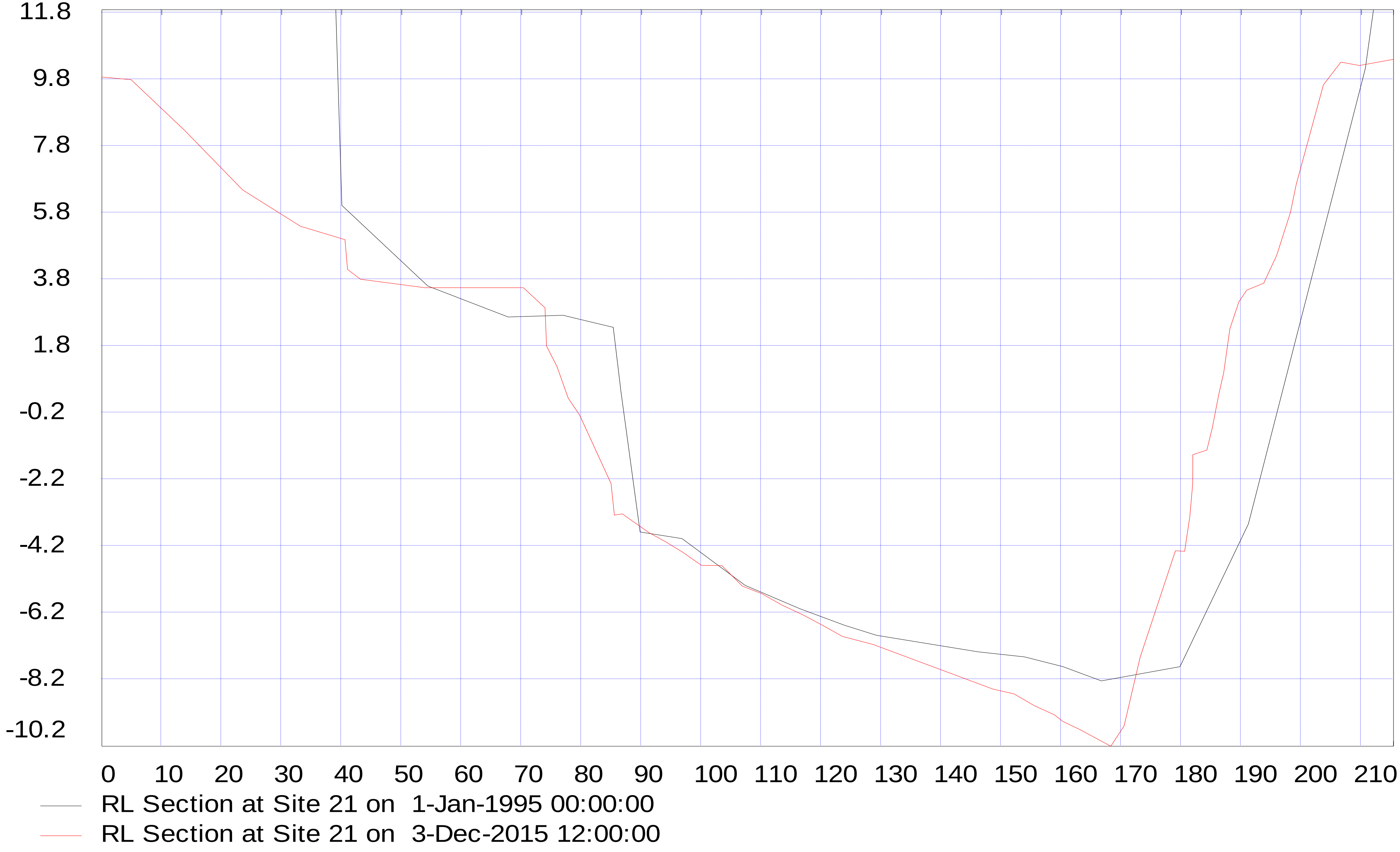
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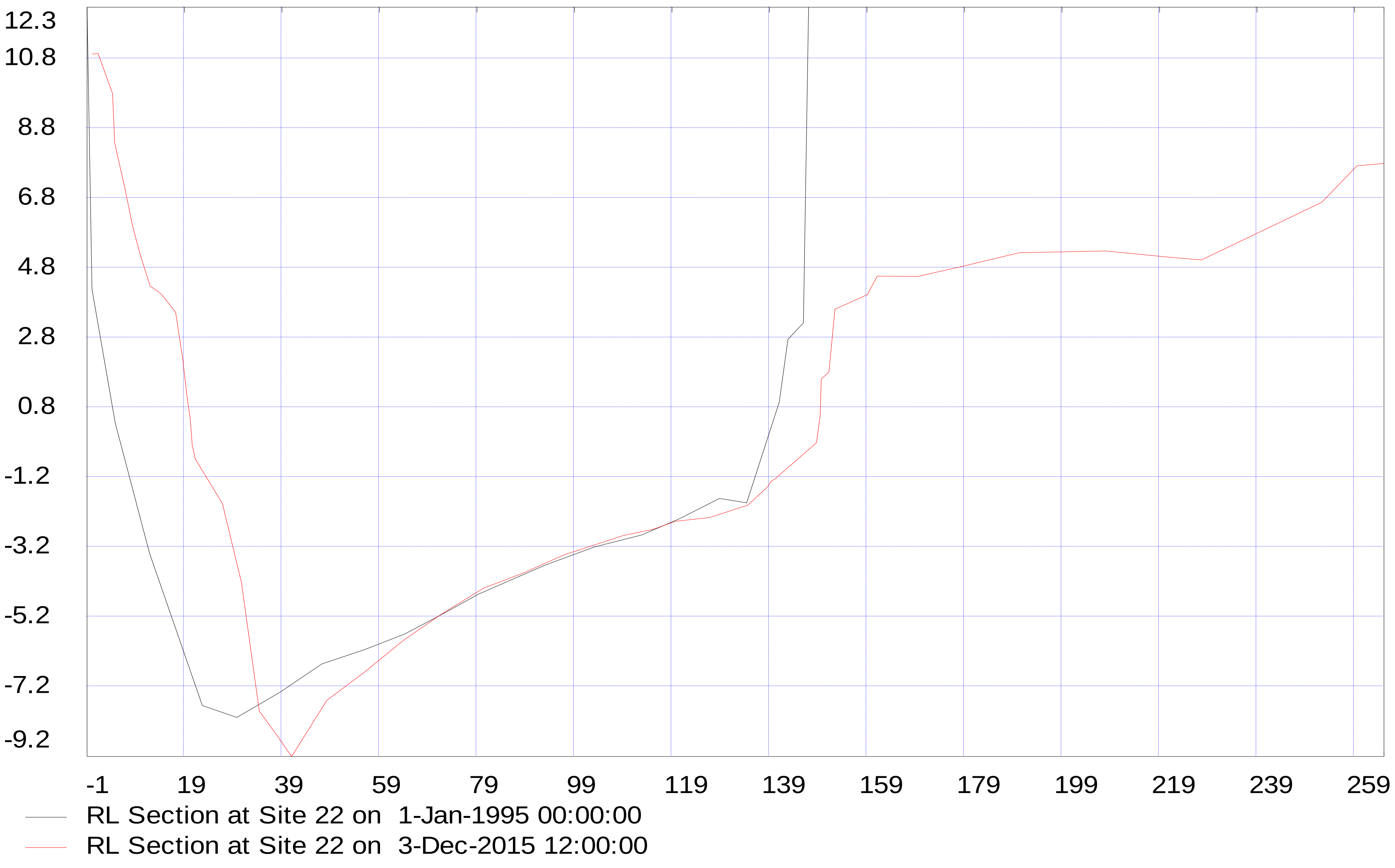
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