SERPENTINE, NICOMEKL & CAMPBELL RIVERS - CLIMATE CHANGE FLOODPLAIN REVIEW

FINAL REPORT

CITY OF SURREY
14245 -56 Avenue
Surrey, BC, V3X 3A2

December, 2012
NHC project file # 300014

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EXECUTIVE SUMMARY

Sea level has increased over the last century and is expected to rise at an accelerating rate over the next century. The City of Surrey (the City) retained Northwest Hydraulic Consultants Ltd. (NHC) to assess the potential for future flooding within the Serpentine and Nicomekl River drainage basins in view of the projected sea level rise (SLR). Detailed hydrological-hydraulic modelling investigations were carried out first to simulate present conditions (2010) and then repeated for conditions in 2100. Of particular interest were changes to the floodplain extents, the adequacy of existing dikes, as well as, the future functionality of spillways, sea dams and other structures.

The Serpentine and Nicomekl drainages, with a combined area over 300 km^2, are complex, containing over 100 km of dikes, two sea dams, some 30 pump stations, 170 floodboxes, spillways and a network of canals and ditches. Flooding in the upper basins is typically caused by heavy rain or rain-on-snow events, whereas high tides in combination with storm surge is the main source for flooding in the lower basins. Mid-basin, flooding can be caused by a combination of high runoff and high ocean levels. Recent severe flooding occurred in October 2003, December 2007 and January 2009.

For the Serpentine/Nicomekl River floodplain, inundation is a function of:

- The volume and temporal pattern of storm rainfall and the watershed's hydrologic response to rainfall;
- The time varying sea level in Mud Bay coincident with the storm event; and,
- The hydraulic response of the system (comprising storage and various hydraulic infrastructure) to the hydrologic inputs and the sea level boundary condition.

This complex system is not amenable to direct statistical analysis; i.e. it is not possible to state a priori with any reasonable confidence what combination of tidal conditions and storm rainfall event will result in peak floodplain inundation with an annual exceedance probability (AEP) of 0.5% equivalent to a return period of 200-years. To avoid the difficulties of a direct statistical joint probability analysis, a continuous simulation approach was adopted. A roughly 50 year-long historic record of local hourly rainfall data was assembled and used as input to a HSPF hydrologic model to produce time series of simulated historic runoff. Historical hourly Mud Bay water levels were developed for the same time period. The two time series were then used as boundary conditions for a hydraulic model of the river system, to simulate water levels at a number of locations in the watersheds for the same period. The simulated levels were then analyzed by conventional frequency analyses to estimate the frequency of water levels at each location.

The time-series approach indirectly took into account the joint probability of high tide levels, storm surge and wind setup. The Design Flood Level (DFL), Flood Construction Level (FCL) and Dike Crest Elevation (DCE) were computed at nine locations in Boundary Bay for year 2010 and 2100. Existing nominal sea dike crest elevations at sites assessed in this study range from 2.30 m to 3.30 m. DCEs using the joint probability approach vary from 3.51 m to 4.16 m for present conditions (0.53 m to 1.05 m above the existing crest elevations). For 2100 conditions, the DCEs vary from 4.74 m to 6.76 m (1.76 to 3.61 m above existing crest elevations) as shown in Figure 1. These levels are approximately 0.5 m less than the equivalent levels computed following Provincial Guidelines which

simply add the 200-year storm surge to the Higher High Water Large Tide. Based on the more sophisticated statistical analysis, the 200 year level estimated using Provincial guidelines actually has a return period of approximately 10,000 years.

Based on a review of recent scientific literature, the Province has adopted a value of 1.0 m sea level rise by the year 2100. Furthermore, portions of Surrey are undergoing subsidence which will also contribute to increasing flooding potential. Based on the available ground monitoring stations in the region, the local ground subsidence was estimated to average 0.225 m by year 2100.

The hydraulic modelling was repeated using the adopted year 2100 ocean levels. Future land use changes were taken into account to develop representative runoff time series for year 2100 conditions. Precipitation patterns were unaltered for this present study.

Compared to 2010 conditions, the 200-year flood level is expected to increase by 1.1 m by year 2100 in the area downstream of the sea-dike between the two rivers (see Figure 2). In the Nicomekl, upstream of the sea dam for 12 km, the 200-year flood level will increase by 0.9 to 1 m. In the Serpentine, upstream of the sea dam for 14 km, the 200-year flood level will increase by about 0.7 m. In the upper basins, the flood level increases taper to 0.1 m. Storage cells on the floodplain will see increases from 0.1 to 0.4m. The modelling assumed raised dikes, including sea dams, preventing overflow.

In the lower floodplain, the present 200-year flood level will have a return period of approximately 2 years in 2100. In the upper Nicomekl the present 200-year flood level will have a return period of roughly 110 years and in the upper Serpentine the present 200-year flood level will have a return period of roughly 30 years.

The ocean, hydrologic and hydraulic modelling had some limitations. There was limited calibration data available for the hydrologic model – the observed flows for the Nicomekl at 203 Street appeared to be inaccurate. The hydraulic model replaced the intricate network of drainage ditches with storage cells to achieve reasonable computing run-times. This resulted in averaged water levels over large areas. Water levels for hydraulic model calibration were limited and further development of the model is recommended in order to improve the accuracy of the results. Some uncertainty was also involved with extrapolating 200-year flood levels from the frequency analyses.

The results presented in this study are believed to accurately represent the potential increases in flood levels due to climate change. Compared to previous work, the simulated profiles are generally higher in the lower basins and lower in the upper basins. Additional analysis should be carried out before revising Flood Construction Levels.

A number of recommendations were made to improve data collection which will help refine the ocean, hydrologic and hydraulic models. Future work to refine the results include completing modelling for 2030, 2050 and 2070 and extending the modelling to 2200. Future work can also incorporate the effects of climate change on precipitation and evaporation.

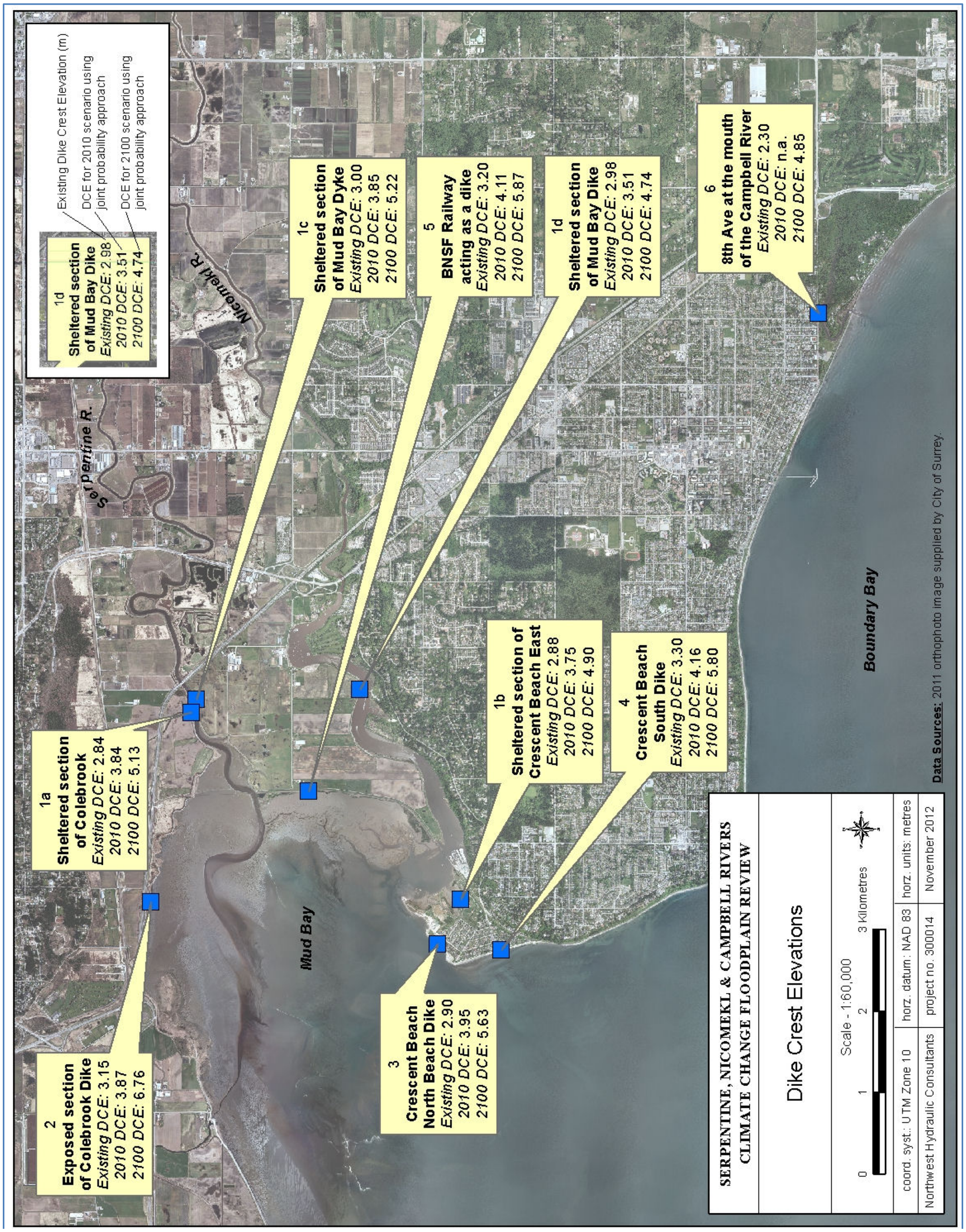

Figure 1. Dike crest elevations

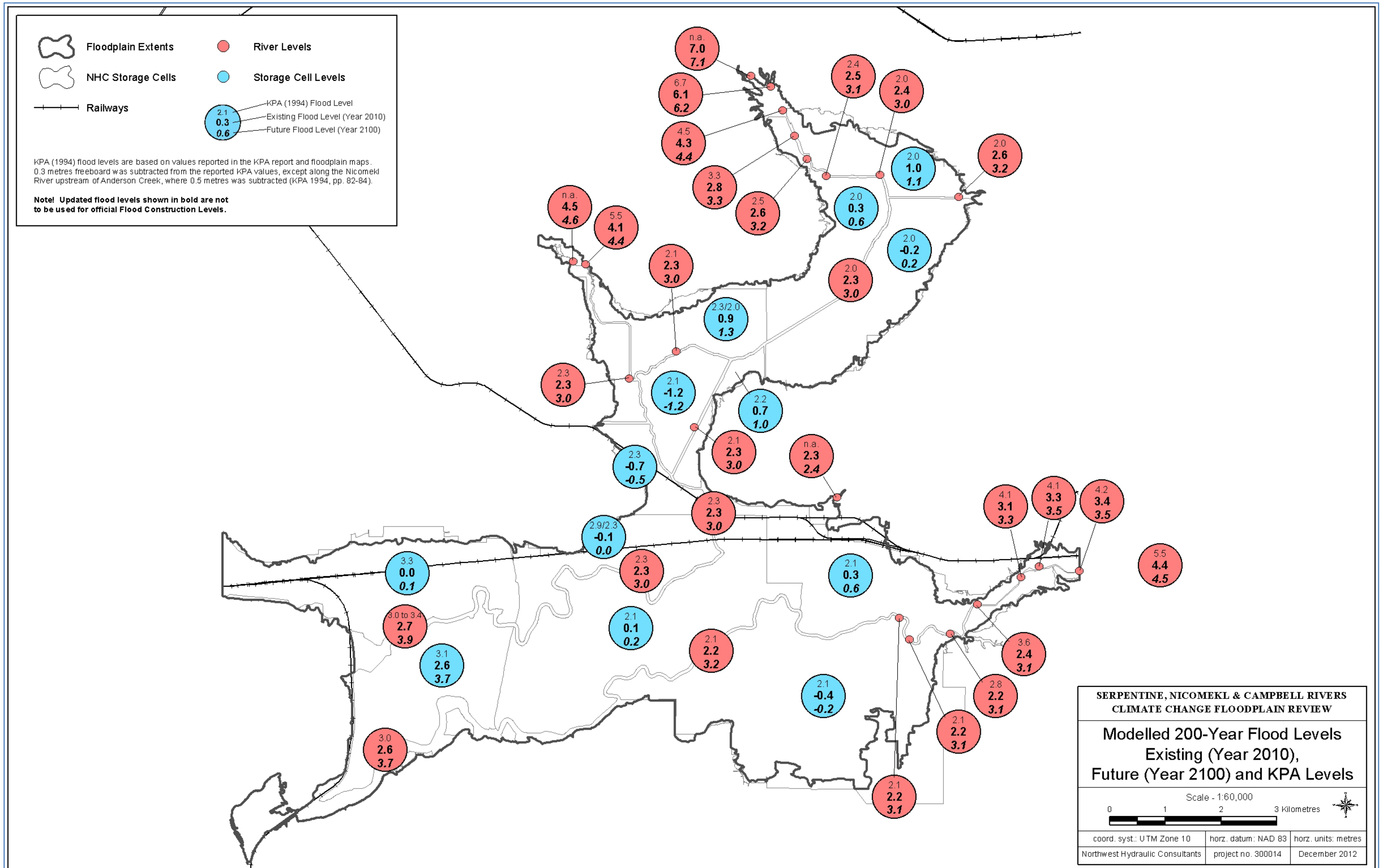

Figure 2. Modelled 200-year flood levels – existing (2010), future (2100) and KPA levels

Credits and Acknowledgements

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- Matthew Osler, PEng. — Project Manager
- Carrie Baron, PEng. — Drainage & Environment Manager

The following Northwest Hydraulic Consultants Ltd (NHC) personnel participated in the study:

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF MAPS

List of Appendices

1 INTRODUCTION

1.1 SCOPE OF WORK

Recent studies on sea level rise due to climate change (Thomson et al 2008, Ausenco Sandwell, 2011) indicate there will be significant impacts to coastal British Columbia over the next century. Based on a review of scientific literature, global sea level rise (SLR) from the year 2000 was estimated to be 1 m by the year 2100 and 2 m by 2200. The City of Surrey (the City) retained Northwest Hydraulic Consultants Ltd. (NHC) to assess potential flooding within the Serpentine and Nicomekl River drainages resulting from projected future sea level rise in terms of floodplain extents, adequacy of existing dikes and functionality of spillways, sea dams and other structures. This study was undertaken to support the City of Surrey in understanding, managing and responding to the combined impacts of sea level rise and ground subsidence. In addition to direct flood damage to residential and commercial development, high water levels may affect infrastructure such as emergency access/egress, sanitary lift stations, potable water supply and electrical control boxes (Baron 2011).

The focus of this study was on probable impacts up to year 2100. The present work forms the first phase of a multi-phase project and involved the following key tasks:

- Compile and review relevant existing information;
- Assess modelling needs and outline appropriate modelling approaches and applications;
- Develop a statistically sound method for representing the joint occurrence of extreme ocean levels and precipitation events in the Serpentine and Nicomekl River basins;
- Develop necessary coastal, hydrologic and hydraulic models and calibrate/validate these to observed data;
- As part of the ocean analysis, determine ocean levels at the mouth of the Campbell River outlet.
- Analyze climate change impacts using the models developed; and,
- Summarize results and deliver all relevant models and data files to the City.

1.2 PROJECT GOALS AND OBJECTIVES

The main goal of the study was to assess the effects of the projected net sea level rise on the Serpentine and Nicomekl floodplain in order to develop appropriate future design standards for the area. To accomplish this goal, a key component was to develop a robust, scientifically sound approach for defining the probability of occurrence of extreme flood events due to the combination of heavy precipitation, high tides plus storm surge.

Specific objectives outlined in the City Terms of Reference included:

- Determine future projected sea levels in Mud and Semiahmoo Bays consistent with the Climate Change Adaption Guidelines for Sea Dikes and Coastal Flood Hazard Land published by the Ministry of Environment;
- Determine future projected sea levels through statistical analysis of various duration tidal patterns, storm surges and wind/wave setup;
- Analyze the magnitude and timing of combined surge, wind setup/wave setup, sea level rise, tidal patterns and precipitation to assist in future policy/design standard development and establishment of model boundary conditions;
- Assess the impact of projected future sea levels on the lands upstream and downstream of the existing sea dams; and,
- Compare existing floodplain levels with results of the sea level impact assessment for year 2100.

1.3 APPROACH

For the Serpentine/Nicomekl River floodplain, inundation is a complex function of:

- The volume and temporal pattern of storm rainfall and the watershed's hydrologic response to rainfall;
- The time varying sea level in Mud Bay coincident with the storm event; and,
- The hydraulic response of the system (comprising storage and various hydraulic infrastructure) to the hydrologic inputs and the sea level boundary condition.

This complex system is not amenable to direct statistical analysis; i.e. it is not possible to state a priori with any reasonable confidence what combination of tidal conditions and storm rainfall event will result in peak floodplain inundation with an annual exceedance probability (AEP) of 0.5% equivalent to a return period of 200-years.

To avoid the difficulties of a direct statistical approach to joint probability analysis, a continuous simulation approach was adopted where long-term (approximately 50 year) simulations were conducted of the systems hydraulic performance, and the simulated annual peak floodplain water levels were subject to conventional frequency analysis. The approach involved the following:

1. A long continuous time series of historic local hourly rainfall data were assembled. This time series was based on the rainfall record from Surrey Municipal Hall which extends back to 1963, or close to a 50-year record.
2. The rainfall time series was used as input to a continuous hydrologic simulation model of the Serpentine and Nicomekl watersheds to produce long (approximately 50-year) time series of simulated historic runoff from the two watersheds.
3. A long time series of historical hourly Mud Bay water levels was developed coincident in time with the time series of simulated runoff on the Serpentine and Nicomekl.

4. A hydraulic model of the system was developed to conduct continuous simulations of the water levels in the combined Serpentine/Nicomekl floodplains with input from the hydrologic simulations of step 2) and external boundary conditions imposed by the Mud Bay water levels of step 3). The hydraulic model produced a long (approximately 50-year) time series of simulated water levels at key points in the floodplain for the historic sea level and rainfall regime.
5. Annual maximum water levels at the key locations were extracted from the hydraulic model results. These were analyzed by conventional frequency analysis to estimate 200-year (0.5% AEP) floodplain water levels at the specified locations.
6. The hydrologic model was then modified to reflect projected land-use changes and a future (year 2100) runoff time series was developed as in Step 2). A time series of sea levels under climate change were developed by adjusting the historic sea level time series developed in Step 3). Ground subsidence was incorporated and Steps 4) and 5) were repeated to determine the 0.5% AEP floodplain water levels for the assumed sea level rise scenario (year 2100).

This proposed continuous simulation approach provides a number of significant advantages:

- It explicitly captures the joint occurrence of extreme sea levels and severe rainfall events;
- It explicitly accounts for varying duration and amounts of rainfall (and runoff) and the matching of the rainfall with the tidal regime;
- It captures the shift in significance of longer lower intensity rainfall events under condition of sea level rise. (Higher sea level implies that longer duration rainfall events become more important in defining interior flood levels); and,
- It avoids arbitrary decisions about the coincidence or lack of coincidence of individual factors which would be required if a direct statistical analysis were attempted.

Some key assumptions of the approach are that:

- The joint occurrence of extreme sea levels and severe rainfall contained in the historic record will be maintained in the future; and,
- Future sea level time series can be adequately constructed by simply increasing all water levels by a uniform amount and scaling storm surges contained in the historic record.

1.4 Report Organization

In addition to this introductory Section 1, Section 2 provides a description of the site, background data and history of past flood events, Section 3 presents climate change scenarios to be considered and Section 4 a summary on ground subsidence. The coastal, hydrologic and hydraulic modelling is outlined in Sections 5, 6 and 7. Anticipated flood level increases are outlined in Section 8 and conclusions and recommendations in Section 9. Suggested further investigations are described in Section 10, with references listed in Section 11.

Five appendices are contained in the report:

- Appendix A lists available background information;
- Appendix B contains coastal analysis results;
- Appendix C supplements the hydrologic work;
- Appendix D provides graphical output for the hydraulic modelling; and,
- Appendix E contains frequency analysis output.

2 SITE DESCRIPTION

2.1 PHYSICAL SETTING

2.1.1 BOUNDARY BAY

Figure 2.1 shows a map of the study area and the surrounding region. Boundary Bay and Mud Bay are situated along the presently inactive, southern edge of the Fraser delta. The Fraser delta has advanced seaward over the last 10,000 years into the Strait of Georgia forming the lands now occupied by Delta and Richmond (Clague et al 1998). Point Roberts peninsula became connected to the mainland less than 5,000 years ago when the Fraser tidal flats extended seaward to these uplands.

Boundary Bay and Mud Bay cover an area of 60 km^2 and face south east on to the southern Strait of Georgia. The tidal flats are approximately 4 km wide and 15 km long. The upper portion, which extends up to the sea dikes, consists of salt marsh and contains silty and sandy peat (Kellerhals and Murray 1969). The lower portion consists of mainly sandy tidal flats that are covered by a system of dendritic drainage channels.

2.1.2 SERPENTINE NICOMEKL FLOODPLAIN

The Serpentine and Nicomekl Rivers have a combined drainage area of over 300 km^2 and discharge into Mud Bay (Figure 2.1). Peak flows in the two rivers are of similar magnitude but following high flows, drawdown in the Serpentine River tends to be more rapid, resulting in increased cross-flow from the Nicomekl into the Serpentine.

European settlement of Surrey began in the 1860's and the first diking district was incorporated some twenty years later. Drainage improvements were initiated by the agricultural community. The Serpentine and Nicomekl sea dams were built in 1913 (KPA 1994). Over time, land use has changed and the population has increased, requiring considerable upgrades to the diking systems.

Significant stormwater management infrastructure exists in the lowland area, including:

- 100 km of earth dike;
- One km+ of sheet pile dike;
- 100 km+ of ditches;
- Two sea dams;
- 30 pump stations;
- 10 spillways; and,
- 170 floodboxes.

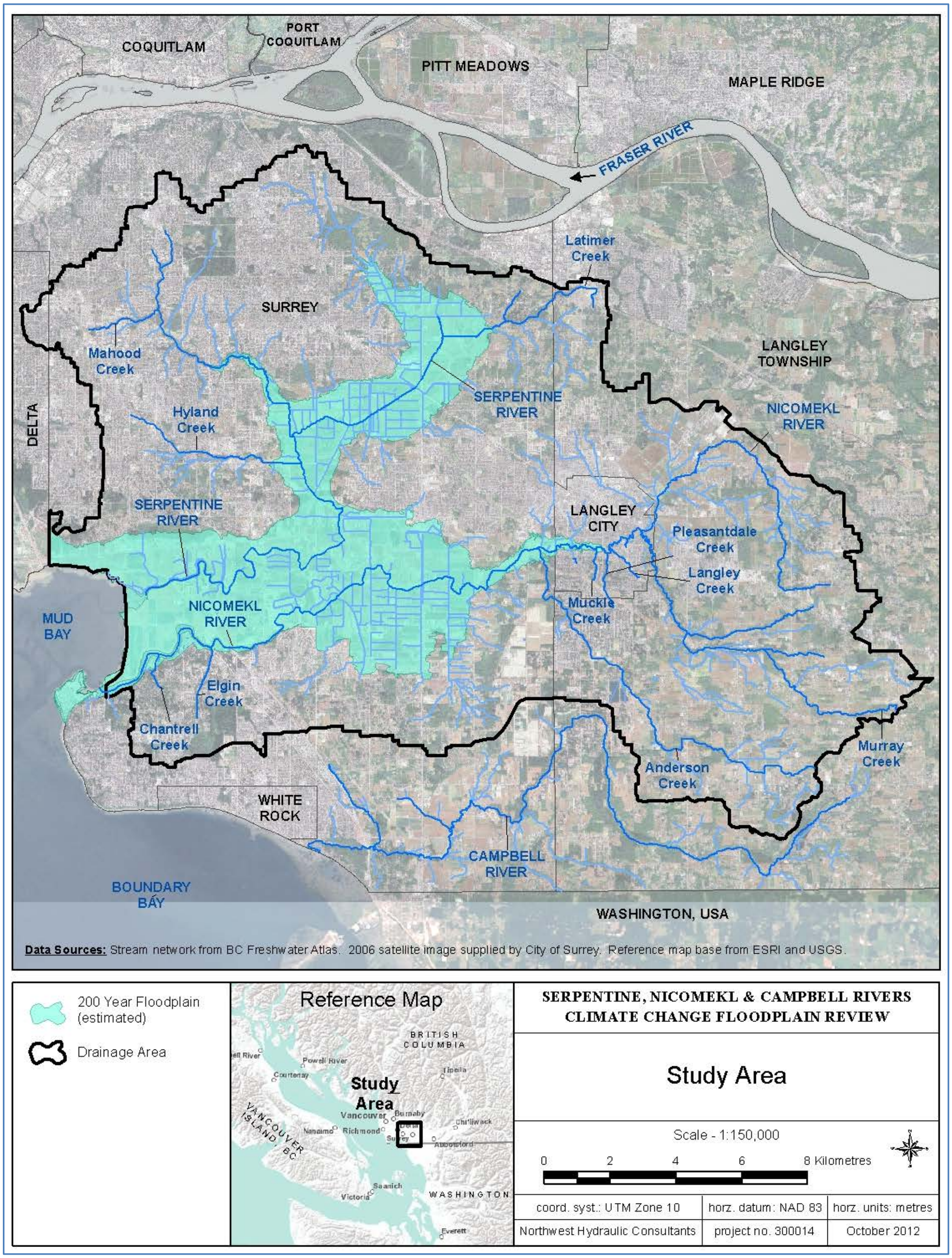


Figure 2.1. Study area

The diking system has gradually been improved but still does not meet provincial requirements for standard dikes. During high ocean or river levels, the only way to drain the internal diked areas is by pumping. However, as water levels in the main stem rivers continue to rise, pump stations are shut off and the land behind the dikes allowed to flood in order not to compromise the integrity of the dikes. Spillways have been incorporated at several locations in the dikes to ensure a degree of equitable flooding of the mainly agricultural land. Although most of the floodplain area of the Serpentine and Nicomekl River basins are within the Agricultural Land Reserve, there are also key settlement areas.

2.2 Flood Hazards

Potential flood hazards in the Serpentine-Nicomekl system include:

- Floods generated from upland runoff;
- Interior flooding behind dikes due to local precipitation;
- During times when the sea dams are closed due to high ocean levels;
- Interior flooding caused by breaching of the river dikes;
- Breaching of the sea dikes along Mud Bay during extreme high water conditions (high tide, storm surge and wave runup); and,
- Breaching of the sea dikes along Mud Bay due to seismic events or tsunami waves.

Only the first three flood mechanisms are discussed in this report. The remaining three will be assessed at a later time.

2.3 History of Floods

Septre (2000) summarized flood events from the late 1800's to year 2000 on the Serpentine and Nicomekl Rivers as provided in Table 2.1**Error! Reference source not found.**. Information since year 2000 was obtained from local newspapers and the City (2009).

Table 2.1. Historic floods (1900 - 2000)

Date	Type of Flooding	Flooding Reported
June 2-10, 1916	Rainfall	Nicomekl banks overtopped, land east of Cloverdale flooded from Serpentine.
December 13-18, 1925	Rainfall	Nicomekl and Serpentine overflowed their banks. Surrounding land submerged. Culverts and roads damaged.
November 18-21, 1932	Rain-on-snow	Serpentine overflowed its banks, salmon swimming in the fields. Several inches of water across Bose Road.
December 19-30, 1933	High tides + storm surge + rain	English Bay at 4.53 m (datum?). Nicomekl dike breached (120m gap) flooding Crescent Beach community by up to

		90cm. Area of 400 ha inundated. Nicomekl and Serpentine overflowing their banks.
January 20-27, 1935	Ice storm, rain-on-snow	For detailed description see KPA 1994. Inter-river area from Johnson Rd to Pacific HWY under water. 15-20 families evacuated.
January 5-11, 1945	Rainfall	Serpentine dikes breached.160 ha inundated at Fry's corner by up to 1.2m. Land from SW Cloverdale to Mud Bay Rd inundated.
February 6-7, 1945	Rain and high winds	Fry's corner inundated.
April 12, 1946	Rainfall	Fry's corner inundated.
Nov 27-Dec 4, 1951	High tides + storm surge	A 21m breach developed in the Serpentine dike, 800m N of Wade Rd causing inundation of 480 ha. Land between King George HWY and Great Northern RW flooded.
November 24,1960	High tides + storm surge	Nicomekl overflowed banks in 15-20 places but only one dike breach occurred.
December 22-23, 1963	Rainfall	Scott Rd flooded by 60 cm. King George HWY near 80th Ave was flooded. Over 100 homes flooded, mainly because of plugged culverts.
December 9-18, 1966	Rain-on-snow + high tides	Flood waters from Serpentine closed Clover Valley Rd from N of Fraser Way and Coast Meridian from N of 66th Ave.
November 21, 1972	High tides + storm surge	Mud Bay dike N of Nicomekl breached. Limited access, over 300 ha inundated. A 21-24 m wide breach developed in the Nicomekl seawall.
December 25-26, 1972	Rain-on-snow + high tides	Two sections of Nicomekl dikes breached.
Nov 29-Dec 8, 1975	Rain-on-snow	Scott Rd flooded. Nicomekl dike breached near 4600 block of 184th St.
February 12-14, 1982	Rainfall	Nicomekl overflowed its banks 168, 192 and 208 St closed.
December 16-18, 1982	High tides + storm surge	Many homes damaged.
January 27, 1987	High tides + storm surge	Flooding reported in Delta where “winds pushed high-tide water onshore” High winds and rain at Cypress and Whistler BC Hydro circuits damaged in Lions Bay, Maple Ridge,

		North Vancouver and Gibsons.
December 20, 1994	Rain on snow + high tide	Snow blankets Lower Mainland in early December Several days of rain cause Lower Mainland sewers to back up and road to flood. Waterfront homeowners in Tsawwassen were sandbagging their property.
January 1, 1997	Snow + rain	Nicomekl River overflowed its banks. 40th Ave between 152 and 156 St closed and Hwy 99 closed Two feet of water on field at corner of 40th and 156th.
December 17, 2000	Snow, wind, freezing rain	104th Ave closed under Guildford mall overpass
October 17, 2003	Rainfall	Flooding of Crescent Beach, Cloverdale and Serpentine-Nicomekl agricultural lowlands Dike overtopping: Serpentine River west of north 176 St crossing (518820, 5445283). Road overtopping: 192nd St was overtopped but 184th St was not 152nd St was overtopped (near 76th Ave in Bear Creek area) Road closures: - 88th Avenue: from 168th Street to 176th Street - 192nd Street at Colebrook - 80th Avenue: Harvie Road to 176th Street - 160th Street: 40th Avenue to 36th Avenue - 82nd Avenue at the 16200 –block
January 17, 2005	Rainfall	Dike breaches and erosion: left dike of Serpentine River near north 176 St crossing (518820, 5445283) and left dike of Nicomekl River at 192 St Bridge (522532, 5438092).
February 5, 2006	Wind and storm surge	Combination of high tides and strong winds left beachfront homes flood-damaged.
November 16, 2006	High winds and rainfall	Storm with winds of more than 100 km/hr and rainfalls exceeding 110 mm in some areas…. caused creeks and rivers to flood in Lower Mainland.
March 14, 2007	Rainfall	Heavy rain flooded a number of homes as low-lying areas of Langley and Surrey were flooded in the Nicomekl and Serpentine watersheds.
January 8, 2009	Rain-on-snow	Flooding of Serpentine-Nicomekl agricultural lowlands Controlled spilling at Fry's Corner and Fleetwood spillways which resulted in closure of 80th Ave between 176th St and Harvie Rd and closure of Harvie Rd between 80th Ave and Fraser Highway.

		Nicomekl flowed over bank onto 40th Ave causing temporary road closures on 40th Ave from 152 to 156B St. Temporary road closures on 192St from 51B to Colebrook Rd Floodwater receded in both lowland areas with minimal damage or loss.

The table illustrates that the causes of severe flooding are varied; heavy rain, rain-on-snow, high tides with storm surge, or a combination of these events. Typically, the worst flooding occurs from November to February. Large flood events since year 2000, are described in more detail in Section 6.2.

2.4 AVAILABLE INFORMATION

Extensive GIS background information is available for the project area. Table A.1 in Appendix A summarizes topography, imagery, hydrography, watershed, land use, soil/surficial geology, subsidence and administrative information received from the City and other sources. The information includes data on pump stations, dikes, sea dams, stream gauges, rainfall gauges, soils, and subsidence, in addition to reference data on roads, railways and administrative boundaries. Also available is ocean bathymetry, river bed bathymetry, orthophotography and land use information.

A large number of background reports were supplied by the City and are listed in Table A.2 of Appendix A.

2.5 DATUM SHIFTS

Shifts in the official Surrey vertical datum have occurred in the past. To ensure correct elevation input for modelling, the City undertook a control survey to confirm the vertical datum of reference points and pertinent structures. The following adjustments were made:

- On the Serpentine River, previous elevation data were lowered by 0.06 m.
- On the Nicomekl River, previous elevation data were lowered by 0.10 m.

Applying a uniform correction of 0.08 m would have been well within the model accuracy.

All elevations provided in this report are to current City datum, also referred to as CGVD 28.

3 CLIMATE CHANGE SCENARIOS

The consensus view from organizations such as the Inter-governmental Panel on Climate Change (IPCC) is that the global climate system is warming, and the expectations are that the global annual mean temperature will rise more than 3°C this century. Continued warming and changing of precipitation patterns will have a large effect on hydrological processes, with significant implications for the economy, infrastructure, and eco-systems of British Columbia (Rodenhuis et al, 2009).

3.1 SEA LEVEL RISE

The sea level rise policy for BC recommends assuming a 1 m rise in global mean sea level between the year 2000 and 2100 as show in Figure 3.1.

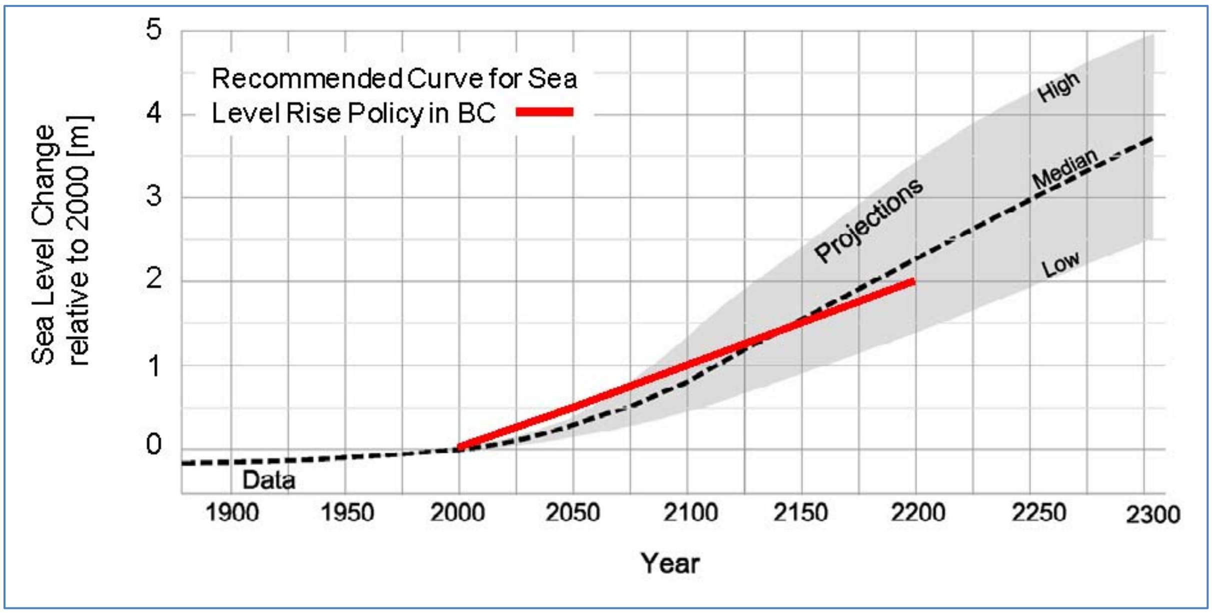


Figure 3.1. Recommended global sea level rise for planning and design in BC (Ausenco Sandwell, 2011)

In addition to this global sea level rise, the effect of local ground movements (subsidence) was added to determine the net relative sea level change in the region (Section 4).

While the policy implies an assumption of a linear 10 mm/year rise in sea level from 2000 to 2100, it is clear that this assumption overstates actual sea level rise early in this period. Recent analyses based on satellite altimeter measurements (Figure 3.2) show a more or less steady global mean sea level rise from 1993 to 2011 of 3.1 ± 0.4 mm/year.

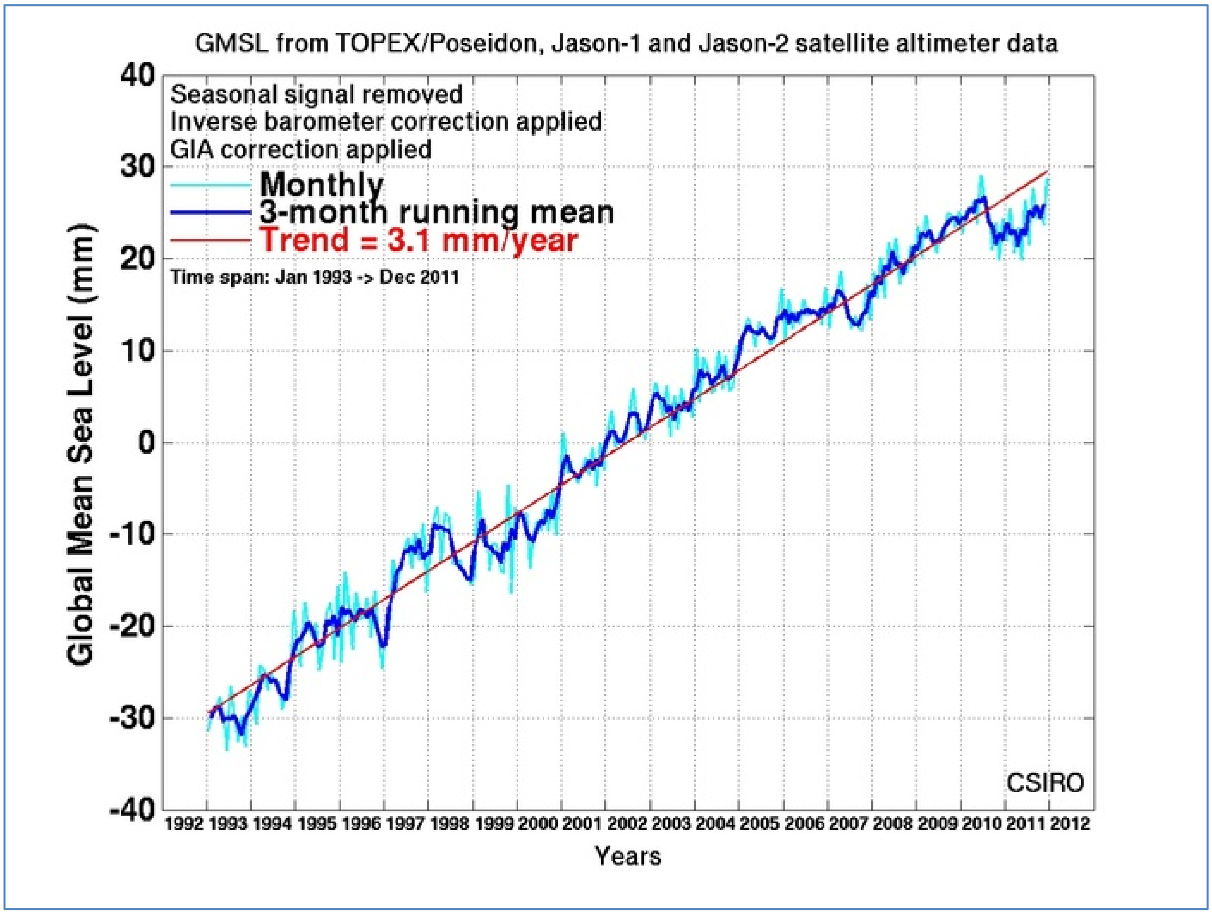


Figure 3.2. Global mean sea level, 1993-2011[1]

For the present study, year 2010 was adopted as the nominal baseline condition. Ocean level data was adjusted as necessary (see Section 5.5) to produce stationary ocean level time series representative of the 2010 mean sea level. Based on Figure 3.2, there was an approximately 0.03 m absolute sea level rise from 2000 to 2010. To ensure consistency with draft sea level rise policy and provincial guidelines, a linear 0.97 m rise in absolute sea level was assumed from the 2010 base condition to 2100 to give a total 1.0 m rise from 2000 to 2100.

As part of this assessment, the impacts of sea level rise were conducted for year 2100. Given the projected linear increase in absolute sea level, interior flood levels and other flood risk metrics are expected to increase smoothly over time, although not necessarily linearly. Future assessments will address the critical timing for failure of infrastructure or implementation of countermeasures by interpolation in time of simulation results from intermediate sea level rise scenarios. It should be recognized that there is significant uncertainty in sea level rise projections with a range in the rise from 2000 to 2100 presented in the draft provincial sea level rise policy and shown in Figure 3.1,

[1] Source: http://www.cmar.csiro.au/sealevel/sl_hist_last_15.html, accessed 28 Feb 2012.

from about 0.5 m to 1.3 m. Given this uncertainty, reliance on interpolation of simulation results, rather than detailed simulation of finer increments of sea level rise, is considered to be a reasonable and an appropriate approach for intermediate and long-range planning purposes.

Over time, aggradation has taken place in Mud Bay and Semiahmoo Bay and this trend is expected to continue (communication with Corporation of Delta). Following discussions with the City, it was assumed that as water levels increase, so will bed levels, roughly maintaining the overall depth. Consequently, storm surge values are unlikely to change significantly. However, should water depths start to increase, i.e. sea levels increase more rapidly than the ocean bed build up, wave heights would also begin to increase.

3.2 Storm Frequency and Intensity

While some climate change studies have indicated possible future increases in the frequency and intensity of storms, implying possible future increases in wave climate severity and storm surge, there is at present little consensus on such impacts. The policy discussion paper (Ausenco Sandwell, 2011, pg. 3) states:

"At the present time, scientific information on the expected changes in storms approaching British Columbia coastal waters and their characteristics, specifically on the intensity of the storms, their related wave conditions and the associated storm surges in the future, is only starting to emerge. Based on the available information it appears reasonable to conclude that no significant change is expected in coastal BC waters; however, further investigations are warranted to fully assess the regional implications and to further assess future trends."

Accordingly, potential future increases in storm frequency and intensity were not considered in this study. If future scientific findings show an increase in storm intensity with climate change, then it is likely that the storm surge component of the ocean level series could be amplified to reflect such impacts.

3.3 Precipitation

Most Global Climate Models (GCMs) show wetter winters and drier summers over much of western Canada and the US Pacific Northwest with future climate change. Increased winter precipitation would increase local runoff in the Serpentine and Nicomekl watersheds and exacerbate lowland flooding already affected by sea level rise. The magnitude of future change in precipitation is however uncertain.

APEGBC has recently developed "Professional Practice Guidelines for Legislated Flood Hazard and Risk Assessments in a Changing Climate in BC" which include a discussion on climate change impacts on precipitation. Preliminary reference is made to peak flow increases in the order of 10%. It is recommended that potential precipitation changes under climate change be investigated as part of future work. The historic precipitation time series applied to the continuous hydrologic simulation could be readily modified to investigate the effects of increased winter precipitation on local flooding in conjunction with more certain projections of sea level rise. Increases in runoff caused by changes to the land-use were accounted for in the hydrologic modelling.

4 GROUND SUBSIDENCE SCENARIOS

4.1 BACKGROUND

Portions of the Fraser delta and the adjacent floodplain are known to be subsiding. Previous regional studies report subsidence rates on portions of the Fraser delta are estimated to be in the order of 2 mm/year or approximately 0.2 m by 2100 (Thomson et al, 2008). Most of the ongoing sediment compaction and related subsidence occurred in the upper 10 to 20 m of the predominantly silt and peat Holocene sediments (Thomson et al, 2008).

Ongoing future subsidence will impact both flood depths behind dikes and flood protection work elevation adjustments over time.

The global sea level rise projections described in Section 3 need to be adjusted to include the effect of regional ground displacements due to subsidence in order to estimate the relative sea level rise. A simplified approach was adopted, which assumed that land surface elevations are fixed at the 2010 base condition and the baseline time series of ocean levels is adjusted by the relative rise. The method is approximate and sensitivity assessments, which adjust the actual hydraulic model geometry based on projected ground levels is recommended as part of future phases of the study.

4.2 GROUND MONITORING LOCATIONS

TRE Canada Inc. (2011) discussed potential ground subsidence monitoring in the floodplain. After reviewing this report, 21 locations were selected in the Serpentine and Nicomekl floodplains with ERS images from May 1992 to August 1999 and Envisat images from January 2003 to February 2006. This data provided a settlement history for each monitoring point shown in Figure 4.1 during the 14 year period from May 1992 to February 2006.

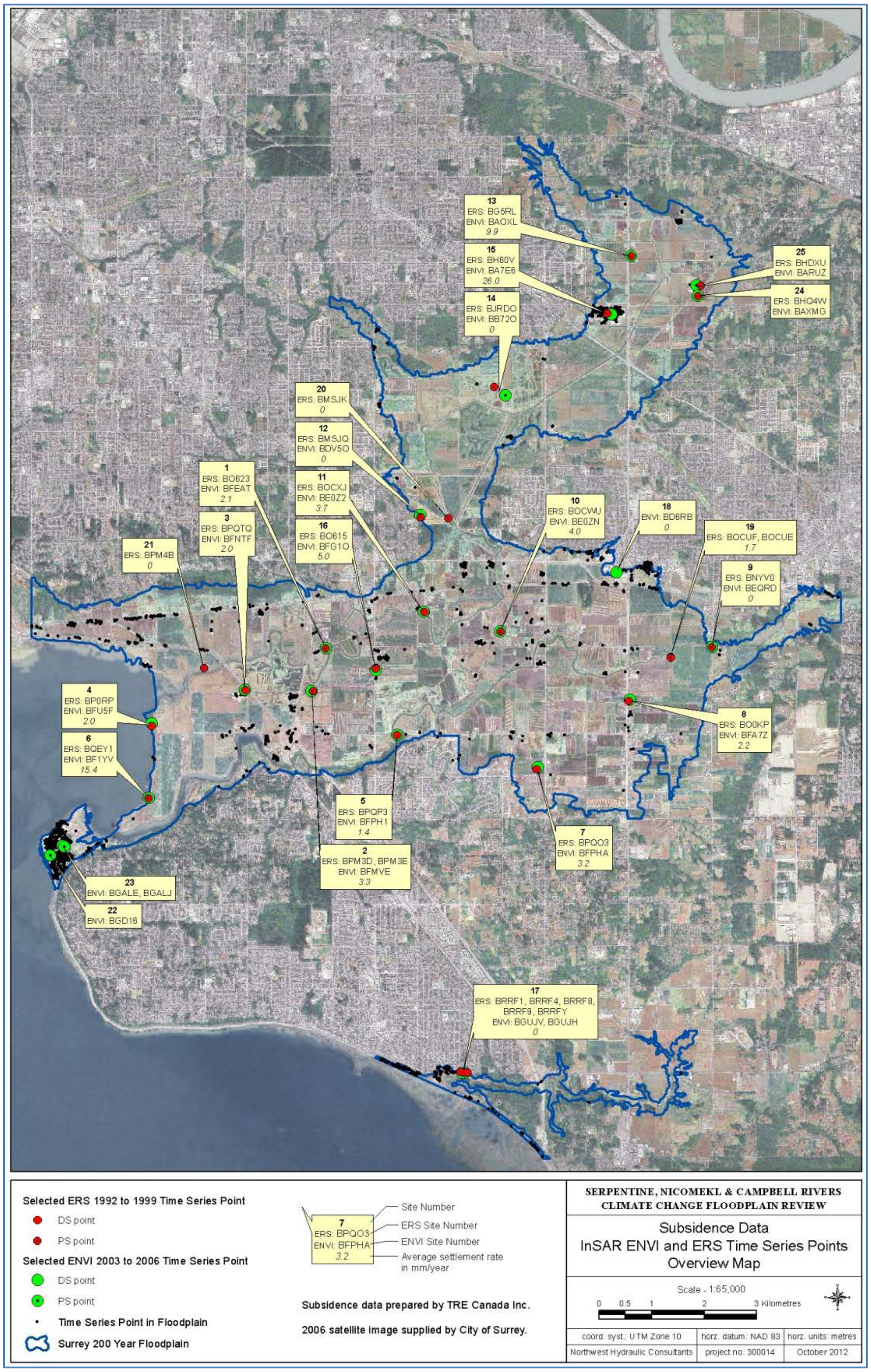


Figure 4.1. InSAR ENVI and ERS time series points overview map

4.3 SUBSIDENCE ANALYSIS

TRE Canada Inc's settlement versus time plots for the ERS and Envisat data were assessed by fitting a trend line through the data plots by eye. These plots were combined to demonstrate the 1992 – 2006 settlement pattern for each site.

With Site 16, adjacent to 152 Street between the Serpentine and Nicomekl Rivers, as an example (Figure 4.2), the ERS data to August 1999 indicates an average settlement rate of 4.5 mm/year. Extrapolating this rate to the start of Envisat data in January 2003 indicates a January 1, 2003 settlement of 50 mm. From that date to January 1, 2006, the Envisat data indicates an additional settlement of 18 mm. This gives a total settlement of 68 mm in 13.6 years or an average of 5.0 mm/year at Site 16. The Envisat data indicate a slightly higher settlement rate than does the ERS data but the reason is not evident. Extrapolating the average settlement rate for Site 16 (Figure 4.2) to year 2100 from the last Envisat data point in February 2006 indicates an additional settlement of 442 mm. This extrapolation utilizes an arithmetic plot of settlement rather than a semi-log plot and the arithmetic extrapolation is likely to be conservative.

Figure 4.2 shows the estimated subsidence rates for the 21 points, indicating a range of values. Based on these results and discounting unreasonably large values, an overall rate of 2.5 mm/year was selected. The City undertook an independent review which agreed with this rate. However, according to the City, buildings within the floodplain may have significantly higher rates of subsidence whereas, areas outside the floodplain have lower rates. The 8th Avenue / Crescent Beach areas are expected to be nearly static.

Over the 90 year time period from year 2010, the assumed topography starting point, to year 2100, an annual rate of 2.5 mm/year translates to a total subsidence of 225 mm. In comparison, the Ausenco (2011) suggested rate of 2.1 mm/year for the Fraser River Delta is slightly less conservative.

The total relative adjustment for sea level rise and the adopted subsidence was therefore 1.195 m (0.225 m +0.97 m). This value was adopted for the hydraulic modelling. Future more in-depth assessments of ground subsidence are envisioned.

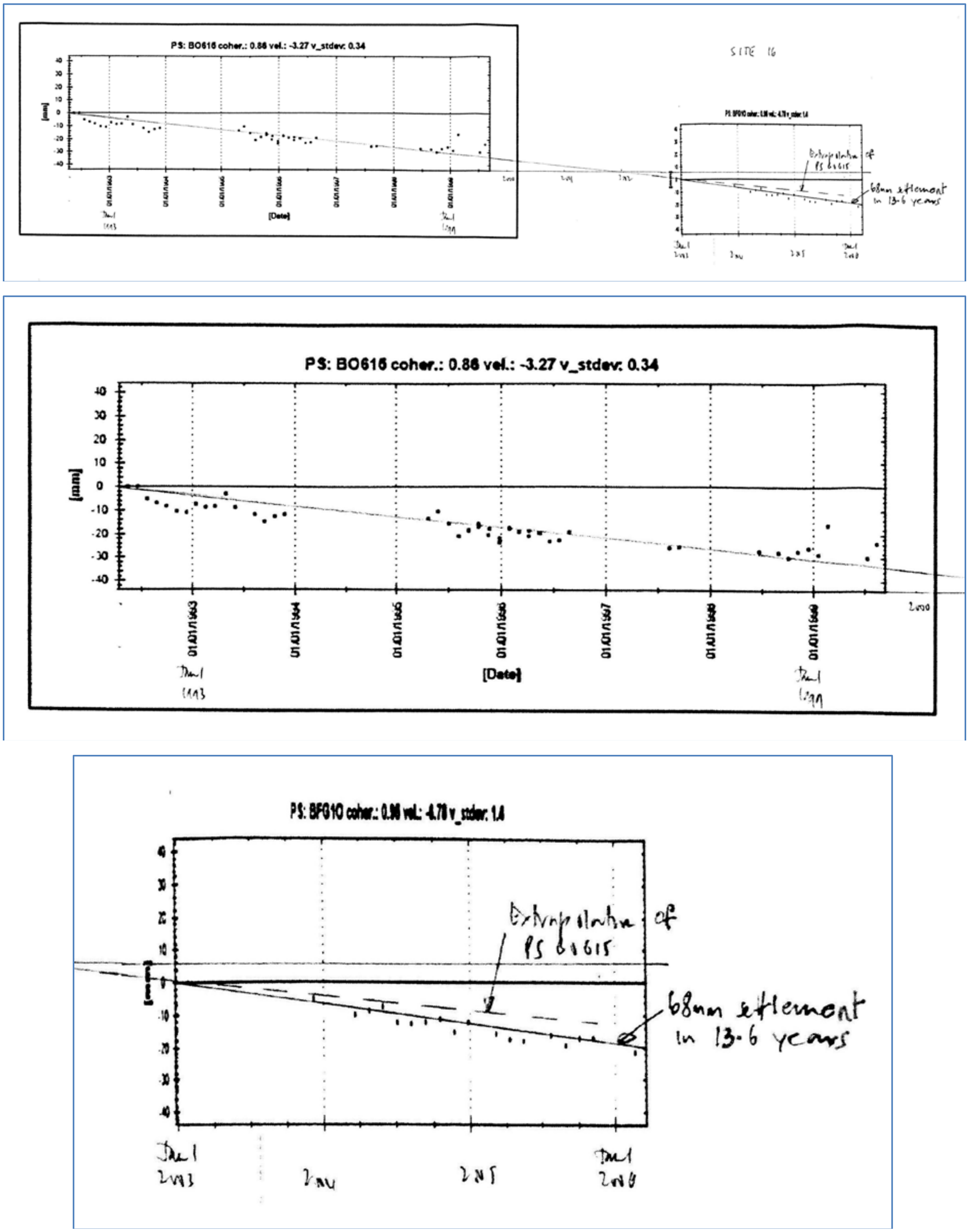


Figure 4.2. Subsidence Site 16

5 Ocean Level Analysis

5.1 Purpose

The purpose of the ocean analysis was two-fold:

- To provide water level boundary conditions for the Serpentine and Nicomekl River hydraulic model; and,
- To calculate Designated Flood Levels (DFLs), Flood Construction Levels (FCLs) and Dike Crest Elevations (DCEs) at specific locations in Boundary Bay.

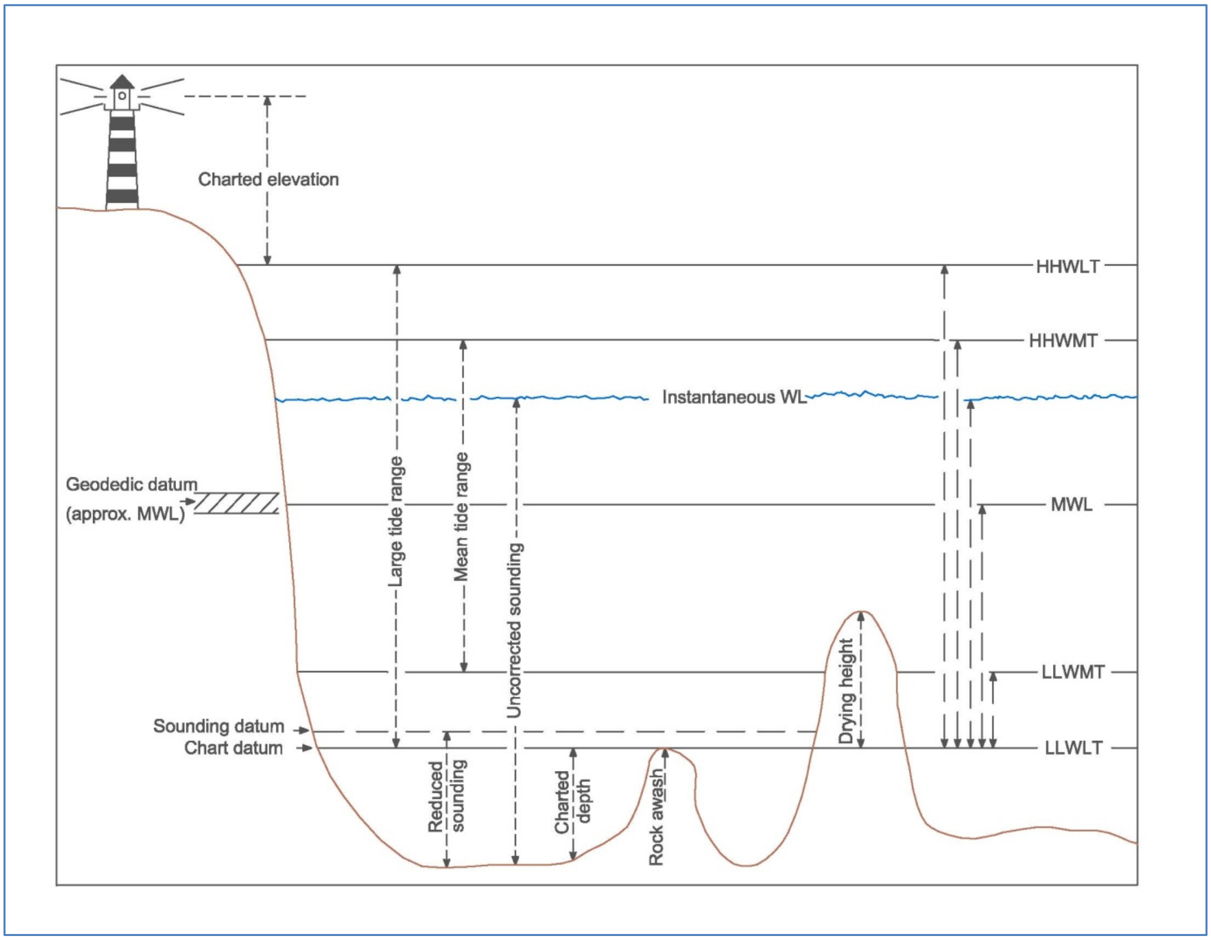
5.2 Definitions and Terminology

The tides in the southern Strait of Georgia are classified as "mixed, mainly semi-diurnal" meaning that they undergo two complete tidal oscillations daily but with inequalities both in high waters and low waters. Tide levels are commonly referenced to "Chart Datum", which represents a low water reference plane that the tide will seldom fall below. Figure 5.1 illustrates several common tide elevation values that are used to compare tide characteristics from place to place (Forrester, 1983). Higher High Water Large Tide (HHWLT) represents the average of the highest high waters, one from each of 19 years of predictions. Higher High Water Mean Tide (HHWMT) represents the average of all the higher high waters from 19 years of predictions. Mean Water Level (MWL) is the average of all hourly water levels over the available period of record.

Actual observed coastal water levels have a deterministic component, the tides, and a probabilistic component, resulting from changes in barometric pressure, wind and wave stress. The probabilistic component is referred to as the 'residual water level'. The residual water level includes external storm surge (which is driven primarily by water levels at the entrance of the Juan de Fuca Strait), local wind setup (resulting from wind stress induced by local winds) as well as wave setup. On the BC coast, storm surges can temporarily increase the ocean level by more than 1 m above the predicted tide level. Figure 5.2 illustrates the difference between the predicted tide level, the observed ocean level and the computed residual levels during the passage of a low pressure storm system in March 2012 at Point Atkinson.

Near the shoreline, the maximum water level is affected by wave setup that is induced landward of the wave breaking zone and wave runup (Figure 5.3). Wave setup is an increase in the mean water level shoreward of the region in which breakers form at the seashore. Wave runup varies according to beach slope and roughness as well as the presence of coastal structures such as sea dikes or revetments)[2]. In this study, wave setup and runup are considered separately from the residual water level.

[2] A useful and practical working definition distinguishing wave setup from wave runup elevations is: "Wave setup contributes to high water marks inside reasonably small buildings; however, wave runup does not."



Note: For Boundary Bay (to geodetic datum), HHWLT = 1.74 m; MWL = 0.07 m; Chart Datum = -2.74 m.

Figure 5.1. Common tide elevation values

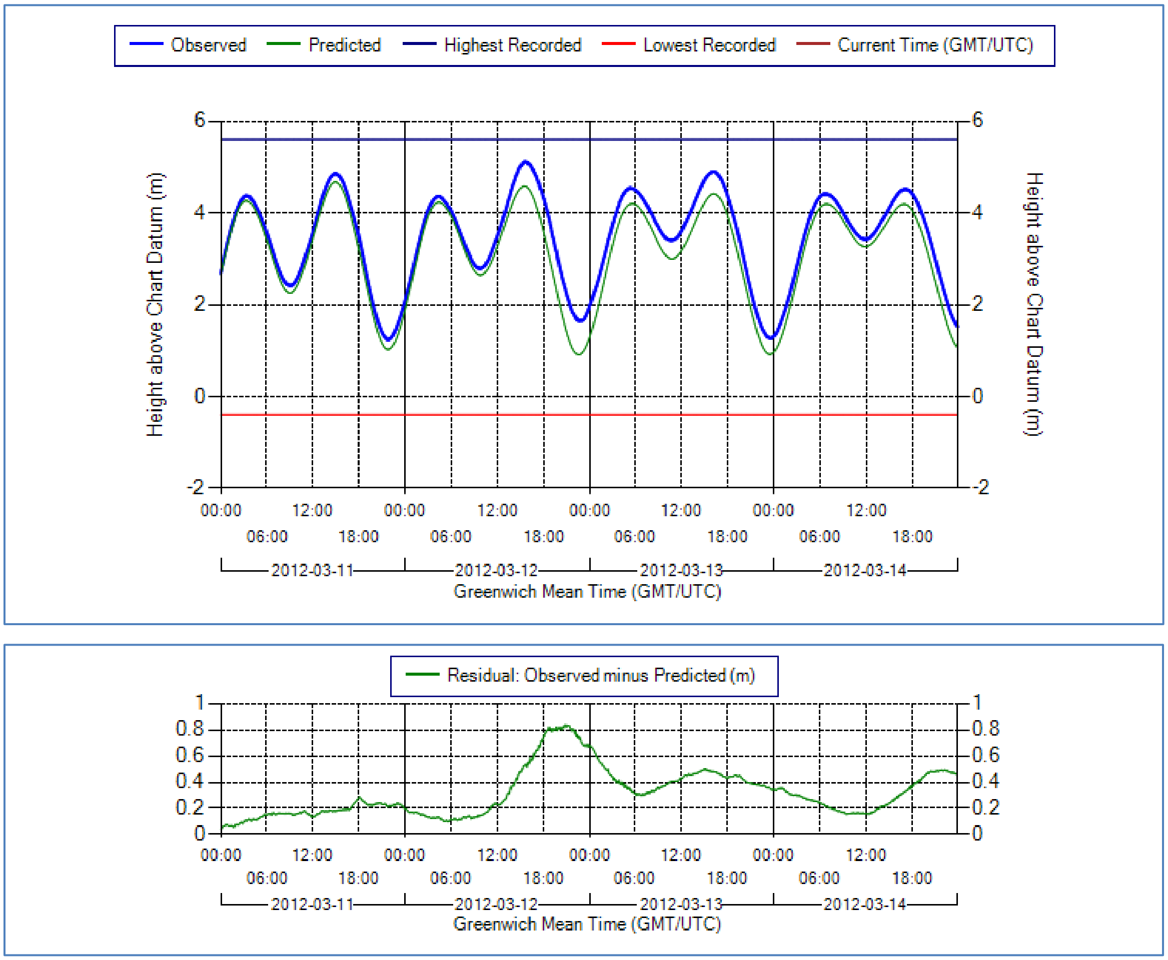


Figure 5.2. Observed level, predicted tide level and residual level at Point Atkinson, March 2012

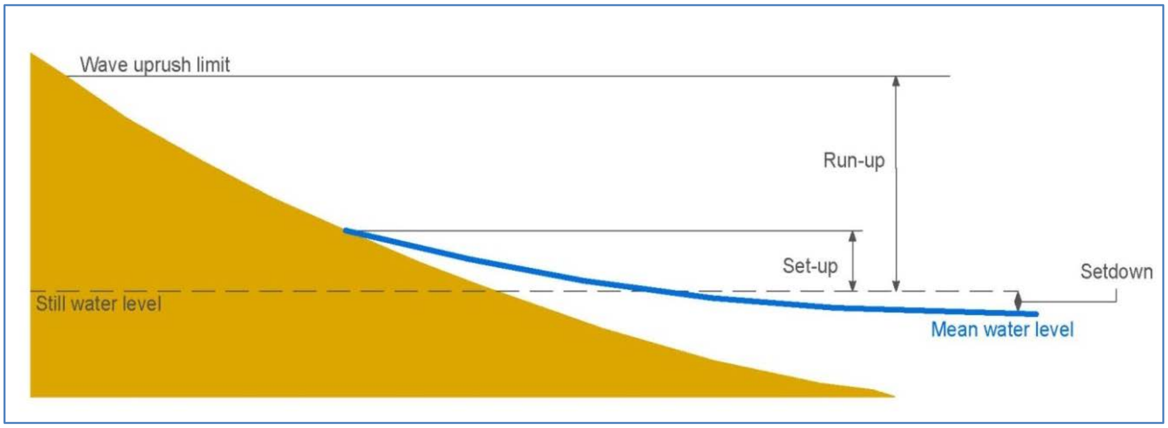


Figure 5.3. Summary of water level variations as waves approach a beach[3]

5.3 Method of Approach

In January 2011, the BC Ministry of Environment published Climate Change Adaptation Guidelines for Sea Dikes and Coastal Flood Hazard Land Use (Ausenco Sandwell 2011). The guidelines recommend adding the Higher High Water Level Tide (HHWLT), the 200-year storm surge and maximum local wind set-up to estimate 200-year design levels, disregarding the joint probability of the three components occurring simultaneously. It was recognized that this simplified method provides first-level conservative estimates and that more in-depth, location specific analyses, are recommended in some situations.

For the present study, a more statistically sound approach was called for, requiring the development of a time series of historic ocean levels that combined tides, surge levels and wind setup and that could be analyzed directly through a frequency analyses. Applying a continuous simulation approach, a 48-year long water level hind-cast, or time series of past ocean levels, was generated at the Serpentine, Nicomekl and Campbell River outlets. The hind-cast formed a composite data-set consisting of both measured and modelled data that accounted for tides, external storm surge and local wind set-up. Additional detail on the ocean analyses are contained in Appendix B.

5.4 Available Background Data

The following data was compiled for the ocean level analysis:

- Water level data at the river and ocean sides of Serpentine and Nicomekl sea dams were supplied by the City. From 2000 to 2010, the records are 90% complete (90% of the time there is at least one sample per hour).

[3] Source: http://www.mfe.govt.nz/publications/climate/coastal-hazards-climate-change-guidance-manual/page23.html, accessed 11 Oct 2012. Redrawn by NHC.

- Water level data were also collected from a large number of Canadian Hydrographic Service (CHS) and National Oceanographic and Atmospheric Administration (NOAA) tide stations. These data-sets are referenced to chart datum and are generally complete. (Chart datum was converted to Geodetic Survey of Canada datum and subsequently to Surrey datum).
- Meteorological data (wind and atmospheric pressure) were collected from Environment Canada weather stations.
- Wave data were collected from the Environment Canada wave buoy at Halibut Bank. The Halibut Bank buoy is the only permanent buoy in the Southern Strait of Georgia. Wave data was also obtained from a buoy temporarily deployed near White Rock (1977-1978). Both data-sets provide wave parameters and non-dimensional spectra.

This information is also summarized in more detail in Table A.1 of Appendix A.

Several previous studies estimating storm surge in Boundary Bay were reviewed (Appendix B.1). In general the approach has been to relate the storm surge in Boundary Bay to storm surge at Point Atkinson and other long running tidal stations.

Starting in 2000, The City has recorded water levels on both sides of the Serpentine and Nicomekl Sea Dams. The data is not sufficiently long for forecasting extremes, but was used to validate modelled water levels. The measurements were recorded at variable, but sub-hourly intervals. Analysis of the data revealed some erroneous measurements which had to be manually removed and some time-stamp issues relating to the application of daylight savings time.

Analysis of the measurements made at the Nicomekl Sea Dam revealed several undocumented and unexplainable datum shifts. As such the Nicomekl measurements were not used for quantitative validation purposes. Figure 5.4 qualitatively compares the water level hind-cast at the Nicomekl River and the measurements at the Serpentine and Nicomekl Sea Dams for a period in January 2009 (a period when the Nicomekl gauge measurement appear reliable). At high tide the measurements at each Dam agree well. Generally the hind-cast accurately reproduces the measurements, the exception being the high river flow event Jan 7-8 (see Section 7.4.1). During this event the river level remained higher than the ocean level, even during high tide, so that the gauge on the ocean side of the Dam was actually measuring river level rather than ocean level.

At the Serpentine Sea Dam, ocean levels were influenced by river levels when the ocean side of the dam was lower than the river side. This meant that the trough of the tidal cycle was not measured accurately.

Other water level records in the area are sparse. Crest gauge measurements were obtained at Crescent Beach from 1973 until 1988, but the measurements only give the highest water level since the gauge was last reset. (The gauge had to be reset manually and often many days or weeks passed between resets). Hourly water level measurements were made in White Rock by the DFO for 5 months in 1972, but are of limited use because of the short duration.

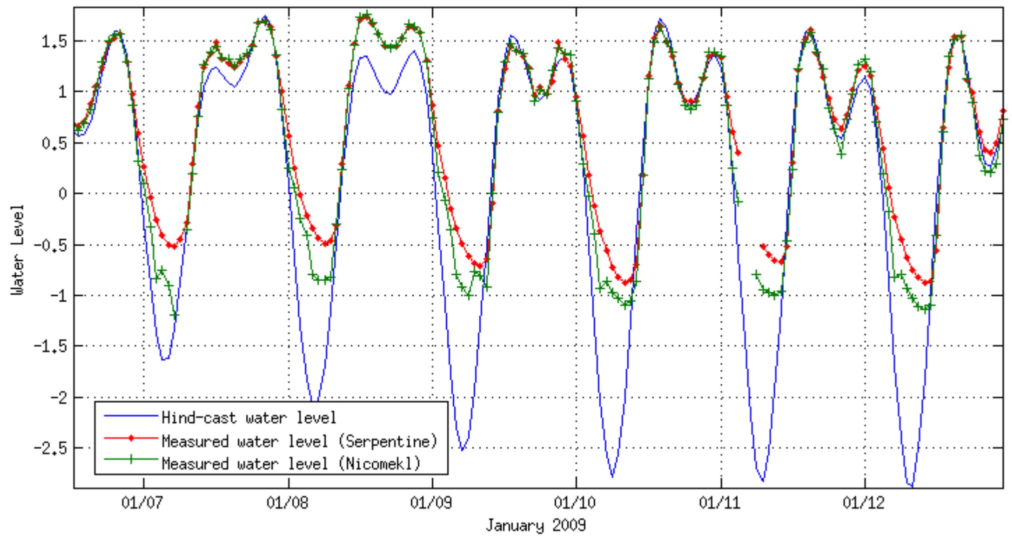


Figure 5.4. Comparison of water level hind-cast at the Nicomekl River and the measurements at the Serpentine and Nicomekl Sea Dams

5.5 OCEAN LEVEL HIND-CAST

Since a complete ocean level record of combined tidal water levels, storm surge and wind setup was unavailable, a time series based on a combination of measured and modelled values of each component had to be generated, forming a composite data-set for the selected modelling period of 1964 to 2011. This hind-cast period was statistically long enough for estimating 200-year return period water levels (design level selected by the City) and also had sufficient precipitation and ocean level background data for generating representative boundary conditions for the hydraulic model.

5.5.1 TIDAL WATER LEVELS

Tidal water levels were estimated from the harmonic constituents provided by the City. The constituents were calculated based on long-term measurements at Tsawwassen and were adjusted to agree with ocean level measurements made at the Serpentine and Nicomekl Sea Dams.

The tide prediction program *Tide4.exe* (Mike Forman, Institute of Ocean Sciences) was used to recreate the tidal signal from 1964 to 2011 inclusive. Based on this data, HHWLT for Boundary Bay was calculated as the average of the highest tidal water level achieved in each year of the current National Oceanic and Atmospheric Administration (NOAA) tidal epoch (1983-2001) to be 1.74 m.

5.5.2 EXTERNAL STORM SURGE

Since water level measurements in Boundary Bay over the hind-cast period are unavailable, the external storm surge was transposed from nearby long-running tide stations. The external storm surge refers to large scale changes in water level that occur at the open ocean and then propagate into the protected waters of the Salish Sea, and is sometimes referred to as 'deep water' storm surge or remote forcing. Principle component analysis was performed to assess the response of the Salish Sea to storm surge propagating through the Juan de Fuca Strait from the Pacific Ocean. This was done to assess the feasibility of using residual water level measurements from nearby tide stations to estimate external storm surge at Boundary Bay.

Observations from 12 tide stations ranging from the open coast to the inland waters were analyzed using principal component analysis. The primary signal is the seasonal variation in sea level due to wind driven coastal upwelling. Because of the deep water connections, this variation occurs essentially simultaneously over the region. A secondary signal relates to a divergence in sea level between the coast and Salish Sea which is apparently due to density differences between the cold, salty coastal water and the fresher, warmer inland waters. From this analysis, observations at Point Atkinson provided a valid reference value for external forcing. Moreover, the analysis of Point Atkinson data and limited White Rock observations indicated that there was little difference in the external forcing at these two locations due to the deep water connection between them.

Hence, principal component analysis of many tidal stations revealed that the Salish Sea responds uniformly to external surge, with only small timing differences between the stations (i.e. the water level residual is essentially the same at Victoria, Patricia Bay, White Rock, Pt Atkinson, etc.). Based on this observation it was deemed acceptable to calculate storm surge at nearby long-running tide-stations and directly transfer values to Boundary Bay (Appendix B.2).

This finding differs from Seaconsult (1992) and warrants explanation. Seaconsult found that for extreme external surge events, the surge level at Boundary Bay was on average 1.09 times greater than that at Point Atkinson. Firstly, Seaconsult's work focused on finding a transfer function for extreme conditions, whereas the current work sought a generally applicable transfer function. Secondly, Seaconsult's conclusions were based on just 13 modelled storms, whereas the current analysis is based on several years of measured data from 12 tide stations. Also, numerical modelling in the early 90’s was less accurate, considering wetting and drying has only recently been addressed in numerical tide models, an important component for the Mud Bay flats. Given the above it is not surprising that there is a 9% difference between the estimates.

To provide complete coverage over the hind-cast period three different tide stations were used, in order of preference:

- Point Atkinson
- Vancouver
- Victoria

External storm surge was calculated as the tidal residual; that is the measured water level with the predicted tide signal removed. This equivalence assumes that wind setup is negligible. This is a reasonable assumption at the above tidal stations as they are situated on relatively steep shorelines where wind setup is likely to have little effect.

To calculate the tidal residual, first the tidal harmonics were calculated using the *T_tide* tidal analysis software (Pawlowicz et al 2002). Tidal harmonics were calculated for each year of available data, and then averaged. The harmonics were then used to recreate the tidal signal (again using *T_tide*) over the period of data availability. The tidal residual was calculated as the difference between the measured water level and that predicted by the tidal harmonics.

Small errors in the phase of the tidal harmonics can cause large oscillations in the residual signal. Two approaches where taken to minimize this effect. First the residual signal was smoothed using a Godin 24-24-25 tidal filter (Thompson 1983). This approach was successfully used to compare the residual signals of different tide stations within the Salish Sea, but tends to over-smooth the signal so that large peaks are reduced in magnitude. As an alternative, water level residual was evaluated only at the maximum and minimum of the daily tidal cycle and small differences in timing between the measured and reconstructed signals were ignored. Though this approach greatly reduces the resolution of the tidal signal, it was found to better capture the peaks in the external surge signal and so was retained for use in the external storm surge hind-cast.

All hourly tidal residual signals were converted from chart datum to CGVD28. The signals were also compensated for the influence of sea level rise to the reference year of 2010 using the relative sea level rise estimates (Mazzotti et al 2008). Point Atkinson was used as the primary data source. Where data was unavailable for Point Atkinson, data for Vancouver was used. Where data was unavailable for both Point Atkinson and Vancouver, data for Victoria was used. The few data points missing after this process were linearly interpolated. The resulting water level residual data-set spans 1964 to 2011 inclusive, at 1 hour intervals. The maximum residual was 1.02 m and occurred in January 1983.

5.5.3 LOCAL WIND SETUP

Given that Boundary Bay and Mud Bay are very shallow it was expected that wind setup would be significant.

A model was constructed using the River and Coastal Ocean Modelling (RiCOM) software to assess the impact of local wind setup on the water levels in Boundary Bay (Appendix B.3). The model grid extended from Point Atkinson in the North to Victoria in the South, containing 84,000 elements and 44,000 nodes. The resolution ranged from 10 m in areas of interest in Mud Bay to 1,000 m around the Western boundaries of the grid. The grid is shown in Figure 5.5.

Bathymetry for the model was gathered from a number of sources. In waters outside of Boundary Bay, bathymetry was sourced from the Canadian Hydrographic Service (CHS) s57 vector charts. In areas of Boundary Bay below Chart Datum, CHS bathymetric survey data was used. Bathymetry of the Serpentine River and some of the shallows of Mud Bay above chart datum was measured by Terra Remote Sensing Inc. Bathymetry of the Nicomekl River was measured by CRA Canada Surveys Inc. Bathymetry around Crescent Beach was obtained by NHC. LiDAR data was provided by the City of Surrey for dry areas of the mud flats and the shore above Chart Datum. All of the information was combined in GIS and converted to Surrey datum.

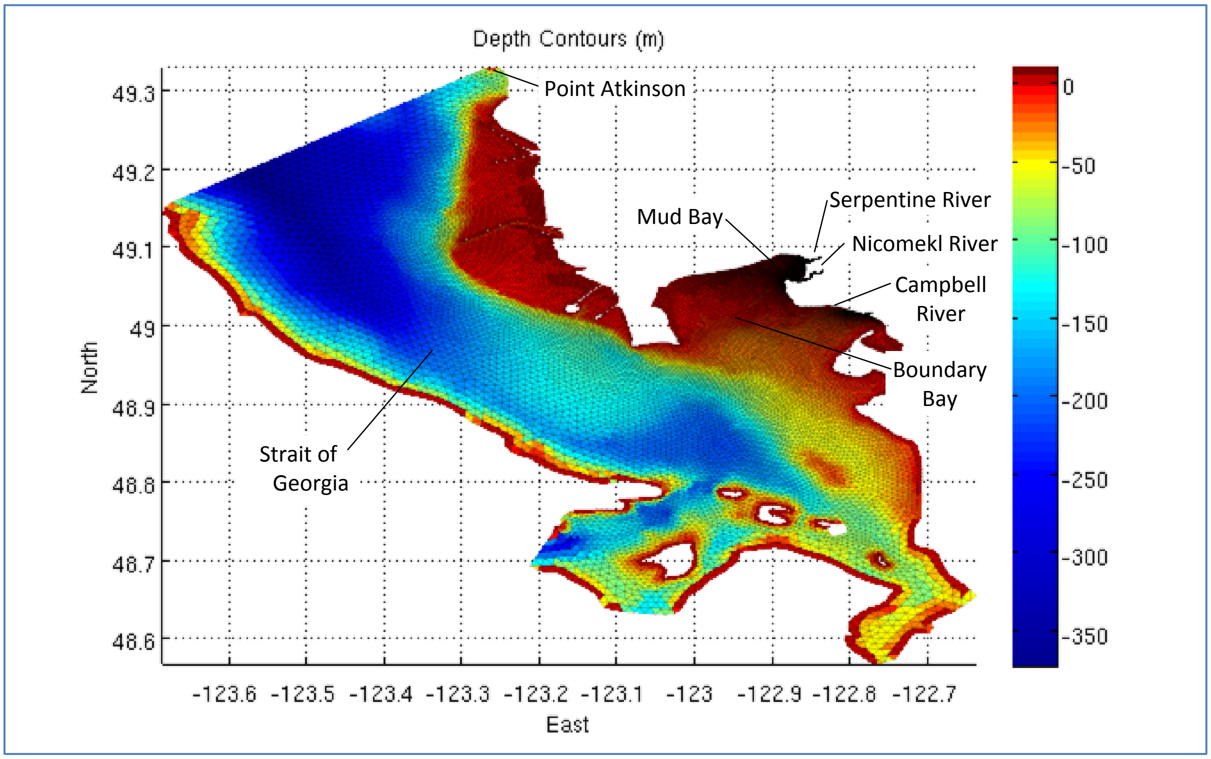


Figure 5.5. Wind-surge and wave computational grid

The RiCOM wind-surge model was setup with water levels along the northern boundary specified as 0 m so that the results could be referenced to Point Atkinson data. Radiation boundary conditions, which allow waves and currents to realistically pass, were specified along the southern boundaries around Saturna Island and through the San Juan Islands.

The hind-cast was performed in a quasi-static manner. A range of separate simulations were performed with a uniform constant wind field. Each simulation was performed with a different wind speed ranging from 10 to 25 m/s and a direction ranging around the compass. In each simulation the water level in Boundary Bay was allowed to reach a steady state value which occurred in about 5 hours. For each region of interest, the results were then collected into a matrix from which the water level could be interpolated for any wind speed and direction. Note that the results showed a very small response at White Rock which is consistent with the data analysis of the Point Atkinson and White Rock data.

Hourly wind data from the Saturna Island were used as input to the hind-cast. Where Saturna Island data was unavailable data from the Vancouver Airport was used. Typically, the model required about 5 hours to reach a steady-state water level. Accordingly, the wind vectors were filtered using a Godin 4-4-5 filter which has a 5 hour cut-off period. Table 5.1 below gives the maximum wind setup at each of the three regions of interest. Though the wind setup at the mouth of the Campbell was not exactly zero, it was within the expected accuracy of the model and so set to zero. Wind setup is most pronounced where there is significant fetch over very shallow water (mouth of

Serpentine and Nicomekl rivers). The depth drops off quite quickly from the mouth of the Campbell so wind setup is minimal. This conclusion is supported by the Empirical Orthoganal Function Analysis of the White Rock tide station provided in Appendix B.2.

Table 5.1. Maximum wind setup at selected locations

Location	Maximum wind setup (m)
Mouth of Serpentine River	0.60 m
Mouth of Nicomekl River	0.32 m
Mouth of Campbell River	0.00 m

A non-stationary simulation of wind-surge over the entire hind-cast period would require considerable computational time but would provide a more accurate method to assess wind setup. This approach may be of interest for future work.

5.5.4 Hind-Cast Assembly and Validation

Ocean water level hind-casts were generated for three separate locations:

- Mouth of the Nicomekl River;
- Mouth of the Serpentine River; and
- Mouth of the Campbell River.

Each water level hind-cast was assembled as the sum of the tidal, external surge and wind setup signals at 1-hour intervals from 1964-2011.

As a means of validating the results, the water level hind-cast at the Serpentine River was compared to the available recorded measurements at the Serpentine Sea Dam (nominally, all time 2000 to 2011 when Sea Dam data available). To minimize the influence of the river on the measurements, only the maximum water levels attained each day were compared. Statistics of the results are shown in Table 5.2.

Table 5.2. Comparison of measured and modelled maximum water levels

Bias	0.003 m
RMS Error	0.12 m
Correlation Coefficient	89%

Figure 5.6 shows a scatter plot of the measured and modelled (hind-cast) maximum daily water levels at the mouth of Serpentine River. The results suggest that the hind-casting method is valid for estimating water levels at the Serpentine River outlet and likely also elsewhere in Mud Bay.

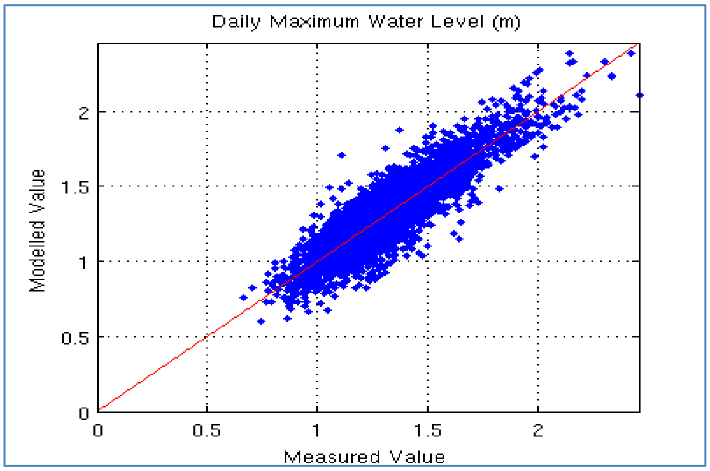


Figure 5.6. Comparison of measured and modelled maximum daily water level at mouth of Serpentine River (Red line indicates a 1:1 relation)

5.6 OCEAN EXTREME LEVEL ESTIMATES

A frequency analysis of the generated ocean level time-series was carried out to estimate the 'Designated Flood Levels' (DFLs).

Extreme water levels were first calculated from each of the compiled water level hind-casts (Serpentine, Nicomekl, Campbell Rivers) using extreme value theory. The peak over threshold approach was used. Using this method, water level events exceeding a threshold of 2.15 m over a 2 hour time threshold were identified. The return period of each event was calculated based on the average time between events. An appropriate statistical distribution was fit to the data, and then the water level corresponding to the desired return period was identified using that distribution. Of the several distributions tested, the Generalized Extreme Value (GEV) distribution fit both the combined water level and the external surge the best. The GEV maximum likelihood fit of the Nicomekl (Crescent Beach) hind-cast water level events with 95% confidence limits and the estimated 100, 200, 500 and 4,000 year water levels is given below in Figure 5.7. Appendix B.4 includes additional detail on the distribution fitting procedure.

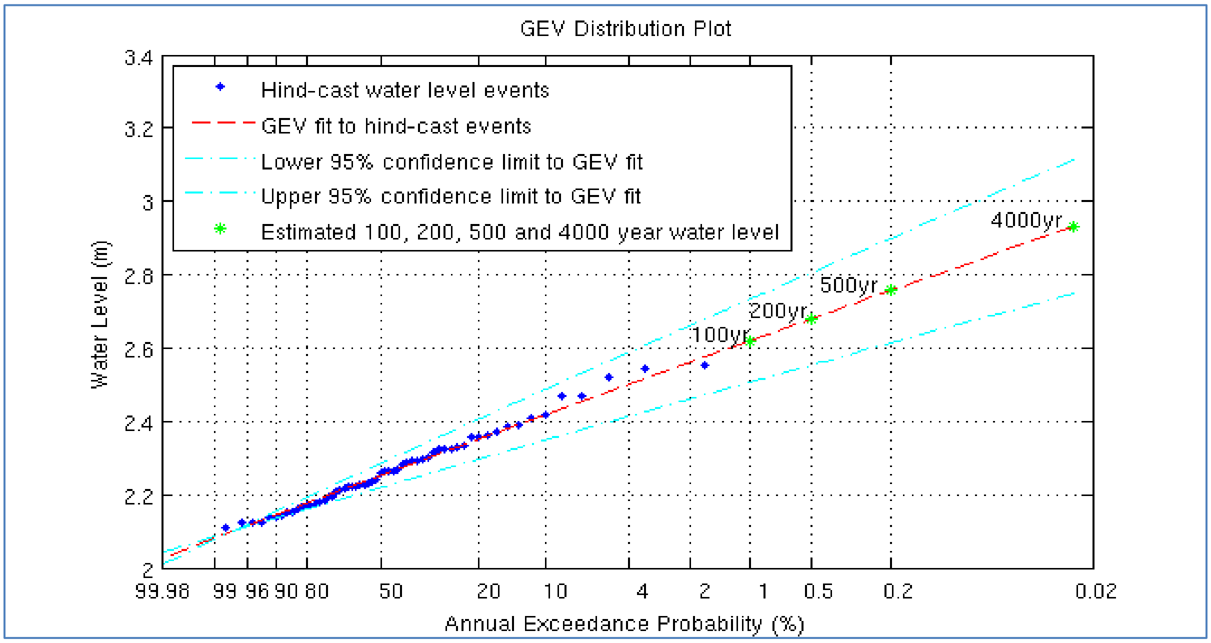


Figure 5.7. Generalized Extreme Value fit of hind-cast water level events and estimated 100, 200, 500 and 4,000 year water levels (Nicomekl – Crescent Beach)

The 100, 200, 500 and 4,000 year water level and surge events near the mouth of the Nicomekl River are listed in Table 5.3 below along with the 95% confidence limits. Water level values for the Serpentine and Campbell Rivers are listed in Table 5.4 and Table 5.5. The difference in extreme water levels between locations is due only to the effect of local wind setup. Since the Salish Sea responds nearly uniformly to external surge, the external surge values are the same for all three locations.

Table 5.3. Extreme water levels and surge levels at the Nicomekl River (95% confidence limits in brackets)

Return Period (years)	AEP (%)	Water Level (m, CGVD28)	External Surge (m, CGVD28)
100	1	2.64 [2.52/2.76]	1.11 [1.02/1.19]
200	**0.5**	**2.70 [2.57/2.83]**	**1.16 [1.06/1.26]**
500	0.2	2.79 [2.63/2.93]	1.24 [1.12/1.36]
4,000	0.025	2.95 [2.77/3.14]	1.46 [1.28/1.62]

Table 5.4. Extreme water levels at the mouth of the Serpentine River (95% confidence limits in brackets)

Return Period (years)	AEP (%)	Water Level (m, CGVD28)
100	1	2.84 [2.69/3.00]
200	0.5	2.94 [2.77/3.12]
500	0.2	3.08 [2.87/3.28]
4,000	0.025	3.43 [3.14/3.71]

Table 5.5. Extreme water levels and surge levels at the mouth of the Campbell River. (95% confidence limits are given in brackets)

Return Period (years)	AEP (%)	Water Level (m, CGVD28)
100	1	2.53 [2.44/2.62]
200	0.5	2.58 [2.48/2.69]
500	0.2	2.65 [2.54/2.77]
4,000	0.025	2.81 [2.66/2.96]

5.7 Wave Conditions and Wave Runup

5.7.1 Locations of Interest

Wave conditions and wave runup information is required to compute Flood Construction Levels (FCLs) and Dike Crest Elevations (DCEs) as described later in this chapter. This information is site specific and at the City's request, the data was compiled for nine locations as listed in Table 5.6, and shown in Figure 5.8.

Table 5.6. Locations for dike crest and flood construction level calculations

Id#	Description	Lat	Lon
1a	Colebrook Dike (Serpentine)	49.085976°	-122.845351°
1b	Crescent Beach East Dike	49.056198°	-122.876770°
1c	Mud Bay Dike (Serpentine)	49.085362°	-122.843113°
1d	Mud Bay Dike (Nicomekl)	49.067271°	-122.841382°
2	Colebrook Dike (near Hwy99)	49.090469°	-122.877106°
3	Crescent Beach North Dike	49.058776°	-122.884280°
4	Crescent Beach South Dike	49.051695°	-122.885258°
5	BNSF Railway (acting as a dike)	49.072965°	-122.858577°
6	Campbell River @ 8th Ave	49.016441°	-122.778356°

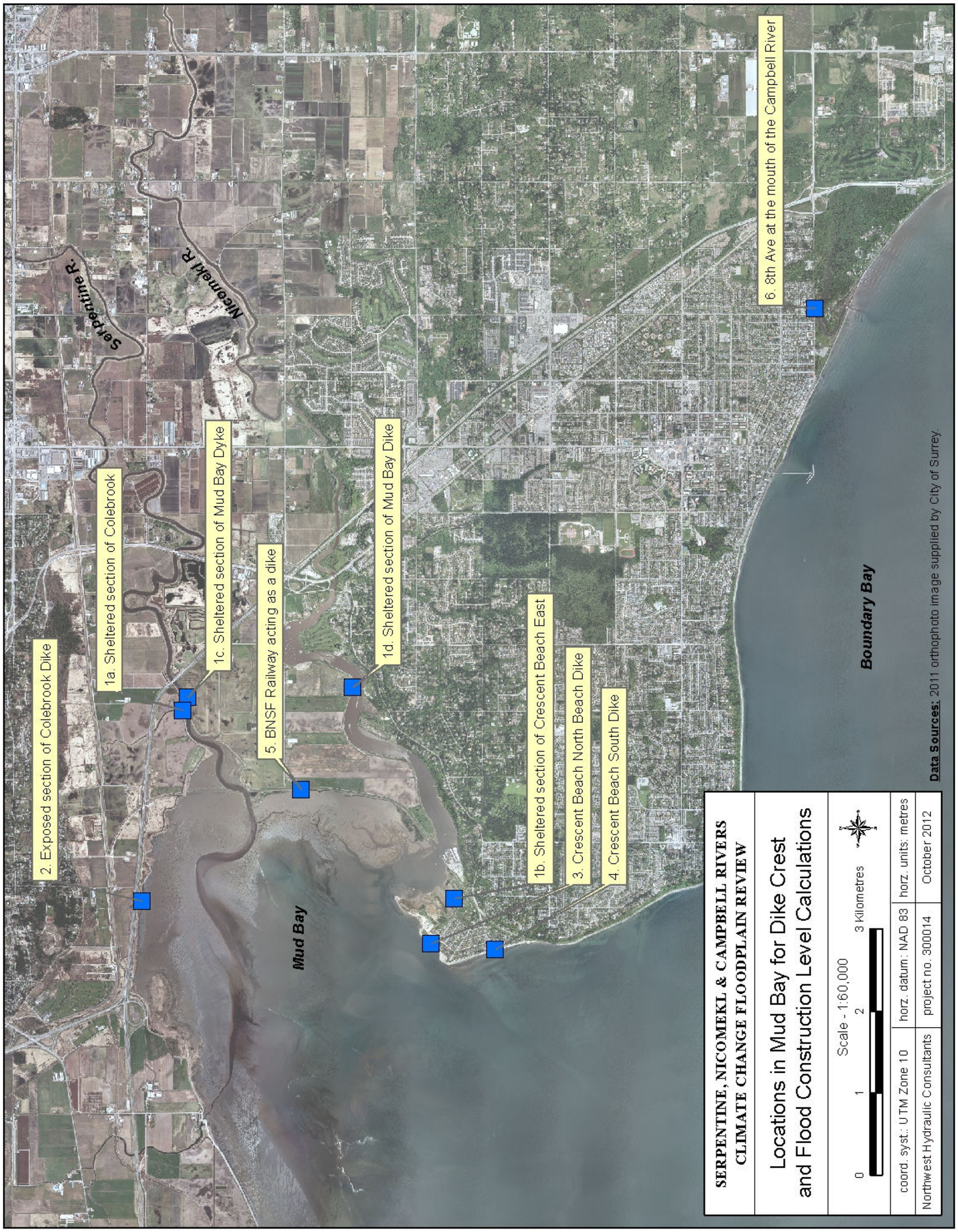


Figure 5.8. Locations in Mud Bay for dike crest and flood construction level calculations

5.7.2 SWAN Modelling

The FCL and DCE calculations require quantifying wave setup and runup during the designated storm. To calculate wave runup it is necessary to specify the wave conditions at the toe of the dike. Limited guidance is provided in the Provincial Sea Dike Guidelines as to the wave condition that should be used, but it is suggested that as a starting point the wave condition should be assumed great enough to be depth limited. Given the sheltered nature of most of Boundary Bay it was anticipated that the depth limited wave condition would lead to overly conservative wave runup estimates. To better understand the wave conditions, a wave model was developed using the SWAN wave modelling software (Booij et al 1999). The model uses the same grid as the RiCOM wind setup model shown in Figure 5.5. The model is driven by winds measured at Saturna Island (48.78°N,123.05°W).

Five large storms, estimated to produce the largest wave heights at each of the locations of interest, were selected to be run in the wave model. The date and wind velocity associated with each storm is given in Table 5.7. Details on how these storms were selected are given in Appendix B.5. The water level for each run was specified based on the 200-year water level for each location. To simulate wave events in the year 2100 relative sea level rise was added to the 200-year water level. Although during some runs, the water level was not exactly that of the 200-year event, the differences are small and will have negligible effect on the results.

The wave model results for the year 2010 and the year 2100 water levels are given in the Table 5.8 and Table 5.9 below. For each location, only the result corresponding to the largest wave height in any of the simulated storms is given. In this way the designated storm for one location may be different than for another location. The label *Hm0* is significant wave height, *Tm01* and *Tm02* are the first and second moment spectral wave periods and θp is the peak wave direction.

The wave heights and periods for the year 2100 water level are larger compared to the same parameters in the year 2010 scenario. This occurs because the greater water depth in the year 2100 scenario reduces the effect of depth induced breaking. In some cases there is no increase in wave height between the two scenarios. In these cases wind speed and fetch, not depth, are the factors limiting wave height.

Table 5.7. Wind speed and direction of selected storms

Date	Max Wind speed (m/s)	Nom. Wind Direction (deg)
1982-12-16	22.78	230
1991-11-16	26.39	180
1994-03-21	24.17	210
1998-11-25	26.67	170
2007-11-12	29.72	160

Table 5.8. Largest wave condition at each location for 2010 water level scenario

#	LOCATION	Storm	Water Level (m)	Hm0 (m)	Tm01 (sec)	Tm02 (sec)	Θp (deg)
1a	Colebrook – Serpentine	1994	2.94	0.28	1.32	1.13	245
1b	Crescent Beach East	1982	2.70	0.48	1.60	1.33	285
1c	Mud Bay - Serpentine	1994	2.94	0.24	1.17	1.00	265
1d	Mud Bay – Nicomekl	1982	2.70	0.23	1.26	1.08	275
2	Colebrook (Hwy99)	1991	2.94	0.85	2.41	1.99	195
3	Crescent Beach North	1982	2.70	0.69	2.06	1.72	255
4	Crescent Beach South	2007	2.70	1.15	5.22	4.16	195
5	BNSF Railway	1994	2.94	0.73	2.07	1.71	245
6	8th Ave @ Campbell	1991	2.58	0.22	0.98	0.84	235

Table 5.9. Largest wave condition at each location for 2100 water level scenario

#	LOCATION	Storm	Water Level (m)	Hm0 (m)	Tm01 (sec)	Tm02 (sec)	Θp (deg)
1a	Colebrook – Serpentine	1998	4.09	0.26	1.61	1.42	245
1b	Crescent Beach East	1982	3.70	0.54	1.88	1.44	285
1c	Mud Bay - Serpentine	1982	4.09	0.21	2.48	2.45	275
1d	Mud Bay – Nicomekl	1982	3.90	0.23	1.25	1.07	275
2	Colebrook (Hwy99)	1994	4.09	1.19	2.74	2.23	315
3	Crescent Beach North	1982	3.70	0.81	2.24	1.85	355
4	Crescent Beach South	2007	3.70	1.50	5.39	4.49	345
5	BNSF Railway	1982	4.09	0.86	2.31	1.91	355
6	8th Ave @ Campbell	1982	3.58	0.23	3.47	3.33	255

5.7.3 WAVE RUNUP CALCULATIONS

The BC Provincial Sea Dike Guidelines accept the use of a number of criteria for calculation of the wave runup component of the Dike Crest Elevation. For this study, the 2% exceedance level was adopted as a general purpose measure.

Combined wave setup and runup calculations were made using the *PC Overtopping* software of the European Overtopping Manual (Pullen 2007). Wave runup is an extremely complex process and current calculation methods are empirically based. These calculations are meant to provide a baseline estimate of expected wave runup as they currently exist. They are not intended to provide information for detailed dike design or upgrades. The PC Overtopping runup calculations, while informative, should be supplemented with hydraulic model testing to ensure that the dike design will perform as required.

Within *PC Overtopping*, each dike was modelled as an armoured slope with a simplified geometry. The geometry of each dike was idealized from a dike cross section extracted from high density LIDAR data. As an example, the cross section of the Crescent Beach South Dike is shown in Figure 5.9 along with its simplified representation. For other dike geometries see Appendix B.6.

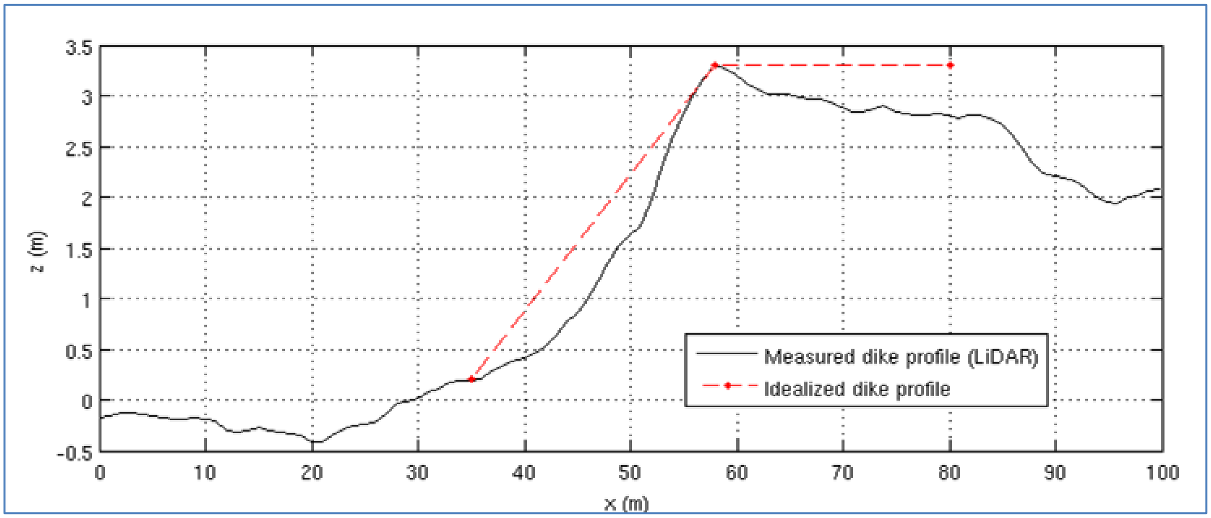


Figure 5.9. Measured and idealized dike profile at Crescent Beach South Dike

From the modelled storms, the maximum wave condition at the toe of each dike was used to estimate wave runup. The parameter θ_R indicates the angle between the shoreline-normal and the wave direction (i.e. $\theta_R = 0$ degrees indicates the wave is propagating directly towards shore).

Where the dike is underwater, *2% wave runup* is given as *inf.* For the 2100 water level scenario most dikes were underwater. To achieve more meaningful results in the year 2100 water level scenario, 1.5 m was arbitrarily added to the crest of each dike (keeping the existing toe fixed and raising the existing crest vertically which increased the slope somewhat) so that the dike is not overtopped.

Table 5.10. Wave runup at locations of interest for 2010 water level scenario

#	LOCATION	Water level (m)	Hm0 (m)	Tm02 (sec)	θ_R (deg)	2% Runup (m)
1a	Colebrook – Serpentine	2.94	0.28	1.13	70	0.30
1b	Crescent Beach East	2.70	0.48	1.33	0	0.45
1c	Mud Bay - Serpentine	2.94	0.24	1.00	80	0.31
1d	Mud Bay – Nicomekl	2.70	0.23	1.08	90	0.21
2	Colebrook (Hwy99)	2.94	0.85	1.99	0	0.33
3	Crescent Beach North	2.70	0.69	1.72	60	0.65
4	Crescent Beach South	2.70	1.15	4.16	0	0.86
5	BNSF Railway	2.94	0.73	1.71	25	0.57
6	8th Ave @ Campbell	2.58	0.22	0.84	55	Inf

Table 5.11. Wave runup at locations of interest for 2100 water level scenario

#	LOCATION	Water Level (m)	Hm0 (m)	Tm02 (sec)	θ_R (deg)	2% Runup (m)
1a	Colebrook – Serpentine	4.14	0.26	1.42	70	0.39
1b	Crescent Beach East	3.70	0.54	1.44	0	0.60
1c	Mud Bay - Serpentine	4.14	0.21	2.45	65	0.48
1d	Mud Bay – Nicomekl	3.90	0.23	1.07	90	0.24
2	Colebrook (Hwy99)	4.14	1.19	2.23	0	2.02
3	Crescent Beach North	3.70	0.81	1.85	0	1.33
4	Crescent Beach South	3.70	1.50	4.49	0	1.50
5	BNSF Railway	4.14	0.86	1.91	0	1.13
6	8th Ave @ Campbell	3.58	0.23	3.33	80	0.67

5.8 Calculations of DFL, FCL and DCE

5.8.1 Provincial Guideline Method

Definitions of the Designated Flood Level (DFL), Flood Construction Level (FCL) and Dike Crest Elevation (DCE) were provided in the provincial guidelines by Ausenco Sandwell (2011) and for reference are included in Appendix B.7. The parameters are commonly used for the design of dikes. According to the definitions, the DFL, FCL and DCE, for a certain point in time, can be estimated as follows:

Designated Flood Level (DFL)	=	Relative Sea Level Rise (RSLR)	+	HHWLT	+	200yr Storm Surge	+	Max Wind Setup	- Eqn. 5.1

Flood Construction Level (FCL)	=	DFL	+	0.5 x 2% Runup	+	Freeboard (0.6 m)	- Eqn. 5.2

Dike Crest Elevation (DCE)	=	DFL	+	2% Runup	+	Freeboard (0.6 m)	- Eqn. 5.3

According to the guidelines, the DFL is calculated by summing the relative sea level rise, the HHWLT, the 0.005 AEP (200-year) storm surge and the maximum local wind setup. The FCL is equal to the DFL, with half of the 2% runup and a freeboard allowance of typically 0.6 m added. For the DCE, the computations are the same, except the entire 2% runup is included.

The guidelines suggest that there is about a 1/20 (0.05 AEP) chance that the 200-year storm surge coincides with the HHWLT, reducing the joint probability of the two conditions to 0.05 x 0.005 = 0.00025 AEP or a 4,000 year return period. Further combining the HHWLT and the 200-year storm surge with the maximum wind setup, reduces the combined probability considerably. In other words, DFLs calculated in this manner do not correspond to a 200-year return period but rather to a return period in the order of maybe 10,000 years.

Calculations for Year 2010

The DFLs for Mud Bay were first computed using the provincial guideline method. Following the guidelines, the DFL for the 2010 base year (no sea level rise) was computed by adding:

- The HHWLT of 1.74 m, computed in Section 5.5;
- The 200-year storm surge of 1.16 m, estimated in Section 5.6; and,
- The maximum wind setups (0.60 m for the Serpentine, 0.32 m for the Nicomekl and 0 m for the Campbell River).

Parameters for Eqn. 5.1 and the DFL results are listed in Table 5.12.

Table 5.12. DFL parameters and results, 2010 scenario using provincial guidelines

#	LOCATION	RSLR (m)	HHWLT (m)	200-year Surge (m)	Max Wind Setup (m)	DFL (m) Prov. Guidelines
1a	Colebrook – Serpentine	0	1.74	1.16	0.60	3.50
1b	Crescent Beach East	0	1.74	1.16	0.32	3.22
1c	Mud Bay - Serpentine	0	1.74	1.16	0.60	3.50
1d	Mud Bay – Nicomekl	0	1.74	1.16	0.32	3.22
2	Colebrook (Hwy99)	0	1.74	1.16	0.60	3.50
3	Crescent Beach North	0	1.74	1.16	0.32	3.22
4	Crescent Beach South	0	1.74	1.16	0.32	3.22
5	BNSF Railway	0	1.74	1.16	0.60	3.50
6	8th Ave @ Campbell	0	1.74	1.16	0.00	2.90

Calculations for the FCLs and DCEs (see Eqn. 5.2 and 5.3) were then performed. The 2% wave runup values provided in Table 5.10 were required for these calculations. A free board allowance of 0.6 m was incorporated. Parameters for the calculations and results are summarized in Table 5.13 and Table 5.14.

Table 5.13. FCL parameters and results, 2010 scenario using provincial guidelines

#	LOCATION	DFL (m) Prov. Guidelines	0.5 x 2% Runup	Freeboard 0.6 m	FCL (m) Prov. Guidelines
1a	Colebrook – Serpentine	3.50	0.15	0.60	4.25
1b	Crescent Beach East	3.22	0.21	0.60	4.04
1c	Mud Bay - Serpentine	3.50	0.15	0.60	4.25
1d	Mud Bay – Nicomekl	3.22	0.11	0.60	3.93
2	Colebrook (Hwy99)	3.50	0.16	0.60	4.26
3	Crescent Beach North	3.22	0.32	0.60	4.14
4	Crescent Beach South	3.22	0.43	0.60	4.25
5	BNSF Railway	3.50	0.29	0.60	4.39
6	8th Ave @ Campbell	2.90	-	0.60	NA

Table 5.14. DCE parameters and results, 2010 scenario using provincial guidelines

#	LOCATION	DFL (m) Prov. Guidelines	2% Runup	Freeboard 0.6 m	DCE (m) Prov. Guidelines
1a	Colebrook – Serpentine	3.50	0.30	0.60	4.40
1b	Crescent Beach East	3.22	0.45	0.60	4.27
1c	Mud Bay - Serpentine	3.50	0.31	0.60	4.41
1d	Mud Bay – Nicomekl	3.22	0.21	0.60	4.03
2	Colebrook (Hwy99)	3.50	0.33	0.60	4.43
3	Crescent Beach North	3.22	0.65	0.60	4.47
4	Crescent Beach South	3.22	0.86	0.60	4.68
5	BNSF Railway	3.50	0.57	0.60	4.67
6	8th Ave @ Campbell	2.90	-	0.60	NA

Calculations for Year 2100

The calculations were repeated for year 2100, incorporating relative sea level rise. Section 3.1 provided a sea level rise estimate of 0.97 m from 2010 to 2100. For locations where ground subsidence is expected (Section 4.3), the relative sea level rise is anticipated to increase an additional 0.225 m or 1.2 m (0.97m + 0.225m). Relative sea level rise values and other parameters for the DFL calculations are listed in Table 5.15. FCL calculations are summarized in Table 5.16 and DCE calculations in Table 5.17.

As ocean depths increase, wave runup will also increase as shown in Table 5.11 (Section 5.7). The wave runup for year 2100 is included in Table 5.16 and Table 5.17.

Table 5.15. DFL parameters and results, 2100 scenario using provincial guidelines

#	LOCATION	RSLR (m)	HHWL T (m)	200-year Surge (m)	Max Wind Setup (m)	DFL (m) Prov. Guidelines
1a	Colebrook – Serpentine	1.2	1.74	1.16	0.60	4.70
1b	Crescent Beach East	1.0	1.74	1.16	0.32	4.22
1c	Mud Bay - Serpentine	1.2	1.74	1.16	0.60	4.70
1d	Mud Bay – Nicomekl	1.2	1.74	1.16	0.32	4.42
2	Colebrook (Hwy99)	1.2	1.74	1.16	0.60	4.70
3	Crescent Beach North	1.0	1.74	1.16	0.32	4.22
4	Crescent Beach South	1.0	1.74	1.16	0.32	4.22
5	BNSF Railway	1.2	1.74	1.16	0.60	4.70
6	8th Ave @ Campbell	1.0	1.74	1.16	0.00	3.90

Table 5.16. FCL parameters and results, 2100 scenario using provincial guidelines

#	LOCATION	DFL (m) Prov. Guidelines	0.5 x 2% Runup	Freeboard 0.6 m	FCL (m) Prov. Guidelines
1a	Colebrook – Serpentine	4.70	0.190	0.60	5.49
1b	Crescent Beach East	4.22	0.30	0.60	5.12
1c	Mud Bay - Serpentine	4.70	0.24	0.60	5.54
1d	Mud Bay – Nicomekl	4.42	0.12	0.60	5.14
2	Colebrook (Hwy99)	4.70	1.01	0.60	6.31
3	Crescent Beach North	4.22	0.67	0.60	5.49
4	Crescent Beach South	4.22	0.75	0.60	5.57
5	BNSF Railway	4.70	0.57	0.60	5.87
6	8th Ave @ Campbell	3.90	0.33	0.60	4.83

Table 5.17. DCE parameters and results, 2100 scenario using provincial guidelines

#	LOCATION	DFL (m) Prov. Guidelines	2% Runup	Freeboard 0.6 m	DCE (m) Prov. Guidelines
1a	Colebrook – Serpentine	4.70	0.39	0.60	5.69
1b	Crescent Beach East	4.22	0.60	0.60	5.42
1c	Mud Bay - Serpentine	4.70	0.48	0.60	5.78
1d	Mud Bay – Nicomekl	4.42	0.24	0.60	5.26
2	Colebrook (Hwy99)	4.70	2.02	0.60	7.32
3	Crescent Beach North	4.22	1.33	0.60	6.15
4	Crescent Beach South	4.22	1.50	0.60	6.32
5	BNSF Railway	4.70	1.13	0.60	6.43
6	8th Ave @ Campbell	3.90	0.67	0.60	5.17

5.8.2 JOINT PROBABILITY METHOD

Calculations for Year 2010

Next, the DFL, FCL and DCE calculations were performed using the joint probability method. In this case the 200-year return period water levels from the extreme value analysis described in Section 5.6 (Table 5.3 to Table 5.5) correspond directly to the DFL. The water levels are listed in Table 5.18 and show the reduction in water levels compared to the Provincial guideline method. On average, the joint probability method gives a 0.52 m lower DFL.

Table 5.18. DFL results, 2010 scenario joint probability approach

#	LOCATION	RSLR (m)	DFL (m) Joint Prob.	Reduction (m) from Prov. guidelines
1a	Colebrook – Serpentine	0	2.94	0.56
1b	Crescent Beach East	0	2.70	0.52
1c	Mud Bay - Serpentine	0	2.94	0.56
1d	Mud Bay – Nicomekl	0	2.70	0.52
2	Colebrook (Hwy99)	0	2.94	0.56
3	Crescent Beach North	0	2.70	0.52
4	Crescent Beach South	0	2.70	0.52
5	BNSF Railway	0	2.94	0.56
6	8th Ave @ Campbell	0	2.58	0.32

The computations for the FCL and DCE are the same for both the Provincial guideline and joint probability methods. The only variation is in the DFL value. FCL and DCE parameters and results are provided in Table 5.19 and Table 5.20. The reduction in elevations compared to the Provincial guideline FCL and DCE values is the same as for the DFL.

Table 5.19. FCL parameters and results, 2010 scenario using joint probability approach

#	LOCATION	DFL (m) Joint Prob.	0.5 x 2% Runup	Freeboard 0.6 m	FCL (m) Joint Prob.
1a	Colebrook – Serpentine	2.94	0.15	0.60	3.69
1b	Crescent Beach East	2.70	0.21	0.60	3.52
1c	Mud Bay - Serpentine	2.94	0.15	0.60	3.69
1d	Mud Bay – Nicomekl	2.70	0.11	0.60	3.41
2	Colebrook (Hwy99)	2.94	0.16	0.60	3.70
3	Crescent Beach North	2.70	0.32	0.60	3.62
4	Crescent Beach South	2.70	0.43	0.60	3.73
5	BNSF Railway	2.94	0.29	0.60	3.82
6	8th Ave @ Campbell	2.58	-	0.60	NA

Table 5.20. DCE parameters and results, 2010 scenario using joint probability approach

#	LOCATION	DFL (m) Joint Prob.	2% Runup	Freeboard 0.6 m	DCE (m) Joint Prob.
1a	Colebrook – Serpentine	2.94	0.30	0.60	3.84
1b	Crescent Beach East	2.70	0.45	0.60	3.75
1c	Mud Bay - Serpentine	2.94	0.31	0.60	3.85
1d	Mud Bay – Nicomekl	2.70	0.21	0.60	3.51
2	Colebrook (Hwy99)	2.94	0.33	0.60	3.87
3	Crescent Beach North	2.70	0.65	0.60	3.95
4	Crescent Beach South	2.70	0.86	0.60	4.16
5	BNSF Railway	2.94	0.57	0.60	4.11
6	8th Ave @ Campbell	2.58	-	0.60	NA

Calculations for Year 2100

Joint probability method calculations for 2100 DFLs, FCLs and DCEs are listed in Table 5.21, Table 5.22 and Table 5.23.

Table 5.21. DFL results, 2100 scenario joint probability approach

#	LOCATION	RSLR (m)	200-year WL (m)	DFL (m) Joint Prob.
1a	Colebrook – Serpentine	1.2	2.94	4.14
1b	Crescent Beach East	1.0	2.70	3.70
1c	Mud Bay - Serpentine	1.2	2.94	4.14
1d	Mud Bay – Nicomekl	1.2	2.70	3.90
2	Colebrook (Hwy99)	1.2	2.94	4.14
3	Crescent Beach North	1.0	2.70	3.70
4	Crescent Beach South	1.0	2.70	3.70
5	BNSF Railway	1.2	2.94	4.14
6	8th Ave @ Campbell	1.0	2.58	3.58

Table 5.22. FCL parameters and results, 2100 scenario using joint probability approach

#	LOCATION	DFL (m) Joint Prob.	0.5 x 2% Runup	Freeboard 0.6 m	FCL (m) Joint Prob.
1a	Colebrook – Serpentine	4.14	0.20	0.60	4.94
1b	Crescent Beach East	3.70	0.30	0.60	4.60
1c	Mud Bay - Serpentine	4.14	0.24	0.60	4.98
1d	Mud Bay – Nicomekl	3.90	0.12	0.60	4.62
2	Colebrook (Hwy99)	4.14	1.01	0.60	5.75
3	Crescent Beach North	3.70	0.67	0.60	4.97
4	Crescent Beach South	3.70	0.75	0.60	5.05
5	BNSF Railway	4.14	0.56	0.60	5.31
6	8th Ave @ Campbell	3.58	0.34	0.60	4.51

Table 5.23. DCE parameters and results, 2100 scenario using joint probability approach

#	LOCATION	DFL (m) Joint Prob.	2% Runup	Freeboard 0.6 m	DCE (m) Joint Prob.
1a	Colebrook – Serpentine	4.14	0.39	0.60	5.13
1b	Crescent Beach East	3.70	0.60	0.60	4.90
1c	Mud Bay - Serpentine	4.14	0.48	0.60	5.22
1d	Mud Bay – Nicomekl	3.90	0.24	0.60	4.74
2	Colebrook (Hwy99)	4.14	2.02	0.60	6.76
3	Crescent Beach North	3.70	1.33	0.60	5.63
4	Crescent Beach South	3.70	1.50	0.60	5.80
5	BNSF Railway	4.14	1.13	0.60	5.87
6	8th Ave @ Campbell	3.58	0.67	0.60	4.85

5.8.3 SUMMARY

Current City dike elevations range from 2.3 m to 3.3 m. Figure 5.10 and Table 5.24 compare the existing dike crest elevations to the computed DCE's for both methods and timeframes.

Protecting against sea level rise over the next 90 years will be a challenge for most BC coastal communities. Simultaneously increasing the design standard from the current typical 200-year level to approximately the 10,000-year level (if the approach in the Provincial guidelines is adopted) is likely not going to be practical.

Wave runup is an extremely complex process and current calculation methods are empirically based. It is the wave runup calculations that alone, are insufficient to inform detailed dike design or upgrades. Accordingly only the DCE and FCL (not DFL) are affected. The empirical methods used in this study, while informative, should be supplemented with more detailed numerical analysis or hydraulic model testing ensure that the dike design will perform as required.

FCLs and DCEs provided in this chapter should be considered preliminary and are not for construction.

Table 5.24. Comparison of existing dike elevations to computed DCE's

#	LOCATION	Existing Crest Elev. (m)	Provincial Guidelines		Joint Probability	
			DCE (m)	Diff. (m)	DCE (m)	Diff. (m)
Existing Conditions 2010						
1a	Colebrook – Serpentine	2.84	4.40	1.56	3.84	1.00
1b	Crescent Beach East	2.88	4.27	1.39	3.75	0.87
1c	Mud Bay - Serpentine	3.00	4.41	1.41	3.85	0.85
1d	Mud Bay – Nicomekl	2.98	4.03	1.05	3.51	0.53
2	Colebrook (Hwy99)	3.15	4.43	1.28	3.87	0.72
3	Crescent Beach North	2.90	4.47	1.57	3.95	1.05
4	Crescent Beach South	3.30	4.68	1.38	4.16	0.86
5	BNSF Railway	3.20	4.67	1.47	4.11	0.91
6	8th Ave @ Campbell	2.30	NA	-	NA	-
Future Conditions 2100						
1a	Colebrook – Serpentine	2.84	5.69	2.85	5.13	2.29
1b	Crescent Beach East	2.88	5.42	2.54	4.90	2.02
1c	Mud Bay - Serpentine	3.00	5.78	2.78	5.22	2.22
1d	Mud Bay – Nicomekl	2.98	5.26	2.28	4.74	1.76
2	Colebrook (Hwy99)	3.15	7.32	4.17	6.76	3.61
3	Crescent Beach North	2.90	6.15	3.25	5.63	2.73
4	Crescent Beach South	3.30	6.32	3.02	5.80	2.50
5	BNSF Railway	3.20	6.43	3.23	5.87	2.67
6	8th Ave @ Campbell	2.30	5.17	2.87	4.85	2.55

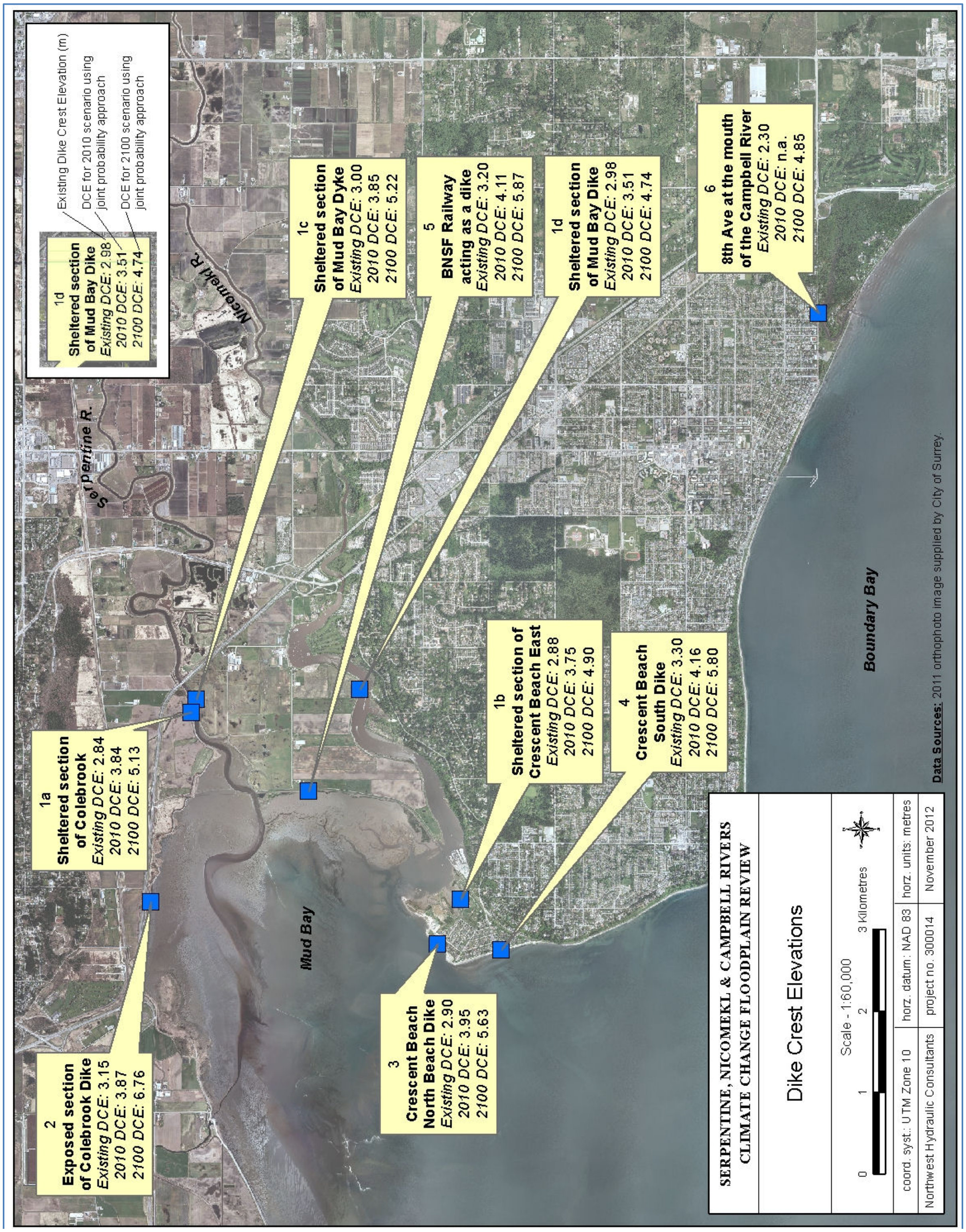


Figure 5.10. Dike crest elevations

6 HYDROLOGIC MODELLING

Hydrologic modelling was carried out to generate inflows to the continuous simulation hydraulic model. Similarly to the ocean modelling, a time-series of flows covering the period from 1964 to 2011 was generated for all key inflow locations. Since streamflow information in the Serpentine and Nicomekl basins is sparse, available precipitation and evaporation data was applied to a hydrologic model in order to produce the required inflow data.

USEPA's HSPF continuous hydrologic model package was used, which offers a number of advantages over other similar packages. Most significantly, HSPF has been widely used in the Pacific Northwest for modelling the hydrologic impacts of land use change, and generalized model parameter estimates are available for a range of land cover and soil type complexes, including those found in the Nicomekl and Serpentine watersheds (Dinicola 1990). The availability of widely-tested generalized model parameters provides more confidence in simulation of ungauged areas and also facilitates model calibration by providing good initial estimates of model parameters.

The HSPF model requires input time series of precipitation and evaporation data and delineation of the basin into pervious and impervious soil-land cover complexes that characterize land surface rainfall-runoff response. The following sections document development and calibration of the HSPF model for the Serpentine and Nicomekl watersheds and summarize results of the modelling for existing and future land use conditions.

6.1 HSPF MODEL DEVELOPMENT

6.1.1 METEOROLOGIC DATA

Precipitation Data

The availability of hourly precipitation data within the lower mainland was reviewed with a particular focus on the availability of long-term records within or in close proximity to the Nicomekl and Serpentine River watersheds. Hourly records potentially suitable for use in hydrologic modelling for this study were available from the City of Surrey, Township of Langley, and Environment Canada and are listed in Table 6.1. Hourly precipitation data are also available from the US National Weather Service station at Blaine. The station locations are shown in Figure 6.1. Note that the long-term stations (those with 20 or more years of record) listed in Table 6.1 now operated by the City were all formerly operated by Environment Canada.

Table 6.1. Hourly precipitation stations

Station Name	Operated By	ID	Period of Record	
			from	to
Abbotsford Airport	Environment Canada	1100030	1976	2002
Pitt Meadows STP	Environment Canada	110FAG9	1974	1993
White Rock STP	Environment Canada	1108914	1964	2002
White Rock STP	City of Surrey		1997	2012
Surrey Municipal Hall	Environment Canada	1107876	1963	2000
Surrey Municipal Hall	City of Surrey		1997	2012
Surrey Kwantlen Park	Environment Canada	1107873	1962	1999
Surrey Kwantlen Park	City of Surrey		1997	2012
Vancouver Int'l Airport	Environment Canada	1108447	1960	2006
Township of Langley	Township of Langley		2005	2012
Chantrell Creek at 32nd Ave	City of Surrey		2000	2012
Port Kells Pump Station	City of Surrey		2004	2012
Semiahmoo Fish & Game Club	City of Surrey		2000	2012
Blaine	NOAA	450729	1948	2011

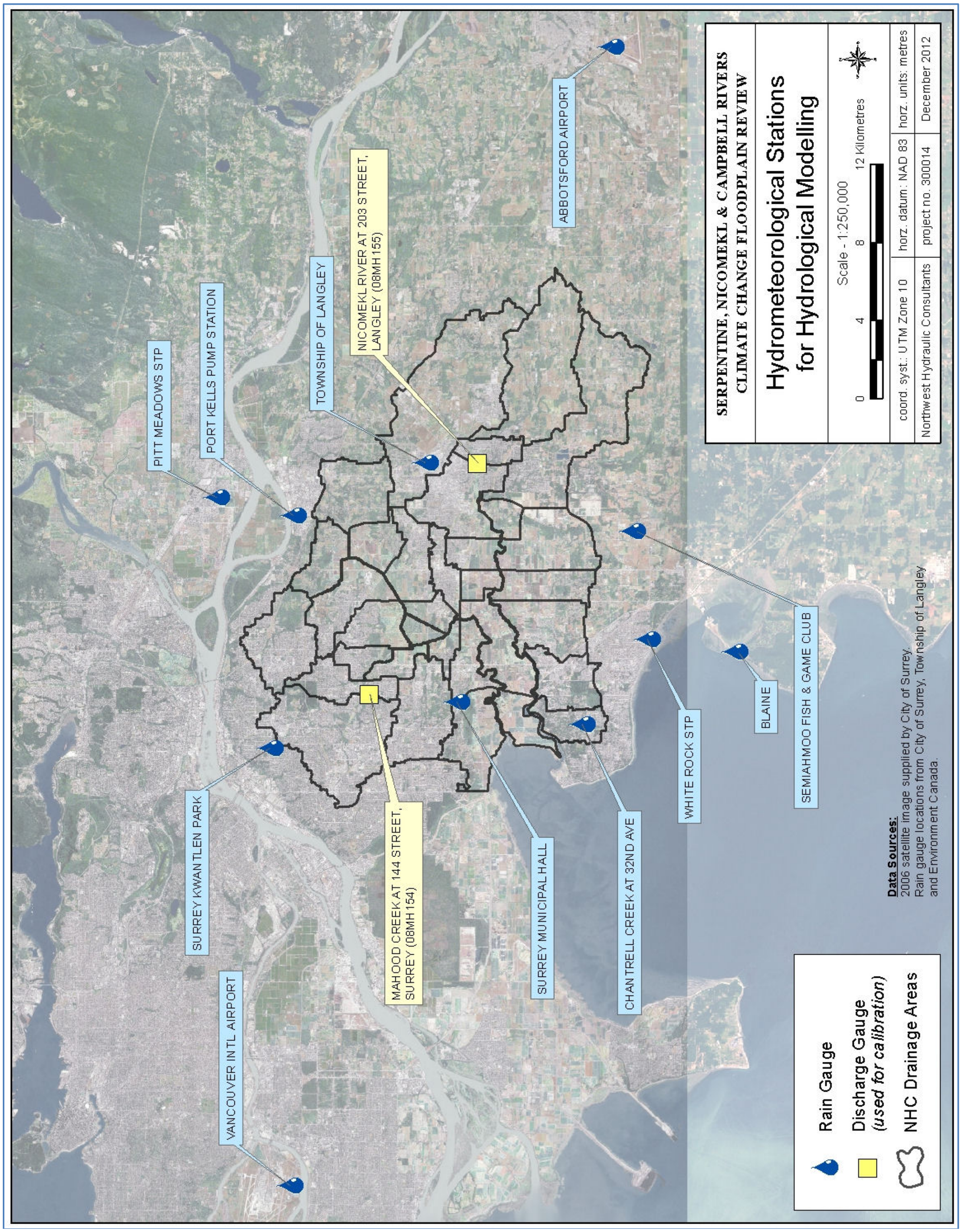


Figure 6.1. Hydrometeorological stations for hydrological modelling

Hourly precipitation data for the period of available record were obtained from the City for all gauges operated by the City and from the Township of Langley for its gauge. Hourly precipitation data from the Environment Canada stations were obtained for the period of record from Environment Canada Climate Services. A composite record for each of the long-term Surrey stations was then created by combining the Environment Canada record from the start of record through September 1997 with the City record from October 1997 onward. The date selected on which to switch from use of Environment Canada data to City of Surrey data was based on consideration of the quality of record from the two overlapping data sources.

NHC elected to use precipitation data from a single representative station to drive the HSPF model, with spatial variation of rainfall over the study area being introduced by simple scaling of those data. Surrey Municipal Hall was selected as the primary precipitation station for hydrologic modelling purposes, based on record length, station location and quality of record.

Review of hourly precipitation data from Surrey Municipal Hall showed a number of gaps and periods of missing data in the composite record. Roughly seven percent of the hourly record is missing between the start of the record in March 1963 through December 2011. Gaps in the record were filled by correlation against records from the other long-term rainfall stations listed in Table 6.1. A short period where data were missing from all Environment Canada stations was filled using data from the US National Weather Service station at Blaine. The stations used for data fill-in, and the multiplier applied, are provided below in Table 6.2 in the order in which they were used.

Table 6.2. Basis for precipitation data fill-in for Surrey Municipal Hall

Station used to fill gaps in Surrey Municipal Hall precipitation record	Multiplier applied
Vancouver International Airport	1.13
Surrey Kwantlen Park	0.83
White Rock STP	1.21
Blaine	1.18

Snow melt processes were not modeled for this study, however reported precipitation data prior to and during two large rain-on-snow events were modified to reflect estimated total moisture inputs (i.e. rain plus melt). These events (in February 1986 and January 2009) were significant known rain-on-snow events; it is possible that there are other similar events in the period of record.

After filling data gaps, the continuous record of hourly precipitation data from Surrey Municipal Hall extended from March 1963 through December 2011, allowing for hydrologic modelling for a total of 48 water years (water years 1964 through 2011).

To evaluate the spatial distribution of rainfall across the study area, NHC compared annual rainfall and event totals for the City of Surrey and Township of Langley gauges. For annual rainfall, there is a pronounced north-south (increasing to the north) gradient across the area and little east-west variation. For individual large storm events, spatial distributions vary, but analysis of the seven largest rainfall events since 2003 generally showed higher precipitation in the northern and eastern parts of the study area (Upper Nicomekl watershed and much of the Serpentine watershed) and lower precipitation south of Surrey Municipal Hall. Since the focus of this study is on large events, NHC elected to use rainfall distribution and scaling factors reflecting an “average” large event distribution, as shown in Figure 6.2. The effect of this rainfall distribution on annual volumes was not explored during calibration but would likely result in some degree of oversimulation of annual volumes. Further analysis, and likely modification of the rainfall scaling factors, would be warranted if the model is to be extended to applications beyond large event analysis.

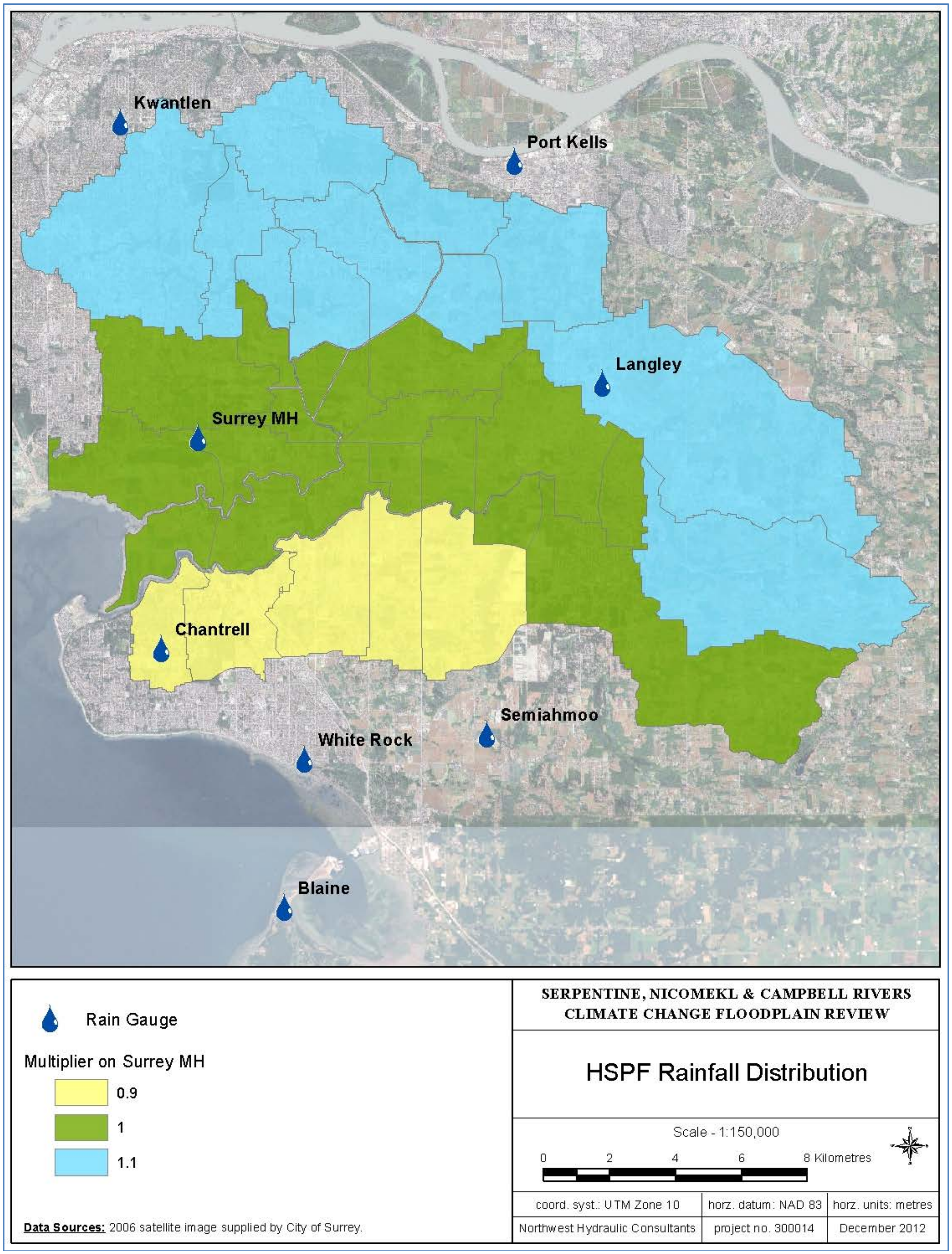


Figure 6.2. HSPF rainfall distribution

Pan Evaporation Data

Daily pan evaporation data from the lower mainland are available from two stations operated by Environment Canada listed in Table 6.3. The available pan evaporation data were ordered from Environment Canada Climate Services. Neither of these monitoring stations is currently active, and no pan evaporation data are available from 1994 to present. The hydrologic model uses pan evaporation data from Vancouver UBC. The published data were screened to remove implausibly large daily values. Gaps in the pan evaporation record were then filled—and the available record extended through 2011—using mean monthly values. Since hydrologic modelling of winter high flows is relatively insensitive to uncertainty in pan evaporation data, the lack of Vancouver UBC data since 1990 is not a significant concern.

Table 6.3. Daily pan evaporation stations

Station Name	Operated By	ID	Period of Record	
			from	to
Agassiz CDA	Environment Canada	1100120	1965	1994
Vancouver UBC	Environment Canada	1108487	1962	1990

6.1.2 SOILS AND LAND USE

Soils/Surficial Geology

The U.S. Geological Survey (USGS) developed regional parameters for the HSPF pervious land segment (PERLND) module by calibrating the model to streamflow from 21 headwater basins in the Puget Sound lowlands of western Washington state (Dinicola, 1990). Parameters were developed for forest, pasture, and grass land covers on glacially-derived (till, outwash) and wetland (saturated) soil types common in the region. In subsequent work, NHC and others have developed parameters representative of alluvial soils typically found in valley bottom areas.

NHC obtained surficial geology data covering the study area from the Geomap Vancouver geologic map. The surficial geology classifications can be readily associated with the HSPF soil types for which regional parameters have been developed. Table 6.4 categorizes the surficial geology types by HSPF soil type. Figure 6.3 shows a map of the study area with surface geology classified by HSPF soil type.

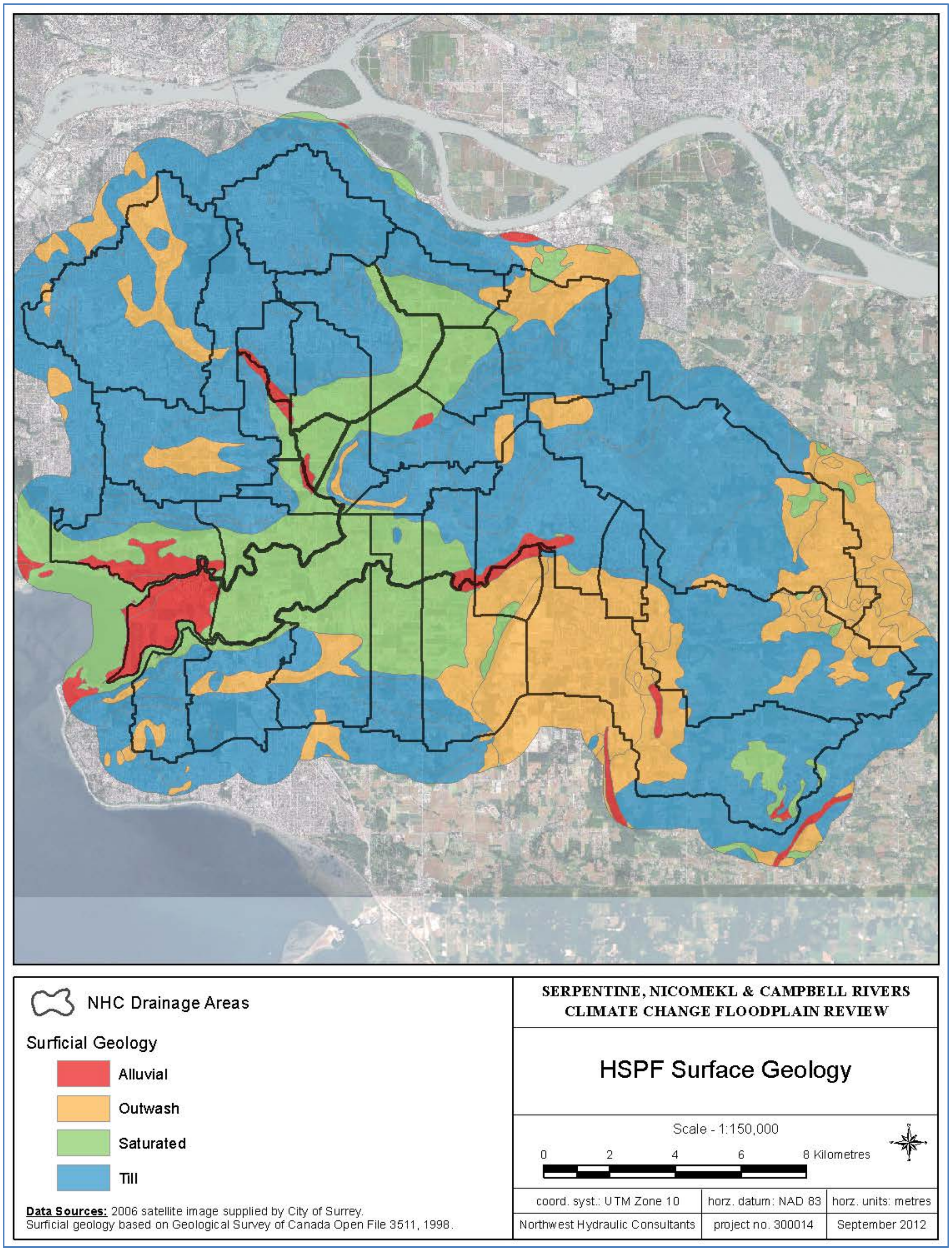


Figure 6.3. HSPF surface geology

Table 6.4. Surface geology classifications

Surficial Geology	HSPF Soil Type
Peat (modern)	Saturated
Gravel & sand (Ice Age)	Outwash
Sand (Ice Age)	Outwash
Sand & silt (modern)	Alluvial (Saturated)[1]
Silt & clay (Ice Age)	Till
Steepland sediments (Ice Age)	Till
Till (Ice Age)	Till

[1]This category covers a small fraction of the lowland area that does not justify inclusion of a separate Alluvial soil type.

Land Use and Impervious Area

NHC obtained geospatial land use data from the City of Surrey, City of Langley, and Township of Langley, including current zoning, Official Community Plan land use, parks and Agricultural Land Reserve areas. Current land use for these areas was derived from current zoning data, supplemented by the available parks and agricultural land layers. For the small portion of the Serpentine watershed extending into the Corporation of Delta, current land use was delineated from aerial photos. Future land use was derived from land use maps in the Official Community Plans (OCPs) for each jurisdiction(4). Some areas were also covered by secondary or neighbourhood plans at a much finer resolution (typically down to parcel-scale). Based on a qualitative visual review of several of these areas, NHC judged that incorporating the higher resolution neighbourhood data would not significantly affect the overall land use picture at the sub-basin scale, so these plans were not used.

While important for planning, the level of distinction provided by the dozens of zoning designations does not carry over to differences in hydrologic response. To simplify impervious area analysis and model development, NHC combined similar zoning and community plan land use designations, in terms of type and intensity of development, into broader categories representing land use and land cover over the entire study area. Grouping of current land use designations relied on descriptions in the zoning bylaws, as well as aerial photo review. Categorization of future land use designations paralleled current land use.

Table 6.5, Table 6.6, and Table 6.7 summarize the model land-use category for each zoning and OCP designation for the three primary jurisdictions. Figure 6.4 and Figure 6.5 show respectively existing and future classified land use for the study area. The most notable difference is the development or redevelopment of most existing low- and medium-density residential areas to higher intensities.

[4] Areas designated as Suburban in the City of Surrey OCP were reclassified as Urban for purposes of hydrologic analysis at the direction of the City.

Most agricultural areas, however, are maintained through the Agricultural Land Reserve, and the extent of commercial and industrial areas is not significantly different.

Several of the jurisdictions have mixed use designations, such as “Comprehensive Development” (CD) or “Mixed Use”. For modelling purposes these were classified—mostly as multifamily or commercial—based primarily on surrounding land use. The City of Langley bylaw provided descriptions of individual CD areas that were used in classification of existing land use. Also, as zoning designations are not always representative of current land use, existing land use initially classified based on zoning was overlain on aerial photos and updated as necessary to correspond to actual current conditions.

Table 6.5. City of Surrey land use classifications

Category	Existing Land Use Zoning Code[1]	Future Land Use OCP Land Use Code
Agriculture/Rural	Ax	AGR, RUR
Commercial	Cx (except CPG, CD)	CC, COM, TC
Industrial	Ix	IND
Institution	PA, PI	n/a
Multifamily	RM-x (except –D), RT	RM
Park	PC, CPG	CNS
SFR-Low	RA, RH, RC	SUB[2]
SFR-Medium	RF, RF-SS, RF-G, RM-D, RS	n/a
SFR-High	RF-9, RF-12, RF-SD	URB

[1]CD (comprehensive development) based on surrounding land use – see text.
[2]All areas designated as SUBURBAN reclassified as URBAN at Surrey’s direction.
SFR = Single Family Residential

Table 6.6. City of Langley land use classifications

Category	Existing Land Use Zoning Code[1]	Future Land Use OCP Land Use Description
Agriculture/Rural	A1	Agricultural
Commercial	C1, C2, C3	Downtown Commercial, Service Commercial
Industrial	I1, I2	Industrial
Institution	P1, P2[2]	Institutional
Multifamily	RM1, RM2, RM3	Urban Residential
Park	P1, P2[2]	n/a
SFR-Low	n/a	Low-Density Residential
SFR-Medium	RS1, RS2	Medium-Density Residential
SFR-High	n/a	High-Density Residential

[1]CD (comprehensive development) based on bylaw description of zone.
[2]P1 and P2 categories cover parks and institutions; category determined from photos.
SFR = Single Family Residential

Table 6.7. Township of Langley land use classifications

Category	Existing Land Use Zoning Code[1]	Future Land Use OCP Land Use Category[4]
Agriculture/Rural	RU-x	Rural, Agricultural/Countryside, Small Farms/Country Estates
Commercial	C-x	Comm, Business Office Park
Industrial	M-x	Light Industrial
Institution	P-x[2]	Institutional
Multifamily	RM-x	MFR
Park	P-x[2]	Park Green Space, Regional Park
SFR-Low	R-CL[3], SR-1, SR-2, SR-3	Comprehensive Rural Estates, Salmon River Uplands
SFR-Medium	CRE-1, FH-1, MH-1, R-CL[3],	SFR-Med
SFR-High	R-CL[3]	n/a

[1]CD (comprehensive development) based on surrounding land use – see text.
[2]P-x categories cover parks and institutions; category determined from photos.
[3]Density for R-CL category determined from photos.
[4]Comp Dev, Mixed Use, and Other based on surrounding land use.
SFR = Single Family Residential

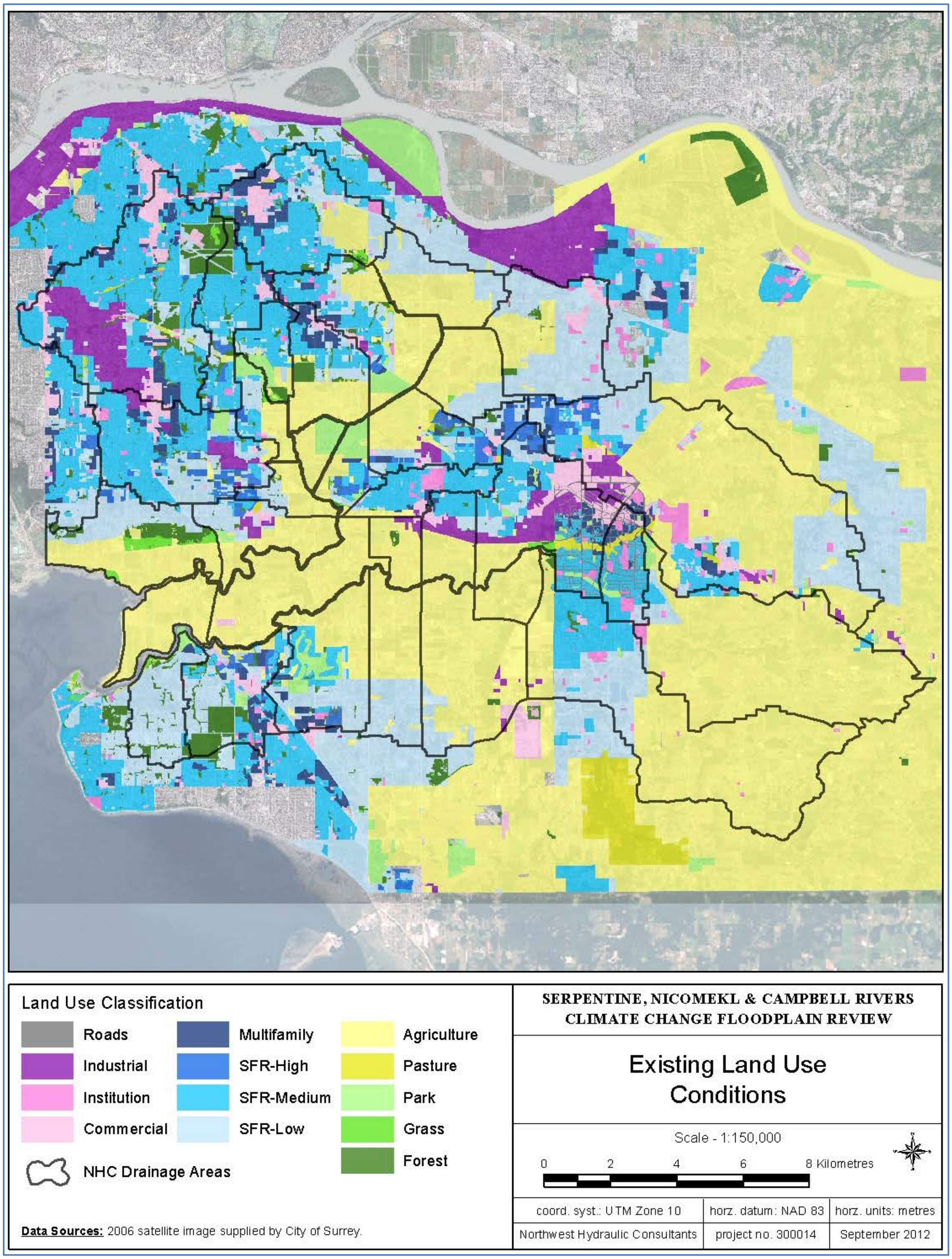


Figure 6.4. Existing land use conditions

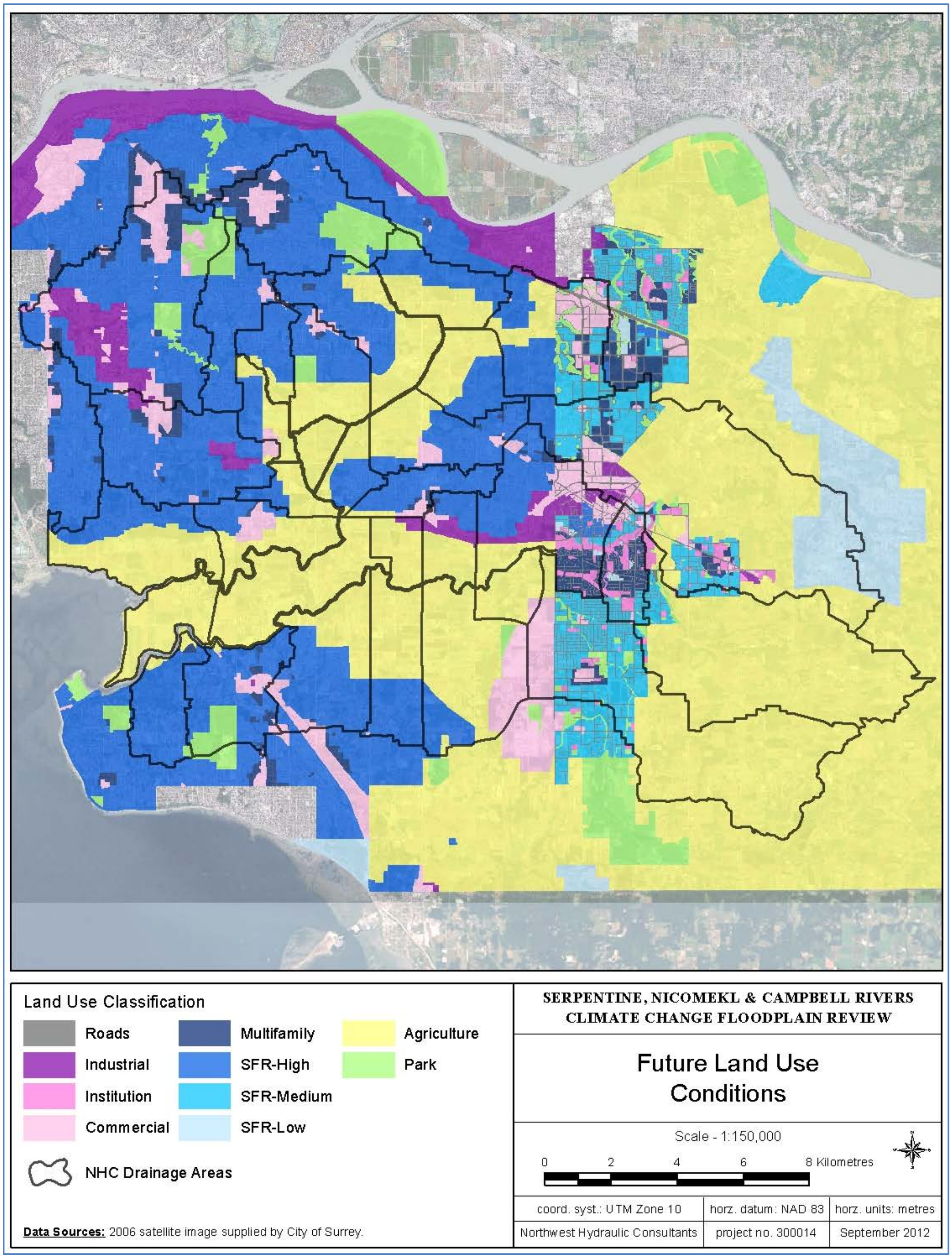


Figure 6.5. Future land use conditions

In HSPF modelling, the land surface runoff response is dictated by the land cover, as opposed to the land use. Therefore, each of the land use categories defined above must be further broken down to representative land cover percentages. Most important is the definition of effective impervious area (EIA), i.e. impervious area that runs off directly to the drainage system. Remaining pervious area is further broken down based on the vegetative cover—either forest, pasture (undeveloped grasslands), or grass (landscaped areas)—which affects evapotranspiration and runoff generation at the surface and in the upper soil layer.

There is no impervious area mapping for the study area, so total and effective impervious area percentages were estimated based on available information and past experience. For current land use categories, initial estimates of impervious percentages were based on the City of Surrey's Design Criteria Manual (2004) and the 2002 Review of Runoff Coefficients (McElhanney Consulting Services Ltd., 2002), as well as NHC's experience. Impervious area percentages determined for zoning categories within the City of Surrey were extended to similar categories in other areas of the watershed. Initial estimates for some categories were refined during model calibration. Table 6.8 summarizes the land cover breakdown for each existing land use category.

Table 6.8. Land cover percentages for existing land use

Category	% EIA	% Forest	% Pasture	% Grass
Agriculture/Rural	4	3	78	15
Commercial	85	0	0	15
Industrial	60	0	0	40
Institution	45	0	0	55
Multifamily	65	0	0	35
Park	3	15	15	67
Roads	80	0	0	20
SFR-High	45	0	0	55
SFR-Low	5	15	20	60
SFR-Medium	20	0	0	80
Forest	0	100	0	0
Grass	0	0	0	100
Pasture	0	0	100	0

Given uncertainties in future development policies and stormwater regulations, including implementation of sustainable stormwater initiatives, it is difficult to predict effective impervious area percentages for buildout conditions in 2100. At the suggestion of the City, impervious area percentages for most future land use categories were kept at the existing levels. An exception is the Industrial category, where existing effective impervious area was lowered significantly compared to typical values during calibration. In this case, future impervious area was increased to align with more typical values. The Park category is also different for future land use. Most of the areas that fall under Park in the future are preserved green space and stream corridors, as opposed to more developed open areas that dominate the Park category under existing land use. Land cover percentages were adjusted accordingly, most notably increasing the percent forest cover. Table 6.9 summarizes the land cover breakdown for each future land use category.

Table 6.9. Land cover percentages for future land use

Category	% EIA	% Forest	% Pasture	% Grass
Agriculture/Rural	4	3	78	15
Commercial	85	0	0	15
Industrial	80	0	0	20
Institution	45	0	0	55
Multifamily	65	0	0	35
Park/Open Space	0	40	10	50
Roads	80	0	0	20
SFR-High	45	0	0	55
SFR-Low	5	15	20	60
SFR-Medium	20	0	0	80

6.1.3 SUBBASIN DELINEATION AND ROUTING

NHC divided the study area into 33 subbasins (or drainage areas) shown in Figure 6.6; 17 in the Serpentine River watershed; 14 in the Nicomekl River watershed; and, 2 covering the floodplain area between the two rivers. The subbasin delineation was driven primarily by locations where inflows are needed for the hydraulic model. All of the subbasins except Upper Nicomekl and Murray Creek, which flow into the 'Nicomekl Gauge Local' subbasin, upstream of the streamflow gauge, provide direct inflows to the hydraulic model. There are six point inflow locations—Nicomekl Gauge Local, West Cloverdale A, Latimer/Clayton, Upper Serpentine A, Mahood Creek, and Hyland Creek—and the remaining subbasins provide lateral inflows to the floodplain storage cells in the hydraulic model. Drainage area information is summarized in Table 6.10.

HSPF uses level pool hydrologic routing based on a static stage-storage-discharge rating (or FTABLE) for each routing reach. This type of routing lacks the sophistication needed to capture the floodplain and tidal interactions and hydraulic structures such as pump stations on the Serpentine and Nicomekl rivers, so most routing was done in the hydraulic model (see Section 7). Within HSPF, routing was only performed for the creek subbasins upstream of the point inflow locations to the hydraulic model. FTABLEs for the Upper Nicomekl subbasins (Upper Nicomekl, Murray Creek, and Nicomekl Gauge Local) were estimated from a coarse HEC-RAS model developed from cross sections from the previous City modelling projects. The model extends only partway up the tributaries, so storage was adjusted during model calibration to better match gauge hydrograph shapes. The Mahood Creek FTABLE was estimated assuming channel dimensions and slope similar to the uppermost cross sections for Bear Creek and was also adjusted during calibration. The Hyland Creek, Upper Serpentine A, and Latimer/Clayton drainage areas also use the Mahood Creek FTABLE, and the Anderson Creek drainage area uses the Murray Creek FTABLE. It is likely that some upland storage exists in most of the subbasins, and hence peak flows and hydrograph shapes for the unrouted drainage areas are not accurate. However, the upland storage and flow attenuation would be dwarfed by the effects of the sea dams, pump stations, and floodplain storage and thus are not significant to conditions in the storage cells.

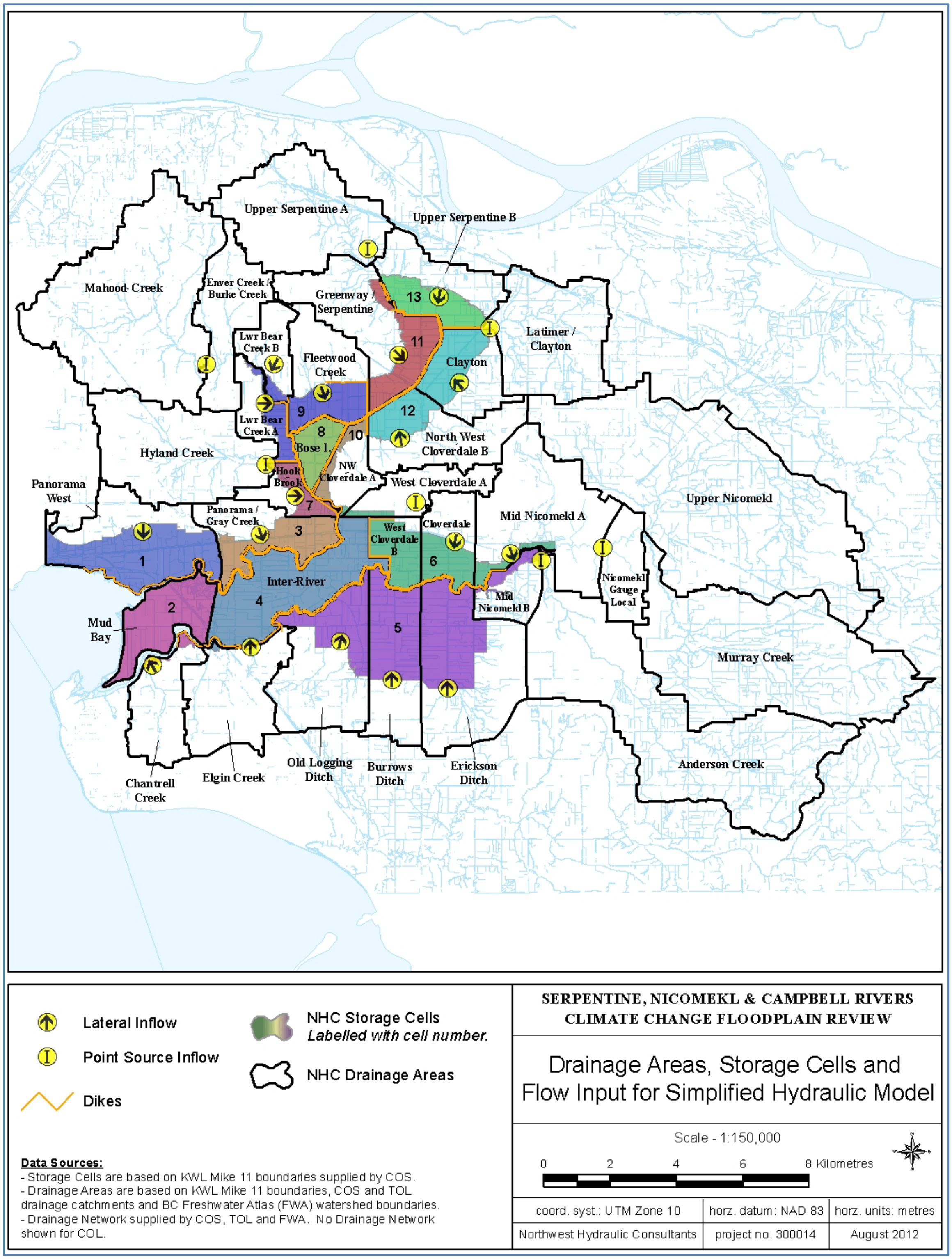


Figure 6.6. Drainage areas, storage cells and flow input for hydraulic model

Table 6.10. Drainage area summary

HSPF ID	Drainage Area Name	Area (ha)	River Basin	Inflow Type	HSPF Routing?
10	Panorama West	1058	Serpentine	Lateral	No
20	Mud Bay	504		Lateral	No
25	Chantrell Creek	560	Nicomekl	Lateral	No
30	Panorama / Gray Creek	593	Serpentine	Lateral	No
40	Inter-River	926		Lateral	No
45	Elgin Creek	854	Nicomekl	Lateral	No
50	Old Logging Ditch	1050	Nicomekl	Lateral	No
53	Burrows Ditch	760	Nicomekl	Lateral	No
56	Erickson Ditch	1308	Nicomekl	Lateral	No
60	West Cloverdale B	278	Nicomekl	Lateral	No
63	West Cloverdale A	424	Nicomekl	Point	No
66	Cloverdale	426	Nicomekl	Lateral	No
70	Hook Brook	245	Serpentine	Lateral	No
75	Hyland Creek	1368	Serpentine	Point	Yes
80	Bose Island	194	Serpentine	Lateral	No
90	Fleetwood Creek	692	Serpentine	Lateral	No
93	Lower Bear Creek A	305	Serpentine	Lateral	No
96	Lower Bear Creek B	338	Serpentine	Lateral	No
100	North West Cloverdale A	309	Serpentine	Lateral	No
110	Greenway / Serpentine	1009	Serpentine	Lateral	No
120	North West Cloverdale B	825	Serpentine	Lateral	No
123	Clayton	558	Serpentine	Lateral	No
126	Latimer / Clayton	1177	Serpentine	Point	Yes
130	Upper Serpentine B	686	Serpentine	Lateral	No
135	Upper Serpentine A	1337	Serpentine	Point	Yes
190	Enver Creek / Burke Creek	714	Serpentine	Lateral	No
195	Mahood Creek	2536	Serpentine	Point	Yes
200	Mid Nicomekl B	319	Nicomekl	Lateral	No
206	Mid Nicomekl A	1361	Nicomekl	Lateral	No
210	Anderson Creek	2944	Nicomekl	Point	Yes

HSPF ID	Drainage Area Name	Area (ha)	River Basin	Inflow Type	HSPF Routing?
220	Nicomekl Gauge Local	411	Nicomekl	Point	Yes
230	Murray Creek	2694	Nicomekl	n/a	Yes
240	Upper Nicomekl	3865	Nicomekl	n/a	Yes

6.2 HSPF Model Calibration

Attempts were initially made to calibrate the HSPF model to gauged discharges on both the Nicomekl River at 203 Street and Mahood-Bear Creek at 144 Street. Both gauge sites were originally established by Environment Canada; City of Surrey has since taken over the Mahood Creek gauge. These sites were selected because they capture significant upland areas with distinct land use patterns and have sufficiently long records, including several large storm events. Hourly discharge data were obtained for both stations. Gauge information is summarized in Table 6.11.

Table 6.11. Discharge calibration gauges

Station Name	Operated By	ID	Drainage Area	Data Available[1]	
			(ha)	from	to
Nicomekl River at 203 St.	Environment Canada	08MH155	6,970	10/2002	9/2010
Mahood-Bear Creek at 144 St.	City of Surrey	08MH154	2,536	10/1997	1/2012

[1]Significant gaps within period of record for both stations.

Because this study is focused on floodplain analysis and flooding events, model calibration was targeted at matching large event peaks and volumes. NHC identified nine large events since water year 2004 listed in Table 6.12. All events are consistently large for rainfall at Surrey Municipal Hall and discharge at the Nicomekl River and Mahood Creek gauges (unless missing). The October 2003 and December 2007 events (shown in gray in Table 6.12) were not used for model calibration and/or validation because of greater uncertainties in rainfall distribution over the watersheds, as described in the table footnotes.

Table 6.12. Large event summary

Event	Event Dates	Surrey MH Rain (mm)		Nicomekl Flow (cms)		Mahood Flow (cms)	
		Max Hr	Event	Peak	Event Avg	Peak	Event Avg
October 2003[1]	15-19	13.2	244	No Data		33.3	7.4
November 2003	27-30	8.2	78	85.0	22.5	38.1	8.4
January 2005	16-23	8.0	217	95.7	19.7	Missing	

November 2006	5-7	9.6	65	62.2	14.1	25.2	5.5
January 2007	1-4	9.0	75	81.3	18.3	27.9	5.2
March 2007	10-14	10.2	104	93.5	18.6	Missing	
December 2007[2]	2-5	8.6	99	90.6	28.4	25.4	5.4
January 2009	5-14	6.3	233	89.6	Missing[3]	24.8	6.2
December 2010	11-13	6.5	63	No Data		27.2	4.5

[1]Spatial rainfall pattern different than other events; 10% more rainfall at Municipal Hall than any other gages that recorded the event. Not used for calibration.

[2]Surrey Municipal Hall rainfall missing for most of event; filled from other gauges. Not used for calibration.

[3]Gauge record includes peak but is sporadic through event.

The HSPF model was first run using the USGS regional parameters and initial estimates for impervious area and FTABLE storage. The HSPF model parameters were then adjusted to first improve simulation of annual and flood event volumes and to then improve simulation of flood hydrograph shape, timing and peak flow. Adjustments were also made to the FTABLES and to the effective impervious area (EIA) percent for certain land use categories were deemed appropriate. For example, the EIA for industrial land use in existing conditions was reduced from an initial estimate of 80% to 60% based on our understanding of the nature of existing industrial development.

Initial HSPF simulations showed oversimulation of flood event peaks and volumes on Mahood Creek and significant undersimulation of peaks and volumes on the Nicomekl River. Significant difficulties were also encountered in attempting to reproduce the timing of recorded peak flows on the Nicomekl, with recorded peak flows typically lagging simulated peak flows by from 6 to 12 hours.

Attempts to improve simulation results while maintaining basin-wide consistency in model parameters were largely unsuccessful (i.e. modification of parameters to improve simulation results for Mahood Creek generally resulted in poorer simulation results for the Nicomekl and vice versa) and attempts to balance oversimulation on Mahood Creek against undersimulation on the Nicomekl resulted in unsatisfactory simulations at both sites.

The difficulties encountered in model calibration prompted a closer examination of recorded flows, particularly for the Nicomekl at 203 Street. This included:

- Comparing reported flood event volumes against observed rainfall amounts, and,
- Examining trends over time in reported instantaneous and daily peak flows.

Observed rainfall and runoff depths for the flood events in the calibration period are compared in Table 6.13. Note that with the exception of the November 2003 event, reported runoff depths for the Nicomekl are greater than for Mahood Creek despite the greater intensity of development in the Mahood Creek catchment and generally comparable or greater rainfall amounts over Mahood Creek according to the storm analyses conducted for this study. It should also be noted that for several events (notably January 2007 and December 2007), runoff depths on the Nicomekl appear to be greater than the storm rainfall depths even after accounting for an approximately 10% increase in rainfall as one moves east from Surrey Municipal Hall to the headwaters of the Nicomekl (Figure 6.2).

Table 6.13. Comparison of flood event rainfall and runoff volumes

Flood Event			Observed Runoff Volume (mm)		Observed Rainfall (mm)
			Mahood Creek at 144 Street	Nickomekl River at 203 Street	Surrey Municipal Hall
Month	From	To			
Oct 2003	15	19	132.2	m	243.8
Nov 2003	27	30	81.5	77.4	78.0
Jan 2005	16	23	m	195.1	216.6
Nov 2006	5	7	52.0	60.8	64.6
Jan 2007	1	4	68.7	87.6	74.8
Mar 2007	10	14	m	106.6	104.4
Dec 2007	2	5	84.9	126.4	99.1
Jan 2009	5	14	188.6	m	233.0
Dec 2010	11	13	46.8	m	62.6

To further investigate the reliability of the measured flows on the Nicomekl River, time series of annual maximum instantaneous and annual maximum daily flows were plotted in Figure 6.7 for the Nicomekl River below Murray Creek (Environment Canada gauge 08MH105, drainage area 64.5 km^2) and Nicomekl River at 203 Street (gauge 08MH155, drainage area 70.0 km^2). The gauge below Murray Creek was located roughly one kilometer upstream from the 203 Street gauge and operated from 1965 through 1984. Some differences between the recorded peak flows below Murray Creek (from 1965 to 1985) and at 203 Street (since 1985) would be expected because of:

- the larger drainage area at 203 Street (roughly 8.5% larger);
- increased development in the watershed over time; and,
- a greater frequency of large storms since the mid-1980's.

However in our opinion, those factors cannot account for the very large differences in reported peak flows at the two gauge sites evident in Figure 6.7.

The comparison of rainfall against runoff in Table 6.13 together with the unexplained differences in peak flows shown in Figure 6.7, leads us to question the reliability of the reported discharge data for the Nicomekl River at 203 Street. Data from this gauge site were thus discounted in the final HSPF model calibration, which focused on simulation of flows on Mahood Creek at 144 Street.

Plots of simulated against recorded flows for the final HSPF model calibration are shown in Appendix C.1 for Mahood Creek at 144 Street and in Appendix C.2 for Nicomekl River at 203 Street. A summary of the simulation results is provided in Table 6.14. The final HSPF model parameters are provided in the calibration model User Control Input (UCI) on the attached DVD.

Excluding model results from the October 2003 event, for which the spatial pattern of rainfall appears to be anomalous, simulated event runoff volumes for Mahood Creek are 2% greater than recorded and simulated peak flows are 12% greater than recorded. For the Nicomekl, simulated event runoff volumes are 29% lower than recorded and simulated peak flows are 43% lower than recorded. However, as discussed above, we are not confident of the reliability of the Nicomekl data. There are indications that the reported Nicomekl flows are overstated, and the Nicomekl data were discounted in the final HSPF model calibration.

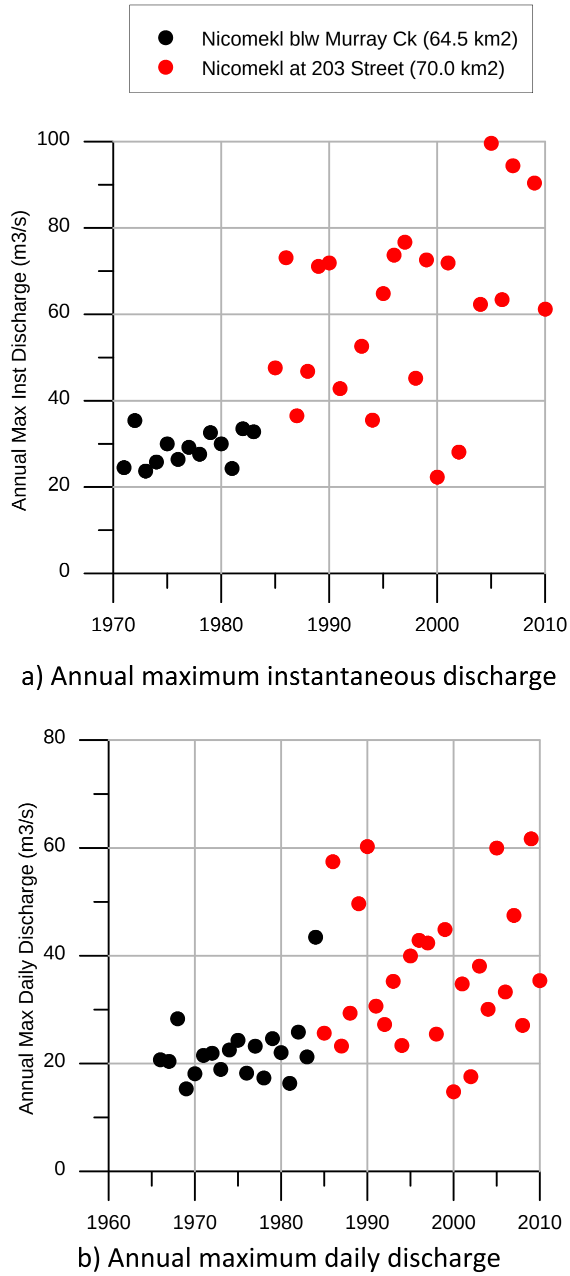


Figure 6.7. Time series of annual maximum instantaneous and annual maximum daily discharge, Nicomekl River below Murray Creek and Nicomekl River at 203 Street

Table 6.14. Summary HSPF model calibration results

Flood Event			Mahood Creek at 144 Street						Nickomekl River at 203 Street						Surrey MH
			Volume (mm)			Peak (m3/s)			Volume (mm)			Peak (m3/s)			Rainfall (mm)
Month	From	To	Gauged	Simulated	% Difference	Gauged	Simulated	% Difference	Gauged	Simulated	% Difference	Gauged	Simulated	% Difference	Observed
Oct 2003	15	19	132.2	200.9	52%	33.3	53.0	59%	m	155.7	m	m	72.6	m	243.8
Nov 2003	27	30	81.5	67.6	-17%	38.1	38.2	0%	77.4	55.9	-28%	85.0	55.9	-34%	78.0
Jan 2005	16	23	m	198.8	m	m	35.9	m	195.1	176.6	-9%	95.7	60.6	-37%	216.6
Nov 2006	5	7	52.0	53.0	2%	25.2	24.4	-3%	60.8	43.0	-29%	62.2	29.0	-53%	64.6
Jan 2007	1	4	68.7	66.1	-4%	27.9	34.4	23%	87.6	56.2	-36%	81.3	43.5	-46%	74.8
Mar 2007	10	14	m	97.1	m	m	41.5	m	106.6	85.2	-20%	93.5	66.9	-28%	104.4
Dec 2007	2	5	84.9	78.4	-8%	25.4	32.9	30%	126.4	61.4	-51%	90.6	37.2	-59%	99.1
Jan 2009	5	14	188.6	224.7	19%	24.8	30.5	23%	m	190.9	m	m	43.9	m	233.0
Dec 2010	11	13	46.8	55.6	19%	27.2	27.6	1%	m	45.9	m	m	41.7	m	62.6
Average all events					9%			19%			-29%			-43%	
Average excl Oct 2003					2%			12%			-29%			-43%	

6.3 HSPF LONG-TERM SIMULATIONS

The HPSF PERLND parameters developed through calibration to the Mahood Creek discharge record were applied basin-wide to generate long-term hydrologic inputs to the hydraulic model of Nicomekl/Serpentine River. Hydrologic modelling was conducted for existing and future land use for the period of meteorological record from water year 1963 through water year 2011. As discussed in Section 3.2, potential climate change impacts on precipitation were not considered in this study; meteorological conditions were assumed to be stationary with the historic precipitation and evaporation records representative of future conditions out to the year 2100. Flows generated by HSPF were written at an hourly time step to a HEC-DSS data base for import to the HEC-RAS hydraulic model.

Maximum annual runoff volumes to the Nicomekl and the Serpentine Rivers above their respective sea dams were computed from the HSPF simulation results for durations of 24-hours, 3-days, 5-days and 7-days for each year of simulation. The runoff volumes were determined by simply accumulating the HSPF-generated flows tributary to each river basin (see Table 6.10). Runoff from the “Inter-River” subbasin (HSPF ID 40) was split 50-50 between the two river basins. Since the HSPF model does not include any flow routing for the lower Serpentine or Nicomekl Rivers, these accumulated HSPF-generated runoff volumes can be regarded as total inflows to the lower Serpentine and Nickomekl floodplains. The annual maximum data for each duration are provided for existing land use in Table 6.15 and Table 6.16 for the Nickomekl and Serpentine respectively, and for future land use in Table 6.17 and Table 6.18. These annual maximum data were subject to frequency analysis to provide basic information on the expected magnitude of extreme storm runoff volumes. Runoff frequency curves (expressed in terms of average flow rate over the duration of interest) are provided in Figure 6.8 (a,b) through Figure 6.9 (a,b) and runoff quantiles are provided in Table 6.19 and Table 6.20. For all runoff frequency analysis, data were fit to a 3-Parameter Log Normal Distribution.

From preliminary hydraulic analysis, we would expect the critical storm duration to be 3-days or longer. From the HSPF simulations, the two largest runoff producing events for duration of 3, 5-, and 7-days on both the Nicomekl and Serpentine are October 2003 and January 2009. As noted previously, the spatial distribution of rainfall during the October 2003 event was atypical and estimates of runoff from that event are probably somewhat conservative (i.e. high).

Table 6.15. Maximum annual runoff, Nicomekl River above sea dam, existing land use

Duration	Maximum Annual Runoff Averaged over Specified Duration (m^3/s) under Existing Land Use							
	24 hours		3 days		5 days		7 days	
Rank	Maxi-mum Annual Runoff	Ending Date	Maxi-mum Annual Runoff	Ending Date	Maxi-mum Annual Runoff	Ending Date	Maxi-mum Annual Runoff	Ending Date
1	113.8	17-Oct-03	77.6	19-Oct-03	57.4	11-Jan-09	48.1	12-Jan-09
2	87.1	26-Dec-72	68.2	9-Jan-09	56.8	21-Oct-03	45.9	23-Oct-03
3	86.2	12-Mar-07	60.4	21-Jan-68	45.5	21-Jan-05	39.8	23-Jan-05
4	85.2	19-Jan-68	58.8	20-Jan-05	42.6	23-Jan-68	34.7	19-Feb-82
5	84.8	18-Dec-79	51.1	28-Dec-72	37.3	28-Dec-72	34.3	27-Dec-72
6	82.6	7-Jan-09	49.7	19-Dec-79	36.6	17-Feb-82	33.3	24-Jan-68
7	82.5	18-Jan-05	47.9	16-Feb-82	35.1	21-Dec-79	30.5	20-Dec-79
8	78.4	30-Jan-97	47.8	13-Mar-07	32.9	13-Jan-06	29.7	18-Mar-07
9	66.0	14-Feb-82	41.9	5-Jan-84	32.7	17-Dec-01	28.7	14-Jan-06
10	62.5	4-Jan-84	40.1	20-Mar-97	32.5	14-Mar-07	28.3	17-Dec-99
11	61.7	16-Dec-99	37.4	17-Dec-99	31.3	6-Jan-84	27.2	17-Dec-66
12	58.1	12-Dec-10	36.7	15-Dec-01	30.3	18-Dec-99	27.0	19-Dec-01
13	55.6	14-Dec-01	36.3	11-Jan-06	30.1	16-Dec-66	26.8	26-Nov-09
14	55.0	13-Jul-72	33.6	1-Feb-92	29.9	1-Feb-92	26.2	3-Feb-92
15	52.2	26-Jan-71	33.5	5-Dec-07	29.4	1-Feb-97	25.0	8-Jan-84
16	51.1	4-Dec-07	33.1	31-Dec-62	28.1	26-Nov-09	24.2	14-Dec-10
17	50.2	16-Mar-74	32.8	14-Dec-10	27.0	2-Jan-63	24.1	4-Feb-99
18	49.3	30-Dec-62	32.1	21-Nov-80	26.8	30-Jan-71	23.7	3-Feb-97
19	48.5	16-Dec-66	32.0	26-Feb-86	26.6	23-Nov-80	23.5	1-Feb-71
20	47.1	15-Jan-10	31.3	16-Dec-66	26.5	13-Dec-10	23.3	3-Jan-63
21	46.1	29-Jan-99	30.8	31-Jan-99	25.4	30-Dec-98	22.6	8-Dec-75
22	46.1	17-Feb-65	30.8	6-Feb-65	25.1	24-Nov-86	21.7	25-Nov-80
23	46.1	10-Jan-06	30.5	18-Mar-74	24.4	7-Jan-69	20.7	17-Dec-73
24	45.9	21-Nov-80	30.3	16-Jan-96	24.3	13-Nov-90	19.9	26-Nov-86
25	45.5	9-Dec-75	30.3	14-Jul-72	23.9	27-Feb-86	19.8	14-Nov-90
26	45.5	4-Jan-69	29.7	11-Nov-90	23.3	16-Jan-76	19.7	9-Jan-69
27	44.7	10-Feb-90	29.4	17-Jan-76	22.8	15-Jan-08	19.6	7-Nov-88
28	44.2	6-Nov-88	29.1	5-Jan-69	22.6	7-Apr-88	19.6	16-Jan-08
29	42.9	5-Jan-01	29.0	22-Nov-09	22.4	17-Dec-73	19.1	9-Dec-89
30	42.8	19-Jan-86	28.3	28-Dec-94	22.4	8-Feb-65	18.8	16-Mar-72
31	42.6	31-Jan-92	27.9	12-Feb-90	22.0	18-Jan-96	18.7	8-Apr-88

32	41.8	10-Nov-90	27.4	28-Jan-71	21.5	7-Nov-88	18.7	1-Mar-86
33	41.0	23-Nov-86	26.5	24-Nov-86	21.4	5-Nov-71	18.0	21-Dec-74
34	38.1	6-Apr-88	26.0	6-Nov-88	21.1	8-Dec-89	17.8	2-Dec-77
35	37.8	3-Dec-82	22.8	22-Dec-74	20.7	30-Dec-94	17.7	4-Dec-82
36	37.8	13-Jan-66	22.4	6-Jan-01	19.4	21-Dec-74	17.5	20-Jan-96
37	36.7	15-Jan-96	22.0	14-Jan-66	18.6	30-Nov-77	17.4	10-Feb-65
38	36.1	27-Dec-94	22.0	8-Apr-88	18.3	21-Oct-00	16.5	1-Jan-95
39	35.7	26-Nov-77	21.9	19-Jan-77	18.3	20-Feb-83	15.4	20-Jan-98
40	35.2	18-Jan-75	21.8	3-Mar-94	18.1	4-Mar-94	15.1	23-Oct-00
41	34.5	23-Mar-93	21.6	5-Dec-82	17.9	15-Jan-66	15.0	5-Mar-94
42	31.3	14-Dec-84	21.3	21-Jan-70	17.4	6-Jan-03	14.5	6-Jan-03
43	31.3	18-Jan-77	20.9	4-Jan-03	17.2	23-Jan-70	14.3	16-Jan-66
44	30.4	2-Mar-94	20.9	24-Mar-93	17.0	16-Dec-84	14.3	18-Dec-84
45	30.4	16-Dec-63	20.3	17-Dec-63	16.6	20-Jan-77	13.9	20-Jan-77
46	29.9	2-Jan-03	19.9	28-Nov-77	16.6	19-Dec-63	13.9	25-Jan-70
47	28.9	19-Jan-70	19.8	16-Dec-84	16.4	19-Jan-98	13.6	21-Dec-63
48	26.4	25-Feb-79	17.6	27-Feb-79	16.1	25-Mar-93	12.9	24-Mar-93
49	24.0	17-Jan-98	17.3	19-Jan-98	13.0	8-Mar-79	11.4	9-Nov-78

Table 6.16. Maximum annual runoff, Serpentine River above sea dam, existing land use

Duration	Maximum Annual Runoff Averaged over Specified Duration (m^3/s) under Existing Land Use							
	24 hours		3 days		5 days		7 days	
Rank	Maximum Annual Runoff	Ending Date	Maximum Annual Runoff	Ending Date	Maximum Annual Runoff	Ending Date	Maximum Annual Runoff	Ending Date
1	131.0	17-Oct-03	80.3	19-Oct-03	57.2	21-Oct-03	45.7	12-Jan-09
2	94.8	26-Dec-72	68.4	9-Jan-09	55.8	11-Jan-09	45.3	22-Oct-03
3	94.8	18-Dec-79	59.6	21-Jan-68	44.6	21-Jan-05	38.3	23-Jan-05
4	91.7	12-Mar-07	58.1	20-Jan-05	39.9	22-Jan-68	33.5	27-Dec-72
5	91.3	19-Jan-68	49.6	19-Dec-79	36.7	26-Dec-72	32.5	19-Feb-82
6	89.5	7-Jan-09	49.4	28-Dec-72	34.8	18-Dec-79	30.1	24-Jan-68
7	87.0	18-Jan-05	47.4	15-Feb-82	34.7	17-Feb-82	29.9	20-Dec-79
8	79.1	30-Jan-97	46.6	13-Mar-07	31.3	17-Dec-01	27.5	18-Mar-07
9	71.9	14-Feb-82	43.6	5-Jan-84	31.1	13-Jan-06	26.6	17-Dec-99
10	67.0	4-Jan-84	39.7	20-Mar-97	30.6	6-Jan-84	26.5	14-Jan-06
11	64.9	15-Dec-99	36.0	15-Dec-01	30.6	14-Mar-07	25.4	16-Dec-66

12	62.4	13-Jul-72	35.3	17-Dec-99	29.1	15-Dec-99	25.2	17-Dec-01
13	62.2	12-Dec-10	34.9	11-Jan-06	28.6	16-Dec-66	25.0	26-Nov-09
14	58.6	26-Jan-71	34.5	5-Dec-07	28.5	1-Feb-92	24.3	3-Feb-92
15	58.1	4-Dec-07	33.6	21-Nov-80	27.0	22-Mar-97	23.8	8-Jan-84
16	58.0	14-Dec-01	33.0	25-Feb-86	26.8	30-Jan-71	22.6	14-Dec-10
17	53.9	16-Mar-74	32.5	1-Feb-92	26.0	22-Nov-80	22.5	1-Feb-71
18	51.2	30-Dec-62	32.4	31-Dec-62	25.9	26-Nov-09	21.9	4-Feb-99
19	50.0	5-Feb-65	31.3	6-Feb-65	25.7	13-Dec-10	21.7	3-Jan-63
20	49.9	16-Dec-66	31.3	14-Jul-72	25.6	24-Nov-86	20.9	8-Dec-75
21	49.8	4-Jan-69	31.1	14-Dec-10	25.6	17-Nov-98	20.8	3-Feb-97
22	49.8	15-Jan-10	30.8	11-Nov-90	25.3	2-Jan-63	20.5	24-Nov-80
23	49.0	21-Nov-80	30.3	16-Dec-66	24.4	13-Nov-90	20.0	25-Nov-86
24	48.9	29-Jan-99	30.2	16-Nov-98	23.4	27-Feb-86	19.7	14-Nov-90
25	48.8	30-Oct-75	30.0	16-Jan-96	23.4	7-Jan-69	19.5	7-Nov-88
26	48.5	10-Nov-89	29.0	18-Mar-74	22.3	15-Jan-08	19.1	17-Dec-73
27	48.2	18-Jan-86	28.9	5-Jan-69	22.0	14-Jul-72	18.2	15-Jan-08
28	47.8	6-Nov-88	28.5	22-Nov-09	22.0	6-Apr-88	18.1	9-Jan-69
29	47.4	10-Jan-06	28.1	28-Dec-94	21.8	8-Feb-65	17.8	9-Dec-89
30	47.0	10-Nov-90	27.7	11-Nov-89	21.3	7-Nov-88	17.7	28-Feb-86
31	46.8	5-Jan-01	27.5	4-Dec-75	21.2	16-Jan-76	17.6	8-Apr-88
32	44.2	23-Nov-86	27.0	28-Jan-71	20.7	17-Dec-73	17.6	2-Dec-77
33	42.4	31-Jan-92	26.8	24-Nov-86	20.5	18-Jan-96	17.5	4-Dec-82
34	41.1	6-Apr-88	26.3	6-Nov-88	19.8	8-Dec-89	17.0	16-Mar-72
35	40.9	3-Dec-82	22.3	6-Jan-01	19.3	30-Dec-94	16.7	21-Dec-74
36	40.4	13-Jan-66	22.2	22-Nov-74	18.8	21-Oct-00	16.5	30-Nov-95
37	40.2	26-Nov-77	22.1	14-Jan-66	18.4	30-Nov-77	16.4	10-Feb-65
38	39.7	23-Mar-93	22.0	24-Mar-93	17.9	21-Dec-74	15.0	22-Oct-00
39	38.7	19-Feb-95	22.0	19-Jan-77	17.8	15-Jan-66	14.7	1-Jan-95
40	38.7	14-Jan-96	21.9	2-Mar-94	17.7	4-Dec-82	14.5	20-Jan-98
41	38.0	20-Nov-74	21.3	21-Jan-70	17.6	4-Mar-94	14.3	5-Mar-94
42	34.0	2-Jan-03	21.2	3-Dec-82	16.9	6-Jan-03	14.0	6-Jan-03
43	33.6	14-Dec-84	21.1	4-Jan-03	16.5	25-Mar-93	13.7	16-Jan-66
44	33.3	18-Jan-77	20.8	8-Apr-88	16.5	23-Jan-70	13.6	24-Mar-93
45	32.5	2-Mar-94	20.0	28-Nov-77	16.1	16-Dec-84	13.4	18-Jan-77
46	32.3	16-Dec-63	19.8	17-Dec-63	16.1	19-Jan-77	13.2	11-Apr-70
47	31.0	19-Jan-70	19.1	14-Dec-84	15.9	18-Jan-98	13.2	14-Dec-84
48	28.8	25-Feb-79	17.2	27-Feb-79	15.7	19-Dec-63	12.3	21-Dec-63
49	25.7	27-May-98	16.3	17-Jan-98	12.9	8-Nov-78	11.3	9-Nov-78

Table 6.17. Maximum annual runoff, Nicomekl River above sea dam, future land use

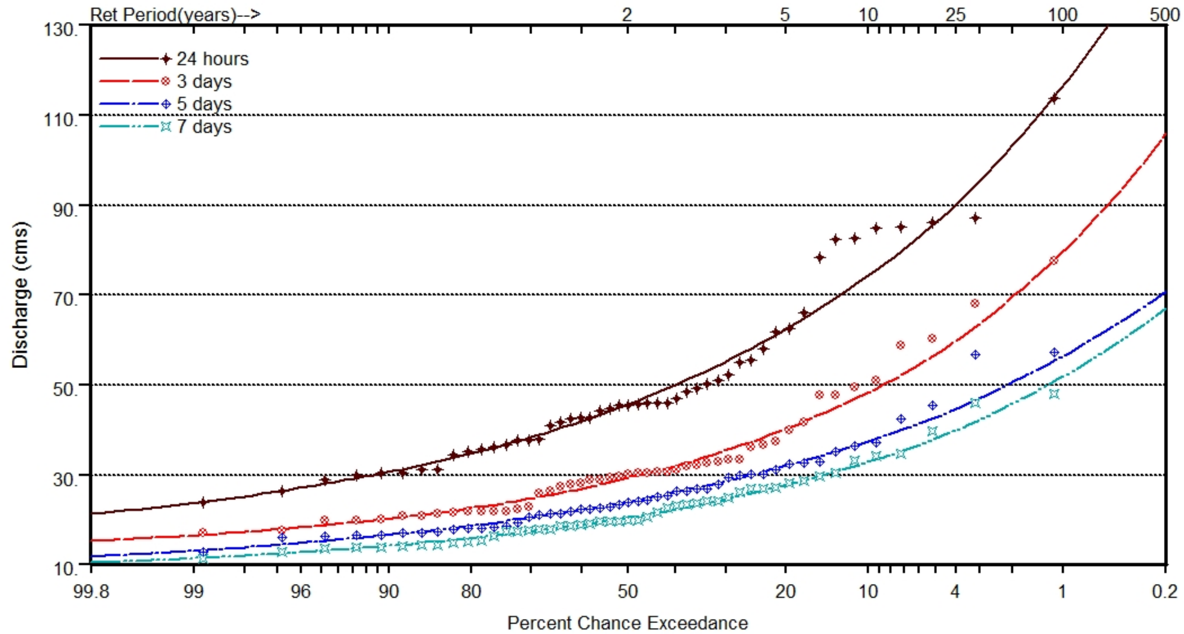
Duration	Maximum Annual Runoff Averaged over Specified Duration (m^3/s) under Future Land Use							
	24 hours		3 days		5 days		7 days	
Rank	Maximum Annual Runoff	Ending Date	Maximum Annual Runoff	Ending Date	Maximum Annual Runoff	Ending Date	Maximum Annual Runoff	Ending Date
1	135.5	17-Oct-03	87.2	19-Oct-03	63.3	21-Oct-03	51.8	12-Jan-09
2	99.1	26-Dec-72	74.6	9-Jan-09	62.3	11-Jan-09	50.8	23-Oct-03
3	97.0	12-Mar-07	65.7	21-Jan-68	49.6	21-Jan-05	43.2	23-Jan-05
4	96.9	18-Dec-79	64.1	20-Jan-05	45.3	23-Jan-68	37.5	27-Dec-72
5	96.2	19-Jan-68	55.2	28-Dec-72	40.4	26-Dec-72	37.3	19-Feb-82
6	93.7	7-Jan-09	54.1	19-Dec-79	39.5	17-Feb-82	35.1	24-Jan-68
7	92.2	18-Jan-05	52.4	15-Feb-82	38.2	19-Dec-79	33.3	20-Dec-79
8	86.4	30-Jan-97	51.7	13-Mar-07	35.4	13-Jan-06	31.6	18-Mar-07
9	76.3	14-Feb-82	46.8	5-Jan-84	35.3	17-Dec-01	30.6	14-Jan-06
10	70.8	4-Jan-84	43.9	20-Mar-97	34.7	14-Mar-07	30.5	17-Dec-99
11	69.0	15-Dec-99	40.1	17-Dec-99	34.0	6-Jan-84	29.2	17-Dec-66
12	66.1	12-Dec-10	39.9	15-Dec-01	32.5	15-Dec-99	29.0	26-Nov-09
13	64.7	13-Jul-72	39.2	11-Jan-06	32.4	16-Dec-66	28.7	17-Dec-01
14	61.9	14-Dec-01	37.5	5-Dec-07	32.3	1-Feb-92	28.0	3-Feb-92
15	60.9	26-Jan-71	36.5	21-Nov-80	31.2	1-Feb-97	27.0	8-Jan-84
16	60.4	4-Dec-07	36.4	1-Feb-92	30.2	26-Nov-09	26.0	14-Dec-10
17	57.4	16-Mar-74	36.1	31-Dec-62	29.6	30-Jan-71	25.6	4-Feb-99
18	55.4	30-Dec-62	35.6	25-Feb-86	29.0	23-Nov-80	25.3	1-Feb-71
19	54.1	16-Dec-66	35.3	14-Dec-10	29.0	2-Jan-63	25.1	3-Jan-63
20	53.6	15-Jan-10	34.2	14-Jul-72	28.9	13-Dec-10	24.9	3-Feb-97
21	53.0	30-Oct-75	34.1	16-Dec-66	28.8	24-Nov-86	24.3	8-Dec-75
22	52.8	4-Jan-69	34.1	11-Nov-90	28.6	17-Nov-98	23.4	25-Nov-80
23	52.7	29-Jan-99	34.0	6-Feb-65	27.5	13-Nov-90	22.7	25-Nov-86
24	52.7	17-Feb-65	33.5	16-Nov-98	26.5	7-Jan-69	22.4	14-Nov-90
25	52.2	21-Nov-80	33.4	16-Jan-96	26.1	27-Feb-86	22.1	7-Nov-88
26	51.5	10-Nov-89	32.7	18-Mar-74	25.0	15-Jan-08	22.1	17-Dec-73
27	51.0	10-Jan-06	32.2	22-Nov-09	24.9	16-Jan-76	21.0	9-Jan-69
28	50.7	6-Nov-88	31.9	5-Jan-69	24.5	6-Apr-88	20.9	16-Jan-08
29	50.6	18-Jan-86	31.5	17-Jan-76	24.4	8-Feb-65	20.7	9-Dec-89
30	50.3	10-Nov-90	31.2	28-Dec-94	24.4	14-Jul-72	20.2	1-Mar-86
31	49.6	5-Jan-01	30.6	11-Nov-89	24.0	7-Nov-88	20.1	16-Mar-72

32	47.6	23-Nov-86	30.0	24-Nov-86	23.8	17-Dec-73	20.1	8-Apr-88
33	46.9	31-Jan-92	29.6	28-Jan-71	23.6	18-Jan-96	19.8	4-Dec-82
34	44.2	6-Apr-88	29.0	6-Nov-88	22.8	8-Dec-89	19.8	2-Dec-77
35	43.8	3-Dec-82	25.3	22-Nov-74	22.2	30-Dec-94	19.5	21-Dec-74
36	43.1	13-Jan-66	24.7	6-Jan-01	21.0	21-Dec-74	18.8	30-Nov-95
37	42.3	26-Nov-77	24.3	14-Jan-66	20.9	21-Oct-00	18.7	10-Feb-65
38	41.9	14-Jan-96	24.2	19-Jan-77	20.6	30-Nov-77	17.4	1-Jan-95
39	41.3	20-Nov-74	24.1	2-Mar-94	19.9	4-Dec-82	17.1	22-Oct-00
40	41.3	19-Feb-95	23.8	21-Jan-70	19.9	15-Jan-66	16.7	20-Jan-98
41	41.2	23-Mar-93	23.7	4-Dec-82	19.8	4-Mar-94	16.3	5-Mar-94
42	36.8	19-Nov-02	23.6	8-Apr-88	19.2	6-Jan-03	16.1	6-Jan-03
43	36.5	14-Dec-84	23.5	4-Jan-03	18.8	23-Jan-70	15.6	16-Jan-66
44	35.8	18-Jan-77	23.5	24-Mar-93	18.6	16-Dec-84	15.2	14-Dec-84
45	35.5	16-Dec-63	22.6	17-Dec-63	18.3	19-Dec-63	15.1	18-Jan-77
46	35.0	2-Mar-94	22.3	28-Nov-77	18.1	19-Jan-77	15.1	11-Apr-70
47	33.5	19-Jan-70	21.5	14-Dec-84	18.1	25-Mar-93	14.8	24-Mar-93
48	31.0	25-Feb-79	19.5	27-Feb-79	18.0	18-Jan-98	14.7	21-Dec-63
49	27.8	27-May-98	18.6	19-Jan-98	14.6	8-Nov-78	12.9	9-Nov-78

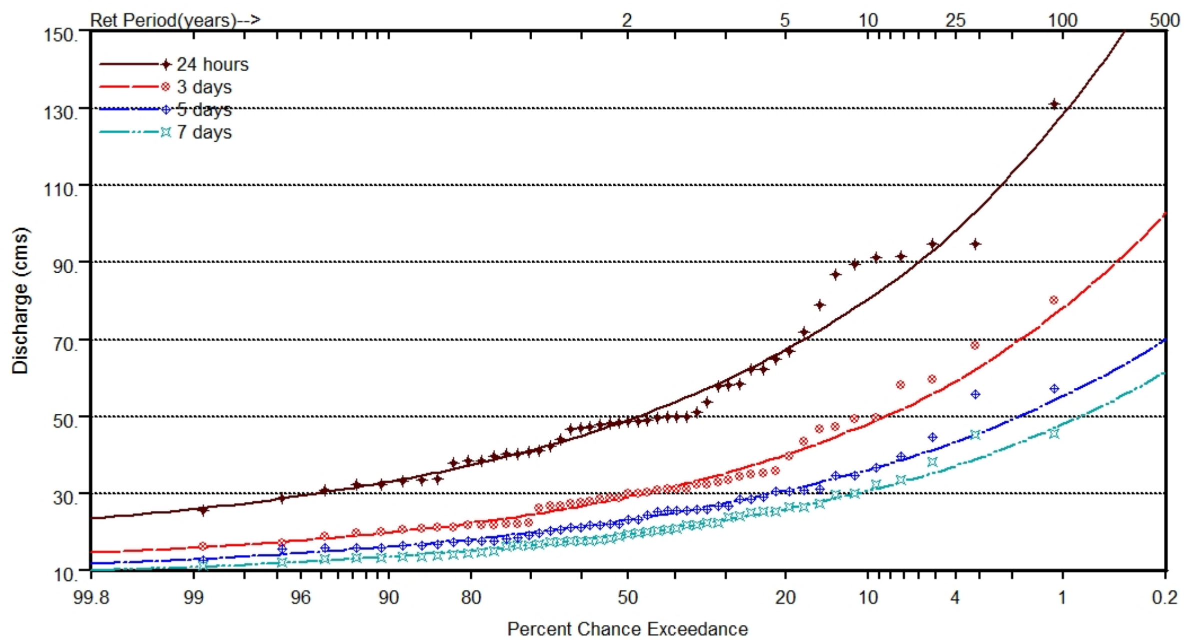
Table 6.18. Maximum annual runoff, Serpentine River above sea dam, future land use

Duration	Maximum Annual Runoff Averaged over Specified Duration (m^3/s) under Future Land Use							
	24 hours		3 days		5 days		7 days	
Rank	Maximum Annual Runoff	Ending Date	Maximum Annual Runoff	Ending Date	Maximum Annual Runoff	Ending Date	Maximum Annual Runoff	Ending Date
1	158.1	17-Oct-03	89.9	19-Oct-03	63.6	21-Oct-03	50.4	22-Oct-03
2	108.9	26-Dec-72	73.3	8-Jan-09	59.4	11-Jan-09	48.0	12-Jan-09
3	107.7	17-Dec-79	63.4	21-Jan-68	47.5	21-Jan-05	40.7	23-Jan-05
4	102.4	19-Jan-68	61.7	20-Jan-05	41.5	22-Jan-68	36.0	27-Dec-72
5	102.4	12-Mar-07	53.4	19-Dec-79	40.1	26-Dec-72	34.3	19-Feb-82
6	99.7	7-Jan-09	51.9	28-Dec-72	38.2	18-Dec-79	32.5	20-Dec-79
7	94.8	18-Jan-05	50.9	15-Feb-82	36.7	17-Feb-82	31.0	24-Jan-68
8	89.0	30-Jan-97	48.8	13-Mar-07	33.1	17-Dec-01	28.6	17-Mar-07
9	83.4	14-Feb-82	48.2	4-Jan-84	33.1	13-Jan-06	28.2	17-Dec-99
10	76.4	4-Jan-84	42.9	20-Mar-97	32.5	6-Jan-84	27.9	14-Jan-06
11	74.5	12-Jul-72	38.3	21-Nov-80	32.0	15-Dec-99	27.4	22-Nov-09

12	73.0	15-Dec-99	38.2	15-Dec-01	31.6	14-Mar-07	27.2	16-Dec-66
13	71.0	12-Dec-10	38.1	5-Dec-07	30.6	16-Dec-66	26.7	17-Dec-01
14	69.5	3-Dec-07	37.5	25-Feb-86	30.3	1-Feb-92	25.6	2-Feb-92
15	68.5	26-Jan-71	37.1	11-Jan-06	29.7	24-Nov-86	25.2	8-Jan-84
16	63.7	13-Dec-01	37.0	16-Dec-99	29.4	30-Jan-71	24.0	14-Dec-10
17	62.2	16-Dec-73	36.0	15-Nov-98	29.2	17-Nov-98	23.7	1-Feb-71
18	60.1	4-Feb-65	35.5	31-Dec-62	28.6	22-Nov-80	23.4	25-Nov-86
19	59.4	30-Oct-75	35.5	11-Nov-90	28.4	22-Mar-97	23.3	3-Jan-63
20	58.8	30-Dec-62	35.0	14-Jul-72	28.1	12-Dec-10	22.9	4-Feb-99
21	58.3	18-Jan-86	34.8	31-Jan-92	27.6	13-Nov-90	22.6	8-Dec-75
22	57.5	10-Nov-90	34.0	6-Feb-65	27.6	26-Nov-09	22.4	6-Nov-88
23	57.4	4-Jan-69	32.9	16-Jan-96	26.9	2-Jan-63	22.3	13-Nov-90
24	56.9	15-Jan-10	32.9	14-Dec-10	26.2	14-Jul-72	22.1	24-Nov-80
25	56.9	16-Dec-66	32.7	16-Dec-66	25.3	27-Feb-86	21.3	3-Feb-97
26	56.6	10-Nov-89	31.9	19-Oct-75	25.0	7-Jan-69	21.1	10-Nov-89
27	56.5	29-Jan-99	31.8	22-Nov-09	24.8	30-Oct-75	20.4	16-Mar-74
28	55.9	21-Nov-80	31.7	11-Nov-89	24.2	15-Jan-08	19.9	4-Dec-82
29	55.4	6-Nov-88	31.3	5-Jan-69	24.0	6-Nov-88	19.9	2-Nov-85
30	54.1	5-Jan-01	30.8	28-Dec-94	23.9	6-Apr-88	19.6	15-Jul-72
31	52.1	23-Nov-86	30.5	5-Nov-88	23.4	8-Feb-65	19.5	2-Dec-77
32	51.8	9-Jan-06	30.4	24-Nov-86	22.8	16-Dec-73	19.5	15-Jan-08
33	48.4	6-Apr-88	30.2	18-Mar-74	22.1	21-Oct-00	19.0	9-Jan-69
34	48.3	26-Nov-77	28.5	28-Jan-71	21.6	18-Jan-96	18.7	30-Nov-95
35	48.0	13-Jan-66	25.6	22-Nov-74	21.2	11-Nov-89	18.6	8-Apr-88
36	48.0	20-Nov-74	24.8	2-Mar-94	20.8	22-Nov-74	18.5	21-Dec-74
37	47.8	3-Dec-82	24.6	23-Mar-93	20.6	3-Dec-82	17.3	10-Feb-65
38	47.6	24-Jan-92	24.3	14-Jan-66	20.4	30-Dec-94	17.1	22-Oct-00
39	47.5	23-Mar-93	24.3	3-Dec-82	20.1	30-Nov-77	15.7	21-Dec-94
40	46.1	30-Nov-94	24.2	6-Jan-01	19.8	14-Jan-66	15.7	24-Mar-93
41	46.0	19-Nov-02	24.1	21-Jan-70	19.1	4-Mar-94	15.6	13-Jan-66
42	44.9	14-Jan-96	23.8	19-Jan-77	18.6	6-Jan-03	15.6	6-Jan-03
43	40.8	14-Dec-84	23.8	4-Jan-03	18.3	25-Mar-93	15.5	20-Jan-98
44	39.3	15-Dec-63	22.5	17-Dec-63	18.2	9-Apr-70	15.5	5-Mar-94
45	38.6	18-Jan-77	22.3	28-Nov-77	17.6	18-Jan-77	15.2	18-Jan-77
46	38.2	2-Mar-94	21.9	8-Apr-88	17.5	19-Dec-63	14.9	11-Apr-70
47	36.3	19-Jan-70	21.9	14-Dec-84	17.4	16-Dec-84	14.7	14-Dec-84
48	36.2	27-May-98	19.0	27-Feb-79	17.2	18-Jan-98	13.4	21-Dec-63
49	34.7	25-Feb-79	18.2	17-Jan-98	14.7	8-Nov-78	13.0	9-Nov-78

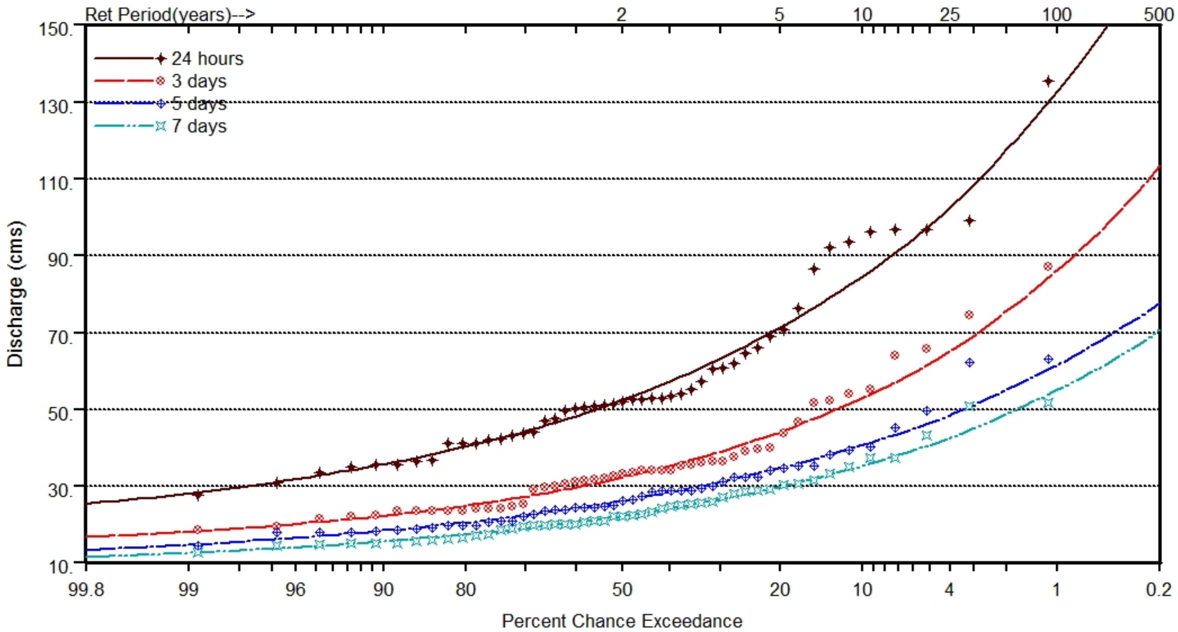


a) Nicomekl River above sea dam

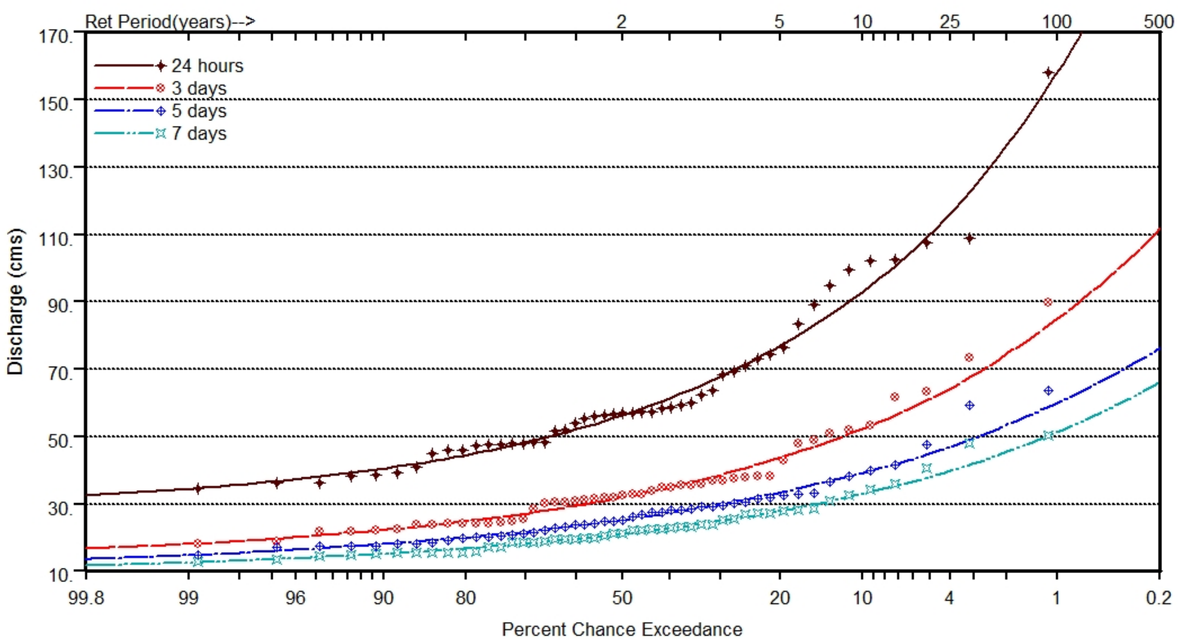


b) Serpentine River above sea dam

Figure 6.8. Runoff frequency curves for existing land use: a) Nicomekl River above sea dam; b) Serpentine River above sea dam



a) Nicomekl River above sea dam



b) Serpentine River above sea dam

Figure 6.9. Runoff frequency curves for future land use: a) Nicomekl River above sea dam; b) Serpentine River above sea dam

Table 6.19. Runoff quantiles for existing land use

	Average Runoff (m3/s) under Existing Land Use by Return Period (years)					
Duration	**2**	**10**	**50**	**100**	**200**	**500**
Nicomekl						
24-hours	45.8	74.7	104	117	132	150
3-days	29.3	48.7	69.9	80.0	91.8	106
5-days	24.1	37.7	50.8	56.6	63.1	71.0
7-days	20.6	33.1	46.1	52.1	59.0	67.3
Serpentine						
24-hours	49.1	81.0	114	129	146	167
3-days	29.2	48.4	68.8	78.5	89.6	103
5-days	23.3	36.5	49.5	55.6	62.2	70.2
7-days	19.5	31.0	42.8	48.1	54.4	61.8
Quantiles are for total land surface runoff above the sea dams						

Table 6.20. Runoff quantiles for future land use

	Average Runoff (m3/s) under Future Land Use by Return Period (years)					
Duration	**2**	**10**	**50**	**100**	**200**	**500**
Nicomekl						
24-hours	52.6	85.2	118	133	150	171
3-days	32.4	53.4	75.8	86.4	98.9	114
5-days	26.3	41.0	55.3	61.8	69.1	77.8
7-days	22.3	35.5	49.0	55.2	62.3	70.8
Serpentine						
24-hours	56.4	93.7	137	159	185	216
3-days	32.1	52.7	74.7	85.2	97.3	112
5-days	25.4	39.5	53.7	60.2	67.6	76.5
7-days	21.2	33.2	45.7	51.5	58.2	66.3
Quantiles are for total land surface runoff above the sea dams						

7 HYDRAULIC MODELLING

7.1 PURPOSE OF HYDRAULIC MODELLING

Sections 5 and 6 described the development of ocean level and inflow boundary conditions for continuous simulation hydraulic modelling, spanning the period 1964 to 2011. The purpose of the hydraulic modelling was to generate time-series of flood levels at a number of locations in the Serpentine and Nicomekl basins. A frequency analysis of annual peak levels was then carried out to estimate the 200-year flood levels as described in Section 8. The 200-year levels derived in this manner reflect the actual joint probability of high ocean levels and precipitation.

Since rain storms are intense and of relatively short duration and the river systems are tidally influenced, a hydrodynamic model with a relatively short computational time-step was required. Considering the long time periods to be simulated, the model had to be fairly simple in order to maintain reasonable run-times.

This section of the report outlines the hydraulic model software selection, the available background information and describes the model development, verification and sensitivity analyses. The model limitations, required simulations and final results are also discussed.

7.2 MODELLING BACKGROUND

7.2.1 SOFTWARE SELECTION

KPA (1993) developed a hydraulic model for the Serpentine and Nicomekl Rivers using Environment Canada's ONE-D software. This model was later converted by UMA (2001) to MIKE11, a one-dimensional hydrodynamic software developed by the Danish Hydraulic Institute (DHI). Over time, the model was updated by UMA, and later by KWL, and was primarily used for the verification of the City individual Functional Plans. The current MIKE11 model includes a roughly 30 km reach of each river and depicts the drainage system in great detail, incorporating the two sea dams, a multitude of pump stations, floodboxes, spillways, culverts, bridges, dikes, and an intricate system of ditches and canals on the floodplain.

An initial consideration was made to use the existing MIKE11 model for the continuous simulations. However, this approach was found to be unsuitable for the following reasons:

- The model uses a time step of 15 seconds and it would take roughly half a year to run a 1964 - 2011 time-series (limited to one computer by the license restrictions).
- The model is not supported by the current MIKE11 software and requires the use of a 2003 DHI licence, no longer available for purchase.
- A series of datum shifts have taken place in Surrey and extensive updates to the model geometry and structure elevation input would be necessary to accurately represent current conditions.

Simplifying, updating and modifying the existing MIKE11 model was considered but this was felt to be impractical given the complexities of the full model. Instead, a new simplified model was developed which focused on the main-stem rivers. The model excluded the intricate network of ditches on the floodplain and replaced these with storage cells. The approach resulted in reasonable computational run-times.

Both DHI's one dimensional hydrodynamic model, MIKE11, and the US Corps of Engineers' equivalent model HEC-RAS, are well suited for this type of modelling and each was carefully considered for the application. Although a good deal of relevant input information was readily available in the MIKE11 format, a decision to use the HEC-RAS software was made because:

- Storage cells, spillways and bridges are more explicitly represented.
- The software is free of charge and can readily be used by different modellers without requiring the purchase of a licence.
- The HEC-RAS DSS input/output interface readily connects with the HSPF hydrologic model (Section 6) and NHC's in-house frequency analysis program 'DASH' (Section 8).
- Modifications can readily be made to adapt the model to test future scenarios.

More detailed two dimensional modelling may be desirable in the future and the ease of converting the 1D model to 2D also influenced the software selection. DHI's MIKEFlood software couples 1D MIKE11 with 2D MIKE21. However, it is important to note that for any potential future two dimensional modelling, the existing MIKE11 model would essentially need to be re-configured to develop a working MIKEFlood model (ie 2D model of the floodplain).

The HEC-RAS model would not need to be re-configured and can be linked with a Flow2D model to assess site specific dike breach scenarios. The HEC-RAS model can also be used directly to assess breaching of the sea dams and as a tool to determine breach hydrographs for the river dikes. These breach hydrographs can then be used as the boundary condition for more detailed 2D analysis of local breach scenarios and resulting inundation of the floodplain.

7.2.2 PREVIOUS STUDIES AND DATA REVIEWED

Reporting

There are a large number of documents and past studies related to the floodplain of the Serpentine and Nicomekl Rivers, particularly on the flood control plan and its verification and implementation. Reports reviewed are listed in Appendix A.2.

Water Level Gauges and Related Information

Water level gauges located within the floodplain are listed in Table 7.1 and Table 7.2 and were used for hydraulic model verification.

The City has nine water level gauges located within the study area. The data are partly available on the City's FlowWorks website (5 minute intervals). Additional data is available for certain periods. The location, period of record and comments on the quality of the data are summarized in Table 7.1.

The City also operates a series of continuous water level gauges installed at pump stations along the Serpentine and Nicomekl Rivers which are accessible through the SCADA system. The gauge locations and available water level measurements are summarized in Table 7.2.

Table 7.1. Water level monitoring stations located within the floodplain (data from FlowWorks)

Name	Period		Interval	Location		Notes
	start	end		UTM X	UTM Y	
Serpentine River at Sea dam	1998	present	5 mins	513223	5437119	data missing for Jan 2005 event
Serpentine River at Hwy 10	2000	2008	5 mins	516822	5439090	gauge removed in 2008 for highway construction
Serpentine River at 168 St.	1996	present	5 mins	517739	5446708	missing Feb-05 to May-05 and Dec-08 to Apr-10, numerous small data gaps
Serpentine River at 104 Ave.	1996	present	5 mins	515937	5448778	missing Feb-09 to May-09 and Jan-10 to Apr-10, numerous data gaps prior to 2009
Serpentine River at 80 Ave.	1998	2001	5 mins	514505	5443576	missing Aug-99 to Oct-99, numerous small data gaps
Latimer Creek at Harvie Road	2001	present	5 mins	521210	5444790	missing Dec-08 to Apr-10 and May-03 to Dec-03, numerous small data gaps
Bear Creek at 152nd Street (Surrey Lake)		present	5 mins	514515	5443579	continuous data (no gaps) (problem with 2009 water levels)
Nicomekl River at Sea dam	1998	present	5 mins	512771	5435196	data missing for Jan 2005 event
Nicomekl River at 192nd St.	2005	present	5 mins	522655	5438111	previous WSC gauge (1952-1981), no data for Jan 2005

Table 7.2. Summary of SCADA water level recording gauges

Name	Type				Location			
	Ditch	Creek	River	Sump	River	UTM X	UTM Y	Address
150TH ST PS	✓			✓	Serpentine	514232	5437274	15050 47 AVE
40TH AVENUE PS	✓			✓	Nicomekl	514866	5436007	15350 41 AVE
48TH AVENUE PS	✓			✓	Serpentine	515653	5437494	15750 48 AVE
64TH AVENUE PS	✓		✓		Serpentine	516469	5440660	16150 64 AVE
BURROWS I PS	✓		✓		Nicomekl	518528	5436887	4550 172 ST
COAST MERIDIAN PS	✓			✓	Serpentine	518069	5442325	16950 72 AVE
COLEBROOK PS	✓				Serpentine	511279	5437049	4600 136 ST
EAST NEWTON PS	✓		✓	✓	Serpentine	515294	5441532	15600 68 AVE
ERICKSON DITCH PS	✓		✓		Nicomekl	520144	5437236	4700 180 ST
FLEETWOOD PS	✓		✓		Serpentine	516140	5442021	7050 160 ST
FRYS CORNER PS	✓		✓	✓	Serpentine	519276	5443161	7627 176 ST
NORTH FRYS CORNER PS	✓		✓	✓	Serpentine	519282	5443268	7701 176 ST
GREY CREEK PS	✓			✓	Serpentine	515525	5438050	5100 157 ST
HALL'S PRAIRIE PS	✓				Nicomekl	520153	5437290	4750 180 ST
HOOKEBROOKE PS	✓				Serpentine	515914	5439892	15900 60 AVE
LOGGING DITCH PS	✓		✓	✓	Nicomekl	516774	5436616	16300 44 AVE
NICOMEKL DITCH PS	✓				Nicomekl	517775	5437568	4850 168 ST
PANORAMA PS	✓	✓			Serpentine	513687	5437474	48 AVE & 148 ST
SOUTH CLOVERDALE PS	✓		✓		Nicomekl	518383	5437374	17100 48 AVE
UPPER SERPENTINE PS	✓		✓	✓	Serpentine	519798	5445191	17800 86 AVE

Pump Station Information and Other Structure Information

Data for the pumps, dikes and other structures or features that were included in the HEC-RAS model was taken from the existing MIKE11 model and other sources of information. Details are provided in the following section.

Existing MIKE11 model files

The City provided a list of modelling files for various MIKE11 model builds. The 2010 post construction model geometry was used as a starting point for the HEC-RAS model.

7.3 HEC-RAS Model Development

Information on the river network, channel cross-sectional geometry, floodplain topography and the hydraulic structures, such as the sea dams, spillways, pumps, floodboxes, bridges and culverts was required to develop the HEC-RAS model. This information was either provided by the City, or extracted from the existing MIKE11 model. The model schematic is shown in Map 1. Compared to the MIKE11 model, the HEC-RAS model eliminated a myriad of drainage ditches, smaller floodboxes and some minor pump stations, but the model incorporated all significant hydraulic features to provide a reasonable representation of the system. Effort was made to represent the drainage system as accurately as possible without substantially increasing model run-times. To some extent, this involved a trial and error procedure.

The different model inputs were largely pre-processed in GIS. A Digital Elevation Model (DEM) was developed for the areas where new cross-sections had to be cut. Hydraulic structure locations were identified in GIS and detailed dimensions summarized in Excel files that allowed the information to be easily transferred to HEC-RAS.

7.3.1 Model Geometry

Network

The Serpentine River is represented in the model from Mud Bay to 168th Street north of 88th Ave. Similarly, the Nicomekl River is included from Mud Bay to roughly 6 km east of the City of Surrey boundary, in the Township of Langley. In addition, reaches representing Bear Creek, Latimer Creek, Cloverdale Canal, and 168th St Canals were included in the model.

Network chainages were set to increase in the upstream direction. All branches have a downstream chainage value of zero except for the Serpentine and Nicomekl Rivers which have their zero chainage at the sea dams and negative chainages on the ocean side of the sea dams.

Cross-sections

The model channel geometry reflects 2011 or 2012 channel cross-sections in reaches where recent survey data was available (see Figure 7.1). Cross-sections in all other reaches were based on the existing MIKE11 model data adjusted for datum shifts. Additional cross-sections were interpolated as necessary. The HEC-RAS model has a total of 910 cross-sections of which 371 were interpolated.

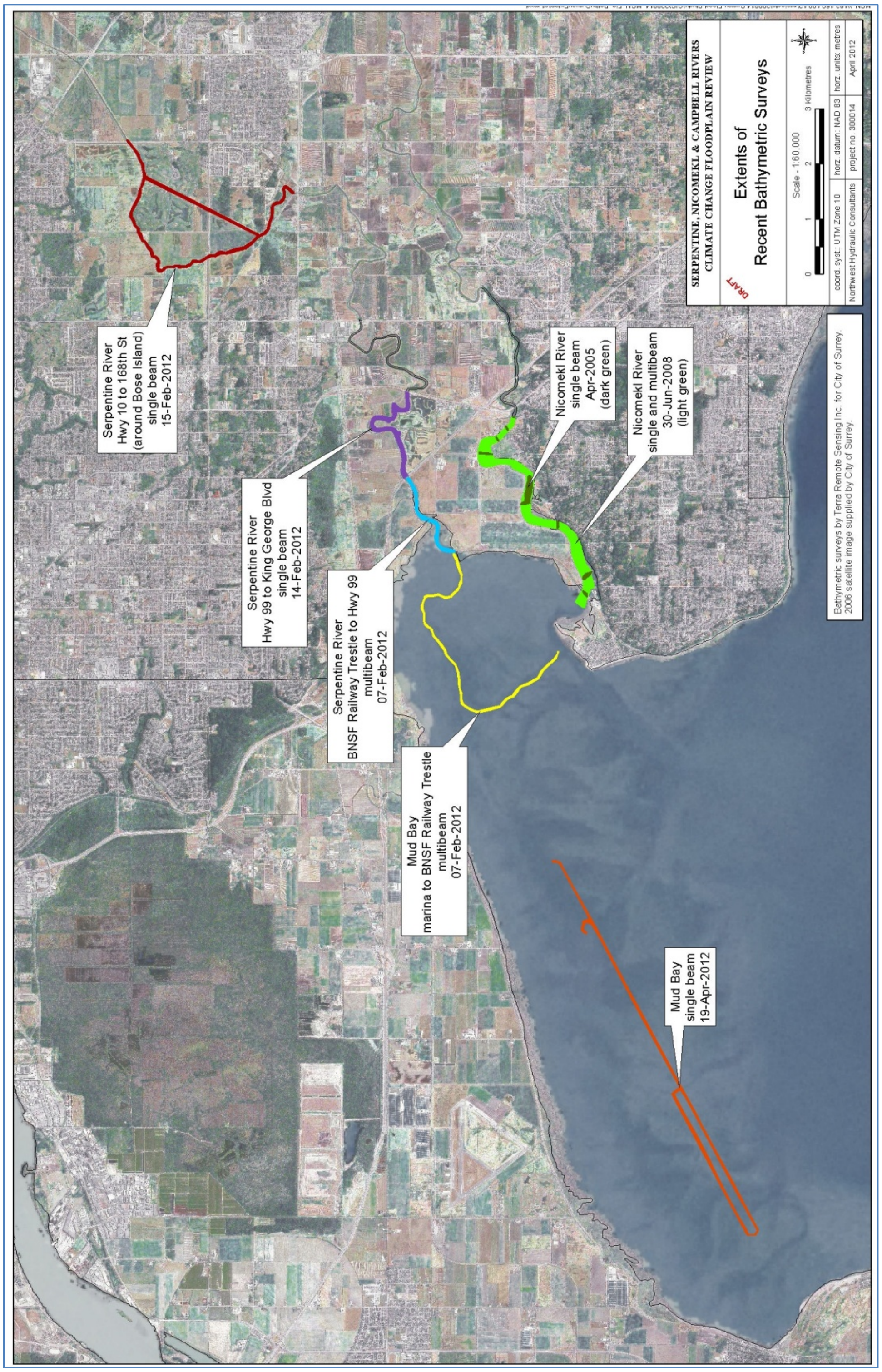


Figure 7.1. Extents of recent bathymetric surveys

Floodplain

An idealized representation of the floodplain was applied, with a minimum number of storage cells and hydraulic structures. Flood waters can pass between the floodplain cells and the river based on the hydraulic capacity of the interconnecting links and the head difference across these.

The floodplain was treated as 13 storage cells and the storage cells were connected to the main channel either through lateral weir structures (representing spillways), floodboxes, and/or pump stations. The hydraulic geometry of the floodplain cells was established by developing volume-stage curves using the developed DEM.

It is recognized that this simplification does not allow highly accurate representation of flood cell water levels. For example, Cell 5 is nearly 8 km long and will clearly not have a horizontal flood level across its full length. Yet, the model was expected to provide reasonably accurate water levels in the river channels and an indication of water levels in the flood cells. Further division of the cells can be made in future phases.

Hydraulic Structures

Hydraulic structures included in the model consisted of sea dams, spillways, floodboxes, bridges, culverts and pump stations. The structures in the model were selected based on their hydraulic significance on water levels and the availability of data to represent their geometry and operation.

The sea dams on the Serpentine and Nicomekl Rivers were represented as flap-gated culverts that were controlled by the difference in water levels across the dams. This ensured that the sea dam gates were closed whenever the water level on the ocean side was higher than on the river side.

Temporary and permanent spillways have been constructed (or are planned) to control the locations and volume of spills onto the floodplain. A total of 21 spillways (temporary and permanent) were included in the RAS model (Table 7.3). Spillway dimensions and elevations were adjusted in the model, based on available information, to reflect the spillways present in 2009 (for calibration) and those expected for future buildout conditions (for 200-yr simulations).

Floodboxes allow water to drain by gravity from the floodplain back into the river channel once the water levels in the river have receded. Complete information was compiled for a total of 91 floodboxes based on correlating data found in the City database and information stored in the MIKE11 model. A total of 21 floodboxes with a diameter of 1 m or greater were included in the model (Table 7.4).

A complete description of bridge crossings, in sufficient detail to represent them in the model, was not available for all bridge crossings. The most recent available information was used to represent the bridges (Table 7.5). A total of 9 out of 10 bridges were included on the Nicomekl River while 11 out of 18 bridges were represented on the Serpentine River.

Pump stations in the model transfer water from the floodplain to the river channel based on a specified pumping capacity curve and specified on/off reference water levels on the floodplain. A total of 21 pump stations were included in the HEC-RAS model (Table 7.6). Pump capacity curves were copied from the MIKE11 model and adjusted as necessary to reflect recent pump tests. Pump on/off reference levels were set to the winter operation levels. However, contrary to actual operations, pumps in the model stayed on even after spillway activation. This was not considered to have a significant impact on simulated water levels.

In order to simplify the representation of dikes in the model, all dikes were assumed to be raised and not overtopped.

Table 7.3. Details of spillways included in HEC-RAS model

		2005			2009			Build Out			Location				Spillway Activation		
Overbank	Storage Area	ID	Width	Invert	ID	Width	Invert	ID	Width	Invert	Easting	Northing	Note	Comment	nearest RAS Key Location (STA)	Return Period (yrs) EX	Return Period (yrs) FUT
Left	Cell 13	none	-	-	none	-	-	S-1	10	2.95	518568	5445438	accurate		17508	>200	5
Left	Cell 12	none	-	-	none	-	-	S-2	20	3.06	521210	5444765	accurate				
Right	Cell 11	none	-	-	none	-	-	S-3	40	2.84	518845	5445146	accurate		16946	>200	25
Right	Cell 12	none	-	-	none	-	-	S-4	40	2.62	519955	5444359	accurate		16017	>200	<2
Left	Cell 12	none	-	-	none	-	-	S-5	45	2.53	519491	5443551	accurate		13846	>200	<2
Right	Cell 11	S-6	30	1.6	S-6	33	1.78	S-6	40	2.47	519194	5443127	accurate		13846	>200	<2
Left	Cell 9	none	-	-	none	-	-	S-9	45	2.56	515331	5442337	uncertain				
Right	Cell 9	none	-	-	none	-	-	S-10	30	2.56	515294	5442327	uncertain				
Right	Cell 9	none	-	-	none	-	-	S-12	25	2.46	516926	5443132	uncertain				
Right	Cell 6	none	-	-	none	-	-	S-14	100	2.51	520589	5437029	uncertain	confirm B.O. wid and inv	11978	>200	<2
Left	Cell 5	none	-	-	none	-	-	S-15	100	2.55	520243	5437215	uncertain	confirm B.O. wid and inv	11012	>200	<2
Left	Cell 6	S-16			S-16			S-16	10	0.34	517803	5438430	accurate	need 2005 & 2009 wid and inv			
Left	Cell 6	none	-	-	none	-	-	S-17	5	0.43	517802	5438713	accurate				
Left	Cell 6	none	-	-	none	-	-	S-18	10	0.55	517800	5438959	accurate				
		none	-	-	none	-	-	S-19			519038	5440936	unknown	need location and B.O. wid and inv			
Right	Cell 12	TS-11			TS-11	11	1.52	S-68	10	2.49	518237	5441525	accurate	need 2005 wid and inv			
Left	Cell 13	TS-1	35	2	TS-1	40	2.1	TS-1	40	4.17	518410	5445632	accurate				
Right	Cell 11	TS-2	3	2.1	TS-2	7	1.9	TS-2	7	4.17	518354	5445619	accurate				
Right	Cell 13	TS-3	15	1.55	TS-3	23	1.6	TS-3	23	3.64	520012	5444805	accurate				
Left	Cell 12	TS-4	45	1.05	TS-4	46	1	TS-4	46	3.64	519996	5444767	accurate				
Left	Cell 13	TS-7	13	1.94	TS-7	15	1.65	TS-7	15	3.83	518901	5445186	accurate				
Right	Cell 9	TS-9			TS-9	17	0.89	TS-9	17	2.96	517720	5442367	accurate	need 2005 wid and inv			
		TS-10			TS-10	10	1.23	TS-10	10	3	518112	5441475	uncertain	need 2005 wid and inv			
		TS-12			TS-12	18	1.02	TS-12	18	3.11	517689	5442150	uncertain	need 2005 wid and inv			
		none	-	-	none	-	-	TS-13			520340	5436838	uncertain	need location and B.O. wid and inv			
		none	-	-	none	-	-	TS-14			520701	5437067	uncertain	need location and B.O. wid and inv			

Table 7.4. Details of floodboxes included in HEC-RAS model

Input in RAS model	ID	River and Reach	Chainage	Overbank	Storage Area	Size or Diameter (mm)	Length (m)	Invert Elevations (m)		# of barrels	Easting	Northing
✓	8	Nicomekl River Main DS	961	Right	Cell 4	1020	18.2	-1.58	-1.44	1	513448	5435136
✓	9	Nicomekl River Main DS	4840	Left	Cell 5	1060	18.6	-1.82	-1.55	1	516118	5436166
✓	10	Nicomekl River Main DS	5864	Right	Cell 4	1020	18.4	-1.61	-1.5	1	516888	5436807
✓	7	Nicomekl River Main US	9470	Left	Cell 5	1220	37	-1.75	-1.66	1	519343	5437046
✓	1	Serpentine River Main	-1668	Right	Cell 1	1300	20	-1.64	-1.64	2	512537	5437535
✓	21	Serpentine River Main	35	Right	Cell 3	1200	18.5	-1.76	-1.62	1	513253	5437103
✓	20	Serpentine River Main	1122	Left	Cell 4	1200	16.2	-1.78	-1.69	1	513674	5437375
✓	2	Serpentine River Main	2216	Left	Cell 4	915	16.6	-1.64	-1.4	1	514177	5437671
✓	3	Serpentine River Main	3370	Right	Cell 3	1220	45.3	-1.65	-1.26	1	514493	5437969
✓	19	Serpentine River Main	3445	Left	Cell 4	1020	102	-1.62	-1.31	1	514529	5437973
✓	18	Serpentine River Main	5434	Right	Cell 3	1060	21.5	-1.65	-1.63	1	515885	5437723
✓	4	Serpentine River Main	6570	Right	Cell 3	1020	18.6	-1.86	-1.85	1	516831	5438285
✓	17	Serpentine River Bose	9014	Left	Cell 10	1050	36	-1.96	-1.88	1	516118	5439908
✓	16	Serpentine River Upper DS	11814	Left	Cell 10	1050	18	-0.28	-0.19	1	517655	5442098
✓	5	Serpentine River Upper MID	12260	Left	Cell 12	1200	19	-2.16	-1.69	1	518025	5442343
✓	15	Serpentine River Upper MID	13460	Left	Cell 12	1050	10	-2.24	-2.24	1	519047	5442972
✓	14	Serpentine River Upper MID	14400	Left	Cell 12	1000	20	-1.8	-1.8	1	519551	5443749
✓	13	Serpentine River Upper MID	14600	Right	Cell 12	1060	20	-2	-2	1	519663	5443950
✓	12	Serpentine River Upper MID	15445	Right	Cell 11	1050	38.5	-1.47	-1.47	1	519936	5444758
✓	11	Serpentine River Upper US	16415	Left	Cell 13	1000	40	-0.9	-0.9	1	519326	5445154
✓	6	Serpentine River Upper US	16960	Right	Cell 11	1400	20	-1.39	-1.39	1	518824	5445163

Table 7.5. Details of bridges included in HEC-RAS model

Input in RAS model	Bridge Name	RAS CH	Width (along stream)	Upstream Face: Low Chord El.	Upstream Face: High Chord El.	Upstream Face: Constant across face?	Upstream Face: Opening width between Abutments	Downstream Face: Low Chord El.	Downstream Face: High Chord El.	Downstream Face: Constant across face?	Downstream Face: Opening width between Abutments	Piers: No.	Piers: Size (width or diameter)	Comment on quality of data entered in RAS
		m	m	m	m		m	m	m		m		m	
Nicomekl River														
ignore	RR Bridge													
✓	Sea Dam	0	10											good
✓	King George Blvd S/B Lanes + 1 N/B	209	11	1.9	2.62	Y	52	2.64	3.13	Y	52	9	1	approx
✓	Hwy 99	1010	45	2.4-4.45	4.95-7	N	110	2.4-4.45	4.95-7	N	110	14	0.86	v. approx
✓	152 St	2101	11	13.85	9.219, 16.726		114.93	13.85	9.219, 16.726		114.93	2	0.9	approx
✓	40 Ave Bridge	2891	8	1.88	2.55	Y	48.8	1.875	2.55	Y	48.8	9	0.31	good
✓	168 St Bridge	7216	7.93	2.28	2.49	Y	43.54	2.28	2.49	Y	43.54	7	0.31	good
✓	176 St Bridge	9486	13.7	3.34	5	Y	50	3.34	5	Y	50	2	1	approx
✓	184 St Bridge	11871	12.3	3.26	4.08	N	45	3.15	3.97	N	45	2	0.61	approx
✓	192 St Bridge	14318	12.3	4.08	5.21	N	44	4.11	5.24	N	44	1	0.61	good
	small unnamed crossing													
Serpentine River														
ignore	RR Bridge													
	Bridge Hwy 99	-2651												
✓	Sea Dam	0												good
	King George Blvd main	0												
✓	152 St Bridge	3405	20.78	2.65	4.16	N	52.6	2.63	4.14	N	52.6	1	0.61	good
ignore	160 St Bridge	5791												
	BCR Bridge	7045												
	56 Ave (Hwy 10) Bridge	7503												
	SRY Bridge	7593												
✓	64 Ave Bridge (Canal)	9917	20.78	2.00	3.69	N	40	2.03	3.72	N	40	1	0.61	good
✓	64 Ave Bridge (River)	9929	20.78	2.05	3.74	N	36	2.04	3.73	N	36	1	0.61	good
ignore	Northview Golf Bridge	10771												
✓	168 St Pedestrian Bridge	11932	3.22	2.8	3.03	Y	43.89	2.8	3.03	Y	43.89	8	0.33	good
✓	168 St Bridge	11960	10.90	3.02	3.82	N	35.26	2.8	3.6	N	35.26	1	0.66	good
✓(2011)	Fraser Hwy Bridge (South)	13551	20	3	3.93	Y	36	3	3.93	Y	36	1	0.61	approx
	176 St Bridge	13912												
✓	80 Ave Bridge	14622	10.12	1.88	3.09	N	21.64	1.93	3.141	N	21.64	3	0.30	good
✓	Fraser Hwy Bridge (North)	16469	25.7											v. approx
✓	88 Ave Bridge	17688	13.4	3.03-3.2	4.14-4.3	N	56	3.2-3.37	4.14-4.3	N	56	2	0.61	good
✓	cattle bridge	18091	3.5	2.9	3.4	Y	12	2.9	3.4	Y	12	0		ok
	168 St Bridge	19244												

Table 7.6. Details of pump stations included in HEC-RAS model

Input in RAS model	Location				Pump Capacity		Reference Water Level					
			To			Stantec	Pump ON	Pump OFF			River	
	Pump Station Name	From	River/Reach	RAS Chainag	KWL M11 model	Pump Tests	Levels (m) Winter	Levels (m) Winter	Easting	Northing	Monitorin g	Year Built
✓	150TH ST PS	Cell 4	Serpentine River Main	1821	**300**	300	-1.36	-1.66	514232	5437274	No	2003
✓	40TH AVENUE PS	Cell 4	Nicomekl River Main	3297	**300**	300	-1.40	-1.70	514866	5436007	No	2003
✓	48TH AVENUE PS	Cell 4	Serpentine River Main	5101	**300**	300	-1.36	-1.66	515653	5437494	No	2003
✓	64TH AVENUE PS	Cell 8	Serpentine River Bose	9844	**38-47***	43	-2.06	-2.52	516469	5440660	Yes	1990
✓	-	-	-	-	**315-474***	452	-1.96	-2.42				-
✓	BURROWS I PS	Cell 5	Nicomekl River Main	8625	**430-597***	573	-0.90	-1.30	518528	5436887	Yes	
✓	-	-	-	-	**430-597***	573	-0.70	-1.10				-
✓	COAST MERIDIAN PS	Cell 12	Serpentine River Upper	12291	**205**	164-228	-1.32	-1.56	518069	5442325	No	2007
✓	-	-	-	-	**205**	164-228	-1.06	-1.36				-
✓	COLEBROOK PS	Cell 1	Serpentine River Main	-3078	**605-1050**	900	-1.56	-1.76	511279	5437049	No	1990
✓	-	-	-	-	**605-1050**	900	-1.46	-1.76				-
✓	-	-	-	-	**605-1050**	900	-1.36	-1.76				-
✓	EAST NEWTON PS	Cell 9	Serpentine River West	10874	**950**	-	-1.66	-2.26	515294	5441532	Yes	2004
✓	-	-	-	-	**950**	-	-1.06	-1.86				-
✓	ERICKSON DITCH PS	Cell 5	Nicomekl River Main	10400	**1900**	1900	-1.90	-2.30	520144	5437236	Yes	1992
✓	-	-	-	-	**1900**	1900	-1.50	-2.10				-
✓	FLEETWOOD PS	Cell 9	Serpentine River West	11824	**1200**	-	-0.56	-1.26	516140	5442021	Yes	1994
✓	FRYS CORNER PS	Cell 12	Serpentine River Upper	13757	**2500**	2000	-0.96	-1.56	519276	5443161	Yes	2000
✓	-	-	-	-	**2500**	2000	-1.06	-1.86				-
✓	NORTH FRYS CORNER PS	Cell 11	Serpentine River Upper	13848	**1000**	1000	-1.66	-2.36	519282	5443268	Yes	2002
✓	-	-	-	-	**1000**	1000	-0.66	-1.06				-
✓	GREY CREEK PS	Cell 3	Serpentine River Main	4526	**300**	300	-1.36	-1.66	515525	5438050	No	2003
✓	HALL'S PRAIRIE PS	Cell 6	Nicomekl River Main	10397	**1000**	900-1500	-1.40	-1.64	520153	5437290	No	1995
✓	-	-	-	-	**1000**	900-1500	-1.20	-1.64				-
✓	-	-	-	-	**900**	-	-2.00	-2.30				-
✓	HOOKEBROOKE PS	Cell 7	Serpentine River West	9045	**228-365**	303	-2.16	-2.56	515914	5439892	No	1992
✓	-	-	-	-	**766-877**	303	-1.36	-1.76				-
✓	LOGGING DITCH PS	Cell 5	Nicomekl River Main	5638	**643-864**	757	-1.42	-1.50	516774	5436616	Yes	1990/2006
✓	-	-	-	-	**643-864**	757	-1.32	-1.51				-
✓	-	-	-	-	**643-864**	757	-1.22	-1.52				-
✓	-	-	-	-	**300**	300	-1.80	-2.13				-
✓	-	-	-	-	**300**	300	-1.90	-2.14				-
✓	-	-	-	-	**300**	300	-2.00	-2.15				-
✓	NICOMEKL DITCH PS	Cell 4	Nicomekl River Main	7221	**650-810**	694	-0.84	-1.05	517775	5437568	No	1983
✓	-	-	-	-		694						-
✓	PANORAMA PS	Cell 3	Serpentine River Main	1206	**350-550**	450	-0.06	-0.96	513687	5437474	No	1995
✓	-	-	-	-	**350-550**	450	-0.66	-0.96				-
✓	-	-	-	-	**300**		-1.56	-1.86				-
✓	SOUTH CLOVERDALE PS	Cell 6	Nicomekl River Main	8017	**1100-1670**	1395	-1.21	-1.05	518383	5437374	Yes	1996
✓	-	-	-	-	**1100-1670**	1395	-0.97	0.90				-
✓	UPPER SERPENTINE PS	Cell 12	Serpentine River Upper	15982	**1000**	1000	-0.96	-1.36	519798	5445191	Yes	2001
✓	-	-	-	-	**1000**	1000	0.84	-0.16				-
✓	Central Surrey PS	Cell 10	Serpentine River Bose		**63-79**		-2.06	-2.53	516877	5441431		
	* dropped pump rate by 40%											

7.3.2 MODEL INFLOWS AND WATER LEVEL BOUNDARIES

Inflows and water level boundaries for the model are shown in Map 1. The hydrologic analyses in Section 6 provided inflow time series for the floodplain cells and for point sources. The ocean analyses of Section 5 provided ocean level time series (incorporating tide, surges and wind setup) for the downstream water levels on each of the rivers.

7.4 MODEL VALIDATION

Model calibration typically forms an important step of hydraulic model development. It involves gradually fine-tuning initially selected channel/floodplain Manning's roughness coefficients and other model parameters to make sure simulated water levels match observed levels for a particular flood event. Once the coefficients have been fine-tuned, the model is typically used for simulating a second independent flood event with known flows and observed water levels to validate that the model is also accurate for a different magnitude event. For the present HEC-RAS model, this procedure was modified somewhat.

Considering the complex nature of the hydraulic network, water levels, particularly in the floodplain, are largely a function of the spilling/pumping capacity of the drainage system rather than the channel and floodplain roughness coefficients, the parameters typically adjusted during calibration. On the other hand, it was not possible to represent all hydraulic structures in detail and fine-tune these to provide a perfect fit.

Although MIKE11 and HEC-RAS roughness values and loss coefficients are not exactly equivalent, the coefficients from the MIKE11 model, calibrated to a 2003 flood (UMA 2004), were transferred to the HEC-RAS model as a starting point. Manning's roughness values (n) for the channels varied from 0.060 for the upstream reaches (narrow channels flowing on moderately steep slopes) to 0.022 in Mud Bay. The model was then validated to two significant recent storms; in January 2009 and January 2005.

7.4.1 JANUARY 2009 FLOOD

The 2009 flood was simulated from January 4 to 17, with peak water levels occurring between January 6 and 8. The Mud Bay water level boundaries were specified as the reconstructed water level time series for January 2009. The inflow boundaries (both point and distributed) were specified as the HSPF modelled inflow hydrographs from 2009 (existing land uses). Spillway locations, dimensions and elevations included in the RAS model were set to be representative of available data for January 2009.

Observed water levels for the 2009 event were available for the Serpentine River at the sea dams (ocean and river sides) and at six pump stations. No observed water levels were available at the Hwy 10 or the 168th Street gauges. On the Nicomekl River, observed water levels were available at the sea dams (ocean and river sides) and at four pump stations. Observed water levels were also available at the 192nd Street gauge. On the tributaries and canals, observed water levels were available for the Bear Creek at Surrey Lake gauge (152nd Street) but the range in water levels and peak levels were suspiciously low. A total of 12 pump stations on the Serpentine and seven pump stations on the Nicomekl have observed water levels in floodplain ditches near the pump stations. Of those, six were identified as problematic gauges during the January 2009 event.

Longitudinal profiles and time series plots at specific locations are included in Appendix D. Differences between simulated water levels and recorded levels are summarized in Table 7.7 (river) and Table 7.8 (floodplain). Agreement for the Serpentine River levels upstream of Bose Island is generally within a 0.1 m tolerance (5 gauges). Agreement for the lower Serpentine River (downstream of Bose Island) is within 0.2 to 0.25 m (3 gauges) with modelled levels being consistently higher.

On the Nicomekl River modelled water levels are generally within 0.2 m of observed levels (5 gauges) but differences increase in the upstream direction and the modelled levels are as much as 0.5 m lower than peak observed levels. It is suspected that these differences are related to the hydrologic inputs to the Nicomekl River. Section 6.2 showed that the simulated flows (based on hydrologic modelling) were significantly less than reported flows for the water levels observed.

Table 7.7. Agreement between maximum observed and modelled river water levels (Jan 2009)

Gauge/Pump Station	River	Chainage (m)	Max Observed Water Level (m)	Max Modelled Water Level (m)	Difference (m)
Upper Serpentine PS	Serpentine	16017.2	1.78	1.75	-0.03
North Fry's Corner PS	Serpentine	13845.9	1.75	1.82	0.07
Fry's Corner PS	Serpentine	13571.7	1.76	1.83	0.07
Fleetwood PS	Serpentine	11817.8	1.76	1.85	0.09
East Newton PS	Serpentine	10822.4	1.86	1.87	0.01
64th Avenue	Serpentine	9824.7	1.68	1.88	0.20
Sea Dam Serpentine River	Serpentine	6.3	1.64	1.88	0.24
Sea Dam Serpentine Ocean	Serpentine	-12.8	1.67	1.89	0.22
Serpentine River at Hwy 10	Serpentine	7502.8	-	1.88	-
Sea Dam Nicomekl Ocean	Nicomekl	-10.7	1.65	1.79	0.14
Sea Dam Nicomekl River	Nicomekl	20	1.64	1.78	0.13
Erickson PS	Nicomekl	10394.5	2.07	1.89	-0.18
Nicomekl River at 192nd Street	Nicomekl	14293.6	2.95	2.44	-0.52
Logging Ditch PS	Nicomekl	5577.4	1.84	1.82	-0.02
South Cloverdale PS	Nicomekl	7950.9	2.05	1.85	-0.20
Burrows I PS	Nicomekl	8642.3	2.14	1.86	-0.29

Differences between modelled and observed water levels on the floodplain are considerably greater, some underestimated and some overestimated. At five pump stations, model agreement with observed water levels is within ± 0.1 m. This can partly be attributed to missing or simplified hydraulic features and assumed horizontal water levels across large storage cells. In some instances, the simulated water level for the storage cell was compared with the observed water level in a local ditch near a pump station, locations unrepresentative of conditions for the entire cell.

Table 7.8. January 2009 agreement between maximum observed and modelled floodplain water levels

Cell	Pump Station	River	Chainage (m)	Max Observed Water Level (m)	Max Modelled Water Level (m)	Difference (m)
1	Colebrook	Serpentine	-3050	-0.01	0.03	0.04
3	Gray	Serpentine	4500	0.22	-0.08	-0.30
3	Panorama	Serpentine	1200	0.19	-0.08	-0.27
4	150th Street	Serpentine	1800	0.14	0.09	-0.05
4	48th Avenue	Serpentine	5050	-0.02	0.09	0.11
7	Hookbrook	Serpentine	9000	-	-0.86	-
8	64th Avenue	Serpentine	9824.7	-0.23	-1.22	-0.99
9	East Newton	Serpentine	10822.4	-	1.07	-
9	Fleetwood	Serpentine	11817.8	0.36	1.07	0.71
11	Fry's Corner North	Serpentine	13845.9	0.22	0.23	0.01
12	Coast Meridian	Serpentine	12250	-	0.55	-
12	Fry's Corner	Serpentine	13571.7	1.05	0.55	-0.50
13	Upper Serpentine	Serpentine	16017.2	1.25	0.98	-0.27
4	40th Avenue	Nicomekl	3300	0.00	0.09	0.09
4	Nicomekl	Nicomekl	7300	-	0.09	-
5	Burrows	Nicomekl	8642.3	0.06	-0.4	-0.46
5	Erickson	Nicomekl	10394.5	0.09	-0.4	-0.49
5	Logging Ditch	Nicomekl	5577.4	-0.10	-0.4	-0.30
6	Halls Prairie	Nicomekl	10400	-	0.1	-
6	South Cloverdale	Nicomekl	7950.9	-	0.1	-

The above results were compared with UMA's model calibration to the 2003 flood (UMA 2004). By fine-tuning roughness coefficients, UMA achieved an agreement of ± 0.08 m at most gauges on the Serpentine with a somewhat poorer fit on the Nicomekl. For the floodplain cells, the simulated water levels were generally higher than the observed levels by 0.2 to 0.3 m although large discrepancies were also noted.

7.4.2 JANUARY 2005 FLOOD

The 2005 flood was simulated from January 13 to 27, with peak water levels occurring between January 17 and 19. Again, the downstream water level boundaries in Mud Bay were specified as the reconstructed water level time series from January 2005. The upstream boundaries and inflows (point and distributed) were specified as the HSPF modelled inflow hydrographs from 2005 (existing land uses). Due to a lack of precise information on the spillway locations for this year, dimensions and elevations included in the HEC-RAS model were set to be representative of available data for January 2009.

Observed water levels for the 2005 event were not available for the Serpentine River at the sea dams (ocean and river sides). No observed water levels were available at the Hwy 10 or the 168th Street gauges. Recorded Serpentine River water levels were however available at six pump station gauges. On the Nicomekl River, observed water levels were available at the sea dams (ocean and river sides) and at four pump stations. Observed Nicomekl River water levels were not available at the 192nd Street gauge. On the tributaries and canals, observed water levels were not available for the Bear Creek at Surrey Lake gauge (152nd Street). A total of 12 pump stations on the Serpentine and seven pump stations on the Nicomekl had observed water levels in floodplain ditches near the pump stations. Of those, four were identified as problematic gauges during the January 2005 event.

Longitudinal profiles and time series plots at specific locations are included in Appendix D. Differences between simulated water levels and recorded levels are summarized in Table 7.9 (river) and Table 7.10 (floodplain).

Water levels for the Serpentine River are consistently lower than observed levels by an average of 0.18 m. The largest difference, 0.31 m, was observed at the East Newton Pump Station. However, for the 2009 flood, agreement between simulated and observed peak water levels at this location was excellent. Overall, agreement was better for 2009 than for 2005.

On the Nicomekl River, modelled water levels were on average within ±0.2 m of observed levels (5 gauges). Differences increased in the upstream direction as the modelled levels are once again lower than observed.

Table 7.9. January 2005 agreement between maximum observed and modelled river water levels

Gauge/Pump Station	River	Chainage (m)	Max Observed Water Level (m)	Max Modelled Water Level (m)	Difference (m)
Upper Serpentine PS	Serpentine	16017.2	1.93	1.73	-0.20
North Fry's Corner PS	Serpentine	13845.9	1.76	1.73	-0.03
Fry's Corner PS	Serpentine	13571.7	1.85	1.73	-0.12
Fleetwood PS	Serpentine	11817.8	1.86	1.73	-0.13
East Newton PS	Serpentine	10822.4	2.03	1.72	-0.31
64th Avenue	Serpentine	9824.7	1.97	1.71	-0.26
Serpentine River at Hwy 10	Serpentine	7502.8	-	1.68	-
Sea Dam Serpentine River	Serpentine	6.3	-	1.62	-
Sea Dam Serpentine Ocean	Serpentine	-12.8	-	2.18	-
Sea Dam Nicomekl Ocean	Nicomekl	-10.7	1.73	2.09	0.37
Sea Dam Nicomekl River	Nicomekl	20	1.52	1.59	0.07
Logging Ditch PS	Nicomekl	5577.4	1.66	1.63	-0.03
South Cloverdale PS	Nicomekl	7950.9	-	1.66	-
Burrows I PS	Nicomekl	8642.3	1.99	1.67	-0.32
Erickson PS	Nicomekl	10394.5	1.90	1.70	-0.20
Nicomekl River at 192nd Street	Nicomekl	14293.6	-	2.42	-

Differences between modelled and observed water levels on the floodplain were generally greater than in the rivers. The agreement at five pump stations was good (within ± 0.2 m), ranged from ± 0.2 m to ± 0.3 m at another five pump stations and was much poorer at the remaining six pump stations, particularly at Fry's Corner, East Newton and Erickson. These locations were also poorly represented in 2009. This implies that the model is neglecting some component of the hydraulic system or that the comparison between the simulated and observed levels is invalid because the observed water levels apply to specific localized areas rather than the average of the overall storage cell.

The floodplain water level gauges that showed good agreement with modelled results for the 2009 verification also generally showed good agreement for 2005. These included gauges found within cells 1, 4, 6, and 11. It would seem that the gauges in these cells are located such that they better reflect entire cell levels.

Table 7.10. Agreement between maximum observed and modelled floodplain water levels (Jan 2005)

Cell	Pump Station	River	Chainage (m)	Max Observed Water Level (m)	Max Modelled Water Level (m)	Difference (m)
4	150th Street	Serpentine	1800	-0.06	0.05	0.11
4	48th Avenue	Serpentine	5050	-0.21	0.05	0.26
8	64th Avenue	Serpentine	9824.7	-1.57	-1.34	0.23
12	Fry's Corner	Serpentine	13571.7	0.94	0.2	-0.74
12	Coast Meridian	Serpentine	12250	-	0.2	-
13	Upper Serpentine	Serpentine	16017.2	-	0.82	-
1	Colebrook	Serpentine	-3050	-0.10	0.03	0.13
3	Panorama	Serpentine	1200	0.09	-0.09	-0.18
3	Gray	Serpentine	4500	0.27	-0.09	-0.36
7	Hookbrook	Serpentine	9000	-	-0.78	-
9	East Newton	Serpentine	10822.4	0.00	0.54	0.54
9	Fleetwood	Serpentine	11817.8	0.17	0.54	0.37
11	Fry's Corner North	Serpentine	13845.9	0.02	0.04	0.02
5	Logging Ditch	Nicomekl	5577.4	-0.13	-0.41	-0.28
5	Burrows	Nicomekl	8642.3	0.01	-0.41	-0.42
5	Erickson	Nicomekl	10394.5	0.48	-0.41	-0.89
4	40th Avenue	Nicomekl	3300	0.02	0.05	0.03
4	Nicomekl	Nicomekl	7300	-	0.05	-
6	South Cloverdale	Nicomekl	7950.9	0.18	-0.06	-0.24
6	Halls Prairie	Nicomekl	10400	-0.33	-0.06	0.27

7.5 Sensitivity Analysis

A sensitivity analysis was carried out to assess the model response to variations in roughness and upstream inflows.

7.5.1 Model Roughness Coefficients

The model was re-run with the roughness coefficients increased and decreased by 20% to assess the effect on the maximum 2009 flood profile. Overall, the model was insensitive to channel roughness in the lower – tidally influenced – reaches of the Serpentine and Nicomekl Rivers. A 20% change in roughness results in less than 0.1 m of change along the water level profile for the river reaches starting at the ocean up to river chainage 13+000 on the Nicomekl and up to river chainage 18+000 on the Serpentine River. However, the upper reach of the Nicomekl River appears to be more sensitive to changes in roughness values, resulting in variations from ± 0.1 m to ±0.26 m. For the upper Serpentine, the reach from chainage 18+445 to 19+158 is quite sensitive to decreases in roughness (variations from -0.1 m to -0.4 m). Values for the entire river reaches are summarized in Table 7.11. Longitudinal plots of modelled water levels are included in Appendix D.

Table 7.11. Model sensitivity to changes in bed roughness

	Nicomekl River		Serpentine River	
	(+ 20%)	(-20%)	(+ 20%)	(-20%)
Average =	0.05	-0.05	0.01	-0.02
Median =	0.02	-0.02	0.00	0.00
Max =	0.23	0.00	0.16	0.02
Min =	-0.02	-0.26	-0.02	-0.44
St. Dev. =	0.07	0.07	0.03	0.05

7.5.2 Inflow Values

The model was re-run with both a 10% increase and a 10% decrease applied to input inflow hydrographs in order to assess the model's sensitivity to the upstream boundaries (inputs from HSPF model results).

Overall, the lower – tidally influenced – reaches of the Serpentine and Nicomekl Rivers are insensitive to changes to inflow hydrographs. A 10% change in inflows results in less than 0.10 m of change along the water level profile for the river reaches starting in the ocean, up to river chainage 13+000 on the Nicomekl, and going all the way up on the Serpentine River (chainage 19+600). However, the upper reach of the Nicomekl River appears to be more sensitive to changes in inflows values (corresponding variations ranging from ± 0.1 m to ±0.26 m). Most of the larger floodplain cells are insensitive to a 10% change in inflow hydrographs while some of the smaller cells (7, 8, and 9) appear to be more sensitive (up to 0.26 m difference). Values for the entire river reaches are summarized in Table 7.12 and for the floodplain cells in Table 7.13. Longitudinal plots of modelled water levels are included in Appendix D.

Table 7.12. Model sensitivity to changes in inflow hydrographs

	Nicomekl River		Serpentine River	
	(+10%)	(-10%)	(+10%)	(-10%)
Average =	0.03	-0.05	0.01	-0.02
Median =	0.01	-0.01	0.00	0.00
Max =	0.12	0.00	0.09	0.00
Min =	0.00	-0.26	0.00	-0.08
St. Dev. =	0.03	0.07	0.02	0.02

Table 7.13. Floodplain cell sensitivity to changes in inflow hydrographs

	Max WSE (m)			Difference	
Cell	Jan 2009 inflows	(+10%)	(-10%)	(+10%)	(-10%)
1	0.03	0.06	0	0.03	-0.03
3	-0.08	-0.02	-0.11	0.06	-0.03
4	0.09	0.12	0.06	0.03	-0.03
5	-0.4	-0.35	-0.45	0.05	-0.05
6	0.1	0.21	-0.01	0.11	-0.11
7	-0.86	-0.67	-1.04	0.19	-0.18
8	-1.22	-0.99	-1.48	0.23	-0.26
9	0.82	0.96	0.67	0.14	-0.15
10	0.61	0.7	0.52	0.09	-0.09
11	0.23	0.33	0.13	0.1	-0.1
12	0.59	0.76	0.45	0.17	-0.14
13	0.99	1.07	0.89	0.08	-0.1

7.6 HEC-RAS Model Simulations

Following verification and sensitivity analyses, the model was used to generate two approximately 50 year long time series of simulated water levels at 40 key locations in the floodplain. The first time series was representative of the 1964 to 2011 runoff and historic water levels in Mud Bay. The second time series was representative of year 2100 projected runoff (incorporating future land-use changes) and projected water levels in Mud Bay under climate change. The time series were subsequently compared to assess the impacts of projected future sea levels on flood levels.

7.6.1 Simulation of Present (2010) Conditions

The model was first run to generate a long time series representative of present conditions (without sea level rise). The geometry file contained spillway information reflecting the buildout conditions.

The downstream water level boundaries in Mud Bay were specified as the reconstructed water level time series for the historic period from 1964 to 2011.

The upstream boundaries and inflows (point and distributed) were specified as the HSPF modelled inflow hydrographs from 1964 to 2011, reflective of existing land uses.

The resulting hourly water level time series were written to a HEC-RAS DSS file and later analysed to determine the 200-year water levels at 40 key locations for present conditions (Section 8).

7.6.2 Simulation of Projected Future (2100) Conditions with Sea Level Rise and Subsidence

The model was re-run to simulate a long time series representative of future conditions (with sea level rise and subsidence). No changes were made to the geometry file (buildout conditions). The downstream water level boundaries in Mud Bay were adjusted to include both sea level rise and subsidence of the floodplain. The entire downstream boundary water level record was raised by a constant value of 1.195 m of which 0.97 m is due to projected sea level rise (refer to Section 3.1) and 0.225 m is due to projected subsidence (assuming a subsidence rate of 2.5 mm/year for 90 years).

The HSPF computed inflow hydrographs used for the upstream boundaries and inflows to floodplain storage cells were reflective of projected future land uses (year 2100) and future runoff.

The resulting hourly water level time series were written to a DSS file and later analysed to determine the 200-year water levels at 40 key locations for future conditions.

7.7 Model Limitations

It is important to note that any inaccuracies in the ocean level boundary conditions or deviations between actual and modelled runoff will affect the hydraulic model results and the effects of any discrepancies could even be amplified in the hydraulic model.

In a few locations, the model appears to neglect some components of the hydraulic system or comparisons between the simulated and observed levels are invalid because the observed water levels apply to specific localized areas rather than the average of the overall storage cell. The hydraulic model accuracy can likely be further improved by fine-tuning the structures incorporated in the model, optimizing the locations where observed and simulated levels are compared and by reducing the size of the storage cells.

The observed water levels in several of the lower floodplain cells were tidally affected. Some of these tidally-caused water level fluctuations were better simulated in the HEC-RAS model after additional floodboxes were included in the model. Perfect simulations were not expected and the main goal of the HEC-RAS model was to represent general flow trends during a flood event. The limitations of the hydraulic model need to be recognized and caution must be applied when interpreting the model results.

It is recommended that further improvements be made as part of future phases of work. However, in terms of evaluating the relative impacts of climate change on flood levels in the Serpentine and Nicomekl basins, the HEC-RAS model is a useful tool.

8 IMPACT OF RELATIVE SEA LEVEL RISE ON FLOOD LEVELS

8.1 FREQUENCY ANALYSES

Annual maximum flood levels for the two time series generated in Section 7.6 were analyzed in frequency analyses to estimate the 200-year return period flood levels at the key locations shown in Figure 8.1. The 2010 and 2100 200-year levels were compared to assess the impact of the projected relative sea level rise and land use change on flood levels.

NHC's in-house frequency analysis package 'DASH' was applied since this software conveniently uses the HEC-RAS DSS output file as input.

8.1.1 FREQUENCY ANALYSIS INPUT

Annual peak water levels were determined based on water years (1 September to 31 August). The highest ranked annual peak water level varied based on the location in the floodplain. Table 8.1 shows the top three or four events that caused the highest ranked annual peak water levels for the various reaches or locations in the floodplain (existing conditions). The combination events refer to events when moderately high ocean levels coincided with moderately high precipitation events.

Table 8.1. Areal distribution of peak water level events by reach

	High Ocean Level Events				Combination Events				High Precipitation and Runoff Events							
Reach or Location	Dec-82	Jan-87	Dec-94	Feb-06	Nov-83	Jan-86	Jan-92	Dec-00	Jan-68	Dec-72	Dec-79	Jan-97	May-97	Oct-03	Dec-07	Jan-09
downstream of seadams	✓	✓		✓												
Nicomekl (0+000 to 10+500)			✓		✓					✓	✓					
Nicomekl (10+500 to 12+000)					✓					✓	✓					✓
Nicomekl (12+000 to 17+280)										✓	✓	✓		✓		
Serpentine (0+000 to 7+500)						✓	✓		✓						✓	
Serpentine (9+000 to 12+000)						✓								✓	✓	✓
Serpentine (16+000 to 17+500)											✓	✓		✓	✓	
Serpentine (18+000 to 19+500)								✓		✓		✓		✓		
Bear Creek (2+800 to 3+100)								✓		✓			✓	✓		
Latimer Creek (1+258)											✓	✓		✓	✓	
Cloverdale Canal (3+882)								✓		✓		✓		✓		

Similarly, Table 8.2 shows the top events for the floodplain storage cells. Flood damage reports were summarized in Table 2.1 as available.

As expected, the data show that the highest water levels in the lower reaches of the rivers are associated with high ocean level events or combination events while the highest water levels in the upper reaches and adjacent floodplain cells are associated with high precipitation events. Choosing a particular event as the 'flood-of-record' is not possible since the various regions respond differently to flooding and the cause of flooding varies. Section 8.3 provides further discussion of high precipitation/tide events and design conditions currently in use.

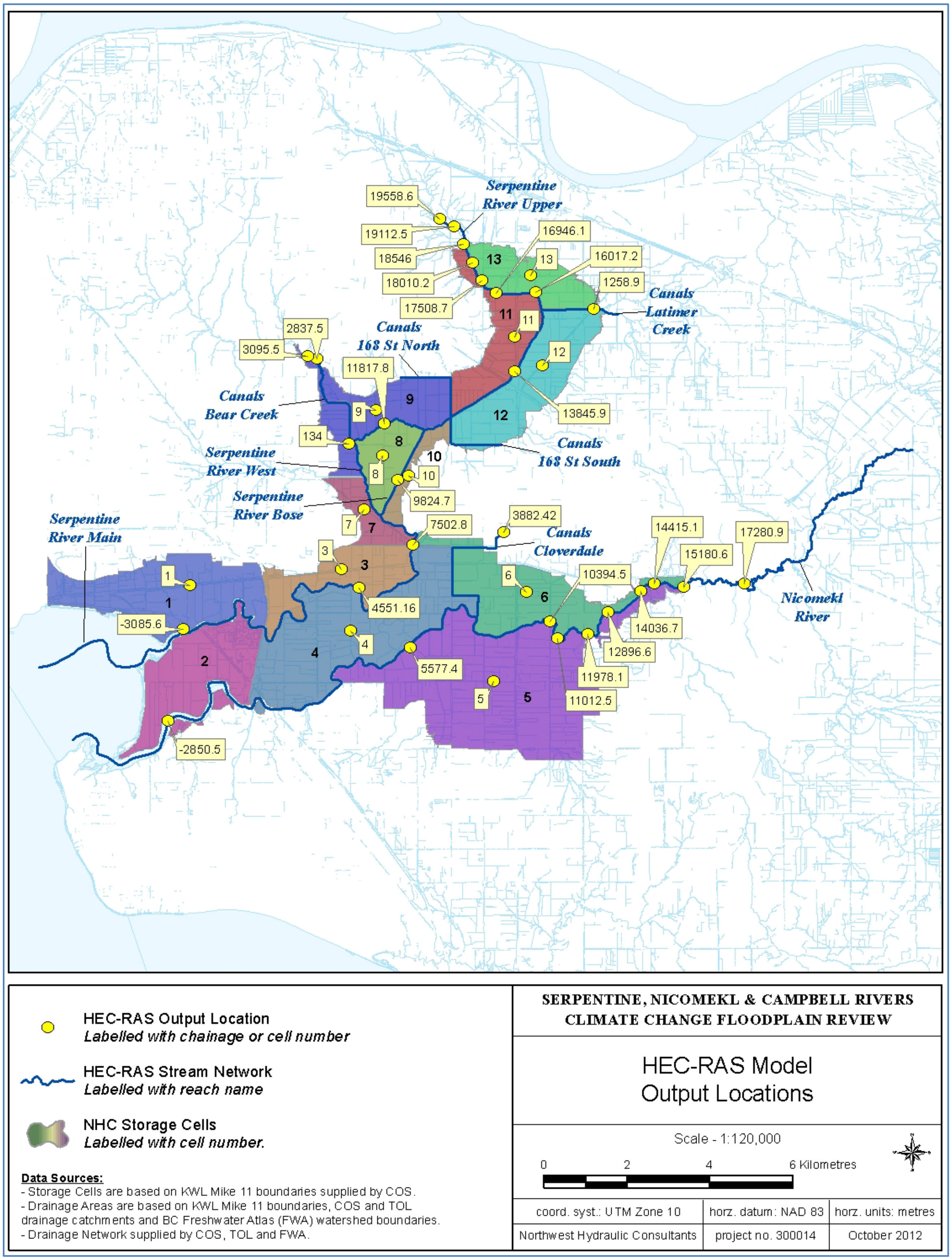


Figure 8.1. HEC-RAS model output locations

Table 8.2. Areal distribution of peak water level events by cell

	High Precipitation and Runoff Events						
Cell	**Jan-68**	**Dec-72**	**Jan-97**	**Oct-03**	**Jan-05**	**Mar-07**	**Jan-09**
1	✓	✓			✓		✓
3		✓		✓	✓		✓
4	✓	✓			✓		✓
5			✓		✓	✓	✓
6	✓			✓	✓		✓
7		✓	✓		✓	✓	
8		✓			✓	✓	✓
9	✓			✓	✓		✓
10		✓		✓	✓		✓
11	✓			✓	✓		✓
12	✓			✓	✓		✓
13	✓			✓	✓		✓

8.1.2 FREQUENCY ANALYSIS RESULTS

At each of the 40 selected locations, a 5-point moving average fit was applied to the annual peak water levels to extrapolate the data to an annual exceedance probability (AEP) of 0.5% (return period of 200-years). The frequency distribution plots are included in Appendix E. Table 8.3 summarises the 200-year water levels for both existing (year 2010) and future (year 2100) conditions. Figure 8.2 shows the 200-year water level increases spatially and compares the results with KPA (1993) design levels. Longitudinal 200-year profiles are plotted in Appendix D.

Table 8.3. Existing (2010) and future (2100) 200-year flood levels

		200-year Flood Levels (m) Surrey Datum		
River and Reach	**Chainage**	**Existing**	**Future**	**Increase**
Canals Bear Creek	134	2.3	3.0	0.7
Canals Bear Creek	2837.5	4.1	4.4	0.3
Canals Bear Creek	3095.5	4.5	4.6	0.2
Canals Cloverdale Canal	3882.42	2.3	2.4	0.1
Canals Latimer Creek	1258.9	2.6	3.2	0.7
Nicomekl River	-2850.5	2.6	3.7	1.1
Nicomekl River	5577.4	2.2	3.2	1.0
Nicomekl River	10394.5	2.2	3.1	0.9
Nicomekl River	11012.5	2.2	3.1	0.9
Nicomekl River	11978.1	2.2	3.1	0.9
Nicomekl River	12896.6	**2.4**	3.1	0.7

		200-year Flood Levels (m) Surrey Datum		
River and Reach	**Chainage**	**Existing**	**Future**	**Increase**
Nicomekl River	14036.7	3.1	3.3	0.2
Nicomekl River	14415.1	3.3	3.5	0.2
Nicomekl River	15180.6	**3.4**	3.5	0.2
Nicomekl River	17280.9	**4.4**	4.5	0.1
Serpentine River Main	-3085.6	2.7	3.9	1.2
Serpentine River Main	4551.16	2.3	3.0	0.7
Serpentine River Main	7502.8	2.3	3.0	0.7
Serpentine River Bose	9824.7	2.3	3.0	0.7
Serpentine River West	11817.8	2.3	3.0	0.7
Serpentine River Upper	13845.9	2.3	3.0	0.7
Serpentine River Upper	16017.2	**2.4**	3.0	0.6
Serpentine River Upper	16946.1	2.5	3.1	0.6
Serpentine River Upper	17508.7	2.6	3.2	0.6
Serpentine River Upper	18010.2	2.8	3.3	0.5
Serpentine River Upper	18546	4.3	4.4	0.1
Serpentine River Upper	19112.5	6.1	6.2	0.1
Serpentine River Upper	19558.6	7.0	7.1	0.1
Storage Area	Cell 1	0.0	0.1	0.1
Storage Area	Cell 2	2.6	3.7	1.1
Storage Area	Cell 3	-0.1	0.0	0.1
Storage Area	Cell 4	0.1	0.2	0.1
Storage Area	Cell 5	-0.4	-0.2	0.2
Storage Area	Cell 6	0.3	0.6	0.3
Storage Area	Cell 7	-0.7	-0.5	0.2
Storage Area	Cell 8	-1.2	-1.2	0.0
Storage Area	Cell 9	0.9	1.3	0.4
Storage Area	Cell 10	0.7	1.0	0.3
Storage Area	Cell 11	0.3	0.6	0.3
Storage Area	Cell 12	-0.2	0.2	0.4
Storage Area	Cell 13	1.0	1.1	0.1

Note: Values in bold indicate that the mathematically fitted value was adjusted by eye.

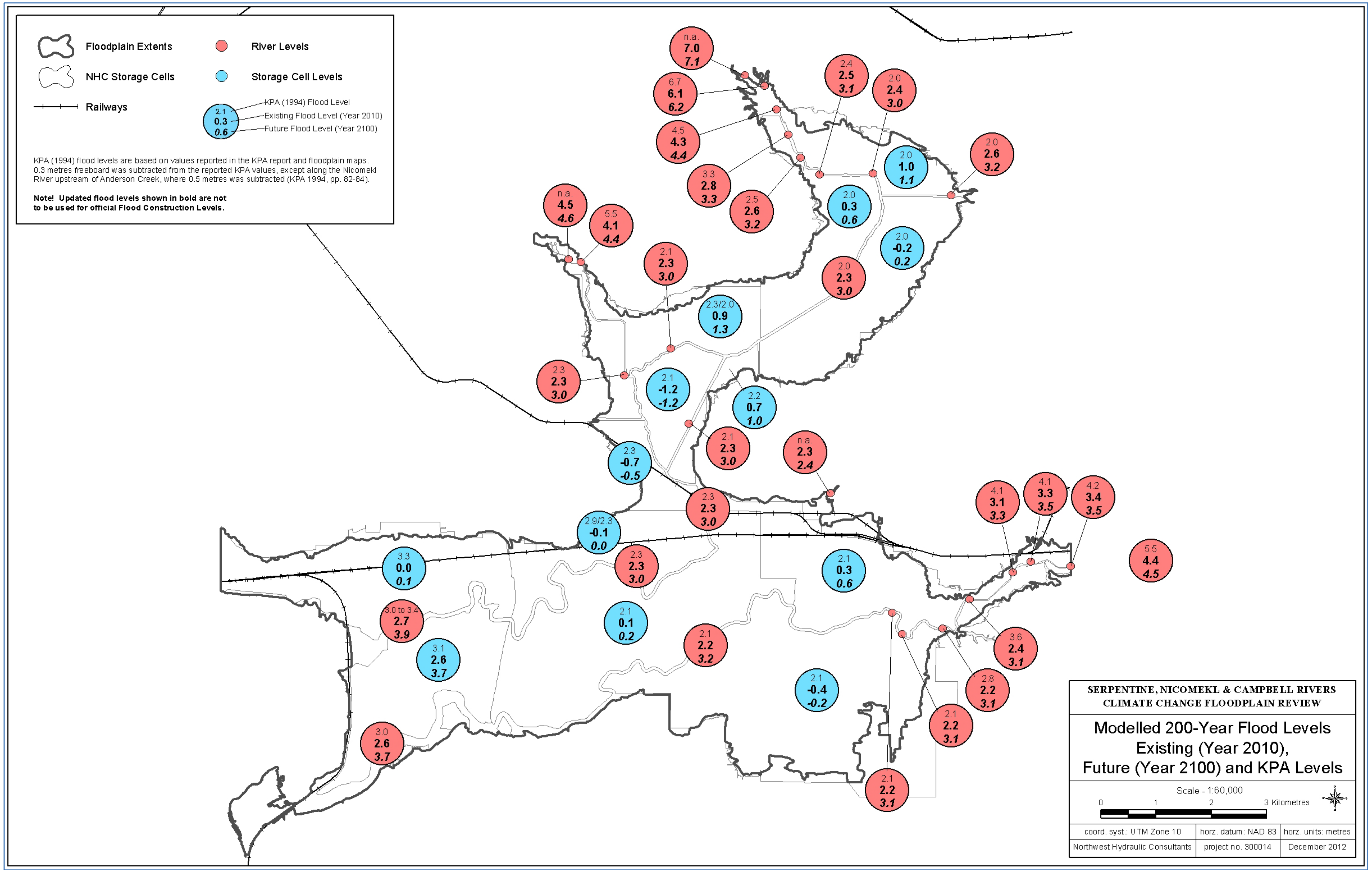


Figure 8.2. Modelled 200-year flood levels – existing (2010), future (2100) and KPA levels

Typically, frequency analysis results are sensitive to the curve fitting method applied. For assessing the relative change from existing to future conditions, the 5-point moving average method was considered representative. Other distributions, such as for example the GEV method which was used for the coastal analysis, gave somewhat different, generally more conservative results. Also, in contrast to the ocean level analysis, based on partial duration series above a threshold value, the frequency analysis of internal flood levels used a data set of annual maxima. The choice of data series was found to have little effect on the results.

For modelling future flood levels, all dikes were assumed to be raised to prevent flow from spilling from the river channels to the floodplain storage areas. However, spillway elevations were not raised and were set according to 'build-out' configurations. The estimated increases to the 200-year flood levels were relatively small for most of the floodplain, ranging from 0.1 m to 0.4 m, with the exception of Cell 2 which has no dikes and saw an increase of 1.1 m. Consequently, the areal extent of the floodplain is expected to increase only by a relatively small amount. However, the spillways and pump stations will be in operation much more frequently in the future.

Within the river channels, flood levels will be significantly higher. Just upstream of the sea dam the Nicomekl 200-year flood level is expected to increase by 1 m compared to the present level. This raised flood level remains nearly horizontal for a distance of about 12 km. As the channel gradient begins to steepen, the increase in the flood level diminishes and over an additional distance of about 5 km the present and future 200-year flood level are within 0.1 m (assuming no changes in precipitation).

The increase in Serpentine flood levels is slightly less. Upstream of the sea dam the 200-year flood level is expected to increase by 0.7 m, the flood profile remaining nearly horizontal for 17 km. Similar to the Nicomekl, the existing and future profiles nearly merge over a few kilometres in the upper basin due to the steep gradient.

Considering the discrepancies in the 2009 and 2005 validation results for some storage cells, particularly Cells 5, 7, 8, 9 and 12, the estimated 200-year present and future flood levels should not be adopted as accurate design levels without additional review. To improve the accuracy of the hydraulic model, refinements to hydraulic structures, reductions to storage cell sizes, improvements to boundary conditions, and more detailed calibration data, particularly for storage cells, would be needed before the results can be used for setting Flood Construction Levels.

8.2 Interpretation of Results

8.2.1 Sea Dam and Dike/Spillway Operation

On both rivers, the sea dam operation is key for reducing flood levels and only during low tides can water drain from the system. Relying on gravity flow, water levels in the Serpentine and Nicomekl Rivers cannot be lower than the level of the low tide. With rising sea levels, the duration when the river outflow is possible, is reduced. Not only are tidal peaks increased, tidal troughs are also raised (for year 2100, roughly 1 m higher than historic low tides). The sea dam elevations surveyed by the City suggested settlement of about 0.2 m compared to previous surveys. The elevations were not confirmed and for the model, top-of-dam elevations of 3.0 m for the Nicomekl and 3.5 m for the

Serpentine were assumed. For the estimated 2100 sea-levels, the dams would frequently overtop. However, the HEC-RAS model prevented overflow, meaning the sea dams were artificially raised. For the 200-year flood, the maximum head drop across both dams under existing conditions is 0.4 m, but is anticipated to increase by year 2100 to 0.54 m for the Nicomekl and 0.90 m for the Serpentine.

For modelling, all river dikes were assumed to be raised to prevent overtopping, both for existing and future conditions. An assessment of current dike elevations was not carried out. Increases in river levels over the next 90 years were listed in Table 8.3 and ranged from 1.1 m downstream of the sea dams to 0.1 m in the upper river reaches. Based on observations during the site inspection, the dike elevations will need to be increased significantly to contain future flood levels.

The purpose of the spillways is to prevent the dikes from overtopping and to provide equitable flooding over the agricultural floodplain. In the model, spillway elevations were the same for both the 2010 and 2100 runs. For example at Storage Cell 12, the peak inflow to the cell was about three times larger for 2100 than 2010 for an event similar to the 2003 flood. Similarly, the peak water level across the spillway was 0.3 m higher. Due to the relatively short duration of the spilling, the peak flood level in the cell was no more than 0.4 m higher for the 2100 run than for 2010. Optimizing spillway configurations for future conditions will likely result in considerable savings to the City compared to raising dikes.

8.2.2 IMPACT OF FLOODING ON INFRASTRUCTURE

Figure 8.3 and Figure 8.4 show the Serpentine and Nicomekl longitudinal profiles for the estimated 200-year water levels (2010 in blue, 2100 in red). The low and high chords of bridges included in the model are plotted to show which bridges will be submerged or partly submerged during the year 2100 200-year condition. Additional details on the bridges included in the model were provided in Table 7.5. On the Nicomekl, of the seven bridges modelled, three will be completely submerged and one partly submerged. On the Serpentine, of the six bridges modelled all six will be partly submerged. Particularly at the submerged bridges and potentially also at the partially submerged bridges, transportation corridors will be affected.

According to current operations, as water starts spilling into storage cells, pump stations are turned off. For this reason, during future conditions pumps will be shut off more often and for longer durations. On the other hand, for drainage, pumps will run more frequently and longer. In some locations the pump stations may be directly affected by the increased flood levels.

The hydraulic gradient across floodboxes will be lesser and outflows reduced. Detailed assessment of floodbox performance was not carried out.

Assessments of sanitary lift stations, road drainage, electrical control boxes and other infrastructure was not part of the scope.

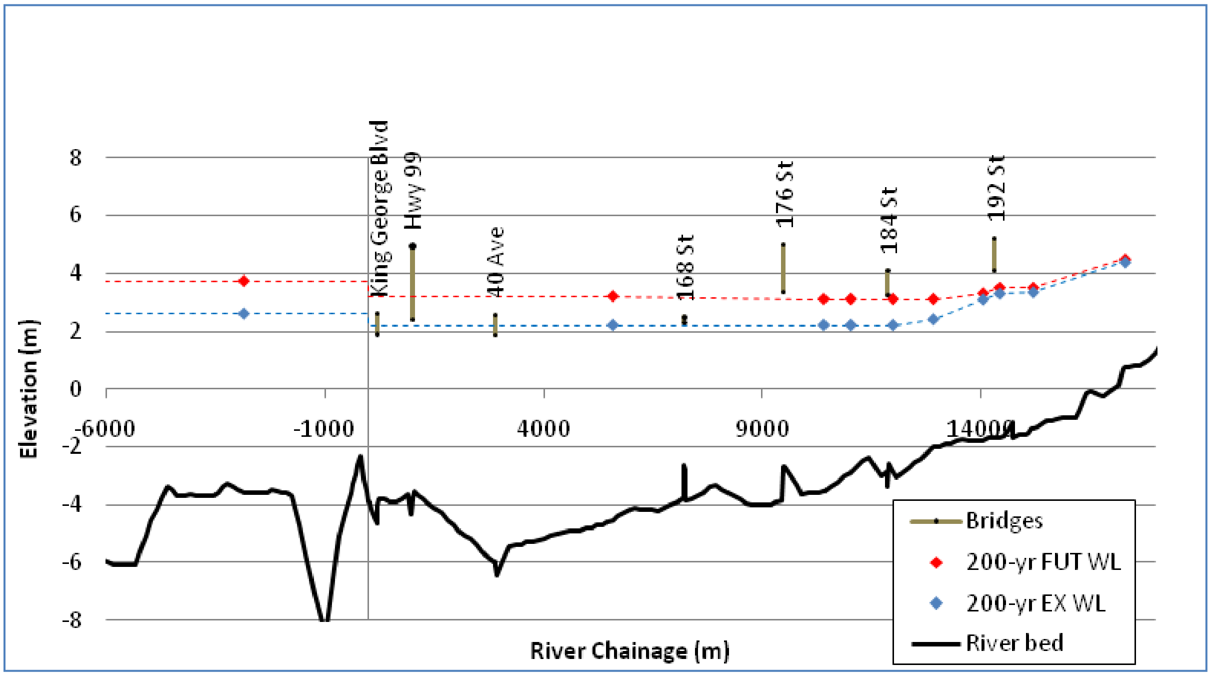


Figure 8.3. Nicomekl River longitudinal profile - 200-year water levels with bridges

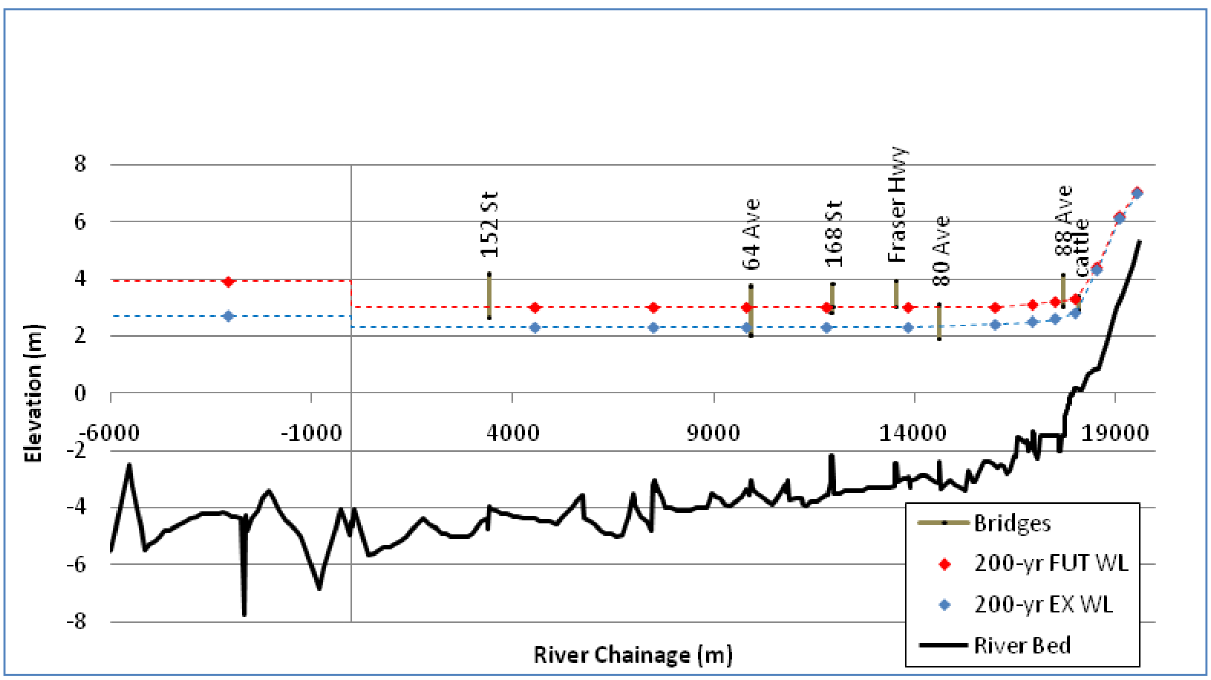


Figure 8.4. Serpentine River longitudinal profile - 200-year water levels with bridges

8.2.3 Flood Return Period Variations in Space and Time

To better understand the variability in return periods at different locations in the Nicomekl and Serpentine basins, the 2003, 2005 and 2009 flood events were isolated from the modelled historic time series (existing conditions) and estimated return periods reviewed. Results reflect modelled 'build-out' configurations rather than actual infrastructure configurations in place at the time of the floods.

At each of the 40 hydraulic output locations, the return period associated with the peak water levels were calculated and are summarised by river reach in Table 8.4 and by storage cell in Table 8.5. Based on the available time-series length, the maximum event had a return period of 72 years.

Table 8.4. Summary of return periods in the river channels

	Oct-03	**Jan-05**	**Jan-09**
Reach or Location	**Return Period (yrs)**		
downstream of sea dams	< 1	1	< 1
Nicomekl (0+000 to 10+500)	4 to 6	2 to 3	7 to 10
Nicomekl (10+500 to 12+000)	7 to 13	3	9 to 13
Nicomekl (12+000 to 17+280)	72	7 to 8	6 to 8
Serpentine (0+000 to 7+500)	2 to 5	3	10 to 12
Serpentine (9+000 to 12+000)	18 to 25	4	11
Serpentine (16+000 to 17+500)	41 to 72	6 to 7	9 to 10
Serpentine (18+000 to 19+500)	24 to 72	4 to 5	4
Bear Creek (2+800 to 3+100)	26 to 28	< 1	4
Latimer Creek (1+258)	72	8	8
Cloverdale Canal (3+882)	28	5	4

Table 8.5. Summary of return periods in the storage cells

	Oct-03	Jan-05	Jan-09
Cell	**Return Period (yrs)**		
1	7	28	18
3	18	28	72
4	8	16	72
5	7	15	72
6	72	13	28
7	< 1	14	11
8	< 1	13	18
9	72	14	37
10	20	13	72
11	72	18	41
12	72	13	32
13	72	12	38

Figure 8.5 shows the results plotted on a map of the floodplain. (The stacked bars at each of the 40 locations are scaled based on the return period in years associated with the peak water levels computed for the flood events.) Results show that the modelled October 2003 flood had the highest return period (20 years to 72 years) in the upper reaches of the Serpentine and Nicomekl Rivers and in some of the upper Serpentine floodplain cells. In comparison, the less severe January 2005 flood had overall lower return period water levels. The January 2009 flood caused higher return period water levels (72 years) in the lower floodplain cells (3, 4, 5 and 10) and moderate water levels in the rivers (5 to 15 years) upstream of the sea dams.

In response to sea-level rise over time, the return period for particular flood level will change. In the lower river reaches, Nicomekl (0+000 to 13+000) and Serpentine (0+000 to 17+500), water levels with a current 72 year return period will on average occur annually by the year 2100. Extrapolating, the existing 200-year flood level will have a return period of less than 2 years. As present extreme events become more frequent, the lower end of the frequency distributions will shift and the predicted reductions in return periods should therefore be considered approximate.

In the upper river reaches, the reductions in return periods are less significant. At Nicomekl 17+300, the present 72 year flood will have a 60 year return period in year 2100 and, a present 200-year flood an approximate return period of 110 years. At Serpentine 19+600, a 72 year flood will have a 21 year return period and, a present 200-year flood an approximate return period of 30 years.

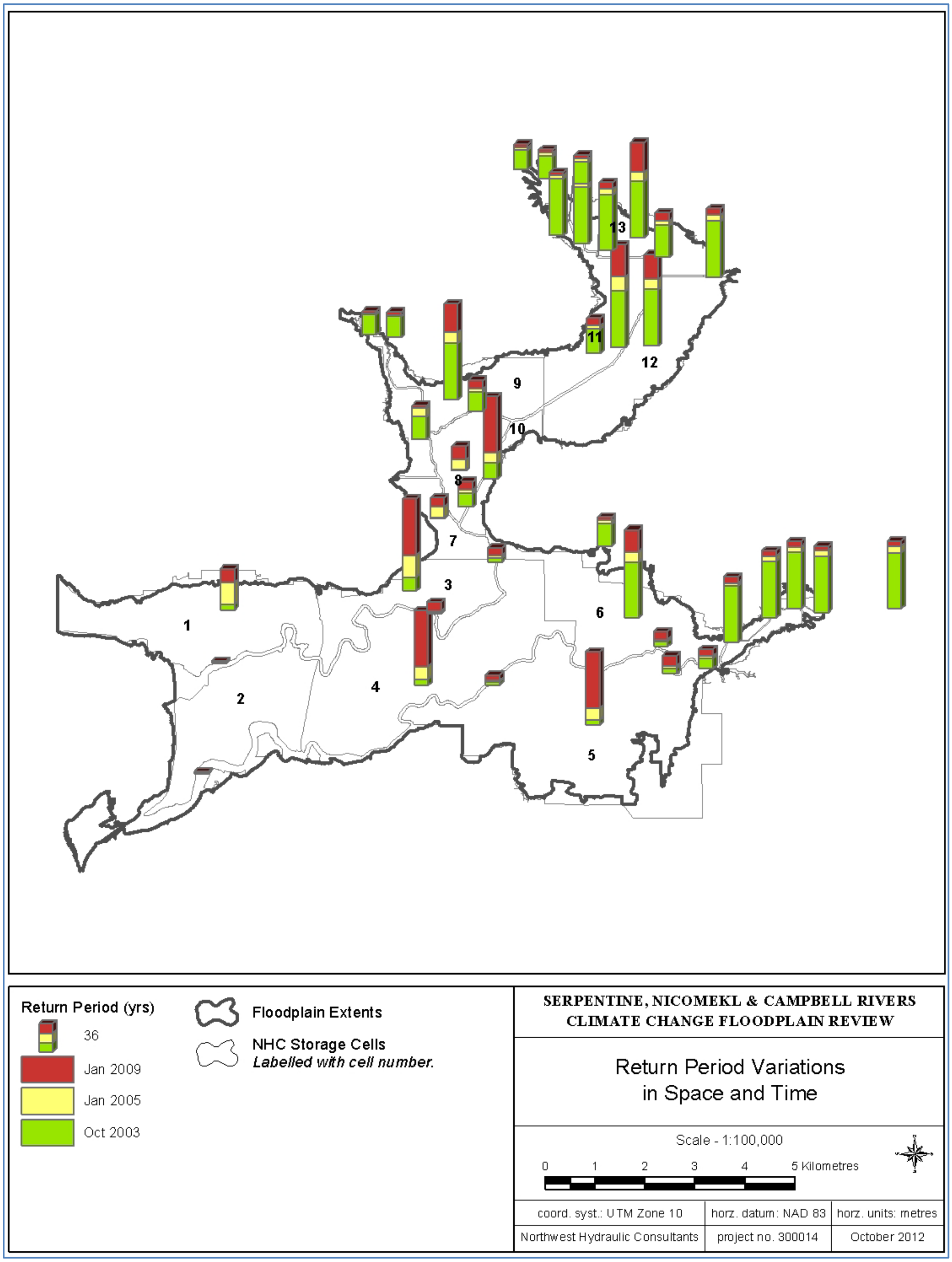


Figure 8.5. Return period variations in space and time

8.3 DISCUSSION OF NHC FINDINGS AGAINST CURRENT DESIGN EVENTS

8.3.1 COMPARISON OF OCEAN LEVELS AND RAINFALL

A comparison was undertaken of ocean water level boundary conditions and rainfall inputs from the present study against those currently defining the 200-year design event in the Serpentine and Nicomekl floodplains based on previous work. Three consistently high ranking events in terms of maximum water level in the lower part of the system, January 2009, October 2003 and January 2005 were included in the comparison.

Time series plots of the 200-year design event based on previous work by others (hourly rainfall and ocean water level boundary condition) are provided in Figure 8.6. Comparable plots of observed hourly rainfall data at Surrey Municipal Hall[5] and synthesized ocean water levels for the January 2009, October 2003 and January 2005 events are provided in Figure 8.7 through Figure 8.9.

The 47 years of hourly rainfall data for Surrey Municipal Hall developed in the current study was analysed to determine maximum annual rainfall amounts for durations of 24-hours, 3-days, 5-days and 7-days. Frequency curves for the annual maximum data are provided in Figure 8.10. As shown in Figure 8.10, the October 2003, January 2009 and January 2005 events are the top three events for durations of 3-, 5-, and 7-days, with the October 2003 event having appreciably higher rainfall amounts than the second ranked event in January 2009. The October 2003 event, however, occurred with much drier ground conditions than in January 2009, and the hydrologic simulation results show the two events producing similar runoff volumes at the watershed scale (see Table 6.15 and Table 6.16). It should also be noted (see Section 6.2) that the October 2003 rainfall event had an atypical spatial distribution which contributed to oversimulation of runoff for that event in the HSPF model.

If the purpose of the design event is to determine interior flood levels upstream from the sea dams, then the critical parameter related to ocean level is the duration of time that the sea dam flap gates are closed. It appears that much previous work focused on peak ocean levels rather than the duration of time the ocean level exceeds some critical threshold. While determination of an appropriate extreme ocean level is of course critical for the analysis and design of sea dikes and other coastal structures, the extreme ocean level does not have a direct bearing on interior flood level. From Figure 8.6 through Figure 8.9, it can be seen that the critical events from the present work have consistently lower peak ocean levels than in the existing 200-year design event, but longer durations of moderately high water levels which affects the duration of time the sea dams are closed.

In comparing rainfall hyetographs, the current 5-day design storm (Figure 8.6) has:

- No rainfall for the first 14 hours, essentially shortening the event to less than 4 ½ days.
- A peak rainfall amount of 20 mm in one hour.
- A total rainfall depth for the event of 297 mm.

[5] The “rainfall” data for January 2009 are actually estimated values of rainfall plus snowmelt determined from the observed precipitation and air temperature records.

Assuming the rainfall depth is for Surrey Municipal Hall, the 297 mm figure is in good agreement with the 200-year 5-day value for Surrey Municipal Hall from Figure 8.10, estimated at about 305 mm. On the other hand, the peak rainfall value appears to be artificially high and possibly intended to generate not only large runoff volumes but also high instantaneous peak runoff rates. While this may allow the existing design event to be applied to multiple locations in the basin, it should be recognized that the resulting hyetograph is probably unrealistic.

The flood level frequency curves in Appendix E suggest that only relatively modest extrapolation from the January 2009 event would be needed to produce 200-year water levels in Cells 3, 4 and 5 where event runoff volume is the dominant concern. For this area of the watershed, a design event could be readily developed by scaling up the January 2009 rainfall amounts and using the historic January 2009 ocean level sequence. Additional analysis would be needed to determine the scaling factor for the 2009 rainfall. Higher up in the watershed, where event volume is not as critical, the October 2003 event, which had higher rainfall intensities, may be a more appropriate candidate to provide the basis for a design event. However, as noted previously, this event had an atypical spatial distribution and further evaluation of candidate events is recommended.

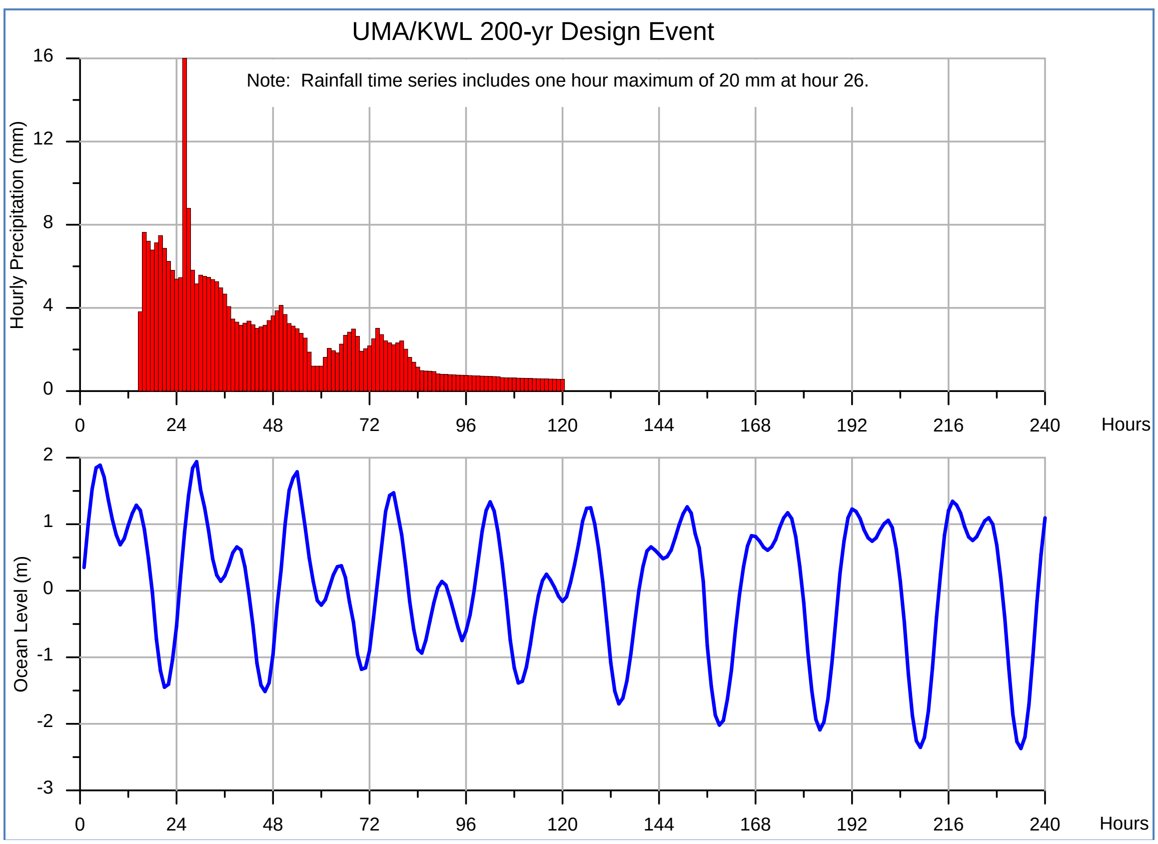


Figure 8.6. Existing 200-year 5-day ocean level and rainfall design conditions (5-day event starts at hour zero)

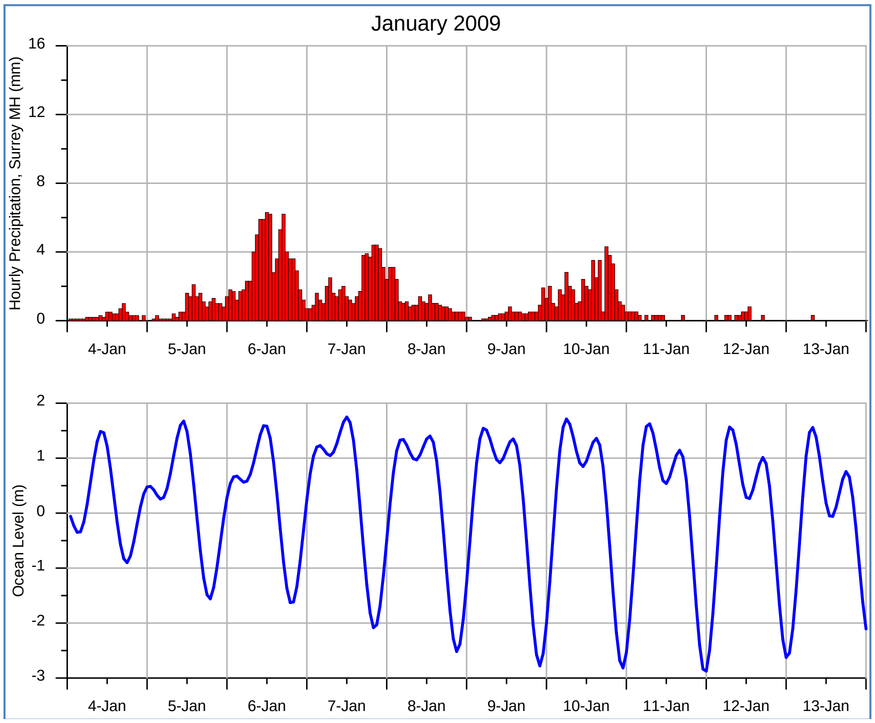


Figure 8.7. Ocean level and rainfall data, January 2009

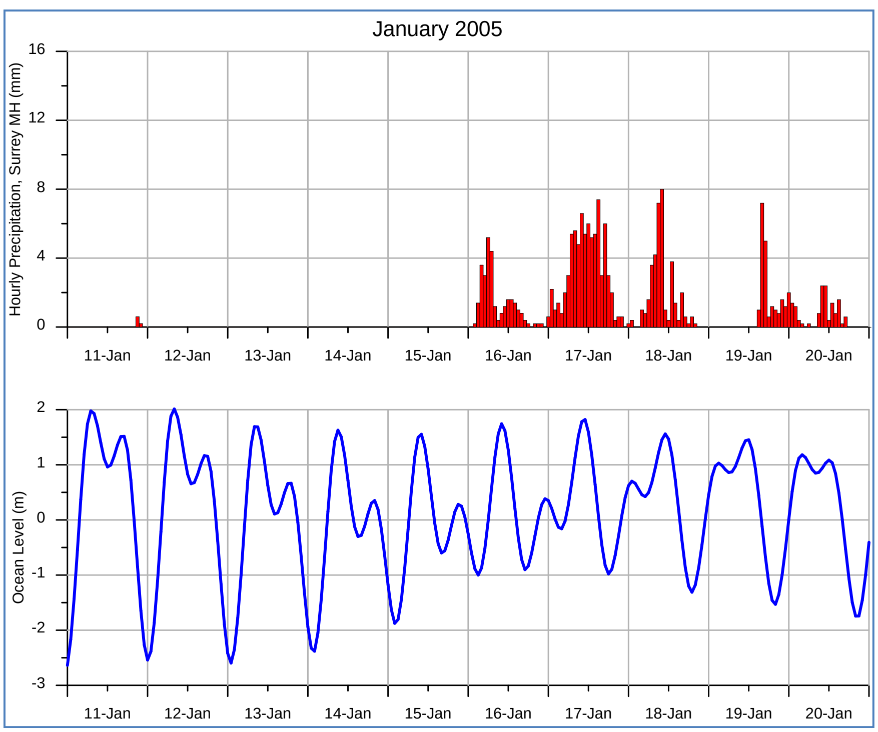


Figure 8.8. Ocean level and rainfall data, January 2005

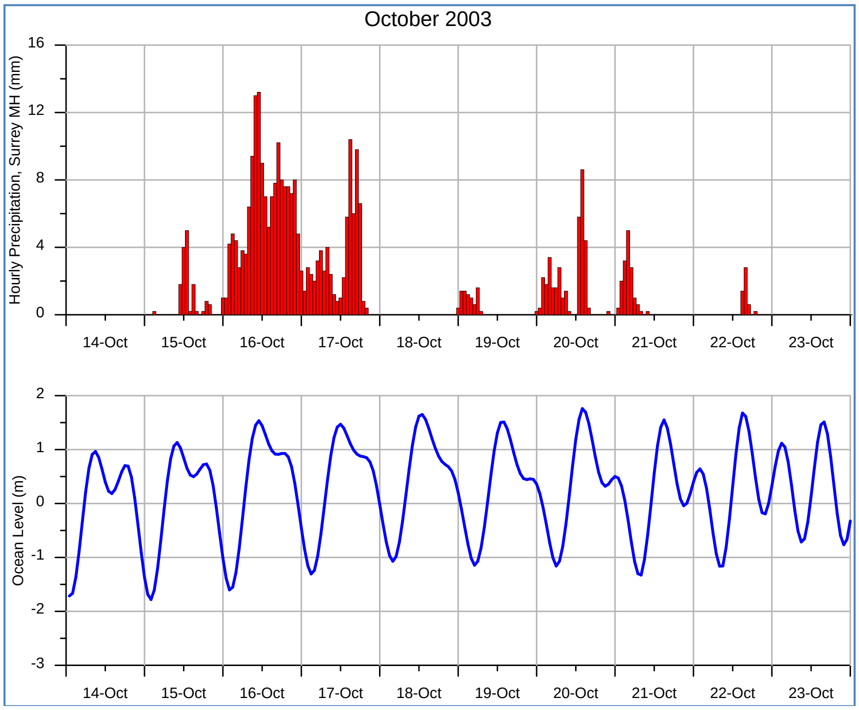


Figure 8.9. Ocean level and rainfall data, October 2003

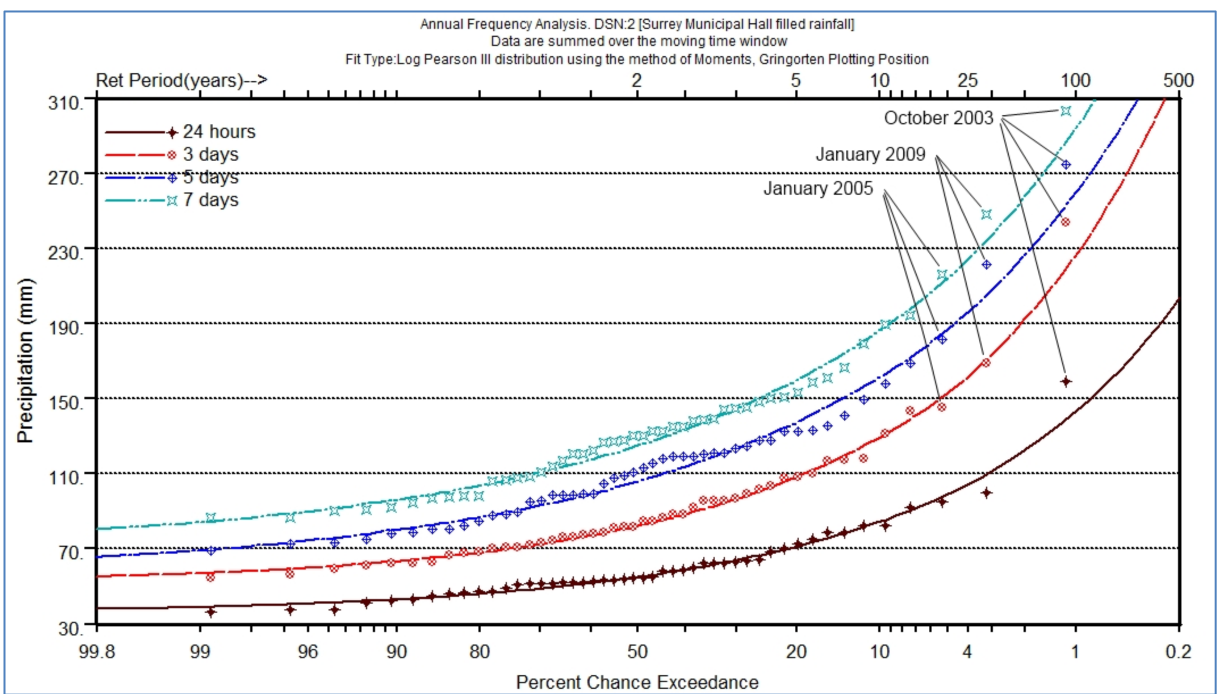


Figure 8.10. Rainfall depth-duration-frequency analysis, Surrey Municipal Hall, water years 1964 - 2011

8.3.2 Comparison of Design Flood Levels

Internal Flood Levels

The Nicomekl and Serpentine River 200-year HEC-RAS profiles (existing conditions) were compared with the 200-year MIKE11 profiles. Results were not expected to be identical since the models have fairly different configurations and boundary conditions. In addition, the HEC-RAS model incorporated datum adjustments, more recent bathymetric data and different spillway configurations. Since the MIKE11 model contains a detailed drainage network for the floodplain and the HEC-RAS model contains storage cells, flood levels outside the river channels were not compared.

The following differences were noted:

- In the lower basins (first 3.5 km above the sea dam on the Serpentine and first 5.5 km above the sea dam on the Nicomekl) the present work yielded respectively up to 0.15 m and 0.2 m higher flood levels. The key reason for this is likely the longer time periods the sea dam gates are closed based on the present ocean design conditions.
- In the mid-basins (up to Km 14 on the Serpentine and Km 12 on the Nicomekl) the design profiles based on the present work are nearly horizontal, compared to average gradients of about 0.00005 on the Serpentine and 0.0001 on the Nicomekl in the previous analyses. The divergence in slopes caused present design levels to be about 0.5 m lower at Km 14 (Serpentine) and about 0.7 m lower at Km 12 (Nikomekl). The observed 2009 and 2005 water levels were seen to also have near horizontal gradients. The higher hydraulic gradients of the previous work is probably a result of the assumed high precipitation spike, causing high instantaneous peak flows and short term very high flood levels.
- In the upper basins, the variation in design levels reduces slightly on the Nicomekl but increases on the Serpentine, the previously estimated design levels remaining higher. Again, these results are likely a result of the flow inputs.

The flood level sensitivity to minor flow changes was described in Section 7.5.2 and was found to be relatively low. However, when ocean levels are allowed to be reduced the sensitivity increases. The HEC-RAS model was also run for the 2003 flood event, which showed a steeper gradient in the mid and upper basins, although as expected, below the 200-year flood.

The present results, based on simplified hydraulic modelling and limited model validation data, should not be used for Flood Construction Levels (FCLs). However, it is recommended that additional work be undertaken to refine the HEC-RAS model and that FCLs be updated in the future.

Ocean Flood Levels

Section 5 described the ocean analysis and gave a 200-year level of 2.70 m for the mouth of the Nicomekl and 2.94 m at the Serpentine. Previous work by NHC and Triton (2006) on the Fraser River estimated a 200-year ocean level of 2.76 m at the outlet of the Fraser Main Arm. The Fraser estimate was derived using the Empirical Simulation Technique and roughly supports the present results.

9 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

9.1 SUMMARY AND CONCLUSIONS

1. The Nicomekl and Serpentine Rivers have a history of flooding and over time, significant flood management infrastructure has been built. High flood levels are a function of high ocean levels, heavy precipitation or a combination of both. Due to the different causes of flooding and the evolving degree of flood protection, it is not possible to pinpoint the historic flood of record. In the upper reaches on the Nicomekl, the October 2003 flood caused the worst flooding (return period in excess of 70 years). For the lowland floodplain, the January 2009 flood was the most severe. In other areas, the December 2007 and January and May 1997 events constituted the flood of record.
2. The continuous simulation approach adopted for the floodplain review formed a statistically defensible method for estimating joint probability ocean/precipitation events, without having to pair certain return period ocean levels with particular runoff events. The combination of an ocean model, hydrologic model and hydraulic model analyzing long-term time-series formed a versatile and flexible tool for assessing flood levels and will allow for future modelling of various conditions as climate change impact estimates are fine-tuned.
3. Current Provincial guidelines suggest adding HHWLT ocean levels, the 200-year storm surge and wind set-up to estimate ocean design levels. The method is conservative and considering the joint probability of these events, may have an overall return period in the order of 10,000 years. Using the Provincial method, a design level of 3.22 m was computed for the mouth of the Nicomekl, 3.50 m for the mouth of the Serpentine and 2.90 m for the mouth of the Campbell River. The method developed in this study gave equivalent design levels roughly 0.5 m lower. A design level of 2.70 m was computed for the mouth of the Nicomekl, 2.94 m for the mouth of the Serpentine and 2.58 m for the mouth of the Campbell River.
4. The Provincial guidelines project a sea level rise of 1 m from year 2000 to 2100. A sea level rise of 0.03 m was observed for the period 2000 to 2010, resulting in a projected rise of 0.97 m from the adopted base year of 2010 to year 2100.
5. A review of subsidence rates in the area suggests an average ground subsidence of 0.225 m over the period 2010 to 2100. Incorporating the ground movement with the projected ocean level increase, resulted in a recommended relative sea level rise of 1.2 m.
6. For wave set-up and runup calculations, the Provincial guidelines suggest, as a starting point, using values large enough to be depth limited. The approach is conservative and instead SWAN modelling was applied.
7. The wave measurements used to validate the wave model were taken over the winter of 1978-1979 about 1.5 km off the shores of White Rock. These measurements were deemed sufficient to validate the wave model for this work.
8. Because of their location up river, measurements made at the ocean side of the Sea Dams were significantly influenced by the river level. The Dams operate using passive flap gates

so that when the river side is higher than the ocean side the river may drain. When the ocean side is higher than the river, the gates shut so that salt water does not propagate up-stream. This means that when the gates are shut the ocean-side gauges accurately measure the ocean level, but, when the gates are open the ocean side gauge is actually measuring the river level.

9. The Design Flood Level (DFL), Flood Construction Level (FCL) and Dike Crest Elevation (DCE) were computed at nine locations for year 2010 and 2100. Existing nominal sea dike crest elevations at sites assessed in this study range from 2.30 m to 3.30 m. DCEs using the joint probability approach vary from 3.51 m to 4.16 m for present conditions (0.53 m to 1.05 m above the existing crest elevations). For 2100 conditions, the DCEs vary from 4.74 m to 6.76 m (1.76 to 3.61 m above existing crest elevations).

10. Wave runup is an extremely complex process and current calculation methods are empirically based. It is the wave runup calculations that alone, are insufficient to inform detailed dike design or upgrades. Accordingly only the DCE and FCL (not DFL) are affected.

11. An HSPF hydrologic model was used to generate time-series of inflow (point and distributed) for the hydraulic modelling. Precipitation records and calibration flow data were sparse and agreement between observed and simulated flows could not be achieved for the Nicomekl River. Better calibration data is required.

12. The existing MIKE11 model of the Serpentine and Nicomekl Rivers was unsuitable for modelling long time-series. Instead, a simplified HEC-RAS hydraulic model was developed using geometric input provided by the City and information extracted from the MIKE11 model. Validation runs using the January 2009 and January 2005 floods showed fairly good agreement for the river channels but poorer results for the floodplain.

13. The HEC-RAS model was run using the 1964-2011 ocean levels (from the ocean model) and inflows (from the hydrologic model) as boundary conditions. The HEC-RAS model output for 40 selected locations for the 1964-2011 time period, provided a tool for estimating existing and future (2100) 200-year flood levels using frequency analyses.

14. In Cell 2, downstream of the sea-dike and between the rivers, the 200-year flood level will increase by 1.1 m by year 2100. In the Nicomekl, upstream of the sea dam for 12 km, the 200-year flood level will increase by 0.9 to 1 m. In the Serpentine, upstream of the sea dam for 14 km, the 200-year flood level will increase by 0.7 m. In the upper basins, the flood level increases taper to 0.1 m. Storage cells on the floodplain will see increases from 0.1 to 0.4 m. The modelling assumed all dikes, including sea dams are raised to prevent overflow.

15. In the lower floodplain the present 200-year flood level will have a return period of approximately 2 years in 2100. In the upper Nicomekl the present 200-year flood level will have a return period of roughly 110 years and in the upper Serpentine the present 200-year flood level will have a return period of roughly 30 years.

16. The 200-year river and floodplain water levels estimated as part of this study should not be used for official Flood Construction Levels. Further refinement of the models is necessary.

9.2 RECOMMENDATIONS

1. Sea level rise in combination with subsidence will significantly increase flood levels and the frequency of flooding, particularly in the lower Nicomekl and Serpentine basins. It is recommended that present structural/ non-structural flood mitigation measures be improved.
2. The wave measurements used for validation were deemed sufficient to validate the wave model for this work. More recent and longer duration wave measurements closer to the areas most threatened by large waves (Crescent Beach, Mud Bay) would provide the opportunity to improve the validation of the numerical wave model and better understand the threat of large waves at the shoreline. Given the necessity to be close to shore, an acoustic Doppler type wave instrument might be most appropriate. These devices are autonomously deployed on the ocean floor for periods of up to several months. These devices can measure wave directionality, something the 1978 buoy could not. To best capture large waves it would be best to deploy the wave measurement device in the winter, within a few hundred metres of the shoreline. The longer the measurements, the better, but even a one or two month deployment may be sufficient to capture some large wave events.
3. Problems with the application of daylight savings time in the data time-stamp needlessly limited the utility of the water level measurements made at the Serpentine and Nicomekl Sea Dams. It is recommended that in the future all measurements be recorded using a consistent time reference such as UTC or PST. In this way problems with daylight savings time will be avoided. If required, the data time-stamp can be converted to PDT prior to dissemination.
4. For ongoing monitoring of sea level in Mud Bay it is recommended that an additional water level gauge be installed. The gauge should be located away from the direct influence of either the Serpentine or Nicomekl and so that it is constantly wet (i.e. not in the drying tidal flats). On initial inspection, the Crescent Beach Pier appears the most appropriate location to install such a gauge. The gauge must be surveyed to a known reference datum, either chart datum or geodetic. Measurements should be recorded at least every hour. For the type of ocean analysis performed in this study, real-time transmission of data is not required.
5. The empirical methods used in this study for wave runup, while informative, should be supplemented with more detailed numerical analysis or hydraulic model testing to ensure that any future detailed dike design will perform as required.
6. Hydrologic modelling in the present study was hampered by inconsistencies and lack of confidence in stream flow data available for model calibration. While water level data are monitored at multiple locations in the Nicomekl and Serpentine watersheds, little work is apparently done to develop and maintain stream gauge stage-discharge ratings. An active long-term program of direct discharge measurements is recommended to develop and maintain reliable stream gauge ratings to resolve apparent inconsistencies and improve the quality of stream discharge data at all gauge locations within the watershed, but particularly

for the Nicomekl at 203rd Street. Consideration should be given to reactivating the former stream flow gauge on Anderson Creek near the mouth. We also recommend that the City investigate the feasibility of measuring directly the total discharge past the two sea dams.

7. The study relied on long-term rainfall data from a single site (Surrey Municipal Hall) for hydrologic model input, with the spatial variation in rainfall over the study area being introduced by application of fixed multipliers on data from that site. While the study area has, on average, a pronounced south to north rainfall gradient, rather significant variations were found in the spatial pattern of rainfall from storm to storm. Hydrologic modelling would be improved by using rainfall inputs from multiple gauge sites. Consideration should be given to reconstructing long-term records of rainfall data from the gauges at Kwantlen Park and White Rock STP to provide better spatial representation of rainfall for hydrologic modelling. Consideration should also be given to improved coverage of rainfall monitoring in the eastern part of the study area by adding rainfall gauges in the Upper Nicomekl and Murray Creek sub-basins. Data in the eastern part of the study area would help resolve uncertainty about west-east variation in rainfall amounts.
8. At the present, there appears to be no active evaporation monitoring sites in the Lower Mainland. While of lower priority for modelling flood flows than improved stream flow and rainfall monitoring, consideration should be given to more comprehensive meteorological monitoring, including pan evaporation data.
9. The present study evaluated the impacts of future sea level rise on flood hazards. More intense and more frequent storms are projected with climate change, implying greater storm runoff volumes and peak flows. The sensitivity of interior flood levels to increased precipitation should be investigated as a surrogate for more detailed examination of the impacts of change in precipitation under climate change.
10. Validation of the hydraulic model to the 2005 and 2009 floods, showed reasonable agreement in the Serpentine but poorer results for the Nicomekl and on the floodplain of both rivers. The Nicomekl discrepancies could be related to the hydrologic model calibration difficulties. We recommend refining the hydraulic model. This can in part be achieved by identifying submerged areas from past flood photography and based on site inspections. Floodplain water levels are recorded at pump stations and their representativeness of average water levels in the modelled storage cells needs to be assessed in more detail.
11. During future large floods, extensive monitoring of water levels and flows is recommended. This will allow re-calibration of the HEC-RAS model to an independent flood event corresponding to the present model configuration. It is recommended that a flood monitoring program be developed.
12. The Provincial guidelines for sea level rise provide a conservative approach for estimating ocean design levels when location specific information is unavailable. The joint probability of coinciding HHWLT, 200-year storm surge and maximum wind set-up may correspond to a 10,000 year return period, not a 200-year event. We recommend that the City select an appropriate design return period based on a risk assessment and that it not be less than 200 years. The existing sea dike is below the present 200-year dike crest elevation based on NHC's joint probability analysis.

13. The ocean, hydrologic and hydraulic models developed for the Nicomekl and Serpentine Rivers form a useful tool for a variety of flood assessments. As more accurate sea level rise and subsidence projections become available it is recommended that the model be re-run and design levels re-assessed.
14. Updating current Flood Construction Levels is recommended.

10 FUTURE INVESTIGATIONS

The first phase of the Serpentine/Nicomekl climate change floodplain review focussed on developing a statistically defensible approach for estimating the joint probability of coinciding high tide, storm surge, wind set-up and precipitation events. The method that was developed allowed assessing the effects of projected sea level rise on flood levels and provided an overview of 200-year flood conditions in year 2100. New additional tasks were identified as a result of the work completed to date. Recommended future investigations are summarized below:

- Further develop the HEC-RAS hydraulic model. Determine if observed water levels at pump stations are representative of water levels in the storage cells as presently defined. This will involve discussions with the City, some site visits and review of the cell topography. To improve model performance, some storage cells may need to be subdivided, for example Storage Cells 5 split into three cells, and Cells 9 and 12 into two cells each. With the help of the City, identify locations of photographs showing previous flooding and confirm inundation extents for the 2009/2005 validation runs where feasible. Together with the City, review the hydraulic structures included in the model that convey flow between the river channels and Cells 5, 7, 8, 9, 12 and other cells as necessary.
- Based on the refined HEC-RAS model, review the Hyland Creek interface; how existing dikes tie into high ground; backwater impacts of piped storm sewer infrastructure and peak river velocity impacts from sea level rise. Review the relative drainage by gravity vs pumping shift as a result of sea level rise. Optimize spillway designs. Assess floodplain impacts of maintaining the current level of drainage service and addition of lower spillways to distribute impacts of sea level rise.
- During future high ocean level and runoff events (comparable to the 2003 and 2009 floods), collect good quality precipitation, water level and flow data in as many locations as possible. Ensure that stage discharge curves for all gauges are accurate. Rerun the hydrologic and hydraulic models, re-calibrate them to the data collected and make model adjustments as necessary.
- Review the MIKE11 200-year design boundary conditions used for computing present FCL's, considering: i) internal flood levels are affected by the length of time tides prevent outflow at the sea dams rather than the peak ocean level; ii) the peak precipitation value (20mm/hour) appears to be outside the expected range; and, iii) the duration of the design storm is less than 4 ½ days. Revise FCLs as necessary based on HEC-RAS boundary conditions.
- Estimate the impact of groundwater flow and dike seepage on floodplain water levels.
- Using the HEC-RAS model, investigate ground subsidence in more detail, determine areal variations in subsidence rates and model subsidence by changing the hydraulic model geometry rather than incorporating subsidence into the relative sea level rise. Use variable subsidence rates for different areas as warranted in connection with absolute sea level rise.
- To allow the City to develop a schedule and budgetary program for upgrading flood protection measures, run the HEC-RAS model for 2030, 2050 and 2070 ocean levels. As a

first approximation, flood levels can be linearly interpolated between 2010 and 2100 water levels. (It should be noted that assuming a linear sea level increase from year 2000 to 2030 yields a rise of about 0.30 m, well above the median projection of about 0.15 m).

- Assess the critical timing and potential vulnerabilities to infrastructure to determine the order of anticipated impacts and the necessary timing for implementing countermeasures.
- Assess potential future changes to precipitation patterns; and incorporate into the hydrologic model to adjust inflows to climate change.
- Extend the analysis to year 2200 based on present average sea level rise projections and future changes to precipitation.
- Determine floodplain extents and zones of influence corresponding to the flood level estimates for year 2100 and other time periods as desired.
- Model the flooding caused by breaching of the river dikes.
- Model the potential inland flooding due to breaching of the sea dikes along Mud Bay during extreme high water conditions.
- Model the potential inland flooding due to breaching of the sea dikes along Mud Bay due to seismic events or tsunami waves.
- Carry out hydrologic and hydraulic modelling for the Campbell River.

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MAPS

APPENDIX A

BACKGROUND INFORMATION

nhc
northwest hydraulic consultants

Table of Contents

1 APPENDIX A.1. – GIS INFORMATION RECEIVED

Table A.1. GIS information received

Description	City of Surrey	City of Langley	Township of Langley	Other
<u>Topography</u>				
Lidar bare earth	2009, City of Surrey and part of Boundary Bay shoreline	2009	None available	-
Contours	1m contours	-	2008 1m contours	-
Ocean bathymetric surveys	2012 Mud Bay survey – some data received	-	-	CHS Electronic Navigation Chart data; CHS bathymetric surveys # 301069 and 301677; NHC April 2012 bathymetric survey near Crescent Beach
River bed surveys (downstream of sea dams)	2012 Serpentine survey received; received 2005 & 2008 Nicomekl surveys	-	-	-
<u>Imagery</u>				
Orthophotos	2009, 2011	2010	2010	-
Satellite image	Lower Mainland 2006	-	-	-
<u>Hydrography</u>				
Stream network	Open channel network	Use FWA stream network, request more detailed data later if required; also see PDF map in report (Urban Systems)	Surficial drainage channels	BC 1:20,000 scale Freshwater Atlas (FWA) stream network
Water bodies	Water bodies	-	-	FWA available, not yet acquired

Description	City of Surrey	City of Langley	Township of Langley	Other
Historic floodplain	200 year floodplain	No	None available	BC MoE 200 year floodplain from 1994
<u>Watersheds</u>				
Watersheds	Drainage catchments – major and sub	Catchment boundaries as PDF map in report (Urban Systems); not available in GIS format	Watersheds	FWA watersheds
<u>Infrastructure</u>				
Pump stations	Yes	Yes	Yes	-
Spillways	Yes, not georeferenced	-	-	-
Dikes	Yes	There are none	Incomplete version acquired, no other GIS layer available although dikes are shown on Township's online GIS	
Sea dams	Yes	-	-	-
Stream gauges	Yes	There are none	There are none	WSC active and discontinued
Rainfall gauges	Yes, City & Metro Vancouver	There are none active	One at Municipal Hall, installed 2005	
Road centrelines	Yes	Yes, plus road names and addresses	Yes	Available, not yet acquired
Rail lines	Yes	-	Yes	Available, not yet acquired
<u>Landuse</u>				
Current landuse	Zoning	Zoning; also see PDF map in report (Urban Systems)	Zoning – incomplete, supplement with orthophotos	-

Description	City of Surrey	City of Langley	Township of Langley	Other
Future landuse	Official Community Plan; received revision May 2012	Official Community Plan	Refer to OCP document and Neighbourhood Community Plan documents (most received in GIS format, some PDF only or not digital)	-
Wetlands	Verbal description only	-	-	-
Other	Parks & natural areas; Agricultural Land Reserve boundaries	-	Agricultural Land Reserve boundaries	-
<u>Soils & Surficial Geology</u>				
Soils	-	As PDF in report (Urban Systems); not available in GIS format	-	Lower Mainland CAPAMP Soils Mapping
Surficial Geology	-	-	Surficial Geology	Geological Survey of Canada
<u>Subsidence</u>				
Subsidence data	Yes, from TRE study	-	-	-
<u>Administrative</u>				
Municipal boundaries	Yes	Yes	Yes	-
Community boundaries	Yes	-	Yes	-

2 APPENDIX A.2. – OCEANOGRAPHIC DATA

Table A.2. Oceanographic Data

Water Level			
Station Name	**Source**	**Duration**	**Notes**
Serpentine Sea Dam	City of Surrey	2000-2011	Several month long gaps. Problems with date/time stamp (now corrected by inspection).
Nicomekl Sea Dam	City of Surrey	2000-2011	Problems with date/time stamp (not yet corrected). Detailed analysis to follow.
Pt. Atkinson	DFO	1914-2012	Complete >1961
Vancouver	DFO	1919-2012	Complete >1943
New Westminster	DFO	1969-2012	A few small gaps
Sydney	DFO	1977-2012	Complete >1975
Victoria	DFO	1910-2012	Complete >1963
Tofino	DFO	1910-2012	Complete >1949
Seattle	NOAA	1902-2012	Complete
Cherry Point	NOAA	1996-2012	2 month gap in 2006
Friday Harbor	NOAA	1996-2012	Complete
White Rock	DFO	19/02/72-26/07/72	Complete
Drayton Harbour	NOAA	05/05/11-13/06/11	Complete
Campbell River	DFO	1965-2012	Complete >1972 (several large gaps 1965-1971)
Steveston	DFO	1969-2001	Not yet assessed
Sooke	DFO	1958-1985	Not yet assessed
Tsawassen	DFO	1967-1978	Not yet assessed

Meteorological			
Station Name	**Source**	**Duration**	**Notes**
Vancouver Intl	EC	1953-2012	Complete
Saturna Island CS	EC	1994-2012	Complete
Saturna Island Campbell Scientific	EC	1980-1992	This data has been ordered but not yet received
Tofino	EC	1960-2012	Complete
Halibut Bank	DFO	1992-2012	Large gaps 1994,2000,2001; small gaps throughout
Sands Head	EC	1994-2012	Several large gaps 1999-2002. It is believed additional data (<1994) can be obtained upon request.
Friday Harbor	NDBC	2005-2012	Not yet assessed.
Wave			
Station Name	**Source**	**Duration**	**Notes**
Halibut Bank (C46146)	DFO	1992-2012	Large gaps 1994,2000,2001; small gaps throughout
White Rock (MEDS116)	DFO	10/14/1977 -3/3/1978	Bad data <Nov 11,1977. Some small gaps.
Roberts Bank (MEDS108)	DFO	2/7/1974-4/3/1976	Not yet assessed.
Sturgeon Bank (MEDS102)	DFO	2/7/1974-4/3/1976	Not yet assessed.

3 APPENDIX A.3. – REFERENCE MATERIAL

Table A.3. Reference material received from Surrey (partial list)

No.	Author	Date	Title	Prepared for
1	Ausenco Sandwell	Jan-11	Draft Policy Discussion Paper	BC Min of Environment
2	Ausenco Sandwell	Jan-11	Guidelines for Management	BC Min of Environment
3	Ausenco Sandwell	Jan-11	Sea Dike Guidelines	BC Min of Environment
4	BC / Canada	Dec-08	Projected Sea Level Changes for BC in the 21st Century	
8	Forman & Henry	1979 / 2004	Tidal Analysis Based on High/Low Water Observations	
9	Golder	Aug-09	Mud Bay and Colebrook Dike Assessment and Functional Plan	
10	HayCo	Mar-99	Crescent Beach Foreshore Assessment	
	Urban Systems/Golder	Jun-09	Crescent Beach Climate Change Adaptation Study	City of Surrey
17	Triton Consultants	Oct-06	Ocean Water Levels and Downstream Boundary Conditions	Fraser Basin Council
	Sea Science	Nov-06	Marine Water Levels - Storm Events	KWL/Delta
	Seaconsult	Jun-05	Extreme Water Levels in Boundary Bay Phase IV	Associated Eng
	Seaconsult	Jun-05	Extreme Water Levels in Boundary Bay Phase II	MOE
	KWL	28-Apr-09	SURREY SERPENTINE AND NICOMEKL LOWLAND FLOOD CONTROL	City of Surrey
14			2009 CONSTRUCTION VERIFICATION, ADDENDUM 1	
15			OUR FILE: 0471.208.300	
18	KPA	1994	Floodplain Mapping Program, Serpentine and Nicomekl Rivers I, II	MOE
	UMA KPA	1997	Nicomekl and Serpentine Integrated Watershed Study - Strategic Plan for Lowlands Flood Control Vol 1,2, 3, 4	City of Surrey
	UMA/AECOM	2000	MIKE11 Schematic	City of Surrey

No.	Author	Date	Title	Prepared for
19	UMA	2001	Conversion of the Serpentine-Nicomekl Lowlands from ONE-D to MIKE11	City of Surrey
	WMC	2002	Drainage System Vulnerability Assessment Project	City of Surrey
	KWL	2011	Little Campbell River Integrated Stormwater Scoping Study Vol 1	City of Surrey
	KWL	2011	Little Campbell River Integrated Stormwater Scoping Study Vol 2	City of Surrey
	Zbeetnoff, Schori	1995	Serpentine-Nicomekl Lowlands Agricultural Profile	City of Surrey
	KWL	2010	Serpentine River Scour Protection Work for the Metro Vancouver's Pipe Crossings Hydraulic Impact Assessment	
	Dayton Knight	1990	1988-1989 Serpentine River Pumping Update	City of Surrey
	MOE	1974	Benefits vs Costs of Drainage and River Dike Improvements Serpentine - Nicomekl Flood Plain	
	Urban Systems		Development of a large Flood Strategy for the Serpentine River Basin	
	KPA	1994	Serpentine and Nicomekl Rivers Technical Appendix to the Design Brief Development of Flood Hydrographs	MOE
	KWL	2008	Verification of the Fraser Highway New Bridge Desin	City of Surrey
	Colliers Maculay Nicolls	1998	Report on Agricultural Land Values, Surrey BC	City of Surrey
	Kistritz Cons	1998	Serpentine and Nicomekl Lowland Flood Control Project Assessment of Environmental Impacts I,II, III	City of Surrey

No.	Author	Date	Title	Prepared for
	UMA	2003	Bridge Crossings in the Serpentine Nicomekl Lowlands	City of Surrey
	Associated Engineering	2009	Lowland Bridge Assessmenet Crossing Review and Design	City of Surrey
	Kistritz Cons	1999	Winter Fish Inventory Results and Risk Assessment of Proposed Pump Stations	City of Surrey
	Associated Engineering	2008	Nico Wynd Dyke Assessment and Maintenance Plan Geotech Appendix by Golder	City of Surrey
	Golder	1999	Bear Creek Dyke Tie-In Geotech Design	
	UMA	1998	Functional Requirements for Floodproofing 152nd Street near Bear Creek	

APPENDIX B

OCEAN ANALYSIS

northwest hydraulic consultants

TABLE OF CONTENTS

1 APPENDIX B.1. - PREVIOUS OCEAN ANALYSES

Several previous studies have attempted to estimate storm surge in Boundary Bay. In general the approach has been to relate the storm surge in Boundary Bay to storm surge at Point Atkinson and other long running tidal stations. The following is a brief, point form summary of the methods used in those studies.

Dunbar and Hodgins (1990)

- This report was not available for review, but is discussed in SeaConsult (1992).
- 200 year water level based on extreme value analysis of surges at Pt. Atkinson, Victoria and Seattle.
- Transfer function developed based on numerical modelling of 5 severe storms.

SeaConsult (1992)

- The report assesses extreme water levels in Boundary Bay.
- Modelling
  - Storm surge modelled using a course grid of Juan de Fuca Strait, and Georgia Strait and a fine nested grid in Boundary Bay.
  - 15 storms were modelled to develop linear transfer function between Boundary Bay and Pt. Atkinson.
  - Externally propagating surge wave considered separately from local wind setup and wave setup.
  - 200 year water level derived by summing the maximum tide, external surge, wind setup, wave setup and low frequency variations without consideration for joint probability.
- Results
  - 200 year external surge based on extreme value analysis of surges at Pt. Atkinson and linear transfer function.
  - 200 year external surge of 124 cm is recommended based on a transfer coefficient of 1.09 and a Pt Atkinson surge of 115cm (Geodetic Datum).
  - 200 year water levels (all contributing components) range from 364 cm at White Rock to 419 cm at the then entrance of the Serpentine River.

SeaConsult (1994)

- The study is an extension on SeaConsult (1992).
- Extreme water levels are calculated using extreme value theory based on two additional methods:
  - By assuming all contributing factors (determined in SeaConsult, 1992) are statistically independent.
  - By coupled modelling of all contributing factors for 12 storms.

- At White Rock, the first method results in a 200 year water level of 286 cm and the second method 290 cm.
- At the entrance of the Serpentine River, the first method results in a 200 year water level of 309 cm and the second method 296 cm.
- It is recommended that the larger of the two values be used.

Hay and Co. (1999)

- Reports a foreshore assessment of Crecent Beach.
- Mainly focused on wave conditions.
- 18 year wave hind-cast performed using course-grid (2km) Donelan Model. Grid did not extend into Boundary Bay.
- An STWAVE model was used to transform off-shore wave conditions calculated in the Donelan Model to wave conditions at Crescent Beach.
- The maximum significant wave height calculated at Crescent Beach was 1.4 m.

Sea Science (2006)

- This report reviews previous work concerning flood inundation in Boundary Bay for the Corporation of Delta.

2 APPENDIX B.2. - EMPIRICAL ORTHOGONAL FUNCTION ANALYSIS OF TIDE STATION DATA

Water levels at a particular location can be partitioned into the component arising from remote sources and the component arising from local sources. In the context of this work, remote sources are coastal sea level variations, and local sources include local wind forcing, runoff, and wave setup.

Moreover, the remote sources are partitioned by frequency into tidal period variations (period less than 30 hrs), and low frequency variations (period greater than 2 days). In practical terms, the motions with periods around 1 to 2 days are damped by the filters used in the analysis. The tidal period variations are determined from harmonic analysis and are treated separately.

The task here is to determine the appropriate low frequency remote forcing to use with sea level reconstruction at the study site.

A powerful method to extract information from multiple time series of sea level observations is Empirical Orthogonal Function analysis (EOF, also known as Principle Component analysis). In general, one attempts to reduce a large number of correlated data to a few time-series (eigenmodes) that contain the majority of the variance in the data set. Each eigenmode can then be identified with some physical mode of the system.

In this analysis, cross-correlations R are used where

$$R=R(h_i,h_j)=\frac{1}{M}\sum_{k=1}^{M}h_i(t_k)h_j(t_k) \qquad (1)$$

and $h_i(t_k)$ is the water surface elevation at the i^{th} measurement site at time t_k and M is the total number of observations. Using this coefficient matrix, one can solve an eigenvalue problem and derive a set of eigenfunctions $F_n(h_i)$ and positive eigenvalues λ_n. The sum of the eigenvalues is the total variance, whereas each eigenvalue divided by this sum is the percentage of the variance contained in each eigenmode. The measurements of sea level at each site can be expressed in terms of the N eigenfunctions as

$$h_i(t_k)=\sum_{n=1}^{N}A_n(t_k)F_n(h_i) \quad (2)$$

where $A_n(t_k)$ is the amplitude of the n^{th} mode at time t_k. Finally, the amplitude is given by

$$A_n(t_k)=\sum_{n=1}^{N}h_i(t_k)F_n(h_i) \quad (3)$$

as the eigenfunctions are orthogonal.

Usually, there are a few dominant eigenmodes whose spatial distribution is given by F_n, and whose time response is given by the corresponding time-dependent amplitude A_n. These modes with large eigenvalues contain the majority of the variance in the data such that the smaller modes can be neglected as noise. The amplitudes A_n can then be correlated with the various forcing functions to find the dominant physical mechanism(s) for each mode.

For the analysis presented here, we initially used the 12 sites listed in Table 1. These sites range from the open coast to the inland marine waters of British Columbia and Washington State. Water level residual is calculated by subtracting predicted tide from measured water level, then low pass filtering these residuals with a Godin 24-24-25 filter (2.5 day half power point).

The first eigenmode (Figure 1) contains 93.5 % of the variance and thus dominates the low frequency signal. This signal corresponds to the seasonal variation in sea level due to wind driven coastal upwelling. Because of the deep water connections between the coastal ocean and the inland marine waters, this variation occurs essentially simultaneously over the region. The spatial pattern is relatively uniform with slightly larger amplitudes at the coastal sites - Tofino and Neah Bay.

The second eigenmode contains 3.5 % of the variance. This signal is due to a difference in sea level between the coastal sites and Salish Sea sites due to density differences between the cold, salty coastal water and the fresher, warmer inland waters, and frehwater runoff runoff in the inland areas.

The third eigenmode contains 1.7 % of the variance and has an amplitude that increases monotonically from the beginning to the end of the calendar year. The random spatial pattern of this mode suggests that it is a noise or error signal. This mode and the remaining small modes are neglected.

From this analysis, we found that observations at Point Atkinson provide a valid reference value for low frequency remote forcing. Moreover, the analysis of Point Atkinson data and limited White Rock observations indicated that there was little difference in the external forcing at these two locations due to the deep water connection betweem them (Figure 2).

Table 1. Sites used in the EOF analysis.

Site	Latitude(N)	Longitude(W)	source
Pt Atkinson	49.34	-123.25	DFO
Vancouver	49.29	-123.11	DFO
Sydney	48.65	-123.45	DFO
Victoria	48.42	-123.37	DFO
Campbell River	50.04	-125.25	DFO
Friday Harbor	48.54	-123.01	NOAA
Seattle	47.60	-122.34	NOAA
Cherry Point	48.86	-122.76	NOAA
Port Townsend	48.11	-122.76	NOAA

Port Angeles	48.13	-123.44	NOAA
Neah Bay	48.37	-124.61	NOAA
Tofino	49.15	-125.91	DFO

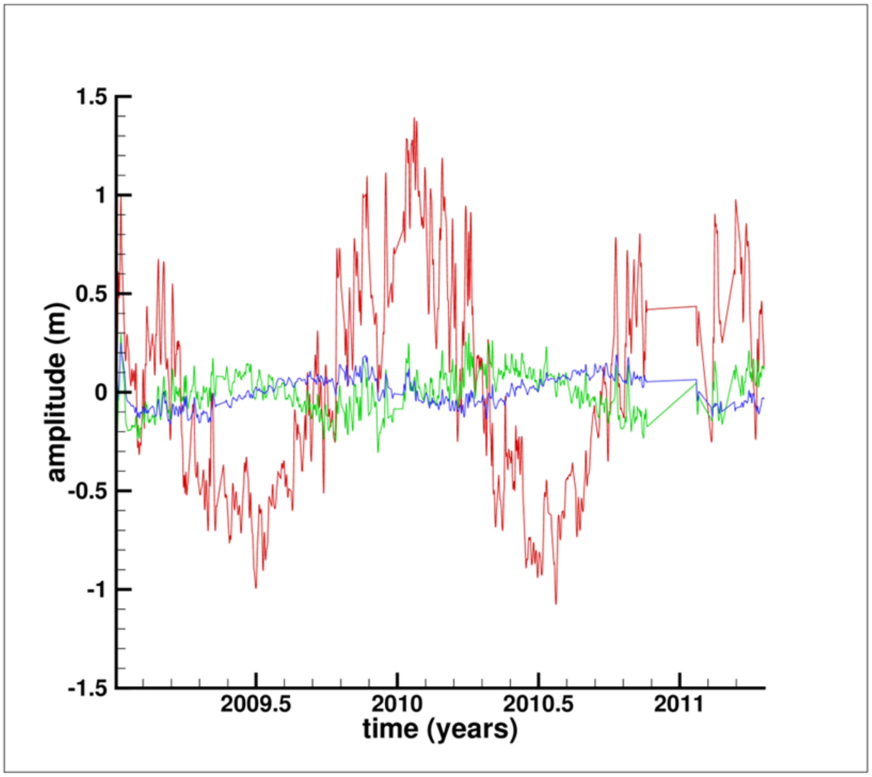


Figure 1 - The first 3 eof modes. First=red, second=green, and third=blue.

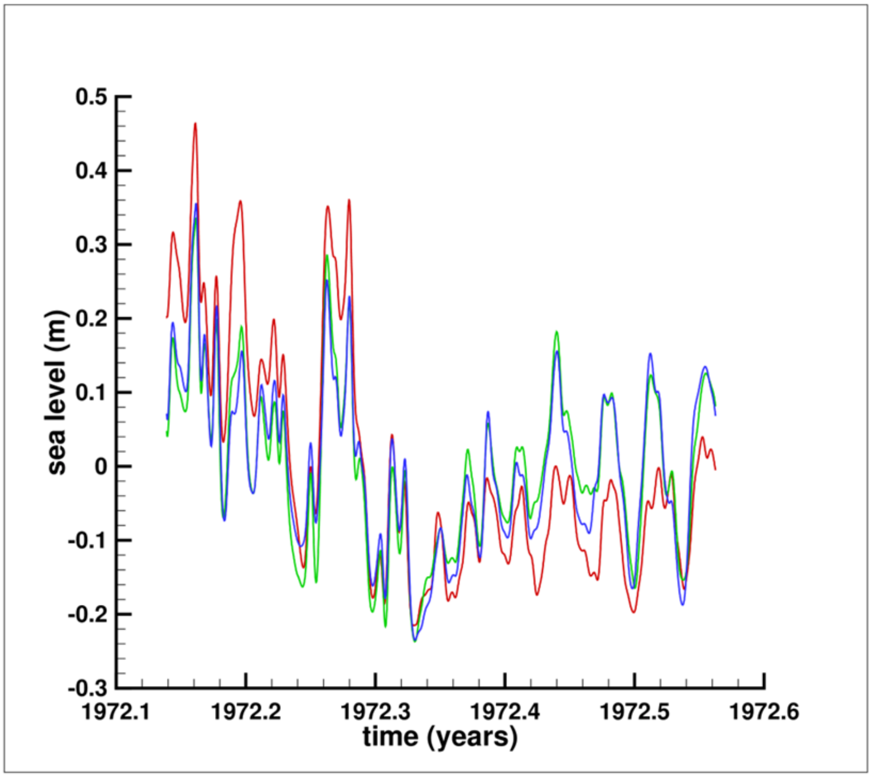


Figure 2 - Low frequency time-series at Neah Bay (red), Point Atkinson (green), and White Rock (blue).

3 APPENDIX B.3. - THE RIVER AND COASTAL OCEAN MODEL

The River and Coastal Ocean Model (RiCOM) was developed by Dr. Roy Walters formally of the National Institute for Water and Atmospheric Research of New Zealand and US Geological Survey who is now a modeling consultant with Cascadia Coast Research Ltd and formerly with Triton Consultants Ltd. RICOM was developed to solve some of the longstanding problems with finite element methods – namely lack of local mass conservation and problems with stability and/or accuracy with advection-dominated flows. In addition, a double-averaging method (DAM) has been incorporated into the model to allow an accurate approximation of subgrid objects and their effects on the volume averaged flow. The latter provides a means to couple the results of small-scale CFD models with the large-scale oceanographic model.

RICOM solves the primitive hydrodynamic equations with a semi-implicit time-stepping scheme that is unconditionally stable with respect to time-step size so that the time-step size is controlled by the physics of the specific problem under consideration rather than by numerical constraints. Secondly, the model uses a semi-Lagrangian approximation for advection that is accurate, stable, and robust which yields accurate results without oscillations for high speed flows such as occur over weirs, in flow constrictions, and tidal rapids. Finally, the model uses a finite element spatial approximation that gives considerable flexibility in designing the computational grid. The particular elements that are chosen have no spurious modes so that the solution is free of grid-scale oscillations (Walters and Casulli, 1998; Walters, 2006; Walters et al, 2009). Because of the design of the algorithm, wetting and drying capabilities are inherent to the finite volume continuity equation and do not require any special attention. In addition, the model conserves mass both locally and globally which is an important property when dealing with solute and particulate transport, especially when the transport equations are in a finite volume form.

RiCOM is formulated from the Reynolds-averaged Navier-Stokes equations that are time averaged over turbulent time scales. The governing equations are derived using the Boussinesq approximation and by introducing a rotating frame of reference. The equations are spatially averaged to derive double-averaged equations that allow sub-grid spatial effects (vegetation, bottom roughness, etc.) to be included in a rigorous manner (Walters and Plew, 2008). The discretized equations are derived using a finite element approximation in space and a finite difference approximation in time. A more detailed description of the technical model background can be found in Walters et al (2009, model NPI).

RiCOM has been successfully applied to a number of recent projects including storm surge estimates for the southern Beaufort Sea (Canada), tidal dynamics in the Fraser River (B.C.), Cook Inlet Alaska, South West Korea, Discovery Islands (BC) and the Bay of Fundy/Minas Passage (years 2009 to 2011) in eastern North America. In all these projects the model was validated in both 2D and 3D mode against measured tidal height and ADCP current data.

REFERENCES

Walters, R.A., 2006. Design considerations for a finite element coastal ocean model, Ocean Modelling 15, 90-100.

Walters, R.A. and Casulli, V., 1998, A robust, finite element model for hydrostatic surface water flows. Communications in Numerical Methods in Engineering 14, 931-940.

Walters, R.A., and D.R. Plew, 2008. Numerical Modeling of environmental flows using DAM: Some preliminary results. Acta Geophysica 56(3), 918-934.

Walters, R.A., Hanert, E., Pietrzak, J., Le Roux, D.Y., 2009. Comparison of Unstructured, Staggered Grid Methods for the Shallow Water Equations. Ocean Modelling 28, 106--117.

4 APPENDIX B.4. - ESTIMATION OF EXTREME WATER LEVELS

Estimation of extreme water levels was performed using extreme value theory, also called frequency analysis. There are a number of different approaches that can be used to perform frequency analysis with the *Annual Maximum* and *Peak-over-threshold* approaches being two of the most common.

Using the *Annual Maximum* approach the largest water level records of each year of data are identified and return periods of each event are calculated. An appropriate statistical distribution is fit to the data, and then the water level corresponding to the desired return period (beyond the range of the data) is estimated using that distribution.

A similar process is used in the *Peak-over-threshold* approach. Instead of yearly maximums, water level events exceeding a specified threshold are identified. The return period of each event is calculated based on the average time between events. An appropriate statistical distribution is fit to the data, and then the water level corresponding to the desired return period(beyond the range of the data) is identified using that distribution.

This Appendix examines this appropriateness of each of these methods for estimating extreme water level events in Boundary Bay. It also seeks an appropriate statistical distribution to fit to the empirical water level data. The 48 year water level hind-cast at the mouth of the Nicomekl River was the basis of the extreme for this investigation.

4.1 TOTAL WATER LEVEL EXTREME ESTIMATES

All fitting was done using the WAFO Toolbox for Matlab. The results using the annual maximum approach are given in Table 2. The correlation coefficient indicates the goodness of fit between the data and distribution, the RMS error indicates the root-mean-square difference. The Gumbel and the GEV distributions rank best in terms of both parameters. Unfortunately these two distributions give very different long term projections. To shed some light on this, the analysis was performed again using the peak over threshold approach.

The peak over threshold approach was used with a water level threshold of 2.1m and a minimum event duration of 2 hours. The results are given in Table 3. Again the Gumbel and GEV have the best fit stats, but in this case their long-term projections are much closer and are also close to the GEV projections using the annual maximum approach.

It appears that the lower values (1.8-2.1m) in the annual maximum approach skewed the Gumbel fit so that the slope of the fit was greater than it should have been. By eliminating these values in the peak over threshold approach and concentrating on the highest hind-cast values a more accurate distribution fit was achieved.

Therefore the peak over threshold approach with the GEV distribution was selected for total water level extreme value estimates.

Table 2. Total water level extremes estimated by various distribution using the annual maximum approach [48 years].

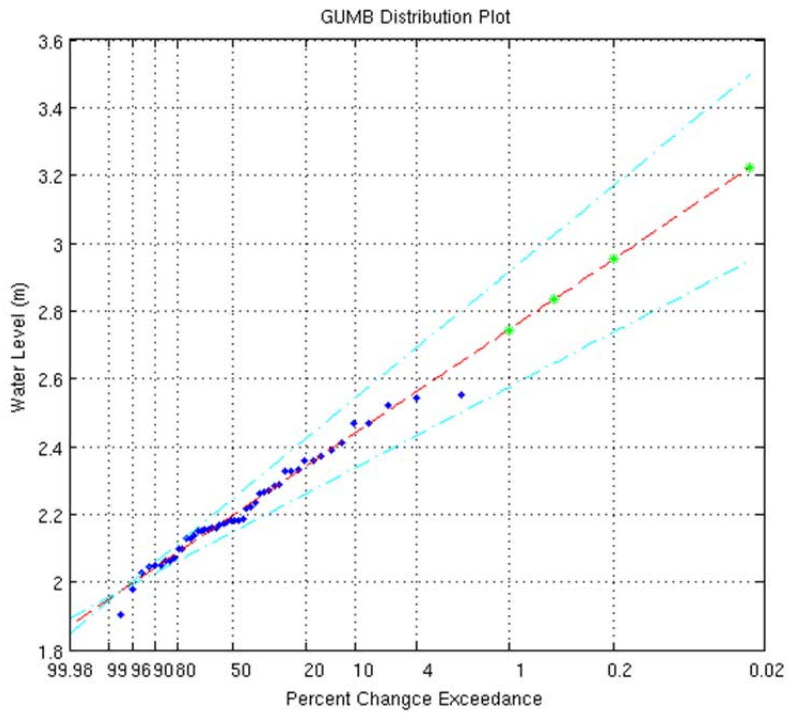
			Water Level (m)			
Distribution	**Corr. Coeff.**	**RMS Error**	**100yr**	**200yr**	**500yr**	**4000yr**
Generalized Extreme Value	0.992	0.02	2.657	2.719	2.795	2.950
Gumbel	0.990	0.022	2.744	2.834	2.953	3.222
Log Normal	0.988	0.025	2.581	2.624	2.677	2.786
Normal	0.984	0.027	2.567	2.604	2.649	2.740
Weibull	0.955	0.051	2.538	2.562	2.590	2.641

Table 3. Total water level extremes estimated by various distributions using the peak over threshold approach with water level threshold of 2.1m and time threshold of 2 hours [59 occurrences].

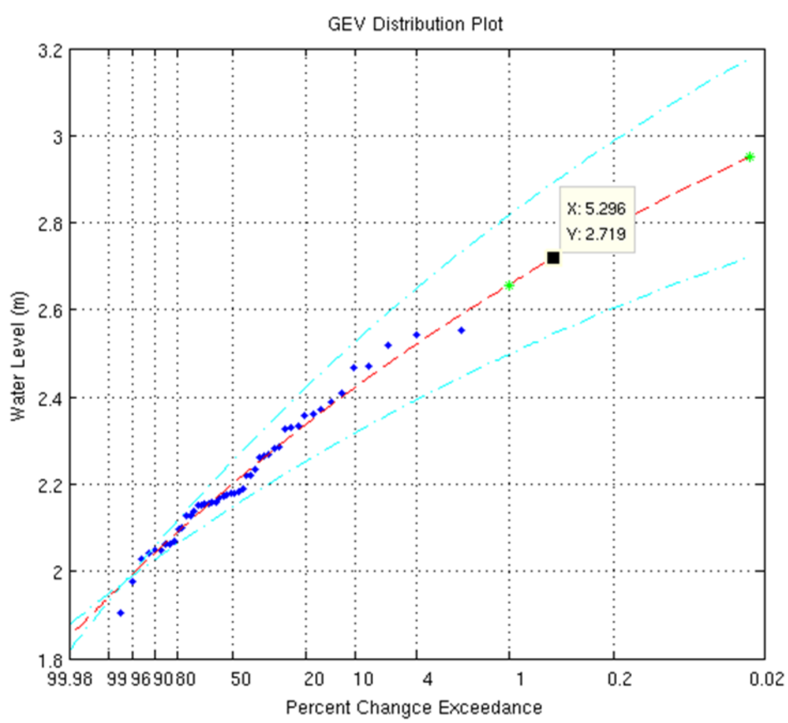
			Water Level (m)			
Distribution	**Corr. Coeff.**	**RMS Error**	**100yr**	**200yr**	**500yr**	**4000yr**
Generalized Extreme Value	0.996	0.011	2.642	2.701	2.778	2.951
Gumbel	0.996	0.013	2.632	2.691	2.769	2.946
Log Normal	0.980	0.022	2.540	2.569	2.605	2.678
Normal	0.975	0.025	2.534	2.560	2.593	2.657
Weibull	0.927	0.053	2.521	2.538	2.558	2.595

Annual Maximum: 48 years

GUMBEL

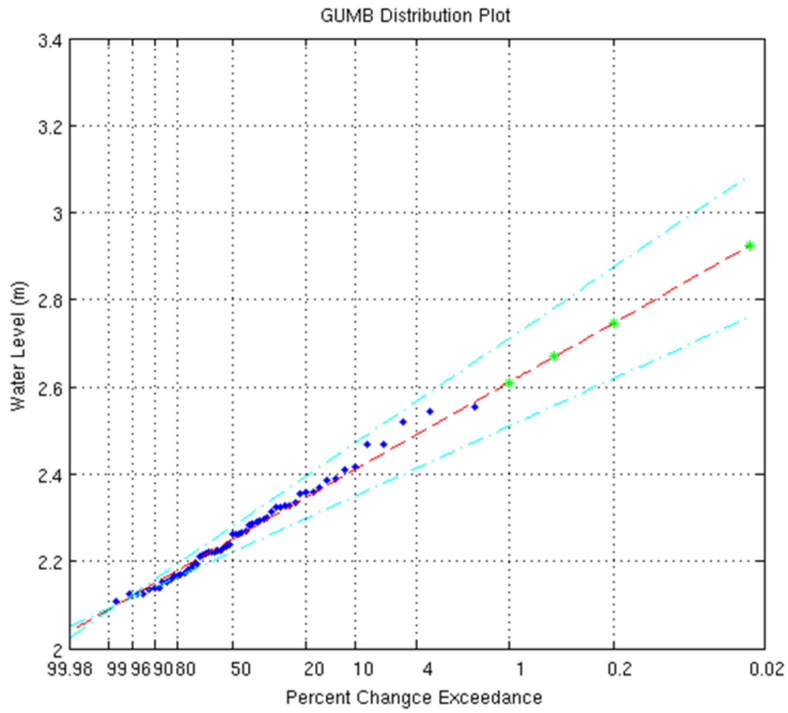


GEV

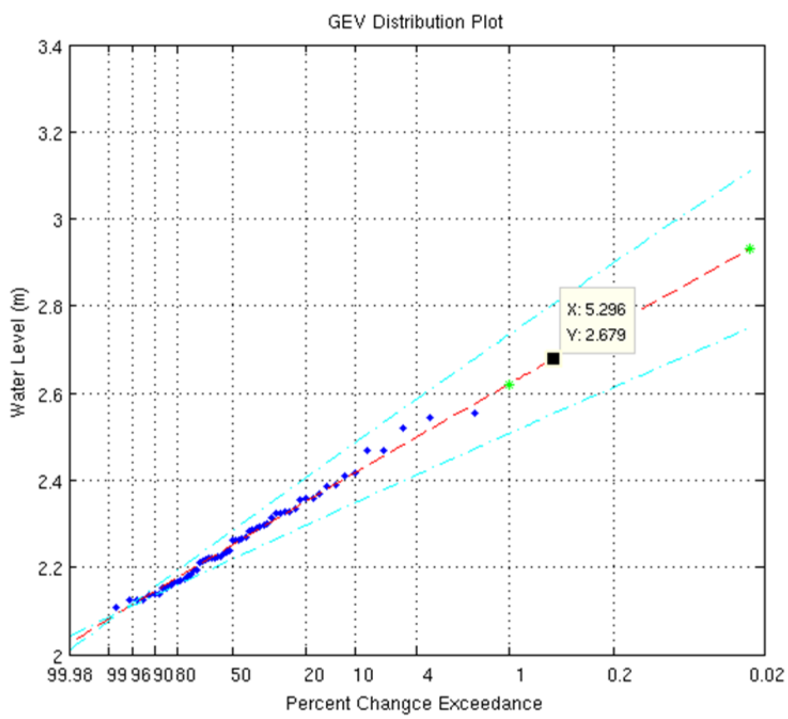


Peak Over Threshold: Water Level threshold = 2.1m, Time threshold = 2 hours (59 occurrences)

GUMBEL



GEV



4.2 External Surge Extreme Estimates

Given the issues encountered with the estimation of extreme values for the total water level, a similar analysis was carried out for the external surge.

The fit results using the annual maximum approach and various distributions are given in Table 4. The best fits in terms of correlation coefficient and RMS error are the GEV distribution and the Normal distribution. The Gumbel has a surprisingly poor fit by comparison.

The peak over threshold approach was used with a water level threshold of 0.8m and a minimum event duration of 2 hours. The results are given in Table 5. The GEV and Gumbel have the best fit statistics. The GEV closer approximates the highest measured values, but the extremes estimated with the GEV distribution are significantly higher than those estimated using the other distributions

It appears again that the lower values in the annual maximum approach have skewed the distribution fits. The peak over threshold approach concentrates on higher values in the distribution and so appears more appropriate for fitting of this data.

The choice between distributions comes down to the 5 highest measured surge values. These 5 values don't adhere well to the trend set by the lower data values. It may be that these values are just randomly higher than is statistically expected and in this case the Gumbel distribution would be the most appropriate. If those five values actually represent the mean of the distribution, then the distribution inflects upwards at the end and the GEV distribution is most appropriate. As it seems unlikely that 5 consecutive values would fall above the distribution mean, we must regard those values as representative of the mean. So, for the external surge extreme value estimates the peak over threshold approach with the GEV distribution is used.

Table 4. External surge extremes estimated by various distributions using the annual maximum approach [48 years].

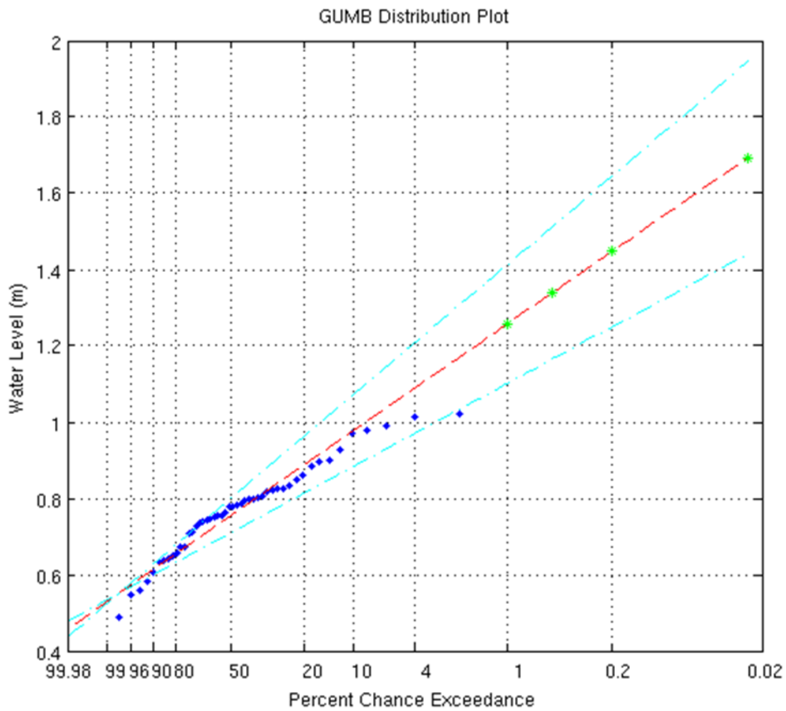
			Water Level (m)			
Distribution	**Corr. Coeff.**	**RMS Error**	**100yr**	**200yr**	**500yr**	**4000yr**
Generalized Extreme Value	0.992	0.016	1.045	1.066	1.088	1.122
Gumbel	0.975	0.033	1.256	1.339	1.447	1.694
Log Normal	0.989	0.019	1.110	1.156	1.214	1.339
Normal	0.993	0.016	1.057	1.087	1.124	1.198
Weibull	0.988	0.019	1.026	1.047	1.071	1.116

Table 5. External surge extremes estimated by various distributions using the peak over threshold approach with water level threshold of 0.75m and time threshold of 2 hours [55 occurrences].

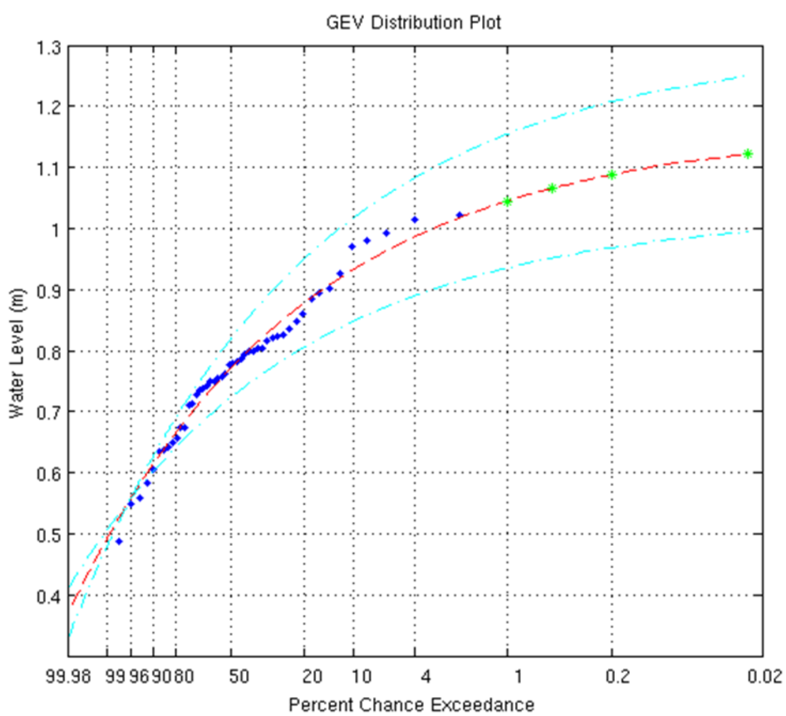
			Water Level (m)			
Distribution	**Corr. Coeff.**	**RMS Error**	**100yr**	**200yr**	**500yr**	**4000yr**
Generalized Extreme Value	0.993	0.009	1.105	1.160	1.239	1.446
Gumbel	0.991	0.013	1.046	1.079	1.124	1.225
Log Normal	0.967	0.017	1.009	1.028	1.051	1.100
Normal	0.956	0.020	1.003	1.020	1.040	1.079
Weibull	0.909	0.035	1.002	1.013	1.027	1.052

Annual Maximum: 48 years

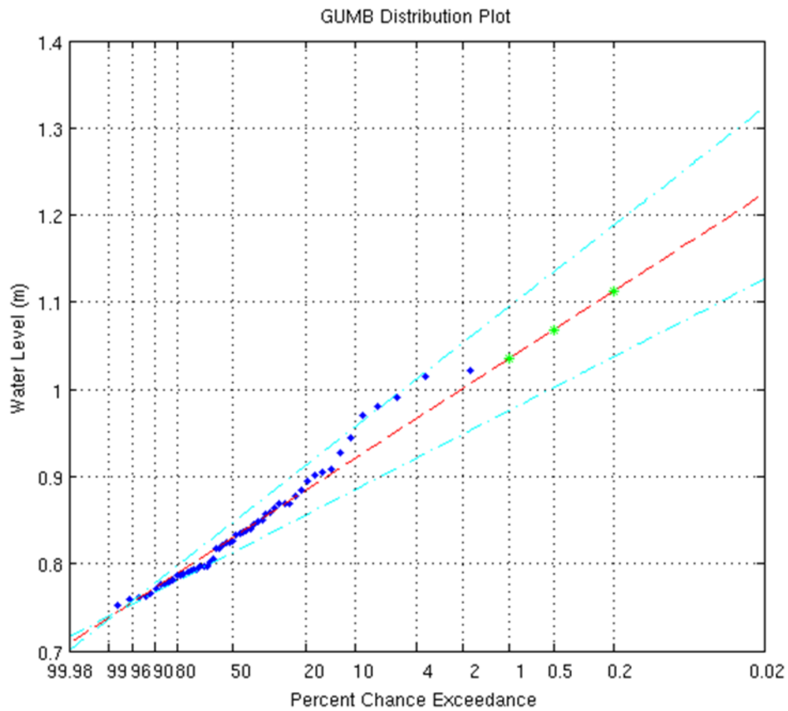
GUMBEL



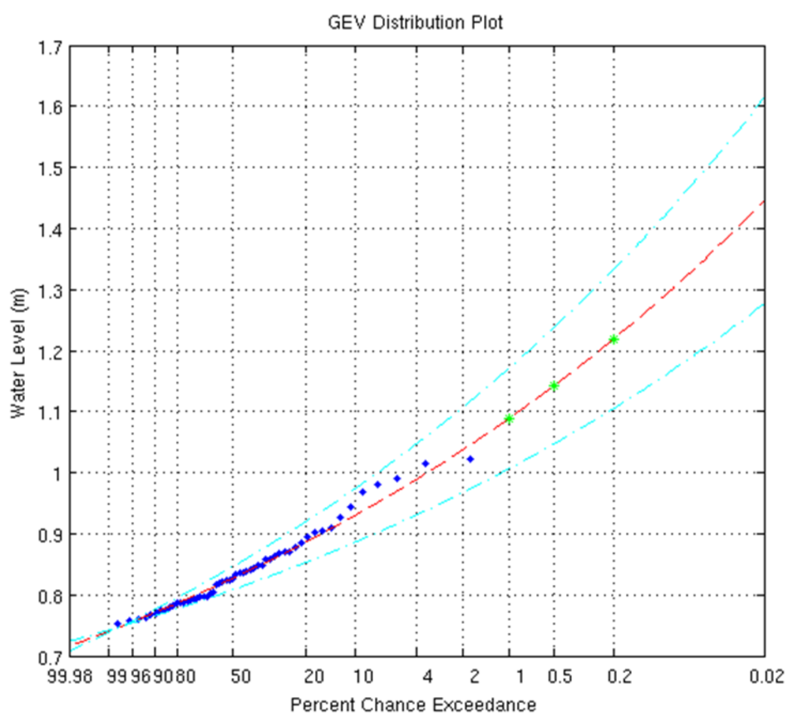
GEV



Peak Over Threshold: Water Level threshold = 0.8m, Time threshold = 2 hours (29 occurrences)
GUMBEL



GEV



5 APPENDIX B.5. - WAVE MODELING: STORM SELECTION

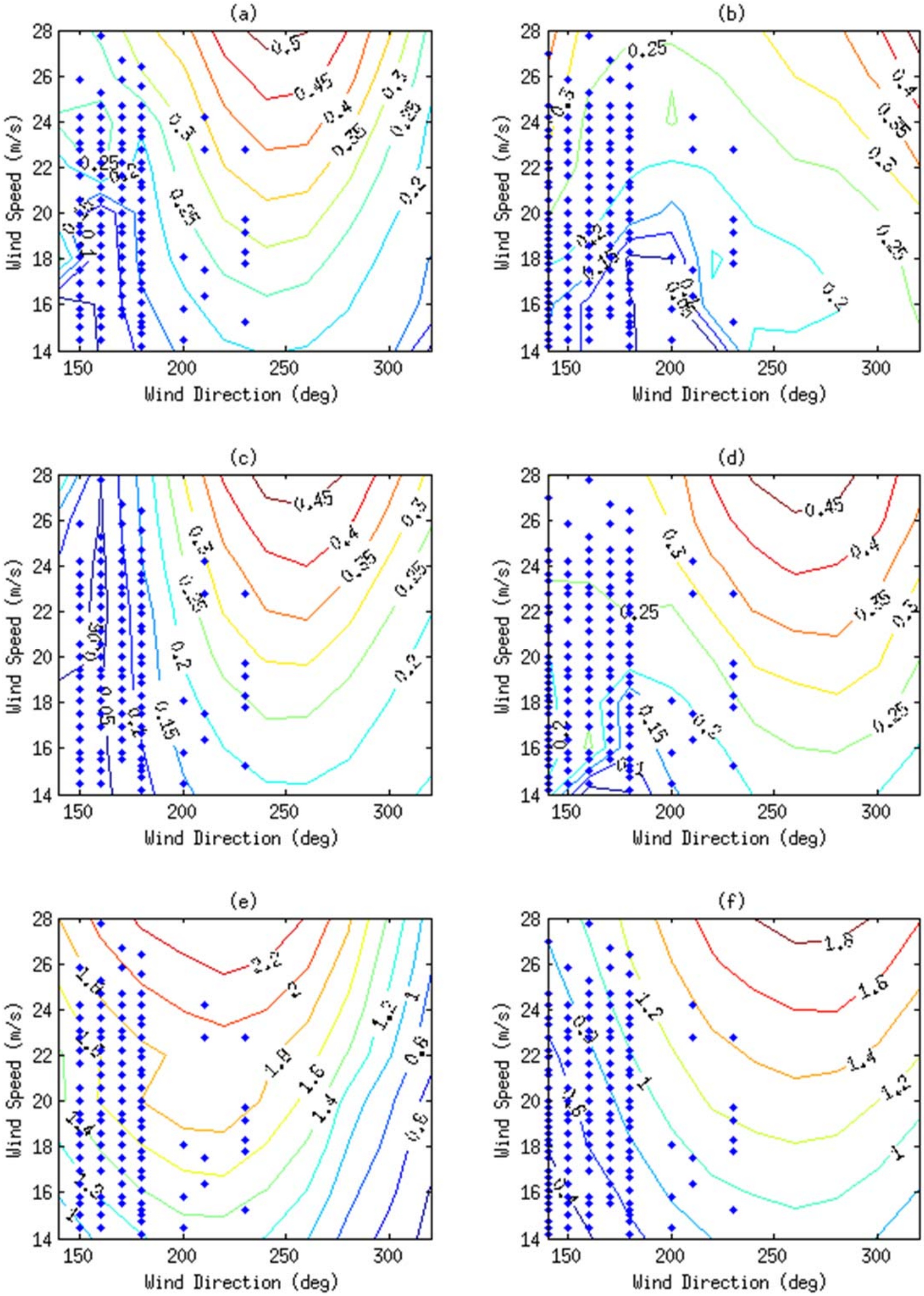
To identify which storm events create the largest waves at the sites of interest, 80 separate wave model runs were performed for a range of wind speeds and directions. The results were then compared to storms measured at Saturna Island. In Figure 3, each subplot shows the storms measured at Saturna Island plotted over contours of significant wave height for each location. Colour contours show the significant wave height at the site resulting from the wind speed and direction (given on the 'y' and 'x' axis) and the blue dots give storm conditions measured at Saturna Island. Note that a depth offset of 10m was used to eliminate the effect of depth induced breaking on wave estimates. Subplots ‘a' through ‘i’ correspond to locations of interest as follows:

#	LOCATION	subplot
1a	Colebrook – Serpentine	A
1b	Crescent Beach East	B
1c	Mud Bay - Serpentine	C
1d	Mud Bay – Nicomekl	D
2	Colebrook (Hwy99)	E
3	Crescent Beach North	F
4	Crescent Beach South	G
5	BNSF Railway	H
6	8th Ave @ Campbell	I

The three storms estimated to cause the largest significant wave height at each location were selected to be run in non-stationary simulations. The top three storms were the same storms for many of the locations. A complete list of all top storms is given in the table below. Note that wind data measured 1969-1993 are hourly averages and data measured 1994-2012 are averages of the last 10 minutes of the hour. No systematic difference between the two measurement techniques was identified.

Table 6. Design storms for wave model

Storm #	Nominal wind direction (deg)	Maximum wind speed (m/s)	Date Range
1	230	22.78	Dec 12 – 27, 1982
2	210	24.17	Mar 13 – 27, 1994
3	160	29.72	Nov 5 – 19, 2007
4	180	26.39	Nov 15 – 22, 1991
5	170	26.67	Nov 18 – 28, 1998


(a)
(b)
(c)
(d)
(e)
(f)
Wind Speed (m/s)
14
16
18
20
22
24
26
28
Wind Direction (deg)
150
200
250
300

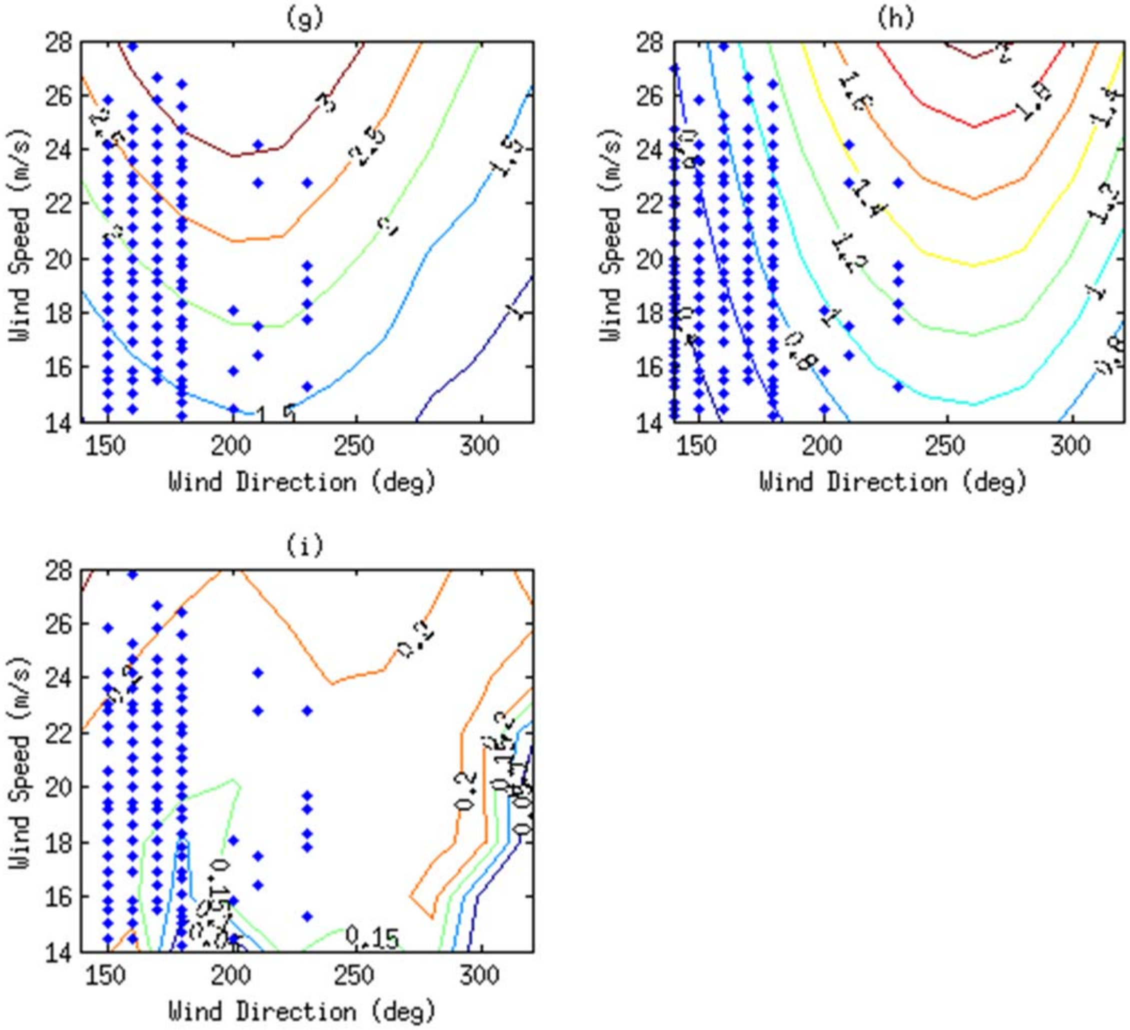


Figure 3 - Colour contours show the significant wave height at each location resulting from the wind speed and direction (given on the 'y' and 'x' axis) and the blue dots give storm conditions measured at Saturna Island.

6 Appendix B.6. – Wave Runup and Dike Geometry

Within the PC Overtopping wave runup software, each dike was modelled as an armoured slope with a simplified geometry. The geometry of each dike was idealized from a dike cross section extracted from high density LIDAR data. Each dike cross section and its simplified representation is presented in this section.

Figure 4 shows the locations of interest where wave runup calculations were made (except for Location 6: Campell River @ 8th Avenue). Figure 5 through Figure 13 show the LiDAR derived dike profile for each location and the corresponding simplified dike profile. The arrows show the direction of wave incidence.

Associated with each section of the simplified dike profile is a roughness factor that is input to the PC Overtopping software. The roughness factor accounts for the different behaviour of wave runup over different materials. The roughness factors for each segment were estimated based on photos collected on-site and from aerial photographs. Most segments were assigned a roughness factor of 0.55, corresponding to a *natural rubble mound of rock,* others were assigned a factor of 1.0 corresponding to grass or similar ground cover.

The geometry and roughness factors *(rf)* used for each runup calculation are presented in Table 7. For the 2100 water level scenario most dikes were underwater. To achieve more meaningful results in the year 2100 water level scenario, 1.5m was added to the crest of each dike. This value is indicated in brackets

For runup model input parameters and results please see the body of the report.

Table 7. Dike geometry and roughness factors

1a	Colebrook Sheltered (Serpentine)	rf	0.55	0.55	~	~	~
		x	45	54	~	~	~
		y	-1.7	2.84 (4.34)	~	~	~
1b	Crescent Beach Sheltered	rf	1	1	1	1	1
		x	-91	-77	-68	-56	-43
		y	0.15	2.75	2.88 (2.38)	0.65	2.7
1c	Mud Bay Sheltered (Serpentine)	rf	0.55	0.55	~	~	~
		x	43	51	~	~	~
		y	-1.6	3 (4.5)	~	~	~
1d	Mud Bay Sheltered (Nicomekl)	rf	0.55	0.55	~	~	~
		x	47	53	~	~	~
		y	-0.8	2.98 (4.48)	~	~	~
2	Colebrook Dike at Hwy99	rf	1	1	1	1	~
		x	35	50	72	78	~
		y	0.45	2.35	2.35	3.15 (4.65)	~
3	Crescent Beach North	rf	1	1	0.55	~	~
		x	31	46	52	~	~
		y	-1.3	0.6	2.9 (3.4)	~	~
4	Crescent Beach	rf	0.55	0.55	~	~	~

	West	x	35	58	~	~	~
		y	0.2	3.3 (4.8)	~	~	~
5	BNSF Railway	rf	0.55	0.55	~	~	~
		x	-54	-45	~	~	~
		y	0.5	3.2 (4.7)	~	~	~
6	Campbell at 8th ave Sheltered	rf	1	1	1	1	~
		x	47	51	56	59	~
		y	0	1.3	1.3	2.3 (3.8)	~

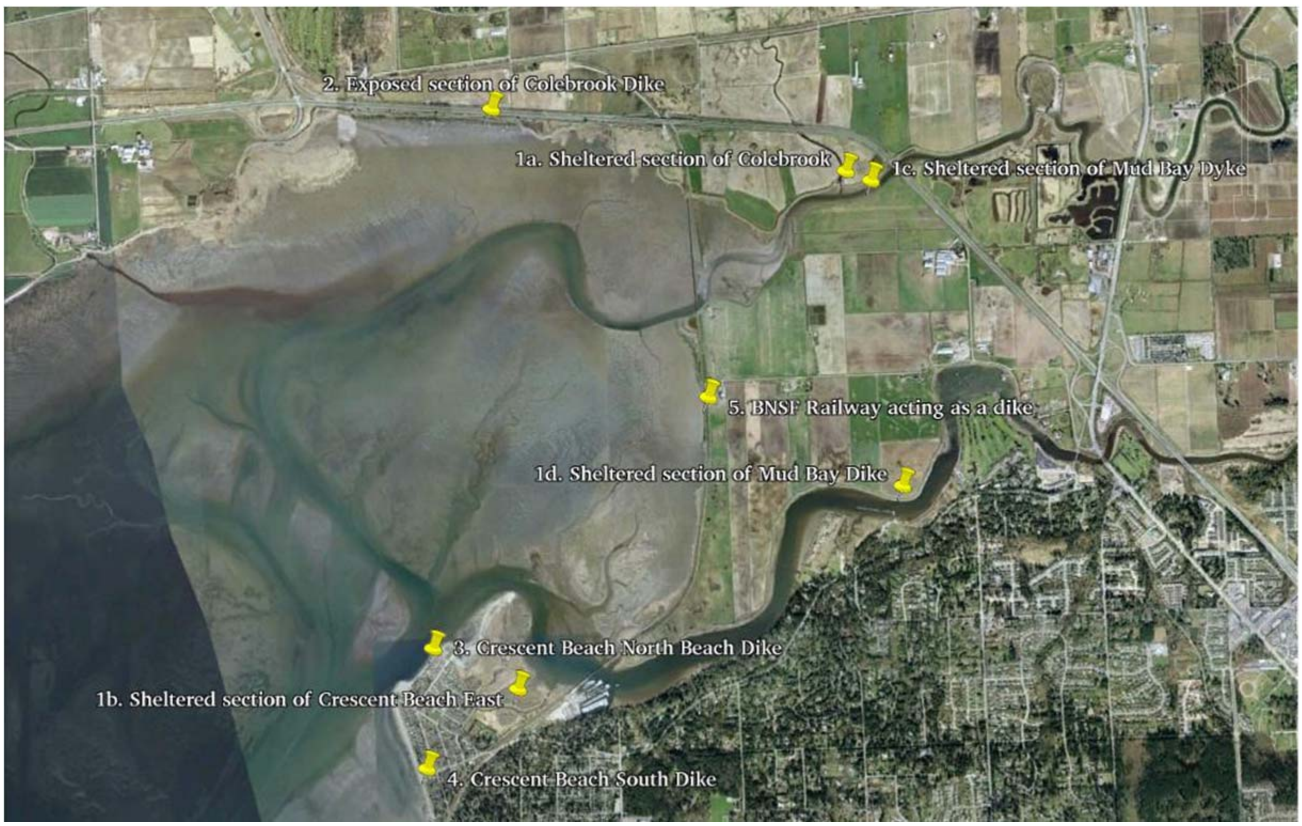


Figure 4 - Locations in Mud Bay for Dike Crest and Flood Construction Level calculations.

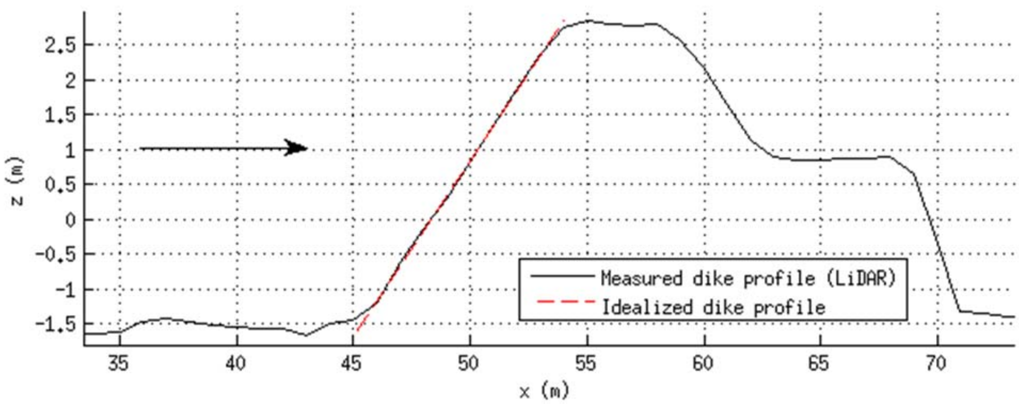


Figure 5 - Location 1a

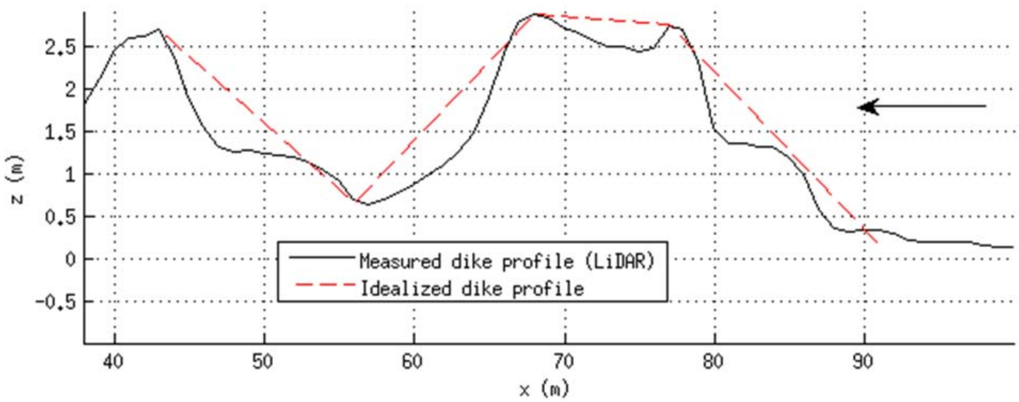


Figure 6 - Location 1b

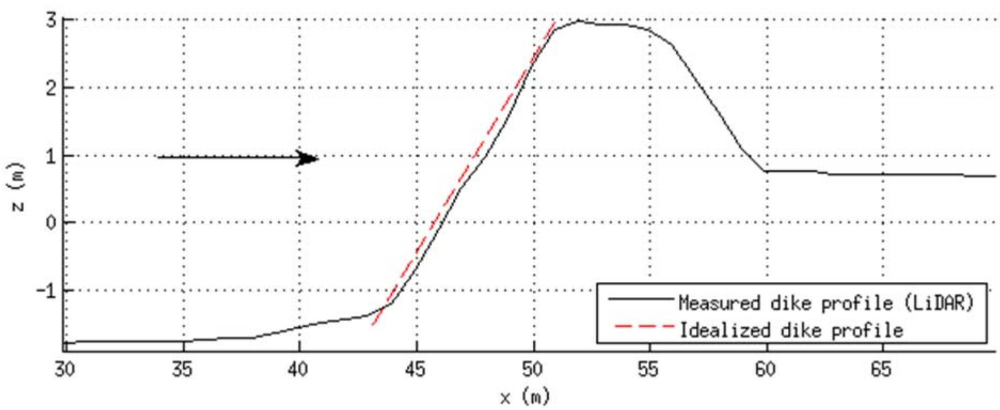


Figure 7 - Location 1c

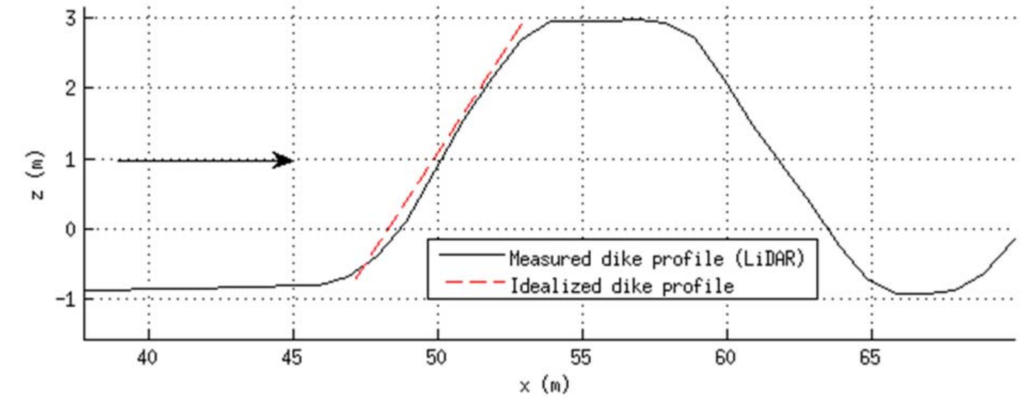


Figure 8 - Location 1d

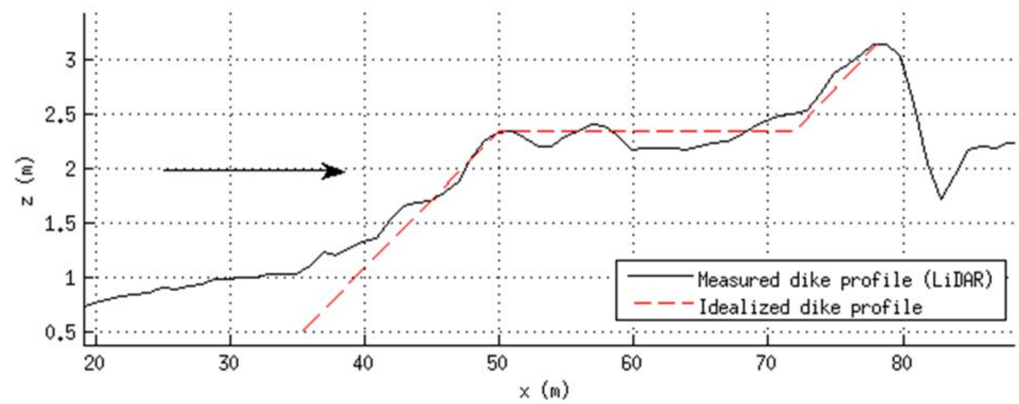


Figure 9 - Location 2

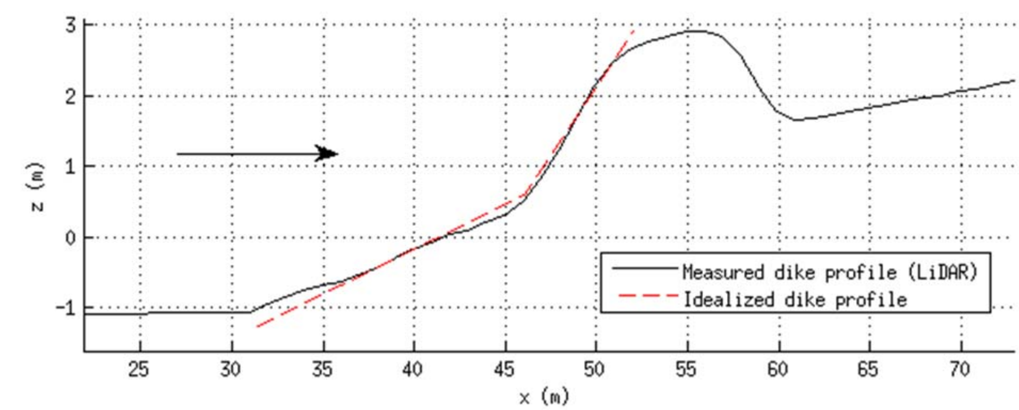


Figure 10 - Location 3

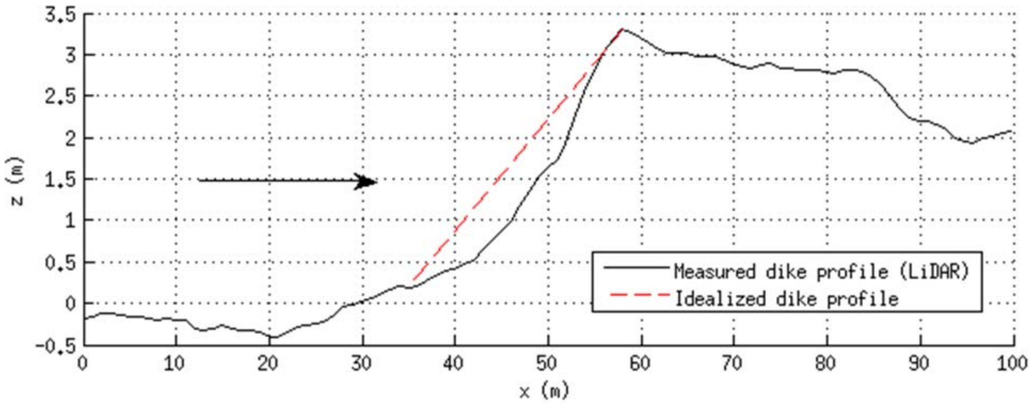


Figure 11 - Location 4

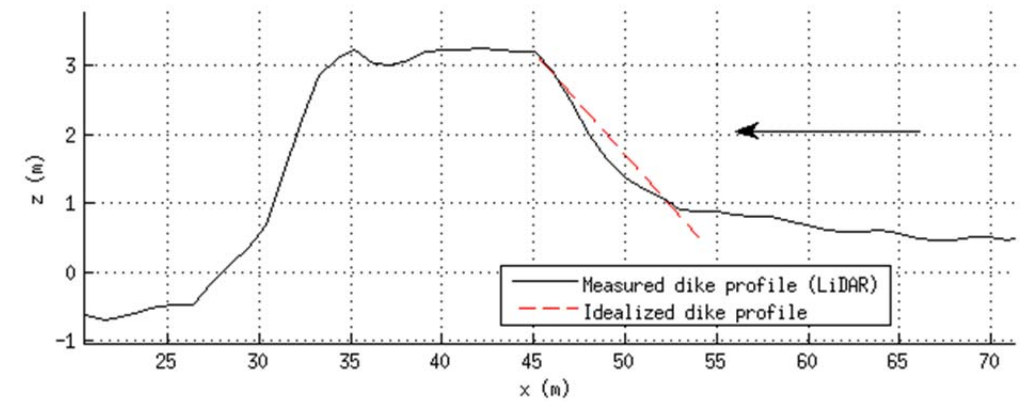


Figure 12 - Location 5

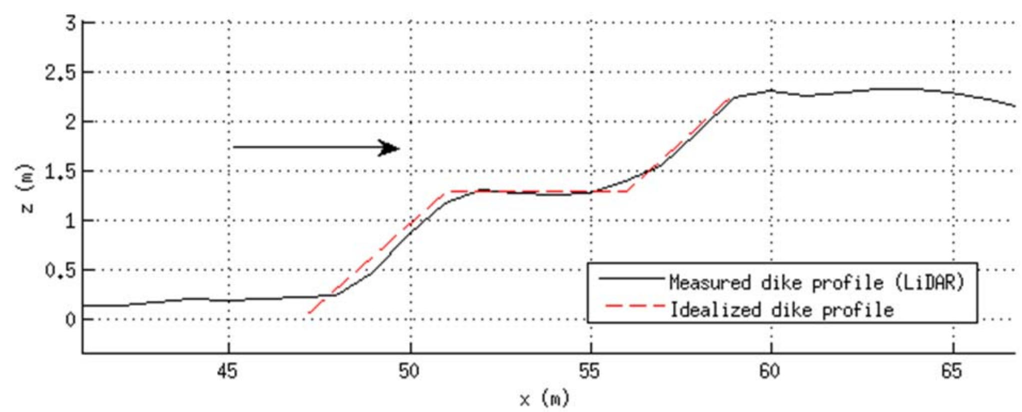


Figure 13 - Location 6

7 APPENDIX B.7. – DEFINITIONS (AUSENCO SANDWELL 2011)

7.1 DESIGNATED FLOOD

A flood, which may occur in any given year, of such a magnitude as to equal a flood having a 200-year recurrence interval based on a frequency analysis of unregulated historic flood records or by regional analysis where there is inadequate streamflow data available. Where the flow of a large watercourse is controlled by a major dam, the designated flood shall be set on a site-specific basis.

In coastal areas, the existing definition of a Designated Flood is not appropriate as the probability of flooding from the sea is the result of the joint occurrence of tide and a storm crossing the coastal waters of British Columbia and at some time in the future, sea level rise due to climate change.

In estuaries, where a river discharges into the sea, the definition of the Designated Flood applies to the river.

In these documents the definition "Designated Flood" is replaced with the term "Designated Storm" as defined below.

7.2 DESIGNATED FLOOD LEVEL (DFL)

The observed or calculated elevation for the Designated Flood and is used in the calculation of the Flood Construction Level.

In coastal areas, the Designated Flood Level (DFL) includes the appropriate allowance for future sea level rise, tide and the total storm surge expected during the designated storm.

Designated Flood Level (DFL)	=		Future SLR Allowance
		+	Maximum Hight Tide (HHWLT)
		+	Total Storm Surge During Designated Storm

7.3 DESIGNATED STORM

A storm, which may occur in any given year, of such a magnitude as to equal a storm having the designated annual exceedence probability (AEP). The Designated Storm has several phenomena associated with it that will define components of the Designated Flood Level, including storm surge, wind set-up, wave run-up and overtopping for the storm. These include:

• *A time series of atmospheric pressure during the passage of the storm over the area in question*

• *A time series of wind speed and direction during the passage of the storm over the area in question*

• *A time series of wave conditions, including wave heights, periods and directions during the passage of the storm in question.*

7.4 FLOOD CONSTRUCTION LEVEL

Uses the Designated Flood Level plus an allowance for Freeboard to establish the elevation of the underside of a wooden floor system or top of concrete slab for habitable buildings. In the case of a manufactured home, the ground level or top of concrete or asphalt pad, on which it is located, shall be equal to or higher than the above described elevation. It also establishes the minimum crest level of a Standard Dike. Where the Designated Flood Level cannot be determined or where there are overriding factors, an assessed height above the natural boundary of the water-body or above the natural ground elevation may be used (as defined in the Land Use Guidelines 2004). In coastal areas the FCL does not relate to the crest level of a sea dike, nor does it relate to the crest level of flood proofing fill exposed directly to the designated flood level. The FCL does; however, include wave – structure interaction effects, to be determined at the location of the site of the building.

Flood Construction Level (FCL) =		Flood Construction Reference Plane (FCRP)
	+	Freeboard

7.5 FLOOD CONSTRUCTION REFERENCE PLANE (FCRP)

A total sea level (tides+surge) event having a 200-year recurrence interval based on a frequency analysis of historic tide station records and/or by regional analysis where there is inadequate data available. Future relative sea level rise is added as appropriate.

Flood Construction Reference Plane (FCRP)	=	Designated Flood Level (DFL)
	+	Estimated Wave Effect

The Estimated Wave Effect may be difficult to specify. The Provincial Guidelines provide the following guidance:

The Estimated Wave Effect can be defined to be 50 per cent of the calculated Wave Run-up on the estimated future shoreline. This percentage is based on an analysis of existing data (2010), described below, and may be revised as more information, including site specific surveys or detailed engineering investigations undertaken by qualified professionals, becomes available. [1]

7.6 FreeBoard

A vertical distance added to the Designated Flood Level. Used to establish the Flood Construction Level.

The Freeboard allowance should be the greater of:
- *0.6m, or;*
- *For flood proofing fill – the crest elevation of an equivalent sea dike (see Sea Dike Guidelines*
- *2010)*
- *For exposed vertical building foundations – the wave-structure interaction;*
- *For tsunami areas – the runup elevation of the appropriate tsunami hazard.*

7.7 Sea Dike Crest Elevation

Sea Dike Crest Elevation has essentially the same meaning as "dike crest height" in the existing document "Dike Design and Construction Guide 2003". However, the existing definition of dike height suggests that consideration of wave run-up and set-up is optional. The term Sea Dike Crest Elevation is defined to specifically cover scenarios where wave run- up, overtopping and wind and wave setup must be included in defining the height of the dike.

Dike Crest Elevation	=		Designated Flood Level (DFL)
		+	Wave Runup
		+	Freeboard

Reference

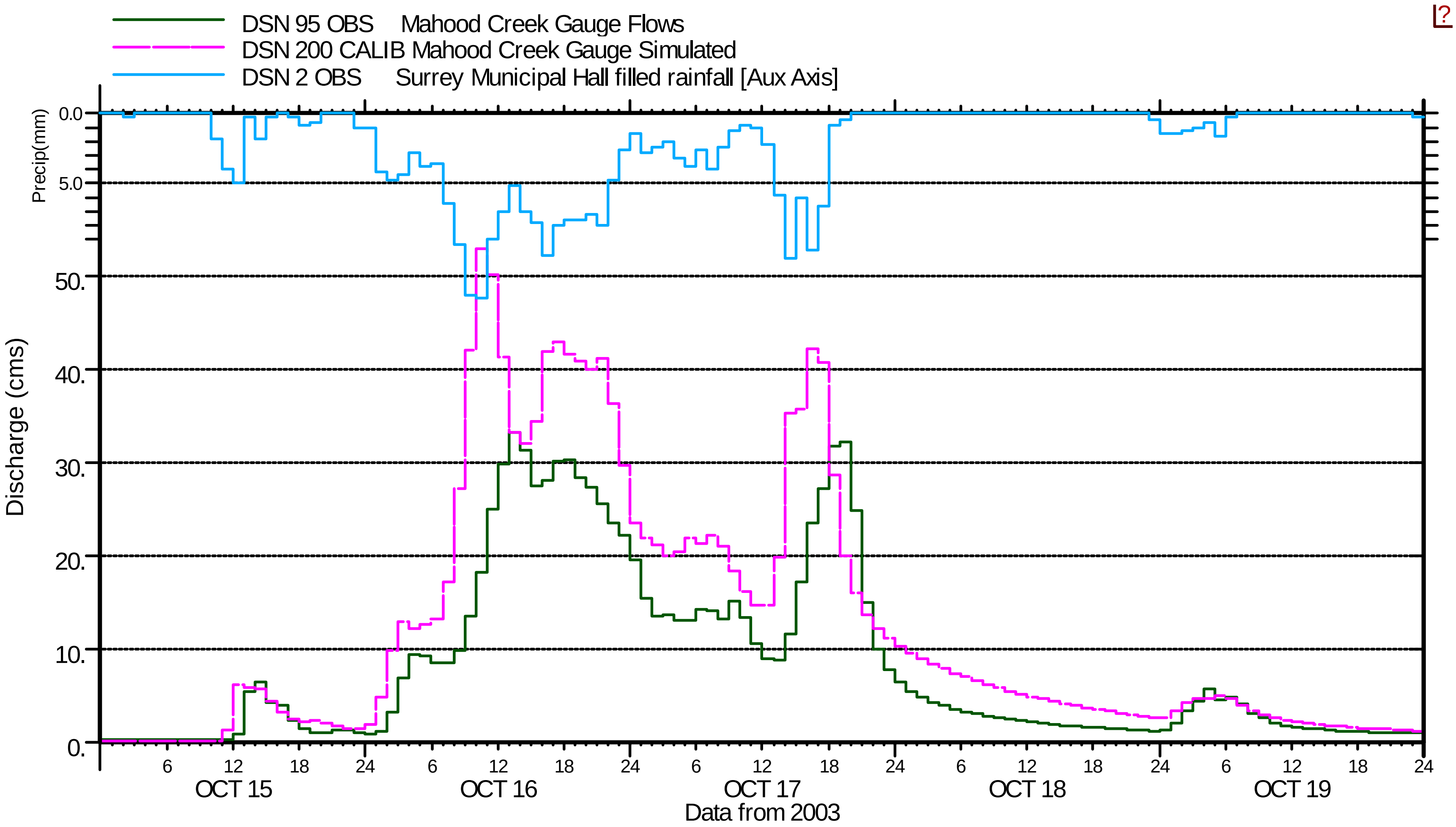
[1] Ausenco Sandwell (2011), 'Guidelines for Management of Coastal Flood Hazard Land Use', Technical report, BC Ministry of Environment.

Appendix C

Hydrologic Analysis

Appendix C.1

HSPF Model Calibration Results
Mahood Creek at 144 Street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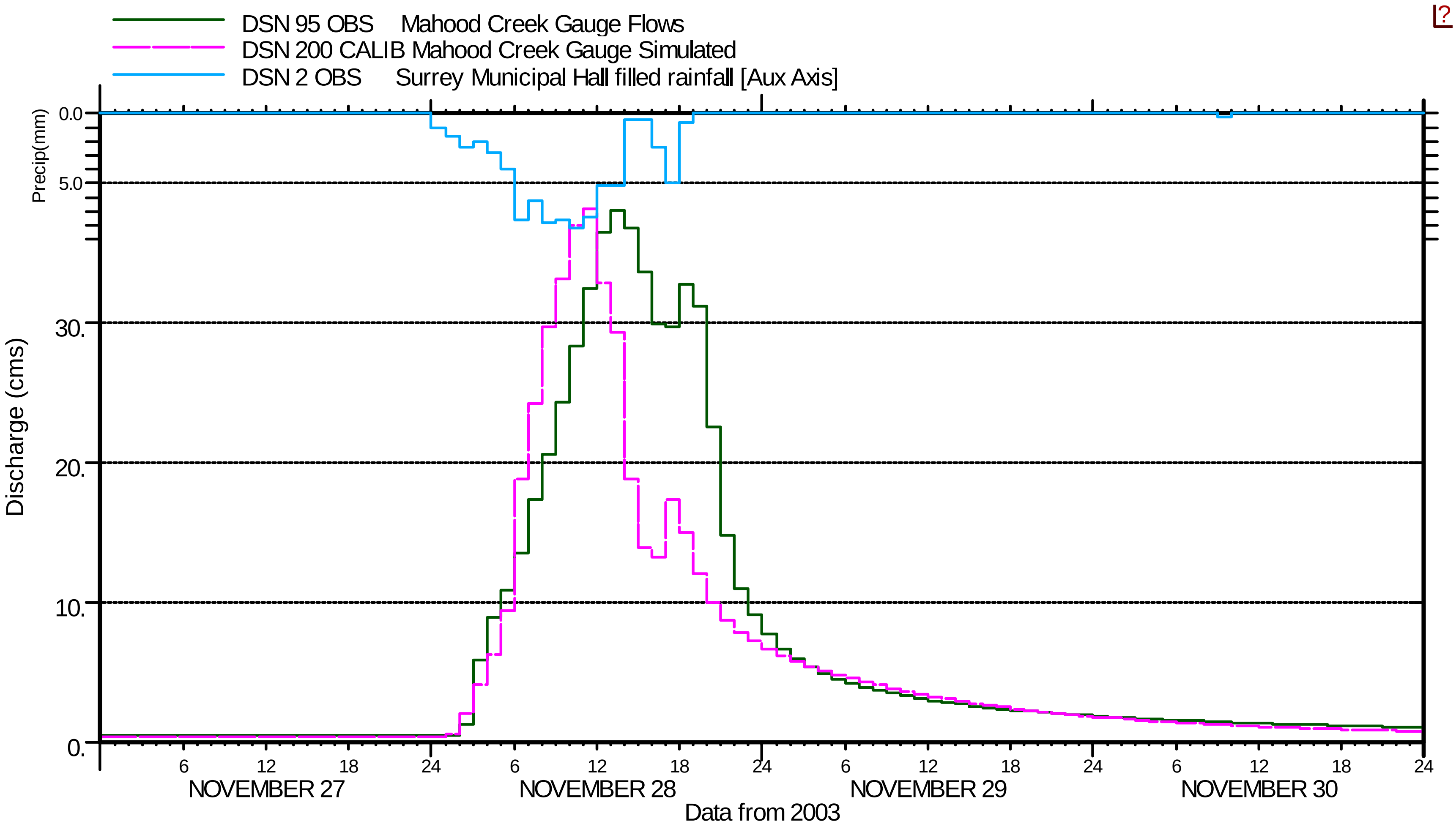
DSN 200 CALIB Mahood Creek Gauge Simulated
DSN 2 OBS Surrey Municipal Hall filled rainfall [Aux Axis]
Precip(mm)
0.0
5.0
Discharge (cms)
50.
40.
30.
20.
10.
0.
6
12
18
24
OCT 15
OCT 16
OCT 17
OCT 18
OCT 19
Data from 2003



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DSN 200 CALIB Mahood Creek Gauge Simulated
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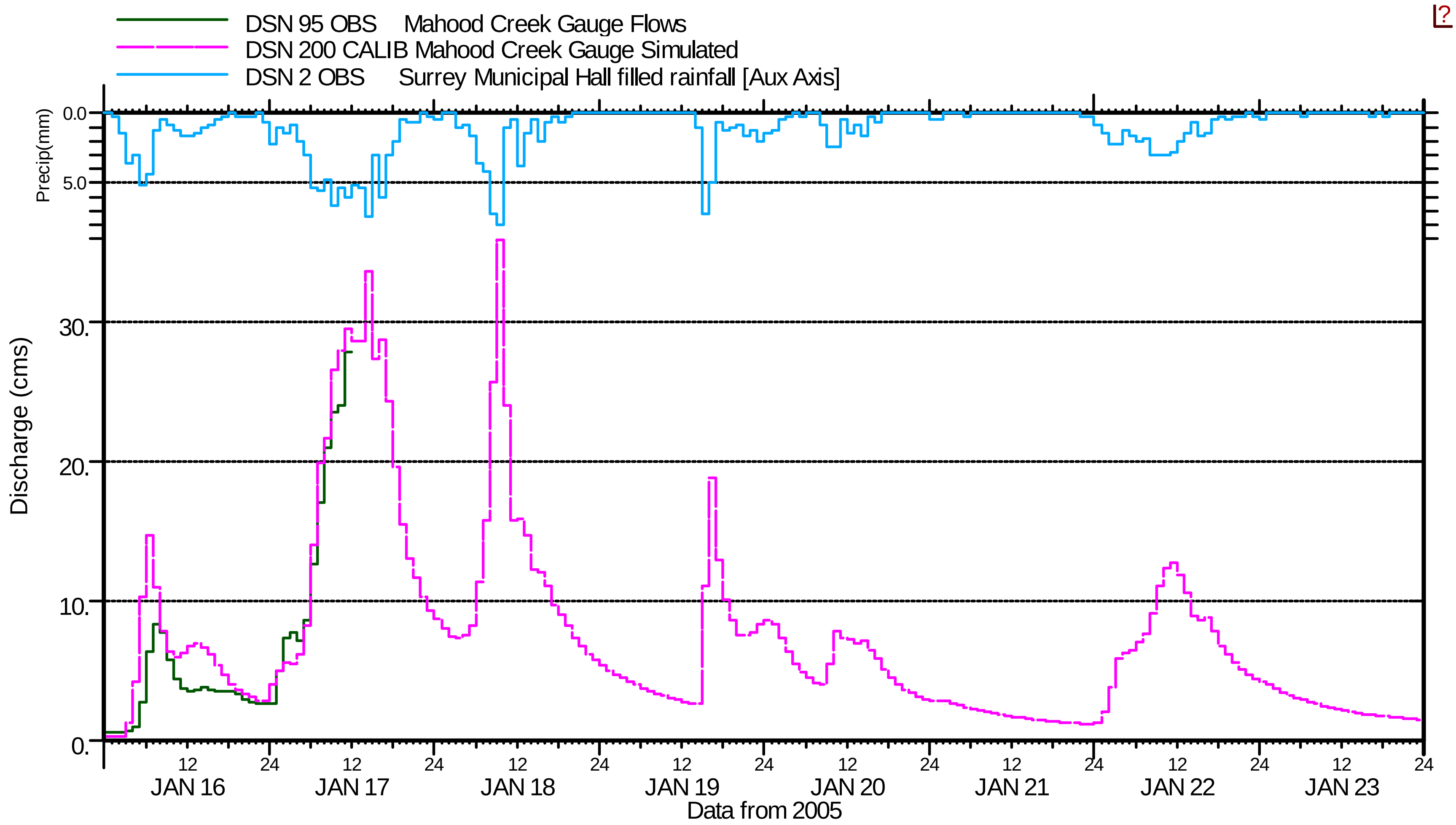
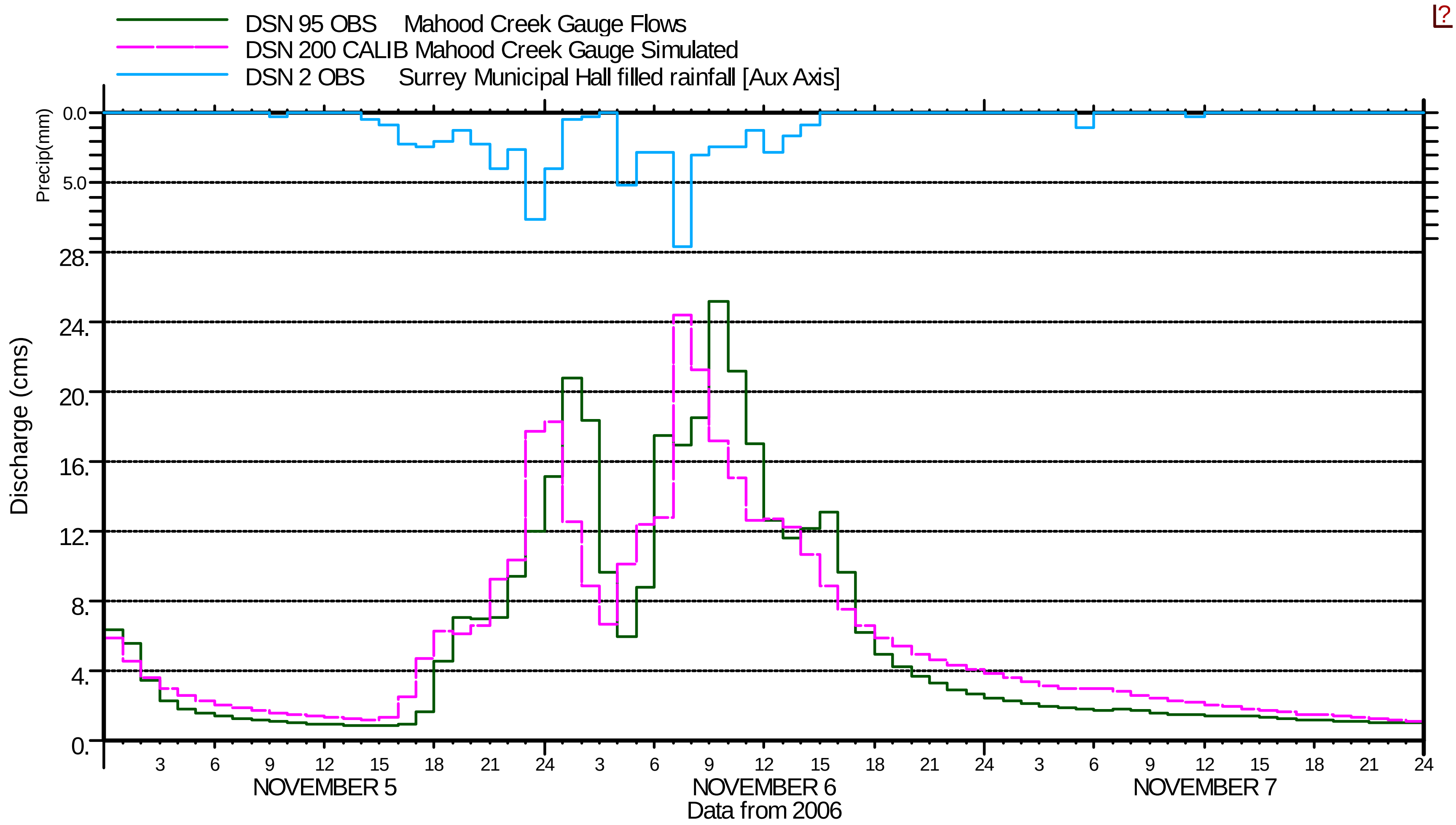
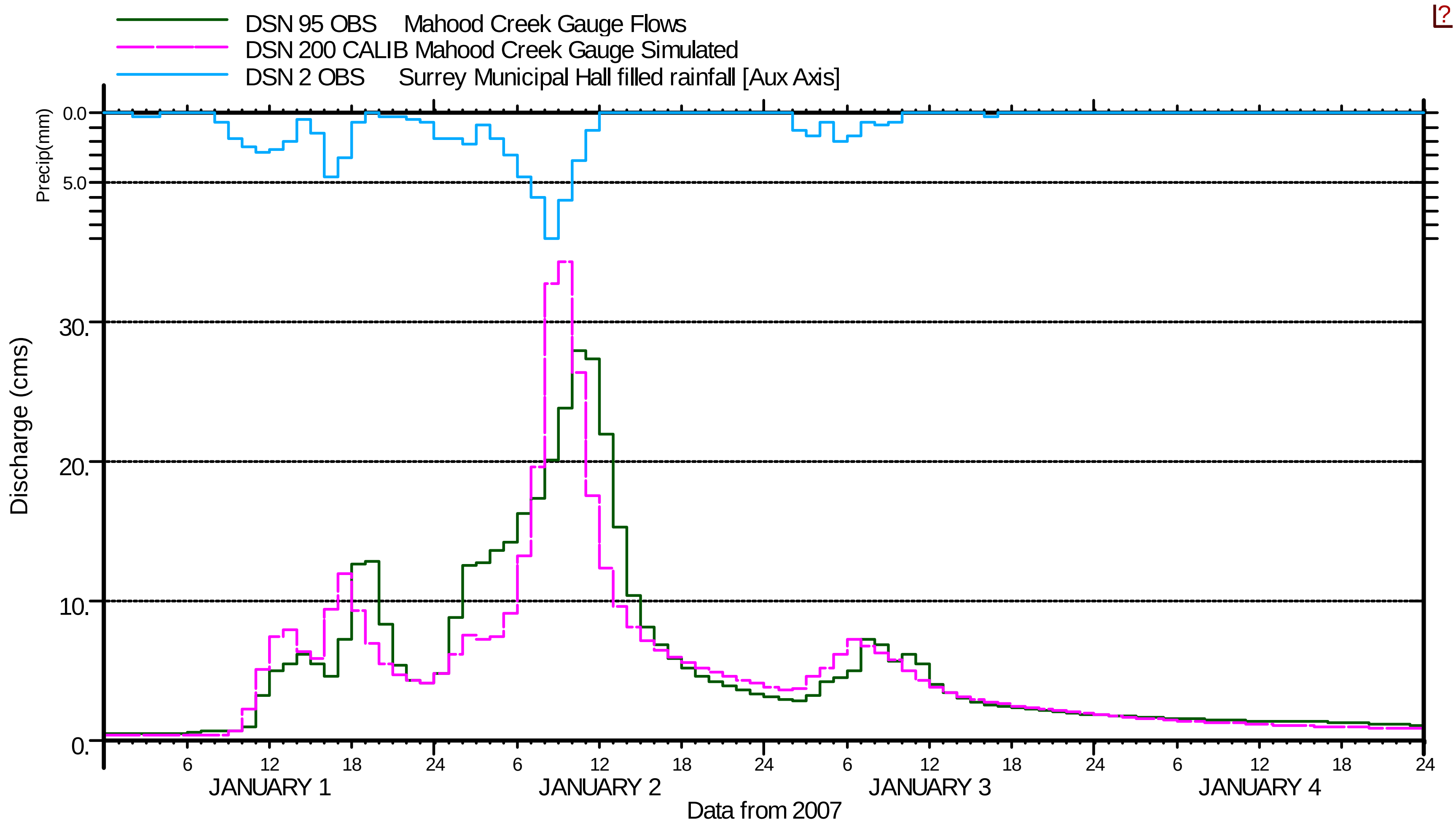
Precip(mm)
0.0
5.0
Discharge (cms)
30.
20.
10.
0.
6
12
18
24
NOVEMBER 27
NOVEMBER 28
NOVEMBER 29
NOVEMBER 30
Data from 2003



DSN 95 OBS Mahood Creek Gauge Flows
DSN 200 CALIB Mahood Creek Gauge Simulated
DSN 2 OBS Surrey Municipal Hall filled rainfall [Aux Axis]
Precip(mm)
0.0
5.0
Discharge (cms)
30.
20.
10.
0.
12
24
JAN 16
JAN 17
JAN 18
JAN 19
JAN 20
JAN 21
JAN 22
JAN 23
Data from 2005


DSN 95 OBS Mahood Creek Gauge Flows
DSN 200 CALIB Mahood Creek Gauge Simulated
DSN 2 OBS Surrey Municipal Hall filled rainfall [Aux Axis]
Precip(mm)
0.0
5.0
Discharge (cms)
28.
24.
20.
16.
12.
8.
4.
0.
3 6 9 12 15 18 21 24 3 6 9 12 15 18 21 24 3 6 9 12 15 18 21 24
NOVEMBER 5
NOVEMBER 6
NOVEMBER 7
Data from 2006



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DSN 200 CALIB Mahood Creek Gauge Simulated
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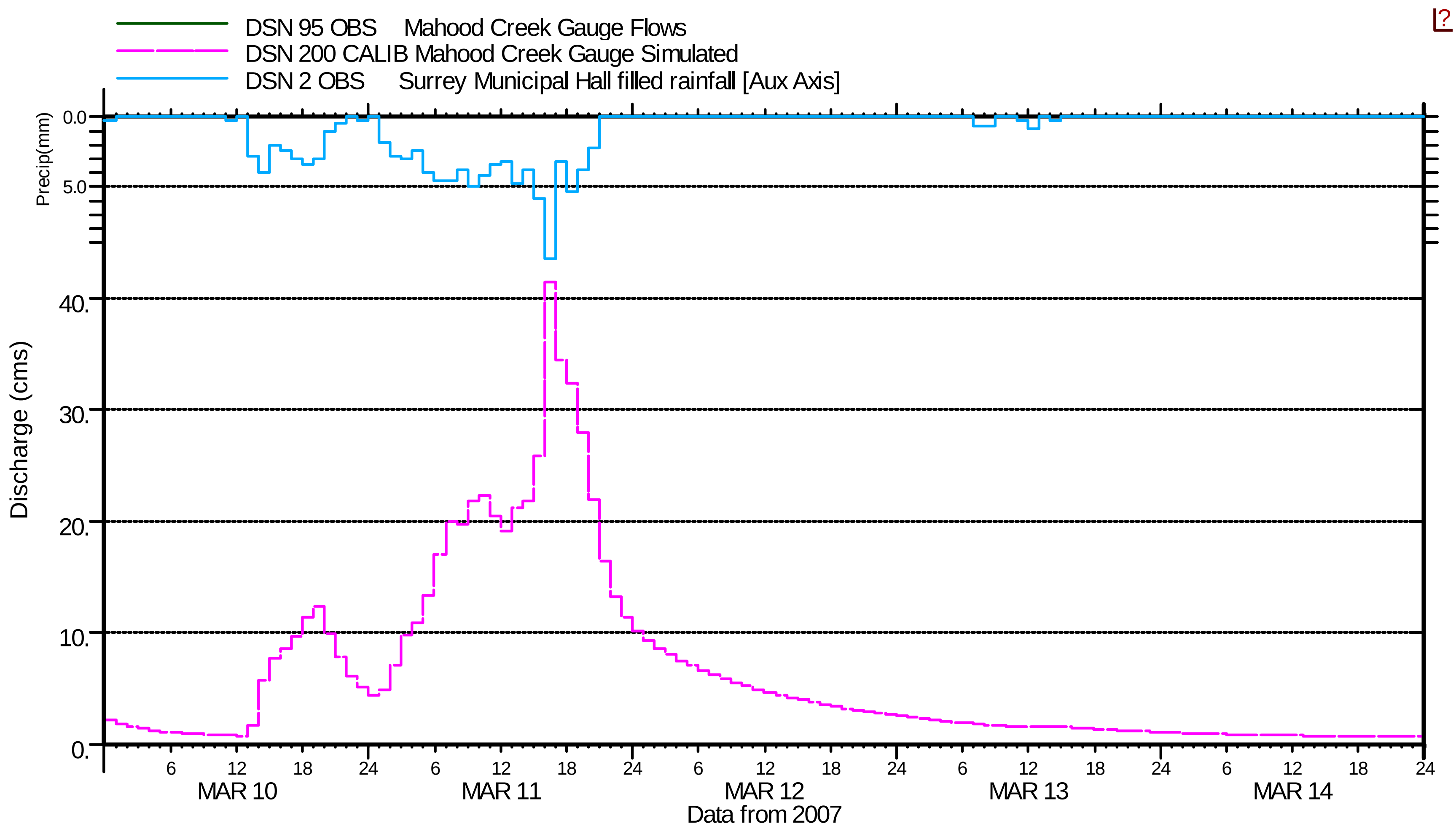
Precip(mm)
0.0
5.0
Discharge (cms)
30.
20.
10.
0.
6
12
18
24
JANUARY 1
JANUARY 2
JANUARY 3
JANUARY 4
Data from 2007



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Precip(mm)
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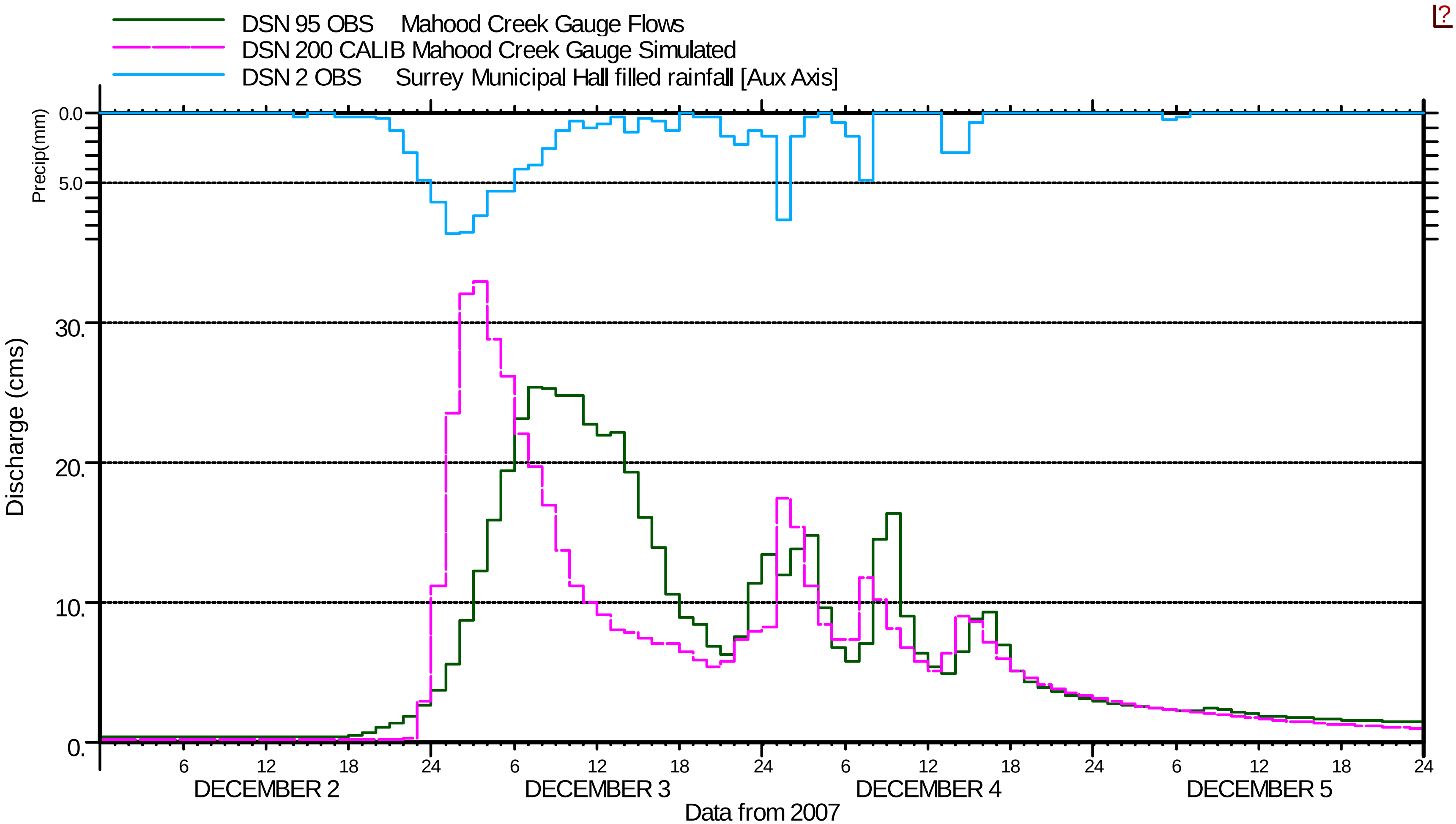
5.0
Discharge (cms)
40.
30.
20.
10.
0.
6
12
18
24
MAR 10
MAR 11
MAR 12
MAR 13
MAR 14
Data from 2007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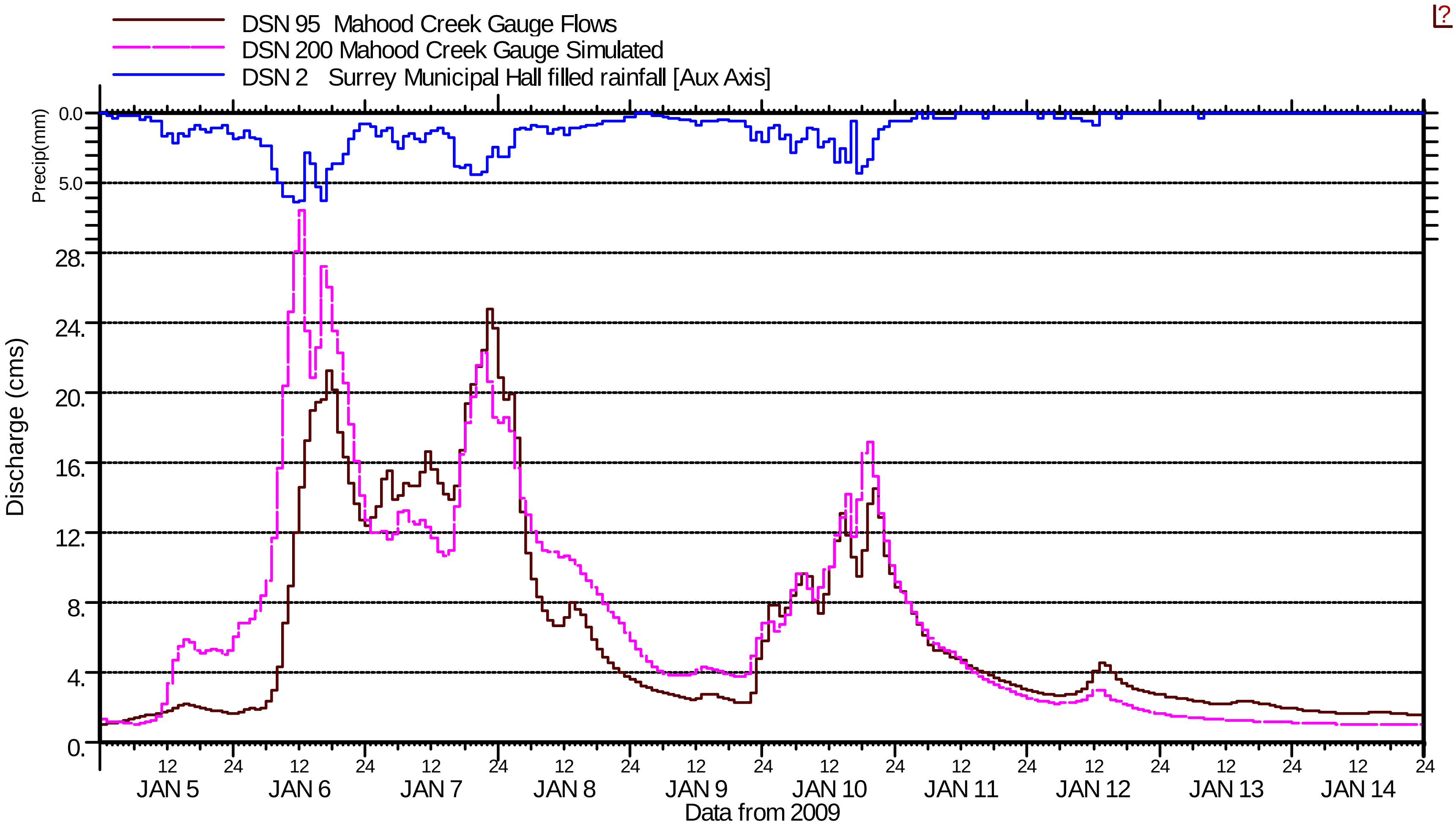
DSN 200 CALIB Mahood Creek Gauge Simulated
DSN 2 OBS Surrey Municipal Hall filled rainfall [Aux Axis]
Precip(mm)
0.0
5.0
Discharge (cms)
30.
20.
10.
0.
6
12
18
24
DECEMBER 2
DECEMBER 3
DECEMBER 4
DECEMBER 5
Data from 2007



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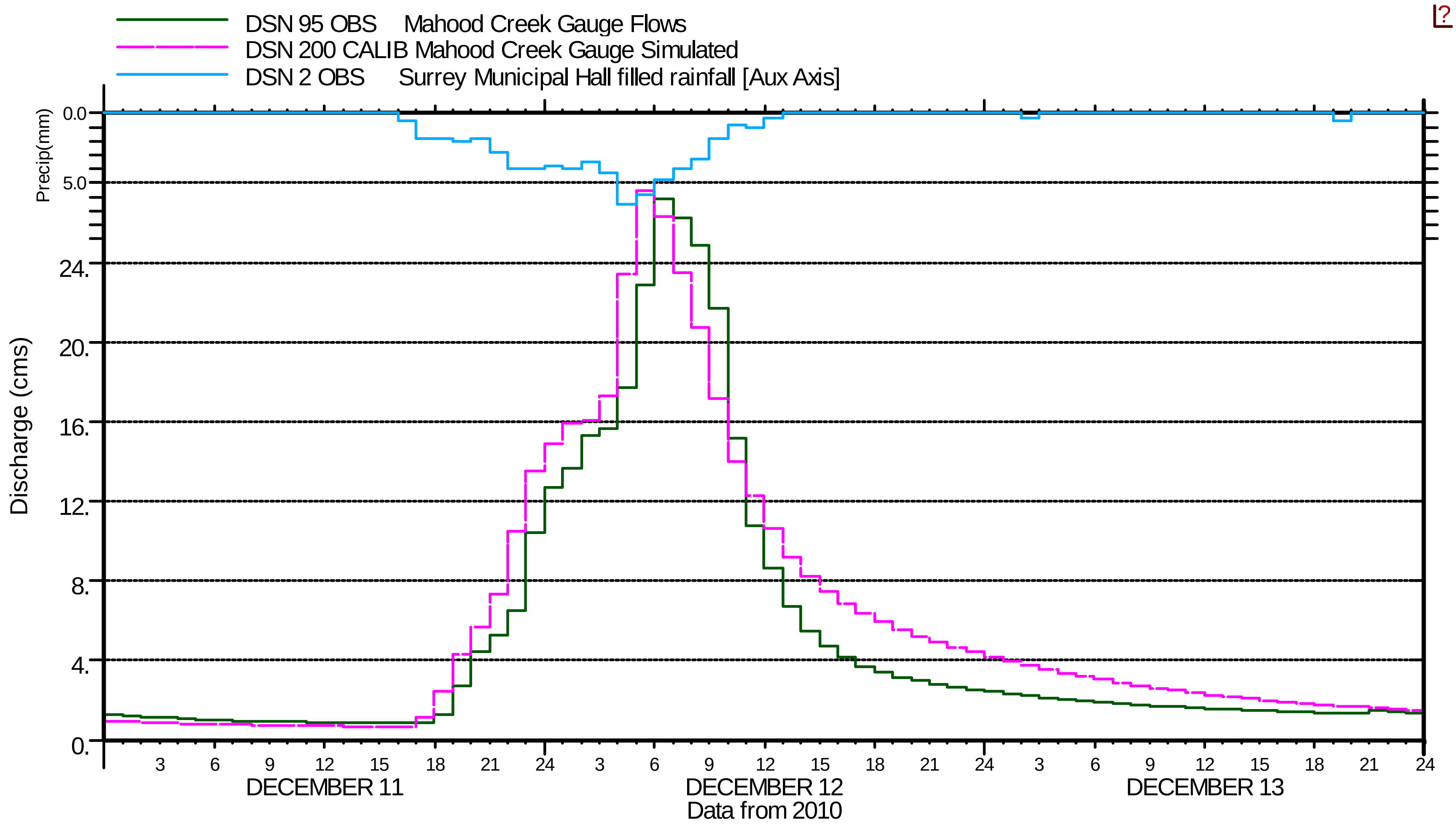
5.0
Discharge (cms)
28.
24.
20.
16.
12.
8.
4.
0.
12 24 12 24 12 24 12 24 12 24 12 24 12 24 12 24 12 24 12 24
JAN 5
JAN 6
JAN 7
JAN 8
JAN 9
JAN 10
JAN 11
JAN 12
JAN 13
JAN 14
Data from 2009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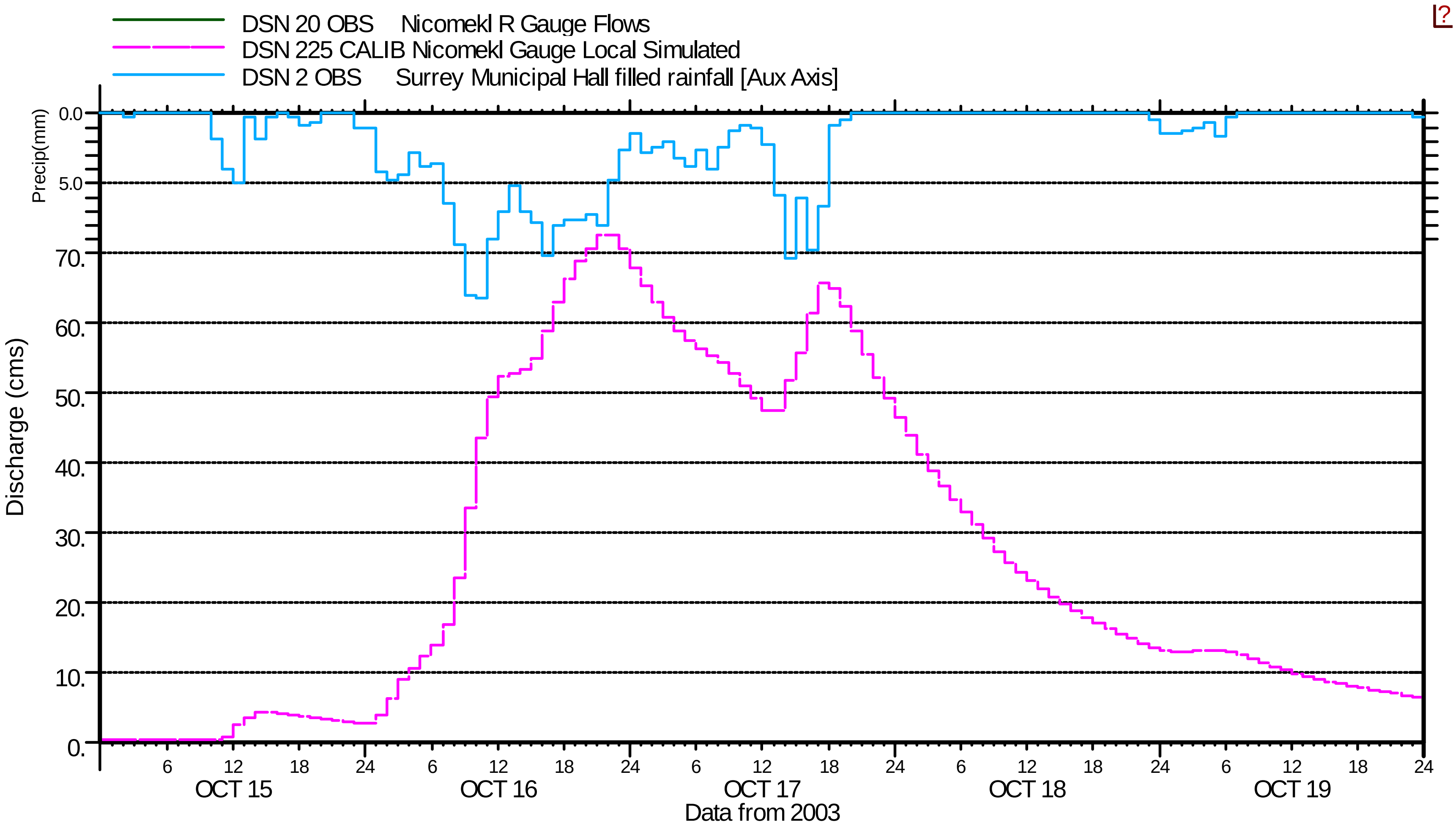
5.0
Discharge (cms)
24.
20.
16.
12.
8.
4.
0.
3 6 9 12 15 18 21 24 3 6 9 12 15 18 21 24 3 6 9 12 15 18 21 24
DECEMBER 11
DECEMBER 12
DECEMBER 13
Data from 2010

Appendix C.2

HSPF Model Calibration Results
Nicomekl River at 203 Street



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Discharge (cms)
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60.
50.
40.
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10.
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OCT 15
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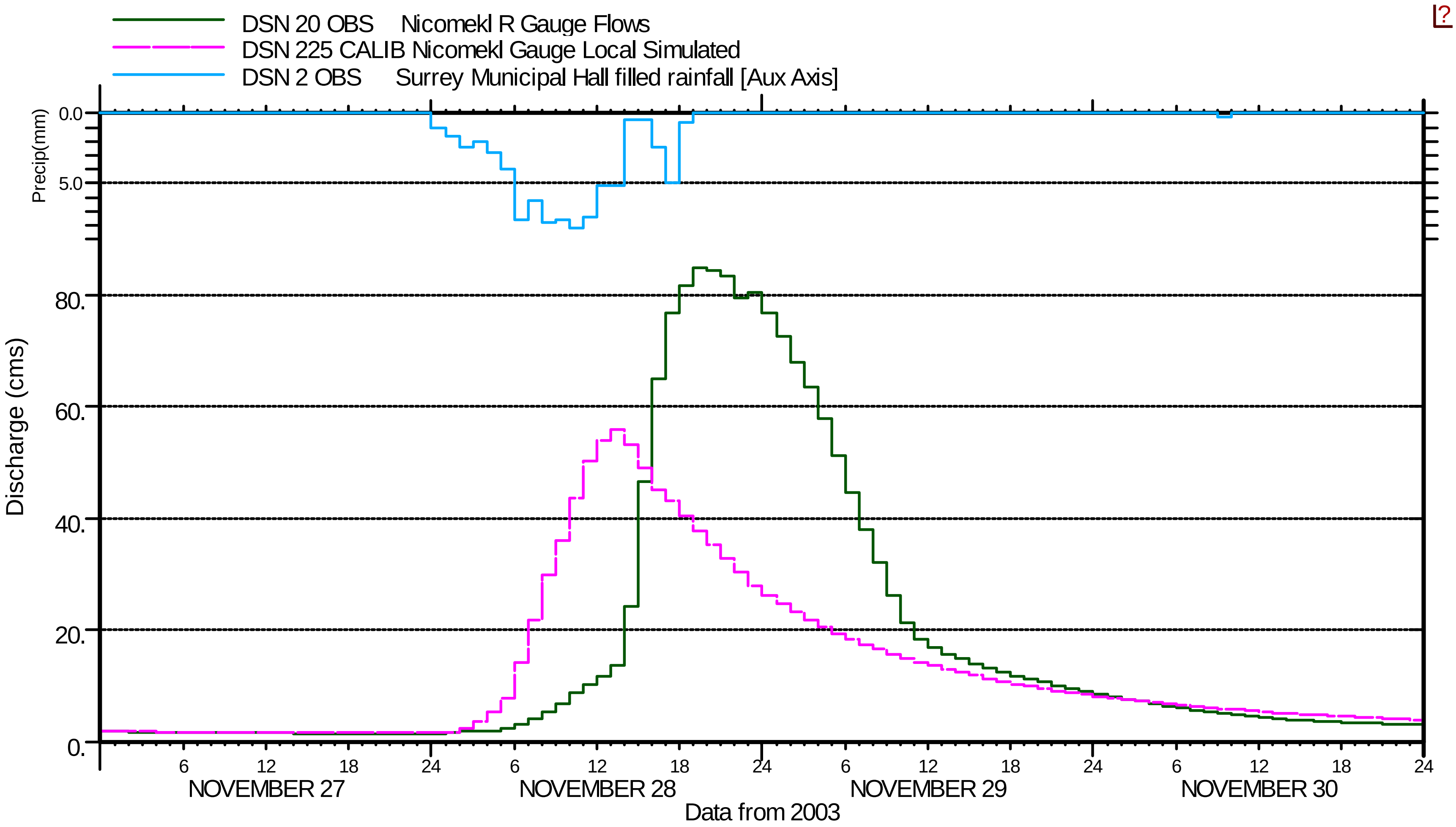
OCT 18
OCT 19
Data from 2003



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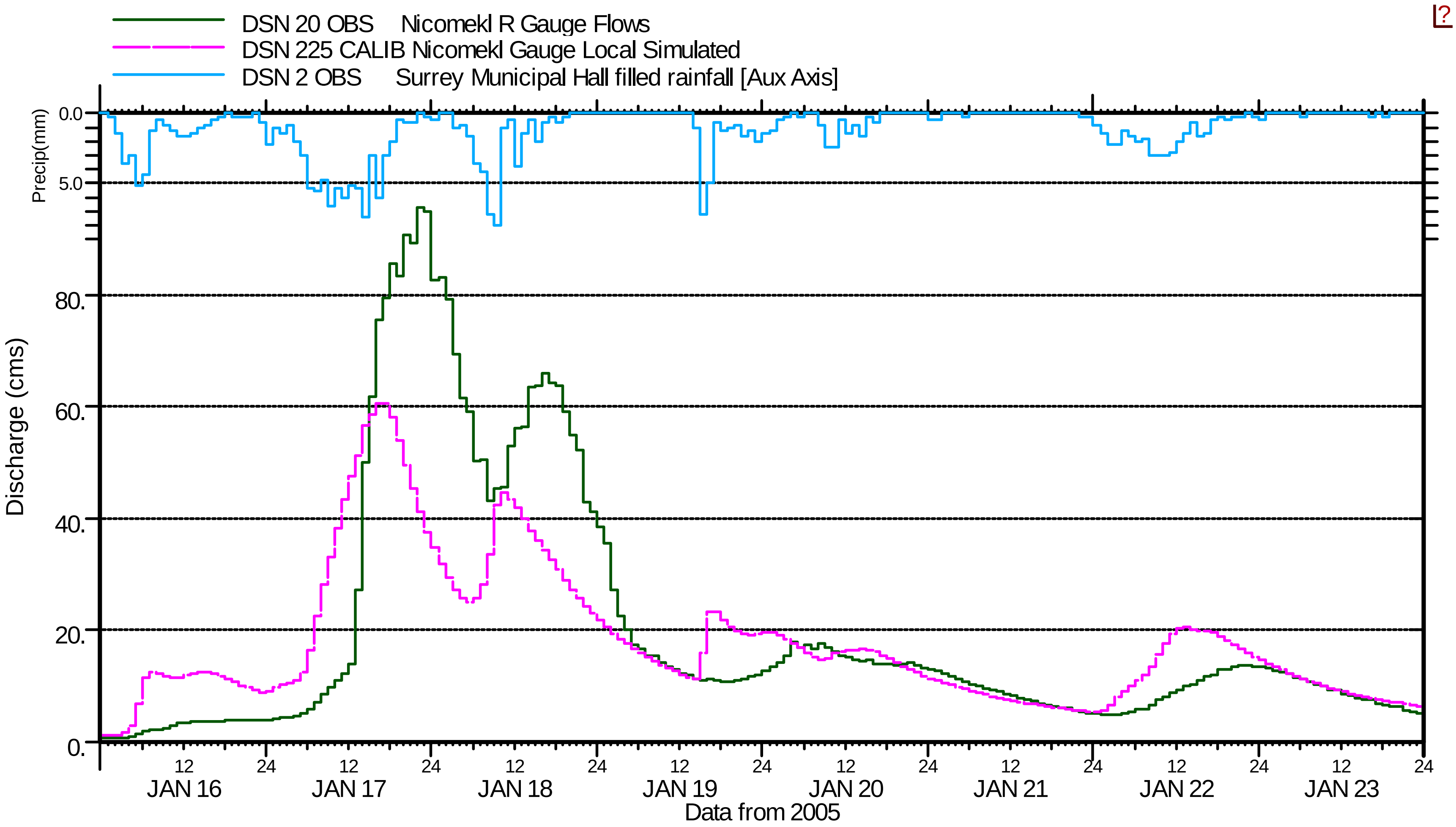
Precip(mm)
0.0
5.0
Discharge (cms)
80.
60.
40.
20.
0.
6
12
18
24
NOVEMBER 27
NOVEMBER 28
NOVEMBER 29
NOVEMBER 30
Data from 2003



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Precip(mm)
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5.0
Discharge (cms)
80.
60.
40.
20.
0.
12
24
JAN 16
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JAN 18
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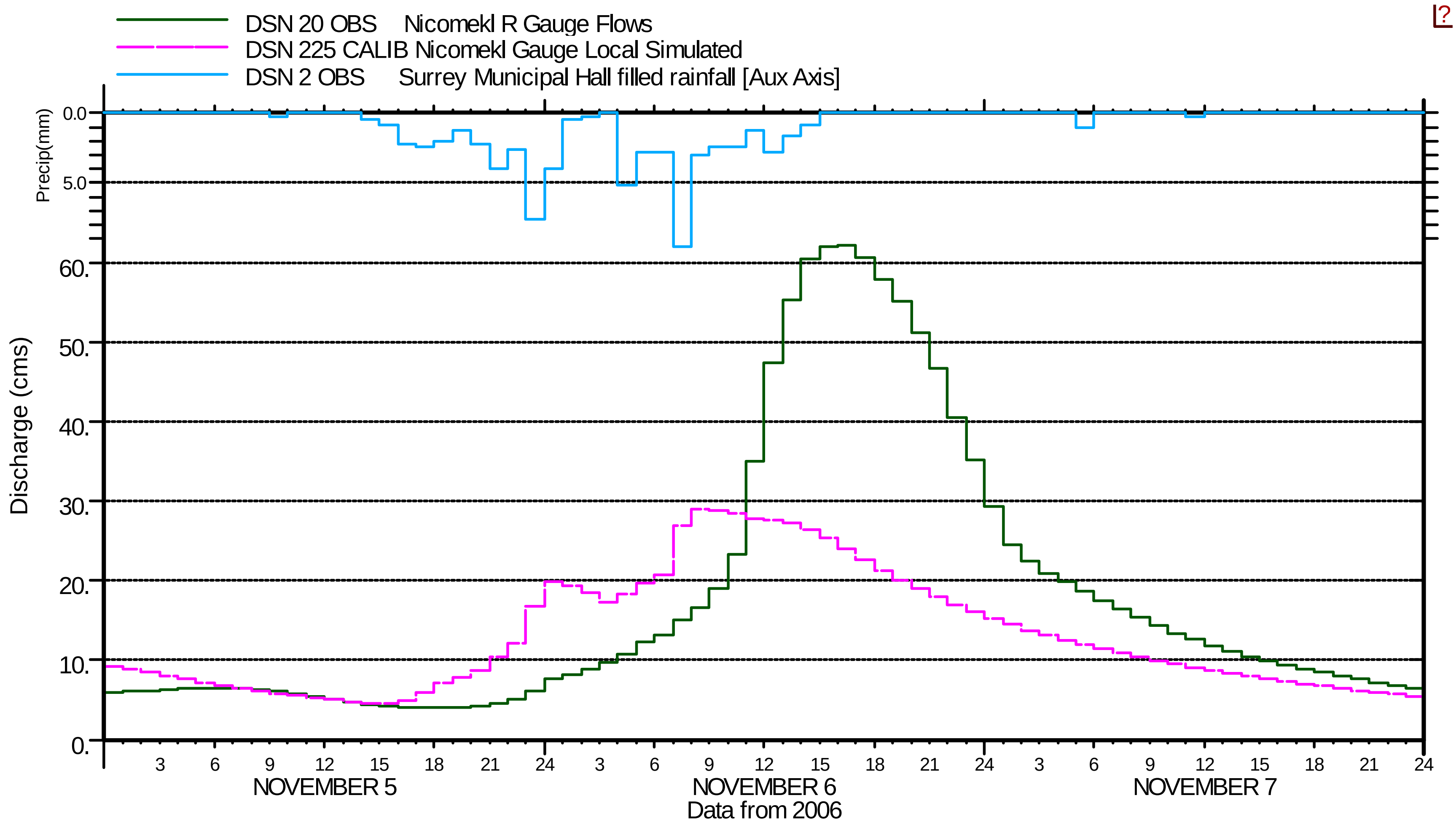
Data from 2005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Precip(mm)
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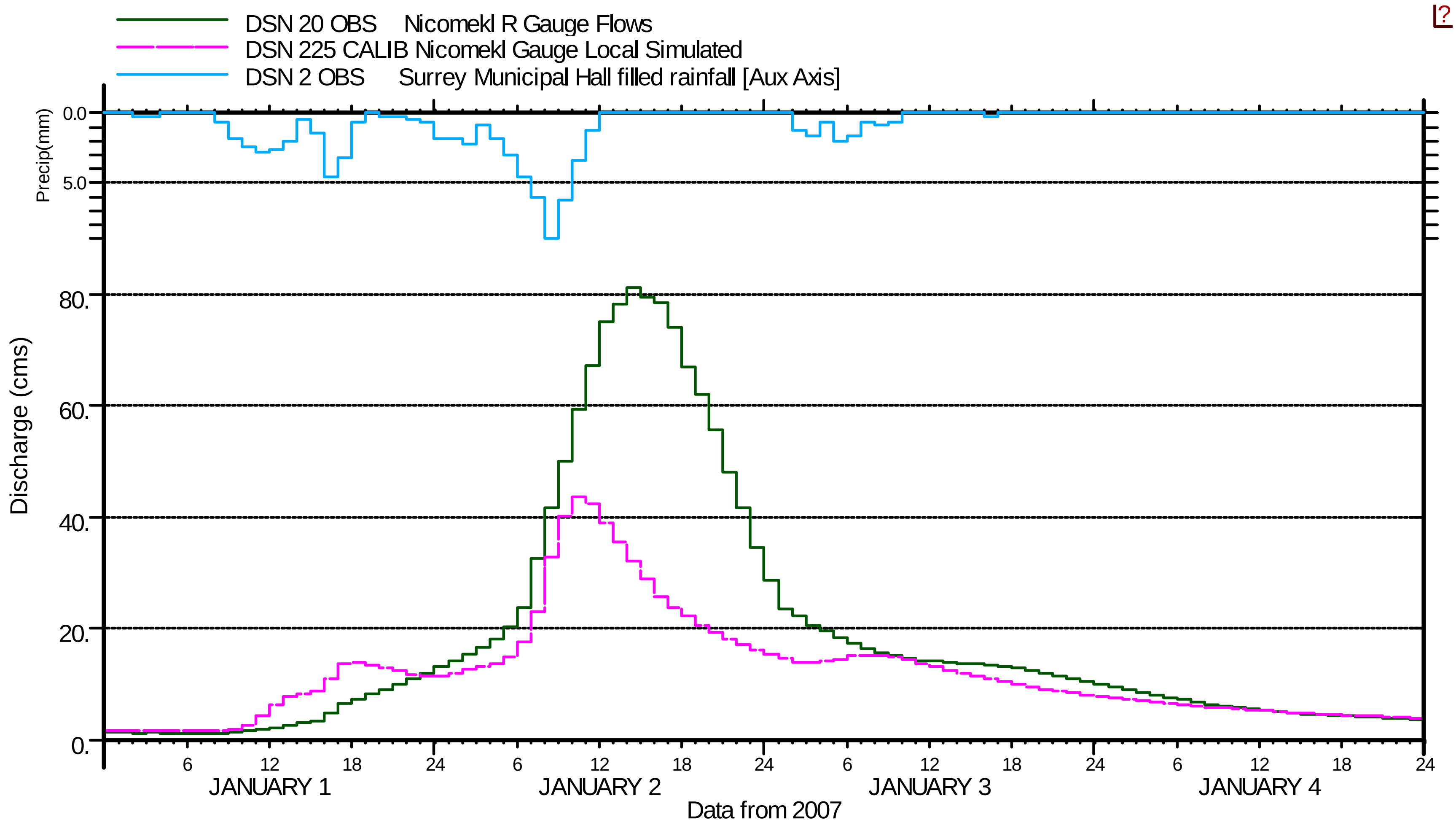
5.0
Discharge (cms)
60.
50.
40.
30.
20.
10.
0.
3 6 9 12 15 18 21 24 3 6 9 12 15 18 21 24 3 6 9 12 15 18 21 24
NOVEMBER 5
NOVEMBER 6
NOVEMBER 7
Data from 2006



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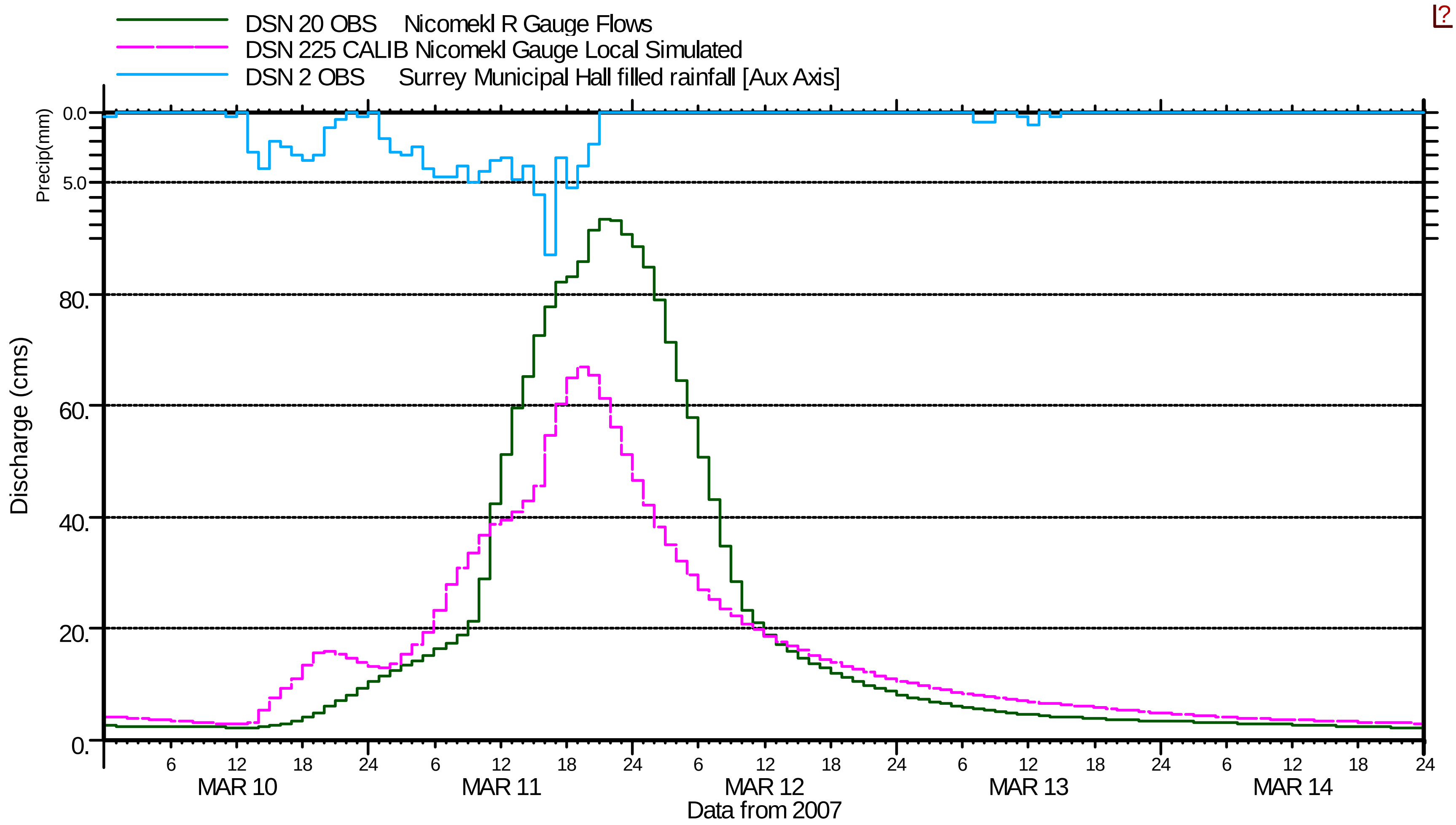
Precip(mm)
0.0
5.0
Discharge (cms)
80.
60.
40.
20.
0.
6
12
18
24
JANUARY 1
JANUARY 2
JANUARY 3
JANUARY 4
Data from 2007



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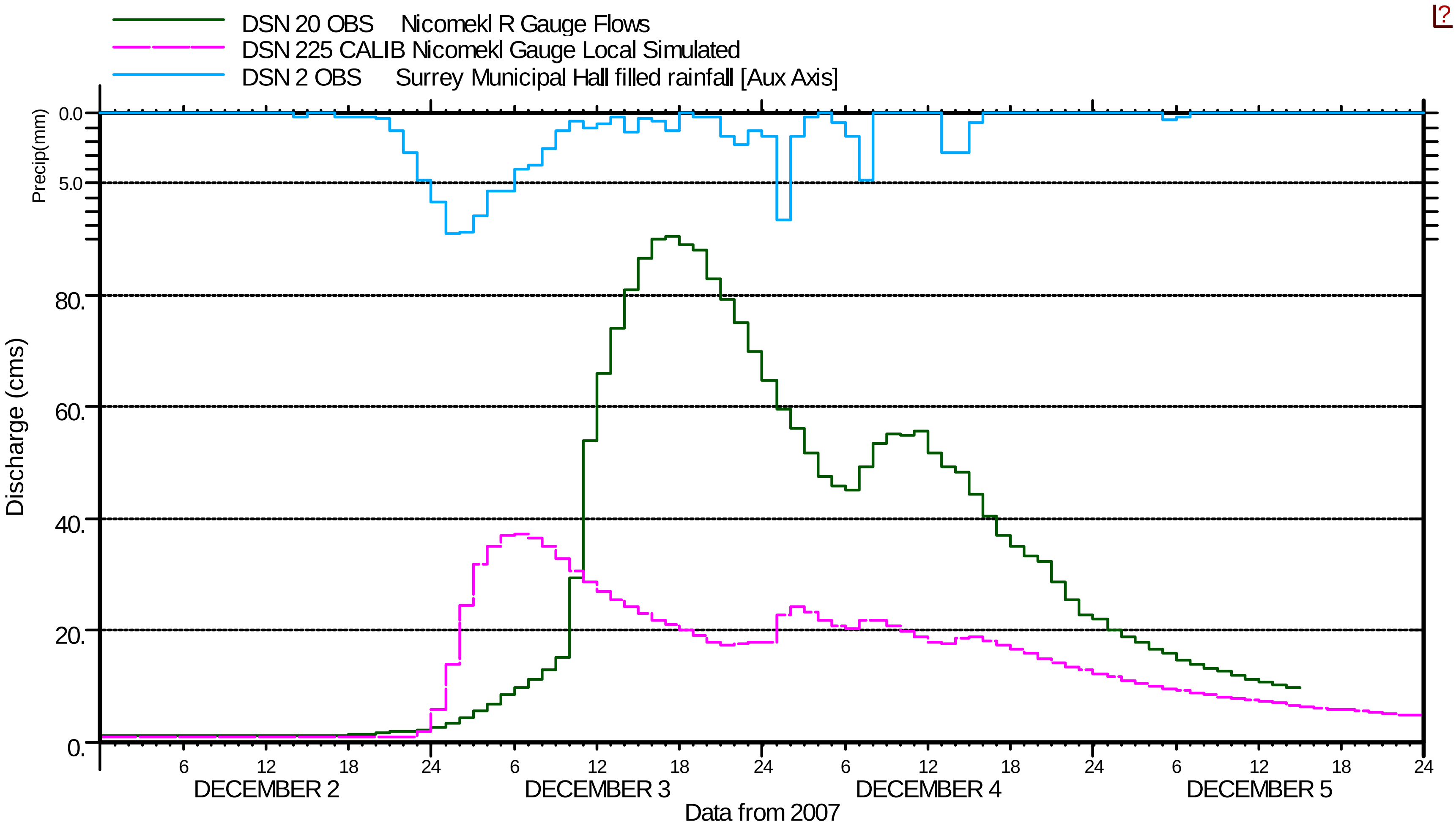
Precip(mm)
0.0
5.0
Discharge (cms)
80.
60.
40.
20.
0.
6
12
18
24
MAR 10
MAR 11
MAR 12
MAR 13
MAR 14
Data from 2007



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DSN 225 CALIB Nicomekl Gauge Local Simulated
DSN 2 OBS Surrey Municipal Hall filled rainfall [Aux Axis]
Precip(mm)
0.0
5.0
Discharge (cms)
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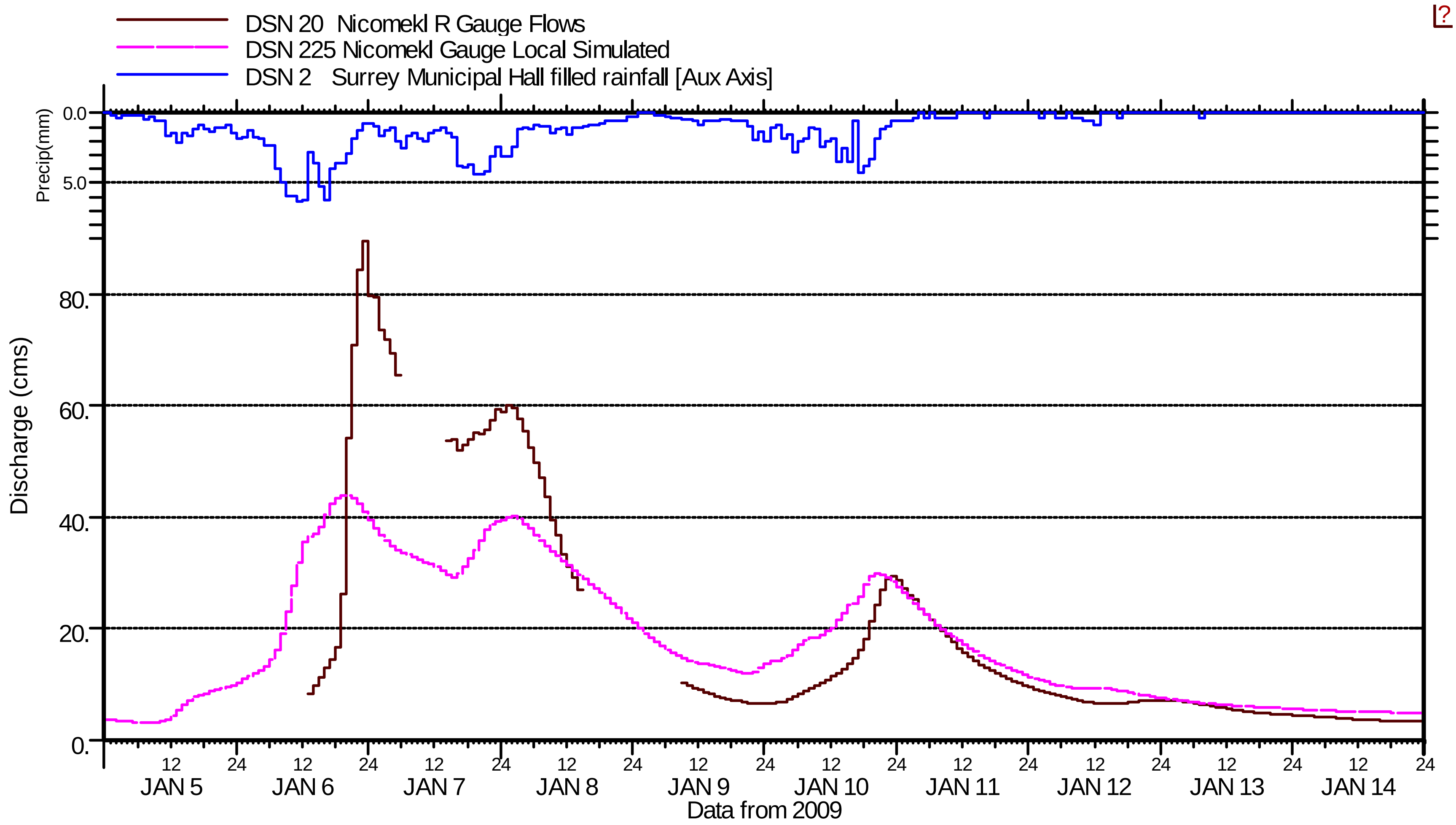
60.
40.
20.
0.
6
12
18
24
DECEMBER 2
DECEMBER 3
DECEMBER 4
DECEMBER 5
Data from 2007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DSN 225 Nicomekl Gauge Local Simulated
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Precip(mm)
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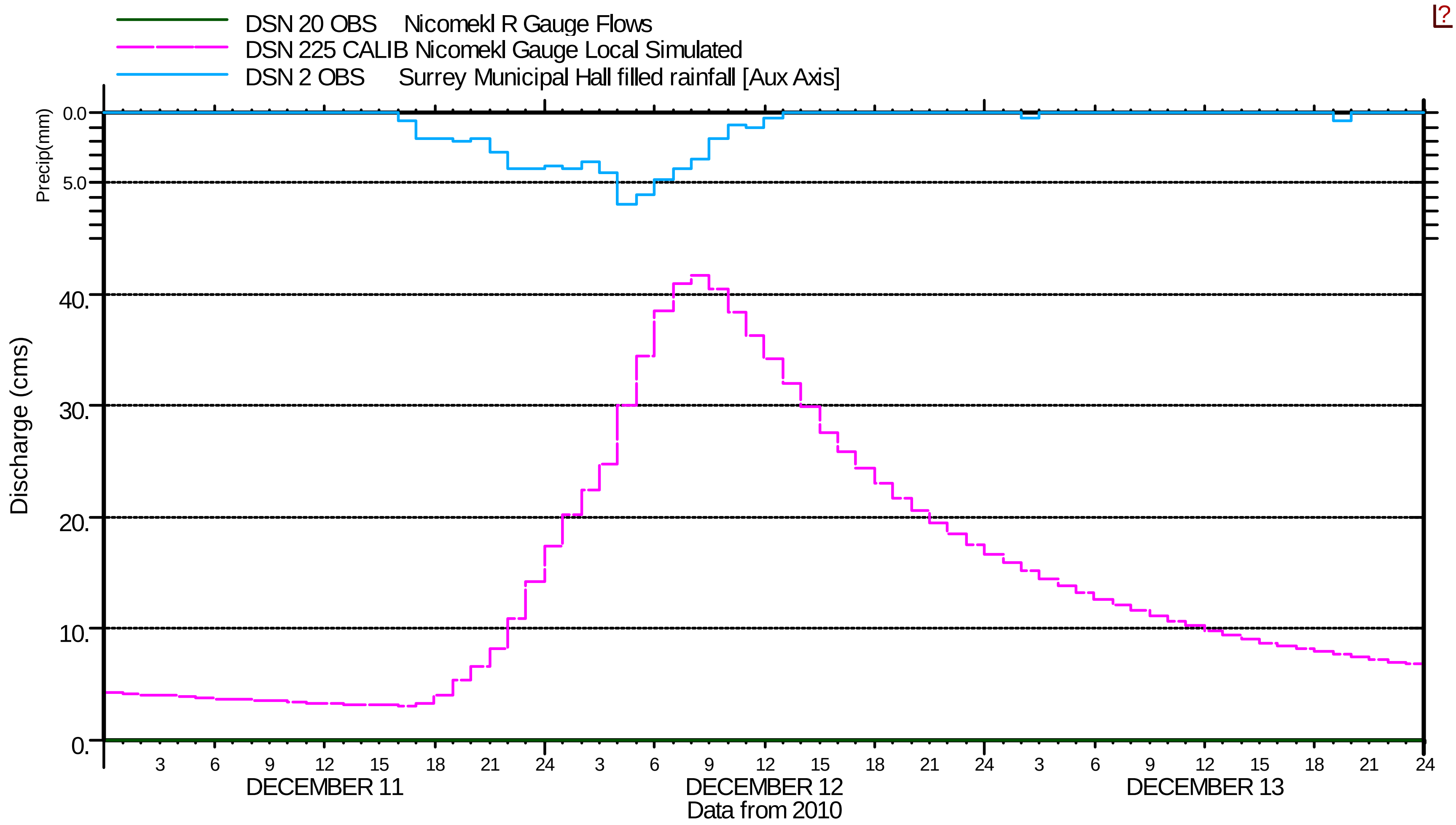
5.0
Discharge (cms)
80.
60.
40.
20.
0.
12
24
JAN 5
JAN 6
JAN 7
JAN 8
JAN 9
JAN 10
JAN 11
JAN 12
JAN 13
JAN 14
Data from 2009



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DSN 2 OBS Surrey Municipal Hall filled rainfall [Aux Axis]
Precip(mm)
0.0
5.0
Discharge (cms)
40.
30.
20.
10.
0.
3
6
9
12
15
18
21
24
DECEMBER 11
DECEMBER 12
DECEMBER 13
Data from 2010

APPENDIX D

HYDRAULIC ANALYSIS

TABLE OF CONTENTS

1 APPENDIX D.1. – LONGITUDINAL PROFILES

Longitudinal profiles of the Serpentine and Nicomekl Rivers as well as the floodplain cells were plotted for a series of HEC-RAS model simulations including:

- January 2009 validation
- January 2005 validation
- Sensitivity analyses
- Impacts of sea level rise

VALIDATION: JANUARY 2009

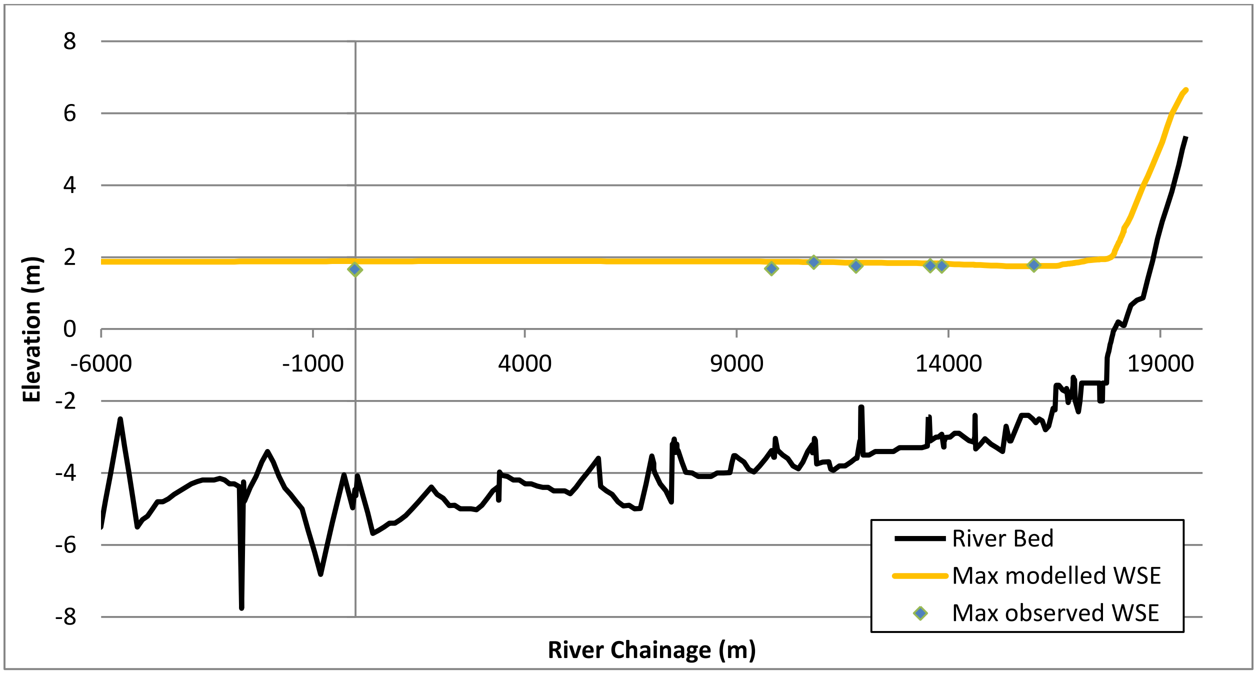


Figure 1: Serpentine River Longitudinal Profile (Jan 2009).

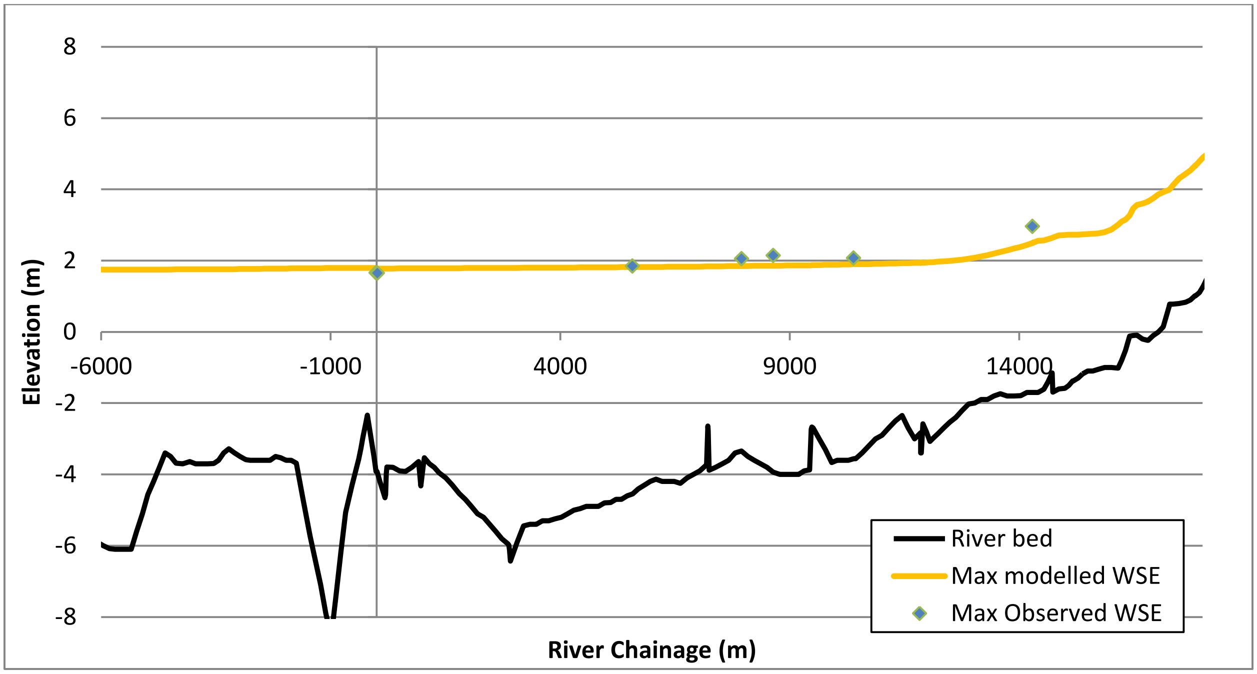
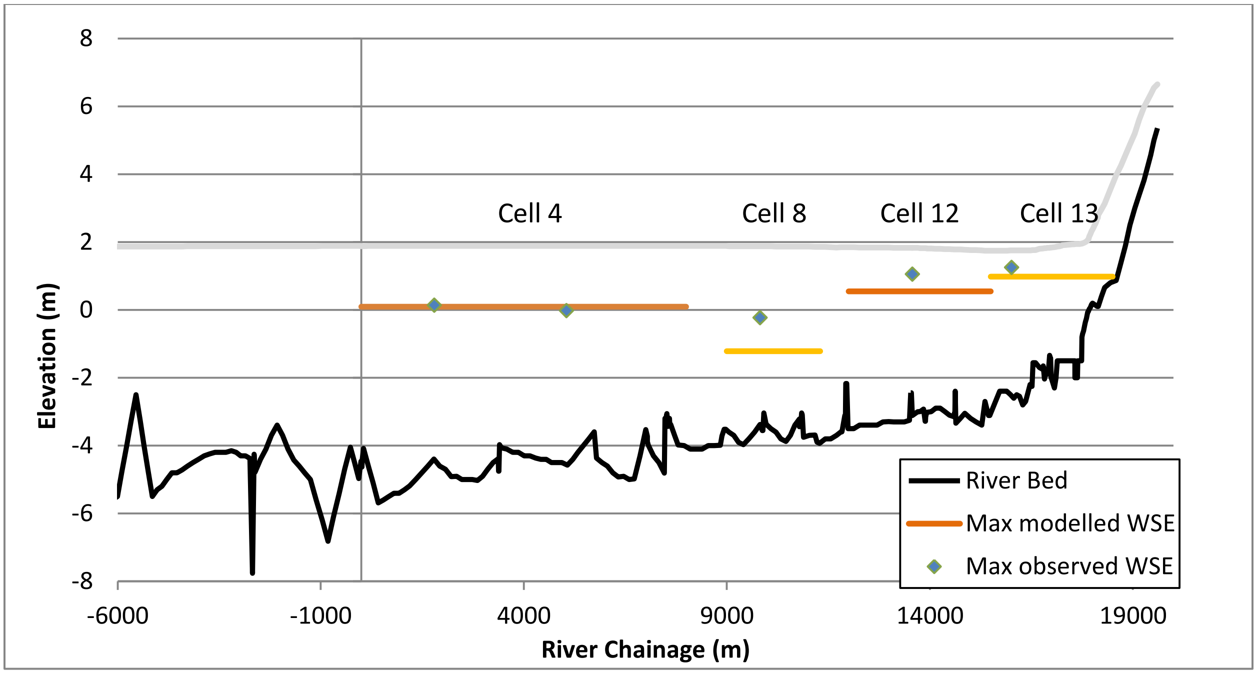


Figure 2: Nicomekl River Longitudinal Profile (Jan 2009).



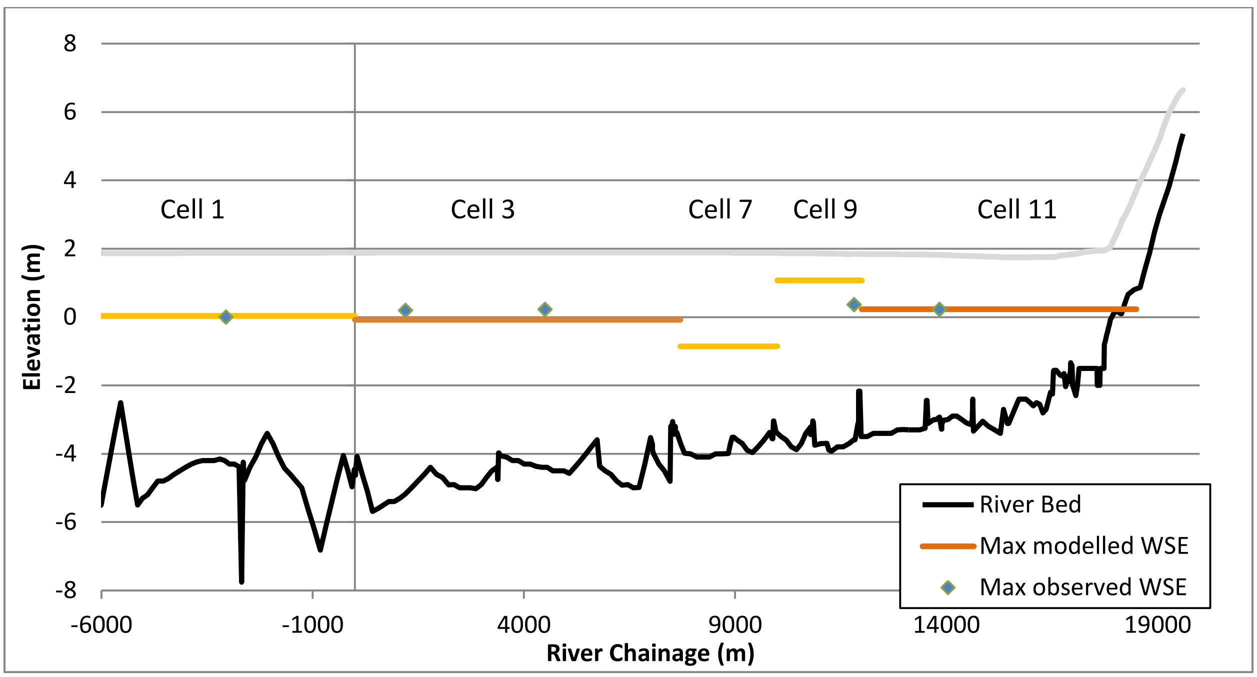


Figure 3: Longitudinal profile of floodplain cells on left (above) and right (below) banks of Serpentine River (Jan 2009).

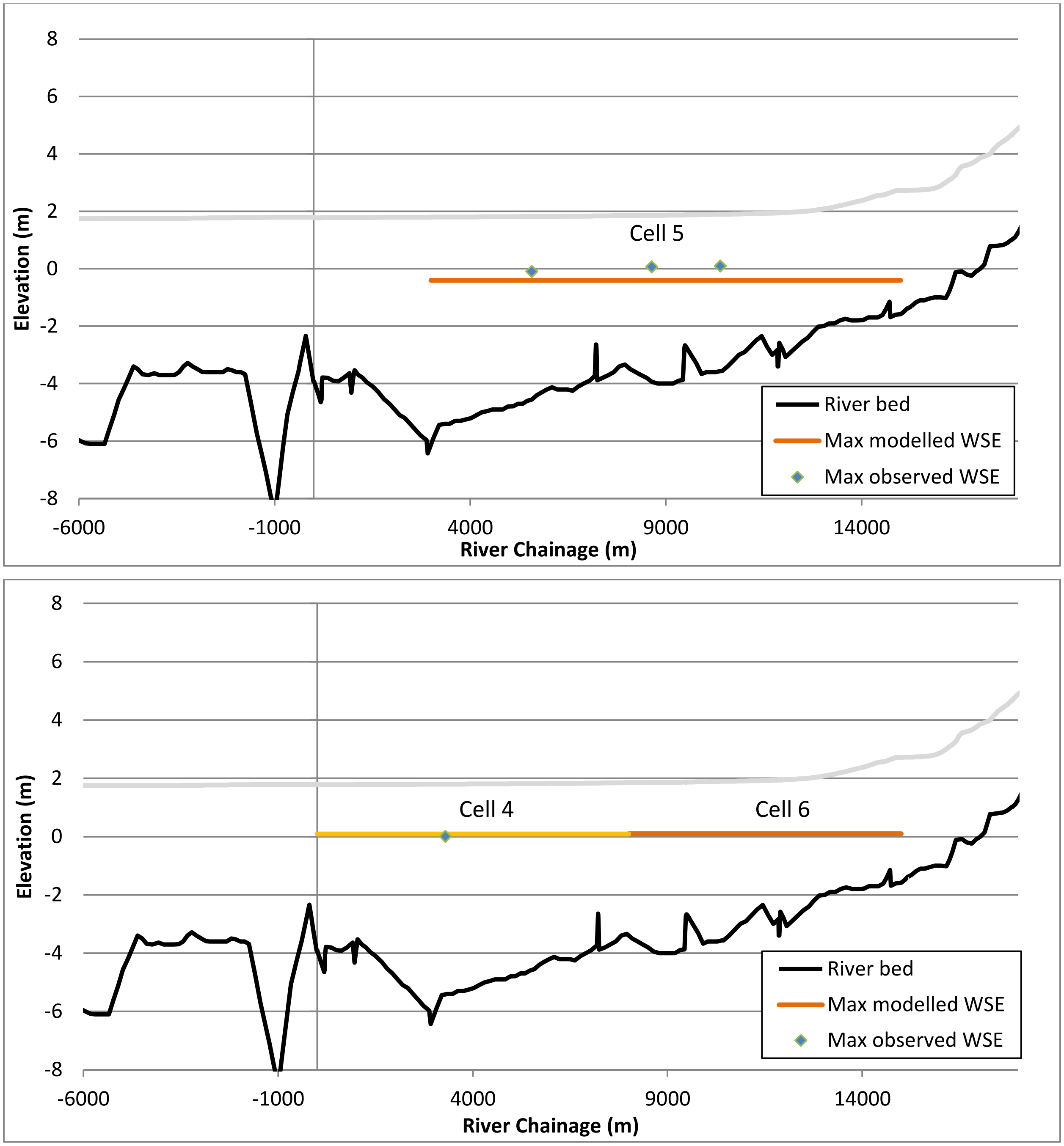


Figure 4: Longitudinal profile of floodplain cells on left (above) and right (below) banks of Nicomekl River (Jan 2009).

VALIDATION: JANUARY 2005

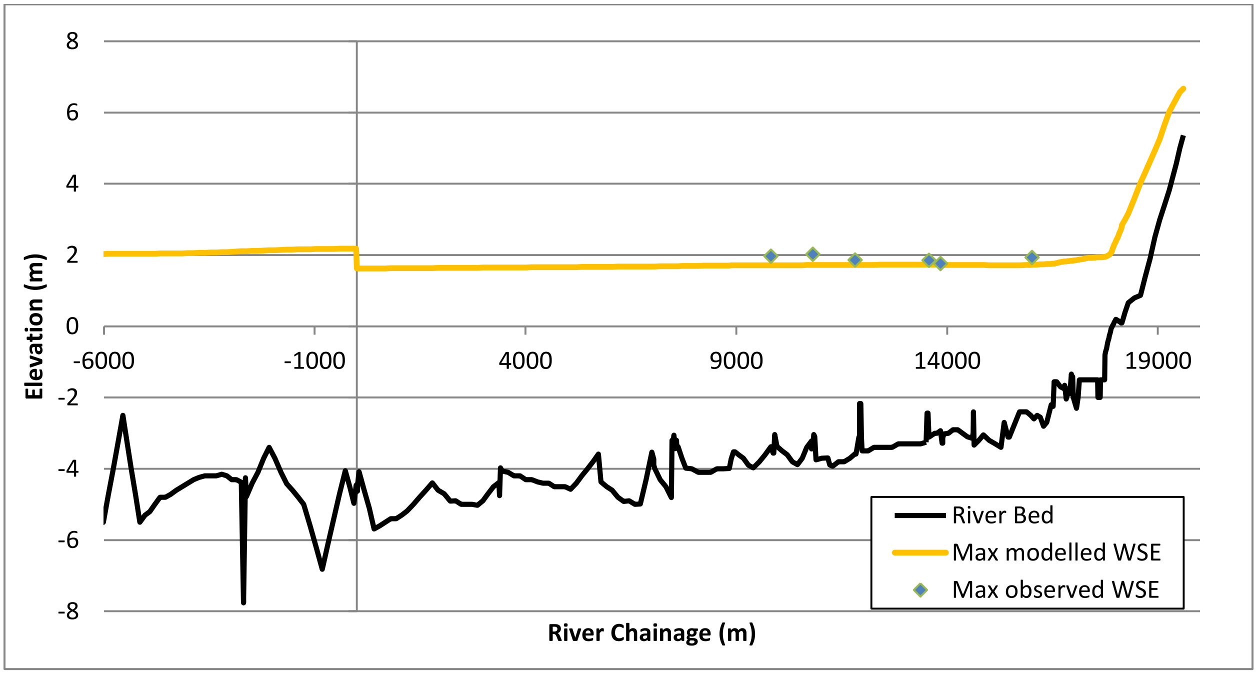


Figure 5: Serpentine River Longitudinal Profile (Jan 2005).

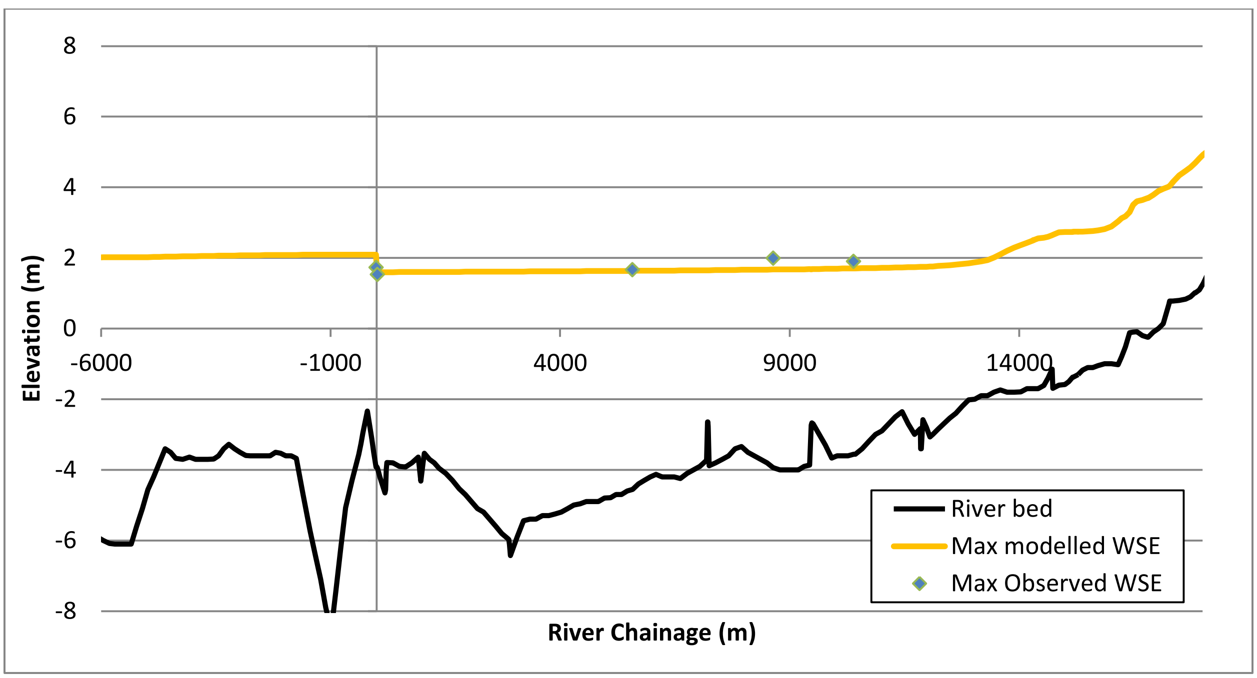


Figure 6: Nicomekl River Longitudinal Profile (Jan 2005).

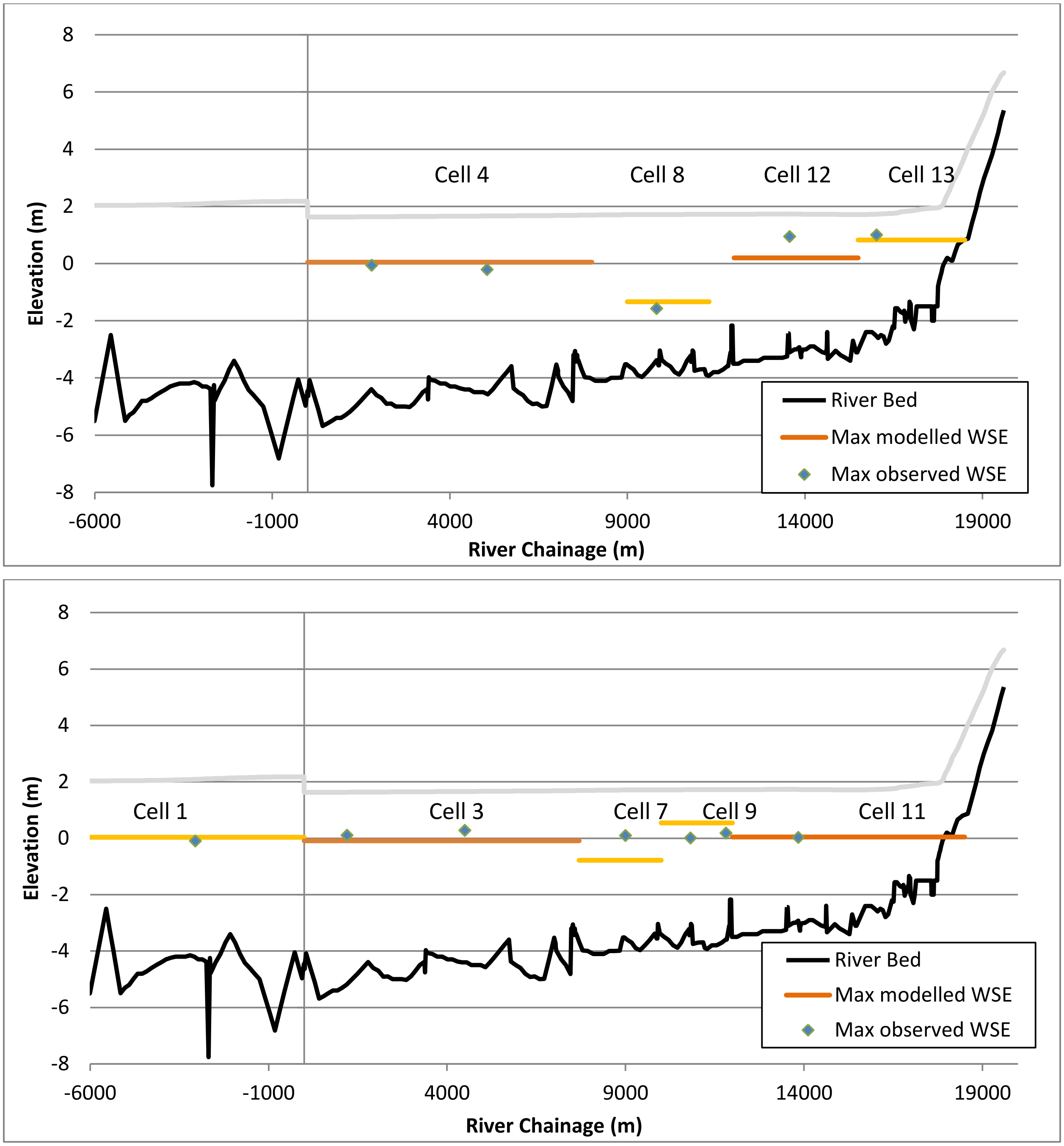


Figure 7: Longitudinal profile of floodplain cells on left (above) and right (below) banks of Serpentine River (Jan 2005).

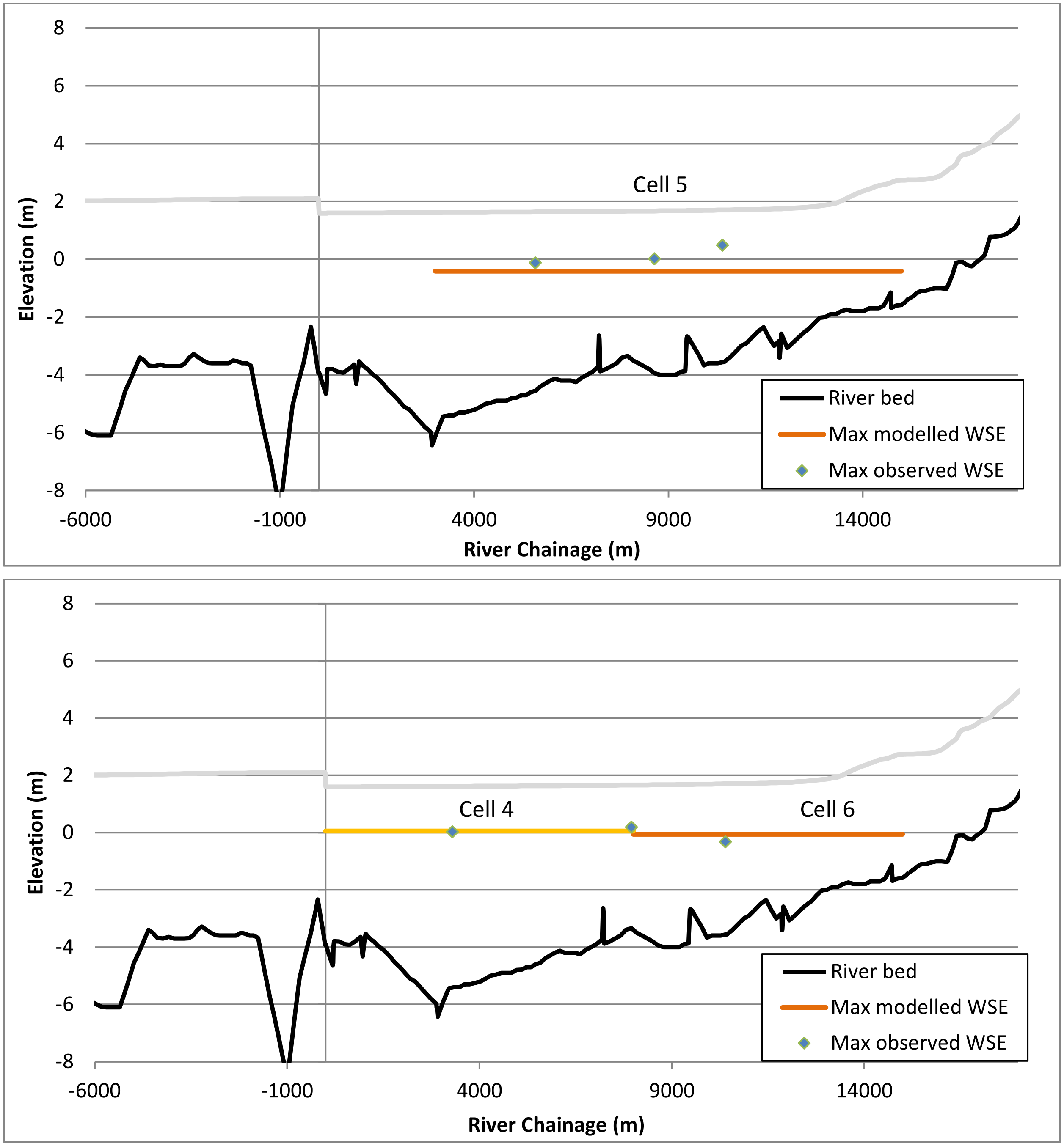


Figure 8: Longitudinal profile of floodplain cells on left (above) and right (below) banks of Nicomekl River (Jan 2005).

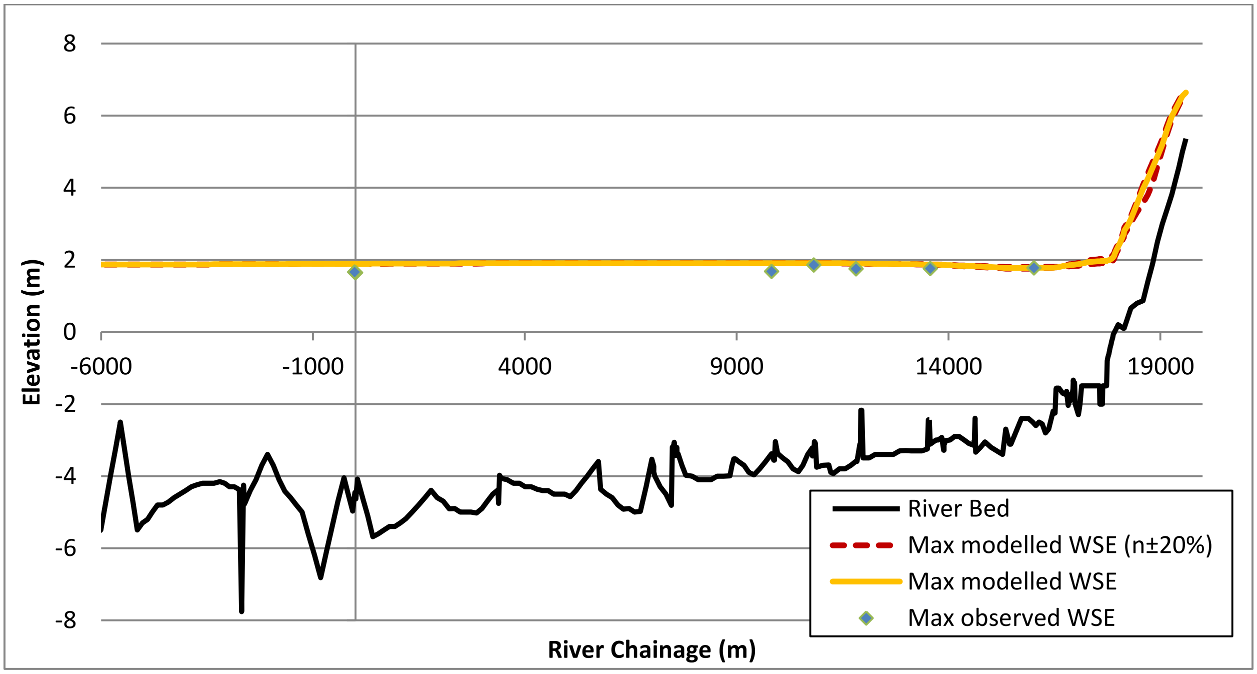


Figure 9: Serpentine River longitudinal profile of modelled water levels showing sensitivity to bed roughness (Jan 2009).

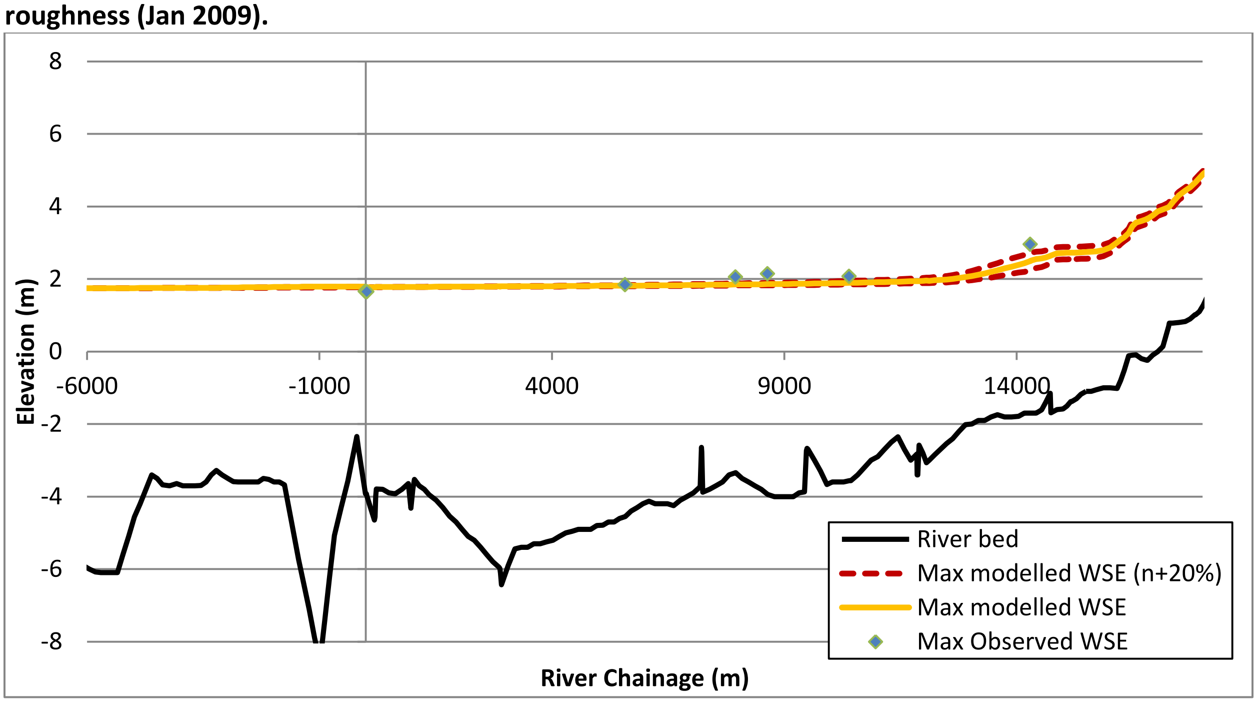


Figure 10: Nicomekl River longitudinal profile of modelled water levels showing sensitivity to bed roughness (Jan 2009).

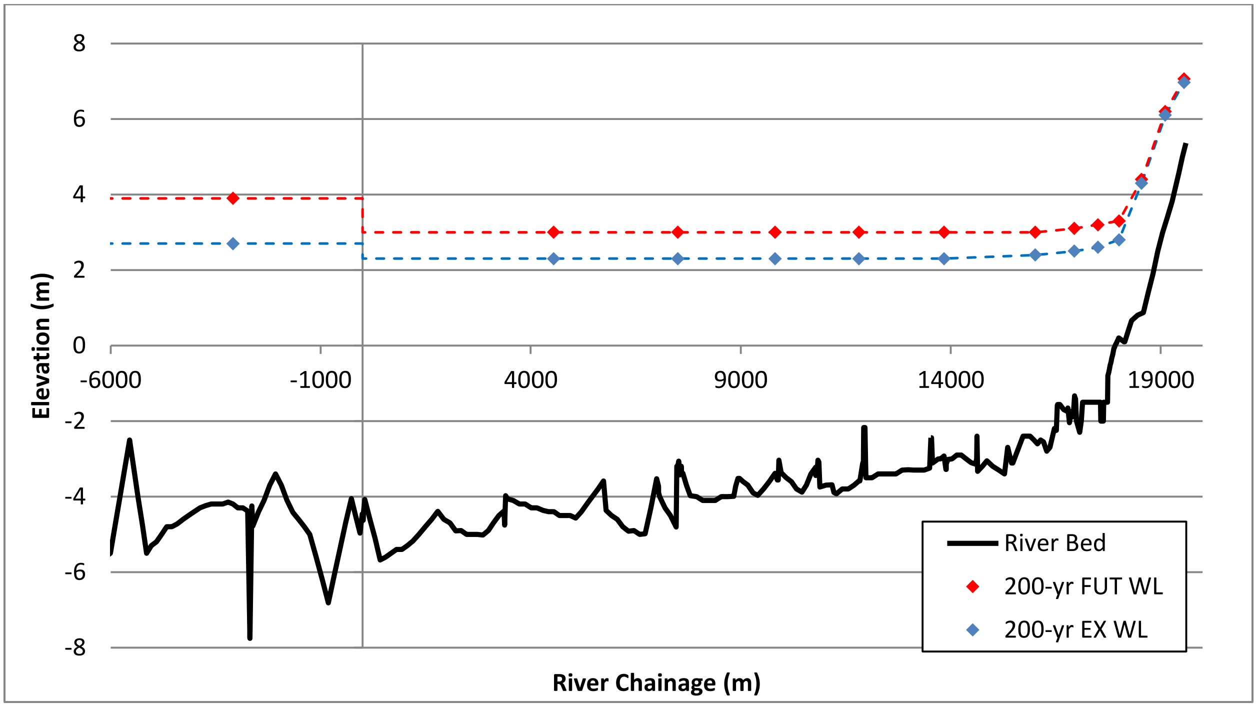


Figure 11: Serpentine River longitudinal profile of 200-year water levels.

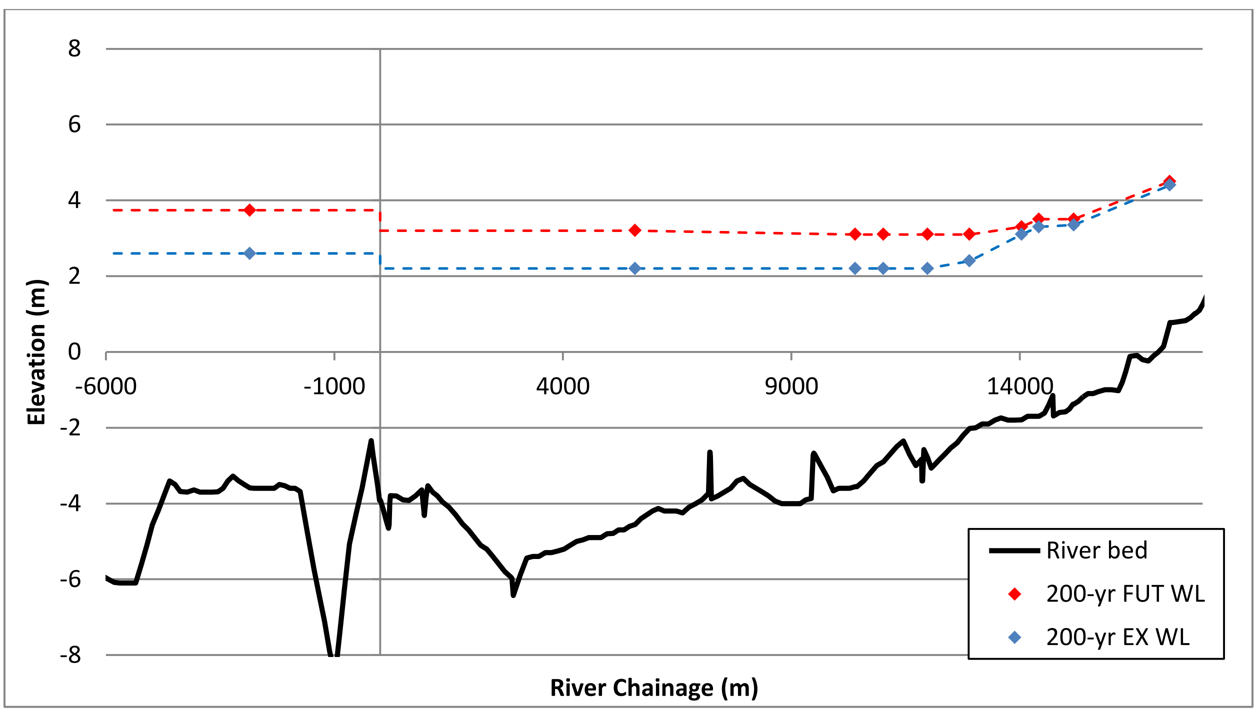


Figure 12: Nicomekl River longitudinal profiles of 200-year water levels.

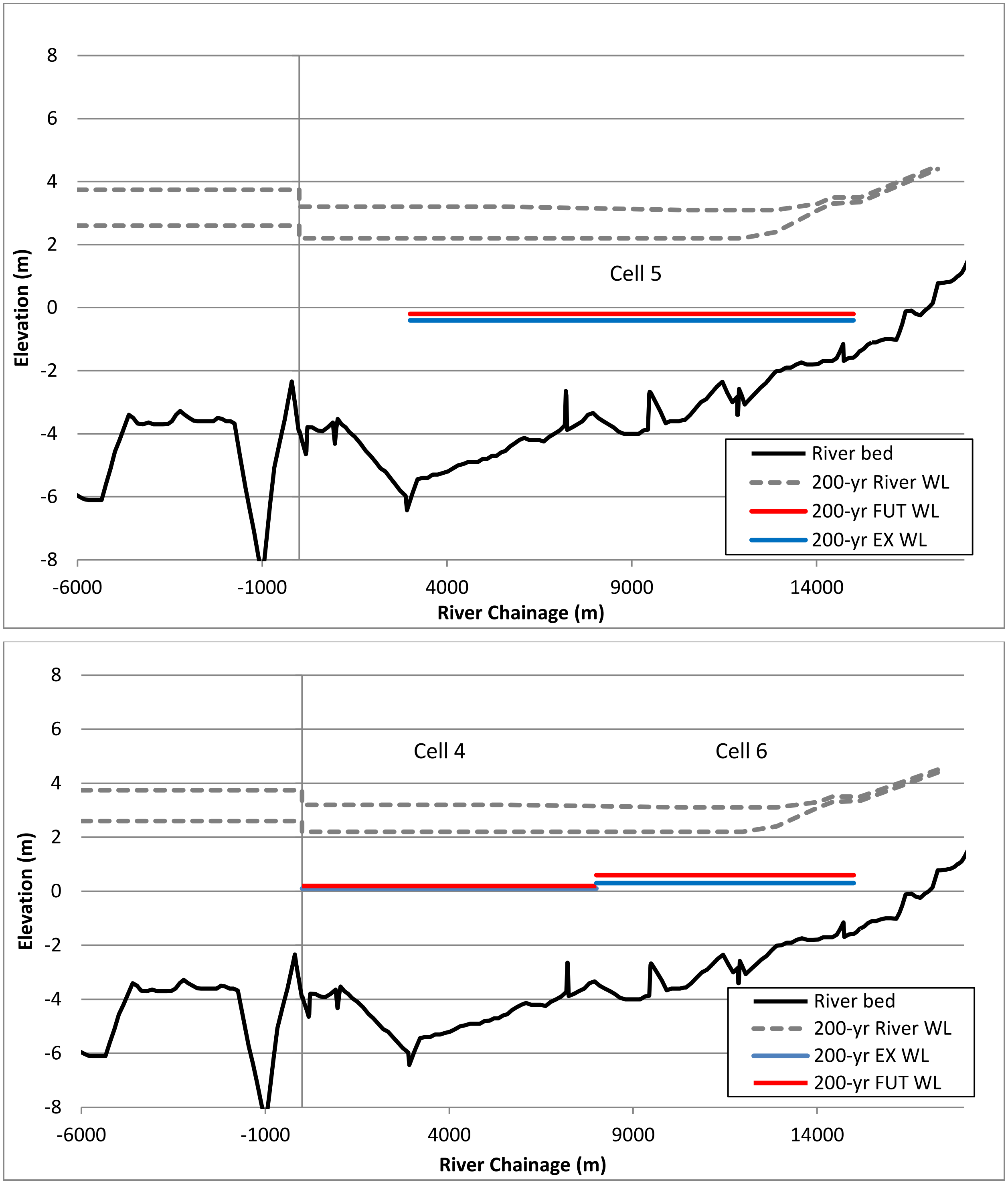


Figure 13: Longitudinal profile of 200-year water levels for floodplain cells on left (above) and right (below) banks of Nicomekl River.

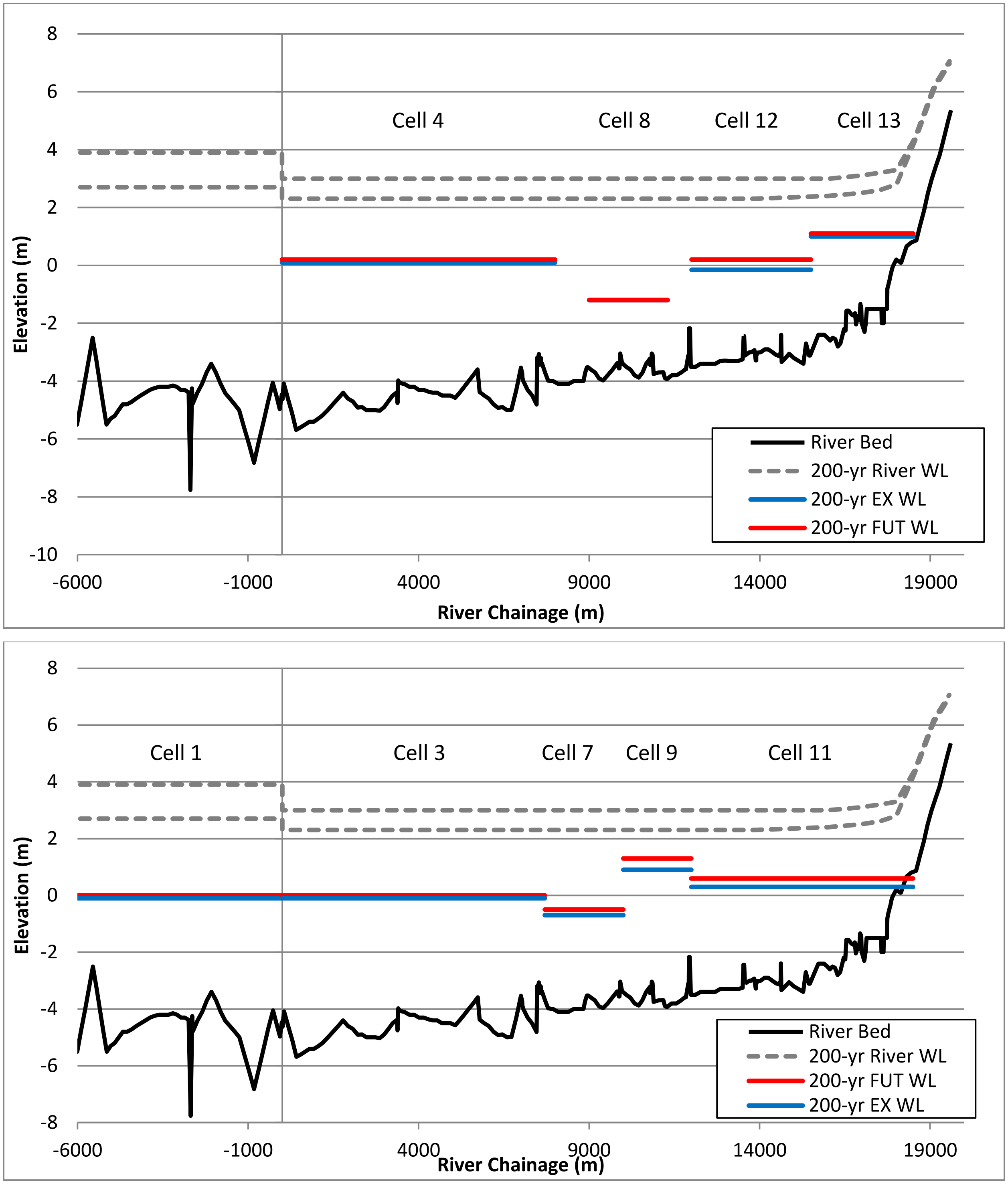
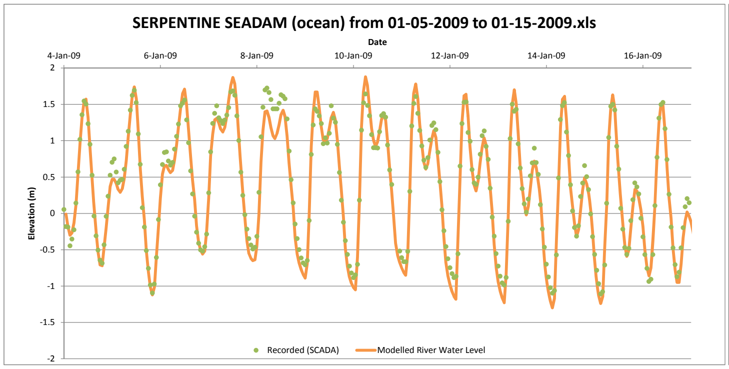


Figure 14: Longitudinal profile of 200-year water levels for floodplain cells on left (above) and right (below) banks of Serpentine River.

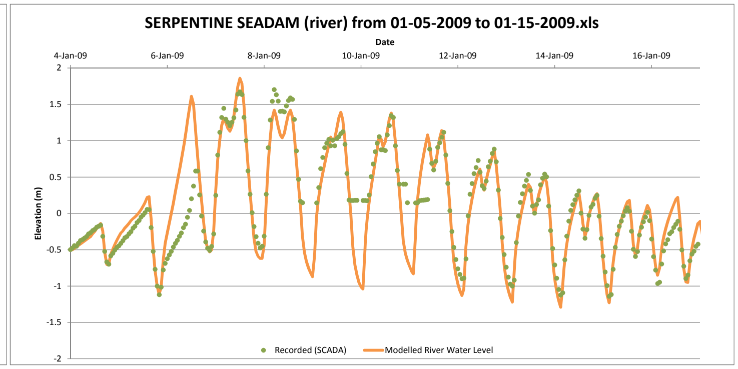
2 APPENDIX D.2. – VALIDATION TIMESERIES PLOTS

The simplified HEC-RAS model was validated to two significant recent storms; in January 2009 and January 2005. Comparison timeseries plots of observed and modelled water levels are included in this section for locations in the Serpentine and Nicomekl River channels and locations in the floodplain. Most observed data were recorded at pump station locations (refer to Map 1). Plots are ordered by river (Serpentine then Nicomekl) and then by location starting downstream and moving upstream.

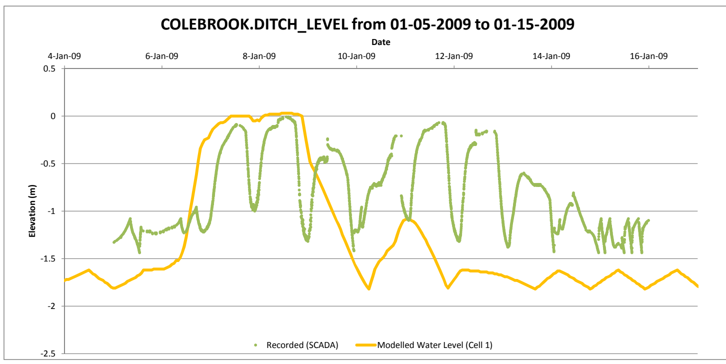
SERPENTINE



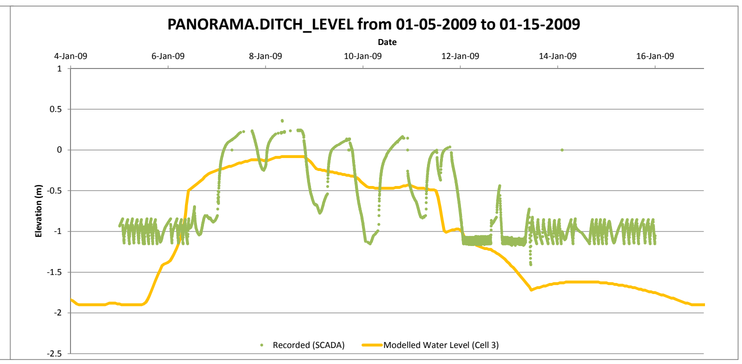
SERPENTINE SEADAM (ocean) from 01-05-2009 to 01-15-2009.xls
Date
Elevation (m)
Recorded (SCADA)
Modelled River Water Level



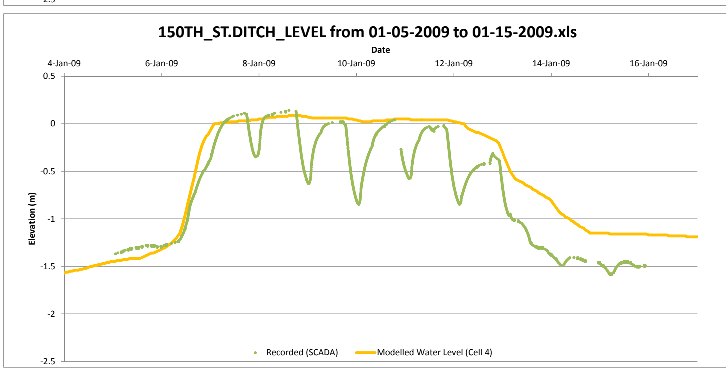
SERPENTINE SEADAM (river) from 01-05-2009 to 01-15-2009.xls
Date
Elevation (m)
Recorded (SCADA)
Modelled River Water Level



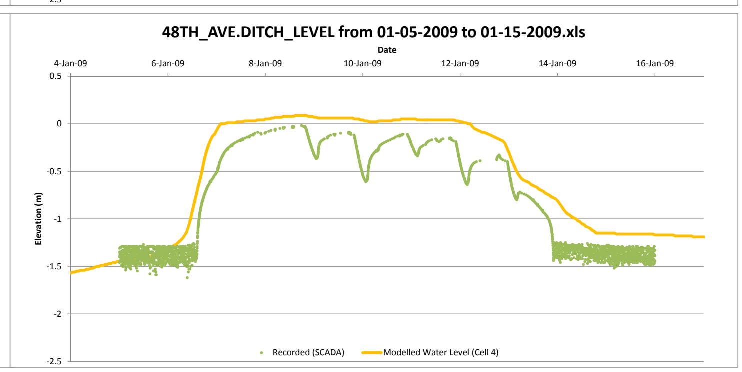
COLEBROOK.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 1)



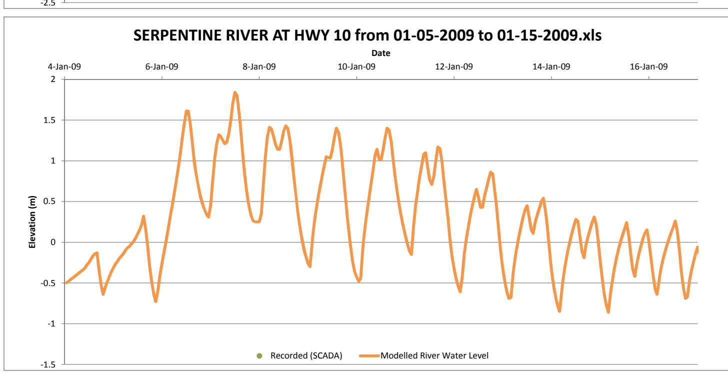
PANORAMA.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 3)



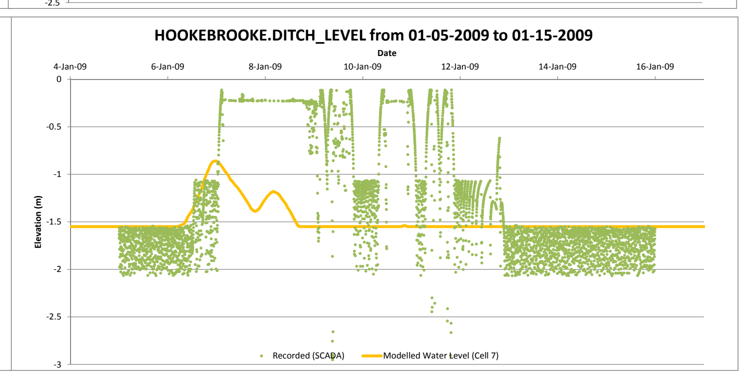
150TH_ST.DITCH_LEVEL from 01-05-2009 to 01-15-2009.xls
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 4)



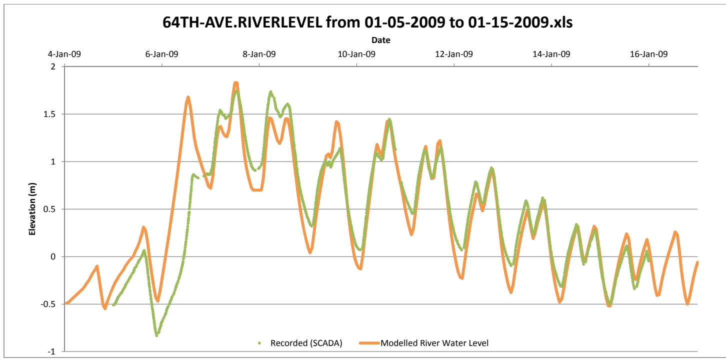
48TH_AVE.DITCH_LEVEL from 01-05-2009 to 01-15-2009.xls
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 4)

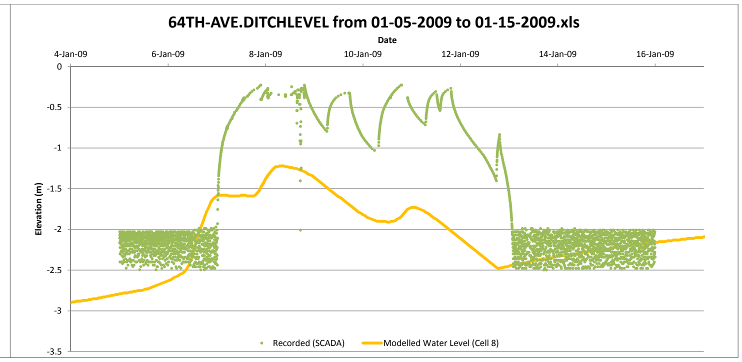


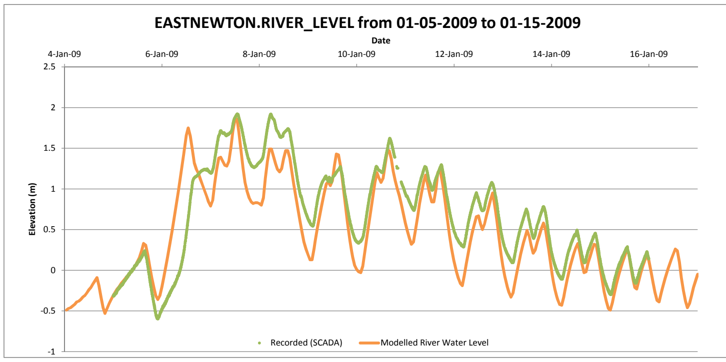
SERPENTINE RIVER AT HWY 10 from 01-05-2009 to 01-15-2009.xls
Date
Elevation (m)
Recorded (SCADA)
Modelled River Water Level

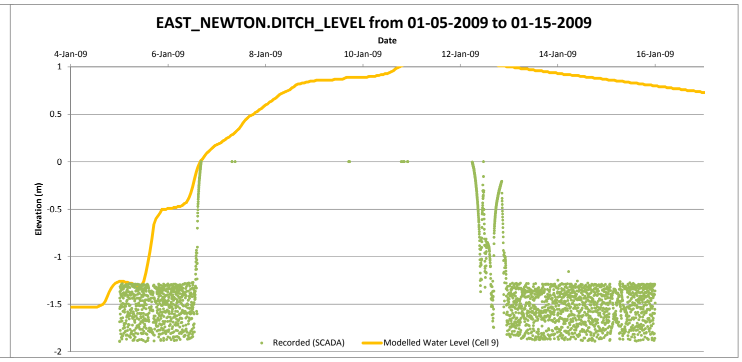


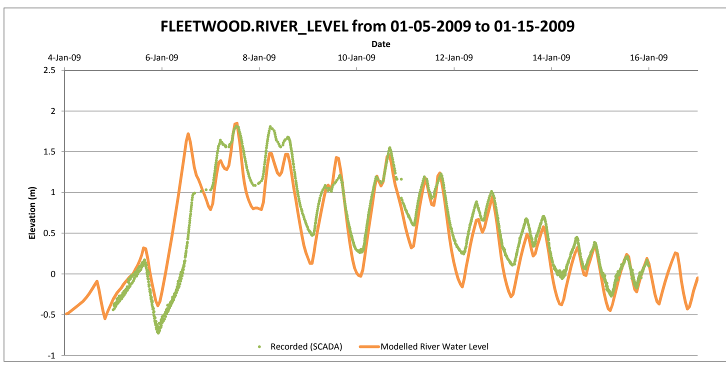
HOOKEBROOKE.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 7)

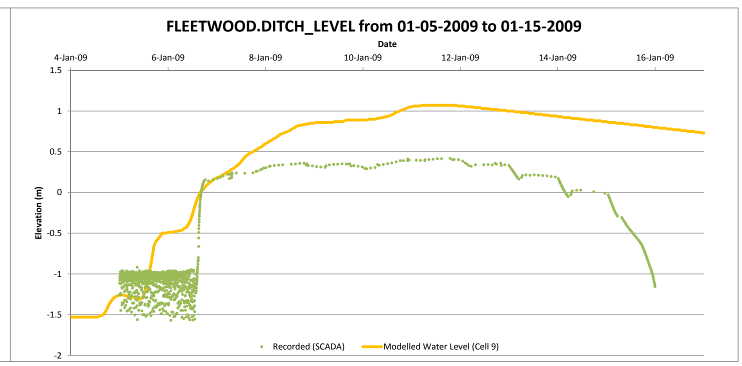

64TH-AVE.RIVERLEVEL from 01-05-2009 to 01-15-2009.xls
Date
Elevation (m)
Recorded (SCADA)
Modelled River Water Level

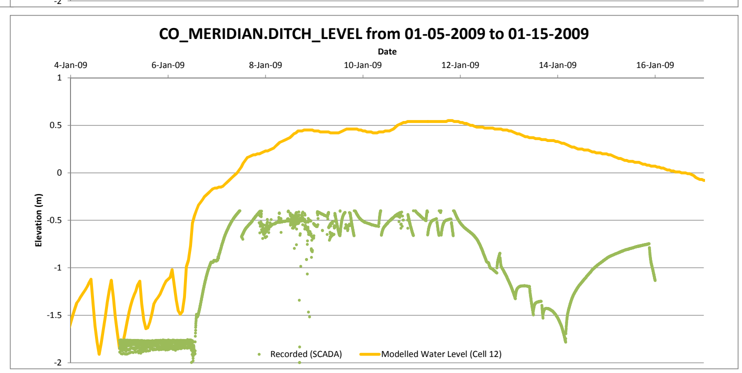

64TH-AVE.DITCHLEVEL from 01-05-2009 to 01-15-2009.xls
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 8)

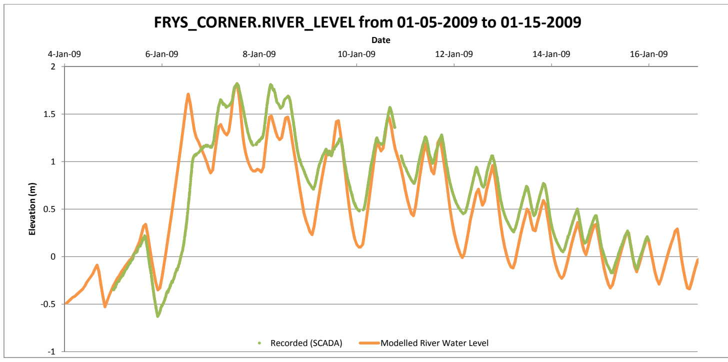

EASTNEWTON.RIVER_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled River Water Level

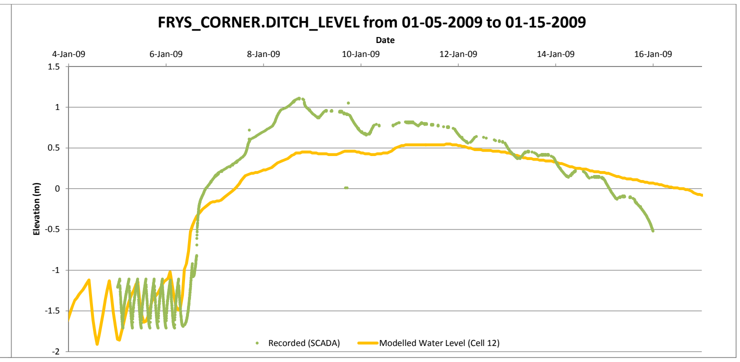

EAST_NEWTON.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 9)

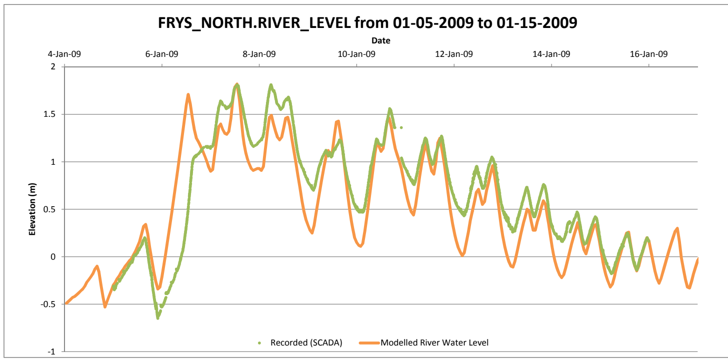

FLEETWOOD.RIVER_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled River Water Level

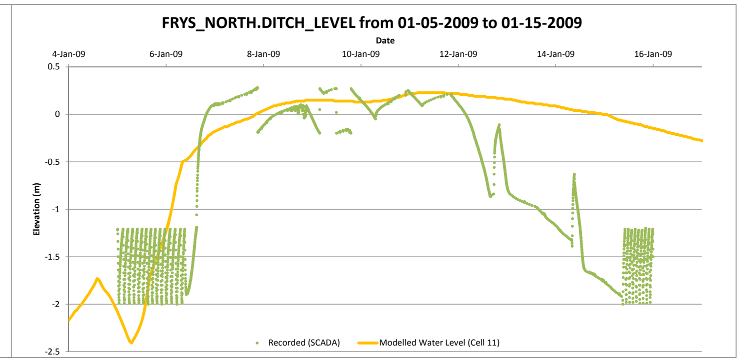

FLEETWOOD.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 9)

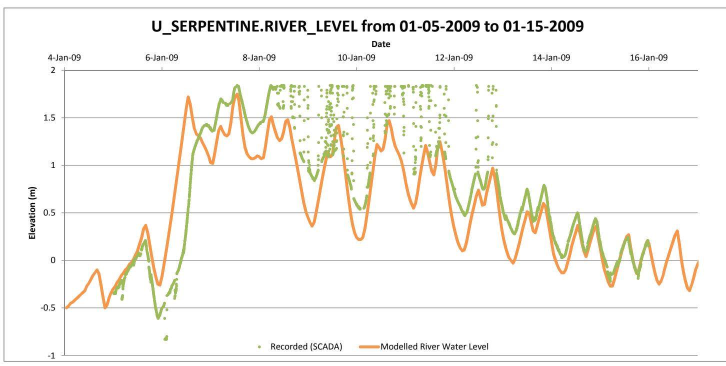

CO_MERIDIAN.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 12)

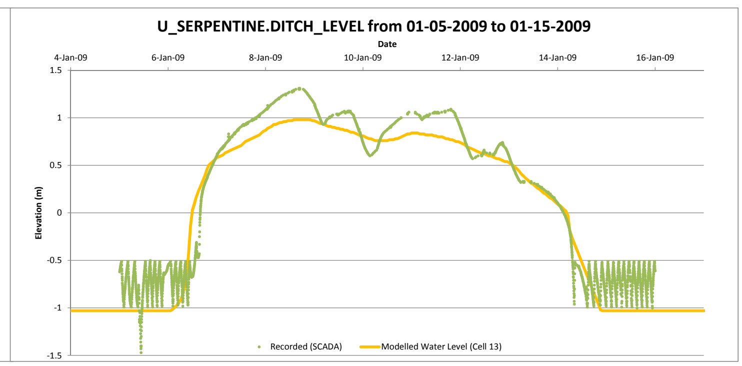






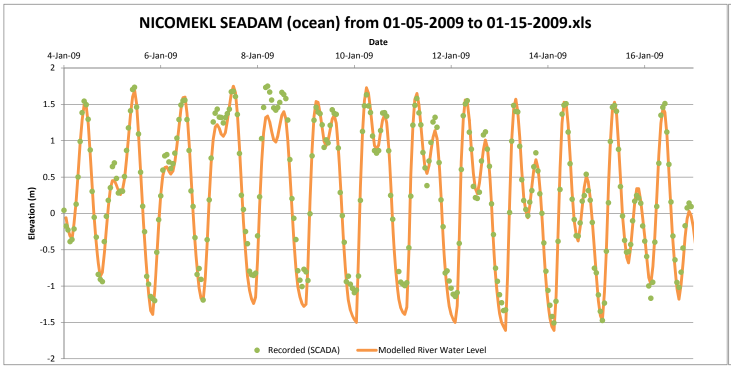


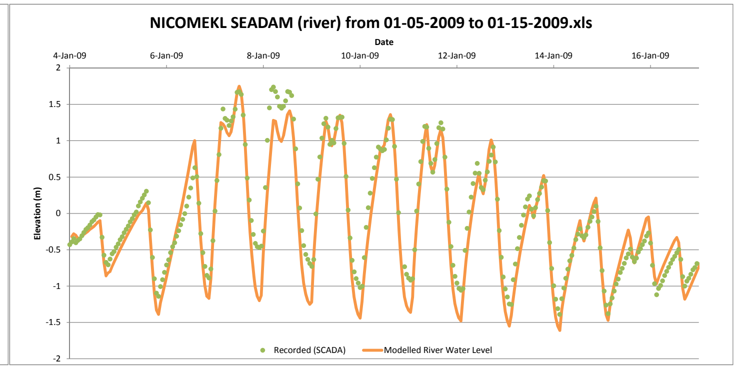


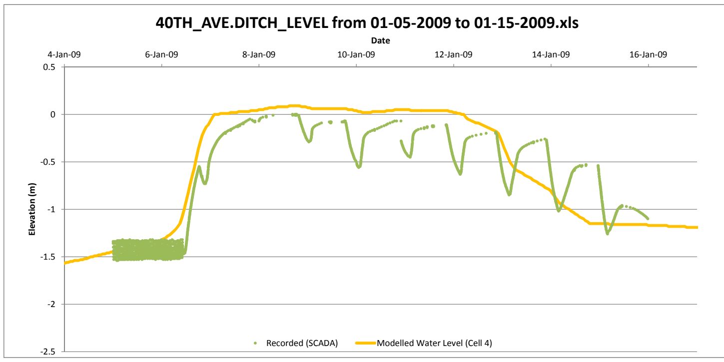


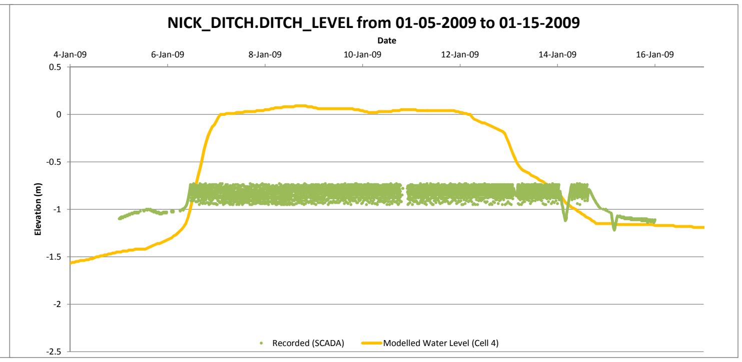
No Data for Serpentine at 168th St for Jan 2009

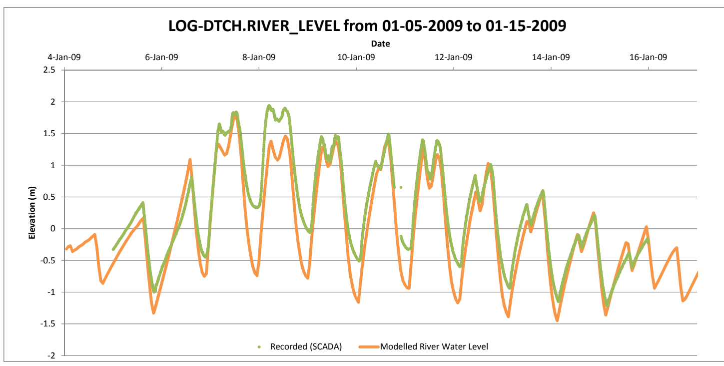
NICOMEKL

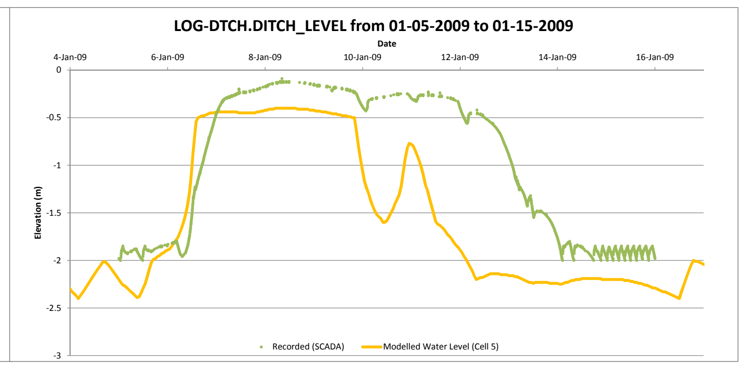

NICOMEKL SEADAM (ocean) from 01-05-2009 to 01-15-2009.xls
Date
Elevation (m)
Recorded (SCADA)
Modelled River Water Level

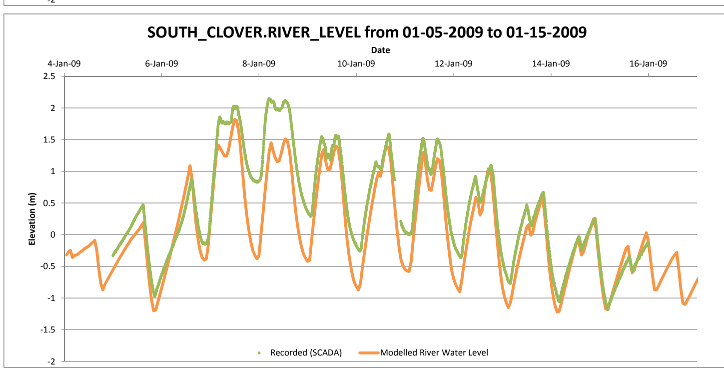

NICOMEKL SEADAM (river) from 01-05-2009 to 01-15-2009.xls
Date
Elevation (m)
Recorded (SCADA)
Modelled River Water Level

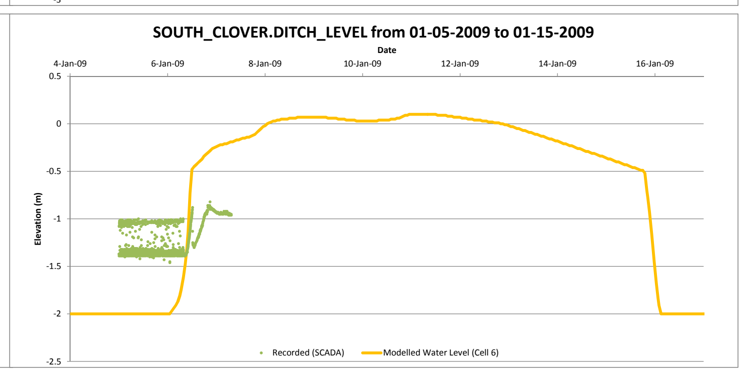

40TH_AVE.DITCH_LEVEL from 01-05-2009 to 01-15-2009.xls
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 4)

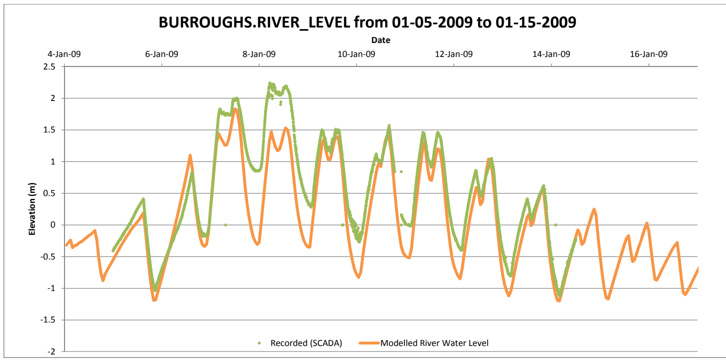

NICK_DITCH.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 4)

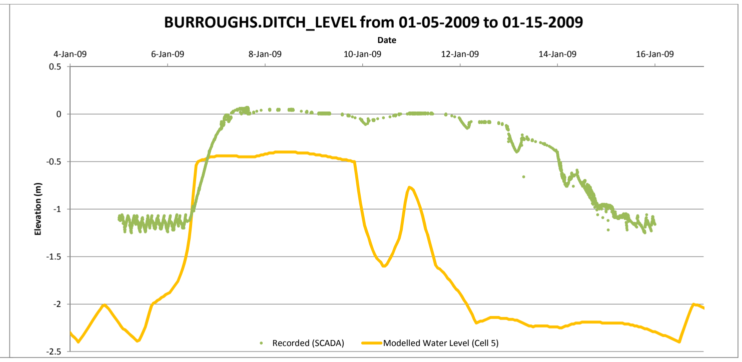

LOG-DTCH.RIVER_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled River Water Level

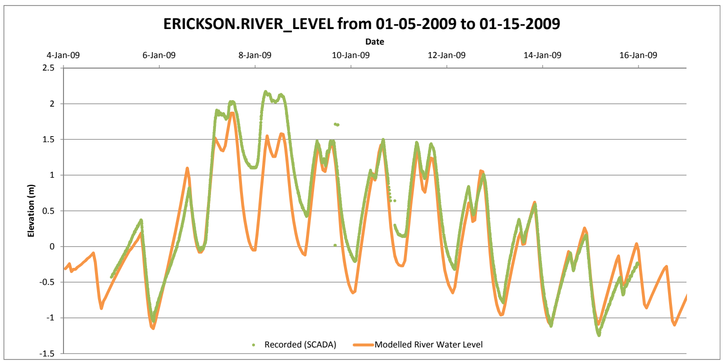

LOG-DTCH.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 5)

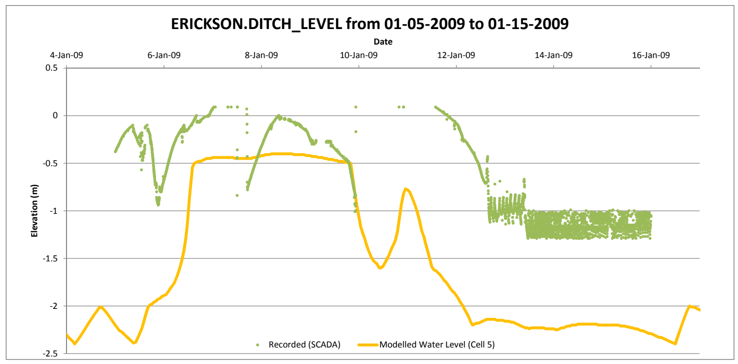

SOUTH_CLOVER.RIVER_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled River Water Level

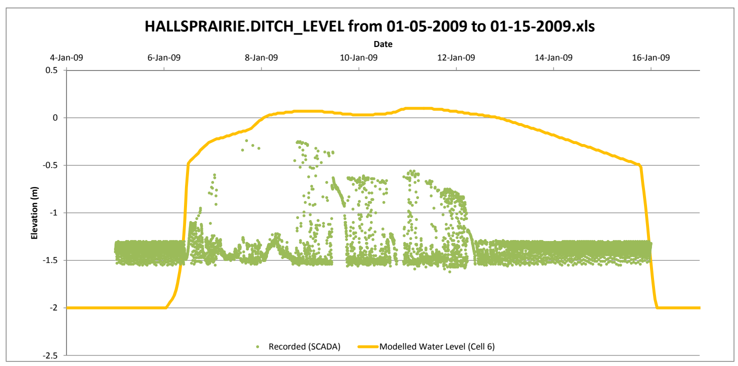

SOUTH_CLOVER.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 6)

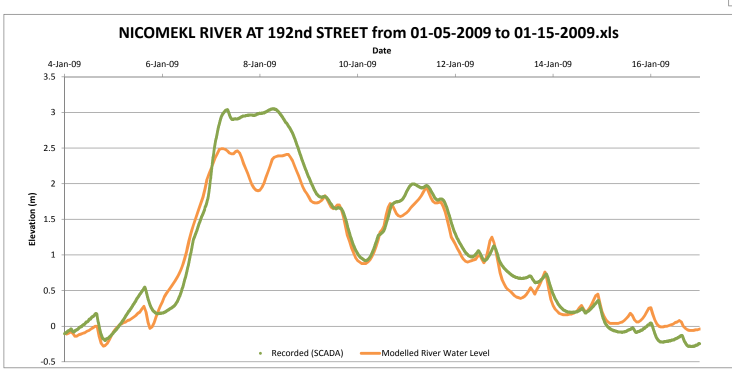

BURROUGHS.RIVER_LEVEL from 01-05-2009 to 01-15-2009
Date
4-Jan-09
6-Jan-09
8-Jan-09
10-Jan-09
12-Jan-09
14-Jan-09
16-Jan-09
Elevation (m)
Recorded (SCADA)
Modelled River Water Level


BURROUGHS.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
4-Jan-09
6-Jan-09
8-Jan-09
10-Jan-09
12-Jan-09
14-Jan-09
16-Jan-09
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 5)

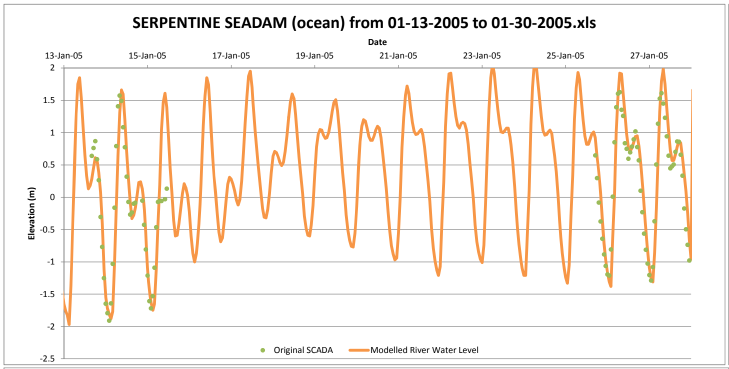

ERICKSON.RIVER_LEVEL from 01-05-2009 to 01-15-2009
Date
4-Jan-09
6-Jan-09
8-Jan-09
10-Jan-09
12-Jan-09
14-Jan-09
16-Jan-09
Elevation (m)
Recorded (SCADA)
Modelled River Water Level

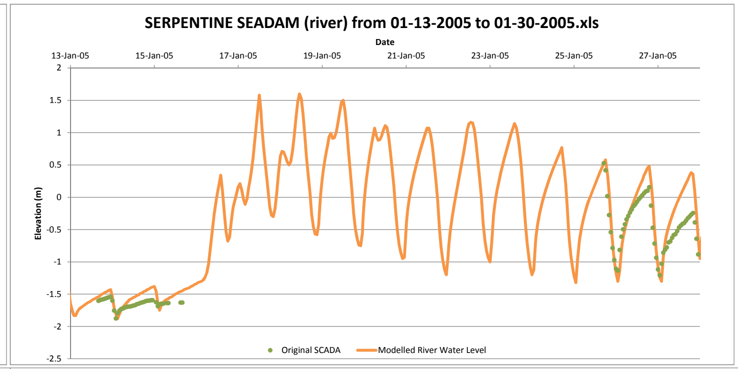

ERICKSON.DITCH_LEVEL from 01-05-2009 to 01-15-2009
Date
4-Jan-09
6-Jan-09
8-Jan-09
10-Jan-09
12-Jan-09
14-Jan-09
16-Jan-09
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 5)

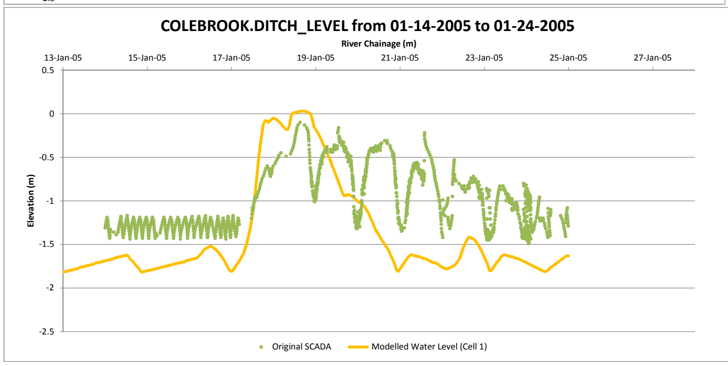

HALLSPRAIRIE.DITCH_LEVEL from 01-05-2009 to 01-15-2009.xls
Date
4-Jan-09
6-Jan-09
8-Jan-09
10-Jan-09
12-Jan-09
14-Jan-09
16-Jan-09
Elevation (m)
Recorded (SCADA)
Modelled Water Level (Cell 6)

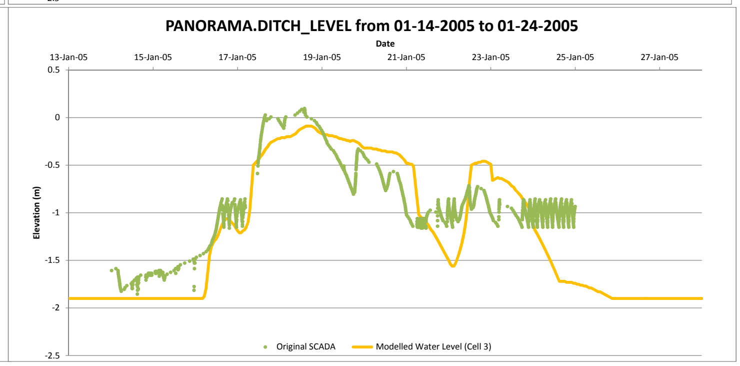

NICOMEKL RIVER AT 192nd STREET from 01-05-2009 to 01-15-2009.xls
Date
4-Jan-09
6-Jan-09
8-Jan-09
10-Jan-09
12-Jan-09
14-Jan-09
16-Jan-09
Elevation (m)
Recorded (SCADA)
Modelled River Water Level

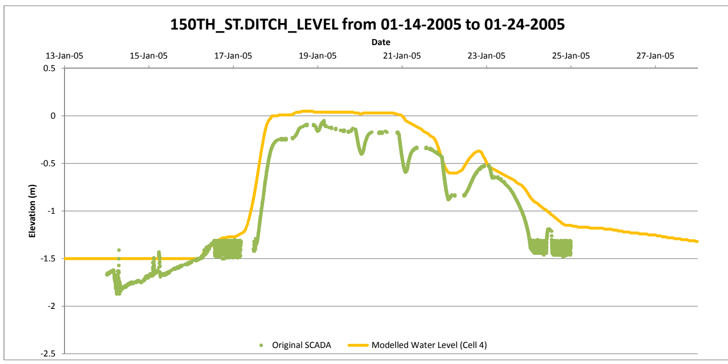
SERPENTINE

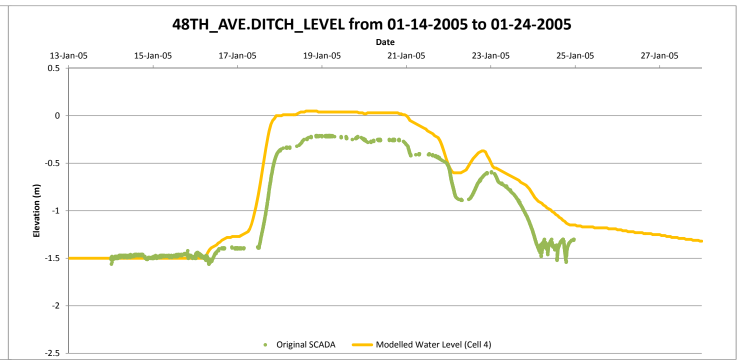

SERPENTINE SEADAM (ocean) from 01-13-2005 to 01-30-2005.xls
Date
Elevation (m)
Original SCADA
Modelled River Water Level

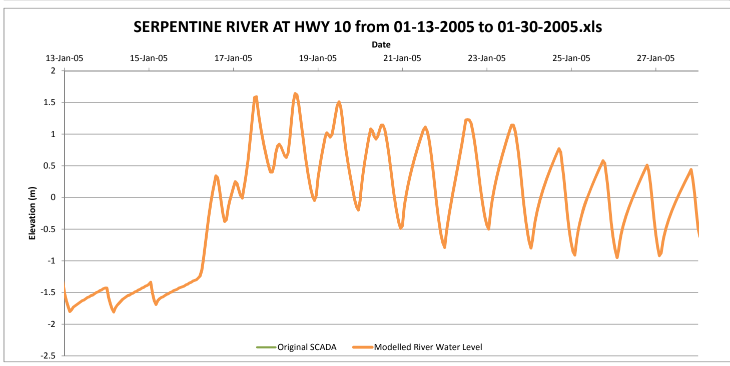

SERPENTINE SEADAM (river) from 01-13-2005 to 01-30-2005.xls
Date
Elevation (m)
Original SCADA
Modelled River Water Level

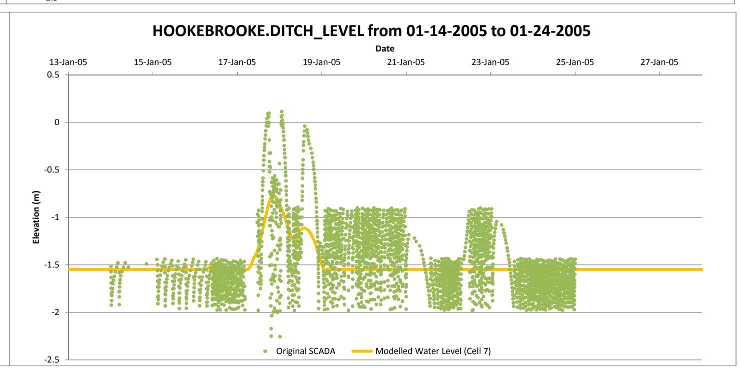

COLEBROOK.DITCH_LEVEL from 01-14-2005 to 01-24-2005
River Chainage (m)
Elevation (m)
Original SCADA
Modelled Water Level (Cell 1)

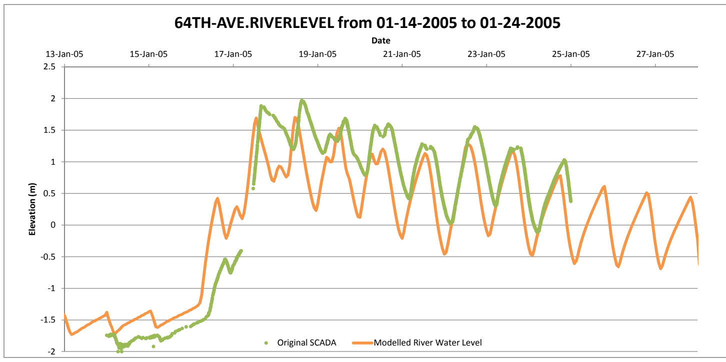

PANORAMA.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 3)

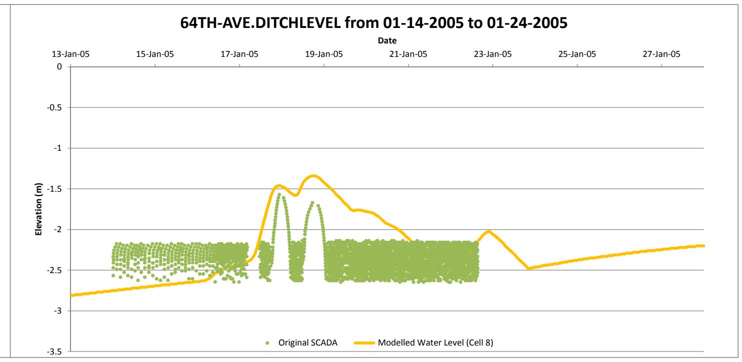

150TH_ST.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 4)

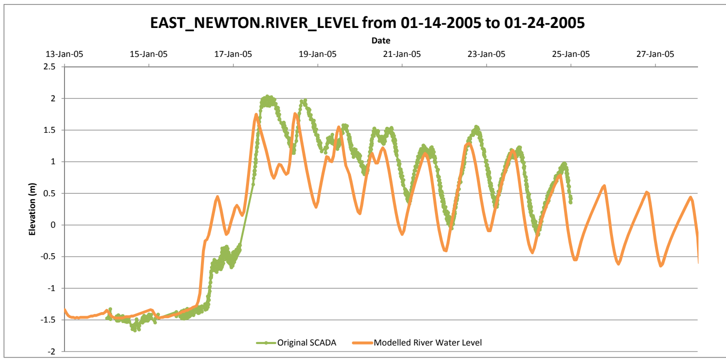

48TH_AVE.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 4)

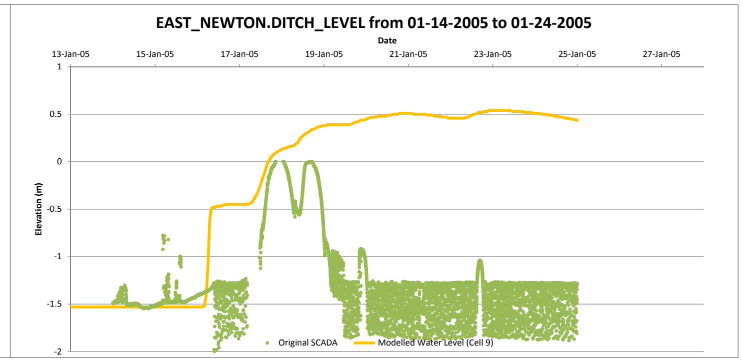

SERPENTINE RIVER AT HWY 10 from 01-13-2005 to 01-30-2005.xls
Date
Elevation (m)
Original SCADA
Modelled River Water Level

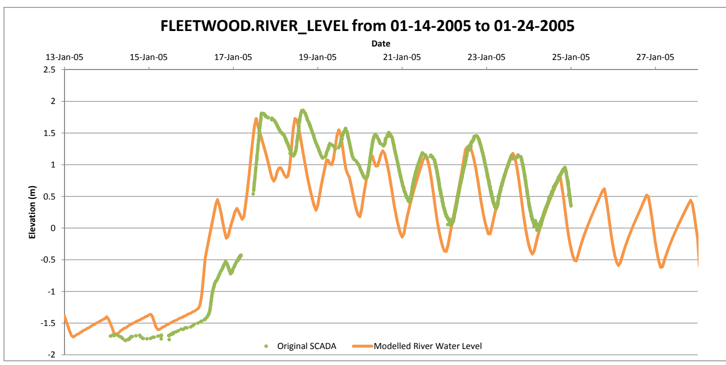

HOOKEBROOKE.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 7)

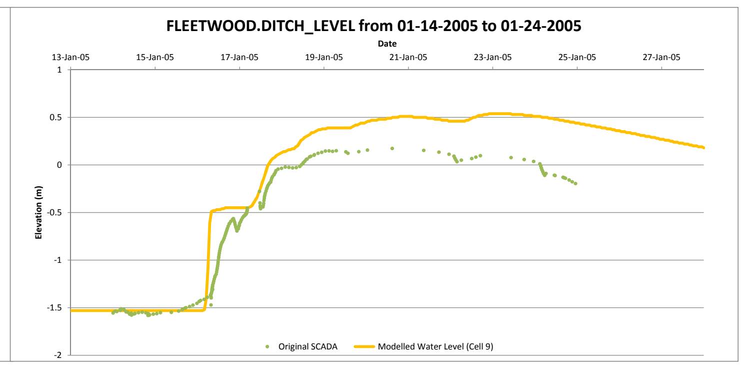

64TH-AVE.RIVERLEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled River Water Level

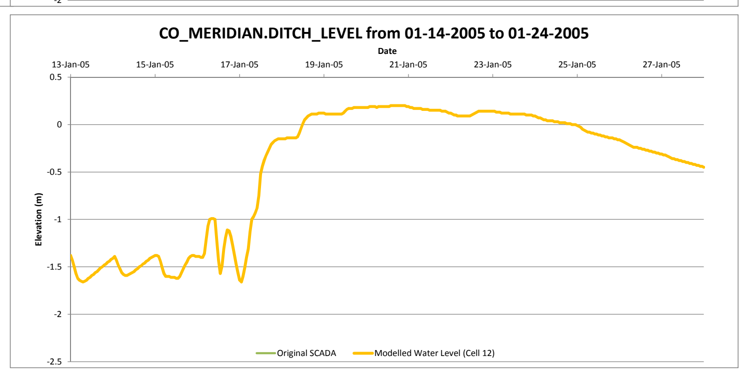

64TH-AVE.DITCHLEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 8)

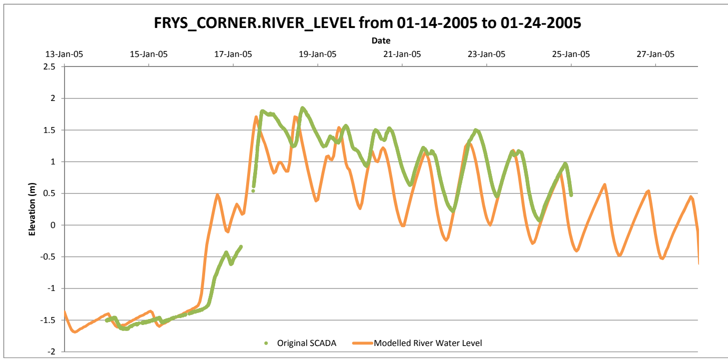

EAST_NEWTON.RIVER_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled River Water Level

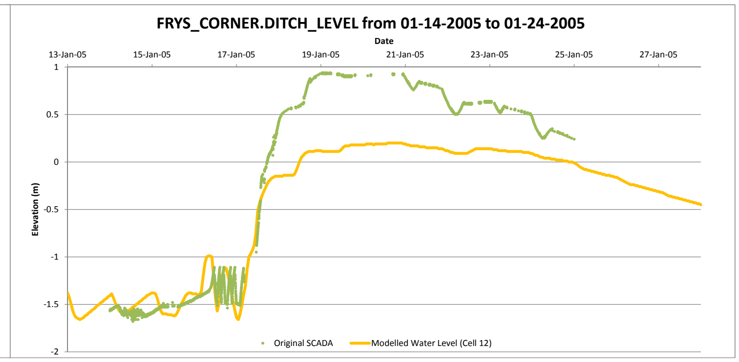

EAST_NEWTON.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 9)

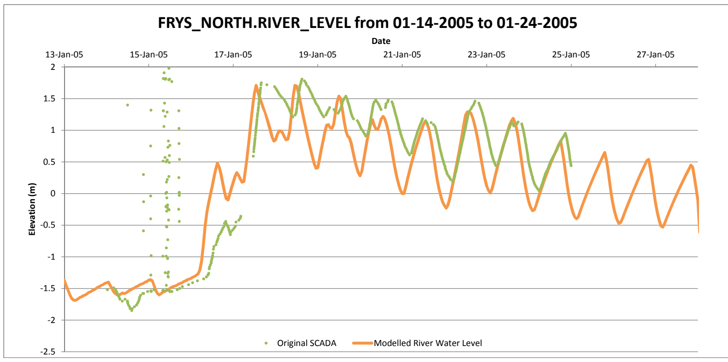

FLEETWOOD.RIVER_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled River Water Level

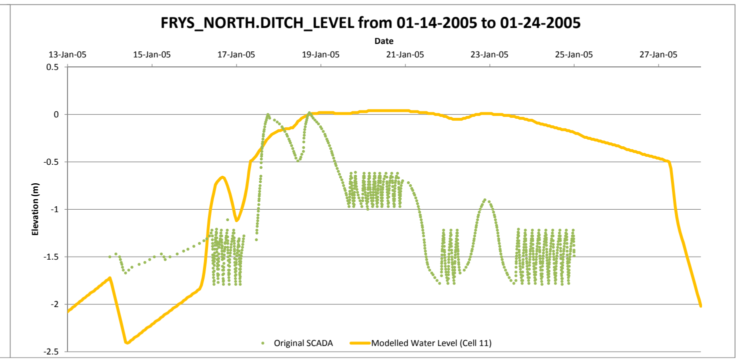

FLEETWOOD.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 9)

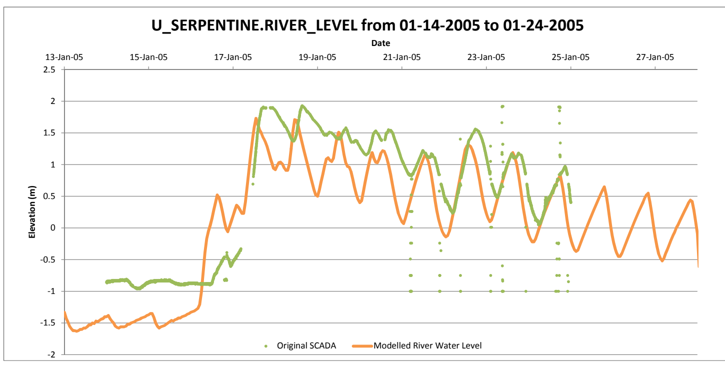

CO_MERIDIAN.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 12)

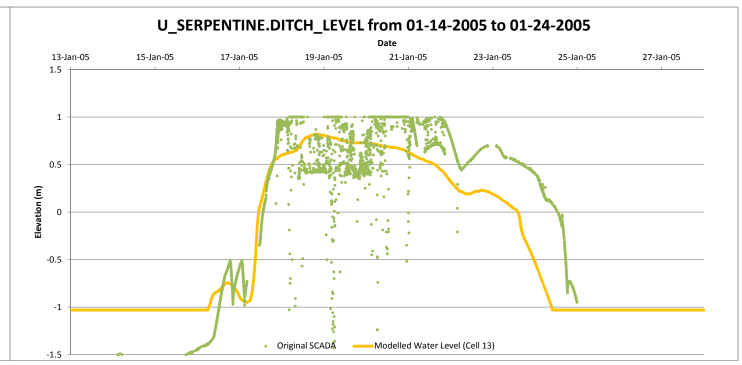

FRYS_CORNER.RIVER_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled River Water Level


FRYS_CORNER.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 12)

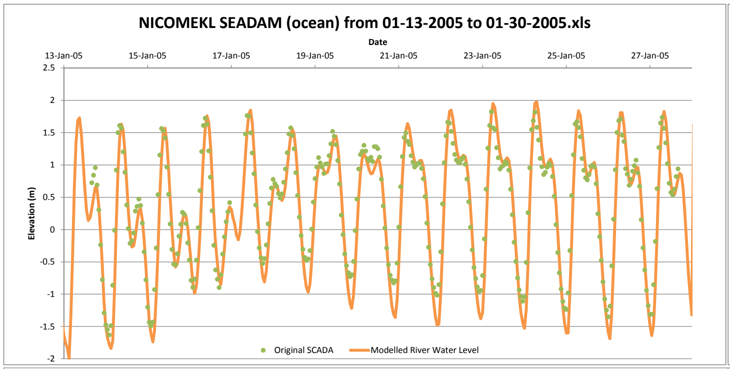

FRYS_NORTH.RIVER_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled River Water Level

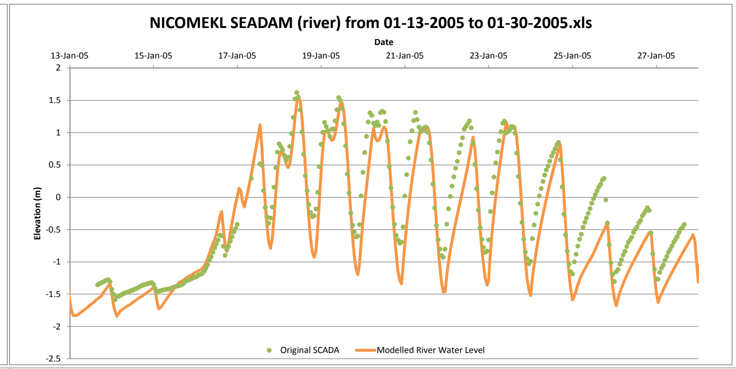

FRYS_NORTH.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 11)

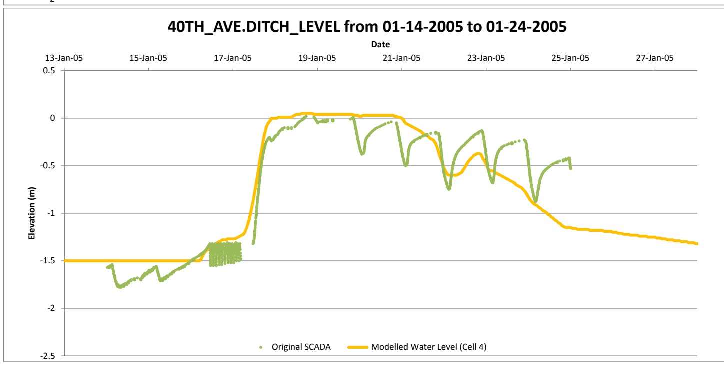

U_SERPENTINE.RIVER_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled River Water Level

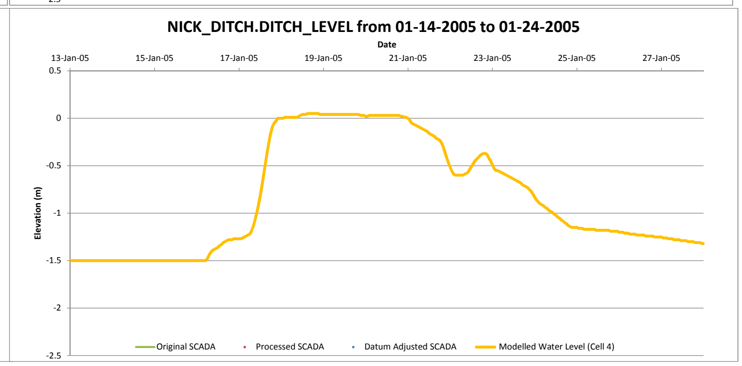

U_SERPENTINE.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 13)

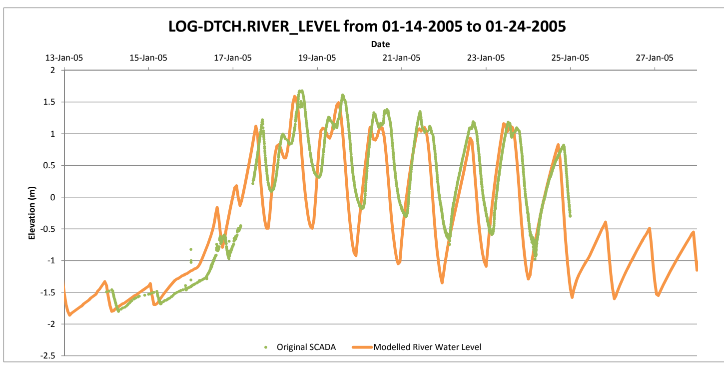
NICOMEKL

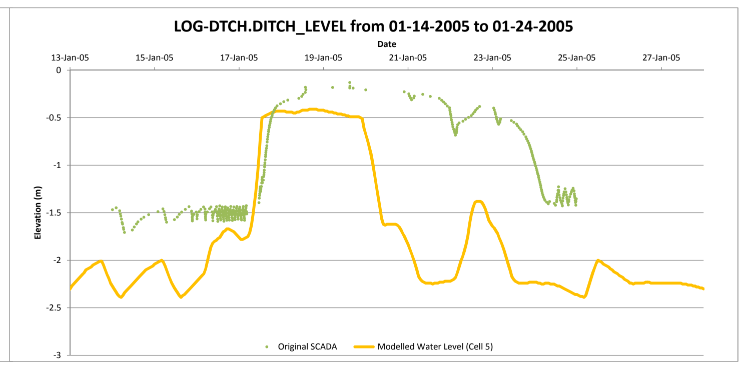

NICOMEKL SEADAM (ocean) from 01-13-2005 to 01-30-2005.xls
Date
Elevation (m)
Original SCADA
Modelled River Water Level

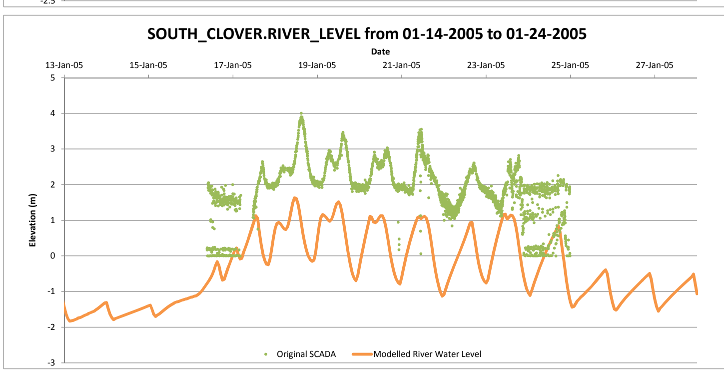

NICOMEKL SEADAM (river) from 01-13-2005 to 01-30-2005.xls
Date
Elevation (m)
Original SCADA
Modelled River Water Level

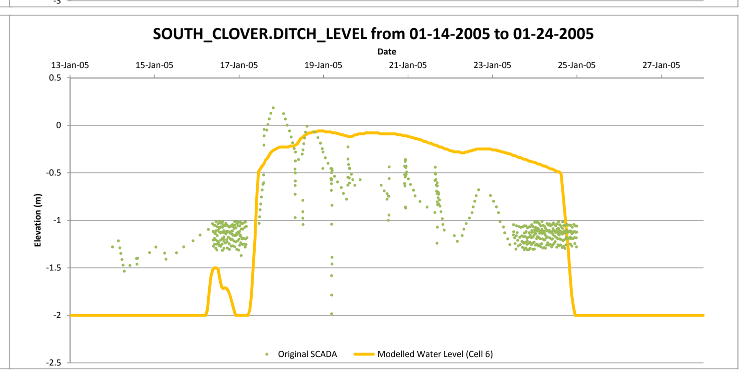

40TH_AVE.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 4)

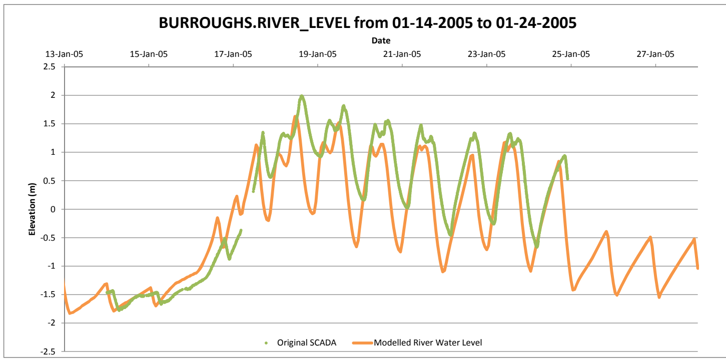

NICK_DITCH.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Processed SCADA
Datum Adjusted SCADA
Modelled Water Level (Cell 4)

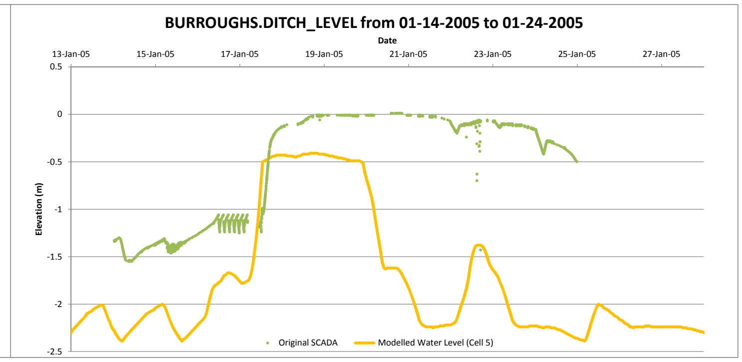

LOG-DTCH.RIVER_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled River Water Level

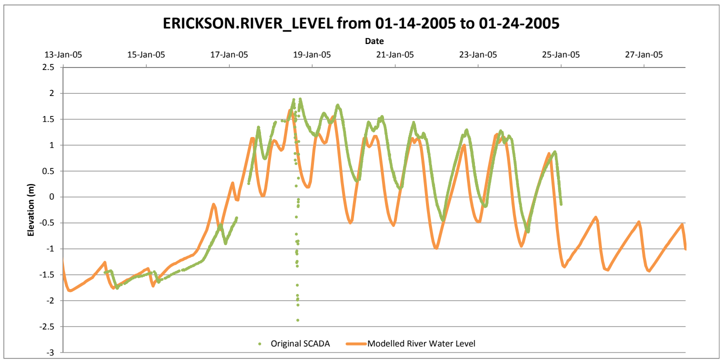

LOG-DTCH.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 5)

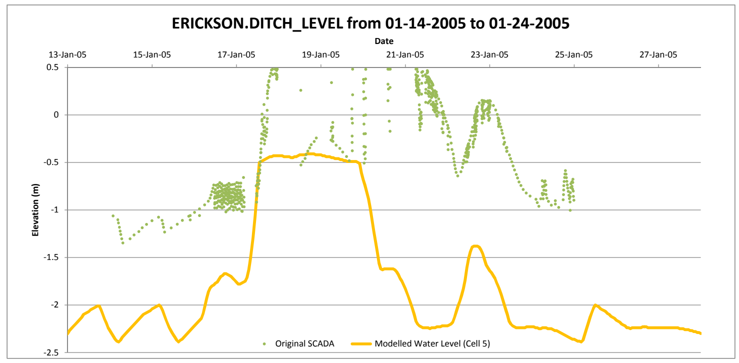

SOUTH_CLOVER.RIVER_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled River Water Level

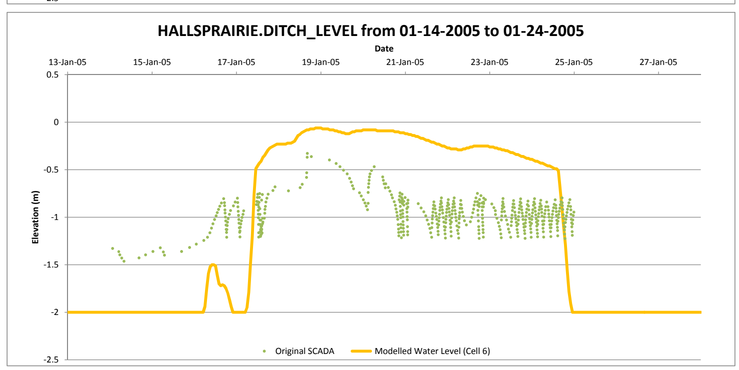

SOUTH_CLOVER.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 6)

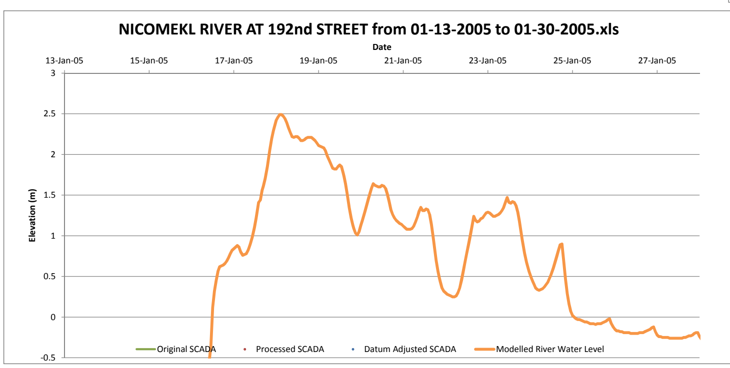

BURROUGHS.RIVER_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled River Water Level


BURROUGHS.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 5)


ERICKSON.RIVER_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled River Water Level


ERICKSON.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 5)


HALLSPRAIRIE.DITCH_LEVEL from 01-14-2005 to 01-24-2005
Date
Elevation (m)
Original SCADA
Modelled Water Level (Cell 6)


NICOMEKL RIVER AT 192nd STREET from 01-13-2005 to 01-30-2005.xls
Date
Elevation (m)
Original SCADA
Processed SCADA
Datum Adjusted SCADA
Modelled River Water Level

APPENDIX E

FREQUENCY ANALYSIS

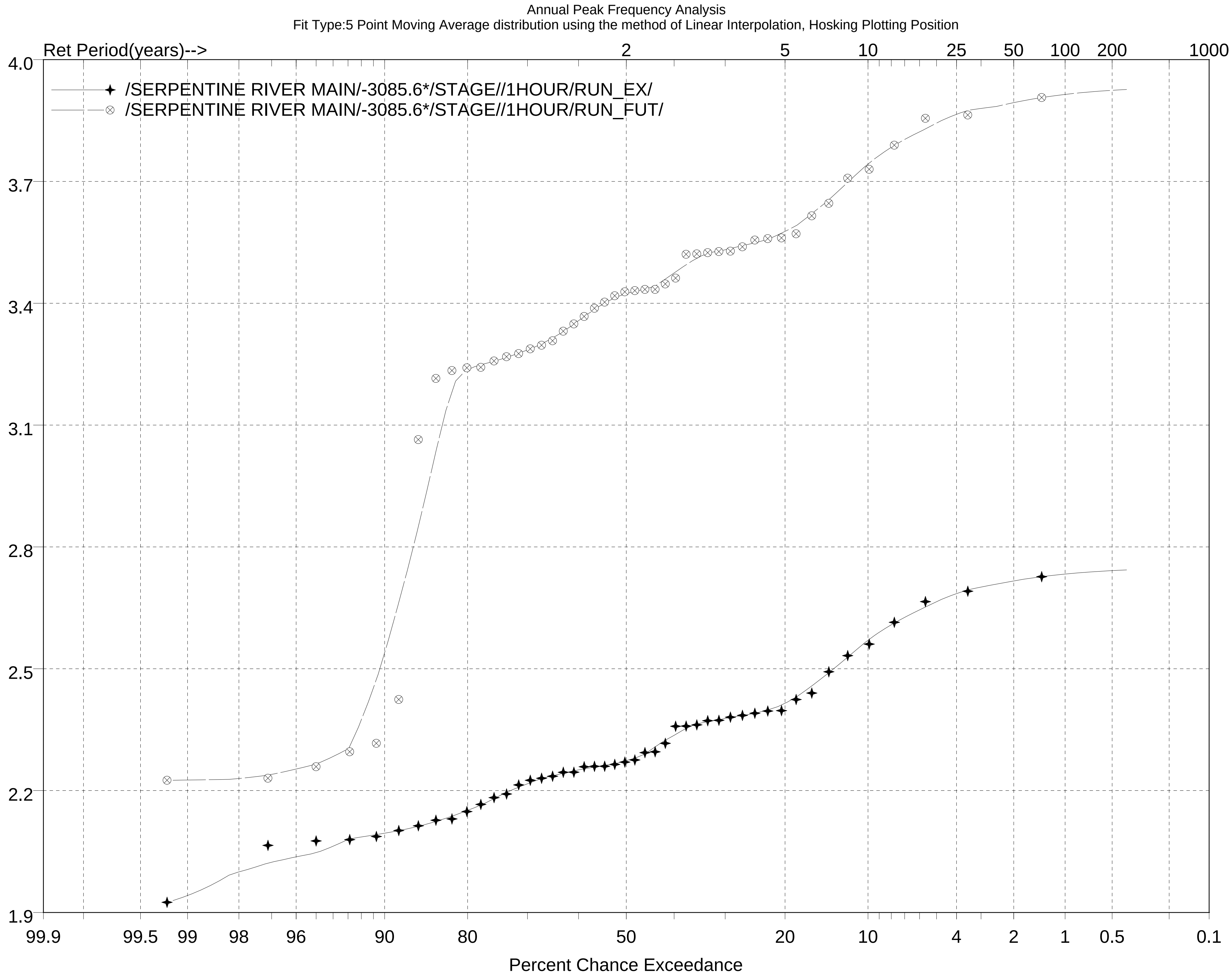
northwest hydraulic consultants

TABLE OF CONTENTS

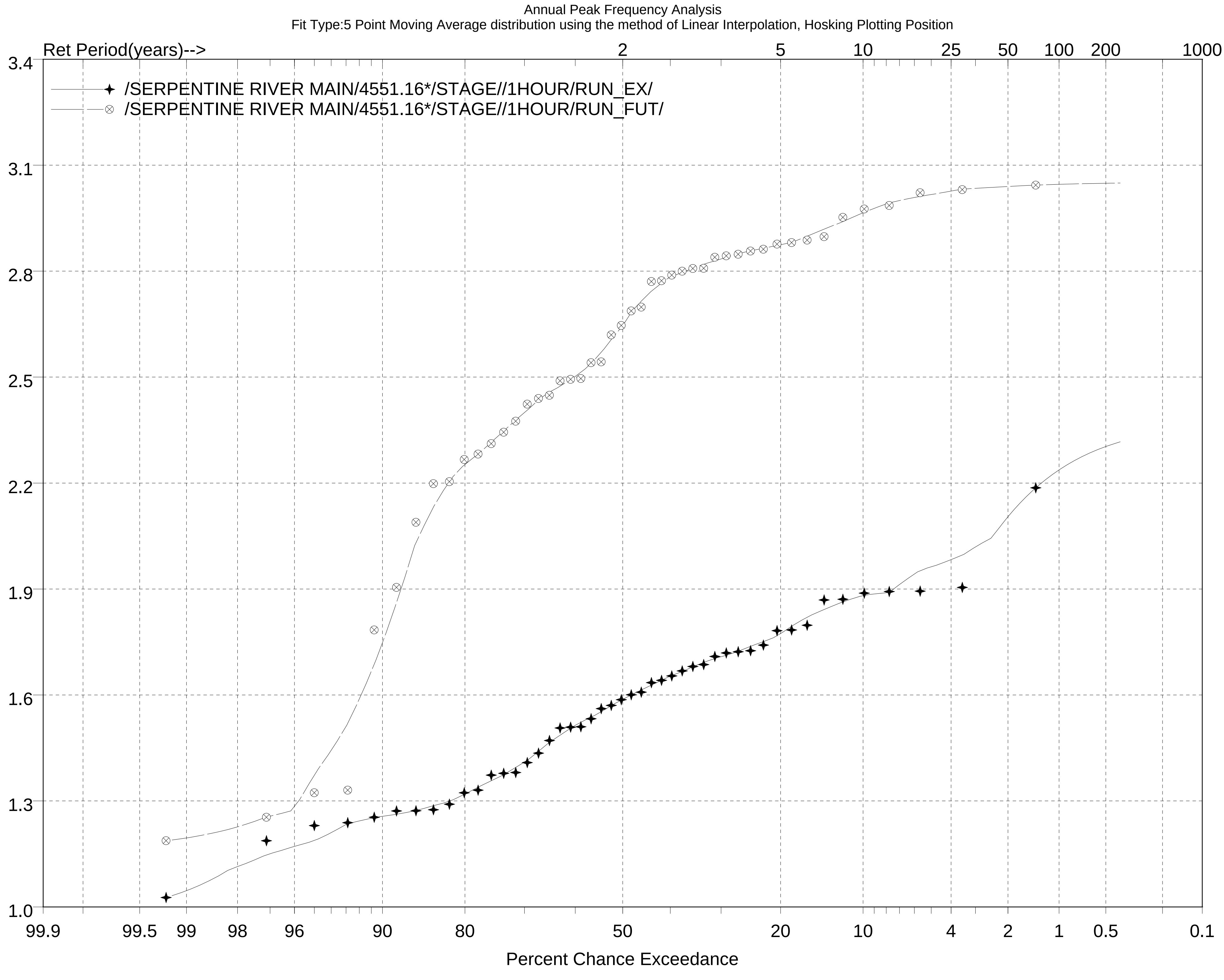
1 APPENDIX E.1. – FREQUENCY ANALYSIS PLOTS

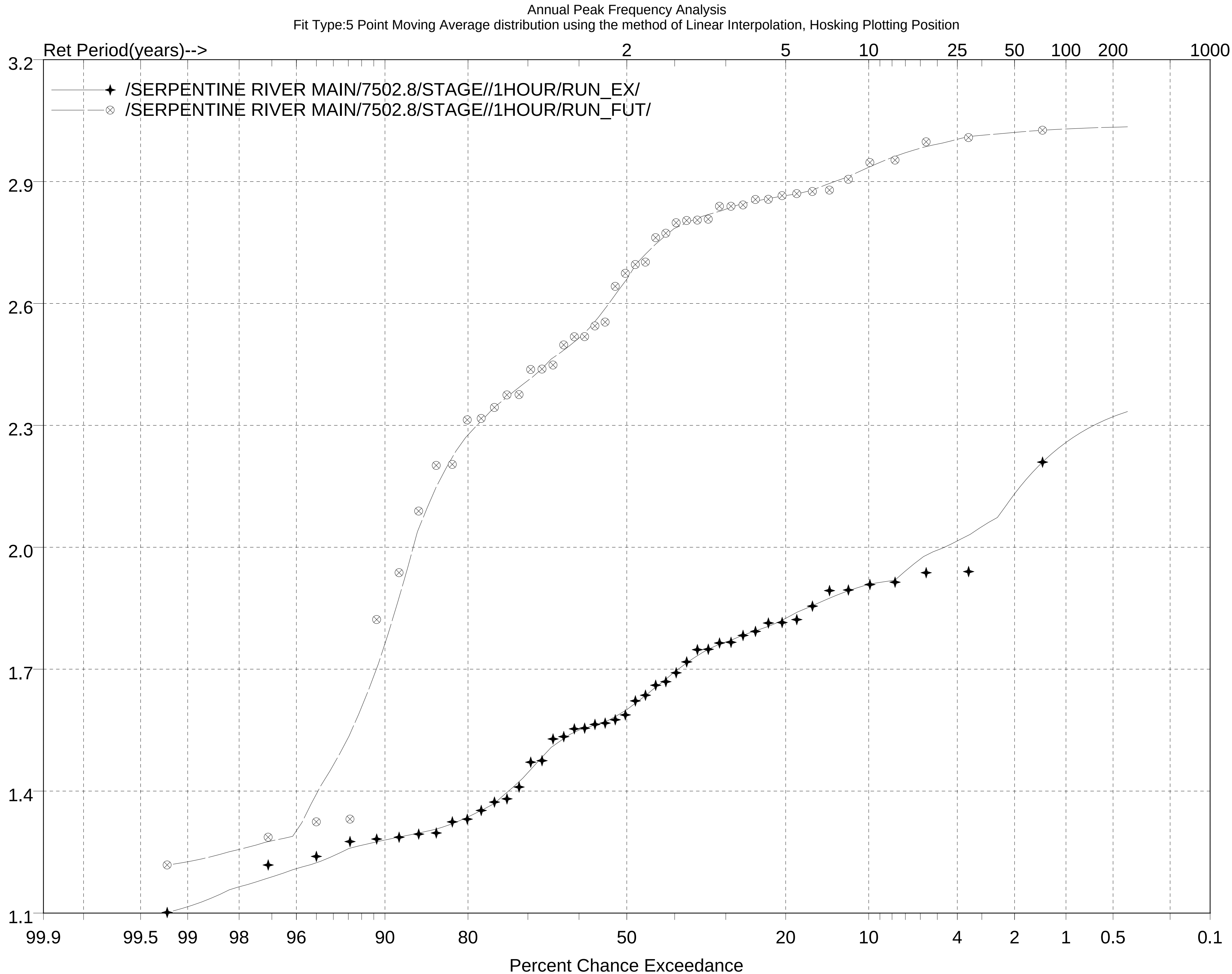
Annual maximum flood levels for the two time series generated for historic (present) and projected future (year 2100) conditions were analyzed in frequency analyses and 200-year return period flood levels were estimated at the key locations. The present 200-year levels were compared with the equivalent estimated year 2100 levels to assess the impacts of the projected relative sea level rise and landuse change on flood levels.

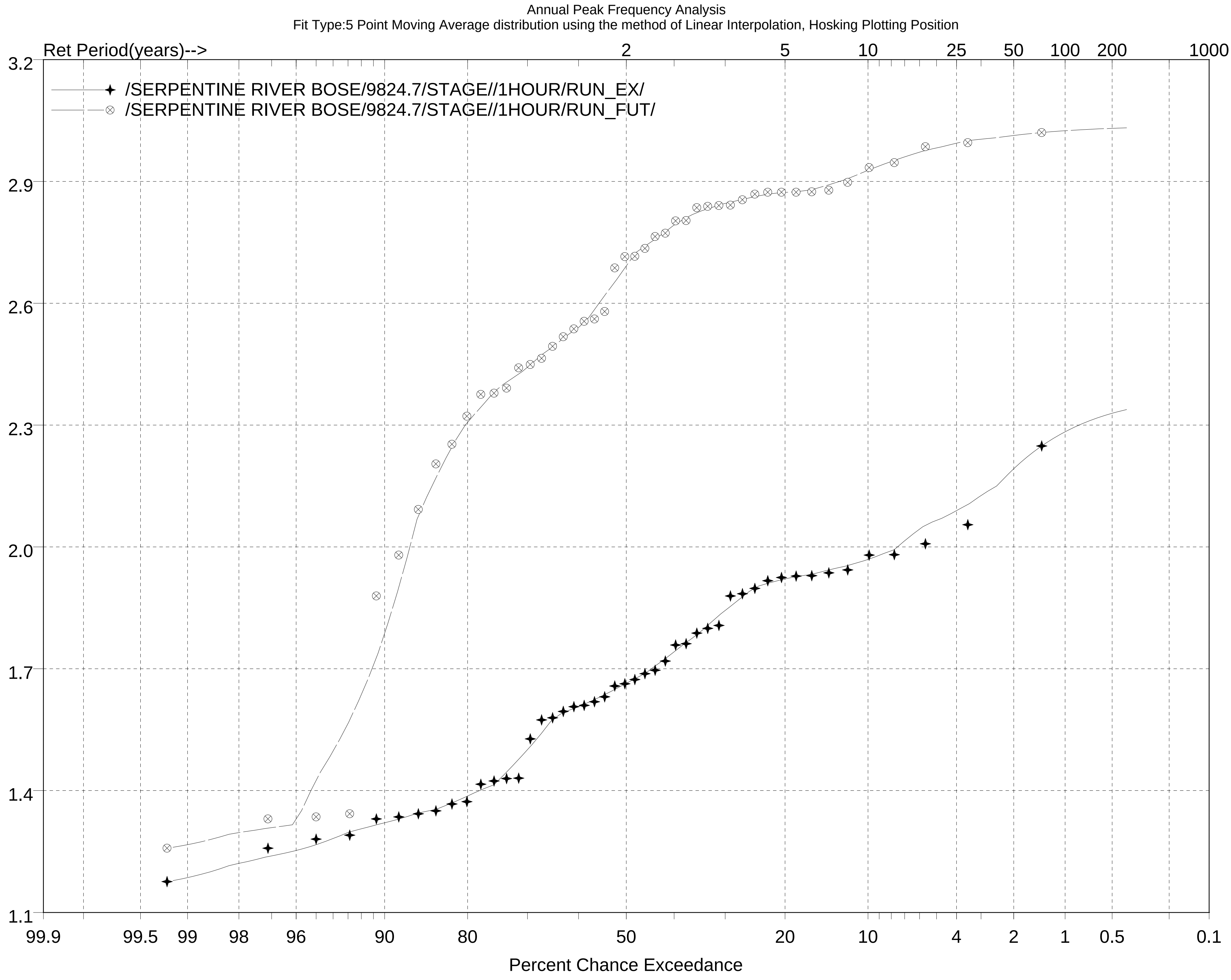
NHC's in-house frequency analysis package 'DASH' was applied for the frequency analyses. The following figures show the frequency analysis plots of the peak annual water levels for both existing and future conditions. The 200-year water levels were extrapolated using a 5-point moving average fit to the data.



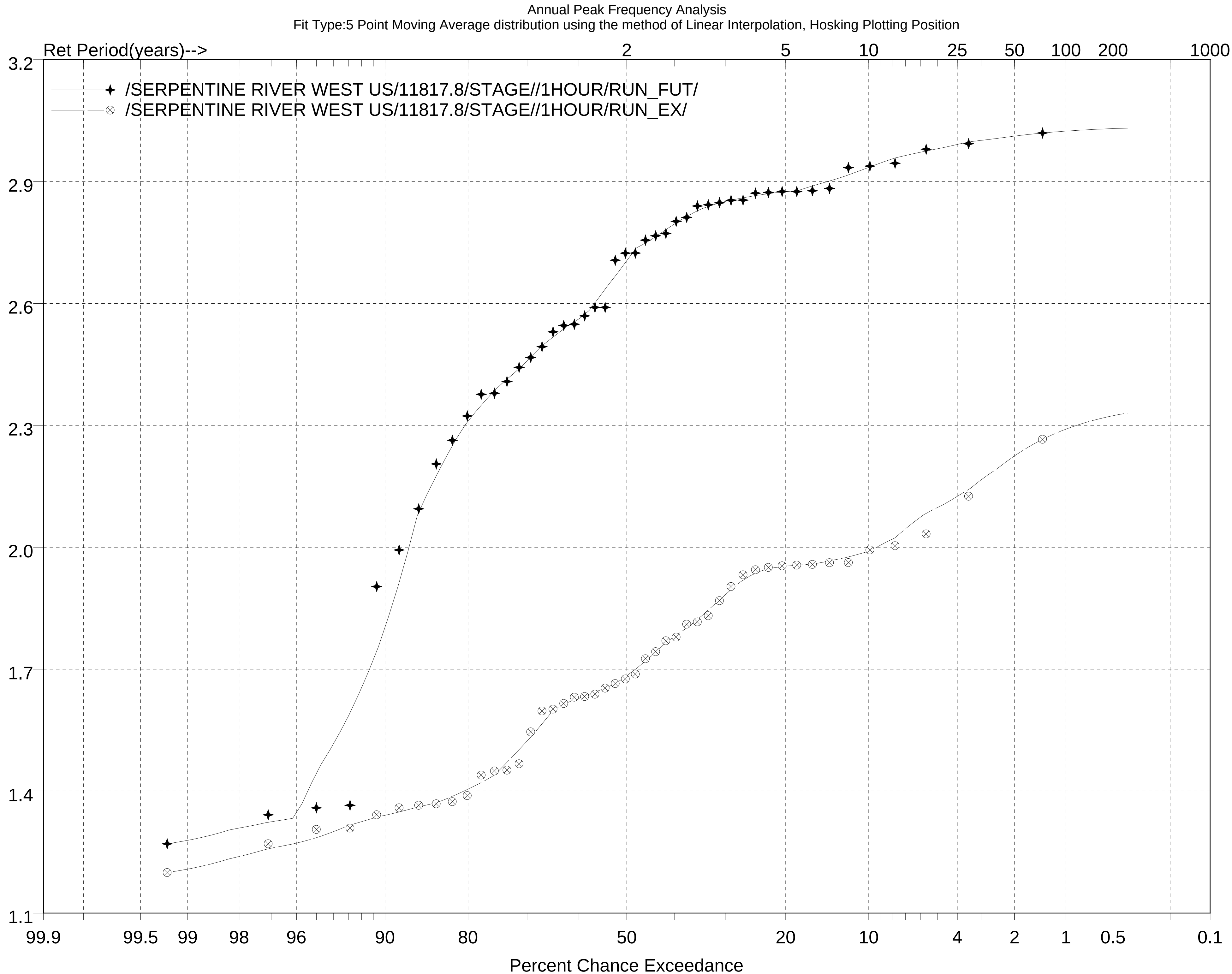
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER MAIN/-3085.6*/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER MAIN/-3085.6*/STAGE//1HOUR/RUN_FUT/
4.0
3.7
3.4
3.1
2.8
2.5
2.2
1.9
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

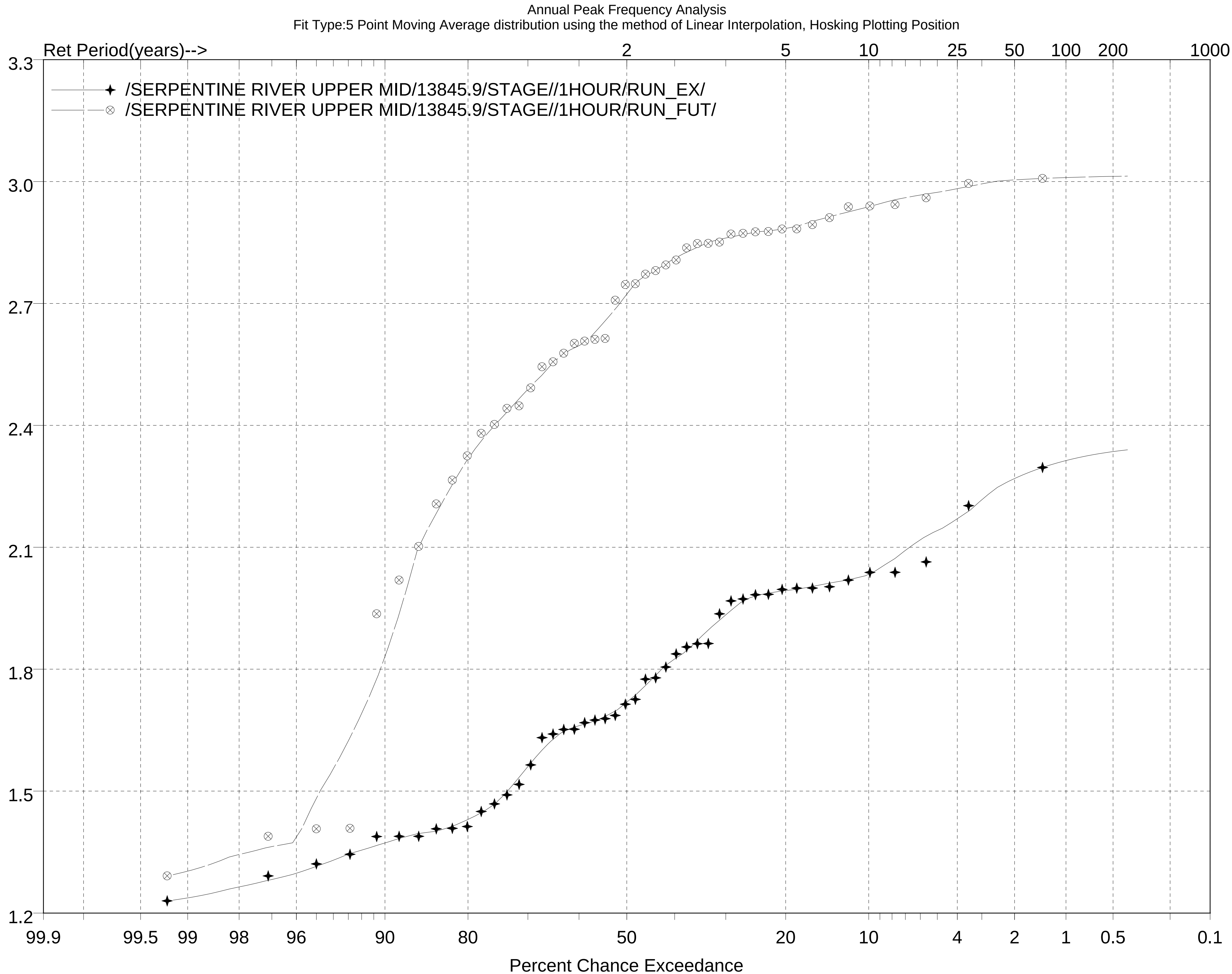

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER MAIN/4551.16*/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER MAIN/4551.16*/STAGE//1HOUR/RUN_FUT/
3.4
3.1
2.8
2.5
2.2
1.9
1.6
1.3
1.0
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance


Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER MAIN/7502.8/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER MAIN/7502.8/STAGE//1HOUR/RUN_FUT/
3.2
2.9
2.6
2.3
2.0
1.7
1.4
1.1
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

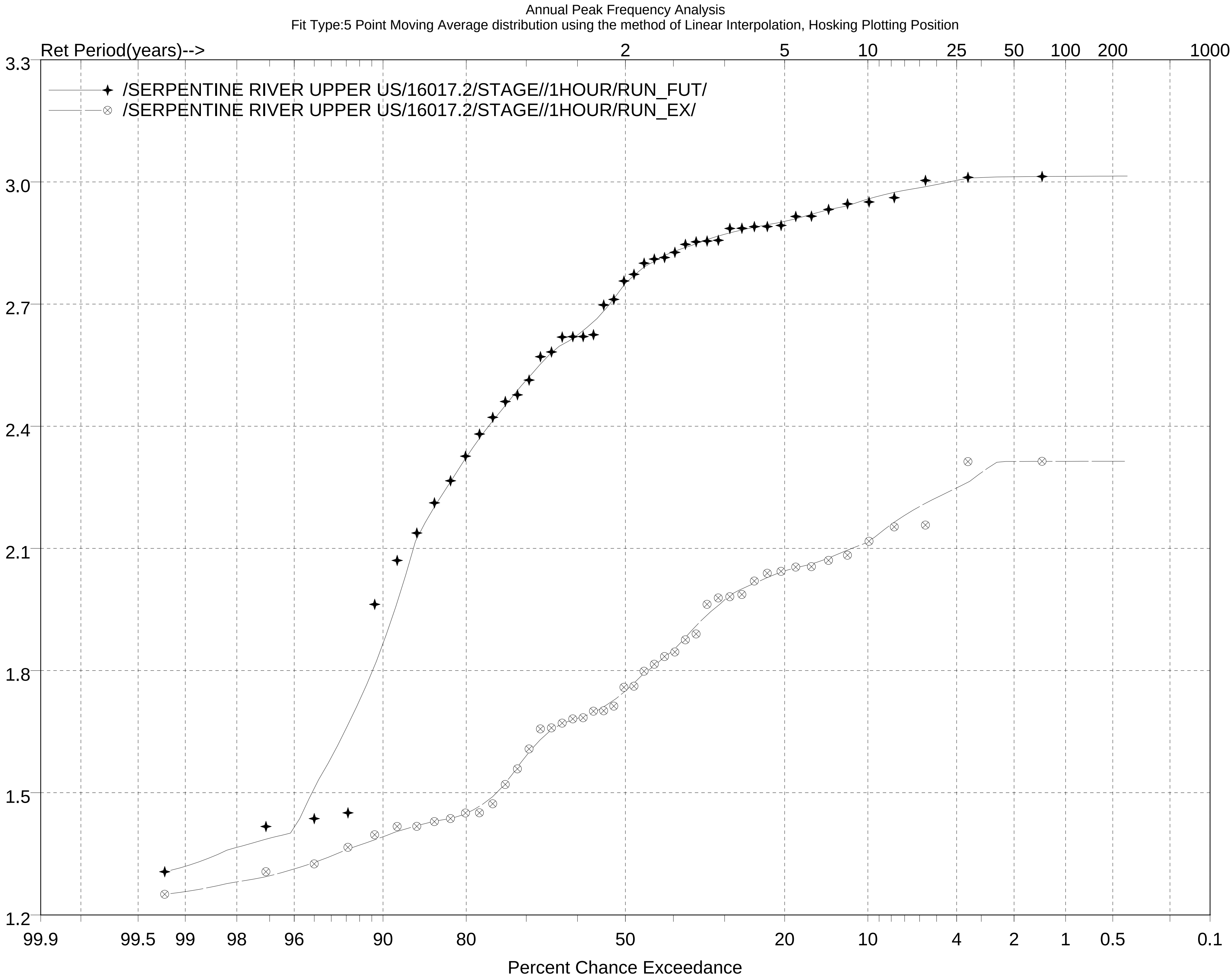


Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER BOSE/9824.7/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER BOSE/9824.7/STAGE//1HOUR/RUN_FUT/
3.2
2.9
2.6
2.3
2.0
1.7
1.4
1.1
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

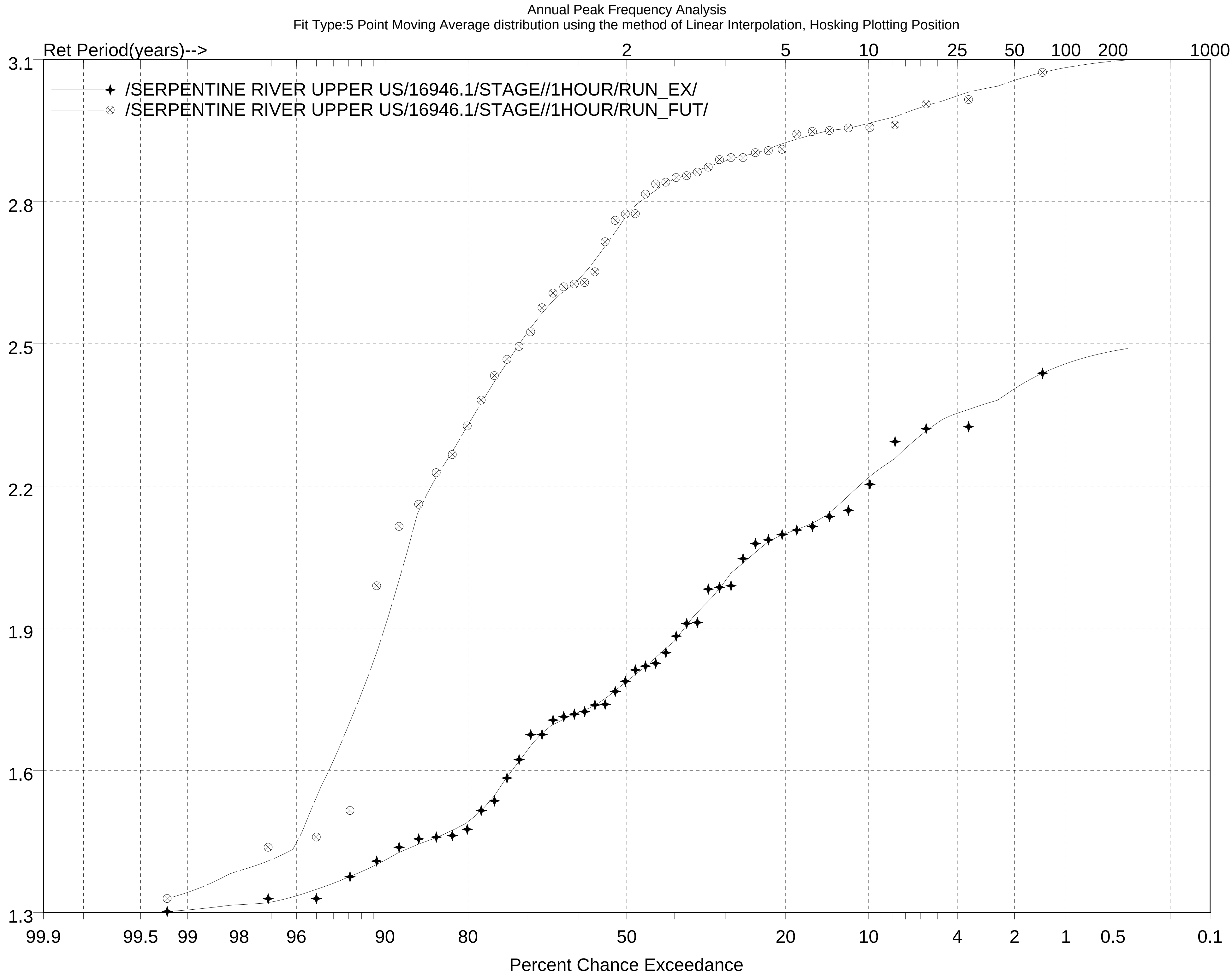

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER WEST US/11817.8/STAGE//1HOUR/RUN_FUT/
/SERPENTINE RIVER WEST US/11817.8/STAGE//1HOUR/RUN_EX/
3.2
2.9
2.6
2.3
2.0
1.7
1.4
1.1
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

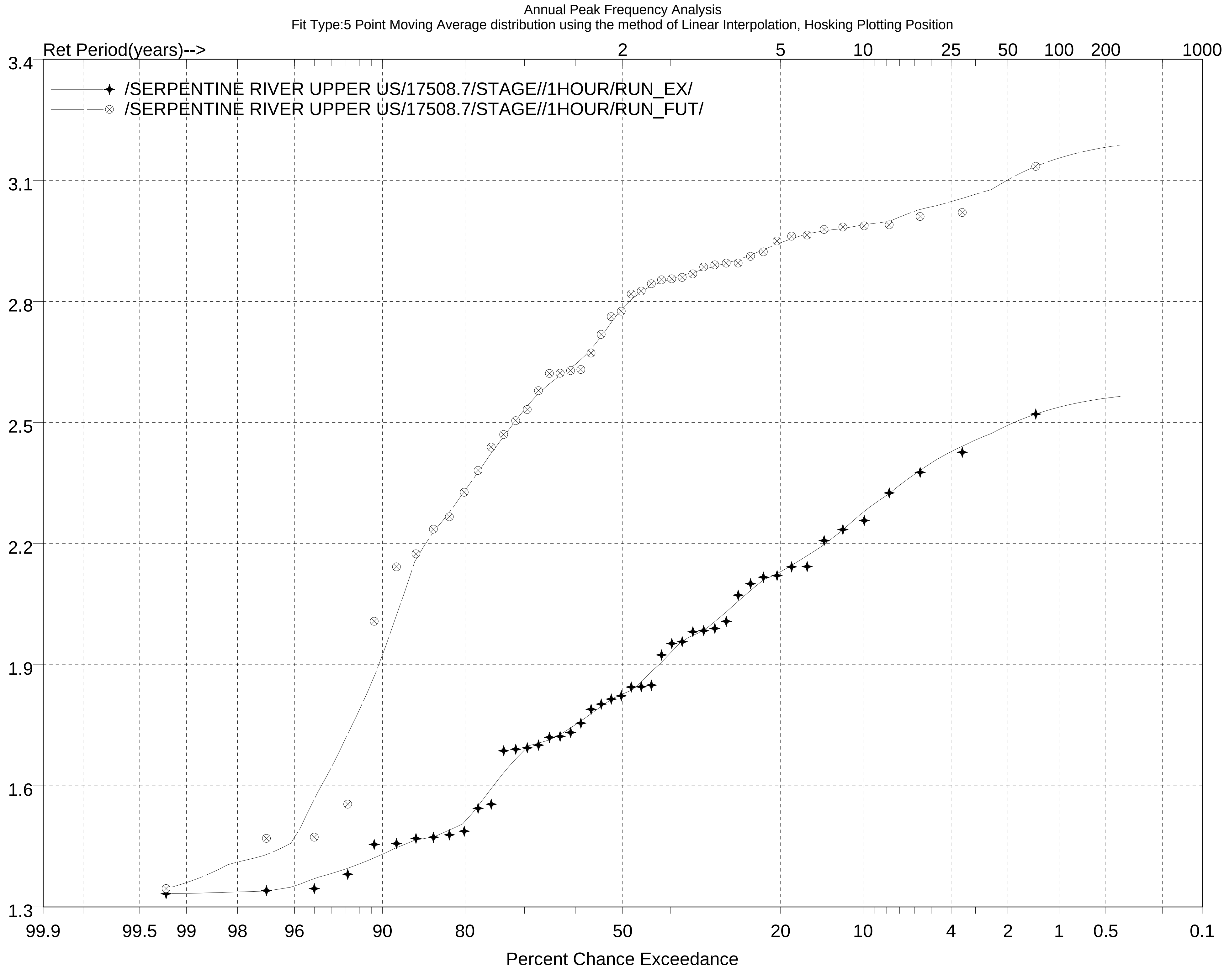


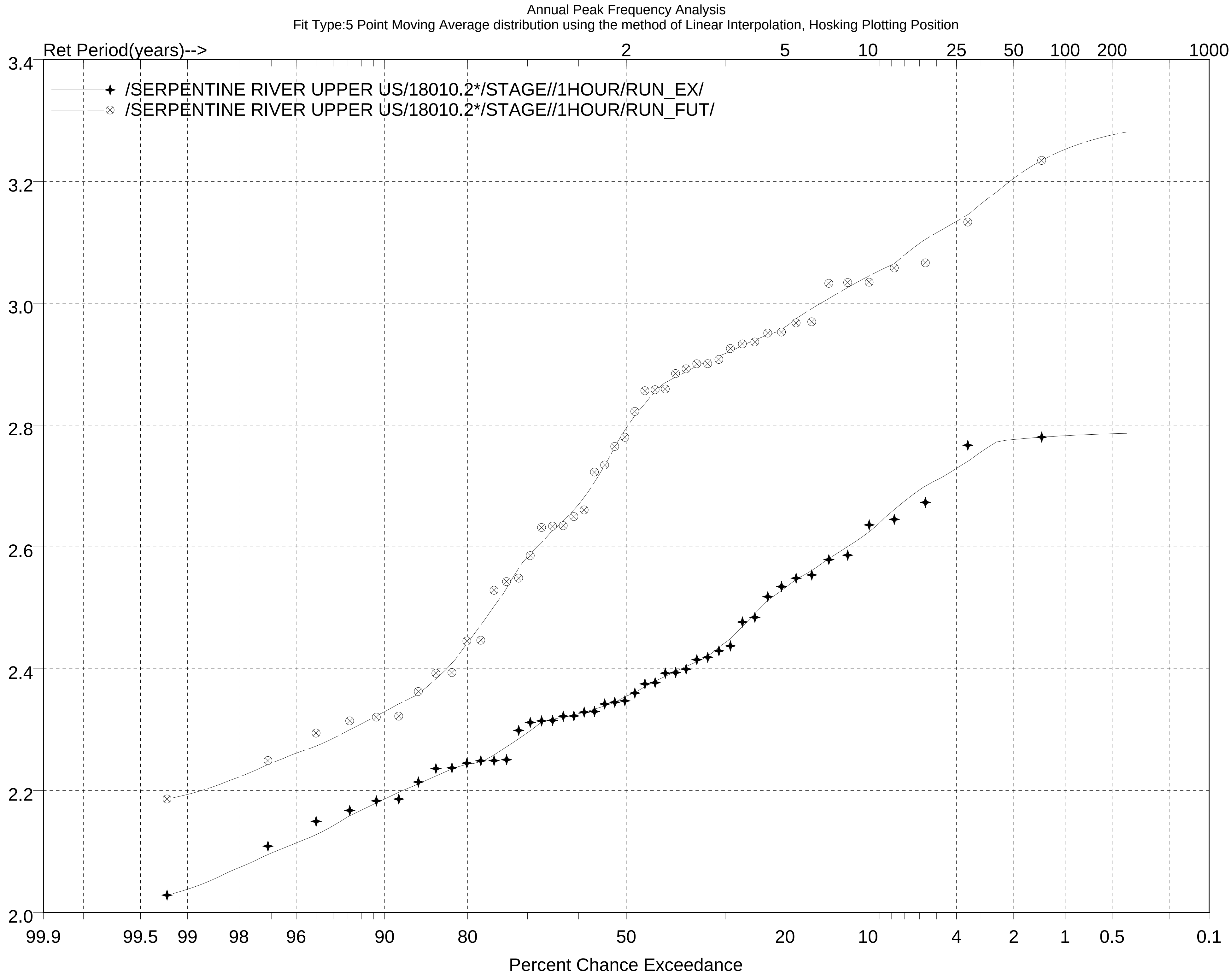
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER UPPER MID/13845.9/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER UPPER MID/13845.9/STAGE//1HOUR/RUN_FUT/
3.3
3.0
2.7
2.4
2.1
1.8
1.5
1.2
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



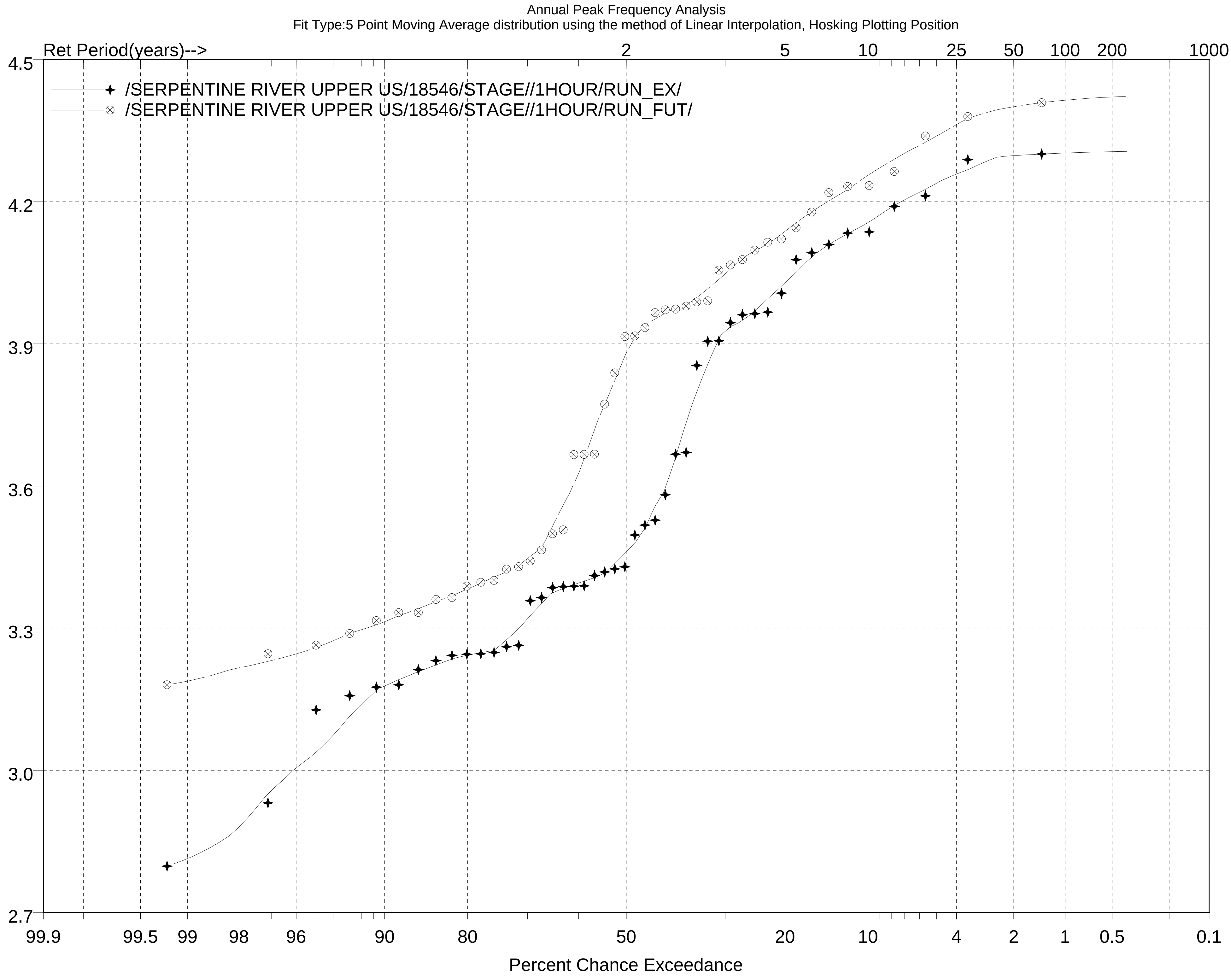
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER UPPER US/16017.2/STAGE//1HOUR/RUN_FUT/
/SERPENTINE RIVER UPPER US/16017.2/STAGE//1HOUR/RUN_EX/
3.3
3.0
2.7
2.4
2.1
1.8
1.5
1.2
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

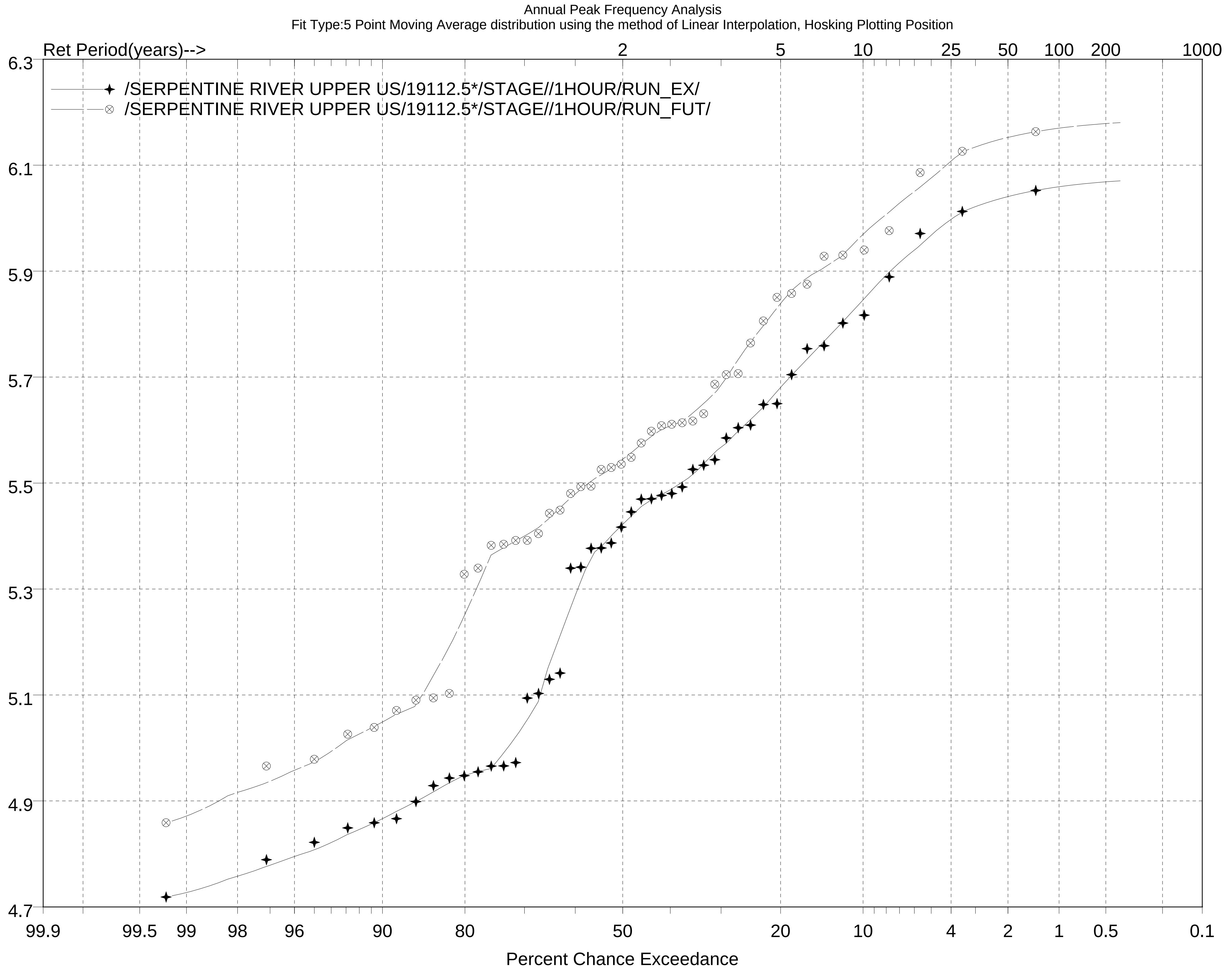

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER UPPER US/16946.1/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER UPPER US/16946.1/STAGE//1HOUR/RUN_FUT/
3.1
2.8
2.5
2.2
1.9
1.6
1.3
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

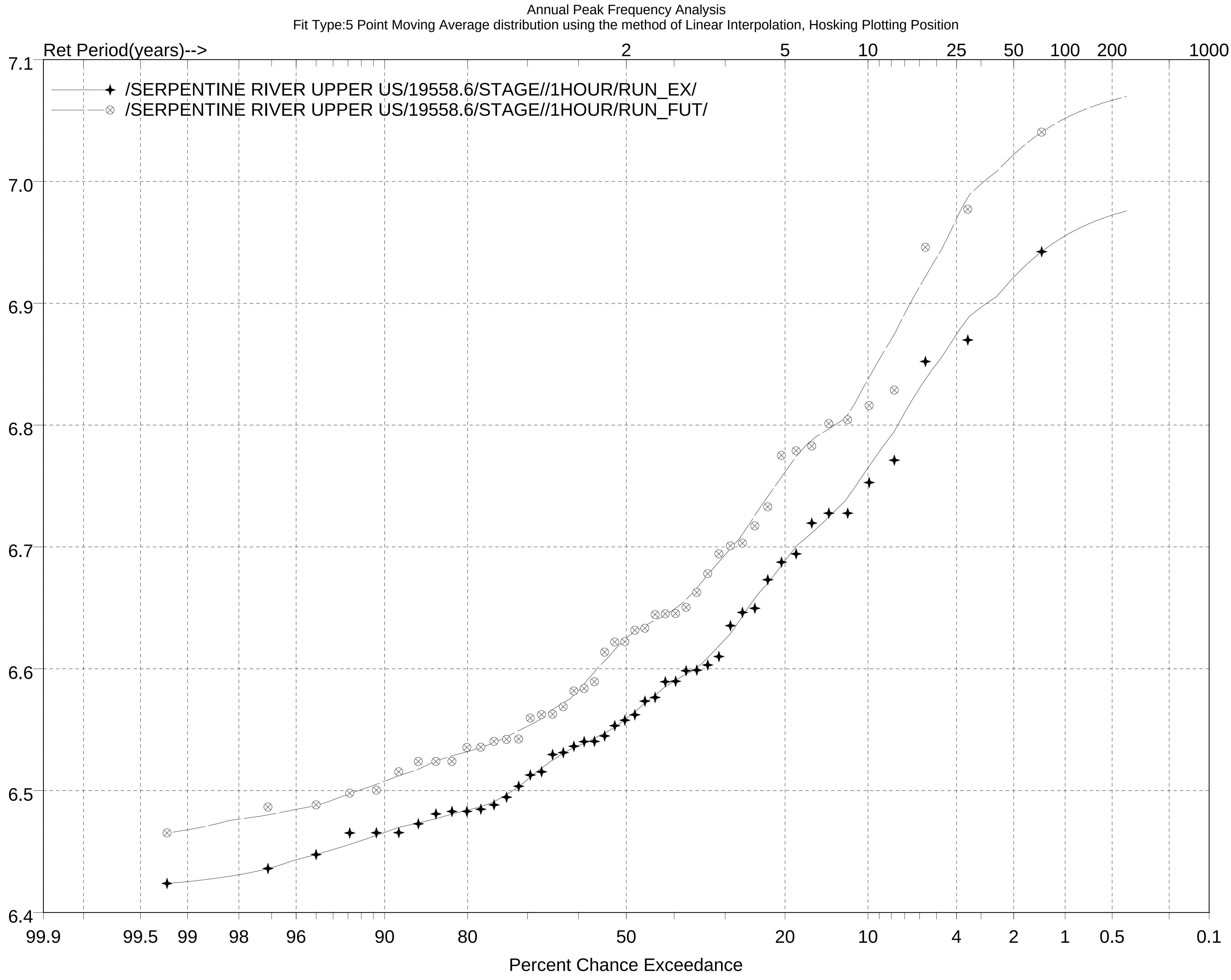

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER UPPER US/17508.7/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER UPPER US/17508.7/STAGE//1HOUR/RUN_FUT/
3.4
3.1
2.8
2.5
2.2
1.9
1.6
1.3
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



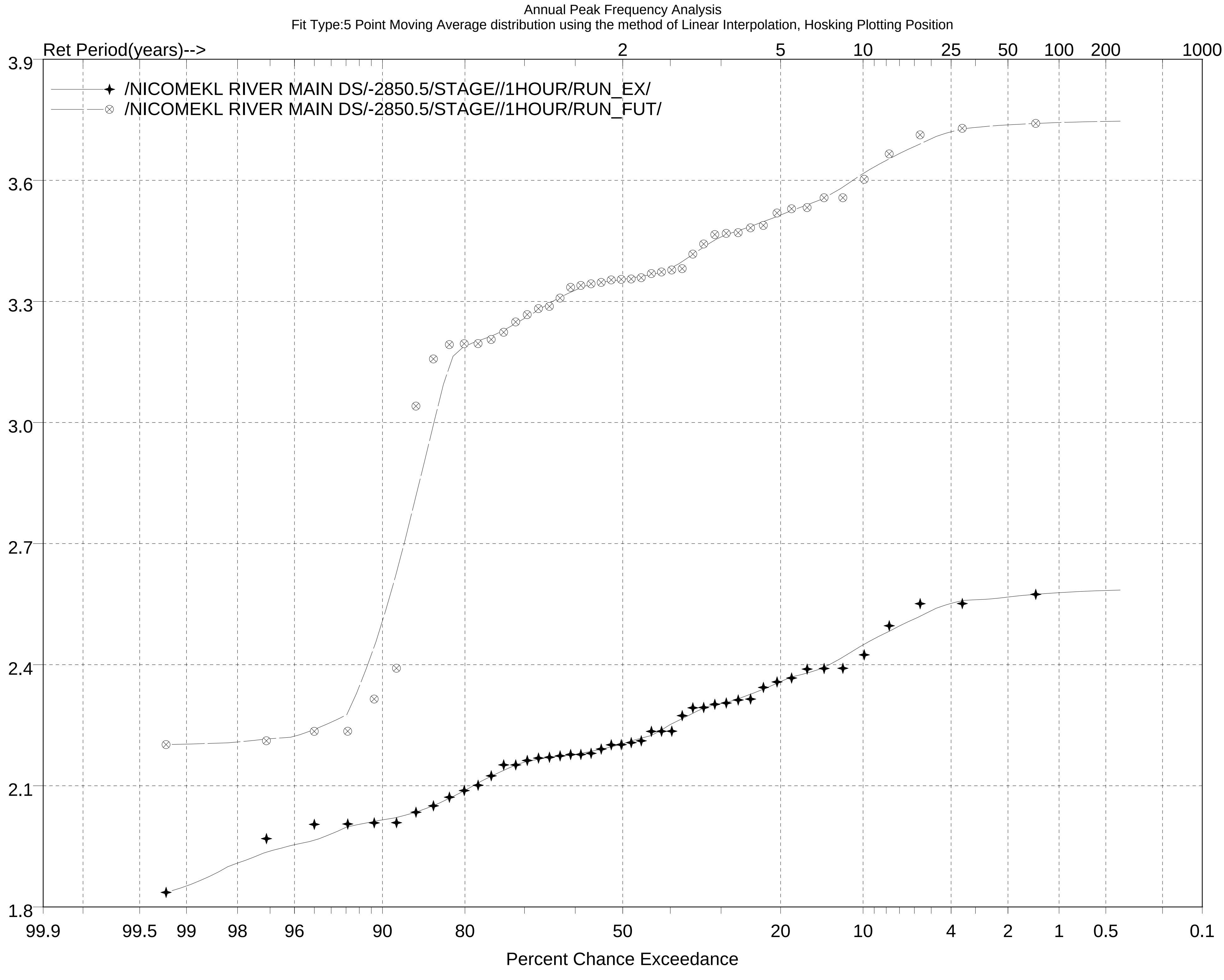
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER UPPER US/18010.2*/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER UPPER US/18010.2*/STAGE//1HOUR/RUN_FUT/
3.4
3.2
3.0
2.8
2.6
2.4
2.2
2.0
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance


Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER UPPER US/18546/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER UPPER US/18546/STAGE//1HOUR/RUN_FUT/
4.5
4.2
3.9
3.6
3.3
3.0
2.7
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

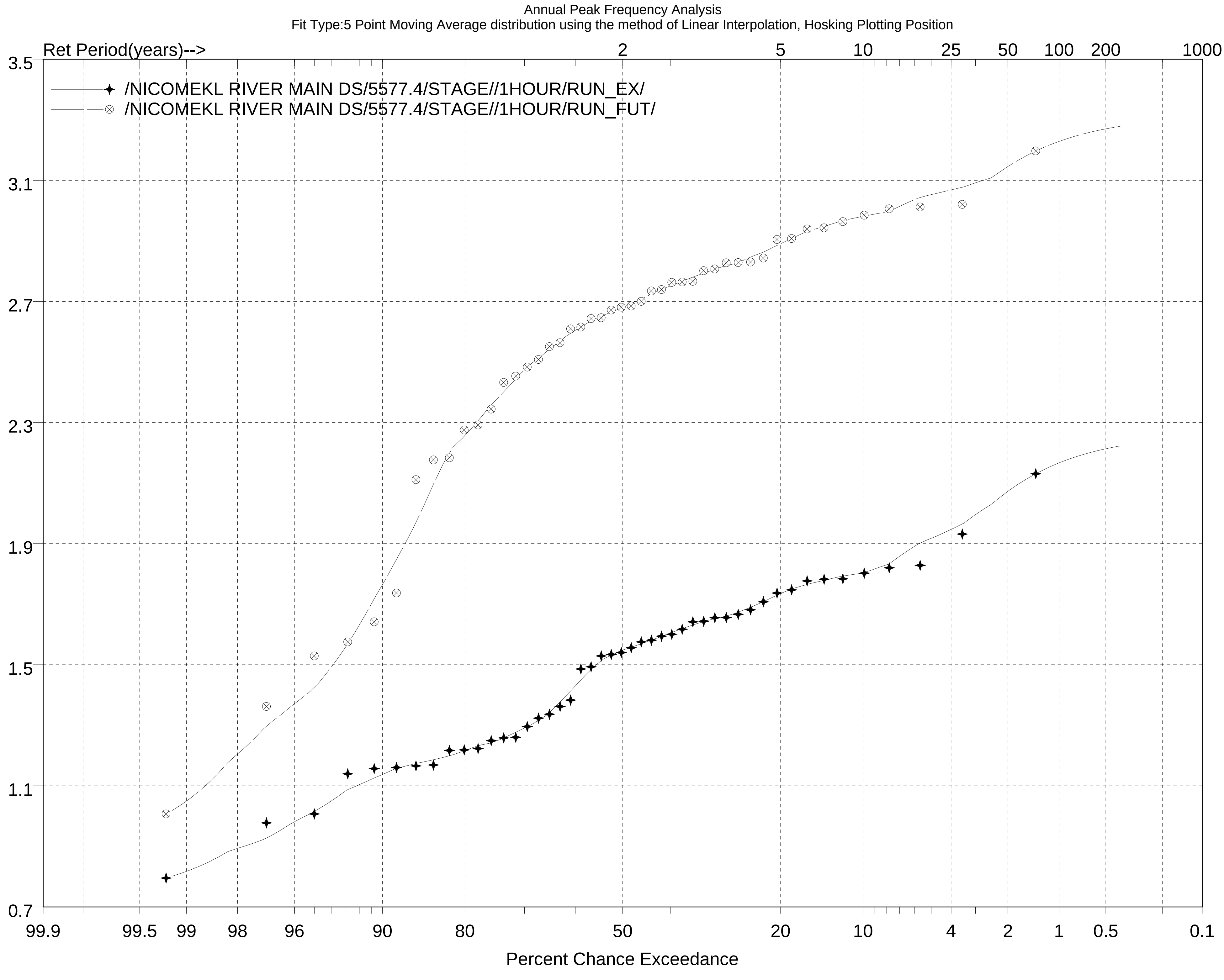

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER UPPER US/19112.5*/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER UPPER US/19112.5*/STAGE//1HOUR/RUN_FUT/
6.3
6.1
5.9
5.7
5.5
5.3
5.1
4.9
4.7
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



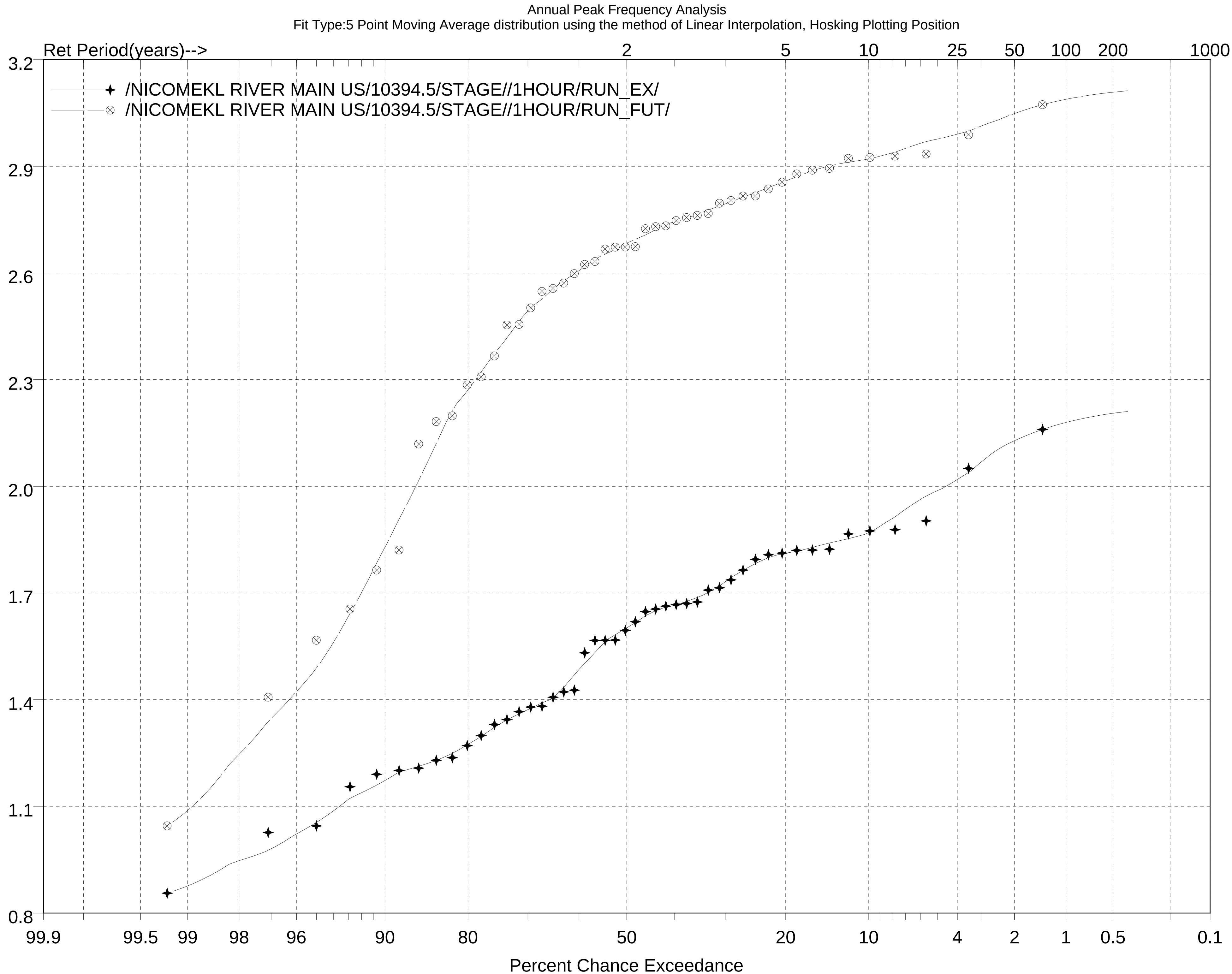
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/SERPENTINE RIVER UPPER US/19558.6/STAGE//1HOUR/RUN_EX/
/SERPENTINE RIVER UPPER US/19558.6/STAGE//1HOUR/RUN_FUT/
7.1
7.0
6.9
6.8
6.7
6.6
6.5
6.4
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



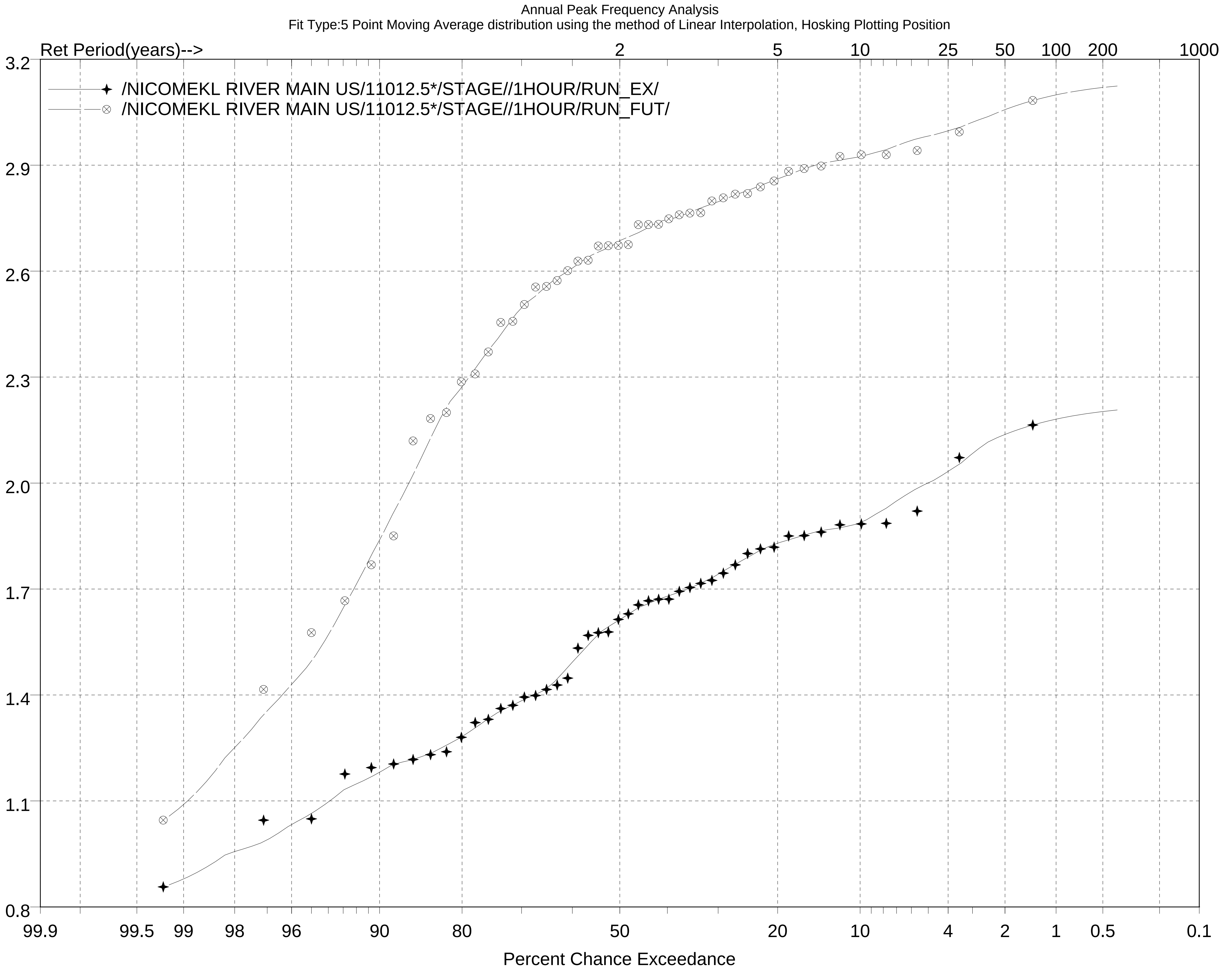
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/NICOMEKL RIVER MAIN DS/-2850.5/STAGE//1HOUR/RUN_EX/
/NICOMEKL RIVER MAIN DS/-2850.5/STAGE//1HOUR/RUN_FUT/
3.9
3.6
3.3
3.0
2.7
2.4
2.1
1.8
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

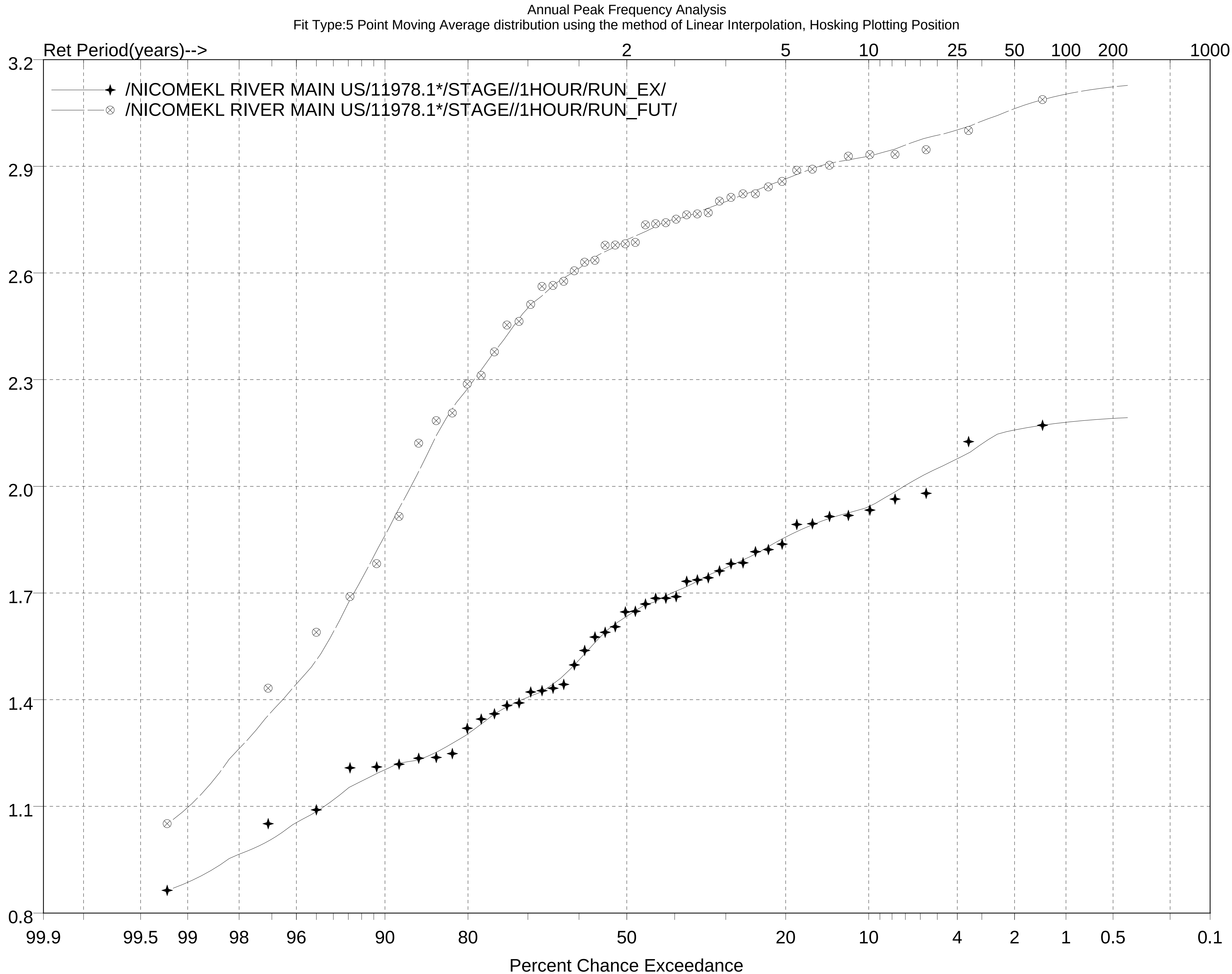


Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/NICOMEKL RIVER MAIN DS/5577.4/STAGE//1HOUR/RUN_EX/
/NICOMEKL RIVER MAIN DS/5577.4/STAGE//1HOUR/RUN_FUT/
3.5
3.1
2.7
2.3
1.9
1.5
1.1
0.7
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

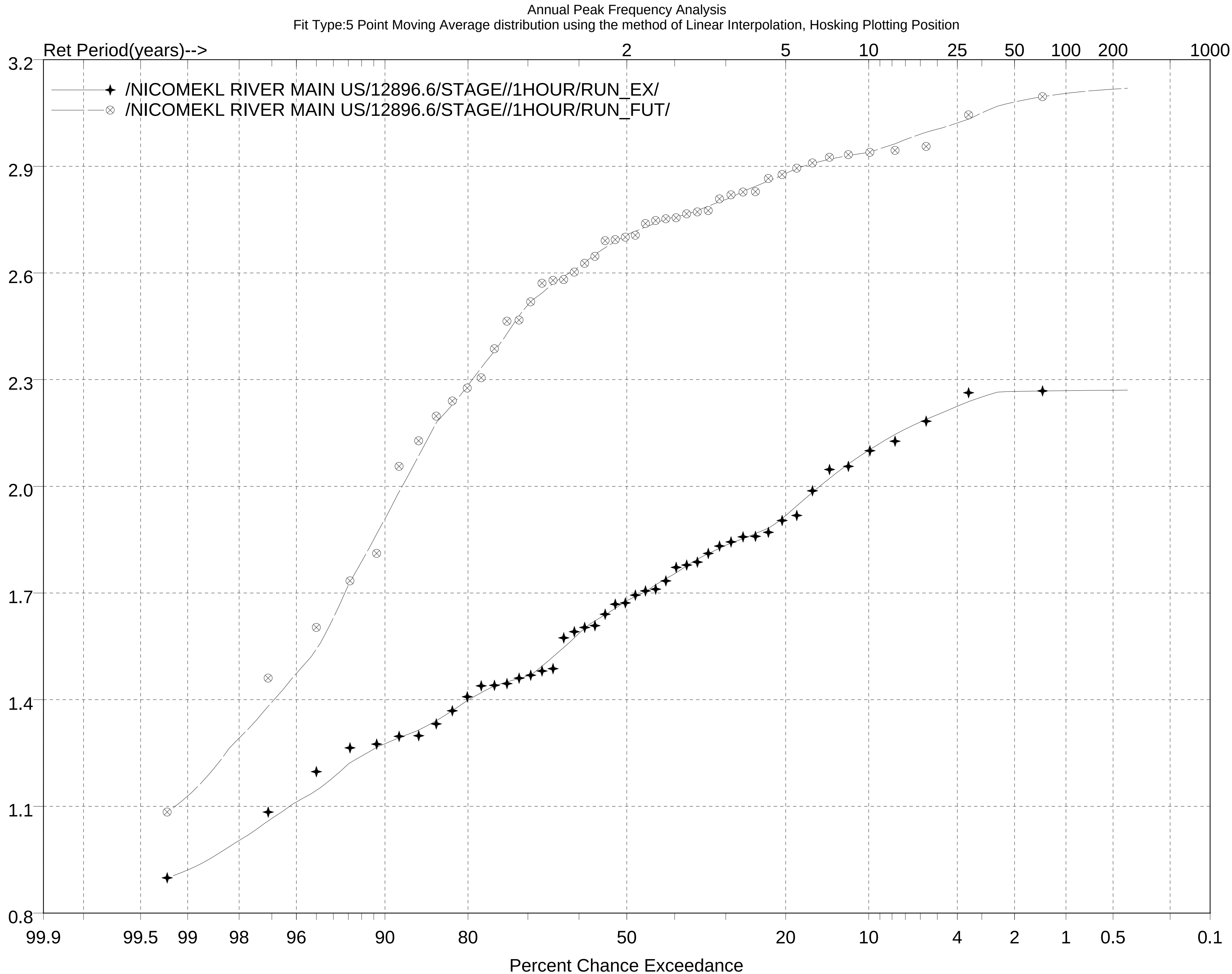


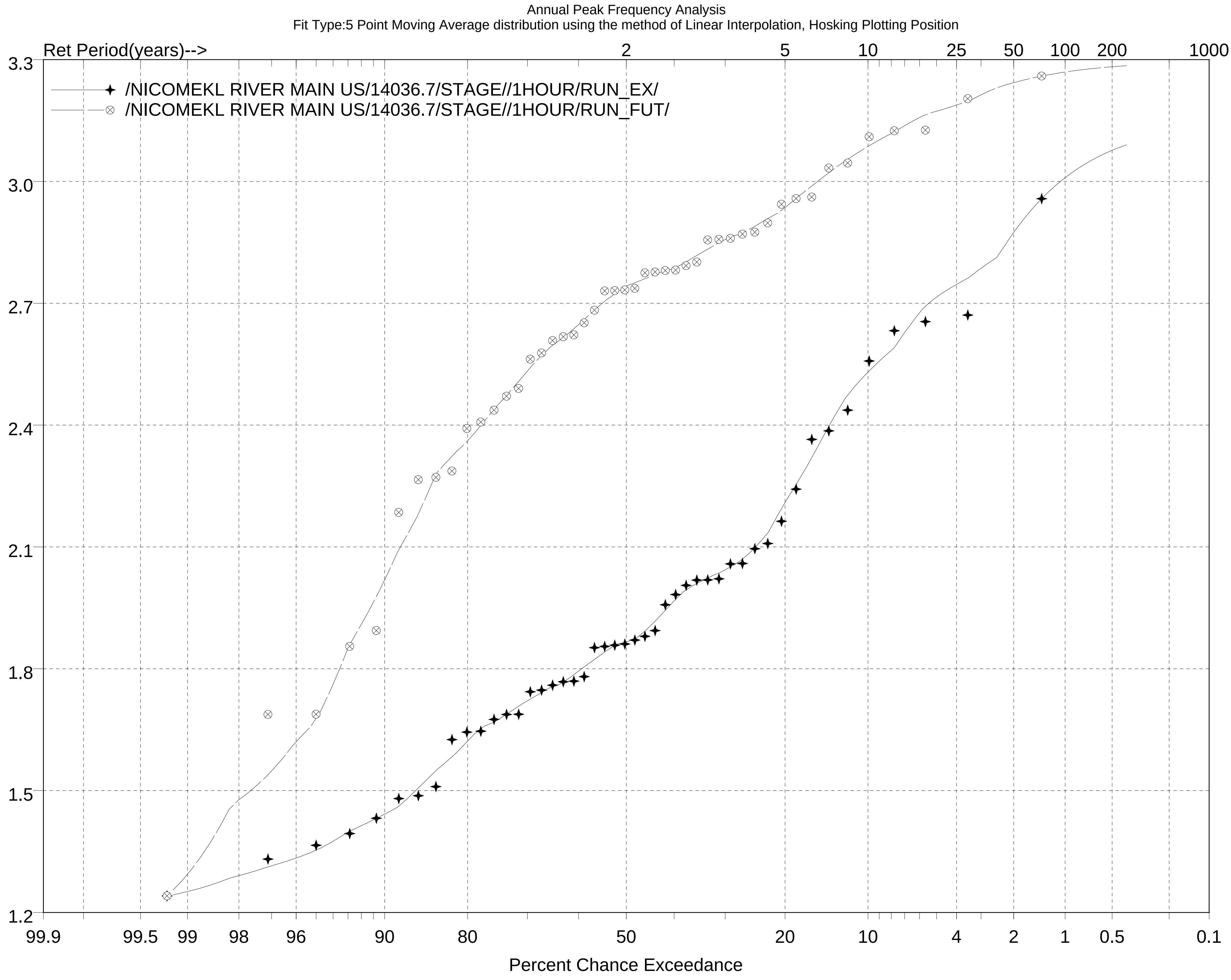
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/NICOMEKL RIVER MAIN US/10394.5/STAGE//1HOUR/RUN_EX/
/NICOMEKL RIVER MAIN US/10394.5/STAGE//1HOUR/RUN_FUT/
3.2
2.9
2.6
2.3
2.0
1.7
1.4
1.1
0.8
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

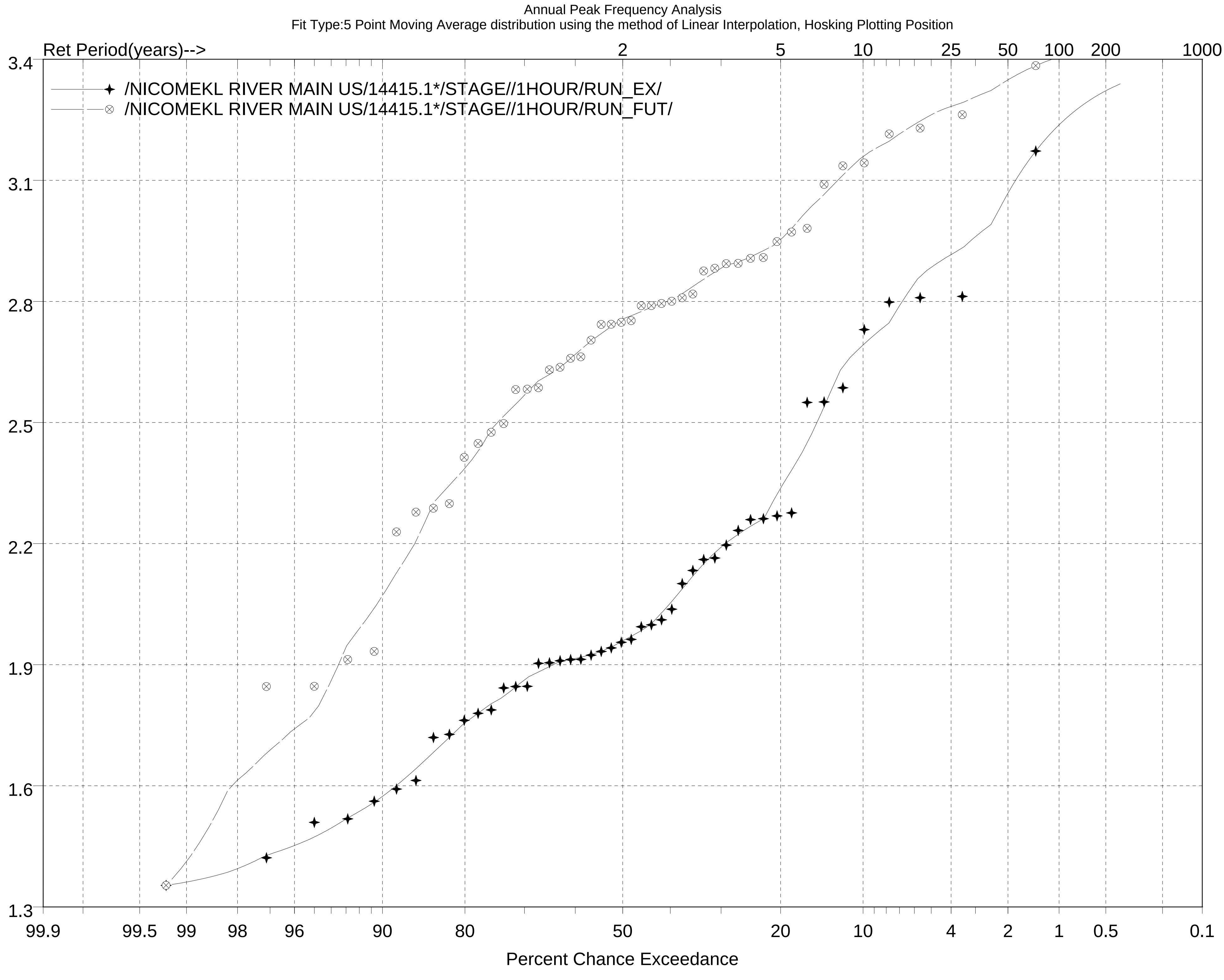

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/NICOMEKL RIVER MAIN US/11012.5*/STAGE//1HOUR/RUN_EX/
/NICOMEKL RIVER MAIN US/11012.5*/STAGE//1HOUR/RUN_FUT/
3.2
2.9
2.6
2.3
2.0
1.7
1.4
1.1
0.8
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



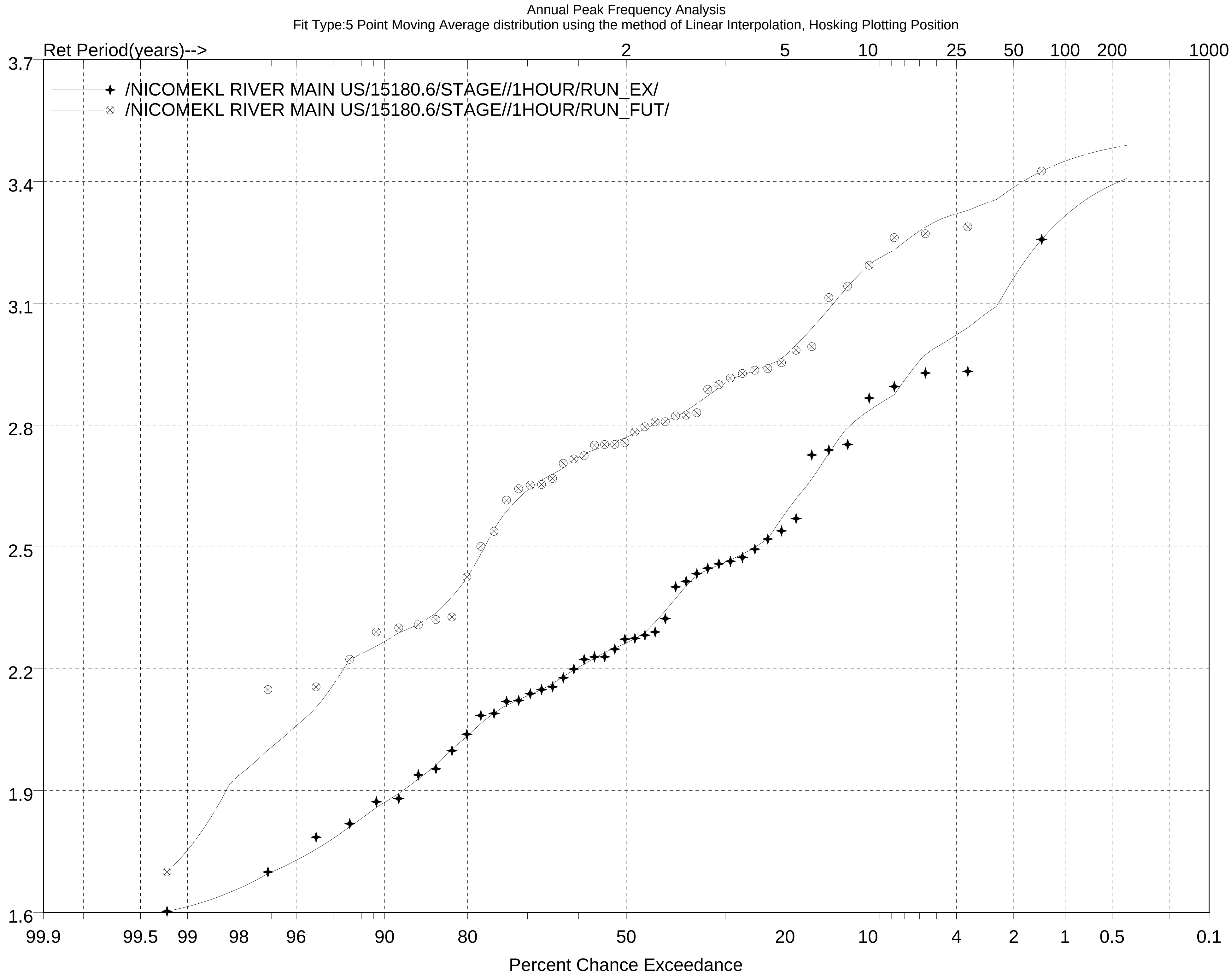
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/NICOMEKL RIVER MAIN US/11978.1*/STAGE//1HOUR/RUN_EX/
/NICOMEKL RIVER MAIN US/11978.1*/STAGE//1HOUR/RUN_FUT/
3.2
2.9
2.6
2.3
2.0
1.7
1.4
1.1
0.8
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

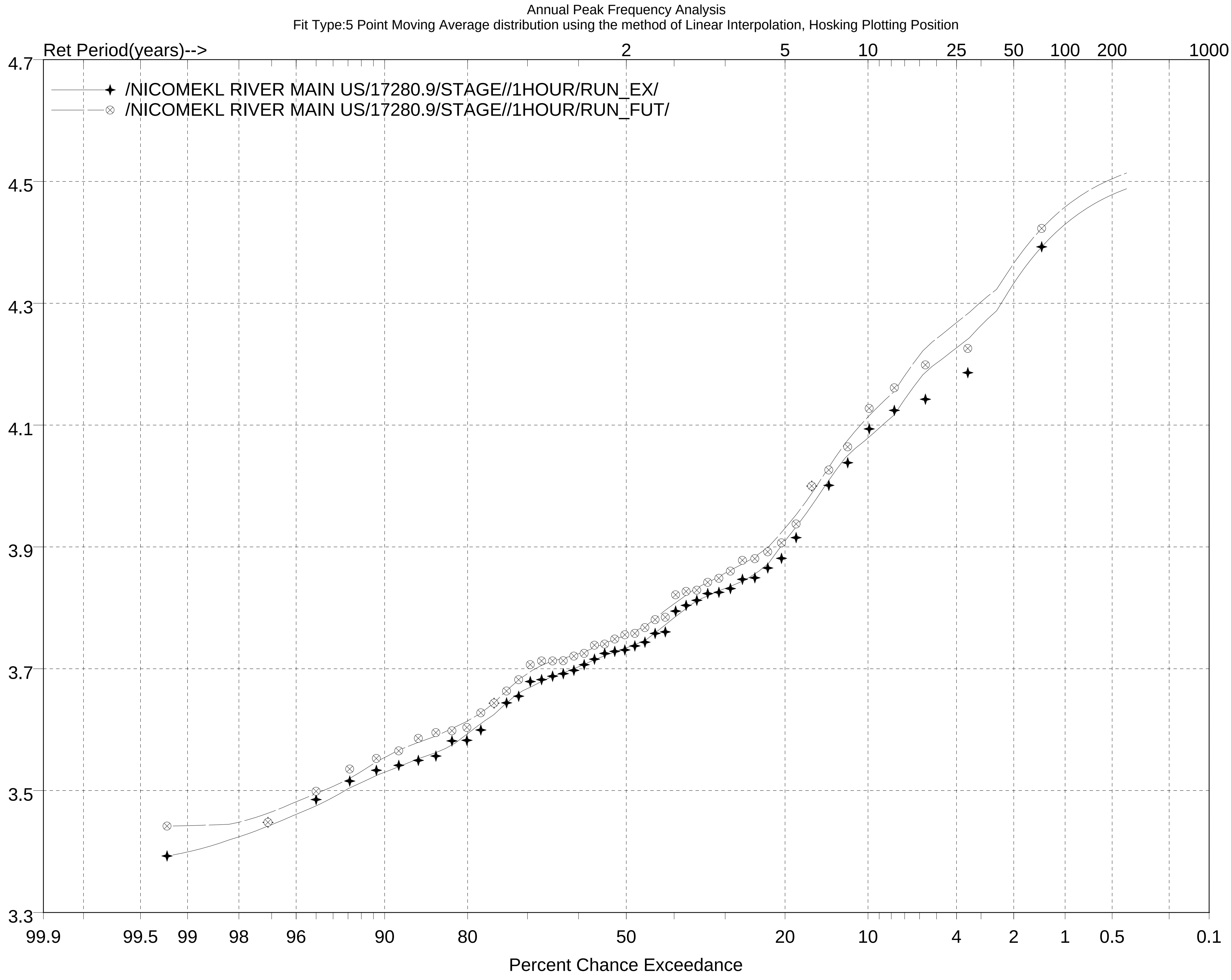

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/NICOMEKL RIVER MAIN US/12896.6/STAGE//1HOUR/RUN_EX/
/NICOMEKL RIVER MAIN US/12896.6/STAGE//1HOUR/RUN_FUT/
3.2
2.9
2.6
2.3
2.0
1.7
1.4
1.1
0.8
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

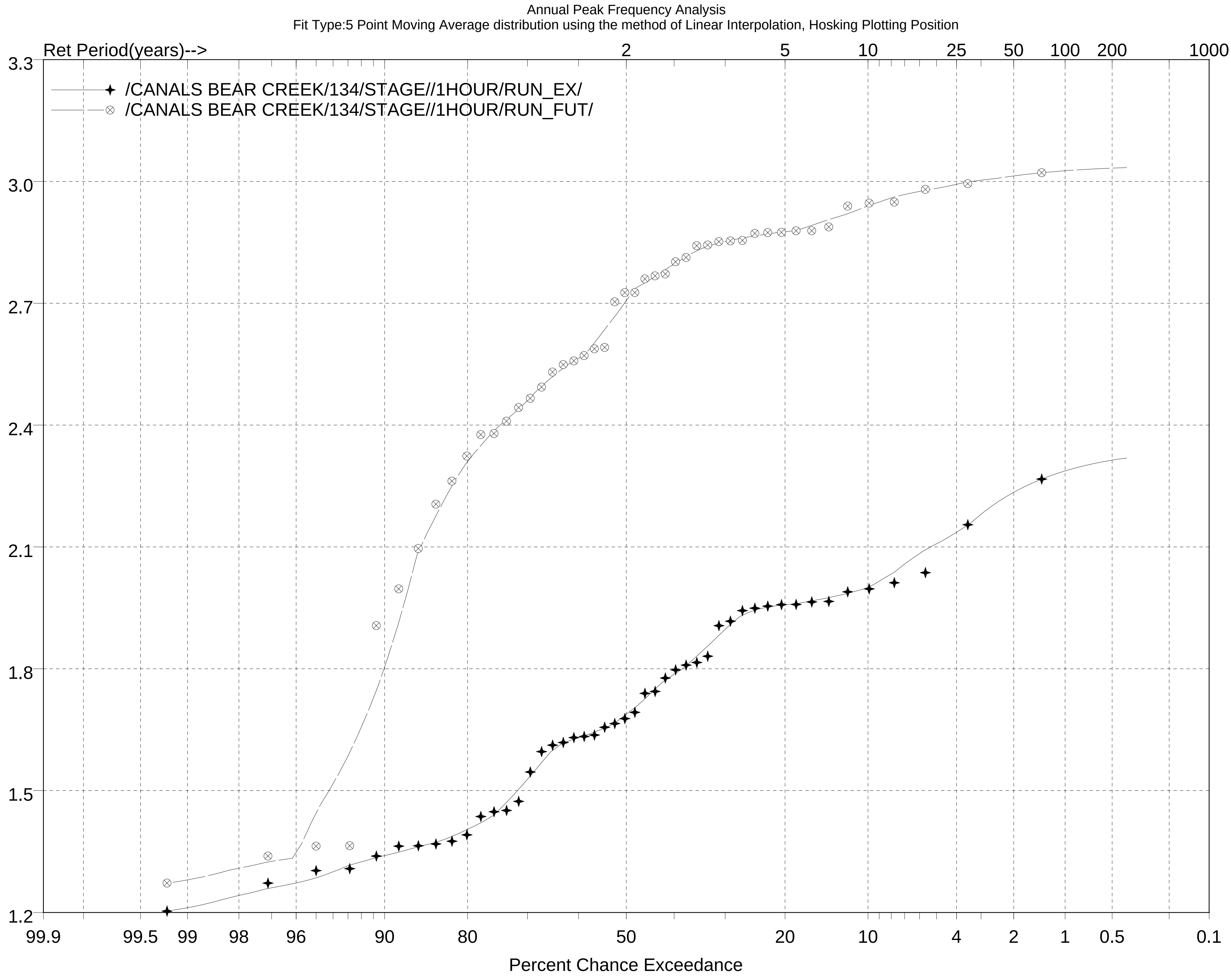

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/NICOMEKL RIVER MAIN US/14036.7/STAGE//1HOUR/RUN_EX/
/NICOMEKL RIVER MAIN US/14036.7/STAGE//1HOUR/RUN_FUT/
3.3
3.0
2.7
2.4
2.1
1.8
1.5
1.2
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



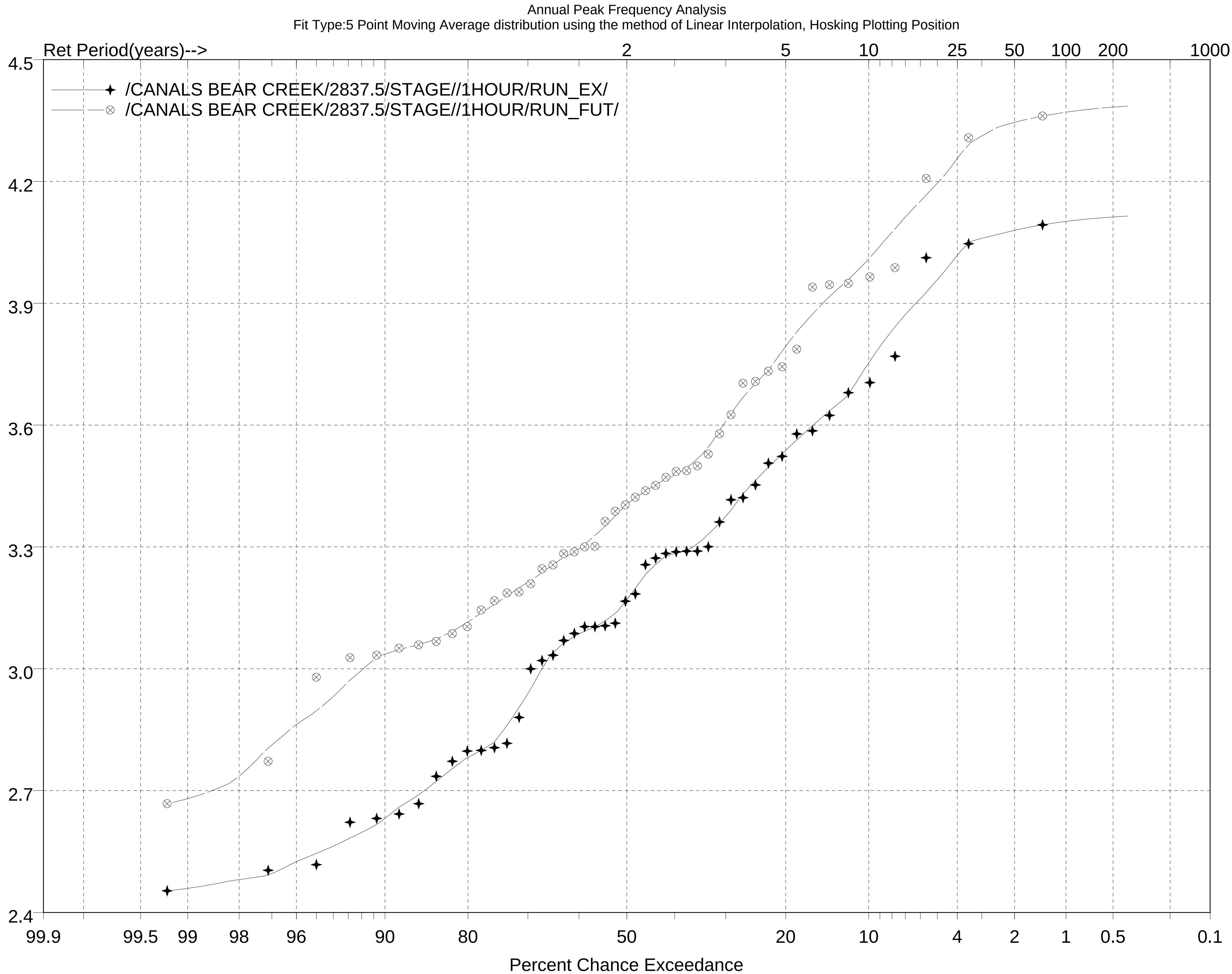
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/NICOMEKL RIVER MAIN US/14415.1*/STAGE//1HOUR/RUN_EX/
/NICOMEKL RIVER MAIN US/14415.1*/STAGE//1HOUR/RUN_FUT/
3.4
3.1
2.8
2.5
2.2
1.9
1.6
1.3
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance


Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/NICOMEKL RIVER MAIN US/15180.6/STAGE//1HOUR/RUN_EX/
/NICOMEKL RIVER MAIN US/15180.6/STAGE//1HOUR/RUN_FUT/
3.7
3.4
3.1
2.8
2.5
2.2
1.9
1.6
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

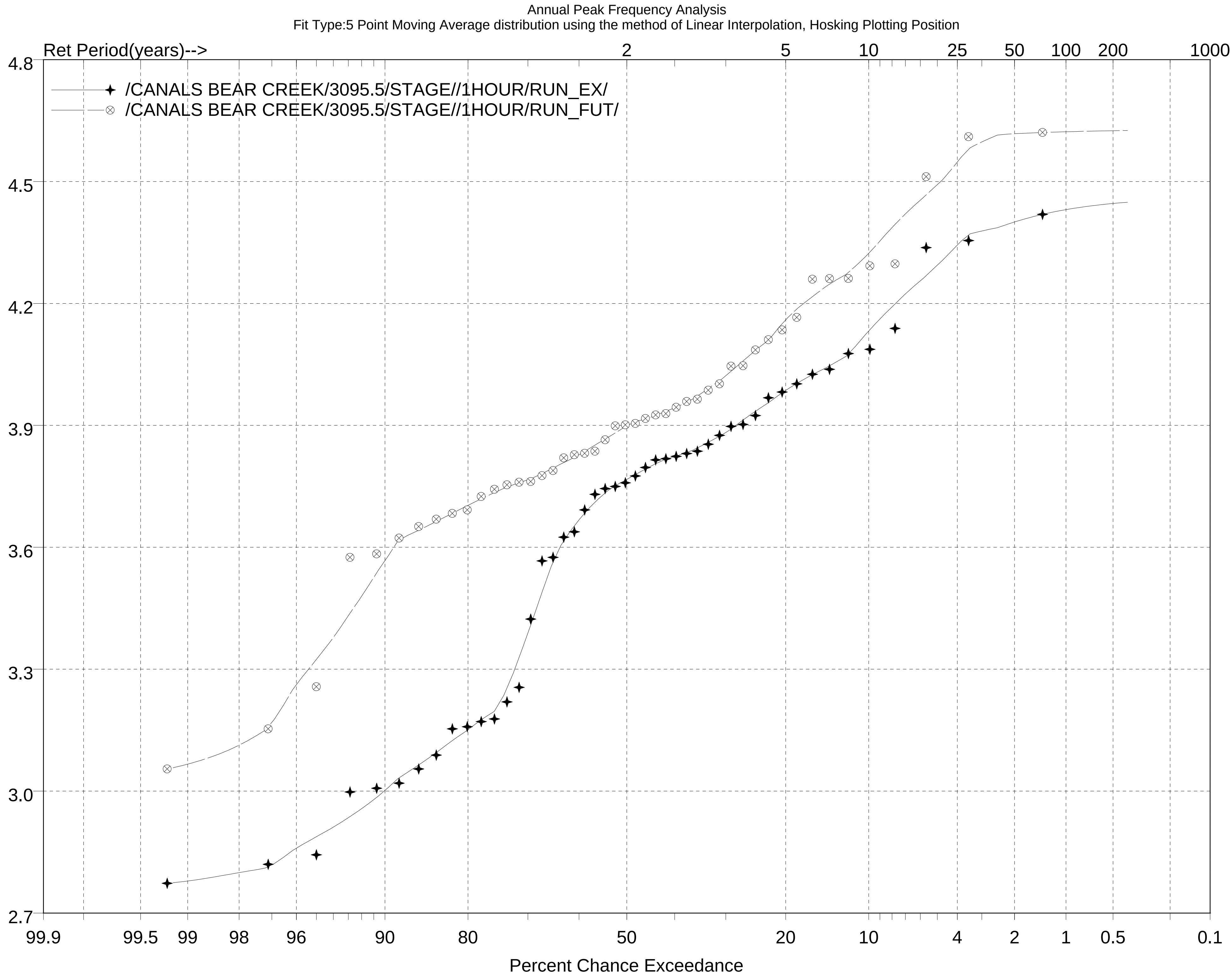

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/NICOMEKL RIVER MAIN US/17280.9/STAGE//1HOUR/RUN_EX/
/NICOMEKL RIVER MAIN US/17280.9/STAGE//1HOUR/RUN_FUT/
4.7
4.5
4.3
4.1
3.9
3.7
3.5
3.3
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



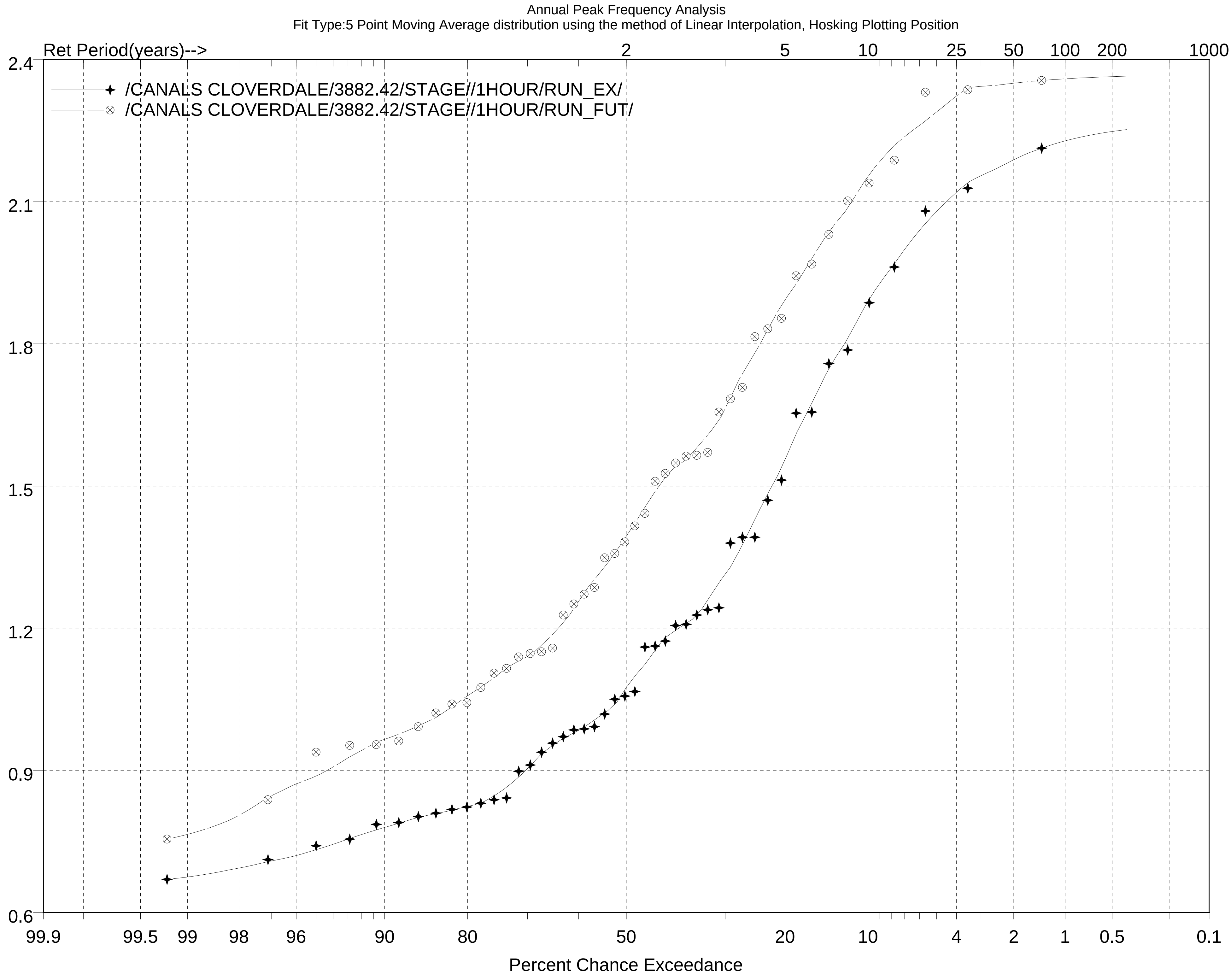
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/CANALS BEAR CREEK/134/STAGE//1HOUR/RUN_EX/
/CANALS BEAR CREEK/134/STAGE//1HOUR/RUN_FUT/
3.3
3.0
2.7
2.4
2.1
1.8
1.5
1.2
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



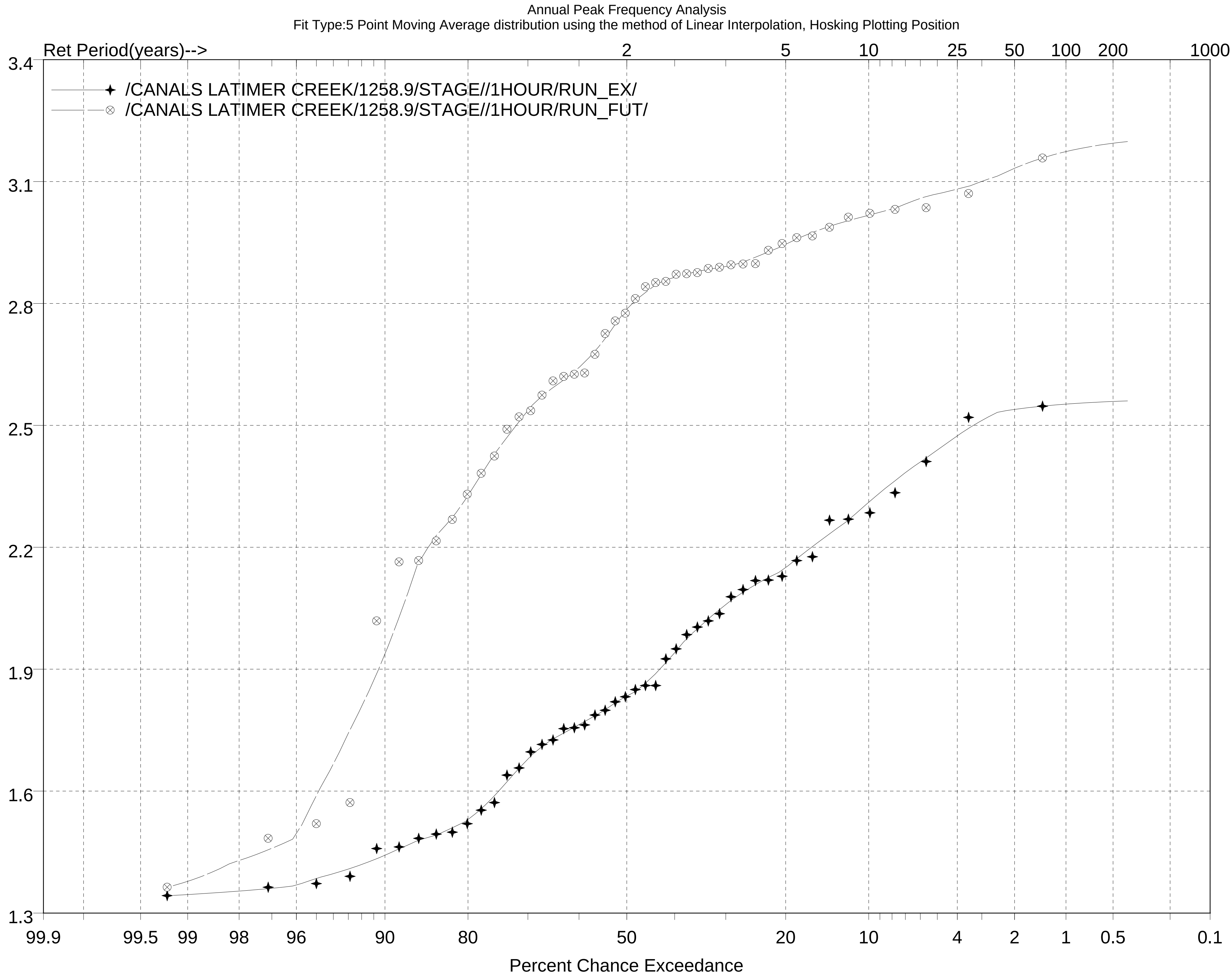
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/CANALS BEAR CREEK/2837.5/STAGE//1HOUR/RUN_EX/
/CANALS BEAR CREEK/2837.5/STAGE//1HOUR/RUN_FUT/
4.5
4.2
3.9
3.6
3.3
3.0
2.7
2.4
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



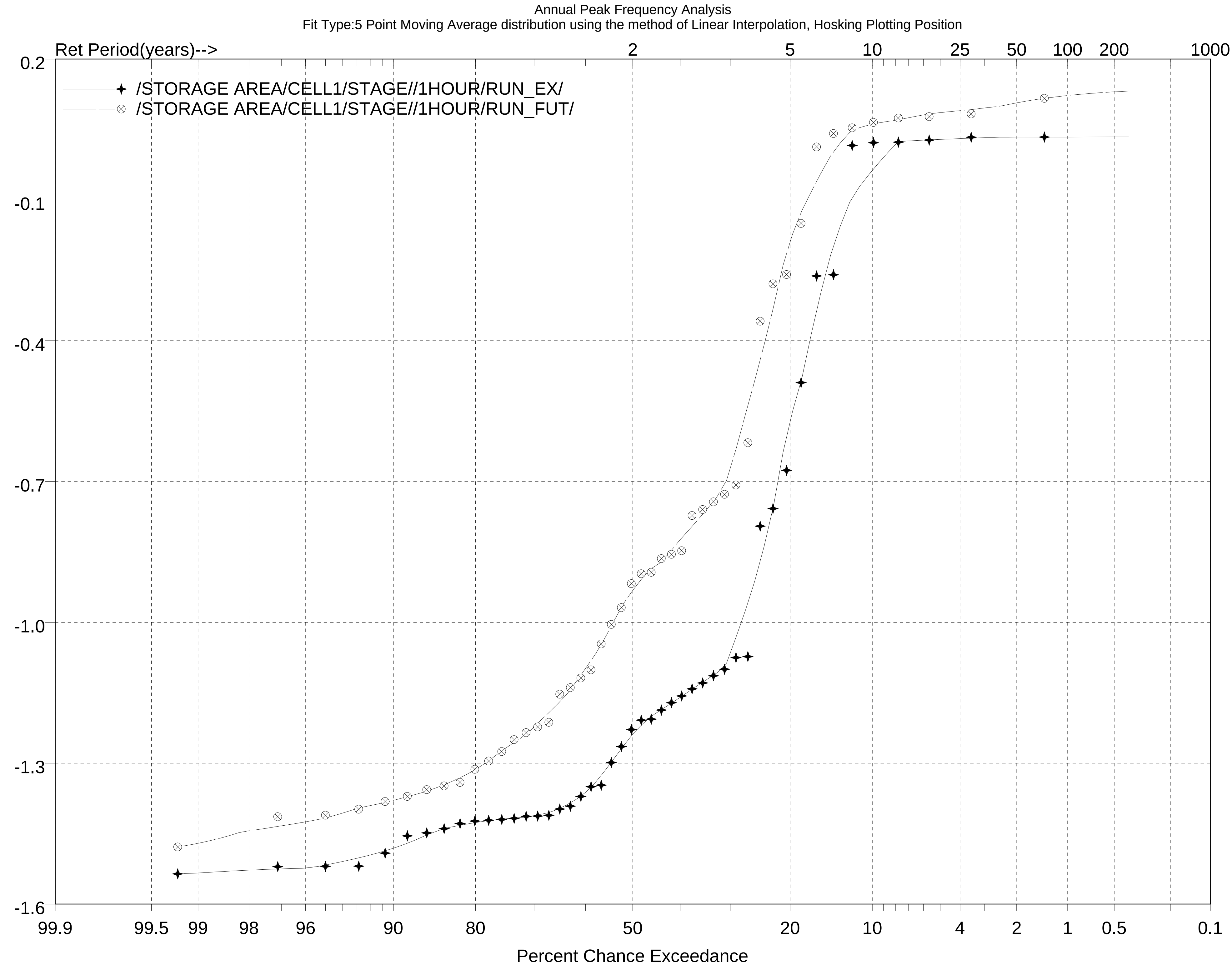
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/CANALS BEAR CREEK/3095.5/STAGE//1HOUR/RUN_EX/
/CANALS BEAR CREEK/3095.5/STAGE//1HOUR/RUN_FUT/
4.8
4.5
4.2
3.9
3.6
3.3
3.0
2.7
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

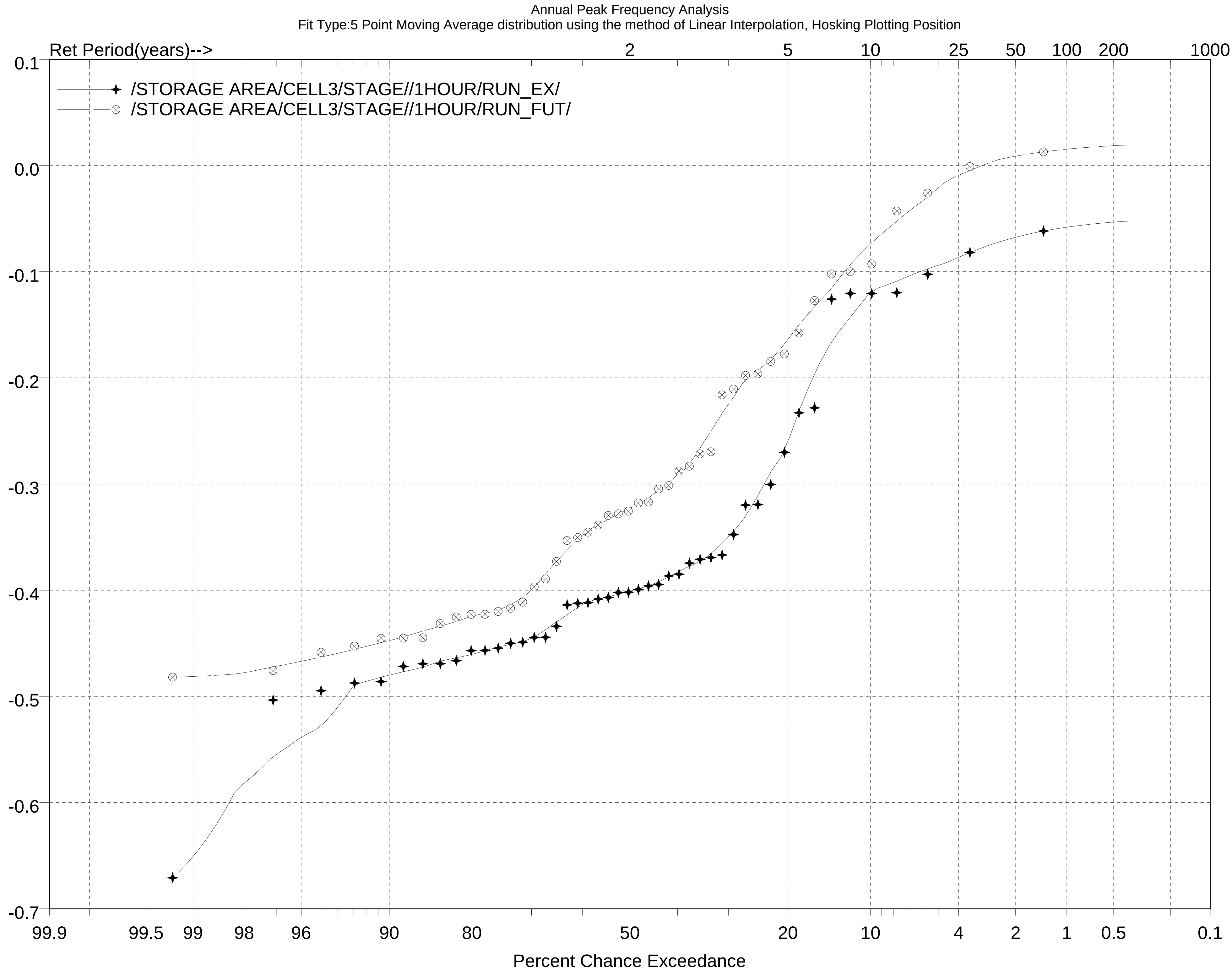


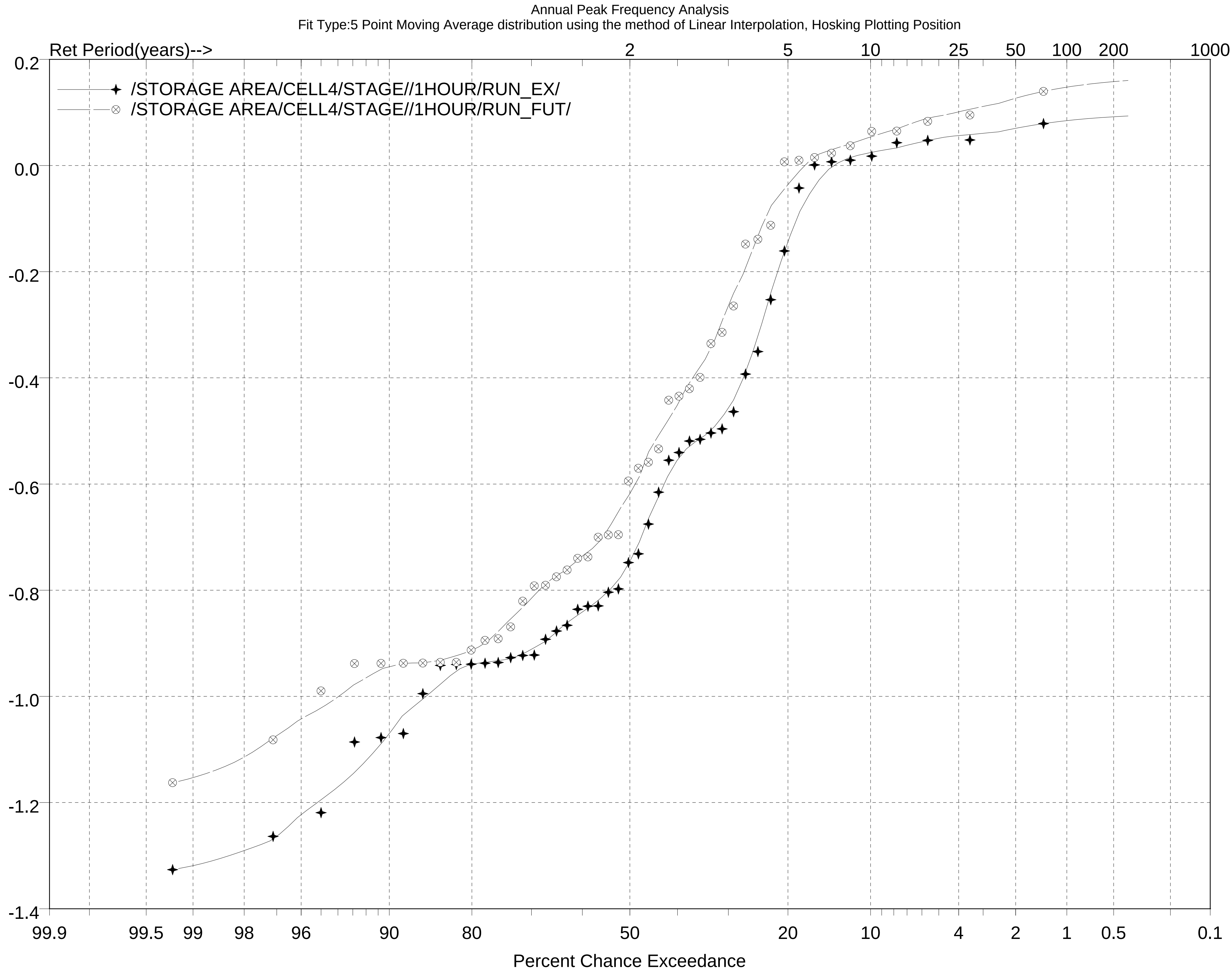
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/CANALS CLOVERDALE/3882.42/STAGE//1HOUR/RUN_EX/
/CANALS CLOVERDALE/3882.42/STAGE//1HOUR/RUN_FUT/
2.4
2.1
1.8
1.5
1.2
0.9
0.6
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



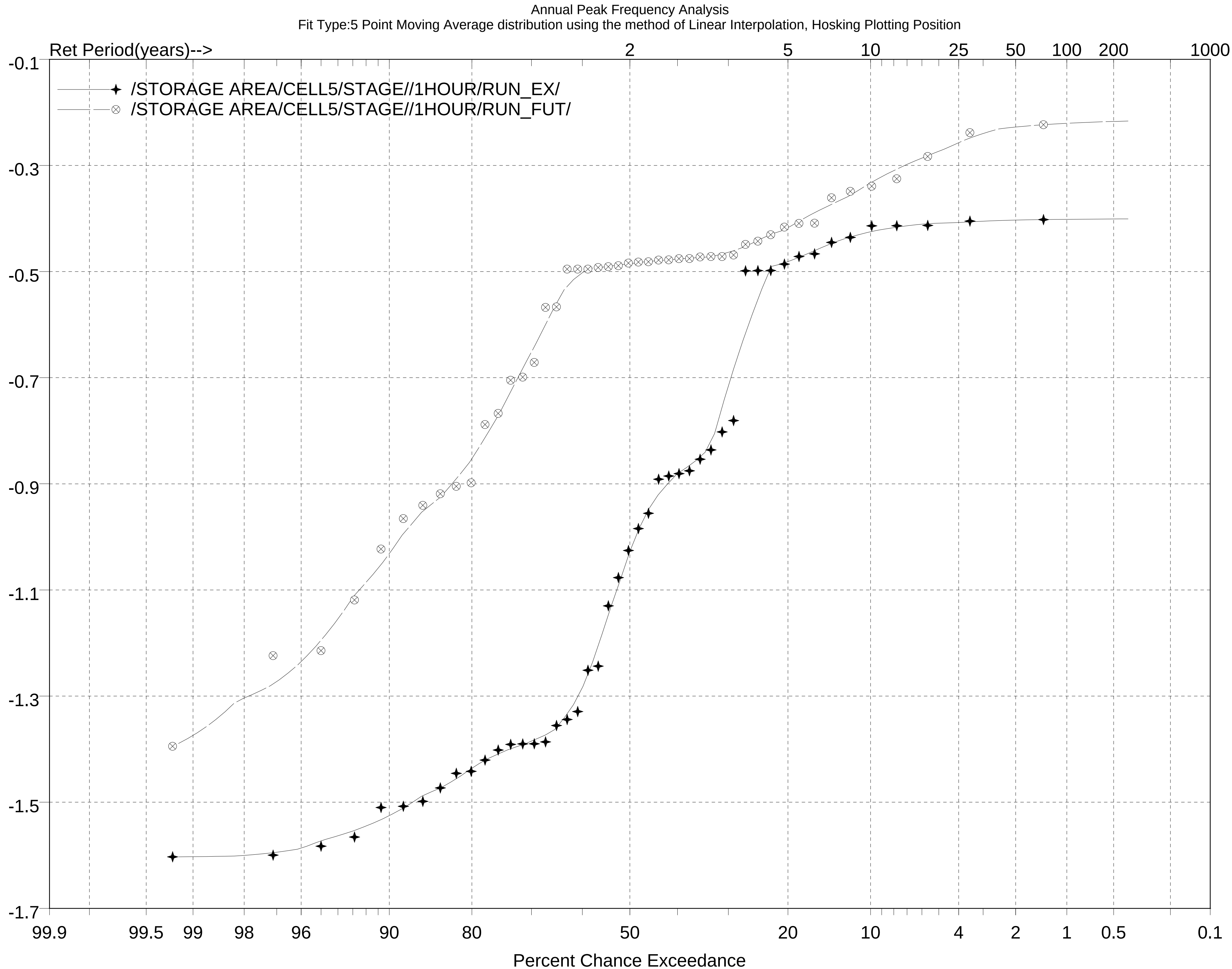
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/CANALS LATIMER CREEK/1258.9/STAGE//1HOUR/RUN_EX/
/CANALS LATIMER CREEK/1258.9/STAGE//1HOUR/RUN_FUT/
3.4
3.1
2.8
2.5
2.2
1.9
1.6
1.3
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

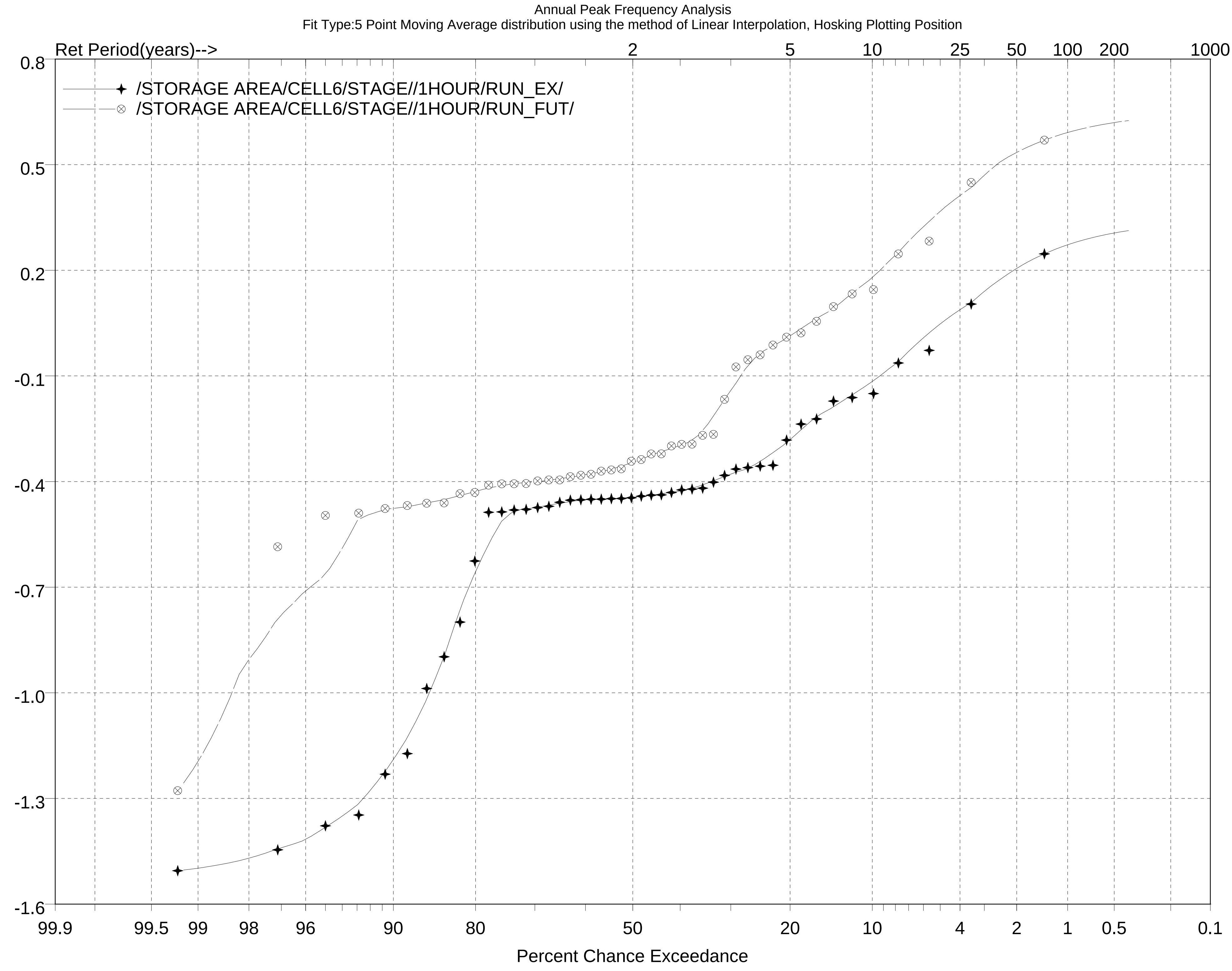

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL1/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL1/STAGE//1HOUR/RUN_FUT/
0.2
-0.1
-0.4
-0.7
-1.0
-1.3
-1.6
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance


Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL3/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL3/STAGE//1HOUR/RUN_FUT/
0.1
0.0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
-0.7
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

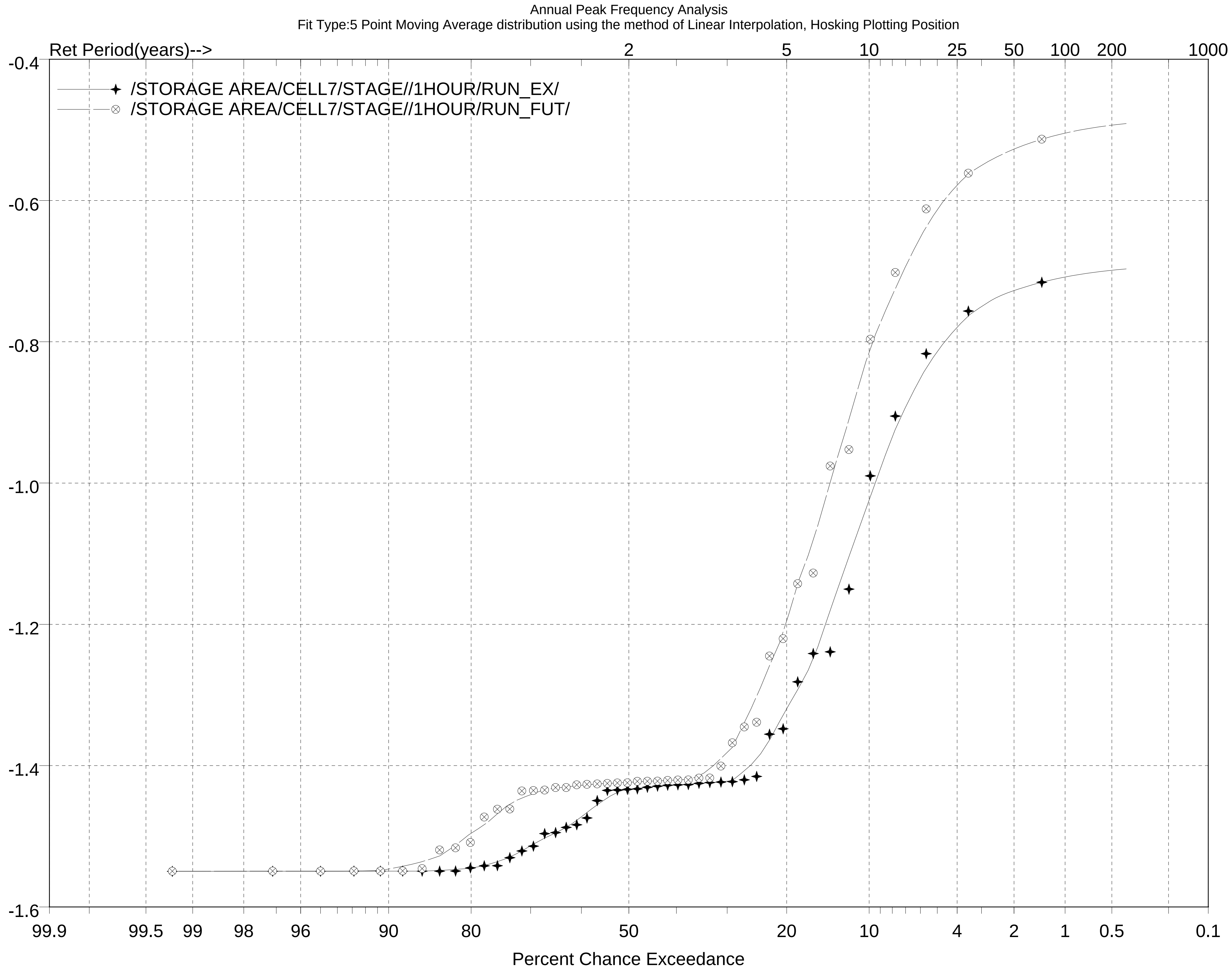


Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL4/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL4/STAGE//1HOUR/RUN_FUT/
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

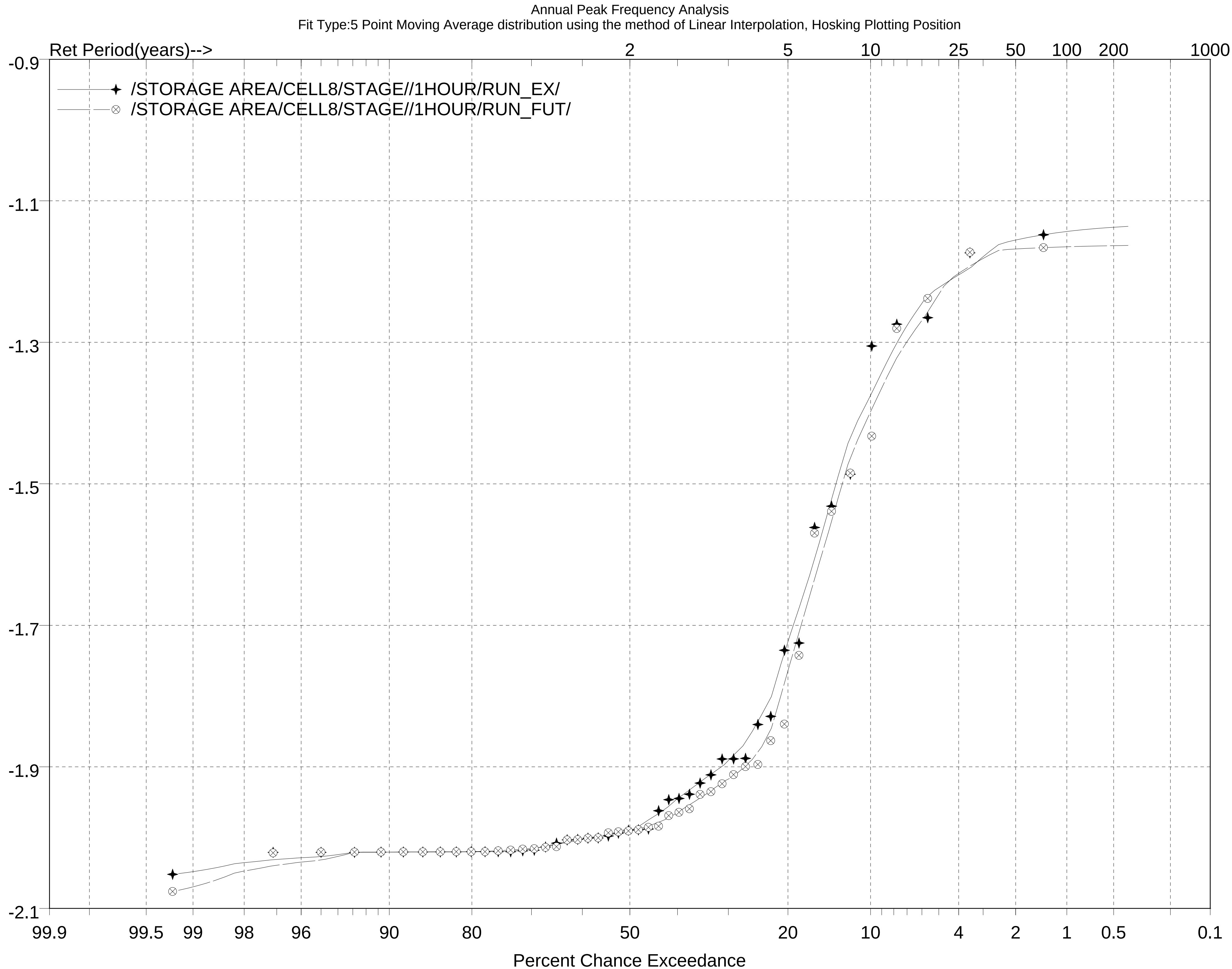

Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL5/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL5/STAGE//1HOUR/RUN_FUT/
-0.1
-0.3
-0.5
-0.7
-0.9
-1.1
-1.3
-1.5
-1.7
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



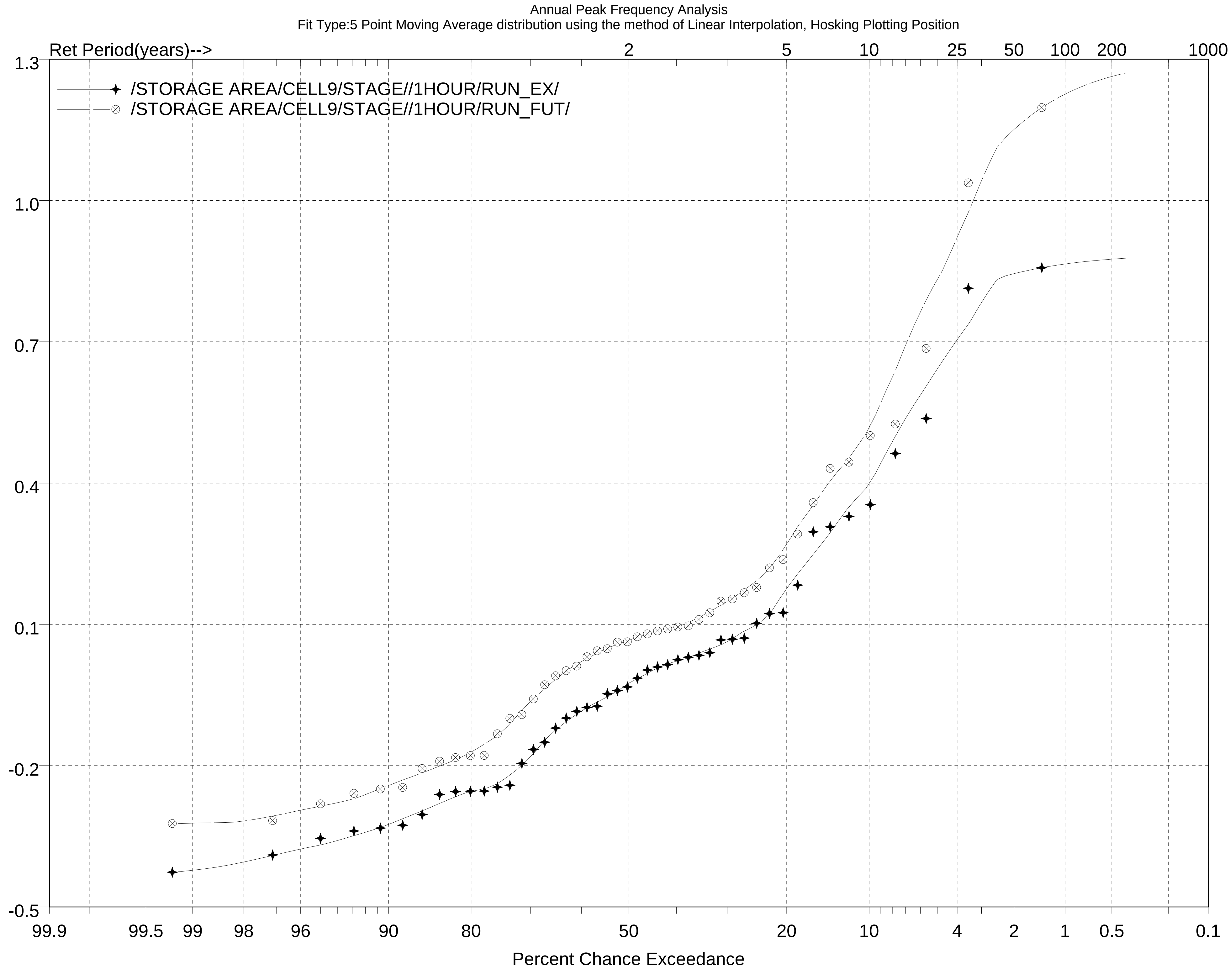
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL6/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL6/STAGE//1HOUR/RUN_FUT/
0.8
0.5
0.2
-0.1
-0.4
-0.7
-1.0
-1.3
-1.6
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



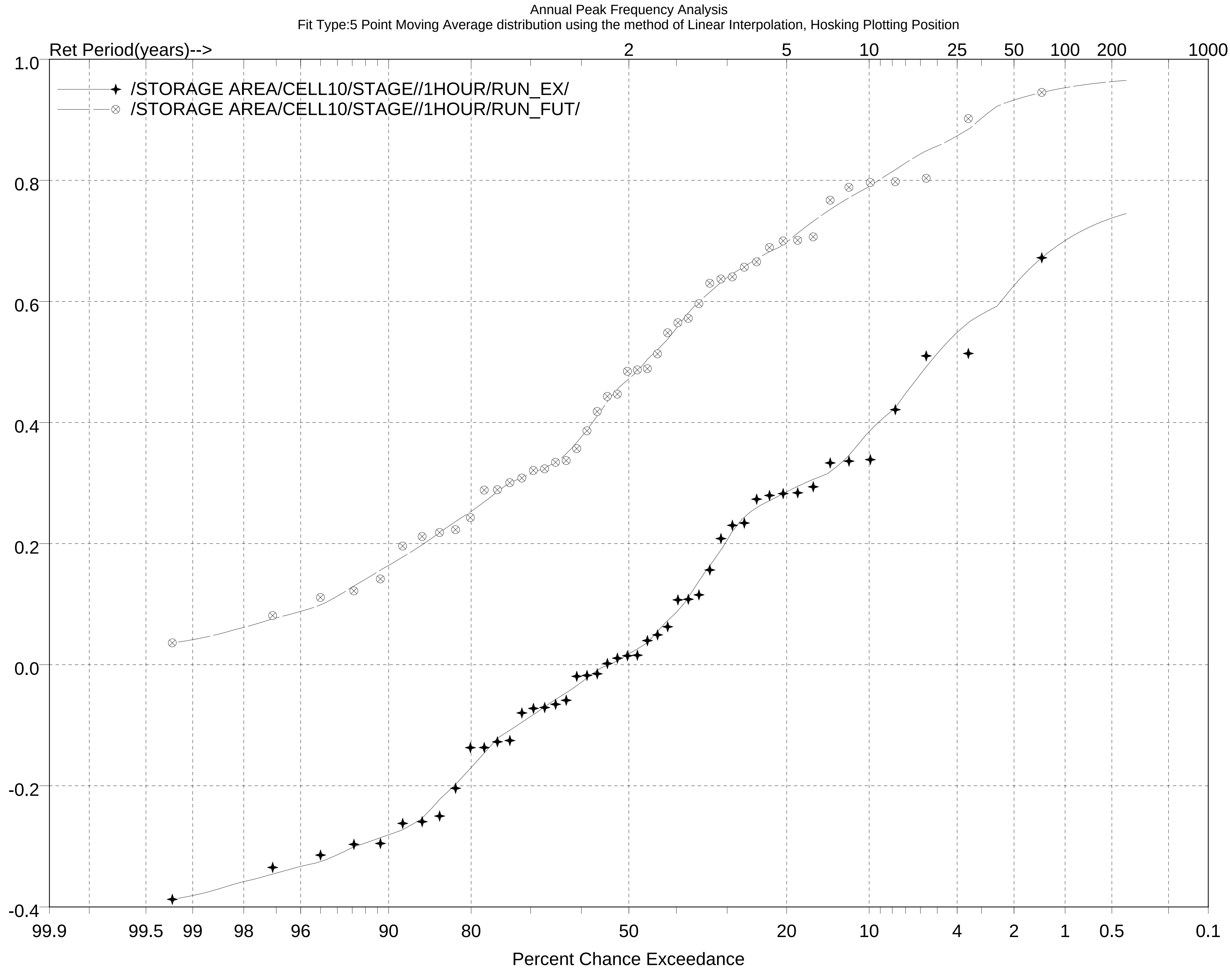
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL7/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL7/STAGE//1HOUR/RUN_FUT/
-0.4
-0.6
-0.8
-1.0
-1.2
-1.4
-1.6
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

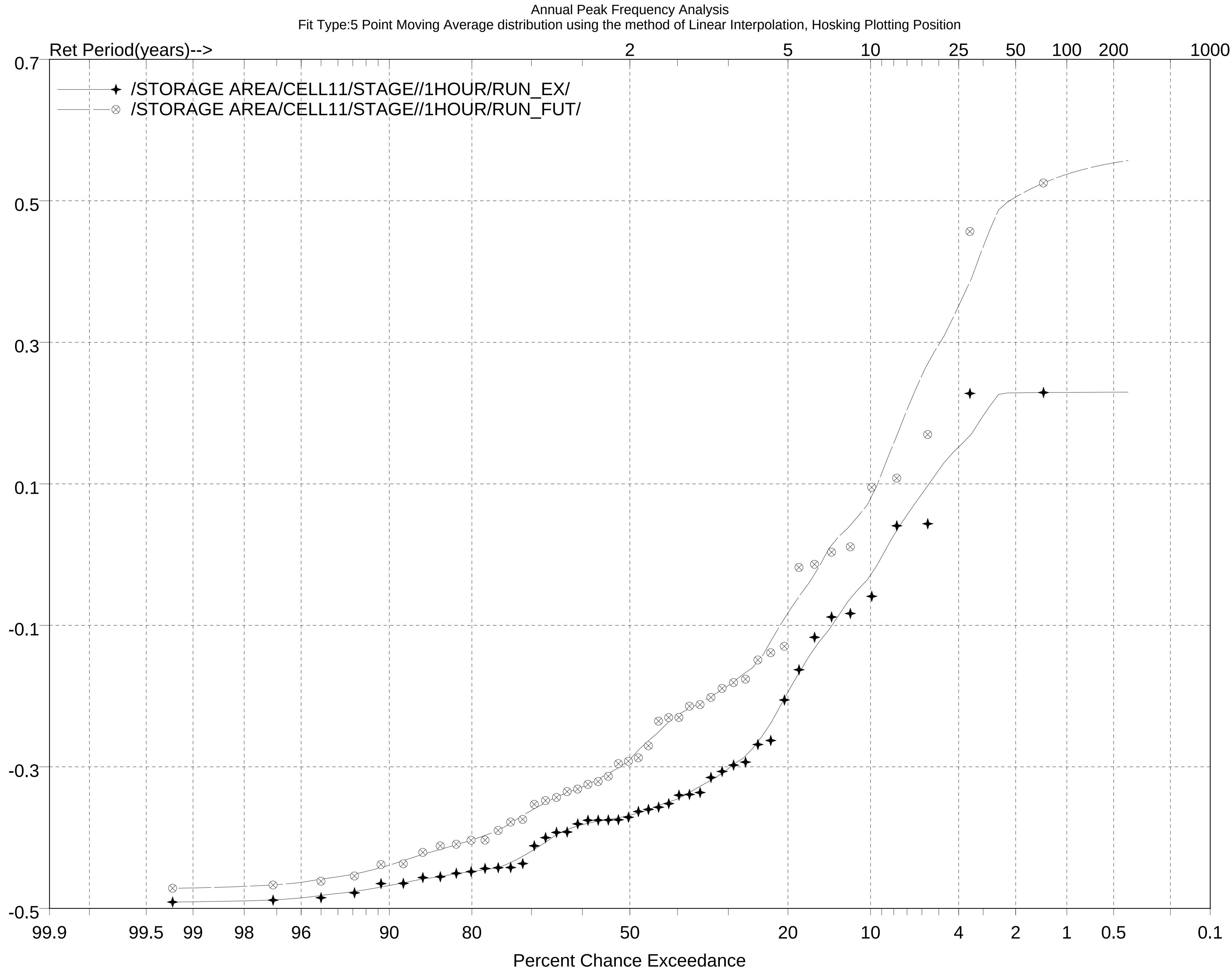


Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL8/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL8/STAGE//1HOUR/RUN_FUT/
-0.9
-1.1
-1.3
-1.5
-1.7
-1.9
-2.1
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance

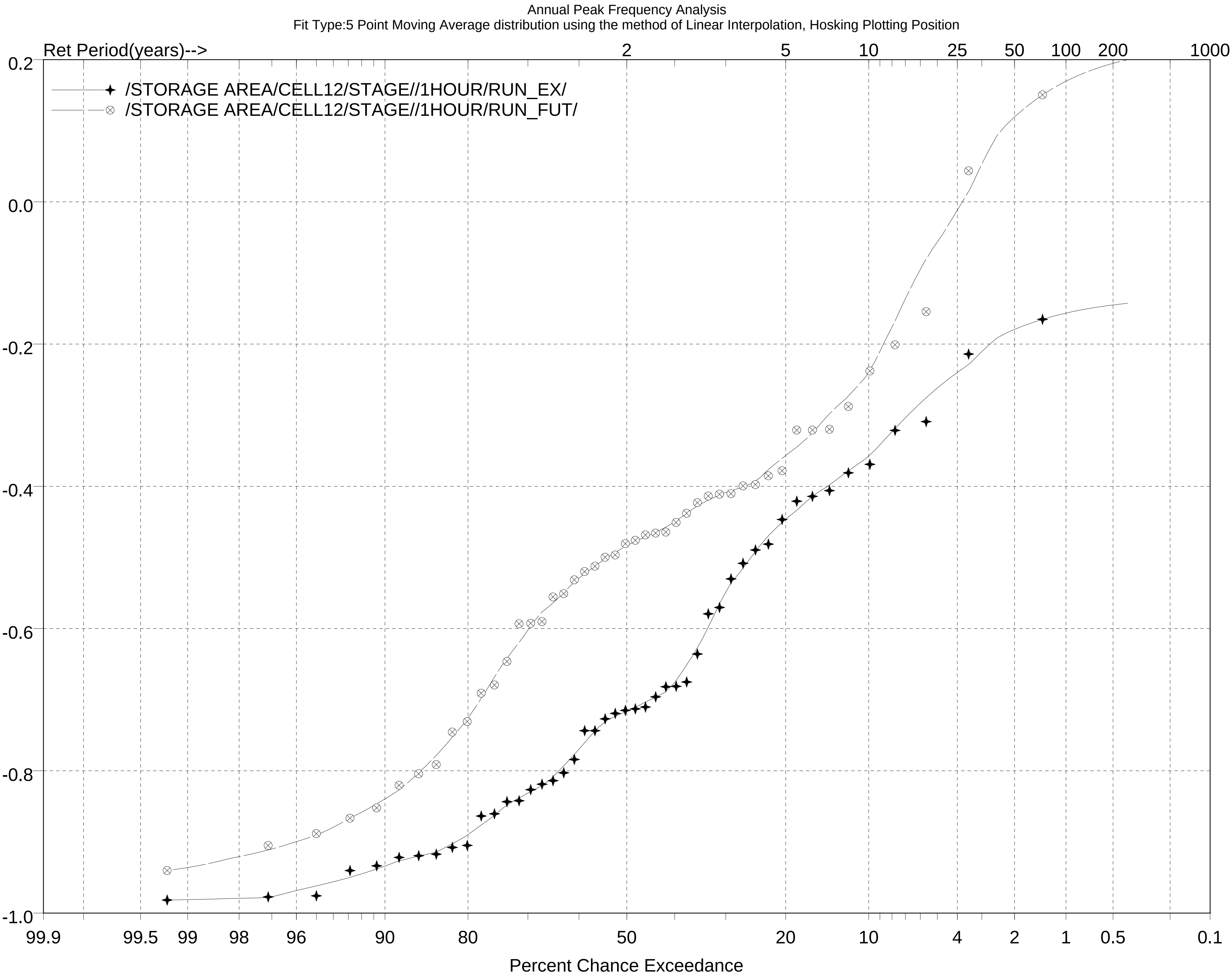


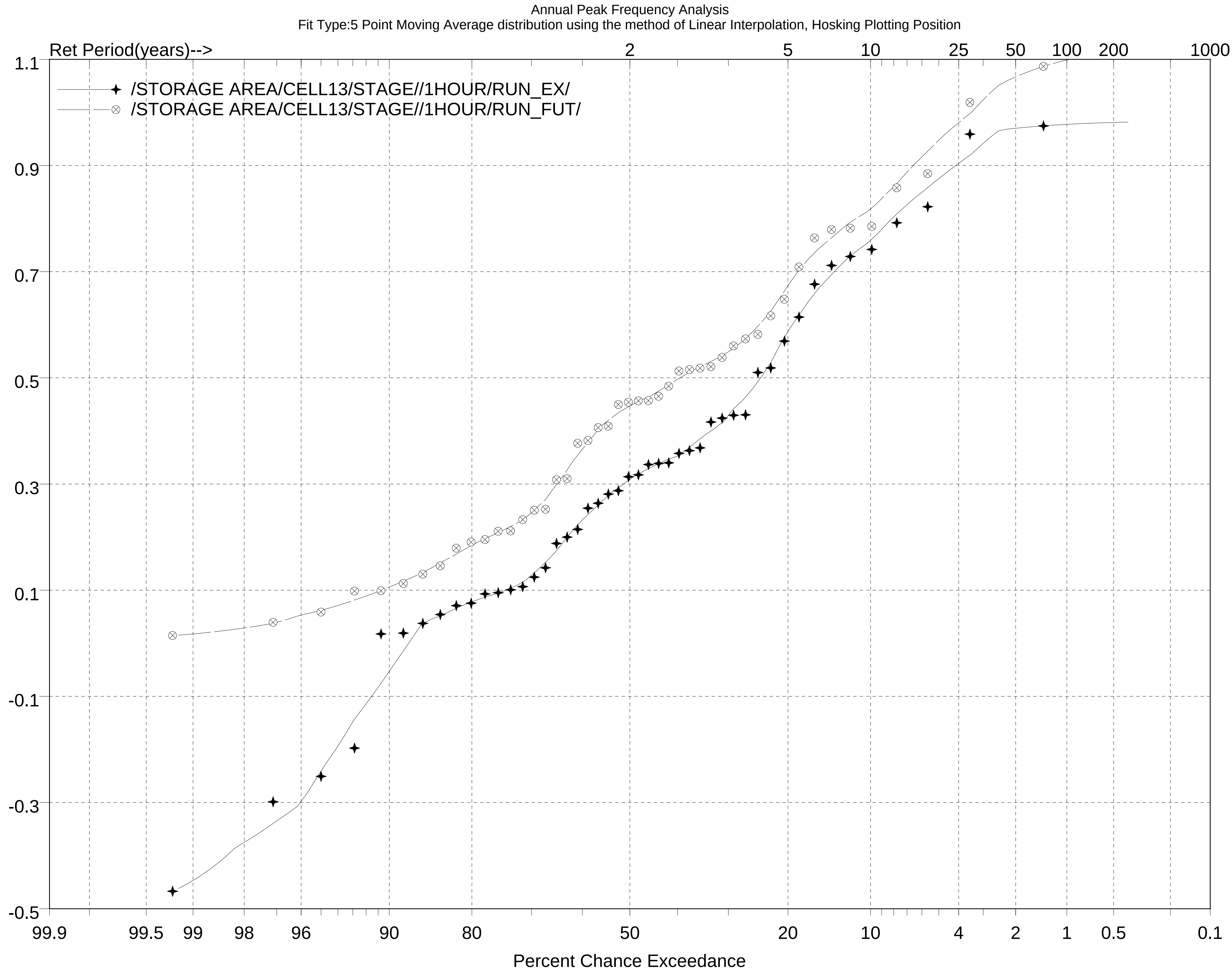
Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL9/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL9/STAGE//1HOUR/RUN_FUT/
1.3
1.0
0.7
0.4
0.1
-0.2
-0.5
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance


Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL10/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL10/STAGE//1HOUR/RUN_FUT/
1.0
0.8
0.6
0.4
0.2
0.0
-0.2
-0.4
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL11/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL11/STAGE//1HOUR/RUN_FUT/
0.7
0.5
0.3
0.1
-0.1
-0.3
-0.5
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance


Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL12/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL12/STAGE//1HOUR/RUN_FUT/
0.2
0.0
-0.2
-0.4
-0.6
-0.8
-1.0
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance



Annual Peak Frequency Analysis
Fit Type:5 Point Moving Average distribution using the method of Linear Interpolation, Hosking Plotting Position
Ret Period(years)-->
2
5
10
25
50
100
200
1000
/STORAGE AREA/CELL13/STAGE//1HOUR/RUN_EX/
/STORAGE AREA/CELL13/STAGE//1HOUR/RUN_FUT/
1.1
0.9
0.7
0.5
0.3
0.1
-0.1
-0.3
-0.5
99.9
99.5
99
98
96
90
80
50
20
10
4
2
1
0.5
0.1
Percent Chance Exceedance