A PRELIMINARY ASSESSMENT OF THE HYDRODYNAMICS OF THE TOUW RIVER AND WILDERNESS LAKES SYSTEM WITH EMPHASIS ON THE MANAGEMENT OF THE ESTUARY MOUTH

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ABSTRACT

The Touw River estuary and Wilderness coastal lakes is a sensitive system from a flooding and ecological viewpoint and, therefore, careful consideration is placed on the hydrodynamics and salinity levels within the system. The estuary consists of a "temporary open/closed" estuary, where during closed mouth conditions, the sand bar at the estuary mouth is artificially managed in an attempt to reduce flood water levels in the system. The reason behind this management strategy is the construction of residential property along the flood plains of the estuary and coastal lakes, which in the past, had been exposed to regular cycles of inundation during flood events. In an attempt to reduce flood water levels in all water bodies and hence reduce the risk of inundation, a management policy was formulated. The past and present management plan is to maintain the sand bar at Touw estuary mouth, during closed mouth conditions, at an elevation of between +2.1m to +2.4m MSL, based on proposals made by the CSIR in 1981.

Recent flood events, after the implementation of the management policy, still occasionally result in significant inundation of residential property, which has raised concern for some interested parties over the effectiveness of the management strategy. Furthermore, a growing concern was also evident over the long term wellbeing of the system from an ecological viewpoint. Historical data shows significant changes in salinity levels since the implementation of the management strategy which could impose negative long term effects on the system.

In this study, numerical models were consequently constructed and applied in order to analyse the effectiveness of the current management policy and recalculate flood water levels under a number of proposed scenarios. Long term salinity changes were also analysed in an attempt to better understand salinity propagation throughout the system, using extreme hypothetical cases.

Through the analysis of the simulation results, it was concluded that flood water levels in the Touw estuary were almost completely dependent on the size of the Touw River flood and the initial height of the sand bar at the estuary mouth. Whereas, water levels in the coastal lakes are almost entirely dependent on the quantity of runoff into the lakes and their initial water levels. The current management plan, involving only artificial manipulation of the sand bar at the estuary mouth, therefore has a fairly insignificant effect on flood water levels achieved in the coastal lakes. Furthermore, it was concluded that the construction of the preparatory channel is a vitally important aspect of the current management plan and that skimming of the sand bar alone is ineffective to completely mitigate the risk of residential inundation along the banks of the Touw River.

The salinity modelling study provided a first indication of the salinity characteristics within the system. It was found that the penetration of seawater into the system was less prominent as the water bodies became further removed from the ocean and that a direct relationship was evident between the volume of direct freshwater inflow to a water body and the degree of salinity variation in that specific water body. In water bodies with high volumes of direct freshwater inflow such as the Touw estuary, a large degree of salinity variation is evident. However, in water bodies with no freshwater inflow, such as Rondevlei, salinity levels remain more stable and are less likely to fluctuate.

OPSOMMING

Uit ‘n vloed- en ekologiese oogpunt is die Touwsriviermonding en Wilderniskusmere ‘n uiters sensitiewe stelsel en daar is dus deeglike oorweging gegee aan die hidrodinamika en soutvlakke in die stelsel. Die monding bestaan uit 'n "tydelike oop / geslote" monding, en tydens geslote mondtoestande word die sandbank by die riviermond kunsmatig beheer in 'n poging om vloedwatervlakke binne die stelsel te verminder. Die rede vir hierdie strategie is omdat baie residensiële eiendomme langs die vloedvlaktes van die monding en kusmere gebou is, wat in die verlede aan 'n gereelde siklus van oorstromings blootgestel is tydens vloede. In 'n poging om vloedwatervlakke in al die watermassas te verminder, en sodoende die risiko van oorstroming te verminder, is 'n bestuursbeleid geformuleer. In beide die vorige en die huidige bestuursplanne is die sandbank in die Touwsriviermond tydens geslote mondtoestande in stand gehou op 'n hoogte van tussen 2,1 m en 2,4 m MSL, gebaseer op die voorstelle wat deur die WNNR in 1981 gemaak is.

Onlangse vloede wat plaasgevind het na die implementering van die beleid, het steeds van tyd tot tyd gelei tot noemenswaardige oorstromings van residensiële eiendomme, en kommer is uitgespreek deur 'n paar belanghebbende partye oor die doeltreffendheid van die strategie vir die bestuur. Daar is verder kommer uitgespreek oor die langtermyn welstand van die stelsel uit 'n ekologiese oogpunt. Historiese data toon 'n beduidende verandering in soutvlakke sedert die implementering van die bestuurstrategie met ‘n negatiewe langtermyn uitwerking op die stelsel.

In hierdie studie is daar derhalwe numeriese modelle opgestel en toegepas ten einde die doeltreffendheid van die huidige bestuur van die beleid te bepaal, asook om die vloedvlakke te herbereken en te analiseer na aanleiding van 'n aantal voorgestelde scenario's. Langtermyn soutgehalte veranderinge is ook ontleed in 'n poging om die soutgehalte verspreiding deur die hele stelsel beter te verstaan, deur gebruik te maak van uiterste hipotetiese gevalle.

Deur die ontleding van die simulasie resultate, is daar tot die gevolgtrekking gekom dat vloedwatervlakke in die Touwsrivier-monding byna heeltemal afhanklik was van die grootte van die Touwsrivier vloed en die aanvanklike hoogte van die sandbank by die riviermond. Watervlakke in die kusmere is egter byna heeltemal afhanklik van die hoeveelheid afloop na die mere en die aanvanklike watervlakke. Die huidige bestuursplan, wat slegs ‘n kunsmatige manipulasie van die sandbank by die riviermond behels, het dus 'n redelik onbeduidend invloed op die vloedwatervlakke wat in die kusmere bereik is. Daar is verder tot die gevolgtrekking gekom dat die konstruksie van die voorbereidende kanaal 'n uiters belangrike aspek van die huidige bestuursplan is, en dat die afskraping van die sandbank alleen oneffektief sou wees om die risiko van residensiële oorstroming langs die oewer van die Touwsrivier uit te skakel.

Die soutgehalte modelleringstudie verskaf 'n eerste aanduiding van die soutgehalte eienskappe binne die stelsel. Daar is gevind dat die penetrasie van seewater in die stelsel minder prominent was as in die watermassas verder van die see af, en dat daar 'n duidelike direkte verband is tussen die volume van die varswater wat direk invloei na 'n watermassa en die mate van soutgehalte variasie in daardie spesifieke watermassa. In watermassas waar hoë volumes varswater direk invloei soos die Touwsrivier-monding, is 'n groot mate van soutgehalte variasie sigbaar. In die watermassas waar geen varswater invloei nie, soos die Rondevlei, bly soutvlakke meer stabiel en is minder geneig om te wissel.

ACKNOWLEDGEMENTS

The completion of this thesis would not have been possible without the help and guidance by others.

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1. INTRODUCTION

1.1. Background

This thesis serves as a preliminary assessment of the hydrodynamics of the Touw River and the Wilderness lakes system where the majority of the emphasis lies in the management of the Touw River mouth. The "periodically open/closed" estuary in Wilderness serves as the focus area to this investigation. A "periodically open/closed" estuary is defined as an estuary that is closed off to the ocean during parts of the year, while at other times; the estuary mouth is open to the ocean and receives marine interaction. The estuary in Wilderness is closed to the ocean approximately 73% of the time (CSIR, 1981) by a natural process of sediment accretion at the estuary mouth. A closed estuary mouth plays a critical role in the hydrodynamics of the estuary and it is perceived to affect the immediate water levels in the estuary mouth as well as the surrounding coastal lakes. During closed mouth conditions under natural circumstances, the sand barrier between the estuary and the ocean can reach elevations in excess of +3m MSL (Russell, 2013). This has a direct result on the water levels achieved within the estuary. During flood events, the water level in the estuary is required to rise above the level of the sand bar, before breaching and scouring of the sand bar commences. The predicament to this scenario is, however, the construction of residential property along the floodplains of the Touw River estuary at elevations well below +3m MSL. Under natural conditions, flooding of these properties would be a definite regular occurrence. Therefore, a management strategy has been implemented to artificially manage the height of the sand barrier and hence reduce water levels within the Touw River estuary and inland water bodies.

Historical studies have been performed on this water area in an attempt to reduce flood water levels (CSIR, 1981). Assessments have also been undertaken in order to find an optimal management policy in order to control flood water levels, while at the same time, preserving the ecological wellbeing of the system (Fijen, 1995). The current management strategy is to maintain the crest height of the sand berm at the river mouth, after natural closure of the Touw River mouth, at +2.1m to +2.4m MSL (South African National Parks, 2010). The primary aim of this management action is to avoid high water levels in the lake and estuarine areas, thereby reducing the potential for flood damage to low lying residential properties. These artificial mouth openings, at water levels much lower than the levels that have been achieved during natural conditions, have resulted in numerous changes to the system when compared to the totally natural situation. It is claimed that these changes include an overall reduction of water levels in the estuary and lakes system, reduced open water areas and reduced wetland areas, as well as reductions in areas subject to rising and falling water levels (Fijen, 1995). A reduction of scouring potential is also evident which leads to increased sedimentation and reduced open mouth conditions. Salinity changes within the system are also occurring, which have a consequential effect on the overall ecological change of the system (Russell, 2013). Another major concern is the loss of natural variability of the water system, which may result in a loss of habitat diversity and thus species diversity within the National Park. (Fijen, 1995)

The rationale behind the thesis is to verify existing flood levels for different return periods and to evaluate the effect of flooding within the Touw River estuary and Wilderness coastal lakes under different management strategies of the sand barrier between the estuary and the ocean. In this report, return period is the term used to describe annual recurrence interval.

1.2. Objective

The objective of this thesis is to provide a preliminary assessment of the changes in the Touw River estuary and Wilderness coastal lakes including flood water levels and long term salinity changes under a number of different mouth management situations using a simplified numerical modelling approach.

1.3. Scope of Work

The study objective is achieved through the execution of several sub tasks, the details of which will be covered in later chapters. In general, sub tasks initially involve developing an understanding of the Touw River estuary and Wilderness coastal lakes system, the formation of open/closed estuaries and the implications of this type of estuarine environment in the Wilderness area.

The study then progresses to data collection and the necessary field work that was needed in order to complete both a hydrodynamic and salinity numerical study. An evaluation of this data was also discussed before investigating the numerical model approach.

The numerical modelling aspect of the study is further subdivided into two categories, namely a hydrodynamic study and salinity study. A model for the hydrodynamic study was formulated, calibrated and then verified. Following these steps the model was used to predict hypothetical scenarios based on a number of different conditions involving the sand bar height, initial water level of each water body and the return period of flood events.

Thereafter, a salinity model was formulated based on the calibrated hydrodynamic model. This provided an opportunity for a first indication of general salinity trends throughout the system.

Chapter 2 of this report introduces the study area and the hydrodynamic aspects within the Touw River estuary and Wilderness coastal lakes.

Chapter 3 covers a literature review where initially, general principles of sand bar formation at temporarily open/closed estuaries are discussed. Artificial manipulation techniques of the sand bar are also mentioned. The study then focuses on the case study at hand and describes specific details related to the Touw River estuary and Wilderness coastal lakes. Previous studies are mentioned which relate to the formulation of the current management plan of the sand bar at Touw River mouth. Lastly, estuarine salinity characteristics are described both from a holistic and case specific point of view.

Chapters 4 and 5 pertain to the methodology and data evaluation. The aims of the numerical model are discussed as well as the relevant model selection in order to meet those aims. Data requirements for the chosen model are mentioned, whereafter data collection methods are discussed. Chapter 5 relates to an evaluation of the prototype data which was gathered from field investigations.

Chapters 6 to 9 relate to the hydrodynamic numerical model, including separate chapters for the model set up and testing, calibration, validation and the theoretical modelling tests used to run certain hypothetical flooding scenarios.

A first indication of salinity modelling in the system can be seen in Chapter 10. The calibration of the salinity model is discussed, whereafter theoretical tests are performed based on a number of different hypothetical conditions.

Chapter 11 includes a discussion on the conclusions and recommendations related to this study.

2. THE TOUW RIVER ESTUARY AND WILDERNESS COASTAL LAKES

2.1. Study Area

The area of interest is the Touw River estuary and Wilderness Lakes catchment which is situated in the Southern Cape of South Africa between the towns of George and Knysna and extends from approximately 22º34' to 22º43' E and from 33º49' to 34º00' S (Fijen, 1995). The total catchment area is approximately 181.4km^2 in extent and is bounded in the north by the Outeniqua mountains, in the south by the Indian Ocean and in the west by the Kaaimans River (Fijen, 1995). A map showing the entire Wilderness catchment area can be seen in Figure 2-1. The south-western region of this catchment area consists of the Touw River mouth, estuary and floodplain which act as the most significant areas to this assessment. The estuary is also linked by a natural channel, the Serpentine River, to the first of a series of three coastal lakes namely, Island Lake. This lake may also be referred to as Eilandvlei or Onderlangvlei when consulting different literature sources. Thereafter small connecting channels provide links to Bolangvlei, (which may also be referred to as Langvlei) and to Rondevlei respectively. A map showing the water areas described above can be seen in Figure 2-1. The surface area of Island Lake, Bolangvlei and Rondevlei are 137ha, 203ha and 106ha respectively (Fijen, 1995). The area also forms part of the Garden Route National Park and is of great importance to national conservation with the lakes and surrounding area being proclaimed the Garden Route National Park – Wilderness Coastal Area, previously known as the Wilderness National Park. The Touw River estuary and Wilderness coastal lakes fall under the jurisdiction of SANParks. SANParks is, therefore, the managing body of this area and more importantly, responsible for the management of the Touw River mouth.

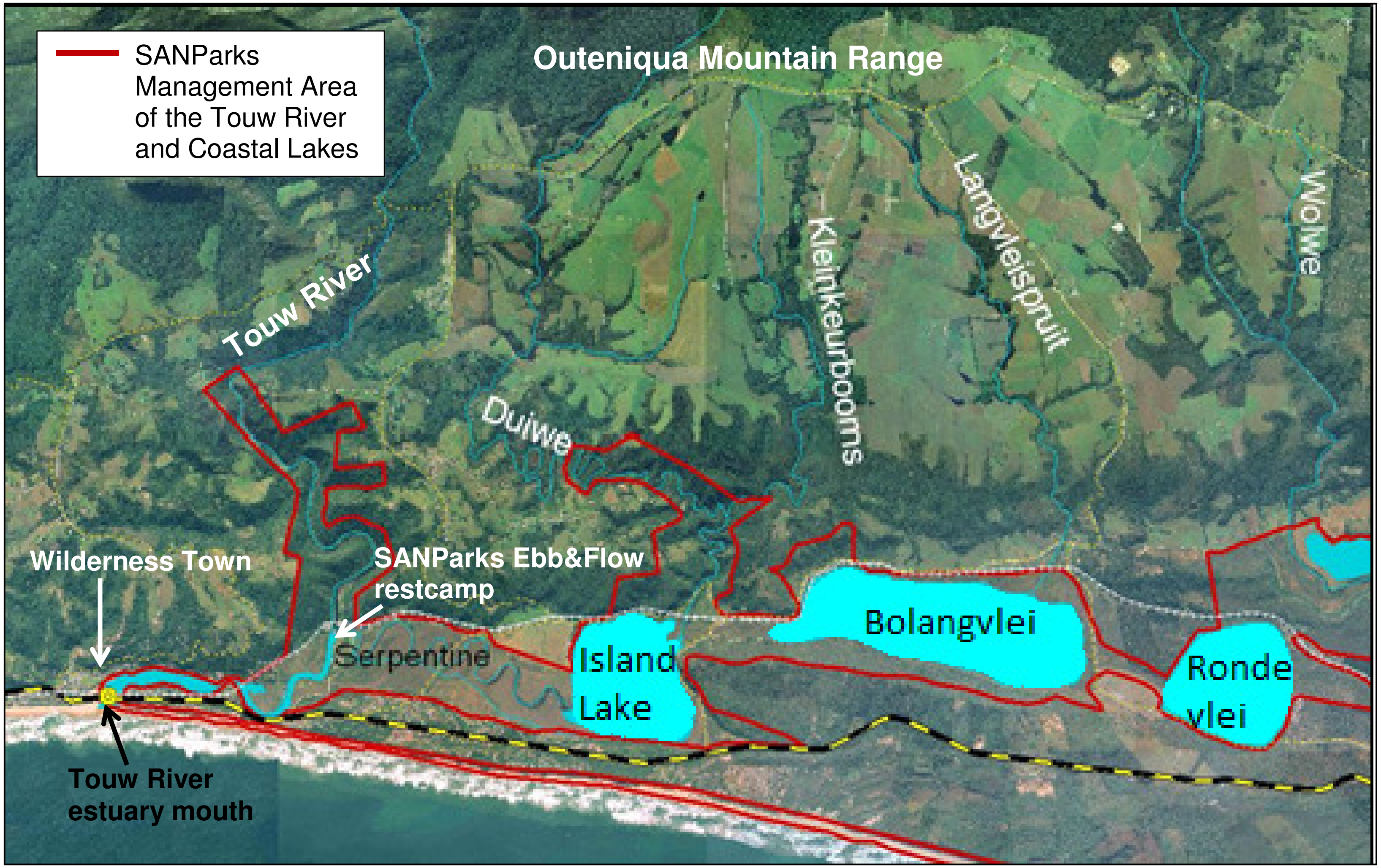


Figure 2-1: Map of the Garden Route National Park - Wilderness Coastal Section (SANParks, 2013)

2.2. Hydrodynamics of the Touw River and Wilderness Coastal Lakes

The hydrodynamic elements within the catchment area consist of an interconnected system between rivers, lakes, an estuary and the ocean (Fijen, 1995). The wilderness lakes are connected to the Touw River via the Serpentine River and discharge to sea via the estuary mouth at the town of Wilderness. This estuary acts as a transition zone between a fresh water environment and an oceanic environment. According to Day in 1980 an estuary can be described as “a partially enclosed coastal body of water which is either permanently or periodically open to the sea and within which there is a measurable variation of salinity due to the mixture of seawater and fresh water derived from land drainage.” (Day, 1980). The estuary is subject to both marine influences, including tides and the influx of saline water and beach sand as well as river influences such as freshwater flows and riverine sediments. The two main types of estuaries are initially distinguished by their connection to the ocean, namely open estuaries and temporarily open/closed estuaries, which become separated from the sea due to the formation of a sand barrier between the estuary mouth and the ocean. The estuary at Wilderness can be classified as a temporarily open/closed estuary, where the estuary is periodically isolated from the ocean by barrier beaches which build up as a result of wave action and longshore sediment transport and during periods of low river flow (Watermeyer Prestedge Retief, 1994). The CSIR reported in 1981 that tidal exchange only took place when the estuary was open to the sea, which occurred approximately 27% of the time in the Touw River estuary (CSIR, 1981). An open mouth as well as a closed mouth condition for the estuary mouth at Wilderness can be seen in Figure 2-2 and Figure 2-3 below.

Figure 2-2: Closed mouth condition at Wilderness (Watermeyer Prestedge Retief, 1994)

Figure 2-3: Open mouth condition at Wilderness (Watermeyer Prestedge Retief, 1994)

From Figure 2-1 it is evident that the estuary is dominantly supplied from the inflow of two rivers, the Touw River as well as the Serpentine River. The latter of these rivers also provides a link to the first of a series of three lakes in the Wilderness lakes system, namely Island Lake. Water in this lake is chiefly supplied by the Duiwe River as well as a connecting channel which meets the second of the Wilderness Lakes, namely Bolangvlei. Bolangvlei is the biggest of the Wilderness Coastal Lakes and is dominantly supplied by the Langvlei Spruit and a connecting channel which meets the last lake of the system, Rondevlei Lake. There is no influent river into Rondevlei and the water in the lake is predominantly recharged by groundwater (Fijen, 1995).

A critical component to the hydrodynamics and salinity levels experienced within the estuary and entire water system is the formation of the sand barrier between the estuary and the ocean. The sand barrier, when closed, has a strong influence on the high flood water levels that are achieved within the estuary and the implications thereof are discussed in later sections. The sand barrier also acts as a blockage and prevents any marine inflow during closed conditions. Therefore, a significant impact on salinity levels within the entire water system is also experienced during different mouth situations (open or closed). The formation and implications of a sand barrier are described in the following sections.

An additional component affecting the hydrodynamics of the entire system is the large volume of debris associated with flood events. Debris is transported throughout the system and as a result, the smaller channels in the system become heavily constricted and water flow within the system is compromised. The likelihood of this occurrence is significantly high given that the Wilderness coastal lake system is located in an area with dense fynbos and forest vegetation (South African National Parks, 2012), posing an increased risk of debris build-up during flood events. Large volumes of debris have also been confirmed in the past. In a media briefing after the major flood that took place in 2007, the existing park manager Jill Gordon stated that there were “huge deposits of sediments and debris from the floods.” (South African National Parks, 2008). Figure 2-4 also shows the extent of debris during past flood events.

Figure 2-4: Debris deposited on the beach as a result of a flood event in Wilderness

3. LITERATURE STUDY

3.1. Sand Bar Formation at Estuary Mouths

3.1.1. Introduction

This section describes the dynamics and the influences involved in the formation of sand bars at temporarily open/closed estuaries. Temporarily open/closed estuaries often occur in environments that contain significant wave exposure and where stream flow in the river is highly seasonal (Rich & Keller, 2013). Global locations where these conditions are apparent include California, South Africa and Australia (Rich & Keller, 2013). The Australian east coast is recognised as a global coastal location well known for temporarily open/closed estuaries. Figure 3-1 shows a number of temporarily open/closed estuaries which occur on the Australian east coast.

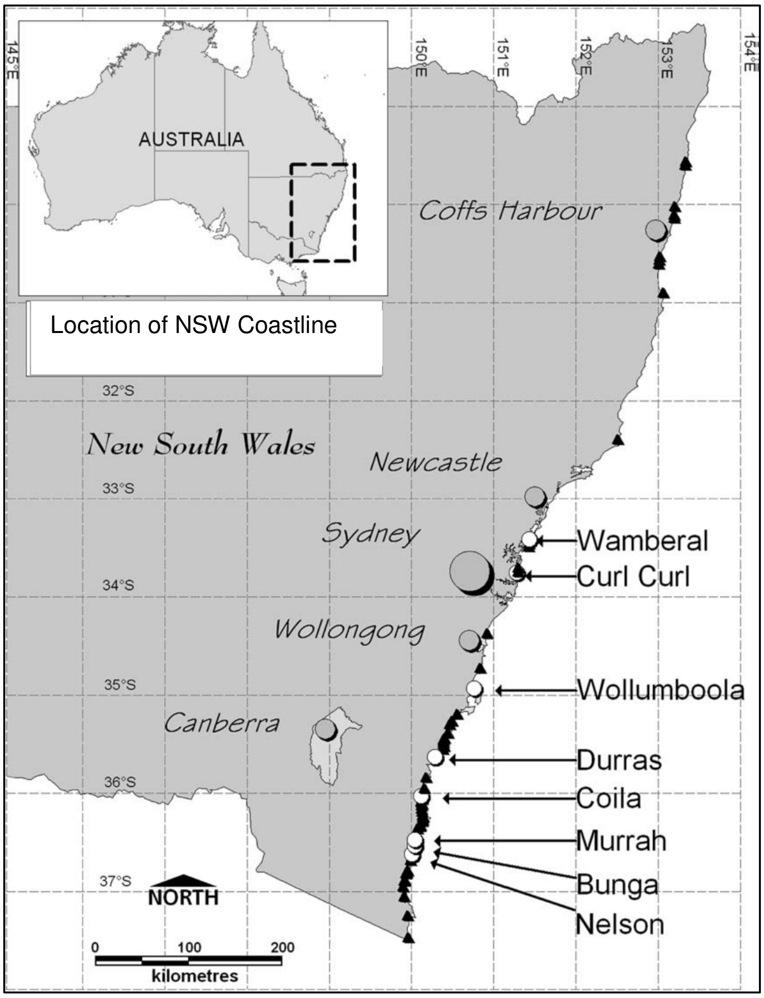


Figure 3-1: Location of temporarily open/closed estuaries along the Australian east coast (Heines, et al., 2006)

There are a number of influencing factors which affect the formation of the sand bar as well as the resulting berm height. Major factors contributing to the formation of the sand bar at an estuary mouth include longshore sediment transport and tidal patterns, river flow and wave energy. These factors often act collectively under different mechanisms to control the state of the estuary mouth.

3.1.2. Longshore Sediment Transport

Longshore sediment transport along the coastline is one of the dominant supplies of marine sediment to estuary mouths. This is the process whereby sediment is moved parallel to the coastline by wave and current action (Schoonees & Theron, 2002). Typical influential factors affecting longshore sediment transport include the angle of wave attack, wave height, wave length, depth of water, water density and viscosity, gravitational acceleration, duration of event and the sediment particle size and density (Kamphuis, et al., 1985). In the nearshore zone, breaking waves suspend most of the transported sediment. Waves approaching the coastline with an oblique angle generate longshore currents and as a result, these currents transport the suspended sediment alongshore, parallel to the coastline. The longshore transport rate is the rate at which sediment is moved parallel to the coast in the littoral zone. This rate is usually expressed as a volume per time, m^3/year (Schoonees & Theron, 2002). Longshore sediment transport rates across the South African east coast are shown in Figure 3-2. It can be seen that typical net longshore transport rates across the South African east coast range from 150 000m^3/year to 850 000m^3/year.

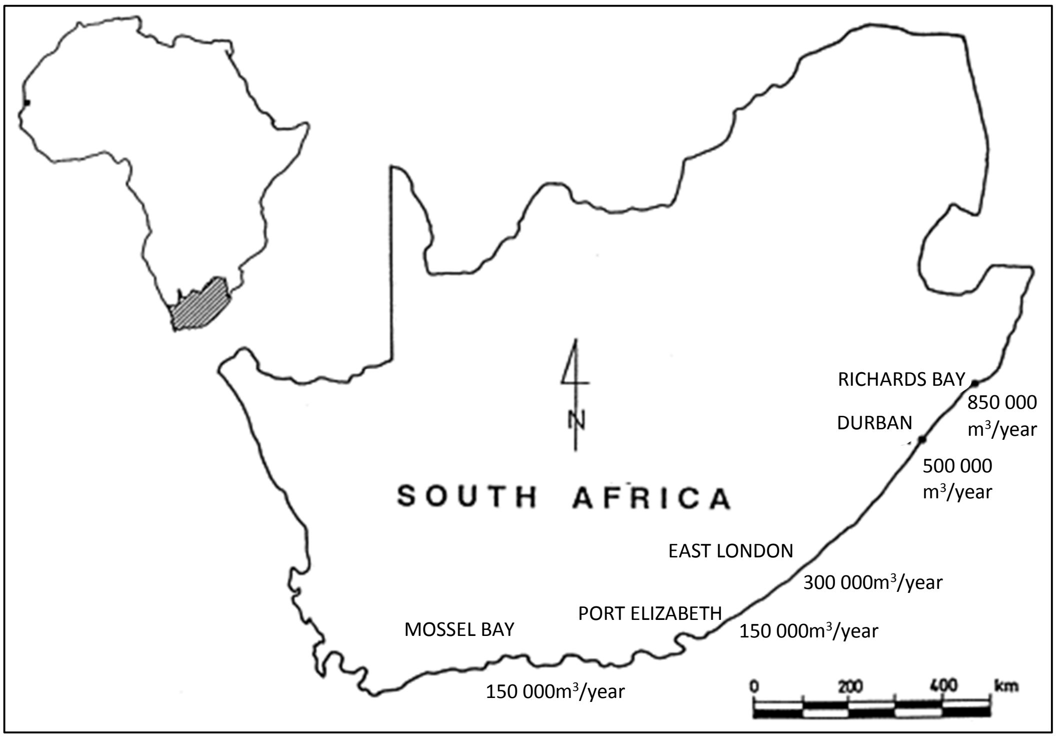


Figure 3-2: Net longshore sediment transport rates along the South African east coast in a north easterly direction (modified from (Schoonees, 2000))

Van Rijn states that in order to maintain an open estuary mouth, a balance is necessary between the scouring action of the tidal currents that keep the channel open and the longshore transport of beach sand that tends to close the inlet (Van Ryn, 1998). A relationship exists between the volume of water in the tidal prism passing the inlet and the cross sectional area of the tidal inlet. As the tidal prism decreases, the inlet cross sectional area decreases at an exponential rate in order to maintain an equilibrium inlet state. It is, therefore, established that mouth closure often occurs during neap tides, when the tidal prism is relatively low. A low tidal prism results in a constriction the inlet cross sectional area. Furthermore, longshore transport rates remain unchanged during neap tides and the balance is offset between the scouring action of tidal currents and longshore sediment transport. An accumulation of sediment is, therefore, evident in the inlet area which tends to close the inlet.

3.1.3. Wave Action

The effect of wave action and wave energy, especially over neap tides, is a big contributing factor behind estuary mouth closure. High waves cause a strong turbulence and, therefore, during a flood tide, large quantities of sediment are transported into the mouth. As the water level and turbulence subside, the sediment settles resulting in a deposition of sediment in the mouth region of the estuary. Many South African estuaries are flood-dominated estuaries implying that the flood currents within the estuary are stronger than the ebb currents (Beck, 2005). As a result, sediment entering the mouth during flood tides cannot all be removed during the ebb tide and a resulting net gain of sediment is evident at the inlet region. Furthermore, high waves are predominantly a winter influence along the South African coast. It should, therefore, be easier to maintain an open mouth during the spring and summer months when wave energy has subsided and longshore sediment transport is less prevalent.

3.1.4. River Flow

For Southern California estuaries it was shown that river flow is the major control of the mouth state and that tidal patterns and wave energy play a smaller role during closure (Elwany, et al., 1998). Elwany, et al. mentions that upstream river flow is the primary driving factor behind the hydrodynamics of estuaries. A reduction in river flow, therefore, results in less scouring potential at the mouth of the river which may result in a build-up of sediment at the estuary mouth, which is caused by marine influences such as wave action and longshore sediment transport. However, during flood conditions, large quantities of sediment are scoured from the estuary mouth due to high volumes of river discharge, thus resulting in an open mouth condition. Rich & Keller mention that the length of time an estuary remains closed before breaching is a function of stream flow, estuary storage at breaching levels and barrier seepage, whereas mouth closure probability is best predicted using a ratio of estuarine discharge to sediment transport by waves (Rich & Keller, 2013).

3.1.5. Additional Factors

Additional minor influences may also contribute to the final sand bar height including beach grain size as well as aeolian sediment transport. Temporarily open/closed estuaries tend to occur on coastlines with coarse beach sediment. Coarse beach sediment is often associated with steep beach profiles which in turn result in high wave run-up. Sediment is, therefore, transported to higher elevations by wave run up and sand barriers are created reaching levels well in excess of the high water mark. (Parkinson, 2007). Aeolian processes also contribute to the height of the sand berm via dune formation at the backbeach (Rich & Keller, 2013).

3.1.6. Mechanisms resulting in estuary mouth closure

According to Ranasinghe & Pattriaratchi there are two main mechanisms which result in the formation of a sand barrier (Ranasinghe & Pattiaratchi, 1999).

- Mechanism 1: The interaction between the inlet current and the longshore sediment transport process.

  During the longshore sediment transport process, sediment is transported parallel to the coastline due to the currents generated from an oblique angle of wave attack. Breaking waves suspend the sediment in the surf zone and during open mouth conditions at a temporary open/closed estuary, the discharge from the estuary interrupts the longshore transport process and sediment begins to accumulate, forming a spit on the up drift side of the estuary mouth. The size and growth rate of the spit is dependent on the size of sediment, the availability of sediment and the intensity of both longshore current flows and river flows. During times of low river flows, the estuary mouth experiences a reduction in scour potential and the continuous supply of sediment from longshore processes result in an increasing growth of the spit across the estuary mouth and can result in complete closure of the estuary. This process is shown schematically in Figure 3-3.

- Mechanism 2: Interaction between the inlet current and the cross shore sediment transport process

  During the cross shore sediment transport process, sediment is transported perpendicularly to the coastline due to currents formed from a wave attack normal to the coastline. The onshore or offshore sediment transport is dependent on a number of wave characteristics such as steepness, bed slope and wave intensity. However, periods of continuous onshore transport can result in the closure of an estuary mouth, as can be seen in Figure 3-3.

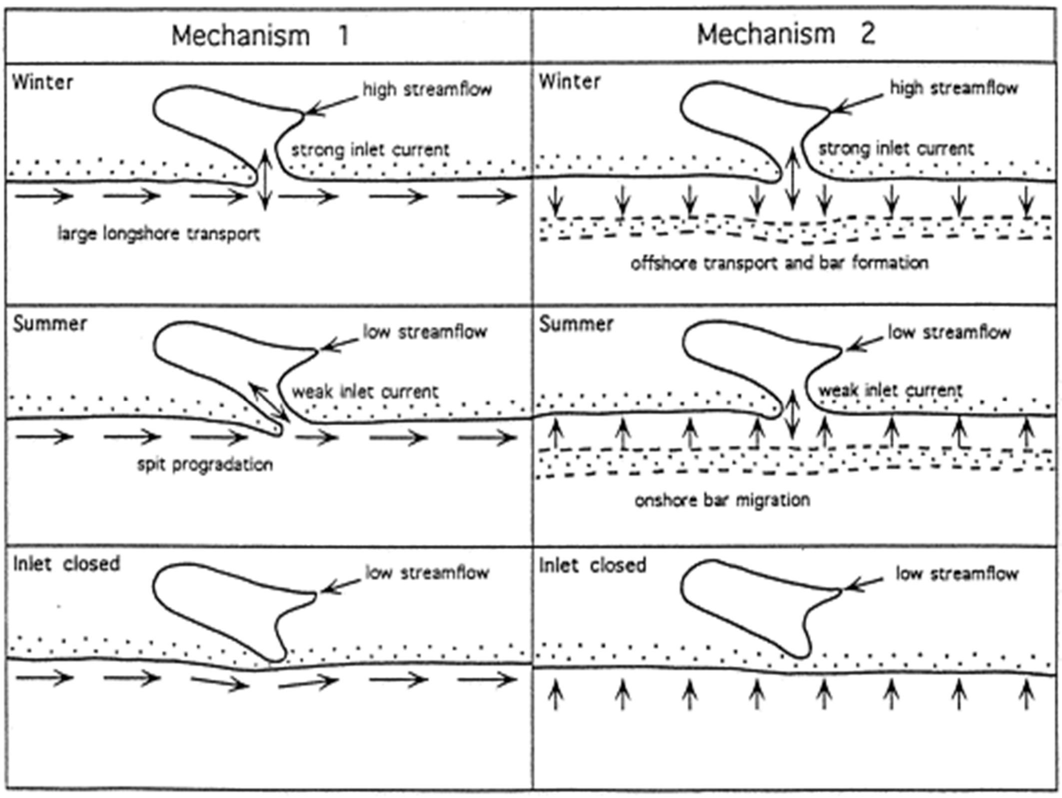


Figure 3-3: Schematic Depiction of inlet closure by longshore processes and cross shore processes (Ranasinghe & Pattiaratchi, 1999)

Once the estuary has been closed off to the ocean, tidal exchange between the estuary and ocean no longer occurs; however, deposition may still continue above the high water mark. Wave run up onto the sand barrier may continue to deposit sediment and build the sand barrier, reaching elevations beyond the high water mark.

3.2. Artificial Manipulation of Estuary Mouths

3.2.1. Artificial Breaching by construction of a preparatory channel

Artificial breaching involves physically breaching the sand barrier in front of the estuary before the natural breaching of the estuary occurs. This is typically practised to prevent local property flooding in areas surrounding the estuary and may also be conducted due to ecological reasons such as the migration of certain fish species (James, et al., 2007). At certain times of the year, fish migrate into estuarine systems in order to breed. During these specific times of the year, estuaries may be artificially opened, in order to facilitate fish migration into the estuaries.

The process involves digging a preparatory channel across the crest of the sand berm where the base of the channel is situated at a level below the surface level of the water in the estuary, thus initiating water flow from the estuary to the ocean (Watermeyer Prestedge Retief, 1994). The hydraulic head difference between the estuary and the ocean forces water in the direction of the ocean and as the water moves within the artificially constructed channel, the scouring potential increases and eventually forms a complete open mouth situation. A sand plug may also be constructed on the seaward side of the preparatory channel. This is often constructed before major flood events and consists of a narrow wall of sand, constructed across the preparatory channel. As water levels in the estuary rise as a result of the flood, the sand plug is easily scoured resulting in a complete open mouth.

Artificial breaching may also have adverse effects on water quality and other ecological components within the entire water system. Artificial breaching causes unnatural flushing conditions of the water system (Beck, 2005), which in turn may have a consequential effect on the ecology of ecosystem (Fijen, 1995). During natural conditions, the sand barrier may reach elevations well above the high water mark which would result in natural water levels in excess of the sand bar elevation within the estuary before natural breaching would occur (Watermeyer Prestedge Retief, 1994). Due to the greater difference in water levels between the estuary and the ocean under natural conditions, a large water gradient develops. This causes high flow velocities throughout the system when the sand barrier is breached. In turn, this causes high scour rates of sediment and minimizes the accumulation of sediment at the estuary mouth. When a system is able to flush efficiently, an abundance of freshwater enters the system during a flood, as previous estuarine water is flushed out to sea. Due to increased scour rates in the system, a greater cross sectional mouth area develops. The inlet remains open for long periods of time thus allowing the propagation of salinity throughout the entire system. It can, therefore, be concluded that a greater flushing potential results in greater range of salinity levels throughout the system which has a consequential effect on the species diversity of the flora and fauna in the area, as different species thrive under different salinity conditions. Artificial manipulation of the sand bar by construction of a preparatory channel can be seen in Figure 3-4.

Figure 3-4: Preparatory channel construction in Wilderness (Watermeyer Prestedge Retief, 1994)

3.2.2. Sand bar Skimming

Sand bar skimming involves the process whereby the sand berm levels in front of the estuary are maintained at a constant level (Fijen, 1995). This is often performed in an effort to minimise peak water levels in the estuary. When water levels in the estuary increase above the sand berm height, water breaches the sand bar and flows out to the ocean. This is often practised in areas susceptible to property flooding along the banks of the estuary. Maintaining the sand barrier height at a specific level limits extreme water levels experienced within the estuary. This limits the natural water levels that are achievable in the estuary and as a result has a direct impact on the ecology of the ecosystem. A lack of total flushing efficiency is evident and similar ecological problems are experienced to those mentioned in the case of the construction of the preparatory channel. The sand bar skimming process can be seen in Figure 3-5.

Figure 3-5: Sand Bar Skimming practised in Wilderness (Watermeyer Prestedge Retief, 1994)

3.3. General Characteristics of Estuarine Salinity

3.3.1. Introduction

An estuary is an intermediate zone between a river and the ocean which has both river and marine influences (National Ocean Service Education, 2008). Salinity within an estuary is, therefore, determined by the interaction of seawater from the ocean and freshwater from the upstream river. In freshwater, the concentration of salts is nearly zero ppt (parts per thousand), however, the salinity of the ocean averages about 35 ppt (National Ocean Service Education, 2008). This mixture between seawater and freshwater in estuaries is called brackish water and the salinity can range between 0.5 – 35 ppt. The salinity of an estuary varies significantly and can change from one day to the next depending on the rainfall, tides and the condition of the estuary mouth (open or closed) as well as a number of other factors (Levinton, 1995).

3.3.2. Intrusion, retention and removal of seawater in estuaries

During open mouth conditions at temporary open/closed estuaries, the tidal influx of salt in an estuary is by way of a two layer estuarine circulation. Seawater enters the estuary during a flood tide as a single layer which plunges beneath the ambient estuarine water (Largier & Taljaard, 1991). This can be seen in Figure 3-6.

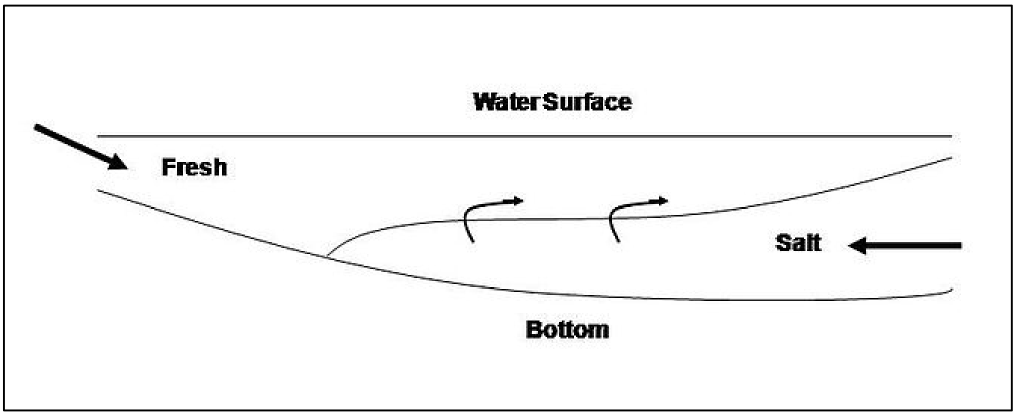


Figure 3-6: Salinity intrusion into estuaries

Seawater is then diluted with the ambient estuarine water and this diluted seawater can then propagate up the estuary as a gravity driven bottom density current. On the ebb tide, the seaward-flowing layer of freshwater entrains and removes saline water from the halocline. Saline water may also become trapped, especially in flood-dominated estuaries where the flood tide is greater than the ebb tide (Largier & Taljaard, 1991).

During periods where the mouth is closed, there is no interaction between marine and freshwater sources. During this time, salinity in the estuary is primarily determined by other factors including freshwater inflow from rainfall and run-off, evaporation and groundwater leakage.

The action of gravity upon the density difference between seawater and freshwater tends to cause a vertical stratification in the water column. This can also result in a convectional flow within the estuary which is termed estuarine circulation (Hsu, et al., 1999). The driving force of estuarine circulation is the horizontal salinity gradient which induces a vertically varying pressure gradient.

3.4. Mouth dynamics at the Touw River estuary and Coastal Lakes

During closed mouth conditions under natural circumstances, the sand bar at the estuary mouth may reach elevations in excess of +3m MSL (Fijen, 1995) (Watermeyer Prestedge Retief, 1994). Given that natural breaching of the sand bar will only occur once the water levels upstream of the sand bar exceed the sand bar height, high water levels will occur in the estuary and surrounding water bodies during flood conditions. Water flows down from freshwater sources and is allowed to propagate throughout the water system until water levels at the mouth exceed the height of the sand bar. Thereafter, natural breaching of the sand bar commences and the estuary opens to the ocean. Furthermore, a large head of water is also built-up between the water system and the ocean creating a large flushing potential of the entire water system. When breaching occurs, the large difference in water head causes rapid flows of water into the ocean, which, in turn result in large scour volumes of sediments at the estuary mouth. As a result, the estuary remains open for long periods of time which allows for marine influences to propagate into the estuary as well as the upper coastal lakes.

However, in the case of Wilderness, the natural conditions of the sand bar at the estuary mouth often lead to detrimental implications with regards to peak water levels in the estuary and surrounding water bodies. Given the fact that residential property has been constructed at elevations below natural flood water levels, SANParks, the managing board of the Touw River estuary and coastal lakes system, is forced to implement a management strategy involving the artificial manipulation of the sandbar in order to reduce flood levels within the entire system.

3.5. Past experience on the Touw River mouth management

The Lakes Area Development Board managed the Wilderness lakes area including the Touw estuary between 1973 and 1983. The procedure which was adopted included skimming the sand bar by bulldozers in preparation for flood events which could result in excessive flooding of low lying areas (Watermeyer Prestedge Retief, 1994). A relatively wide trapezium shaped channel was constructed on the beach berm. Artificial closure of the mouth was sometimes also performed during peak holiday season to allow for sufficient depth for boating activities in the estuary (Watermeyer Prestedge Retief, 1994).

Since 1983, SANParks has been solely responsible for the management of the Touw River and Wilderness lakes area (Watermeyer Prestedge Retief, 1994). For the past thirty years, the management practice adopted by SANParks, is based on the CSIR studies performed in 1981 and 1982. During a closed mouth condition, the sand bar is maintained at an elevation between +2.1m and +2.4m MSL (Fijen, 1995). Surveys of the sand bar are performed weekly in order to monitor the sand bar elevation. A preparatory channel is also constructed in the event of severe flooding predictions (South African National Parks, 2010). This management plan is described in detail in Chapter 3.7.

3.6. Previous Studies and Proposed Solutions for the Touw River Estuary and Coastal Lakes

3.6.1. CSIR Flood Studies

The flood studies performed by CSIR incorporated field data analysis as well as numerical model simulations performed during 1979 and 1981. Results of the study showed that floods in the Touw River normally result in a quick rise in water level within the estuary. However, the water levels in the lakes increase at a much slower rate due to the small dimensions of the Serpentine River and connecting channels between the lakes (CSIR, 1982).

During floods in the Touw River, there is little time available for mouth breaching before water levels in the estuary reach a destructive level. Therefore, since 1983, the policy accepted by the South African National Parks board is to maintain the level of the sand berm at the mouth of the estuary at a height between +2.1m and +2.4m MSL. The primary reason of this action is to prevent flooding of low lying houses along the estuary. The maximum safe flood level based on previous studies by CSIR is +2.6m MSL in the estuary and +2.4m MSL at Island Lake. This is the level above which flooding of residential property will commence.

Figure 3-7 outlines the areas of low lying residential property in the Touw River estuary and Island Lake.

Figure 3-7: Low lying areas along the Touw River estuary and Island Lake - adapted from (Watermeyer Prestedge Retief, 1994)

The flood levels for various flood predictions can be seen in Table 3-1. The Table gives results based on a sand berm height at the estuary mouth of +2.2m MSL and initial water levels of both 1.4m and 1.6m MSL, which corresponds closely to a realistic situation. This Table is based on the interpretation and the results of the numerical models performed by CSIR in 1982. It must be noted that these results are adapted from model simulations that were performed using either a fixed sand berm level or a completely open mouth condition. The scour of the sand bar during the flood event was not modelled.

Table 3-1: Flood peak levels in the Wilderness system for a sand bar height of +2.2m MSL (CSIR, 1982)

Return Period (Years)	Initial Water Level (+m MSL)	Flood Peak Levels (+m MSL)		
		Estuary	Eilandvlei	Langvlei and Rondevlei
5	1.4	2.45	1.76	1.51
	1.6	2.42	1.98	1.70
10	1.4	2.52	1.93	1.54
	1.6	2.5	2.09	1.73
20	1.4	2.59	2.03	1.57
	1.6	2.58	2.17	1.76
	2.0	2.53	2.39	2.15
50	1.4	2.65	2.14	1.61
	1.6	2.64	2.27	1.80
100	1.4	2.94	2.39	1.65
	1.6	2.93	2.5	1.87

The results of the study are interesting with regards to the flood levels and different initial water levels in the system. As expected, in the lakes a lower initial water level results in a lower flood water level. However, this is interestingly not the same case for the estuary. A lower initial water level (+1.4m) in the estuary results in a higher flood level compared to the results obtained with a slightly higher initial water level (+1.6m). This may be explained by the fact that during lower water levels in the estuary, water flows through Serpentine from Island Lake to the Touw River estuary. This may impede the flow from the Touw River to Island Lake and during a flood event in the Touw River, this impedance may force the majority of the flood discharge to the estuary, thus leading to higher water levels.

Table 3-1 also shows that flooding of property will begin to occur for a flood with a return period in excess of 20 years. It must, however, be noted that this case is only relevant for a sand berm height of +2.2m MSL. If, however, the sand berm was maintained at +2.1m MSL as at present, it can be inferred that flooding will occur for floods with a return period well in excess of 50 years.

During 1979, flood hydrographs were also being developed for the Wilderness system to support the CSIR numerical model study that was being constructed at the time. The results of the unit hydrograph constructed during this period can be seen in Table 3-2.

Table 3-2: Flood Hydrographs for Wilderness (1979 estimate) (Görgens, 1979)

	Return Period in years				
	5	10	20	50	100
Peak (m^3/s)	109.1	145.7	170.4	183.1	278.3
Duration (h)	51	51	51	51	51
Volume (million m^3)	4.5	6.0	7.0	7.6	11.5

Subsequent to this study, it was discovered that certain maximum water levels, as a result of floods, occurred more frequently than those based on the 1979 flood estimates. These flood estimates seemed inaccurate and misleading and in 1994 a new study was undertaken in order to provide updated flood estimates in the Wilderness area. In addition, ten more years of river flow data was available to verify and calibrate results. The results of the 1994 study are shown in Table 3-3.

Table 3-3: Flood Hydrographs for Wilderness (1994 estimate) (Görgens, 1994)

	Return Period in years				
	5	10	20	50	100
Peak (m^3/s)	98	150	213	329	403
Duration (h)	40	40	40	40	40
Volume (million m^3)	3.4	5.2	7.4	11.4	14

There are some interesting differences between the 1979 and 1994 estimates. Firstly it is estimated that the duration of a flood is shorter in the recent estimate by approximately 20%. It is also interesting that the 1994 estimates predicted a flood with a return period of 5 years has a lower flood discharge peak and a lower flood volume compared with the 1980 estimates, whereas for higher return periods, both peak discharges and flood volumes are underestimated in the 1979 study. A comparison of the results is shown graphically in Figure 3-8 and Figure 3-9.

Figure 3-8: Comparison between different estimates for the peak flood discharges at different return periods (Görgens, 1994) (Görgens, 1979)

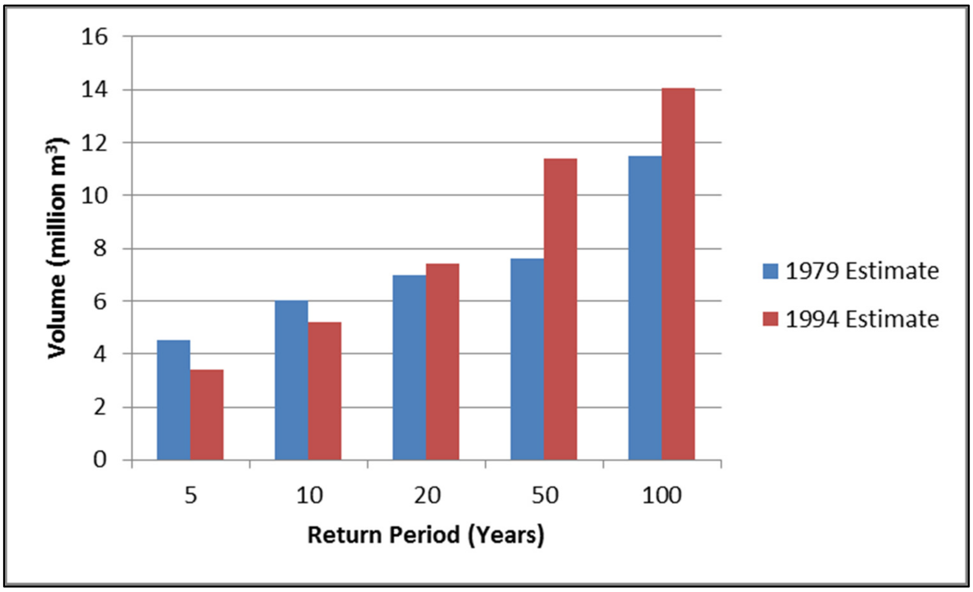


Figure 3-9: Comparison between different estimates for the flood volumes at different return periods (Görgens, 1994) (Görgens, 1979)

3.6.2. CSIR Proposed Solution

Numerical models were also constructed to try and alleviate the large flood level peaks in the estuary at low initial water levels. One of the proposed solutions involved the dredging of the Serpentine River from +0.0m to -1.0m MSL. However, studies showed that this solution offered no significant benefit and would only decrease flood levels in the Wilderness estuary by 0.1m (CSIR, 1981). A solution in order to reduce flood levels in Island Lake was also proposed. This involved dredging the connecting channel between Island Lake and Bolangvlei from its present level of +1.0m MSL to +0.0m MSL (CSIR, 1981). The results of this model showed a significant improvement on the water exchange of the lake system (CSIR, 1981). Flood levels in Island Lake would decrease by 0.3-0.4m as floodwater could more easily flow to Bolangvlei and Rondevlei.

The dredging of the interconnecting channels was also seen as beneficial from an ecological point of view, as the increased water exchange would improve conditions for plants, fish and invertebrates (Fijen, 1995). However there was also a concern with the drainage of the lakes during open mouth conditions since dredging of the interconnecting channels between the lakes would also improve the outflow from the upstream lakes, and lakes would drain more quickly. A sluice gate was recommended by the CSIR in 1981, in order to prevent the drainage of the coastal lakes, however this was not modelled.

3.6.3. SANParks Management Plan

Subsequent to the studies performed by the CSIR, dredging operations were commenced and a sluice gate was constructed in the Serpentine River in order to prevent drainage of the upstream lakes during open mouth conditions. During closed mouth conditions, the water stored in the upstream lakes could be used to raise the water level in the estuary again, thus reducing the risk for potential flood damage. However, the dredging of the channels was never satisfactorily completed as initial dredging operations were stopped after experiencing ecological problems (Fijen, 1995). As a result, the sluice gate also lost its purpose and up until 1995 had never been used (Fijen, 1995).

In 1994, The South African National Parks board also suggested a change in the procedure for managing the estuary mouth (Watermeyer Prestedge Retief, 1994). Rather than allowing the sand berm to close naturally, the procedure would involve the mechanical closure of the mouth by means of a bulldozer to create a narrower sand berm compared to the resulting width of the sand berm under natural conditions. The narrower sand berm would scour out more easily and thus more quickly, leading to a reduction in maximum flood level elevations. This procedure was subsequently evaluated by a consulting engineering firm, Watermeyer Prestedge Retief, where studies showed that maximum flood levels for a 100 year flood could be reduced by 0.2m if the width of the sand berm was reduced from 100m to 50m and by 0.3m if the sand berm was reduced from 100m to 20m (Watermeyer Prestedge Retief, 1994).

Photographs of the artificial berm construction as well as the final result after construction can be seen in Figure 3-10 and Figure 3-11.

Figure 3-10: Construction of the narrow sand berm (Watermeyer Prestedge Retief, 1994)

Figure 3-11: Narrow sand berm after construction (Watermeyer Prestedge Retief, 1994)

This practice is no longer in use and the current management plan is described in the following section.

3.7. Current Management Plan for the Touw River estuary and Wilderness Coastal Lakes

3.7.1. Objective

The objective of the estuary management plan at Garden Route National Park is to "Reduce the probability of inundation of residential areas on adjacent floodplains whilst at the same time maintaining sufficient head of water to maximise the scouring of sediments from the estuary mouth on breaching and maintain natural inundation flux in marginal wetlands, by maintaining the system of artificially breaching estuaries in the Touw system" (South African National Parks, 2010).

3.7.2. Current Management Plan

The current management plan is found in Appendix F of the Garden Route National Park management plan (South African National Parks, 2010). Relevant text relating to Touw River estuary and Wilderness coastal lakes can be seen below.

During periods when the estuary mouth is closed, sandbar skimming is the current practise of the Touw River mouth management. The height of the sandbar at the estuary mouth may be reduced by earth moving equipment operated by, or under the direct supervision of SANParks personnel. A portion of the sandbar, stretching from the estuarine water's edge to the edge of the sea high water mark is maintained at a height of between +2.1m and +2.4m MSL. The width of the maintained sandbar level will not exceed more than 5m. Surveying is also performed on a regular basis to ensure that the maximum sandbar levels are not exceeded. Surveying of the sandbar is performed at intervals not greater than seven days and in the event of high seas and severe weather warnings, re-measuring of the sandbar height will take place at least once every two days (South African National Parks, 2010).

A preparatory channel may also be constructed at any time prior to estuary breaching if deemed necessary by SANParks personnel to facilitate future breaching. Furthermore, a preparatory channel will be prepared when water levels in excess of +1.8m MSL are evident in the Touw estuary and when a high rainfall event (>50mm) is forecast. The extent and design of the preparatory channel must be such that premature breaching of the estuary (< +2.1m MSL in the Touw estuary) cannot occur as a result of either sediment erosion caused by high seas or acts to breach the estuary by unauthorized persons. The width of the channel should not exceed 5m and the elevation of the plug, which is constructed on the seaward side of the preparatory channel, should remain between +2.0m MSL and +3.0m MSL.

3.8. Recent floods in the Touw River estuary and Wilderness coastal lakes

Although the current management strategy has been deemed to be the best solution for controlling water levels within the entire water system, the management plan has not always worked as well as originally anticipated. Major recent flood events include the floods of 2003, 2006 and 2007. In all three of these flood events, water levels reached elevations above the safe limit which resulted in damage to property. It is difficult to evaluate whether this is a result of an unusually large flood, inaccurate flood level predictions, an inadequate management plan or simply the poor implementation of the current management plan of the estuary mouth. Sometimes floods are just too big and, irrespective of the management plan, they cannot be dealt with or prevented. The management plan has, however, significantly reduced problems in less extreme floods.

In March 2003, heavy rains caused flooding in the Garden Route National Park – Wilderness coastal section as well as the surrounding residential areas. The Touw River breached its banks eight hours after the mouth was artificially opened. “The mouth was opened at around +2.1m MSL, which is well within the recommended range for opening the mouth” (SANParks, 2003). According to previous studies and simulated water level predictions performed by the CSIR, residential property should not have been affected with the implementation of this management strategy. However, it is difficult to perform a comparative study between the results of these floods and the simulated results performed by CSIR in 1982, as the return period for the flood is unknown. The worst case water level predictions for a 20 year and 50 year flood are +2.59m and +2.65m MSL respectively. These are the worst case results based on initial water levels in the estuary. It must also be noted that the flood levels estimated in the simulation are based on a slightly higher initial berm height of +2.2m and not +2.1m as in the case of the 2003 floods (SANParks, 2003). However, it is known that flood levels within the estuary, during the 2003 floods exceeded +2.6m due to the fact that property was damaged. Water damage to property only occurs at water levels above this elevation. Therefore, it is concluded that water levels achieved in the estuary as a result from the flood were therefore greater than the anticipated water levels obtained from previous simulated studies.

The August 2006 floods portrayed a similar result. The rest camp and staff houses were evacuated and as a result no injuries were sustained. However, low lying surrounding properties around the estuary were affected by the floods. A photograph showing the extent of the flood can be seen in Figure 3-12.

Figure 3-12: Result of 2006 flood in Wilderness (Ravens-Ernstzen, 2006)

The park manager at the time, Roy Ernstzen, said, "Although the Touw River mouth was breached at 17:30 on the Tuesday evening at 2.14m above mean sea level (Flood peak was sometime during Wednesday), the unusually heavy rains experienced combined with good rainfall during the month, caused flooding of all the low lying areas, including the Ebb & Flow rest camp and some of the surrounding properties. The Touw River mouth is normally artificially breached when water levels reach 2.2m above mean sea level; however, the peak flood waters exceeded 2.8m above sea level, despite the mouth being open." (Ravens-Ernstzen, 2006)

Heavy floods were also recorded in November 2007 where surrounding property around the estuary was once again damaged by floods. Nearby roads and transport links also suffered damage and were flooded and the National Park was closed for four weeks while clean-up operations were undertaken. Figure 3-13 and Figure 3-14 below show the extent of the 2007 flood at Ebb & Flow.

Figure 3-13: Result of 2007 flood in Wilderness (Photo: P. Huizinga)

Figure 3-14: Ebb & Flow camping site during 2007 flood (Photo: P. Huizinga)

In the period of five years from 2003 to 2007, three major floods have occurred which have all resulted in damage to property. The maximum safe water level determined in previous studies (CSIR, 1982) is +2.6m MSL in the estuary which implies that all three floods exceeded this level. The numerical simulation studies performed by CSIR in 1982 calculated that water levels of this height in the estuary would also only ever be reached with a flood having a return period of more than 20 years. This appears not to be the case and it is likely that the resulting water level predictions by CSIR for the estuary after a flood event are an underestimate of the true conditions experienced in the estuary. It is beyond the scope of this investigation to establish the contributing factors to previous high flood levels in the system, but one outcome is certainly made clear from the previous recent flood events. Flooding still remains a problem in the Touw River and Wilderness coastal lakes and to date has not been completely solved with the implementation of the existing management strategy. The aim of this investigation is not to propose an entirely new management plan. Rather, this investigation will aim to improve on the understanding of the hydrodynamics within the estuary and the resulting water levels that can be expected within the system after flood events and, therefore, assess the effectiveness of the current management plan.

3.9. Salinity Characteristics of the Wilderness estuary and coastal lakes system

The salinity characteristics of the Wilderness estuary and coastal lakes system are relatively unique due to the interaction between components involving multiple freshwater inflow sources, coastal lakes and a temporary open/closed estuary mouth.

The Touw estuary has a typical estuarine longitudinal salinity gradient with significantly higher salinities at the mouth compared to those at the head of the estuary (Russell, 2013). The typical transition periods with regards to salinity in the Touw River estuary and Wilderness coastal lakes are described. Immediately following a flood event when the sand bar at the estuary mouth is breached, the system is in an outflow phase, implying that estuarine water drains into the ocean. Salinity levels during this phase are relatively low throughout the system (Russell, 2013), due to the abundance of freshwater in the system. Freshwater floods the system before the influx of saline water from tidal processes propagates into the estuary. During the open mouth condition following the flood event, a tidal cycle in the estuary occurs and as a result, substantial volumes of seawater move into the estuary. As expected, high salinity levels are present within the estuary during this time. The rise in salinity levels throughout the system is dependent on the duration of the open mouth. Once the estuary mouth closes, salinity levels gradually decrease as the system becomes diluted with lower salinity water from the influent rivers (Fijen, 1995).

Propagation of seawater into the lakes is infrequent and is usually associated with spring tides and abnormally low barometric pressures (Whitfield, et al., 1983). Previous studies of the salt budget of the Wilderness Lakes suggested that significant dampening of tidal influence to the lakes was evident due to the narrowness of the interconnecting channels between the lakes.

Russel, in his paper in 2013 mentions that the Touw River estuary and Wilderness coastal lakes consist of a predominantly unstratified estuarine system (Russell, 2013) and no definite pattern of stratification has ever been recorded in the coastal lakes. However, according to Fijen, it is not inconceivable that these lakes could, at times, be stratified (Fijen, 1995).

In the past a reverse salinity gradient has existed in the lake system (Russell, 2013) where the average salinity of the lakes increased, the further removed from the sea. This was accepted to be caused by the absence of freshwater inflows to the upper lakes which would contribute to higher salinities frequently occurring in these water bodies (Russell, 2013). Table 3-4 describes the salinity situation of the lakes before 1995 which illustrates the reverse salinity gradient within the lakes.

Table 3-4: Salinity levels of the Wilderness coastal lakes before 1995 (Fijen, 1995)

	Island Lake	**Bolangvlei**	**Rondevlei**
Average Salinity (ppt)	6.2	10.6	13.9
Salinity Range (ppt)	4-10	8-13	12-16

3.9.1. Salinity modelling using a one dimensional approach

It is possible to model salinity characteristics of an estuary using a one-dimensional numerical model; however, severe limitations exist when using a one dimensional approach. It has been established that salt intrusion into estuaries is predominantly by way of a two layer estuarine circulation. This cannot be modelled using a vertically integrated system as in a one-dimensional modelling approach (Hsu, et al., 1999) and would require a three-dimensional model. Although previous studies suggest that the water system is predominantly unstratified, there still exists the possibility for salinity stratification in the water system which is unable to be numerically modelled using a one dimensional approach.

Furthermore, often rivers and estuaries are elongated such that lateral variation is relatively insignificant. In cases when the ratio between river width to length is fairly small, laterally homogeneous salinity conditions are likely to appear. However, in the case of the Wilderness estuary and coastal lakes system, significant lateral variability exists due to the presence of the coastal lakes and narrow interconnecting channels. The geometry of the system is unlike that of a conventional estuary and it is unlikely that homogenous salinity conditions are present across the width of the lakes. A one dimensional model is incapable of simulating lateral variation and at least a two-dimensional modelling tool would be necessary in order to incorporate lateral salinity variation.

A one dimensional modelling approach will, therefore, provide a first indication of the salinity levels along the longitudinal direction of the water course. Resulting salinity levels will also include depth averaged salinity levels over the entire water column and will not account for any stratification in the water column. Upstream of the Serpentine River, in the coastal lakes, is likely to show the largest degree of error in a one dimensional model.

3.10. Summary and Conclusions

A review of existing literature and previous studies on the Wilderness estuary and coastal lakes system has indicated that a periodically open/closed estuary, such as the case in Wilderness plays a critical role in the hydrodynamics of the estuary and connecting water bodies. It has also been shown that the formation of the sand bar at the estuary mouth is complicated and involves the combination of a number of processes. An understanding of artificial sand bar manipulation was also achieved, particularly involving manipulation measures such as sand bar skimming and preparatory channel construction.

In the past, property had been constructed along the flood plains of the Touw estuary and in low lying areas around the coastal lakes. In an attempt to limit water levels within the area and hence reduce inundation of residential property, various studies were performed on this water system from both a hydrodynamic and ecological point of view. The past and present management policy is to maintain the sand bar at the Touw estuary mouth, during closed mouth conditions, at an elevation of between +2.1m to +2.4m MSL, based on proposals made by the CSIR in 1981. However, in the recent past, multiple flood events in 2003, 2006 and 2007, resulted in inundation of residential property. Pressure from the public was directed at SANParks and questions were raised by the public regarding the value of the current management policy.

An assessment of the current management policy is, therefore, required in order to re-evaluate flood water levels using a modern numerical modelling simulation tool. Additionally, revised recommendations are possible with an updated numerical modelling study.

4. METHODOLOGY

4.1. Aims of the Numerical Model

The aim of the model was to perform a study in order to investigate the hydrodynamics of the Touw River estuary and Wilderness coastal lakes. Flood water levels, under different hypothetical scenarios, were required as an output from the hydrodynamic model. Furthermore; the model was also required to provide a first indication of the salinity trends within the system. Long term salinity variations under a number of different situations were required as an output from the numerical model.

4.2. Model Selection

4.2.1. Mike 11

The numerical modelling software used for this study was a model suite developed by the Danish Hydraulics Institute, namely “MIKE by DHI”. MIKE by DHI includes, amongst others, a one-dimensional suite called MIKE 11 which is ideal for simulating longer stretches of water and is synonymous with top quality river modelling. Given that the Touw River estuary and coastal lakes system predominantly consists of a river system with connecting lakes, it would be appropriate to use a one-dimensional modelling approach.

Specific application areas in MIKE 11 include:

- Flood analysis and flood alleviation design studies
- Real time flood forecasting
- Dam break analysis
- Optimisation of reservoir and canal gate / structure operations
- Ecological and water quality assessments in rivers and wetlands
- Sediment transport and river morphology studies
- Salinity intrusion in rivers and estuaries
- Wetland restoration studies

(DHI, 2011).

The application of flood analysis, flood alleviation and salinity intrusion into rivers and estuaries is very relevant to this study and it had been ascertained that MIKE 11 by DHI is applicable to assess the impact on the hydrodynamics and long term salinity changes of the Wilderness estuary under different mouth management scenarios.

The only limitation of this numerical modelling approach to the Wilderness system is the detailed flow modelling of the coastal lakes. MIKE 11 uses a one dimensional approach, in other words, MIKE 11 assumes all flow characteristics to be varying in one direction only. Due to this fact the detailed flow modelling of a lake in one dimension will not provide an accurate answer with regards to the flow directions and detailed flow characteristics within the water body. Flow characteristics varying in one direction assume that width and depth dimensions are averaged across the entire field. This is a good assumption when modelling a case such as a river, but the modelling of lakes may be better suited to two or three dimensional modelling.

However, this investigation primarily focuses on the hydrodynamics (water levels) within the entire system. For these purposes, the flow patterns, within the lakes are irrelevant and an accurate representation of water levels across the system can be obtained as long as the correct volumes of water in the lakes are used in the MIKE 11 approach.

MIKE 11is also advantageous due to the advanced formulations of simulating flow through a variety of different hydraulic structures. Previous management plans, such as the introduction of a sluice gate in the Serpentine River as well as other proposals could be re-evaluated. Additionally, hydraulic structures such as an overtopping gate control structure can be used to simulate the sand bar as it scours during a flood event.

The MIKE 11 hydrodynamic module (HD) uses an implicit finite difference scheme for the computation of unsteady flows in rivers and estuaries. The scheme is adaptable and changes in time and space depending on the local flow conditions. Therefore, the module is capable of describing all types of flow conditions including supercritical and subcritical flow (DHI, 2011).

The mathematics behind the MIKE 11 modelling tool involves the use of the 1-D Saint Venant equations, namely the continuity equation and the momentum equation (Sleigh & Goodwill, 2000), which are formulated equations based on the conservation of mass and conservation of momentum. The mathematical theory behind these fundamental equations can be seen in Appendix A.

A one dimensional model requires certain input data in order to function correctly. The data requirements are described in the following section.

4.3. Data Requirements

The data necessary in order to complete the numerical modelling simulations, included cross sections along the water course, flow data at the upstream inflow boundaries, namely the Touw River, Duiwe River and Langvlei Spruit. Roughness coefficients were also required at each cross section for the main river channel as well as the flood plains on either side of the main channel. A tidal water level was required at the estuary mouth and, lastly, historical water levels of all water bodies and historic salinity measurements were also necessary for the modelling study.

The cross sections, flow data, tidal data and roughness coefficients were all necessary for the formulation of the model in the model set up. However, historical water levels and salinity data were required for the calibration and validation of the model.

4.4. Data Collection

This data was captured in one of two ways; historic datasets obtained from SANParks or on-site field investigations. Historical data was obtained from the SANParks Scientific Services Office at Rondevlei and permission was granted by SANParks to make use of this data. Data made available by SANParks for use in this study, included historical water level data for the estuary and three coastal lakes, as well as salinity measurements for the same water bodies. Historical records of the Touw River flow data were also available.

Site visits were also conducted in order to obtain necessary data related to the physical characteristics of the water system, this primarily included water level adjustment factors, cross sections and roughness coefficients. The methodology related to each aspect is described later in the chapter and detailed site visit reports can be seen in Appendix B.

4.4.1. Water level Data

Historical water level data for all water bodies was obtained from the SANParks Scientific Services Office at Rondevlei. However, water level recorders were not all referenced to Mean Sea Level or to the same datum during installation. (This shortcoming is discussed further in Chapter 5). A survey was, therefore, performed on all water level recorders in order to evaluate the adjustment factors necessary to convert all water level data to the same datum of MSL. Figure 4-1 shows the survey of the Rondevlei water level recorder. Surveys of known benchmarks in the area were also performed in order to relate surveyed data of water levels to a known benchmark.

Figure 4-1: Checking elevation of water level recorder at Rondevlei

4.4.2. Cross Sections

Cross sectional data was required for the input into the one-dimensional model and, therefore, cross sectional data for the Wilderness estuary, Serpentine River and coastal lake system were required.

The method used when conducting field research involved the use of two sets of apparatus, namely an echo sounder, used to establish the bottom profile of the water channel, and vertical GPS surveying equipment, used to establish the elevation of the flood plains on either side of the main water channel. The process involved paddling from one side of the water channel to the other side and recording water depths at two meter intervals. It must be taken into consideration that the echo sounder only records water depth and, therefore, does not provide a depth profile with a consistent datum. The method on how this data was transformed to a consistent datum of MSL is discussed in the following paragraph. Figure 4-2 shows the method by which the water channel was surveyed using the echo sounder attached to the kayak.

Figure 4-2: Water depth measurements being taken on a kayak

Once a full cross sectional profile of the water channel had been established with the echo sounder, the GPS surveying equipment was used to establish the cross sectional data of the flood plains on either side of the main channel. Points were surveyed from the water edge to a level above the flood plain on either side of the channel. The surveyed data also provided valuable information regarding the actual time-based water level of each cross section and made it possible for the merger between the water channel data and flood plain data, where all values could be referenced to the same datum, namely mean sea level (MSL). Figure 4-3 illustrates the method used to survey the flood plains.

Figure 4-3: GPS instrument being used to survey flood plain topography at Touw River

Unaccommodating terrain such as dense brush and reeds made it difficult to survey the flood plains for all cross sections. In this instance, one point was surveyed at the water edge on either side of the channel and a 1m contour elevation map, created by the CSIR in 1981 was then used to approximate flood plain levels on either side of the channel. This contour map can be seen in Appendix C.

Cross sections for the three coastal lakes were also estimated using the bathymetry data acquired from the CSIR (CSIR, 1981). Three cross sections were evaluated for each lake. It was felt that using three equally spaced cross sections along the course of the lake portrayed a realistic representation of these water bodies and also provided sufficient accuracy for the required outcomes in the one-dimensional modelling study. This contour map can be seen in Appendix C.

4.4.3. Roughness Coefficients

Roughness coefficients were estimated at each cross section interval along the water course and were based on a visual observation of the site at each cross section. The main channel as well as the flood plain was described according to the type of vegetation and bed characteristics at each cross section. These descriptions were then cross referenced with the descriptions of Manning n values in literature performed by Chow (Chow, 1959). Table 4-1 highlights the possible channel descriptions along with corresponding Manning n values.

Table 4-1: Manning n Values for various channel types (Chow, 1959)

Type of Channel and Description	Minimum	Normal	Maximum
1. Main Channels			
a. clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
b. same as above, but more stones and weeds	0.030	0.035	0.040
c. clean, winding, some pools and shoals	0.033	0.040	0.045
d. same as above, but some weeds and stones	0.035	0.045	0.050
e. same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. same as "d" with more stones	0.045	0.050	0.060
g. sluggish reaches, weedy, deep pools	0.050	0.070	0.080
h. very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.100	0.150
3. Floodplains			
a. Pasture, no brush			
1.short grass	0.025	0.030	0.035
2. high grass	0.030	0.035	0.050
b. Cultivated areas			
1. no crop	0.020	0.030	0.040
2. mature row crops	0.025	0.035	0.045
3. mature field crops	0.030	0.040	0.050
c. Brush			
1. scattered brush, heavy weeds	0.035	0.050	0.070
2. light brush and trees, in winter	0.035	0.050	0.060
3. light brush and trees, in summer	0.040	0.060	0.080
4. medium to dense brush, in winter	0.045	0.070	0.110
5. medium to dense brush, in summer	0.070	0.100	0.160

Manning n values that were used in the set up of the model are discussed in Chapter 6.3.3.

5. EVALUATION OF PROTOTYPE DATA

5.1. Introduction

During the evaluation of the prototype data it was also discovered that certain shortcomings were evident in the data sets. Firstly, the flow gauging station in the Touw River had a maximum recordable discharge of 66.2m^3/s. Many flood events in the Touw River have discharges far greater than this maximum recordable value. Secondly, there were no flow gauging stations in place in order to capture inflow data for the Duiwe River and Langvlei Spruit. Lastly, the water level data sets for the different water bodies were not referenced to mean sea level, nor were they referenced to the same datum. This chapter discusses the limitations of these data sets and also mentions the procedures that were taken in order to overcome any shortcomings.

5.2. River Inflow Data

Three river inflow points occur along the length of the water course which collectively form the dominant freshwater influent to the system. The Touw River, which flows directly into the Touw estuary, the Duiwe River which flows into Island Lake as well as a small river that flows directly into Bolangvlei, namely Langvlei Spruit. The river inflow points along the water course can be seen in Figure 5-1 along with their respective catchments.

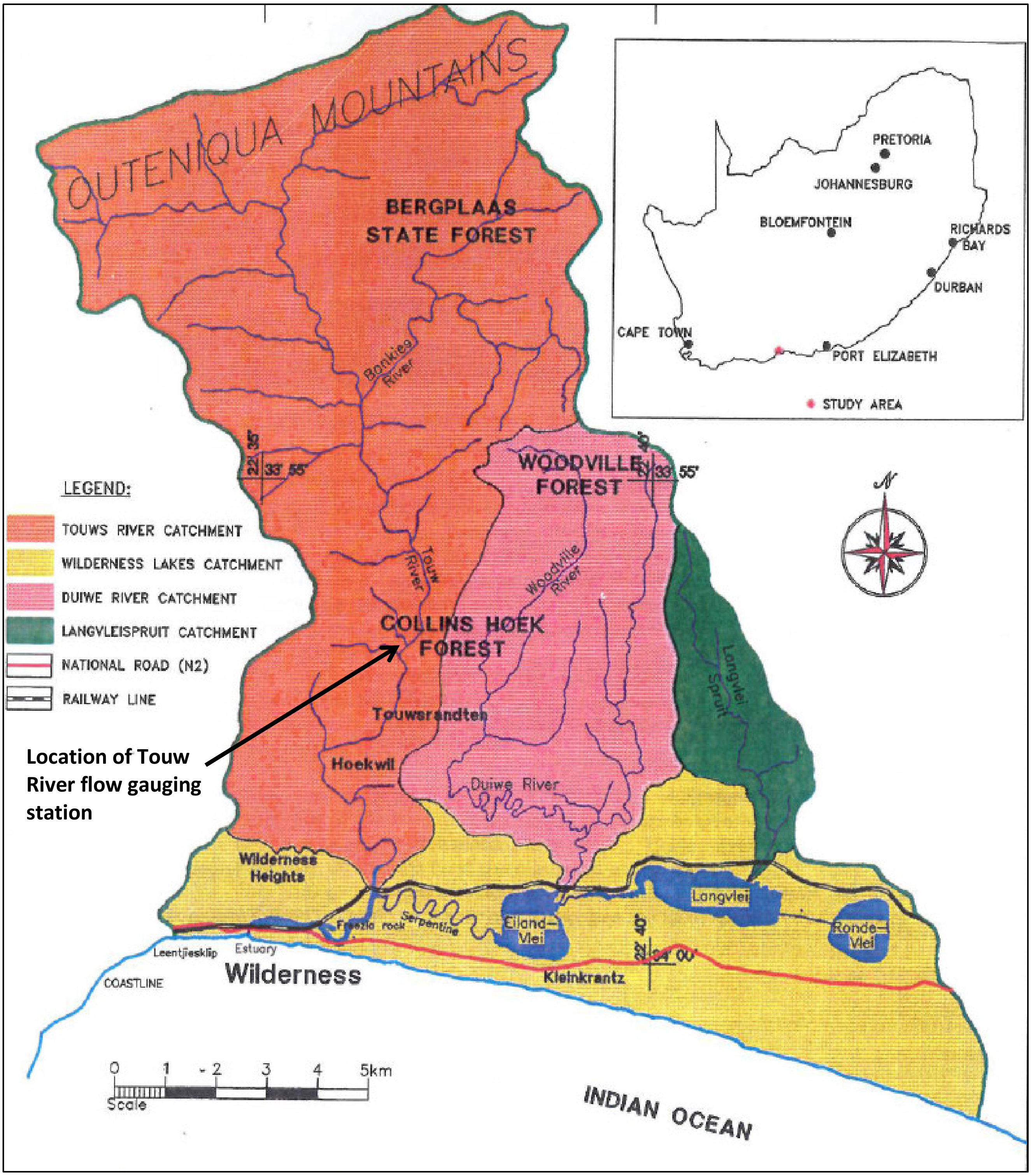


Figure 5-1: Catchment areas of respective inflow sources for the Wilderness estuary and coastal lakes system (Fijen, 1995)

The dominant source of river inflow into the system was provided by the Touw River. A flow gauging station exists in the Touw River; station K3H005 (Department of Water Affairs, n.d.), and the location of this station can be seen in Figure 5-1. It makes use of a weir gauging station, where the resulting discharge can be calculated given the height of water above the crest of the weir (water depth). A photograph of the gauging station is illustrated in Figure 5-2.

Figure 5-2: Position of flow gauging station on Touw River

Data from this station was used for the Touw inflow boundary condition; however, this station has a maximum recordable discharge of 66.2m^3/s, which corresponds to a water depth of 2.0m, measured from the crest of the weir. Nevertheless, the water depth had no limiting factor and all correct water depths were recorded. This can be seen in the following Q-h curve (Discharge-water height curve) which was constructed based on the information from the rating table for station K3H005 obtained from the Department of Water Affairs (Department of Water Affairs, n.d.).

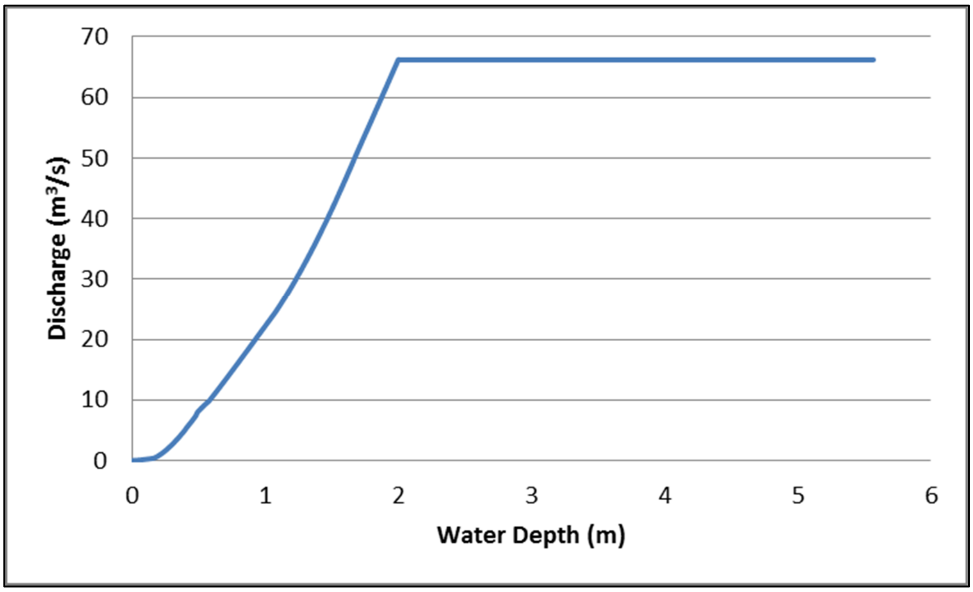


Figure 5-3: Q-h relationship of the Touw River gauging station (Department of Water Affairs, n.d.)

It can be seen in Figure 5-3 that the discharge reaches a peak recordable discharge of 66.2m^3/s at a 2.0m water depth and remains constant at this discharge for any additional water depth increment. This is obviously not realistic and, therefore, extrapolation of this curve was consequently necessary in order to determine the appropriate discharges for water heights greater than 2.0m. Initially a second degree polynomial trend line was constructed on the known data for water depths between 0.0m and 2.0m and an equation of this trend line was determined. This equation was then applied to all water depth values greater than 2.0m in order to calculate the corresponding discharges. This method is shown in Figure 5-4 to 5-5.

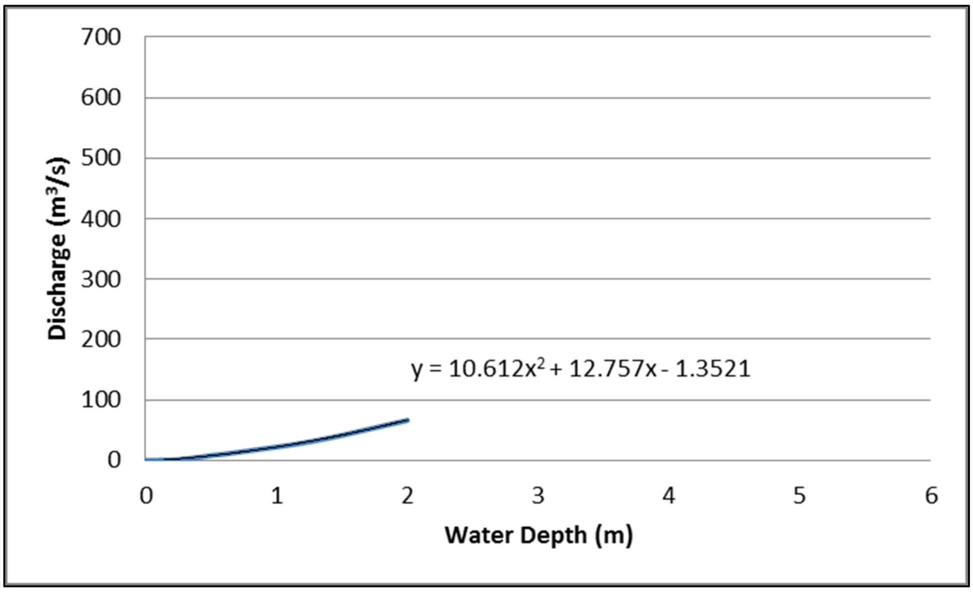


Figure 5-4: Establishment of trend line equation for the Touw River gauging station

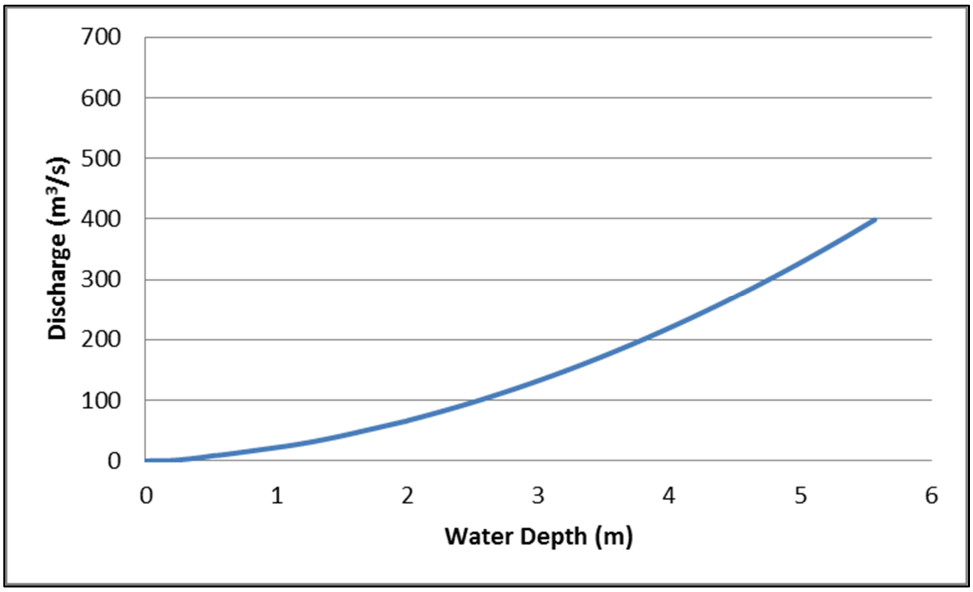


Figure 5-5: Estimated Q-h relationship for Touw River gauging station using trend line equation

However, later in the study, after preliminary runs of the model, it was found that higher inflows did not provide sufficient discharge and the resulting water levels achieved in the model did not correlate to the prototype measured water levels. It was, therefore, decided to alter the Q-h relationship of the Touw River gauging station, where water depths greater than 2.0m correspond to greater discharges than those predicted in Figure 5-5. It is likely that the cross section of the gauging station does not remain uniform, especially when approaching the flood plains and, therefore, the Q-h relationship does not remain constant for all water depths. Much higher discharges are likely to be associated with higher water depths where water flows in the flood plains. This was unable to be verified as the cross section of the weir site was unknown and was unable to be surveyed due to the fact that the gauging station is located on private property.

The final calibrated result of the Q-h relationship for the Touw River gauging station can be seen in Figure 5-6, which is best approximated by the polynomial trend line that has also been included. Floods with peak discharges greater than 66.2m^3/s were, therefore, approximated using this Q-h relationship. Although this is a subjective approach, this was the only data available for use in this study. A full river survey is however needed in order for a more accurate extrapolation techniques.

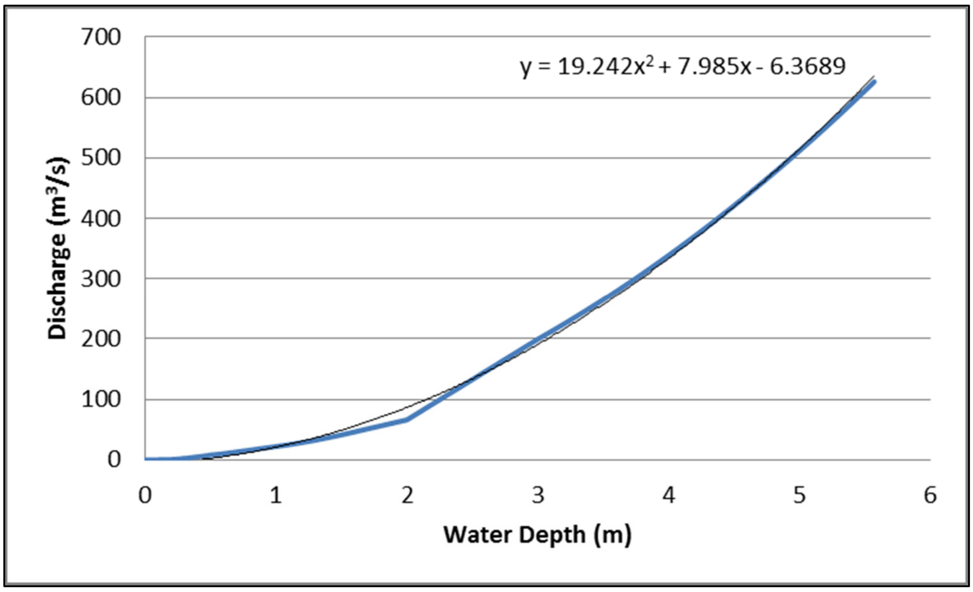


Figure 5-6: Final estimated Q-h relationship for Touw River gauging station

5.3. Duiwe River and Langvlei Spruit Inflow

The data from station K3H005 (Touw River gauging station) was used for the Touw inflow boundary condition; however no flow data existed for the Duiwe River or for the Langvlei Spruit. It was, therefore, decided to use a percentage of the Touw River inflow and apply this data to the Island Lake and Bolangvlei inflow points. The percentage used was partially based on the variables shown in Table 5-1 as well as flood hydrographs constructed by A.H.M Görgens in 1979 and 1994.

Table 5-1: Catchment Characteristics of different catchment in Wilderness (CSIR, 1981)

	Catchment		
	Touw River	Duiwe River	Langvlei Spruit
Area (km^2)	103	34	9
Mean Annual Precipitation (mm)	915	910	900
Mean Annual Runoff (m^3)	$22.6x10^6$	$6.0x10^6$	$1.5x10^6$
Longest Water Course (km)	33	17	9
Average Slope (%)	2.9	4.2	3.9
Basin Lag Time (hrs)	12	6	4
Critical Storm Duration (hrs)	4	2	1.25

Based on the data in Table 5-1, flood hydrographs were constructed by A.H.M. Görgens in 1979 and recalculated in 1994. These hydrographs were used to establish flow characteristics of the Duiwe River and Langvlei Spruit; specifically the average attenuation in flows for the Duiwe River and Langvlei Spruit compared to the Touw River discharge and the lag time between different inflow sources under flooding events. Although these hydrographs may be slightly outdated, it was beyond the scope of the study to recalculate flood hydrographs based on a hydrological study of the area. Figure 5-7 shows a 5 year flood hydrograph of the Touw River, Duiwe River and Langvlei Spruit according to Görgens' 1979 predictions and highlights the lag time of each flood peak.

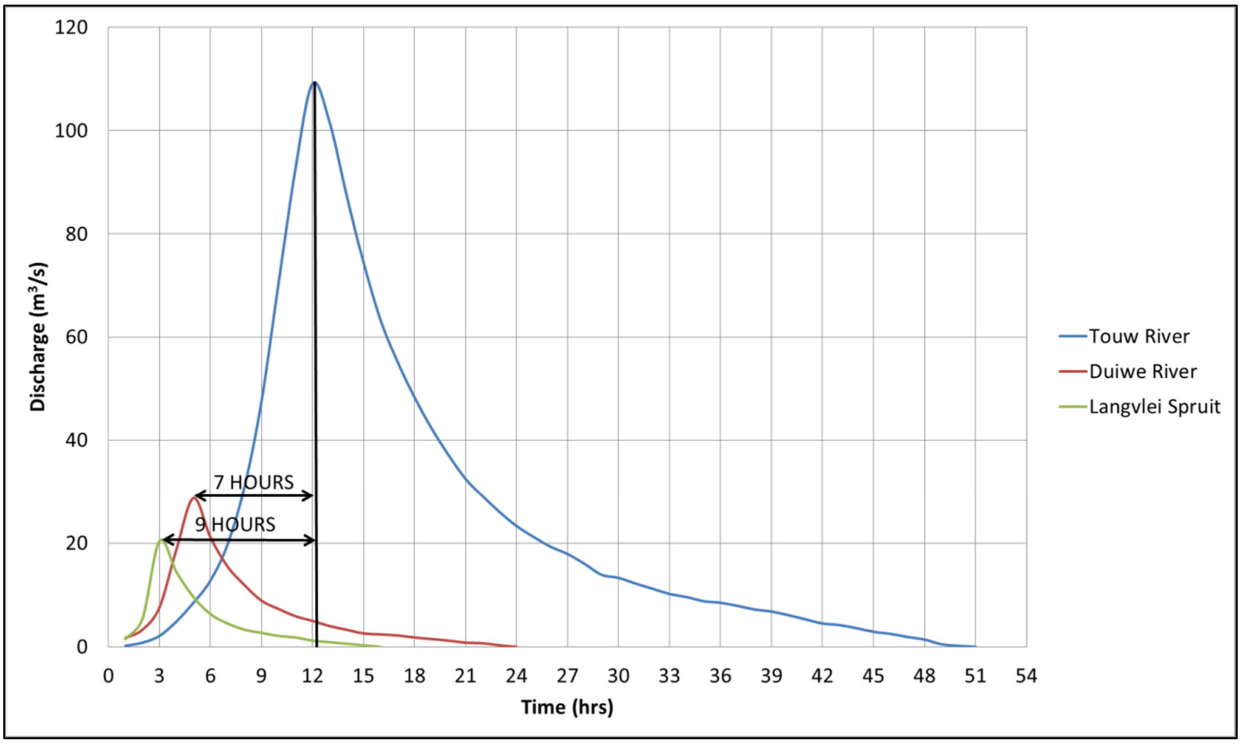


Figure 5-7: 5 year flood hydrograph prediction illustrating lag between flood peaks (Görgens, 1979)

Although a 5 year flood hydrograph is illustrated in Figure 5-7, similar relationships can be seen across all the predicted flood events for the 1979 study. From the previous figure it is evident that a lag time of 9 hours is evident between the Touw River peak discharge and the Langvlei Spruit peak discharge. A 7 hour lag time is evident between the Touw River peak discharge and the Duiwe River peak discharge. The total duration of a flood event is 51 hours, 26 hours and 16 hours for the Touw River, Duiwe River and and Langvlei Spruit respectively. Therefore, the durations of flood event in the Duiwe River and Langvlei Spruit are approximately 51% and 31% of the flood duration in the Touw River respectively. Regarding the actual peak discharge from each source, it can also be established that the peak flow in the Langvlei Spruit and Duiwe River is approximately 19% and 26% of the Touw River inflow respectively.

The 1994 flood hydrograph predictions show a very different result. Figure 5-8 illustrates the 1994 flood hydrograph prediction for a 5 year flood.

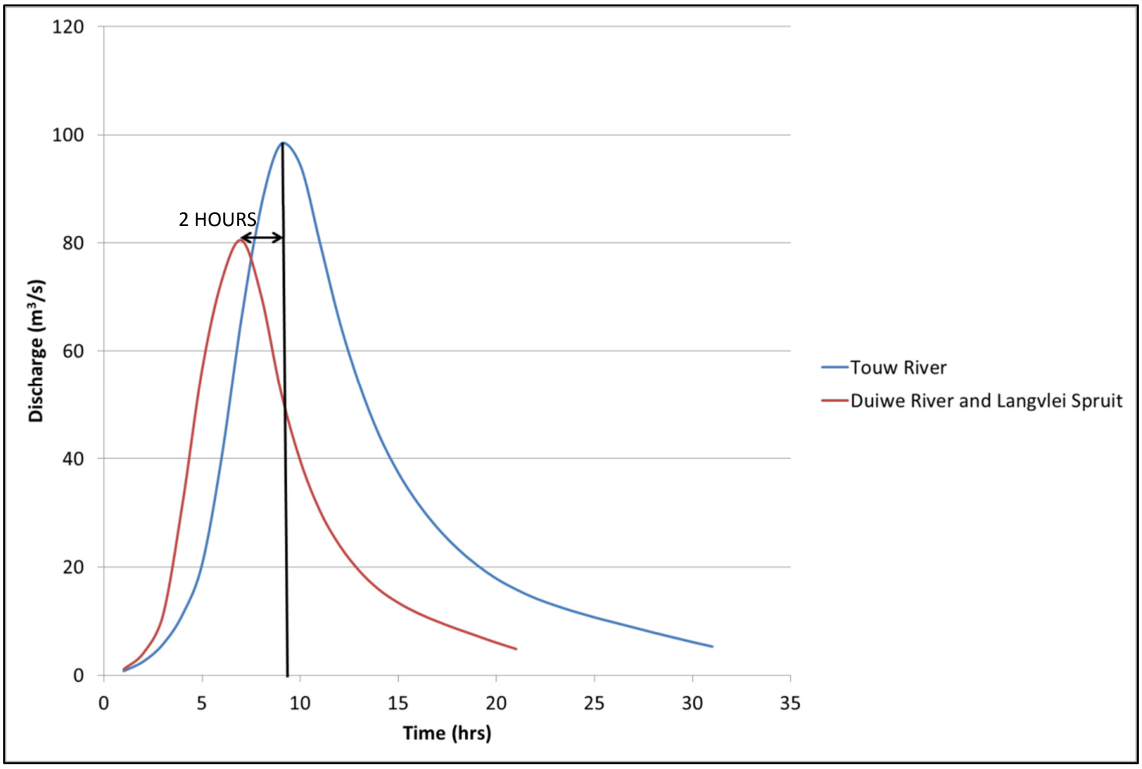


Figure 5-8: 5 year flood hydrograph prediction illustrating lag between flood peaks (Görgens, 1994)

The 1994, 5 year flood prediction illustrated in Figure 5.8, provides a good representation of the general trends evident throughout all the flood hydrographs in the 1994 study. The major distinguishing difference between the 1979 and 1994 predictions is that the later predictions predict a significantly larger flood in the Duiwe River and Langvlei Spruit compared to the 1979 hydrograph predictions. The first thing worth noting in the 1994 predictions is that the Duiwe River and Langvlei Spruit have exactly the same hydrograph and a lag time of 2 hours is evident between the Touw River peak discharge and the peak discharge of both the Duiwe River and Langvlei Spruit peak. These lag times of 2 hours for both the Duiwe River and Langvlei Spruit are a lot lower than the lag times estimated during the 1979 prediction, where lag times of 7 hours and 9 hours were predicted for the Duiwe River and Langvlei Spruit respectively. The total duration of a flood event is 31 hours, 21 hours and 21 hours for the

Touw River, Duiwe River and Langvlei Spruit respectively. The Touw River and Duiwe River have, therefore, seen a reduction in flood duration while the Langvlei Spruit has seen an increase in flood duration from the 1979 estimates. With regards to the relative flood durations compared to the Touw River, both the flood duration of the Duiwe River and Langvlei Spruit are approximately 68% of the flood duration of the Touw River. Regarding the actual peak discharge from each source, it is established that the peak flow in the Langvlei Spruit and Duiwe River is approximately 75% of the Touw River inflow. This is a massive increase from the 1979 estimates where 26% and 19% of the Touw inflow were predicted for the Duiwe River and Langvlei Spruit respectively.

It can, therefore, be established that the predictions between the 1979 and 1994 estimates are very different and provide a difficult task of estimating the correct values to use for the attenuation and lag time of the Duiwe River and Langvlei Spruit. One might simply say that the latest predictions (i.e. the 1994 predictions) provide a better and more accurate result. However, both studies are outdated and do not incorporate the large flood events which have been recorded in recent years. Furthermore, during the calibration of the model, it was found that the relationships between the Touw River and other inflow sources from the 1979 hydrograph predictions provide a more accurate result for recent flood events.

After the model was established, a sensitivity analysis was performed on both of the hydrograph predictions. Historic water level data, which was obtained for all water bodies in Wilderness, was compared using the findings of both flood hydrograph predictions. A real flood event was simulated, first using findings based on the 1979 predictions. This simulation was then repeated using the findings based on the 1994 predictions. Figure 5-9 and Figure 5-10 illustrate the result of both simulations.

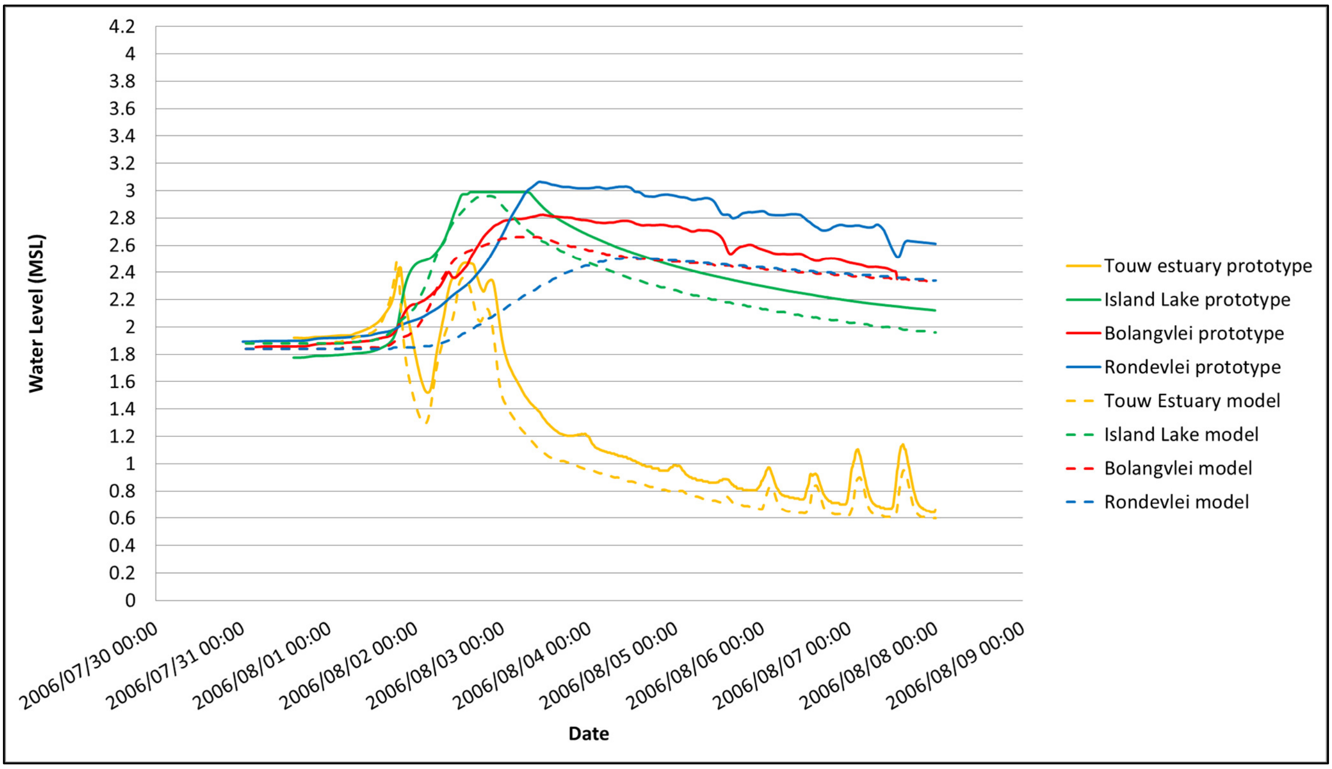


Figure 5-9: Comparison between model predicted and prototype water levels for all water bodies during a sample flood, using the findings based on the 1979 hydrograph predictions

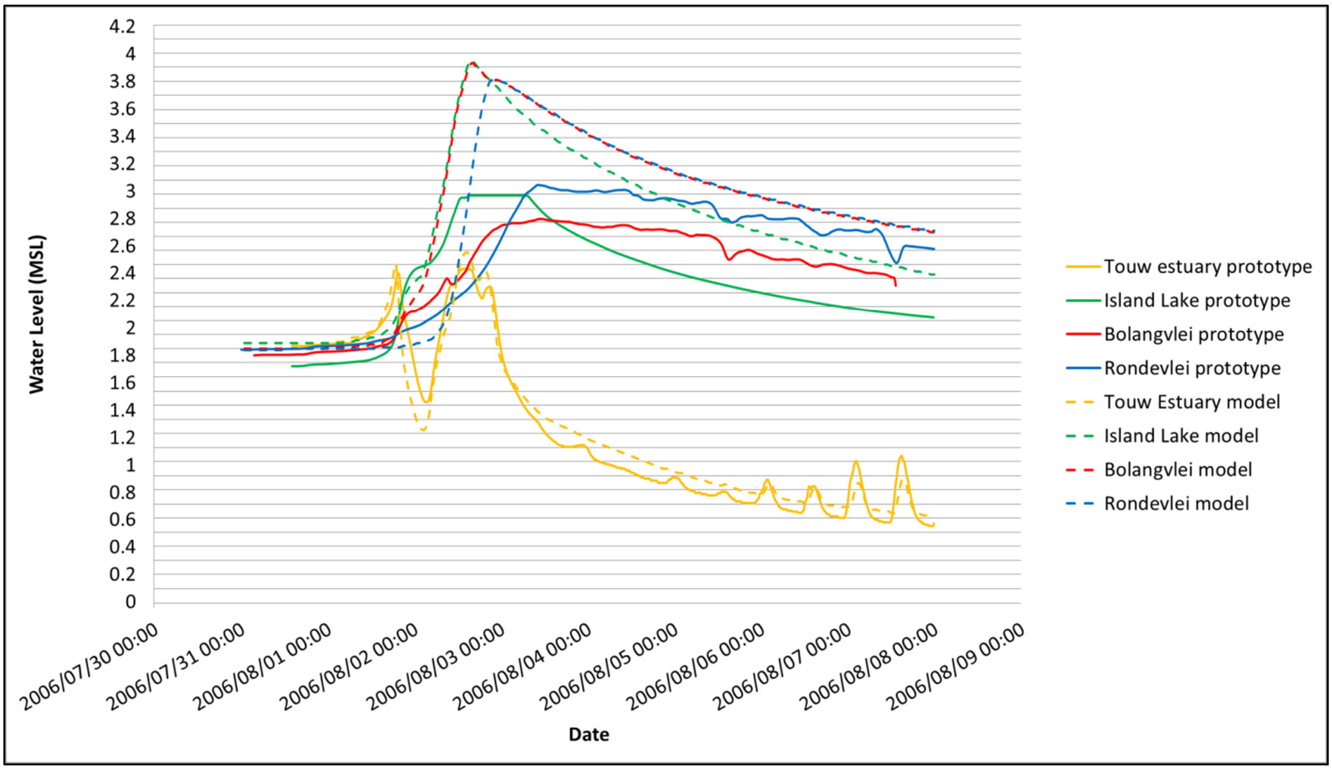


Figure 5-10: Comparison between model predicted and prototype water levels for all water bodies during a sample flood, using the findings based on the 1994 hydrograph predictions

Each graph shows the actual water levels recorded during the flood event for each water body as well as the corresponding model predicted water levels for the flood event. From a comparison between the two graphs, it can be seen that the findings based on the 1979 hydrograph predictions provide a far more realistic approximation of the prototype peak water levels during a flood compared to the findings of the hydrograph predictions made in 1994. It was found that the 1994 hydrograph predictions over-predict the discharge of the Duiwe River and Langvlei Spruit which, in turn, result in higher peak water levels in the lakes.

It was, therefore, decided to construct hydrographs for the Duiwe River and Langvlei Spruit based primarily on the findings of the 1979 flood hydrograph study. Although a more recent study exists, the findings based on the 1979 study provide a more accurate representation of reality with regards to the resulting water levels achieved during a flood event.

In summary, a lag time of 9 hours was used between the Touw River peak discharge and the Langvlei Spruit peak discharge. A lag time of 7 hours was used between the Touw River peak discharge and the Duiwe River peak discharge. The duration of flood event in the Duiwe River and Langvlei Spruit are approximately 51% and 31% of the flood duration in the Touw River respectively. With regards to the peak discharges of the Duiwe River and Langvlei Spruit, a percentage of the Touw inflow was used. This percentage is equal to 19% for the Langvlei Spruit and 40% for the Duiwe River. Note that this percentage value has increased from 26%, which is the percentage value that Görgens used in the 1979 predictions, to 40%. It was found that insufficient flow was evident when using 26% of the Touw River inflow for the Duiwe River inflow source and, therefore, this percentage value was increased slightly.

The values mentioned above are based purely on annual averages; however, it must be taken into account that substantial variability may exist in the rainfall distribution and intensity between the different catchments for isolated storm events. In a study by Hughes and Wright (1988) it was found that for individual storms in the Wilderness area, the profiles (i.e. distributions) vary over a considerable range from storm to storm. (Hughes & Wright, 1988). Görgens also mentions the uncertainty related to mean annual rainfall estimates for catchments in this region. This is largely contributed by localised rain-shadow areas within both the foothills and mountains of the southern Cape combined with large mountainous areas of inaccessibility (Görgens, 1994).

5.4. Water level Data

During the analysis of the water level data it was found that the water levels for different water bodies did not correlate to the same datum. It was later found that the water level recorders in each water body had not been installed to the same datum and adjustment factors exist in order to correct all water level data to the same datum of mean sea level (MSL). The adjustment factors were unknown for the present day water level recorders; however, the adjustment factors were known for an extended period before the year 2001. The historical adjustment factors, prior to 2001, for each water level recorder are shown in Figure 5-11.

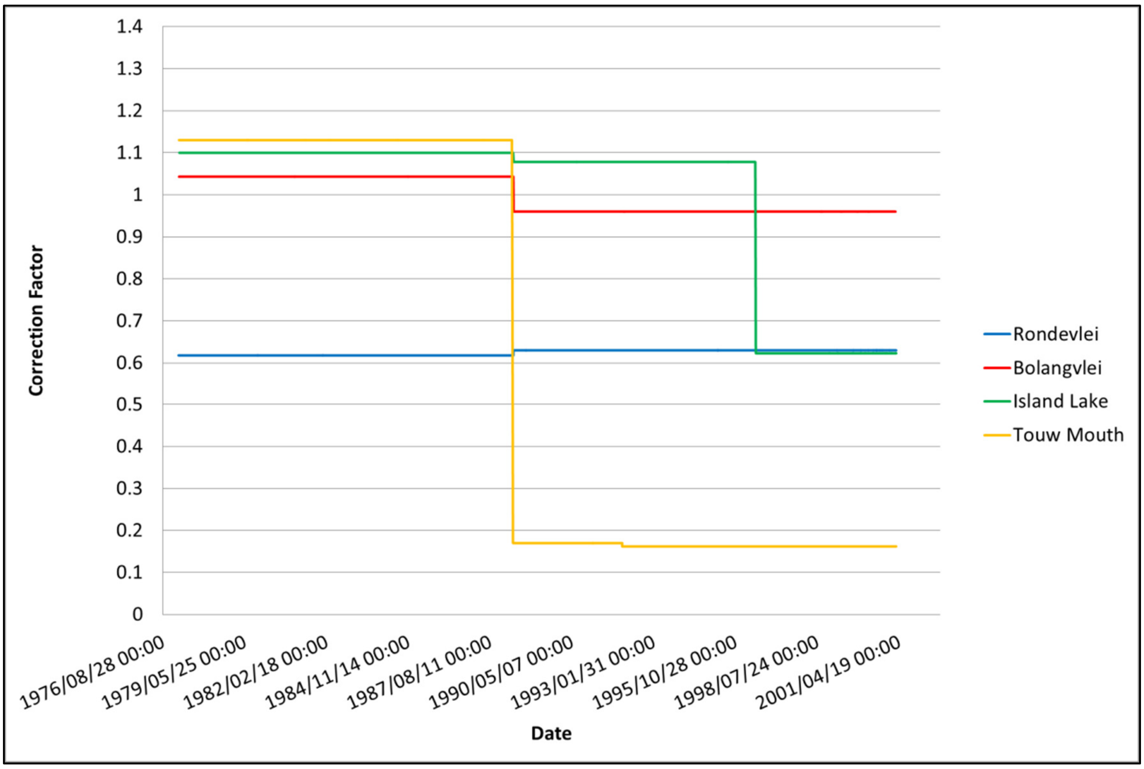


Figure 5-11: Historical record of MSL correction factors for all water bodies

The most recent record of adjustment factors for the water level records is from the 10 January 2001. The values for the most recent adjustment factors are shown in Table 5-2.

Table 5-2: Most recent MSL adjustment factors for water level recorders

Water Body	Adjustment Factor
Rondevlei	+0.63m
Bolangvlei	+0.96m
Island Lake	+0.623m
Touw Estuary	+0.162m

This implies that a water level recording in Rondevlei of 1.0m is, in fact, equal to a level of 1.63m MSL due to the adjustment factor of +0.63m

Vertical elevation surveys were also performed on each water level recorder in order to verify that the latest adjustment factors were accurate. Due to the large vertical tolerance of the available GPS surveying equipment, the survey was performed primarily as a reassurance that the latest adjustment factors were still accurate. It was confirmed by the manufacturers of the equipment that the maximum vertical tolerance of the GPS surveying equipment is 0.5m. Multiple surveys for each water level recorder were also performed in order to improve accuracy. The zero mark of each water level recorder was recorded in order to establish the correction factor to mean sea level. All water level recorders except the recorder at Bolangvlei were surveyed. The recorder at Bolangvlei was inaccessible during the time of the survey. Table 5-3 shows the results of the survey, where an estimated correction factor to mean sea level for each water body is given.

Table 5-3: Surveyed MSL adjustment factors for water level recorders

Water Body	Adjustment Factor
Rondevlei	+0.52m
Bolangvlei	Inaccessible
Island Lake	+0.80m
Touw Estuary	+0.14m

It was found that the surveyed data corresponds reasonably well with the most recent adjustment factors for the water level recorders and, therefore, the latest adjustment factors were used for the prototype water level data in this study.

5.5. Survey Locations

Locations of surveys and elevation checks can be seen in Appendix D.

5.6. Salinity Data

5.6.1. Method of collection

Salinity measurements were performed by SANParks officials at Wilderness. Measurements for all water bodies were performed every month for the period January 1991 to October 1999. Thereafter, salinity measurements of all water bodies were recorded at three month intervals continuing until October 2012 (Russell, 2013). Measurements were performed at nine locations along the Touw estuary and at five different locations in each of the coastal lakes. The locations of water quality sample sites can be seen in Figure 5-12 (Russell, 2013).

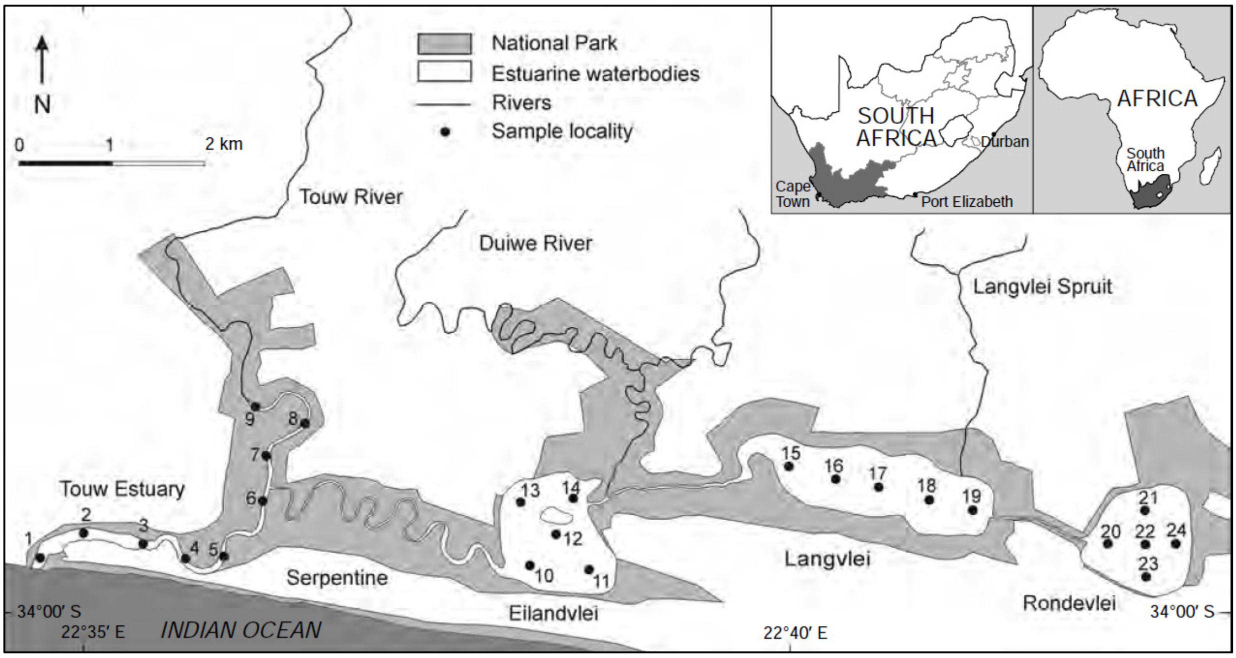


Figure 5-12: Locations of water quality sample site (Russell, 2013)

Furthermore, salinity measurements were only recorded near the surface at each location at a depth of 0.3m from the surface using YSI 33 (1991-2005) and Model 30 (2005-2010) S-C-T meters (Russell, 2013). This data is, therefore, not reliable to deduce any stratification characteristics within the water bodies. Previously, it has been mentioned that density differences between seawater and ambient estuarine water often result in a significant variance in salinity throughout the water column. Intrusion of salinity into an estuary is often by way of a single seawater layer which plunges below the ambient estuarine water and, therefore, stratification of salinity is unaccounted for, with a single reading at each locality.

5.6.2. Prototype Data

A figure illustrating the prototype data is shown below in Figure 5-13. The salinity for each water body was determined by calculating the average salinity between all locations for a specific water body. Although salinity variation in the Touw River, at times, varied substantially between the different testing sites, there existed very little salinity variation between different testing sites within each lake.

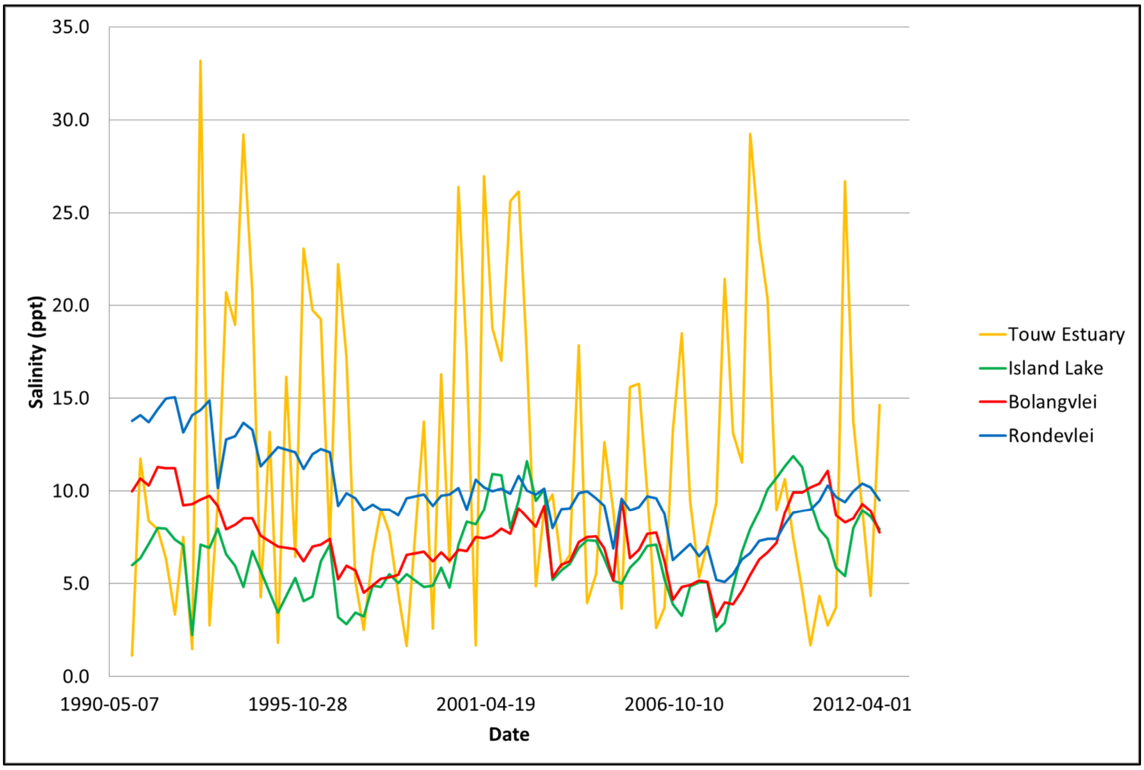


Figure 5-13: Long term salinity levels for all water bodies (data provided by Russel)

Salinity levels in the Touw estuary are extremely dependent on the condition of the sand bar at the estuary mouth and salinity levels fluctuate rapidly between open and closed mouth states. An open mouth results in high salinity levels in the estuary whereas low salinity levels are experienced during a closed mouth condition. Given that the interval between salinity recordings in the Touw estuary is three months and the average time that the mouth remains open is only 28 days (CSIR, 1981), the salinity data for the estuary is primarily dependent on the time based condition of the sand bar on the day of the recording. Salinity levels within the lakes are less likely to fluctuate over time due to their location in relation to the estuary mouth. A good understanding of long term trends in the lakes can, therefore, be established.

Russell (2013), mentions that significant declines in salinity have occurred in both Bolangvlei and Rondevlei while no significant trend was evident for Island Lake. Figure 5-14 illustrates the long term salinity trends in the Wilderness lakes which reiterate Russell's statement. A linear trend line for each lake is also included, thus illustrating the general long term salinity trends within each lake.

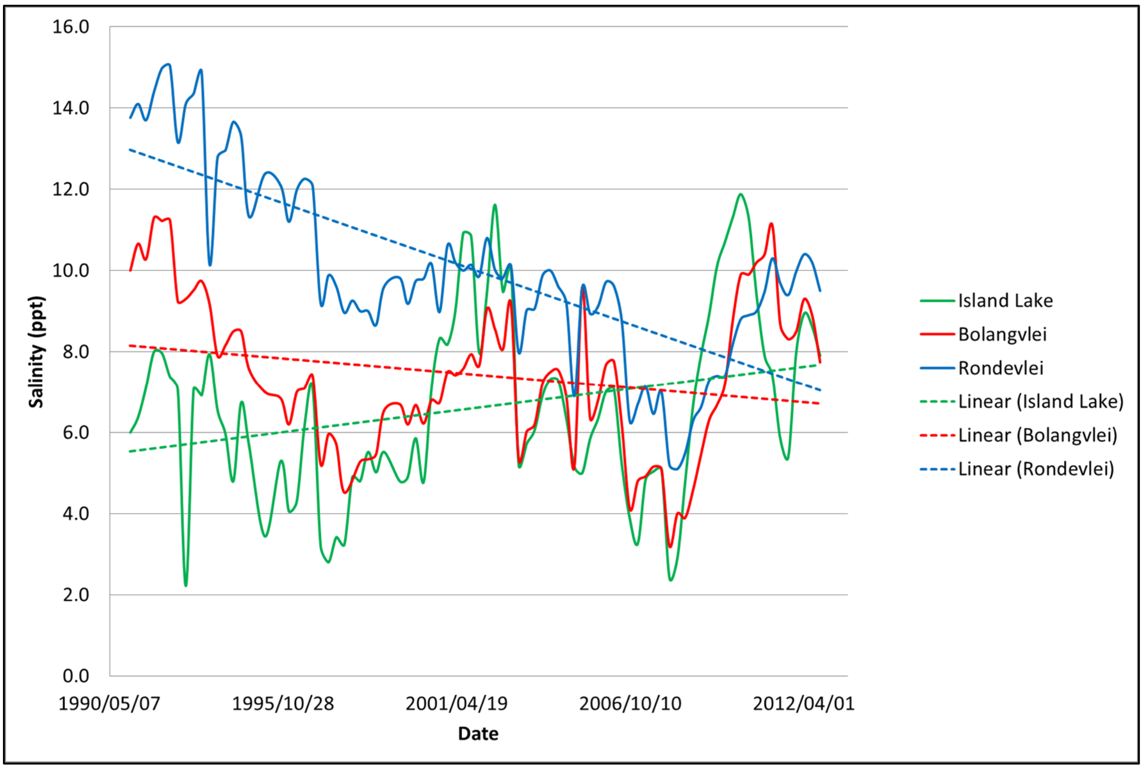


Figure 5-14: Observed long term salinity level for coastal lakes with trend lines (data from Russell)

The general long term salinity trends illustrate that salinity levels in Bolangvlei and Rondevlei are decreasing and although no significant trend is evident in Island Lake, the general long term trend shows increasing salinity levels for this water body. Furthermore, the salinity levels in Rondevlei are decreasing at a faster rate than those of Bolangvlei.

The current salinity levels within the lakes are very similar and the strong salinity gradient that once existed in the lakes is not as prominent and may even be regarded as non-existent. Since the beginning of the year 2000, salinity levels in the lakes have shown a different salinity situation compared to the situation before 1995 (as mentioned in the literature review). Table 5-4 illustrates the current salinity situation in the coastal lakes from the year 2000 to 2012 and also illustrates the difference between past and present salinity trends.

Table 5-4: Comparison of salinity characteristics of coastal lakes between the periods of 2000 to 2012 and before 1995 (Fijen, 1995) (data from Russel)

	Island Lake		**Bolangvlei**		**Rondevlei**	
	< 1995	**>2000**	**< 1995**	**>2000**	**< 1995**	**>2000**
Average Salinity (ppt)	6.2	7.3	10.6	7.2	13.9	8.8
Salinity Range (ppt)	4-10	2-12	8-13	3-11	12-16	5-11

It is evident from this comparison that average salinity levels in the upper lakes, namely Bolangvlei and Rondevlei have dropped significantly while a small increase in average salinity is evident in Island Lake. Furthermore, the range in salinity is far greater in the data after 2000. However, these large salinity ranges are largely a result of the massive flood events which occurred in 2006 and 2007.

These general salinity trends of the coastal lakes may be linked to the artificial management of the sand bar at the estuary mouth. As a result, the average duration that the estuary mouth remains open after breakthrough has decreased significantly from the natural conditions. A study performed by the CSIR confirms this (CSIR, 1981) and it was found that before the introduction of artificial sand bar manipulation, the mouth remained open on average for 50 days. This average then decreased to 28 days since the inception of artificial sand bar manipulation (CSIR, 1981). As a consequential effect, it appears that seawater intrusion into the entire system has also diminished. During longer periods of an open mouth condition, seawater is able to propagate to the upper reaches of the estuary as well as the upper reaches of the coastal lakes. However, now that the average "open mouth" duration has decreased, the propagation of seawater to the entire system is less prominent.

6. NUMERICAL MODEL SET UP AND MODEL TESTING

6.1. Introduction

The numerical modelling study included a Mike 11 one-dimensional model of the entire Wilderness coastal lakes system expanding from the upper most lake, Rondevlei, to a distance into the ocean at the Touw River mouth. The aim of the study was to evaluate the hydrodynamics of the Touw estuary and coastal lakes in Wilderness with attention being paid to the maximum water levels achieved in each water body during different flood events. Therefore, the numerical model was set up according to the conditions present during a flood event.

Given that site visits were performed during periods with good weather conditions, slight adjustments were taken into consideration when setting up the numerical model, to account for the increased debris load during a flood event. The adjustments include manually decreasing the surveyed cross sectional area in the Serpentine River and the interconnecting channels which simulate a resulting constriction caused by debris.

The model set up is divided into four core components which run simultaneously in the Simulation Editor of Mike 11. These components include the Network, Cross Sections, Boundary Conditions and Hydrodynamic Parameters.

6.2. Network

6.2.1. Water Course

The extent of the primary branch of the network ranges from the eastern reaches of Rondevlei to a distance into the ocean at the Touw mouth. The branch begins at the most eastern part of Rondevlei and follows the water course through the coastal lake system. The branch then follows the centreline of the Serpentine River until the Serpentine meets the Touw River. At this point, the branch follows the downstream course of the Touw River and continues past the Touw Mouth to a distance to the ocean. The length of the entire branch is 18350m. An additional branch is also included at the Touw/Serpentine River intersection, simulating the upper reaches of the Touw River. The entire network can be seen in Figure 6-1 below.

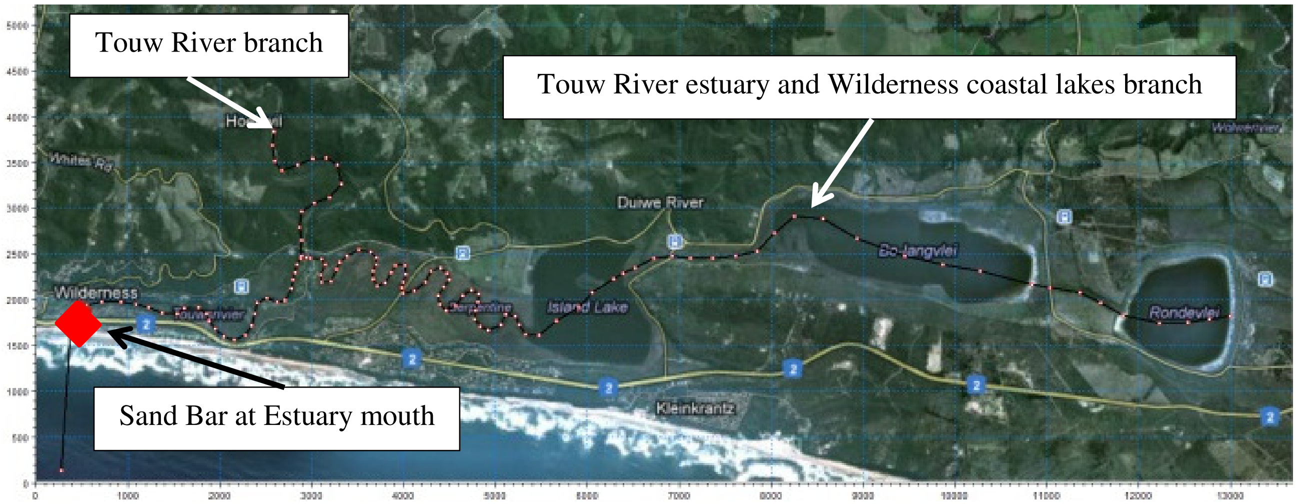


Figure 6-1: Numerical model network of Touw River estuary and Wilderness coastal lakes

6.3. Cross Sections

6.3.1. Introduction

Cross sections were described at certain intervals along the course of the network. In total, 39 cross sections were used along the entire course. This equates to an average cross section every 470m along the water course; however, attention must be paid to the fact that cross sections were placed at finer intervals in the Touw estuary as the bathymetry in this water body is more dynamic and less uniform. A figure showing the location of each cross section can be seen in Appendix E.

6.3.2. Distance and Time Intervals

The modelling term "maximum dx" is the maximum distance allowed by the calculation engine between two adjacent water level calculation points. At locations where cross sections are present, the calculation engine will always create water level points at these locations. If however, the maximum dx is defined with a value smaller than the distance between the cross sections, the calculation engine will automatically insert a number of additional calculation points, by interpolation, in between existing cross sections such that the minimum distance between water level calculation points in the calculation will be less than or equal to the defined value of maximum dx (DHI, 2011). In the case of this model, the maximum dx was 10000m which is greater than the greatest distance between two subsequent cross sections. Therefore, the dx used in the model is controlled by the distance between subsequent cross sections within the model. This ranges between 42.3m and 1784.1m. The time interval (dt) is the time interval used in the calculation of the model. A sufficient time interval was necessary to assist the calculation process and prevent model instability which can lead to premature model termination. The time step used in the hydrodynamic modelling study was 60 seconds. This time step provided both, an adequate time step to prevent modelling instability as well as a sufficient time step to assess the necessary changes in flood water levels.

6.3.3. Bed Level

The input data used for the cross sections consists of the data collected on site using the three collection methods, namely the echo sounder, land surveying equipment and bathymetry charts. The collection method is described in the Methodology chapter. Maximum depths for each water body were found to be -4.0m, -2.0m, -4.0m and -4.0m MSL in Rondevlei, Bolangvlei, Island Lake and the Touw estuary respectively. Figure 6-2 shows an example of one cross section near the old railway bridge in the Touw estuary. The bottom axis is the horizontal distance along the cross section from the left bank to the right bank. The left vertical axis shows the elevation of the bed level at each horizontal point along the cross section and the right vertical axis shows the Manning n value for different positions along the cross section. Individual cross sections for each section can be seen in Appendix E.

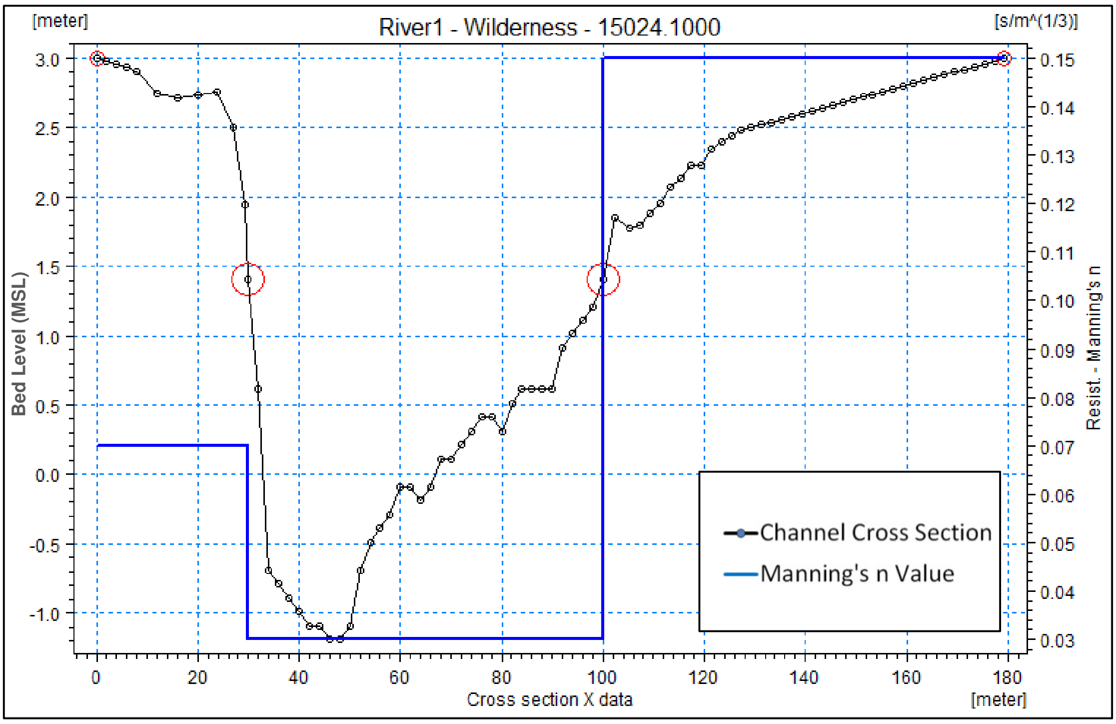


Figure 6-2: Example of cross section output in MIKE 11

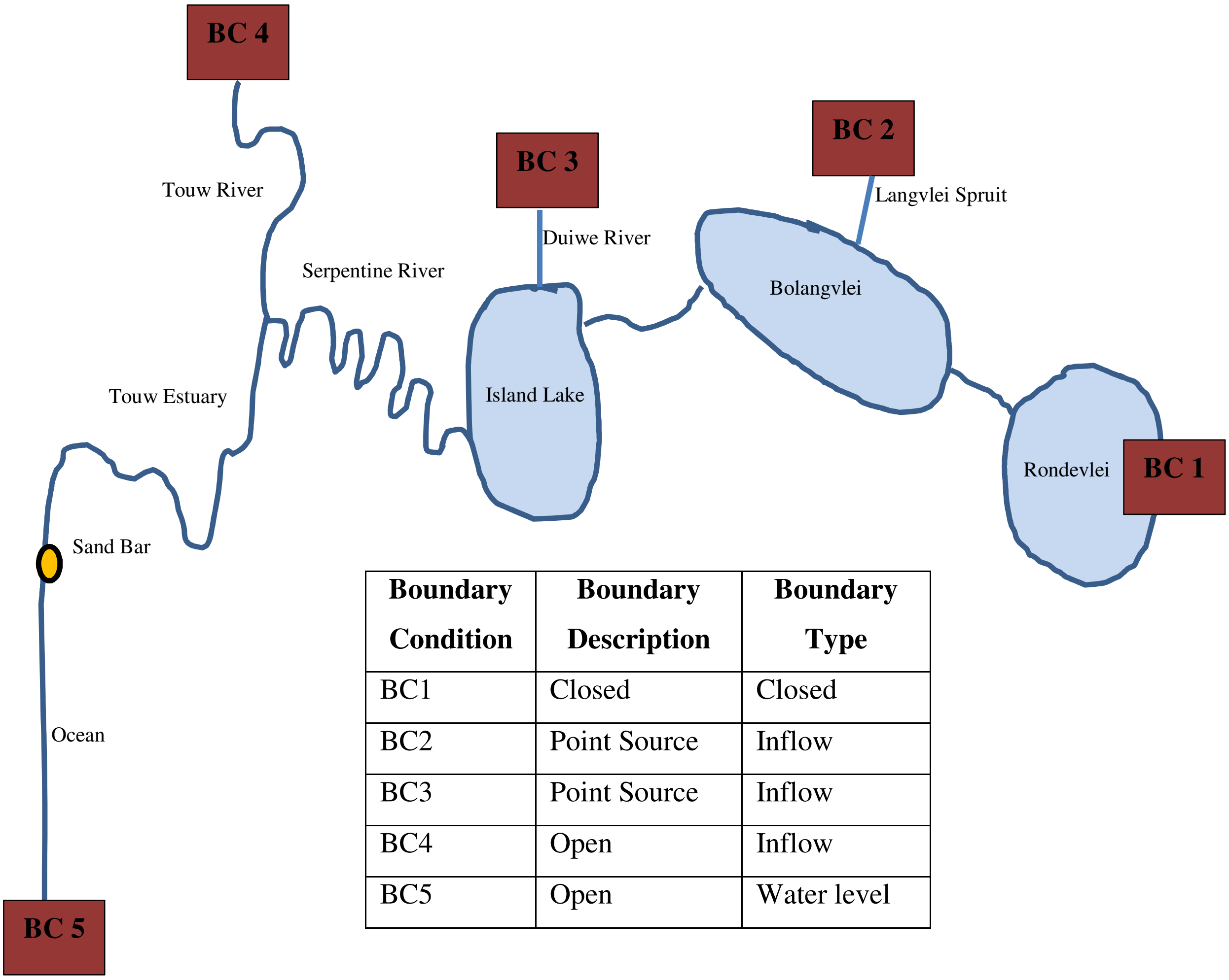
6.3.4. Roughness Coefficients

Roughness coefficients were initially based on a visual observation of the site at each cross section as described in the Methodology chapter. The preferred convention for bed roughness were Manning n values. The model was set up so that three roughness coefficients were described across the width of the cross section. At each cross section, a Manning n value for the main channel as well as the left and right flood plain was given. Using Figure 6-2 as an example, the Manning n values are represented across the width of the cross section by the blue line. It was observed that at this cross section, the left flood plain consisted of light scattered brush, the main channel was clean and the right flood plain consisted of tall, dense brush. According to (Chow, 1959), this equates to a Manning n value of 0.07, 0.03 and 0.15 respectively. It must be noted, however, that this only provided a first indication to Manning n values used throughout the model. Manning n values were further refined during the calibration process. Table 4-1 highlights possible channel descriptions along with corresponding Manning n values and Manning n values for all cross sections can be seen in Appendix E.

6.4. Boundary Conditions

6.4.1. Introduction

Different boundary conditions are present in the numerical model of the Touw River estuary and Wilderness coastal lakes. Figure 6-3 illustrates the locations of various boundaries within the numerical model and the following table describes the type of boundary. There are five boundary conditions used throughout the model. A closed boundary on the eastern side of Rondevlei, an open inflow boundary used in the Touw River branch, point source inflow boundaries used for inflow sources to Island Lake and Bolangvlei and an open water level boundary in the ocean.



Boundary Condition	Boundary Description	Boundary Type
BC1	Closed	Closed
BC2	Point Source	Inflow
BC3	Point Source	Inflow
BC4	Open	Inflow
BC5	Open	Water level

Figure 6-3: Description and location of boundary conditions used in numerical model

6.4.2. River Inflow

The technique used for the calculation of river inflows from different sources is discussed in detail in Chapter 5. In summary, the flow gauging station on the Touw River had a maximum recordable peak discharge of 66.2m^3/s which corresponds to a 2.0m water depth at the gauging station. Therefore, an interpolation technique was performed in order to obtain discharges for water depths greater than 2.0m. This technique is explained in detail in Chapter 5.2. Furthermore, inflow data did not exist for the Duiwe River and Langvlei Spruit. Consequently, relationships were established between the Touw River inflow and the other inflow sources. A summary of these relationships, as used in the model, is depicted in Table 6-1 and Figure 6-4. Refer to Chapter 5.3 for a detailed explanation behind these values.

Table 6-1: Relationship between Touw River flood hydrograph and other inflow sources

	Touw River	**Duiwe River**	**Langvlei Spruit**
Flood Duration	100%	51%	31%
Lag time of peak discharge	0 hours	-7 hours	-9 hours
Peak Discharge	100%	40%	19%

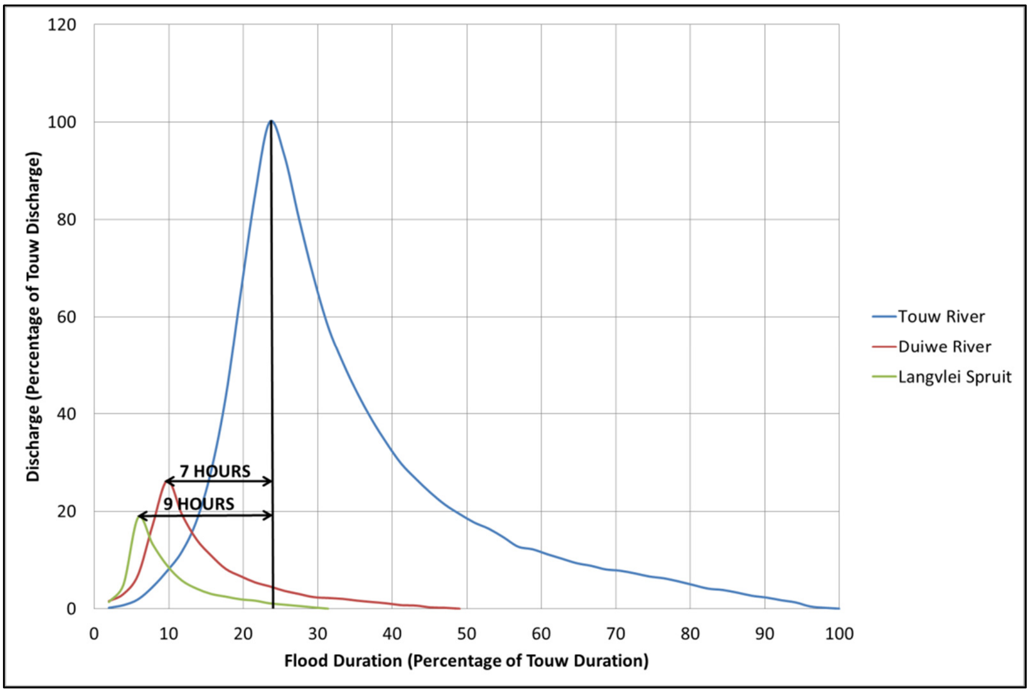


Figure 6-4: Relationship between Touw River flood hydrograph and other inflow sources

6.4.3. Tidal Water level

A tidal water level boundary existed at the ocean boundary in the network and the tidal prediction simulator in Mike 21 was used in order to create a time series of the predicted tide at this boundary. Tidal elevation is calculated from a number of tidal constituents, which are different according to the specific geographical location. Each constituent contains an amplitude and a phase angle and when combined, the result is a sinusoidal tidal cycle at the specified location. The closest location with known parameters for a tidal prediction was Knysna and the five most significant input parameters are shown in Table 6-2. Approximate simulated tidal water levels were established for the area but were not cross referenced to any prototype tidal data, such as Lowest Astronomical Tide or Highest Astronomical Tide.

Table 6-2: Input Parameters for Tidal Simulation (United Kingdom Hydrographic Office, 2011)

Input Parameter	Amplitude (m)	Phase Angle (degrees)
Z_0	0.272	0
M_2	0.54	108
S_2	0.27	134
K_1	0.06	173
O_1	0.02	296

The value of Z_0 is calculated as follows. The value of Z_0 above Chart Datum (CD) is given by the United Kingdom Hydrographic Office as +1.06m (United Kingdom Hydrographic Office, 2011). However, given that the model datum level was chosen as mean sea level (MSL), a correction factor is needed to convert the value of Z_0 to a datum level of MSL. The height of MSL relative to CD was found to be +0.788m in the South African tide tables by SANHO (South African National Hydrographic Office, 2013). Therefore, the value of Z_0 relative to MSL is +1.06m – 0.788m, which is +0.272m. This is also illustrated in Figure 6-5.

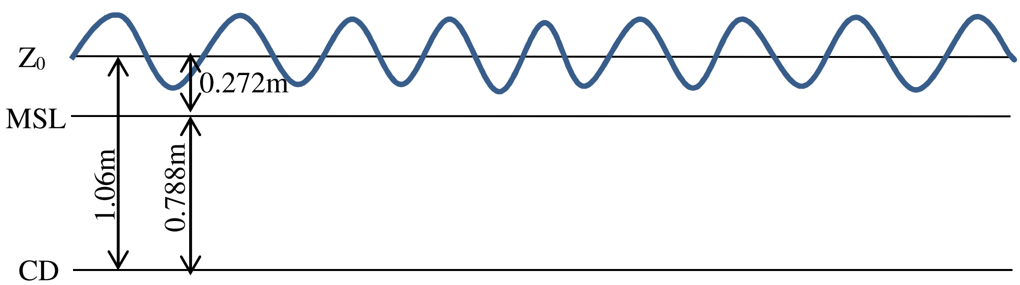


Figure 6-5: Calculation of Z_0 relative to MSL

6.4.4. Closed Boundary

A closed boundary exists at the eastern side of Rondevlei. Rondevlei receives no runoff directly by a river and there is no inflow or outflow of water at this boundary. Therefore, this boundary was made closed.

6.5. Hydrodynamic Parameters

6.5.1. Ground Water Leakage

There is very limited information available on the groundwater flow characteristics within the Wilderness region. However, according to Hughes and Filmater in 1994, none of the major geological strata in the area are permeable enough to permit the existence of a primary aquifer; therefore the ground water movement is primarily determined by the extent to which the rocks are weathered and fractured (Hughes & Filmater, 1994). Due to the lack of valuable data, the ground water leakage coefficient was determined during the calibration of the model. An arbitrary value of 2.3e-8/s was used which appeared to work well within the model. This value was found over long term based simulations with a closed sand bar, where the rate of storage of the lakes could be assessed. It must be noted, however, that rainfall and evaporation was not included in this model. For values lower than 2.3e-8/s, the storage rate within the lakes appeared to be too rapid and for higher values, above 2.3e-8/s, the rate of storage was insufficient, as a large percentage of the water entering the lakes was lost to ground water leakage.

6.6. Model Testing

6.6.1. Introduction

The model testing phase included a simple run, to ensure that no faults were presents and the model was working sufficiently. A simulation was performed with an open mouth condition, where a tidal water level was able to propagate through the entire system. In order to maintain simplicity, no river inflow boundaries were included in the model during the testing phase. A week long simulation period was analysed and the water level changes in different parts of the system were recorded for the entire week. The date was arbitrarily chosen as the first week in February 2004.

6.6.2. Results

Figure 6-6 shows the results of water level over time for different locations along the Touw River estuary and Wilderness coastal lakes. Chosen locations included the Touw mouth, upper Touw estuary, Serpentine River, Island Lake, Bolangvlei and Rondevlei.

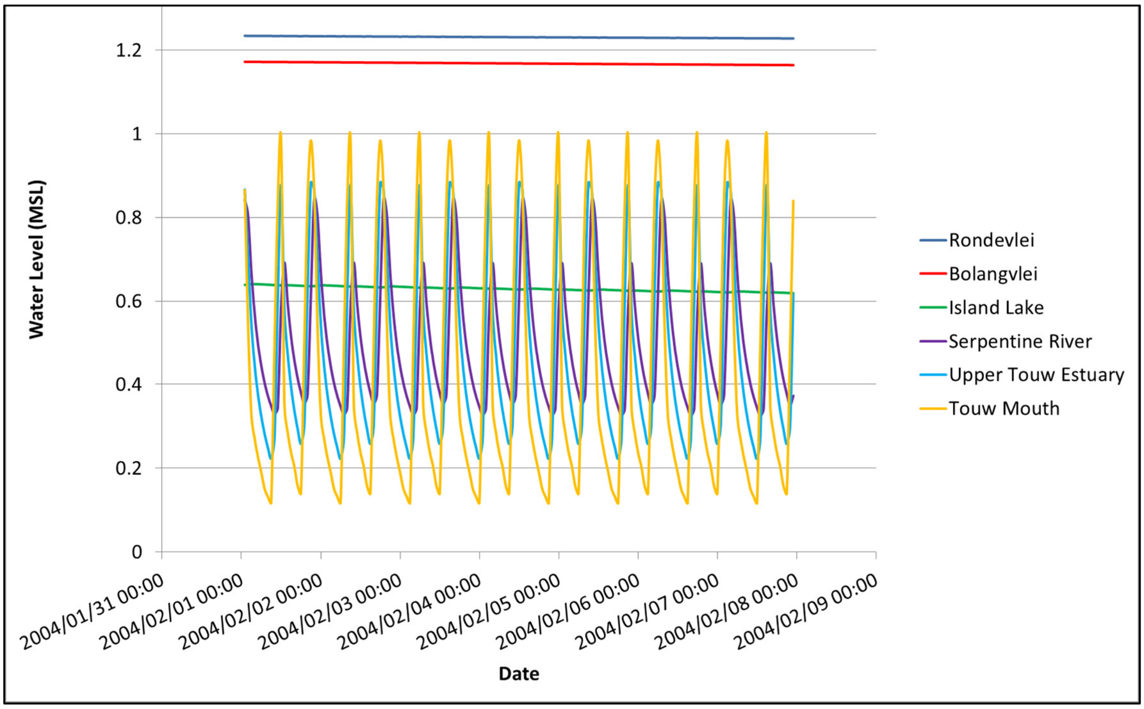


Figure 6-6: Results of model testing

From Figure 6-6, a tidal cycle can be seen in the Touw mouth, upper Touw estuary and in the Serpentine River. However, no tidal fluctuation is evident in any of the lakes. It appears from these results, in this case, that tidal penetration in the system reaches a maximum upstream location at the intersection between the Serpentine River and Island Lake. Furthermore, the tide height in the system decreases the further removed from the ocean. The tide height in the Touw mouth is 0.9m whereas the tide height in the Serpentine River is 0.5m.

The results of the model testing were favourable, which implied that a working model had been established. However, the model still required calibration to ensure that results best replicate prototype events within the Touw River estuary and Wilderness coastal lakes.

6.7. Representation of sand bar processes at estuary mouth

Due to the nature of this water system, the Touw mouth has a temporary open/closed mouth condition. The estuary is not always exposed to a tidal influence due to the formation of a high sand bar which closes the Touw mouth during periods of low river flow. The level of the sand bar in front of the estuary has a vital impact on the hydrodynamics and water levels achieved in the entire water system and it is, therefore, important that this factor is incorporated in the model and that the behaviour of the simulated sand bar best replicates reality as closely as possible. In order to simulate a temporary open/closed estuary mouth situation, a control structure was implemented at the estuary mouth. This control structure made use of an over topping gate whereby the level of the gate could be controlled. The level of the gate was representative of the level of the sand bar in front of the estuary mouth and where a gate level of 0m represented a complete open mouth condition. The position of the control structure can be seen in Figure 6-1.

An important component in this type of modelling is the initial level of the sand bar directly before the occurrence of any flood event. Historical records of sand bar elevations do exist for the sand bar in front of the estuary mouth. However, this data set consists of on-going weekly recordings of sand bar elevations and does not account for the reduction in sand bar elevation often performed directly before a flood event. In recent years, the SANParks team in Wilderness have applied a number of strategies to try and alleviate flood levels in the estuary by artificially managing the height of the sand bar. This action usually takes place a couple of days before a predicted flood event and often includes activities resulting in a considerable reduction in the elevation of the sand bar, such as the construction of a preparatory channel (South African National Parks, 2010). Data could not be sourced on the levels of the sand bar directly before a flood event and it is unknown whether this data is actually recorded.

This made it difficult to assume initial sand bar elevations used in the calibration and validation numerical models, based on the existing sand bar elevation data alone. Due to the number of variables and uncertainty involved in the estimation of the sand bar elevation directly before a flood event, it was decided to use a conservative value of +2.1m MSL. This value correlates to the minimum level of sand bar skimming set out in the SANParks management plan (South African National Parks, 2010).

It is also important to establish a good grasp of the general behaviour of the sand bar in terms of scour rates. However, no exact data existed for this factor and there is limited research performed in this field. The CSIR estimated the rate of scour of the sand bar to be a constant 0.011m/minute in depth (CSIR, 1982). The scour rate of the sand bar width was not reported. This scour rate was initially tested and was found to be too rapid and a linear estimation of the scour rate did not provide a realistic representation of the prototype conditions.

A more suitable scour rate that was established during the investigation was an exponential scour rate. Once the water level exceeded the elevation of the sand bar, scour begins and the sand bar decreases in height. An initial rapid scour rate is evident; however, the scour rate does not continue at the same rate. Instead the rate of scour gradually decreases over time until equilibrium is achieved between deposition of sediments and scour.

A sensitivity analysis was performed on the scour rate prediction and compared to the results of a linear scour prediction. Figure 6-7 compares the results of water levels in the Touw mouth using different theories related to scour rate of the sand bar with prototype water level conditions. Based on Figure 6-7, it is evident that, although the peak water levels are similar in both cases, an exponential scour rate better replicates the prototype data following the flood peak. During the model calibration, time was devoted to establishing realistic scour rates of the sand bar to understand the behaviour of the sand bar in front of the estuary mouth.

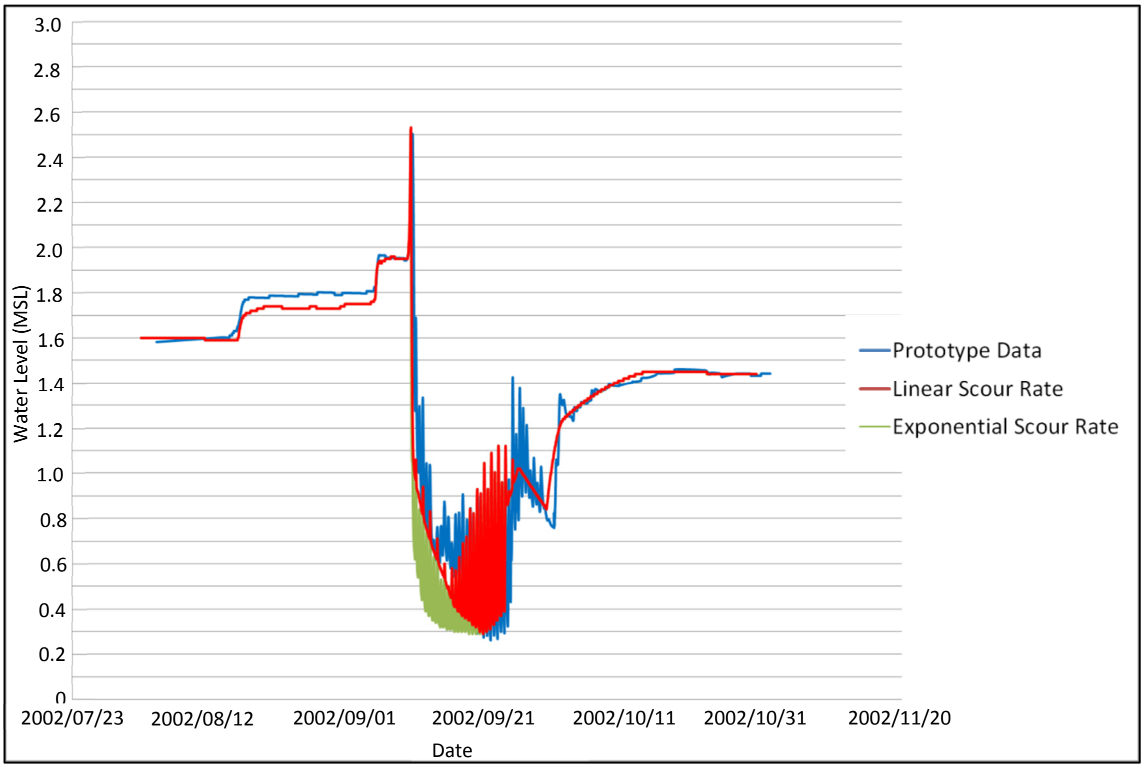


Figure 6-7: Comparison between an exponential scour rate and a linear scour rate and the resulting effect on the Touw Mouth water levels

Deposition of sediment is an important aspect in controlling the height of the sand bar and determining the rate of scour. Following the scour of the sand bar, the estuary mouth opens and marine effects become present within the system allowing marine and river processes to interact. Tidal effects have an impact on the system, thus bringing marine sediment into the estuary mouth region and depositing the sediment onto the sand bar. Furthermore, this process of marine sediment deposition is significantly enhanced during a major storm event (Beck, 2005). A secondary factor leading to an increase in sediment deposition at the estuary mouth is the potential for large volumes of alluvial sediment from the Touw River. Riverine sediments in the Touw River are, however, generally found further upstream in the estuary near Ebb & Flow rest camp.

Figure 6-8 shows the exponential approximation of the rate of scour used in the model under a particular flood event. The initial rate of scour is rapid and the sand bar elevation decreases by 0.5m in the first hour, after scouring commences. Only scouring of the sand bar in the vertical direction was considered and the width of the gate remained constant throughout the whole scouring process. The figure also shows the hydrograph of the flood and the resulting prototype Touw water level at the railway bridge water level recorder.

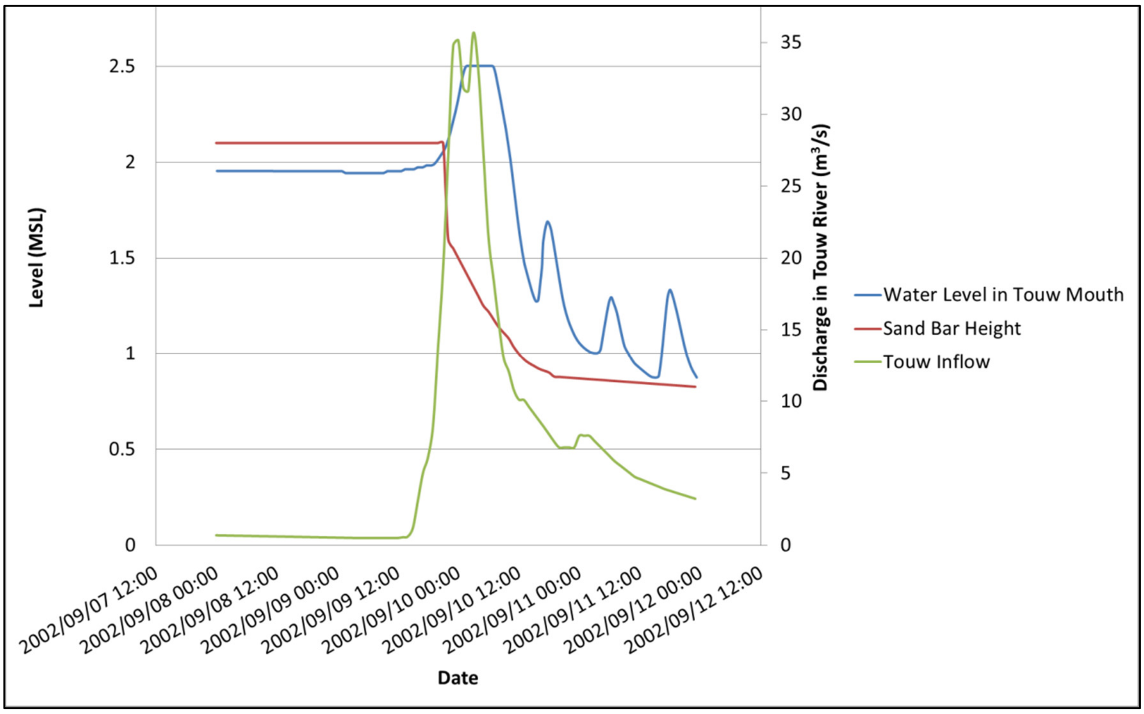


Figure 6-8: Exponential scour rate of sand bar along with Touw inflow and resulting water levels in the Touw mouth.

Logical operands were also tested to control the height of the sand bar during a flood event where the estuary mouth transitions from a closed mouth condition to an open mouth condition. These logical operands made use of "if statements" based on the upstream water level behind the gate; if the upstream water level just landward of the sand bar is greater than the height of the sand bar then, at the next time step, set the sand bar to a fully open mouth condition. The results of these tests showed that the gate opened too early compared to the prototype situation and insufficient build-up of water was evident in all water bodies.

7. MODEL CALIBRATION

7.1. Introduction

The hydrodynamic numerical model was calibrated against prototype water level data which was gathered from the SANParks Scientific Services Office at Rondevlei. There was no previous data related to water velocities in the Wilderness system and therefore water velocities were unable to be used as a calibration tool. The strategy used for calibration was to model flood events where the estuary mouth transitions from a closed mouth condition to an open mouth condition. Normal flow events were not calibrated. The rationale behind this strategy was that a flood event provides the most significant change in hydrodynamics in the shortest period of time and is also relevant for the desired outcome of the model. As mentioned previously, the outcome of the model was the prediction of water levels in the Touw estuary and coastal lakes under different flooding scenarios; therefore, it would be appropriate to calibrate the model using a flood event. During a flood event that causes a breach of the sand bar at the estuary mouth, the water level in all water bodies increases to a peak level and then decreases once the sand bar is breached and the water in the system drains into the ocean. By calibrating the model using a flood event that transitions from a closed mouth to an open mouth, the peak water level achieved in each water body, as well as the filling and drainage rates of each water body, is able to be observed. This provides a number of different variables useful for the calibration of the numerical model and to assess whether the model does in fact predict a realistic representation of the prototype event.

7.2. Calibration flood event

The flood event used in the calibration of the numerical model is an event that took place on the 22 November 2007. This was a large flood event, with a peak flood inflow discharge estimated to be slightly greater than 300m^3/s and a total volume of runoff in the Touw River of 27.64 million m^3. According to Görgens' flood hydrographs, this correlates to a flood with a return period of greater than 100 years (Görgens, 1979). As a result of the large flood, significant changes in water levels are evident in all water bodies. This allows for a good opportunity to calibrate and adjust the model in order to replicate the prototype event. Figure 7-1 illustrates the 2007 flood hydrograph compared to the 1979, 100 year hydrograph prediction (Görgens, 1979). From Figure 7-1 it can be seen that the peak discharge between the two hydrographs is fairly similar but the duration of the 2007 flood event is a lot longer than that of the predicted 100 year flood, thus giving rise to a significant increase in the total volume of water associated with the flood. Given that the total volume of water associated with a 100 year flood prediction is 11.47 million m^3, the 2007 flood, with a total volume of 27.64 million m^3, is approximately 140% greater than the 100 year flood prediction of the 1979 study, in this regard.

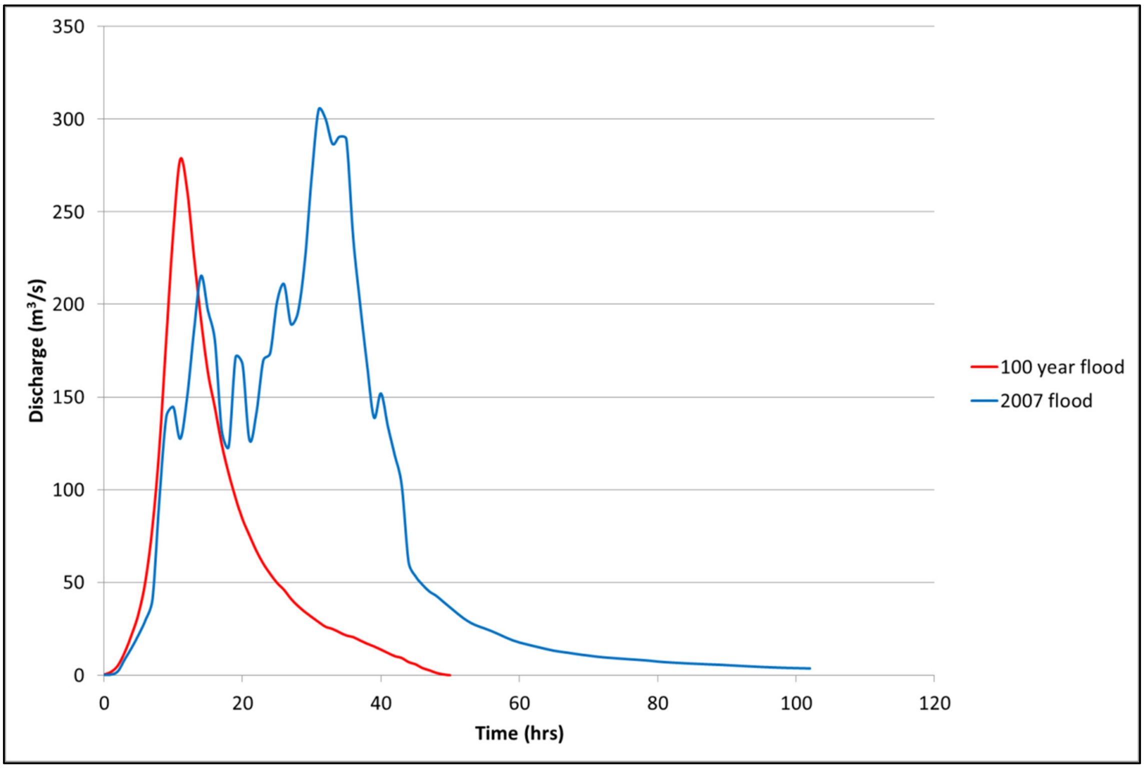


Figure 7-1: Comparison between hydrographs of 2007 flood event and a 100 year flood prediction

The time frame used for the simulation included a three month period, where the flood event occurred during the middle month. This allowed for sufficient time both before and after the flood event to analyse the storage and drainage rates as well as the peak water levels achieved in each water body.

The actual measured water levels achieved during the flood event are shown in Figure 7-2. The figure was constructed using the water level data gathered from SANParks at the Scientific Services Office at Rondevlei. Four water level recorders are present in the Wilderness estuary and coastal lakes system and are located at Rondevlei, Bolangvlei, Island Lake and the Touw River estuary at the old railway bridge.

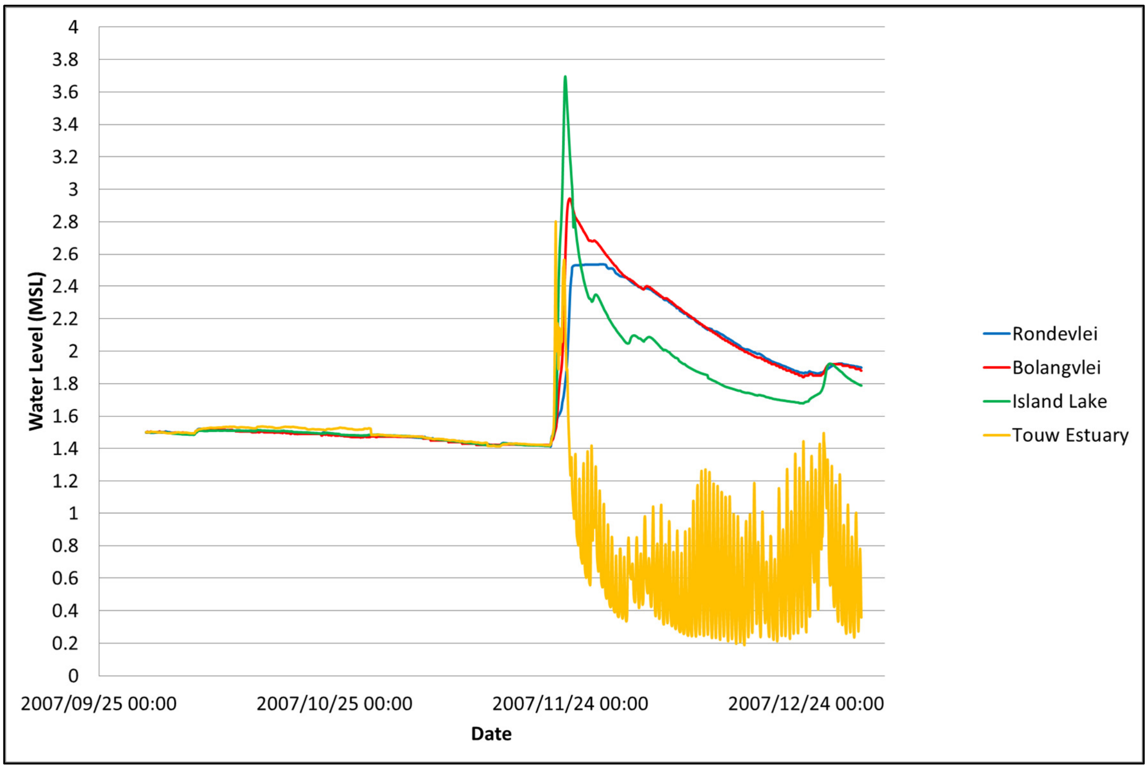


Figure 7-2: Actual measured water levels during 2007 flood for all water bodies

From Figure 7-2 it is apparent that the estuary transitioned from a closed mouth condition to an open mouth condition. When looking at the water levels achieved in the Touw estuary, given by the orange line in the figure, it can be seen that the water level remained relatively constant before the flood, thus illustrating a closed mouth. After the flood, the water level drops suddenly and significantly and a tidal cycle is evident in the estuary, illustrating an open mouth condition. There also appears to be a rapid rise in water level for all water bodies immediately after the flood, but closer inspection needs to be taken in order to evaluate the real storage rate in each of the water bodies.

Figure 7-3 shows the actual flood event in more detail and the rise in water level for each water body can clearly be seen from this figure. It can be established that the water level of the Touw estuary rises at the fastest rate, followed by Island Lake, Bolangvlei and lastly Rondevlei. This difference in storage rate between the water bodies is most likely due to the volume of inflow to each water body as well as the size of each water body. The prototype water level data for Rondevlei shows that a constant water level of 2.533m is evident for a period of four days. This is viewed as either an instrument failure or a previous instrumentation limitation. This is illustrated by the dashed line on the figure.

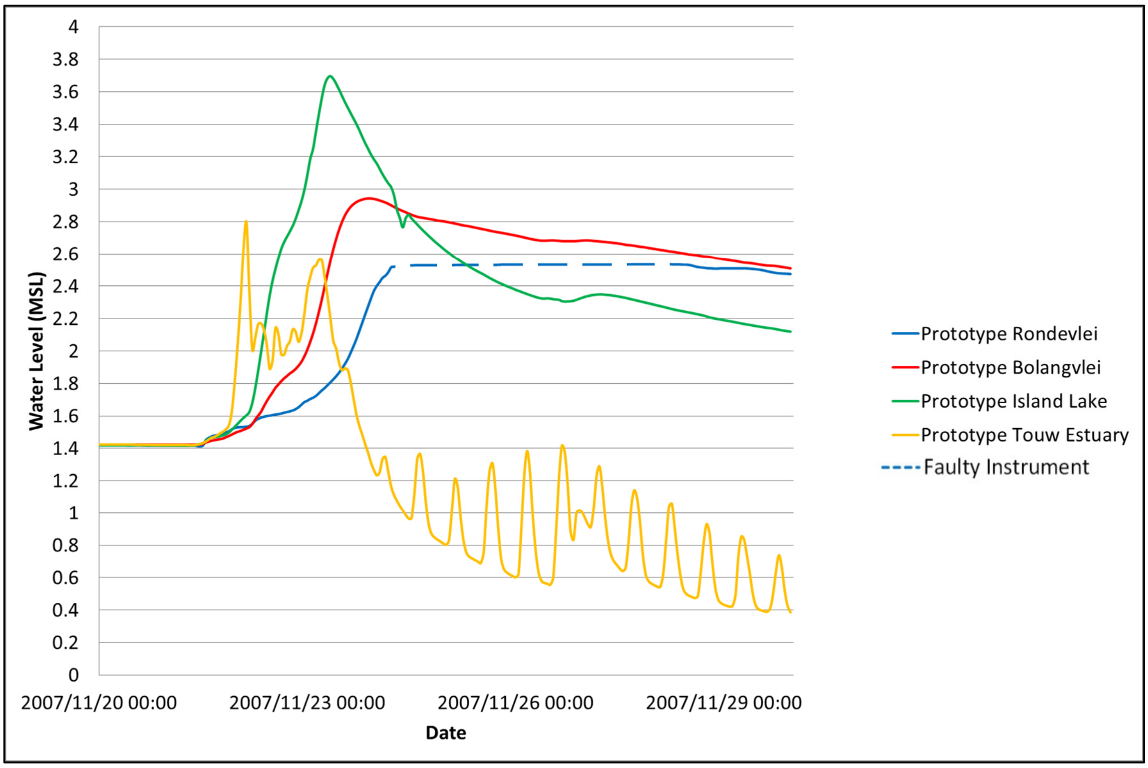


Figure 7-3: Inspection of the rate of water level rise for actual measured water levels during 2007 flood for all water bodies

The drainage rates of each water body was also analysed once the sand bar had been breached and water was allowed to flow out of the system and into the ocean. The rate of drainage is predominantly determined by the proximity of the water body to the estuary mouth, where the closer the proximity, the faster the rate of drainage. From Figure 7-2 and Figure 7-3, it can be seen that the drainage rate of the Touw estuary is the fastest, followed by Island Lake. Finally, Bolangvlei and Rondevlei seem to share a very similar rate of drainage.

Peak levels achieved in each water body are summarised in Table 7-1. It must be noted however, that the following water levels are purely based on the 2007 event and do not provide an indication of all flood events.

Table 7-1: Actual prototype peak water levels achieved in all water bodies for 2007 flood

Water Body	Peak Water Level (msl)
Touw Estuary	2.8m
Island Lake	3.7m
Bolangvlei	2.95m
Rondevlei	2.53m

These three factors, namely the storage rate, drainage rate and peak water levels were important justification parameters, useful for the calibration of the model. These parameters were analysed during the numerical modelling calibration process, which is discussed below.

7.3. Calibration of the numerical model for the 2007 flood

7.3.1. Introduction

The aim of the calibration process of the numerical model was to create a numerical model that best replicated the 2007 flood event in all regards, while paying special attention to the filling rates, drainage rates and peak water levels in each water body. The specific input parameters used in numerical models are described below.

7.3.2. Inflow

The flood hydrographs used for the different inflow sources can be seen below in Figure 7-4. It must be noted that only the flood hydrographs for the duration of the flood event are illustrated below and not the inflow for the whole simulation period which was significantly smaller. These inflows were derived from an extrapolated Q-h rating which is discussed in detail in Chapter 5.2.

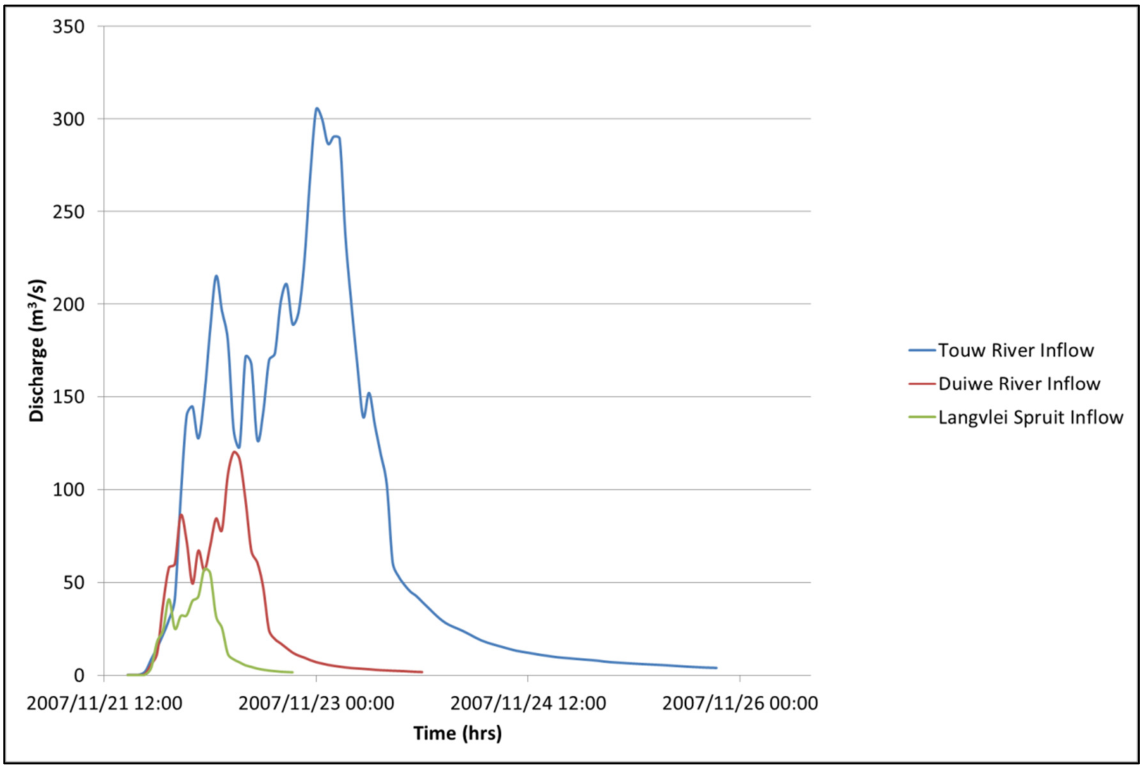


Figure 7-4: Predicted flood hydrographs for different sources during 2007 flood

7.3.3. Sand Bar Scour

The sand bar scour follows the decreasing scour rate theory as described in the model set up and testing chapter. Figure 7-5 shows the control structure height over time and also only focuses on the flood event as opposed to the entire simulation period. This estimation of sand bar scour rate appeared to show the best results during the model calibration.

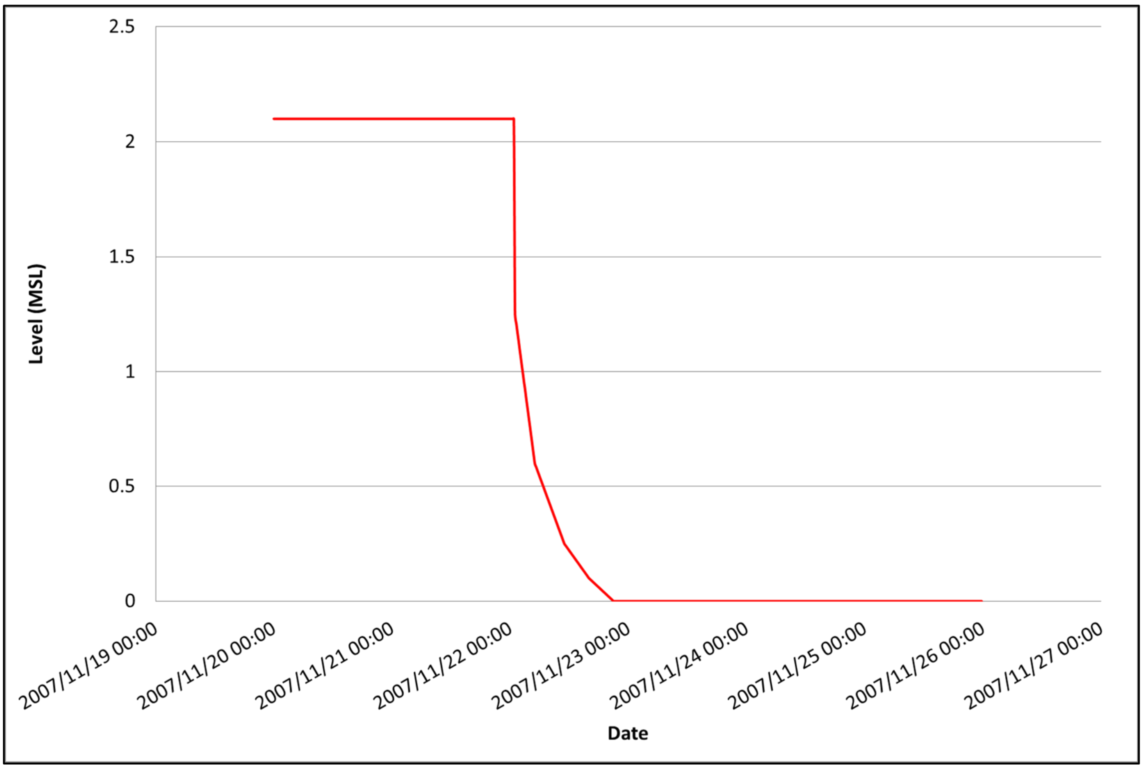


Figure 7-5: Model sand bar height over time during 2007 flood

7.3.4. Results of the model calibration

The results of the numerical model calibration are discussed below. The results show a close approximation to the prototype event with a maximum tolerance of 8.1% when predicting peak water levels. The rate of storage and drainage of the water bodies also provides a good approximation to those of the prototype event.

Figure 7-6 shows the model predicted water levels for all water bodies for the entire three month simulation period and the same trends can be seen in the model simulation as in the prototype event. Figure 7-6 best illustrates the overall drainage rates of different water bodies once the sand bar is breached and open to the ocean. A similar relationship is evident when compared to the prototype event illustrating that the Touw estuary has the fastest drainage rate, and Bolangvlei and Rondevlei equally share the slowest rate of drainage.

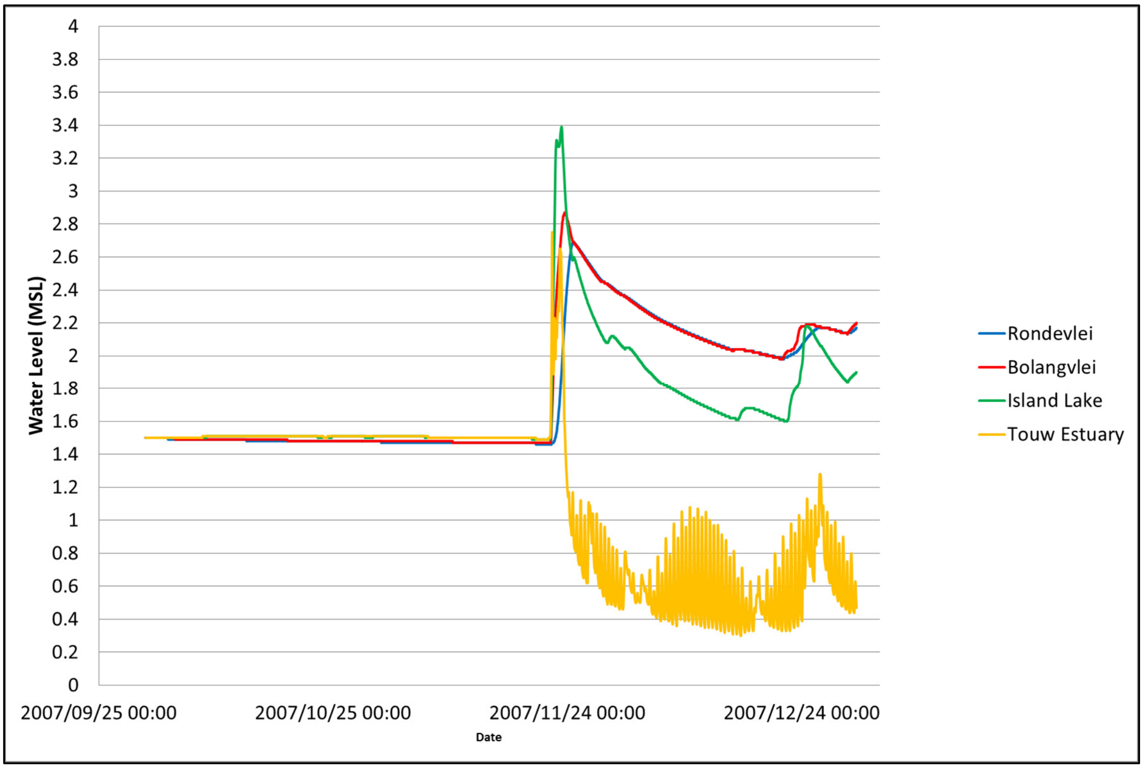


Figure 7-6: Model predicted water levels during 2007 flood for all water bodies

A direct comparison between prototype water levels and model predicted water levels for each water body is shown in Figures 7-7 to 7-10.

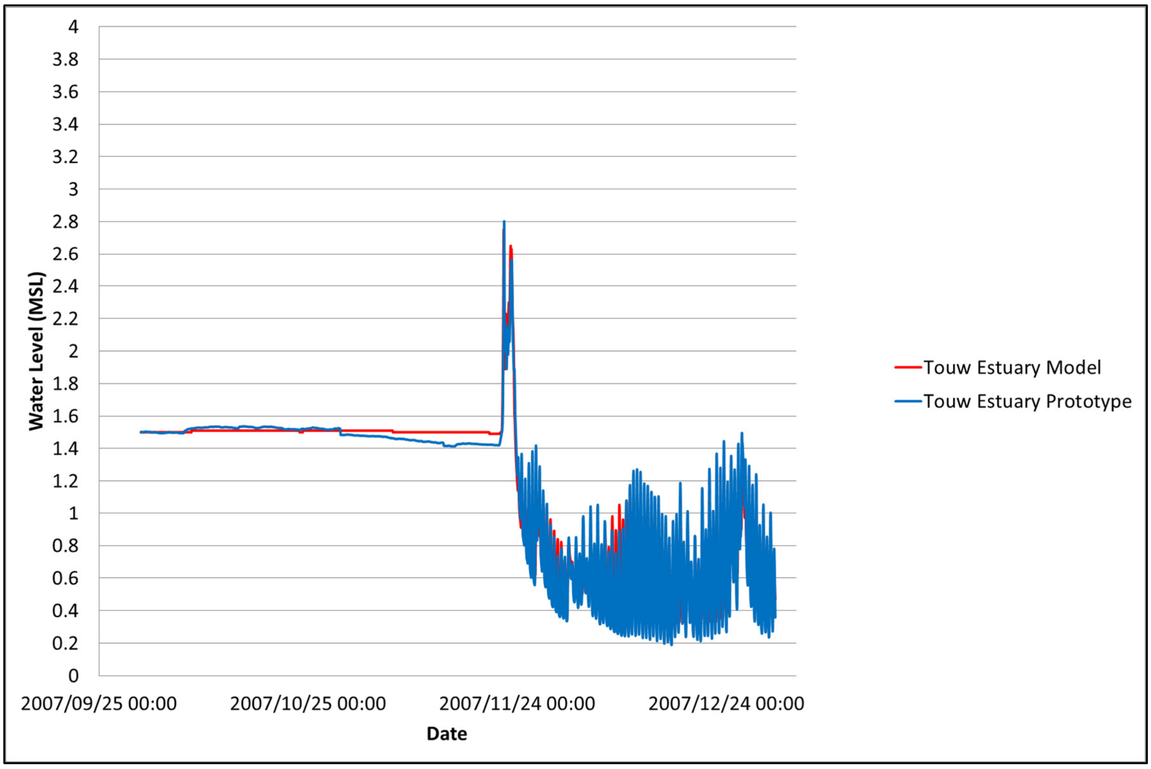


Figure 7-7: Touw Estuary direct comparison between prototype and model water levels during the 2007 flood

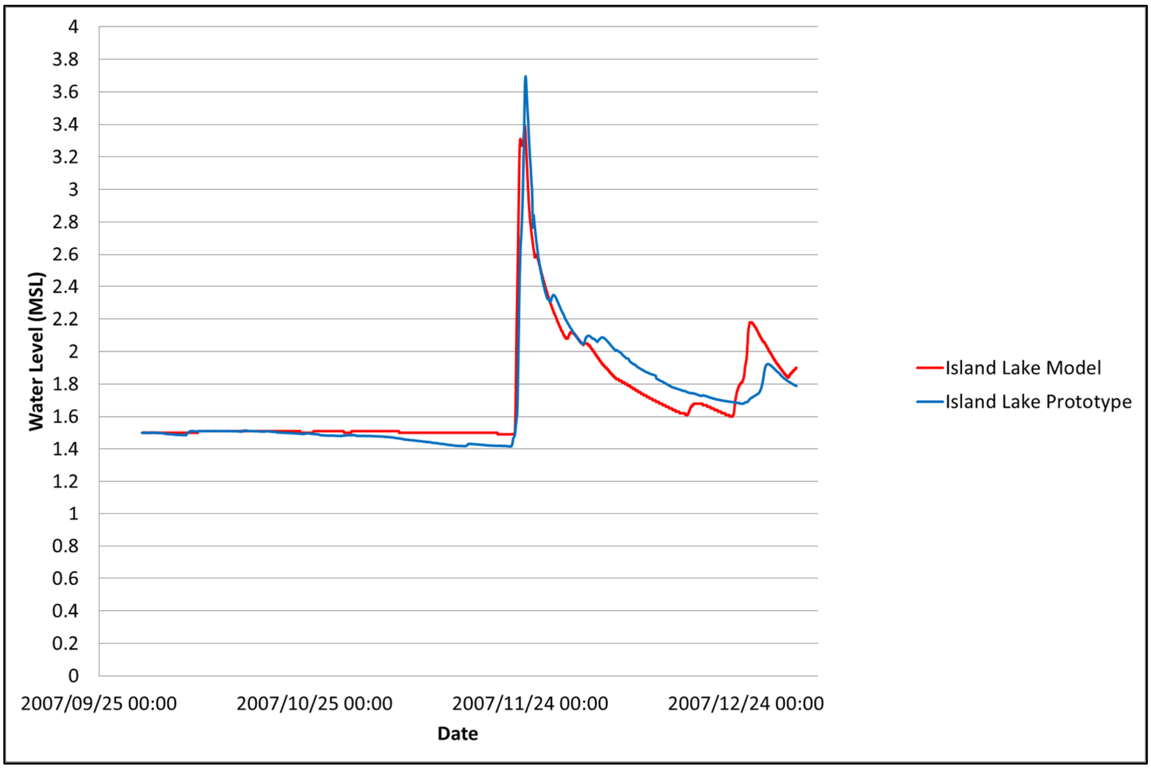


Figure 7-8: Island Lake direct comparison between prototype and model water levels during the 2007 flood

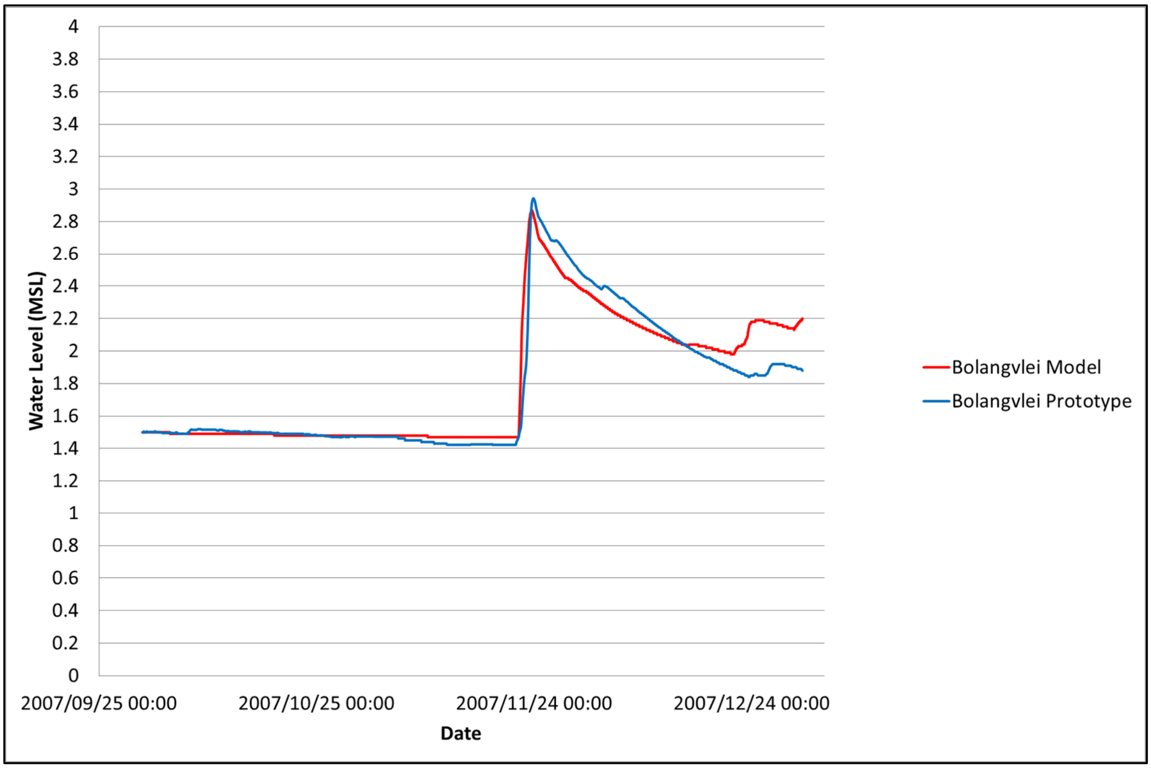


Figure 7-9: Bolangvlei direct comparison between prototype and model water levels during the 2007 flood

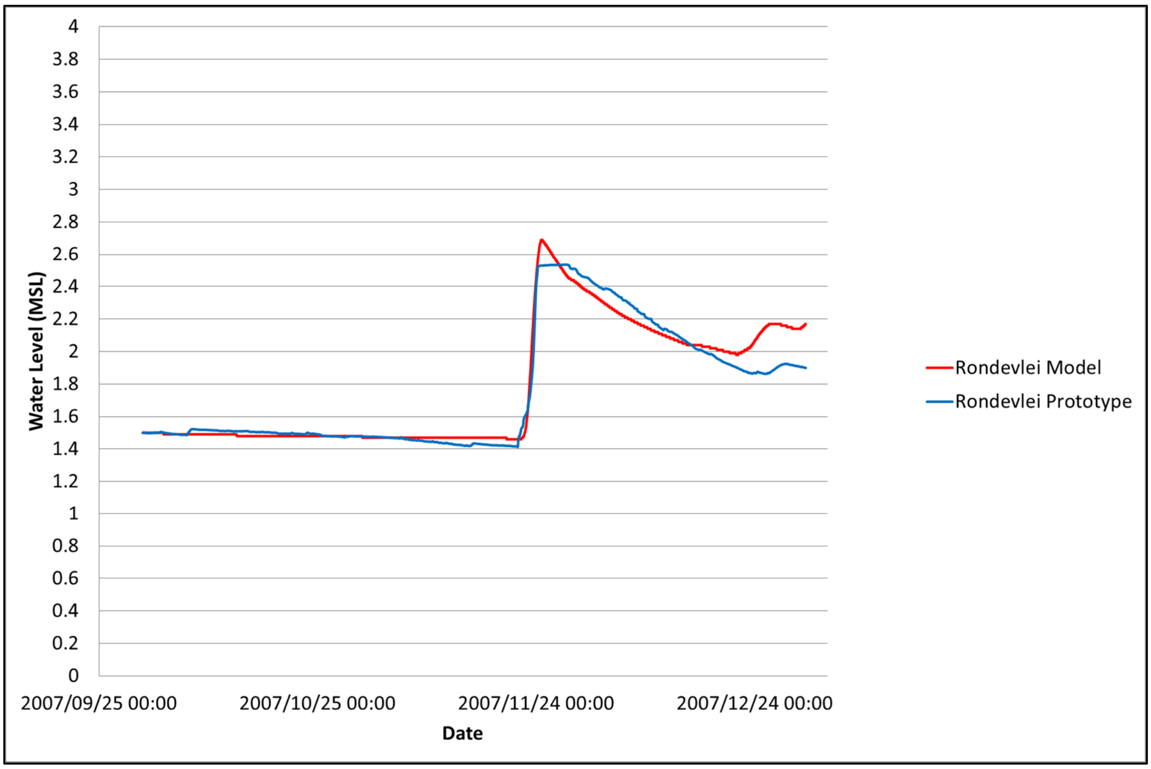


Figure 7-10: Rondevlei direct comparison between prototype and model water levels during the 2007 flood

The direct comparison between water levels predicted by the model versus the actual recorded water levels illustrates precisely the differences between the prototype event and the model predictions. Although the drainage rates in the water bodies are not perfect, a good assumption of drainage rates has been made.

In order to evaluate the storage rates in the water bodies, a closer inspection of the actual flood event needs to be undertaken over a shorter period of time. Figure 7-11 shows a representation of the flood event over a shorter time period, thus illustrating the storage rates as well as the peak water levels achieved in each water body.

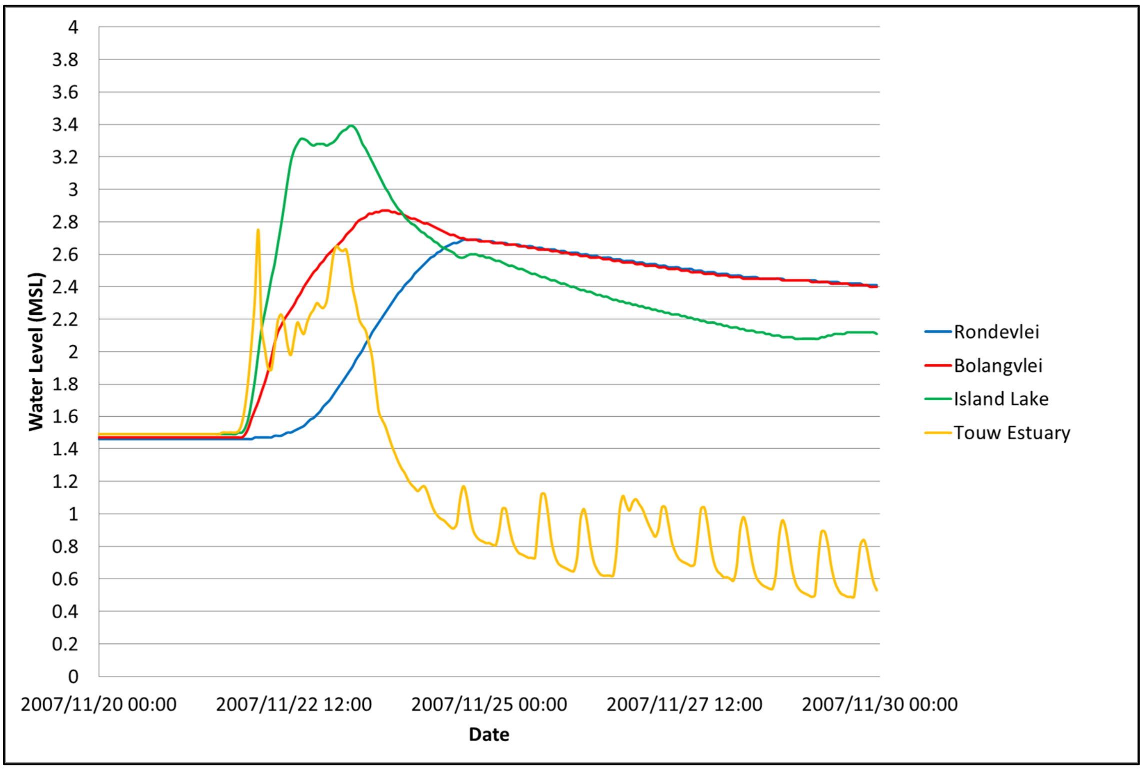


Figure 7-11: Model predicted water levels for the 2007 flood over shorter time period for all water bodies

Figure 7-11 shows that the Touw estuary has the fastest storage rate and the water level in this water body rises rapidly during a flood event. The water body with the second fastest storage rate is Island Lake, followed by Bolangvlei and finally Rondevlei. These relationships are exactly the same as those depicted during the prototype event.

A direct comparison between the water levels achieved during the prototype event and the predicted water levels, based on the numerical model is illustrated in Figure 7-12. The solid lines indicate the actual prototype water levels recorded during the 2007 flood event and the dashed lines show the model predictions of each respective water body. It is important to keep in mind that an instrument failure occurred at Rondevlei from the 24/11/2007 – 28/11/2007 and is represented by the unchanged water level in Rondevlei over these days.

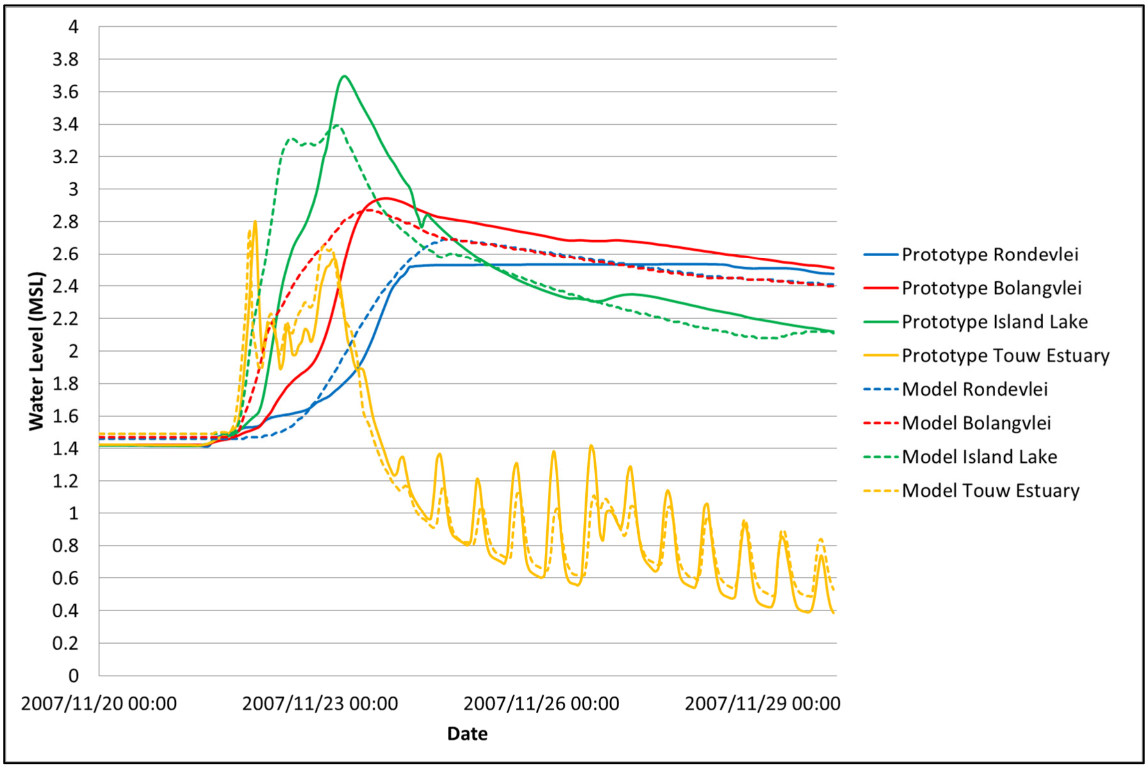


Figure 7-12: Direct comparison between model predicted water levels and water levels recorded during the 2007 flood event for all water bodies

From Figure 7-12, the rate of storage in the Touw estuary is reasonably well represented, but the rate of storage in the lakes is slightly greater in the numerical model compared to that of the prototype event, leading to an earlier peak water level. This may be a result of the input method used for the inflows into Island Lake and Bolangvlei in the numerical model. In reality, the inflows to these lakes both occur on the northern boundary of each lake. However, given the fact that the numerical modelling was performed using a one dimensional modelling approach, there are certain limitations involved. One of these limitations includes the fact that it is impossible to simulate an inflow on the northern boundary of the lake. Instead the inflow to the lakes was represented by a point source which occurred along the centreline of the network. This may result in a faster storage rate in the lakes, as the inflow source is fed directly to the centre of the lake as opposed to the northern boundary.

The peak water levels predicted by the model were also analysed and compared to those of the prototype event. The peak water levels for both the model and prototype event can be seen in Figure 7-12, and Table 7-2 summarises the information.

Table 7-2: Comparison between peak water levels of the model simulation and the prototype event for all water bodies during 2007 flood event

Water Body	Prototype Peak Water Level (msl)	Model Peak Water Level (msl)	Water Level Difference (m)	Percentage Difference (%)
Touw Estuary	2.8m	2.76m	-0.04	-1.4
Island Lake	3.7m	3.4m	-0.3	-8.1
Bolangvlei	2.95m	2.89m	-0.06	-2.0
Rondevlei	2.53m	2.7m	0.17	6.7

The greatest degree of difference between the maximum water levels predicted by the model, versus those recorded during the actual event, occurs in Island Lake and under-predicts the peak water level in Island Lake by 8.1%. The water level prediction in the Touw estuary shows the best results with a 1.4% difference between the model and the prototype event. It is also vital that the Touw estuary water levels are accurate as this is the water body with the greatest residential and community significance. Many houses are situated on the banks of the Touw estuary and have been flooded during past flood events and this is the primary driving factor behind the management of the sand bar at the estuary mouth. Figure 7-13 compares the model predicted water levels and the actual recorded water levels in the Touw estuary during the 2007 flood event. The figure also illustrates the sand bar height and the Touw River inflow over the same time period.

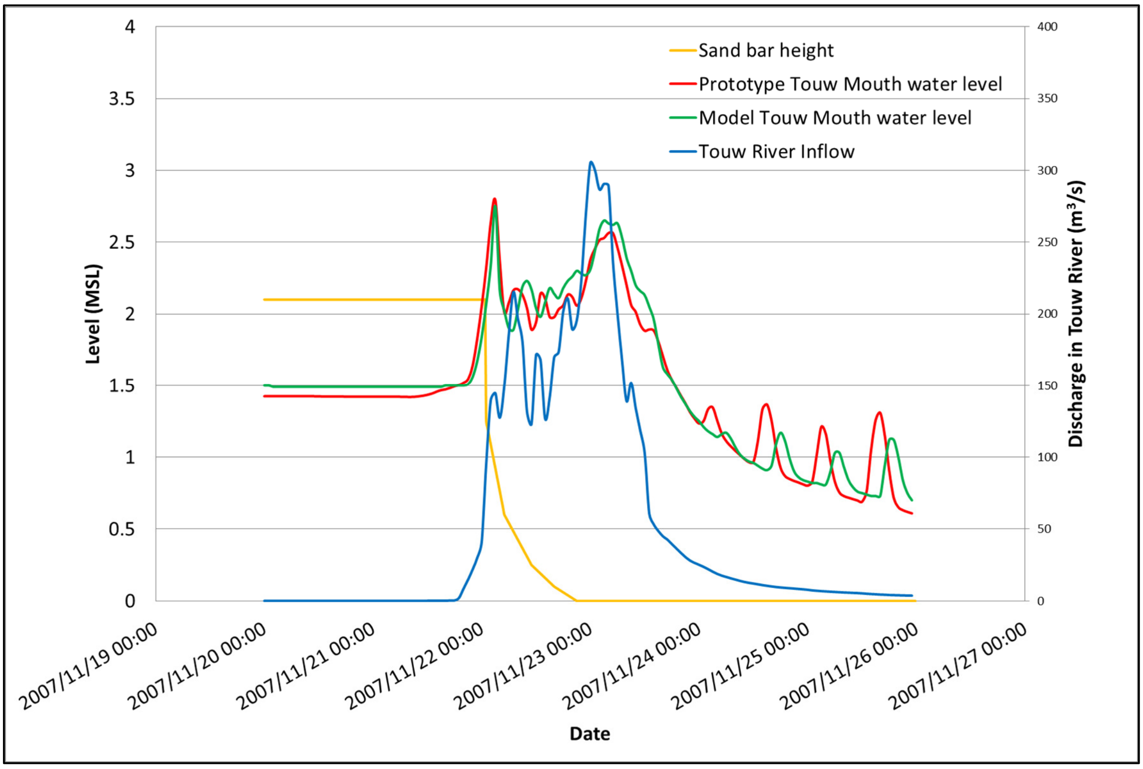


Figure 7-13: Comparison between the model predicted water levels and actual recorded water levels in Touw estuary during 2007 flood event, while also showing the sand bar height and Touw River discharge over time

From Figure 7-13 on the 22/11/2007, it can be seen that as the discharge in the Touw River increases, the water level in the Touw estuary rises; until the water level at the estuary mouth reaches the height of the sand bar. Once the water level in the estuary exceeded the level of the sand bar, the sand bar began to scour and the sand bar height decreased as a result. The estuary transitioned from a closed mouth estuary to an open mouth estuary, and once the majority of the flood discharge had passed, a tidal effect is evident in the Touw estuary.

7.4. Sensitivity Analysis

A sensitivity analysis was performed on the calibrated model to discover the sensitivity of certain model input criteria and to evaluate how results change when these criteria are altered. This is performed by systematically varying modelling parameters, such as bed friction and cross sectional area at certain critical sections.

Some of the most critical sections of the entire system from a sensitivity point of view, include the interconnecting channels between the coastal lakes and the Serpentine River. These are relatively small channels and are extremely sensitive to changes in cross section and bed friction. These channels provide vital connections between the lakes to allow for the transfer of water between different water bodies. Decreasing the size of the cross sections of these interconnecting channels causes a constriction in the channels and as a result, impedes the transfer of water between the water bodies. During a flood event, insufficient water enters the coastal lakes and the storage rate of the lakes is significantly less than that of prototype conditions. During the outflow phase, following a flood event where the estuary transitions from a closed estuary to an open estuary, the water levels in the lakes decrease at a much slower rate than those of prototype conditions due to the increased constriction in the channels.

This same result is also evident when increasing the bed friction of the interconnecting channels and Serpentine River to values above those mentioned in the calibrated model. For a detailed breakdown of the Manning n values used throughout the entire model see Appendix E. By increasing the bed resistance, the flow of water over the bed surface is impeded which has a consequential effect on the transfer of water between the water bodies.

On the contrary, when increasing the cross sections or decreasing the bed friction of the interconnecting channels, it was found that an abundance of water from the Touw River was travelling via the Serpentine River and entering the coastal lakes. The storage rates of the lakes were too rapid due to the rapid influx of flood water levels into the coastal lakes via the Serpentine River. Furthermore, during an outflow phase, where the estuary mouth is open, it was found that the model water levels within the lakes decrease at a much faster rate than prototype water levels.

7.5. Conclusion

Based on the previous Figures and Tables it can be concluded that the numerical model has been calibrated as the model is able to replicate the outcomes of the 2007 flood. However, this model also needs to be verified with other flood events in order to confirm that the same model settings are reliable for other types of flood events.

8. MODEL VALIDATION

8.1. Introduction

The validation process of the numerical model acts as a confirmation step in order to confirm that the numerical model is applicable to other flooding situations, rather than just the one calibrated event. Given that the model was calibrated using a large flood event with a maximum flood peak discharge slightly greater than 300m^3/s, the validation process made use of two smaller flood events with maximum peak discharges of 240m^3/s and 45m^3/s, providing a substantial variation in the flood intensity evident in Wilderness. These flood events took place during 2006 and 2002 respectively and are named throughout the following chapter as the 2006 flood event and the 2002 flood event.

8.2. The 2002 flood event

8.2.1. Prototype Event

The 2002 flood event took place during September 2002 and had a maximum peak discharge of 35m^3/s and a total volume associated with the flood of 2.2 million m^3. This is a relatively minor flood with a return period of approximately one or two years. This is based on the fact that the flood is significantly less than the five year predicted flood by Görgens in 1979 (Görgens, 1979). Figure 8-1 shows a hydrograph of the flood event as well as a 5 year flood hydrograph prediction.

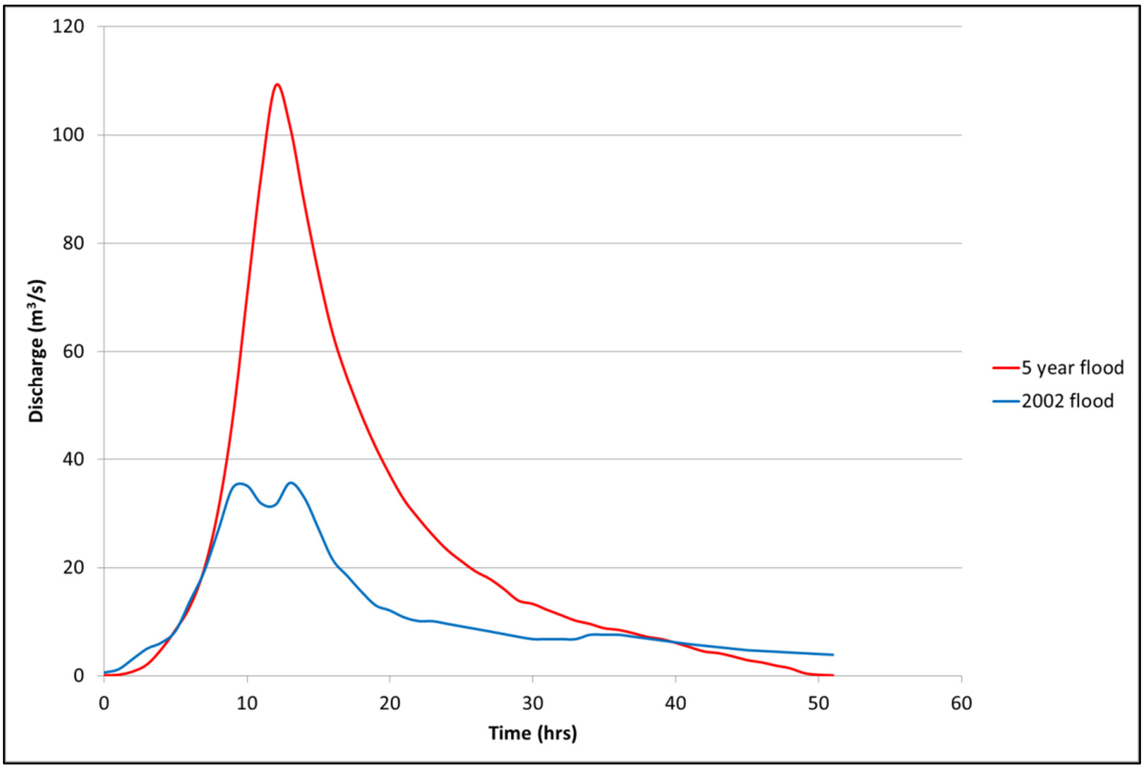


Figure 8-1: Comparison between hydrographs of 2002 flood event and a 5 year flood prediction

The actual water levels recorded in each of the water bodies for the entire simulation period are shown in Figure 8-2. A period of three months is shown in Figure 8-2 which provides sufficient time to analyse the storage and drainage rates.

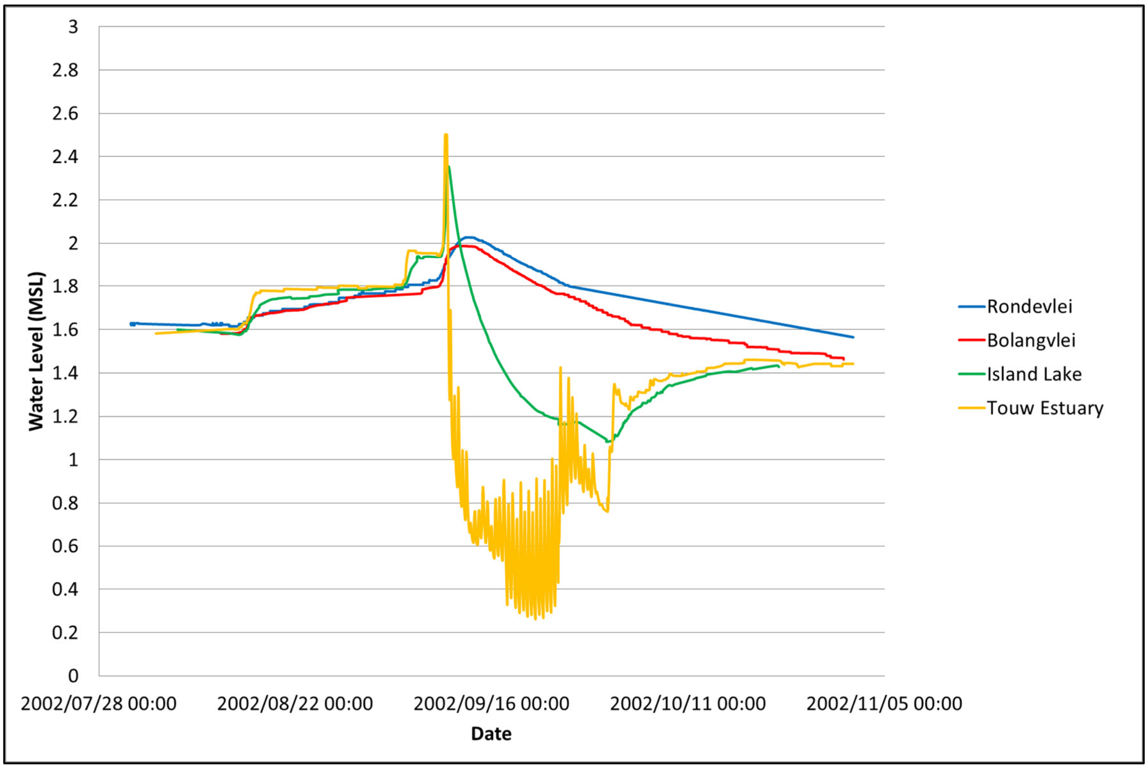


Figure 8-2: Actual measured water levels during 2002 flood for all water bodies

In this flood it is evident that the estuary transitions from a closed mouth estuary to an open mouth estuary once the flood occurs. However, the magnitude of the flood was insufficient to open the estuary for an extended period of time and the estuary closed again approximately two weeks after the flood. The closure of the estuary mouth can be seen in Figure 8-2 by the relatively stable orange line, which occurs around the 01 October 2002. It is inferred that the estuary mouth then remains closed for the rest of the simulation period, as no tidal cycle is evident in the Touw estuary water level recordings.

8.2.2. Model Simulation

Without changing any of the model parameter settings, as calibrated, the results of the model simulation are shown in Figure 8-3. The time period used in the model also includes a period of three months, which is comparable with the prototype event, thus providing the capability of a direct comparison between the prototype event and the model results.

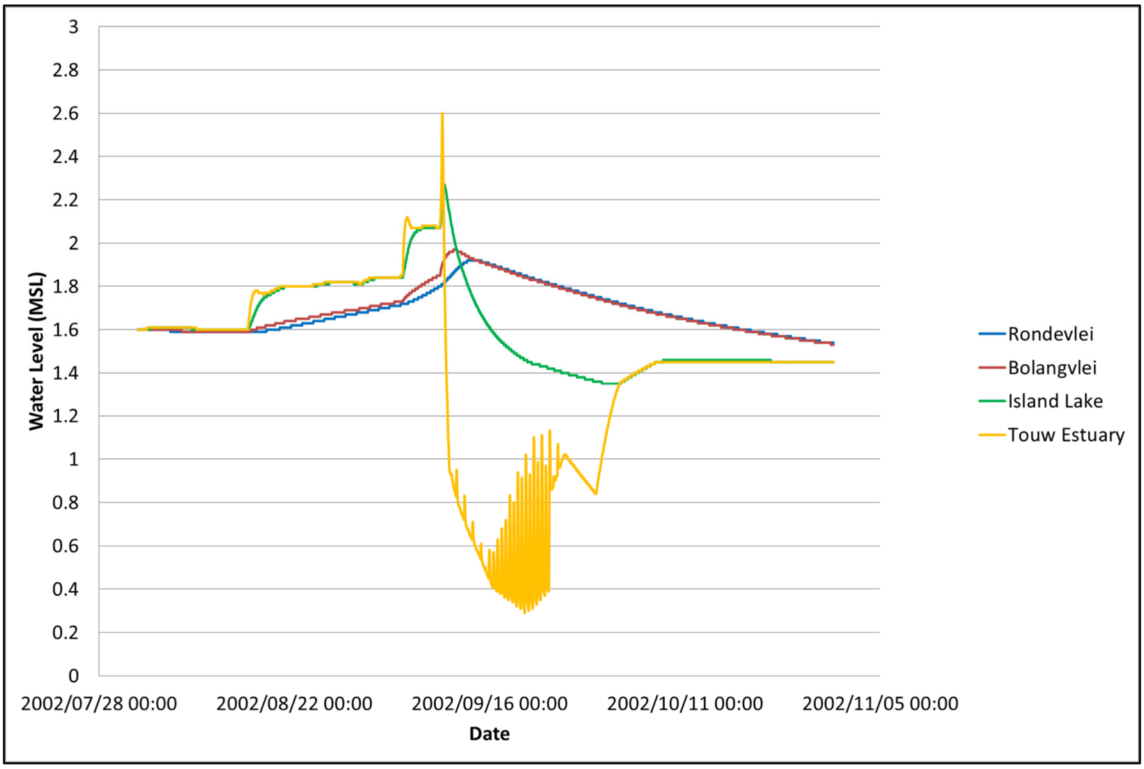


Figure 8-3: Model predicted water levels during 2002 flood for all water bodies

8.2.3. Direct Comparison

The following section highlights a direct comparison between the model predicted water levels and the actual recorded water levels for each flood. Figures 8-4 to 8-7 provide a more clarifying view on the representation of the model to the prototype event, rather than a visual comparison between the two previous figures.

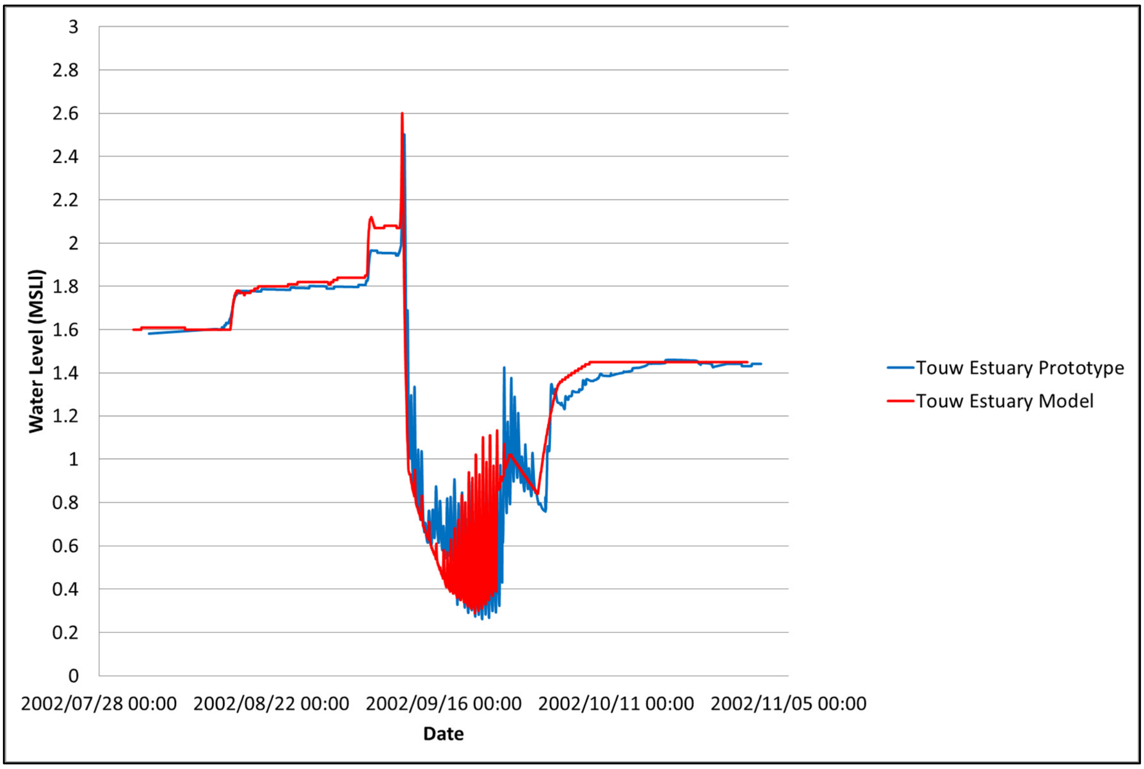


Figure 8-4: Touw Estuary direct comparison between prototype and model water levels during the 2002 flood

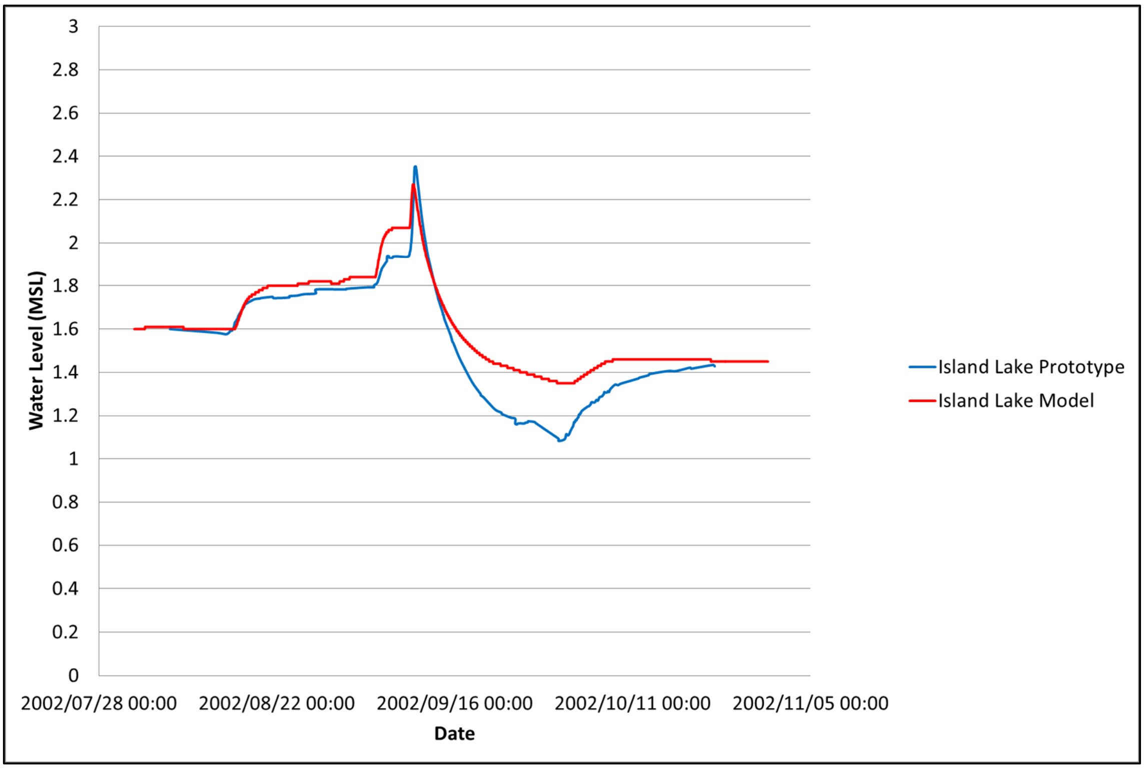


Figure 8-5: Island Lake direct comparison between prototype and model water levels during the 2002 flood

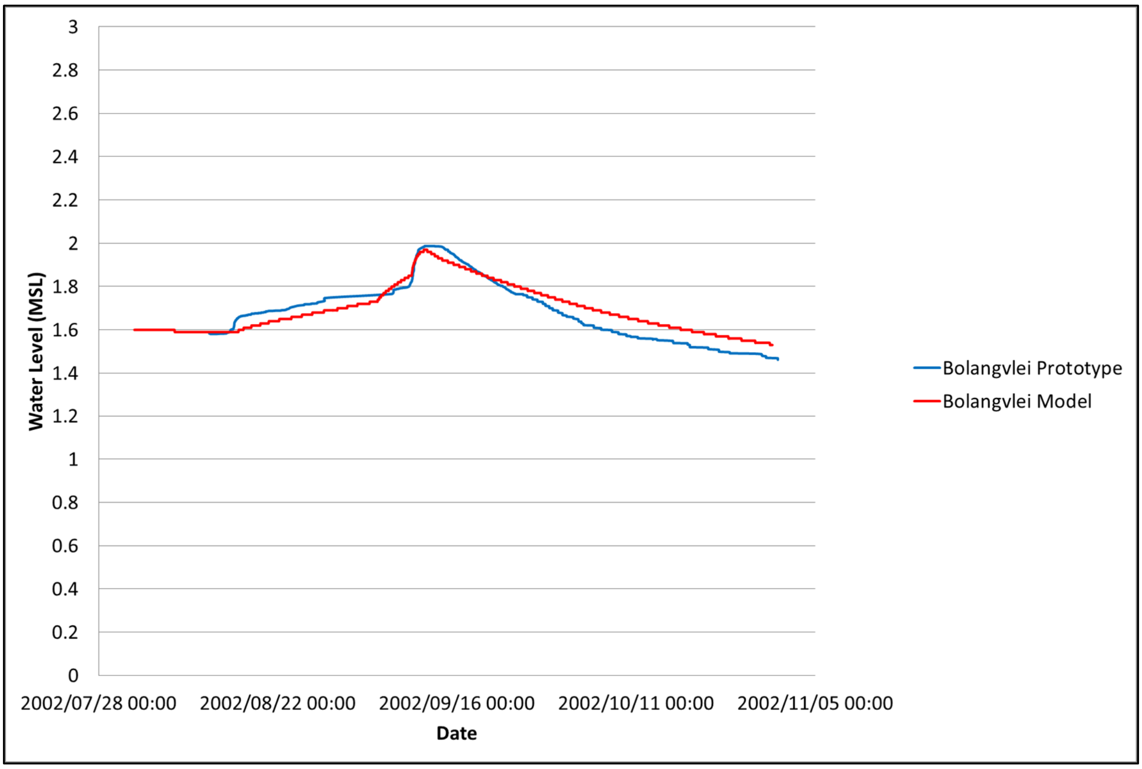


Figure 8-6: Bolangvlei direct comparison between prototype and model water levels during the 2002 flood

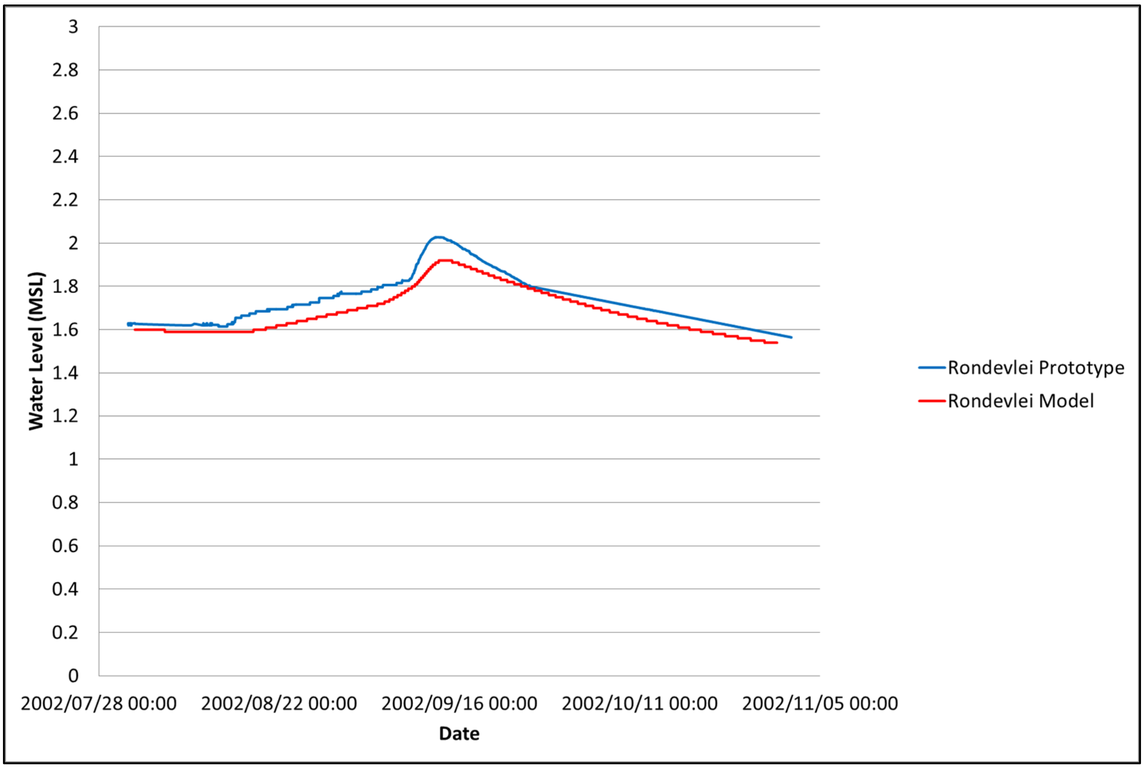


Figure 8-7: Rondevlei direct comparison between prototype and model water levels during the 2002 flood

The rate of storage and the rate of drainage are relatively similar across all water bodies. The greatest exception occurs in Island Lake where the model predicts a drainage rate that is slightly slower than the actual rate of drainage that was recorded during the event.

The most significant outcome of the model is the prediction of peak water levels achieved during the flood event for each water body. Table 8-1 shows a comparison between prototype water levels and model predicted water levels for all water bodies in the system.

Table 8-1: Comparison between peak water levels of the model simulation and the prototype event for all water bodies during 2002 flood event

Water Body	Prototype Peak Water Level (msl)	Model Peak Water Level (msl)	Water Level Difference (m)	Percentage Difference (%)
Touw Estuary	2.50m	2.60m	0.10	4.00
Island Lake	2.35m	2.27m	-0.08	-3.40
Bolangvlei	1.99m	1.97m	-0.02	-1.01
Rondevlei	2.03m	1.92m	-0.11	-5.42

Overall a good result is evident. The greatest difference between the model predicted water levels and the actual recorded water levels occurs in Rondevlei, where the model under-predicts the actual water levels recorded in the lake by 5.42%. The greatest degree of accuracy occurs in Bolangvlei where a difference of 1.01% is apparent between the actual recorded water levels and the model predicted water levels.

8.3. The 2006 flood event

8.3.1. Prototype Event

The 2006 flood event took place during August 2006 and had a maximum peak discharge of 240m^3/s and a total volume of flood water in the Touw River of 15.36 million m^3. Figure 8-8 shows the estimated hydrograph of the flood as well a 100 year flood prediction hydrograph by Görgens in 1979 (Görgens, 1979).

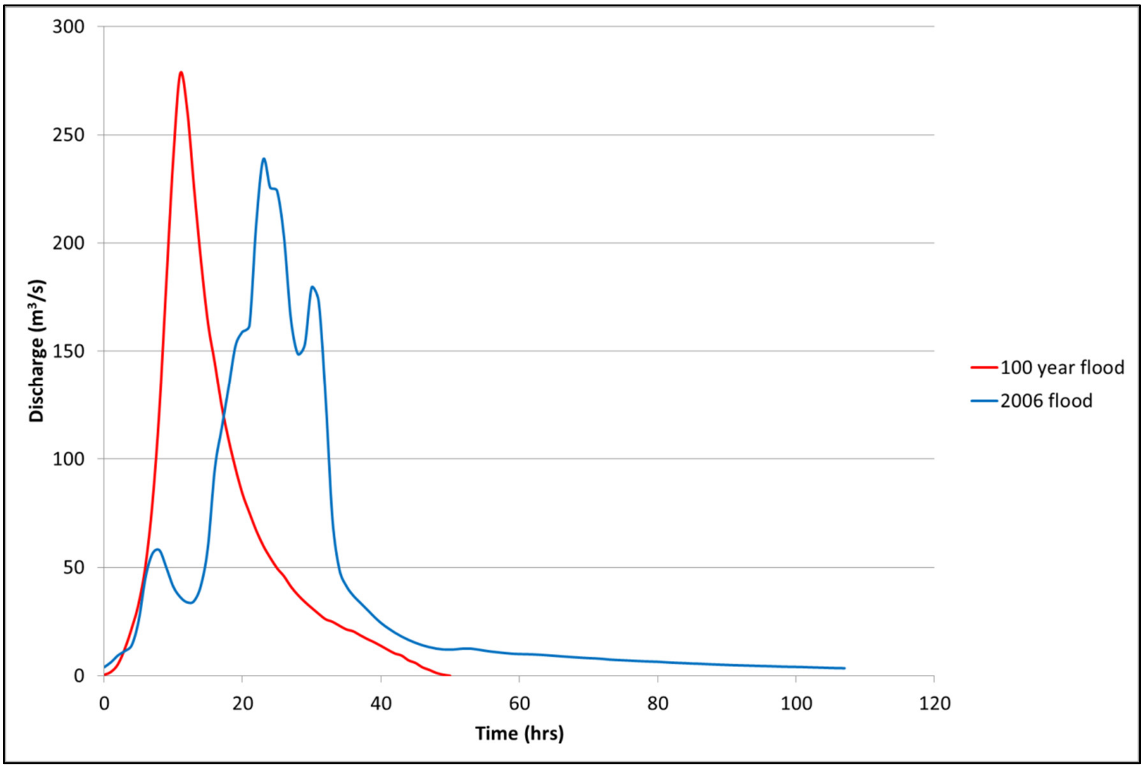


Figure 8-8: Comparison between hydrographs of 2006 flood event and a 100 year flood prediction

Although the estimated peak discharge during the flood is slightly less than the peak discharge predicted for a 100 year flood, the total volume of water associated with the flood is 34% greater than the total amount of water associated with the predicted 100 year flood. Therefore, based on Görgens' predictions in 1979, the return period of this flood is approximately 100 years.

Figure 8-9 shows the actual water levels recorded in each of the following water bodies for the entire simulation period.

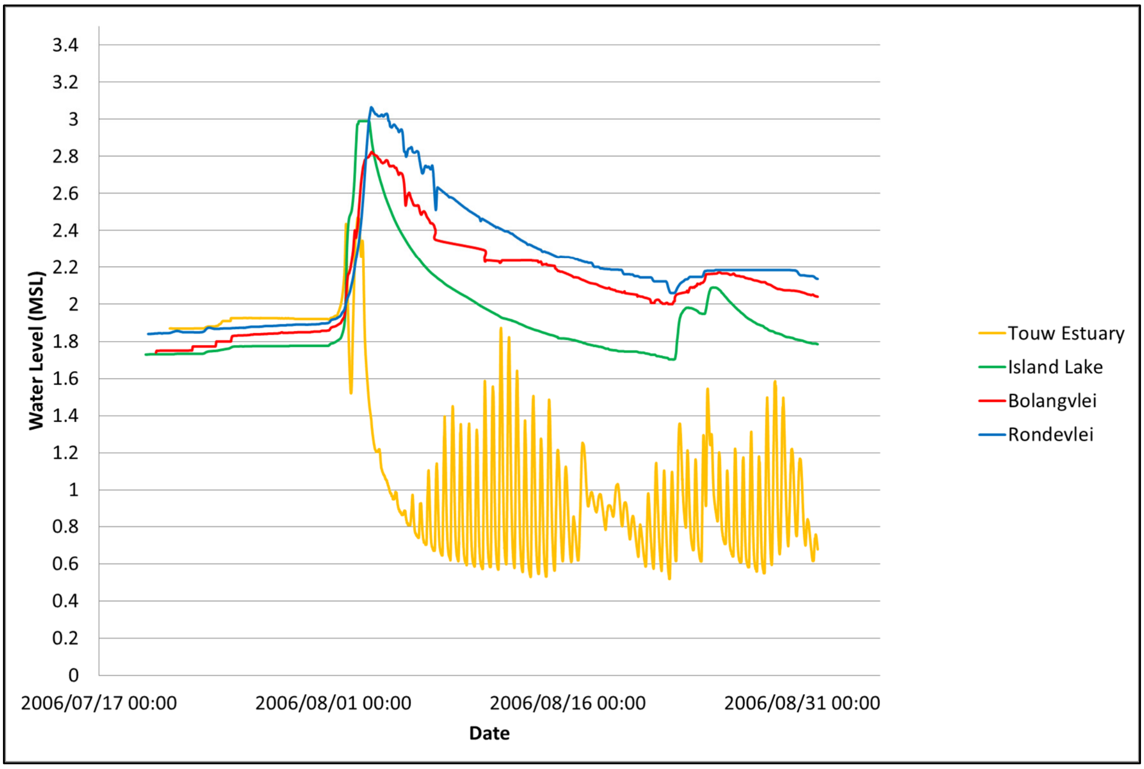


Figure 8-9: Actual measured water levels during 2006 flood for all water bodies

8.3.2. Model Simulation

Without changing any model parameter settings, as calibrated, the results of the model simulation are depicted in Figure 8-10. Figure 8-10 shows the model predicted water levels for the entire simulation period.

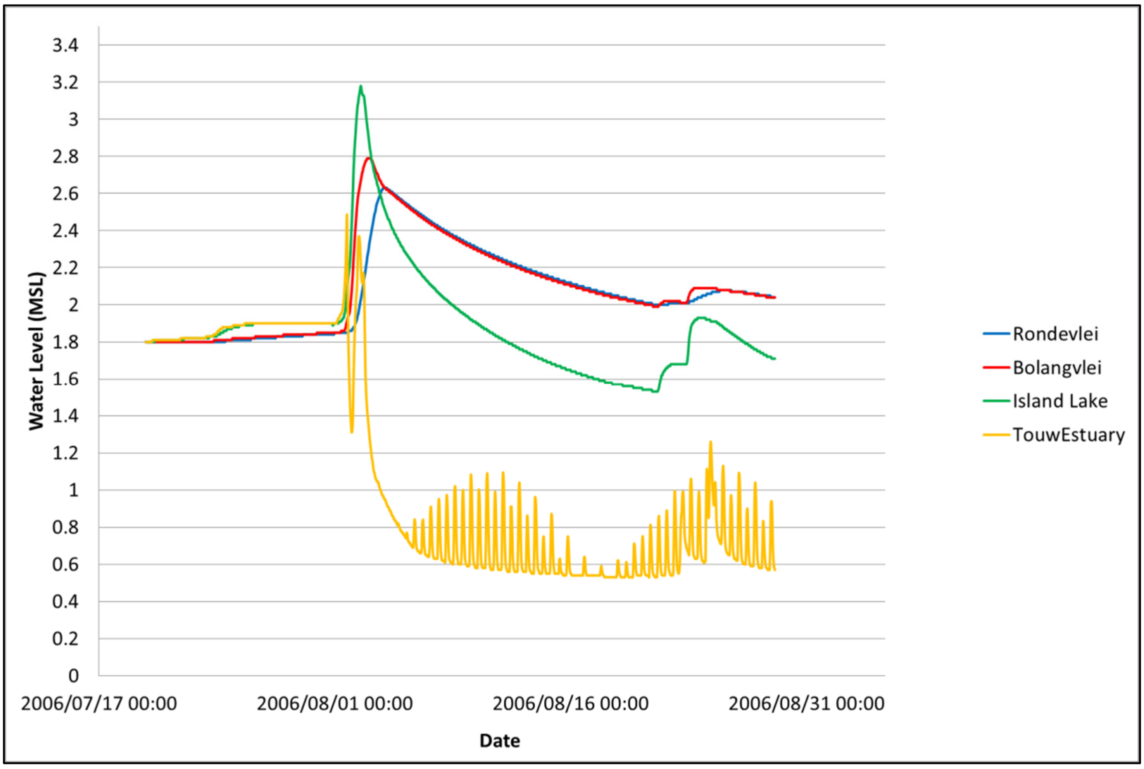


Figure 8-10: Model predicted water levels during 2006 flood for all water bodies

It is difficult to evaluate, based on these two figures alone, whether or not the model provides a good representation of the prototype event. Therefore, the following section provides a direct comparison for each water body between the actual recorded water levels and the model predicted water levels. This provides a more clarifying view regarding the representation of the model.

8.3.3. Direct Comparison

Figures 8-11 to 8-14 represent a direct comparison of water levels between the prototype event and the model predictions. Each figure shows a direct comparison for a different water body and from these figures a good judgement can be concluded on the representation of the model on the actual water levels. Figure 8.11 shows a discrepancy in the tidal water levels after the flood event. This could be a result of the fact that the sand bar at the mouth was in actual fact scoured to a substantial depth below +0 MSL, which may then increase the tidal water levels passing through into the estuary. In the numerical model, one of the limiting factors of an overtopping gate structure at the estuary mouth was that the gate could not drop below +0m MSL. This may have a consequential effect on the peak tidal water levels achieved in the estuary.

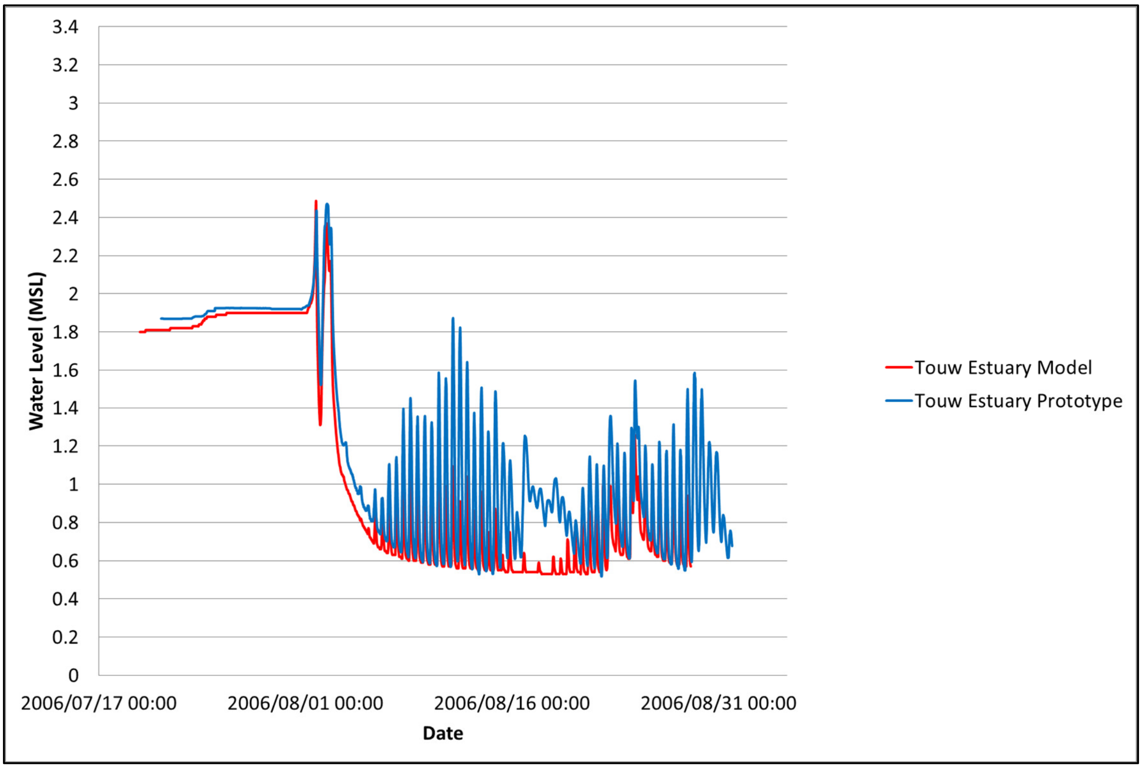


Figure 8-11: Touw Estuary direct comparison between prototype and model water levels during the 2006 flood

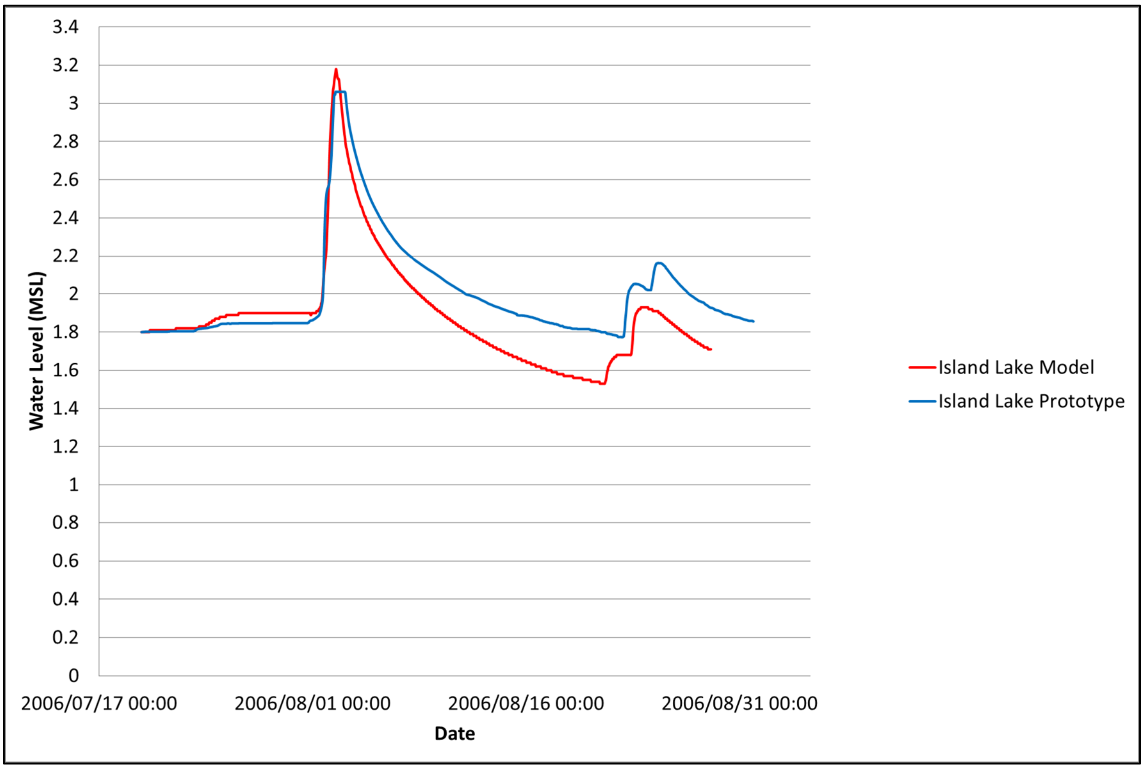


Figure 8-12: Island Lake direct comparison between prototype and model water levels during the 2006 flood

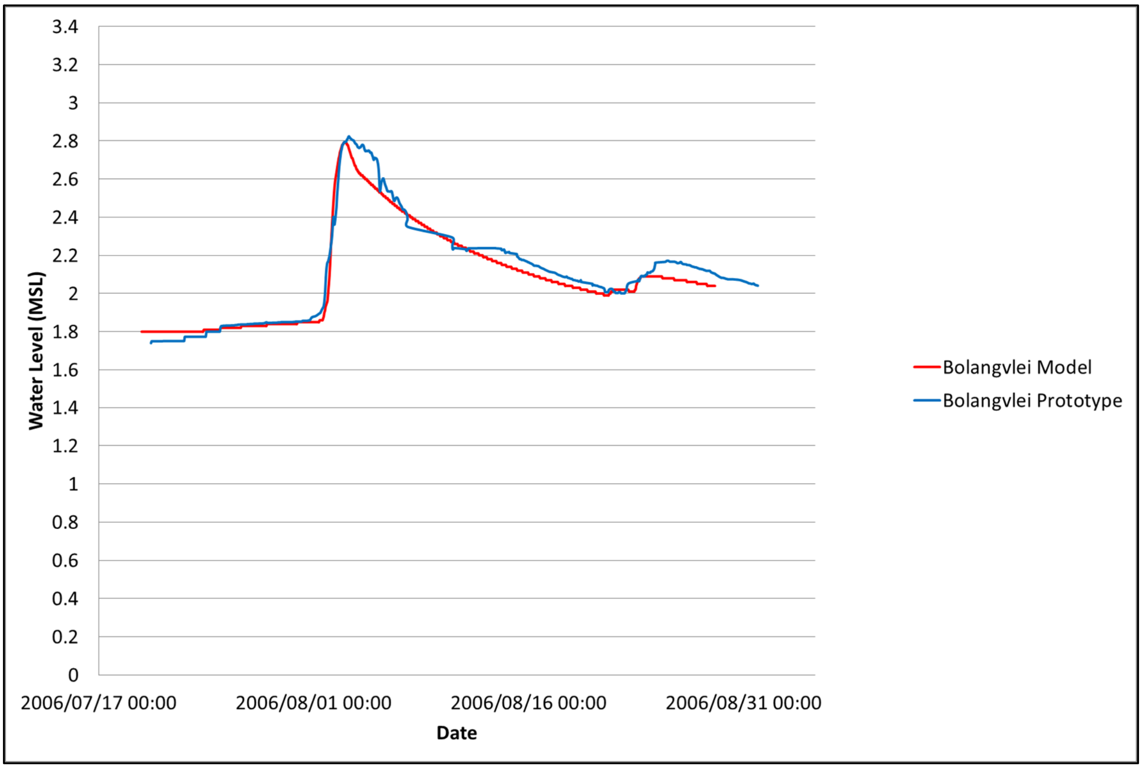


Figure 8-13: Bolangvlei direct comparison between prototype and model water levels during the 2006 flood

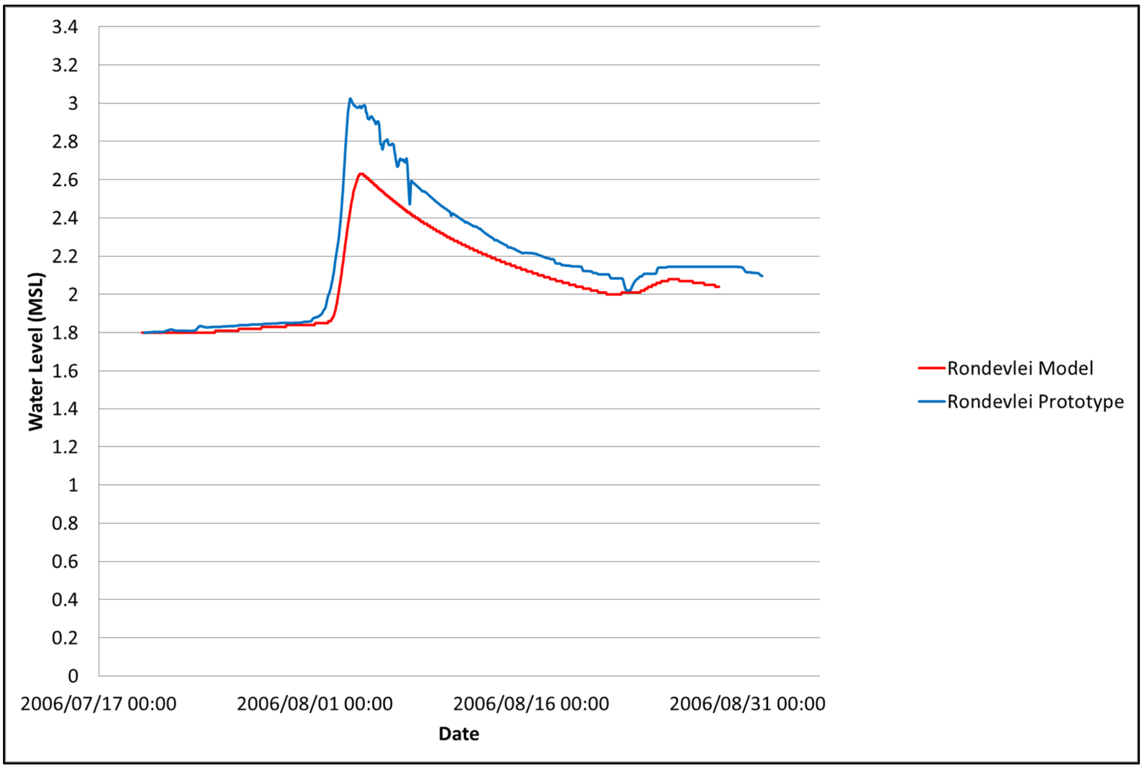


Figure 8-14: Rondevlei direct comparison between prototype and model water levels during the 2006 flood

The average rate of drainage is relatively similar for all lakes with the exception of Island Lake. In Island Lake, the model predicts a slightly faster drainage rate than that of the prototype event. However, this can only be noticed approximately one week after the flood event.

The most significant outcome of the model is the prediction of peak water levels achieved during the flood event for each water body. Table 8-2 shows a comparison between prototype water levels and model predicted water levels for all water bodies in the system.

Table 8-2: Comparison between peak water levels of the model simulation and the prototype event for all water bodies during 2006 flood event

Water Body	Prototype Peak Water Level (msl)	Model Peak Water Level (msl)	Water Level Difference (m)	Percentage Difference (%)
Touw Estuary	2.47m	2.48m	0.01	0.4
Island Lake	2.99m	3.18m	0.19	6.4
Bolangvlei	2.82m	2.79m	-0.03	-1.1
Rondevlei	3.07m	2.63m	-0.44	-14.3

In the case of the 2006 flood event, the greatest degree of differentiation occurs in Rondevlei, where the model under-predicts the water level by 14.3%. The best results can be seen in the Touw estuary where a difference of 0.4% is evident.

8.4. Interpretation of results

When comparing peak water level data across different events, the calibration event as well as the two validation events, a degree of variability is evident across the different events. The predicted outcome is often not the same, which exemplifies the unpredictable nature of the entire system. In order to account for the variation across different events, an average accuracy is determined, based on the accuracy of individual events. Table 8-3 compares the percentage differences between model predicted water levels and the actual recorded water levels. The Table highlights these differences across all three events and the average, across all events, is determined.

Reasons for this inconsistency may be related to a number of factors including rainfall variability between different flood events across the different catchment areas as well as the volumes of debris associated between isolated flood events.

Three different simulations were tested using the same model; the 2007 flood event used in the calibration of the model and the two flood events used during the validation process, namely the 2002 flood event and the 2006 flood event. Based on a comparison between all three flood events, there are many areas of repetition between the different flood events. However, certain behaviours do alter slightly between different flood events, which are most likely due to individual flood factors, some of which are mentioned above, including rainfall variability and debris blockages. In order to truly grasp the accuracy of the model for peak water level prediction, the average percentage difference between prototype water levels and model predicted water levels across all three models was calculated and can be seen in Table 8-3.

Table 8-3: Accuracy evaluation summary for all water bodies

Water Body	2007 Event Percentage Difference (%)	2002 Event Percentage Difference (%)	2006 Event Percentage Difference (%)	Average Percentage Difference (%)
Touw Estuary	-1.4	4.0	0.4	1.0
Island Lake	-8.1	-3.4	6.4	-1.7
Bolangvlei	-2.0	-1.0	-1.1	-1.4
Rondevlei	6.7	-5.4	-14.3	-4.3

8.5. Conclusion

The greatest variation between prototype and model predicted water levels, across all three models, was 14.3%. This occurred in Rondevlei during the 2006 flood study. The most favourable degree of differentiation was 0.4%, which occurred in the Touw estuary during the same flood event. This is an acceptable range of variation and is safe to assume that the model has a maximum tolerance of approximately 15%.

When comparing the average percentage difference across all three models, illustrated in Table 8-3, the results indicate an accuracy of less than 5% for all water bodies. A satisfactory hydrodynamic model has, therefore, been created and is capable of predicting different scenarios with theoretical river discharges.

9. MODEL TESTS WITH THEORETICAL RIVER DISCHARGES

9.1. Introduction

Now that a calibrated, validated and reliable model had been established, this model was consequently used in order to predict the outcome of certain hypothetical flood events. The aim of this investigation is to provide a preliminary assessment of the flood water level changes in the estuary and lakes under a number of different mouth management situations using a simplified numerical modelling approach. In order to meet the aim of the investigation, it was necessary to establish water level predictions for all water bodies in the Wilderness system based on a number of influencing variables. The resulting water level achieved in a water body after a flood event primarily rests on three variables viz. the size of the flood, the initial water level of the water body and the level of the sand bar immediately before the flood. These variables are discussed further in the following section.

9.2. Input Parameters and Modelling Process

9.2.1. Flood Events

Five flood events were used during the production runs and were differentiated by the return period of the flood, namely a 5 year, 10 year, 20 year, 50 year and 100 year flood. A hydrological study was not performed on the Wilderness area and therefore flood hydrographs were based purely on previous studies. Although more recent hydrographs (performed in 1994), do exist, it was found that the earlier predictions provide a more realistic representation of recent prototype flood events. Therefore, theoretical river discharges were based primarily on the unit hydrograph predictions performed by Görgens in 1979 (Görgens, 1979). The only alteration made to the 1979 hydrograph predictions was an increase in peak flow in the Duiwe River, which was found to be necessary during the calibration of the model. The Duiwe River inflow was increased to approximately 40% of the Touw River inflow. Whereas in the original hydrographs by Görgens, the inflow of the Duiwe River was approximately 26% of the Touw River inflow. This is discussed in detail in Chapter 6.4.2. Figures 9-1 to 9-3 show the hydrographs used during the production runs for the Touw River, Duiwe River and Langvlei Spruit respectively.

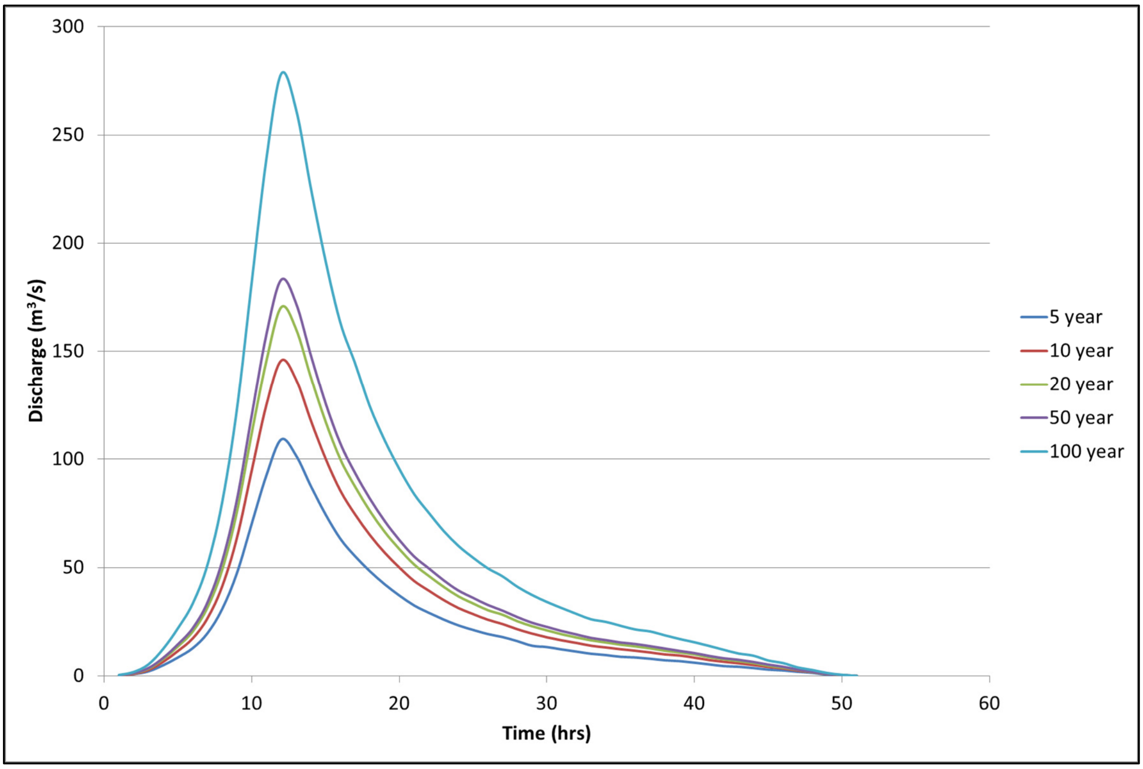


Figure 9-1: Flood Hydrographs for Touw River for different flood events

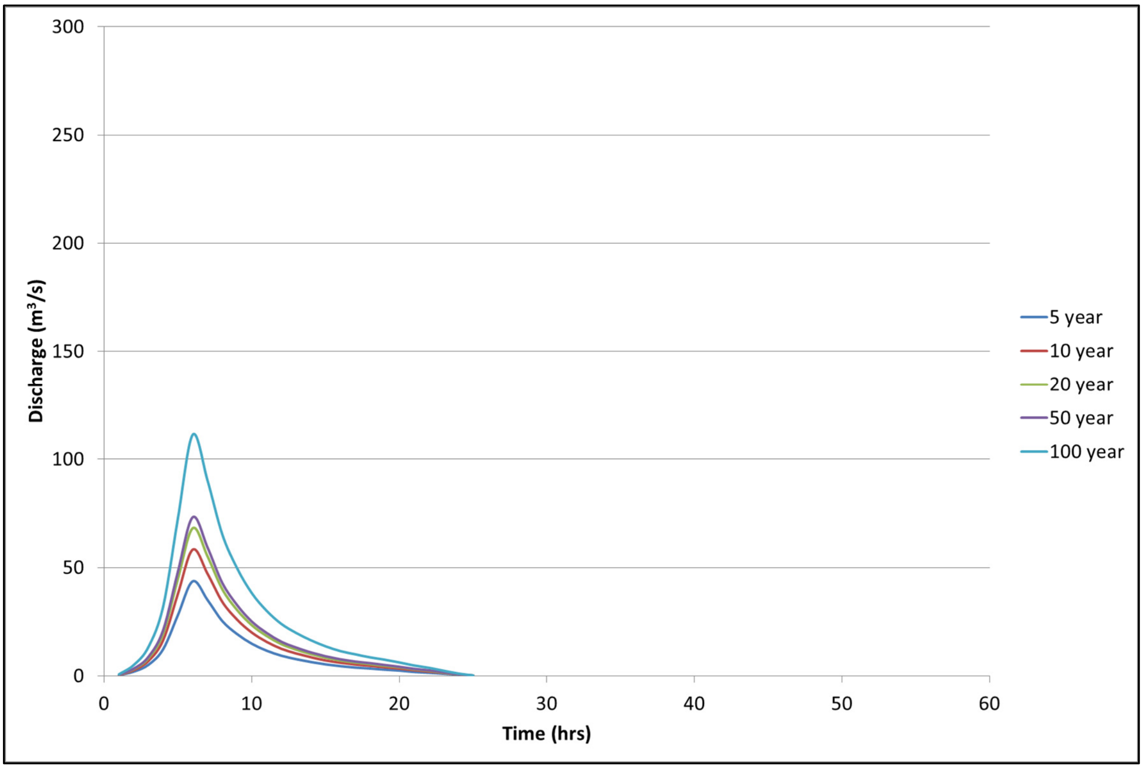


Figure 9-2: Flood Hydrographs for Duiwe River for different flood events

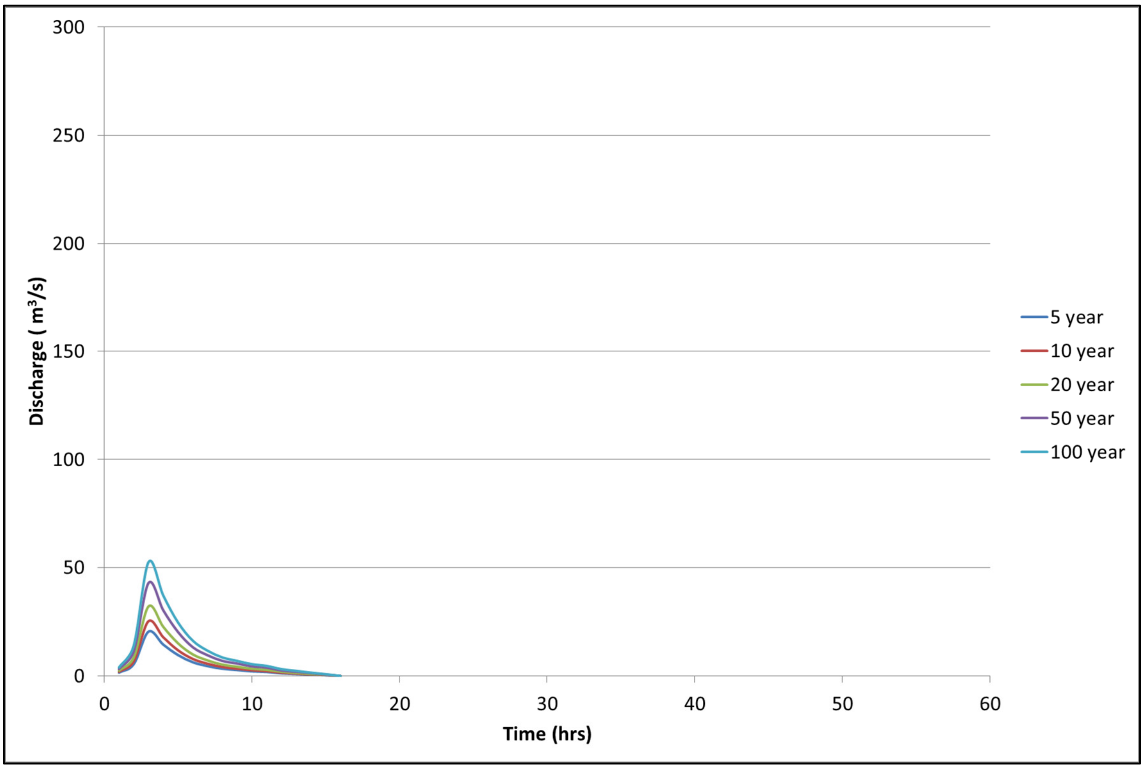


Figure 9-3: Flood hydrographs for Langvlei Spruit for different flood events

Figures 9-1 to 9-3 show the river discharge over time. From these figures it is easy to evaluate the peak discharge during a flood and the total duration of the flood for each water source. However, these figures do not provide a true understanding of the total volume of water associated with each flood event. Table 9-1 indicates the total volume of water associated with each flood event for the different water sources.

Table 9-1: Volumes of water associated with each water source for different flood events

	Touw River (1000 m^3)	Duiwe River (1000 m^3)	Langvlei Spruit (1000 m^3)
5 year	4455	894	270
10 year	6007	1204	334
20 year	7045	1412	424
50 year	7572	1518	567
100 year	11477	2299	694

9.2.2. Initial Water Level

The initial water level in each water body has a resulting effect on the peak water level observed during a flood. It is, therefore, important that different initial water levels are tested which cover all potential scenarios. Initial water levels are assumed to be uniform across the entire system and are based on historic water levels before flood events. The initial water levels range between the values of +1.2m MSL and +2.2m MSL. A separate case for a tidal condition was also performed where the water level in the Touw estuary was determined by the tidal boundary and the water level in the lakes all had an initial level of +1.2m MSL.

9.2.3. Initial Sand Bar Level

The initial sand bar height also has a resulting effect on the peak water levels achieved in the water system. During a flood event, when the estuary mouth is initially closed, the water in the Touw estuary rises to a level above the sand bar before the sand bar begins to scour and the estuary mouth opens to the ocean. Although the initial height of the sand bar primarily affects the peak water levels observed in the Touw estuary, this may also have a consequential effect on the peak water levels of the coastal lakes. Simulations were, therefore, performed using different sand bar heights ranging from +0m MSL, which represent an open estuary mouth to an extreme case of +3m MSL. The current management plan of the sand bar stipulates that the sand bar levels in front of the estuary be maintained at a level between +2.1m and +2.4m and, therefore, a case of +3m MSL would only arise if the current management plan was not implemented or insufficiently executed. Sand bar heights of +0m, +1.2m, +1.4m, +1.6m, +1.8m +2.0m, +2.2m, +2.4m and +3.0m MSL were tested.

9.2.4. Modelling Process

Due to the number of influential variables mentioned above, there are a large number of modelling scenarios which can be formulated based on these variables. Table 9-2 summarizes the modelling scenarios involved for one flood event. The grey blocks indicate scenarios which are unrealistic and will not occur naturally and are, therefore, excluded from the model simulations. In total there are 34 different scenarios for one flood event. Due to the fact that there are five flood events, the total number of different scenarios for all flood events is 170.

Table 9-2: Different types of modelling scenarios for one flood event

		Initial Water Level (MSL)						
		Tidal	**1.2**	**1.4**	**1.6**	**1.8**	**2**	**2.2**
Initial Sand Bar Level (MSL)	**0**	✓						
	1.2		✓					
	1.4		✓	✓				
	1.6		✓	✓	✓			
	1.8		✓	✓	✓	✓		
	2		✓	✓	✓	✓	✓	
	2.2		✓	✓	✓	✓	✓	✓
	2.4		✓	✓	✓	✓	✓	✓
	3		✓	✓	✓	✓	✓	✓

9.3. Hydrodynamic Results

The following results show the model predicted water levels for all flood events under the initial conditions mentioned above. From the results, certain trends are evident such as the fact that peak flood water levels in the Touw estuary are more dependent on the level of the sand bar at the estuary mouth, than the initial water level within the estuary. The opposite is true for the coastal lakes. Peak flood water levels in the lakes are more dependent on the initial water level in the lake rather than the level of the sand bar at the estuary mouth. This is illustrated in the following section for a 5 year flood. In order to avoid repetition, individual results for the additional flood events can be seen in Appendix F.

9.3.1. 5 year flood

Figure 9-4 shows the peak water level predicted by the model for the Touw Estuary. It must be taken into account that the location of these predicted peak water levels is at the location of the existing water level recorder in the Touw estuary, at the old railway bridge. This is also a good intermediate position within the estuary, therefore indicating a good approximation of the average peak water levels achieved across the entire length of the Touw estuary.

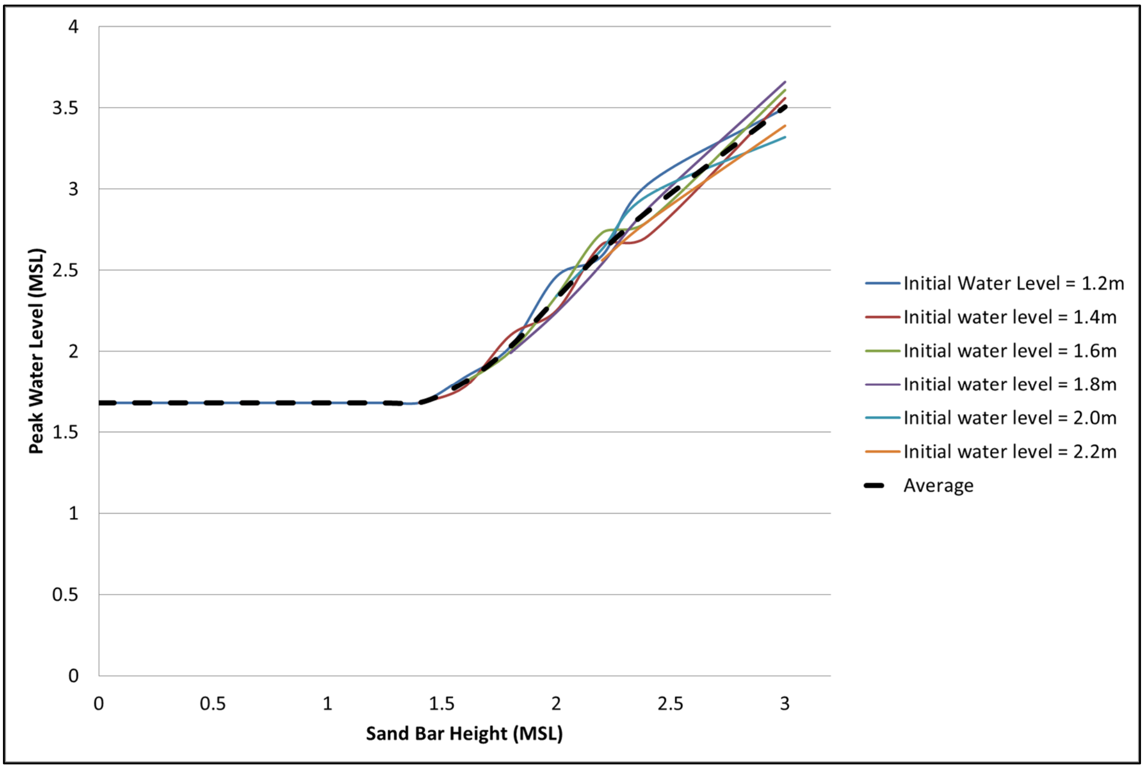


Figure 9-4: Touw Estuary peak water levels after 5yr flood for different sand bar heights and initial water levels

From Figure 9-4 it can be concluded that the peak flood water levels in the Touw estuary are not dependent on the initial water level in the estuary. It can also be seen that the peak flood water levels in the Touw estuary are also not dependant on the sand bar height if the sand bar elevation is below a certain level. In the case of a 5 year flood, this sand bar elevation value is approximately +1.4m MSL which correlates to a minimum peak water level of +1.68m MSL. However, for sand bar levels greater than +1.4m MSL, the peak water levels in the Touw estuary are dependent on the initial level of the sand bar at the estuary mouth and a linear relationship is evident between peak water level and sand bar elevation. As the initial sand bar elevation increases, the peak water levels in the Touw River increase at a linear rate. An average line is also plotted on the graph, which illustrates the best fit line between the data.

It was mentioned earlier that location of these water level predictions were taken at the position of the existing water level recorder which is situated approximately midway in the estuary at the old railway bridge. Although this provides a good estimation to the average peak water levels achieved across the length of the estuary, the peak water levels may differ significantly depending on the exact location within the estuary. It was found that higher peak water levels were evident in the upper reaches of the estuary, near Ebb & Flow, while lower peak water levels were evident towards the estuary mouth. Figure 9-5 illustrates this point. The figure shows a longitudinal section of the Touw River estuary from the intersection point between the Touw River and the Serpentine River to the ocean boundary. The peak water levels during a five year flood with an initial sand bar elevation of 0.0m (open estuary) are displayed in the Figure 9.5.

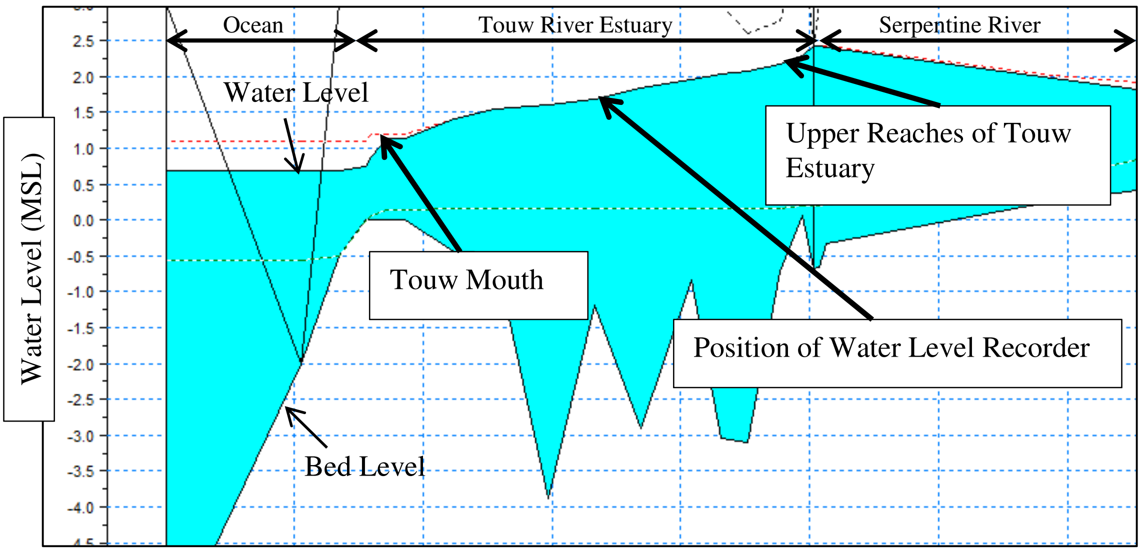


Figure 9-5: Snapshot of peak water level during a flood across the Touw estuary (Sand bar = 0.0m)

From Figure 9-5 it is evident that higher water levels are predicted for the upper reaches of the Touw estuary, while lower water levels are predicted in areas near the river mouth. The hydraulic gradient between peak water levels in the upper reaches of the estuary and peak water levels at the estuary mouth is minimized as the height of the sand bar increases. Figure 9-6 shows the peak water levels during a five year flood with a sand bar elevation of +3.0m MSL.

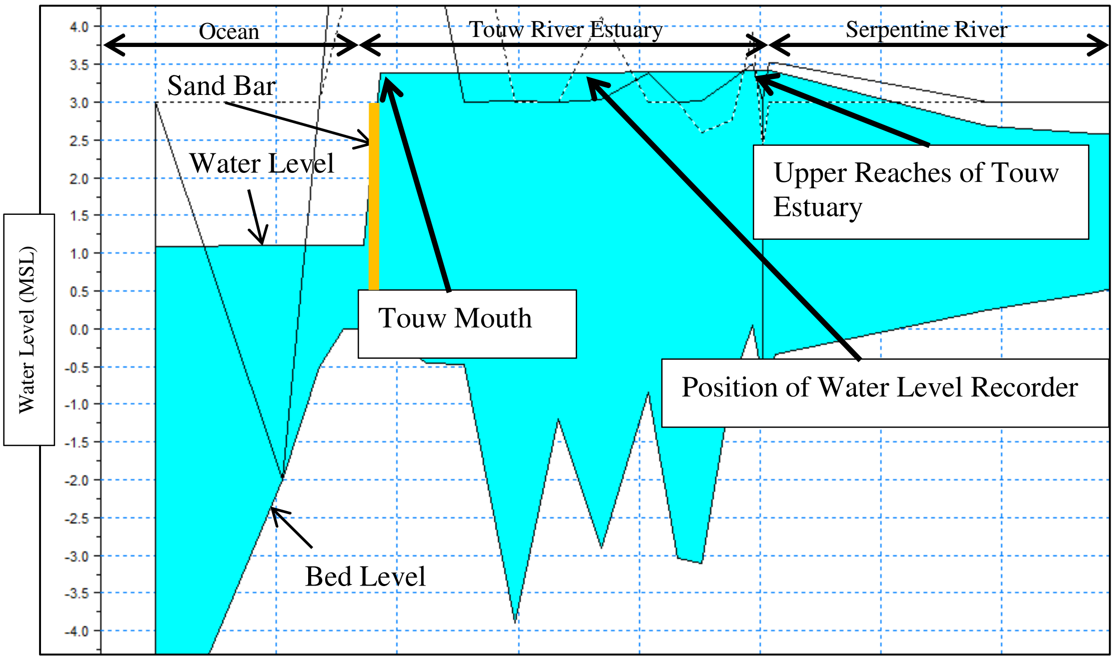


Figure 9-6: Snapshot of peak water level during a flood across the Touw estuary (Sand bar = 3.0m)

The peak water levels for the lakes on the other hand are more dependent on the initial water level, rather than the initial level of the sand bar during a flood event. It appears that as the distance between the sand bar and the water body increases, the initial sand bar elevation plays less of a pivotal role in the resulting peak water levels during a flood event. Figures 9-7 to 9-9 show the peak water level predictions for the coastal lakes during a five year flood, given different initial conditions. It is important to note that the dependant variable, on the horizontal axis, in the following figures is "initial water level" and not "initial sand bar level".

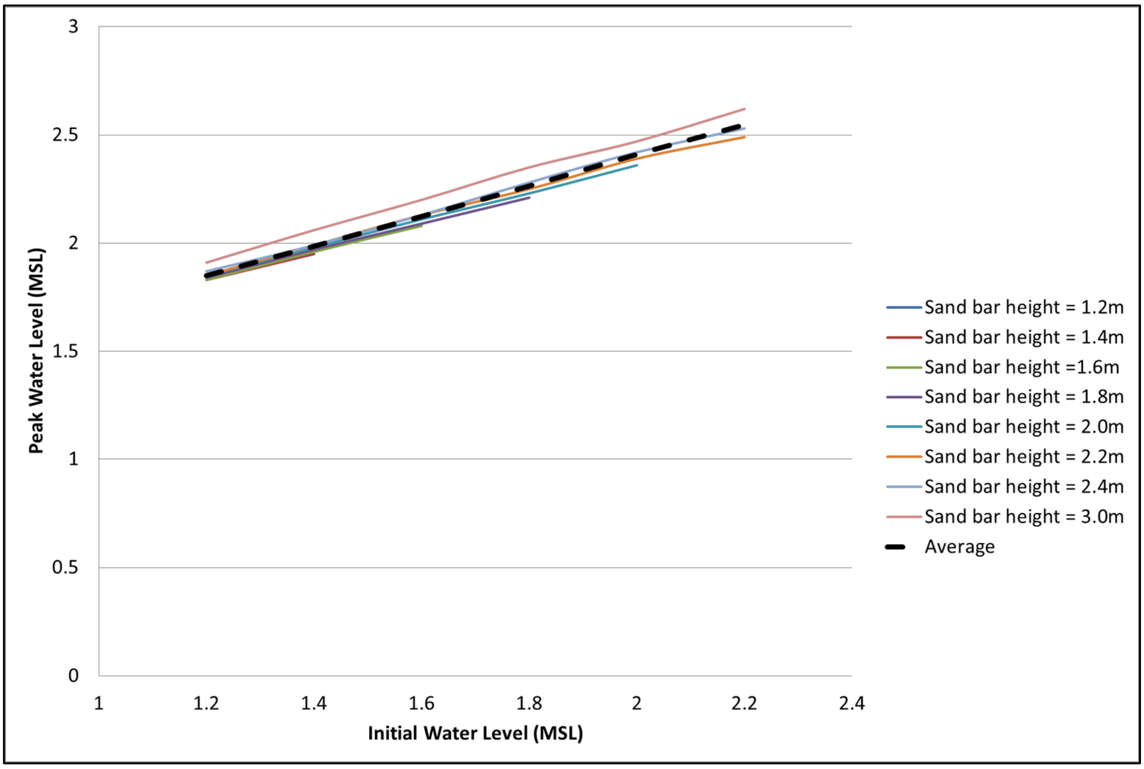


Figure 9-7: Island Lake peak water levels after 5yr flood for different sand bar heights and initial water levels

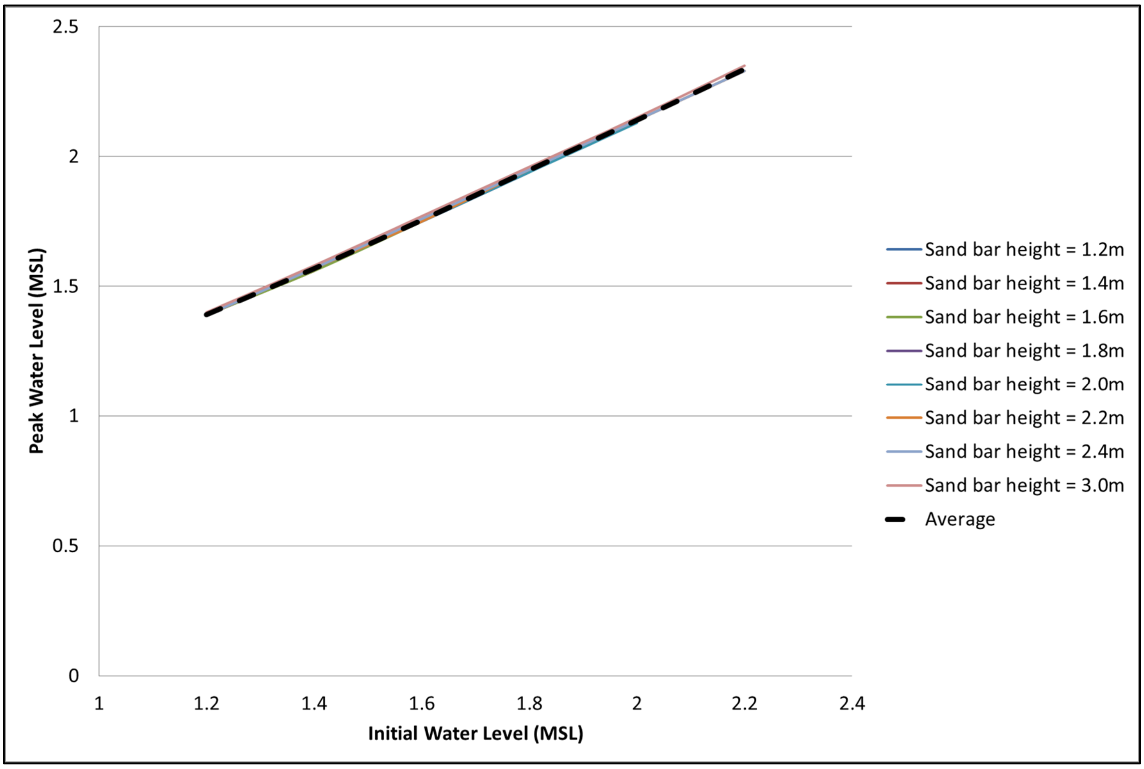


Figure 9-8: Bolangvlei peak water levels after 5yr flood for different sand bar heights and initial water levels

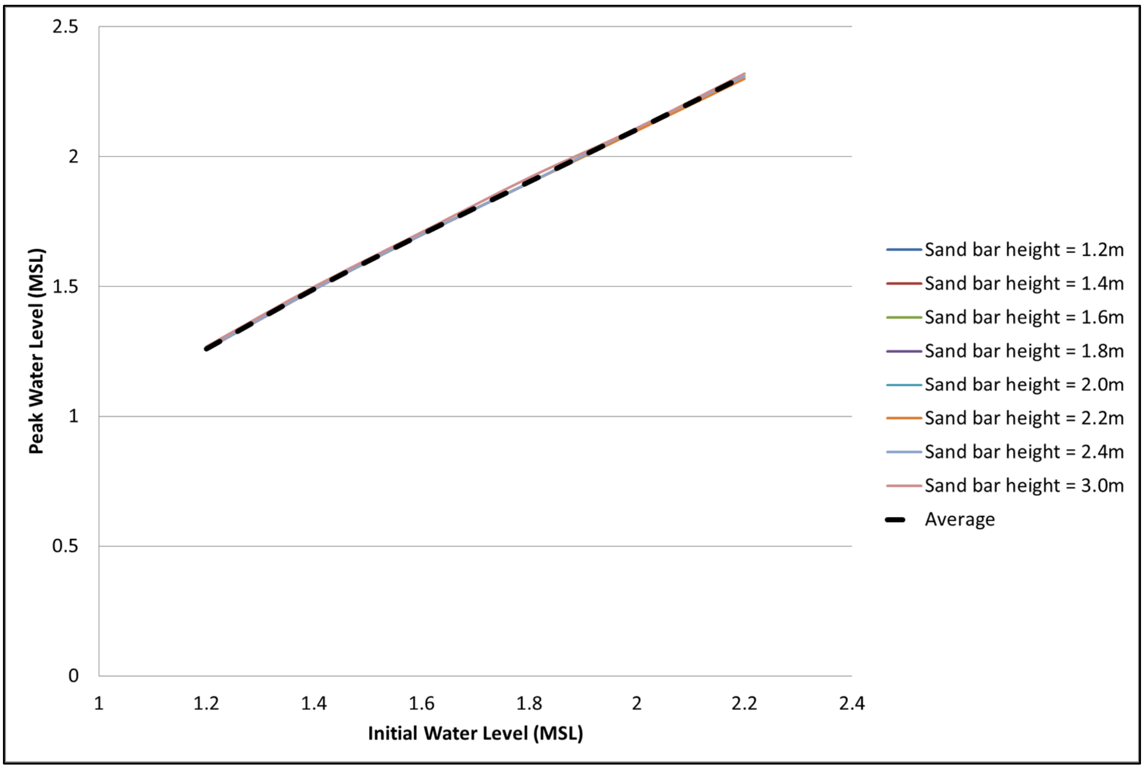


Figure 9-9: Rondevlei peak water levels after 5yr flood for different sand bar heights and initial water levels

From Figures 9-7 to 9-9, it is evident that the peak water levels in the lakes for a specific flood event are predominantly dependent on the initial water level within the lake rather than the initial sand bar height. An accurate estimation of the peak water levels in the lakes can be made by applying an average line to the data, illustrated in each figure as the dotted black line. The greatest deviation from the average occurs in Island Lake where a partial dependency on the initial sand bar height is evident. However, a good approximation is still evident using a single average line. The relationship between the initial water level and the resulting peak water level during a flood in each lake is linear, indicating that a higher initial water level will result in a higher peak water level.

The results for the other flood events show very similar trends and can be seen in Appendix F.

9.3.2. Combined Results

As mentioned previously, the peak water levels achieved during a flood event are dependent on three main variables; the size of the flood, the initial elevation of the sand bar at the estuary mouth and the initial water level within each water body. Based on the results above, it can be concluded that the peak water level in each water body is only really dependent on two of the three variables. Peak water levels in the Touw estuary are dependent on the size of the flood and the initial level of the sand bar at the estuary mouth. Whereas, peak water levels within the lakes are dependent on the size of the flood and the initial water level of the lakes. Therefore, from this information, combined graphs can be plotted for the different water bodies which determine the peak water level based on the flood event and the other dependent variable. This was performed by taking the average trend line for each water body in each flood event, illustrated by the black dashed line, and combining them in order to produce a single graph for each water body. Figures 9-10 to 9-13 illustrate the final results of the hydrodynamic water level predictions for each water body in the Touw River estuary and Wilderness coastal lakes.

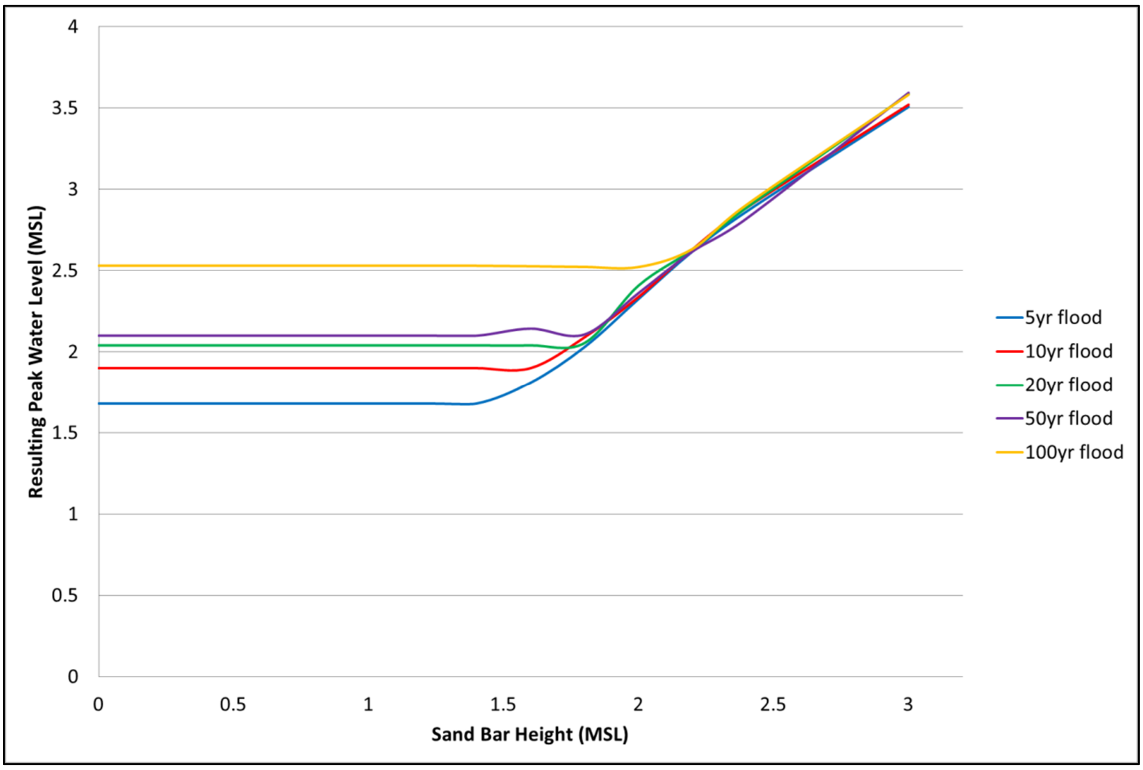


Figure 9-10: Touw estuary peak water level predictions for different initial sand bar levels and for different flood events

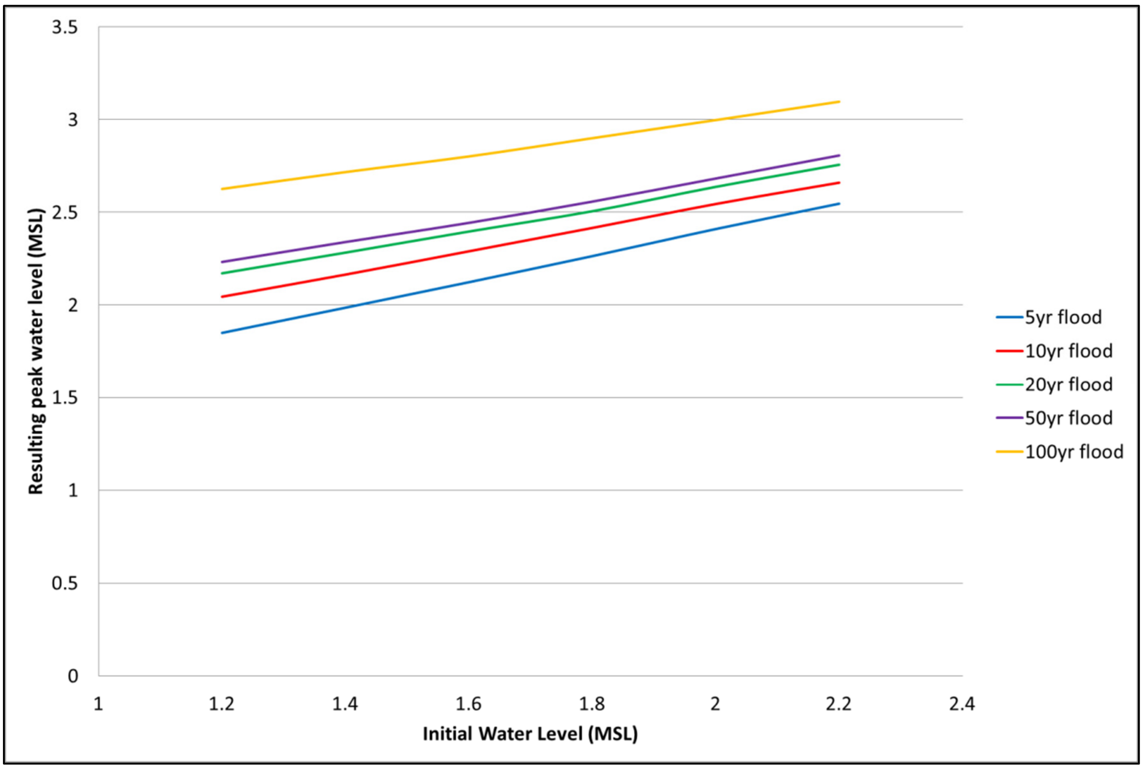


Figure 9-11: Island Lake peak water level predictions for different initial water levels and for different flood events

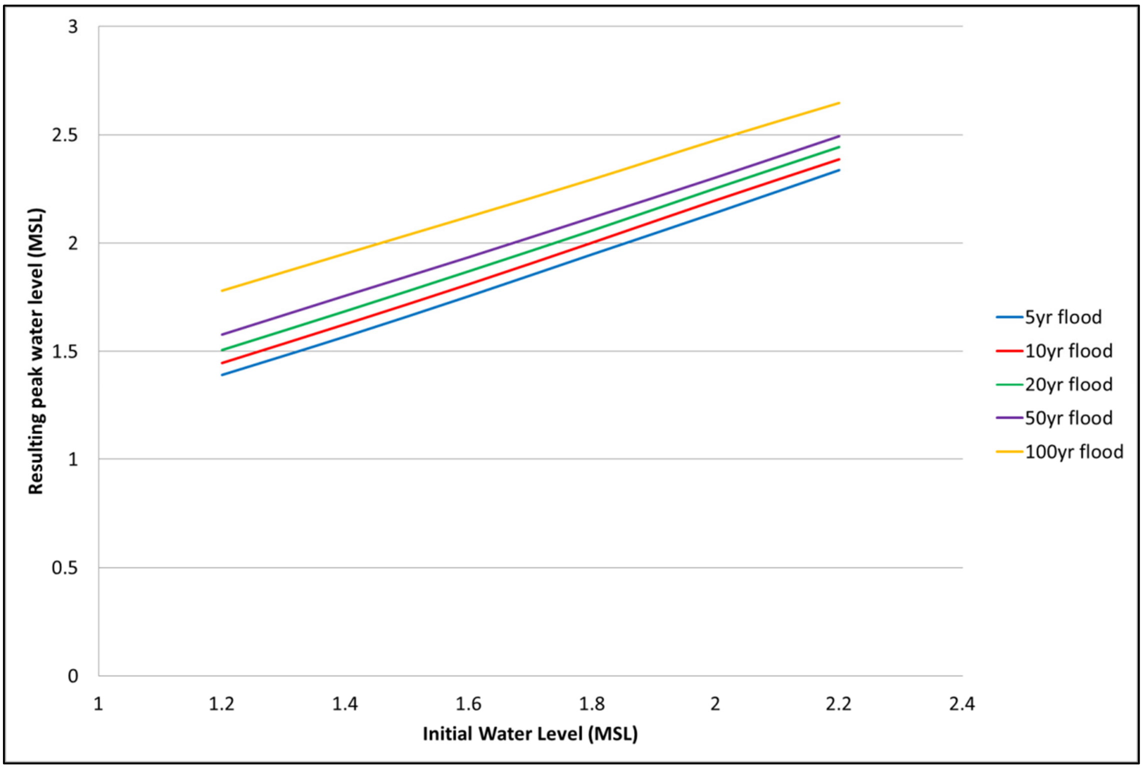


Figure 9-12: Bolangvlei peak water level predictions for different initial water levels and for different flood events

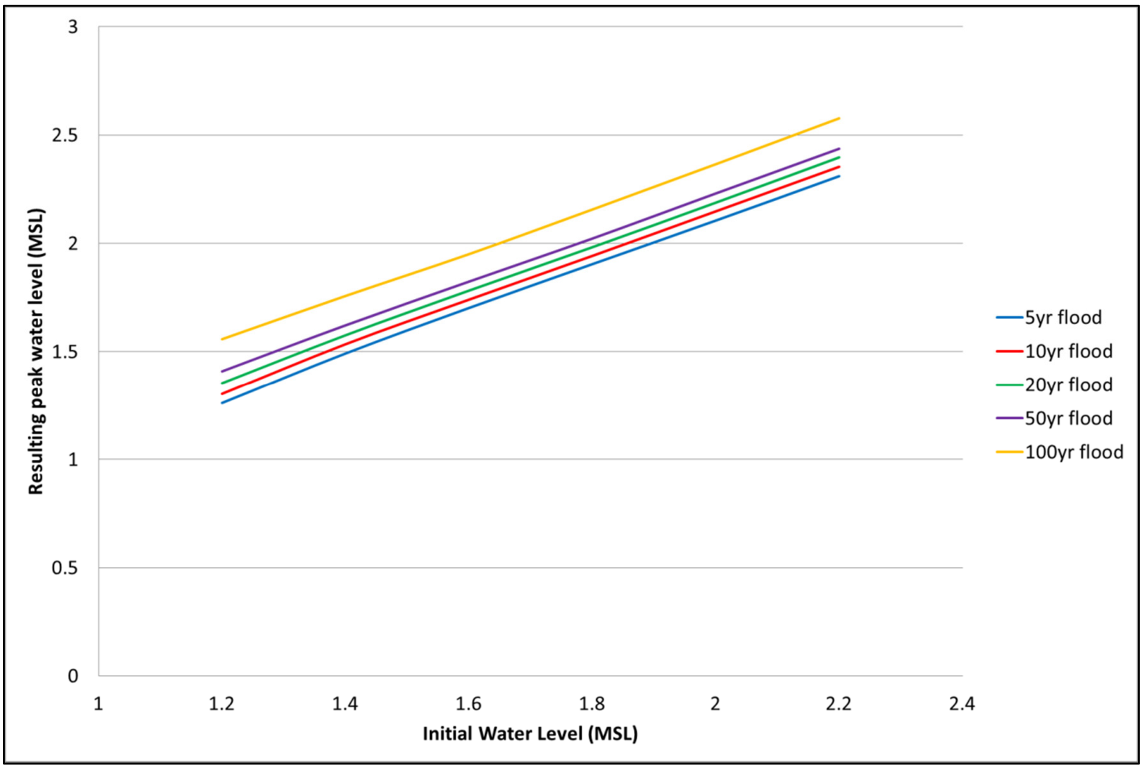


Figure 9-13: Rondevlei peak water level predictions for different initial water levels and for different flood events

9.4. Interpretation of Results

9.4.1. Touw estuary

Figure 9-10 portrays a graph which determines the resulting peak water levels in the Wilderness estuary based on different flood events and initial sand bar elevations. It has previously been established that the resulting peak water levels of the estuary are not dependent on the initial water level within the estuary, but rather the initial sand bar elevation and the size of the flood event. It can be seen from Figure 9-10, that for an open estuary and for lower sand bar elevations a minimum constant water level is evident for each flood event. The minimum peak water level associated with a 5 year flood is +1.68m MSL and this value holds true for all sand bar elevations from a completely open estuary to a maximum sand bar elevation of approximately +1.4m MSL. On the opposite end of the scale, the minimum peak water level associated with a 100 year flood is +2.53m MSL and this value holds true for all sand bar elevations lower than +2.1m MSL including an open estuary mouth. This sand bar elevation, below which peak water levels of the same flood remain constant, is referred to in this study as the threshold sand bar elevation. These points for different flood events are illustrated in Figure 9-14.

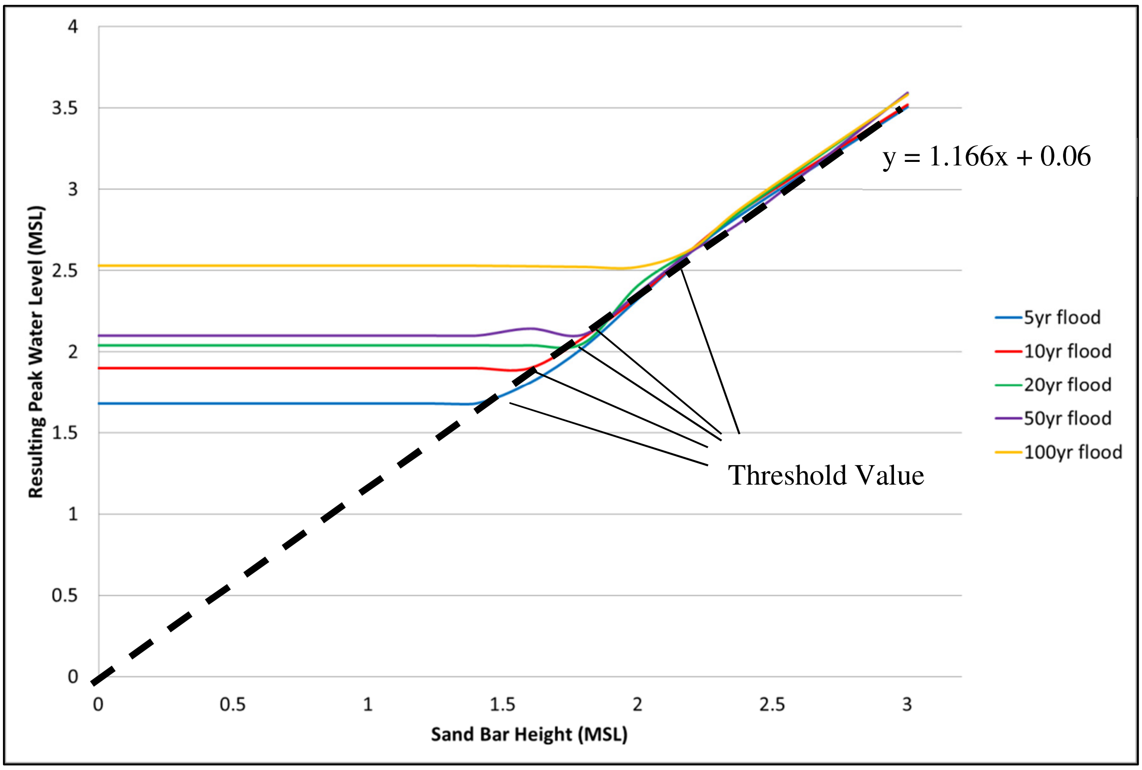


Figure 9-14: Analysis of Touw peak water level predictions graph

Once the sand bar elevation rises above this threshold value, the peak water level predictions for different flood events converge and follow a single linear prediction. This can be seen on Figure 9-14 by the convergence of lines as the sand bar elevation rises above the threshold value for each flood event. For higher sand bar elevations, above +2.1m MSL, the resulting peak water levels in the Touw estuary are no longer dependent on the size of the flood and are only dependent on the initial elevation of the sand bar at the estuary mouth. The equation of this linear prediction line is approximately equal to $y = 1.166x + 0.06$, where y is the resulting peak water level in the Touw estuary and x is the initial sand bar elevation. As a hypothetical scenario, for an initial sand bar elevation of +3m MSL, it is predicted that average peak water levels of +3.56m MSL will be evident in the Touw estuary for all flood events. This is also confirmed by the equation:

$$y = 1.166x + 0.06$$
$$y = 1.166(3) + 0.06$$
$$y = +3.56m \text{ MSL}$$

9.4.2. Coastal lakes

A slightly different scenario is evident for the coastal lakes. It has already been established that the resulting peak water levels in the coastal lakes are far more dependent on the initial water level of the lake rather than the initial elevation of the sand bar at the estuary mouth. For all lakes, a linear relationship is evident between the initial water level of the lake and the resulting peak water level, where a greater initial water level results in a greater peak water level. The peak water levels in the lakes are also dependent on the size of the flood event where a greater flood event will result in higher peak water levels.

The sources of inflow to the coastal lakes are from the Duiwe River and Langvlei Spruit which flow directly into Island Lake and Bolangvlei respectively. Due to the fact that the Duiwe River has a slightly greater discharge than the Langvlei Spruit, higher peak water levels are evident in this lake, under the same conditions. There is no external water source which flows into Rondevlei and, therefore, Rondevlei receives water via an interconnecting channel between Bolangvlei and Rondevlei. Due to this reason, there is very little difference between the predicted peak water levels of Bolangvlei and Rondevlei.

For the tests performed, the minimum peak water levels in the three lakes are associated with a five year flood and an initial water level of +1.2m MSL in the lakes. The corresponding peak water levels under these conditions are +1.85m, +1.39m and +1.26m MSL for Island

Lake, Bolangvlei and Rondevlei respectively. The maximum water levels in the lakes are associated with a hundred year flood and an initial water level of +2.2m MSL. The corresponding peak water levels under these conditions are +3.10m, +2.65m and +2.58m MSL for Island Lake, Bolangvlei and Rondevlei respectively.

9.4.3. CSIR Study Comparison

Table 9-3 illustrates a direct comparison of peak water level predictions between this study (abbreviated as 2013 in the Table) and the study performed by the CSIR in 1982 (abbreviated as 1982 in the Table). The Table shows a comparison for an initial sand bar height of +2.2m MSL.

Table 9-3: Comparison of peak water levels between this study and 1982 predictions for an initial sand bar height of +2.2m MSL

Return Period (Years)	Initial Water Level (+m MSL)	Flood Peak Levels (+m MSL)							
		Estuary		Island Lake		Bolangvlei		Rondevlei	
		1982	2013	1982	2013	1982	2013	1982	2013
5	1.4	2.45	2.66	1.76	1.99	1.51	1.57	1.51	1.49
	1.6	2.42	2.73	1.98	2.13	1.70	1.75	1.70	1.70
	1.8	2.36	2.54	2.12	2.25	1.89	1.95	1.89	1.9
10	1.4	2.52	2.55	1.93	2.16	1.54	1.62	1.54	1.53
	1.6	2.5	2.63	2.09	2.29	1.73	1.81	1.73	1.74
	1.8	2.49	2.71	2.23	2.41	1.92	2.0	1.92	1.94
20	1.4	2.59	2.68	2.03	2.28	1.57	1.69	1.57	1.58
	1.6	2.58	2.75	2.17	2.4	1.76	1.87	1.76	1.78
	1.8	2.57	2.55	2.31	2.5	1.96	2.06	1.96	1.98
50	1.4	2.65	2.73	2.14	2.34	1.61	1.76	1.61	1.62
	1.6	2.64	2.49	2.27	2.44	1.80	1.93	1.80	1.82
	1.8	2.62	2.58	2.39	2.55	2.01	2.12	2.01	2.02
100	1.4	2.94	2.69	2.39	2.72	1.65	1.95	1.65	1.76
	1.6	2.93	2.76	2.5	2.81	1.87	2.12	1.87	1.95
	1.8	2.91	2.54	2.59	2.89	2.09	2.29	2.09	2.15

Table 9-4 illustrates the percentage difference between this study and the 1982 study performed by the CSIR. A green shaded box illustrates a greater water level prediction in this study compared to the 1982 study, whereas a pink shaded box illustrates that predicted water levels in this study are lower than the predicted water levels in the 1982 study. The magnitude of the difference is also illustrated by the relative shading, where a darker shade illustrates a greater differential.

Table 9-4: Percentage differences in peak water level predictions between this study and 1982 study

Reurn Period (Years)	Initial Water Level (+m MSL)	Percentage Difference (%)			
		Estuary	Island Lake	Bolangvlei	Rondevlei
5	1.4	8.6	13.1	4.0	-1.3
	1.6	12.8	7.6	2.9	0.0
	1.8	7.6	6.1	3.2	0.5
10	1.4	1.2	11.9	5.2	-0.6
	1.6	5.2	9.6	4.6	0.6
	1.8	8.8	8.1	4.2	1.0
20	1.4	3.5	12.3	7.6	0.6
	1.6	6.6	10.6	6.3	1.1
	1.8	-0.8	8.2	5.1	1.0
50	1.4	3.0	9.3	9.3	0.6
	1.6	-5.7	7.5	7.2	1.1
	1.8	-1.5	6.7	5.5	0.5
100	1.4	-8.5	13.8	18.2	6.7
	1.6	-5.8	12.4	13.4	4.3
	1.8	-12.7	11.6	9.6	2.9

By analysing Table 9-3 and Table 9-4, there are definite similarities and differences evident between the 1982 and 2013 peak water level predictions. An initial general impression suggests that water level predictions between the two studies are relatively similar. Based on the sample set comparison above, a maximum difference of 18.2% between the two studies is evident, thus indicating fairly comparable results. A general difference between the two studies is that higher peak water levels are predicted in this study compared to the 1982 study. This study could therefore be considered more conservative than the earlier study performed in 1982.

A deeper understanding is also required in order to analyse differences and similarities on a more intricate level. Beginning with the Touw estuary, it appears that as the initial water level in the Touw estuary increases, the resulting peak water level in the 1982 predictions decreases. However, in this study it was found that peak water levels in the Touw estuary were not dependent on the initial water level of the estuary. Furthermore, with a sand bar height of +2.2m MSL, it can be seen, in the 1982 predictions, that as the size of the flood increases, the resulting peak water levels in the estuary increase as a result. However, this trait is not evident in this study and peak water level predictions in the estuary for a sand bar height greater than +2.1m MSL remain fairly constant between all flood events. As mentioned previously, when the initial sand bar elevation is greater than +2.1m MSL, similar peak water levels, between all flood events, occur in the estuary. This can be seen in Figure 9-10.

With regards to the coastal lakes, generally higher peak water levels are predicted in this study compared to the study performed in 1982. On average, an increase in peak water level of 9.9%, 7.1% and 1.3% are predicted for Island Lake, Bolangvlei and Rondevlei respectively. This is most likely explained by the increase in discharge of the Duiwe River assumed in this study. It was assumed that peak discharge levels in the Duiwe River were approximately 40% of the Touw River peak discharge, whereas in the CSIR this percentage value was approximately 26%. Given, that Rondevlei receives no direct inflow from external sources, the difference between the two studies is minimal.

9.5. Evaluation of the current management strategy

A flood management procedure became necessary in Wilderness after residential property was constructed alongside the banks of the estuary below the maximum natural water levels of the Touw estuary. During the study performed by Watermeyer Prestedge Retief in 1994, the lowest recorded property along the Wilderness estuary had an elevation of +2.6m MSL and a lowest recorded property of +2.4m MSL was recorded on the banks of Island Lake. (Watermeyer Prestedge Retief, 1994). This results in a huge flooding potential risk as any water level above the lowest property level in each water body, results in inundation of residential property. The survey performed in the camping site at Ebb & Flow showed the ground elevations vary between +2.4m and +2.7m MSL. Therefore, there is also a huge flooding potential for these low lying camping areas. Due to the large flooding risk of the estuary and Island Lake, SANParks, the managing board of the Wilderness estuary and coastal lakes system, have been forced to implement a management strategy involving the artificial manipulation of the sandbar in an attempt to reduce flood levels within the estuary and the entire system.

The current management plan stipulates that when the mouth is closed, the sand bar at the estuary mouth be maintained at a height of between +2.1m and +2.4m MSL over a width not greater than 5m (South African National Parks, 2010). A preparatory channel may also be constructed at any time prior to estuary breaching if deemed necessary by SANParks personnel to facilitate future breaching. Furthermore, a preparatory channel will be prepared when water levels in excess of +1.8m MSL are evident in the Touw estuary and when a high rainfall event (>50mm) is forecast. The extent and design of the preparatory channel must be such that premature breaching of the estuary (< 2.1m MSL in the Touw estuary) cannot occur as a result of either sediment erosion caused by high seas or acts to breach the estuary by unauthorized persons. The width of the channel should not exceed 5m and the height of the plug should remain between +2.0m and +3.0m MSL (South African National Parks, 2010).

The following discussion pertains to the evaluation of the current management plan with the exclusion of any preparatory channel construction. With sand bar heights at the estuary of between +2.1m and +2.4m MSL, which is the current tolerance interval of the sand bar height in the management plan, resulting peak water levels during a flood in the estuary are no longer dependent on the size of the flood but rather the initial height of the sand bar at the estuary mouth. The average water level in the Touw estuary for an initial sand bar height of +2.1m MSL is +2.5m MSL and for an initial sand bar height of +2.4m MSL, the resulting average peak water level in the Touw estuary is +2.9m MSL. This is shown in Figure 9-15.

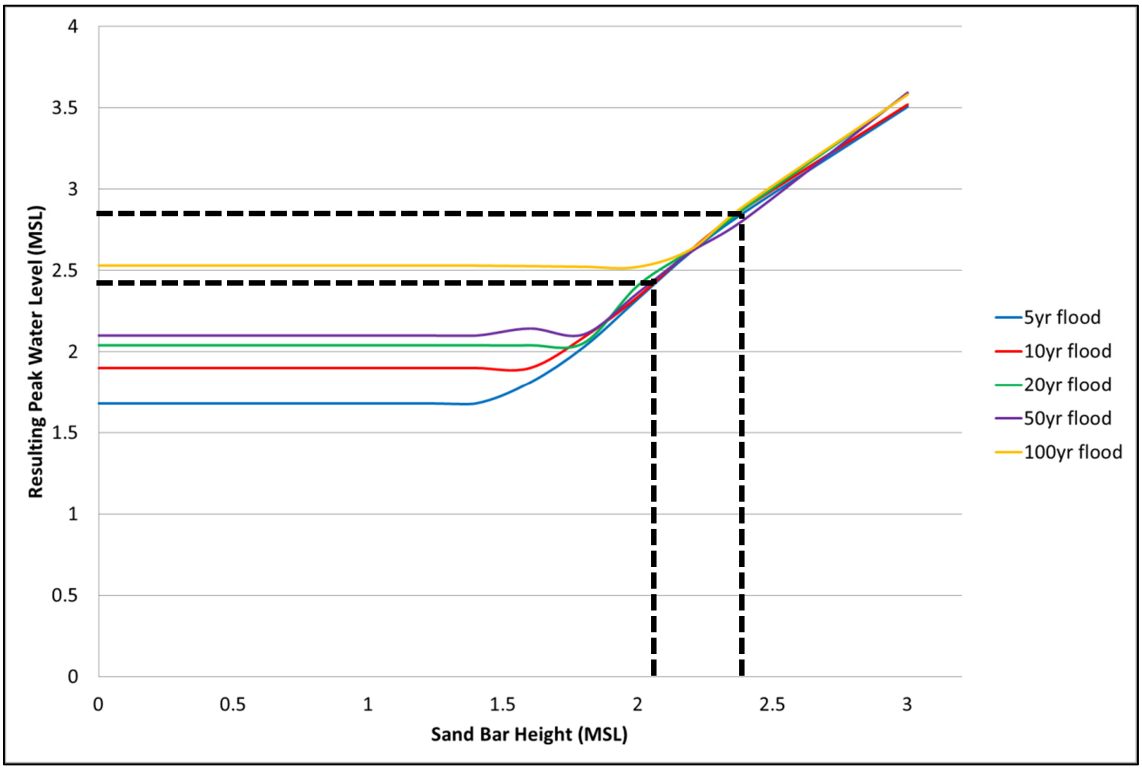


Figure 9-15: Resulting average peak water levels in the Touw estuary with sand bar heights of 2.1m and 2.4m MSL

Given that inundation of property occurs when water levels in the estuary exceed +2.6m MSL, there is a significant chance of flood damage for all flood events while only using the current sand bar skimming strategy. The resulting peak water levels illustrated in Figure 9-15 also refer to average peak water levels across the length of the estuary and does not account for the water gradient that may exist in the estuary during floods. An example of this water gradient is evident in Figure 9-5, where a maximum water level difference of 0.8m may exist across the Touw estuary between the water level recorder and Ebb & Flow. Therefore, the peak water levels could be far more substantial towards the upper ends of the estuary near the SANParks Ebb & Flow rest camp. Based on these findings, the existing sand bar skimming level (+2.1m to +2.4m) is too high to completely prevent inundation of property along the banks of the Touw estuary without the construction of a preparatory channel. The implications of the existing sand bar skimming strategy result in dangerously high water levels in the Touw estuary during any flood event where a transition from a closed mouth to open mouth estuary is evident.

The construction of the preparatory channel has therefore been implemented in the past, in an attempt to minimize peak water levels in the Touw estuary. This is an essential component of the management strategy, however, it is difficult to analyse the effectiveness of this strategy as there are a number of unknown variables present in the design and construction of the channel. The base level of the preparatory channel is unknown and there are a number of unknown variables associated with the specifications of the sand plug on the seaward side of the preparatory channel. Figure 9-16 and Figure 9-17 show the end result of the construction of a preparatory channel on the 02 April 2013. This channel was constructed due to the forecast of heavy rains predicted on the following days.

Figure 9-16: Preparatory channel constructed on the 02 April 2013

Figure 9-17: Longitudinal profile illustrating the construction of a preparatory channel on 02 April 2013

With regards to the construction of the preparatory channel, there remains some inconsistency between the management plan (South African National Parks, 2010) and the as-built design and practice. It was clarified by SANParks officials at Wilderness that the sand plug is breached manually immediately before a large flood event, however, this information is not stipulated in the SANParks management plan (South African National Parks, 2010). It was therefore decided to evaluate two different scenarios. The first scenario analysed the outcome of resulting water levels in the system given that the sand plug is breached manually before the impact of the storm event. The second scenario analyses the impact, should the sand plug undergo a natural breach.

With a manual breach of the sand plug before a significant storm event, peak water levels in the Touw estuary during the time of the flood event are predominantly dependant on the base elevation of the preparatory channel. There are limited specifications with regards to the design base level elevation of the preparatory channel which provides a difficult task of assessing the current management strategy. However, despite not knowing the design base level elevation of the preparatory channel, certain outcomes are predicted, based on hypothetical values. In order for optimum function the preparatory channel in a manner to alleviate peak flood levels in the estuary, the base level of the preparatory channel should not exceed +1.4m MSL. For values lower than this level, all floods with return periods of 5 years to 100 years are no longer dependent on the height of sand bar and resulting peak water levels in the estuary are beyond the control of sand bar intervention policies. This is illustrated in Figure 9-18. With base level elevations below +1.4m MSL in the preparatory channel, resulting peak water levels are +1.68m, +1.9m, +2.04m, +2.1m and +2.53m MSL for a 5, 10, 20, 50 and 100 year flood respectively.

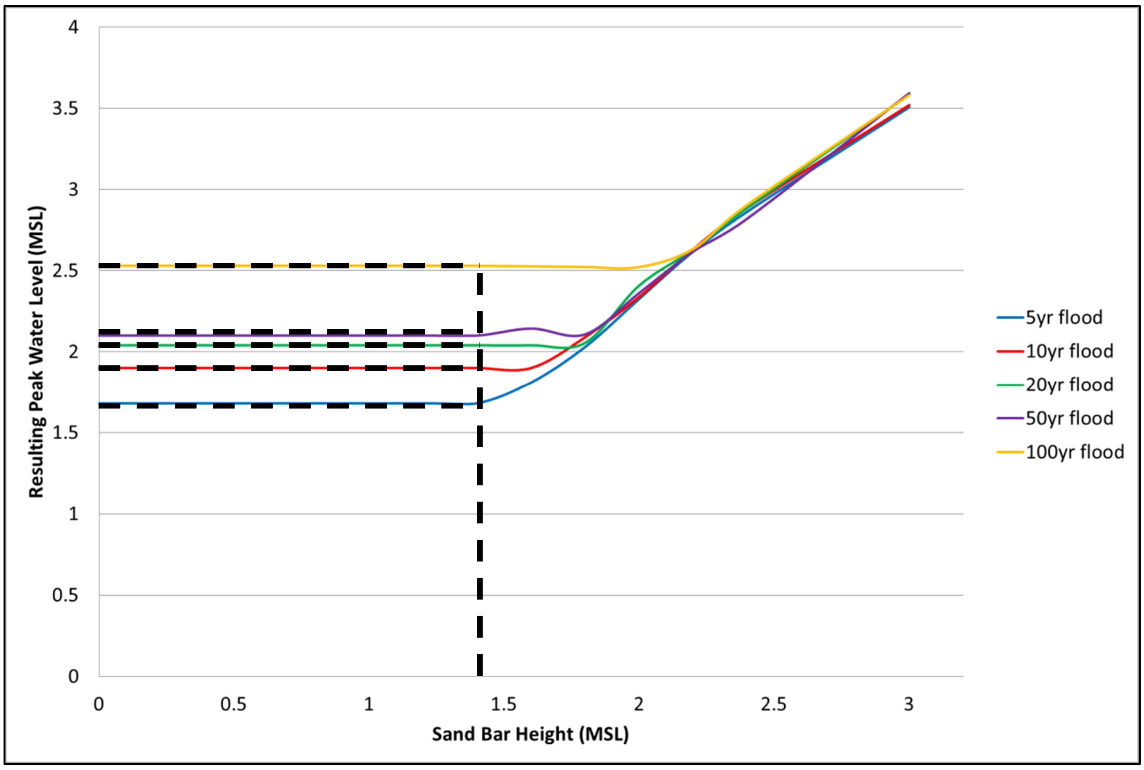


Figure 9-18: Resulting peak water levels in the Touw River estuary for different flood events with a preparatory base level elevation of 1.4m MSL or lower

The resulting outcomes of peak water levels were also analysed, should the sand plug undergo a natural breach during a flooding event.

The lack of exact plug construction specifications makes it difficult to predict an exact scour pattern of the sand plug. One possible scenario is that the plug is undermined by fast flowing water flowing through the preparatory channel, thus scouring the base of the plug and resulting in complete failure of the plug with minimal water build-up upstream of the plug. In this case, resulting peak water levels in the estuary are dependent on the base elevation of the preparatory channel and not the crest height of the plug. Resulting peak water levels in the estuary should therefore portray a similar result to Figure 9.18, where the effects of a manual breach were described.

The other scenario is that the plug only begins to scour once the water level just upstream of the plug exceeds the level of the plug. The latter scenario is the same scenario predicted for the scour of the entire sand bar and is the scenario which has previously been used in this investigation.

Dependent variables associated with the scour pattern of the plug include the size of the flood, the width of the plug as well as the base level of the preparatory channel. Using an example of extreme cases, it is hypothesised that a large flood event with a narrow plug will undermine the base of the plug before water levels upstream of the plug exceed the plug level. However; for a smaller flood event and a wide plug it is hypothesised that the water level in the estuary will rise to an elevation above the level of the plug before any significant scour of the plug is evident.

The existing management policy stipulates that the height of the plug should not exceed +3.0m MSL and should not be less than +2.0m MSL (South African National Parks, 2010). Based on a worst case scouring scenario, where the plug does not scour until the upstream water level is greater than the level of the sand plug, the resulting peak water levels in the Touw estuary will follow the prediction as the full sand bar scour predictions. These predictions can be seen in Figure 9-19 and resulting peak water levels are highlighted for a plug height of +2.0m MSL and +3.0m MSL.

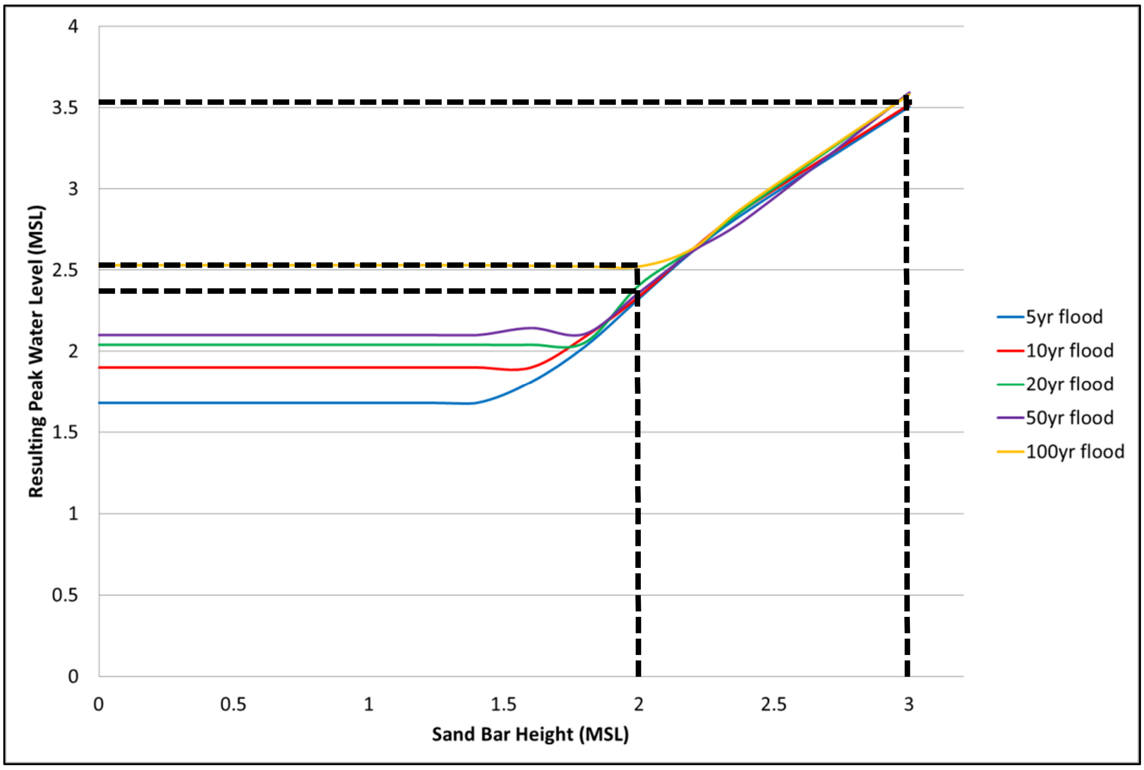


Figure 9-19: Resulting peak water level predictions in the Touw River estuary, based on different flood events and plug heights

Should the sand plug follow the same scour pattern as the full sand bar at the mouth of the estuary, the following result is predicted. For a minimum plug elevation of +2.0m MSL, average peak water levels of +2.4m MSL are predicted in the Touw estuary for any flood with a return period less than 100 years. For a flood with a return period of 100 years, the average peak water level in the Touw estuary is +2.53m MSL. Both of these values correspond to a plug height of +2.0m MSL which is the minimum allowable level of the plug height according to the SANParks management policy of the bar at the Touw River mouth (South African National Parks, 2010). The maximum allowable level of the sand plug is +3.0m MSL (South African National Parks, 2010) and the average peak water levels in the estuary that correlate to a +3.0m MSL plug height are +3.5m MSL for all flood events.

The current management policy primarily consists of the artificial manipulation of the sand bar at the estuary mouth. However, the water levels in the Wilderness lakes are not dependent of the height of the sand bar but rather on the initial water level within each lake. Therefore, the current management policy has only a small impact on the resulting water levels in the lakes. Property is constructed on the banks of Island Lake at an elevation of +2.4m MSL, therefore, any water above this elevation will result in inundation. Figure 9-20 illustrates the combined peak water level results based on different floods and initial water levels. The figure also shows a dashed line indicating a water level of +2.4m MSL and the respective initial water levels associated with each flood event.

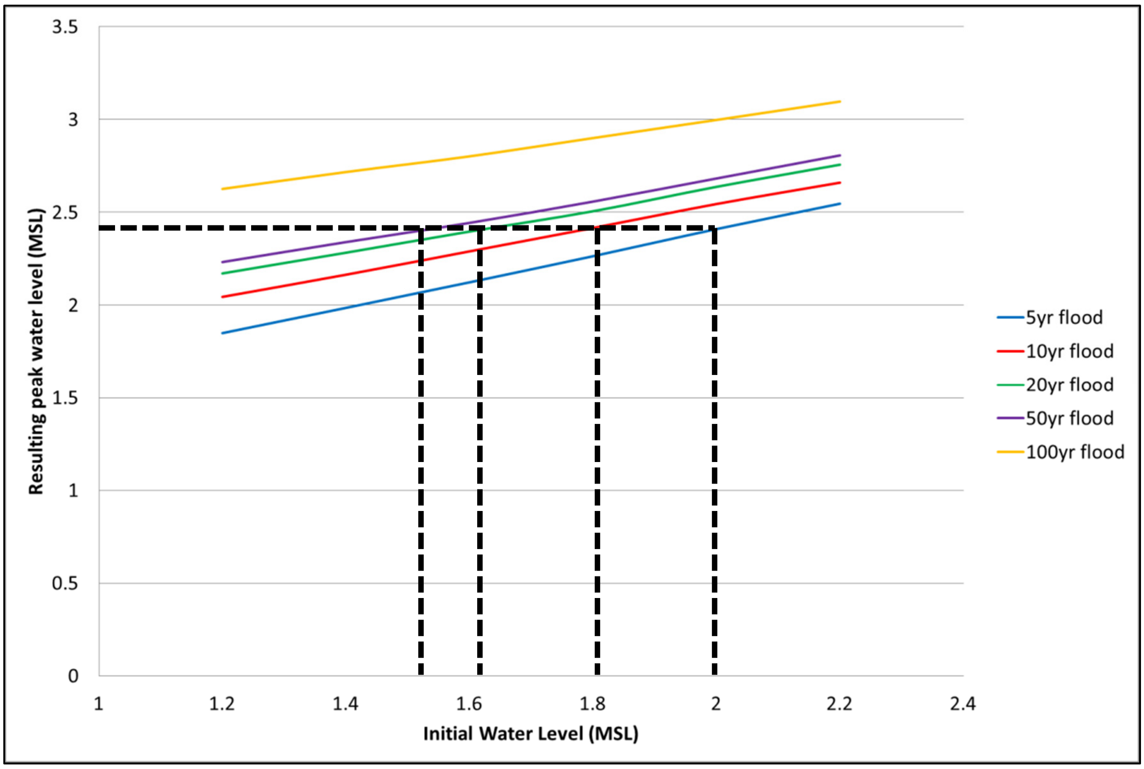


Figure 9-20: Resulting peak water levels in Island Lake illustrating a +2.4m MSL water level and the associated initial water level for each flood event

It can be seen from Figure 9-20 that a 100 year flood will always result in inundation of property for all initial water levels greater than +1.2m MSL. However, for other flood events, inundation of property only commences with an initial water levels of +1.5m, +1.6m, +1.8m and +2.0m MSL for a flood with a return period of 50 years, 20 years, 10 years and 5 years respectively. In an effort to minimise flood damage of Island Lake property, water levels in the lakes could be maintained at levels below +1.5m MSL. This is regarded as a safe value whereby inundation of property is prevented for all floods with a return period of 50 years or less. This would require the sluice gate to be operational and further studies should be performed to look at the practicality of this as well as any adverse environmental effects that may be associated with the implementation of this structure.

10. SALINITY MODEL TESTS

10.1. Introduction

The current salinity trends in the Wilderness coastal lakes system are cause for concern for the wellbeing of the ecology of the entire system. Salinity is a natural component of estuarine ecosystems to which many biotic components are well adapted. Adjustments to salinity levels may have a negative effect on the ecology of the system and could alter the species diversity and general composition of biota inhabiting this ecosystem. Organisms survive salinity changes either by tolerance or avoidance and new species, that favour the new salinity conditions, begin to enter the ecosystem, thus resulting in a change in species diversity (Nielson, et al., 2003).

A one dimensional modelling study was performed in order to predict long term salinity changes within the estuary under a number of different hypothetical scenarios. This model provides a first indication to salinity trends within the system, however, it is not expected to be completely reliable. This is predominantly due to the one dimensional modelling approach and the lack of the ability to completely resolve stratification concerns throughout the system.

Resulting salinity levels are dependent on a number of different factors including the initial salinity of each water body, the condition of the sand bar at the estuary mouth (open or closed) as well as the freshwater inflow from the rivers. However, in order to model different scenarios, based on hypothetical conditions, calibration of the model is initially required in order to ensure that results obtain an acceptable representation of prototype conditions.

10.2. Model Calibration

10.2.1. Introduction

The salinity numerical model was calibrated against prototype salinity data which was gathered from the SANParks Scientific Services Office at Rondevlei. A two year simulation period was used as a calibration event, where a variety of river inflow conditions were present. Furthermore, the estuary mouth changes condition from a closed mouth state to an open mouth state on a number of different occasions. This provided an adequate length of time to evaluate and compare trends within the prototype data and the model data. The outcome of the model was the prediction of long term salinity trends within the different water bodies in the Touw estuary and coastal lakes, therefore, it is most appropriate to calibrate the model using a long term simulation period.

A similar model as used in the hydrodynamic study was also used in the salinity study. All hydrodynamic parameters were left unchanged and the salinity model was calibrated by altering the dispersion coefficient within the Advection / Dispersion module within the model set up.

10.2.2. Calibration event and prototype data

As mentioned previously, the calibration simulation covers a two year period from the 01 January 2003 to the 01 January 2005. Salinity measurements were only recorded every three months and therefore nine measurements were recorded during the two year simulation period. During this period, one flood with a return period of three years was recorded (Görgens, 1979) and four additional floods were recorded which resulted in a breach of the estuary mouth. Figure 10-1 illustrates the Touw River inflow conditions over the two year period as well as the sand bar condition for the same period. A sand bar elevation of +2.1m MSL indicates a closed sand bar and a sand bar elevation of +0.0m MSL indicates an open mouth condition.

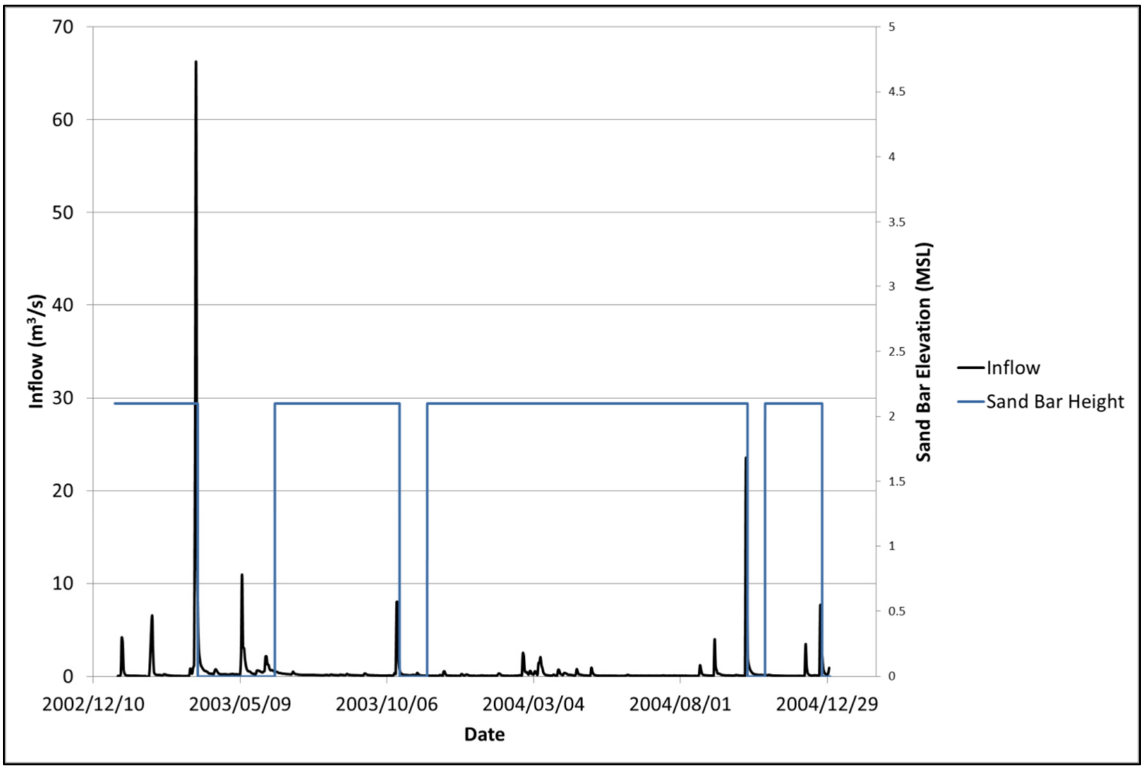


Figure 10-1: Touw River Inflow and mouth condition over two year simulation period

The resulting prototype salinity measurements that were recorded under these conditions can be seen in Figure 10-2. Due to the lack of data density, only the actual recorded points are highlighted on the figure. Due to the long interval between salinity measurements, a best fit line may be deceiving because of the rapid salinity fluctuations that occur in the estuary between an open mouth and closed mouth. It must also be noted that salinity measurements are performed at a depth of 0.3m below the water surface at each locality. The data is, therefore, reflective of surface salinities only and does not account for the full water column.

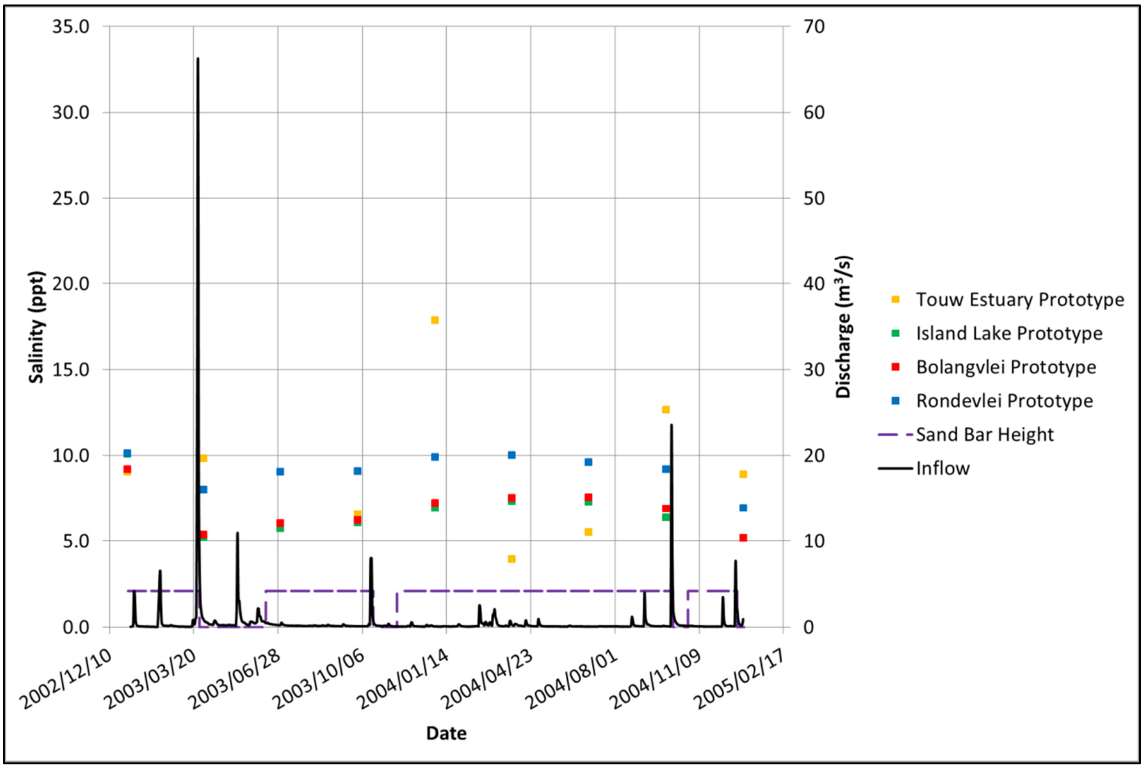


Figure 10-2: Prototype salinity measurements over two year simulation period

From Figure 10-2 it can be seen that the salinity levels within the Touw estuary fluctuate significantly between seawater and freshwater conditions. This body of water is closest to the ocean and is also the closest receiving basin for fresh water from the Touw River. The salinity in Rondevlei fluctuates the least of all water bodies and is most likely due to the fact that Rondevlei has no direct freshwater inflow and is also the furthest removed from the ocean. Despite the sparse prototype salinity data and the lack of full water column salinity measurements, a one dimensional numerical model was created which best replicated the prototype salinity measurements illustrated above.

10.2.3. Calibration of numerical model

10.2.3.1. Introduction

The aim of the calibration process of the numerical model was to create a numerical model that best replicated the salinity data of the two year simulation period. Given that the hydrodynamics of the numerical model had previously been calibrated, the primary tool used for calibration in the salinity model was the dispersion coefficient.

10.2.3.2. Dispersion Coefficient

This dispersion coefficient is a coefficient which describes the efficiency of mixing and propagation of saline water throughout the water system. This coefficient is described as a function of the mean flow velocity (DHI, 2011)

$D = aV^b$ (DHI, 2011)

Where:

D = Dispersion Coefficient

a = Dispersion Factor

V = Mean Flow Velocity

b = Dispersion Exponent

Velocities in the system alter significantly between low flow periods and periods of flooding, which, in turn, alters the dispersion properties of saline water. The dispersion coefficient of all water channel areas, including the estuary, Serpentine River and interconnecting channels between the lakes was described by the function below.

$D = 300V^1$

Velocities in the lakes are relatively small, while a significant amount of dispersion still takes place. A constant dispersion coefficient of $20m^2/s$ was therefore used in each of the three lakes. This was a calibrated value which was found to work well within the model.

10.2.3.3. Results

The results of the salinity numerical model for the two year period are shown in Figure 10-3. Results were computed for every day thus providing an adequate density of data to evaluate different scenarios between open mouth and closed mouth conditions. Salinity peaks are therefore evident in the Touw estuary during an open mouth condition. This was not as clear in the prototype data due to the sparse salinity measurements, which were recorded every three months.

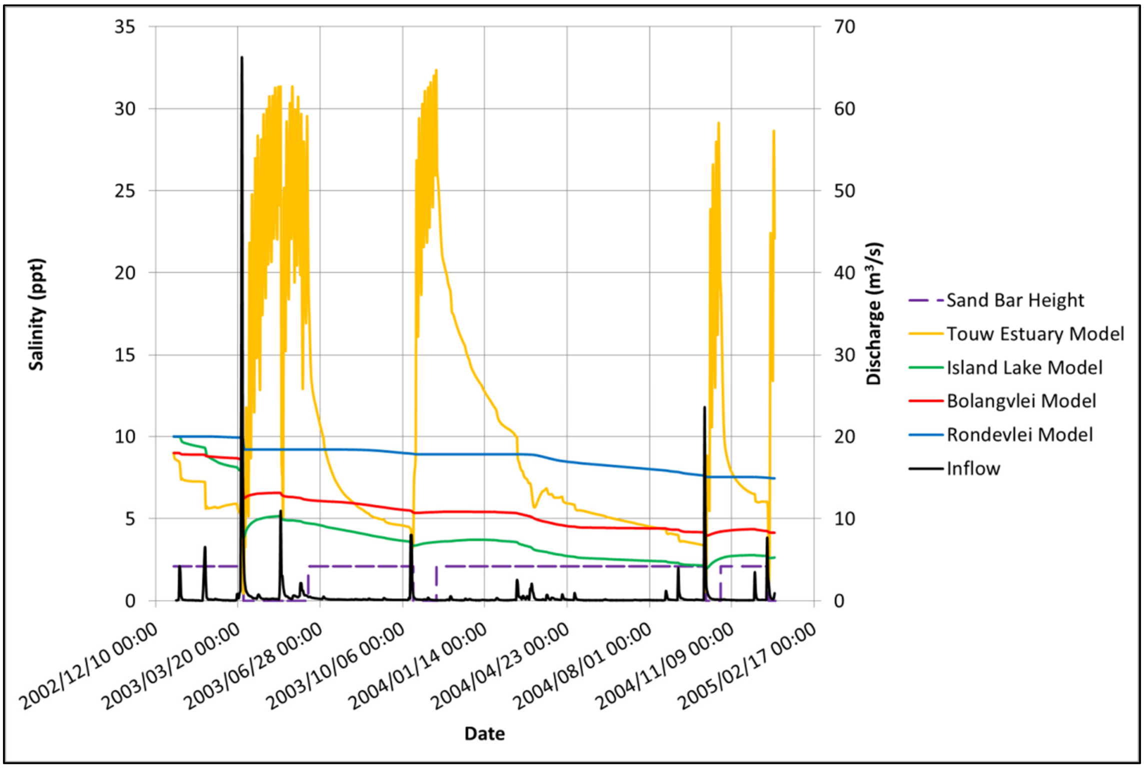


Figure 10-3: Calibration model salinity results over two year period

The model predicts that salinity levels in the Touw estuary fluctuate significantly between an open mouth and closed mouth condition. Salinity levels above 30ppt are predicted during an open mouth condition and salinity levels below 5ppt are predicted for an extended closed mouth state. It is evident that saline water also propagates up the Serpentine to the coastal lakes as increases in salinity levels are evident in Island Lake and Bolangvlei during open mouth conditions. A direct comparison between the prototype data and the model predicted data is necessary in order to whether a good representation between the model and prototype data exists.

Figure 10-4 shows a direct comparison between the prototype salinity data and the model predicted salinity levels for the same period.

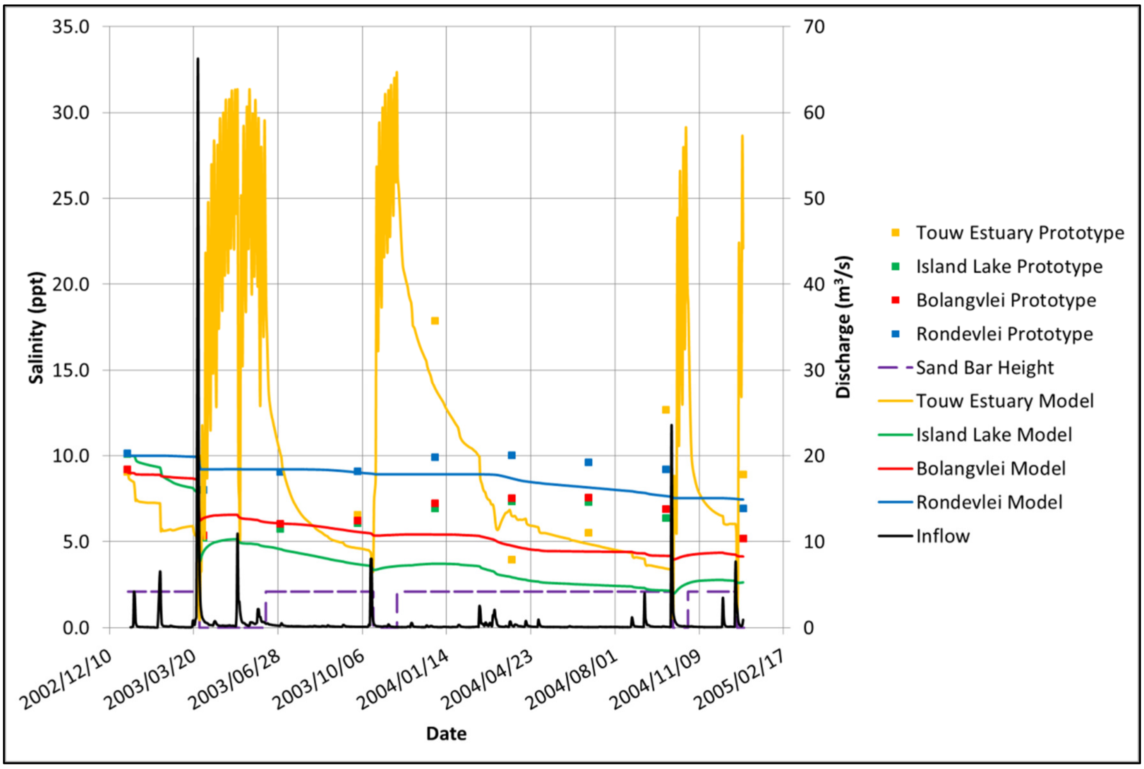


Figure 10-4: Direct comparison between prototype salinity data and model predicted salinity levels for the same period

Due to the sparse density of prototype salinity data, it made it difficult to evaluate a true comparison between the numerical model and prototype data. However, a number of similarities and differences are still evident, given the limited prototype dataset. In both the prototype data and model predicted results, it is evident that saline water propagates into the estuary as well as the coastal lakes. This can be seen from the rise in salinity during an open mouth condition. However, it appears that one major difference between the model and reality is the retention time of salinity levels in the coastal lakes after an open mouth condition. After the initial big flood in March 2003, the mouth remained open for a period of 86 days. Saline water was then allowed to propagate into the estuary and up to the coastal lakes. However, it appears that the propagation of salinity into the coastal lakes is a longer lasting process in reality compared to the model prediction. After the big flood in March 2003, prototype salinity levels increased steadily from March 2003 until April 2004. However, increases in salinity in the model were very short lived following an open mouth condition.

These differences between the model predicted salinity levels and the prototype data may be a result of numerous factors. Firstly, the model uses a one-dimensional modelling approach, where in actual fact the system is three-dimensional. Many aspects of salinity are therefore unable to be modelled using a one-dimensional modelling approach including horizontal and vertical salinity distributions. Stratification, which may occur in the Touw River estuary and coastal lakes, is therefore unable to be modelled using a one-dimensional approach.

Additional factors include the fact that the model does not account for the process of evaporation, which implies that greater volumes of fresh water remain in the system compared to reality. In reality, freshwater is constantly evaporated from the system, thus resulting in higher concentrations of salinity in the water bodies. Evaporation was not included in the model as it may disturb the hydrodynamics of the model, which, in turn, would require a re-calibration of the hydrodynamic model. The aim of the salinity model was to provide a first indication in to salinity trends throughout the system and therefore a model with the exclusion of evaporation was sufficient to meet this objective.

The model does not account for any windblown effects on the water system. Windblown effects create additional mixing in the water bodies and could alter the stratification of salinity in the in the water system thus creating a more saline environment near the water surface. Additionally, windblown effects may also alter the propagation of salinity throughout the water system and create longer retention times of salinity in the lakes.

Furthermore, no discharge data existed for the Duiwe River and Langvlei Spruit. Flow data for these rivers was therefore calculated as a percentage of the inflow data for the Touw River based on a calibration of the hydrodynamic model. The percentage value was initially calculated based on the catchment characteristics of each respective inflow source and was refined during the calibration of the hydrodynamic model. However, considerable variability may exist in the relationship between flows of the Touw River, Duiwe River and Langvlei Spruit, which may alter significantly over the two year simulation period used for the calibration event. A greater inflow of freshwater into the lakes from the Duiwe River and Langvlei Spruit may therefore be evident in the model compared to that of reality.

Differences in geology and soils may also have a slight impact on the influent salinity levels of the Touw River, Duiwe River and Langvlei Spruit respectively, thus affecting salinity levels in the overall system. In the model, a constant salinity of 0.5 ppt is assumed for all freshwater inflow sources but variation may occur according to the soil characteristics of each catchment. The Touw River arises in the forest and fynbos covered slopes of the Outeniqua Mountains. Its lower catchment is intensively utilised for agriculture, plantation and urban development (Russell, 2013). Approximately 21% of the total catchment size no longer supports natural vegetation (Russell, 2013). A similar situation is also evident in the Duiwe River and Langvlei Spruit catchment, where 63% of the catchment no longer supports natural vegetation (Russell, 2013). This is a threefold increase compared to that of the Touw River. The geology of the catchment areas predominantly consists of sandstone from the Table Mountain Group while a substantial portion of the Duiwe River catchment consists also of Cape Granite.

10.2.4. Conclusion

Despite the discrepancies between the prototype data and model predicted results, the model portrayed adequate results in other areas and justifiable reasons were mentioned for the discrepancies between the model and prototype data. The model was therefore used for the prediction of certain hypothetical scenarios with regards to future salinity levels of all water bodies.

10.3. Numerical Modelling Approach

10.3.1. Introduction

Now that a calibrated salinity model had been established, this model was therefore used in order to predict the outcome of certain hypothetical events. The modelling approach that was adopted for this study included a sensitivity analysis which was performed on the salinity levels within each water body in the Wilderness estuary and coastal lakes system. The salinity levels within the system are primarily based on three fundamental variables viz. the condition of the sand bar (open or closed), the volumes of freshwater inflow from the respective river sources and the initial salinity levels within the system. A study was therefore performed in order to estimate the resulting salinity after a one year simulation period, based on different hypothetical combinations of the dependent variables discussed above. For the purpose of a date allocation, the year 2100 was allocated as the yearlong simulation period.

10.3.2. River Inflow

Two different modelling approaches were considered with regards to inflow conditions. Firstly, a zero inflow condition was simulated, where no inflow from any freshwater source was evident throughout the entire simulation period. This provided the opportunity to evaluate how salinity levels change across the system, based entirely on seawater influx.

The second modelling approach with regards to inflow conditions included a yearlong period of average inflow. This included a period with average annual flows as well as a one year return period flood event. A historic annual inflow time series was selected which best represented these conditions as this approach provides the most realistic scenario. The 2004 inflow conditions were selected to represent an average inflow year for the simulation period. A few minor inflows are evident as well as a one year return period flood event, which was based on Görgens' (1979) predictions. The annual hydrograph can be seen in Figure 10-5.

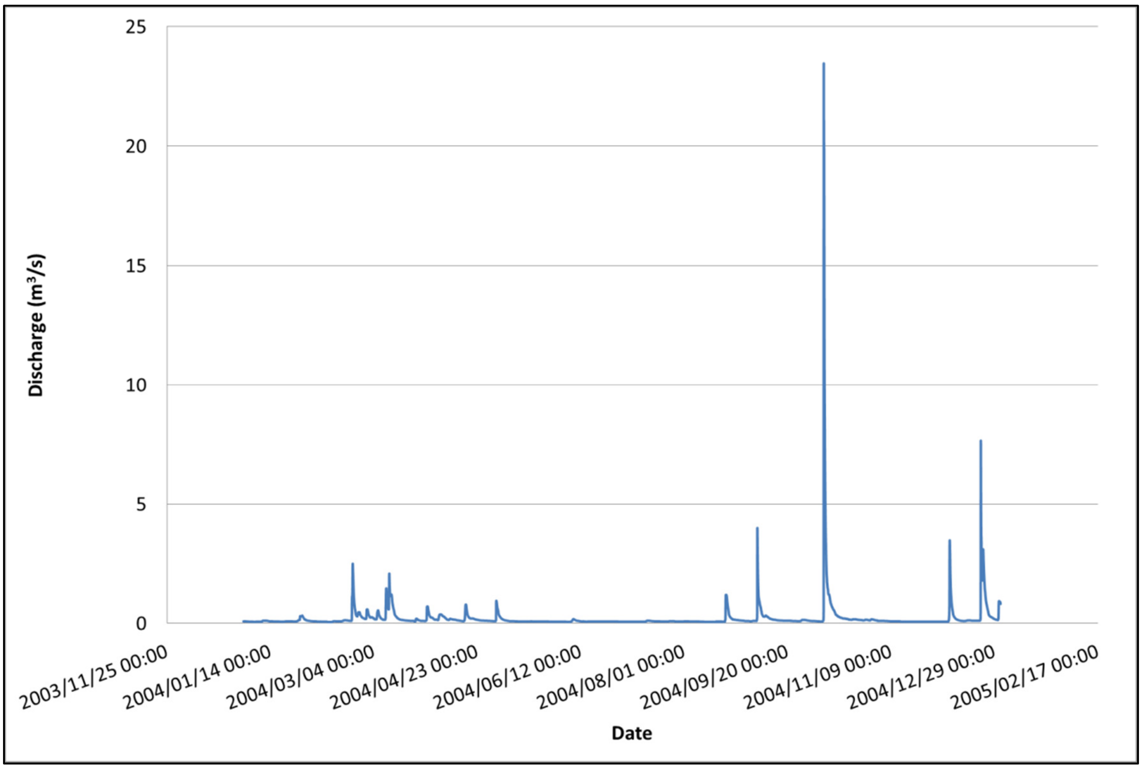


Figure 10-5: Annual Hydrograph used for Touw River inflow in salinity model

10.3.3. Sand Bar Condition

Two different sand bar conditions were further evaluated for each of the inflow conditions. These sand bar conditions included a fully open mouth condition for the entire simulation period and a fully closed mouth condition for the entire simulation period. For the zero inflow condition only an open mouth condition was simulated in order to evaluate the propagation of seawater into the system.

10.3.4. Initial Salinity Levels

Resulting salinities in the water system are also dependent on the initial salinity level of the respective water bodies. Therefore different simulations were tested given different hypothetical initial salinity levels. Salinity levels of freshwater inflow remained constant at 0.5 ppt and salinity levels of the ocean remained constant at 35 ppt.

Table 10-1 and Table 10-2 summarize the modelling approach for the zero inflow condition as well as the average annual inflow condition.

Table 10-1: Numerical modelling approach for resulting salinity levels in different water bodies, for zero inflow condition

		Sand Bar Open			
		Ronde vlei	**Bolang vlei**	**Island Lake**	**Touw estuary**
Initial Salinity Level (ppt)	**0.5**	✓	✓	✓	✓
	5.0	✓	✓	✓	✓
	10.0	✓	✓	✓	✓
	15.0	✓	✓	✓	✓
	20.0	✓	✓	✓	✓
	25.0	✓	✓	✓	✓
	30.0	✓	✓	✓	✓
	35.0	✓	✓	✓	✓

Table 10-2: Numerical modelling approach for resulting salinity levels in different water bodies, for average annual inflow condition

		Sand Bar Open				Sand Bar Closed			
		Ronde vlei	**Bolang vlei**	**Island Lake**	**Touw estuary**	**Ronde vlei**	**Bolang vlei**	**Island Lake**	**Touw estuary**
Initial Salinity Level (ppt)	**0.5**	✓	✓	✓	✓	✓	✓	✓	✓
	5.0	✓	✓	✓	✓	✓	✓	✓	✓
	10.0	✓	✓	✓	✓	✓	✓	✓	✓
	15.0	✓	✓	✓	✓	✓	✓	✓	✓
	20.0	✓	✓	✓	✓	✓	✓	✓	✓
	25.0	✓	✓	✓	✓	✓	✓	✓	✓
	30.0	✓	✓	✓	✓	✓	✓	✓	✓
	35.0	✓	✓	✓	✓	✓	✓	✓	✓

10.4. Salinity Modelling Results

10.4.1. Introduction

The following results show the model predicted salinity levels during and after the one year simulation period for the different hypothetical scenarios, which were previously discussed. The following chapter is divided into three subsections, which represent the three different numerical modelling approaches. Firstly, salinity levels are analysed based on zero inflows and an open mouth condition. Secondly, resulting salinity levels are analysed based on an average annual inflow and an open mouth condition and lastly, salinity levels are analysed based on an average annual inflow and a closed estuary mouth.

10.4.2. Resulting Salinity Levels after zero freshwater inflow and an open mouth

Table 10-3 shows the resulting salinity levels after one year where an absence of freshwater inflow is evident in the entire system and the mouth remains open for the entire simulation period. Additionally, the percentage difference from the initial salinity value is also indicated. A green shaded box illustrates a greater salinity prediction after the simulation period, whereas a pink shaded box illustrates lower salinity prediction after the simulation period. The magnitude of the difference is also illustrated by the relative shading, where a darker shade illustrates a greater differential.

Table 10-3: Resulting salinity levels after one year for zero freshwater inflow and a completely open mouth condition

		Resulting Salinity Level				Percentage Difference			
		Ronde vlei	Bolang vlei	Island Lake	Touw estuary	Ronde vlei	Bolang vlei	Island Lake	Touw estuary
Initial Salinity Level (ppt)	0.5	0.50	0.56	4.85	34.76	0.21%	11.82%	869.41%	6852.57%
	5	5.01	5.05	8.78	34.79	0.16%	1.03%	75.60%	595.87%
	10	10.02	10.04	13.15	34.83	0.16%	0.43%	31.50%	248.28%
	15	15.02	15.03	17.52	34.86	0.16%	0.23%	16.80%	132.41%
	20	20.03	20.03	21.89	34.90	0.16%	0.13%	9.45%	74.48%
	25	25.04	25.02	26.26	34.93	0.16%	0.07%	5.04%	39.72%
	30	30.05	30.01	30.63	34.96	0.16%	0.03%	2.10%	16.55%
	35	35.06	35.00	35.00	35.00	0.16%	0.00%	0.00%	0.00%

With no freshwater inflow into the system, salinity levels within the system are completely dependent on the influx of seawater. This provided valuable insight into the propagation of salinity throughout the system as well as the rate of salinity fluctuations in various water bodies. As hypothesised, salinity levels for all tests either remained the same or increased throughout all water bodies. Greater increases in salinity levels were also evident for an initial salinity with a more freshwater initial condition. Furthermore, the penetration of seawater was less prominent as the water bodies became further removed from the ocean. Large variations in salinity were evident in the Touw estuary while only minor salinity changes occurred in Rondevlei. Figure 10-6 illustrates the changes in salinity throughout the entire simulation period for an initial salinity level of 0.5 ppt in all water bodies. Had evaporation been included in the numerical model, the changes in salinity levels would have been more substantial, however, hypersaline conditions are not expected for any of the coastal lakes.

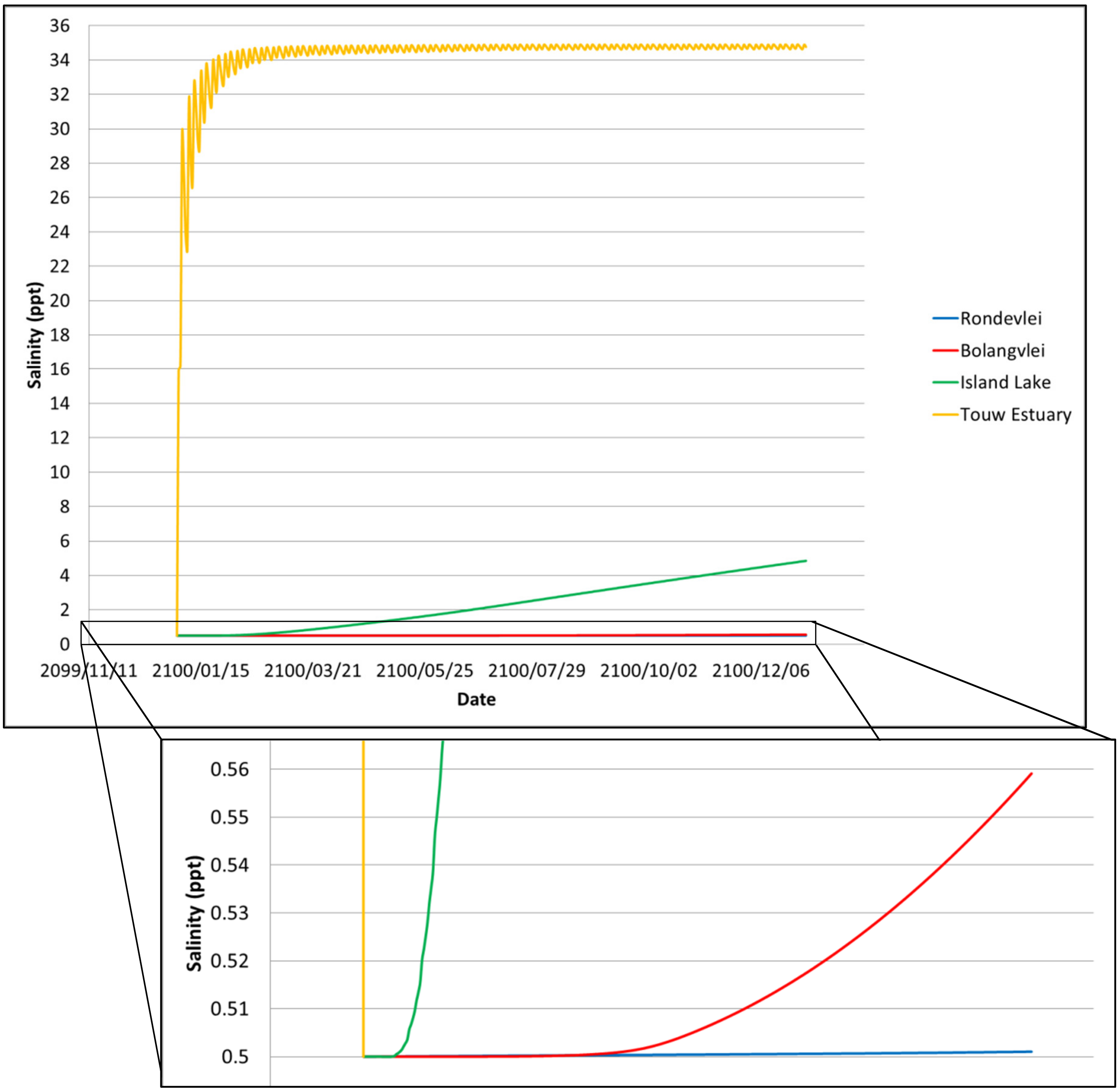


Figure 10-6: Salinity variation during simulation period for initial salinities of 0.5 ppt (Zero freshwater inflow and open mouth condition)

From Figure 10-6, it is evident that seawater propagation into the estuary is significant and occurs at a rapid rate. The penetration of seawater into the coastal lakes is far less prominent. Only a minor intrusion of seawater is predicted for Island Lake which occurs at a much slower rate than that of the estuary. Seawater intrusion into the upper lakes, namely Bolangvlei and Rondevlei is essentially negligible. A closer analysis shows that the furthest removed lake from the ocean, Rondevlei, has the lowest salinity differential from the initial conditions.

Salinity level changes over the entire simulation period for different initial salinity levels can be seen graphically in Appendix G.

10.4.3. Resulting Salinity Levels after average annual freshwater inflow and an open mouth

Table 10-4 shows the resulting salinity levels after one year, where an average annual inflow is simulated and the mouth remains open for the entire simulation period. Additionally, the percentage difference from the initial salinity value is also indicated. A green shaded box illustrates a greater salinity prediction after the simulation period, whereas a pink shaded box illustrates lower salinity prediction after the simulation period. The magnitude of the difference is also illustrated by the relative shading, where a darker shade illustrates a greater differential.

Table 10-4: Resulting salinity levels after one year for an average annual freshwater inflow and a completely open mouth condition

		Resulting Salinity Level				Percentage Difference			
		Ronde vlei	Bolang vlei	Island Lake	Touw estuary	Ronde vlei	Bolang vlei	Island Lake	Touw estuary
Initial Salinity Level (ppt)	**0.5**	0.50	0.50	1.48	28.94	0.16%	0.24%	196.86%	5688.68%
	5	5.00	4.06	4.48	29.07	0.02%	-18.87%	-10.45%	481.44%
	10	10.00	8.01	7.80	29.22	0.01%	-19.93%	-21.97%	192.15%
	15	15.00	11.96	11.13	29.36	0.01%	-20.28%	-25.80%	95.72%
	20	20.00	15.91	14.46	29.50	0.00%	-20.46%	-27.72%	47.51%
	25	25.00	19.86	17.78	29.64	0.00%	-20.56%	-28.88%	18.58%
	30	30.00	23.81	21.11	29.79	0.00%	-20.63%	-29.64%	-0.71%
	35	35.00	27.76	24.43	29.93	0.00%	-20.69%	-30.19%	-14.48%

When a freshwater inflow is added to the system while the estuary mouth is maintained in an open condition for the entire simulation period, the resulting salinity levels differ substantially compared to the case when no freshwater inflow was simulated. For initial salinity levels of 0.5 ppt in all water bodies, in other words a completely freshwater system, salinity levels are expected to increase in all water bodies after the yearlong simulation period. Thereafter, with initial salinity levels between 5 and 35 ppt, resulting salinity changes in Rondevlei are essentially negligible. This is contributed by the lack of external freshwater inflow into the system. Additionally, very little water is able to propagate from Bolangvlei to Rondevlei during an open mouth condition, as this is generally regarded as an outflow phase, when water in the system is drained and travels in the direction of the ocean. Reductions in salinity levels are evident in Bolangvlei and Island Lake for initial salinity ranges between 5 and 35 ppt. This is most likely caused by a greater volume of freshwater inflow into these water bodies compared to that of seawater propagation. In contrast, the Touw estuary shows that a greater volume of seawater penetrates this water body due to its proximity to the ocean. The resulting salinity in the Touw estuary remains fairly constant regardless of the initial salinity level and only differs by 1 ppt between the lowest and highest initial salinity level. It can therefore be concluded that initial salinities, between the range of 0.5 and 25 ppt, result in an increase in salinity in the estuary under conditions of average annual freshwater inflow and a yearlong open mouth condition, while initial salinity levels above 30 ppt result in a decline in salinity level under the same conditions.

Figure 10-7 illustrates the changes in salinity throughout the entire simulation period for an initial salinity level of 0.5 ppt in all water bodies. From Figure 10-7, it can be seen that salinity levels in the Touw estuary increase rapidly due to the open mouth condition thus allowing seawater to propagate into the system. However, instances of high freshwater inflow are also evident which quickly alter the salinity levels in the estuary to a more freshwater environment. The effect of seawater penetration into Island Lake is less severe compared to the previous scenario, where no inflow was simulated. Freshwater inflow from the Duiwe River thus has a significant effect on the salinity control of Island Lake. Salinity fluctuations in Bolangvlei and Rondevlei were essentially negligible; however, a closer analysis reveals that slight salinity changes are evident in the lakes. Rondevlei shows a slight steady increase in salinity level, most likely due to seawater penetration. Bolangvlei also shows a slight increase in salinity level; however, the rate of increase is not constant which illustrates that the freshwater inflow from Langvlei Spruit has a consequential effect on the salinity levels within this water body.

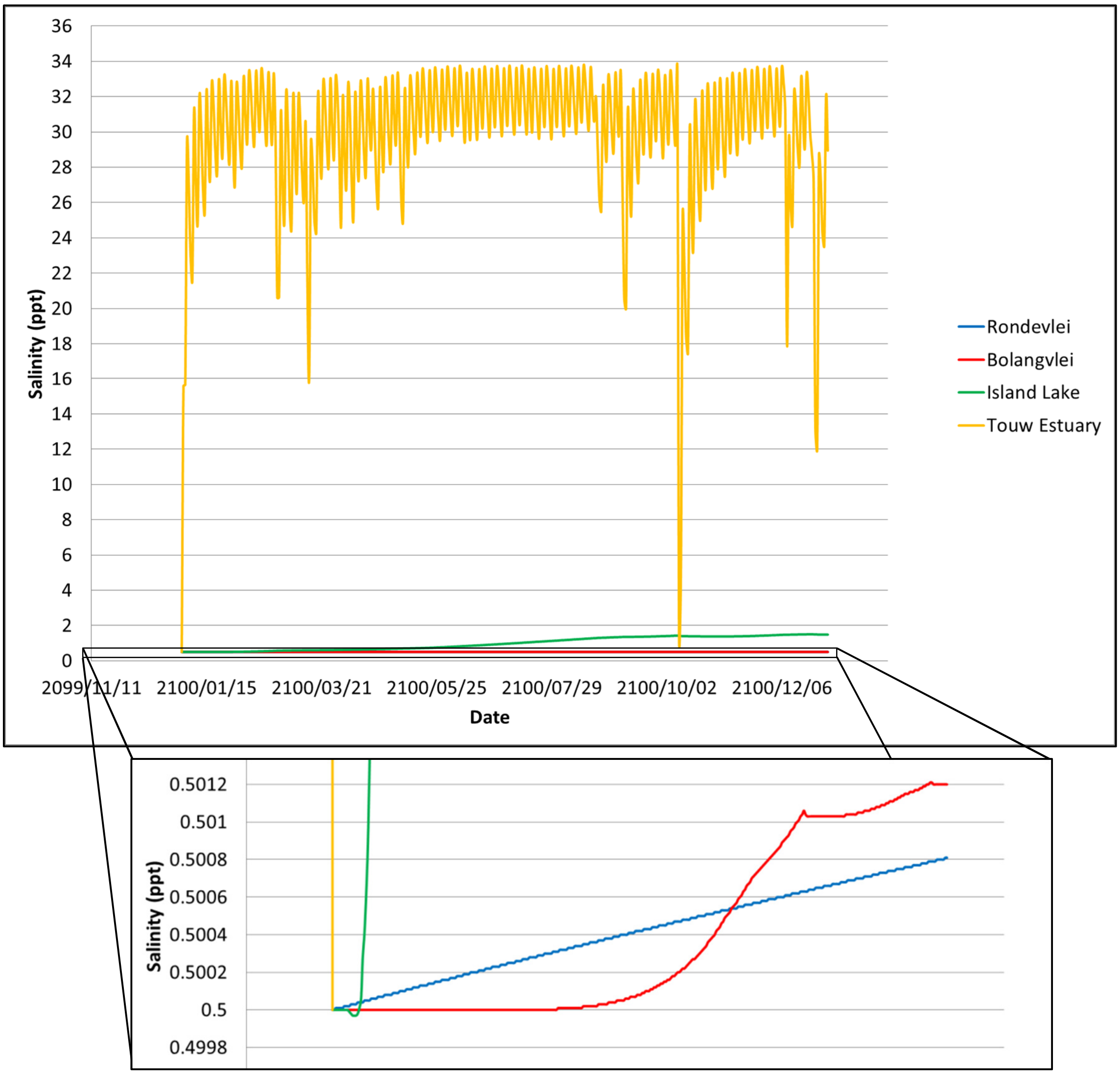


Figure 10-7: Salinity variation during simulation period for initial salinities of 0.5 ppt (Freshwater inflow and open mouth condition)

Salinity level changes over the entire simulation period for different initial salinity levels can be seen graphical in Appendix G.

10.4.4. Resulting Salinity Levels after average annual freshwater inflow and a closed mouth

Table 10-5 shows the resulting salinity levels after one year, where an average annual inflow is simulated and the mouth remains closed for the entire simulation period. Additionally, the percentage difference from the initial salinity value is also indicated. A green shaded box illustrates a greater salinity prediction after the simulation period, whereas a pink shaded box illustrates lower salinity prediction after the simulation period. The magnitude of the difference is also illustrated by the relative shading, where a darker shade illustrates a greater differential.

Table 10-5: Resulting salinity levels after one year for an average annual freshwater inflow and a completely closed mouth condition

		Resulting Salinity Level				Percentage Difference			
		Ronde vlei	Bolang vlei	Island Lake	Touw estuary	Ronde vlei	Bolang vlei	Island Lake	Touw estuary
Initial Salinity Level (ppt)	**0.5**	0.50	0.50	0.50	0.47	-0.45%	-0.90%	-0.88%	-5.01%
	5	4.27	3.33	2.19	1.02	-14.68%	-33.37%	-56.14%	-79.65%
	10	8.45	6.48	4.08	1.62	-15.47%	-35.17%	-59.21%	-83.79%
	15	12.64	9.63	5.97	2.22	-15.73%	-35.77%	-60.23%	-85.18%
	20	16.83	12.79	7.85	2.83	-15.86%	-36.07%	-60.74%	-85.87%
	25	21.01	15.94	9.74	3.43	-15.94%	-36.25%	-61.05%	-86.28%
	30	25.20	19.09	11.62	4.03	-16.00%	-36.37%	-61.25%	-86.56%
	35	29.39	22.24	13.51	4.64	-16.03%	-36.46%	-61.40%	-86.76%

With a completely closed mouth condition during the entire simulation period, there is an absence of seawater penetration into the estuary and salinity levels within the system are only dependent on the freshwater inflow from the influent rivers. From Table 10-5, it is evident that salinity levels, for all tests, decreased throughout all water bodies. Greater decreases in salinity were also evident as the initial salinity level increased.

Furthermore, the degree of salinity variation also appears to have a direct relationship with the volumes of direct freshwater inflow to each water body. The Touw estuary receives freshwater predominantly from the Touw River, the largest source of freshwater inflow to the system, and shows the greatest variation in salinity levels. Island Lake predominantly receives freshwater from the Duiwe River, the second greatest freshwater source in the system, and also shows the second greatest variation in salinity levels. Bolangvlei receives freshwater from Langvlei Spruit, which is the smallest freshwater source in the system, and shows the third greatest variation in salinity levels. Rondevlei receives no direct freshwater inflow and, during model simulations, freshwater in Rondevlei is only received from the Bolangvlei via an interconnecting channel between the two lakes. As a result, Rondevlei shows the least variation in salinity levels between initial conditions and the final salinity result after the simulation period. Figure 10-8 illustrates the previous point and shows the changes in salinity throughout the entire simulation period for an initial salinity level of 35 ppt in all water bodies.

Figure 10-8: Salinity variation during simulation period for initial salinities of 35 ppt (Freshwater inflow and closed mouth condition)

Salinity level changes over the entire simulation period for different initial salinity levels can be seen graphical in Appendix G.

11. CONCLUSIONS AND RECOMMENDATIONS

11.1. Conclusions

A review of existing literature and previous studies on the Wilderness estuary and coastal lakes system has indicated that a periodically open/closed estuary, such as the case in Touw River estuary, plays a critical role in the hydrodynamics of the estuary and connecting water bodies. Under natural conditions, it was found that the sand bar at the Touw estuary mouth may exceed elevations above +3m MSL. This would have a consequential effect of water levels upstream of the sand bar as water levels would rise above the sand bar before any scour at the mouth takes place. A predicament lies in the fact that residential property has been constructed along the banks of the Touw estuary at elevations of +2.6m MSL and along the banks of Island Lake at +2.4m MSL, thus creating a large risk of inundation during flood events. Previous studies have been performed on this water area in an attempt to control water levels in the estuary and coastal lakes. This was predominantly performed by artificial manipulation of the estuary mouth, however, additional management policies such as the dredging of interconnecting channels between the coastal lakes as well the introduction of a sluice gate in the Serpentine River were also analysed in previous studies.

The current management policy is, after natural closure, to maintain the crest height of the sand bar at the Touw River mouth between +2.1m and +2.4m MSL (South African National Parks, 2010) over a width not greater than 5m. A preparatory channel may also be constructed at any time prior to estuary breaching if deemed necessary to facilitate future breaching.

A one dimensional numerical model was created in order to assess flood water levels under different theoretical circumstances and mouth management situations. Furthermore, the model was also used to gain a first indication of salinity changes in the system given a number of hypothetical conditions.

During the hydrodynamic study, it was found that peak water levels in the Touw estuary are dependent on the return period of the flood and the height of the sand bar at the estuary mouth. Peak water levels in the estuary are effectively independent of the initial water level in the estuary before a flood. The opposite was found for the coastal lakes, where peak water levels in the coastal lakes are dependent on the initial water level as well as the return period of the flood. Peak water levels in the lakes, however, are almost completely independent of the sand bar elevation at the estuary mouth. The current management plan, which involves only artificial manipulation of the sand bar at the estuary mouth, therefore, has a fairly insignificant effect on flood water levels achieved in the coastal lakes.

Furthermore, for sand bar elevations greater than +2.1m MSL preceding a flood event, the resulting peak water levels in the estuary are no longer dependent on the size of the flood and are only dependent on the initial elevation of the sand bar at the estuary mouth.

For sand bar elevations below +2.1m MSL, peak water levels in the estuary are predicted to remain the same for each flood event. A maximum peak water level of +2.53m MSL is predicted for the Touw estuary under these sand bar conditions and is associated for a flood with a 100 year return period. As inundation of property in the estuary only occurs at water levels above +2.6m MSL in the Touw estuary, it can be concluded that all property along the banks of the estuary should remain safe from inundation for all floods with return periods of 100 years or less and as long as the sand bar is maintained at a level below +2.1m MSL.

A sand bar elevation of +2.1m MSL also refers to the lowest limit of the tolerance interval for the current sand bar skimming maintenance procedures at the estuary mouth. An evaluation of this procedure, which includes maintenance of the height of the sand bar between +2.1m and +2.4m MSL during a closed mouth condition, was also undertaken. From a hydrodynamic perspective, peak levels associated with a sand bar elevation of +2.1m and 2.4m MSL are +2.53m and +2.9m MSL respectively. Furthermore, these water levels pertain to all flood events, as resulting peak water levels above a sand bar elevation of +2.1m MSL are no longer dependent on the size of the flood. Although the current management strategy provides a significant reduction in flood water levels in the Touw estuary from the natural conditions, modelling results show that this strategy cannot completely alleviate inundation of property along the banks of the Touw estuary. Without the inclusion of a preparatory channel before a flood event, peak water levels resulting from a sand bar between +2.1m and +2.4m MSL often result in water levels above the residential inundation threshold of +2.6m MSL. In order to effectively minimise the flooding risk of residential property along the banks of the Touw River estuary based on sand bar skimming alone, the sand bar would need to be maintained at a maximum level of +2.1m MSL.

In the past the construction of the preparatory channel has, therefore, been necessary in an attempt to alleviate peak water levels in the entire system. The best results, in order to minimise peak water levels in the estuary and hence reduce flooding damage, are evident when the base level of the preparatory channel is designed to have a maximum elevation of +1.4m MSL. With a maximum base level elevation of +1.4m MSL in the preparatory channel, resulting peak water levels are at their lowest for each flood with return periods between 5 and 100 years.

In an effort to minimise flood damage of Island Lake property, water levels in the lakes could be maintained at levels below +1.5m MSL. This is regarded as a safe value whereby inundation of property is prevented for all floods with a return period of 50 years or less.

Although this study predominantly focuses on the hydrodynamic implications of artificial manipulation of the sand bar at the estuary mouth, a first indication of long term salinity trends were also analysed and salinity changes in the system were predicted based on a number of hypothetical conditions.

Historical salinity data suggests that salinity levels in Bolangvlei and Rondevlei are decreasing, while salinity levels in Island Lake appear to be increasing. Furthermore, the salinity levels in Rondevlei are decreasing at a faster rate than those of Bolangvlei. The salinity modelling study comprised a sensitivity analysis, where salinity variations were analysed under a number of extreme hypothetical scenarios.

During the salinity study, it was found that during an open mouth condition, the penetration of seawater into the system was less prominent as the water bodies became further removed from the ocean. A large influx of seawater was evident in the Touw estuary while only a minor penetration of seawater was evident in Island Lake. The penetration of seawater in Bolangvlei and Rondevlei was essentially negligible, although Bolangvlei did show higher degrees of salinity change during an open mouth compared to those of Rondevlei. The effect of freshwater inflow on salinity levels within the system was also analysed. A direct relationship was found between the volumes of direct freshwater inflow to each water body and the degree of salinity variation in each water body. In water bodies with high volumes of direct freshwater inflow such as the Touw estuary, a large degree of salinity variation is evident. However, in water bodies with no freshwater inflow, such as Rondevlei, salinity levels remain more stable and are less likely to fluctuate.

11.2. Recommendations

It was previously mentioned that the current sand bar skimming strategy alone would result in dangerously high water levels in the Touw estuary and the establishment of a preparatory channel is particularly necessary before flood events. However, in order to effectively minimise the flooding risk of residential property along the banks of the Touw River estuary based on sand bar skimming alone, it is recommended that the sand bar be maintained at a maximum level of +2.1m MSL. Flood water levels in the estuary that are associated with this sand bar level all peak below the residential inundation threshold of +2.6m MSL.

It was also discovered that the current management plan, involving only artificial manipulation of the sand bar at the estuary mouth, has a fairly insignificant effect on flood water levels achieved in the coastal lakes. Flood water levels in the lakes are essentially independent of the height of the sand bar at the estuary mouth, and it is recommended that a different strategy be evaluated in order to control water levels in the lakes.

Both the hydrodynamic and salinity study excluded any considerations for evaporation, sea level rise and storm surge. During a flood event, surrounding low pressures may induce some degree of storm surge and it is recommended that updated hydrodynamic modelling cases be analysed to include considerations for storm surge and sea level rise.

Immediate measures can also be implemented in the system which would assist with future studies as well as assist SANParks personnel with the current execution of the management policy of the sand bar at the estuary mouth.

Firstly, it would be beneficial to install a flow gauging station in the Touw River which is able to record discharges greater than the current limit of 66.2m^3/s. The installation of flow gauging stations in the Duiwe River and Langvlei Spruit would also be advantageous.

It is also recommended that accurately surveyed water level recorders be installed in all water bodies, where all recorders refer to the same datum of Mean Sea Level (MSL). Currently, water level recorders are not installed to MSL and water level data for different water bodies do not even correlate to the same datum. Additional work is therefore necessary in adding adjustment factors to each water level recorder in order to convert the data to MSL. This is an inefficient method, and there is a high potential for error. In a system that is so dependent on water levels, accurate, time based water level recordings should be accessible for all water bodies without the uncertainty of knowing whether or not adjustment factors have been added.

Different future research areas were identified during this study which, if executed, would be hugely beneficial in further understanding the hydrodynamics of Wilderness coastal lakes system.

The first focus area involves an accurate study of the behaviour of the sand bar at the estuary mouth as it transitions from an open mouth to a closed mouth. It would be beneficial to assess the scour rates of the sand bar over time and determine how long it takes the estuary to transition from a closed condition to an open condition under different flooding scenarios and sand bar heights. The total scour volumes associated with each flood should also be analysed. Accurate cross sections of the estuary could, therefore, be formulated for an open mouth and closed mouth condition.

The flood hydrographs for this system are outdated, where the latest hydrograph for this system was performed in 1994. Additionally, during the calibration of the numerical model, better results were achieved using an earlier hydrograph study performed in 1979. The relationships between hydrographs of the Touw River, Duiwe River and Langvlei Spruit appeared to be more realistic in the earlier study. Nevertheless, a large concern remains over the accuracy and validity of both hydrograph studies. Updated hydrograph studies for the three influent rivers, namely the Touw River, Duiwe River and Langvlei Spruit would be extremely beneficial to further understand the hydrodynamics of the entire system.

Although a salinity model for this system was developed, the accuracy of this model remains uncertain. The effects of evaporation were not included in the salinity study. Evaporation may have a significant influence in long term salinity levels within the system and it is therefore it is recommended that updated model tests be performed to include the effects of evaporation. There are various intricate components involved in estuarine salinity that were unable to be modelled, due to the limitations of a one dimensional approach. Limitations of a one dimensional approach include salinity variations in width and depth. Due to the geometry of the system, which includes broad ranges of channel widths from wide open lakes to narrow channels, it is recommended that a two dimensional modelling approach be used in order to model salinity throughout the system. Previous literature indicated that the system is predominantly unstratified; however, it may be interesting to note how stratification of salinity alters throughout the system from the estuary mouth to the upper most lake of Rondevlei. A three dimensional modelling approach will, therefore, be necessary in order to assess salinity stratification throughout the system.

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APPENDICES

LIST OF FIGURES

APPENDIX A. MATHEMATICAL THEORY BEHIND MIKE 11

The mathematics behind the MIKE 11 modelling tool involves the use of the 1-D Saint Venant equations, namely the continuity equation and the momentum equation (Sleigh & Goodwill, 2000) which are formulated equations based on the conservation of mass and conservation of momentum. The mathematical theory behind these fundamental equations can be seen in Appendix A.

1. Derivation of the continuity equation

Consider a section of channel with a length of Δx as illustrated in the following figure.

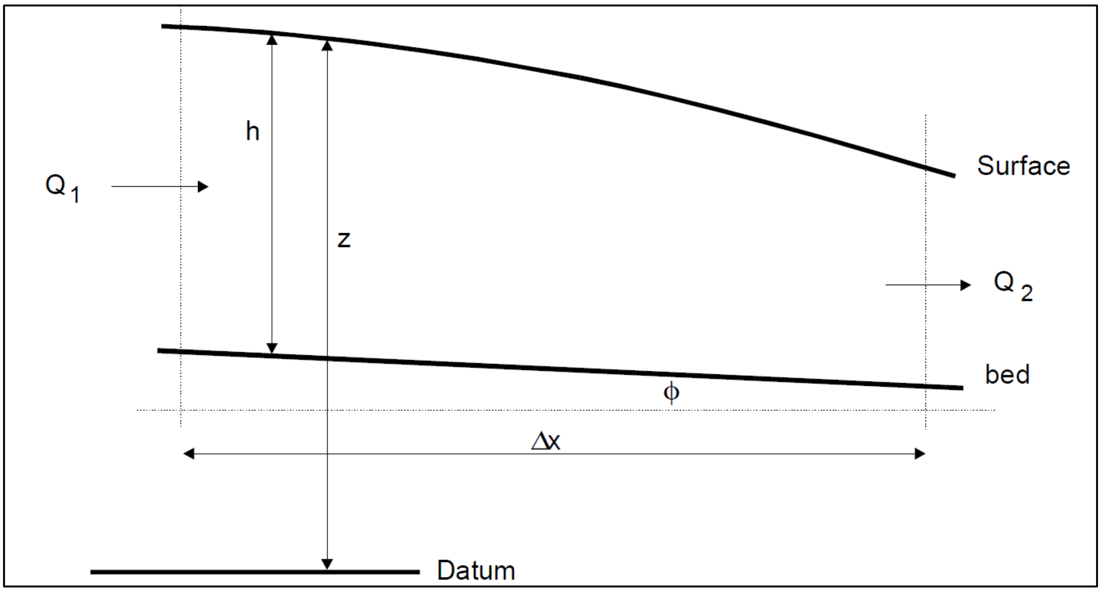

Figure A-1: Short length of channel for St Venant equation (Sleigh & Goodwill, 2000)

The following symbols are used in this derivation:

A = the cross-sectional area of the section

h = depth of flow at the section

z = elevation of surface above a datum at the section

v = mean velocity at the section

Q = discharge at the section

b = width of the top of the section

x = position of the section measured from the upstream end

t = time

g = acceleration due to gravity

ρ = mass density of the fluid

Other symbols are defined at the point when they are introduced.

Assuming no lateral inflow exists, then

$$Q_2 - Q_1 = \frac{\partial Q}{\partial x}\Delta x$$

and the volume of water between section 1 and 2 is increasing at a rate of

$$b\frac{\partial h}{\partial t}\Delta x$$

where b refers to the top width.

Due to the fact that the cross sectional area A = bh, the equation above can be expressed as.

$$\frac{\partial A}{\partial t}\Delta x$$

The terms are equal in magnitude but of opposite sign, so

$$\frac{\partial Q}{\partial x}\Delta x + b\frac{\partial h}{\partial t}\Delta x = 0$$

As $\frac{\partial Q}{\partial x} = \frac{\partial (Av)}{\partial x}$ then,

$$\boldsymbol{v\frac{\partial A}{\partial x} + A\frac{\partial v}{\partial x} + b\frac{\partial h}{\partial t} = 0}$$

2. Derivation of the momentum equation

Applying Newton's Second Law to the element length of channel

$Force = Mass \times Acceleration$

$Force = \rho A\Delta x\frac{dv}{dt}$

$Force = \rho A\Delta x[v\frac{dv}{dx} + \frac{dv}{dt}]$

Since v varies in both space and time, external forces which cause this acceleration are included. In the simplest case three external forces exist.

- $\frac{\partial H}{\partial x}$ Change in static pressure
- F Frictional resistance of channel walls and bed

- ρg Gravity force

As illustrated in the previous figure, if φ is the angle of the bed slope, then a combination of the above three forces can be written as

$$\frac{\partial H}{\partial x}\Delta x cos(\varphi) - F\Delta x + \rho g A \Delta x sin(\varphi)$$

For small bed slopes it can be approximated that cosφ = 1 and sinφ = φ = I, so

$$\frac{\partial H}{\partial x}\Delta x - F\Delta x + \rho g A \Delta x i$$

Furthermore,

$$\frac{\partial H}{\partial x} = \rho g A \frac{\partial h}{\partial x}$$

and

$$F = \rho g A j$$

where j is energy loss / unit length of channel / unit weight of fluid.

Equation these external factors to the change in momentum yield,

$$\rho A \Delta x \left[v\frac{dv}{dx} + \frac{dv}{dt} \right] = -\rho g A \frac{\partial h}{\partial x} - \rho g A j + \rho g A \Delta x i$$

Rearranging this equation gives the momentum equation.

$$\boldsymbol{g\frac{\partial h}{\partial x} + v\frac{\partial v}{\partial x} + \frac{\partial v}{\partial t} = g(i-j)}$$

APPENDIX B. SITE VISITS AND FIELD WORK

B.1. Initial site visit

B.1.1. Introduction

Throughout the study a number of site visits to Wilderness were necessary in order to gain data which was required for input into the modelling simulation. The primary aims of the initial site visit were to establish reliable cross sections of the Touw River estuary, Serpentine River, the three coastal lakes as well as the interconnecting channels between the lakes. Furthermore, discharge data from the Touw River, as well as water levels and salinity levels in all dominant water bodies were also important outcomes of the site visit. The initial site visit took place from the 05 December 2012 to the 14 December 2012 and a summary of the visit is described below.

B.1.2. Summary of Activities performed

The first two days of the site visit involved meeting with members of SANParks. The primary aim of these meetings was to introduce the study to all interested and affected parties within the area. Additionally, these meetings were also used to evaluate whether SANParks had any valuable data in existence that would benefit the study, or whether any additional data collection could be performed to strengthen or support their data records.

SANParks were extremely generous and helpful and provided historical records of flow data from the Touw River, water levels in all major water bodies in Wilderness as well as historical records of salinity levels within all major water bodies. Furthermore, SANParks provided a 1m elevation contour map of the entire system. This elevation map is from a study performed by the CSIR in 1981 (CSIR, 1981).

After analysing the data received from SANParks, it was decided that the remainder of the site visit would focus on data collection activities that had not already been performed by SANParks. This was done to ensure that time spent on site was utilized efficiently, and that there was no unnecessary duplication of data. It was also concluded that the elevation contour map of the system was outdated, specifically near the lower reaches of the river at the estuary mouth. Due to the fact that the mouth of the estuary is extremely dynamic in Wilderness, the bathymetry and cross sections in this area are constantly changing. It was, therefore, decided that the primary objective of the site visit would be to focus on the establishment of cross sections in the lower reaches of the system, namely the Touw River estuary and the Serpentine River. The coastal lakes and upper reaches of the system are less dynamic and are more likely to remain constant over longer periods of time. Therefore, the elevation contour map performed by the CSIR in 1981 would be sufficient to evaluate bathymetric data in these water bodies.

Furthermore, an observation was performed on the techniques used by SANParks staff to monitor the sandbar elevations at the estuary mouth, as this is an important procedure in the management plan of the system. The procedure performed by SANParks staff involved surveying the highest level of the sand berm seaward of the estuary. The highest level was identified from a visual inspection. Following this, the water edge at the lowest reach of the estuary was also surveyed. All surveyed points were benchmarked to a known +3m MSL benchmark located below the N2 road bridge over the Touw River estuary. Photographs of the procedure can be seen in the following figure.

Figure B-1: Survey of sand bar with SANParks officials

As the field work study progressed in Wilderness, a concern became evident regarding the real elevations of benchmarks used in the area. Elevations of some benchmarks seemed unrealistic and little correlation existed between different benchmarks. As a result, it was decided to survey points with known elevations within Wilderness to establish the accuracy of the GPS and also examine whether an overall correlation between known benchmarks is evident. The surveyed positions include the +3m MSL benchmark below the N2 road bridge and the trig beacon situated next to the N2 highway near Island Lake.

B.1.3. Apparatus

The data collection activities performed in the area primarily involved the establishment of cross sections of the water course at different chainages along the water course. The areas of focus included the Touw River estuary and the Serpentine River. For the completion of this task a combination of bottom profile and land surveying equipment was necessary in order to compute the channel bottom profile and flood plains respectively. The following sets of apparatus were used to collect data on the initial site visit:

- Lowrance G4 Fishfinder
- Trimble GeoXH Land surveying equipment
- Canoe and Paddle
- Pen and Paper
- Camera

B.1.4. Site Conditions during time of visit

The recognition of external site conditions at the time of the site visit is important, as certain data may differ according to the surrounding environment. One of the greatest external factors at the time of the site visit was the time-based condition of the estuary mouth. This has a significant impact on the overall operations of the entire system and affects the flow velocity and discharge of water throughout the system. At the time of the site visit, the mouth had recently been closed and the sandbar in front of the estuary mouth was recorded at +1.53m MSL, according to a SANParks survey performed during the time of the site visit. The condition of the estuary mouth also affects cross sectional data of the estuary in sections near the estuary mouth. Sediment accumulates in this area when the mouth is closed, hence creating a shallower cross sectional profile compared to an open mouth condition.

B.2. Second Site Visit

B.2.1. Introduction

The primary aim of the second site visit was to establish the correct level of the water level recorders with reference to Mean Sea Level. This was performed based on prior analysis of the water level data and finding discrepancies between the water level data. Due to the fact that the water level recorders were not all referenced to Mean Sea Level or to the same datum during installation, correction factors exist in order to convert all the water level data from different water bodies to the same datum, Mean Sea Level. The duration of the site visit was one day on the 15 March 2013.

B.2.2. Summary of Activities performed

A meeting was scheduled with SANParks where the correction factors were obtained.
However, correction factors were only available until the year 2000. The year 2000 correction factors were applied to water levels in the following years and the results looked favourable, which indicated that very little vertical shift had occurred to the water level recorders in the subsequent years. However, in order to test this hypothesis, a survey was performed on all water level recorders to evaluate the accuracy of the correction factors for water levels after 2000. All water level recorders were surveyed except the recorder at Bolangvlei. This recorder was inaccessible at the time of the survey.

B.3. Uncertainty Analysis

B.3.1. Introduction

An uncertainty analysis was performed on the data acquired throughout the site visits. An analysis of the data showed that parts of the data seemed unfitting and unrealistic which yielded areas of uncertainty and questioned the accuracy and validity of the data. Therefore, an uncertainty analysis of various components linked to the data collection is discussed below.

B.3.2. Survey Equipment

The equipment used for survey purposes on site included a Trimble GeoExplorer 6000 series GeoXH handheld GPS surveying device (from now on referred to as GPS) and a Lowrance X4 fish finder (from now on referred to as echo sounder). These apparatus can be seen in the following figure.

Figure B-2: Equipment used during site visits.
GPS on the left and echo sounder on the right

These sets of apparatus were used in accordance with one another in order to calculate the cross sections of various components of the water course. The echo sounder was used to record water depths along the width of the water course and the GPS was used to survey flood plains on either side of main water channel.

B.3.3. Terrain Conditions

The floodplains of the water system comprised a number of different terrain and vegetation conditions, including grassy banks, reeds, marshland and steep cliffs. Despite decent efforts to survey as much of the flood plain as possible, some areas of the floodplain were simply inaccessible and could not be reached on foot with the GPS surveying device. In these cases, either an observable estimate was made or in the case of flat marshlands, such as the Serpentine, an estimated constant slope was assigned as the floodplain topography. In cases where the flood plain was inaccessible, the water edges on either side of the main channel were still recorded by the GPS device to establish a water level above Mean Sea Level and correlate the echo sounder data to this datum.

B.3.4. Survey Sources

A combination of three different sources or apparatus was used in order to obtain the cross sectional data for the entire system. These include the GPS surveying equipment, echo sounder and a one meter contour elevation chart performed by the CSIR in 1981 (CSIR, 1981). This elevation map can be seen in Appendix C. Due to time constraints; an accurate survey of the entire system could not be completed. It was, therefore, decided to focus any surveying work on the lower reaches of the system, including the estuary and Serpentine River, as these tend to be more dynamic over time. The lakes were thought to be less dynamic than the lower reaches of the system, and the bathymetry would remain fairly constant over the past 32 years, since the last survey performed by the CSIR in 1981. Due to this reason, cross sections for the lakes were evaluated from the one meter elevation contour map (CSIR, 1981) and cross sections for the estuary and Serpentine River were evaluated using a combination between GPS and echo sounder devices. The precision and unit intervals for each source are different. The contour elevation map shows one meter contours and, therefore, data gathered from this source has 1m unit intervals, the echo sounder has units of decimetres, relating to 0.1m unit intervals and the GPS is precise to the nearest millimetre, relating to 0.001m intervals.

APPENDIX C. CSIR ELEVATION STUDY OF WILDERNESS ESTUARY AND COASTAL LAKES

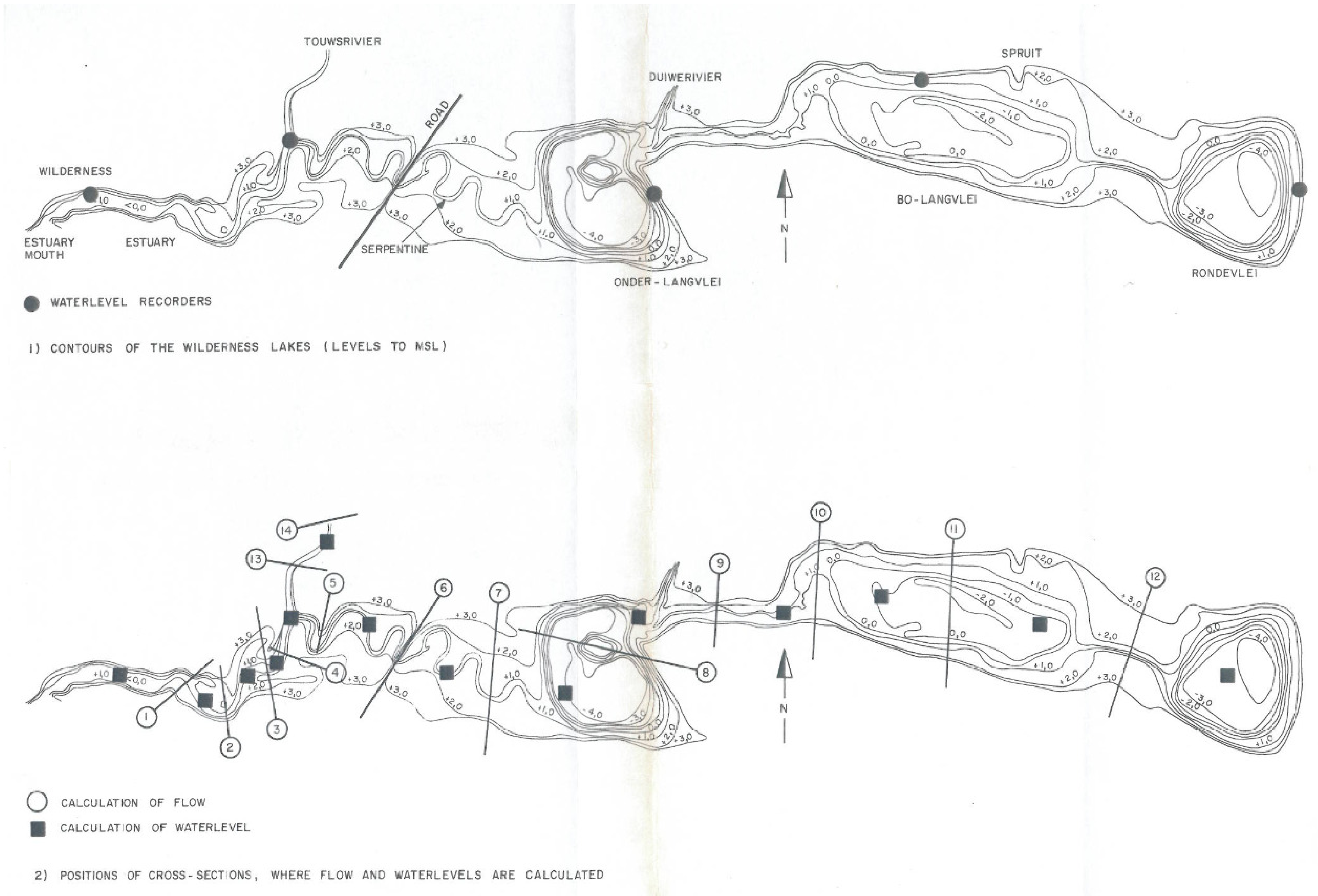

Figure C-1: CSIR Elevation study of Touw River estuary and Wilderness coastal lakes (CSIR, 1981)

APPENDIX D. SURVEY LOCATIONS

D.1. Introduction

The following figures show locations of the surveys performed in the Touw River estuary and Wilderness coastal lakes area. The place markers in the figures are either yellow or blue. Yellow place markers refer to locations where a survey involving cross sectional data was performed, whereas blue place markers refer to a location where a survey involving water level recorders or datum verification was performed.

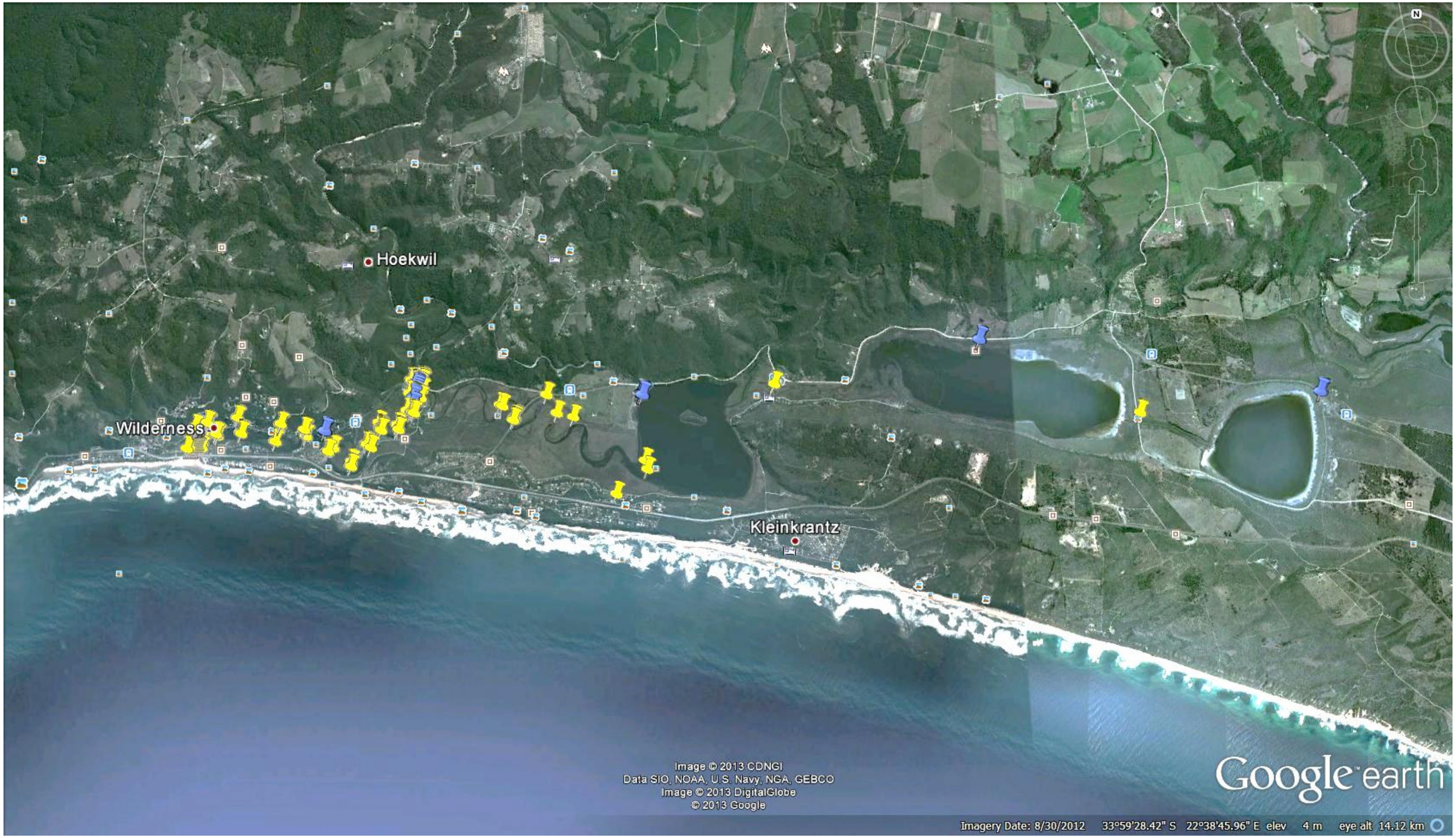

Figure D-1: Survey locations of entire Wilderness estuary and coastal lakes system

Figure D-2: Survey locations of Wilderness estuary

Figure D-3: Survey locations of Serpentine River

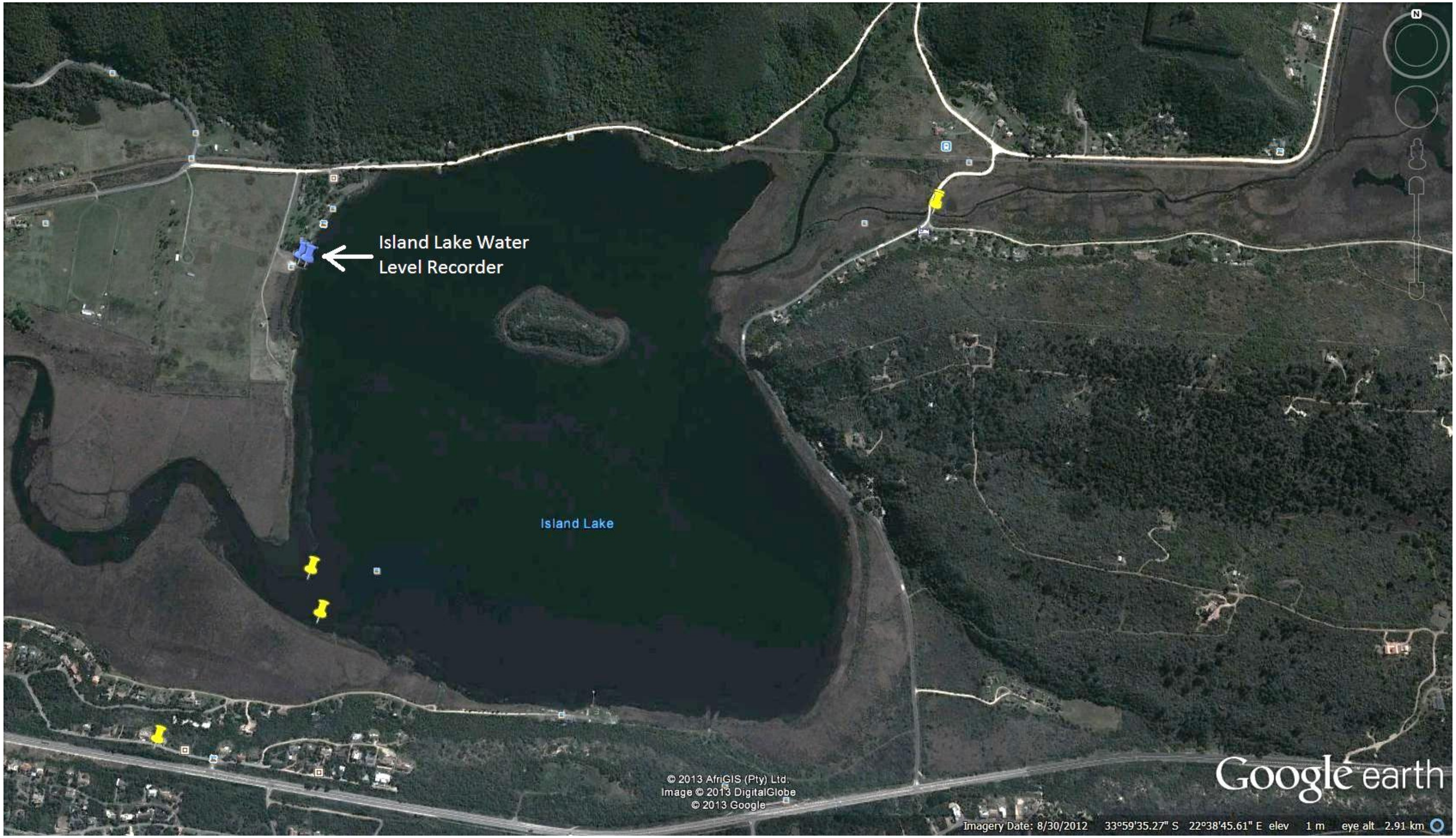

Figure D-4: Survey locations of Island Lake

Figure D-5: Survey locations of Bolangvlei

Figure D-6: Survey locations of Rondevlei

APPENDIX E. CROSS SECTIONS OF ENTIRE WATER SYSTEM

E.1. Introduction

The following figures in this Appendix show the cross sections used in the construction of the numerical model. The cross sections are arranged in a specific order, starting from the eastern most boundary at Rondevlei and ending at the ocean boundary past the Touw River mouth. A numerical notation has also been assigned to each cross section in this order. The figure on the following page shows the location of each cross section, illustrated by a white box with a red border and a black line passing through the box. Numbers are also assigned for every fifth cross section which provides an easy reference to the exact location of each cross section.

It must be taken into consideration that the scale of each cross section displayed in the following figures is not constant, as considerable variation differs in the cross section widths. Had a single scale been used for all cross sections, detailed data in cross sections with smaller widths would be illegible.

In the figures related to the individual cross sections, the cross sections are illustrated by the black line and the vertical level at each position along the cross section is depicted on the left vertical axis. The blue line shows the Manning n roughness coefficient for different sections along the width of the cross section and Manning n values are displayed on the right vertical axis. Lastly, the red circles illustrate the transition point from the main channel to the flood plain for both the left and right side of the cross section respectively.

Figure E-1: Plan View of the location of each cross section used in the numerical model

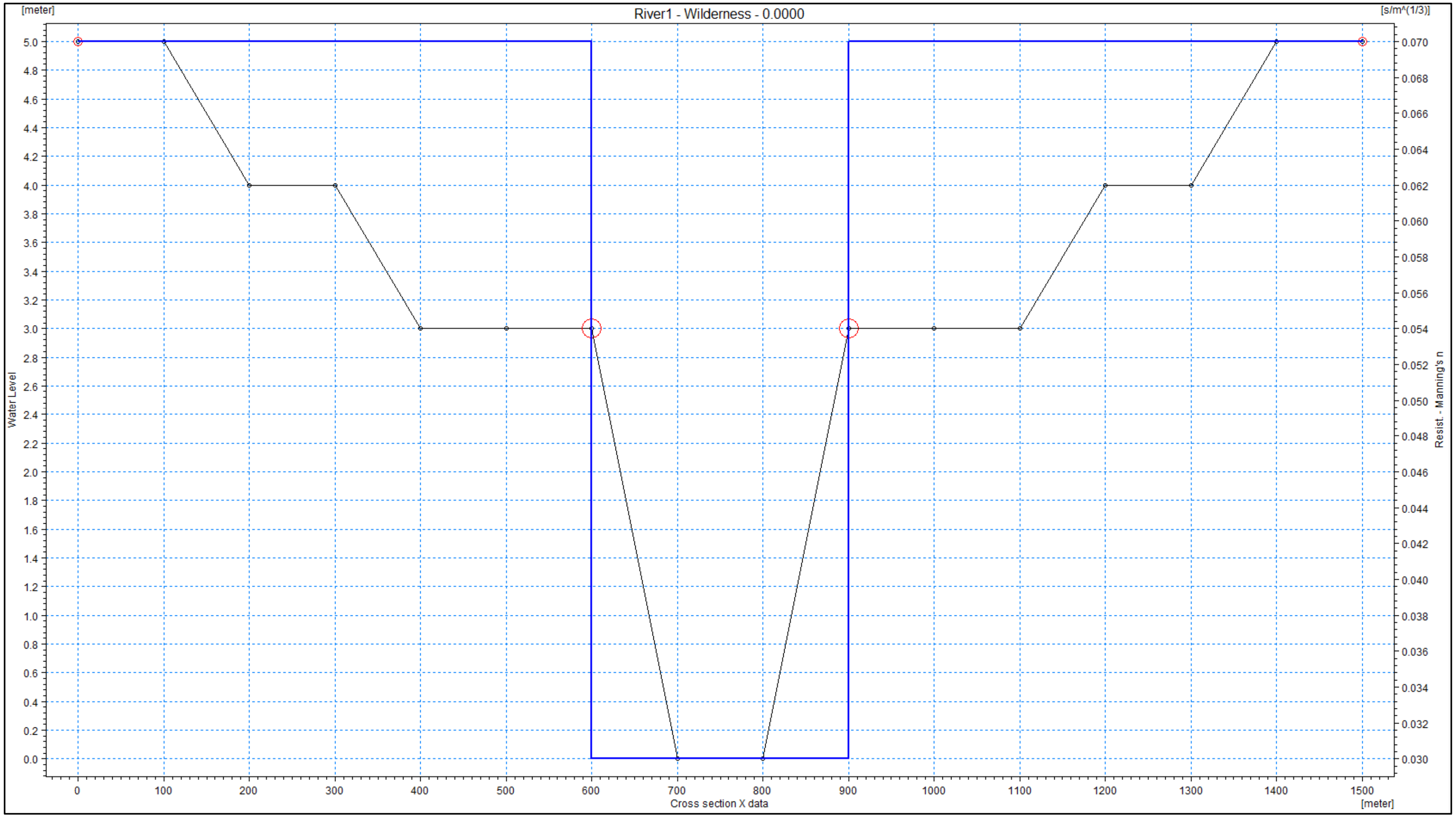

Figure E-2: Cross Section 1

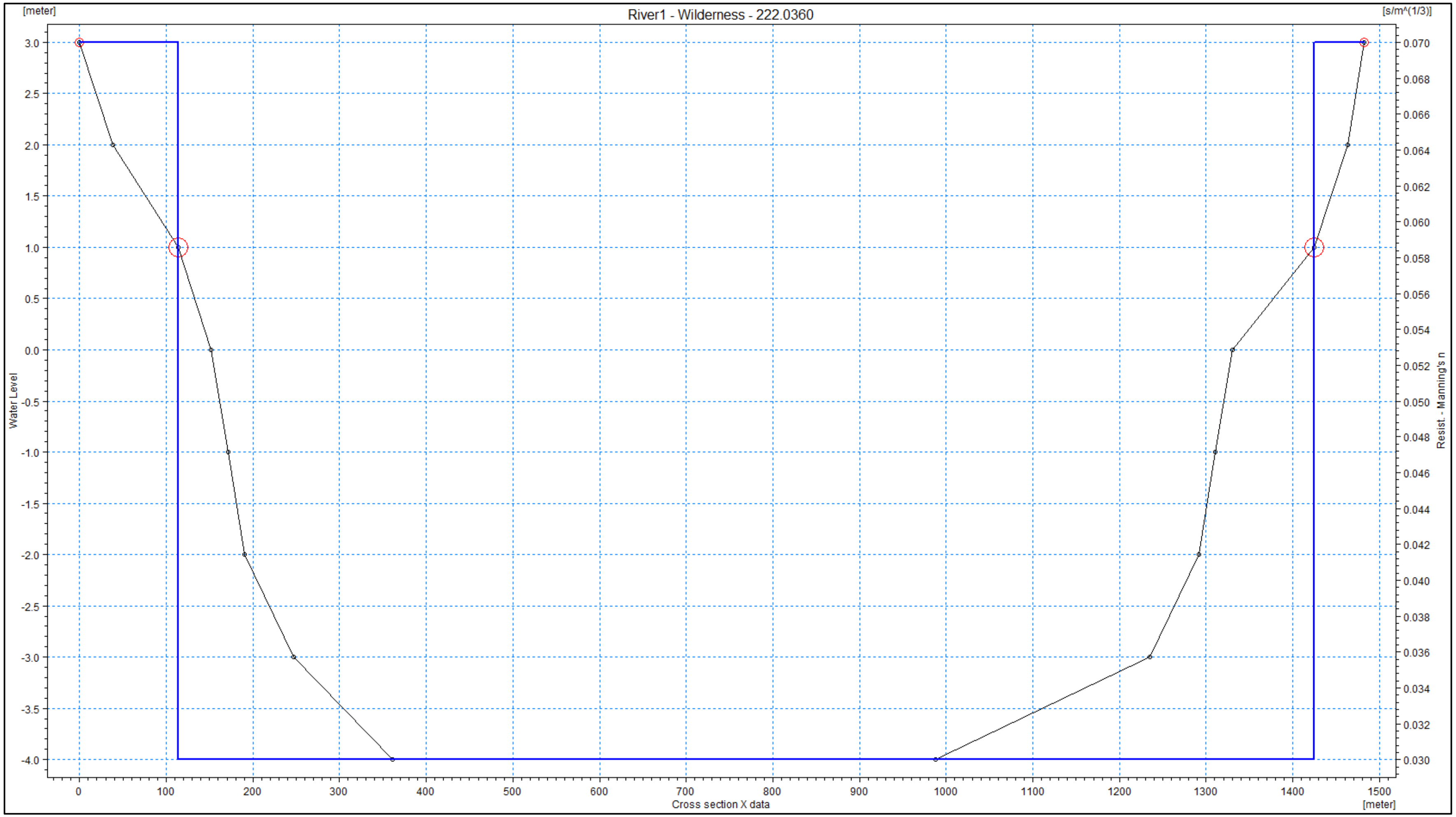

Figure E-3: Cross Section 2

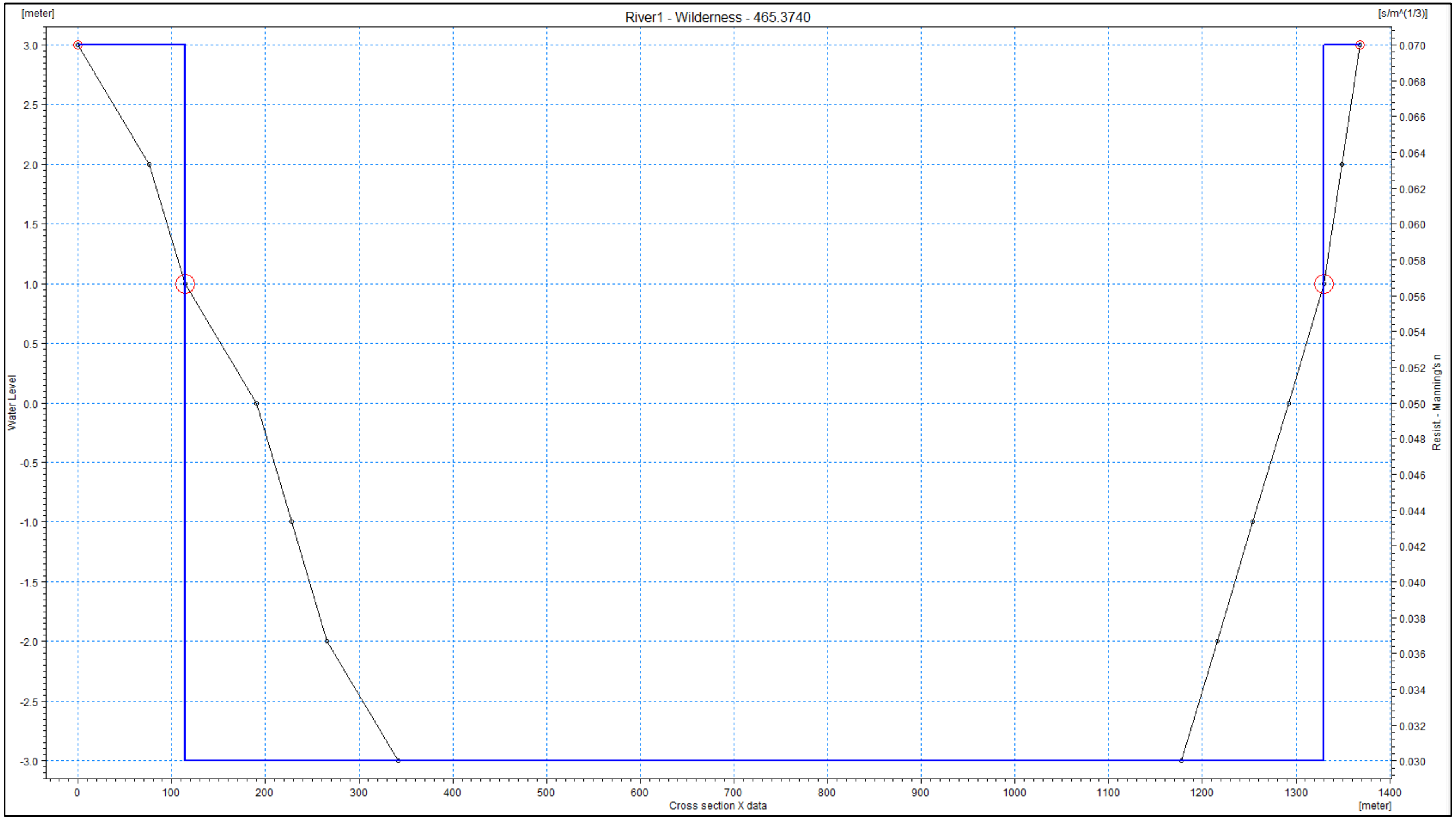

Figure E-4: Cross Section 3

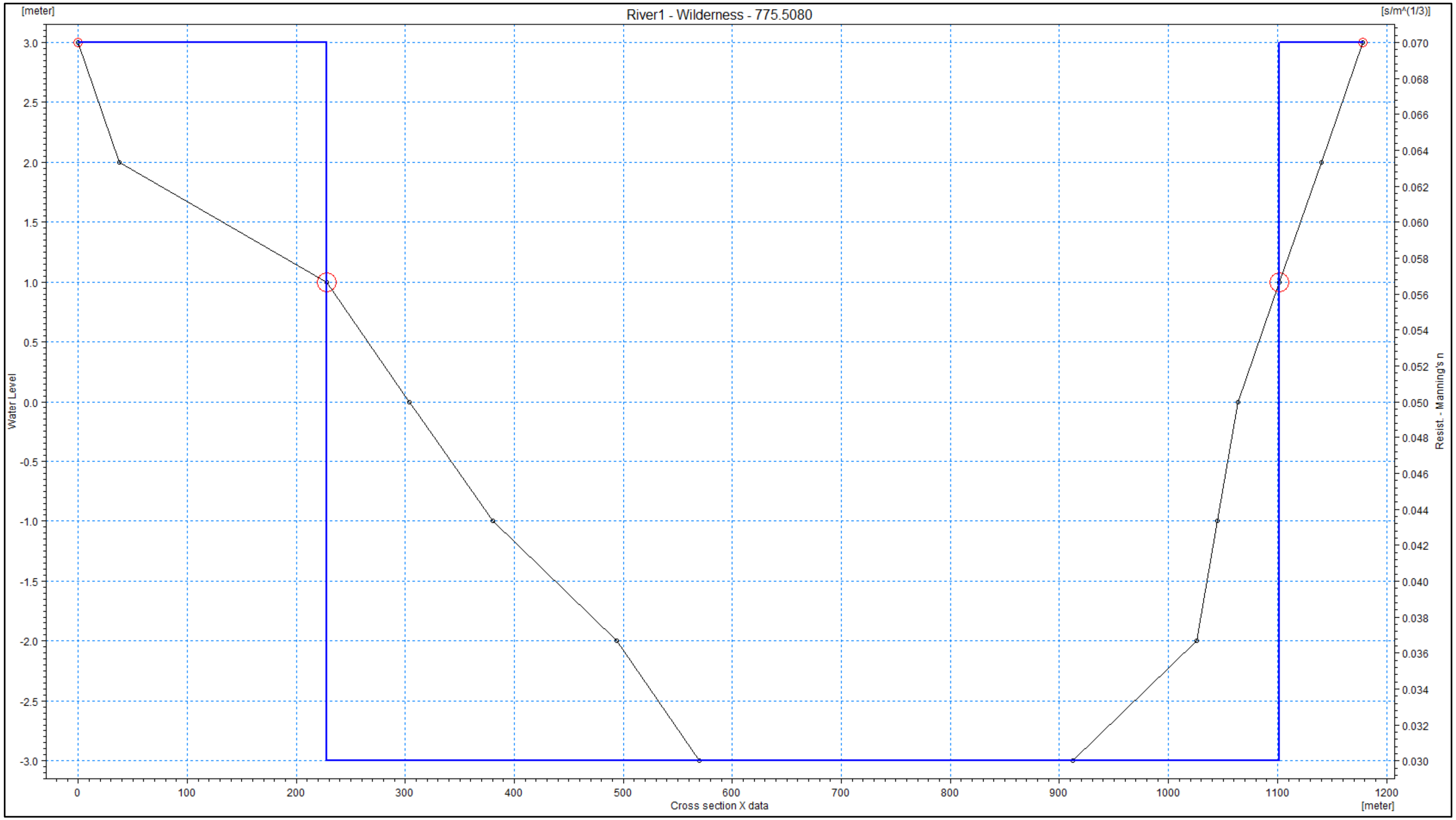

Figure E-5: Cross Section 4

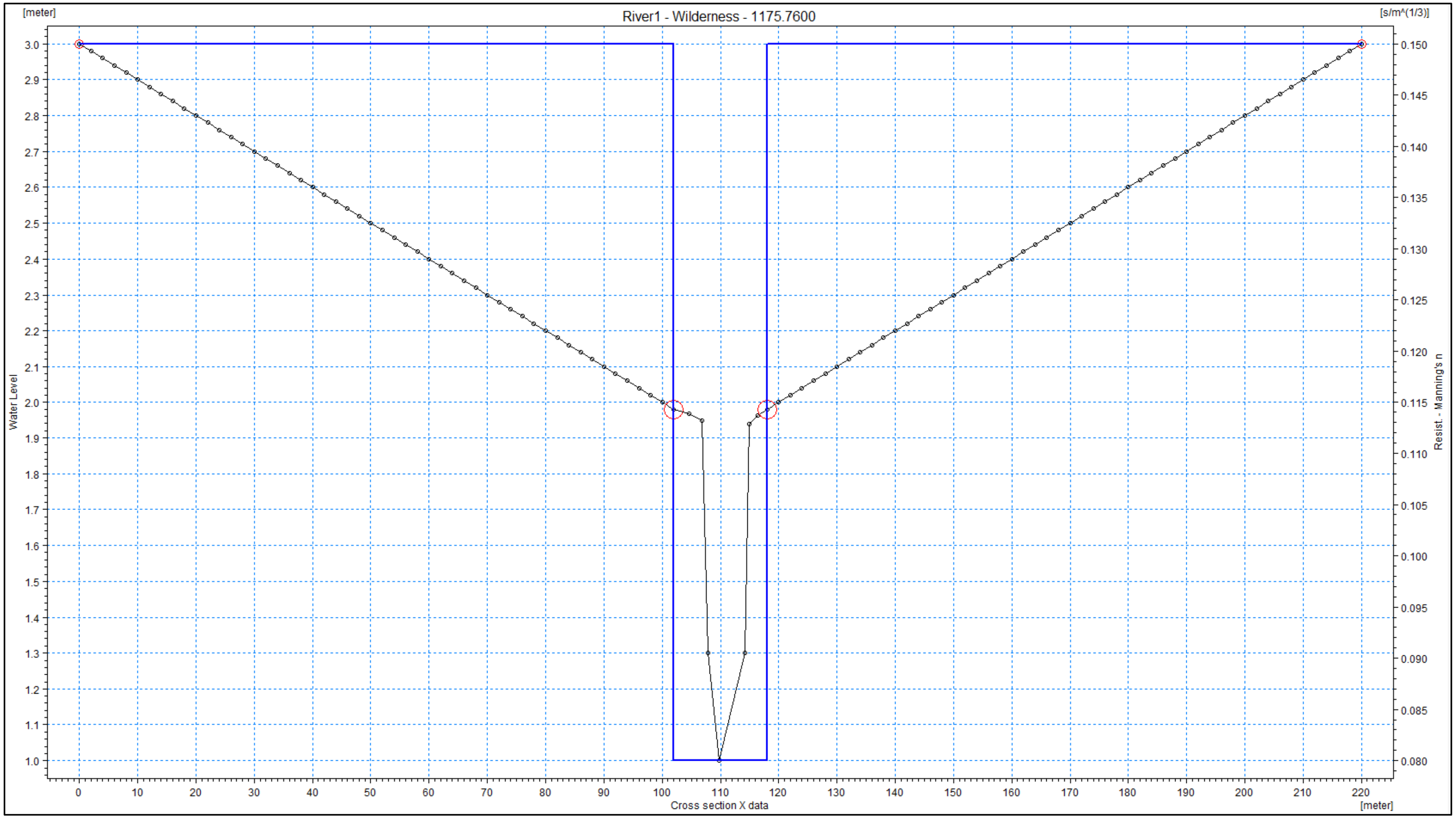

Figure E-6: Cross Section 5

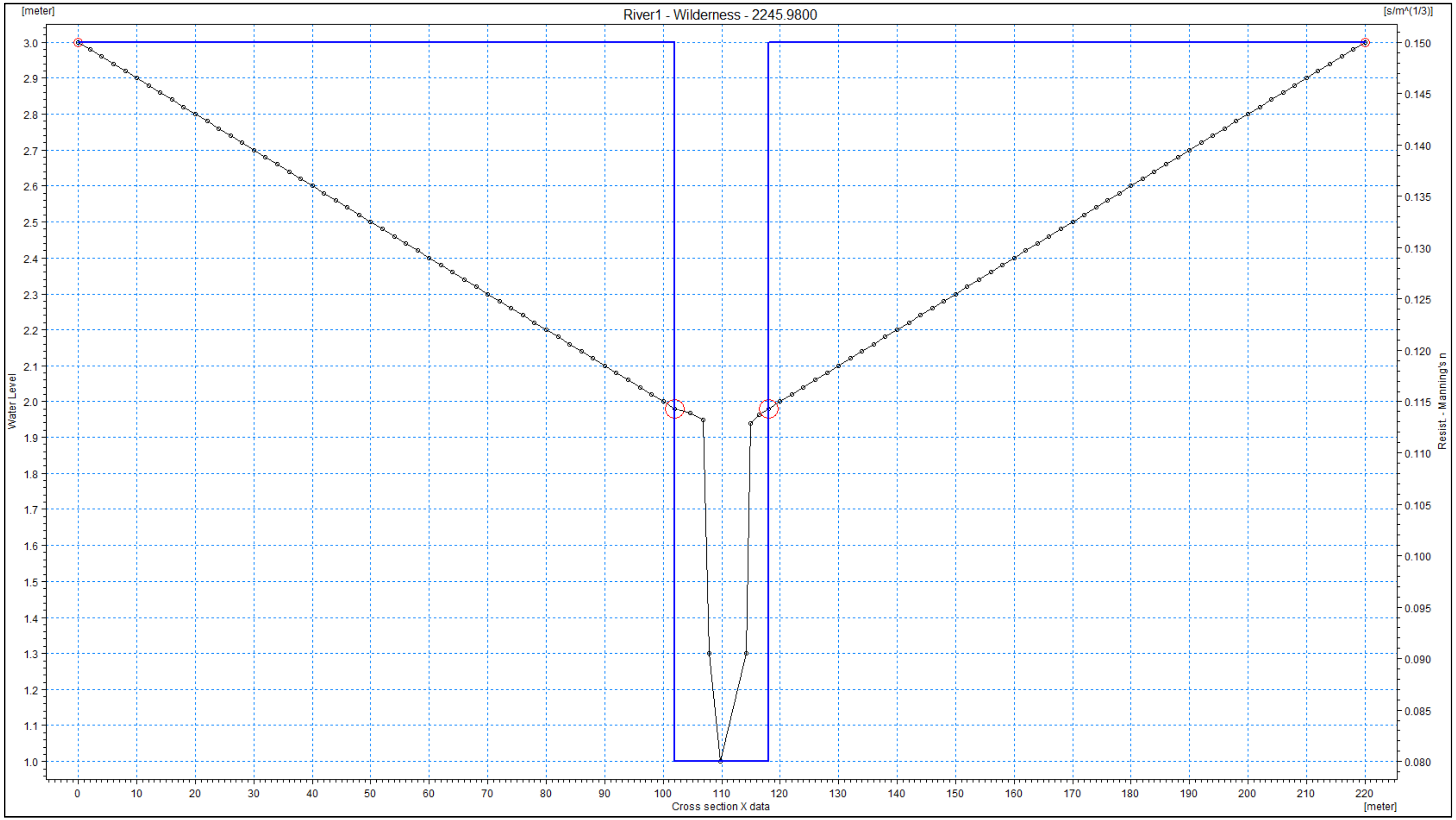

Figure E-7: Cross Section 6

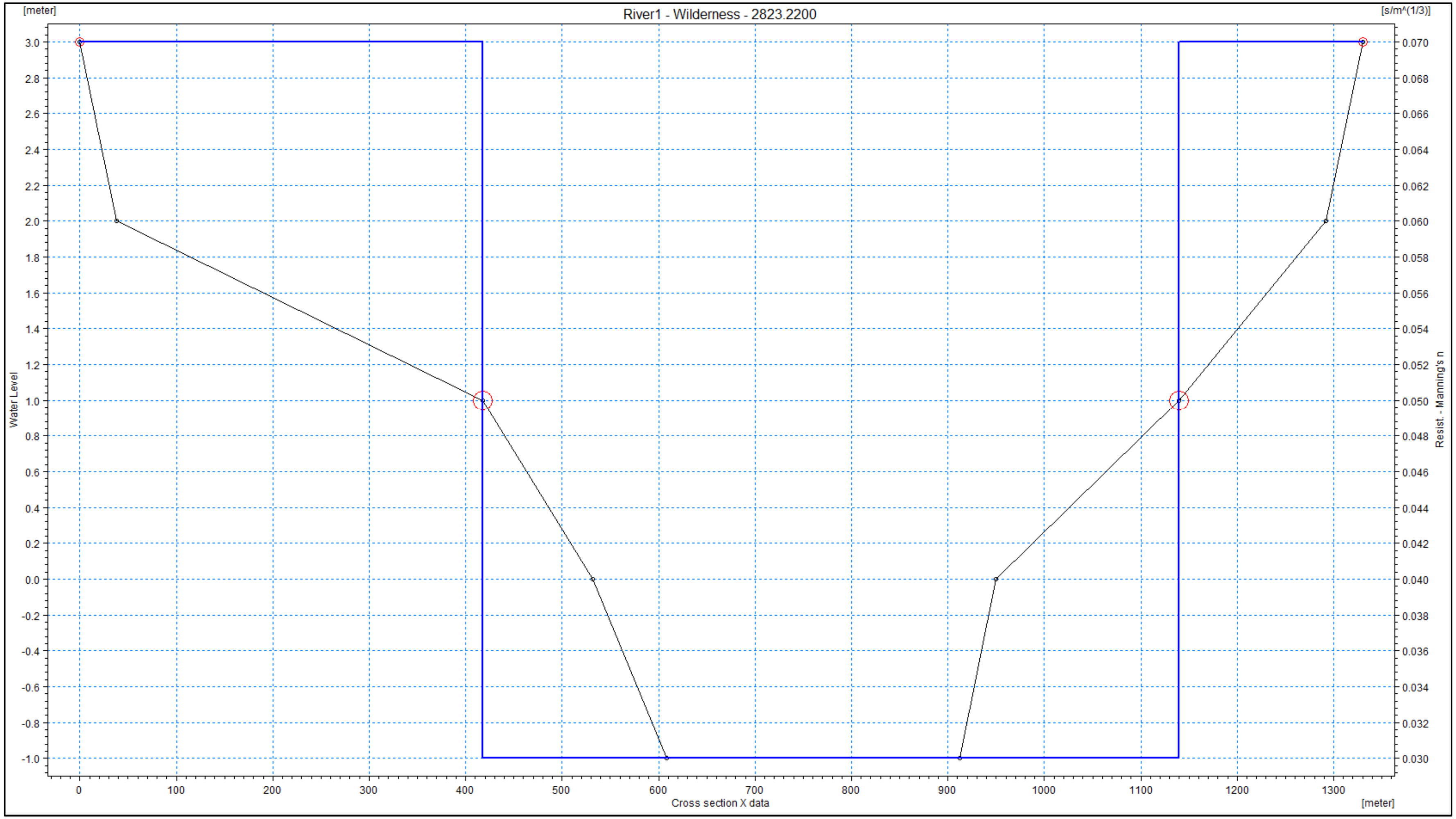

Figure E-8: Cross Section 7

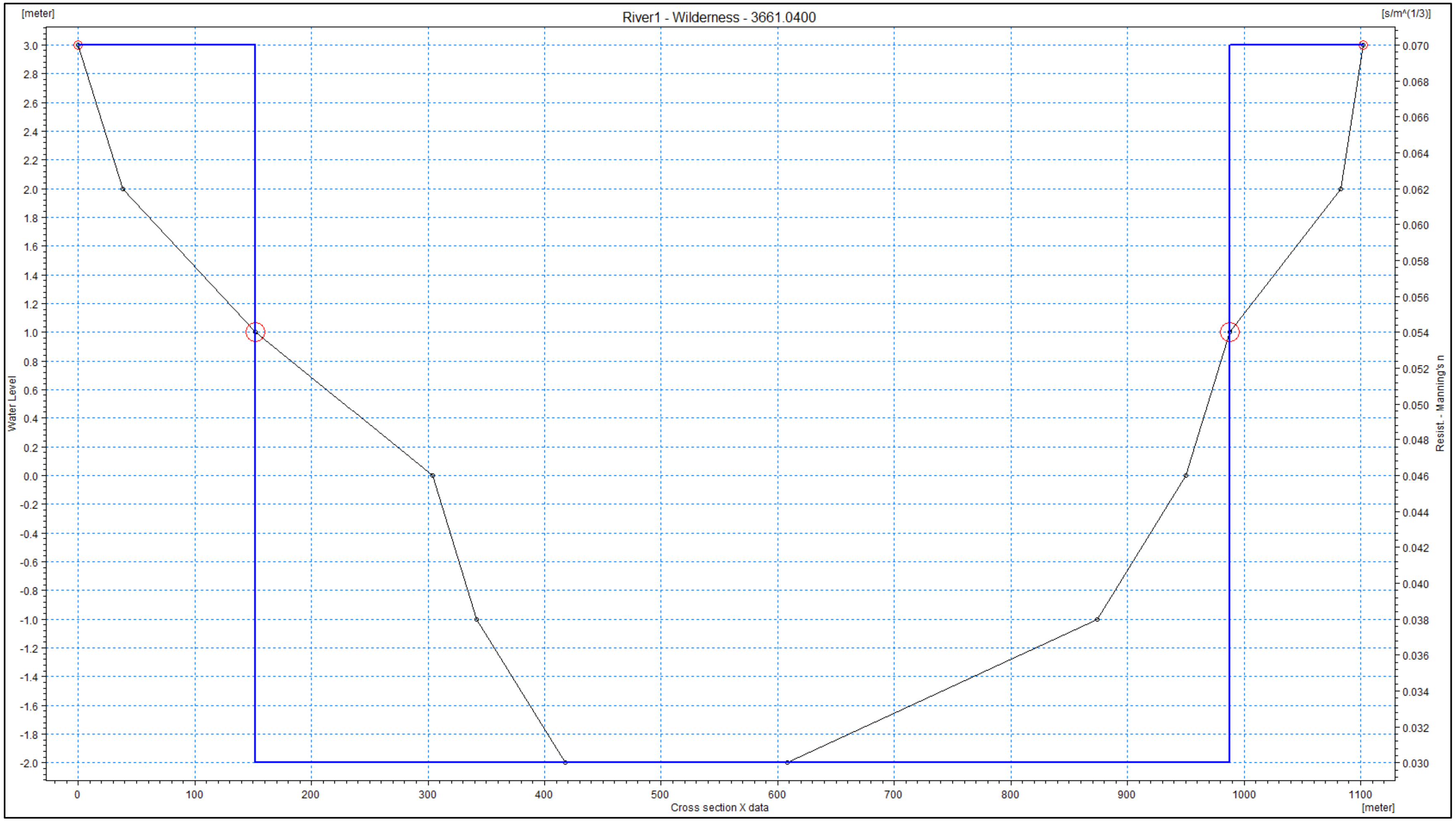

Figure E-9: Cross Section 8

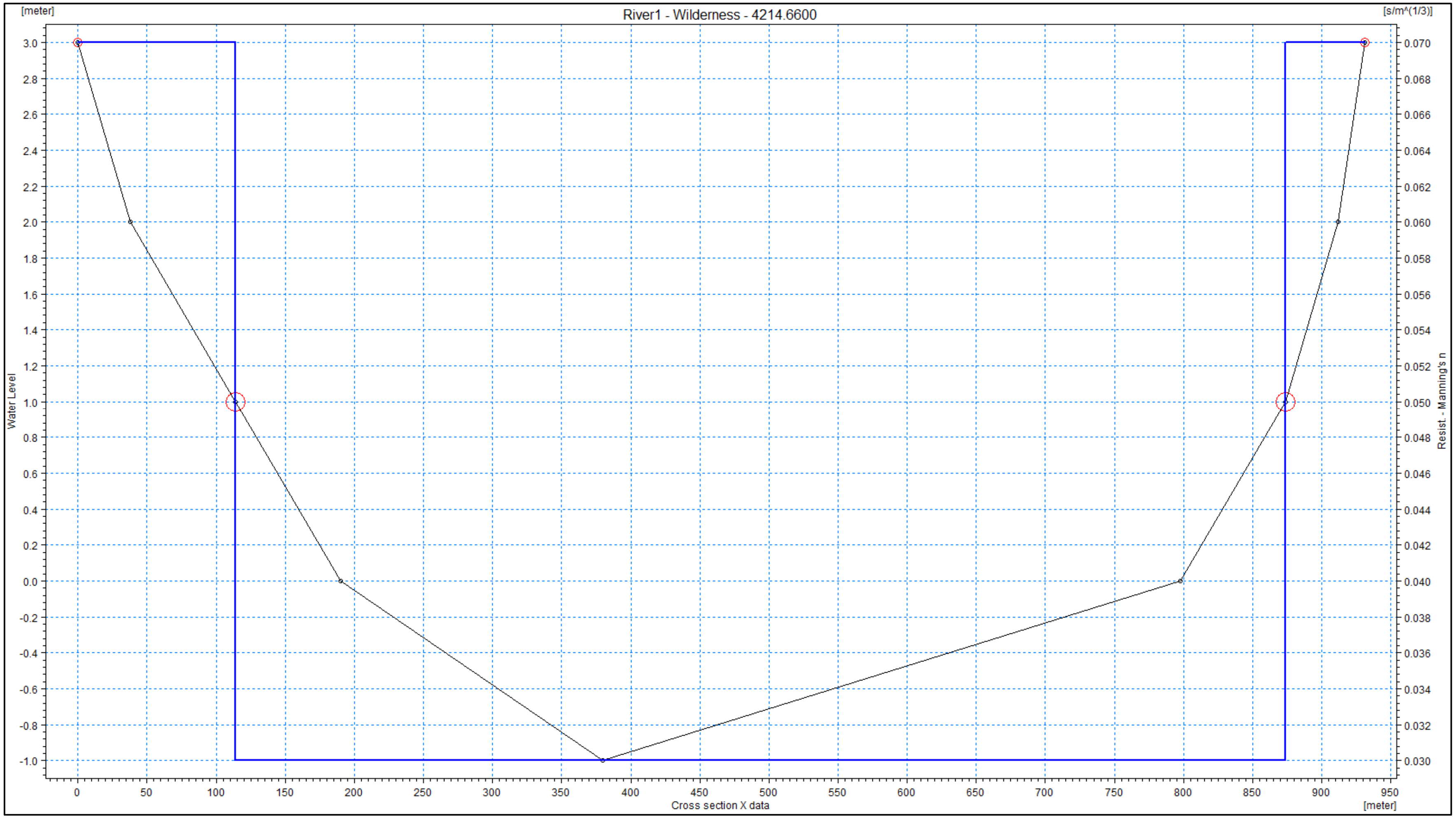

Figure E-10: Cross Section 9

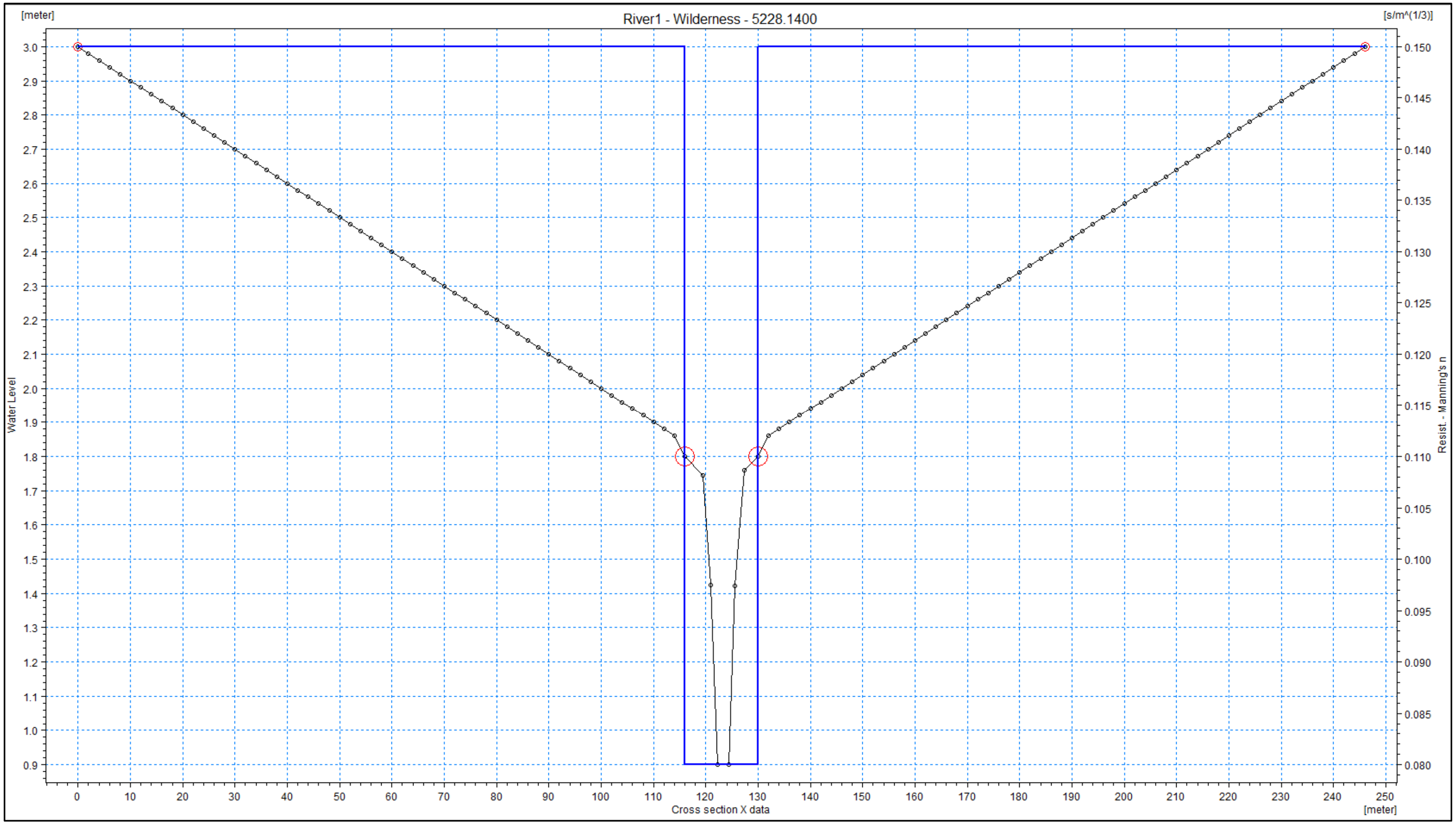

Figure E-11: Cross Section 10

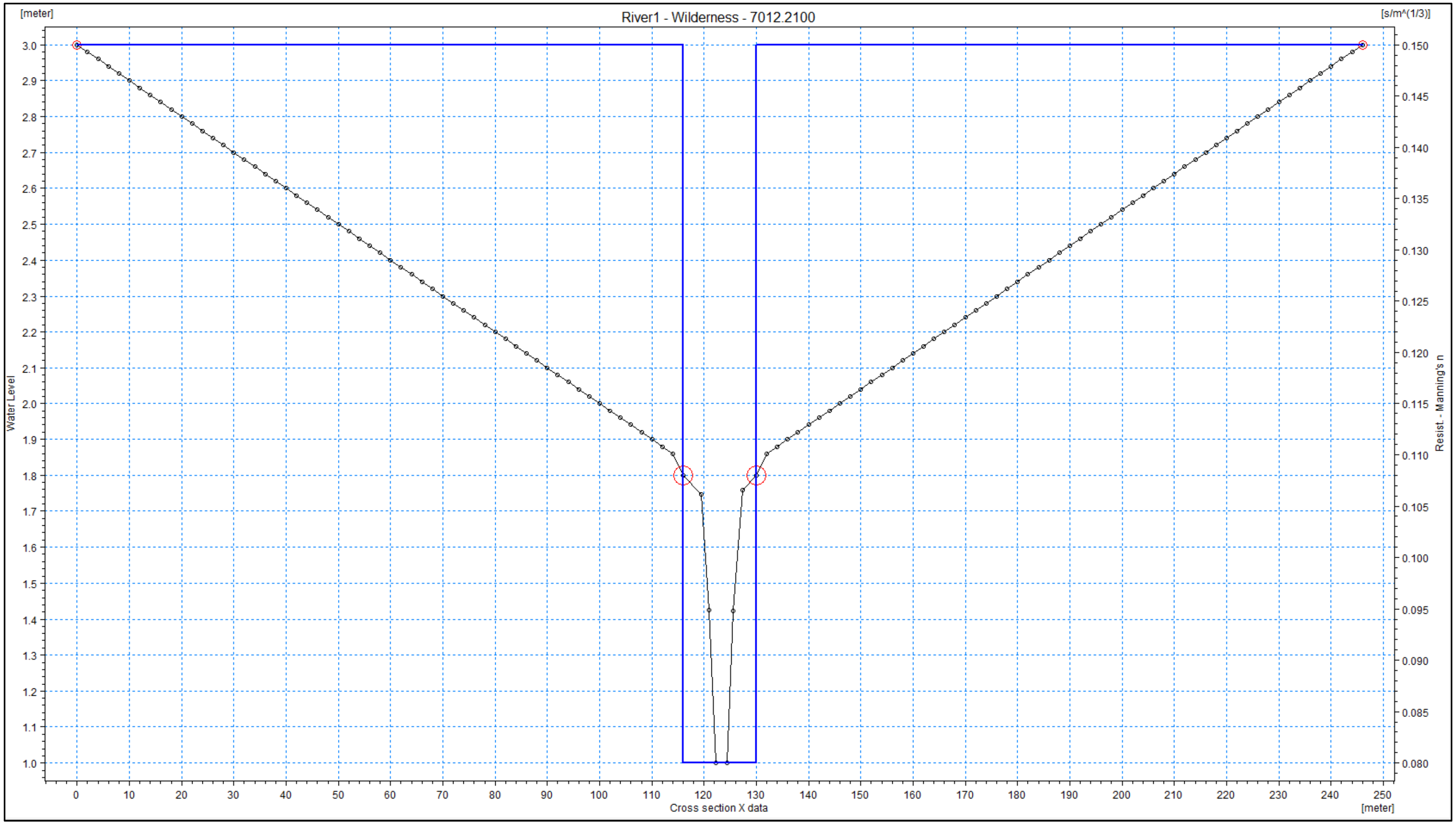

Figure E-12: Cross Section 11

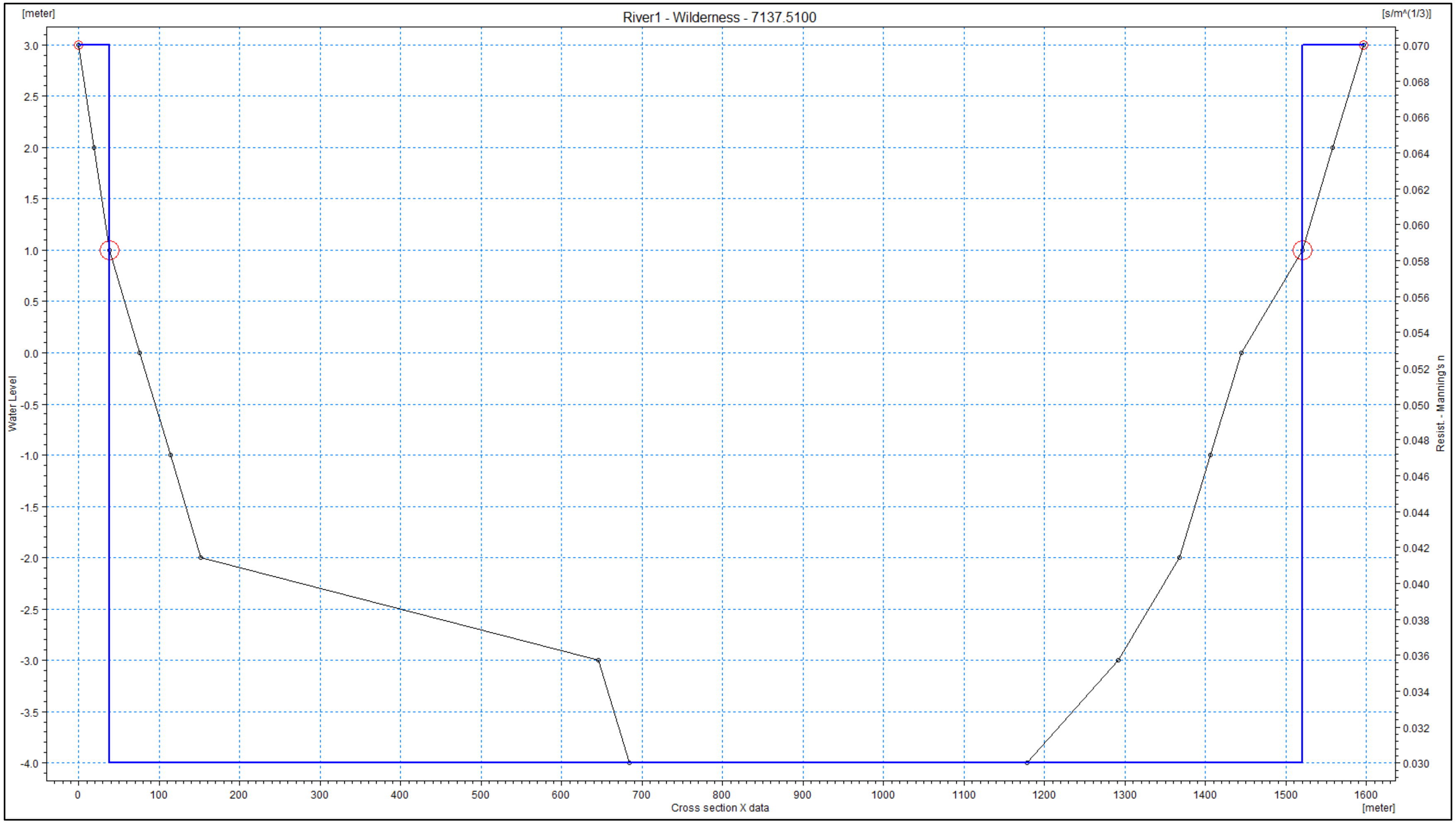

Figure E-13: Cross Section 12

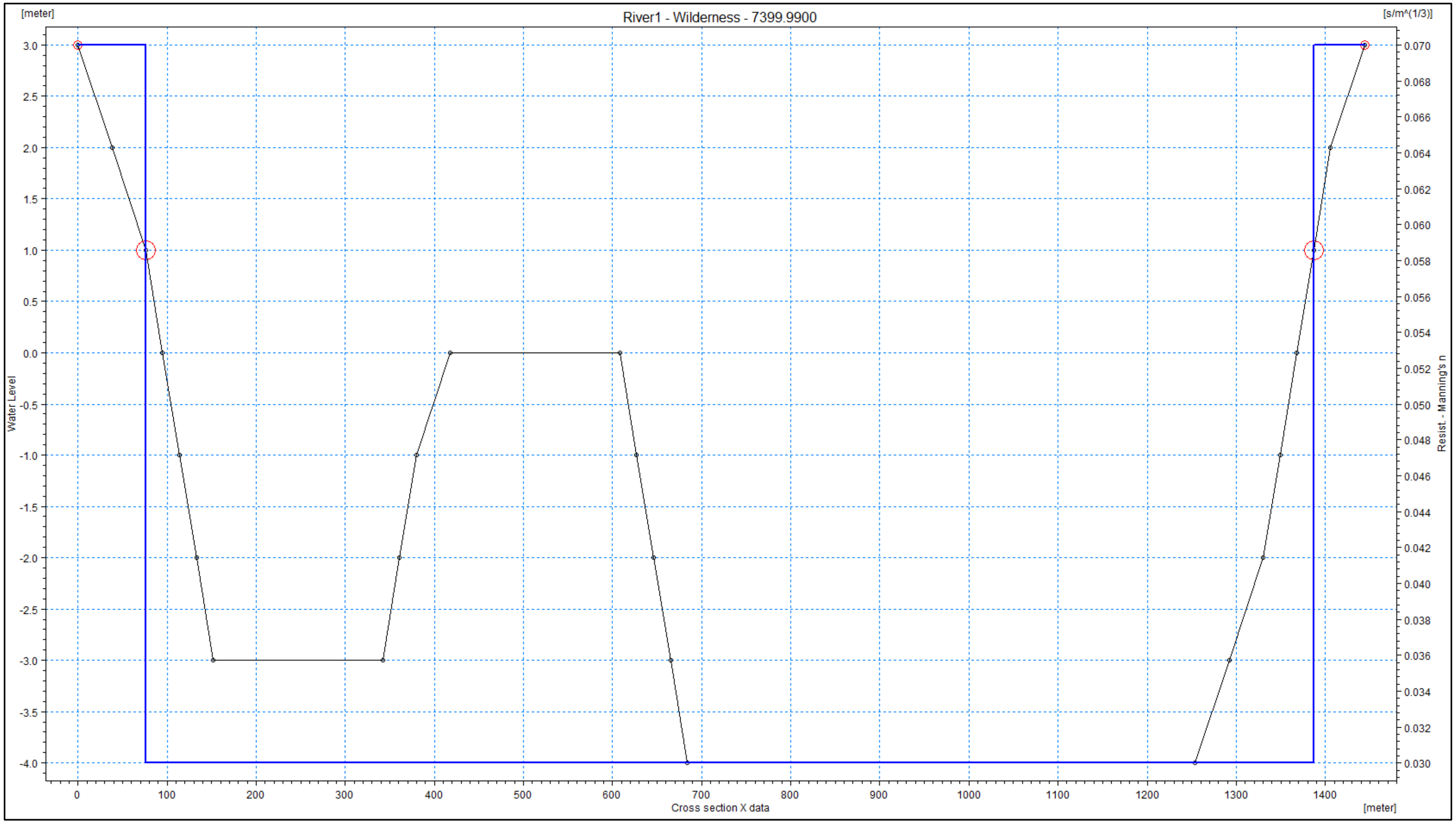

Figure E-14: Cross Section 13

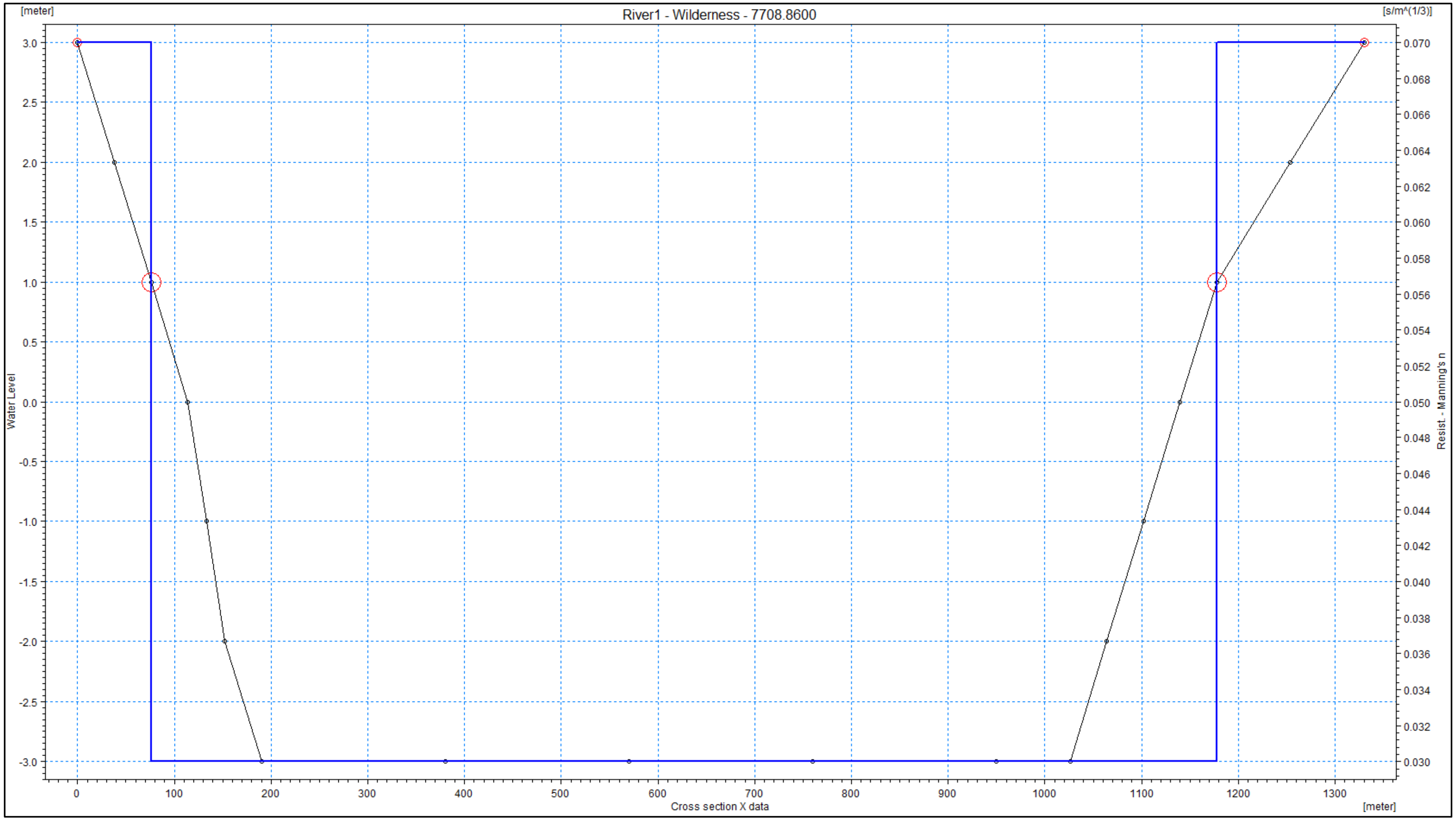

Figure E-15: Cross Section 14

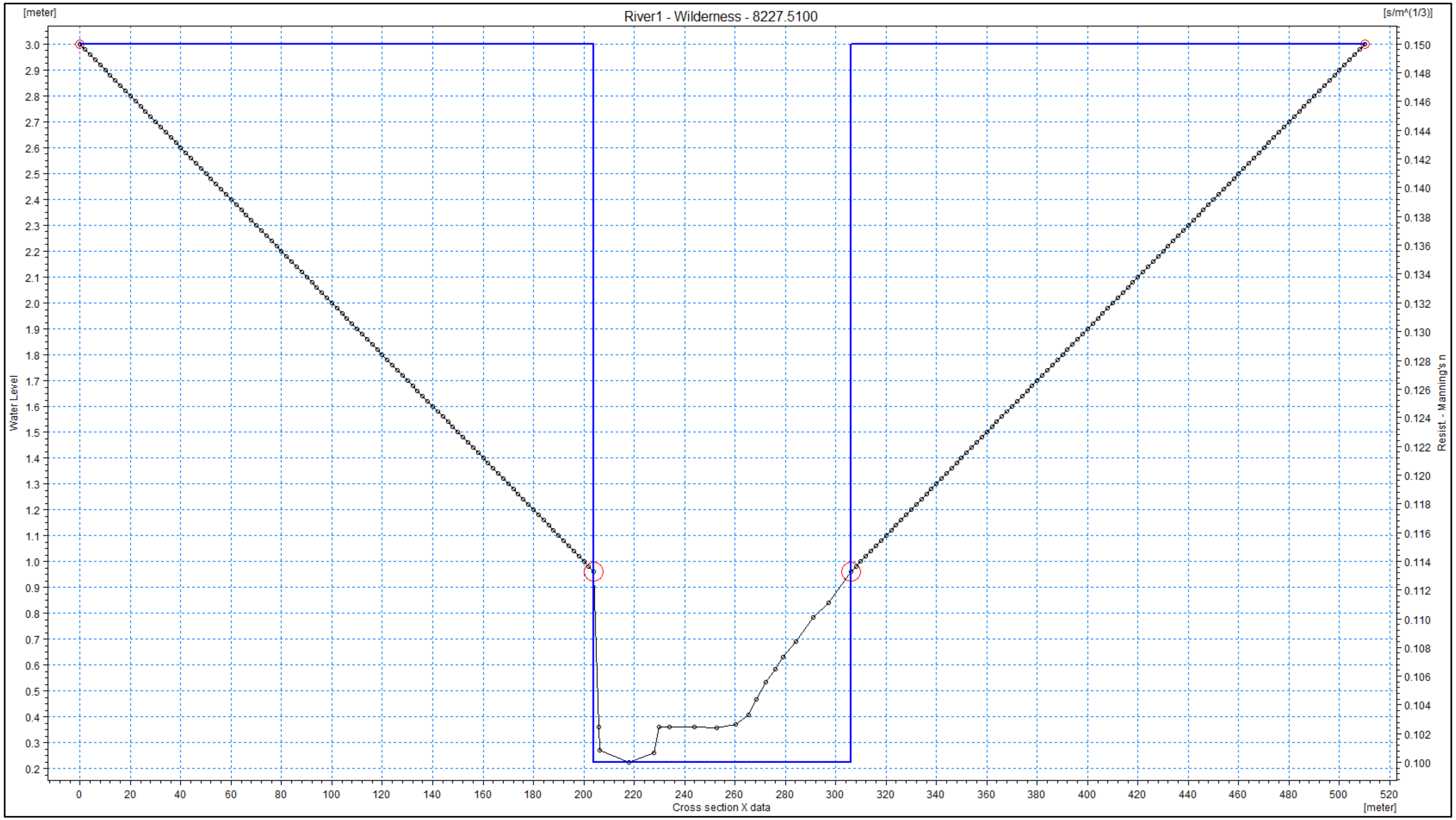

Figure E-16: Cross Section 15

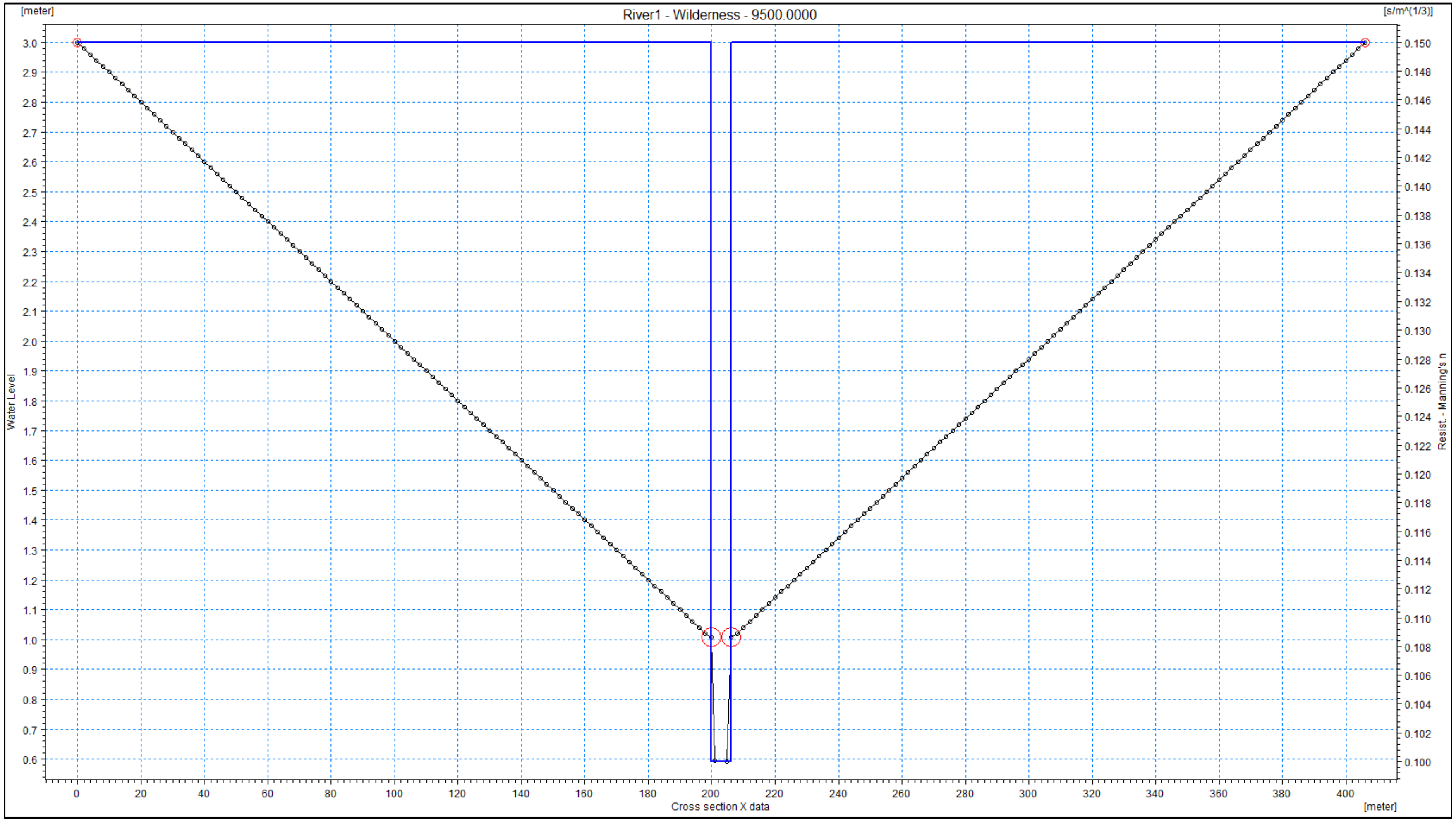

Figure E-17: Cross Section 16

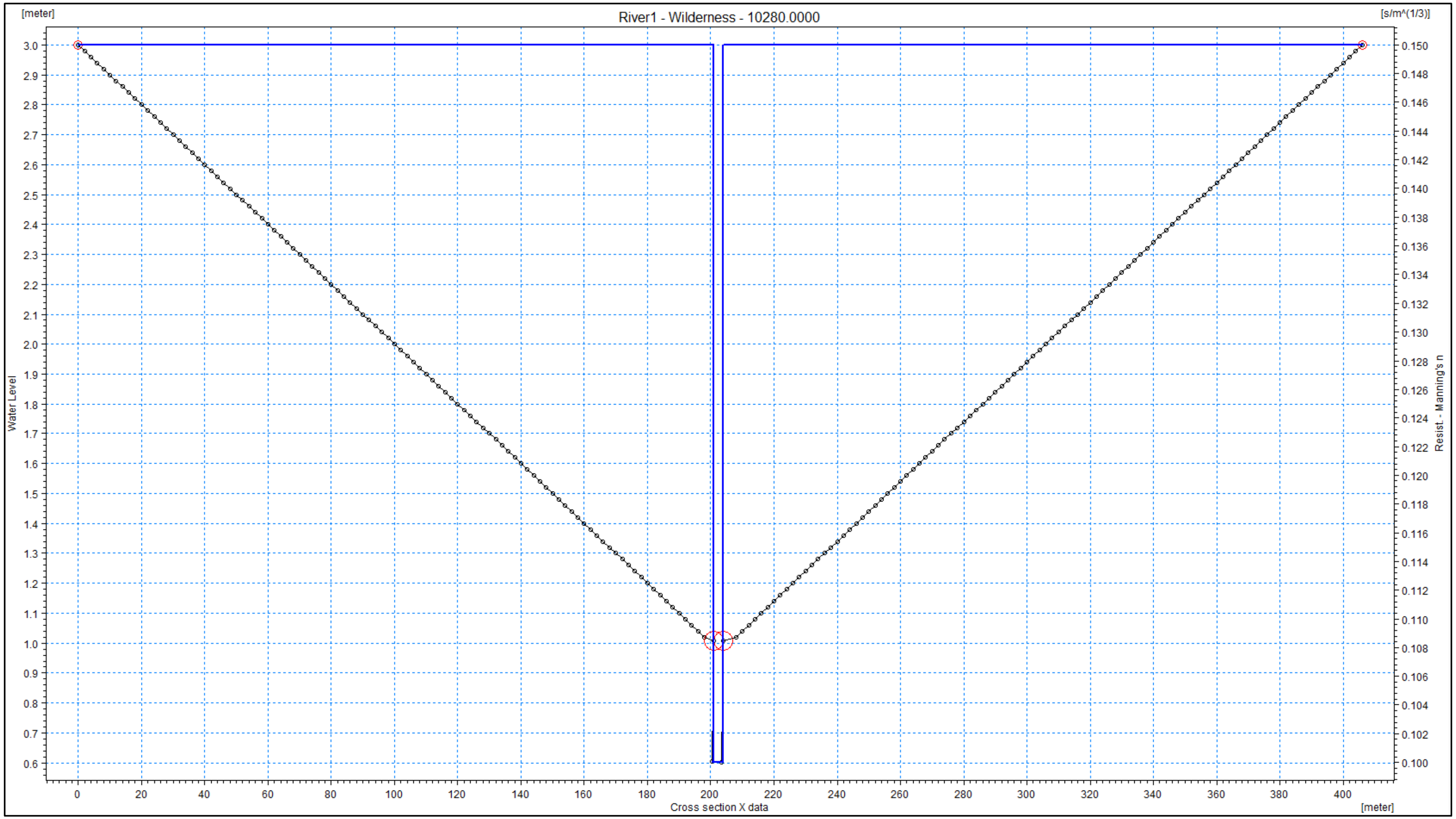

Figure E-18: Cross Section 17

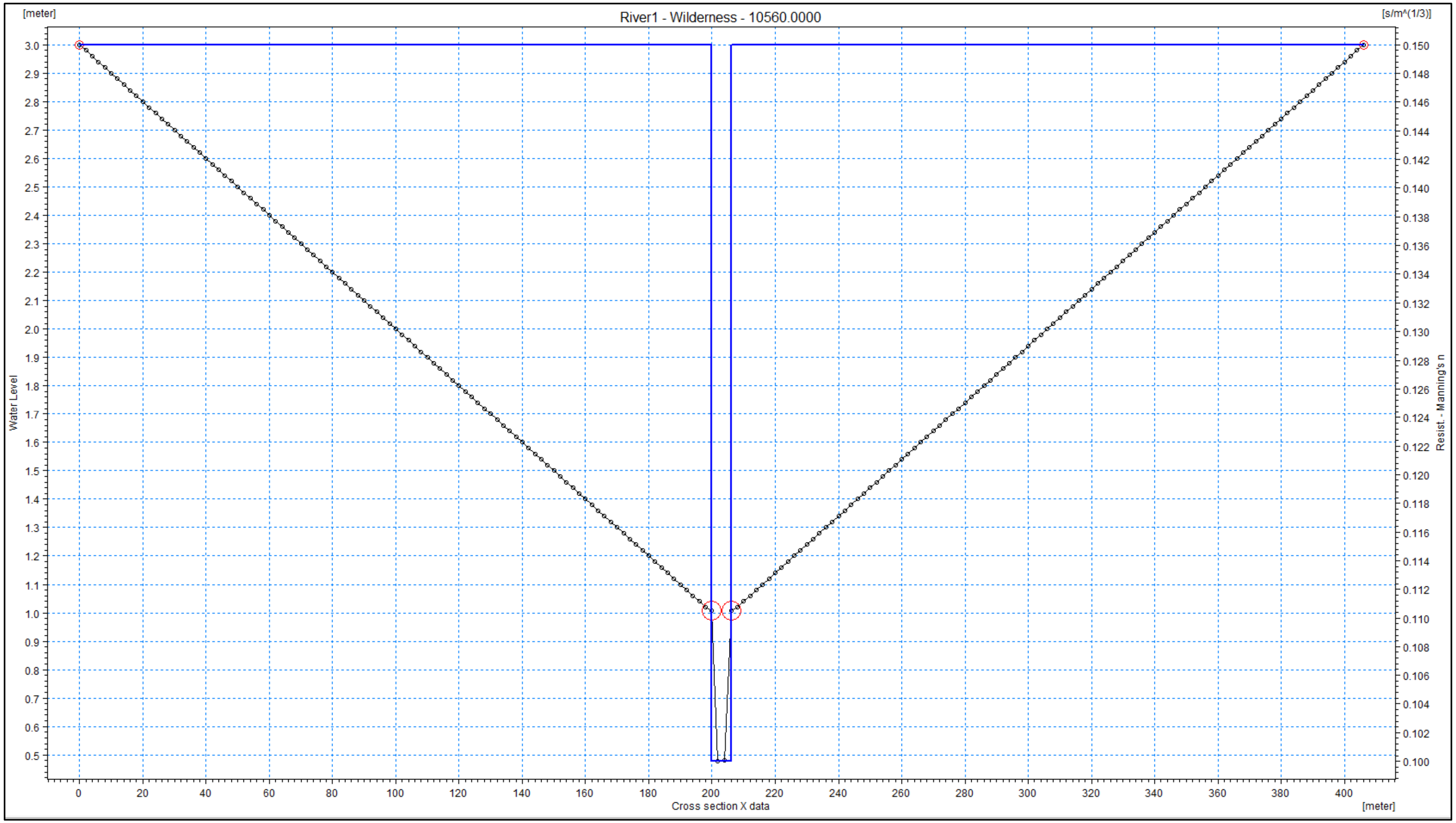

Figure E-19: Cross Section 18

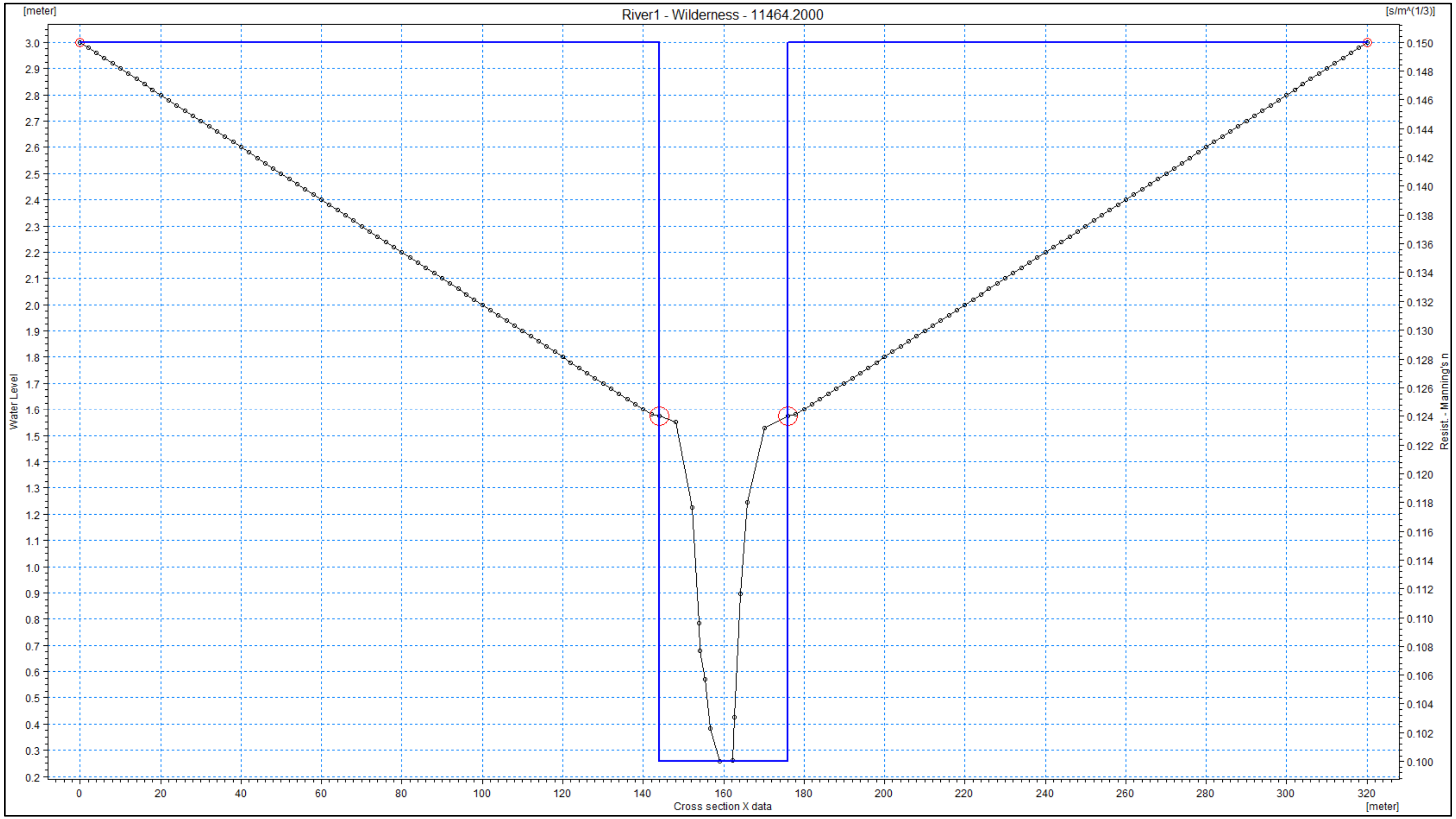

Figure E-20: Cross Section 19

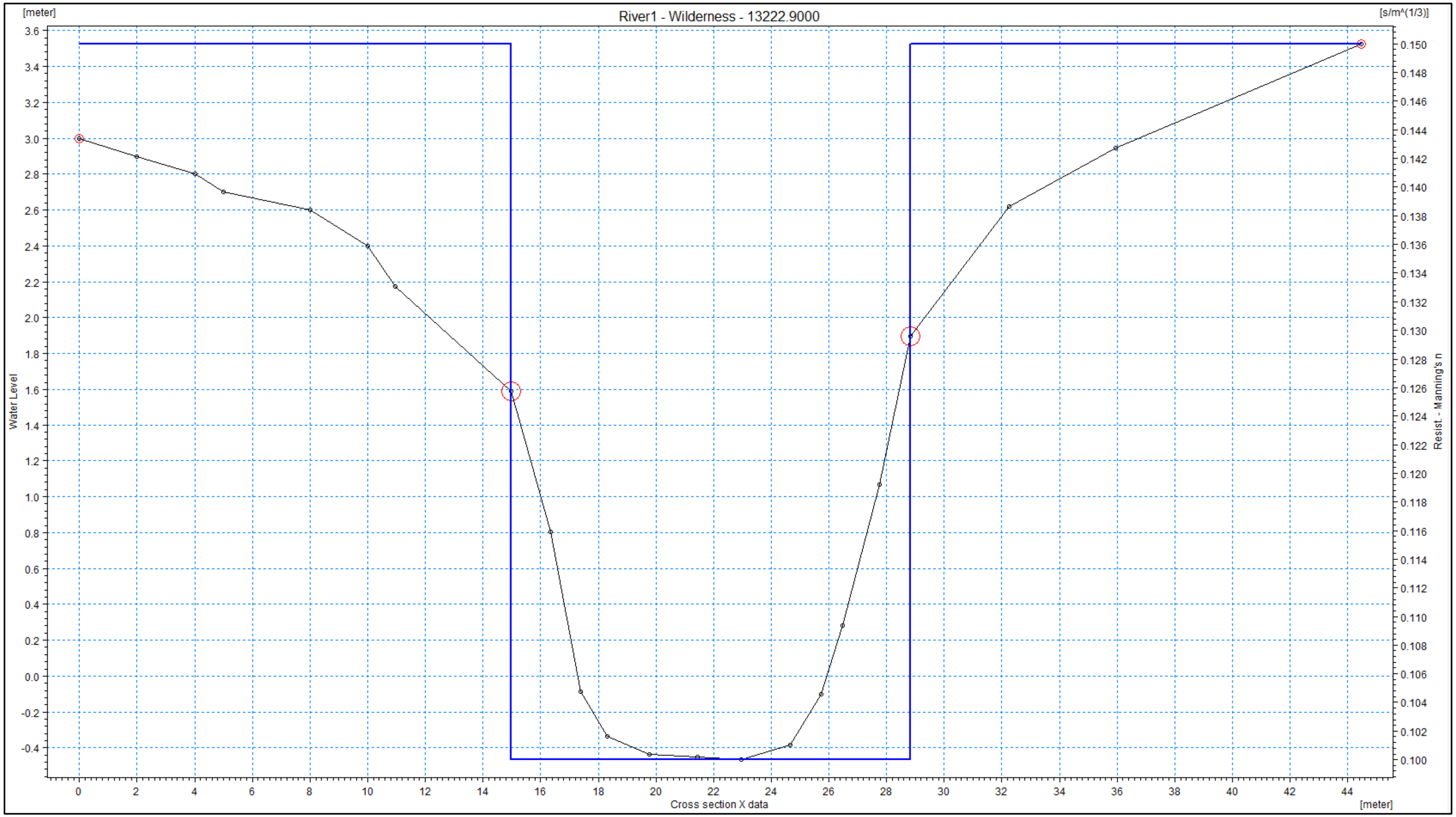

Figure E-21: Cross Section 20

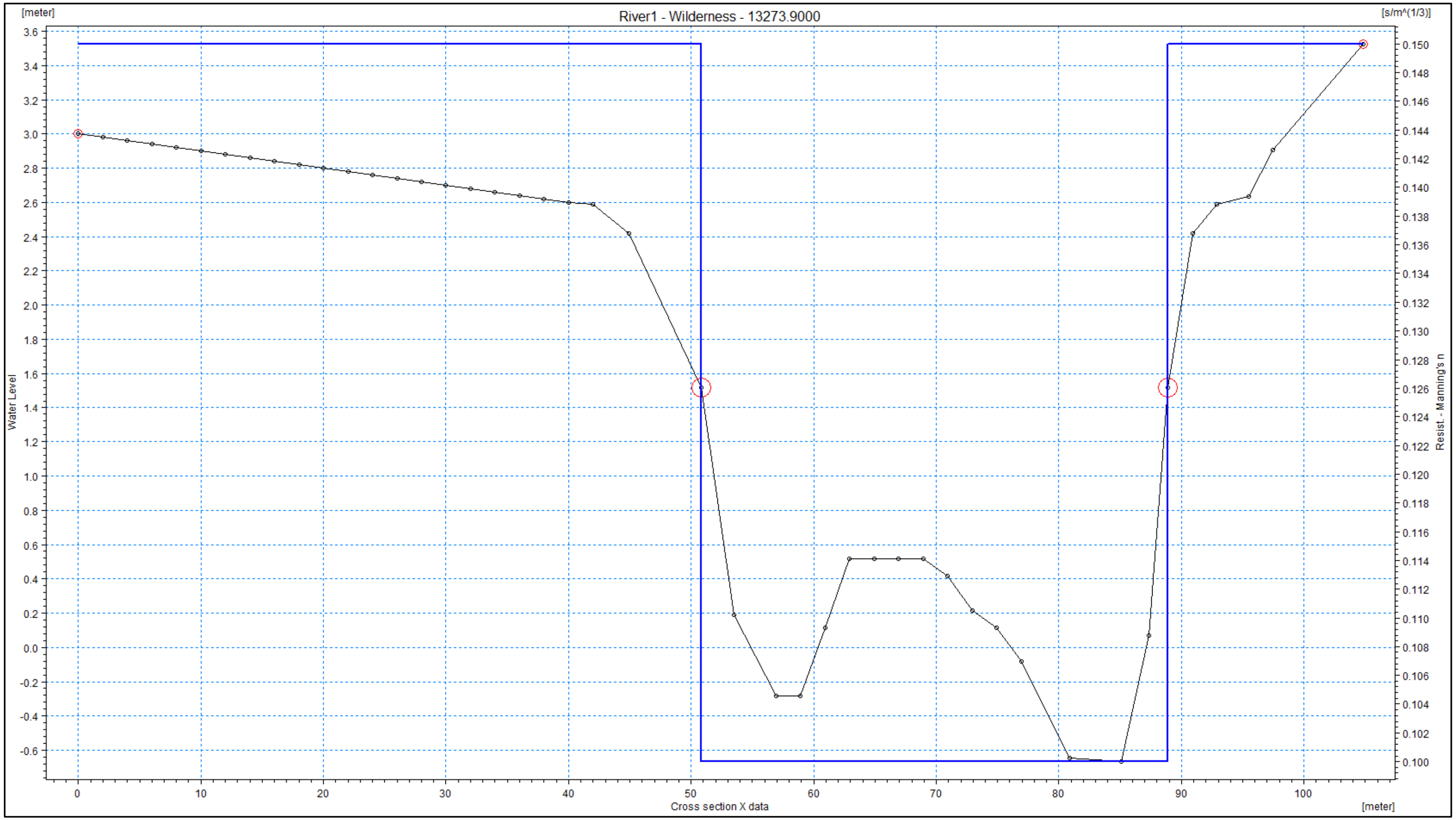

Figure E-22: Cross Section 21

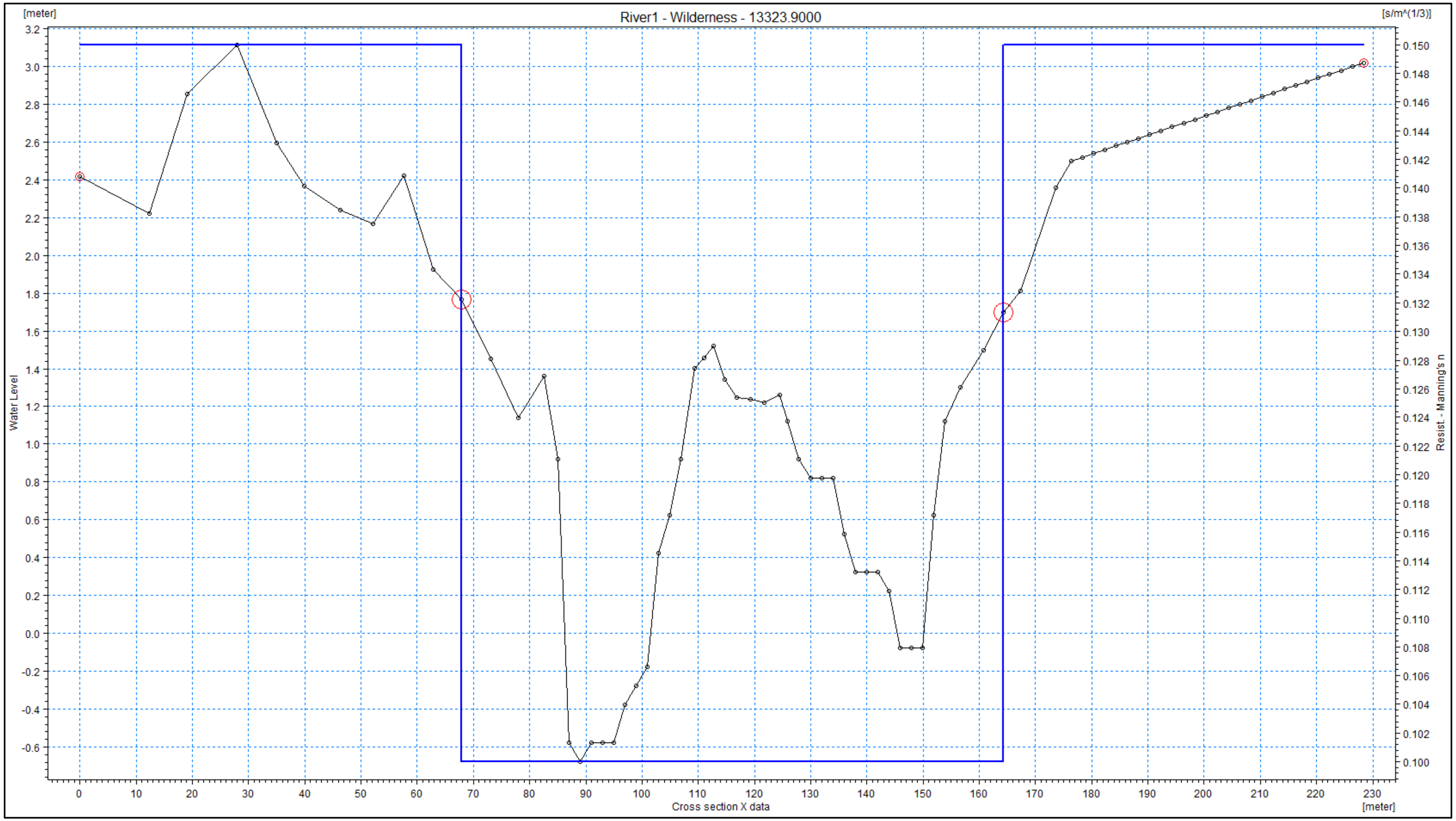

Figure E-23: Cross Section 22

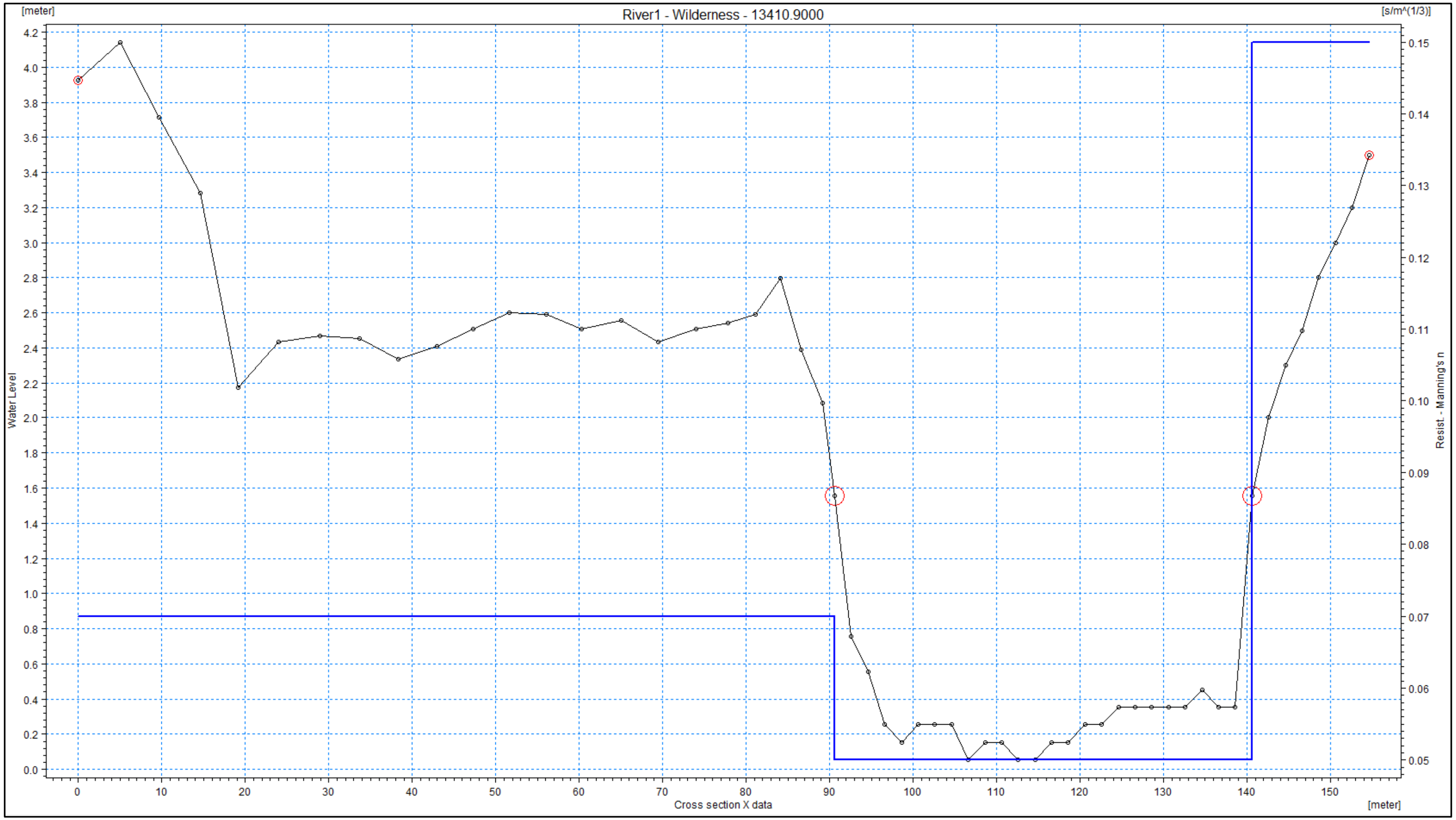

Figure E-24: Cross Section 23

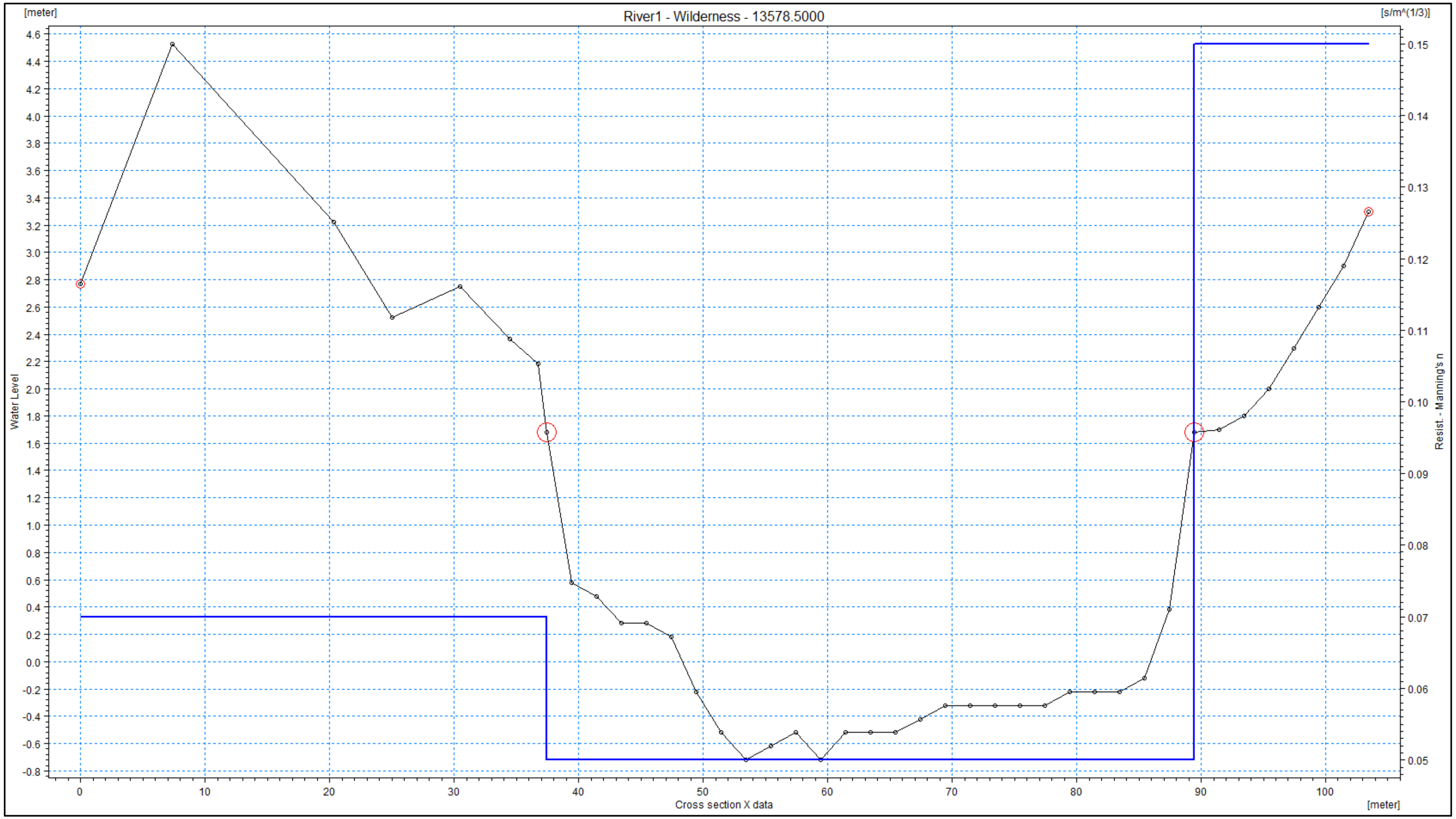

Figure E-25: Cross Section 24

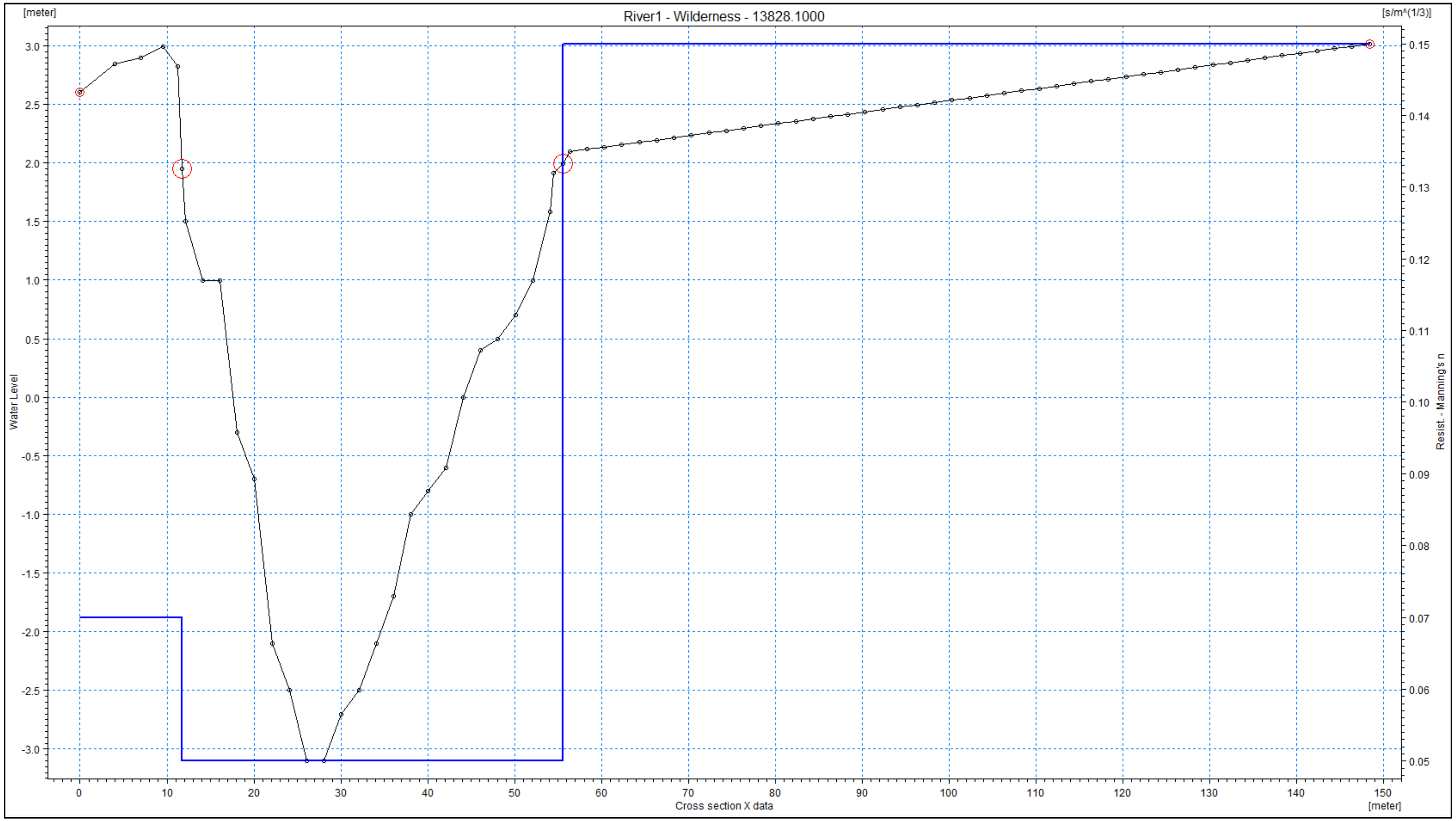

Figure E-26: Cross Section 25

Figure E-27: Cross Section 26

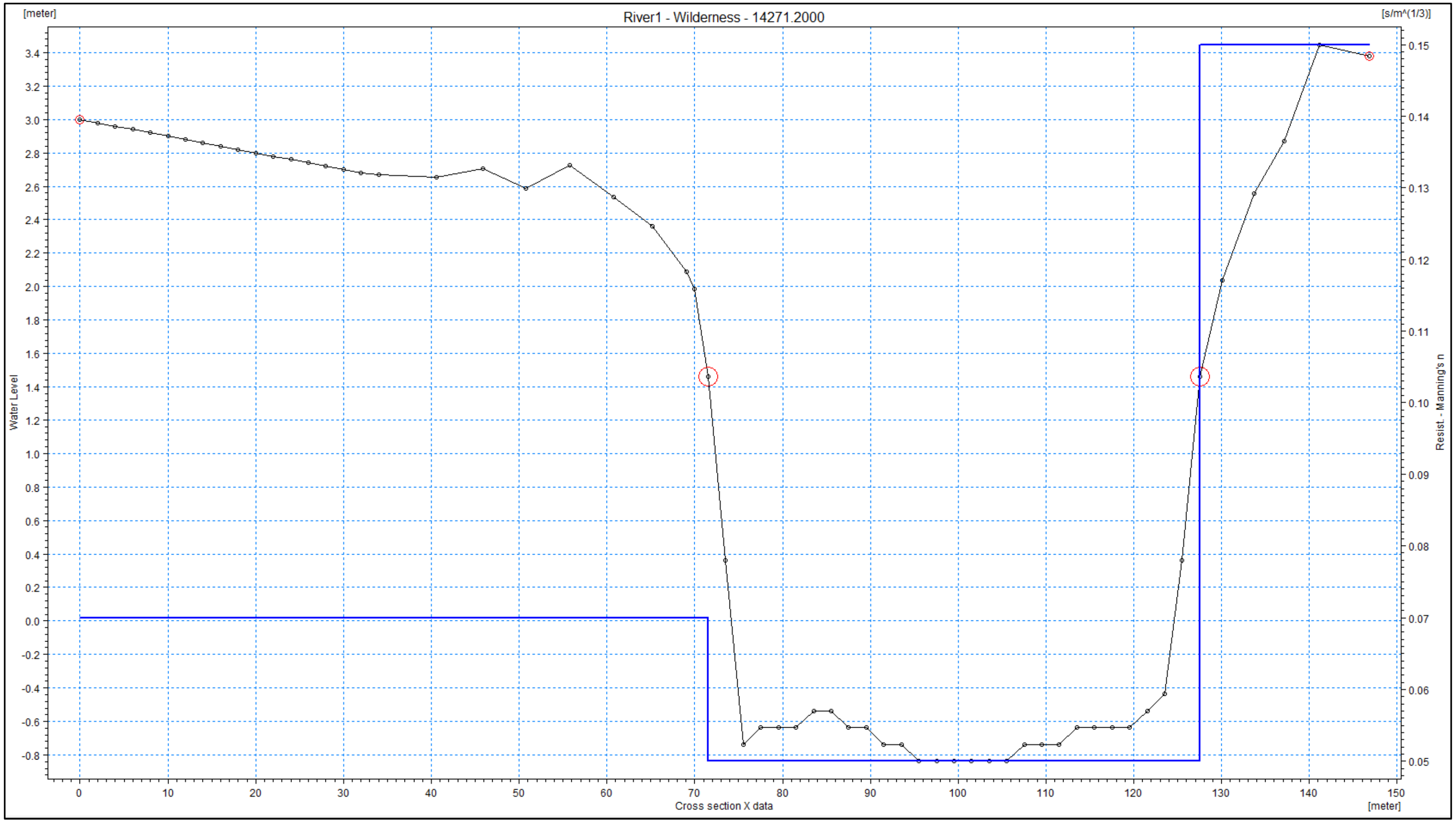

Figure E-28: Cross Section 27

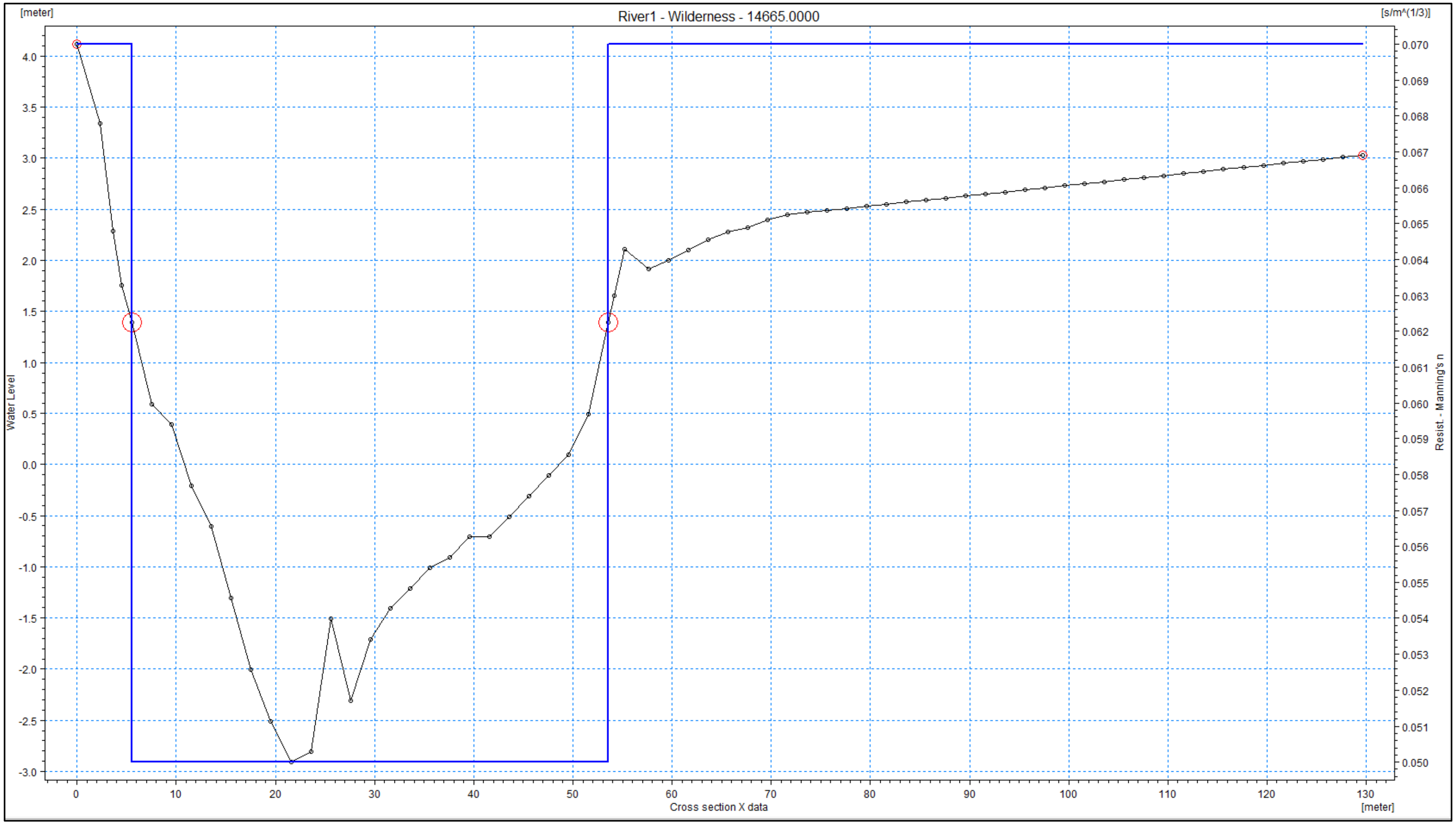

Figure E-29: Cross Section 28

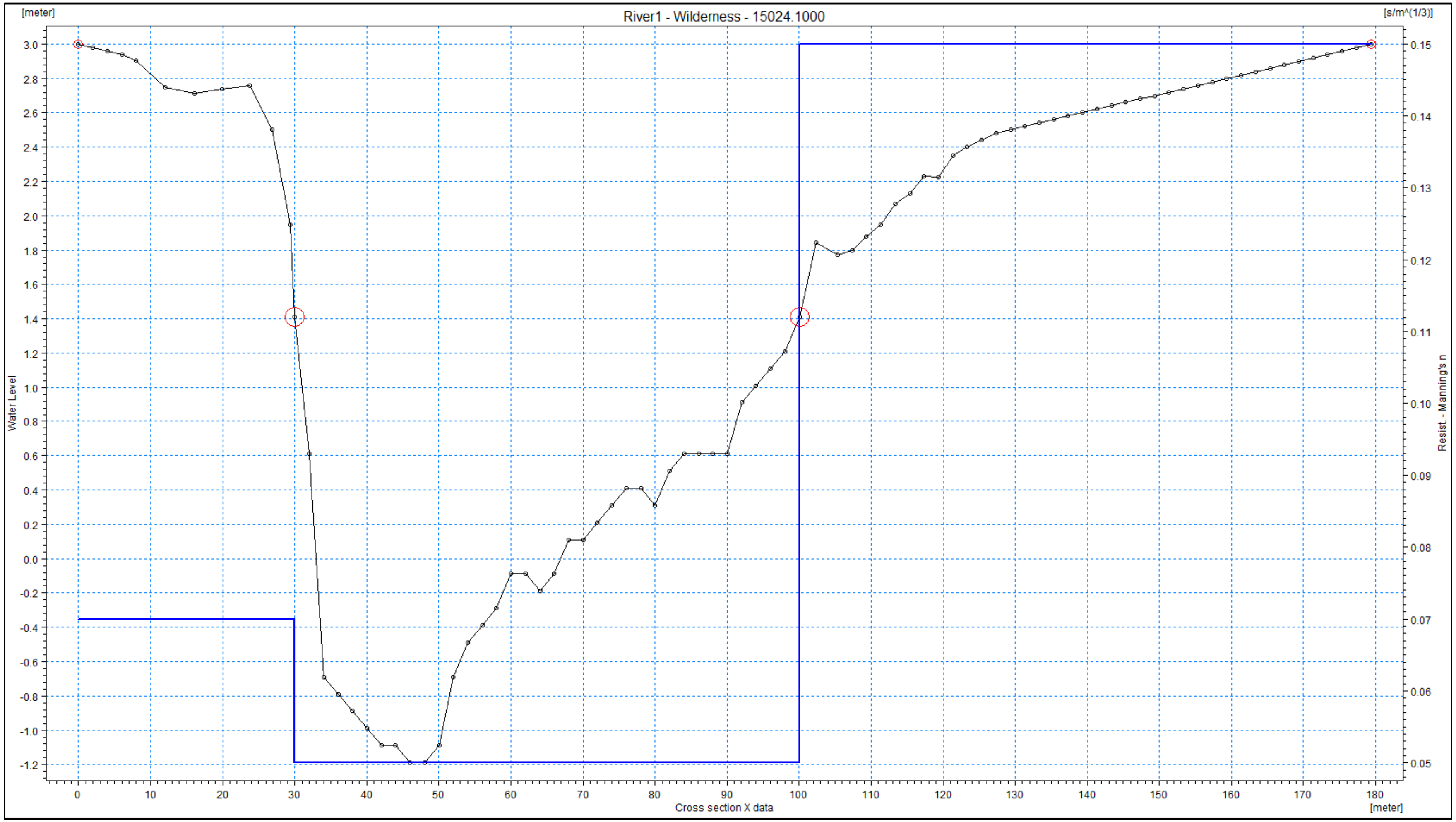

Figure E-30: Cross Section 29

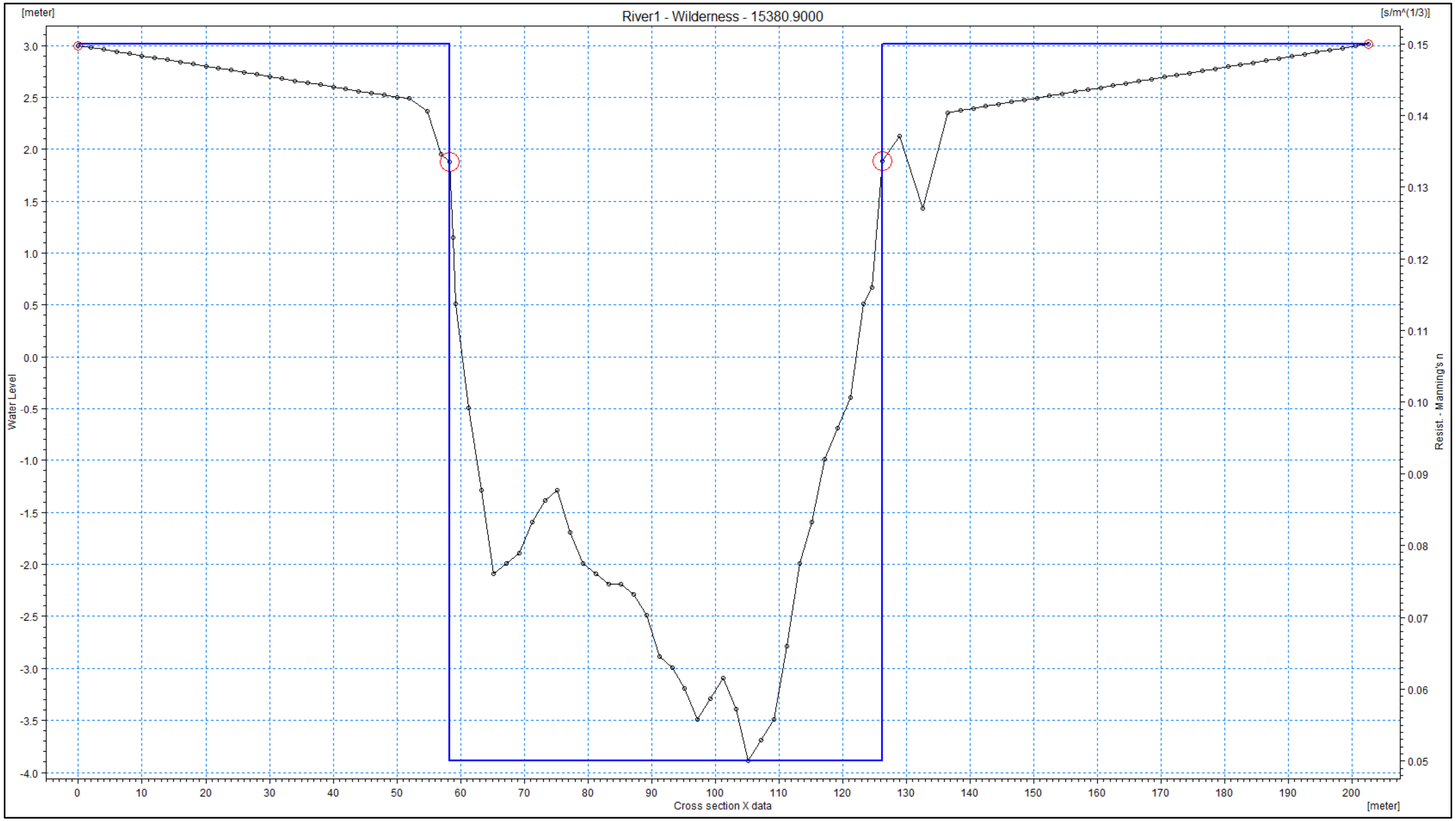

Figure E-31: Cross Section 30

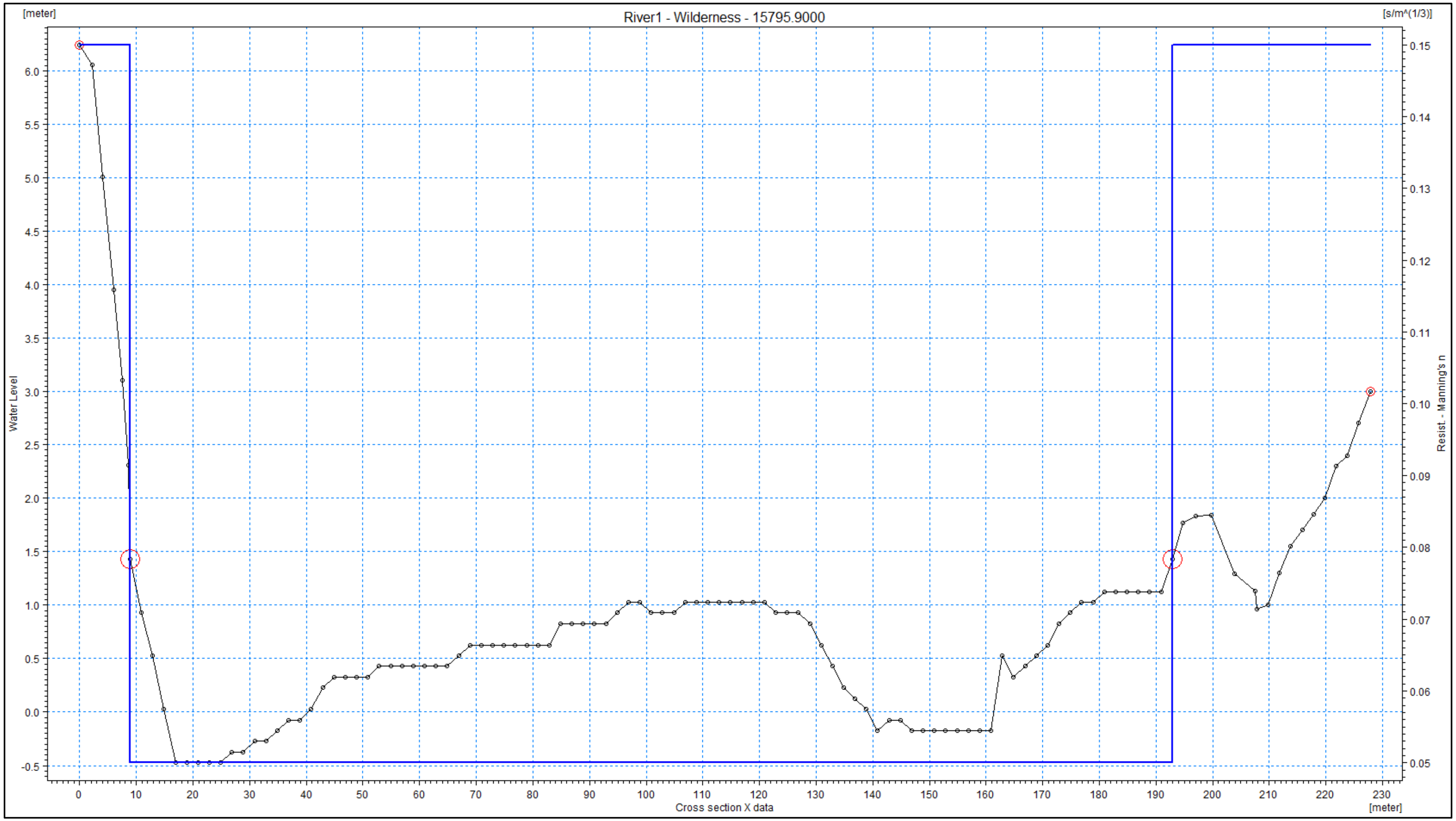

Figure E-32: Cross Section 31

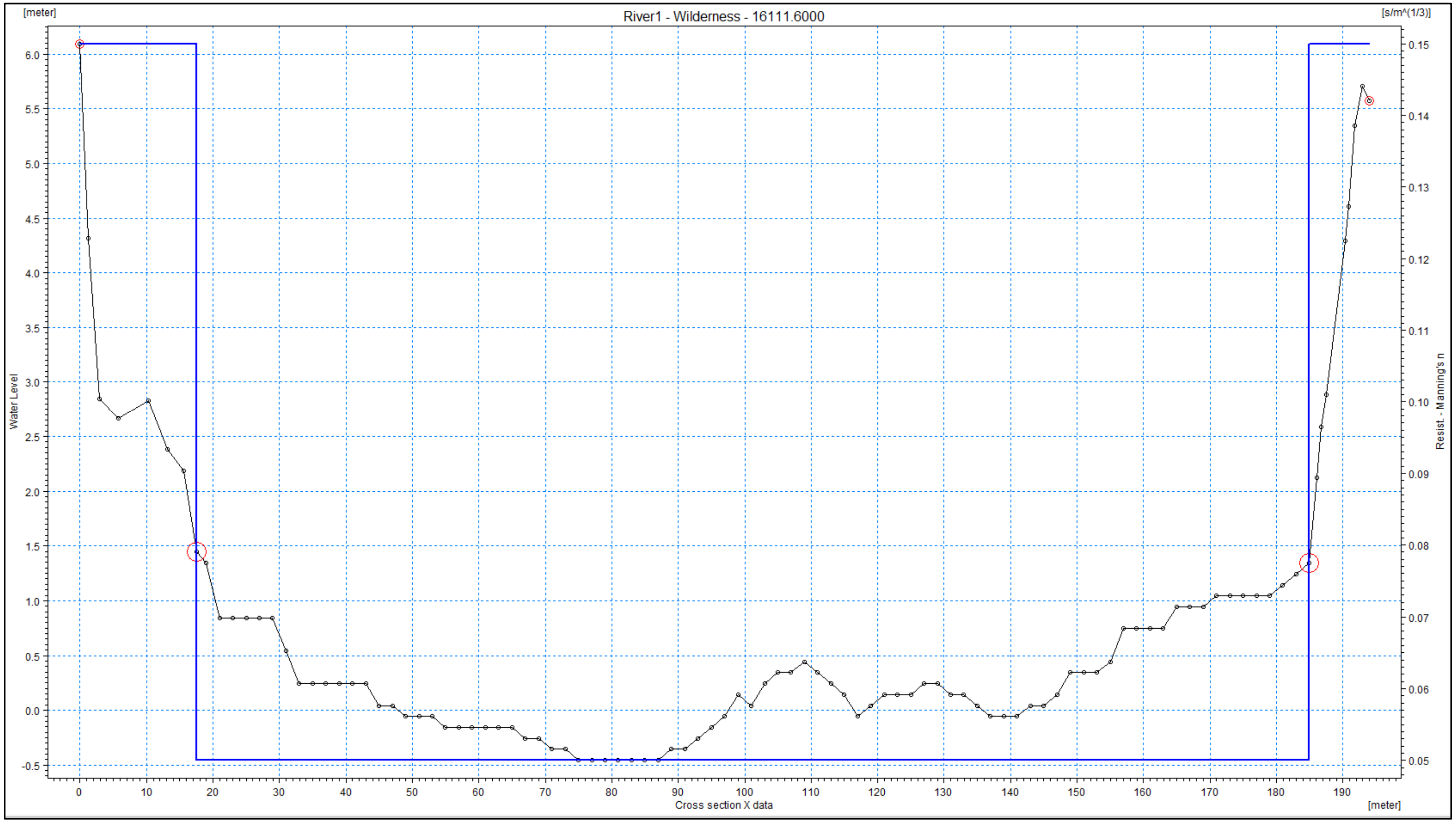

Figure E-33: Cross Section 32

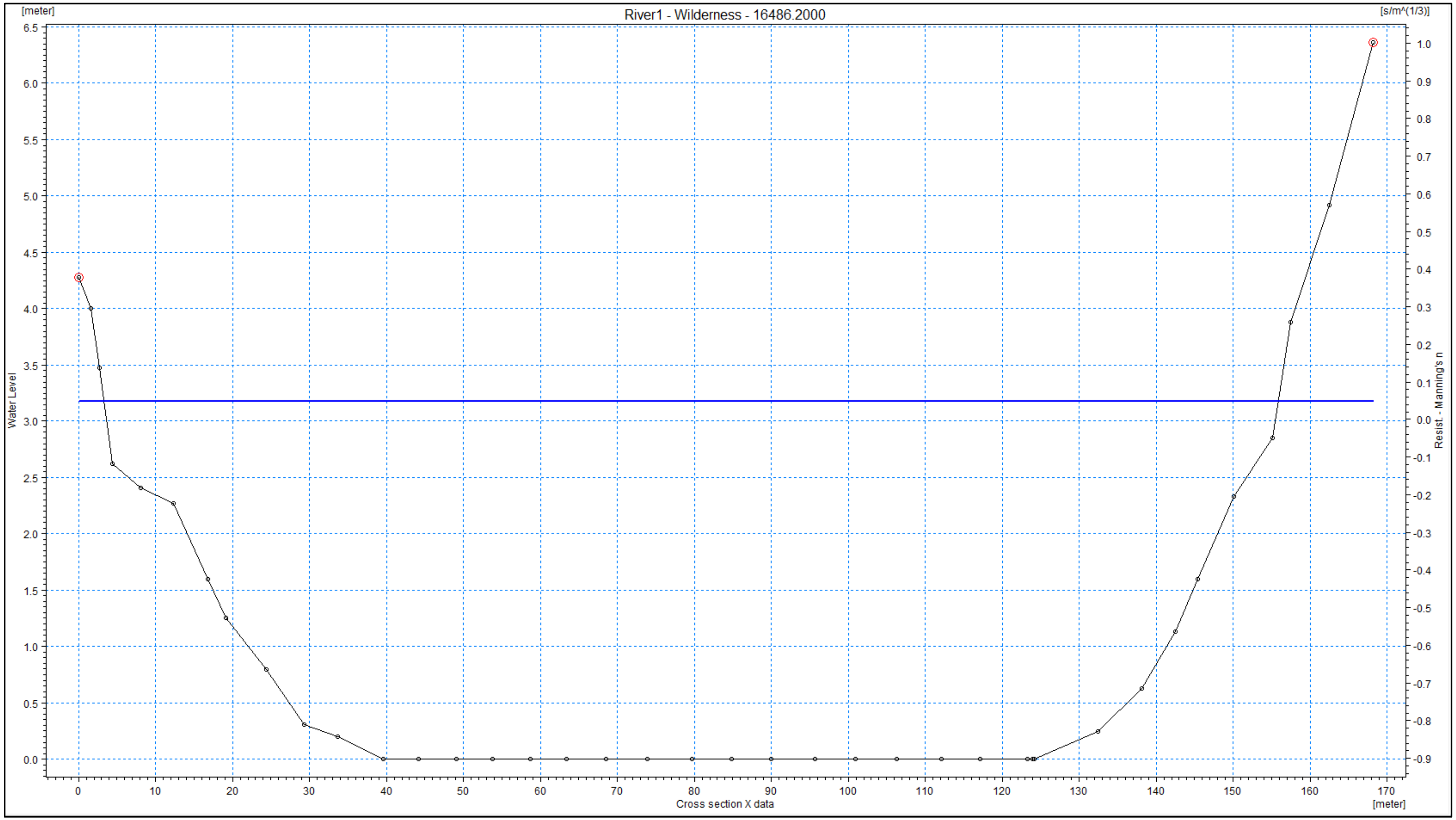

Figure E-34: Cross Section 33

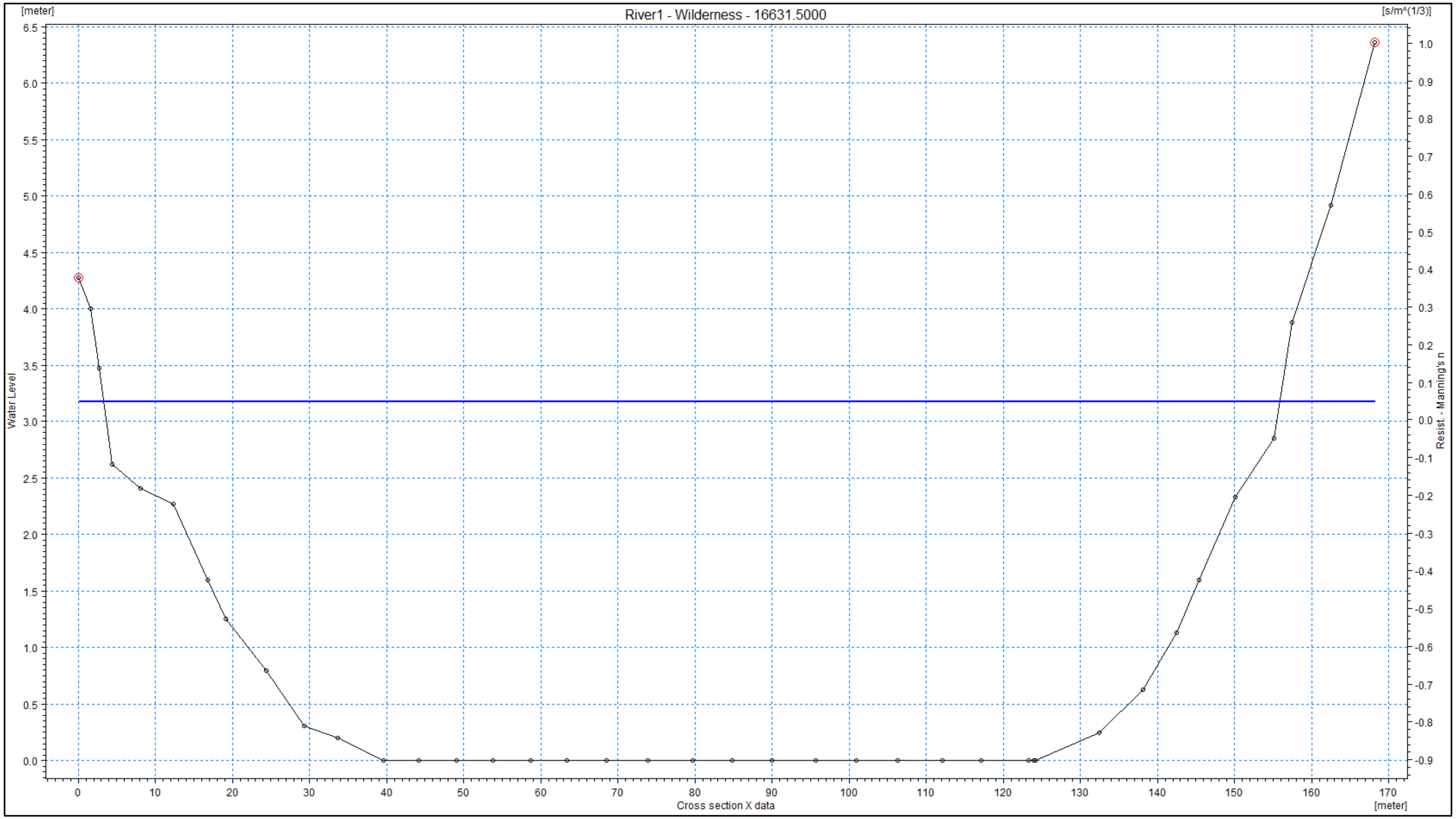

Figure E-35: Cross Section 34

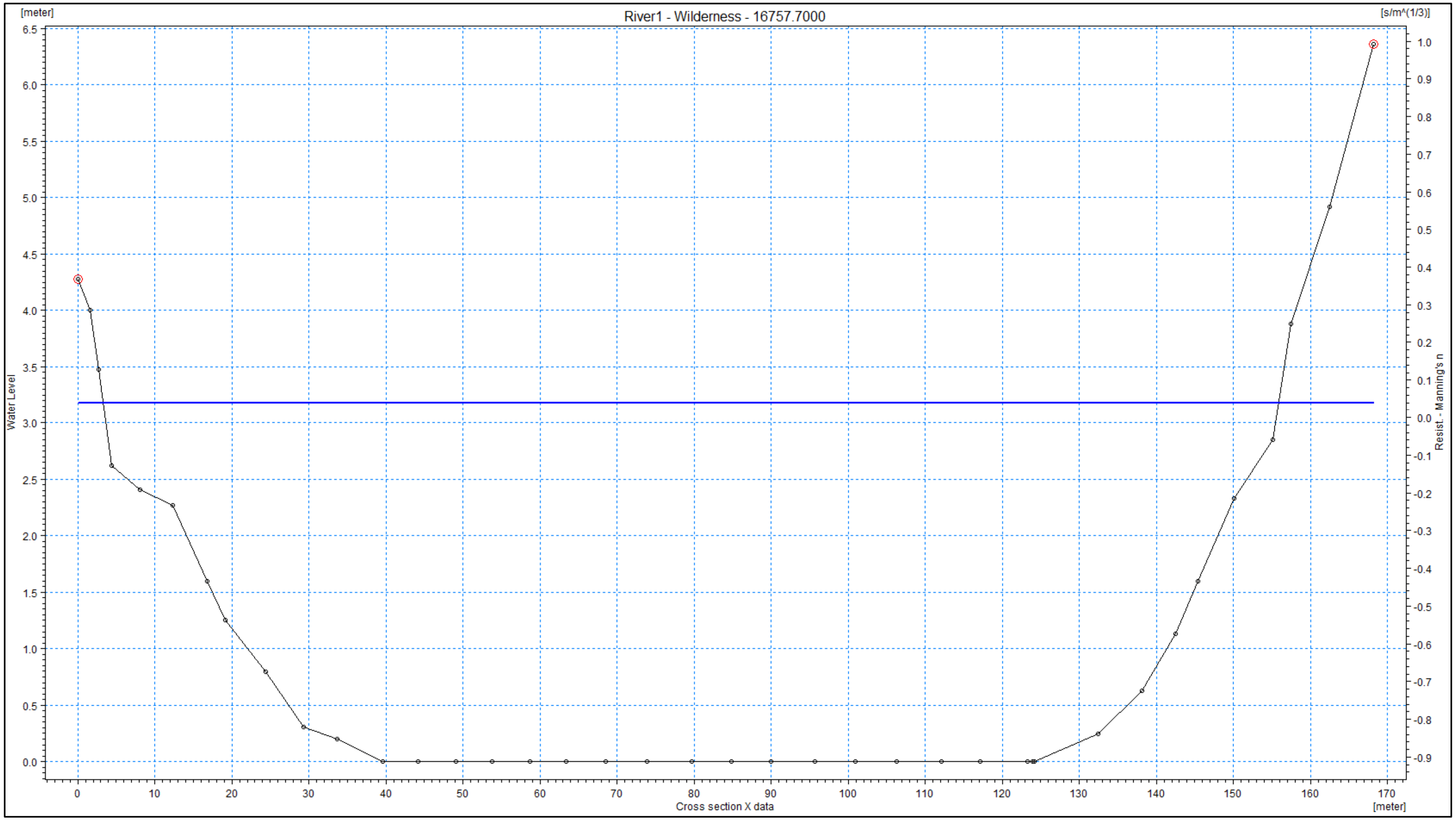

Figure E-36: Cross Section 35

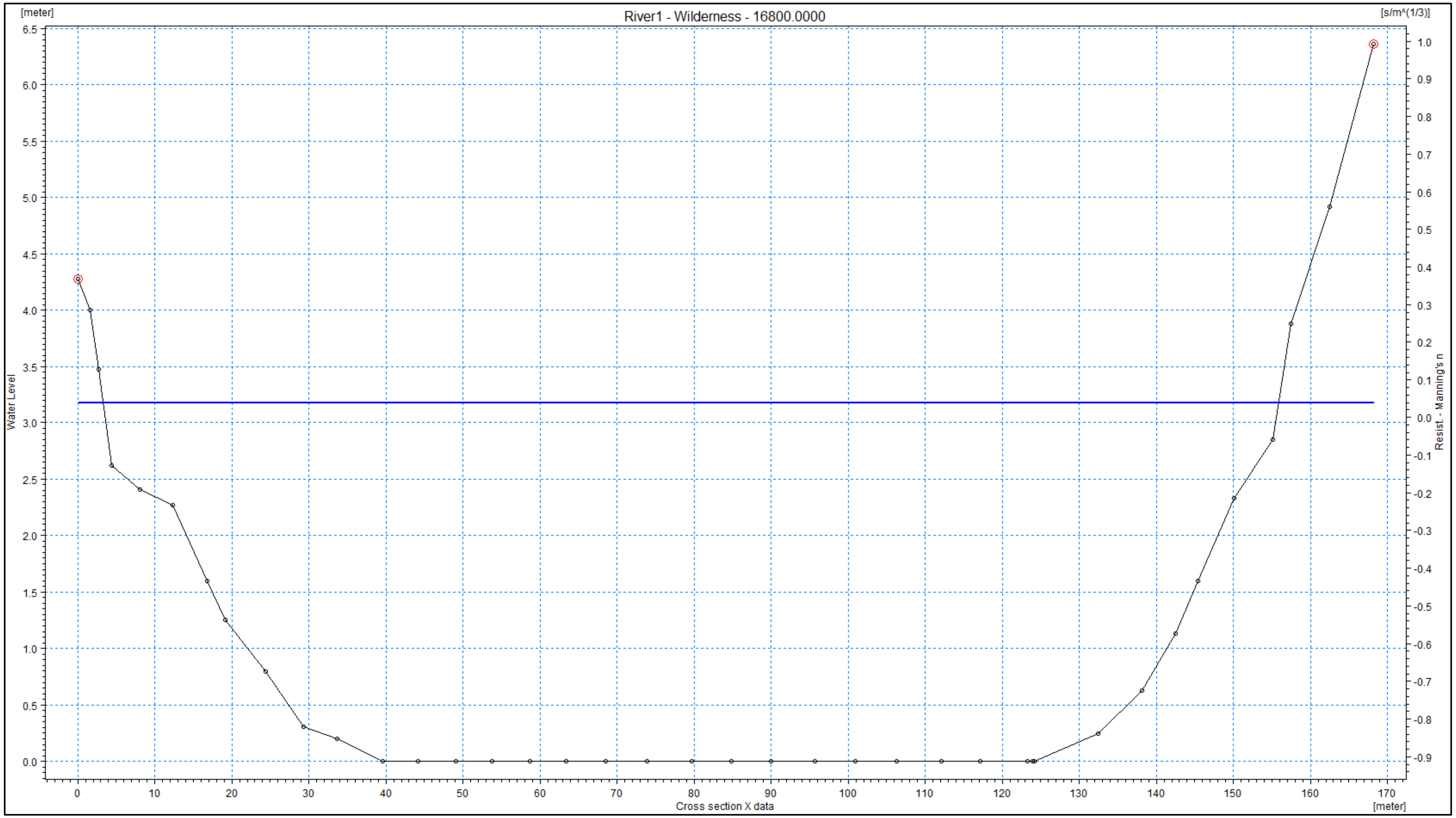

Figure E-37: Cross Section 36

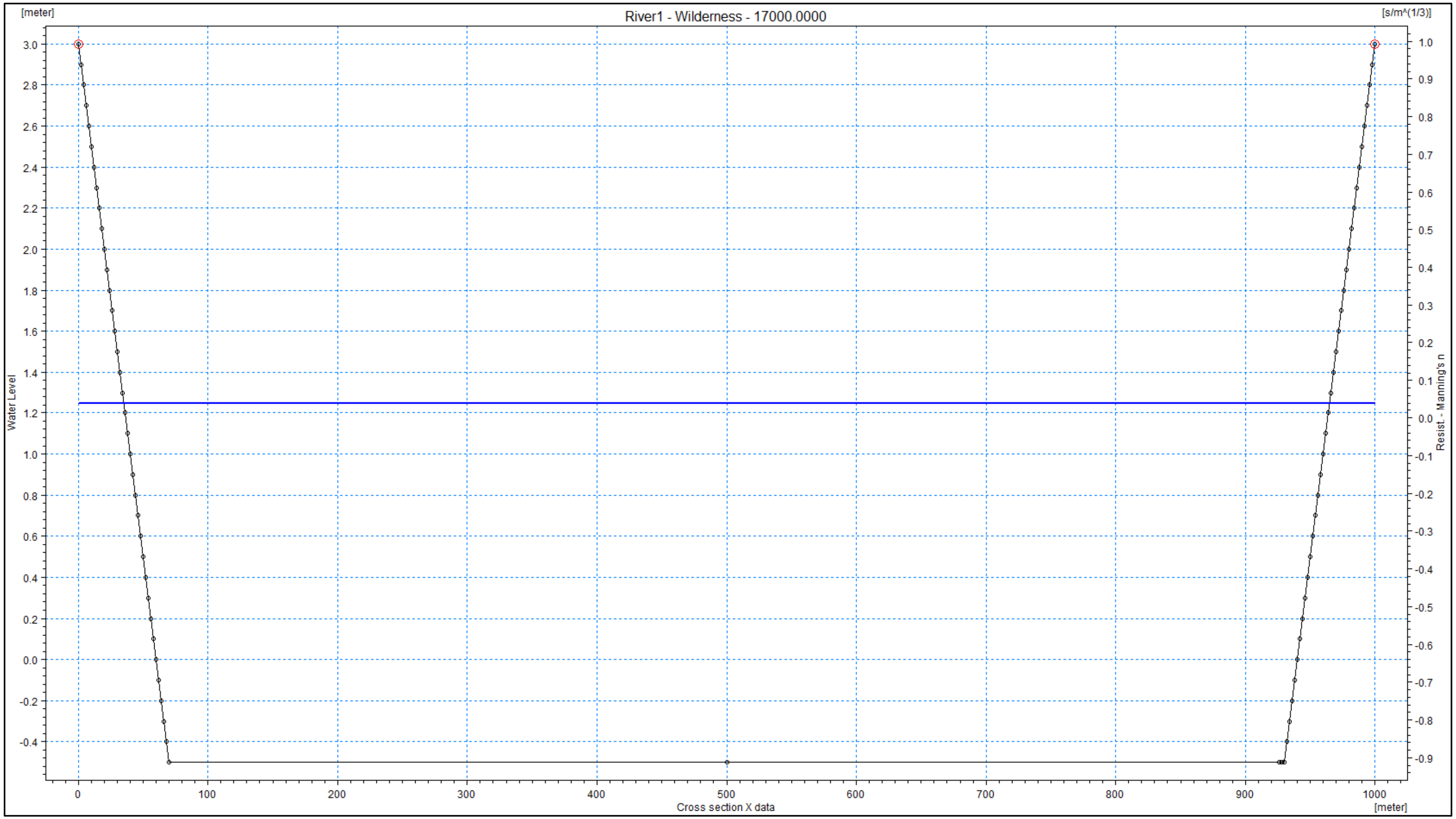

Figure E-38: Cross Section 37

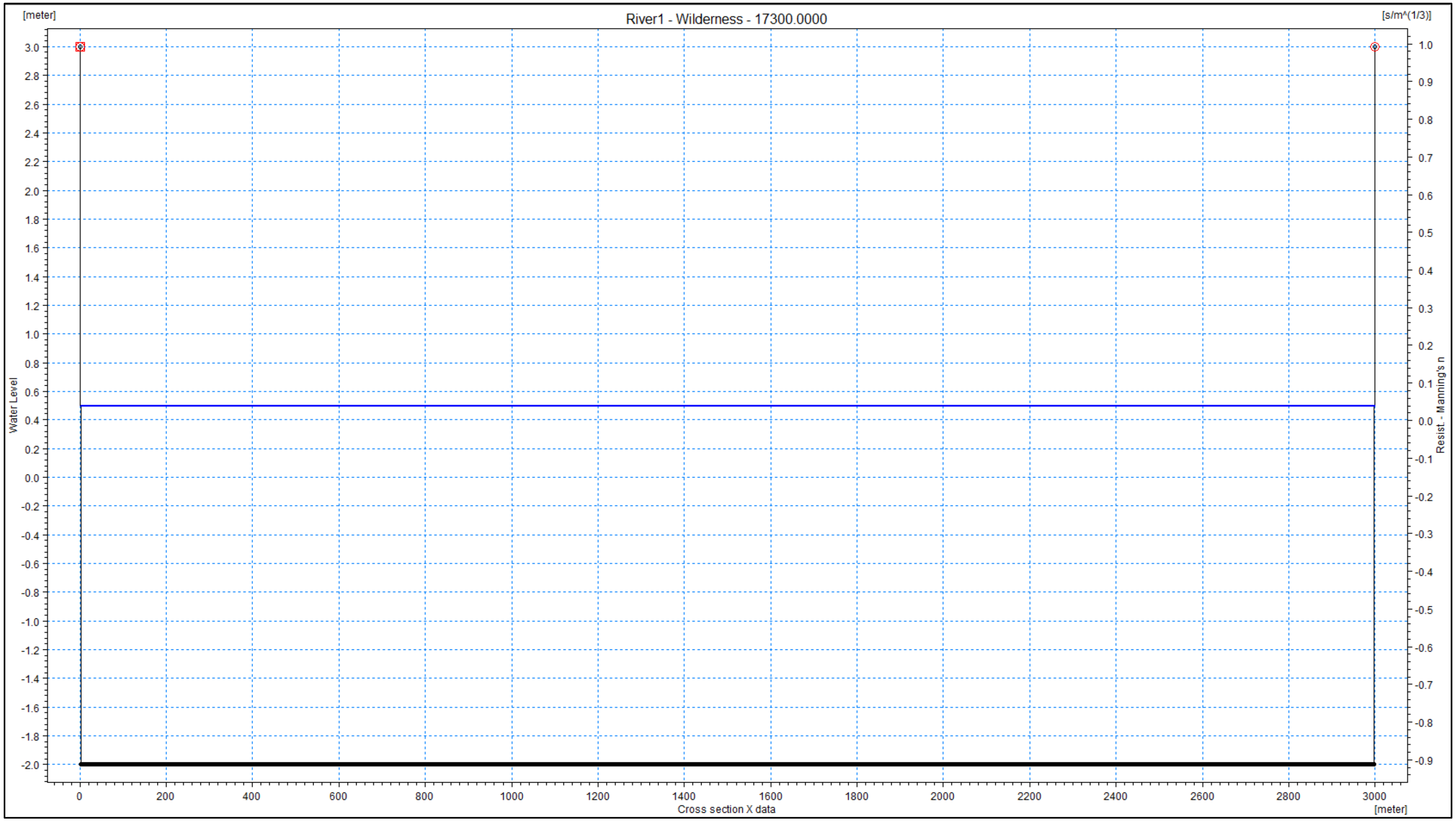

Figure E-39: Cross Section 38

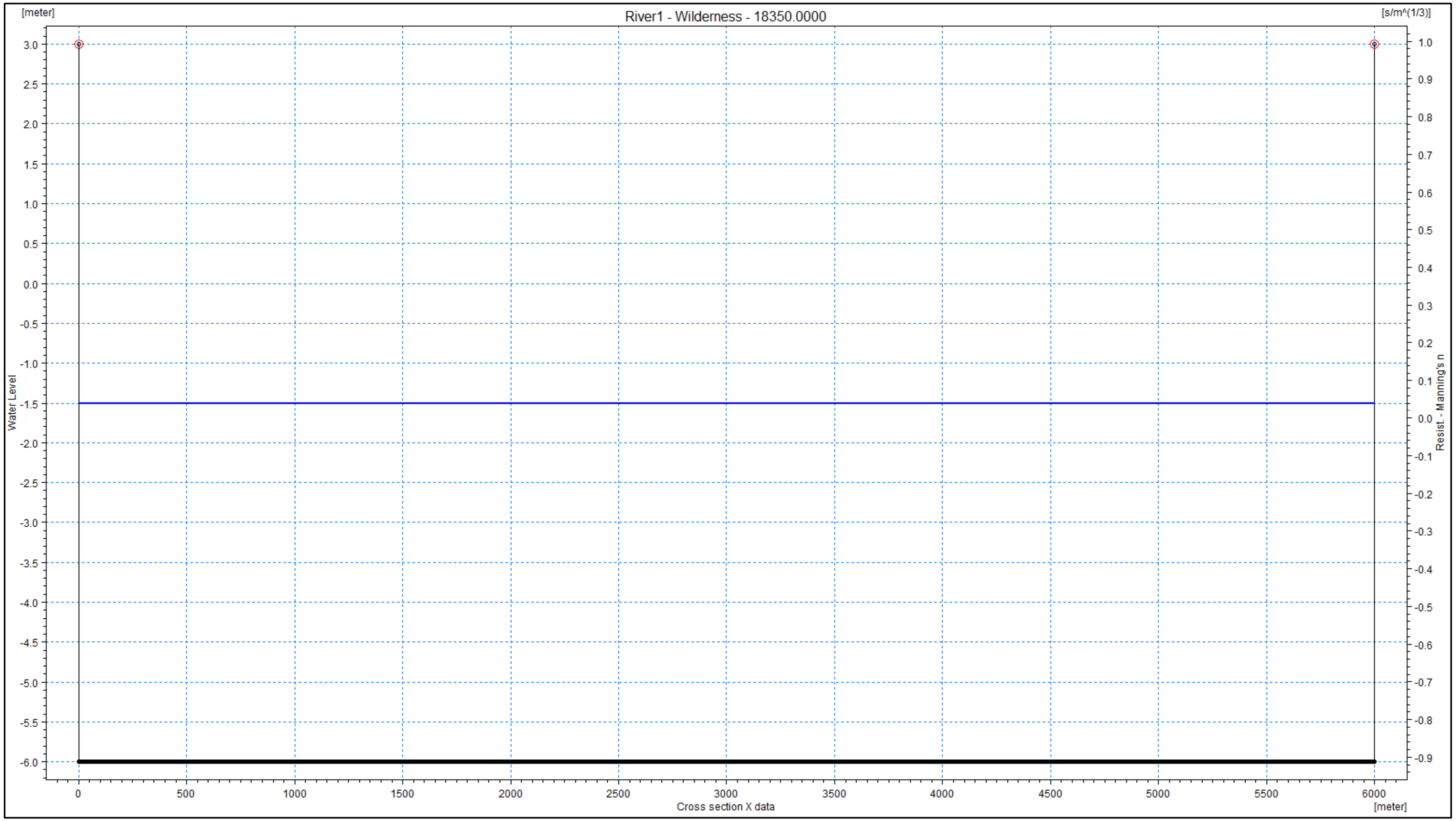

Figure E-40: Cross Section 39

APPENDIX F. MODEL PREDICTED PEAK WATER LEVELS

F.1. Introduction

The following section continues from the main report and provides the results for the model predicted peak water levels for different flood events. Each figure represents a different water body for a different flood event. The other dependent variables within the figures are initial sand bar level and the initial water level.

Similar trends that were evident in the 5 year flood predictions are also evident in the following figures. In the figures related to the Touw estuary, it can be seen that the peak water levels are not dependent on the initial water level before the flood and are also not dependent on the sand bar elevation below a certain sand bar height. This sand bar elevation threshold is different according to the size of the flood

From the figures related to the coastal lakes, it can be seen that the peak water levels achieved during a flood event in the coastal lakes are not dependent on the initial level of the sand bar at the estuary mouth. Peak water levels during floods are far more dependent on the initial level of the water in the lake before the flood event.

Based on these trends, an average line for the least dependent variable was plotted on the graph, illustrated by a dotted black line. These averaged lines where then used in order to create the combined peak water level predictions.

F.2. 10 year flood

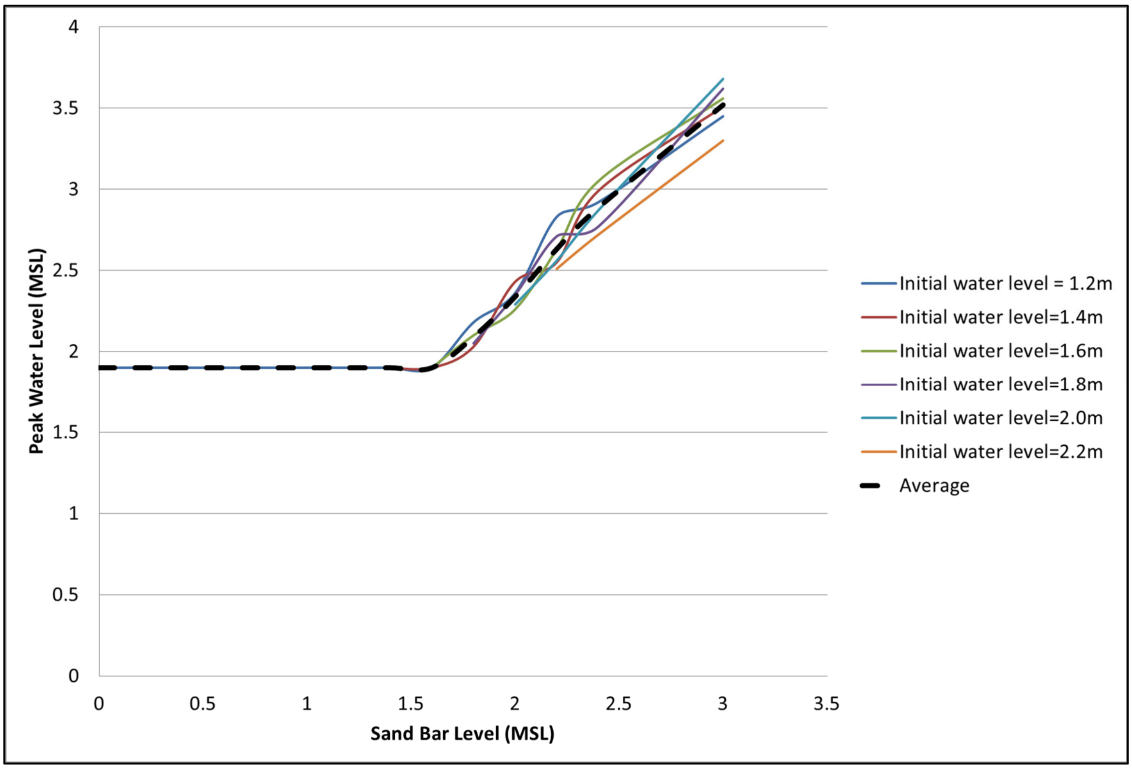

Figure F-1: Touw estuary peak water levels after 10yr flood for different sand bar heights and initial water levels

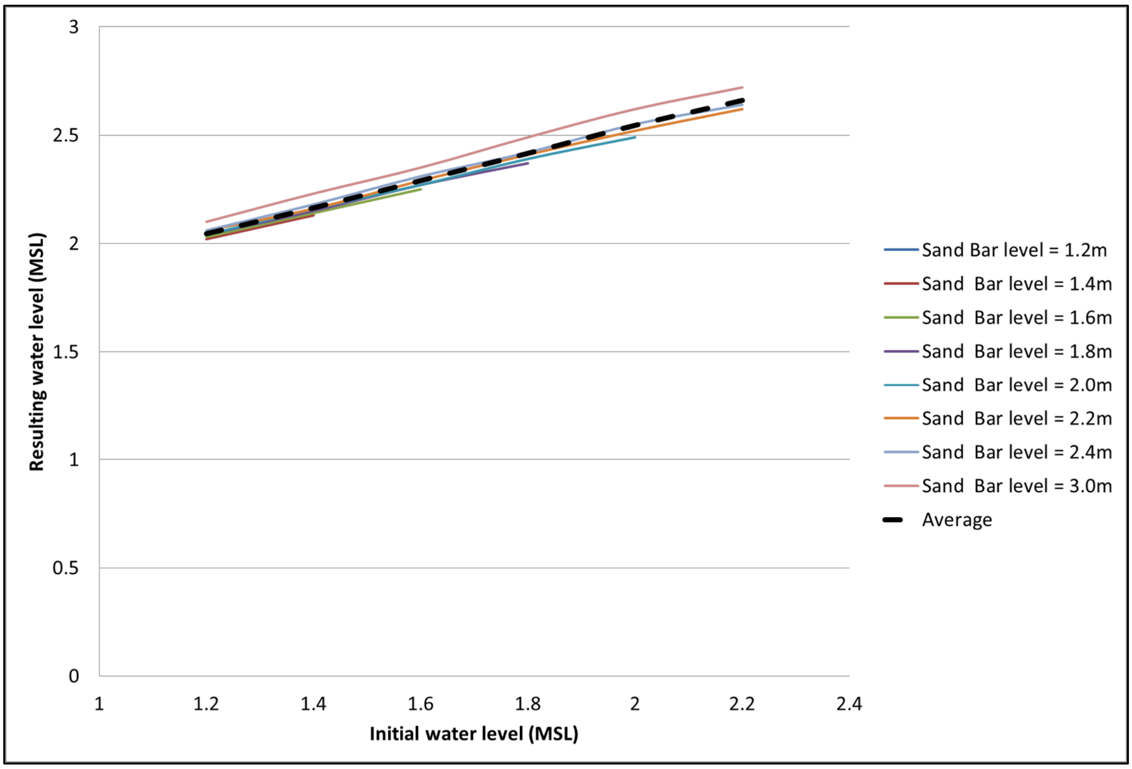

Figure F-2: Island Lake peak water levels after 10yr flood for different sand bar heights and initial water levels

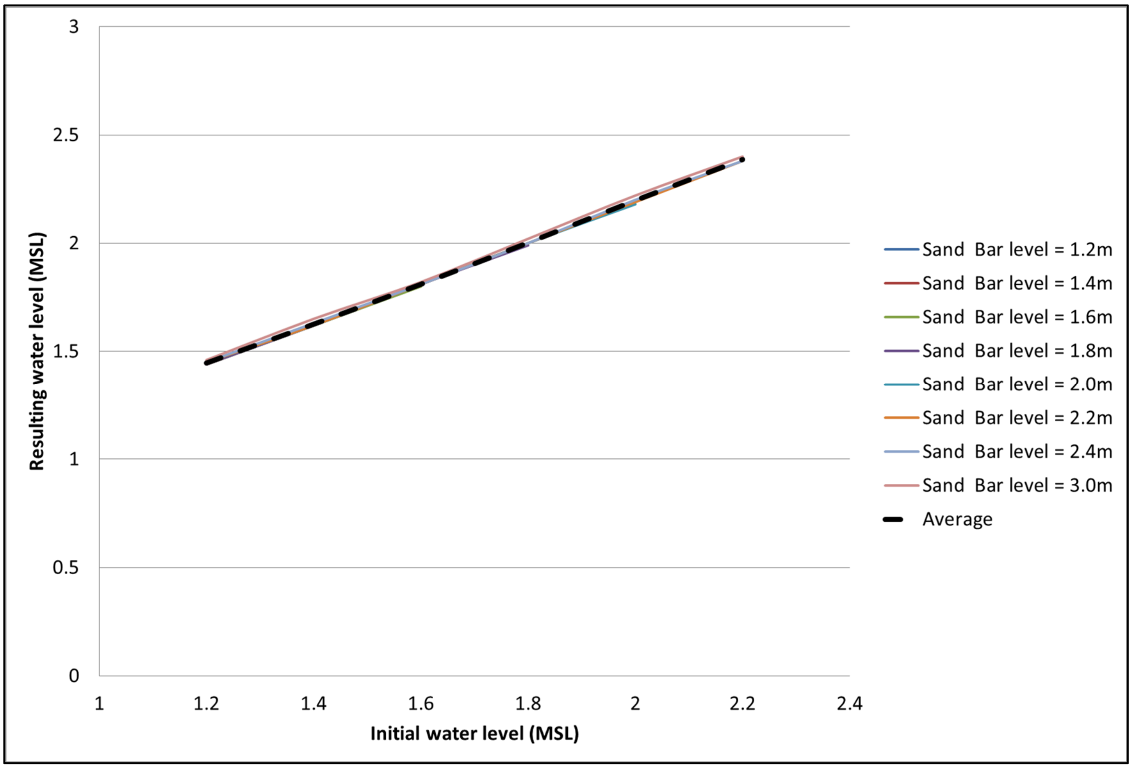

Figure F-3: Bolangvlei peak water levels after 10yr flood for different sand bar heights and initial water levels

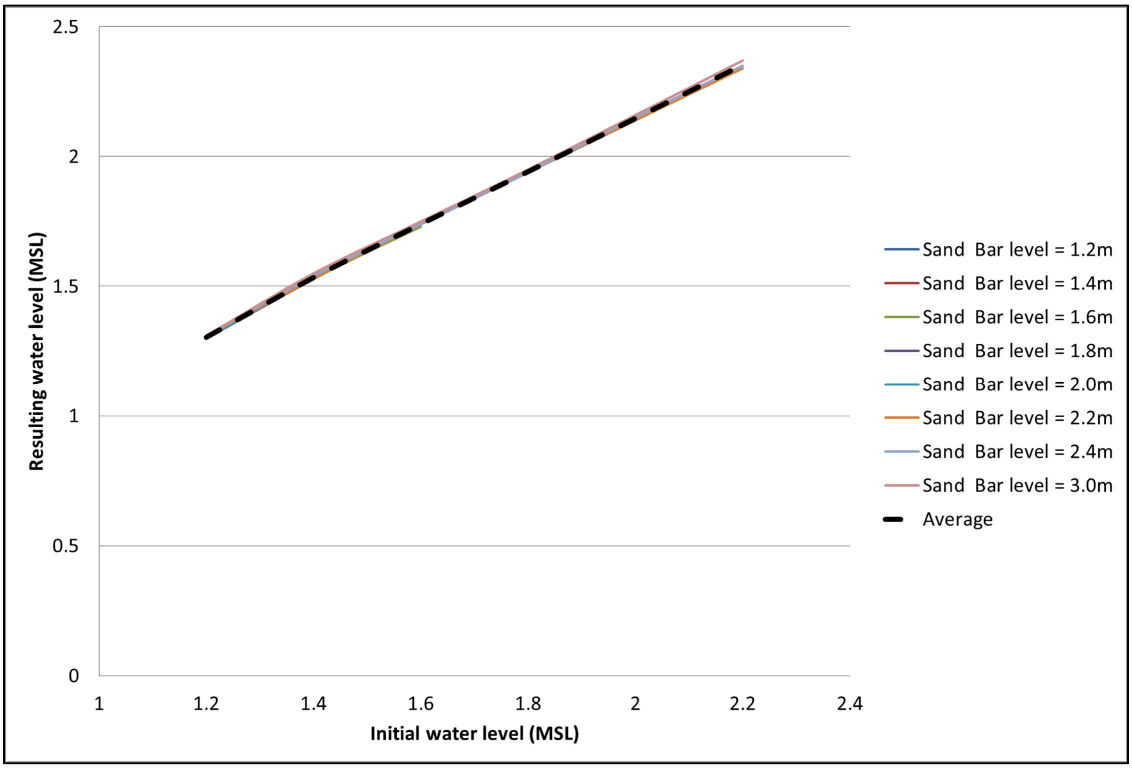

Figure F-4: Rondevlei peak water levels after 10yr flood for different sand bar heights and initial water levels

F.3. 20 year flood

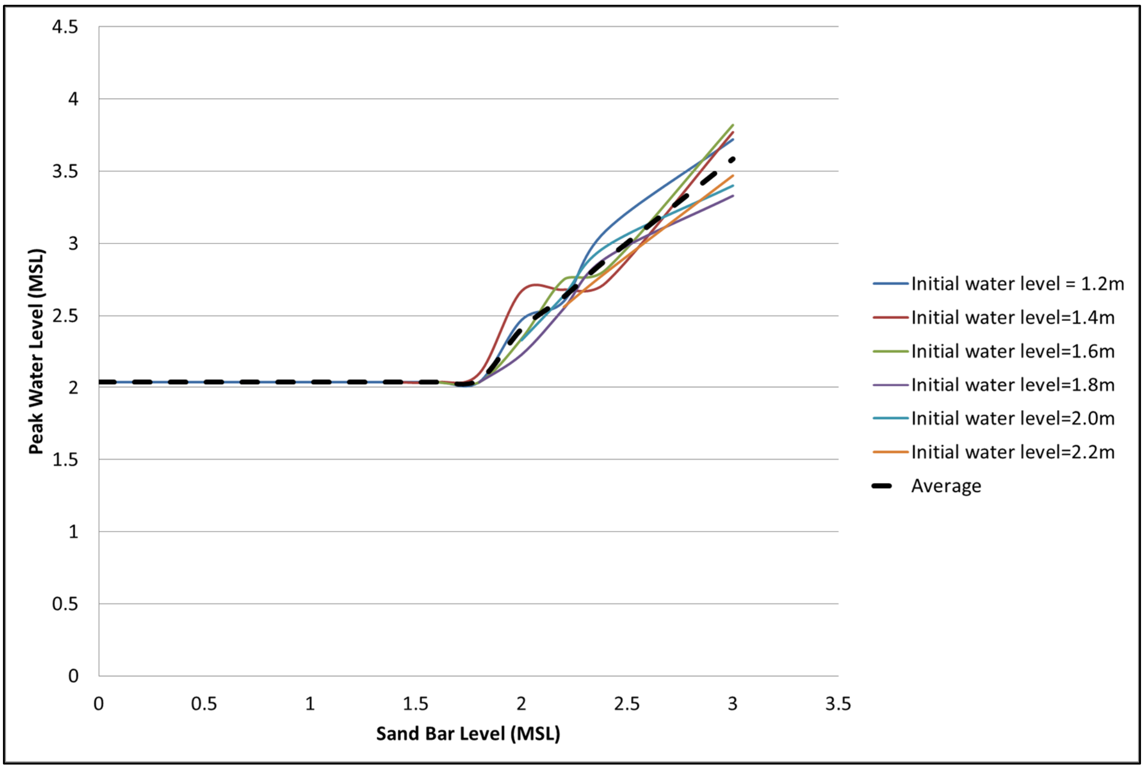

Figure F-5: Touw estuary peak water levels after 20yr flood for different sand bar heights and initial water levels

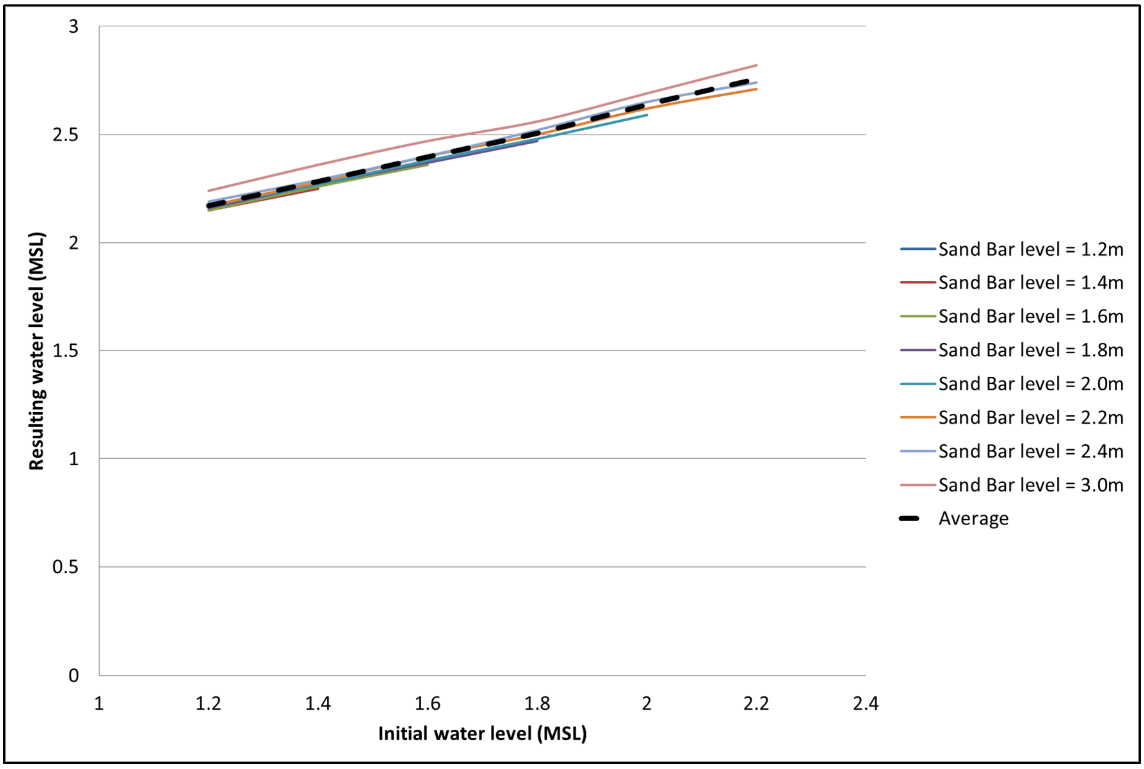

Figure F-6: Island Lake peak water levels after 20yr flood for different sand bar heights and initial water levels

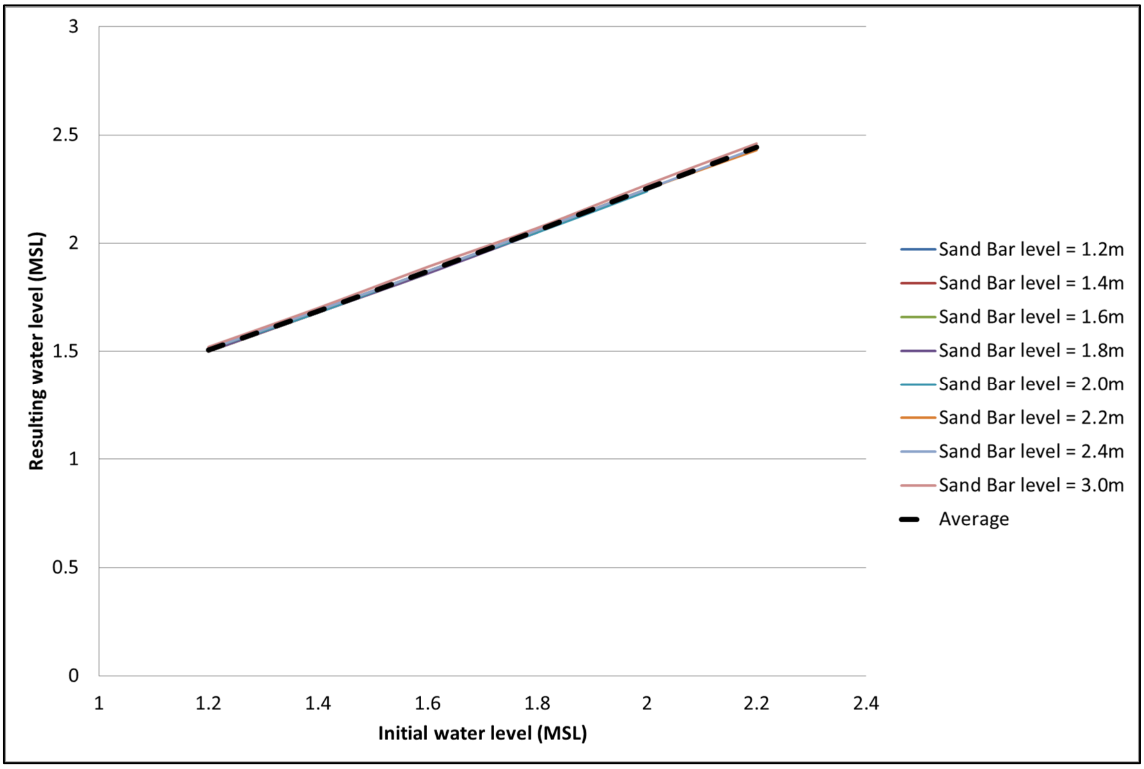

Figure F-7: Bolangvlei peak water levels after 20yr flood for different sand bar heights and initial water levels

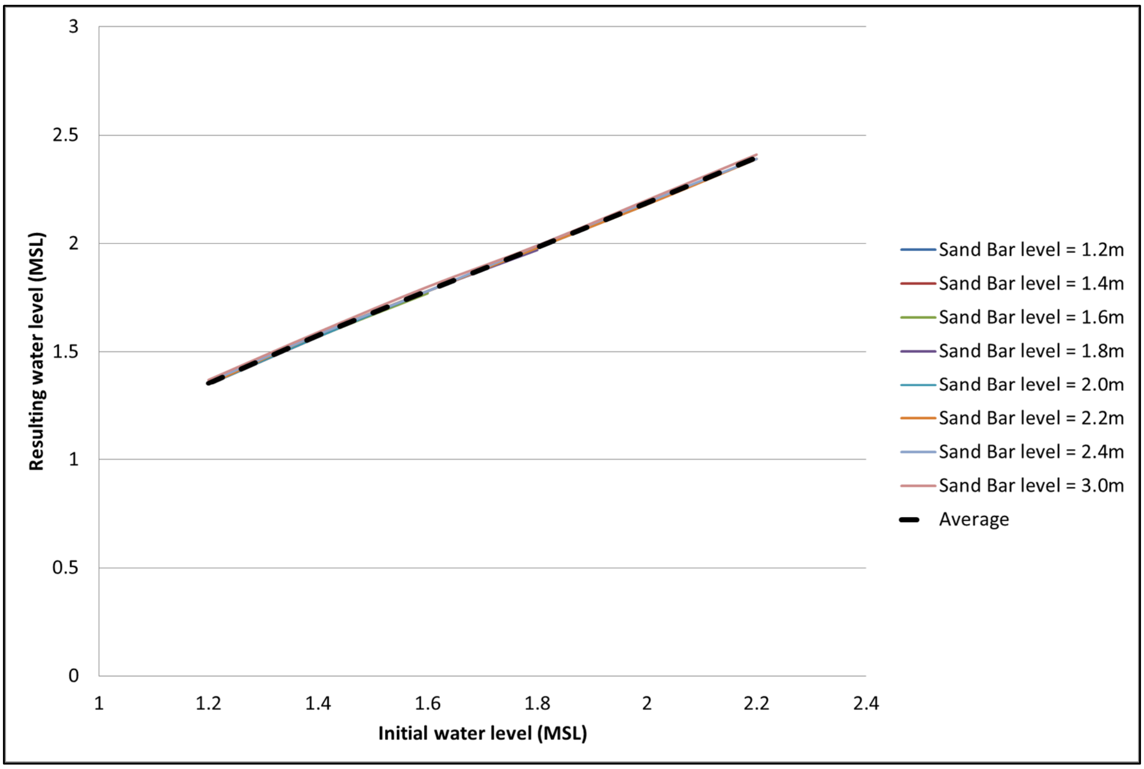

Figure F-8: Rondevlei peak water levels after 20yr flood for different sand bar heights and initial water levels

F.4. 50 year flood

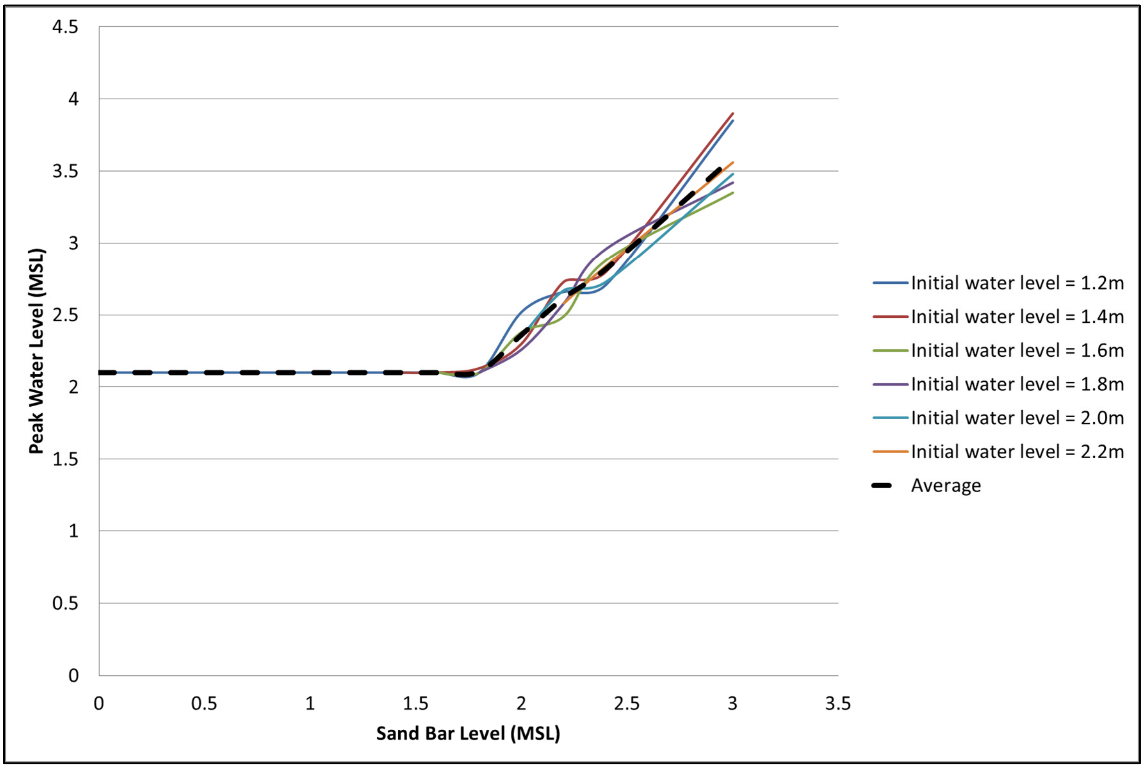

Figure F-9: Touw estuary peak water levels after 50yr flood for different sand bar heights and initial water levels

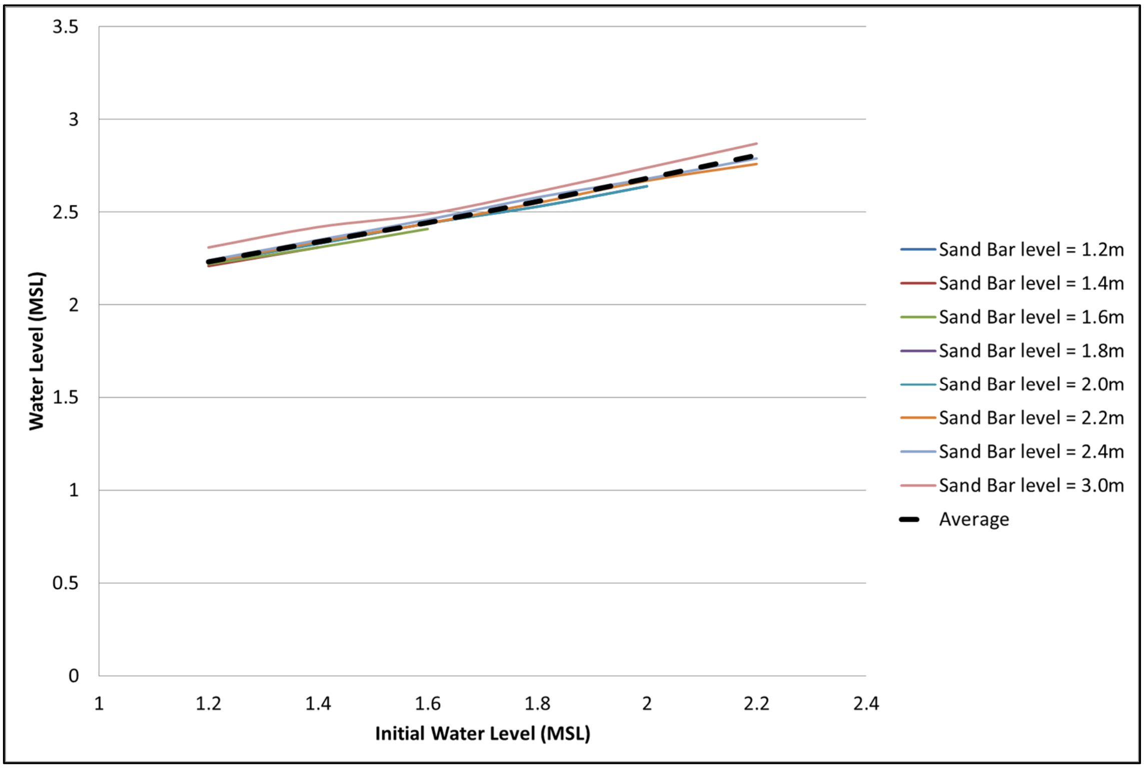

Figure F-10: Island Lake peak water levels after 50yr flood for different sand bar heights and initial water levels

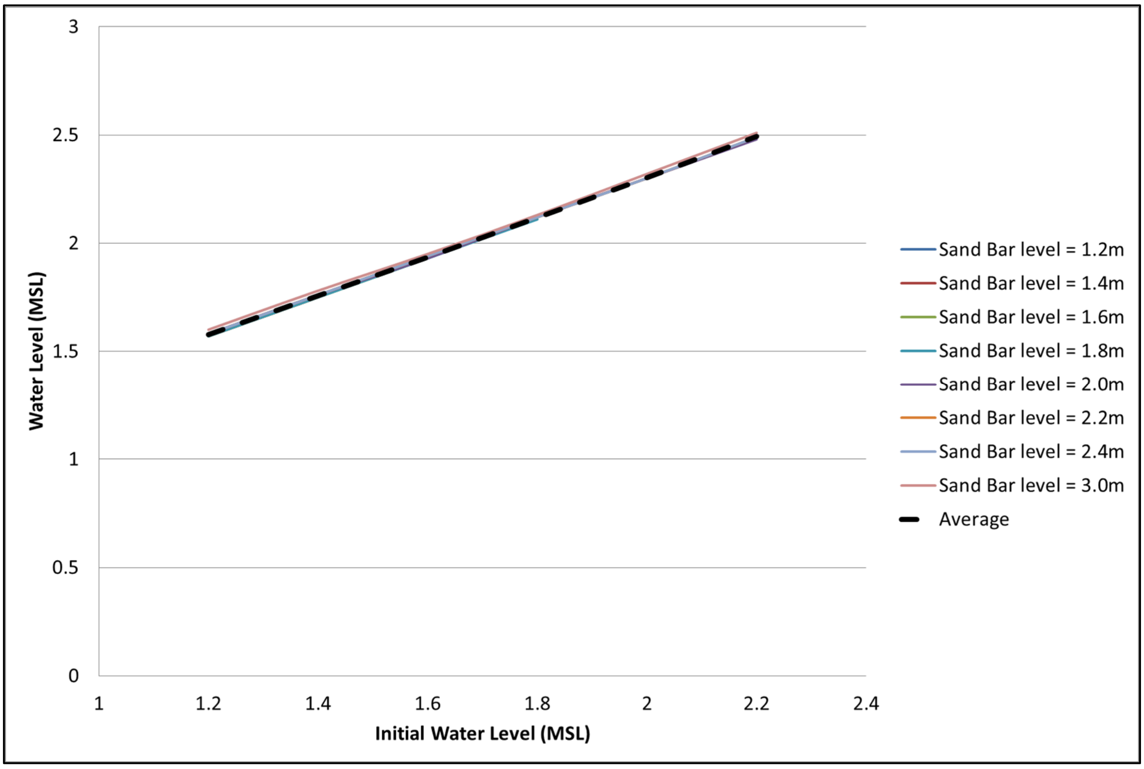

Figure F-11: Bolangvlei peak water levels after 50yr flood for different sand bar heights and initial water levels

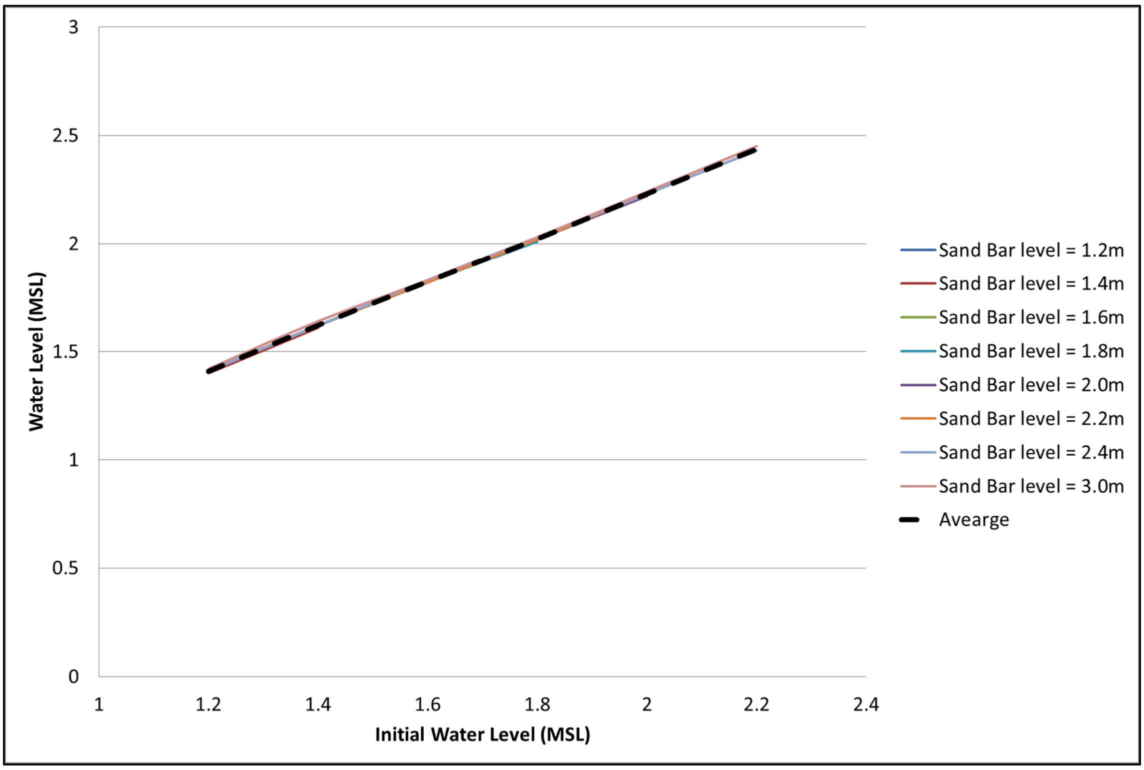

Figure F-12: Rondevlei peak water levels after 50yr flood for different sand bar heights and initial water levels

F.5. 100 year flood

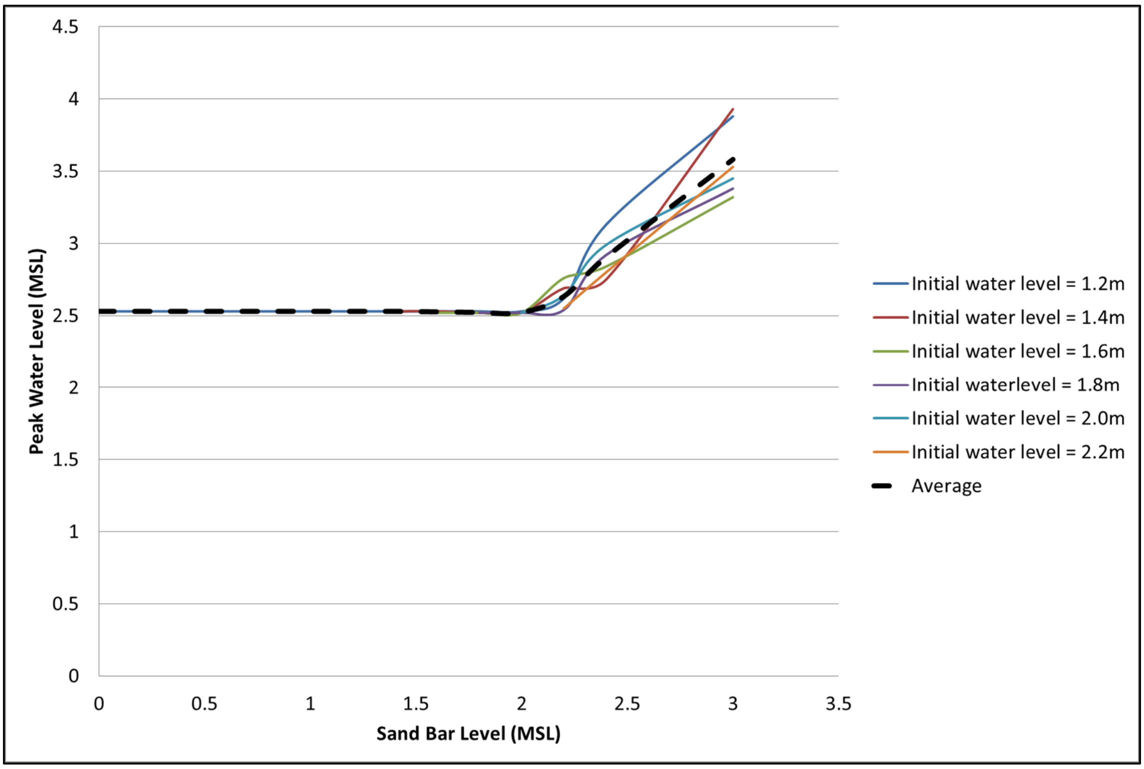

Figure F-13: Touw estuary peak water levels after 100yr flood for different sand bar heights and initial water levels

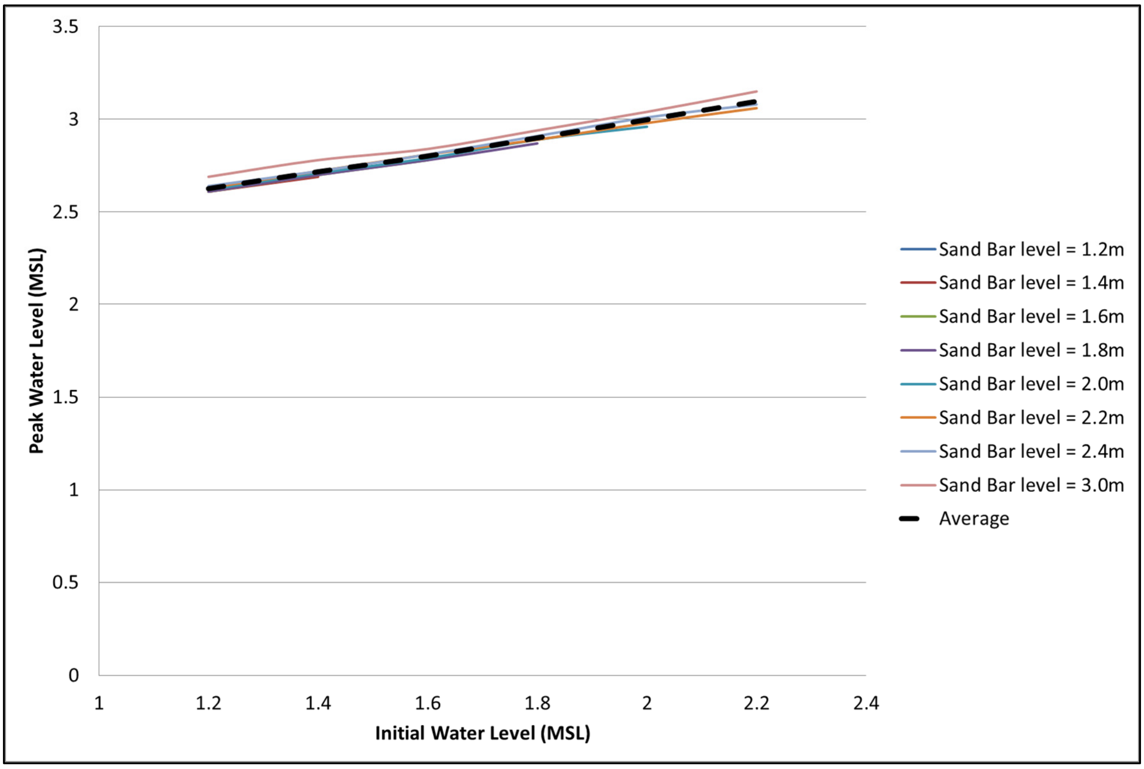

Figure F-14: Island Lake peak water levels after 100yr flood for different sand bar heights and initial water levels

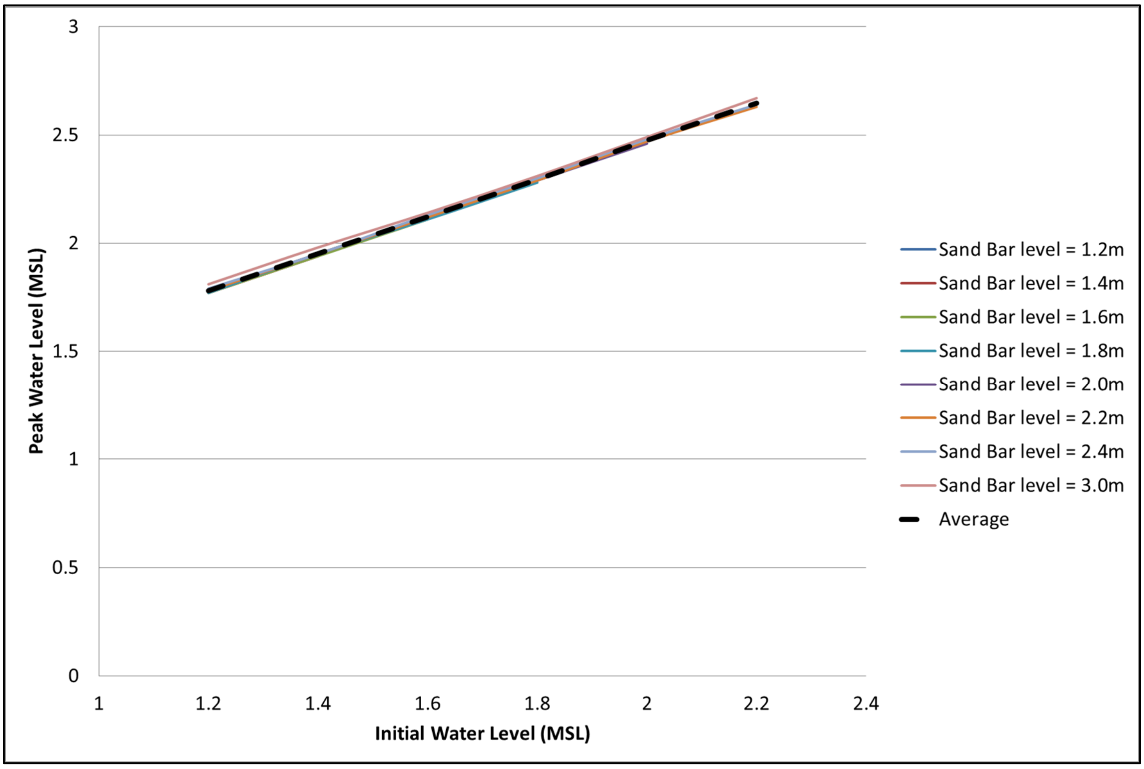

Figure F-15: Bolangvlei peak water levels after 100yr flood for different sand bar heights and initial water levels

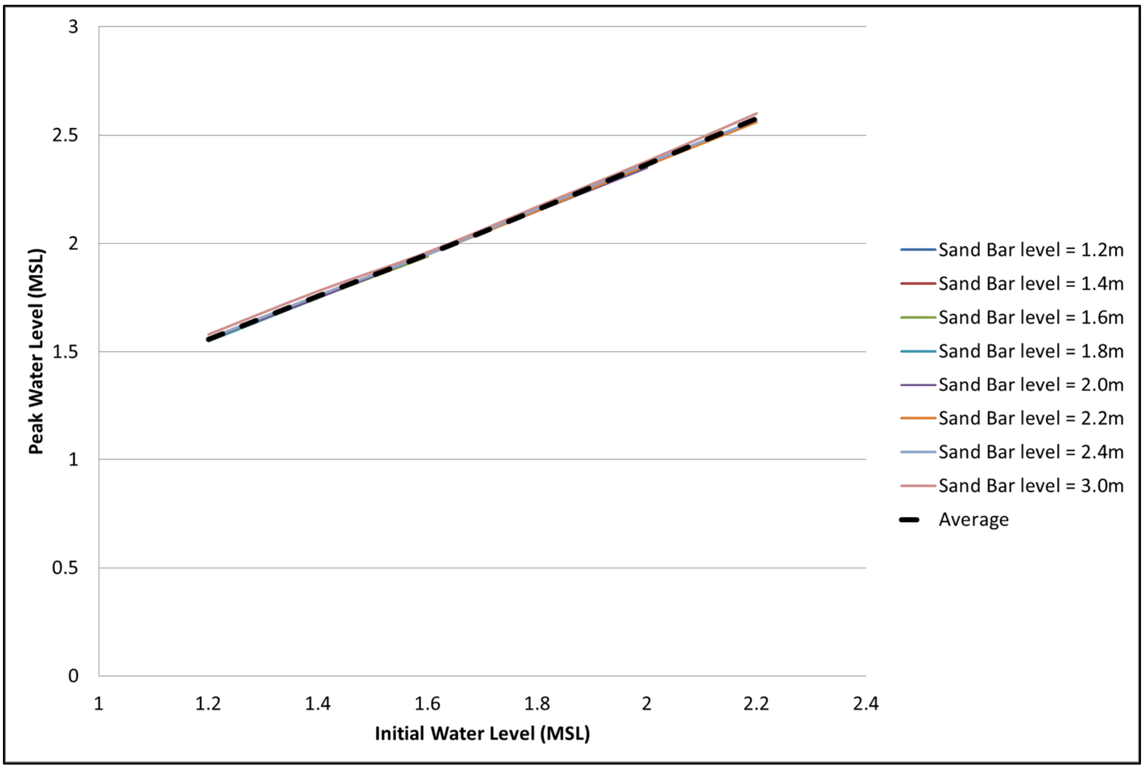

Figure F-16: Rondevlei peak water levels after 100yr flood for different sand bar heights and initial water levels

APPENDIX G. HYPOTHETICAL SALINITY RESULTS

G.1. Introduction

The following figures show how the salinity levels change throughout the simulation period under the different hypothetical conditions. The first hypothetical condition was a yearlong period with zero freshwater inflow and a mouth condition which remained open for the entire period. The second hypothetical condition was a yearlong period where an average annual freshwater inflow was simulated and the estuary mouth remained open for the entire period. The final hypothetical condition included a yearlong period where an average annual freshwater inflow was simulated and the estuary mouth remained closed for the entire simulation period. For each of these different hypothetical scenarios, different initial salinity levels were analysed in order to evaluate how different salinity levels fluctuate given the hypothetical conditions.

G.2. Salinity levels for zero freshwater inflow and an open mouth condition

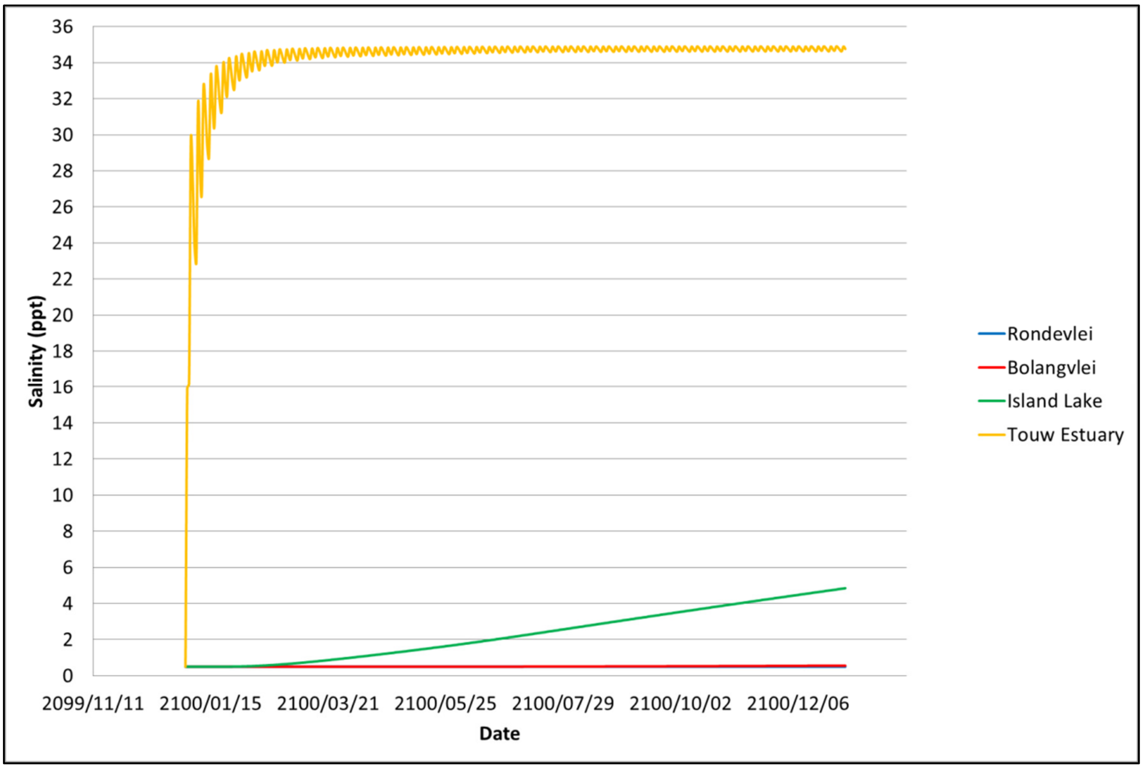

Figure G-1: Salinity variation during simulation period for initial salinities of 0.5 ppt (Zero freshwater inflow and open mouth condition)

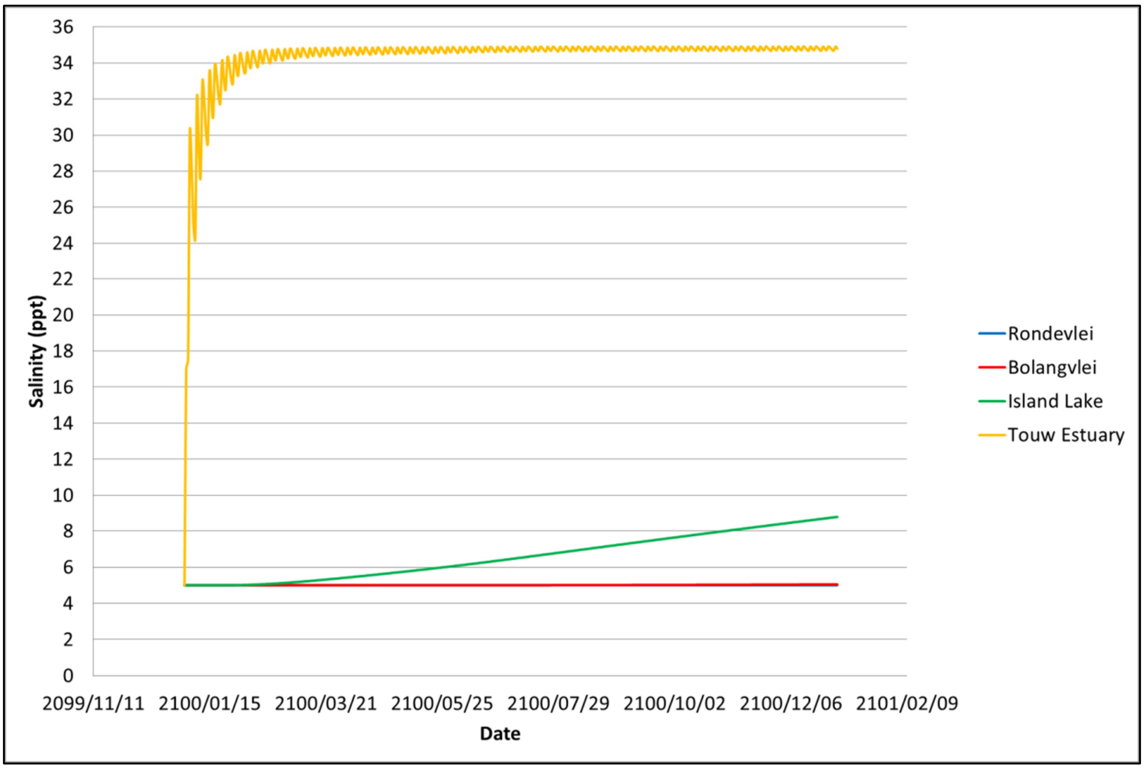

Figure G-2: Salinity variation during simulation period for initial salinities of 5 ppt (Zero freshwater inflow and open mouth condition)

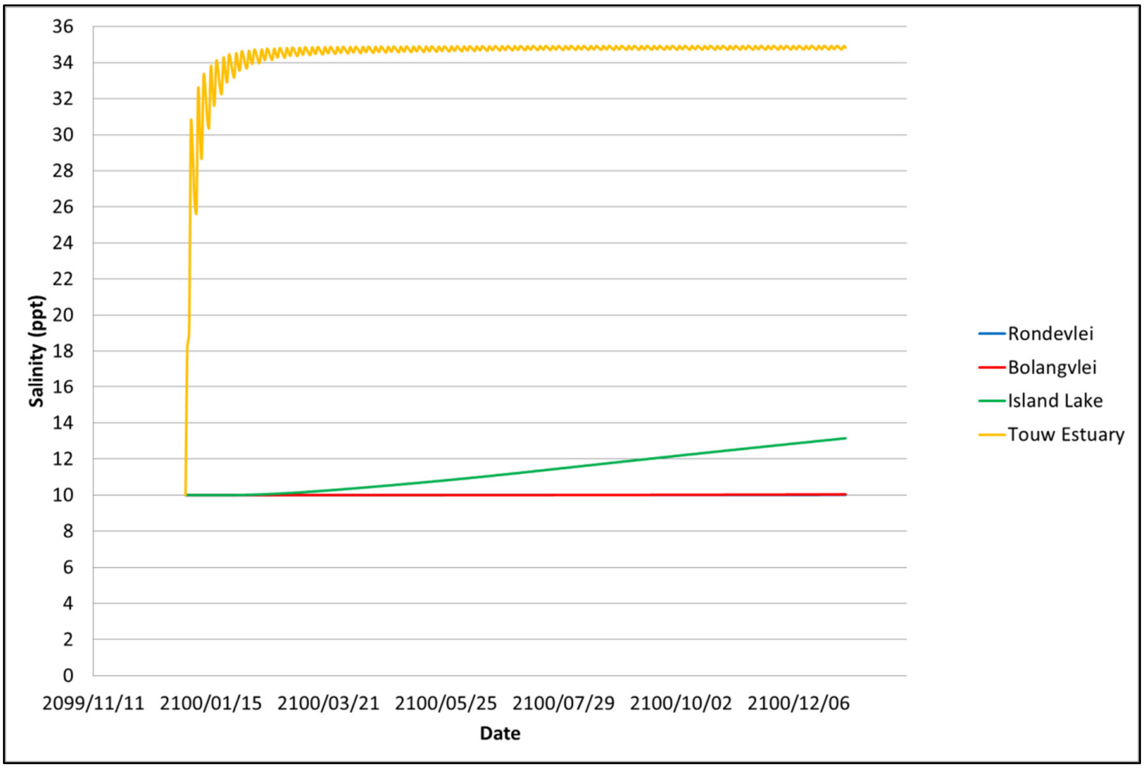

Figure G-3: Salinity variation during simulation period for initial salinities of 10 ppt (Zero freshwater inflow and open mouth condition)

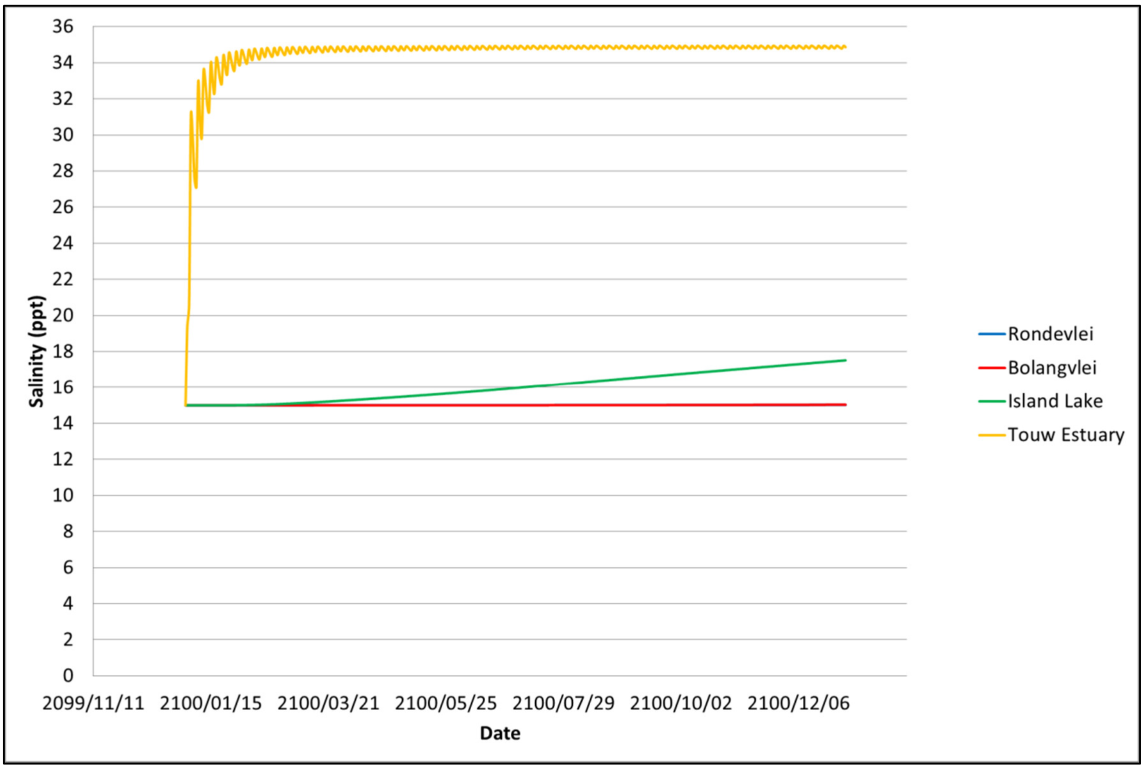

Figure G-4: Salinity variation during simulation period for initial salinities of 15 ppt (Zero freshwater inflow and open mouth condition)

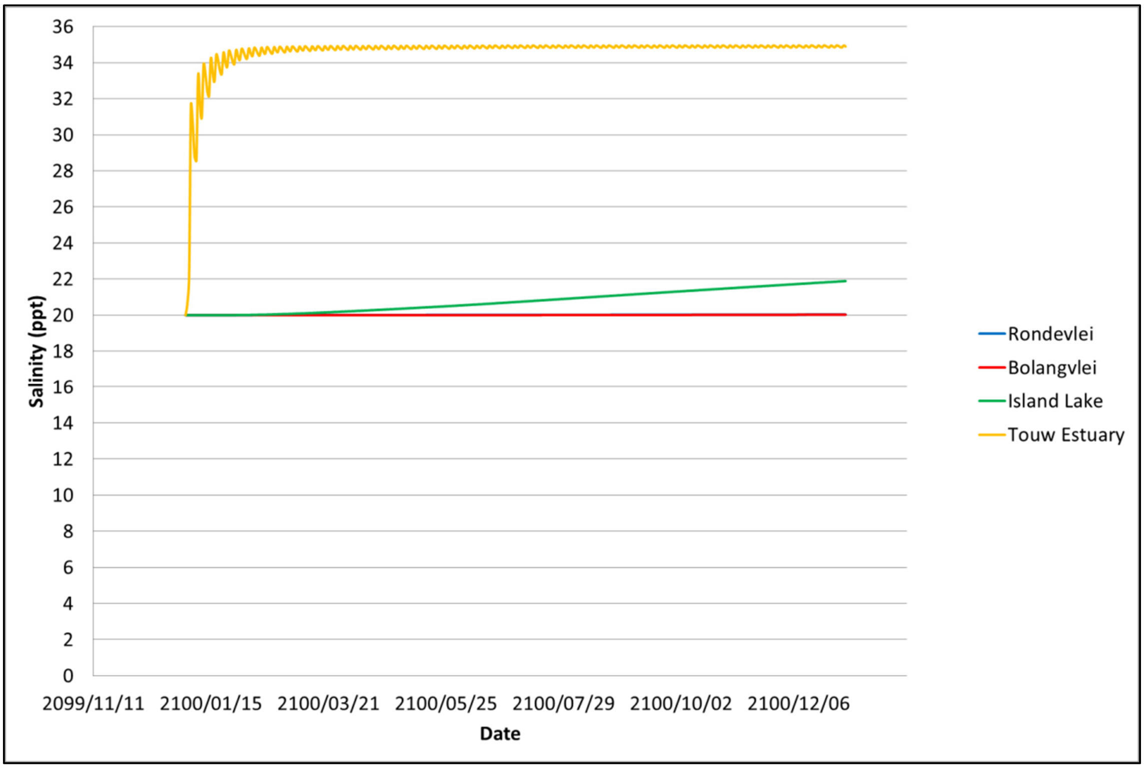

Figure G-5: Salinity variation during simulation period for initial salinities of 20 ppt (Zero freshwater inflow and open mouth condition)

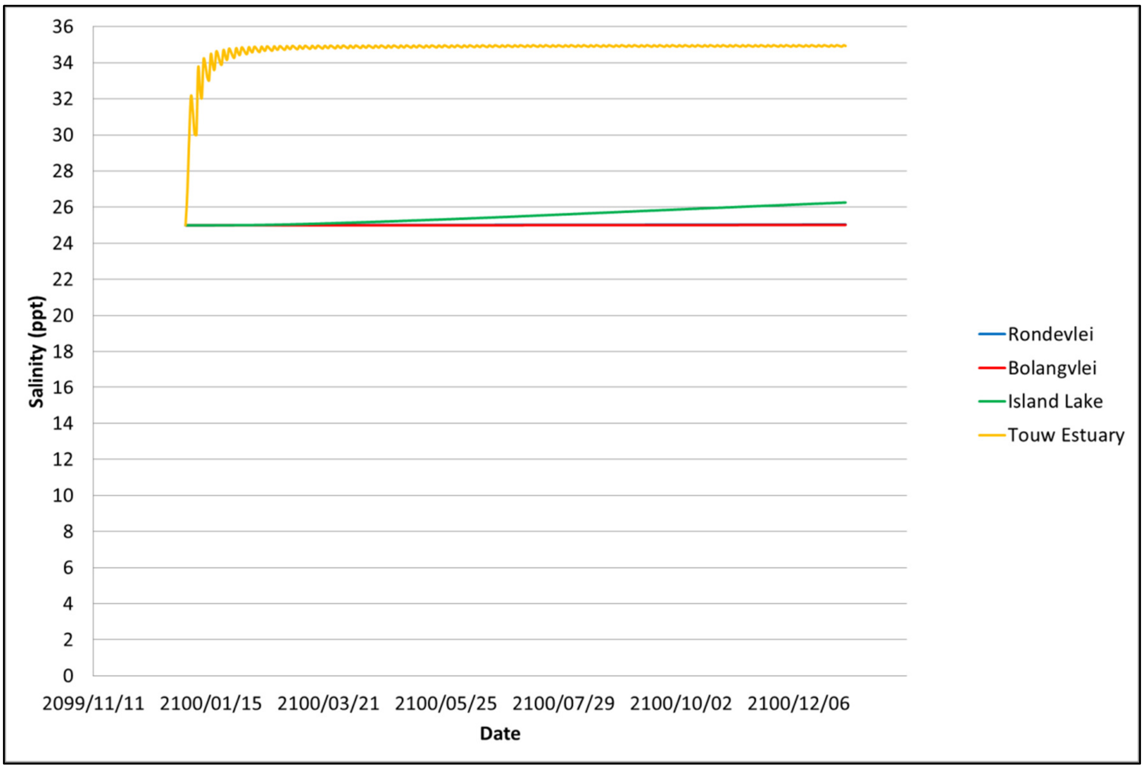

Figure G-6: Salinity variation during simulation period for initial salinities of 25 ppt (Zero freshwater inflow and open mouth condition)

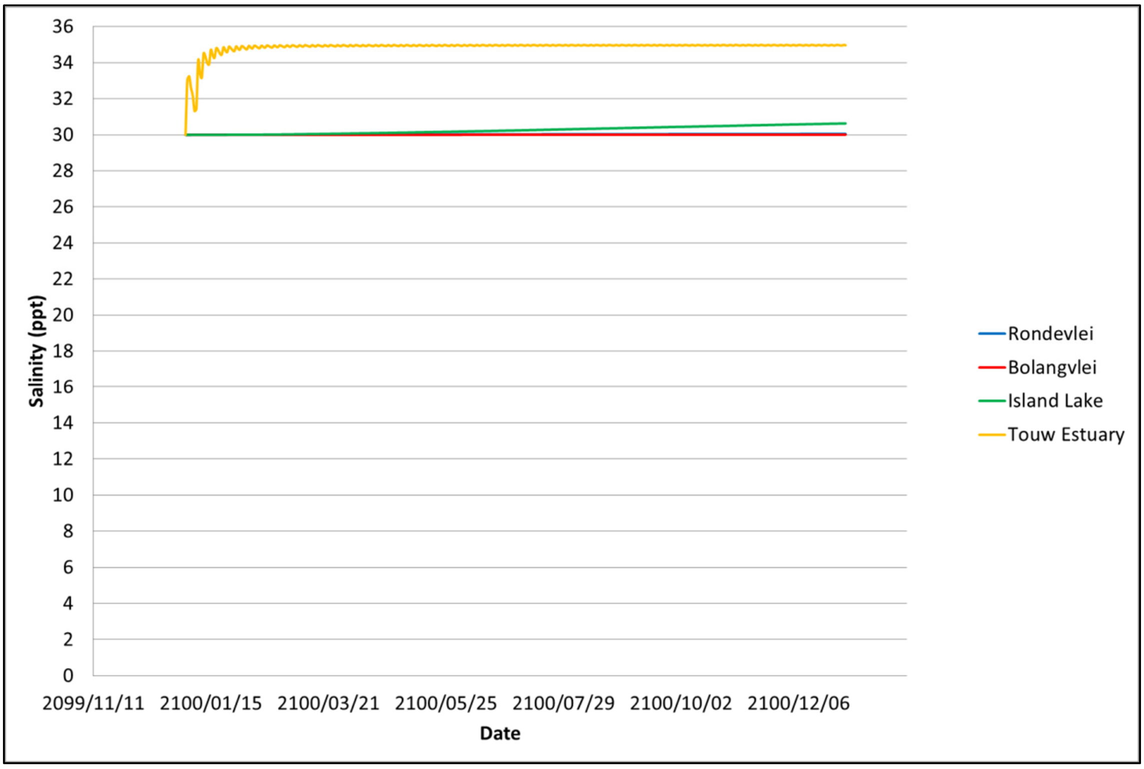

Figure G-7: Salinity variation during simulation period for initial salinities of 30 ppt (Zero freshwater inflow and open mouth condition)

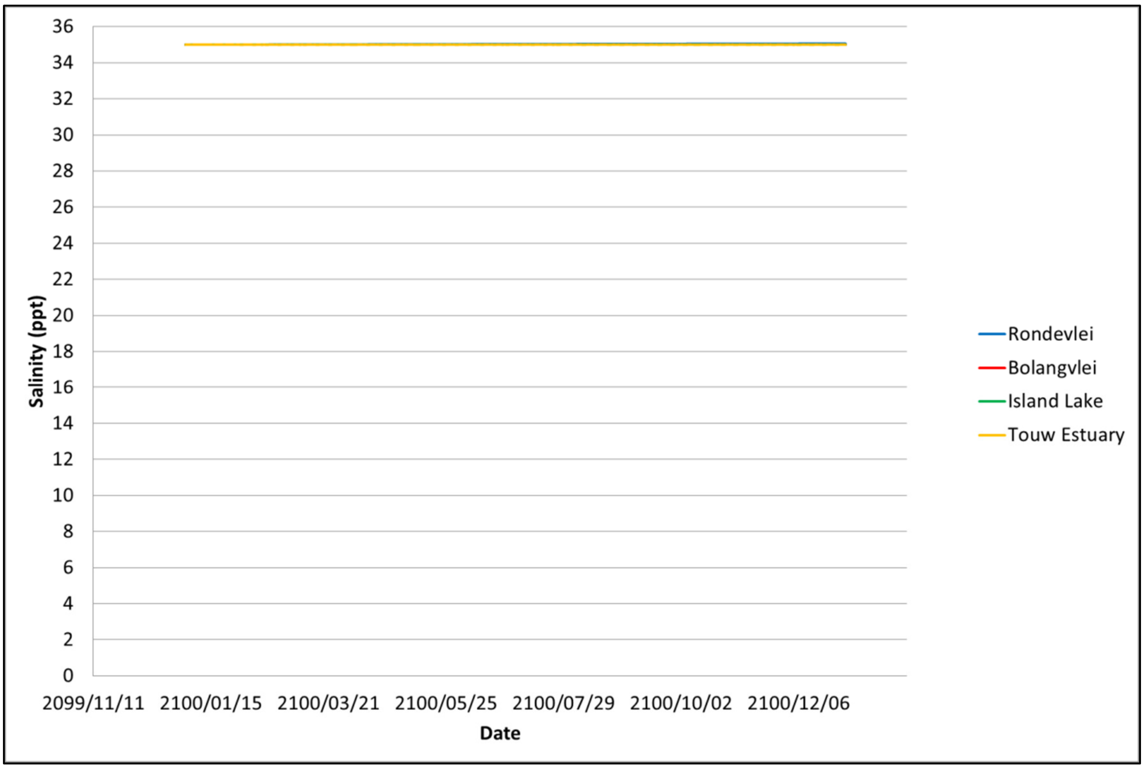

Figure G-8: Salinity variation during simulation period for initial salinities of 35 ppt (Zero freshwater inflow and open mouth condition)

G.3. Salinity levels for average annual freshwater inflow and an open mouth condition

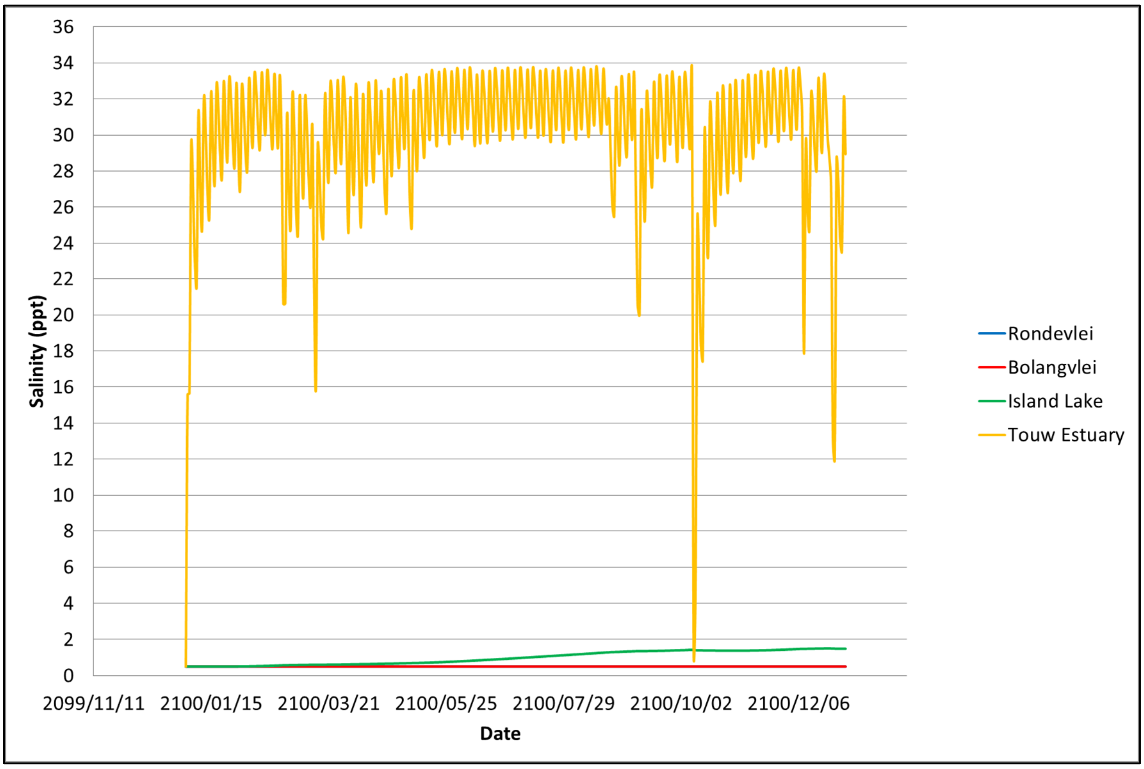

Figure G-9: Salinity variation during simulation period for initial salinities of 0.5 ppt (Average annual freshwater inflow and open mouth condition)

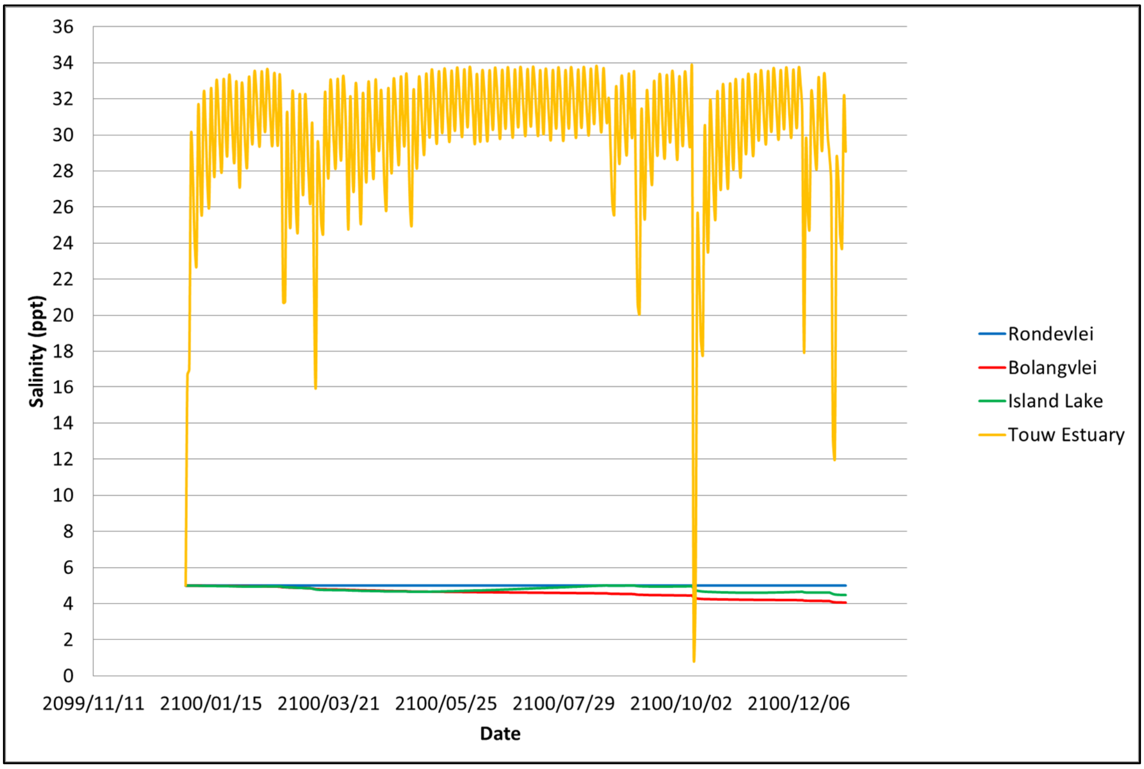

Figure G-10: Salinity variation during simulation period for initial salinities of 5 ppt (Average annual freshwater inflow and open mouth condition)

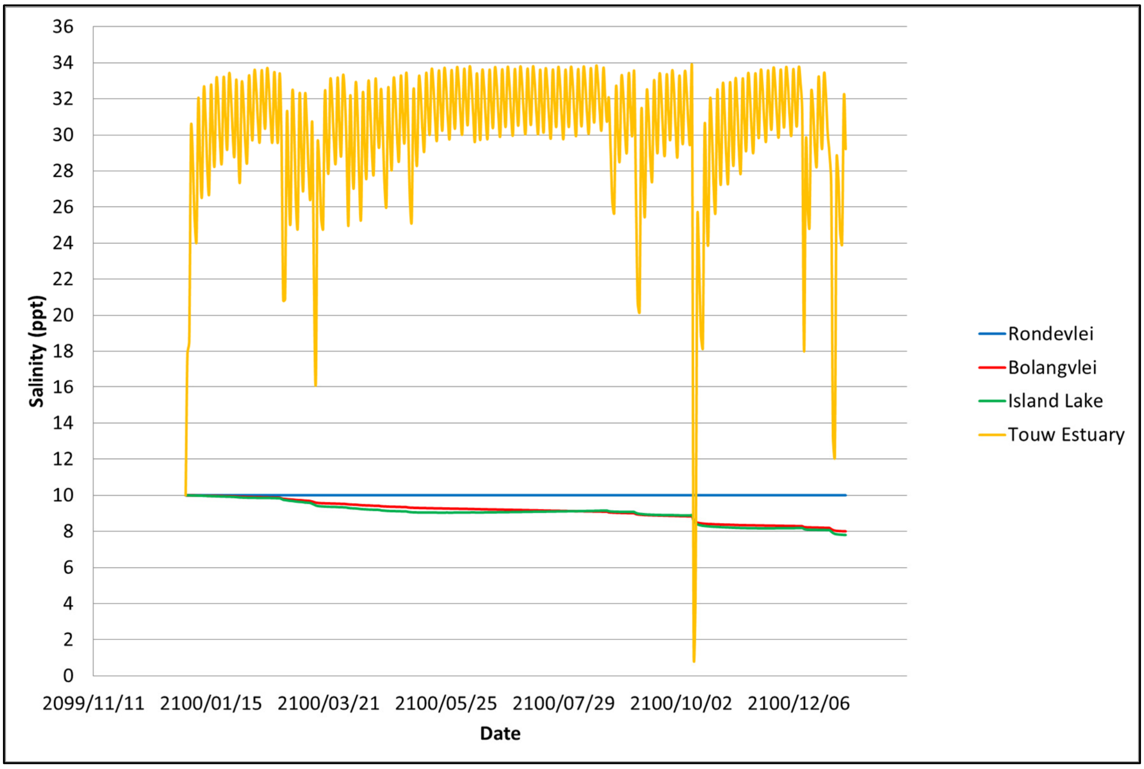

Figure G-11: Salinity variation during simulation period for initial salinities of 10 ppt (Average annual freshwater inflow and open mouth condition)

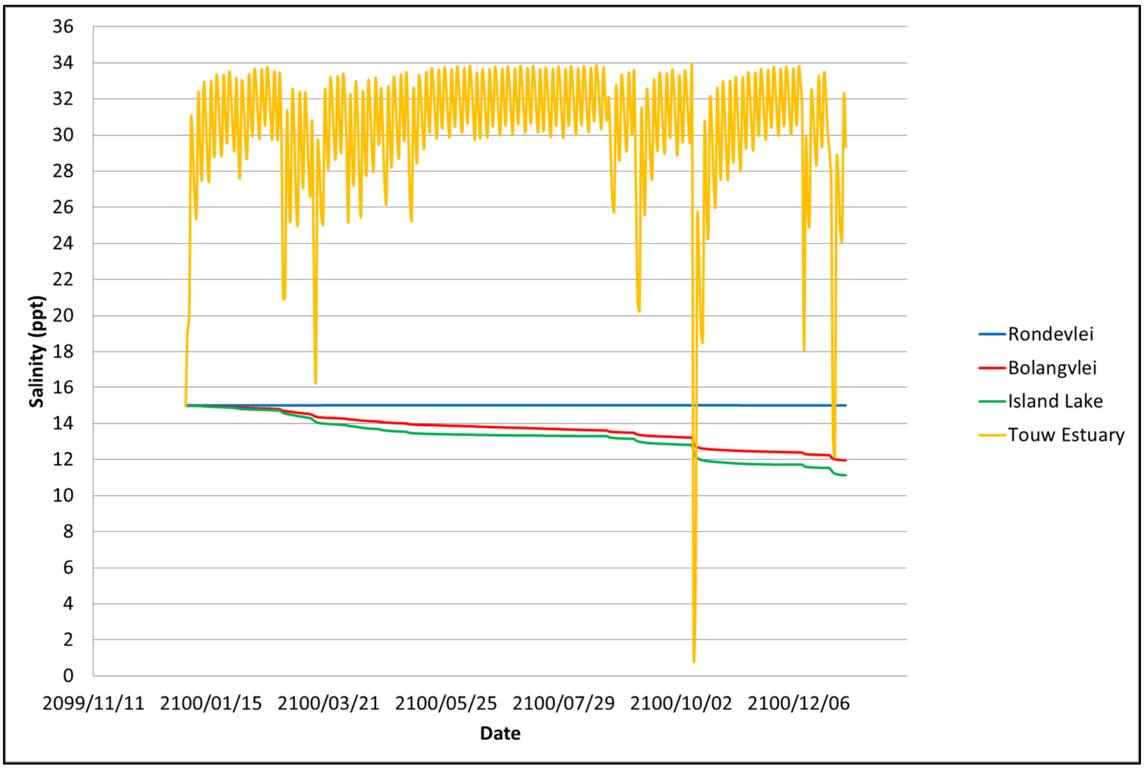

Figure G-12: Salinity variation during simulation period for initial salinities of 15 ppt (Average annual freshwater inflow and open mouth condition)

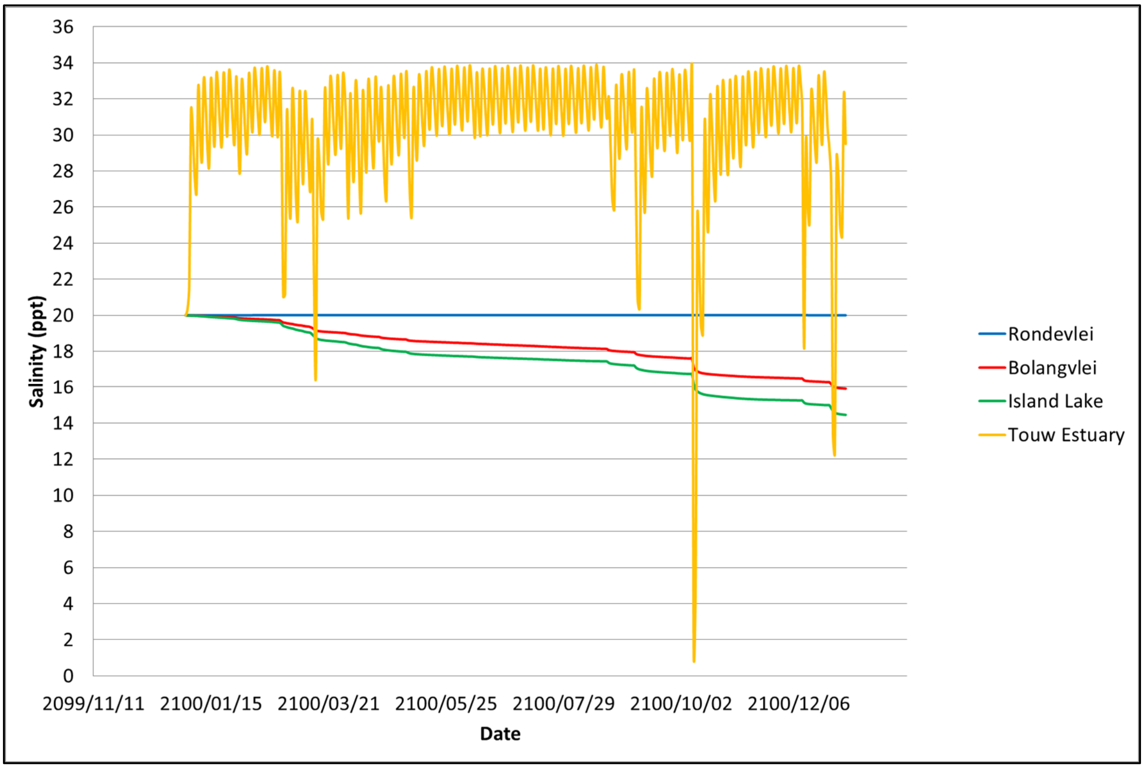

Figure G-13: Salinity variation during simulation period for initial salinities of 20 ppt (Average annual freshwater inflow and open mouth condition)

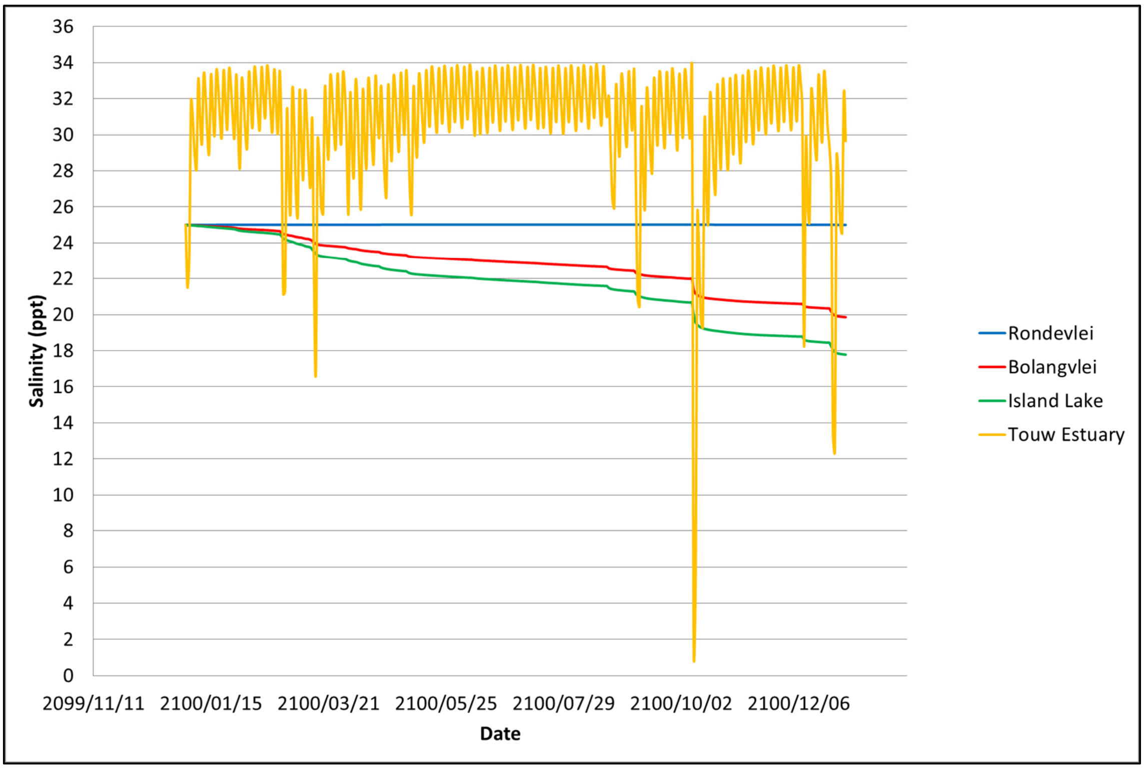

Figure G-14: Salinity variation during simulation period for initial salinities of 25 ppt (Average annual freshwater inflow and open mouth condition)

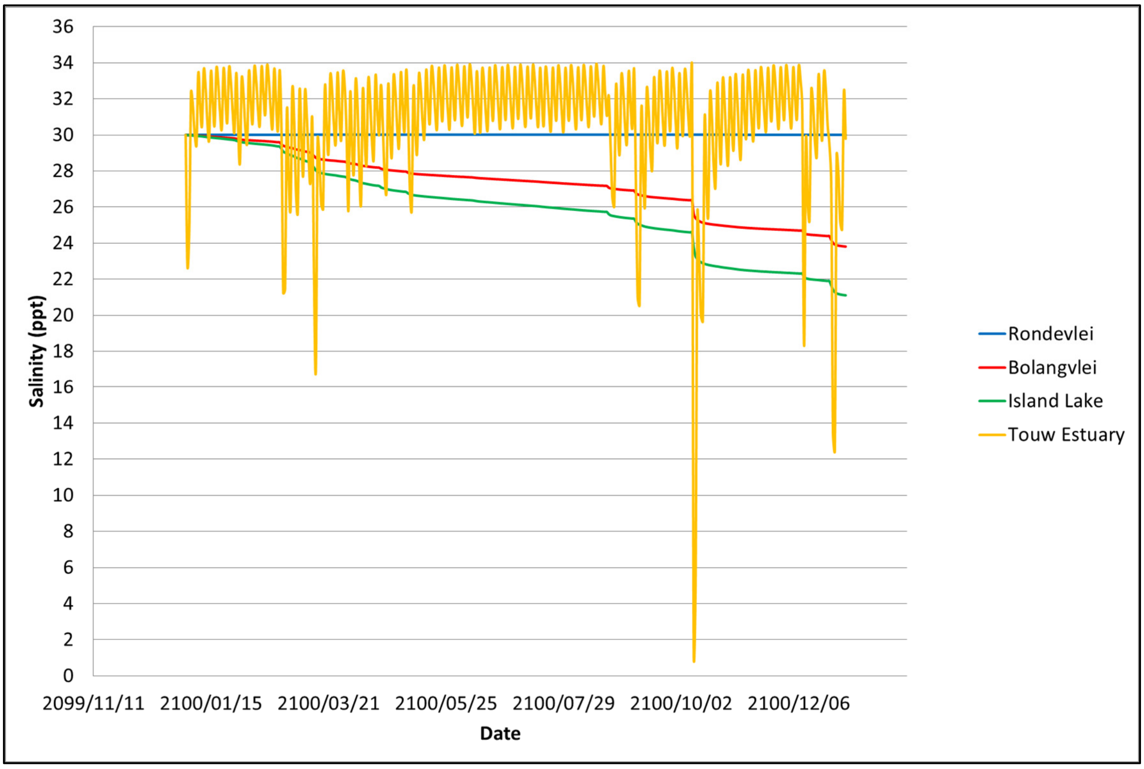

Figure G-15: Salinity variation during simulation period for initial salinities of 30 ppt (Average annual freshwater inflow and open mouth condition)

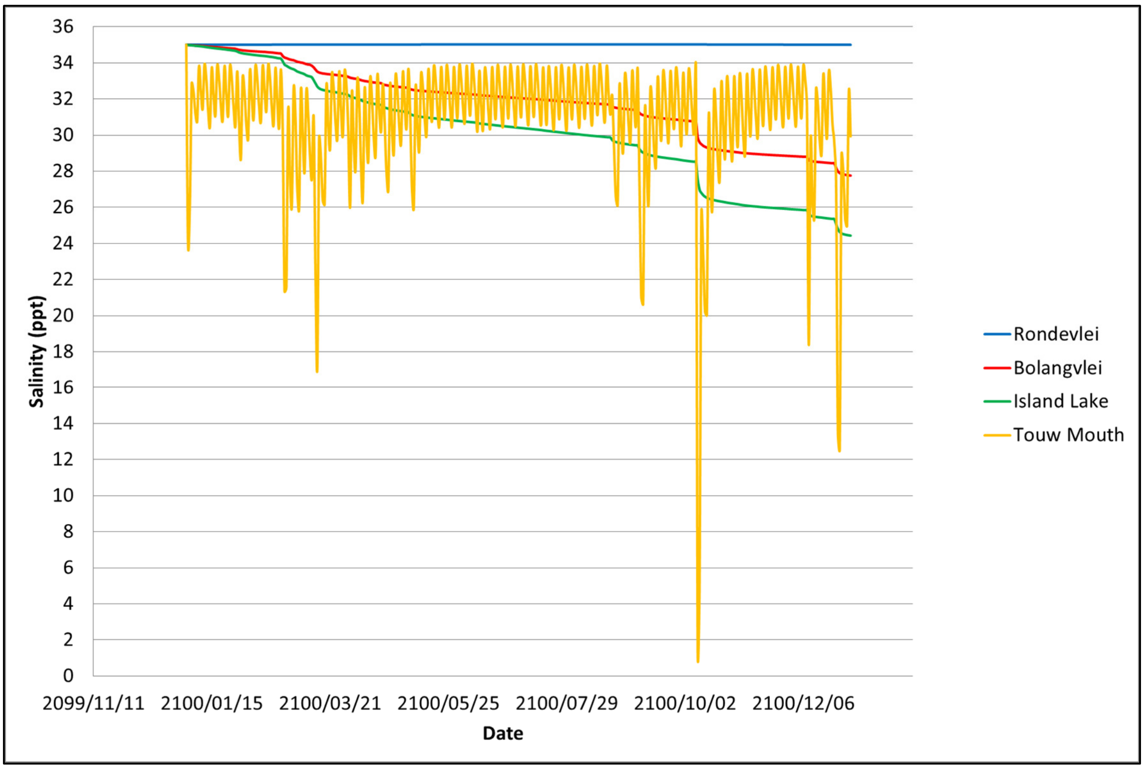

Figure G-16: Salinity variation during simulation period for initial salinities of 35 ppt (Average annual freshwater inflow and open mouth condition)

G.4. Salinity levels for average annual freshwater inflow and a closed mouth condition

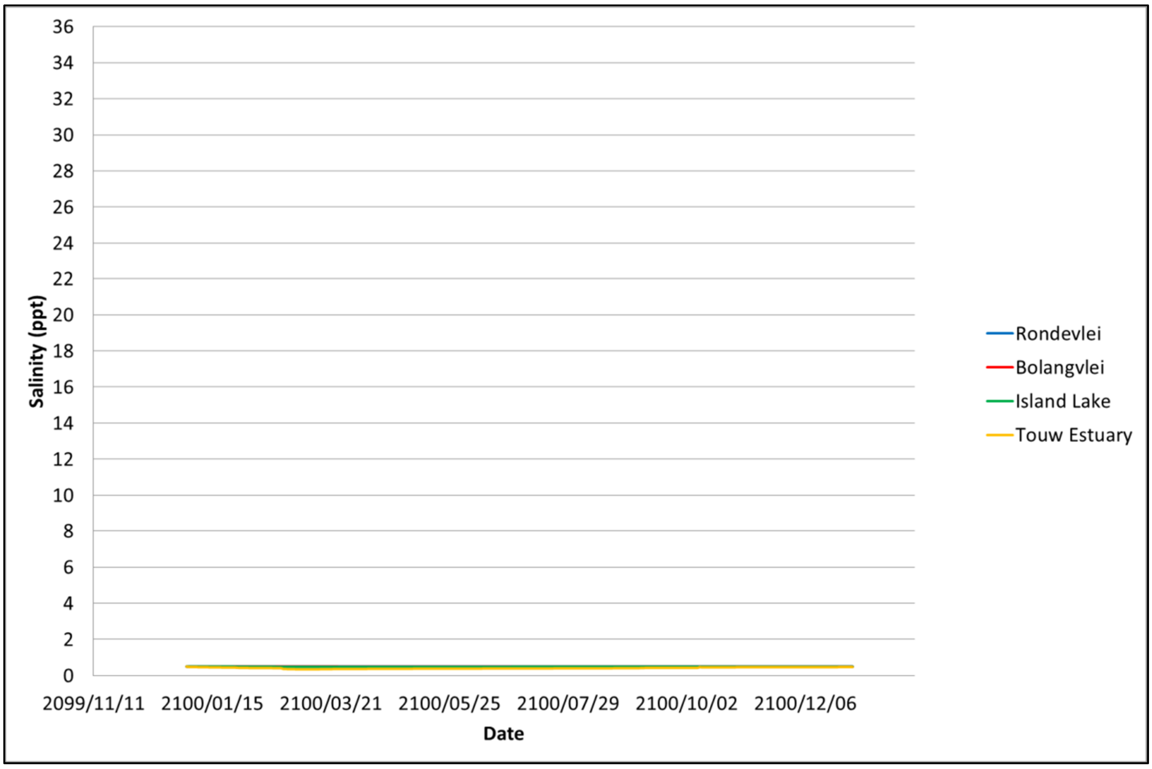

Figure G-17: Salinity variation during simulation period for initial salinities of 0.5 ppt (Average annual freshwater inflow and closed mouth condition)

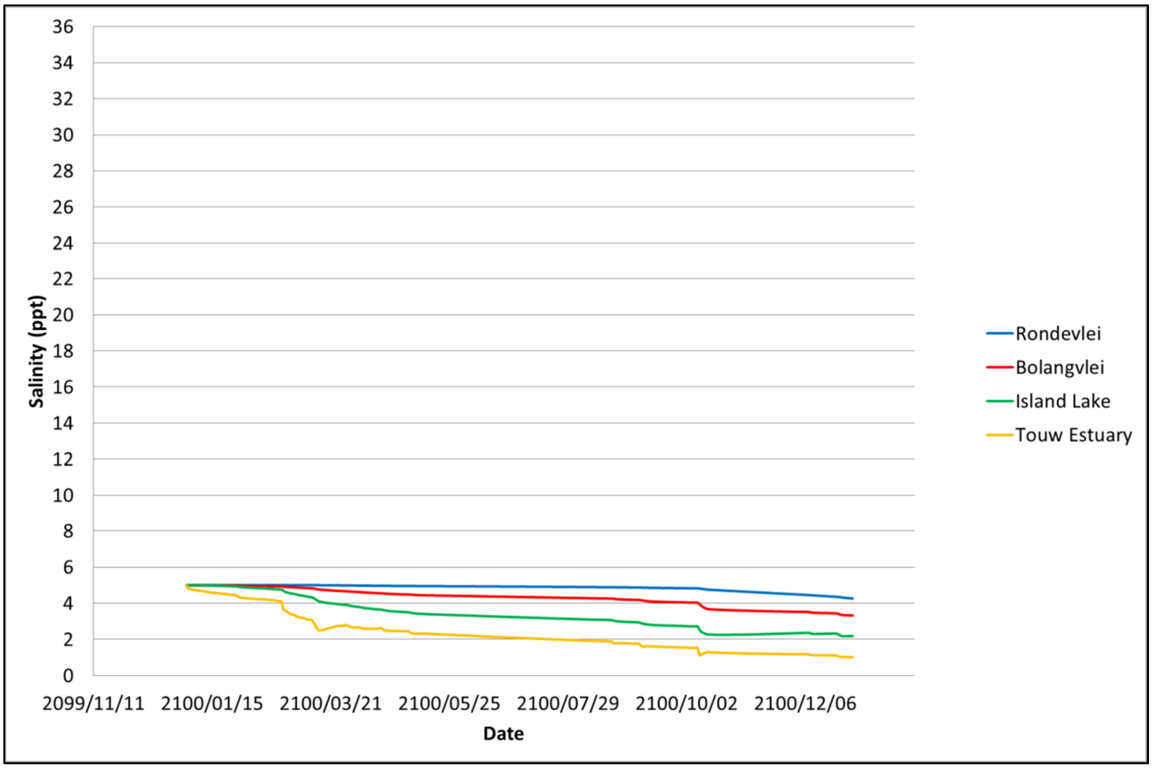

Figure G-18: Salinity variation during simulation period for initial salinities of 5 ppt (Average annual freshwater inflow and closed mouth condition)

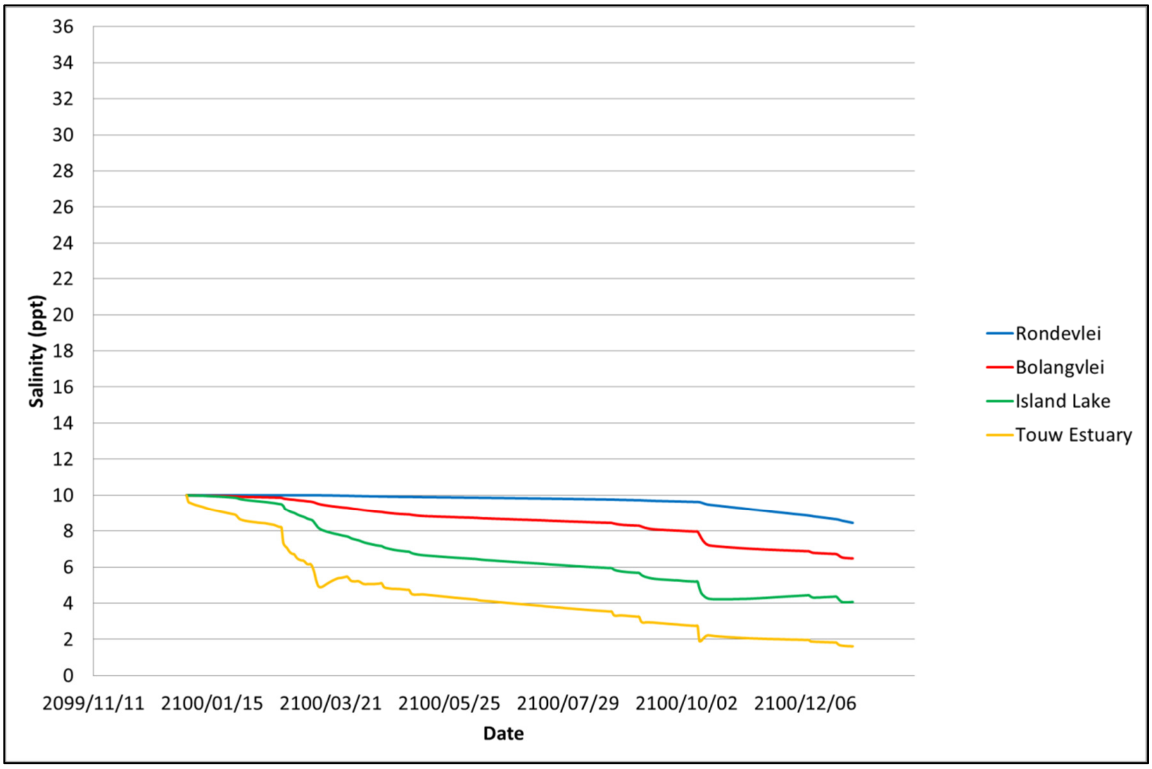

Figure G-19: Salinity variation during simulation period for initial salinities of 10 ppt (Average annual freshwater inflow and closed mouth condition)

Figure G-20: Salinity variation during simulation period for initial salinities of 15 ppt (Average annual freshwater inflow and closed mouth condition)

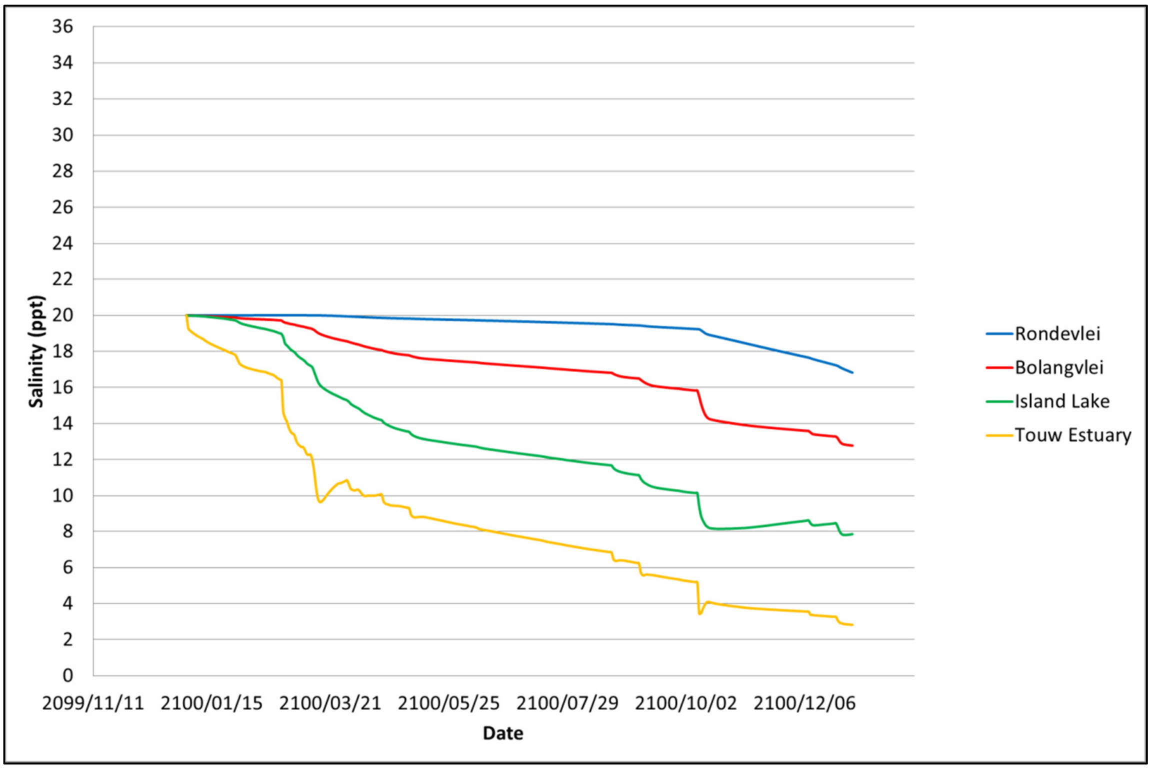

Figure G-21: Salinity variation during simulation period for initial salinities of 20 ppt (Average annual freshwater inflow and closed mouth condition)

Figure G-22: Salinity variation during simulation period for initial salinities of 25 ppt (Average annual freshwater inflow and closed mouth condition)

Figure G-23: Salinity variation during simulation period for initial salinities of 30 ppt (Average annual freshwater inflow and closed mouth condition)

Figure G-24: Salinity variation during simulation period for initial salinities of 35 ppt (Average annual freshwater inflow and closed mouth condition)