

The effect of sea level rise on flood levels in the Great Brak Estuary: assessing the adequacy of a 5 m setback line

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*Thesis presented in partial fulfilment of the requirements for the degree of Master of
Science in the Faculty of Civil Engineering at Stellenbosch University*

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March 2015

Declaration

By submitting this thesis electronically, I declare that the entirety of the work contained therein is my own, original work, that I am the authorship owner thereof (unless to the extent explicitly otherwise stated) and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

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Abstract

Global warming will result in a sea level rise of between 0.25 and 0.82 m by 2090, as well as an increase in intensity and frequency of both extreme sea level and extreme rainfall events. In consequence, low-lying areas will be permanently inundated, extreme waves will penetrate further inland and flood intensity and frequency will increase. Estuaries are subject to the effect of both extreme sea levels and extreme floods and water levels in estuaries are expected to increase, under both open and closed conditions. As a response to expected higher flood levels, setback lines have been legislated in South Africa. For cases where a flood level study has not been undertaken, a minimum setback line at the 5 m above mean sea level (MSL) contour is prescribed in terms of the National Environmental Management Act (Act 107 of 1998).

This study assessed the adequacy of the 5 m setback line, under the effects of climate change, for Great Brak estuary. Local features of the Great Brak estuary may influence flood levels. Specifically, the lagoon of the Great Brak estuary, below the N2 Bridge, is small at 1.1 x 0.7 km. Further, it is constrained at the upstream end by road and rail embankment, and on the left bank by steep slopes. A sand barrier at the mouth is at times breached, both naturally and artificially. Artificial breaching is initiated when the sand barrier is between 1.5 and 2.0 m high, or when a flood is forecast. The barrier has previously reached 2.7 m, higher than the still water level of the sea, which has not exceeded 2 m above MSL. There is a populated island about 180 m upstream of the mouth. The greater extent of the island is below 2.5 m above MSL.

Mike11 software was used to generate flood levels on which the conclusions of this study are based. The study determined that the influence of the increased sea levels does not extend much beyond the N2 Bridge. This may be a peculiarity of the Great Brak estuary, due to the influence of the three bridges and the road and rail embankments. For the scenario where Mean High Water Springs coincides with an extreme sea storm and there is a 100-year riverine flood, the flood level in the estuary is 3.16 m at the mouth, increasing to 4 m upstream of the N2 bridge. In the scenario where the barrier height was raised to 4 m above MSL, the flood levels were 4.52 m downstream of, and 5 m upstream of, the N2 Bridge. Extensive inundation of properties in the floodplain and on the Island will occur, as well as the inundation of the N2 embankment. The probability of such an extreme sea level event occurring at the same time as peak runoff of a 100-year riverine flood is unlikely.

It is the conclusion of this study that, for the Great Brak River, the 5 m setback line, as prescribed, is sufficient for an extreme situation where a future 100-year flood coincides with the MHWS coincides and an extreme sea storm raising the sea level to 2.65 above current MSL.

Opsomming

Aardverwarming sal lei tot 'n styging van seevlakte van tussen 0.25 en 0.82 m teen 2090, sowel as 'n toename in intensiteit en frekwensie van beide stormseevlak en reëerval.

Gevollik sal laagliggende gebiede permanent oorval word, stormgolwe verder in die binneland dring en vloed intensiteit en frekwensie toeneem. Riviermondings is onderhewig aan die effek van beide hoë seevlakte en vloede. Om die negatiewe effekte van hoë vloedvalktes te bekamp word 'n minimum terugsetlyn van 5 m bo seevlak voorgeskryf, in terme van die Wet op Nationale Omgewingsbestuur (Wet 107 van 1998). Hierdie is van toepassing waar 'n vloedlyn studie nie onderneem is nie.

Hierdie studie beoordeel die gesiktheid van die 5 m terugsetlyn, onder die invloed van klimaatsverandering, vir Groot Brak rivier monding. Plaaslike kenmerke van die Groot Brak monding mag vloed vlakke beïnvloed. Spesifiek, die Groot Brak monding meer het 'n oppervak van net $1,1 \times 0,7$ km; is in die stroomop rigting beperk deur pad en spoor walle; en word op linkeroewer deur steil hellings vesper. Die sandversperring by die word kunsmatig oopgemak wanneer die sand versperring tussen 1,5 en 2,0 m hoog is, of wanneer 'n vloed voorspel word. Hierdie sandversperring het al voorheen 2,7 m hoogte beriek, hoër as die 2 m maksimum historiese stilwater vlak van die seë. Daar is 'n bevolkte eiland sowat 180 m stroomop van die mond. Die die eiland is meestelik onder 2,5 m bo seevlak.

Mike11 sagteware is gebruik om vloed vlakke, waarop die bevindinge van hierdie studie gebaseer is, te bepaal. Hiedie studie bevind dat die effek hoë voedvlakke trek nie veel verder stroomop as die N2 brug, oontlike weens die voorkoms van die drie bruë. In die geval waar 'n uiterste seëstorm terselfde tyd voorkon as die lente hoogwater gety endie 100 jaar river vloed, sal die watervlak in the mondingsmeer tot 3.16 m bo huidiglike seevlak styg by die mond, en tot 4 m bo huidiglike seevlak by die N2 brug. In die geval waar die sandversperring by the riviersmond 4 m verhoog is, sal die watervlak in the mondingsmeer tot 4.5 m bo huidiglike seevlak styg by die mond, en tot 5 m bo huidiglike seevlak by die N2 brug. Faktore nie in ag geneem in hierdie studie sluit in die uitwerking van die verhoogde afloop, sediment verandering en die effek van windgolwe oor die ondingsmeer. Wydverspreide vloeding van ontwikkelde areas aangrensend to vloedvlakte sal voorkom, insluitend die oorstroing van die N2 padwal.

Die waarskynlikheid is klein dat 'n uiterste seëstorm terselfde tyd voorkom as the lengte hoogwater gety en die 100 jaar rivier vloed. Dit is dus die gevolgtrekking van hierdie studie dat die 5 m terugsetlyn soos voorgeskryf, voldoende is vir Groot Brak rivier vir so 'n uiterste geval.

Acknowledgements

The preparation of this study was supported by a number of institutions and individuals. Their contribution is gratefully acknowledged.

For sharing their experience and knowledge of estuaries

- Lara van Niekerk (CSIR Stellenbosch, Coastal Systems Research Group), also for providing survey and bathymetry data of Great Brak estuary
- Andrew Mather (eThekweni Municipality, telephonic interview)
- Marius Rossouw (CSIR Stellenbosch, Coastal Systems Research Group)
- Andre Theron (CSIR Stellenbosch, Coastal Systems Research Group)

For providing information

- Johan Pieterse, for sharing survey information of the estuary
- The Mossel Bay Municipality, for authorising access to information on the estuary
- The South African Weather Service
- The Surveyor General.

Danish Hydraulics Institute (DHI Group) is gratefully acknowledged for making the software MIKE11 freely available for use in this study.

The support and guidance of the staff of the Department of Civil Engineering (University of Stellenbosch) has been invaluable and much appreciated.

The guidance, support and patience of Geoffrey Toms are especially valued.

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Abbreviations

%	Percentage
AR 4	Fourth Climate Change Assessment Report (IPCC)
AR 5	Fifth Climate Change Assessment Report (IPCC)
CEM	Coastal Engineering Manual (of USACE)
CERM	Consortium for Estuarine Research and Management
CMP	Coastal Management Programme
CPSL	Coastal Process Setback Line
CPZ	Coastal Protection Zone
CSIR	Council for Scientific and Industrial Research
CSL	Coastal Setback Line
Cumec	Cubic metres per second
DEA	Department of Environmental Affairs
DEDEA	Department of Economic Development and Environmental Affairs
DHI	Danish Hydraulic Institute
DWA	Department of Water Affairs
EIA	Environmental Impact Assessment
EMP	Environmental Management Plan
EWISA	Electronic site of Water Institute of South Africa
GMSL	Global Mean Sea Level
HAT	Highest Astronomical Tide
HWM	High Water Mark
ICM	Integrated Coastal Management
ICM Act	Integrated Coastal Management Act
IDP	Integrated Development Plan
IPPC	Inter-governmental Panel on Climate Change
km	Kilometre

LLD	Land Level Datum
m	metre
m^3	cubic metre
MAP	Mean Annual Precipitation
MAR	Mean Annual Runoff
MEC	Member of Executive Committee
MHWN	Mean High Water Neaps
MHWS	Mean High Water Spring
ML	Mean Level
MLWN	Mean Low Water Neap
MLWS	Mean Low Water Spring
mm	millimetre
MSL	Mean Sea Level
NEMA	National Environmental Management Act
NMMU	Nelson Mandela Metropolitan University
POE	Permanently Open Estuary
s	second
SA	South Africa
SANHO	South African Navy Hydrographic Office
SCS	Soil Conservation Service
SDF	Standard Design Flood
SLR	Sea Level Rise
SUH	Synthetic Unit Hydrograph
SUN	University of Stellenbosch
TOC	Temporarily Open/Closed Estuary
USACE	United States Army Corps of Engineers
WG	Working Group

1 Contents and structure of this document

This study commences with three sections which provide context for this study: Section 2 Introduction and background, which provides a broad overview of the issues informing this study; Section 3, on the prescripts and perspectives for setback lines in South Africa; and Section 3.3 which provides an overview of the Great Brak estuary. The definition of the problem to be studied and the scope of the work are set out in Section 4. Following the contextual sections above, an overview is given of South African estuaries, their features and categorization (Section 5). This overview provides a broad insight into the factors influencing form, features and dynamics of South African estuaries.

A literature review (Section 6) has been undertaken covering the effect of climate change on estuaries, rainfall and runoff, and sea levels. Included in this section is a short discussion on the hydraulics of estuaries, which informs the modelling decisions.

Section 7 of this study sets out the methodology used for modelling of water levels, in the estuary below the N2 bridge, under 100-year riverine flood and sea level rise conditions. Section 8 describes the features, of Great Brak River and estuary, affecting the hydrology (runoff) and the hydraulics of the estuary. Sections 9 to 11 describe the model inputs, challenges in modelling and calibration of the model. Results of the research are given in Section 12. The final section (Section 13) of the report provides conclusions based on findings of the study.

2 Introduction and background

“Warming of the Climate System is unequivocal, as is now evident from observations of increase in global average air and ocean temperatures, widespread melting of snow and ice and rising global average sea level”. (Intergovernmental Panel on Climate Change, 12-17 November 2007, p. 30)

The fifth Climate Change Assessment Report (AR 5), released in early 2014 by the Intergovernmental Panel on Climate Change (IPPC)¹, confirms the findings of AR 4 and assesses global warming as *virtually certain*². (Pachauri, et al., 2014). To date the oceans have absorbed the greater part of the global energy increase (Collins, et al., 2013). As the ocean temperature increases in response to the energy absorbed, the water expands and sea levels rise. Long term trends from tidal gauges across the world confirm that sea levels have risen 0.19 m (0.17 to 0.21 m) over the period 1901- 2010. Further, the IPCC find it *very likely*³ that the rate of sea level rise increased after 1993, from 1.9 mm per annum to 3.2 mm/annum (Rhein, et al., 2014), and that the rate of sea level rise will continue to increase over the observed 1971-1990 rate of increase (Church, et al., 2013). The predicted eustatic sea level rise by 2081-2100 is between 0.25 m and 0.82 m, relative to 1985-2005, for various radiation penetration scenarios, as shown in Figure 1. Due to the eustatic rise in sea levels, coastal and low-lying areas currently above the water line will become permanently inundated (Theron & Rossouw, 2008).

In addition to eustatic sea level rise, sea levels under storm conditions will be affected by climate change. Sea level extremes, are and will be, further increased, mainly due to the higher mean sea level (Goshen, 2013; Church, et al., 2013). The IPCC find it to be *very likely*, and *likely*⁴ that sea levels may also be affected by increased intensity of storms, continuing the currently observed increase in frequency of the most intense storms (Church, et al., 2013). Coastal areas affected by extreme sea level rise will be subjected to increased damage due to the increased intensity of storms, and damage will extend further inland (Theron & Rossouw, 2008).

¹ The Assessment Report has four parts: the scientific basis (Working Group I); Impacts, Adaptation, and Vulnerability (Working Group II); Mitigation (Working Group III); and the Synthesis Report.

² 99% probability of exceedance

³ i.e.: with a 90% probability of exceedance

⁴ i.e.: with a probability of exceedance of 66%.

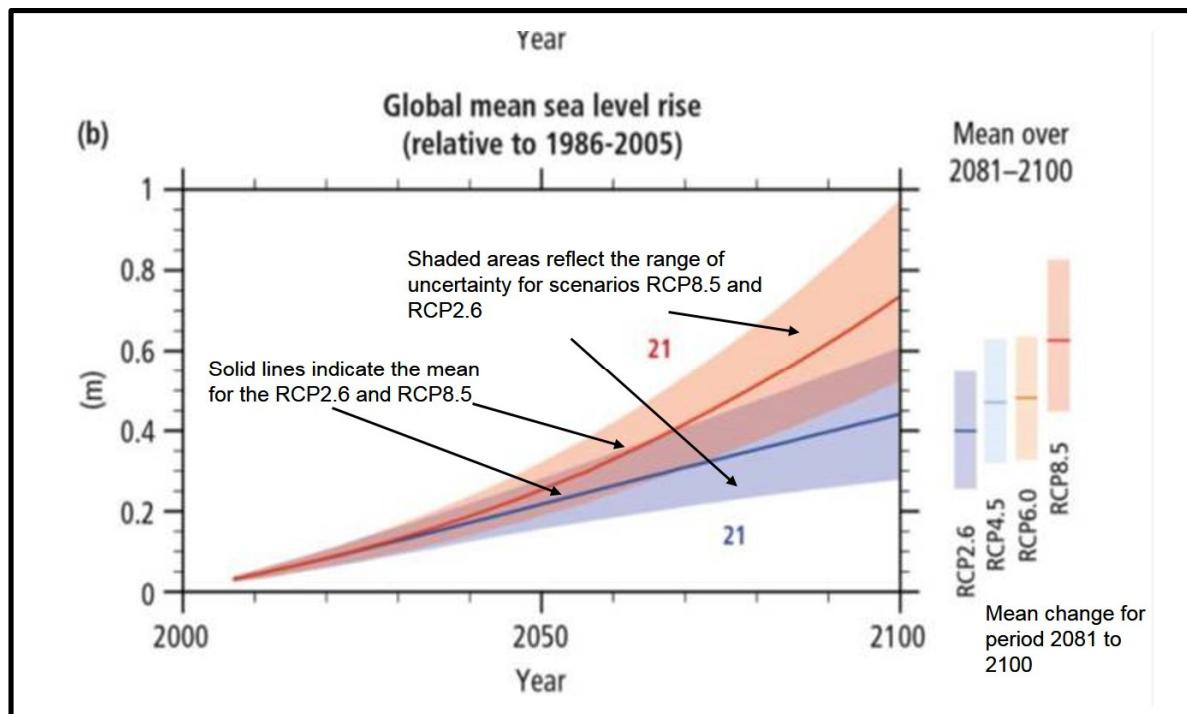


Figure 1: Global mean Sea Level Rise 2006-2100, from multi-modal simulations (Church, et al., 2013).

Globally, six hundred million people are estimated to live in the low elevation coastal zone, defined as the area below the 10 m above MSL contour, and are thus vulnerable to the impact of increased sea level and extreme sea level events. The number of people living in the coastal zone that will be affected by the 100-year extreme sea level event almost doubled between 1970 and 2010, with the largest increase in people affected by coastal flood risk being in sub-Saharan Africa (Wong, et al., 2014). In South Africa 30% of the population lives in coastal regions. The most vulnerable areas on the South African coast line have been identified as Northern False Bay, Table Bay, Saldanha Bay, the south Cape coast (Mossel bay to Nature's Valley), Port Elizabeth and the developed areas of the Kwa-Zulu Natal coast (Theron & Rossouw, 2008).

In respect of rainfall, while it is predicted that rainfall will generally decline under global warming, an increase is predicted in the rainfall intensity with a greater number of high rainfall events, resulting in increased flood intensity. Simulations indicated that the frequency of extreme 1-day events could increase from every 20 years to every 10 years (Collins, et al., 2013). As a result, flooding will take place more frequently and the extent of areas affected by flooding will increase (Theron & Rossouw, 2008). The Research Partnership (Santam Group, the WWF, and CSIR, 2011) reported in excess of R 2.5 billion in direct

damage in the Western Cape due to eight severe storms, for the period 2003-2008, with 70% of the damage in the Eden district (Gouritsmond to Nature's Valley, including the Great Brak River catchment).

Roets & Duffel-Canham (2009) state that:

"The only potential solution to these unpredictable events is the proactive determination and implementation of realistic development setback lines".

In terms of South African legislation⁵, provincial authorities may delineate coastal setback lines to protect public and private property; ensure public safety; delineate a coastal protection zone; and for aesthetic reasons. However, South Africa has over 3 000 km⁶ of coastline, and as a result the determination of accurate setback lines for each locality may be unaffordable. A cost of R 20 000 to R 35 000 per kilometre has been estimated, resulting in a cost of between R 32 million and R 56 million for the 1 600 km coastline of the Western Cape (DEADP, 2011).

Further, South Africa has 300 functional estuaries (Consortium for Estuarine Research and Management (CERM), n.d; Van Niekerk & Turpie, 2011) for which the delineation of setback lines is not only a function of sea levels, but also of riverine floods. In the absence of adequate data and resources to determine the setback lines in estuaries accurately, provisions in the National Environmental Management Act (Act No. 107, 1998) and Integrated Coastal Management Act (Act No. 24, 2008) serve as guidance for setback lines in estuaries. In terms of the National Environmental Management Act (NEMA) the setback line for estuaries is coincident with the 5 m above MSL contour. The rationale behind the setting of the 5 m contour includes considerations of flooding, potential effects of climate change on estuarine retreat and ecological requirements.

⁵ National Environmental Management Act: Integrated Coastal Management Act (Act No. 24 of 2008)

⁶ <http://www.gcis.gov.za/sites/default/files/docs/resourcecentre/yearbook/2003/ch1.pdf>

3 Context

3.1 Defining the term estuary

Estuaries occur where rivers flow into the sea (Cooper et al, 1999). Beyond this broad definition, there is disagreement on whether the term can be applied to situations where the mouth is not permanently open to the sea, or should be limited to situations where the lagoons⁷ are permanently connected to the sea. Wolanski (2007) and Day (1980), quoting Pritchard (1967), specify that an estuary should have “a free connection with the open sea”. This requirement is supported by the United States Environmental Protection Agency, (2014) and Badia Cebada (2003), who specify that an estuary should have “an exchange of fresh water and sea water”. This definition would exclude the application of the term “estuary” to:

- rivers that terminate in lagoons where there is a permanent barrier such that only fresh water flows into the lagoon. These are termed “coastal lakes” (Whitfield, 1992).
- rivers where the mouth is open intermittently. These are termed temporarily open/closed (TOC) by Whitfield and Bate (2007).

For this study, a broader definition of estuary is required, as the case study (Great Brak estuary) is a TOC and this type of estuary is common in South Africa⁸. Day (1980) proposed a modification of Pritchard’s definition to include the case where the mouth is closed for any period:

“An estuary is a partially enclosed coastal body of water which is periodically open to the sea, and where there is a measurable variation of salinity due to the mixture of sea water with fresh water derived from land drainage.”

Day distinguishes between river mouths, where fresh water flow is sufficiently strong to prevent seawater entering the river (giving examples of Knysna and Langebaan).

⁷ Day (1982) indicates that the term should not be “rigidly defined”. For the purposes of this document “lagoons” refers to the body of water, stretching from the sea inland, which is substantially wider than the river.

⁸ The Coastal Engineering Manual (USACE, April 2008) uses the term “tidal inlet”, without emphasizing the interchange of sea and fresh water. The term cover a larger range of coastal features than river mouths and is not used further in this document.

For the purposes of this study, the definition proposed is that of the National Biodiversity Assessment 2011 (Volume 3: Estuary component. Technical report, (Van Niekerk & Turpie, 2011):

“An estuary is considered a partially enclosed, permanent water body, either continuously or periodically open to the sea on decadal time scales, extending as far as the upper limit of tidal action or salinity penetration. During floods, an estuary can become a river mouth, with no seawater entering the formerly estuarine area. Alternatively, when there is little or no fluvial input, an estuary can be isolated from the sea by a sandbar and become a lagoon or lake. These may become fresh or hypersaline.”

465 South African estuaries were identified in Allanson and Baird (1999) from Heydorn and Tinley, and Begg (1978), CERM (accessed December 2013) lists 258 estuaries and van Niekerk & Turpie (2011) identify 300 functional estuaries in South Africa.

3.2 Setback lines for South African estuaries

There are multiple definitions of setback lines (USACE, April 2008; Roets & Duffel-Canham, 2009). They all have in common the delineation of an area of risk with associated developmental constraints. For the purposes of this study the definition of a setback line is adapted from the ICMA, as follows:

“A line ... demarcat(ing) an area within which development (is) prohibited or controlled”. (Act No. 24, 2008)

The ICMA provides for setback lines to protect public and private property, ensure public safety, provide a coastal protection zone, for aesthetics and for other purposes supporting the objectives of the Act. The Act makes provision for more than one type of setback line, with different levels of control applicable for each. The ICMA specifies that the effects of climate change must be considered in the development of estuarine management plans, on which the setback lines must be delineated. The Western Cape (WC) setback line methodology (WSP Africa, 2010) covers six areas of risk which must be addressed when determining setback lines: erosion, windblown sand, flooding, biodiversity, heritage, and “other” (Breetzke, et al., 2012). The WC methodology results in two setback lines, delineating three zones, with decreasing level of risk. The WC methodology is adapted for Overberg (DEADP, 2011), resulting in three setback lines with four risk zones, as shown in Figure 2 below.

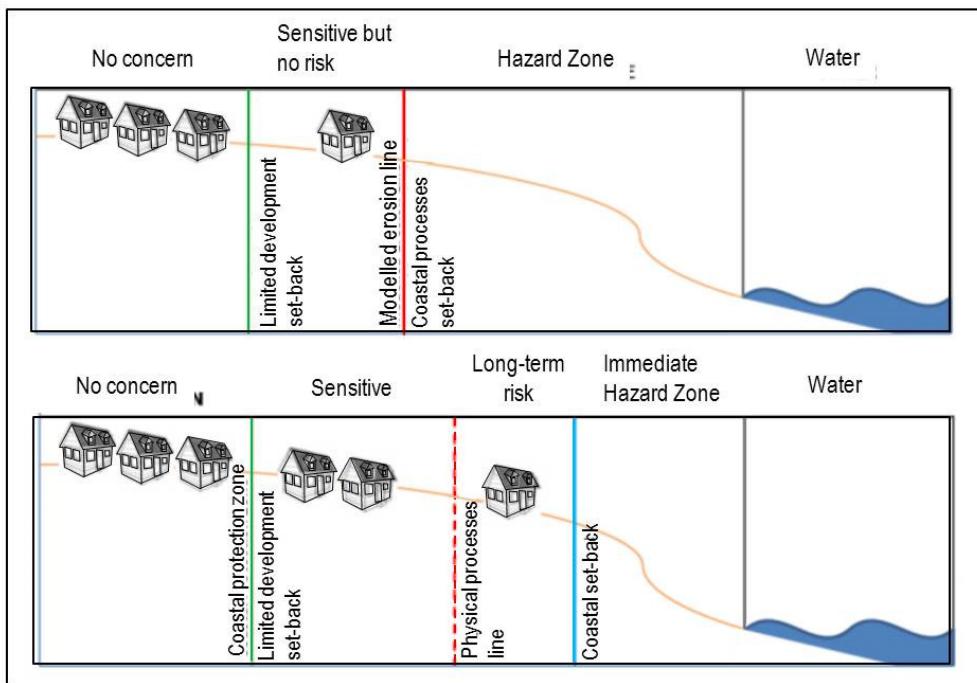


Figure 2: Risk zones and associated setback lines identified by (top) Western Cape setback line methodology and (bottom) Overberg Municipality (DEADP, 2011)

Roets & Duffell-Canham (2009) specified that the area below the 100-year flood line and the 5 m contour should be below the coastal setback line. Their recommended setback lines do not cover wave run-up. They further recommended that primary and secondary dunes should fall within the area delineated by the setback line.

Sea level rise will impact estuaries differently to open coast line because estuaries are sheltered; wind waves in the estuary are smaller due to the small fetch across the estuary, and due to sedimentation processes. Coastal research can therefore not necessarily be applied to estuaries (Stevens, April 2010). Further, estuaries are subject to riverine as well as tidal flooding, introducing the potential for severe flooding (Bishop, et al., 2010).

Policy documents, legislation, regulations and guidelines exist to provide guidance on the determination of the location of setback lines in South Africa.

3.2.1 National Environment Management Act

In 2010 National Environment Management Act, 1988 (Act no. 107 of 1998) was promulgated. Listing Notice 3: “Activities and Competent Authorities Identified in terms of sections 24(2) and 24D”, defines an estuary as “the estuarine functional zone as defined in

the National Estuaries Layer, available from the South African National Biodiversity Institute's BGIS website (<http://bgis.sanbi.org>)". In the BGIS National Estuaries Layer, the estuarine functional zone is delineated to the 5 m above MSL contour. The rationale behind the setting of the 5 m contour includes considerations of flooding as well as potential effects of climate change on estuarine retreat and ecological requirements. The motivation for setting the 5 m contour as the limit of the estuarine function zone is detailed in the section on estuaries in the Technical Report of the 2011 National Biodiversity Assessment (Van Niekerk & Turpie, 2011). The assessment addresses predominantly ecological issues and climate change is mentioned only in the context of retreat of estuary mouths.

3.2.2 DWA: Methods for the Determination of the Ecological Water Requirements for Estuaries

In 2010 the Department of Water Affairs and Forestry⁹, in its document "Methods for the Determination of the Ecological Water Requirements for Estuaries" (DWA, 2010) supported the 5 m contour in general, but indicated that it would not always be applicable. The Department expressed concern that, should the 5 m contour be adopted as the development setback line, this would not be far enough from the estuary for ecological requirements. DWA defined downstream, upstream and lateral boundaries within estuaries in greater detail than NEMA. These boundaries are shown in Text Box 1. DWA also indicated appropriate instruments for determining these boundaries, including salinity, aerial photography and influence of tidal variations. For situations in which the estuarine boundaries identified by DWA have not been determined scientifically, DWA advise that the 5 m contour above MSL should be used as the boundary of the estuary. *For estuaries where the mouth closes from time to time DWA specifies that the 100-year flood line should be determined under closed-mouth conditions.*

⁹ Now the Department of Water and Sanitation

Boundaries of estuary as defined by DWA (2010)

Downstream boundary: The estuary mouth, including the surfzone, seaward extent of the flood delta and/or transitional waters.

Upstream boundary: The extent of tidal influence or the extent of back-flooding during the closed mouth state whichever is furthest upstream.

Lateral boundaries: The lateral boundaries should include all areas below the high tide mark, all estuarine vegetation and any floodplain areas below the upstream boundary as determined by the 1:100 flood line and (for relevant sites) littoral active zones.

Text Box 1: Boundaries of estuaries as defined by DWA (2010)

3.2.3 Western Cape Provincial Government

Western Cape Province's Setback Line Methodology (WSP Africa, 2010) defines two setback lines: the physical processes, or hazard line, and a management line. The former delineates areas that are assessed to have "unacceptable level of erosion, flooding or wave action", and the latter line "accommodates requirements of bio-diversity, heritage, etc."

The Western Cape methodology for defining and adopting coastal development setback lines recommends that, in the absence of a coastal setback line, the 10 m above MSL contour or the line 100 m inland from the high water mark should be designated the development/coastal process line (see Figure 2) (WSP Africa, 2010). The methodology requires that a sea level rise of 1 m should be used when accurately determining setback lines (Umvoti Africa, 2011).

3.2.4 City of Cape Town

The 5 m contour is also used by the City of Cape Town as one of the criteria to define the extent of an estuary. Other criteria used by City of Cape Town include the 50 year flood line and the limit of estuarine vegetation. However, the City expressed concern that, where the development setback line is located at the 5 m meter contour extensive areas will fall below this line as the topography in some areas is very flat. Further, not all areas below the 5 m contour are, (for the City), estuarine in character, neither do they "provide a realistic representation of the area prone to flood risk". (City Of Cape Town, n.d.).

Determination of accurate coastal setback lines is recommended by Theron & Rossouw (2008). In determining setback lines they further recommend that sea level rise be taken into account, as well as the increased intensity and duration of sea storms.

3.3 Great Brak estuary: an overview

The Great Brak River is located on the South Coast of the Western Cape Province of South Africa, lying between Mossel Bay in the west and George in the east (EWISA, 2014). The location of the catchment is shown in Figure 3. The river drains a catchment¹⁰ of 188 km², which is functionally divided into the area above Wolwedans dam, with an area of 131 km² (Department of Water Affairs, South Africa, n.d) and the area below the dam. Above Wolwedans dam, the river is called the Groot River. Rivers in the Catchment are shown in Figure 4.

The Groot River starts at an elevation of 950 m and has a very steep channel for the first 4 km. Thereafter the channel slope is moderate, dropping 400 m over 14.5 km to Wolwedans dam. Directly below Wolwedans dam wall the channel falls 30 m over 5 km, after which the slope becomes very flat, falling only 5 m over 3.5 km. Mean Annual Runoff (MAR) is quite variable, ranging from $4,3 \times 10^6 \text{ m}^3$ (1979/80) to $44,5 \times 10^6 \text{ m}^3$ (1962/63) (EWISA, 2014). Wolwedans dam is located on the river roughly 8 km upstream of the mouth¹¹. Wolwedans dam provides water to Mossel Bay, George and the town of Great Brak. A smaller dam, the Ernest Robertson dam, is located on the Groot River, 14.5 km upstream of Wolwedans dam. Figure 5 shows the two dams. The estuary is shown in Figure 6 with the 5 m setback line indicated. Figure 7 shows the extent of the estuary.

An island (known only as “the Island”), with area approximately 300 x 400 m, is located approximately 180 m upstream of the mouth. The height of most of the island is between 2 to 3 m above MSL, with a high point of 6 m above MSL on the south-western side. Housing has been developed on the Island.

¹⁰ As measured on a GIS system using catchment boundaries provided from state agencies through PLANETGIS and National Spatial Development Plan datasets

¹¹ Following the river centreline and including meanders, following the low flow path east of the Island.

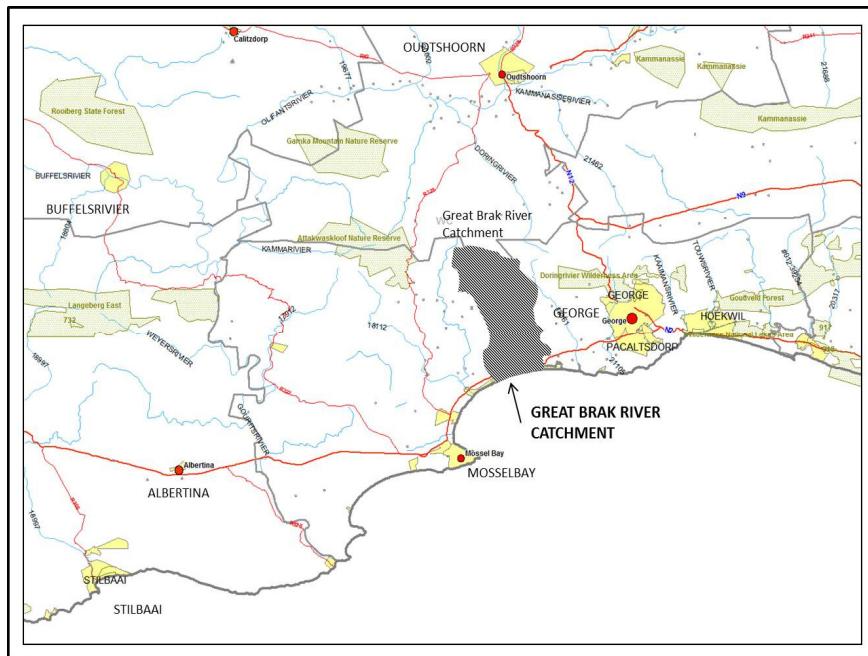


Figure 3: Great Brak catchment locality (modified from Agricultural Research Commission, n.d)

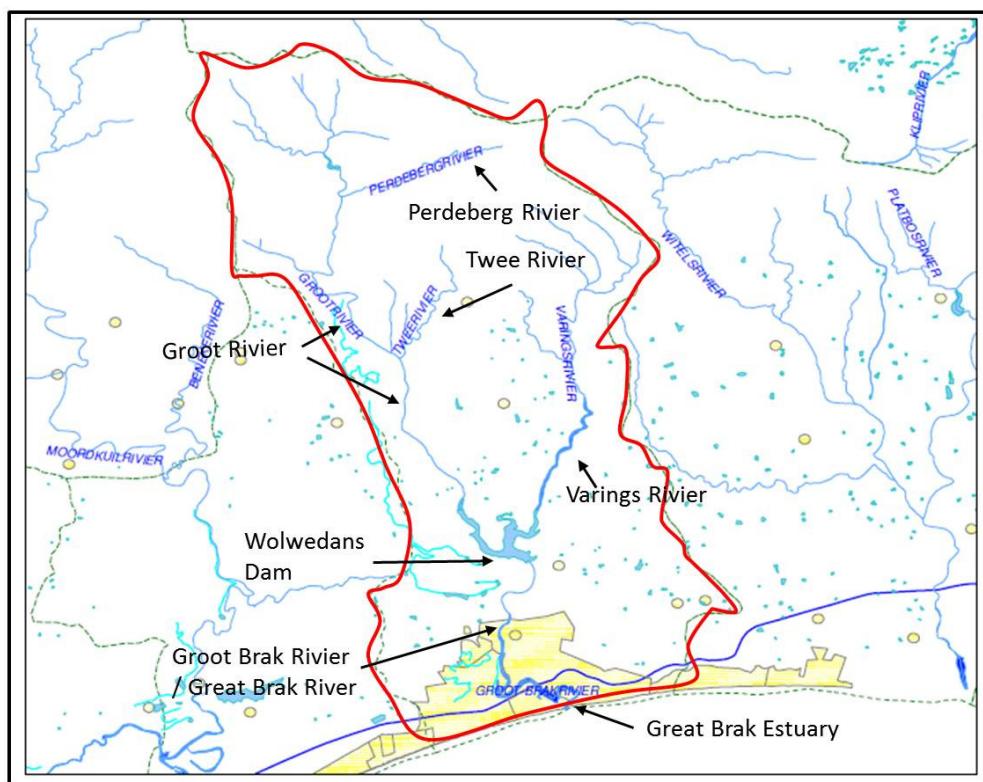


Figure 4: Rivers in the Great Brak Catchment (Agricultural Research Commission, n.d)

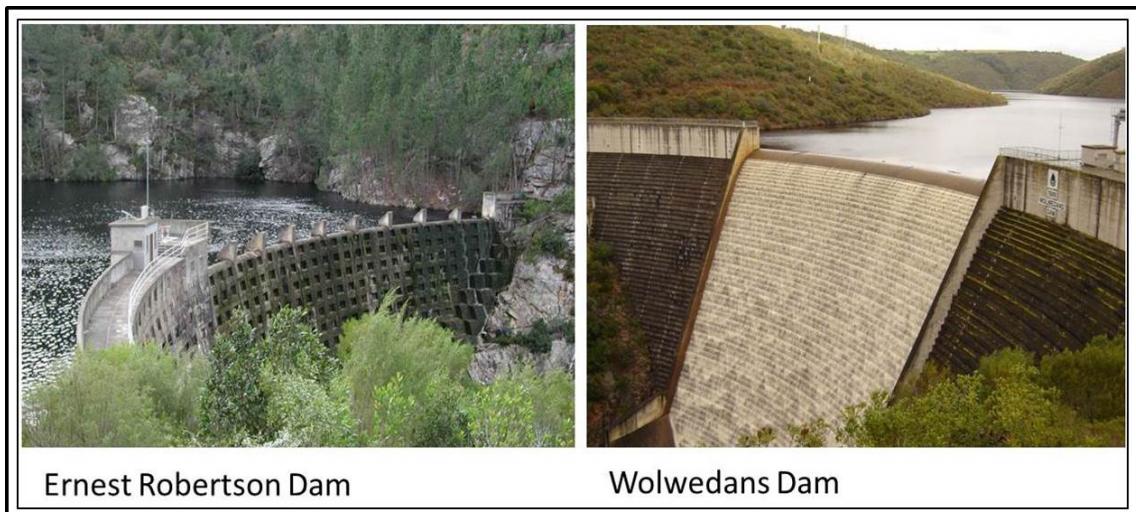


Figure 5: Dams on the Great Brak River (DWA Hydrological Services Surface Water (Data, Dams, Floods and Flows, n.d.)

The sand barrier at the estuary mouth is artificially breached, as shown in Figure 8, to manage flood levels and minimize inundation of the Island. The sand barrier has reached up to 2.7 m in height (bathymetry, July 1989). Water levels in the estuary have been managed by artificial breaching of the mouth since 1978. Artificial breaching is initiated when the sand barrier is between 1.5 and 2.0 m high, or when a flood is forecast. Despite this, at high flood levels there is still flooding of the Island, as shown in photographs by Piet Huizinga (see Figure 9). The eastern channel is generally the dominant channel at low flow, even when the mouth is open because the western channel is blocked by sediment. This sediment is washed in from the sea at high tides and when the sea level is raised during storms, thus forming a sand barrier. Sediment is also deposited by the river as its flow stagnates against the sand barrier. The sediment blocking the western channel extends over 300 m from the mouth into the estuary, as shown in Figure 10.

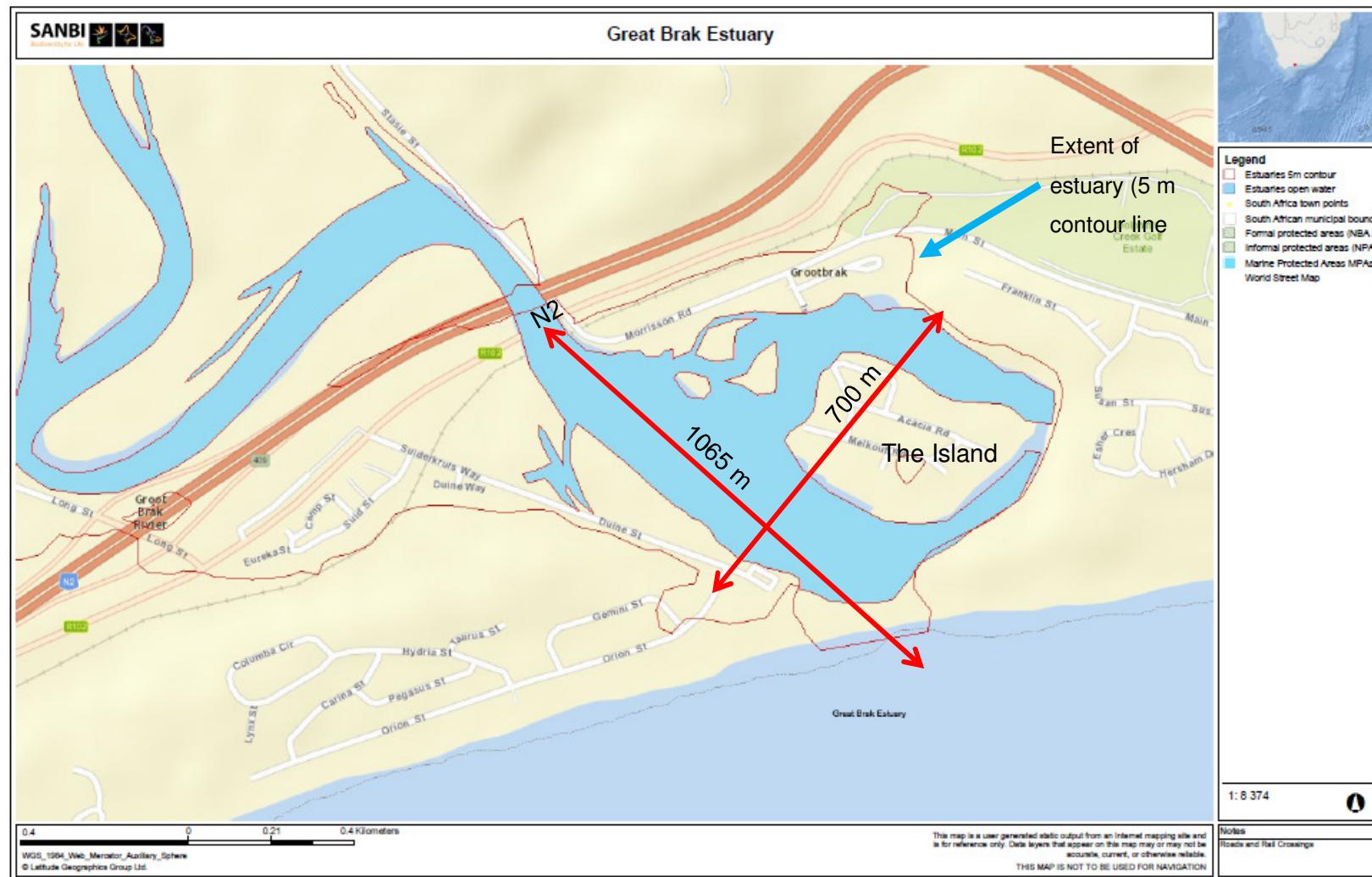


Figure 6: Great Brak estuary showing open water area, the 5 m contour and the adjacent urban road layout (SANBI, 2007)

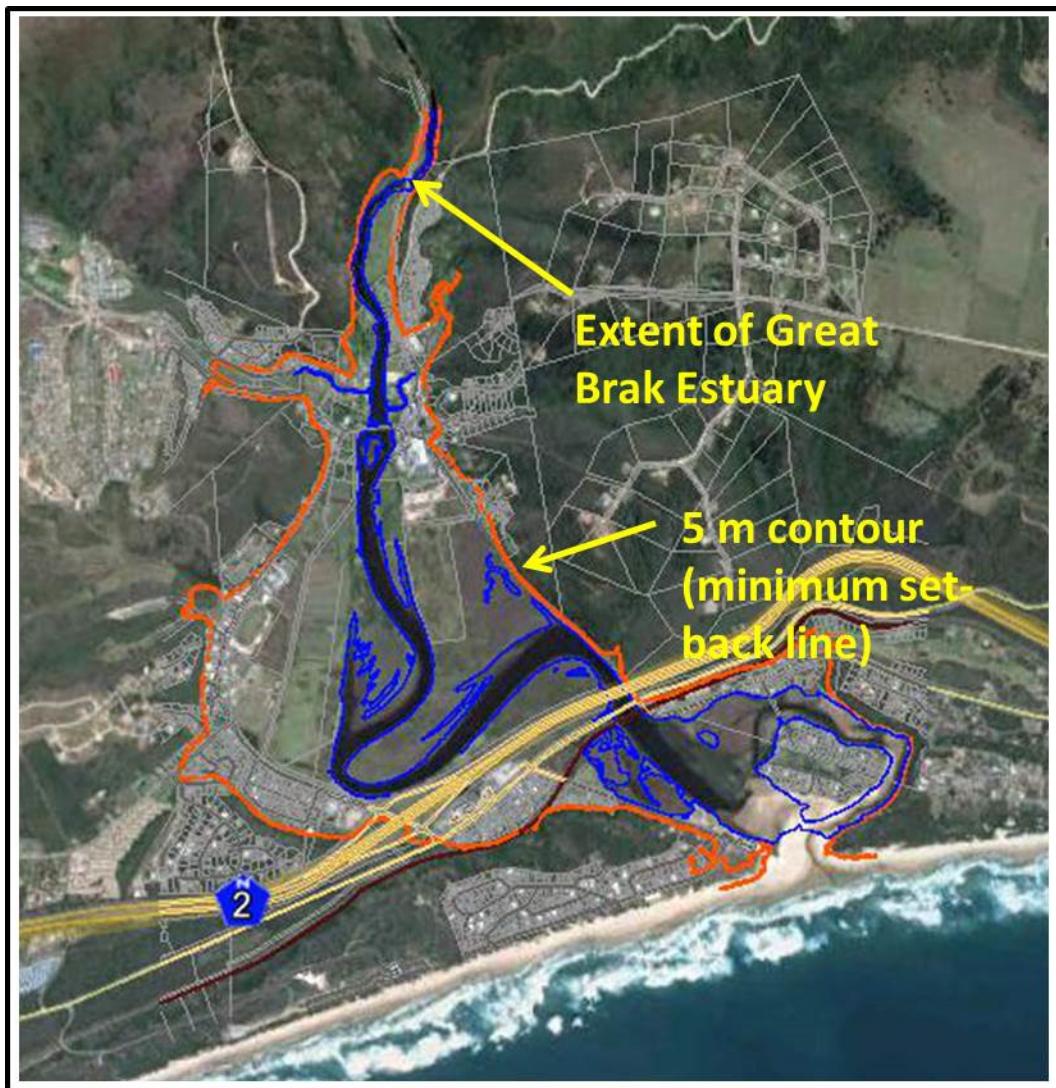


Figure 7: Extent of the Great Brak Estuary (Adapted from Google Earth)



Figure 8: Breaching of the barrier and tidal inflow after breaching (www.dwa.gov.za)



Figure 9: Flooding of the Island in 2007 flood (EWISA, 2014)



Figure 10: Sedimentation blocking the west channel (www.dwaf.gov.za, accessed October 2014)

Structures on the Great Brak River

Apart from the two dams, the structures affecting river flow include the Searle's bridge, the N2 highway bridge, a secondary road bridge, a railway bridge and a bridge providing access to the Island.

The bed levels in the estuary depend on the extent to which sediment has accumulated or been flushed away, and the level of the sand barrier at the mouth. Ten surveys of the bathymetry of the estuary were undertaken in the period 1989 to 1999. These surveys showed the general bed level in the lagoon, below the N2 Bridge, to be about 0.7 to 0.9 m above MSL, while the bed levels in the

channel in the eastern channel of the lagoon were between -0.1 to -0.4 m MSL. The higher bed levels are associated with higher level of the barrier at the mouth.

In May 2010, Gorra Water calculated the flood line for the Great Brak estuary and river, for the 100-year flood. The flow used in the calculation was 744 m³/s, and the effect of climate change was not taken into account. The resultant flood lines were approximately 1.5 m at the estuary mouth rising to 2.5 m at the Island, and 3.5 m at the bridge embankments. In April 2014, Johan Pieterse determined a 100-year flood line for the estuary, taking into account an extreme sea level under climate change, of 3.44 above MSL, and obtained a flood level of 3.6 m at the estuary mouth and 4 m just upstream of the N2 Bridge (Pieterse, 2014).

4 This Study

4.1 Problem statement

The 5 m contour has been set as the limit of the estuarine functional zone (SANBI, 2007; DWEA, 18 June 2010), which includes the floodplain area. In respect of the floodplain the provision of 5 m intends to accommodate flooding when the mouth is closed, for sand barrier heights up to 4.5 m (SANBI, 2007). The blanket application of the 5 m contour as a setback line has raised concerns, as some areas are very flat and large areas will then be excluded from development (City Of Cape Town, n.d.), with associated economic impacts. On the other hand, for estuaries with steep banks, the 5 m may be insufficient (Van Niekerk & Turpie, 2011). Furthermore, sea level rise projections range up to 0.82 m. As an example, if the current sand barrier height is 4.5 m, the increase in height of the sand barrier resulting from higher sea levels may exceed 5 m above MSL, with resultant flood levels in the estuary exceeding the 5 m contour.

In the case of the Great Brak estuary, the mouth is closed a great deal of the time. The sand barrier is breached artificially at between 1.5 m and 2.0 m above MSL, which should assist in limiting flood levels. However, extreme sea levels under climate change may exceed 2.5 m above MSL, negating the effect of artificial breaching, as the sea level will be higher than the barrier. As flood peak values may also increase under the influence of climate change, future flood levels may be significantly higher than currently experienced.

4.2 Purpose

Local features of the Great Brak Estuary will influence the flood levels such that the application of a generic 5 m setback line may not be sufficient to meet the requirements of the ICMA. The purpose

of this study is to assess the adequacy of the 5 m setback for the Great Brak estuary lagoon, below the N2 Bridge, given predicted increases in changes in sea levels and runoff.

4.3 Scope of this study

Mike11 software will be used to generate flood levels for current and future situations, which will be the basis of a conclusion on the adequacy of the 5 m contours under the impact of climate change. The study will assess the effects only in respect of the hydraulic requirements. The hydraulics at the mouth of the estuary has been simplified by treating the sea level as a hydraulic control. Flooding is determined for the scenarios listed in Table 1.

Table 1: Sea level scenarios modelled

Current sea level scenarios	Future sea levels scenarios
Mean sea level	Mean sea level plus eustatic sea level rise
MHWS plus surge	MHWS plus sea level rise plus future storm surge

Flooding was also determined for closed mouth conditions where the sand barrier is at 2 m and 4 m above MSL. The barrier is assumed to function as a dam for purposes of modelling. No breaching of the barrier was modelled.

5 Estuaries in South Africa

The National Biodiversity Assessment 2011 indicates that there are 300 functional estuaries in South Africa (Van Niekerk & Turpie, 2011). Whitfield and Bates (2004, pp iv) state:

“Each estuary is unique because of the various factors that influence its structure and sensitivity to flow, and two similar sized estuaries adjacent to one another can be quite different.”

Notwithstanding the uniqueness of each estuary, it is useful to categorize estuaries according to common features. Figure 11 shows the potential features of an estuary. All features may not be present in every estuary. In addition, estuaries themselves are dynamic and vary in both time and space dependent on decadal scale runoff patterns and runoff/coastal interactions. The Coastal Engineering Manual (USACE, April 2008) highlights the dynamic nature of the coast and that estuaries can vary in both time and space.

There are several approaches to categorizing estuaries in South Africa, based on climatic, morphological features and other parameters. Details of two approaches are provided in this study. The first approach, based on morphological / climatological parameters, provides a useful introduction into estuaries in South Africa. The second approach categorizes estuaries from the perspective of the behaviour of the sand barrier at the mouth, which is relevant to the hydraulic behaviour of estuaries. There is considerable overlap between the two categorization methods. The morphological categorization approach presented in Table 3 was developed from Allanson and Baird (1999), building on earlier work by De Villiers and Hodgson (1999), who specified three climatological zones: two warm climatic zones, influenced by the warm Agulhas current (sub-tropic and warm temperate), and third zone, the cool temperate zone. Allanson and Baird further subdivided the subtropical zone to allow for four morphological zones for South African estuaries. This categorization then introduces morphology as a second parameter. It takes into account climate, bedrock profiles, geology, lithology and wave energy. The four zones and their features are given in Table 3.

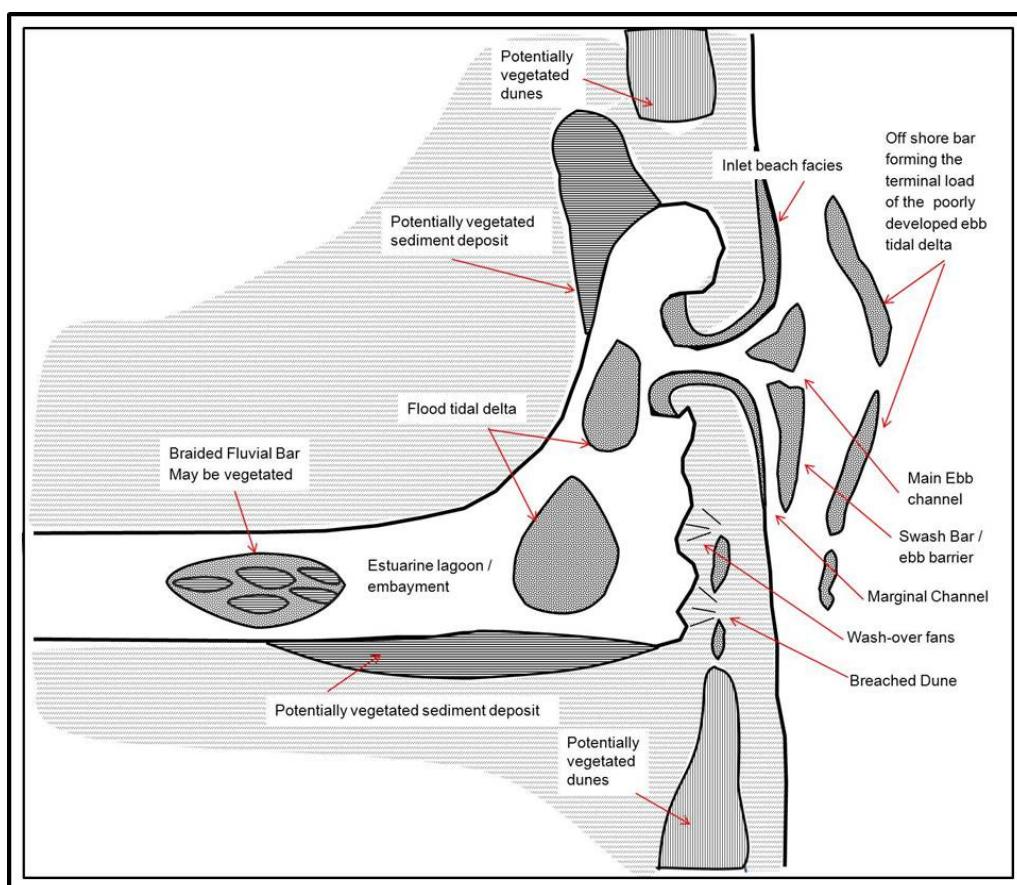


Figure 11: Idealized schematic of features of a typical estuary showing the major morphological features. (Modified from Allanson B.R and Baird, (1999)).

The second approach presented in this document categorizes estuaries in South Africa according to mouth conditions, as detailed in Table 4. Permanently open estuaries have mouths that are

open throughout the year, while temporarily open/closed estuaries have mouths that are open for periods ranging from weeks to months, but may be closed for periods of more than a year (Van Niekerk, 2007). Temporarily open/closed estuaries behave (hydraulically) like closed estuaries when the mouth is closed, and like open estuaries when the mouth is breached (van Niekerk, 2007). Permanently open estuaries include the Whitfield (1992) categories of POEs, river mouths and estuarine bays. Temporarily open/closed estuaries include the Whitfield (1992) categories of TOCEs and estuarine lakes.

A map showing the estuaries, named in Table 3 and Table 4, is given as Figure 12.

Van Niekerk (2007) provides information on outflow channel geometry associated with open, closed or semi-closed mouth conditions, as summarized in Table 2. Tidal effects are considered in this categorization.

Table 2: Features associated with mouth states (van Niekerk, 2007)

Mouth state	River inflow	Outlet channel depth	Tidal inflow	Tidal variation in estuary
Open	Flooding	> 2 m deep and very wide	Ebb and flood	Yes
Closed	Inflow < evaporation	No channel	Only due to over wash at high tides and during storms	No
Semi-closed	Low inflow	< 0.3 m deep and < 30 m wide	Only due to over wash at high tides and during storms	No

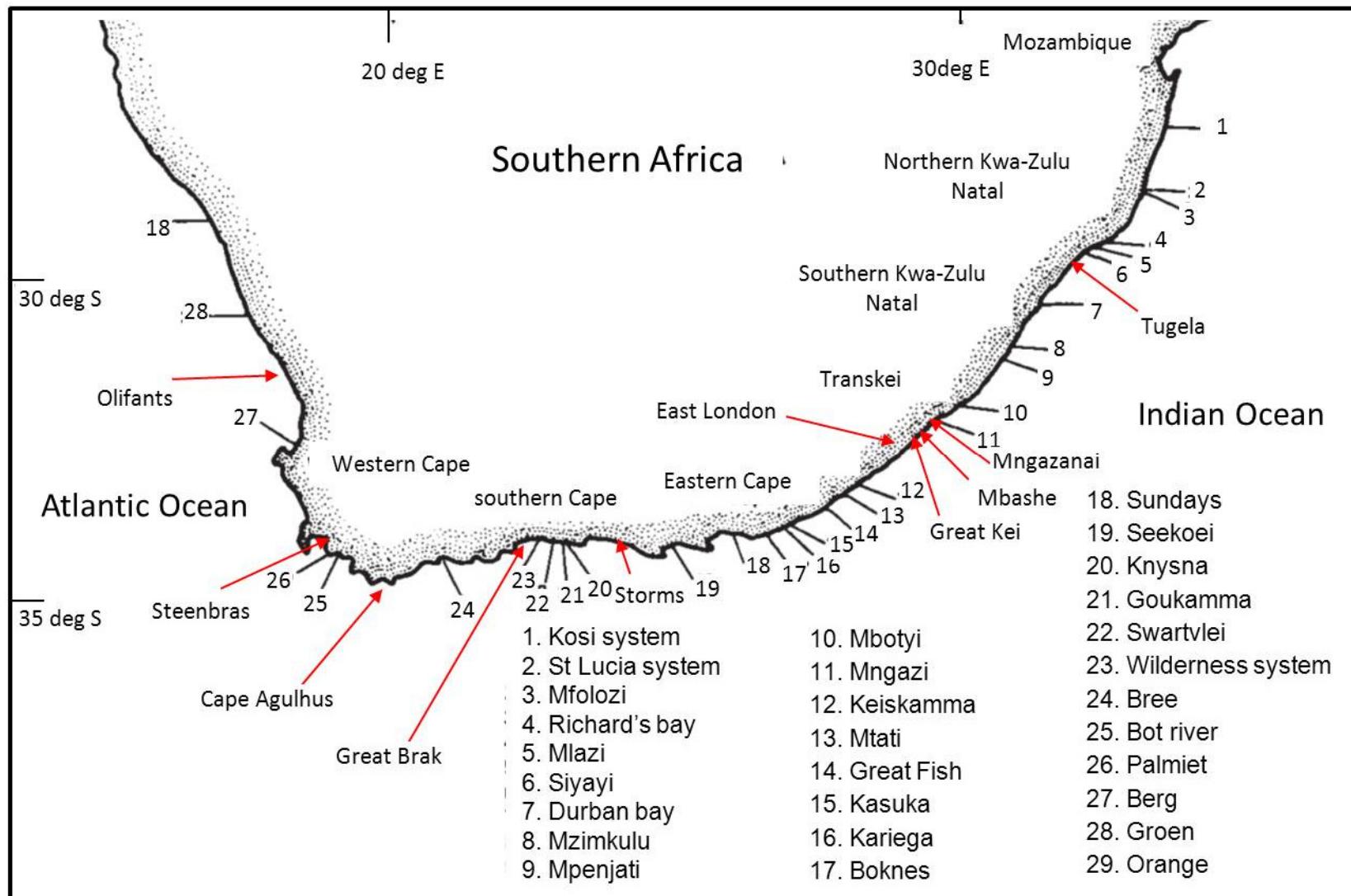


Figure 12: Locality of South African estuaries discussed in the report (modified from Whitfield (1992))

Table 3: Features of four morphological zones (compiled from Allanson and Baird, 1999; supplemented as indicated in text)

Morphological zone	Western Cape North coast Orange River to Cape Point	South coast of the Western Cape and South-East Eastern Cape coast Cape point to the Mbashe River	Southern Kwa-Zulu Natal and Transkei coast Mbashe to Tugela rivers	Northern KZN and Southern Mozambique Tugela to Northern border and into Mozambique
Climate	Arid becoming semi-desert and desert in the North. Seasonal rainfall, mostly winter	Humid temperate zone. Seasonal rainfall Up to Cape Agulhus seasonal winter rainfall. East from East London rainfall happens throughout the year.	Subtropical climate. Seasonal rainfall (summer)	Subtropical climate. Seasonal rainfall (summer)
Wave climate and shore conditions	Tidal range relatively constant across zone (1.4 to 1.6 m) Wave energy low and approach relatively constant	Tidal range less than 2 m Wave energy low and approach relatively constant Outlets may be above high tide level, resulting in outflow to the sea but no tidal inflow. No tidal deltas.	Tidal range less than 2 m (Perillo, 1996) Wave energy high and approach relatively constant. Steep nearshore gradient result is an absence of ebb-tidal deltas or offshore bars. These deltas occur after floods but are quickly eroded	Tidal range less than 2 m (Perillo, 1996) Wave energy high and approach relatively constant. Offshore barriers not common.

Morphological zone	Western Cape North coast Orange River to Cape Point	South coast of the Western Cape and South-East Eastern Cape coast Cape point to the Mbashe River	Southern Kwa-Zulu Natal and Transkei coast Mbashe to Tugela rivers	Northern KZN and Southern Mozambique Tugela to Northern border and into Mozambique
Parameter				
Estuarine Geo-morphology	Estuaries typically confined in bedrock valleys	Estuaries typically confined in bedrock valleys	Estuaries typically confined in bedrock valleys	Alluvial plains, not confined to bedrock channels River can move laterally and mouth positions can change substantially (unless confined by rocky outcrop.)
Sediments	Not given by Allanson and Baird (1999)	Fluvial deltas in the upper reaches, reducing estuary volume.	Deep weathering profiles in soils gives high sediment yield. High sediment loads in rivers Sand barriers at the mouths are due to river sediment, rather than to marine sediments.	Low river gradients. Wide floodplain.
Flow conditions	Orange, Olifant's and Berg rivers flow throughout the year due to large catchments that extend beyond the arid zone. River flow varies seasonally. Remaining estuaries have ephemeral river flow with net evaporation	Smaller rivers in the east maintain flow throughout the year due to rain throughout the year. Seepage through the barrier may allow exchange of sea and lagoon water. Channels in the fluvial deltas reflect river discharge	Steep hinterlands results in rapid river discharge,	Rivers typically flow throughout the year

Morphological zone	Western Cape North coast Orange River to Cape Point	South coast of the Western Cape and South-East Eastern Cape coast Cape point to the Mbashe River	Southern Kwa-Zulu Natal and Transkei coast Mbashe to Tugela rivers	Northern KZN and Southern Mozambique Tugela to Northern border and into Mozambique
Parameter		Storms and Steenbras have no barriers. Great fish and Great Kei are river dominated, maintaining open mouth due to river flow. The remaining estuaries are wave dominated: Tidal estuaries (wave dominated): have sandy barrier (with dunes in some cases), constricted inlet, landward flood-tidal delta, limited ebb-tidal deltas, channel deeper on landward side of barrier, Fluvial deltas at upper end of estuary. Non-tidal lagoons: Barrier closed for part of the year. Breaching occurs when there is strong river flow. Over wash (more common on dissipative beaches of Eastern Cape) may introduce water into the lagoon. Occasional overtopping of the barrier or shallow channel flow	River dominated coastal estuaries, non-tidal river mouths and drowned river mouths: Low intertidal volume. River dominated coastal lagoons: As barriers are high, high flow velocities develop when the barrier overtops, resulting in breaching. Breaching occurs during (a) seasonal inflow over long period raising the water level or (b) flood. Over wash can also cause a barrier breach. Rapid re-establishment of barrier after breaching.	Barriers are migratory.

Table 4: Characteristics of types of estuary in South Africa (compiled from Baird, n.d; and CERM, accessed 11/12/2013; and Whitfield, (1992) pg 94;)

	Permanently open estuaries (POEs)	Temporarily open/closed estuaries (TOCEs)	River mouths	Estuarine lakes	Estuarine bays
	27% of 168 estuaries listed by Bates (2007)	72% of 168 estuaries listed by Bates (2007)			
Climate	Higher rainfall regions Moderate climatic zone	Strong seasonal rainfall patterns or areas with very low rainfall (Baird, n.d).	Higher rainfall regions with flow most of the year	Rainfall sufficient such that flow into estuary exceeds loss of water by evaporation	Not a factor
Tidal prism	1-10 ⁶ m ³ per spring tidal cycle Tidal flow into the estuary.	No tidal inflow when mouth is closed. Small tidal prism when mouth is open.	Small with little intrusion <10 ⁶ m ³ per spring tidal cycle	Negligible. Little intrusion. <0.1-10 ⁶ m ³ per spring tidal cycle Surface flow or seepage through the barrier allowing tidal influence on water levels in the lagoon (Badia Cebada 2003)	Large tidal prism. Tidal flow dominating >10 ⁶ m ³ per spring tidal cycle. Regular replacement of riverine flow, at time up to middle reaches of the estuary Tidal range similar to that of open sea in estuary.
Catchments/ River flow	>500 km ² , some >10,000 km ² Riverine flow into the sea.	< 500 km ²	Generally >10,000 km ² River flow perennial	River flow intermittent Riverine flow into the sea only during high river flow.	Riverine flow minor relative to tidal effects River flow not a criteria

	Permanently open estuaries (POEs)	Temporarily open/closed estuaries (TOCEs)	River mouths	Estuarine lakes	Estuarine bays
	River flow perennial	The river flow is typically low, averaging less than 1 m3/s (Baird, n.d.). River flow intermittent	Riverine flow into the sea dominates. Sea inflow does not penetrate far upstream during moderate to high river flows. During floods river flows extends into the sea.		
Barrier	Has a barrier at the mouth with a (usually narrow) channel to the sea. Channel position may vary Mouths permanently open	Has a barrier that closes mouth for several months of the year. Can have surface flow or seepage through the barrier	Mouths permanently open. Generally <2 m depth. During large floods mouth can scour up 15 m depth (Whitfield 1992)	Separated from sea by dunes. Where lakes are completely separated from the sea, they are termed coastal lakes. Can be permanently or temporarily open to sea	Mouth depth > 3 m below mean sea level
Features	Wetland with salt marshes (temperate zone) or mangroves (subtropical zone) Grass in inter-tidal and sub-tidal area. Can be linked to estuarine lakes	When mouth is open behaves like POE. Behaves like river mouth after floods. Can be linked to estuarine lakes	Heavy silt loads can be deposited in the estuary. As an exception, Tsitsikamma coast river mouths carry little sediment so there is little deposition on the rocky base.	-	Some are artificially created by dredging. Typically have extensive sand or mangrove marches

	Permanently open estuaries (POEs)	Temporarily open/closed estuaries (TOCEs)	River mouths	Estuarine lakes	Estuarine bays
Examples	Mlazi (KwaZulu-Natal), Mzimkhulu, Mngazana, Keiskamma (Eastern Cape), and Berg (Western Cape) ¹² .	Siyayi, Mpenjati (Kwa-Zulu Natal), Mbotsyi, Mtati, Kasuka (Eastern Cape), Goukamma (southern Cape) and Groen (Western Cape)	Only two systems can be classified as river mouths: the Thukela River and the Orange River (Baird, n.d.).	St Lucia (KwaZulu-Natal) and Swartvlei (Western Cape).	Durban Bay (KwaZulu-Natal), Richard's Bay (KwaZulu-Natal), and the Knysna lagoon (Western Cape).

¹² CERM website indicates that only 37 of the 289 river mouths in South Africa are permanently open to the sea (CERM, accessed 11/12/2103), while Baird (n.d) indicates that there are 465 estuaries, of which 20% (129) are permanently open to the sea.

6 Literature Review

6.1 Effect of climate change on flood levels in estuaries

A review of the effect of climate change on the hydrodynamics of estuaries requires an understanding of the external factors, and the estuarine features, that influence the hydrodynamics. *Factors* that influence the hydrodynamics of an estuary are the interaction between riverine inflow, tides and waves and, for large open bays and lagoons, wind stress (USACE, April 2008; Schumann, et al., 1999). *Features* of an estuary that influence the hydrodynamic response of an estuary include the status of the mouth / swash bars / ebb deltas (open or closed), estuarine bathymetry (Allanson B.R and Baird, 1999) and the structure of the main ebb channel (USACE, April 2008; Schumann, et al., 1999). Figure 13 shows the direct factors that influence the hydrodynamic response of the estuary.

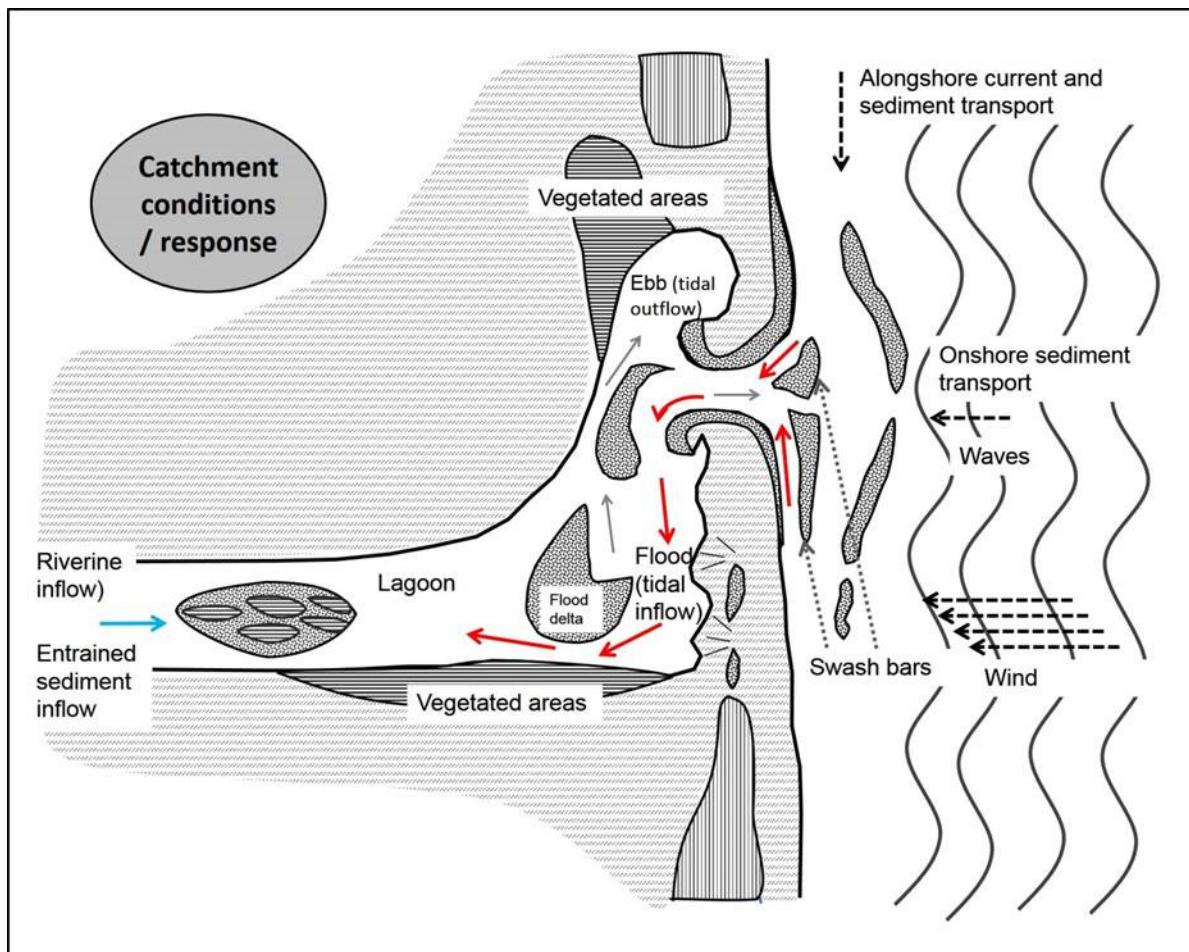


Figure 13: Factors influencing the hydrodynamic behaviour of an estuary. Modified from Allanson B.R and Baird (1999).

The major portion of literature on the impact of climate change on estuaries relates to the ecological functions of estuaries, and associated issues such as salinity and pollution. The literature also explores the economic and livelihood implications of climate change effects on estuaries. Expected changes to the external factors and estuary features influencing the hydrodynamics are covered in literature only in broad strokes. Detailed studies assessing the hydraulic response of estuaries to sea level rise are sparse and often, where they exist, proprietary and not accessible for research. This section therefore deals primarily with the conceptual effects of climate change on estuaries, with a focus on the hydraulic features.

Sea level rise raises the still water level of the sea (Mather & Stretch, 5 March 2012). Wong, et al (2014) identify the impact¹³ of climate change on estuaries¹⁴ and coastal lagoons¹⁵ as: changes in the sediment balance and distribution due to changes in storm frequency, intensity and tidal range; expansion, in some cases, due to inundation of marshes; and shrinkage, in other cases, of lagoons as the mouth retreats and developed areas limit the extent to which the lagoon can migrate inland. The drivers of these changes are identified as storms, sea level rise and runoff (Wong, et al., 2014). A further effect of climate change on estuaries is the change to the estuary mouth.

At the mouth, changes in both open and closed conditions are expected to increase the water level in the estuary relative to the land levelling datum. The predicted effect of increased eustatic levels and storm surge is that the height of the sand barrier will increase as sediment is transported to higher levels under wave action (Goshen, 2013) and that the mouth will retreat. Under open mouth conditions, the higher water level at the mouth will reduce the rate of flow out of the estuary, thus acting as a hydraulic control to flood water (Midgley, et al., October 2007; Umvoti, May 2010). The extent to which the tidal flow will penetrate inland will increase as sea levels rise, resulting in increased flooding under tidal influences (Goshen, 2013; Mather & Stretch, 5 March 2012).

Sedimentation of the lagoon is predicted to change in response to riverine flow changes. Longer dry periods and an increase in short duration high intensity storms are predicted as extreme weather events increase (Goshen, 2013). Both lower inflow, and longer periods of

¹³ Relevant to hydraulic behaviour of estuaries. Additional changes impacts are indicated for ecological systems

¹⁴ Defined in the AR 5 as having mixing of fresh and salt water

¹⁵ Defined in the AR 5 as shallow water bodies separated from the ocean by a barrier and connected at least intermittently to the sea

low inflow, will reduce the flushing capacity of the river, such that the mouth will be breached less frequently. As a result the sand barrier height will increase as shown in Figure 14. The situation will be exacerbated for catchments where sediment yield is increased due to climate change effects¹⁶ (Midgley, et al., October 2007). Scour of the lagoon sediments may also be reduced, if breaching of the sand barrier takes place more frequently in response to the increasing estuarine water levels. The potential denuding of vegetation in arid catchments (i.e., increasing the erodibility of soils) coupled with an increase in the intensity of rain events due to climate change would lead to a significant increase in the deposition of sediment in estuaries" (van Niekerk et al., in prep). The increase in sediment from inland, and retreat of the mouth inland, may completely fill the lagoon, thus raising the bed level (Mather & Stretch, 5 March 2012).

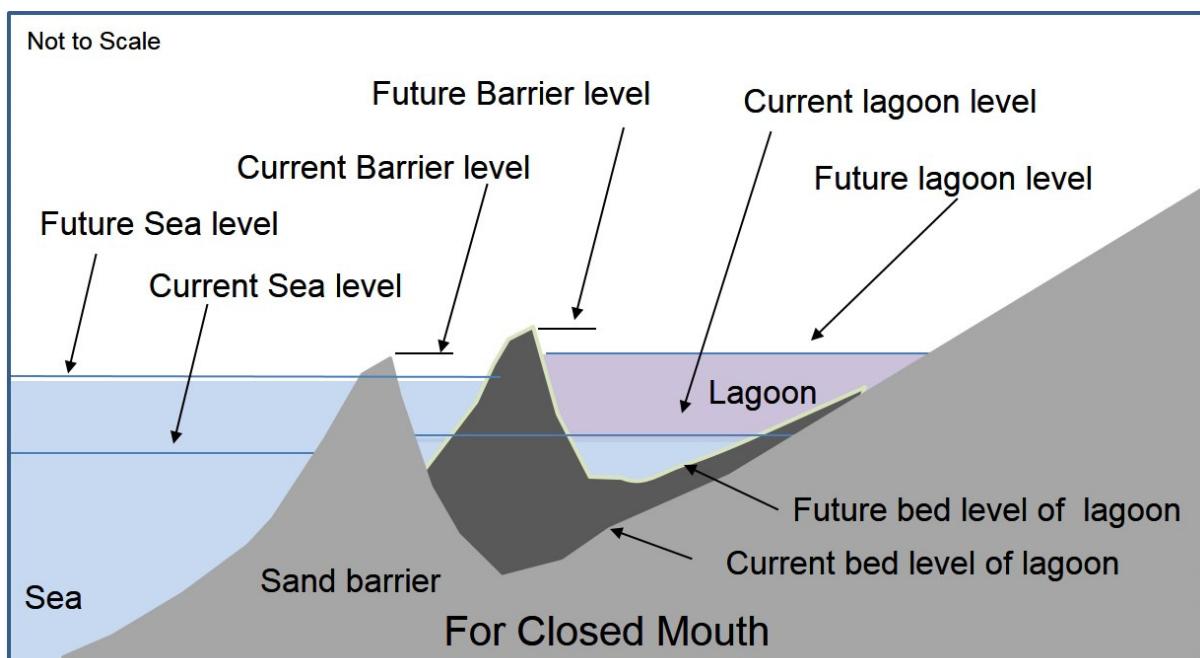


Figure 14: Retreat and increase in height of the barrier under sea level rise, resulting in shrinkage of the lagoon (diagram by author)

¹⁶ For example, where higher temperatures result in a reduction in vegetation

6.2 Effect of climate change on rainfall

While it is predicted that rainfall will generally decline under global warming, an increase is predicted in the rainfall intensity, with a greater number of high rainfall events, resulting in increased flood intensity (Midgley, et al., October 2007).

Generally, lower rainfall is also predicted for South Africa under climate change, with a decrease in mean annual precipitation (MAP) of 20% predicted for 2090-2099, relative to 1980-1999 levels (Intergovernmental Panel on Climate Change, 2014). The level of confidence on the potential change in rainfall is not as high as the level of confidence for sea level changes, due to the variable nature of rainfall both in time and space (IPCC, 2007). The predicted change is therefore based on statistical analysis of rainfall data. The increase in intensity and number of extreme events is reflected in a reduction in the return period for a particular event (i.e.: a current 20 year event may become a 10 year event in the future) and an increase in the short term rainfall amount. Simulations indicate that the frequency of extreme 1-day events could increase from every 20 years to every 10 years (Collins, et al., 2013). The converse is that longer dry periods are also predicted.

Predictions have been made of changes in extreme rainfall events (expressed as 20 mm rainfall in 24 hrs) (Engelbrecht, et al., Published online 4 January 2012), and the change in frequency of the events has been mapped (see Figure 15). According to this map the expected change in the frequency of extreme rainfall events, for the far Western Cape, is a decline of between 1 and 1.5 % per annum. However, simulations undertaken by a Santam-WWF partnership (Research partnership between the Santam Group, the WWF, 2011), predict a 10% increase in the frequency of intense short duration (24 hour) storm rainfall, for the Western Cape. The study predicted that the increase in frequency, for just the winter months, was higher, at 36%. The period of analysis was for 2020 to 2050, measured against the 1960 to 1990 period. The data analysed is shown in Figure 16

Figure 17 shows the prediction of the CMIP 5 model¹⁷ for the change in value for 2081-2100 relative to 1985-2005, of the 1-day precipitation for the 20-year return period. This model also gives an increase in frequency and value of intense storms. For South Africa a decrease of up to 2.5% is predicted for the western areas, and an increase of between 5 and

¹⁷ The Coupled Model Inter-comparison Project (CMIP) provides a standard experimental protocol for studying the output of coupled atmosphere-ocean general circulation mode. <http://cmip-pcmdi.llnl.gov/>

7.5% for the eastern part of the country. Figure 17 also shows the expected change in return period for the event with the amount of rainfall currently applicable to the 20-year event. The figure shows that the return period is relatively unaffected for the western part of the country, increasing from a 20-year return period to between a 20 and 17.5-year return period. For the eastern part of the country the return period increases from 20-years to between 17.5- and 15-year return periods.

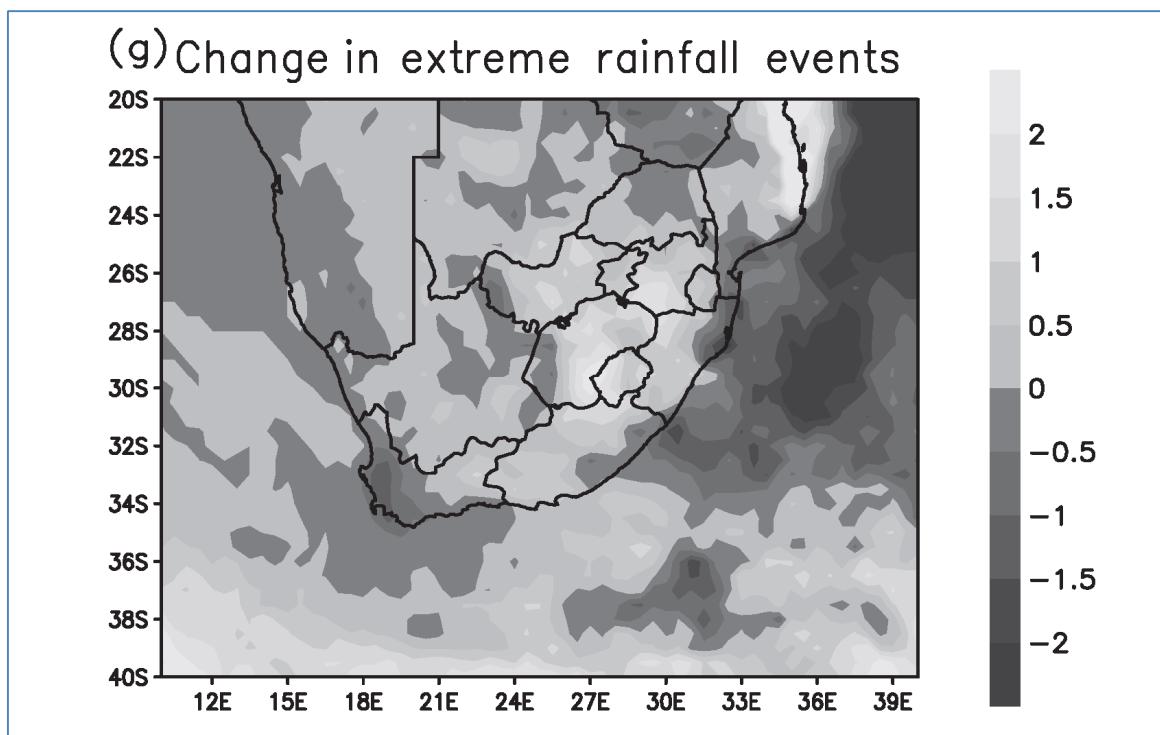


Figure 15: Projected annual change in heavy rainfall event (20 m/24 hr) associated with closed-off lows (Engelbrecht, et al., Published online 4 January 2012)

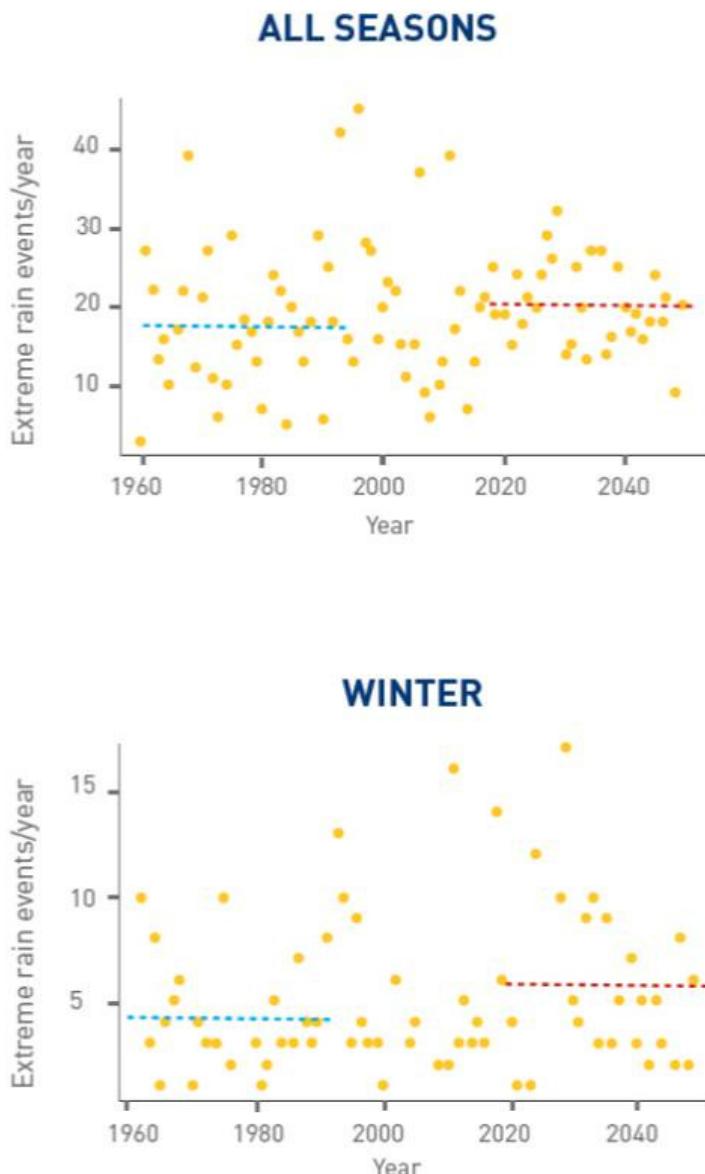


Figure 4: Number of intense rainfall days in the area as calculated using historical and simulated future climate data

Figure 16: Number of intense rainfall days in the area from Cape Town to Cape Infanta (Research partnership between the Santam Group, the WWF, 2011)

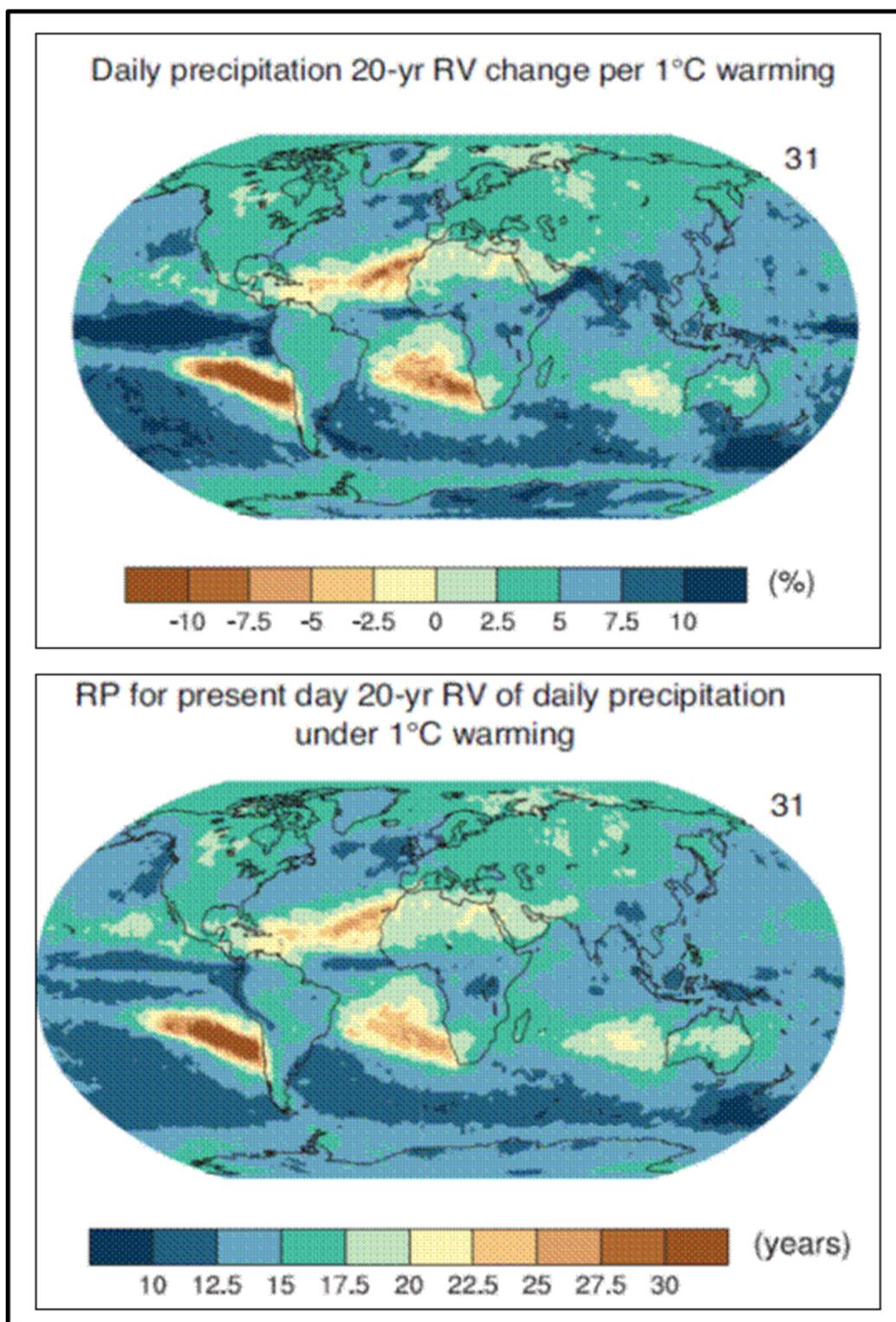


Figure 17: The CMIP 2081-2100 multi-modal ensemble median: percentage change per 1°C of local warming relative to 1986-2005. (Top) 20-year return period values of annual maximum daily precipitation and (Bottom) Return period (years) of the 1986-2005 20-year return period

6.3 Change in Sea levels under climate change

Long term trends from tidal gauges across the world indicate that sea level rise, for the period 1901- 2010 was 0.19 m (0.17 to 0.21 m). The IPPC finding is that it is *very likely*¹⁸ that the global average sea level rise was 1.9 mm (1.7 to 2.1 mm) per annum for this period. The rate was not uniform across the period and the IPPC find it as *very likely* that the rate increased after 1993 to 3.2 mm/annum (Rhein, et al., 2014). The IPCC considers it *very likely* that the rate of sea level rise will continue to increase over the observed 1971-1990 rate of increase (Church, et al., 2013). Sea level rise is not uniform spatially and local variations in the rate of sea level rise can be greater or less than the global average due to local/ regional winds (Rhein, et al., 2014).

As methods of determining the contribution of snow and ice, currently located on land, has improved. Estimations of future sea level rise has therefore also improved. As a result predicted sea level rise is greater in AR 5 than in AR 4. Various scenarios of radiation penetration provide a range of predicted sea level changes, for the period 2081-2100 relative to sea levels for 1985-2005, as shown in Figure 18. These predictions take into account *some* contribution from predicted melting of Greenland and arctic glaciers. IPCC have medium confidence in the contribution to the predicted SLR from thermal expansion, but low confidence in the modelled contribution from melting glaciers. The predictions shown in Figure 18 do not include the SLR which would result from the *complete* melting of the Greenland glaciers, which would potentially take place if global mean surface temperature rise exceeds between 2° and 4° C relative to pre-industrial temperatures (Church, et al., 2013). Should the Greenland ice sheet melt entirely, sea levels are likely to increase by up to 7 m. This scenariois not predicted for this century. Figure 18 shows the result of the application of the predicted annual increase in SLR applied over the period 2006-2100, with the associated uncertainties.

In addition to eustatic sea level rise, sea levels under storm conditions will be affected by climate change. Sea level extremes are and will be further increased, mainly due to the higher mean sea level (Church, et al., 2013; Goshen, 2013). The IPCC find this as *very likely*, and as *likely*¹⁹ that sea levels may also be affected by increased intensity of storms.

¹⁸ i.e.: with a 90% probability of exceedance

¹⁹ i.e.: with a probability of exceedance of 66%.

Uncertainty in IPPC findings

The uncertainties associated with the findings of the Assessment are indicated through the probability and level of confidence by the IPPC. “Confidence” (expressed as *very low, low, medium, high or very high*) is a function of robustness and extent of data, and the extent of agreement on the findings, thus providing an assessment of the validity of a finding. The probability is an outcome of the data analysis (IPCC 4, 2010).

Text box 2: Uncertainty in IPPC findings

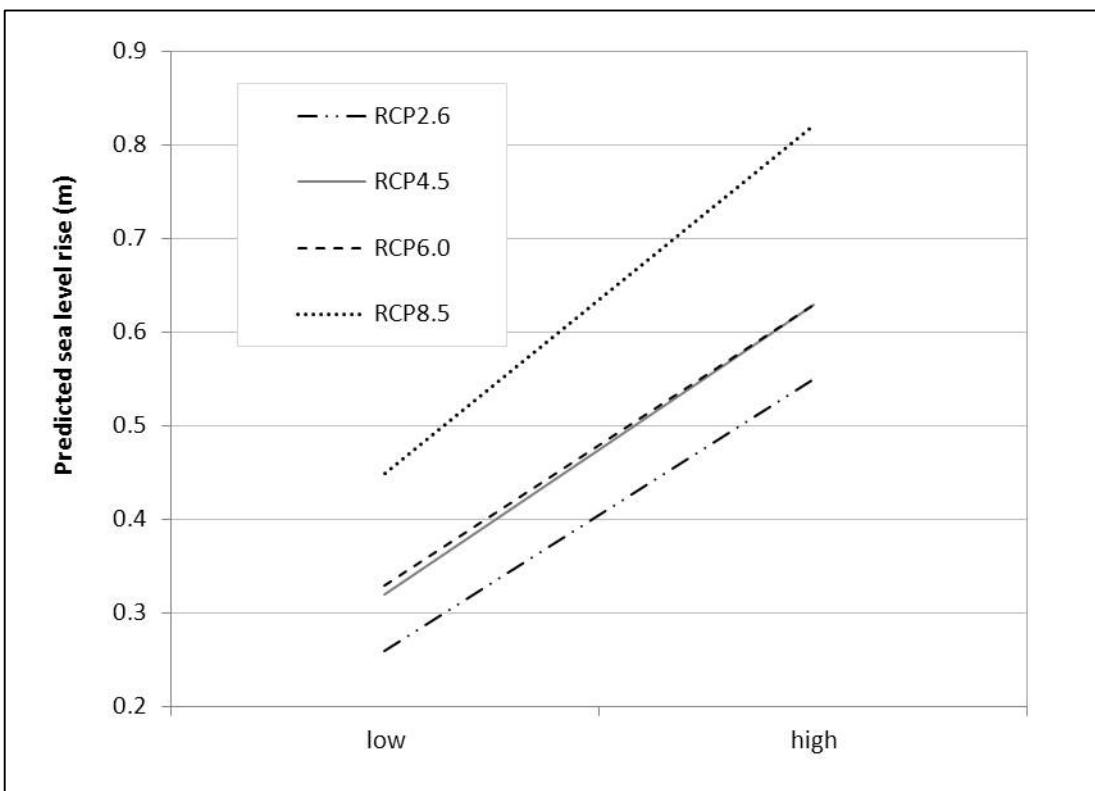


Figure 18: Predictions from various scenarios for sea level rise, for 2081-2100 relative to 1985-2005 (Church, et al., 2013)

Sea level rise in South Africa has been estimated as listed in Table 5. Differences in sea level rise between the three areas identified are attributed to differential rate of crustal movement and barometric changes as well as temperature changes of the Agulhas current (Goshen, 2013). Recent models predict rise in sea level for South Africa is 0.5 m by the year 2100 (Theron & Rossouw, 2008).

Table 5: Rates of sea level rise for South Africa (Theron & Mather, 2012) and projections for total rise in level by 2090 (Mather, May 2014).

Location	Rate of sea level rise	Mather prediction for SLR for 2090 for SA	IPPC global SLR predicted
Western Region (Cape Columbine to Walvis Bay)	+1.87 mm.yr-1,	0.21 m	
Southern Region (Cape Columbine to Port Elizabeth)	+1.47 mm.yr-1	0.18 m	0.26 to .82 m
Eastern Region (Port Elizabeth to Richard's Bay)	+2.74 mm.yr-1	0.28 m	

In addition to the eustatic change in sea levels, it is expected that the storm surge may increase in intensity and frequency since data reflects an increase in frequency of extreme waves ((IPCC, 2014; Proudman & Blackman, 3 September 2003). Storm surge is defined as “the abnormal rise in seawater level during a storm, measured as the height of the water above the normal predicted astronomical tide.” (National Ocean Service, May 13, 2014). Storm surge is due, in part, to a small rise in the water level resulting from low atmospheric pressure during storms. This is termed the inverse barometric effect. A more significant contribution to storm surge is wind set-up (National Weather Service: National Hurricane Center, 05 Sepember 2014). Storm surge is superimposed on the eustatic sea level rise.

The IPCC find that it is *very likely* that the currently observed increase in frequency of the most intense storms will continue (Church, et al., 2013). However, there is low confidence in the latter findings due to a large variability in findings for different basins. The exception is the Southern Ocean, where there is medium confidence that the projected increase in significant wave height will be 5-10%. The Co-ordinated Ocean Wave Climate Projection (COWCLIP) project predicts no change, or a negative change, in significant wave height for the coastal area around South Africa for the period 2075-2100 relative to 1980-2009 (Church, et al., 2013). This perspective is not supported by local predictions of future storm surge levels.

Table 6 provides an example of estimated storm surge water-level increase for Salt River in Cape Town. The estimates are for current conditions (before taking account of future climate change effects). From the similarity in wave climates between Cape Point and the FA (Aghulush) platform (Joubert, 2008), similar surge magnitudes would be expected at the Great Brak estuary mouth. As a measure of expected change in sea level during storms, a recent study showed a 0.5 m increase in significant wave height for peaks of individual storms over a period of 14 years (Theron & Mather, 2012).

Table 6: Estimated current storm surge heights Salt River Cape Town (Luger, March 2012)

Return period	Best estimate positive residual (m)	Upper 95% confidence positive residual (m)
20	0.64	0.71
50	0.70	0.80
100	0.74	0.87

Sea level comprises a number of parameters, all of which are affected by climate change. The parameters are given in Table 7. Theron (2008) and Luger (2012) identify slightly different parameters, as Luger introduces long waves as a parameter. Theron lists severe wind set-up separately, while Luger consolidates these into a single storm surge value.

The parameters in Table 7 are not necessarily cumulative. For example, storm surge may not occur at the same time as the highest astronomical tide. To determine the extent to which the parameters identified in Table 7 occur simultaneously it would be necessary to undertake a joint probability analysis. This would require extensive statistical modelling and is therefore outside the scope of this study. The wave set-up and run-up parameters are generally not applicable for water levels in estuaries in South Africa, as wave energy is typically dissipated at the mouth of the estuary.

Midgley et al (2007) indicate that the 2007 flooding event on the coast of KwaZulu-Natal, where the Highest Astronomical Tide (HAT) combined with a storm surge, had a return event of 1:500 years. They further indicate that the 0.4 m sea level rise will result in water levels currently found only at HAT, with a return period of 18.6 years (the Saros spring high tide, coincident with the metonic cycle of the moon), occurring instead at the frequency of the normal tidal variation over the lunar month. The probability that extreme rainfall and extreme storm surge events coincide is thus

increased (Midgley, et al., October 2007). Sea level rise used by municipalities and Eskom, for various risk scenarios, are shown in

Table 8 (Corbella & Mather, March 2012).

Table 7: Parameters and estimated maximum effects on still-water levels. Compiled from (Theron & Rossouw, 2008; Luger, March 2012)

Parameters and effects	Typical figures for	Time scale	Positive vertical effect on water level at Cape
			Town
Mean high water spring tide	1 m	12.4 hrs / 14 days / 18.6 yrs	1 m
Highest astronomical tide	1.4 m		
Storm surge (Severe wind set-up plus maximum hydrostatic set-up)	+0.35 m + 0.5 m	6 to 48 hrs	+0.7 (1:100-year return period)
Long waves		3 to 60 minutes	+0.9 m (1:100-year return period)
Wave set-up	+0.1 m	6 -48 hrs	+0.5 m
100-year sea-level rise (IPCC 2007)	+0.2 to 0.6 (say 0.4)	Decades	+0.3 to 2.0 m (Projection 2010)
Run-up		14-20 seconds	+2.5 m (1:100-year return period)

Table 8: Sea level rise and setback line values by various South African institutions (Corbella & Mather, March 2012)

Institution	SLR adopted
Overberg setback (WSP)	1.0 m
KZN setback	0.1, 0.3, 0.6 and 1.0 m
NMBM setback	0.3, 0.6 and 1.0 m
City of Cape Town	0.15 to 0.7 m
Eskom Nuclear Power Station	0.4, 0.8 and 1.0 m
Overstrand	0.3, 0.6 and 1.0 m
Richard's Bay	1.0 m

6.4 Consideration in determining water levels in estuaries

6.5 Bathymetry

Bathymetry is influenced by tidal effects, estuary size, geomorphology, and coastal and riverine sedimentation (Whitfield, 2007). The bathymetry of an estuary influences water levels in the estuary in that the bathymetry reflects the storage capacity in the basin as well as the loss of energy across the basin. Bathymetry is dependent on geomorphology, sediment availability and transport patterns, and change in the relative levels of land and sea, among other factors. The bathymetry is constantly changing under the influence of sediment deposition by river inflow and wave action, sediment removal under scouring floods, and anthropomorphic changes.

Data is not available on the change in bathymetry during flooding events. It was therefore not possible to model this. A simplified approach was adopted where the estuary bed was taken as fixed.

6.5.1 Sand barrier at the mouth

The Coastal Engineering Manual (USACE, April 2008) indicates that the swash bar / ebb tidal barrier acts as a hydraulic control, influencing the water level in the estuary, as well as producing an ebb-tidal delta between the shore and the ebb barrier. When the mouth is closed, the barrier acts a dam. When the mouth is open, flow will take place from the ocean into the estuary mouth, when the sea level is higher than the water level in the estuary. When the water level is higher in the estuary than the sea level, water will flow out of the estuary into the sea (Schumann, et al., 1999), as shown in Figure 19.

When water flows out of the estuary into the sea an ebb channel forms. The depth and width of this ebb channel depends on the size of the flood and physical constraint of the mouth. The geometry of the channel will change over the period of the flood, widening and deepening as the flood increases and the mouth sediments erode. Tidal flow also influences the channel's size and depth. Both flood and ebb channels can be established.

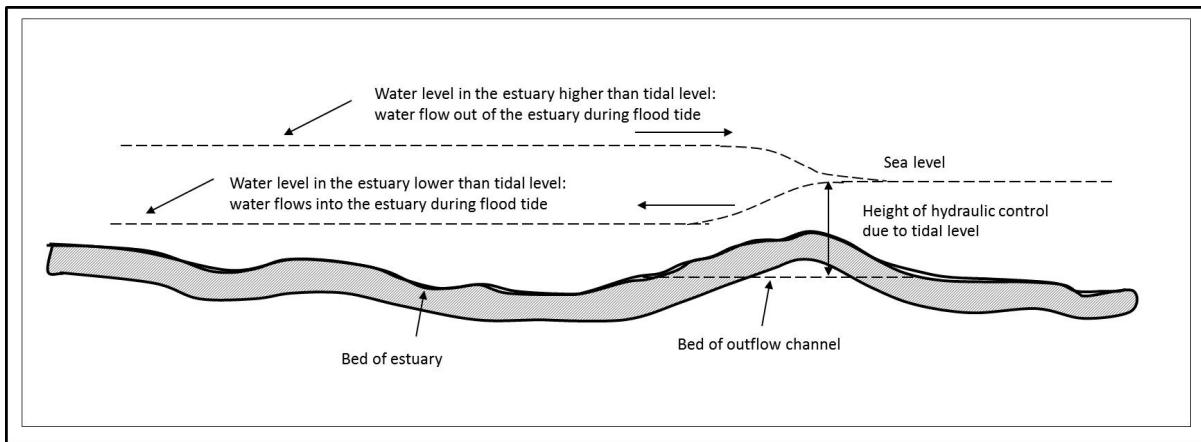


Figure 19: Direction of flow for high and low flood level in the estuary (relative to sea level)

The flow at the mouth, under open and closed conditions, is shown in Figure 20.

6.5.2 Tidal effects

In respect of tidal effects, for estuaries with an extent in the order of tens of kilometres, the water levels in the estuary generally rise and fall with the tidal levels. This may also be true if a larger estuary has a wide mouth (USACE, April 2008).

Narrow inlets are common to South African estuaries, particularly for temporarily open estuaries. For these, the attenuation of energy of waves entering the estuary is significant such that, for most South African estuaries, the wave energy is entirely dissipated on the sand bar and no wave energy is propagated into the estuary.

Water levels in South African estuaries with open mouths are therefore primarily driven by changes in the sea level, under high and low tides.



Figure 21 shows the waves breaking off Kei Mouth, demonstrating the dissipation of wave energy before the estuary mouth is reached. Wave run-up, as experienced on beaches, will not be applicable in the estuary due to the dissipation of wave energy offshore of the mouth. It has therefore been assumed that the still water level of the sea would be the applicable level for determining water levels in the estuary.

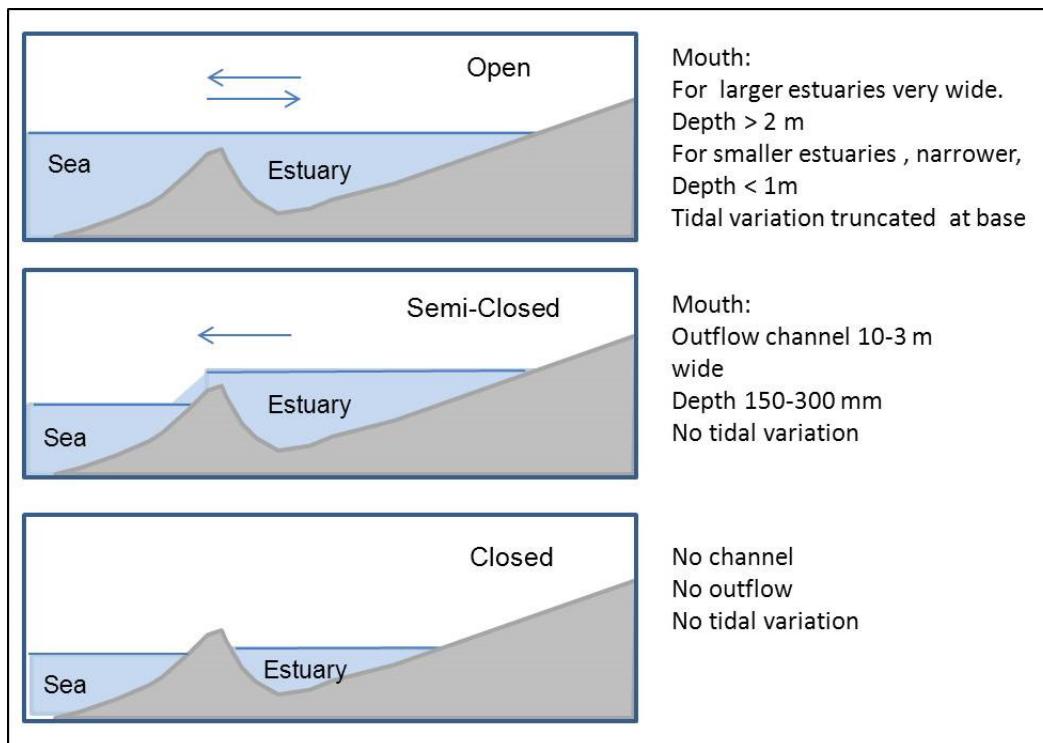


Figure 20: Flow conditions and mouth geometry for three mouth states (Van Niekerk, 2007)



Figure 21: Open estuary mouth at Kei Mouth showing dissipation of the wave energy seaward of the mouth (image from Google Earth)

6.5.3 Determining water levels in estuaries

Because of friction effects, the depth of flow at any section in the estuary is determined using conservation of mass, energy and momentum relationships. Standard hydrodynamic formulae, (such the St Venant equations), are applied to determine water level for each cross-section (DHI, 2012; USACE, April 2008). Water levels are a function of energy provided by the inflow, frictional energy loss, the cross-section of the flow (area, hydraulic radius and depth) and downstream controls such as downstream water level or structures (New South Wales Government, April 2005). Standard methods of estimating bed friction losses are used, such as the Chezy, Manning or Darcy-Weisbach formulas.

The flow at the mouth is potentially complex, involving an interaction between tides, waves, riverine flow and changing barrier levels (USACE, April 2008). Where wave propagation into the estuary is greatly reduced, an energy differential is required for tidal inflow into the estuary, as well as tidal and riverine outflow from the estuary. CEM advises that flow at the mouth can be determined using hydraulic laws and hydraulic equations for open channel flow (USACE, April 2008). For the modelling of flood levels in the Great Brak River, the mouth was treated as a cross-section with a raised bed, relative to upstream sections.

7 Modelling Methodology

The purpose of the hydraulic modelling was to determine flood levels in the estuary under extreme sea levels, given peak flood conditions and various scenarios for the height of the barrier at the mouth. The Great Brak River reach to be modelled for this study extends for 8 kilometres, from the Wolwedans Dam to the river mouth, with well-defined channels over its length. Within the estuary lagoon, below the N2 Bridge, an island diverts the flow to result in two clearly defined channels. While the floodplain widens considerably below Searle's bridge, the predominant flow at flood peak will be in the downstream direction. At the meander it is expected that, while there will be considerable lateral flow, the predominant flow will be in perpendicular to the cross-sections. Under high flow conditions, the flow path is expected to bypass the north-eastern meander bend to some extent. This has been accommodated by including a "link" channel in the model.

Modelling of the flooding of the estuary was undertaken use the Mike11, one-dimensional modelling software, developed by DHI. The one-dimensional model was selected for flood modelling as:

- there is clear channelization of the Great Brak River and estuary
- the velocity of flow under flood conditions is predominantly in the downstream direction, with the relative effects of lateral flow across the floodplain being taken into account through changes in the cross-sectional area.
- insufficient data may be available in real life situation to allow for two-dimensional modelling
- use of the simpler one-dimensional model rather than the two-dimensional model is more convenient and less costly when modelling floods in practice.

7.1 Mike11 computerized flow modelling software

MIKE11 is an internationally well-recognised, computerized one-dimensional (1-D) hydraulic modelling programme. It forms part of a suite of programmes developed by the Danish Hydraulic Institute (DHI). Mike11 has the capacity to allow user-defined structures, and thus to allow accurate determination of flow change at the structures on the Great Brak River. Mike11 also allows for the effect of the encroachment of structures into the floodplain. The encroachment module was not used for this study, as the purpose of the module is primarily to allow for the determination of flood protection measures. Mike11 was selected for this study due to its widespread use and its reputation as one of the best hydraulic modelling programmes.

7.2 Method of flow computation of Mike11

Mike11 calculations are based on one-dimensional flow, which assumes all flow is perpendicular to the cross-section under consideration. This, and a number of other assumptions, allow for simplification of the formulae for conservation of mass, energy and momentum. These simplified equations are called the St Venant equations, which are used by Mike11 to calculate flow volume and depth. The St Venant equations are applied for small changes ∂x in the distance along the horizontal axis in the direction of flow. The use of small increments in distance justifies the simplifying assumptions, as the changes in slope and velocity between sections are then negligible. The assumptions of the St Venant equations, and their applicability for this study, are shown in Table 9 below. The adjustments made to the inputs for the modelling of the Great Brak River, to accommodate these assumptions, are also given in Table 9.

Table 9: Applicability of Mike11 theoretical assumptions for modelling the Great Brak River

Assumption implicit in St Venant equations	Applicability of assumptions to modelling of Great Brak river	Adaptation in input to ensure assumptions are valid
Flow is one-dimensional. This implies velocity is constant and the water surface is horizontal across any section and perpendicular to the direction of flow	When the flow spreads to the floodplain, resulting in lateral flow, this assumption no longer holds	Lateral flows were accommodated by increased friction factors for the floodplain, and by creating a “link” branch to model “spill” from the main stream
Vertical accelerations are negligible and wavelengths are large compared to the water depth. This implies gradually varied flow with negligible change in hydrostatic pressure in the direction of flow	Applicable. Tidal wave-lengths are large compared to flow depth. The riverine flood wavelength is large compared to flow depth.	None required
Bed slopes are small	Applicable between adjacent cross-sections	None required
Channel bed is stable	Sediment movement is found	Fixed bed assumed, except at Searle’s bridge, where the upstream cross-section was adjusted to remove sediment accumulation. The rationale, (supported by photographic evidence), is that sediment upstream of the bridge is washed away before the peak flood passes.

Assumption implicit in St Venant equations	Applicability of assumptions to modelling of Great Brak river	Adaptation in input to ensure assumptions are valid
Streamline curvature is small	This condition is not met at the meander and mouth	Distance between cross-sections was reduced
The fluid is incompressible with constant density	Applicable	None required
The Manning and Chezy equations are applicable. These factors are defined for bed shear and for steady, uniform flow conditions	Unsteady, non-uniform flow conditions apply due to lateral flow and tidal inflow	Manning assumed to apply in line with model assumptions
Flow everywhere can be regarded as having a direction parallel to the bottom, i.e. vertical accelerations can be neglected and a hydrostatic pressure variation along the vertical can be assumed	Not applicable at bends	No adjustments made
The flow is subcritical. Mike11 is designed for subcritical flow. For supercritical flow, the term $\partial(\beta Q^2 / \partial x)$ in the momentum equation is neglected. As a result, non-uniform flow across the section is neglected. As β is a function of the Froude number, the model results will be less accurate with increasing Froude number (i.e.: where flow is turbulent).	<p>Potentially not applicable at the N2 bridge. At this bridge, flow is also expected to be non-uniform due to the proximity of the river bend and the occurrence of flow parallel to the road embankment (and at right angles to the bridge).</p> <p>May affect the computation at the mouth as two channels merge close to the mouth and</p>	No adjustments made – model results interpreted as erroneous

Assumption implicit in St Venant equations	Applicability of assumptions to modelling of Great Brak river	Adaptation in input to ensure assumptions are valid
	<p>the mouth is at right angles to one of these channels.</p> <p>Flow over mouth barrier is also likely to be turbulent once the sea level reaches the level of the mouth. This will apply under incoming tide, as the direction of tidal flow is opposite to the riverine flood.</p>	

7.3 Model input data

Modelling of the floods for the Great Brak River using Mike11 requires data that defines the physical characteristics of the system, including river alignment; data on physical elements that affect flow (e.g. sediment); cross-section data; estuary bathymetry; and information on energy losses in the system, as represented by roughness coefficients. Information on the location and physical characteristics of structures affecting flow is required.

In addition, information is required on flows that affect the flood levels in the estuary. This includes tidal levels and riverine flood inflows. For riverine flood flows, the flood hydrograph was determined from rainfall-runoff modelling, which required data on land use, slopes, river length, point rainfall, and catchment characteristics. Historical measurements of water levels in the estuary, correlated with historical runoff and tidal level data, allowed for the calibration of the model, and thus an assessment of accuracy of the water levels determined by the Mike11 model. Sources of information are listed in Table 10 below. The quality of the data, as assessed by the author, is also indicated

Table 10: Sources of data used in this study

Data	Source	Quality
GIS data		
• Rivers, dams and catchments	DWA GIS ArcGIS (Department of Agriculture)	Manual digitization Large scale, required estimation of boundaries
• Land-use, vegetation, catchment characteristics	SANBI BGIS	Good
• Estuary layer	NSDP mesoframe data	Good
• SA infrastructure, towns, cadastral	Surveyor General Mossel Bay municipality	Good
• 5 m and 1 m contours	(through J Pieterse)	
Rainfall data	South African Weather Services	Good, but outdated (last 13 years not included)
Sea level data	University of Hawaii website	Variable, periods missing

Data	Source	Quality
Cross-section and bathymetry data	DWA via CSIR Stellenbosch	DWA cross-sections not geo-referenced nor referenced to level benchmark. Bathymetry good
Flow and level gauge data	DWA hydrological service website	Good

7.4 Modelling the Great Brak River in Mike11

Figure 22 below shows the network diagram adopted for the Great Brak River and estuary. The river section modelled commences directly below the Wolwedans Dam and extends to the estuary mouth. The alignment of the river was followed when developing the network diagram. A “link” branch was added to the model, linking flow from just before the first (north-eastern) meander bend, to the second (south-western) meander bend. Photographs of the river reach show a well-defined channel occurring at this locality. The head of this channel fills with sediment at low flows, but this sediment is washed away at higher flows. The effect of this “link” channel will be to speed up the flow through the second meander bend. The channel compensates, to some extent, for the inability of the model to accommodate cross-flow.

For purposes of modelling only two boundary points were set: at Wolwedans Dam and at the estuary mouth. Inflow from minor tributaries and diffuse inflow at the estuary were not modelled as additional boundary conditions, as the effect of the locality of this inflow is expected to be minimal. The contribution of these source to flow was taken into account when determining the inflow at the Wolwedans Dam boundary. At Wolwedans Dam the outflow hydrograph for the dam, as well as the flow due to the contribution of the catchment downstream of the Wolwedans dam, was combined and input into the model as a time series boundary condition. At the estuary mouth simulated tidal levels for different sea level scenarios were input as time series boundary condition.

The location of the cross-sections used, and the width of the cross-sections, are shown in Figure 23. Cross-section location was chosen based on available data, supplemented by additional cross-sections to ensure that the difference in bed level, flow direction and river width between adjacent cross-sections is not excessive. Selection of the width of the cross-

sections was challenging as the cross-sections potentially encroach on one another where river branches have been modelled. This is because the river sections modelled as branches have sections flowing next to, and almost parallel to, each other. To ensure that there was no overlap in cross-sections, certain cross-sections were input as having vertical side-walls. These vertical side-walls then abut onto the adjoining cross-section of the parallel branch. An example of a cross-section is shown in Figure 24. An example of two abutting cross-sections is shown in Figure 25.

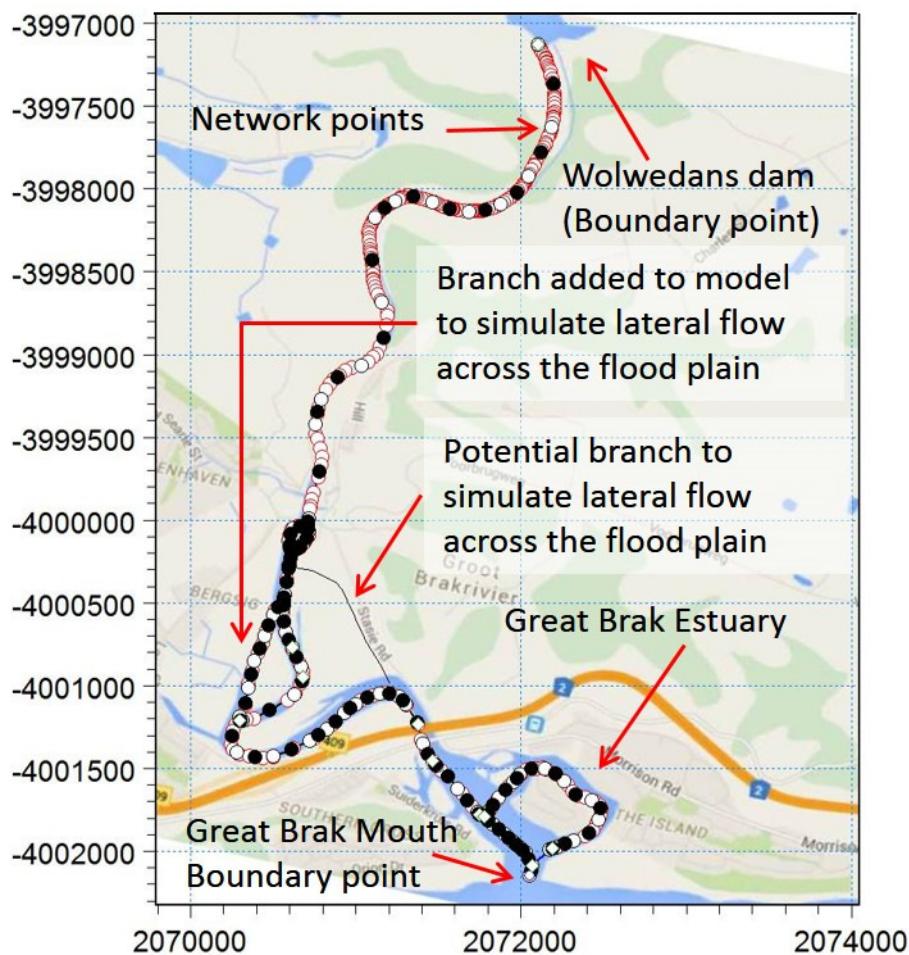


Figure 22: Network diagram for the Great Brak River and estuary

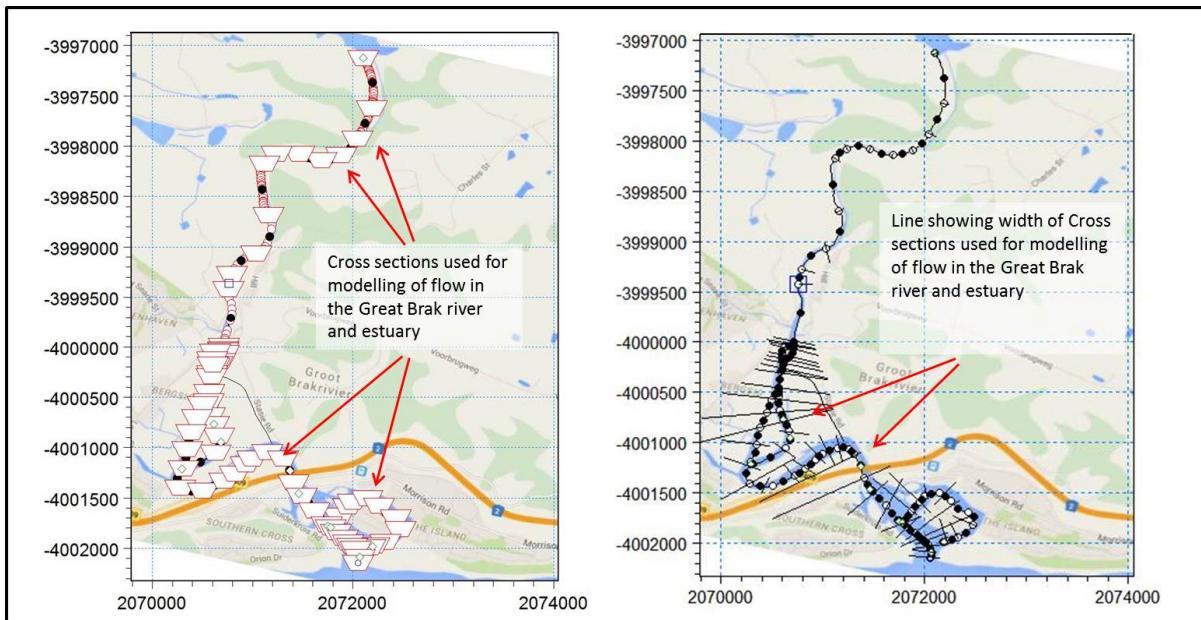


Figure 23: Location and width of cross-sections used in modelling

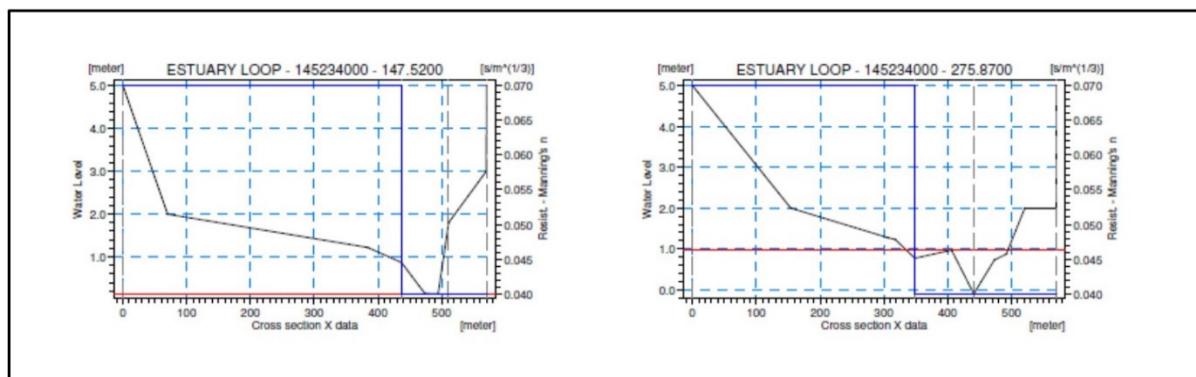


Figure 24: Example of cross-sections captured in the Mike11 for the Estuary branch

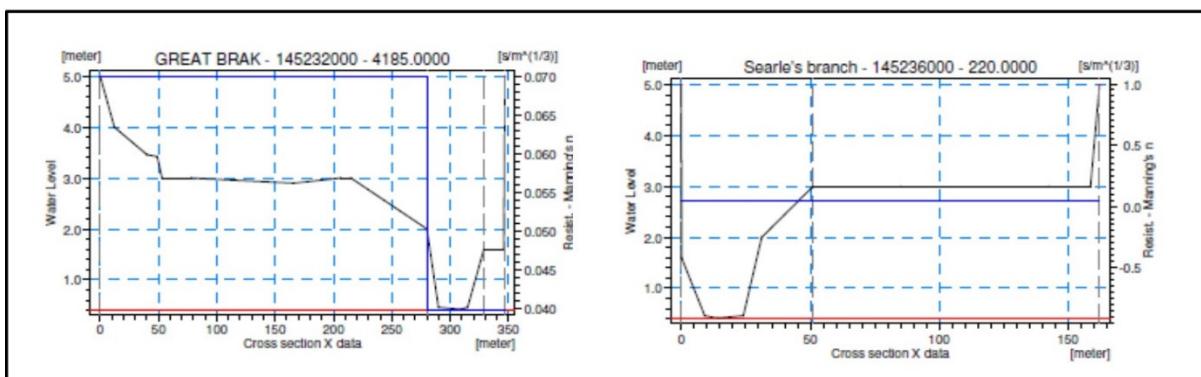


Figure 25: Example of abutting cross-sections of two channels, showing vertical sides

7.5 Roughness coefficient

Measurements of roughness coefficients are not available for the Great Brak River or its estuary. Roughness coefficients were therefore derived through trial and error, using an existing flood event to calibrate the flood levels by adjusting the roughness coefficients. The water level gauge in the lagoon was used as the indicator against which the results of the modelling were measured. Table 11 shows the typical roughness coefficients (Manning's n) for different flow boundary conditions. These were used as guide to start the calibration.

Table 11: Manning n Values for various channel types (Chow, 1959)

Channel and Description	Minimum	Normal	Maximum
Major streams : top width at flood stage > 30 m			
a. Regular sections, no boulders or brush	0.025	-	0.060
b. Irregular, rough section	0.035	-	0.100
Minor stream: top width at flood < 30 m			
a. Clean, straight, full stage, no rifts or deep pools	0.025	0.030	0.033
B. 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	0.80
h. Very weedy reaches, deep pools, or floodways with heavy stand of timber and underbrush	0.075	0.030	0.150
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's n values of 0.04 and 0.07 were chosen for use throughout the river length and in the estuary. For the riverbanks $n = 0.07$ was generally adopted, and $n = 0.04$ was generally adopted for channels. The high bank coefficient compensates for the lateral flow energy losses and vegetation, which may occur on the riverbanks and the floodplain. However, for the Great Brak branch the resistance factor for the right bank was adjusted to $n = 0.04$ as it is generally narrower and less lateral flow is anticipated. For the "link" branch, $n = 0.04$ was chosen for the channel and both banks, as the area outside the channel is limited and the high flow velocity in the steep channel will limit the effect of lateral flow. An attempt to refine the model outputs, by introducing greater variation in friction factors for different sections of the river, did not produce significantly improved results.

7.6 Modelling of riverine inflow

Although flow gauge information is available upstream of the Great Brak estuary, the flow measured is not a true reflection of the peak flow record since the flow peak is moderated through the Wolwedans dam. Rainfall-runoff modelling was therefore used predict the flow for the extreme runoff event to be used in the modelling. River runoff was determined using standard hydrological methods, as set out in the Road Drainage Manual (South African National Roads Agency Limited (SANRAL), 2007) and Flood Frequency Estimation Methods as applied in the Department of Water Affairs (Van der Spuy, 2010). Runoff was calculated using the Design Flood Estimation Tool (DFET) of Stellenbosch University (Gericke, n.d.). Flow was routed through Wolwedans dam, using the modified PULS method, to produce an adjusted peak and time of concentration, reflecting the moderating effect of the dam. The design flood was calculated assuming the worst-case scenario for flooding in the estuary, where Wolwedans and Ernest Robertson dams are full at the time of the peak flood, such that the full volume of the peak flood passes over the dam.

7.7 Modelling of sea levels

Sea levels were identified from SANHO tables and adjusted to give values relative to current MSL. As the SANHO information is relative to chart datum the information must be adjusted to align with levels measured onshore and estuary levels, both of which are relative to MSL. Figure 26 shows the datum applicable for the Mossel Bay sea level gauging station. Chart datum has changed several times since 1988. Means sea level was last adjusted in 1964, at which time sea level records became available. Historical sea level data has been adapted to Mean Sea Level by applying the value of the relevant chart datum for that specific period.

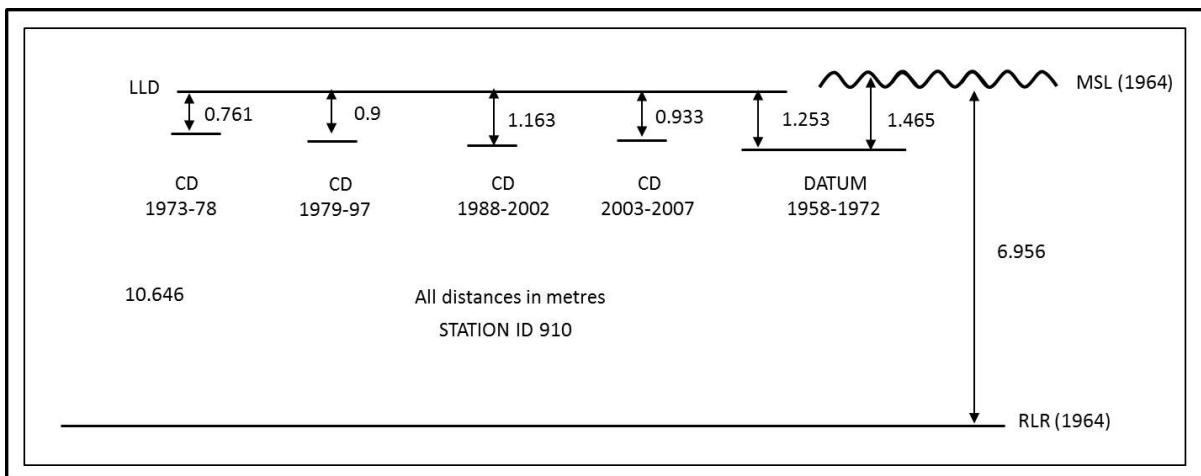


Figure 26: Changes in Chart datum changes relative to land levelling datum for Mossel Bay (University of Hawaii Sea Level Centre, n.d.)

Abbreviation used in Figure 26

LLD	Land levelling datum	RLR	Revised local reference
CD	Chart datum	MSL	Mean sea level
MLWS	Mean low water springs	MHWN	Mean high water neaps
MLWN	Mean low water neaps	MHWS	Mean high water springs
ML	Mean level	HAT	Highest astronomical tide

Sea levels taking into account the effect of Sea Level Rise and increased storm surge were adopted from the literature.

8 Hydrodynamic features of the Great Brak River and estuary

8.1 General

The Great Brak catchment is long and narrow, starting in mountainous reaches at an elevation of 1350 m (Surveyor General, South Africa, Accessed August 2014) with rainfall in excess of 800 mm per annum (Agricultural Research Commission, n.d). The river drains a catchment of 188 km² as measured on a GIS system²⁰. The middle reaches of the river flow through gentler terrain where agriculture (and formerly irrigation) is practised and rainfall is between 600 and 800 mm per annum. The lower reaches are very flat and rainfall is below 600 mm per annum. The Wolwedans dam is located on the river 8 km upstream of the mouth²¹. The Wolwedans dam provides water to Mossel Bay, George and the town of Great Brak. Excluding the branch around the Island in the estuary, the flow path is 8 km from the dam to the estuary mouth. Flow in the Great Brak River below Wolwedans is regulated by releases from the dam. A smaller dam, the Ernest Robertson dam is located on the Groot River 14.5 km upstream of Wolwedans dam. Apart from the two dams, the structures affecting river flow include the Searle's bridge, the N2 highway bridge, a secondary bridge, a railway bridge and a bridge providing access to the Island. Figure 27 and Figure 28 show the bridges crossing the river.

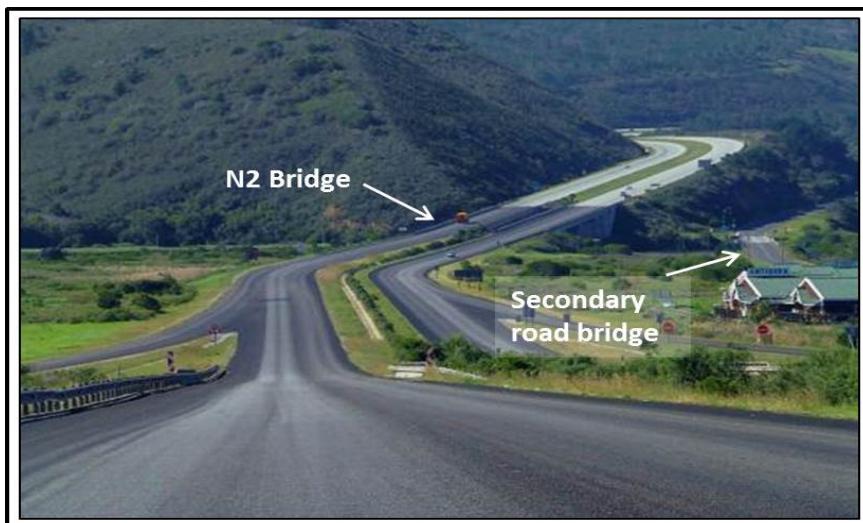


Figure 27: N2 and secondary road bridges aerial view (source: www.wheretostay.co.za)

²⁰ Using catchment boundaries provided from state agencies through PLANETGIS and National Spatial Development Plan datasets

²¹ Following the river centreline and including meanders, following the low flow path east of the Island.

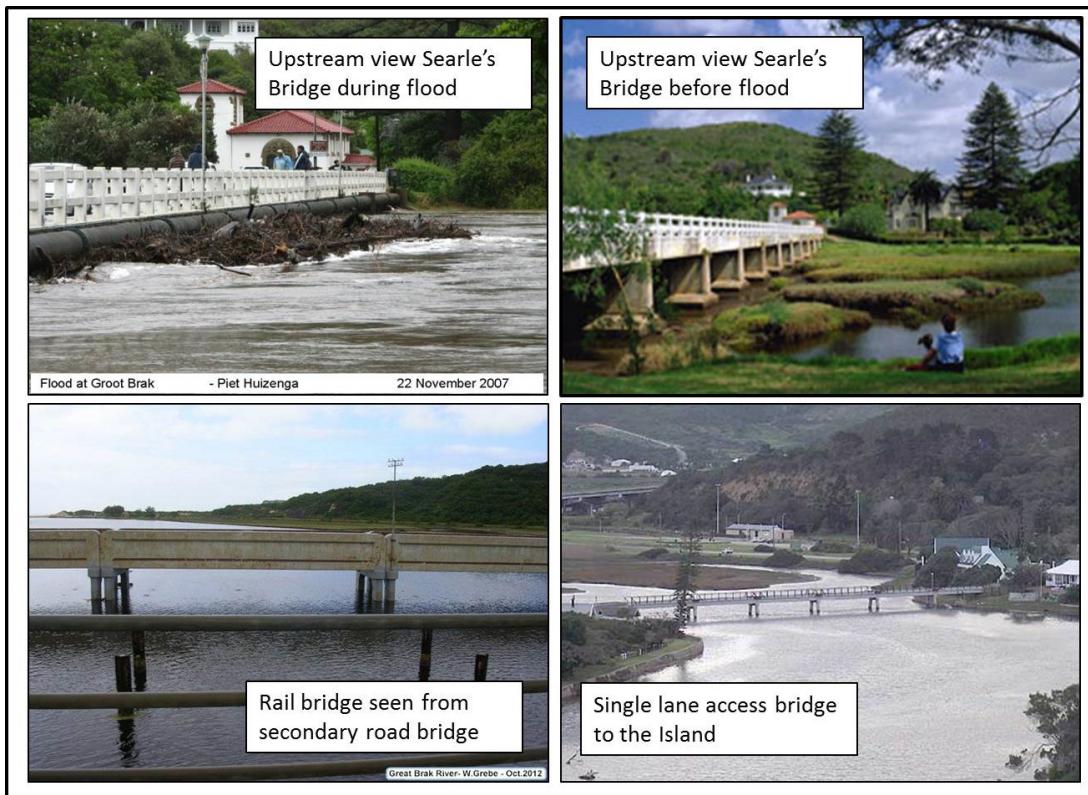


Figure 28: Bridges crossing Great Brak River (sources: Piet Huizinga (EWISA, 2014); W Grebe, www.twosunsets.co.za)

8.2 The estuary

The estuary lagoon, area below the N2 Bridge, is small, having a length of just over 1 km from the mouth to the N2 Bridge, and a width of just over 700 m. The estuary extends 6 km upstream, after which the tidal effect is no longer evident. The northern extent of the lagoon is constrained by three bridges, lying within 130 m of each other, and their embankments, as shown in Figure 29.

The width of the mouth is limited on the east by dune and soft rock formations, as shown in Figure 30. To the west, low dunes provide a soft constraint on the width of the estuary mouth. There is a large sediment deposit upstream of the N2 Bridge, with a significant meander around this formation. The sediment high points are between 2 and 3 m above MSL and the sediment area is submerged during the 1:100-year flood²².

²² As reflected in Annexure A (Gorra Water: Kleynhans, 10 May 2010)

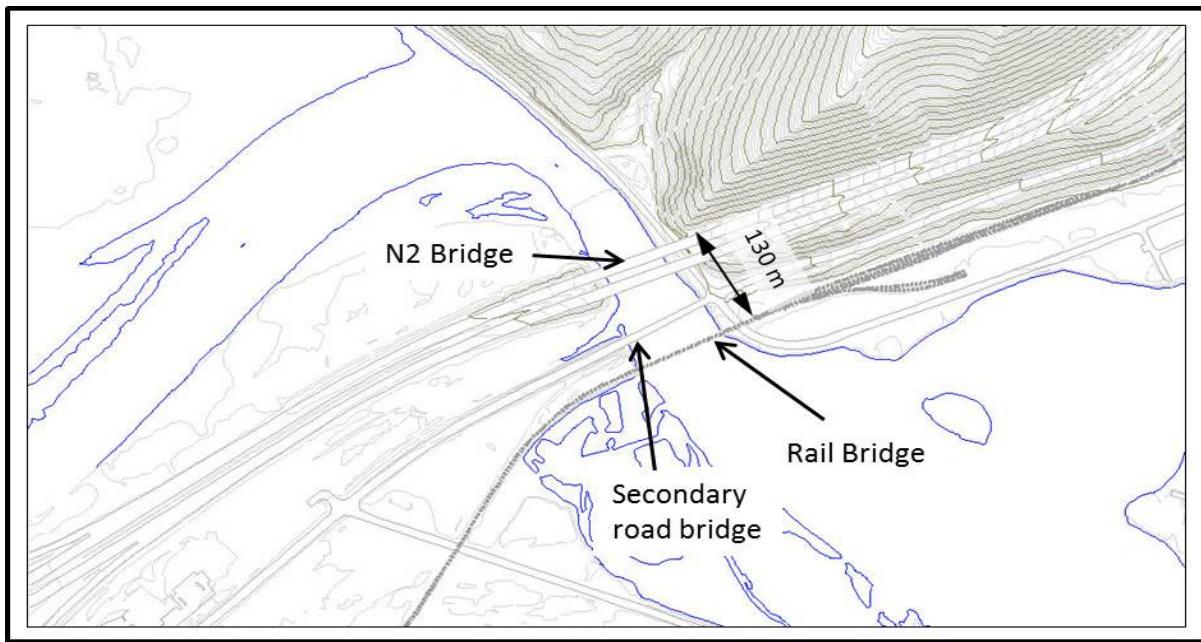


Figure 29: Three bridges within distance of 130 m (from PlantGis, modified from data from **(Mossel Bay Municipality, n.d.)**)



Figure 30: Dunes on left bank inhibiting width of estuary mouth (www.dwa.gov.za; www.PamGolding.co.za)

The bed level of the estuary depends on the extent to which sediment has accumulated, or has been flushed away, and the level of the sand barrier at the mouth. In 10 surveys spanning 10 years from 1989 to 1999, the level in the estuary was typically 0.7 to 0.9 m above MSL, with the bed level in the eastern channel being -0.1 to -0.4 m above MSL. The

exception is a deep section on the southeast bend of the eastern channel in the lagoon, where levels are below -1 m MSL. The level for the sediment deposit to the west of the Island varies between 0.6 and 1.3 m above MSL, while the level of the sand barrier at the mouth varies between 0.3 and 2.7 m above MSL. When the mouth is open the level at the mouth varies from -0.2 to -1.2 m MSL. The bed levels in the lagoon are therefore dependant on whether the mouth is closed. For the sediment accumulated to the west of the island to become inundated, the level of the barrier at the mouth would have to exceed about 1.3 m above MSL. Bathymetry plots are shown in Figure 31 and Figure 32 for open and closed mouth conditions, respectively.

8.3 Gauging points

The river is well provided with gauging points measuring flow, abstraction from the dams and abstraction via pipelines and canals. A water-level gauging station for the estuary is located on the railway bridge, with records available for the period 1988 to 2014 (Department of Water Affairs Gauging Stations, n.d.). Sea level data is available at the Mossel Bay sea level gauging station for the period 1964 to 2012 (University of Hawaii Sea Level Centre, n.d.). Historical flow records are available for the gauging weir K2H002, for the period 1961 to 2014, for daily average flow. Figure 33 gives a schematic of the gauging stations, with key information on record length, catchment, etc.

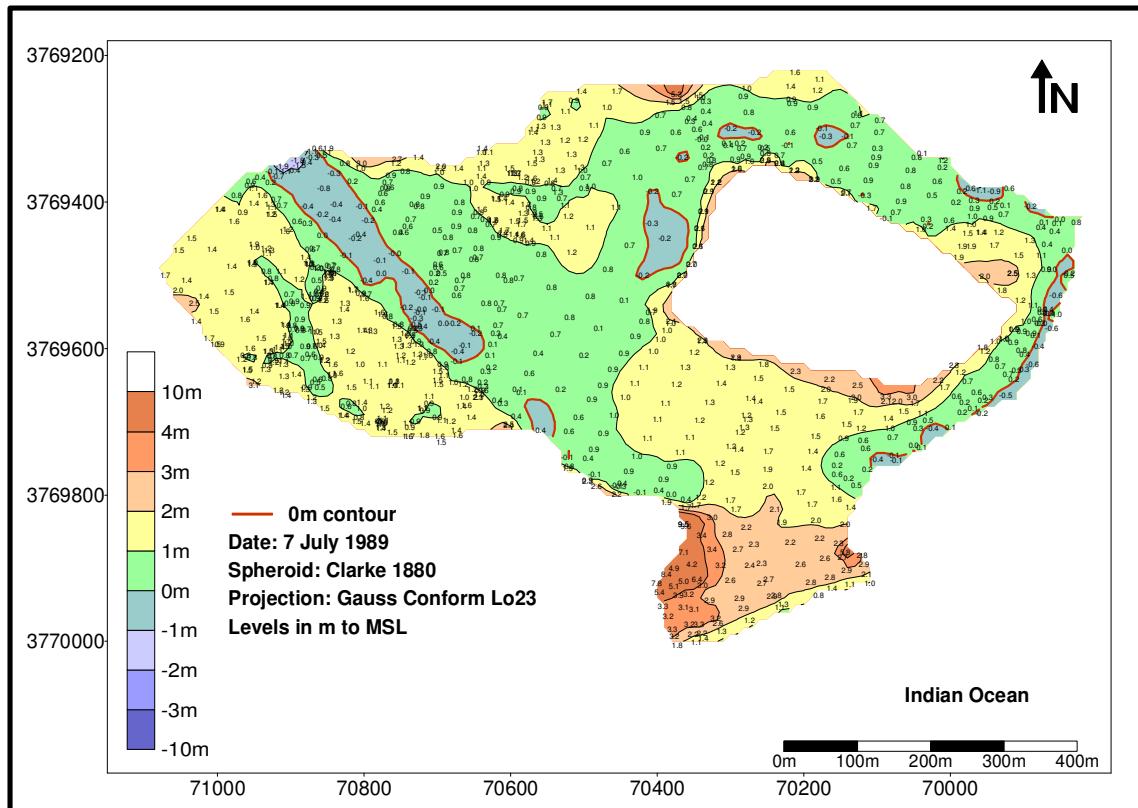


Figure 31: Great Brak bathymetry with high sand barrier level, 1989

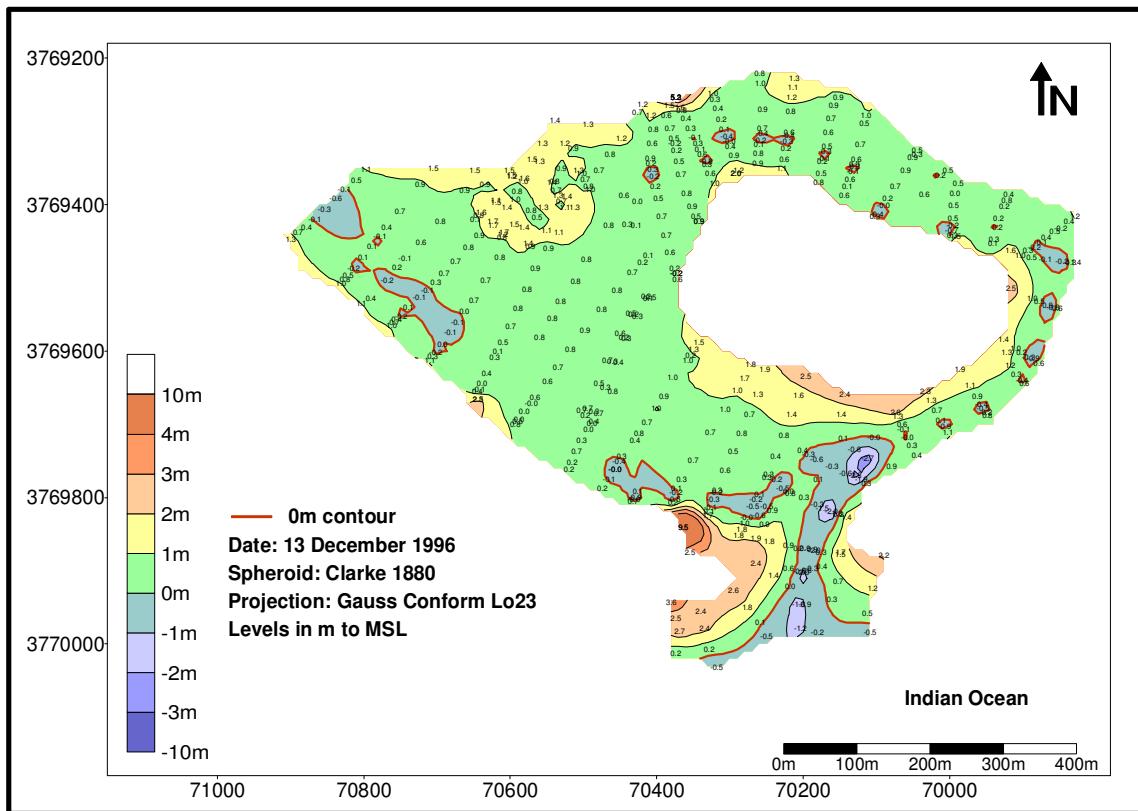


Figure 32: Great Brak bathymetry with open mouth

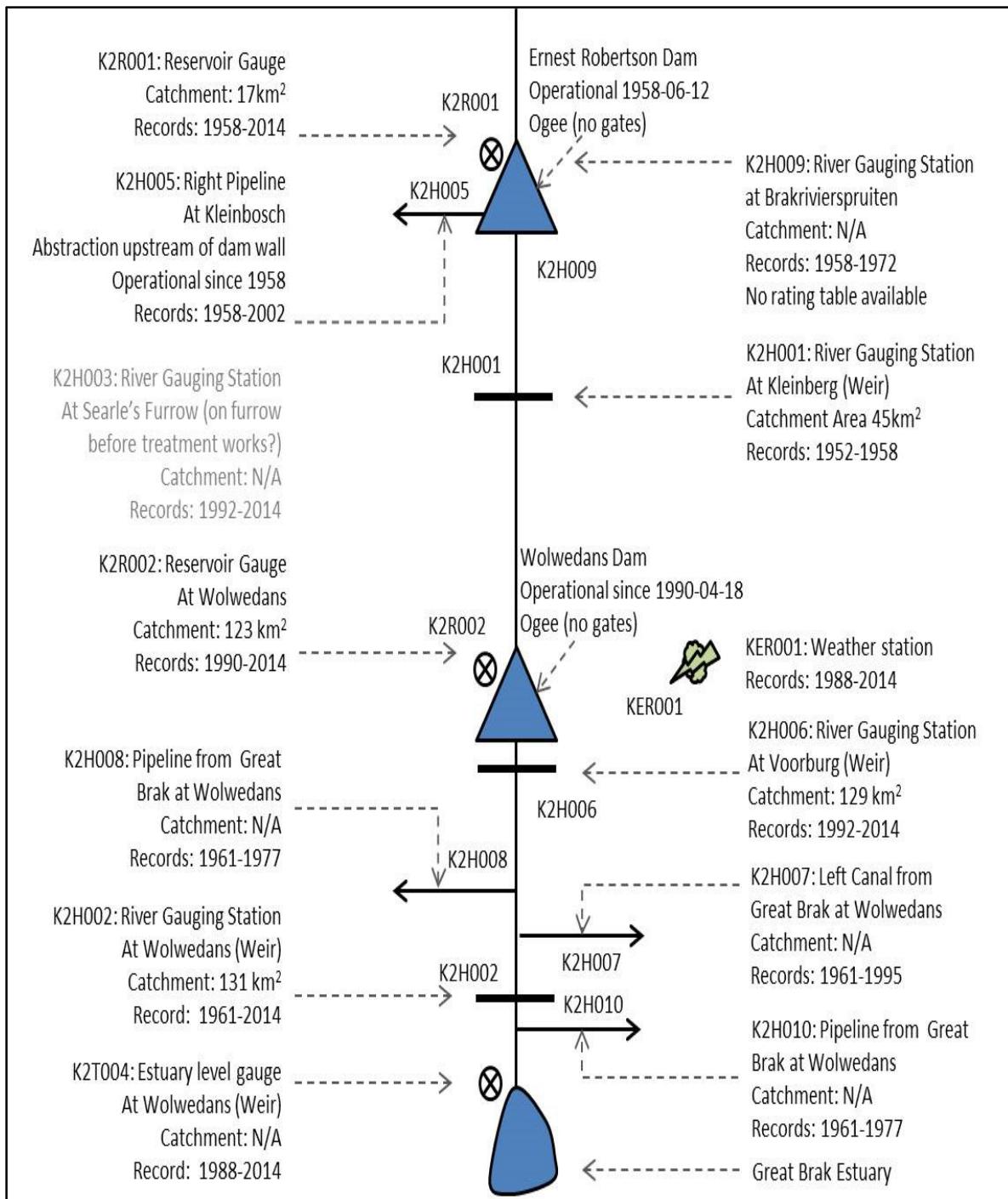


Figure 33: Graphic of Great Brak River (by author)

8.4 Sediment

Given the potential effect on flow paths and resultant modelling inputs, considerable attention was given to the nature and pattern of sediment deposits, as well the changes to sediment during flood events. In reaches immediately below Wolwedans dam, sediment deposits are not significant. Sediment deposits are significant in three areas:

- At Searle's bridge (above and below)
- At the meander above the N2 bridge
- In the lagoon, below the rail bridge and to the west of the Island

These sediment areas are shown in Figure 34.

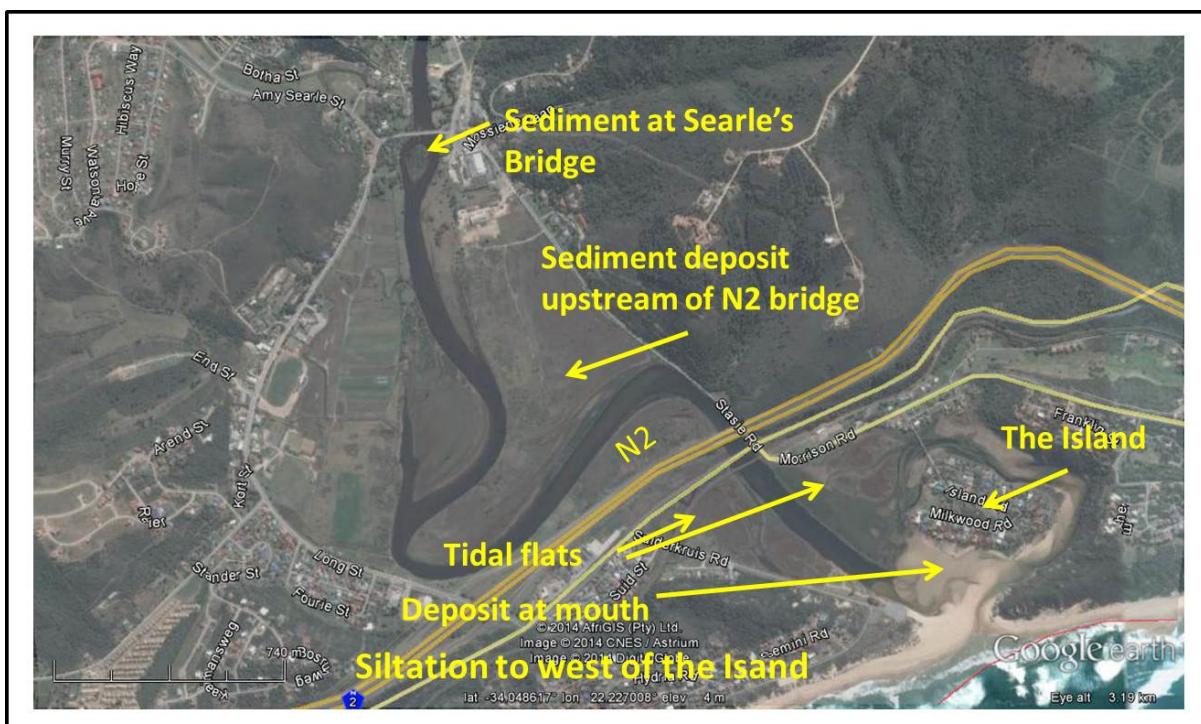


Figure 34: Main Sediment deposits on Great Brak (source: annotated on Google Earth image)

8.5 Sediment at Searle's bridge

Small sediment islands have formed upstream and downstream of Searle's bridge. Both deposits have become vegetated as shown in

Figure 35.

Figure 35(b) is an older picture (note there is no pipe fixed to the bridge) and demonstrates considerable accumulation of silt upstream of the bridge. The later photograph in Figure

35(a) shows sediments on the eastern bank washed away, but the sediment on the western bank has trees growing on it, indicating that it would be more resistant to erosion by flood water. For the purposes of modelling, the west-bank sediments were assumed as immovable, while the east-bank sediments were assumed to have been washed away. The downstream sediment, seen in

Figure 35(c), is well vegetated, and has established shrubs on it, as seen on the upstream east-bank sediment bank. For purposes of modelling this sediment was assumed stable / immovable.

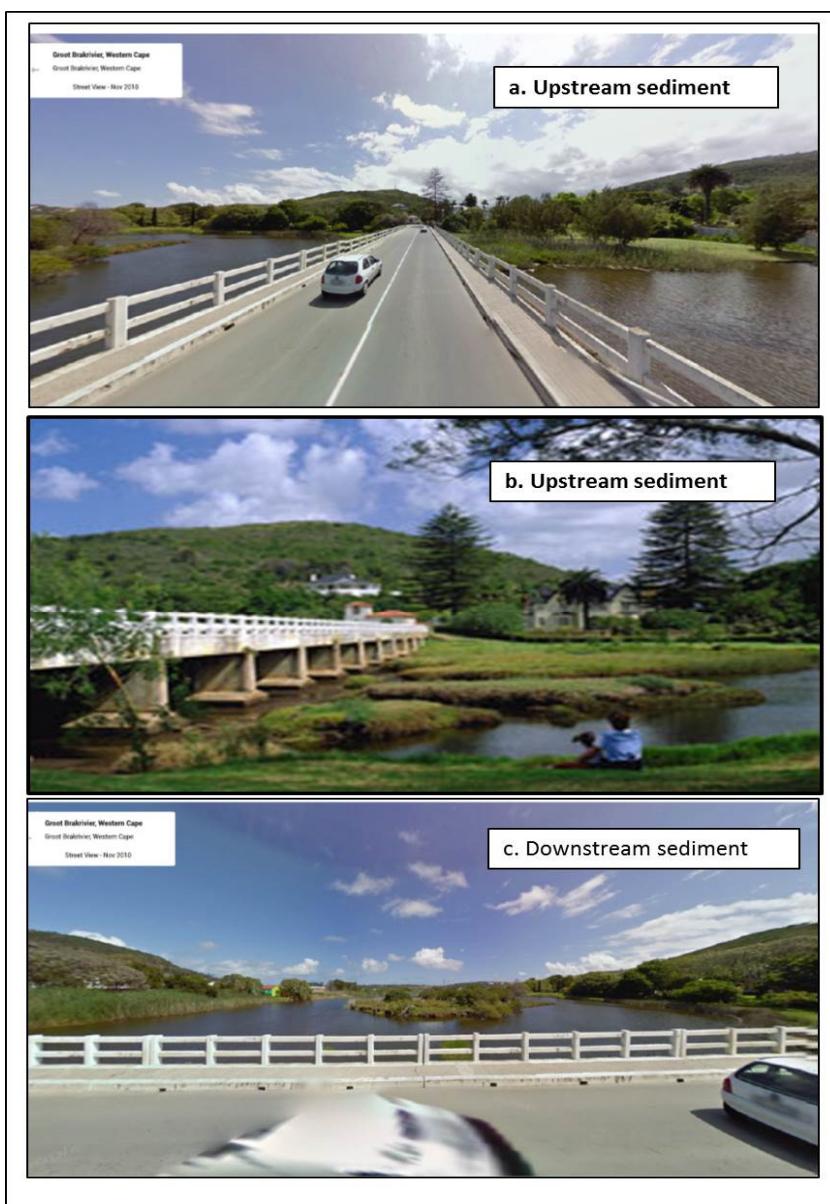


Figure 35: View of sediments upstream and downstream of Searle's bridge (source (a) and (b): Google Earth street view 2010; (c) www.twosunsets.co.za)

8.6 Sediment and meander upstream of N2 Bridge

As it is anticipated that the watercourse will straighten during floods, bypassing the meanders to some extent. A number of photographs were sourced to establish the pattern of behaviour of the river over and around this sediment area when in flood. The contour plan of this area shows signs of a broad and shallow depression on the eastern floodplain between Searle's bridge and the N2 Bridge (in the south). A deeper, more defined channel is found to the west of the main channel just upstream of the meander bend (see Figure 36).

The following is evident from available photographic sources:

- From the pattern of sediment, it is anticipated that the more defined western channel will become active under flooding, as it is lower lying (as it has water in it even at low flow levels, when the east channel is dry). The eastern floodplain depression is expected to convey water under high floods when the higher ground to the east and downstream of Searle's bridge (where buildings are situated) is breached and/or backwater resulting from the narrowing at the three bridges submerges this higher ground. At this stage, a portion of flow will be diverted through this depression, taking the shortest route between Searle's bridge and the N2 Bridge. As this depression is shallow, and at a higher level than the main channel bed, the bulk of the flow is still expected to follow the main river channel. The expected flow pattern during floods is shown in Figure 37.

Despite the development of the eastern and western channels under high floods, the main channel is very deep at the meander (3.8 m deep) and will carry the bulk of the flow. The embankment of the N2 Bridge will act to dam the flow and reduce the velocity head. The velocity will be further reduced at the confluence of the main and east channels, the narrowing of the channel resulting from the three bridges and the required change in direction of the flow (in the main channel) required to pass the water under the three bridges. The result will be increased cross flow towards the east channel.

The Gorra study shows the meander fully submerged during the 1:100-year flood, the expected flow pattern under the 1:100-year flood is shown in Figure 37.

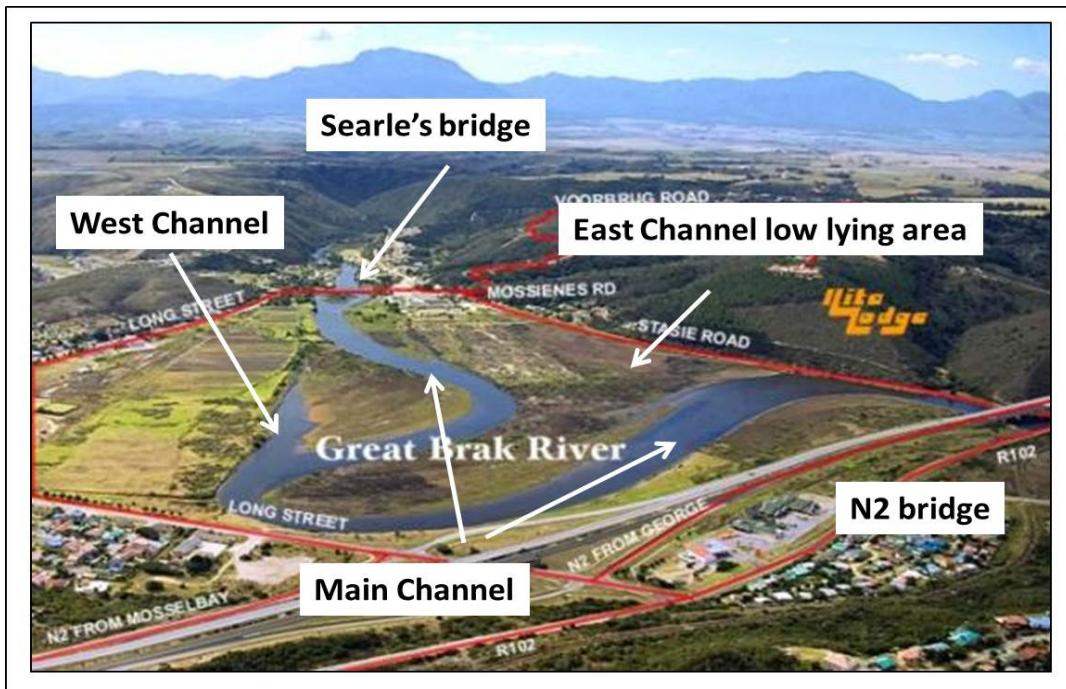


Figure 36: Meander east channel shown as sedimented (modified from www.sleeping-out.co.za)

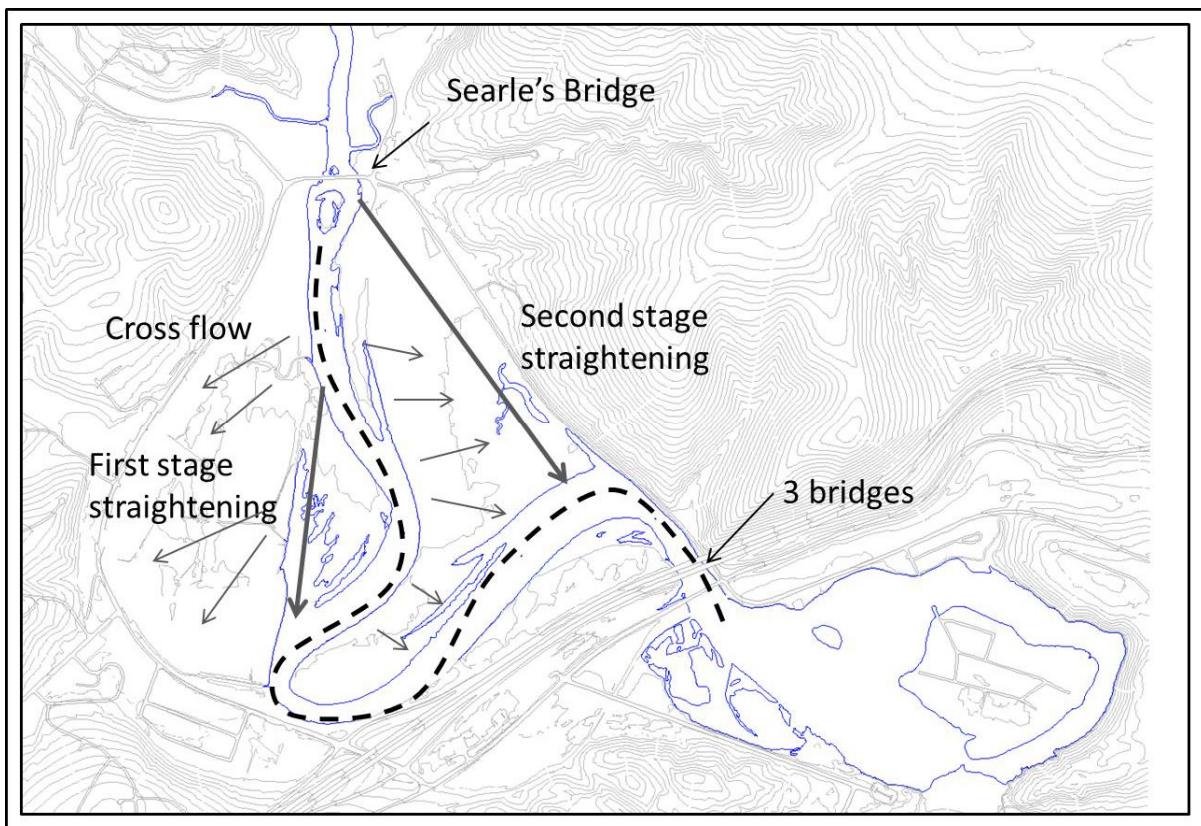


Figure 37: Expected flow patterns under high return period floods (modified from (Mossel Bay Municipality, n.d.)

8.7 Sedimentation below the three bridges

The lagoon below the rail bridge is extensively sedimented, forming tidal flats not submerged under normal tidal conditions but submerged under flood conditions. These flats have defined channels, which will carry the higher flows initially. The Gorra Water analysis indicates that these flats will be submerged during the 1:100-year flood event. The low flow channels are shown in Figure 38.

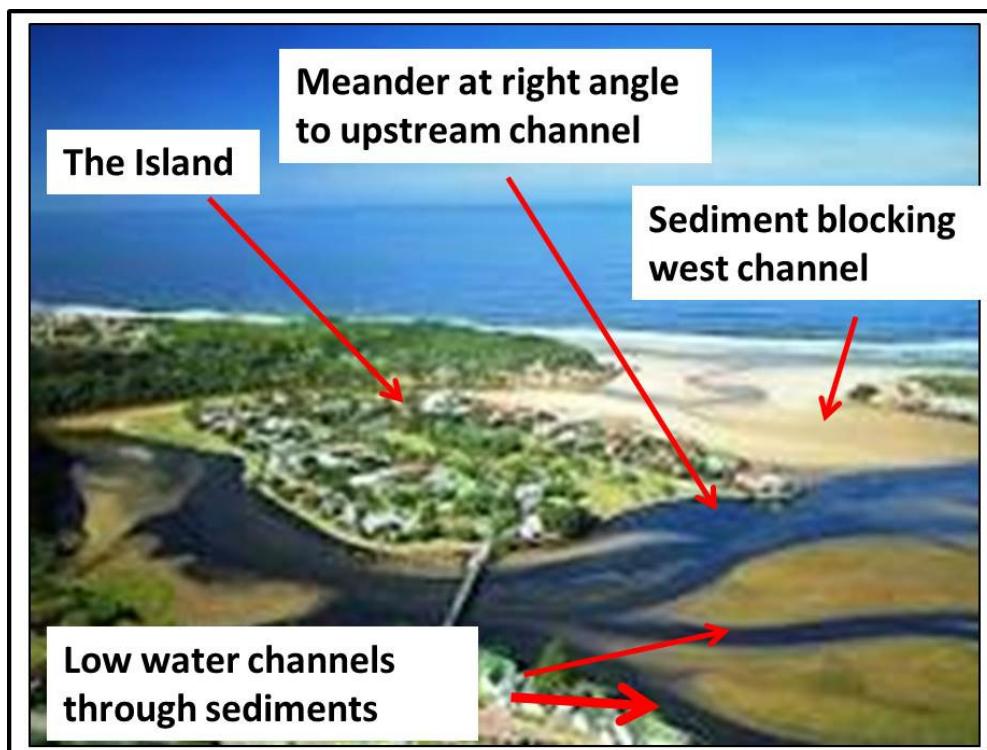


Figure 38: Estuary sediments showing low water channels (modified from http://en.wikipedia.org/wiki/Great_Brak_River_%28town%29)

Two main channels flow through the estuary, diverging around a large stable flood tidal delta approximately 180 m upstream of the mouth. The perennial channel diverges around, and to the east, of the Island, as shown in Figure 38. The eastern channel is generally the dominant channel at low flow, even when the mouth is open, as the western channel is blocked by sediment deposited from the sea at low flow. This sediment stretches up to 300 m into the estuary. Even during flood conditions, the western channel is not necessarily flushed open. The western channel from time to time develops a meander through the barrier, when flow is sufficient and the barrier is poorly developed. The sediment at the mouth forms a barrier up

to 2.7 m in height (bathymetry 1990), which is above the highest sea levels measured, which have not exceeded 2 m above MSL. The barrier has been artificially breached since 1978, when the water level in the estuary reaches between 1.5 and 2.0 m above MSL, and when flooding is expected. A large sediment deposit (“the Island”) in the middle of the estuary is stable, with a general height of between 2 and 3 m above MSL, and a high point of over 7 m above MSL. Housing has been developed on the Island. The Island is subject to flooding.

For the purposes of modelling, the sediments in the estuary were taken as follows:

- The Island was assumed as stable and a fixed bed level
- The eastern channel bed level was taken as indicated in the bathymetry (CSIR 1988)
- The western channel was assumed open to the sea during the 100-year flood, and levels were interpolated from measurements at the three bridges and the mouth.
- Channels in sediments downstream of the rail bridge (east and west) were neglected in modelling.

9 Model inputs

9.1 Boundary conditions: inflow

The resultant hydrographs after routing of flow through the Wolwedans dam are shown in Figure 39.

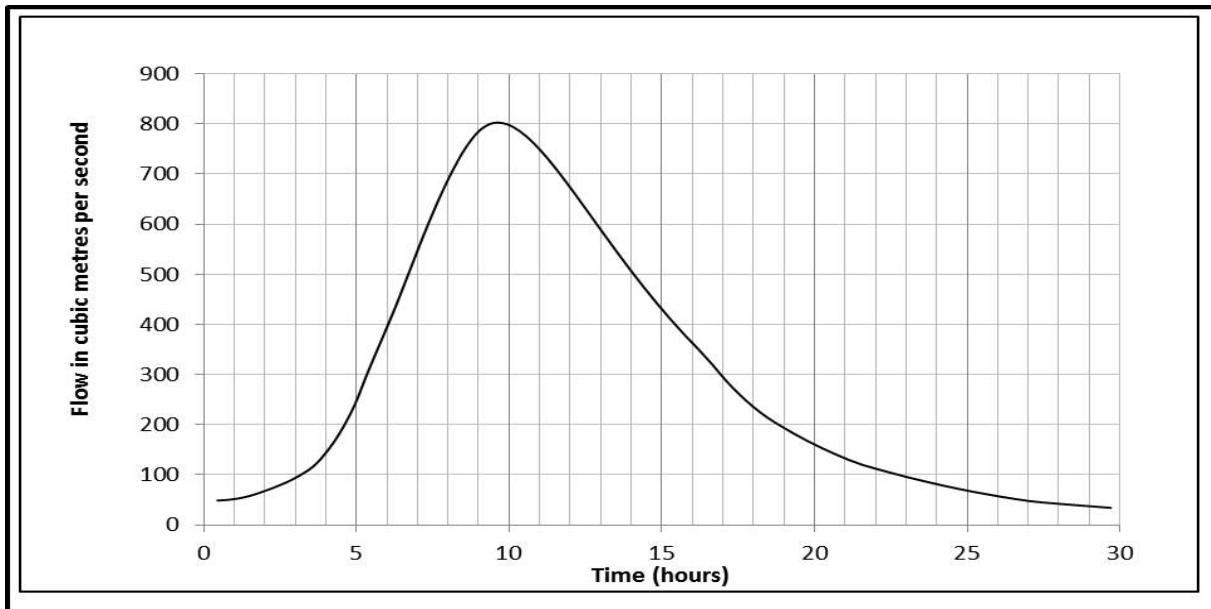


Figure 40 shows the design flow hydrograph at the estuary mouth. Design flows are given in Table 12.

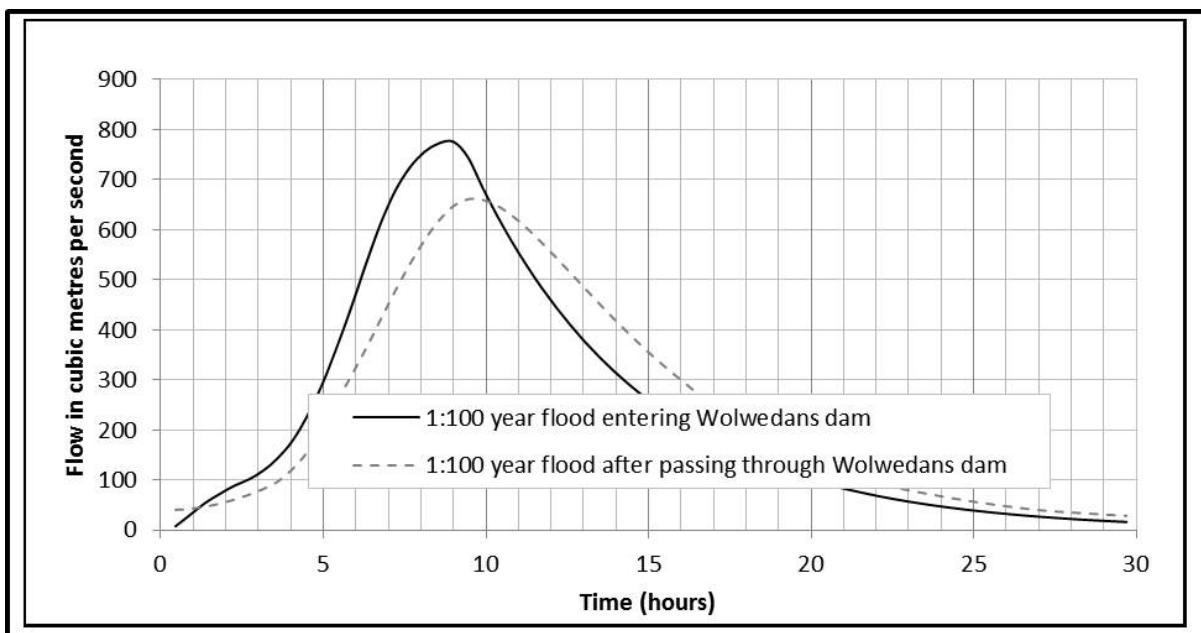


Figure 39: Resultant flow hydrograph at Wolwedans after routing flow through the reservoir

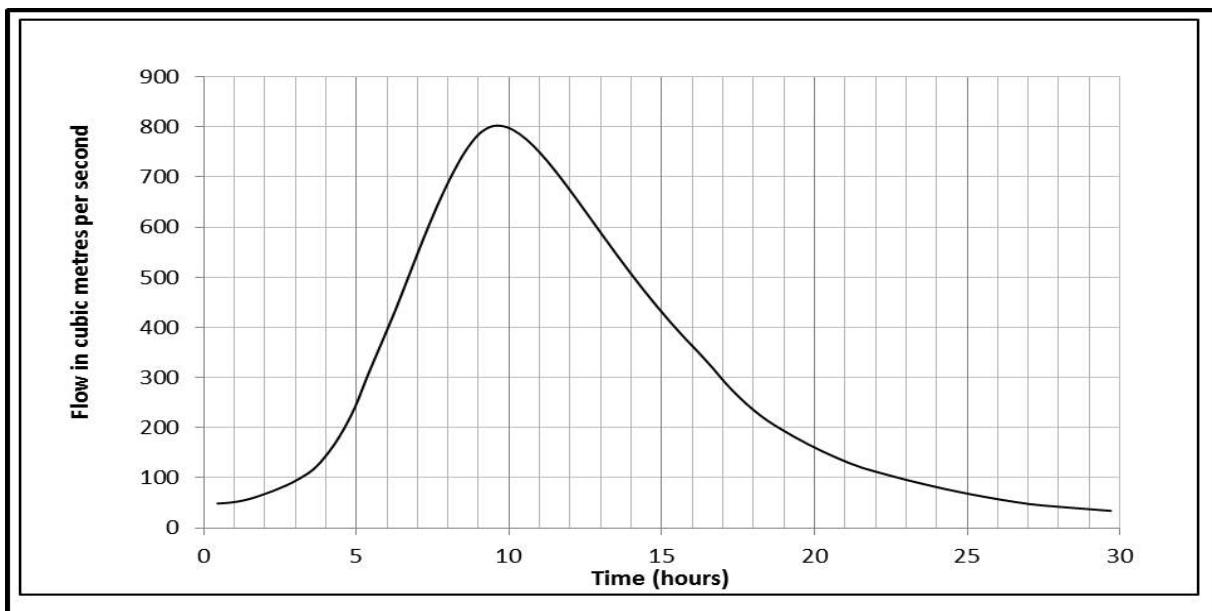


Figure 40: Design Inflow hydrograph for Great Brak Estuary

Table 12: Design floods adopted for this study

Return period	Peak flow in m ³ /s from	Peak flow in m ³ /reduced
	DFET	due to routing through dam
50	665	600
100	876	800
200	1090	1015

9.2 Boundary conditions: sea levels

Historical records have been used for calibration of the Mike11 model. For future flow predictions, information on sea levels has been accessed from the South African National Hydrographic Office publications. The relevant information is shown in Table 13. For the purposes of this study, the MHWS used. The level of the MHWS was referenced against MSL for the modelling.

Table 13: Tidal level indicators (m) at Mossel Bay (South African National Hydrographic Office (SANHO), 2013), relative to Land level and Chart datums.

Datum	MLWS	MLWN	ML	MHWN	MHWS	HAT

Chart datum	0.26	0.88	1.17	1.46	2.10	2.44
Land level datum	0.67	-0.05	0.24	0.56	1.17	1.51

Data on sea level was obtained from the University of Hawaii's website, which sourced it from the South African Navy gauge at Mossel Bay. The record available is from 1964 to 2012. Levels measured include tidal variation and storm surge. The gauge is located in Mossel Bay harbour and local effects due to currents, bay shape and bathymetry at the Brak River mouth are not taken into account. However, as records of the sea level at the river mouth are not available, and information to model the change in sea level due to these factors is also not available, the sea level was accepted to be the same as that at Mossel Bay. Before use in modelling, the sea levels were adjusted to take account of changing chart datum levels relative to MSL over time so that data would correlate with land surveys of the river cross-sections and estuary bathymetry. The flood model was calibrated using historical sea levels, with coincident historical riverine inflows. The Great Brak estuary is open to the sea at times, but at other times has a barrier across the mouth that prevents interchange between the ocean and the estuary. In the latter case the barrier acts as a dam, which may overtop under flood conditions, at which stage the barrier is breached and tidal effects can influence the water level in the estuary. The average monthly maxima and annual maximum levels from the historical data are shown in Figure 41. The sea level does not breach the 2 m mark within the record.

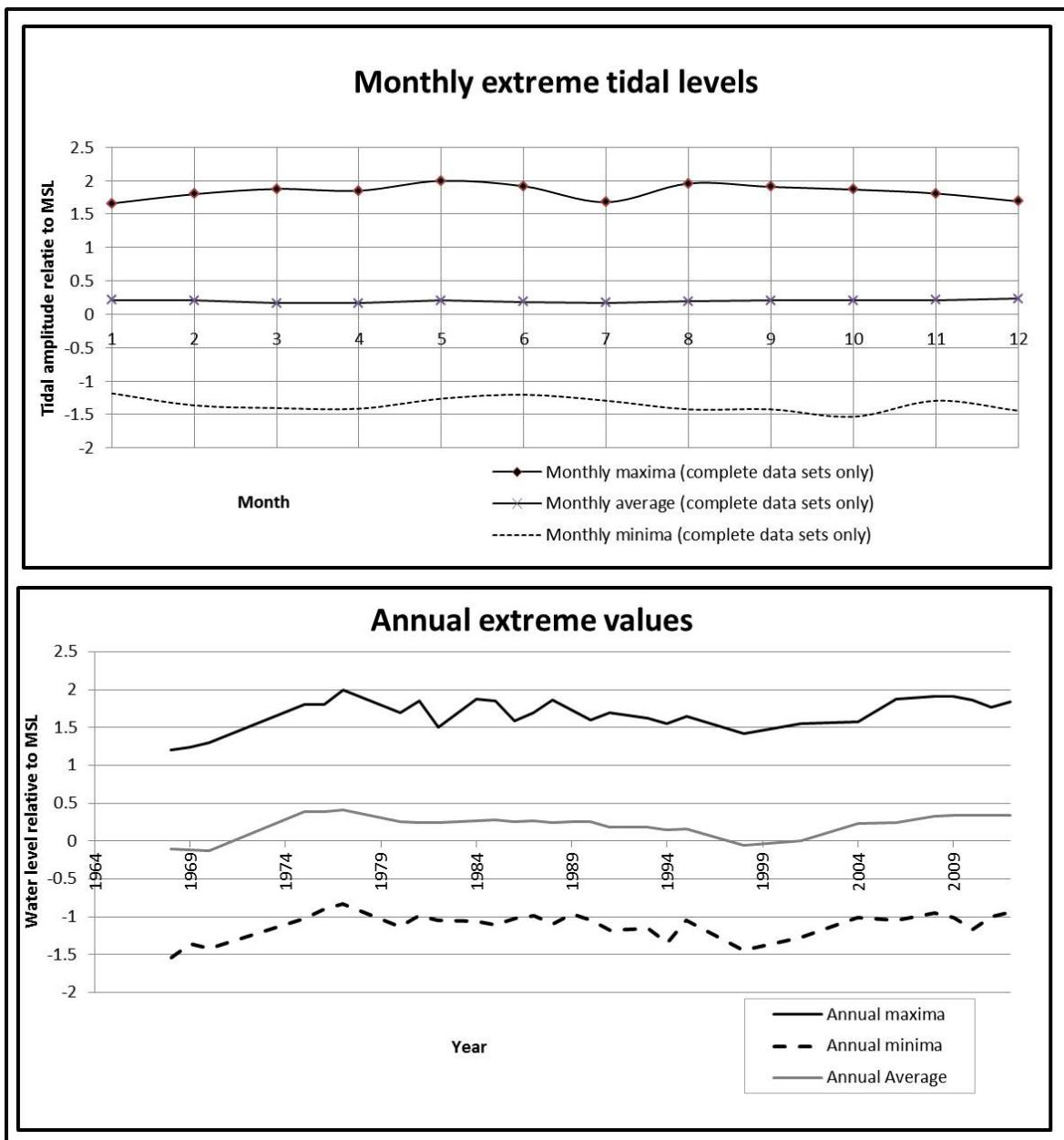


Figure 41: Monthly and annual minima, maxima and means for sea level from Mossel Bay gauging station. Prepared from data accessed from (University of Hawaii Sea Level Centre, n.d.)

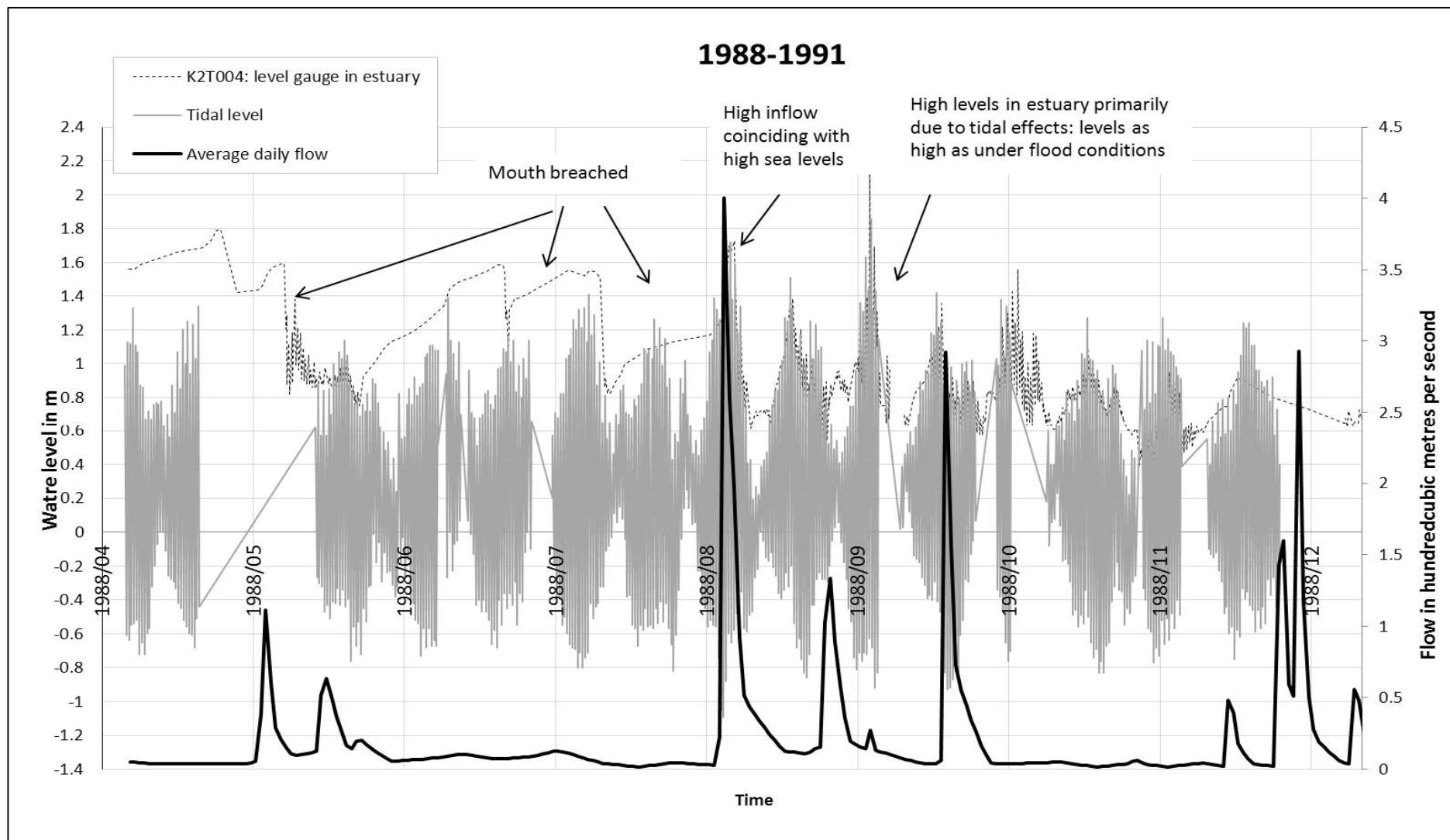


Figure 42: Tidal and inflow impact on water levels in the estuary - a snapshot

Figure 42 shows the effects of tide on the estuary water levels as well as the effects of the barrier. The gradual filling of the estuary while the barrier is closed can be seen clearly, as can the reduction in level following the breaching. When there is no inflow, the tidal effect is the determinant of water level in the estuary. Of interest in Figure 42 are the water level two peak. The first peak occurs where high inflow closely coincides with a very high tide (potentially an extreme high water level, or including storm surge). The tidal levels nevertheless dominates in this case. The second peak has very low inflow and is entirely due to tidal levels.

As was seen from Figure 42, sea level is a significant factor in high water levels in the estuary.

9.3 Design sea levels

For this study, the information in the preceding sections has been combined to provide a design sea level as given in Table 14.

Table 14: Design values for sea level adopted for this study

Sea level rise	0.333 m	(Intergovernmental Panel on Climate Change, 2014); (Theron & Mather, 2012)
Current storm surge	0.720 m	(Wijnberg, n.d.)
Storm surge rise	0.60 m	(Luger, March 2012; Theron & Mather, 2012)
Current MHWS	1.00 m	SANHO
Total design sea level	2.67 m	

Wind blowing across the sea and entering the estuary will lose impetus due to the headland and dunes at the mouth, such that the fetch will be limited to the lagoon extent, which is 1 km. Wind set-up within the estuary is not considered a significant factor for flooding on the Great Brak estuary. The Mike11 model was not run to include wind set-up.

9.4 River centreline and cross-sections

The river centreline was obtained from digitized information from the PLANETGIS data set, and imported into Mike11.

Input data on cross-sections and bathymetry was obtained from the DWA report²³, a survey provided by Gorra Consulting (Gorra Water: Kleynhans, 10 May 2010), bathymetry survey provided by CSIR (van Niekerk, n.d.), and the report on flood determination for the Great Brak river. Cross-sections from the DWA report (Department of Water Affairs, March 1990) were referenced locally to the survey stations and had to be placed by estimating their positions. A careful check of the levels provided on these sections showed a good match to available 1 m contour information in almost all cases. These levels were therefore adopted as relating to MSL. The cross-sections were extended, using available contour information, to ensure adequate coverage for design flood levels. Additional cross-section profiles were added to account for changes in flow at bends, and at structures.

Photographs of the 2007 flooding of the Great Brak were accessed, which showed the straightening of the flow path. Based on this, it was determined that additional cross-sections and centreline information should be used as model inputs. Altogether 81 sections were used, with a maximum spacing of 180 m.

9.5 Barrier considerations

The barrier acts as a dam, and is expected to raise the water level in the estuary. Where the barrier is intact the water level will reach the barrier level of between 1.5 and 2.0 m, and the flood will be superimposed on this as a starting water level. However, as the management plan for the estuary requires the barrier to be breached when a flood is expected, the model was run for the open mouth condition.

For modelling purposes, the barrier was treated as a fixed bed, using two levels to test different outcomes. The levels adopted are 0.0 m above MSL and 1 m below MSL. The latter is taken from the bathymetry data and is used to represent a fully open mouth condition. During calibration of the model, it was noted that the barrier level dropped after the peak flow, and that retaining the initial barrier level produced model water levels that were too high after the flood peak had passed. Ideally, the dam break capability of Mike11 should be used to model the change in barrier height during the flood. This study did not apply the dam break

²³ Provided by CSIR EMATEK

model. Instead, the record was split at the peak and two separate sets of parameters were run to obtain a complete flow hydrograph.

10 Challenges in modelling

Three main challenges were encountered in modelling the flow in the Great Brak River:

- Dealing with the meanders
- The treatment of sediment deposits and the meander
- Treatment of the three bridges at the upstream end of the estuary.

In respect of the sediment deposits, these were initially assumed fixed bed features. To accommodate potential scouring of sediment during floods, a branch was introduced to shorten the flow through the western meander. As the formation of a channel to shorten the flow between Searle's bridge and the N2 Bridge is probable, an attempt was made to introduce an eastern branch. However, introducing such a branch with a commencing bed level equal to that of the main stream, in addition to the western branch, destabilized the model. This is possibly due to the close location of the two branch exit points.

Sediment on right hand side (RHS) of Searle's bridge was retained as a fixed bed level. Sediments in the middle and o the left had side of the river, upstream of Searle's bridge, are assumed to wash away and are not taken into account in this modelling. This decision is supported by photographic records that indicate absence of sediments after a flood. The downstream sediment was assumed to be fixed. Flow was treated as branched to accommodate this island. This removed instability in the model at the Searle's bridge, and allowed the model to reflect accurately the hydraulic behaviour at the bridge, i.e.:

- Reduction of flow depth during low flow, and
- Overtopping during high flow.

Due to stability issues in the modelling, some modifications were made to cross-sections at Searle's bridge. These modifications are listed in Table 15.

Table 15: Changes to sections at Searle's bridge

Chainage	Modification
4013 and 4025	Section widened to accommodate the programme requirement sections upstream and downstream of bridges,
Above and below Searle's bridge	and culverts have a greater cross-sectional area than the structure openings

Modelling of the flow through the N2 bridge was challenging, as it would appear from contours that the river would tend (under flood conditions) to straighten and flow directly from Searle's bridge to the N2 bridge. However, photographic evidence indicates that the western channel develops first, such that the meander remains in place for lower return floods. The eastern depression appears to remain shallow and flood waters overflows the sediments as sheet flow, rather than forming a strong flow route between the two bridges mentioned. Ideally, the modelling should be undertaken to allow a branch to form high flow levels, scouring a channel at the far eastern edge of the floodplain between Searle's Bridge and the N2 bridge.

As the eastern branch would only start functioning once the floodwater reached about 2 m above MSL, a raised side level should be introduced here. As the model requires bed levels at a junction to be the same, it was not possible to model this off-take using a normal branch. An attempt was made to model the eastern branch using the link branch function of Mike11. This was unsuccessful. The model was therefore run ignoring the potential development of an eastern channel. An attempt was made to compensate for this by reducing the friction factor for the left bank, but the model was not sensitive to this reduction.

Dealing with the meander was also a challenge, since the depth of the scour hole measured is 3.8 below MSL, considerably lower than the adjacent bed levels and the lagoon bed levels (typically 0.7 m above MSL). As the meander is close to the three bridges, model instability resulted from the increase in bed level downstream of the meander. The abrupt change in the flow direction of floodwaters channelled along the N2 embankment to the bridge opening, is also expected to cause model instability.

The sediments in the estuary form a very uneven bed, which introduced downstream sections at higher levels than upstream sections. This was generally handled well by the model, but it caused instability in relation to the three bridges. The bed levels in the estuary

were therefore adjusted to introduce a gentler positive slope upstream of the bridges. Changes in bed level are shown in Table 16.

Table 16: Changes to sections in the estuary

Chainage	Modification
480 at Mouth. For calibration: Level raised to lowest level reflected on the estuary gauge	The calibration results indicated a constant difference with the estuary gauge readings. This was determined to be due to the level at the mouth. The bed level was therefore raised.

Challenges in modelling: bridges

The bridges were input according to the user specified bridge geometry option, rather than using the culvert function of Mike11. Some instability developed in the model due to the bridges, and the input was manipulated to address this issue. The energy equation was used both for flow through the bridges and for overtopping.

Searle's bridge

The cross-section of the bridge is greater than the up and downstream cross-sections due to sediment deposits upstream and downstream. This created instability in the model that was addressed by reducing the bridge cross-section to account for vegetated sediments near the abutments, and enlarging the upstream and downstream cross-sections to account for sediment removal under high flow.

It was anticipated that this bridge might overtop, so the modelling option for a submerged structure was selected.

The island downstream of this bridge is located close to the bridge such that steady state flow is not attained before the flow must divide around the island. This results in instability of the model. To deal with this problem, the island start chainage was moved slightly further downstream.

Three bridges

The three bridges downstream of the meander are too close together to allow modelling as separate entities, since steady state flow is then not achieved between the bridges. To address this challenge, the distance between the N2 Bridge and the other two bridges was increased. In the modelling, the N2 Bridge was treated as a single structure. It is not likely that this bridge will overtop, but the approach embankment has a low section at 3.5 m above MSL.

The secondary road and rail bridges were moved (see Table 17) to the position of the rail bridge and a composite structure was input with the following features:

- Common pier positions, but specifying a longer structure to encompass both bridges
- Soffit and parapet levels of the bridge from the rail bridge, which is slightly lower than the secondary road bridge.

The submerged option was selected for this structure. However, despite numerous attempts to create submergence conditions, the model did not reflect the vertical flow convergence that would indicate overtopping, even when flood levels were high enough to result in damming behind the bridge. It is thought that the solid side panel of the bridge, modelled as deck thickness, prevented overtopping. Where the flood level is below the bridge soffit, there is also little evidence of flow contraction in the vertical plane for any of the three bridges. It is thought that this is due to the water level below the bridge being much the same height as the level above the bridge, due to the downstream control of the tides and/or the barrier.

A key indicator of instability in the modelling, in relation to the three bridges, is that the water level falls below the bed level during modelling.

In the modelling, it was also found that the deep scour holes affected the stability of the model. As the scour holes represent a relevant part of the conveyance cross-section, they were retained in cross-section. Instead, to ensure stability of the modelling, a minimum water level was introduced in the hydrodynamic conditions (.hd file) for sections with negative levels. This measure removed all error messages with the exception of messages related to section 6888.14 located upstream of the N2 bridge, where the water level persistently fell below the bed level. This situation should not occur, given the levels indicated by the estuary gauge.

Various measures were introduced in an attempt to resolve this modelling error, including increasing the cross-section, raising the bed level, limiting the water level in the hydrodynamic file, adjusting downstream section levels to remove scour holes, and adjusting upstream section bed levels to raise them above that of section 6888.14. None of these measures met with success. It is thought that the problem arises due to the model being unable to handle the highly unsteady flow that would occur at this section due to

- its meander and the meander directly upstream
- the almost immediate narrowing through the bridges after the meander.

Table 17: Sections adjusted at three bridges

Chainage	Modification
6889, 7150 and 7309	Section widened to accommodate the programme requirement that sections upstream and downstream of
Above, between and below the three bridges	bridges and culverts should have a greater cross-sectional area than the openings in the structure

11 Model Calibration

The water level gauge in the estuary was used to calibrate the model. The model was calibrated by adjusting friction factors such that gauged estuary levels were obtained for associated historical inflow hydrographs and sea levels. Calibration was undertaken using ten historical events that guided selection of input criteria. A short period of record was selected where there was

- A period of very low riverine inflow, and
- A period of relatively high riverine inflow.
- A water level within the measuring range of the estuary gauge.

The calibration of the model was complicated due to a physical change in the barrier height from 0.8 m to 0.5 m during the peak flood, as seen from the historical records of the gauge in the estuary. For calibration purposes, barrier levels were taken as equal to the lowest level read by the estuary gauge for the period in question. It was necessary to split the calibration records at the peak flow, and to calibrate the “before peak” and “after peak” sections

separately using different levels at the mouth. It was further found that different friction factors had to be applied to calibrate the “before peak” and “after peak” flows.

Figure 43 shows the tidal level, the inflow, the estuary gauge level and the results of the calibrated model for the period **before** the peak flow when the barrier height is at the starting level. Calibration was undertaken using a barrier height of 0.8 m. Low friction factors (Manning n values) were applied to the estuary as these allowed the maximum alignment between tidal patterns. The low velocity in the estuary and the sandy bed indicate a low friction factor, even on meanders in the estuary.

Figure 43 shows the tidal level, the inflow, the estuary gauge level and the results of the calibrated model for the period **after** the peak flow when the barrier height is reduced. The post flood peak barrier height was taken as 0.5 m, although the height reduced further to 0.5 m after a second inflow with a lower peak.

Many iterations of the model were undertaken with the friction factor higher on the floodplain, and none would provide results correlating with the estuary gauge heights. An explanation may be that the high friction factor of the river accommodates the losses in the meanders and the pooling of water upstream of the three bridges, while flow of the floodplains is fairly unimpeded and the bed conditions comprise of sand, mud and short grasses.

The model output was assessed for expected effects using constant inflow, specifically, the effect of the bridges and the flow profile. The bridge profiles entered into the model are shown in Figure 44 to Figure 47.

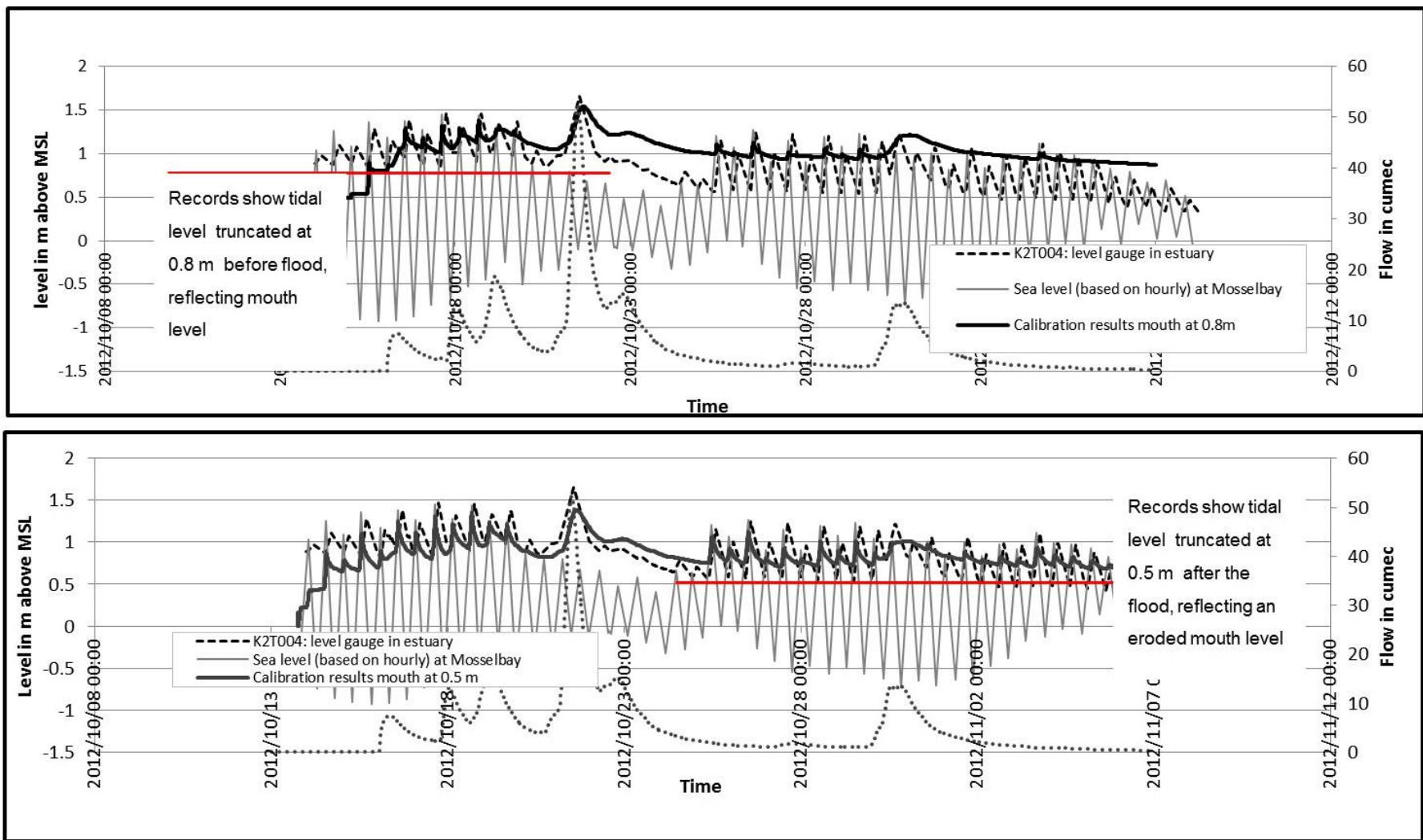


Figure 43: Different base levels for tidal effects in the estuary before and after a flood (from historical level data). Calibrated flow results are superimposed

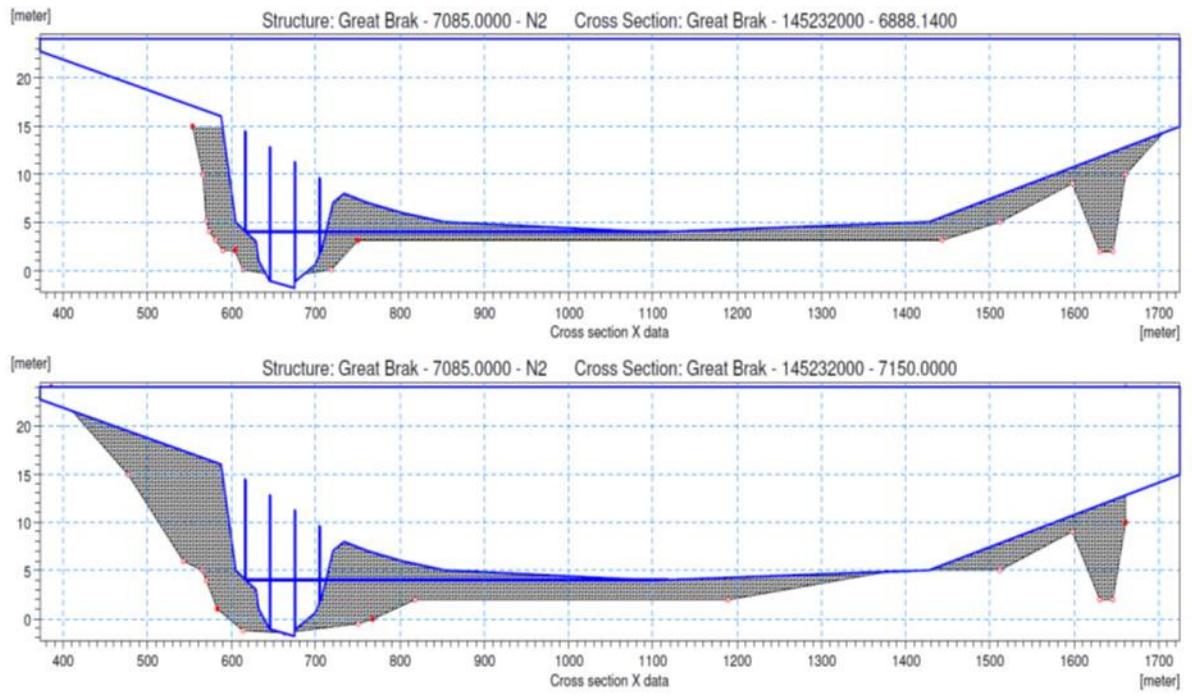


Figure 44: N2 bridge profile. The level at which the embankment overflows is input as the soffit level

A very thin deck is specified to allow the flow through the bridge to be relatively unobstructed by this "deck"

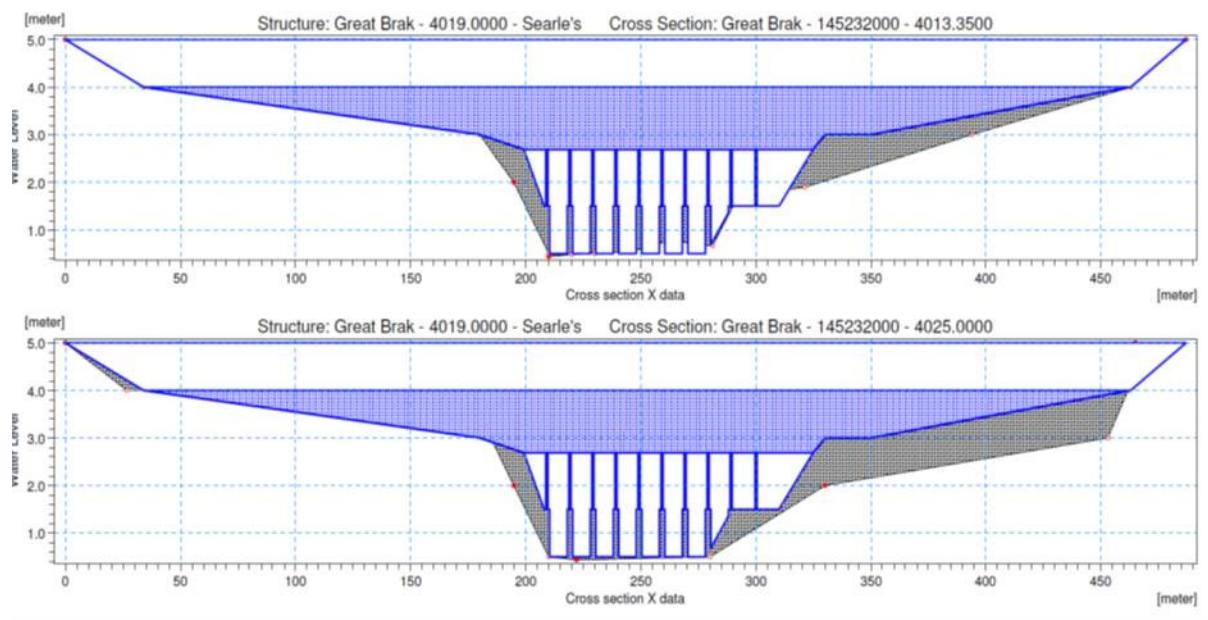


Figure 45: Composite structure representing the secondary road and the rail bridge

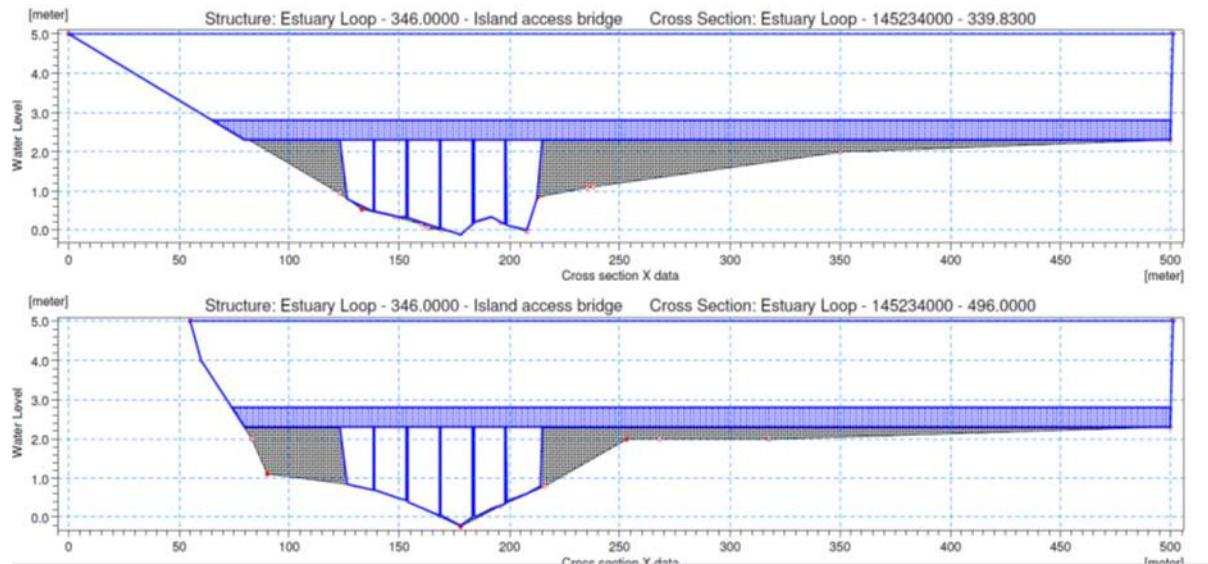


Figure 46: Searle's' bridge profile. The rails on the bridge are assumed to block under flood conditions, resulting in a thick bridge "deck"

The soffit level of the rail bridge (lower soffit) was selected and combined with the highest level on both bridges. There is a resulting thick "deck". Pillars of the two bridges were aligned.

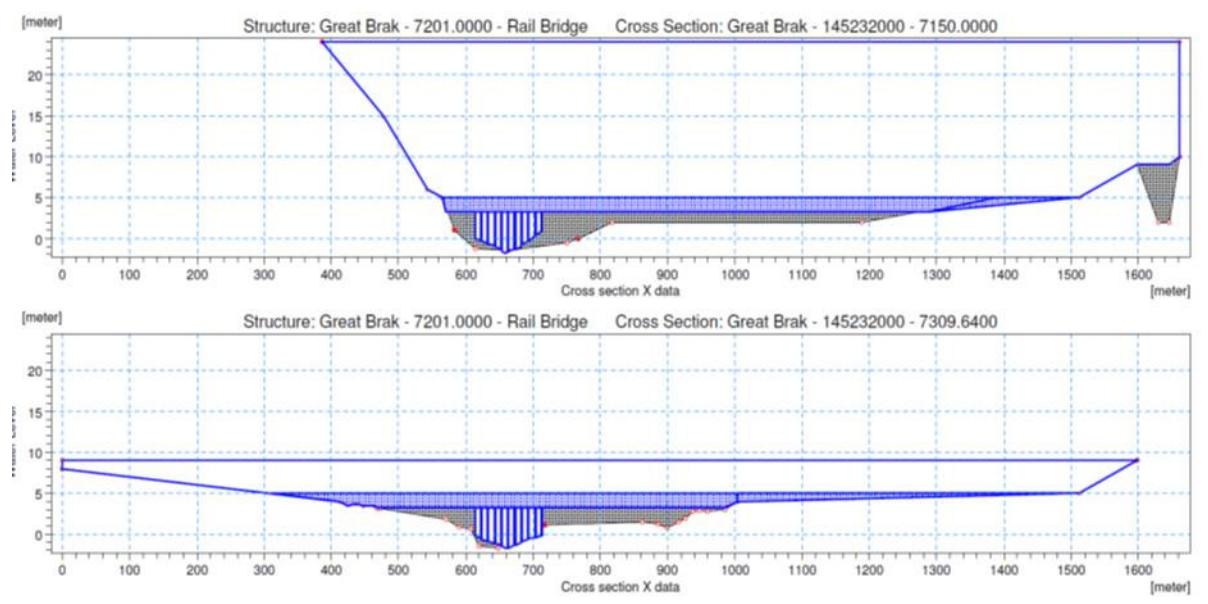


Figure 47: Access Bridge at the Island.

The response of the model to the bridges was checked for consistency with expected open channel flow behaviour. Figure 48 to Figure 50 show the model output for the bridges. The blue area indicates flow depth. The red line shows the maximum flow level attained during the modelling. The grey line shows the left bank elevation. Bed and water levels are shown

on the vertical axis and chainage, measured from Wolwedans dam towards the estuary mouth, on the horizontal axis.

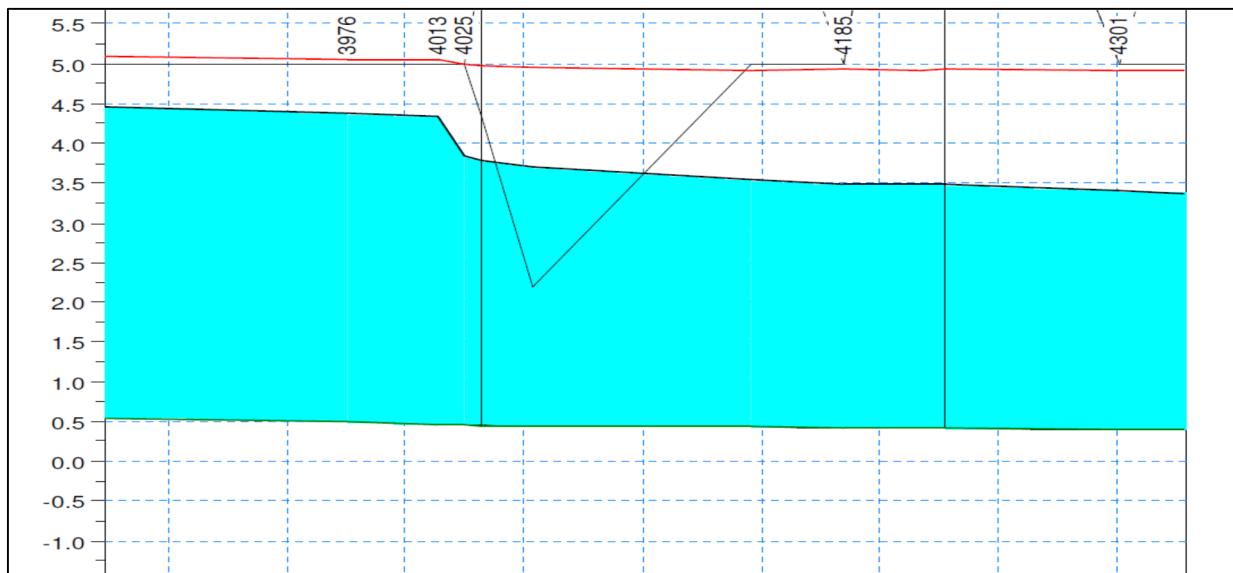


Figure 48: Results at Searle's bridge align with expectation. There is a build-up of water behind the bridge, which overtops, and a reduction in depth of flow downstream

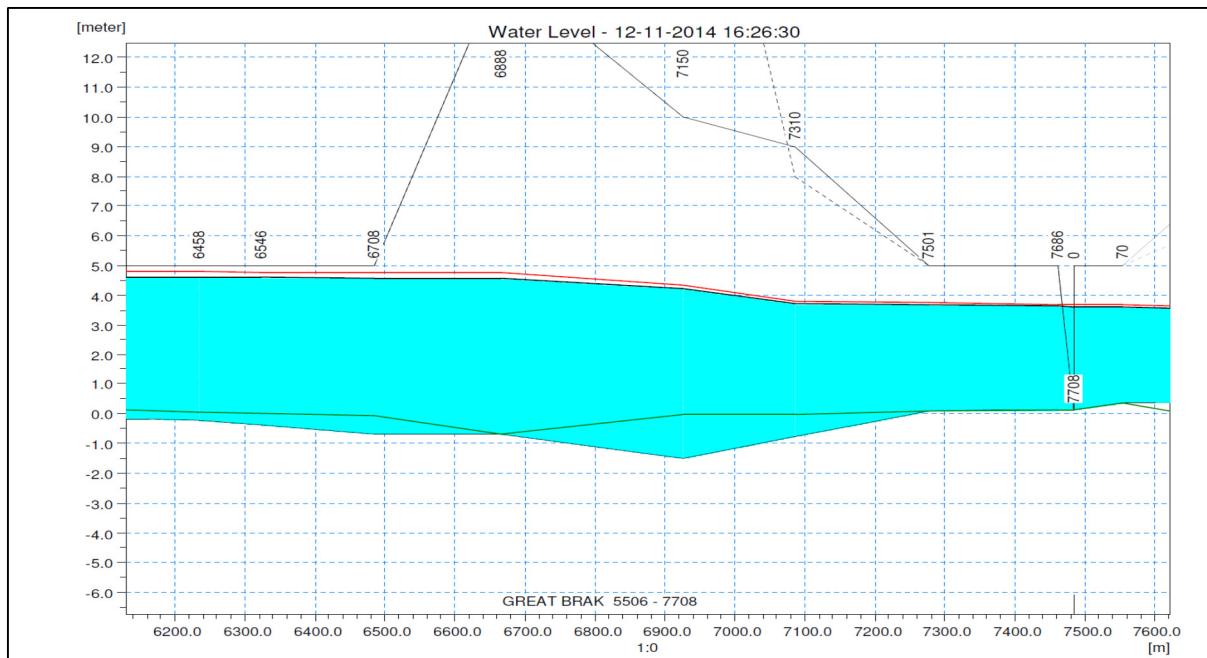


Figure 49: The effect of the three bridges on the water profile.

The effect of the three bridges is not as clearly defined in the model output as for Searle's bridge, possibly because of the high downstream water level. The vertical contraction in flow is potentially more the result of the increase in downstream bed level than energy loss through the bridge openings.

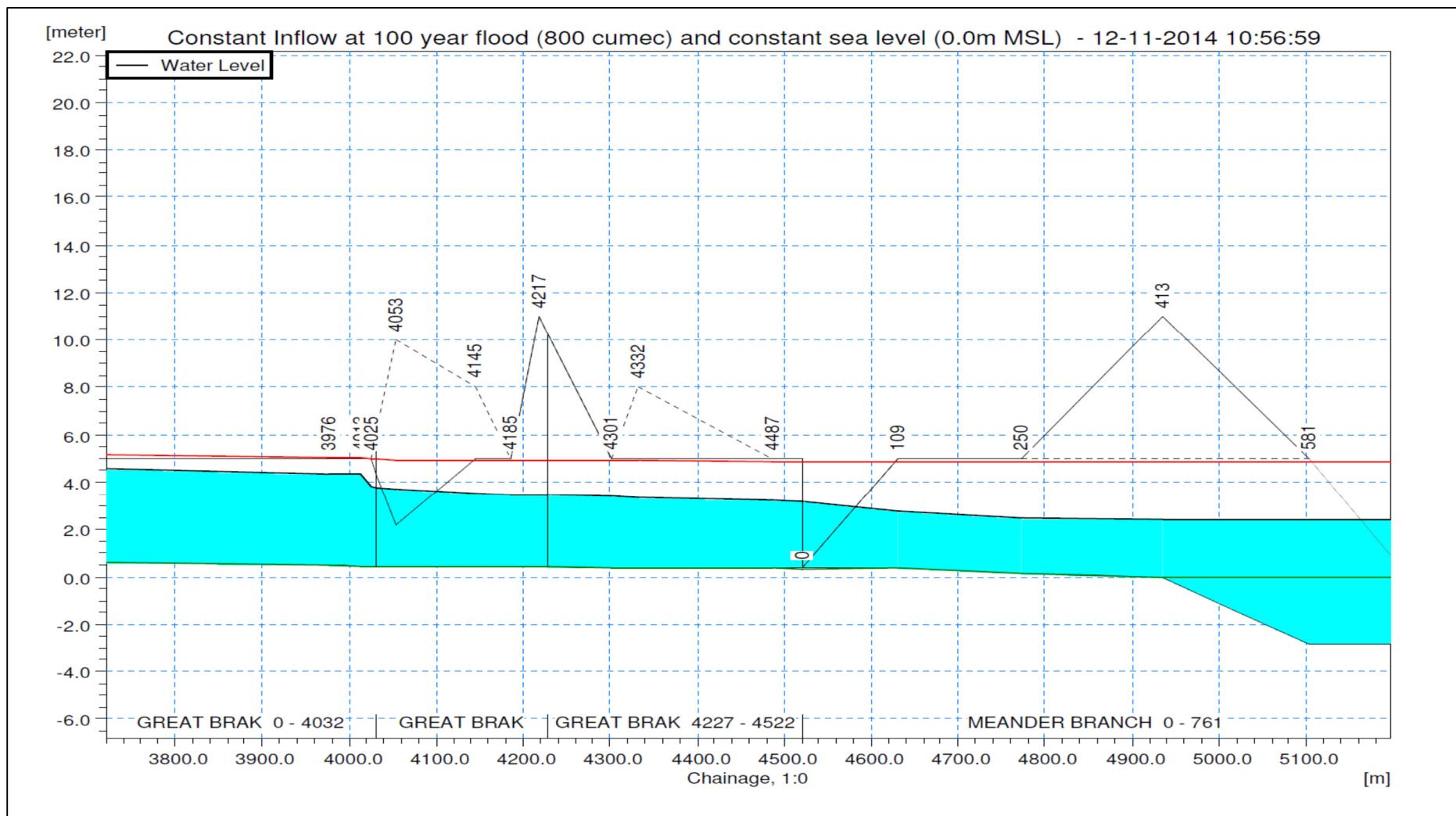


Figure 50: The section reflects a reduction in depth of flow from Searle's bridge towards the second meander bend, at chainage 6888.0. This section is through the branch channel that is shorter than the adjacent meander, thus steeper, resulting in a higher flow rate and a smaller flow depth.

12 Results

12.1 Effect of barrier height on flood levels

Figure 51 shows the flood levels for barrier levels from 1 m above MSL to 4 m above MSL. The modelled water profile is shown only to just past the three bridges. Barrier heights under current conditions have exceeded sea level heights. As an example, the barrier has at times reached 2.7 m MSL to date, while sea levels for the period have not exceeded 2 m MSL. It is therefore possible that the barrier height will, in the future, also exceed the still water sea level. For that reason, a 4 m barrier height was chosen as potentially occurring under extreme sea level rise. For these flood model runs the sea level has been kept constant at 0 MSL. The different flood levels generated are therefore entirely a function of riverine flow and barrier height.

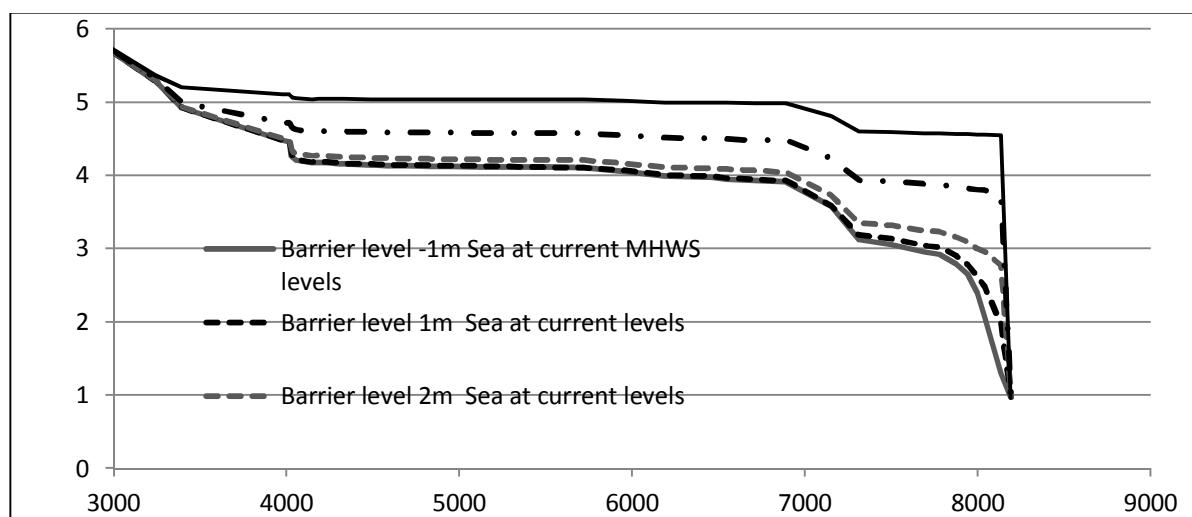


Figure 51: Results of modelling: the effect of increasing height of the barrier at the mouth.

Figure 52 and Figure 53 show the water level profile that develops across the estuary because of the barrier and the raised bed formed by the extensive sediments inside the mouth, blocking the west channel. In the modelling, the raised bed acts as a broad-crested weir and reduces flow depth. In reality, this effect will be greatly reduced by scour of the sediment and of the estuary mouth.

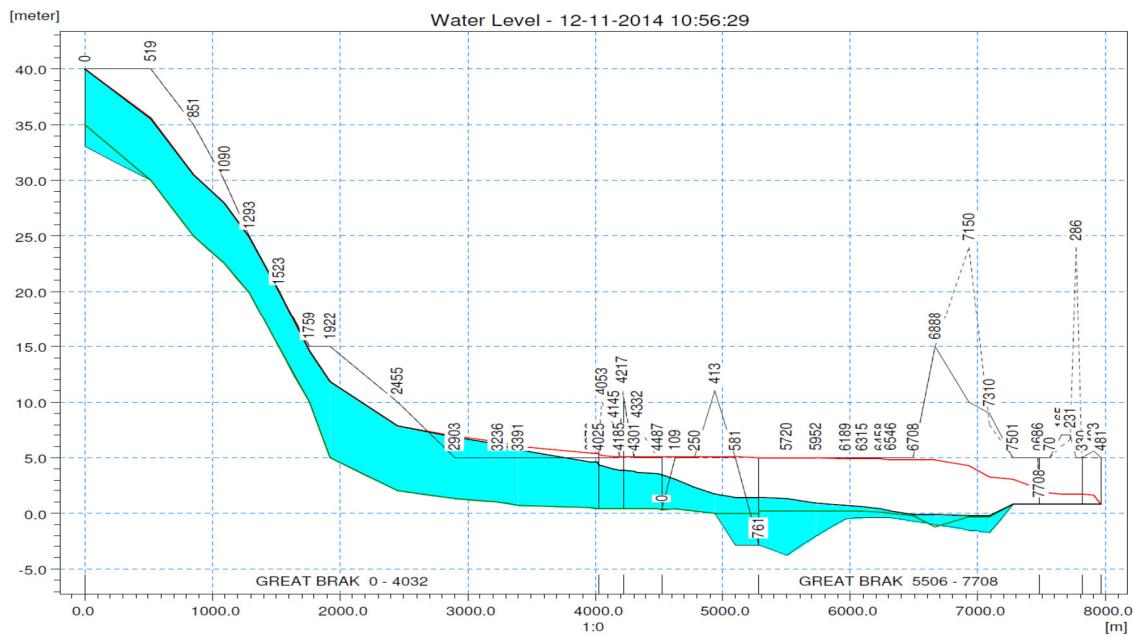


Figure 52: The steep profile of the inflowing tide is shown for the estuary mouth fixed at 0.8 m and constant inflow. The mouth level is not yet reached by water in the estuary

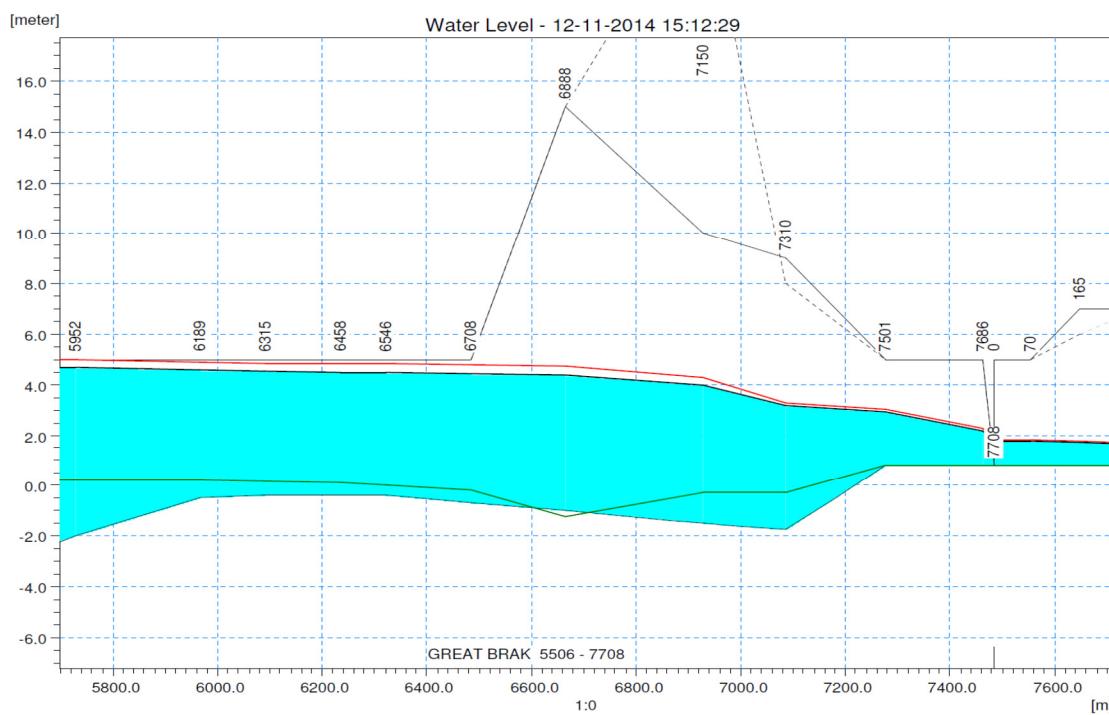


Figure 53: The counter-intuitive drop of the water level at the mouth is shown here to be due to the weir effect of the fixed mouth level at 0.8 m, and the assumption of a bed silted up to the mouth level.

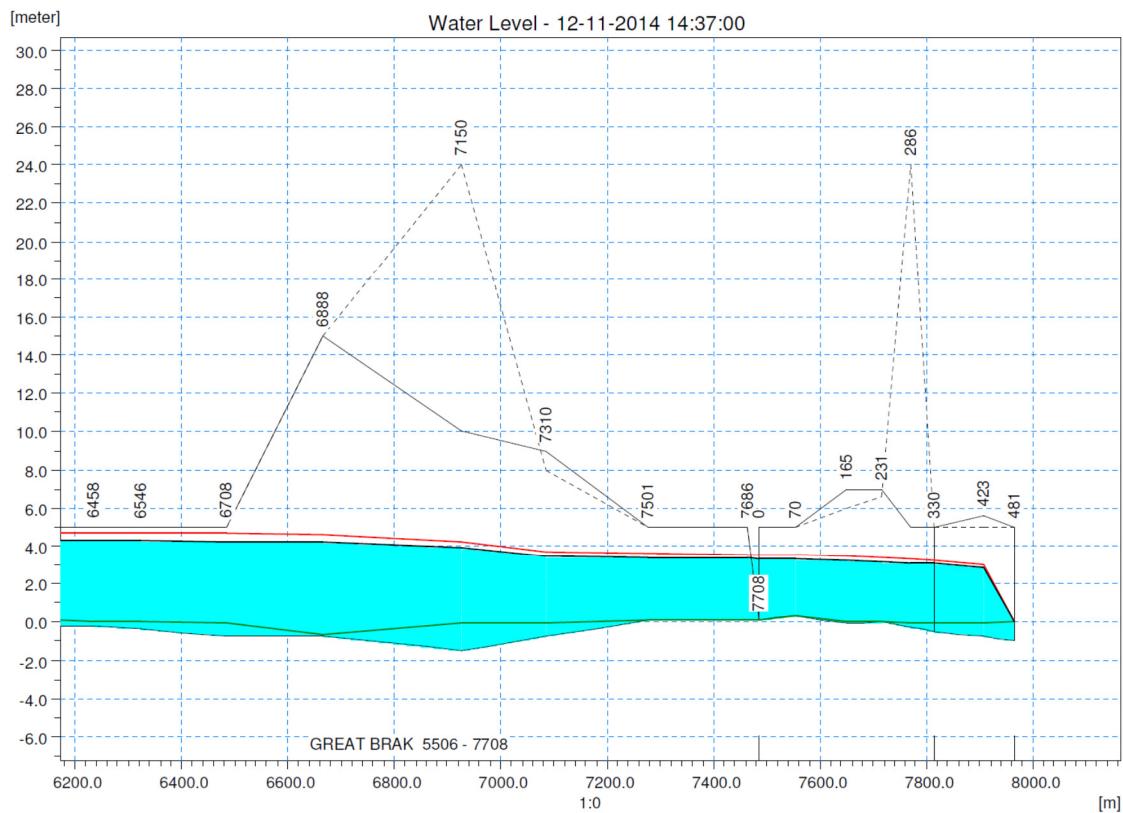


Figure 54: The more usual flow profile expected where the mouth level is set at 0:0 m and does not act significantly as a barrier to flow.

12.2 Flood levels under increased sea level

Figure 55 shows the results of the modelling for the 1:100-year riverine flood for sea levels at 0.98 m (current Mean High Water Spring (MHWS)) and 1.31 m (predicted future MHWS) above current MSL. The sea level rise adopted for this scenario is therefore 0.33 m. The modelling was undertaken for the mouth level at -1 m below current MSL and at current MSL. The resultant increase in flood levels are

- For the mouth level at -1 m below MSL: a flood level increase of 0.6 m, to 2.25 m, near the mouth of the estuary
- For the mouth level at 0 m MSL: a flood level increase of 0.8 m, to 2.5 m, near the mouth of the estuary

The effect of increased sea level is dampened further inland, such that there is no change in the flood level at chainage 7300, about 800 m inland from the mouth of the estuary.

The effect of increased sea level is dampened further inland, such that there is no change in the flood level at chainage 7300, about 800 m inland from the mouth of the estuary.

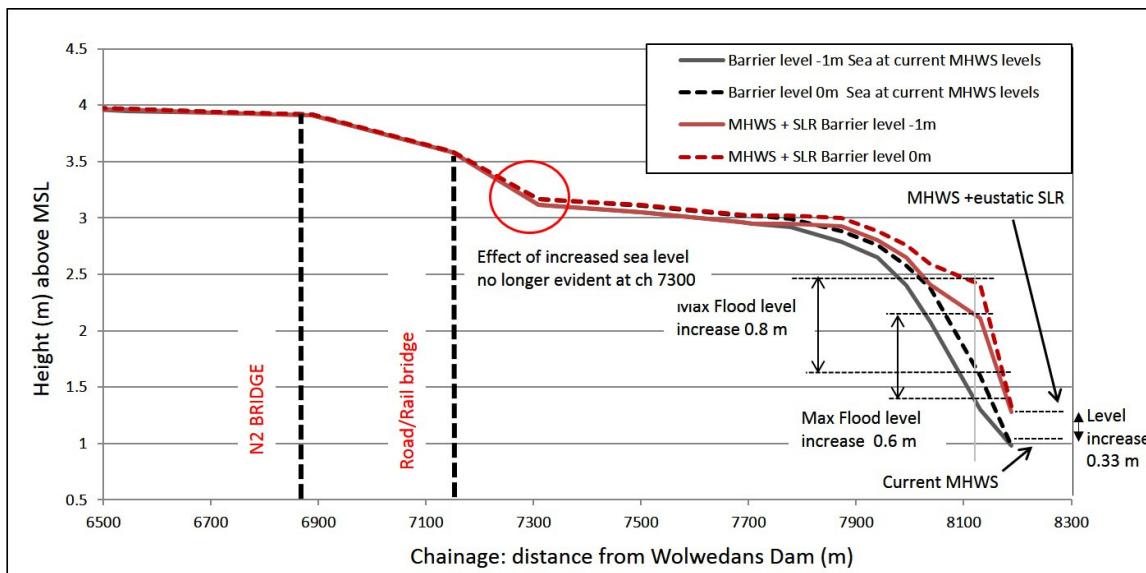


Figure 55: Result of Mike11 modelling showing the effect of sea level rise on flood levels

Figure 56 shows the flooded area under the current 100-year flood. The flood level is at 2.65 m and does not overtop the N2 road embankment. Figure 57 shows the extent of the flooding for the Mean High Water Spring and sea level rise, given a 100-year riverine flood, which results in a flood level of 3.5 m at the N2 Bridge, reducing to 2.5 m at the mouth. It can be seen from Figure 57 that the N2 road embankment, as well as the secondary road and rail embankments, are overtopped at this stage. On the Island, a much larger area will be flooded than would be flooded under the current 100-year flood levels (with no SLR). The extent of flooding is shown only to the N2 road embankment. Above this it can be seen, from relative location of the 3 m and 5 m contours, that the entire floodplain becomes inundated once the flood level reaches 4.0 m. Due to the steep sides of the floodplain, even flood levels at 3 m above MSL can cause extensive flooding and inundation of housing areas adjacent to the N2 embankment (western bank of the lagoon). Upstream of the N2 Bridge, developments to the west of the western meander bend will be submerged.

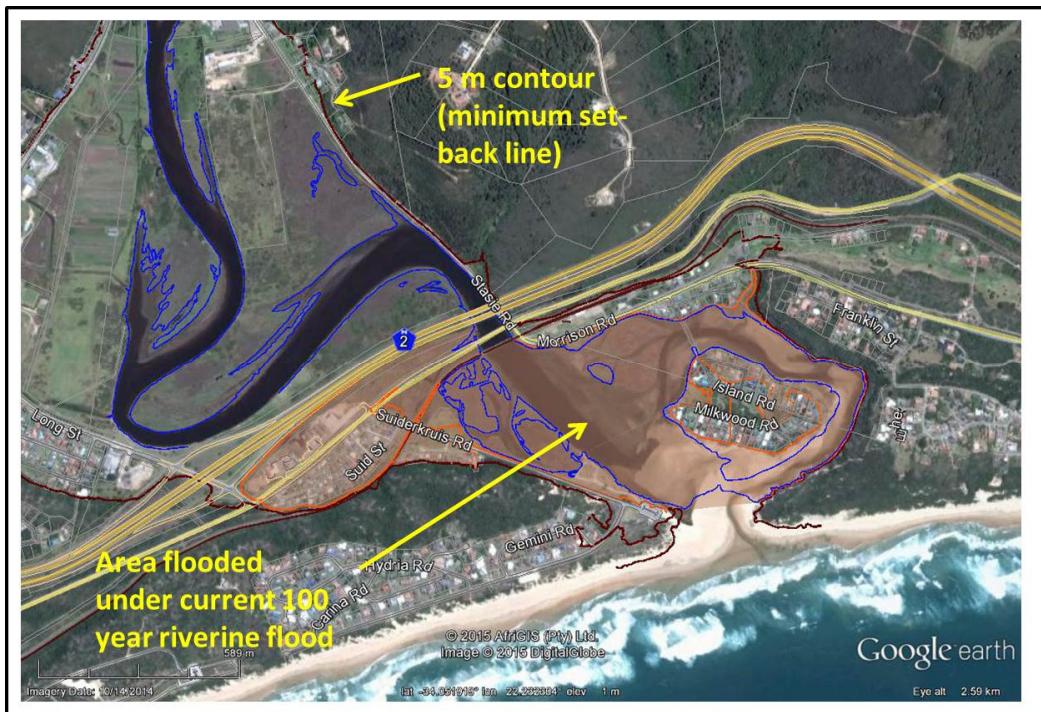


Figure 56: Area below the N2 Bridge inundated under the current riverine 100-year flood. (Adapted from Google Earth)

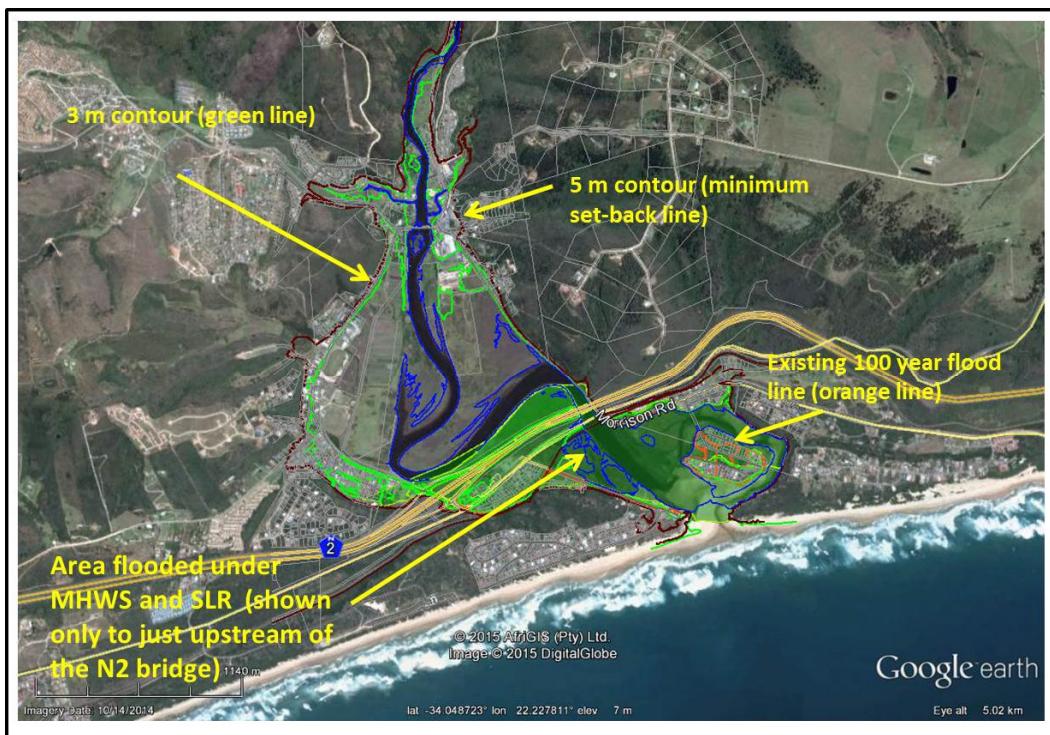


Figure 57: Area inundated under MHWS and Sea Level Rise, coinciding with 100-year riverine flood (Adapted from Google Earth)

Figure 58 shows the results of the modelling for the 1:100-year riverine flood, for sea levels at 1.72 m (current MHWS plus storm surge) and 2.65 m (Sea level rise plus MHWS plus predicted future storm surge). The sea level rise adopted for this scenario is therefore 0.934 m above a current sea level scenario where MHWS occurs coincident with storm surge. The modelling was undertaken for the mouth level at 1 m below current MSL and at current MSL.

The resultant increase in flood levels are

- For the mouth level at -1 m below MSL: a flood level increase of 0.8 m, to 3.16 m, near the mouth of the estuary. This rise is relative to the flood level before climate change, at chainage 8000
- For the mouth level at 0 m MSL: a flood level increase of 0.5 m, to 2.94 m, near the mouth of the estuary. This rise is relative to the flood level before climate change, at chainage 8000

While the effect of increased sea level is dampedened further inland than for the case where only sea level rise is taken into account, the evident effect of sea level rise stretches well beyond the 6500 chainage mark (over 1.6 km inland from the mouth). At these sea levels, with an open mouth, the waterside properties on the Island will be flooded to a depth of between about 1.2 m above the island ground level.

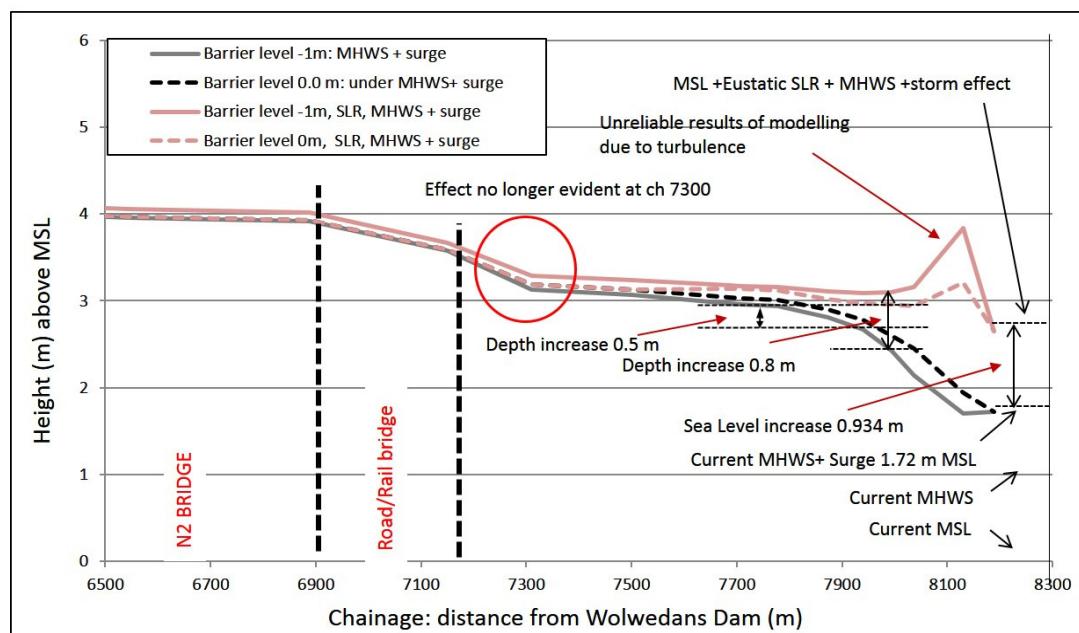


Figure 58: Comparison between current flood levels modelled with MHWS and with surge, against flood levels modelled with Sea level rise, future MHWS and future surge. Barrier levels at -1.0 MSL and 0 MSL

At this stage, the model outputs started showing a peak in levels at the barrier. This is assumed to be a modelling artefact rather than an accurate reflection of levels. These inaccuracies could result when the tidal inflow is at its peak and the riverine outflow is at its peak because considerable turbulence will occur across the barrier. This behaviour cannot be modelled using the theory on which Mike11 is predicated. The model outputs for water levels at the mouth have therefore been neglected in the interpretation of the flood levels generated by the programme.

Figure 59 shows the results of the modelling for the 1:100-year riverine flood for sea levels at 1.72 m (current MHWS current storm surge) and 2.65 m (Sea level rise plus MHWS plus predicted future storm surge). The sea level rise adopted for this scenario is 0.934 m. The modelling was undertaken for the mouth levels at 2 m and 4 m above MSL. The 2 m value was chosen as the highest level at which the barrier is artificially breached. The 4 m value was chosen as a potential barrier level on the basis that current sea levels have raised the sand barrier at the mouth to 2.7 m above MSL at times, while the maximum sea level measured is under 2 m above SL. Historical records show that the barrier can be built above the maximum still water level of the sea. Therefore, 0.93 m increase in sea level (including storm surge) could potentially raise the barrier level by at least the same height, giving a potential barrier height of 3.63 m. This amount has then been rounded up to 4 m for purposes of modelling.

The resultant increase in flood levels are

- For the barrier level at 2 m above MSL: there is a flood level increase of only 0.16 m under a sea level rise of 0.93 m above a current sea level scenario where MHWS occurs coincident with storm surge. The resultant flood level is 2.9 m near the estuary mouth, and 3.38 m at the three bridges. The resulting flood level is above sea level (under sea level rise, MHWS and storm surge), but only 0.16 m higher than the flood level before sea level rise. This reflects the dominance of the riverine flood in determining flood levels.
- For the barrier level at 4 m MSL: no flood level increase is observed under sea level rise and the level for both before and after sea level change is 4.5 m above MSL. This result is expected as the barrier height is above sea level and only riverine flooding influences flood levels in the estuary.

Should the barrier reach 4 m in height then properties on the banks of the Island will be submerged up to 2.5 m. However, these results should be interpreted in light of the management practice of breaching the barrier at between 1.5 and 2.0 m above MSL. This

level of flooding would therefore only be expected if there were a failure to breach the barrier.

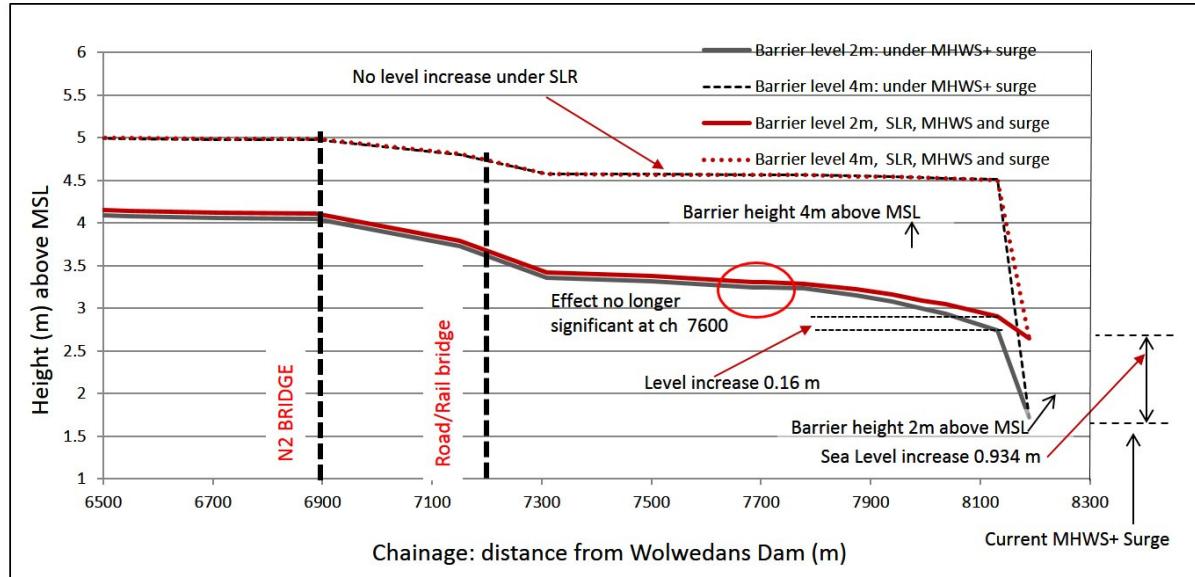


Figure 59: Comparison between current flood levels modelled with MHWS and with surge, against flood levels modelled with Sea level rise, future MHWS and future surge. Barrier levels at 2.0 MSL and 4.0 MSL,

13 Conclusions

The purpose of this study was to assess the adequacy of the 5 m MSL contour given the expected increase in sea levels under climate change, using Great Brak estuary as a case study. The literature reviewed indicated that climate change will affect through increased flooding from higher sea levels and increased flooding from more intense land storms; through changed sedimentation, typically an increase in sedimentation; and through changes in mouth conditions. Both open and closed conditions are expected to lead to increased flooding: the former when the tidal prism moves into the estuary, and the latter when a barrier at the mouth blocks flow, forming a dam.

This study was limited to testing the effect of sea level rise and the effect of increased barrier heights. Modelling tested two sea level conditions: mean sea level (for the current situation, and in 2090), and sea levels under combined sea level rise MHWS and surge (2.65 m above current MSL). The study also considered the influence of a barrier at the mouth on the flood

levels, combining, where sea levels exceeded barrier levels, the effect of both sea level and barrier height on flood levels in the estuary.

The overall conclusion is that sea level rise will have an effect on flood levels in estuaries. However, the influence of the increased sea levels does not extend much beyond the N2 Bridge. The most noticeable effect is with a few hundred meters of the mouth, after which the effect moderates considerably. This may be a peculiarity of the Great Brak estuary, due to the influence of the three bridges and the road and rail embankments.

It is of interest that the potential effect of a combined MHWS and future storm surge, with SLR, will produce a flood of over 3 m at the mouth of the estuary, increasing to 4 m upstream of the N2 Bridge. This situation will occur in the estuary if the mouth is open at the time of the extreme sea event (MHWS and surge, taking account of sea level rise), coincident with 100-year riverine flood).

From Figure 59, the water level in the estuary can be seen to reach as high as 4.52 m downstream of, and 5 m upstream of, the N2 Bridge, should the barrier build up to 4 m high. This is not necessarily an extreme case as the barrier currently reaches up to 2.7 m above MSL at times, more than a meter above current HAT, and a meter above a hypothetical current event with MHWS coinciding with storm surge. Wave set-up has not been considered in this study; neither has increased runoff due to more intense rainfall or sedimentation of the estuary bed.

MHWS coincident with an extreme storm has been described as a 500 year event, with the potential of occurring once in 18.6 years in the future under the influence of climate change (Midgley, et al., October 2007). The probability of such an extreme sea level event occurring at the same time as peak runoff of a 100-year riverine flood is unlikely.

It is therefore the conclusion of this study that, for the Great Brak River, the 5 m setback line, as prescribed, is sufficient for an extreme situation where a future 100-year flood coincides with an extreme sea level event where the MHWS coincides with an extreme storm (raising the sea level to 2.65), including eustatic sea level rise.

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ANNEXURES

15 Annexure A: Uncertainty in IPCC findings

Uncertainty in IPPC findings

AR 5 The uncertainties associated with the findings of the Assessment are indicated through the probability and level of confidence by the IPPC. "Confidence" (expressed as *very low, low, medium, high or very high*) is a function of robustness and extent of data, and the extent of agreement on the findings, thus providing an assessment of the validity of a finding. The probability is an outcome of the data analysis (IPCC 4, 2010). Terminology used in IPPC reports to express probability as follows:

Term	Probability	Term	Probability
Virtually certain	99-100%	More unlikely than likely	0-<50%
Extremely likely	95-100%	Unlikely	0-33%
Very likely	90-100%	Very unlikely	0-10%
Likely	66-100%	Extremely unlikely	0-5%
More likely than not	>50-100%	Exceptionally unlikely	0-1%
About as likely as not	33-66%		

(IPCC 4, 2010; Mather, May 2014)

16 Annexure 2: Design flood determination

16.1 Catchment and river features

The catchment is long and narrow; starting in mountainous reaches at an elevation of 1350 m (Surveyor General, South Africa, Accessed August 2014). The river drains a catchment of 188 km² as measured on a GIS system²⁴. The middle reaches of the river flow through gentler terrain where agriculture (and formerly irrigation) is practised. The lower reaches are very flat. The total river length is measured at 29 km in PLANETGIS²⁵. Above Wolwedans dam, the river is called the Groot River, which has three main branches, the Twee River, the Varings River and Perdeberg River. The Varings River flows into Wolwedans dam. The Groot River starts at an elevation of 950 m and has a very steep channel for the first 4 km. Thereafter the channel slope is moderate, dropping 400 m over 14.5 km to Wolwedans dam. Directly below Wolwedans dam wall the channel falls 30 m over 5 km, after which the slope becomes very flat, falling only 5 m over 3.5 km. The rivers are shown in Figure 60, and their contribution to the flow given in Table 18.

The river typically flows throughout the year. Mean annual runoff at Wolwedans dam is given as $18,19 \times 10^6 \text{ m}^3$ for the period 1961 -1980, but it is indicated that MAR is quite variable, ranging from $4,3 \times 10^6 \text{ m}^3$ (1979/80) to $44,5 \times 10^6 \text{ m}^3$ (1962/63) (EWISA, 2014)

Rainfall in the upper reaches is in excess of 800 mm per annum declining to between 600 and 800 mm per annum in the middle reaches. In the lower reaches, rainfall is less than 600 mm per annum. Mean annual precipitation is shown in Figure 61.

²⁴ Using catchment boundaries provided from state agencies through PLANETGIS and National Spatial Development Plan datasets

²⁵ 28,5 km (EWISA, 2014)

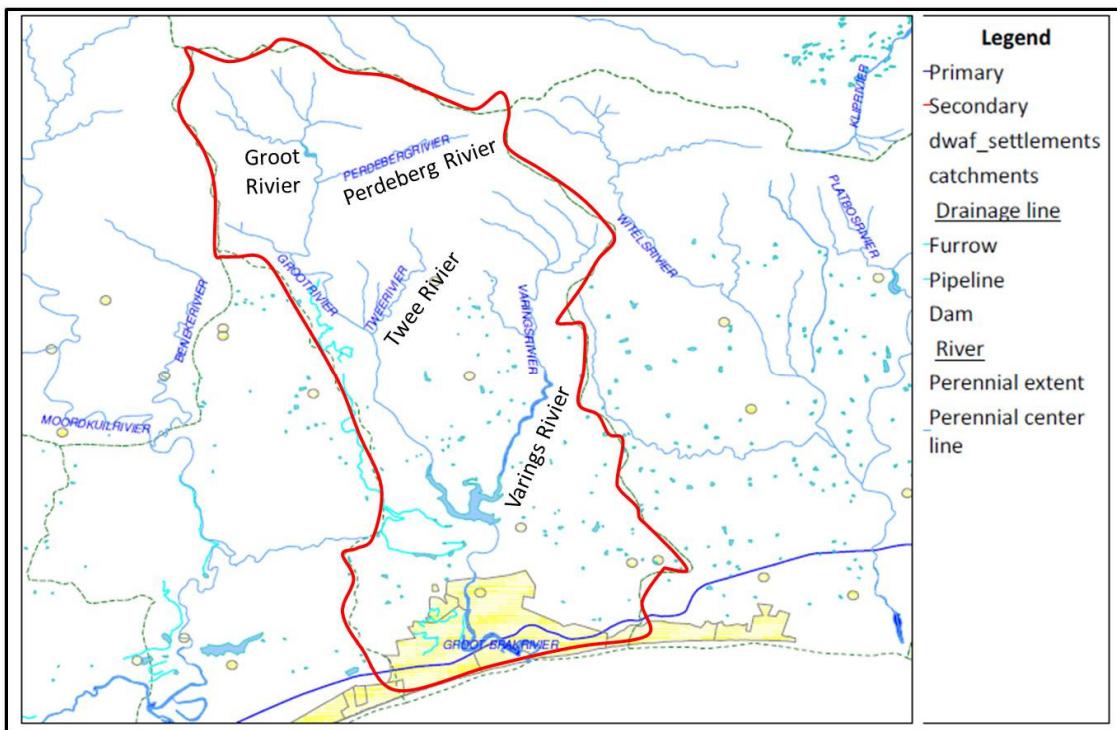


Figure 60: Rivers in the Great Brak Catchment (Agricultural Research Commission, n.d.)

The catchment is functionally divided into the area above Wolwedans dam, with an area of 131 km² (Department of Water Affairs, South Africa, n.d) and the area below the dam. The total area to the catchment mouth is 188 km² from GIS²⁶.

Table 18: Contribution of tributaries to runoff at Wolwedans (Water Institute of South Africa, n.d)

Tributary	% contribution to runoff
Perdebergrivier	23 %
Tweeriviere	25 %
Varingsrivier	31 %
Unnamed stream “D1”	13 %
Unnamed stream “D2”	8%

²⁶ 192 km² to estuary mouth (EWISA, 2014)

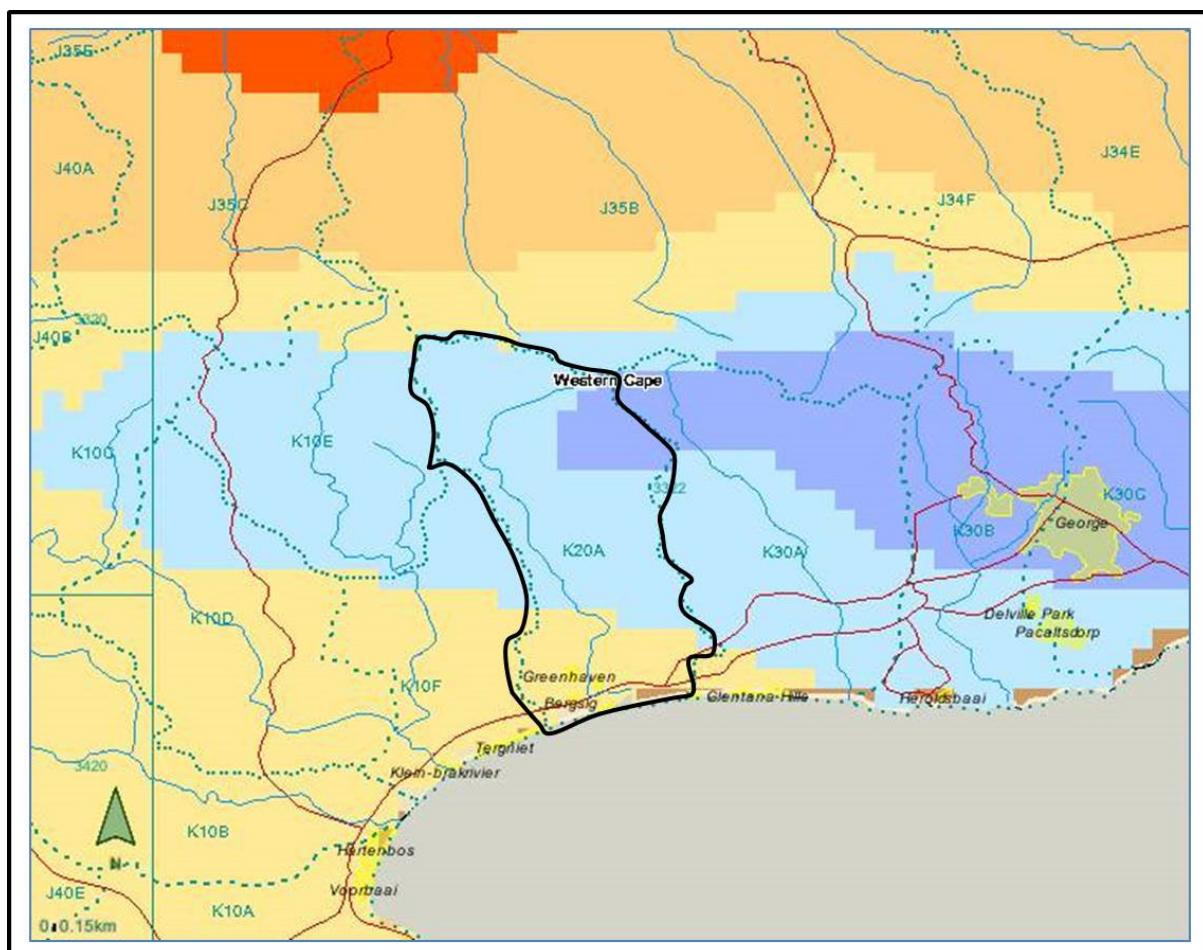


Figure 61: Mean Annual Precipitation for Great Brak Catchment (Agricultural Research Commission, n.d.)

Forestry, Water Affairs and Weather Services weather stations are plentiful in areas around the catchment, but only two stations are located within the catchment (both in the rainfall areas above 600 mm per annum. Figure 62 shows the location of the weather stations. Only weather stations included in the South African Weather Services (SAWS) database (Water Research Commission, n.d.) quoted in Gericke (n.d) were used in the rainfall-runoff modelling, which includes Forestry weather stations but excludes most of the Department of Water Affairs and Sanitation weather stations. The SAWS data was used in preference to the TR102 data (Adamson, 1981; Kovacks, 1988; Van der Spuy, 2010; Patel, 2014; Alexander, March 2002; Alexander, 2002; Simthers, n.d) as the SAWS data had a 20 year longer period of data.

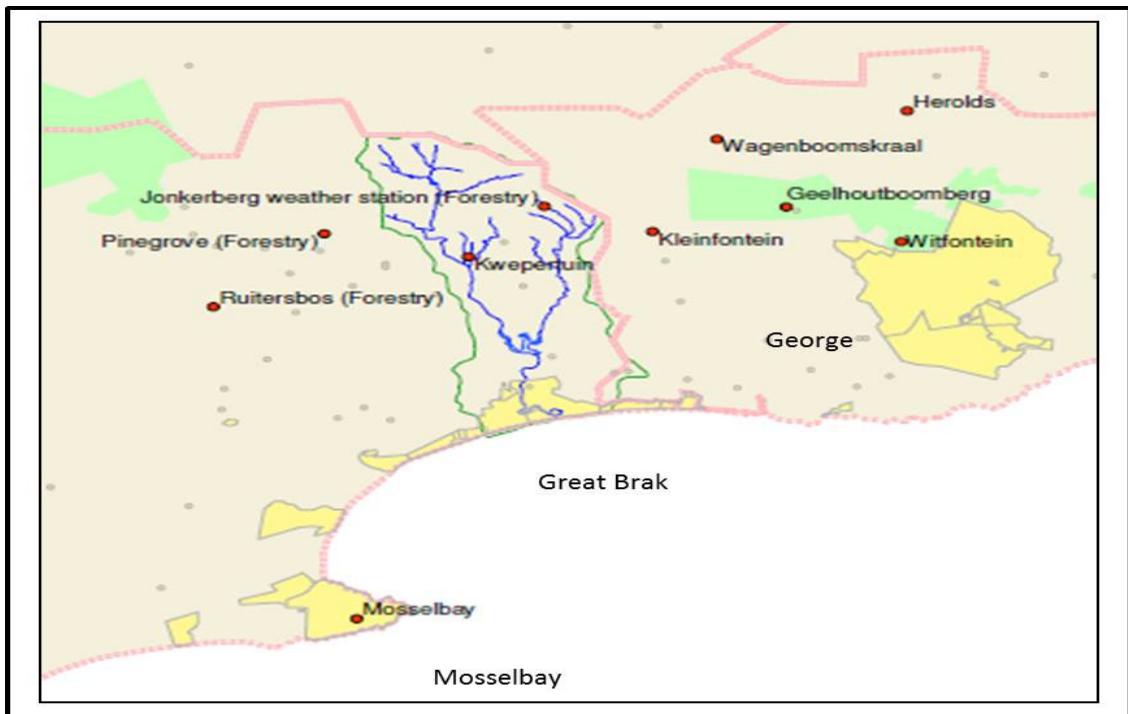


Figure 62: Weather station in and adjacent to the Great Brak River catchment.

Land uses in the catchment, required for modelling of rainfall-runoff for various methods (e.g.: the Rational, Alternative Rational and Soil Conservation Services (South African National Roads Agency Limited (SANRAL), 2007; Team1, 2010, December 15) methods) were abstracted from the AGIS system of the Department of Agriculture and measured in a GIS. The uses are listed in Table 19. The dominant veld types for the Great Brak Catchment are temporal forest and brush (Acocks, sourced from AGIS), as shown in Figure 63.

Flow is gauged at a number of weirs and at the Robertson and Wolwedans dam walls. Historical gauging information was used to calibrate the flood model, but was not used to predict future extreme events since the gauge information reflects flow after attenuation by upstream dams. As the worst-case scenario is that, the peak design flood will occur when Wolwedans dam is full, it was necessary to determine the peak design flood from rainfall-runoff modelling.

Table 19: Land Uses in Great Brak River Catchment

Land use ²⁷	Above Wolwedans dam	Catchment total	Land use ²⁸	Above Wolwedans dam	Catchment total
Agriculture	32%	35%	Scrub/ Brush: Winter region	21%	34%
Open Space	0%	4.5%	Woods	0%	2%
Forest	45%		Residential: Lot size: 1000 m ²	0%	2%
Impermeable	2%	1.5%			

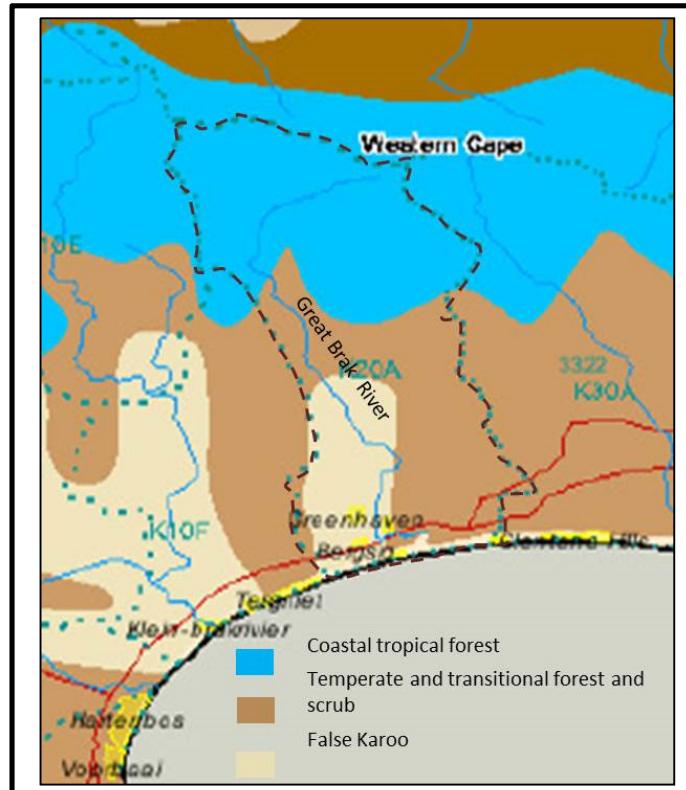


Figure 63: Acocks Veld types for Great Brak (Agricultural Research Commission, n.d)

²⁷ Land uses were aggregated from AGIS information to align with categories required for rainfall-runoff modelling²⁸ Land uses were aggregated from AGIS information to align with categories required for rainfall-runoff modelling

16.2 Design Flood Determination

Discharge records are available upstream of the estuary at Wolwedans dam, for the period 1961 to 2104, at gauge K2H002 (Department of Water Affairs, South Africa, n.d.). As there are minimal abstractions between the dam and the estuary, these records can be used to represent inflow in the estuary from the catchment above Wolwedans dam. However, these records are not a representation of peak floods as the flow value and duration is moderated by change in storage in the dam. As a broad confirmation that the discharge gauge data at K2H002 would exclude certain peak flows and reduce others, and therefore provide an incomplete record, a comparison was undertaken between rainfall and K2H002 discharge. A comparison was undertaken for the period 1991 to 2002 for monthly readings. Monthly readings were considered because the statistical analysis uses monthly maxima. As an example, Figure 64 provides a plot of these monthly readings. From these records, it is clear that the discharge measured at gauge K2H002 does not reflect all the expected peak flows.

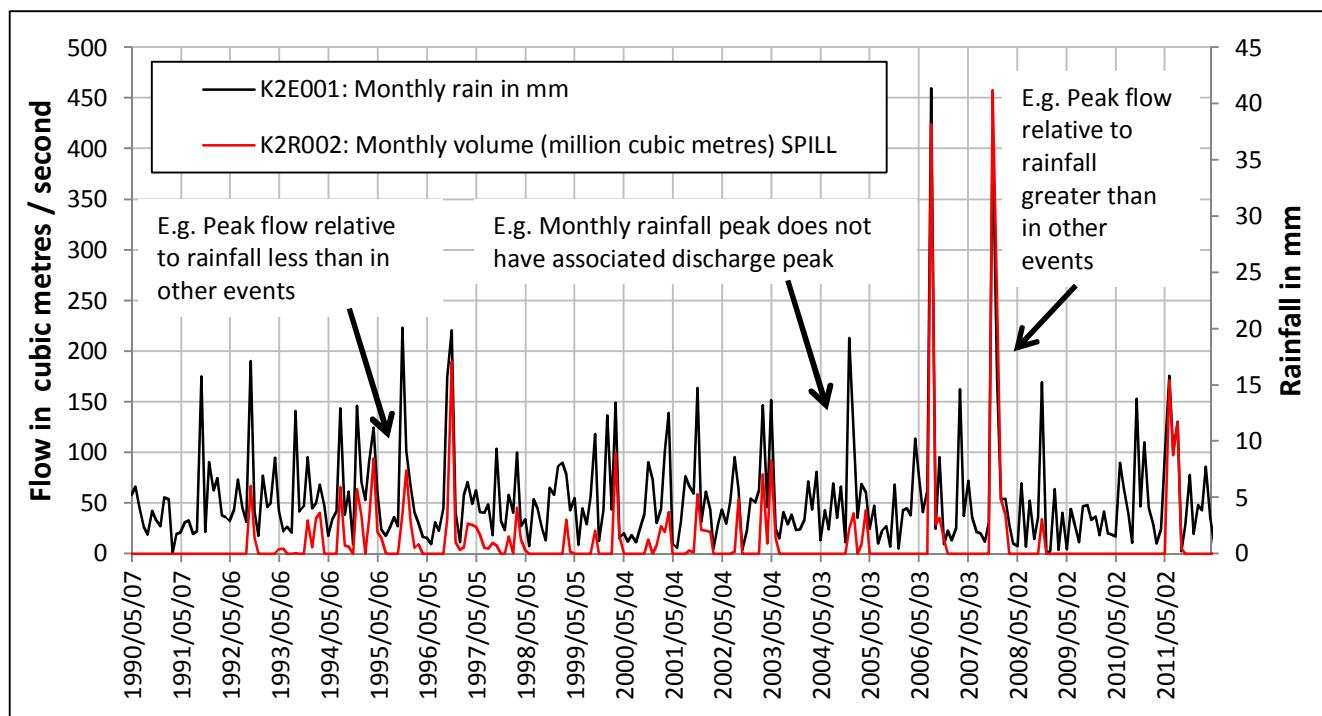


Figure 64: Comparison of peak rainfall with K2H002 runoff records. Data source (DWA Hydrological Services Surface Water (Data, dams, Floods and Flows), n.d.)

As the gauge information did not provide the necessary information, it was necessary to predict the runoff for extreme events using rainfall-runoff methodologies. The methods recommended by the Department of Water Affairs (Van der Spuy, 2010) and set out in the Road Drainage Manual (South African National Roads Agency Limited (SANRAL), 2007) were used. Flow was routed through Wolwedans dam to produce an appropriate peak and flood period reflecting the moderating effect of the dam. Design flood was calculated assuming the worst-case scenario for flooding in the estuary where Wolwedans and Ernest Robertson dams are full at the time of the peak flood, such that the full volume of the flood thus passes through the dam.

The deterministic and empirical methodologies set out by DWA (Van der Spuy, 2010) and SANRAL (South African National Roads Agency Limited (SANRAL), 2007) were used to predict maximum rainfall, design storm duration and associated runoff before selecting floods of the design return period. Statistical methods were not used for the reasons indicated above.

16.3 Time of concentration

The time of concentration T_c for the catchment was determined from the equation

$$T_c = T \left[\frac{0.87L^2}{1000S_L} \right]^{0.385} \quad \text{where } L \text{ is the main watercourse length and } S_L \text{ is mean channel slope using the 10:85 method}$$

The above US Bureau of Reclamation formula is based on the Kirpich formula (Van der Spuy, 2010). The Kerby formula was not used because the catchment is steep. DWA recommends the Taylor-Schwarz method (Van der Spuy, 2010) of determining the channel slope, which is scientifically more accurate as it averages the actual slopes from the respective river reaches. For the Great Brak the use of the Taylor Schwarz method of determining the channel slope results in a significantly larger T_c of 7.5 hrs, against the 4.6 hrs resulting from the 10:85 method and 5.1 hrs when using equal-area method. This significant time difference results in considerably lower peaks when the Taylor-Schwarz methods are used, with longer hydrograph base. This applies to peak flow values for the

Rational, Alternative Rational, SCS (Soil Conservation Service) and SDF (Standard Design Flood) deterministic methods. The Synthetic Unit Hydrograph is not affected as the method uses basin lag rather than time of concentration or channel slope. The lag-routed hydrograph methods are not affected if the determination is made on veld type rather than on time of concentration. Given previous work by DWA and Gorra Water, using the 10:85 method, this method has been adopted.

The final time of concentration for the catchment will be longer than determined by this formula because the arrival, at the estuary mouth, of the flow from the catchment above Wolwedans dam is delayed due to the attenuating effects of the reservoir. The adjustment to the time of concentration was made by estimating the additional time for the water to traverse the reservoir, using the modified PULS method (University of Asia Pacific, n.d.), and adding this additional time to T_c as determined above.

The resulting time of concentration for the catchment is 5.0 hours. For the purposes of predicting design flood $T_c = 5.0$ hrs was used.

16.4 Runoff calculations

The Design Flood Estimation Tool of the University of Stellenbosch (Gericke, n.d) was used to determine design floods. A two-step process was followed:

- Determination of the design flood to Wolwedans dam using the $T_c = 5.0$ hrs (rounding up the 4.6 hrs established from T_c). The routed hydrograph peak occurred 0.45 hrs after the inflow hydrograph peak.
- Determination of the flood hydrograph for the whole catchment using the $T_c = 5.0$ hrs (4.6 hrs original T_c plus 0.45 hrs additional time from lagging of flow through Wolwedans dam).
- Reducing the hydrograph peak for the whole catchment by an amount equal to the reduction of the peak of the flow through Wolwedans dam.

In this way the response time of the estuary is set equal to the time when the whole catchment is contributing and the flood peak takes into account flow that arrives at the estuary mouth at that time.

The use of two separate hydrographs for the catchment above and below the dam could be considered if the response times for the catchment below and above the dam are set equal

to the response time for the overall catchment. If response times are calculated for each catchment individually then overall T_c will be lower resulting in a higher design rainfall, and a consequent over-estimation of the flood peak.

16.5 Design flood to Wolwedans dam

The Design Flood Estimation Tool of the University of Stellenbosch was used for the flood determination. The Tool uses the methodologies recommended by the Department of Water Affairs and set out in detail in the SANRAL Road Drainage Manual. A summary of the results is provided in Table 20 and

Table 21, and shown in Figure 65.

Table 20: Summary of results of deterministic methods for design flood estimation for the catchment above Wolwedans dam

DETERMINISTIC METHODS							
Return period	Design flow (m³/s)						
	(T, years)	Rational	Alternative Rational	SCS	SDF	Unit Hydrograph	Lag-routed Hydrograph
2	104		91	63	30	85	214
5	152		163	146	106	117	295
10	204		227	225	176	150	378
20	267		297	322	256	193	473
50	366		392	482	375	248	623
100	474		476	631	475	308	775
200	640		646	808	581	348	878

Table 21: Results of empirical methods of design flood estimation for the catchment above Wolwedans dam, from DFET (Gericke, n.d)

EMPIRICAL METHODS				
Return period	Design flow (m ³ /s)			
(T, years)	Midgley & Pitman (MIP)	Catchment parameter (CAPA)	Q _T /Q _{RMF} -ratio	Regional Maximum Flood (RMF)
2		45		Francou-Rodier Kovács
5		89		
10	144	129		
20	178	179		1787 1793
50	233	261	704	
100	274	324	907	
200		384	1136	

As can be seen from Figure 65 and

Table 21, the different methods produce widely varying results. The results below 500 m³/s were considered to be too low given the catchment size and response time, and results above 1000 m³/s considered to be too high.

For the purposes of this study, the design flood of the lag-routed hydrograph method was adopted, as a hydrograph is required to model the inflow. The 100-year flow is thus 775 m³/s. To account for the flow attenuation through Wolwedans dam, this was reduced somewhere need to explain study approach early in the report – overview of methodology to 660 m³/s. As a hydrograph is sought for the modelling for this study, the lag-routed hydrograph was adopted. The results for the Synthetic Unit Hydrograph (SUH) and the lag-routed hydrograph are shown in Figure 66.

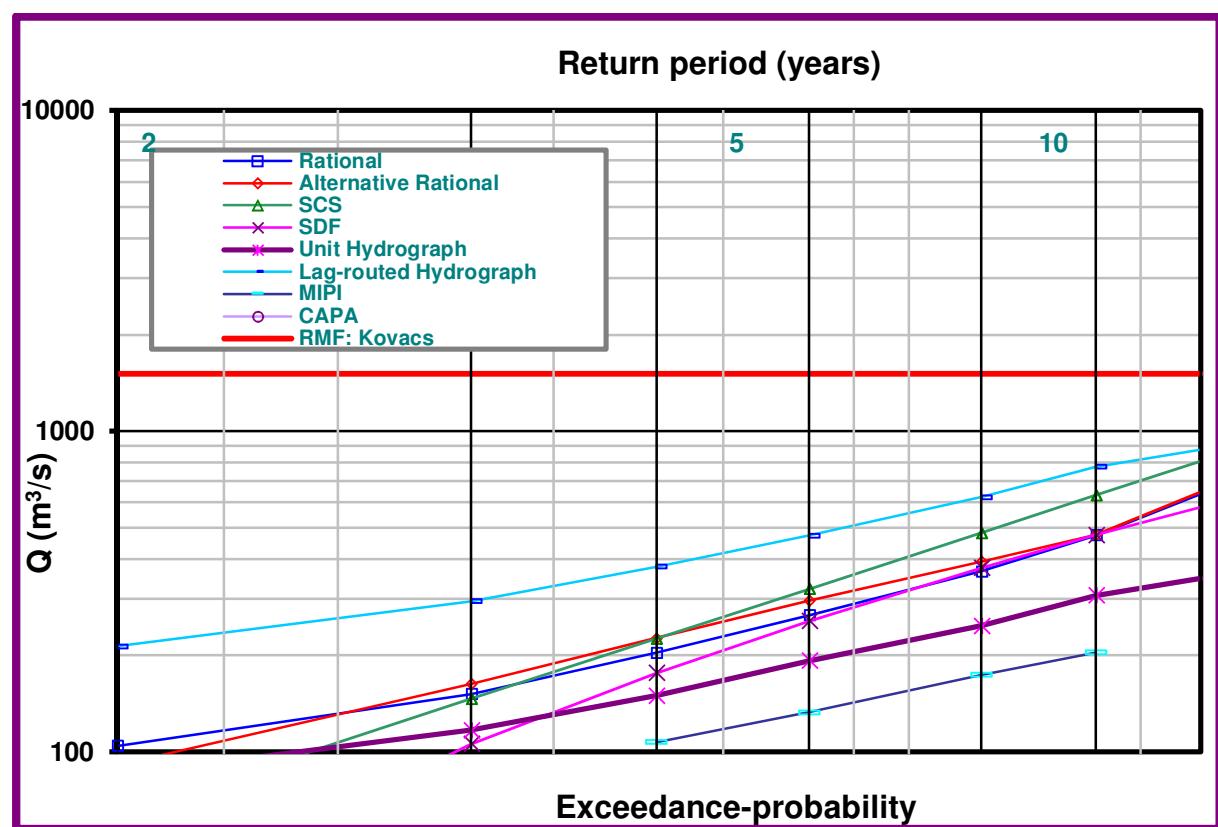


Figure 65: Results of various methods to determine flood peak: catchment to Wolwedans dam (Gericke, n.d.)

Figure 66 shows the SUH and the lag-routed hydrograph, adjusted for the peak flows adopted (Table 22). The lag routed hydrograph result is adopted for this study. From the

figure, it can be seen that the latter method produces a broader hydrograph with a longer period, representing a greater flow volume.

Table 22: Peak flows adopted for the catchment above Wolwedans dam

Return period	Peak flow in m ³ /s
50	623
100	775
200	878

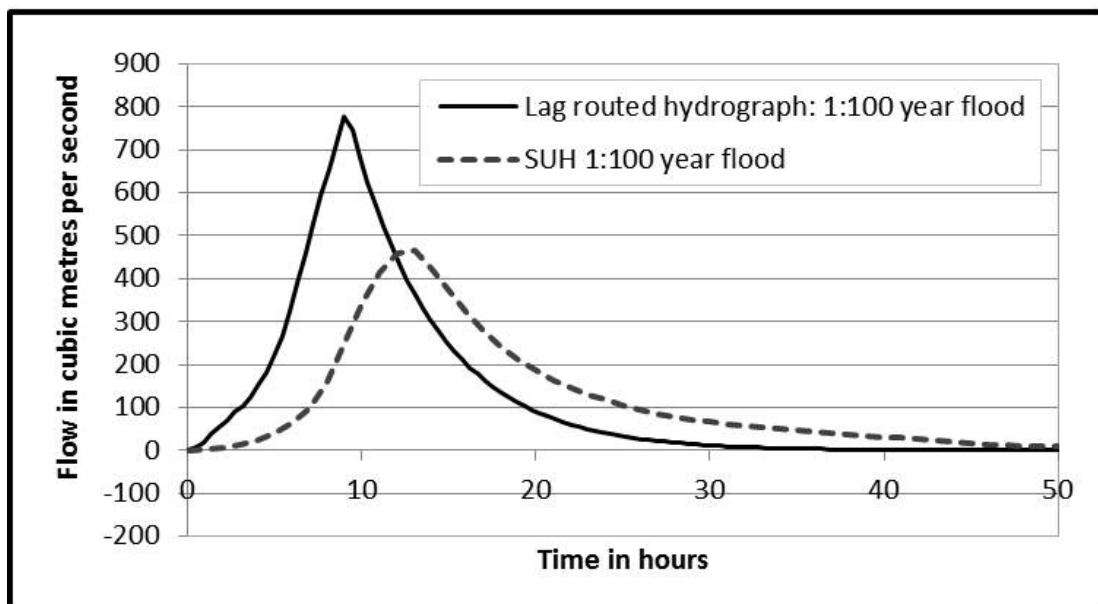


Figure 66: Lag-routed hydrograph and SUH result to Wolwedans dam (Gericke, n.d.)

16.6 Flood routing through Wolwedans dam

Flood routing of the flow through Wolwedans dam was undertaken to obtain the final flood peak and hydrograph duration. The modified PULS method (University of Asia Pacific, n.d.) was used. The resulting hydrograph is shown in Figure 67.

16.7 Runoff calculations for Great Brak Catchment to Estuary Mouth

The design flood for the total catchment to the estuary mouth was determined using the DFET software. The overall catchment response of 5 hrs was used. The flood peak was then reduced to reflect the reduction in peak flow due to the flood routing through Wolwedans dam. The results of the modelling from DFET are shown in Table 23 to Table 25 and Figure 68. The adopted hydrograph for Great Brak is shown in Figure 69.

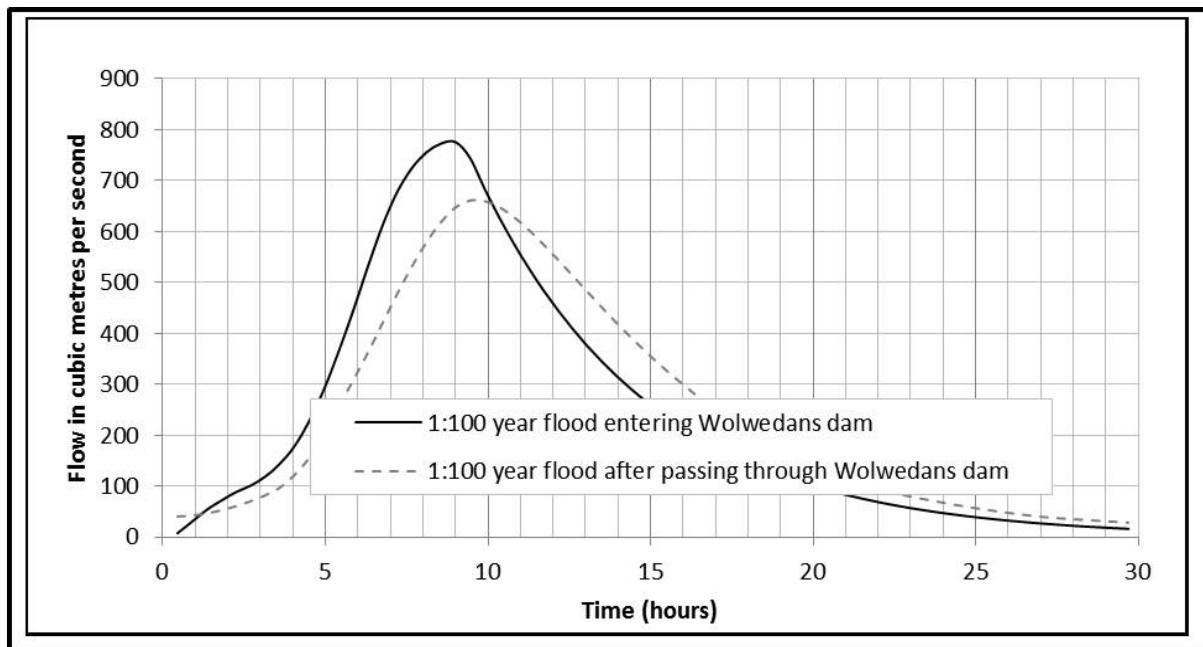


Figure 67: Resultant flow hydrograph at Wolwedans after routing flow through the reservoir

Table 23: Summary of results of deterministic methods for design flood estimation for the whole catchment

DETERMINISTIC METHODS						
Return period (T, years)	Design flow (m^3/s)					
	Rational	Alternative Rational	SCS	SDF	Unit Hydrograph	Lag-routed Hydrograph
2	134	116	101	42	113	300
5	194	209	226	149	156	415

10	261	290	345	248	200	532
20	341	380	489	360	251	665
50	468	502	726	527	330	876
100	607	609	946	668	410	1090
200	819	827	1205	817	465	1233

Table 24: Results for empirical methods for design floods, from DFET, for whole catchment

EMPIRICAL METHODS						
Return period	Design flow (m ³ /s)					
(T, years)	Midgley & Pitman (MIPI)	Catchment parameter (CAPA)	Q _T /Q _{RMF} -ratio	Regional Maximum Flood (RMF)		
2		45		Francou-Rodier 1787.5	Kovács 1793.1	
5		89				
10	144	129				
20	178	179				
50	233	261	704			
100	274	324	907			
200		384	1136			

Table 25: Peak flows adopted for the catchment

Return period	Peak flow in m ³ /s from DFET	Peak flow in m ³ /reduced due to routing through dam
50	665	600
100	876	800
200	1090	1015

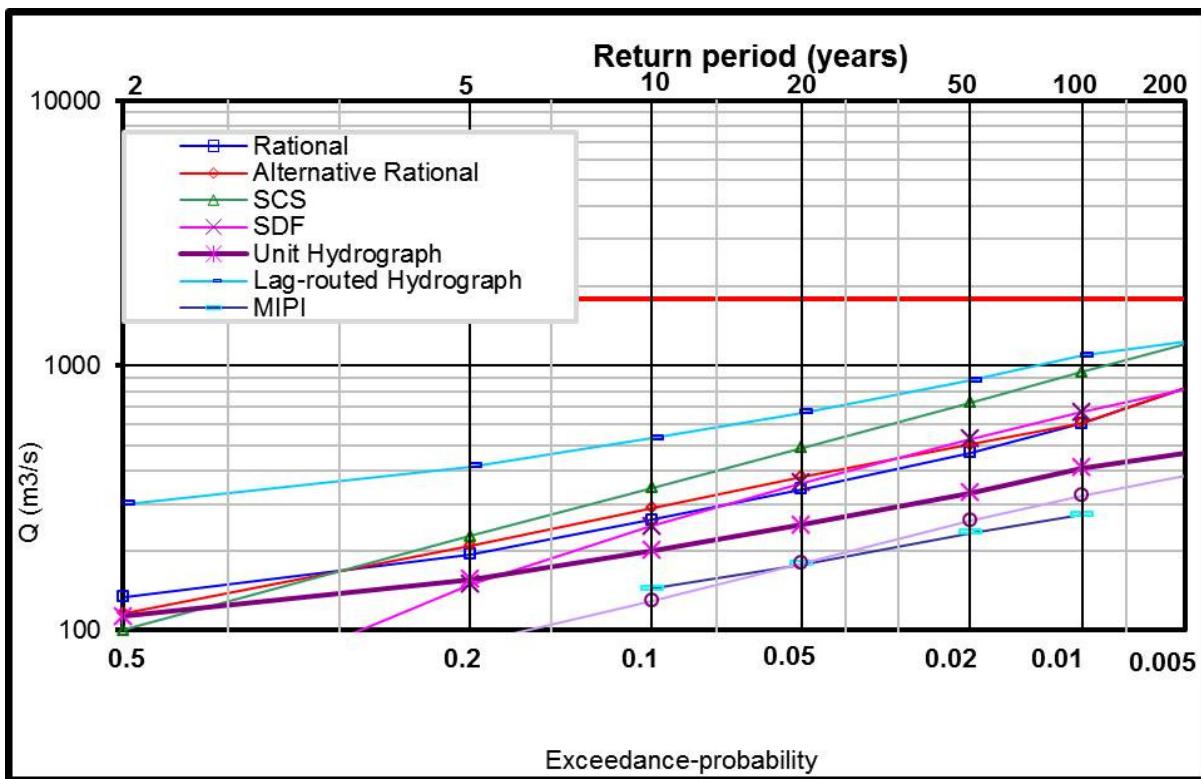


Figure 68: Plot of results for design flood from DFET software: above Wolwedans

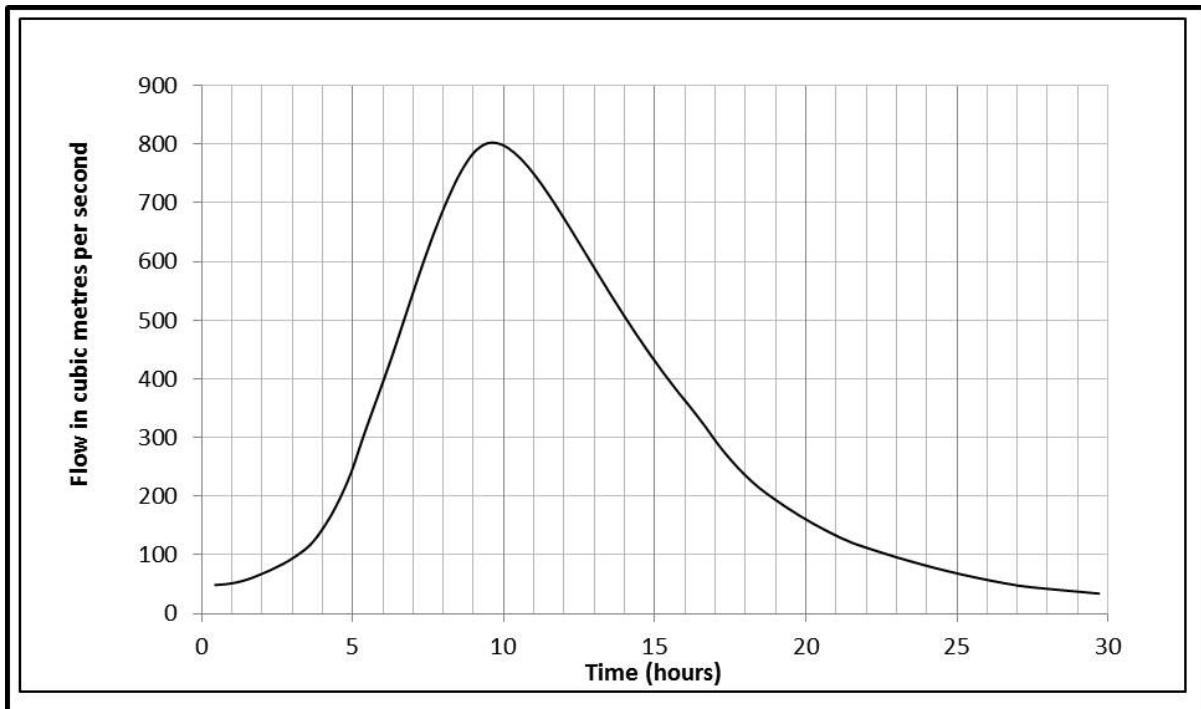


Figure 69: Design Inflow hydrograph for Great Brak Estuary