Berm Height at Temporarily Open/Closed Estuaries in South Africa: Analysis and Predictive Methods

by

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Abstract

This study investigates the berm crest elevation at South African Temporarily Open/Closed Estuaries (TOCE), as well as the processes involved in berm growth, and the drivers that contribute to variation in berm height among estuaries. The relationship between wave runup elevation and maximum berm height at estuaries is evaluated. Additionally, the study presents suitable methods for the prediction of berm height at South African TOCEs, given the limited data availability.

TOCEs along the wave dominated coastline of South Africa are subject to frequent inlet closure. During inlet closure, the presence of the wave built sand barrier (berm) restricts tidal influx and temporarily prevents catchment runoff from reaching the sea. The elevation of the inlet berm dictates the peak flood level in the estuary. A comprehension of estuary mouth behaviour, specifically the berm building processes present after estuary closure, is of paramount importance for the efficient management of these systems. This includes knowledge and quantification of the berm building processes, potential berm height and berm height variability.

The recorded berm crest elevations of twenty prominent TOCEs along the South African coastline are presented. Several years of berm/mouth survey data and estuary water levels have been analysed for the selected locations, resulting in an extensive record of historical berm crest elevations. This provides improved estimates of the probable berm height at these estuaries, especially compared to previous estimates typically based on limited survey data.

The primary drivers responsible for high berms and variation in berm height among estuaries were identified, viz. median sediment grain size, beach face slope, nearshore wave height and nearshore Iribarren number. The relationship between the berm height at the selected estuaries and the relevant coastal parameters were assessed. The beach face slope and the nearshore Iribarren number have a significant influence on the maximum berm height, and adequately describe the variation in berm height among estuaries. A multi-criteria analysis – the Berm Crest Elevation Criteria – and corresponding linear regression model is developed to investigate the relative importance of the dominant coastal parameters on maximum berm height. Additionally, the Berm Crest Elevation criteria provides an accurate first estimate of the maximum berm crest elevation at other, less studied TOCEs, based on only a few coastal input parameters.

The vertical extent of wave runup is assessed to determine the potential limit of berm accretion. Existing runup parameterisations are implemented to simulate several years of wave runup elevation at the selected estuaries, based on recorded sea levels and offshore wave data. The predicted wave runup elevation provides an accurate estimate of the long-term variation of estuarine berm height. The Stockdon *et al.* (2006) wave runup parameterisation provides superior performance across the entire range of estuaries.

The occurrence probability of the simulated wave runup elevation records were assessed to further elucidate the probability of wave runup associated with maximum berm height at estuaries. The findings indicate that the maximum berm height can be predicted by the 5% exceedance probability of wave runup. A theoretical threshold of runup exceedance probability and associated berm response is presented. Additionally, a design scenario of wave runup is proposed to estimate the vertical extent of sediment deposition caused by wave runup. The design scenario is based on the 2% exceedance probability significant wave height, 50% exceedance probability peak wave period and Mean High Water Spring (MHWS) tidal elevation.

Lastly, a berm growth model is presented to predict berm height/growth on a short-term time scale. The model provides an incremental prediction of the morphodynamic response of estuarine berms subjected to wave runup and overwash.

Samevatting

Hierdie studie ondersoek die bermhoogtes by Suid-Afrikaanse strandmere met tydelike oop/geslote mondings, asook die bermvormingsprosesse en die faktore wat bydra tot die variasie in bermhoogtes tussen strandmere. Die verhouding tussen golfoploophoogte en die maksimum bermhoogte by strandmere word ook ondersoek. Verder bied die studie ook geskikte metodes vir die voorspelling van bermhoogte by Suid-Afrikaanse strandmere, gegewe die beperkte data beskikbaar.

Tydelike oop/geslote strandmere langs die golf-energieke kuslyn van Suid-Afrika is onderhewig aan gereelde geslote mondtoestande. Tydens geslote mondtoestande beperk die golf-geboude sandberm die invloei van die gety en veroorsaak ’n tydelike versperring vir afloop vanaf die opvangsgebied. Die hoogte van die sandberm bepaal die piek watervlak in die strandmeer tydens vloede. ’n Begrip van die strandmeer mond-dinamika, veral die bermvormingsprosesse na mondsluiting, is uiters belangrik vir die effektiewe bestuur van hierdie stelsels. Dit sluit in die kennis en kwantifisering van die bermvormingsprosesse, asook die potensiële bermhoogte en bermveranderlikheid.

The bermhoogtes van twintig prominente tydelike oop/geslote strandmere langs die Suid-Afrikaanse kuslyn word aangebied. Verskeie jare se mond/berm opmetings en strandmeer watervlak lesings is ontleed vir die geselekteerde strandmere. Die uitkoms van hierdie analise is ’n omvattende rekord van historiese bermhoogtes. Hierdie rekord verskaf ’n meer akkurate benadering van die maksimum potensiële bermhoogte by die geselekteerde strandmere, veral in vergelyking met vorige voorspellings wat tipies gebaseer was op beperkte opmetings.

Die primêre drywers wat verantwoordelik is vir hoë berms en die variasie in bermhoogtes tussen strandmere is geïdentifiseer. Die primêre drywers sluit in: die mediaankorrelgrootte, strandhelling, golfhoogte en Iribarren getal. Die verhouding tussen bermhoogte by die geselekteerde strandmere en die relevante kus-parameters is geëvalueer. Die strandhelling en Iribarren getal wys die sterkste korrelasie met die bermhoogtes van die onderskeie strandmere. ’n Multi-kriteria-ontleding genaamd die Bermhoogtekriteria (“Berm Crest Elevation Criteria”) en ’n ooreenstemmende regressiemodel is ontwikkel om die gesamentlike effek en relatiewe belangrikheid van die oorheersende veranderlikes te ondersoek. Op grond van net ’n paar inset parameters kan die Bermhoogtekriteria ook gebruik word as ’n akkurate eerste benadering van die maksimum bermhoogte by Suid-Afrikaanse strandmere.

Die hoogte van golfoploop is geëvalueer om die potensiële grens van berm-opbou vas te stel. Bestaande parametriese golfoploopmodelle is benut om verskeie jare se golfoploop te simuleer by die onderskeie strandmere. Die simulasies is gebaseer op gety- en golfopmetings naby die onderskeie strandmere. Die voorspelde golfoploop verskaf ’n akkurate benadering van die langtermyn variasie in strandmeer bermhoogte. Die Stockdon *et al.* (2006) golfoploopmodel voorsien die beste resultate vir al die strandmere en inset parameters.

Die oorskrydingswaarskynlikheid van die gesimuleerde golfoploop rekords is geëvalueer. Die analise is gemik daarop om die oorskrydingswaarskynlikheid van golfoploophoogte wat geassosieer is met die maksimum bermhoogtes te bereken. Die resultate dui aan dat die maksimum bermhoogte voorspel kan word deur die 5% oorskrydingswaarskynlikheid van golfoploop. ’n Teoretiese drumpel van golfoploop oorskrydingswaarskynlikheid en ooreenstemmende bermverandering is voorgestel. ’n Ontwerpscenario van golfoploop is voorgestel om die vertikale omvang van sediment afsetting wat deur golfoploop gegenereer word te voorspel. Die ontwerpscenario is gebaseer op die 2% oorskrydingswaarskynlikheid golfhoogte, 50% oorskrydingswaarskynlikheid golfspitsperiode en die gemiddelde hoogwater springgety hoogte.

Laastens is ’n bermgroeimodel voorgestel. Die model beoog om die korttermyn bermhoogte en groei by strandmere te voorspel deur gebruik te maak van ’n stapsgewyse golfoploop groeikoers.

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Table of Contents

List of Figures

List of Tables

List of Appendices

Nomenclature

B_c	Berm crest elevation
C	Mather C-coefficient
c_p	Constant of proportionality for proposed berm growth model
d	Water depth
D	Sediment grain diameter
D_{50}	Median sediment grain diameter
D_*	Dimensionless grain diameter of sediment
g	Gravitational acceleration constant
h	Depth at closure
H_0	Deep water significant wave height
H_b	Wave breaker height
H_m	Mean wave height
H_{0rms}	Deep water root mean square wave height
H_s	Significant wave height
K_T	Wave transformation coefficient
L_0	Wave length in deep water
L_b	Wave length in breaker zone
m	Beach face slope
n	number of data points
N_0	Dean number
OP	Overtopping potential
R^2	Coefficient of determination
$R_{2\%}$	Wave runup elevation exceeded by 2% of waves based on Rayleigh distribution
R_x	Wave runup elevation exceeded by x% of waves based on Rayleigh distribution
rf	Reduction factor for weighting system for the proposed Berm Crest Elevation Criteria
S	Berm index score for the proposed Berm Crest Elevation Criteria
T_p	Peak wave period
$T_{m-1.0}$	Mean energy wave period
v	Kinematic viscosity of fluid
w	Sediment fall velocity
x_h	Horizontal distance from shoreline to depth at closure
y_j	Measured data points
$\hat{y}_j$	Predicted value from model
Δz	Berm growth rate
α	Beach face slope
α_φ	Phi coefficient of skewness
β_f	Beach face slope
β_φ	Phi coefficient of kurtosis
ξ_0	Deep water Iribarren number
ξ_b	Breaker Iribarren number
ρ	Density of fluid (sea water)
ρ_s	Density of beach material/sediment
σ_φ	Phi standard deviation
φ	Phi sediment grain size
$\emptyset$	Reduction factor to compensate for roughness and porosity of sediment

List of Abbreviations

4PL	Four Parameter Logistic
CD	Chart datum
CSIR	Council for Scientific and Industrial Research
DGPS	Differential Global Positioning System
DWS	Department of Water and Sanitation
ECRU	Estuarine and Coastal Research Unit
IQR	Inter Quartile Ranges
LLD	Land Levelling Datum
LWT	Large Wave Tank
MAE	Mean Absolute Error
MAR	Mean Annual Rainfall
MHWS	Mean High Water Spring
MLWS	Mean Low Water Spring
MSL	Mean Sea Level
NCEP	National Centres for Environmental Prediction
NOAA	National Oceanic and Atmospheric Administration
NRIO	National Research Institute for Oceanology
POE	Permanently Open Estuary
RMSEP	Root Mean Squared Error Predictor
SANHO	South African Navy Hydrographic Office
SWAN	Simulating WAves Nearshore
SWL	Still Water Level
TNPA	Transnet National Ports Authority
TOCE	Temporarily Open/Closed Estuary
UHSLC	University of Hawaii Sea Level Centre

1. Introduction

The vast majority (70%) of South Africa's approximately 300 estuaries are classified as Temporarily Open/Closed Estuaries (TOCE). Temporarily open/closed estuary inlets along the wave dominated South African coastline experience intermittent closure. Inlet closure is primarily dependent on rainfall, tidal flow and beach face morphodynamics. During inlet closure, the presence of a wave built sand barrier (berm) restricts tidal influx and temporarily prevents catchment runoff from reaching the sea.

Periods of high rainfall and runoff lead to an increased water level within the estuary during closed mouth state. The estuary water level may continue to rise to an elevation exceeding the berm crest, at which point the berm is overtopped and a natural breaching process is initiated. Breaching typically results in a scoured channel through the inlet berm and a sudden drop in water level, often followed by tidal influence. Several TOCEs have developments around their shoreline and in the catchment area, which may be subject to flooding due to high water build up behind the inlet berm. Therefore, the elevation of the inlet berm at the time of breaching dictates the flood peak levels within the estuary. The flood peak level within the estuary is a critical boundary condition for the determination of setback lines for human development along estuaries. Prolonged inlet closure may also have a pronounced effect on the physio-chemical properties of the estuary, i.e. the accumulation of pollutants, and changes in the salinity and temperature.

Artificial mouth manipulation is a management intervention aimed at reducing flood risk and potentially restoring natural function to the estuary. However, artificial breaching may be detrimental to the overall hydrodynamic and ecological function of the system. Artificial breaching practised at insufficiently low water levels may lead to significant sedimentation in the lower reaches of the estuary.

A comprehension of estuary entrance behaviour, specifically the berm building processes present after estuary closure, is of paramount importance for the efficient management of these systems. This includes knowledge and quantification of the berm building processes, potential berm height and berm height variability. To date, the primary research focus has been the mouth functioning of South African TOCEs, specifically with regards to prediction of mouth state (open or closed). The prediction of estuarine berm height and berm growth is a relatively unexplored topic in South Africa. In practice, the estimation of estuarine berm height is typically based on limited mouth/berm surveys or professional opinions.

Berms are depositional features, ubiquitous at closed estuary inlets. Berms originate due to the accumulation of marine sediment at the landward extent of incident wave runup. The flow velocity of wave runup decreases towards the upper beach face, resulting in the deposition of sediment.

Knowledge of berm morphodynamics and swash zone hydrodynamics is required to assess the behaviour of berms when subjected to wave action. Therefore, an investigation into the probable berm heights of South African TOCEs, including potential predictive methods, became necessary.

Figure 1-1: Partially closed estuary on the Wild Coast, South Africa (Chadwick, 2015)

1.1. Study Objectives

The first objective of this study is to determine the probable berm crest elevations of South African temporarily open/closed estuaries, as well as to identify the primary drivers responsible for the variation in berm height among estuaries.

The second objective of this study is to identify and evaluate suitable methods for the prediction of berm height, taking into consideration the limited data availability in South Africa. Additionally, this objective aims to elucidate the relationship between wave runup and the maximum berm height at estuaries.

Lastly, this study aims to collate the available coastal parameter data pertaining to berm height and functioning at South African temporarily open/closed estuaries. This includes the sediment grain size, beach face slope, nearshore wave characteristics and supplementary general information of the selected estuaries.

1.2. Limitations of this Study

The following limitations are imposed on this study:

- Available mouth/berm surveys of South African estuaries are extremely sparse. There are no available field observations documenting the short-term morphological response of estuarine berms subjected to wave action.

- The berm height derived from the water level recordings in an estuary only provides a vertical elevation at the time of a breach. This provides limited information regarding the short-term behaviour and response of estuarine berms.

- Numerical wave modelling was not considered to determine the nearshore wave conditions at the respective estuaries. The large number of study locations (20) deemed detailed wave modelling an infeasible option. Wave transformational coefficients were implemented as a suitable alternative to account for the nearshore wave transformation processes.

- Wave runup measurements are not readily available to calibrate/verify the simulated runup records. However, the runup parameterisations were used in accordance with the findings and recommendations of previous South African based runup evaluations (e.g. Theron, 2016; Roux, 2015), so as to ensure best practice.

Considering these limitations, it is worth noting that estuarine berm height is a relatively unexplored topic in South Africa. This study focusses on providing initial insight toward the functioning and potential height of estuarine berms in South Africa, despite of the data limitations. The primary objective is to provide a quantitative comprehension of berm height, however certain aspects are limited to a qualitative understanding due to these limitations.

The conclusions of this study are based on the collective knowledge of the author, as well as the professional opinions from industry professionals. Ultimately, this study aims to identify suitable methods to predict berm height, as well as provide a theoretical basis for subsequent berm height/growth studies.

1.3. Chapter Layout

The report structure aims to guide the reader through the relevant steps and investigations involved in achieving the desired objectives.

Chapter 1 introduces the topic and discusses its relevance and context in South Africa. The study objectives are delineated, along with the scope and limitations, followed by a methodology.

Chapter 2 presents a literature review, primarily discussing the processes relevant to estuary mouth functioning and berm development.

Chapter 3 provides a brief qualitative description of the selected South African estuaries.

Chapter 4 discusses the available South African field data pertaining to berm height and other coastal parameters. The collection process of the relevant field data is also discussed.

Chapter 5 presents the results and analysis of the recorded berm heights at South African estuaries. The relationship between berm height and the relevant coastal parameters are also discussed.

Chapter 6 presents the relevant methods of predicting estuarine berm height, as well as the procedures involved in these methods. Additionally, this chapter explores the relationship between wave runup and berm height.

Chapter 7 provides a summary of the appropriate berm height predictive methods. The summary is intended to act as a basic methodology, demonstrating the intended use and application of the relevant predictors.

Chapter 8 presents the conclusions of the study, followed by recommendations for future research.

1.4. Methodology

A total of 20 temporarily open/closed estuaries were selected for analysis. The selected locations include estuaries from the Western Cape, Eastern Cape and KwaZulu-Natal provinces, to ensure a wide range of conditions among samples. The selection was primarily based on the availability of data pertaining to berm height. A brief qualitative assessment was conducted for each respective estuary, in order to provide a basic high-level understanding of inlet berm formation and mouth functioning.

The recorded berm crest elevations of the respective estuaries were derived from available estuary water level data and mouth/berm surveys. The elevations of the berm saddle points at the time of breaching were derived from scrutinising the respective water level recordings. Consequently, comprehensive records of historical berm crest elevations were collected and evaluated for the respective estuaries.

The relevant coastal parameters responsible for berm morphology were identified and collected for the respective estuaries. The relative importance of these parameters was assessed to determine the primary drivers responsible for high berms, and to assess the potential to describe the variability in berm height among estuaries. The combined effect of the selected coastal parameters is assessed by means of a proposed Berm Crest Elevation Criteria and regression analysis.

A total of 8 suitable parametric runup and berm height prediction models were identified based on their reported performance and data requirements. The use of runup parameterisations are based on the assumption that berm growth is triggered by the deposition of sediment at the landward extent of wave runup. The runup parameterisations selected for evalution include:

- Nielsen and Hanslow (1991)
- Ruggiero *et al.* (2001) – 2 models
- Stockdon *et al.* (2006)
- Mather *et al.* (2011)

The selected parametric berm height models:

- Swart (1974) – *D profile limit*
- Larson and Kraus (1989)
- Okazaki and Sunamura (1995)

The wave runup was simulated for the respective estuaries, according to each of the relevant predictors. The simulated records require several years of recorded wave- and tidal data at each estuary. The simulated runup records are compared to the overlapping recorded berm heights at the respective estuaries. This aims to elucidate the relationship between wave runup and maximum berm height, as well as identify suitable methods for the prediction of berm height at South African estuaries.

2. Literature Review

The literature review primarily focuses on the berm development processes that contribute to the potential maximum berm crest elevation of TOCE in South Africa, as well as the methods available for prediction. A broad overview of the South African coastline and estuaries is provided, as well as an evaluation of estuary classification in a South African context. A description of the major hydrodynamic and sedimentary processes involved in estuary mouth functioning (opening/closure/breaching) are also discussed.

2.1. South African Estuaries and Coastal Setting

A widely accepted definition of an estuary in a South African context is that defined by Day (1980). Day describes an estuary as, "… a partially enclosed coastal body of water which is either permanently or periodically open to the sea within which there is a measurable variation of salinity due to the mixture of sea water with fresh water derived from land drainage". This definition has been adopted as the official definition of an estuary (in a South African context) as per the National Water Act (No. 36 of 1998).

There are almost 300 functional estuaries located along the South African coastline. This number is reduced to 258 when eliminating the systems that do not function according to the recognised definition of Day (1980) (Whitfield, 2000). Typical examples of systems that do not satisfy this criterion include: Langebaan, Buffels Wes, Papkuils and Skaapkop.

The South African coastline spans over 3000 km and is highly variable in terms of geomorphological and climatological features. This can lead to high variation in characteristic features of estuaries along the coastline at a given time. Cooper (2001) describes several factors that may be considered as relatively analogous when considering the geomorphology of estuaries in South Africa. These factors include: low tidal range (microtidal), high wave energy, a predominantly bedrock coast and a consistent sea level history.

The tidal amplitude around South Africa can be classified as microtidal (0 to 2m). Microtidal range is a typical feature among open coasts around the world. The South African coastline experiences a tidal range that varies relatively little between locations, with the majority of locations experiencing a spring tidal range between 1.8 and 2.0 m and neap tides typically ranging between 0.6 and 0.8 m (Davies & Clayton, 1980).

Wave energy along the South African coastline is consistently high, with a slight decreasing gradient from south to north. The wave height and -period peak in the Southern Cape and gradually reduce northward along both the east - and the west coast. There is however variability in the wave incidence angle along the coastline, which in turn contributes to the variability in long-shore sediment transport rates (Cooper, 2001).

The majority of South African estuaries are located in incised bedrock valleys which laterally constrict the growth of the water body. The extent to which the area within the bedrock valley is filled may differ between individual systems. Some estuary channels span the entire valley, while others consist of a channel and a large flood plain within the valley. There are only a few estuaries that have developed on coastal plains (Cooper, 2001).

Reddering and Rust (1990) inferred that most South African estuaries are relatively small, with tidal prisms in the range of 10^6 m^3 or less. Additional characteristic features include the intermittent nature of the estuary mouth state (periodic closure), as well as a strongly developed flood-tidal delta and poorly developed, or absent, ebb-tidal delta (Reddering & Rust, 1990).

2.2. Classification of South African Estuaries

Coastal water bodies, broadly termed estuaries, display a variety of differences in geomorphology, physiography and hydrology. Authors have classified estuaries according to their variability in tidal range (Hayes, 1979), sedimentary infilling (Nichols, 1989) and the influence of wave, tidal and fluvial processes (Cooper, 1993).

One of the most frequently used classifications of estuaries in South Africa is that of Whitfield (1992). Whitfield identified five types of systems by assessing the dominant conditions, viz. permanently open estuaries, temporarily open/closed estuaries, estuarine lakes, estuarine bays and river mouths. When considering estuaries from only a hydrodynamic perspective there are only two main categories: permanently open estuaries (POE) and temporarily open/closed estuaries (TOCE). The remainder of categories are sub-classes of these from an abiotic perspective (Van Niekerk *et al.*, 2012).

A similar classification is discussed by Cooper (2001), where estuaries are grouped according to the predominant mouth conditions. Estuaries are divided into two main categories namely, normally open estuaries and normally closed estuaries. However, for the purpose of this particular study it is convenient to distinguish between estuaries that maintain a permanently open mouth state, and estuaries that experience periodic mouth closure. Therefore, the classification of Whitfield (1992) will serve as a suitable guide for identifying relevant estuaries where berm crest elevation is of interest. There have been instances where individual systems have changed from one estuary category to another. These changes can occur due to long-term climatic variations, natural events and anthropogenic influences. Such was the case with Richards Bay, where it evolved from an estuarine lake to two separate estuarine bays due to the harbour construction (Whitfield, 2000).

The distribution of estuaries along the South African coastline is further grouped into three distinct biogeographic regions namely: cool temperate, warm temperate and sub-tropical. Figure 2-1 illustrates the location and boundaries of the three biogeographic regions.

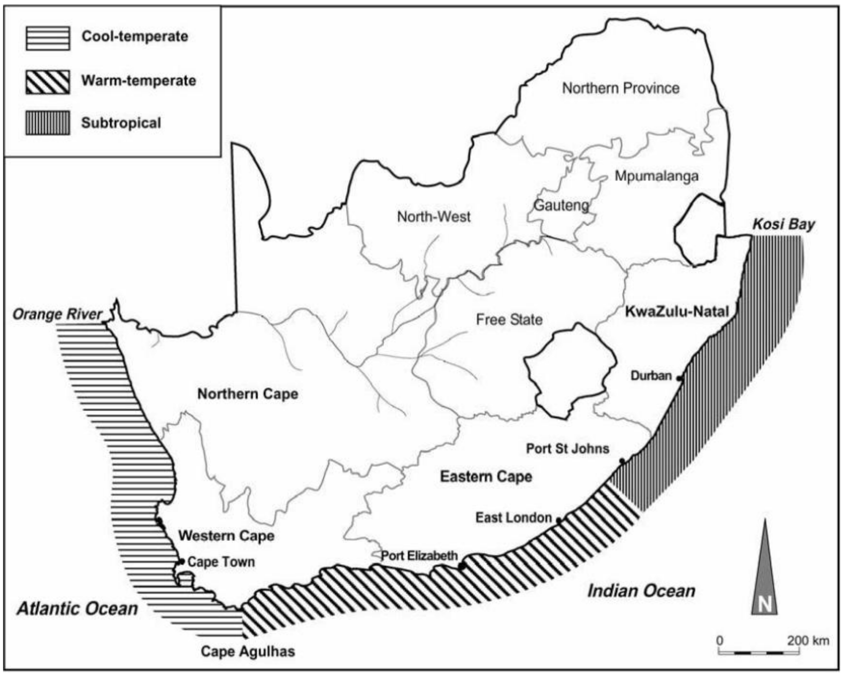


Figure 2-1: Three biogeographic regions of South Africa (Whitfield & Bate, 2007)

2.2.1. Permanently open estuaries

Approximately 30% of South Africa's nearly 300 estuaries maintain a permanent open connection to the sea (Whitfield, 1998, cited in Van Niekerk *et al.*, 2012). These estuaries can generally be characterised by large catchment areas and relatively high runoff throughout the year. Permanently open estuaries remain open even during low flow conditions, however severe mouth restrictions and reduced tidal flushing may occur (Van Niekerk *et al.*, 2012). Furthermore, Whitfield (1992) characterises permanently open estuaries by moderate tidal prisms (1 – 10 x 10^6 m^3) and perennial flow as part of their natural state. Prime examples of permanently open estuaries in South Africa include the Breede - and Olifants Estuaries.

2.2.2. Temporarily open/closed estuaries

The remaining 70% of South African estuaries are classified as temporarily open/closed estuaries (Whitfield, 1998, cited in Van Niekerk *et al.*, 2012). These systems are separated from the ocean for prolonged periods due to the formation of a sand berm across the mouth. The sand berm forms during periods of little or no river flow and is aided by wave action and marine sediment transport. The system remains isolated from the marine environment until breaching of the barrier occurs due to high water levels within the estuary, or an increase in river inflow (Van Niekerk *et al.*, 2012).

Temporarily open/closed estuaries are generally characterised by their small river catchment area (<500 km^2) and seasonal variation in river inflow.

During open mouth state the tidal prism remains relatively small (<1 x 10^6 m^3) and reduces to zero during mouth closure (Whitfield, 1992). Similar estuaries in Australia are named Intermittently Closed and Open Lakes and Lagoons (ICOLLs) and are a characteristic feature of the Australian coastline. Prime examples of temporarily open/closed estuaries in South Africa are the Groot Brak - and Bot River Estuaries.

2.2.2.1. Perched and non-perched estuaries

Cooper (2001) further subdivides TOCEs into two categories namely, perched- and non-perched estuaries. The classification is based on the elevation of the estuary water levels compared to the open sea water levels.

Perched estuaries describe the state where the minimum water level within the estuary is significantly elevated above the Mean Sea Level (MSL). Relatively high wave energy and coarse marine sediment cause these systems to have generally high berm elevations. Tidal inflow during open mouth state is generally restricted or completely inhibited.

At non-perched estuaries, the water level is equal or close to the high tide level at sea. Non-perched systems generally have lower berm levels due wave dissipation caused by low gradient beach profiles and wide surf zones. Non-perched estuaries are prone to marine overwash due to the lower berm levels (Cooper, 2001).

The concept of classifying estuaries as perched or non-perched does introduce some uncertainty due to the dynamic nature of estuaries. Large variations in bed level of the mouth region can be caused by fluvial floods and coastal storms.

2.2.2.2. Mouth state

At any given time, an estuary mouth can be either classified as being open or closed. During the open mouth state (Figure 2-2) the estuary water body is openly connected to the sea. Thus, a tidal exchange is evident along with mixing of fresh and seawater. A prominent outflow channel (> 2.0 m depth and relatively wide) is exhibited during this mouth state, especially after river floods. Smaller systems generally exhibit relatively smaller outflow channels (< 1.0 m depth and a few metres wide) shortly after breaching events. Tidal amplitude is in the order of 1.0 m during spring tide and 0.3 m during neap tides, depending on the degree of mouth restriction and estuary bed levels (Whitfield & Bate, 2007).

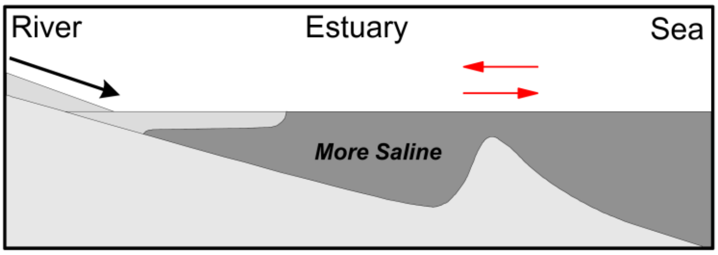


Figure 2-2: Schematised cross section of open mouth (Whitfield & Bate, 2007)

During closed mouth state the estuarine water body is separated from the sea by a wave built sediment berm (Figure 2-3). There is no tidal influence in the estuary and minimal seawater intrusion. Occasional marine overwash due to energetic wave conditions and high sea water levels can cause limited seawater intrusion.

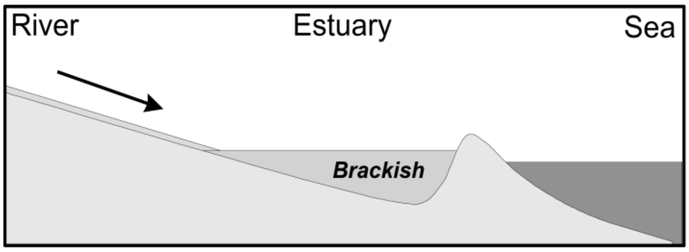


Figure 2-3: Schematised cross section of closed mouth (Whitfield & Bate, 2007)

The mouth state of TOCEs typically vary between an open and closed state. However, Van Niekerk *et al.* (2012) identified a third mouth state based on extensive research conducted on small estuarine systems. The third mouth state is termed semi-closed state, where the estuary berm has almost completely blocked the mouth, except for an elevated narrow outflow channel (Figure 2-4). Estuaries with semi-closed mouths are typically perched. Thus, the shallow channel allows only a minimal outflow of water to the sea and minimal intrusion of seawater. There is however a possibility of seawater intrusion during marine overwash and spring high tide. Semi-closed mouth state must continue for at least 14 days, in order not to be confused with the transitionary period between open and closed mouth state. Unfortunately, there is a lack of data pertaining to the duration of semi-closed mouth state at South African estuaries. However, the limited data available indicates that this state can last anything from two weeks to several months. Semi-closed mouths require only a small steady outflow (0.05 to 1 m^3/s) to maintain this mouth state. Outflow channels are typically between 10 to 30 m wide and only 150 to 300 mm deep (Whitfield & Bate, 2007).

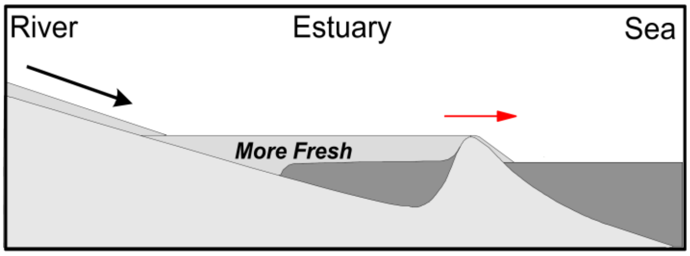


Figure 2-4: Schematised cross section of semi-closed mouth (Whitfield & Bate, 2007)

The formation of a semi-closed mouth is often present at estuaries where the mouth is sheltered from direct wave action due to rock formations or headlands, or in the case of limited wave action and sediment availability. Examples of semi-closed estuaries include: Palmiet -, Mdloti -, Onrus - and Lourens Estuary. Estuaries with semi-closed mouth state are also subject to periodic mouth closure, thus knowledge of the potential berm crest level during closed mouth state is useful.

The duration of each respective mouth state is greatly dependent on river inflow. Perissinotto *et al.* (2004, cited in Perissinotto, 2010) identified a bi-modal distribution when investigated the proportional time that South African TOCEs are either open or closed. Thus, the majority of estuaries are either open for less than a third of the time, or more than two-thirds of the time. There are only a small number of systems that have a balanced open/closed regime. This bi-modal distribution may suggest that a portion of systems only open during episodic flood events (typically exhibit closed mouth state), while others only experience closure due to extreme wave action during sea storms (typically exhibit open mouth state).

2.3. Breaching Processes

Breaching plays an integral part of the general function of TOCEs. Breaching can occur either naturally, or artificially as a management intervention. Breaching leads to a sudden drop in estuary water level, causing a flush of water and significant scour of sediment in the mouth region and lower reaches of the estuary.

2.3.1. Natural breaching

Natural breaching of estuary barriers is a common occurrence at TOCEs and is initiated by overtopping of the berm, or by means of seepage and liquefaction. The rapid discharge of water associated with breaching can cause significant alterations in estuary morphology. The general assumption is that the higher the water level is prior to breaching, the more sediment is flushed out due to the higher outflow velocities and longer outflow durations, resulting in prolonged periods of open mouth state (Van Niekerk *et al.*, 2012).

Seepage of water takes place through the porous unconsolidated sand berm. Berm seepage often takes place at perched estuaries, where there is an evident head difference between the estuary water body and the sea. Seepage can lead to soil stability loss and ultimately berm failure.

Overtopping of the inlet barrier is the main method of natural breaching. The estuary water body will gradually fill up until the level exceeds the minimum level (saddle point) on the berm crest. Breaching is initiated by a gradual overtopping flow that scours a channel on the downstream face of the berm, keeping the upstream face relatively intact. Once the upstream crest experiences significant scouring, the breach formation phase is initiated. Water outflow and erosion rates rapidly increase during this phase. A soon as the breach channel reaches the optimal width and depth, the outflow stabilises and then decreases, and the estuary water level drops (Parkinson & Stretch, 2007). Once the breach has run its course the tidal variation will commence within the estuary, and cease as soon as the mouth is closed.

2.3.2. Artificial breaching

Artificial breaching involves excavating a channel across the inlet barrier in order to connect the estuary water body with the ocean. The channel base should be excavated to a level that is below the water level of the estuary water body.

TOCEs experience episodic mouth closure during sustained periods of low river inflow. Therefore, each individual system has a specific frequency and duration within which mouth closure occurs naturally (van Niekerk, 2007, cited in Whitfield *et al*., 2012). If closure coincides with this range, management interventions are typically not required. However, artificial breaching may be required if closure falls outside this range, i.e., if the estuary closes too often or for prolonged periods. Artificial breaching aims to prevent flooding, flush sediment and contaminants, and restore ecological function of estuaries. Numerous developments have originated on the flood plains of South African estuaries, resulting in the occasional need for artificial breaching to prevent flooding of property.

An alternative method of mouth manipulation, called sand bar skimming, is also practised at certain estuaries. This involves the active management of the inlet berm height by means of scraping/skimming the top off and reducing the height. This specific method is implemented at the Touw Estuary (Southern Cape), where the sensitive low level areas become rapidly inundated during floods.

Artificial breaching at insufficiently low water levels can lead to serious sedimentation effects in the lower reaches of the estuary. Care should be taken to breach during the highest possible estuarine water levels, similar to conditions during natural breaching (Schumann, 2003). Additionally, artificial breaching should be scheduled during periods of low sea level, to ensure a sufficient hydraulic head between water bodies.

Examples of South African estuaries subject to regular artificial breaching include: Seekoei-, Groot Brak-, Bot -, Klein- and Diep Estuary. The artificial breaching sequence of the Groot Brak Estuary is illustrated in Figure 2-5.

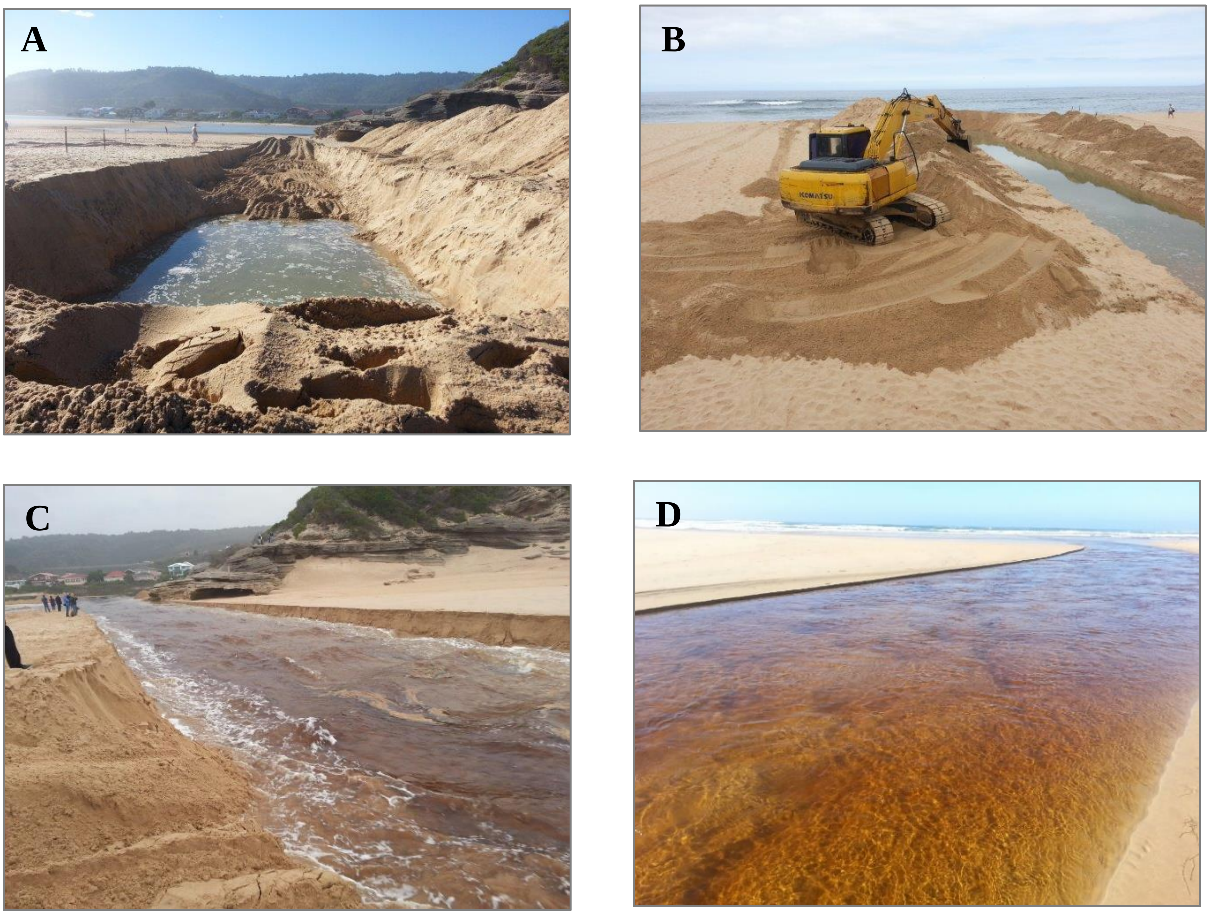


Figure 2-5: Artificial breaching procedure at the Groot Brak Estuary, South Africa (Mossel Bay Municipality, 2017) – A and B show excavation of channel, C and D show the initiation of breaching and open mouth state respectively

2.4. Maintaining Open Mouth State

Van Niekerk *et al.* (2012) identified river inflow and tidal flows as the primary forces responsible for maintaining open mouth conditions in South African estuaries.

Additionally, the degree of mouth protection may contribute towards maintaining an open mouth condition. Rocky headlands and sub-tidal rocky shelves act as a buffer by dissipating incoming wave energy and reducing sediment influx at the mouth (Whitfield & Bate, 2007).

2.4.1. River inflow

A high degree of correlation between estuary mouth closure and river inflow is evident among South African estuaries. River inflow is considered the sole driving force maintaining open mouth conditions in smaller TOCEs. This is due to the limited scouring effect of tidal flows present in smaller estuaries. However, tidal flow does play a significant role in larger systems (Van Niekerk *et al.*, 2012).

River inflow in an estuary is dependent on the relevant catchment processes and rainfall characteristics. Rainfall in river catchments provides an input of fresh water in the estuary. South African rainfall is particularly erratic and seasonally variable, which causes variation in base flow of individual estuaries. TOCEs tend to have smaller catchment areas (typically < 100 km^2) along with a high seasonal variation in runoff (Whitfield, 1992), causing them to be particularly sensitive to mouth closure.

The river inflow required to maintain open mouth conditions is greatly dependent on the wave conditions and sediment availability in the mouth region. An overview of these closing forces is provided in § 2.5.2. Several case studies (e.g. Perissinotto *et al.*, 2004, cited in Perissinotto *et al.*, 2010) have provided the correlation between flow rates and open mouth condition of individual estuaries. In general, low flow is associated with closed mouth conditions and high flow (particularly flood events) are associated with open mouth state. However, flow magnitudes are specific to each individual system, with minor relevance to other systems (Perissinotto *et al.*, 2010). Factors that influence an estuary's sensitivity to flow reduction include: runoff, mouth protection and estuary bathymetry.

As an example, an estuary located on the high-energy coastline of KZN requires a flow of between 5-10 m^3/s to maintain open mouth conditions, while estuaries located on the south-western Cape coast require as little as 1-2 m^3/s (Huizinga & van Niekerk, 2005, cited in Whitfield & Bate, 2007). Contrarily, a semi-closed mouth state requires minimal base flow to persist compared to open mouth conditions. A preliminary assessment conducted at a small number of estuaries revealed a steady outflow of between 0.05-1 m^3/s is required to maintain semi-closed mouth state (Huizinga *et al*., 2001, cited in Whitfield & Bate, 2007).

At present, there is little evidence available pertaining to the duration of open mouth state in small TOCEs. Limited evidence indicates open mouth state is of short duration (a few days to weeks), primarily due to the sporadic runoff in the catchment (Whitfield & Bate, 2007).

High river inflow present during episodic flood events is important for scouring out accumulated sediment in the lower reaches of estuaries. The intensity of floods and associated scouring ability can be greatly influenced by the dams built in the catchment.

2.4.2. Tidal flow

Tidal flow plays a significant role in maintaining open mouth conditions of TOCE in South Africa. The tidal flow in estuaries exceeding 150 ha is typically high enough to maintain open mouth state, regardless of runoff decreases during periods of low flow. Estuarine lakes such as the Bot Estuary are an exception to this rule. These systems experience episodic closure despite their significant size. This is due to possible factors such as extended periods of low river inflow, high sediment availability, high wave energy and high evaporation rates (Whitfield & Bate, 2007).

Tidal flow only has a partial effect on medium sized estuaries (< 150 ha) such as the Groot Brak Estuary. The tidal flow during spring tides is sufficient to maintain an open mouth state, however during neap tides the flow reduces and the mouth closes (Whitfield & Bate, 2007). Consequently, the influence of tidal flow during periods of low river inflow in small to medium sized estuaries is insufficient to maintain open mouth conditions.

In some instances, tidal flows can aid in the transport of marine sediment into the estuary resulting in a flood tidal delta. Such is the case with asymmetrical flood dominated tidal exchange flows. The flood tidal flow rate is higher than the ebb tidal flow, due to the long period of outflow during ebb tide. Consequently, the flood tidal flow has higher sediment transport potential (Schumann, 2003).

2.5. Estuary Mouth Closure

Seasonal closure of TOCE inlets on the microtidal, wave dominated South African coast is a common feature. This section will discuss estuary mouth closure and the related processes.

2.5.1. Mechanisms of mouth closure

Ranasinghe *et al.* (1999) describe two separate mechanisms responsible for inlet closure at small microtidal estuaries on the wave dominated coastlines of Australia and South Africa.

<u>Mechanism 1</u>: The tidal inlet current disrupts the longshore current and consequently the longshore sediment transport. The ebb current velocity is reduced by the redirecting influence of the longshore current which results in a shoal up drift of the inlet. The size and growth rate of the shoal is entirely dependent on the rate of longshore sediment transport across the inlet. Growth of the shoal will cease, given the inlet currents are persistently strong enough to remove the deposited sediment in the

channel. A spit will develop across the inlet channel in the presence of weak inlet currents (synonymous with low river inflow), ultimately causing mouth closure (Figure 2-6). This is the most probable mechanism of inlet closure on straight beaches with high longshore transport rates.

Mechanism 2: This mechanism occurs in the presence of weak inlet currents (< 1 m/s), therefore common at estuaries with small tidal prisms. The inlet current interacts with the onshore sediment transport caused by swell wave action. Typically, the longshore currents and longshore sediment transport rates are small compared to Mechanism 1. Wave action during stormy conditions cause the beach and surf zone to erode. The eroded sediment migrates and settles offshore to form a longshore bar at the edge of the breaker zone. When the storm subsides, sediment from the offshore bar is transported back onshore. The presence of strong ebb tidal currents will disrupt the onshore movement of sediment opposite the inlet, thus preventing closure. Weak ebb tidal currents (during summer) will be insufficient to prevent inlet closure during continuous onshore migration of sediment (Figure 2-6). This mechanism of inlet closure is mainly applicable to embayed beaches where longshore sediment transport rates are lower due to the near normal wave incidence angle.

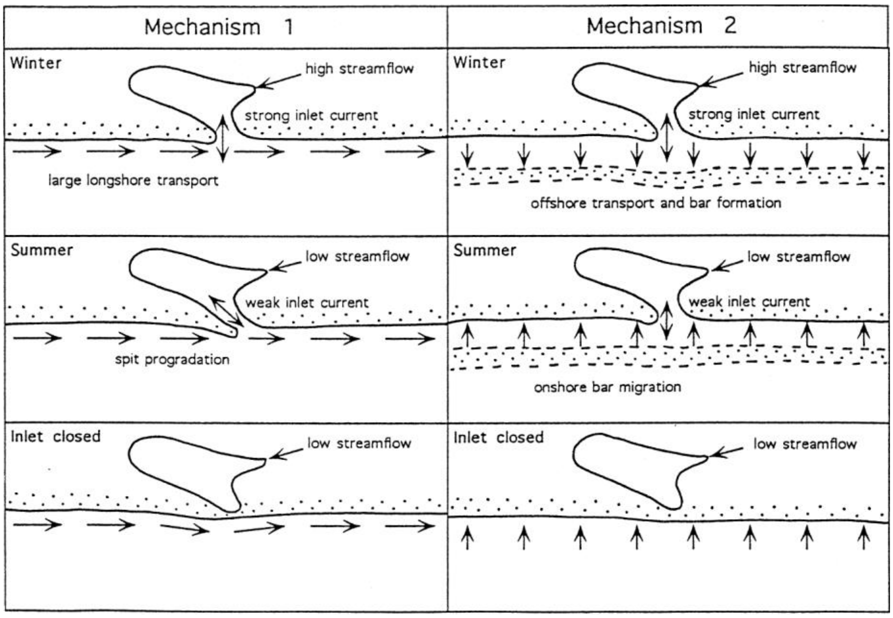


Figure 2-6: Schematic representation of inlet closure mechanisms (Ranasinghe et al., 1999)

Huizinga (2000, cited in Zietsman, 2004) similarly attributes longshore sediment transport to be responsible for inlet sedimentation at larger South Africa estuaries. Huizinga & Van Niekerk (2002, cited in Zietsman, 2004) reported cross shore sediment transport to be the main driving force behind inlet sedimentation among smaller South African estuaries. Findings indicate that South African estuaries often experience inlet closure during sea storm events and not gradually after the storm has subsided, to the contrary of Mechanism 2 described by Ransinghe (1999) (Whitfield & Bate, 2007).

From the above mentioned mechanisms, it is clear that mouth closure is highly dependent on the following processes: inlet currents, river flow, longshore sediment transport, cross shore sediment transport and wave action. Additionally, the availability of sediment within the mouth region significantly contributes towards the closure potential of the inlet.

2.5.2. Governing processes and major forces

After breaching, estuary mouth closure is caused by hydrodynamic processes that deposit sediment in the inlet and rebuild the sand bar. The major factors responsible for estuary inlet closure along the South African coastline are wave energy and sediment availability.

2.5.2.1. Wave energy

Wave action is considered the most important process contributing to sediment accumulation at an estuary inlet. A general assumption is the higher the wave energy at the inlet, the greater the inlet sensitivity to closure. Turbulent wave action causes the suspension of marine sediment in the nearshore zone. The suspended sediment is transported through the inlet by means of tidal flow. Reduced current velocity and turbulence cause the sediment to settle and accumulate in the estuary inlet. Net accumulation of sediment within the inlet will occur in the event of inadequate tidal currents during ebb-tide and low river inflow (Whitfield *et al.*, 2012).

Hydrodynamic conditions at estuary inlets are complex due to the interaction of tidal currents, wave action, wave induced currents and wind stress currents. Understanding the effect of wave action on estuary inlet closure is crucial to determine the likelihood and rate of mouth closure. The nearshore wave climate is dependent on the deep-water wave conditions, as well as the near-shore wave processes. The most important parameters describing the wave characteristics are the wave height, -direction and -period. These parameters can describe the intensity of the near shore wave action at the beach, and with it the associated sediment transport capabilities.

The distribution of offshore wave height along the South African coastline can provide a first estimate to the wave energy at a specific estuary inlet. The predominant direction of the offshore waves may also provide insight toward the wave exposure of the estuary inlet. Rossouw and Theron (2009) determined the regional offshore wave climate along the South African coastline by evaluating approximately 11 years of WaveWatch III forecast model data (Tolman *et al.*, 2002) of the National Centre for Environmental Predictions (NCEP) (NCEP, 2013). The median significant wave heights along the South African coastline for the 50% and 1% exceedance probability, as well as the most probable range of peak wave periods, are provided in Figure 2-7. A decreasing gradient in offshore wave height is evident when moving from south to north along both the east- and west coast.

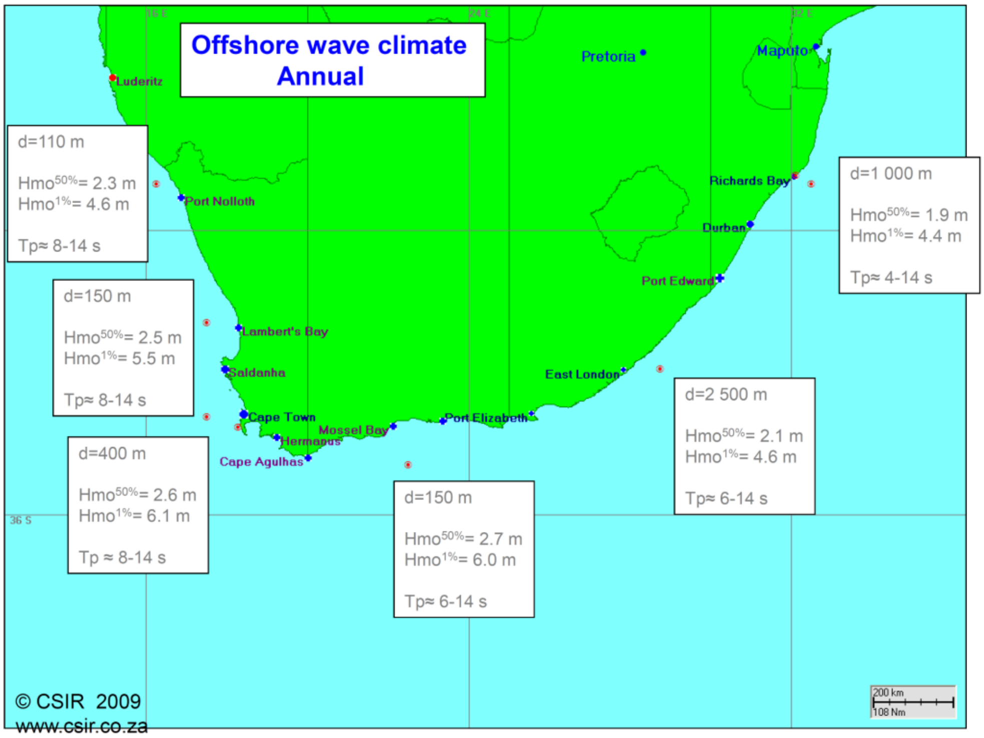


Figure 2-7: Overview of offshore wave height and -period along the South African coastline (Theron, 2016)

The wave roses in Figure 2-8 illustrate the annual variation in offshore wave directionality along the South African coastline. The predominant wave direction along the entire coast is south-west.

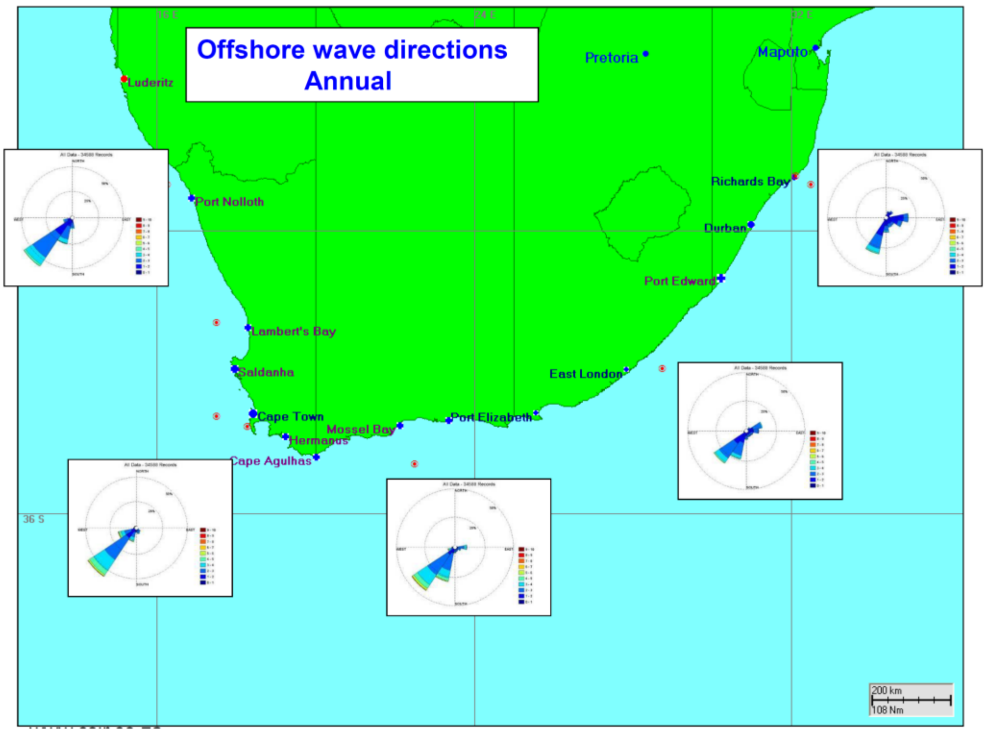


Figure 2-8: Overview of offshore wave direction along the South African coastline (Theron, 2016)

Van Niekerk *et al.* (2012) suggests the following co-factors should be considered when assessing the wave energy and sediment availability at an estuary inlet, viz. beach profile slope, sediment grain size, surf zone width, beach type, tidal flow and berm height. These factors provide an indication on the sediment transport potential, and resultant inlet closure potential, due to wave action at the inlet.

<u>Beach profile slope</u>: Wave breaker type, nearshore currents and backwash are all directly dependent on the profile slope of the beach adjacent to the inlet. A steeper beach slope may suggest more severe surf conditions and higher sediment entrainment (Battjes, 1974).

<u>Sediment grain size</u>: The profile slope is typically dependent on the sediment grain size (Wiegel, 1964). Coarse grain sediment is generally associated with steeper beaches, while finer grain sediment is associated with gentler sloped beaches.

<u>Beach type</u>: Exposed wave dominated beaches can generally be categorised according to one of six types, viz. dissipative, intermediate (4 classes) and reflective beaches. Dissipative beaches comprise flat beach slopes that aid in gradually dissipating the wave energy across the surf zone, while reflective beaches have steeper slopes accompanied by surging breakers that reflect most of the wave energy (Whitfield & Bate, 2007).

<u>Surf zone width</u>: Van Niekerk *et al.* (2012) suggests a similar correlation between the surf zone width and surf conditions. The surf zone width provides an indication of the potential wave energy available at the shoreline for the transport of marine sediment into the estuary. A narrow surf zone is typically associated with higher wave energy and turbulence near the shore, causing high volumes of suspended sediment to be transported to the inlet. A wider surf zone is typically associated with higher rates of wave energy dissipation, causing lower volumes of suspended sediment near the shore. Along the South African coastline steeper beach slopes are generally associated with relatively narrower breaker zones and coarser sediment, compared to gently sloping beaches.

<u>Wave breaker type</u>: The breaker type describes the wave form during incipient breaking and can be classified as four types: surging, collapsing, spilling and plunging breakers (CEM, 2006: II-4-1). The breaker type is correlated to the surf similarity parameter (Iribarren number) which is a function of deep water wave height and wave length, as well as the beach profile slope. Spilling breakers often occur for high wave steepness on gently sloped beaches, while plunging breakers commonly occur on steeper beaches with intermediate wave steepness. Surging and collapsing breakers occur during low wave steepness on steep sloped beaches. Spilling breakers gradually dissipate wave energy across the surf zone and are generally ineffective in suspending sediment compared to plunging or collapsing breakers. Plunging breakers are most effective in sediment suspension due to the concentrated motion of the crest towards the bottom, causing scour of sediment. Collapsing or surging breakers also tend to dissipate high amounts of energy near the shoreline.

Berm height: Berm crest elevations exceeding +3.0 m MSL indicate high wave energy along with high volumes of suspended sediment, resulting in rapid inlet closure compared to lower berm levels (Van Niekerk *et al.*, 2012). The berm dimensions also directly impact the likelihood and frequency of marine overwash.

There are several anthropogenic influences that may inhibit tidal or river flow and ultimately contribute to mouth closure. Structures such as bridges and causeways can cause reduced ebb-tidal flow that leads to sediment accumulation in the lower reaches of the estuary (Whitfield & Bate, 2007).

2.5.2.2. Sediment transport

Nearshore sediment transport is generally categorised as longshore sediment transport, or cross-shore sediment transport. Both components drive shoreline behaviour (erosion/accretion) over long- and short-term periods. Fluvial sediment supply also plays a role in the morphology of the lower reaches of the estuary, as well as in the near-shore region.

The sediment availability at the estuary mouth is dependent on the longshore- and cross-shore sediment transport rates. The ingress of marine sediment at an estuary is also dependent on the transport capacity of the flood and ebb tidal flows. Marine sediment availability can vary greatly among estuarine inlet locations. Predominantly rocky shorelines often have lower sediment availability compared to abundantly sandy shorelines. Well sheltered areas inside bays and/or protected by headlands or reefs may also be associated with limited marine sediment availability, due to low transport potential towards the inlet (Whitfield & Bate, 2007).

Longshore sediment transport

Longshore sediment transport is the process whereby sediment is moved parallel to the coastline due to wave and current action. Waves break in the surf zone at an angle oblique to the coastline and cause longshore currents which flow along the length of the beach. This results in suspension and movement of sediment along the coastline. The longshore sediment transport process and boundaries are illustrated in Figure 2-9.

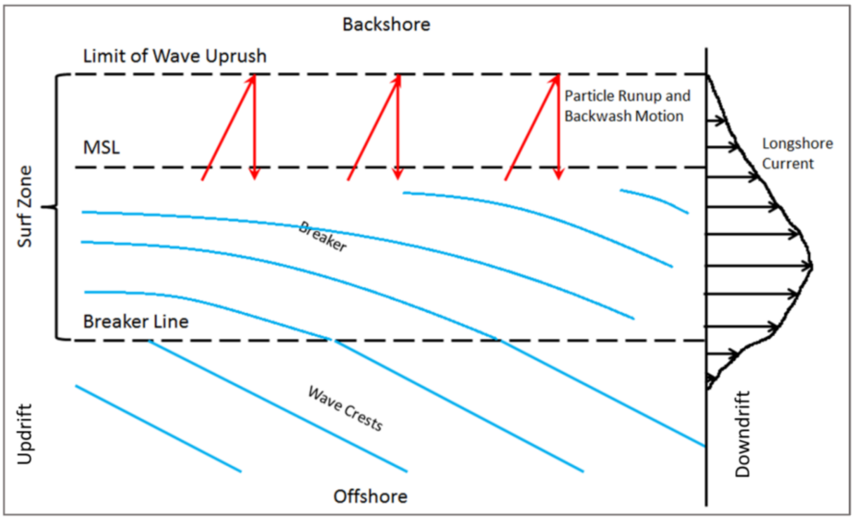


Figure 2-9: Schematic depiction of wave generated longshore sediment transport (Schoonees, 2016)

The variation of net longshore sediment transport rates along the South African coastline are provided in Table 2-4 (Schoonees, 2016). The net longshore sediment transport is generally in an upward (north) direction along both the west- and east coast of South Africa. Rocky and sheltered sections of shoreline can experience an estimated net longshore sediment transport rate between 10 000 m^3 and 400 000 m^3 per annum (Whitfield & Bate, 2007).

Table 2-1: Approximate net longshore sediment transport rates for South Africa (Adapted - Schoonees, 2016)

Location	Annual net longshore transport rate (m^3/year)
Walvis Bay	860 000
Northern False Bay	100 000 – 150 000
Port Elizabeth	150 000
East London	300 000 – 500 000
Durban	500 000
Richards Bay	850 000

Cross-shore sediment transport

Cross-shore sediment transport is the movement of beach- and nearshore sediment perpendicular to the shoreline by the forces generated by waves, wind and tides. This complex movement of sediment consists of an onshore and offshore component at any given time. Sediment is predominantly transported in suspension within the surf zone, as opposed to bed load transport. Extreme cross-shore sediment transport rates were reported to reach highs of approximately 150 000 m^3 over a period of two days during large sea storms acting on a 500m stretch of South African shoreline (Whitfield & Bate, 2007). Similarly, high rates of 100 000 m^3 have been recorded on the shoreline of the relatively sheltered Durban bight during a storm event. The wave-exposed South African coastline experiences high amounts of cross-shore transport with average rates in the order of a few m^3/m per hour (Whitfield & Bate, 2007). The cross-shore sediment transport process is illustrated in Figure 2-10.

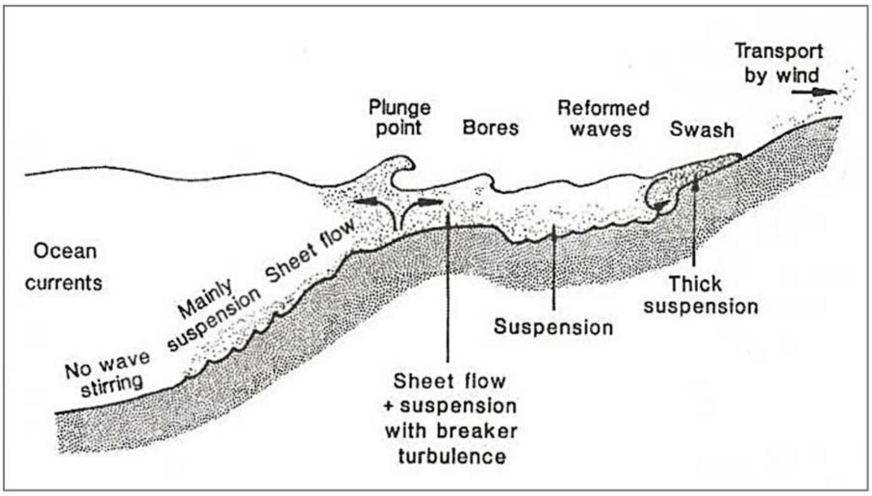


Figure 2-10: Schematic depiction of cross shore sediment transport in the nearshore zone (Nielsen, 2009)

Fluvial sediment supply

Fluvial sediment often accumulates outside the estuary mouth after major flood events, forming a temporary ebb-shoal. A portion of the sediment accumulated on the ebb-shoal is then transported back into the inlet during energetic wave conditions. It is however unlikely that the fluvial sediment will directly cause mouth closure, prior to exiting the mouth. Finer fluvial sediment is most commonly flushed from the estuary. Fluvial sediment yield for TOCEs on the KwaZulu-Natal coastline ranges from approximately 1 800 tonnes per annum for smaller estuaries (Siyai and Ku-boboyi estuaries), all the way up to 1.7 million tonnes per annum for large catchment estuaries (Mgeni Estuary) (CSIR, 1990, cited in Zietsman, 2004).

2.6. Estuarine Berms

For the purpose of this study, a distinction is made between the processes governing estuary inlet closure (§ 2.5.2) and the processes involved in berm growth (horizontal and vertical) following estuary closure. Initial closure of estuary inlets is governed by processes occurring below the high tide level, while longer term variation in berm crest elevation is primarily dependent on wave runup levels and swash processes (Wainwright *et al.*, 2013). However, wave action and sediment transport (§ 2.5.2) are considered primary drivers for inlet closure and berm growth after closure. Berm growth may also be aided by aeolian sediment transport during extended periods of mouth closure. In some instances, aeolian sediment transport can build up the inlet berm to levels exceeding that of maximum runup elevation.

2.6.1. Background and processes

Berms form after the closure of estuary inlets and act as a barrier, separating the two water bodies. Berm growth is triggered by the deposition of sediment at the landward extent of wave runup. This often results in an increasing gradient on the seaward side and a gentle gradient on the landward side (Weir *et al.*, 2006). The rate of berm growth/recovery is dependent on the sediment transported by wave runup, which in turn is governed by swash zone (upper beach) sediment transport processes. According to Hine (1979), vertical berm growth is primarily driven by wave action and requires that wave runup overtop the berm, causing sediment deposition beyond the existing crest. Figure 2-11 illustrates the nearshore zone and defines the swash zone boundaries.

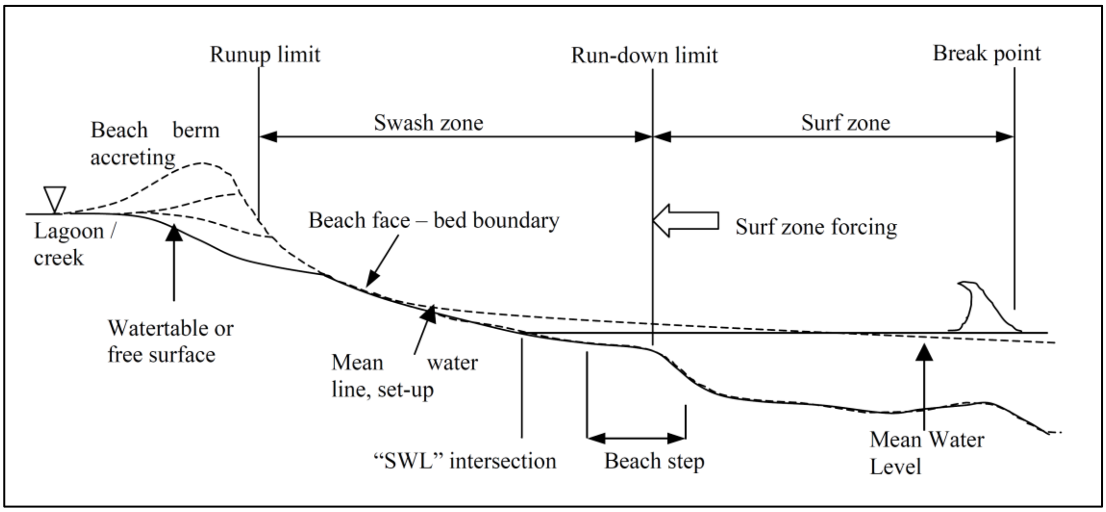


Figure 2-11: Definition sketch of nearshore zone fronting an estuary (Baldock et al., 2008)

The flow velocity of wave runup decreases towards the upper beach face, resulting in the deposition of marine sediment and ultimately berm accretion. Sediment deposition on the beach face is also triggered by the rapid drainage (percolation) of runup into the beach sediment. The rate of percolation is dependent on the beach face slope and the permeability of the beach sediment.

2.6.1.1. Swash zone hydrodynamics

The physical boundaries of the swash zone are dynamic, however the local swash zone may be defined by the limits of wave runup and run-down around the Still Water Level (SWL). The dominant boundary conditions for the swash zone are similar to the hydrodynamic drivers in the inner surf zone viz., wave height, wave period, wave shape, spectral bandwidth, orbital velocities, currents, turbulence, and beach slope/composition (Elfrink & Baldock, 2002).

Elfrink and Baldock (2002) suggests that swash motion may result from both infragravity standing waves and short waves, with the dominant form of motion dependent on the surf zone conditions, which can be described qualitatively by the Iribarren number.

The sediment transported in the swash zone can be carried as suspended load and bed load, with the respective quantities dependent on the hydrodynamic conditions (Pritchard & Hogg, 2005).

2.6.1.2. Wave runup

Runup is defined as the maximum instantaneous elevation of wave uprush above the still water level (Figure 2-12) (CEM, 2006: II-4-14). Quantifying the runup elevation on the beach face fronting an estuary is an important step towards predicting the landward limit of sediment deposition and ultimately berm crest elevations. Wave runup is primarily a function of the following parameters: wave height, - direction, - period, - breaker type, surf zone width, inshore/nearshore roughness and permeability, profile slope, and wave height distribution (Battjes, 1974).

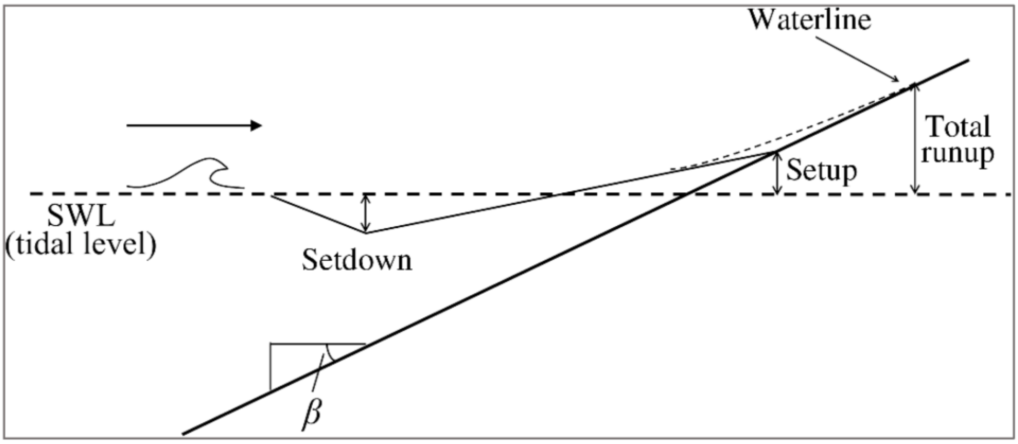


Figure 2-12: Schematic depiction of wave runup

2.6.1.3. Iribarren number

Early studies implemented the use of a surf similarity parameter, also known as the Iribarren number, to assist in quantifying several nearshore wave processes of which wave runup was one (Nielsen & Hanslow, 1991). The Iribarren number (Eq 2.1) can be defined by an offshore parameter (Eq 2.2) and an inshore parameter (Battjes, 1974).

$$\xi_0 = tan\alpha \left(\frac{H_0}{L_0}\right)^{-\frac{1}{2}} \tag{2.1}$$

$$\xi_b = tan\alpha \left(\frac{H_b}{L_b}\right)^{-\frac{1}{2}} \tag{2.2}$$

where

- α = beach face slope
- H_0 = deep water wave height (m)
- H_b = nearshore wave height prior to breaking wave height (m)
- L_0 = deep water wavelength
- L_b = wavelength associated with H_b

The Iribarren number also gives indication of the wave breaker type. Table 2-2 provides the wave breaker type criterion for inshore- and offshore Iribarren numbers.

Table 2-2: Wave breaker type criterion for Iribarren number (Battjes, 1974)

Wave breaker type	ξ_0 range	ξ_b range
Spilling	$\xi_0 < 0.5$	$\xi_b < 0.4$
Plunging	$0.5 < \xi_0 < 3.3$	$0.4 < \xi_b < 2.0$
Surging or collapsing	$\xi_0 > 3.3$	$\xi_b > 2.0$

The Iribarren number and associated wave breaker type are also typically associated with a specific beach type. Spilling breakers are typically associated with more dissipative conditions, while surging breakers are often associated with more reflective conditions.

2.6.1.4. Coastal overwash

Donnely *et al.* (2004) defines coastal overwash as the flow of water and sediment over the beach crest that does not return directly to the sea. Coastal overwash occurs when the wave runup level and/or the mean water level exceeds the beach crest. Donnelly *et al.* (2004), as well as Donnelly (2007), specifically focus on the prediction of storm overwash.

Donnelly *et al.* (2004) provides a clear distinction between overwash caused by wave runup and overwash caused by water levels (storm surge and wave setup included). The former is termed “runup overwash”, and the latter “inundation overwash”. Both regimes have a likelihood of occurring during a single storm event and often occur in combination in South Africa.

Donnelly (2007) describes seven cross-shore profile responses to storm overwash: 1) crest accumulation; 2) landward translation/migration; 3) dune lowering; 4) dune destruction; 5) barrier accretion; 6) barrier rollover; and 7) barrier disintegration. A schematic representation of the above-mentioned storm washover morphologies are presented in Figure 2-13.

The specific cross-shore profile response is primarily distinguished by the following coastal parameters: maximum surge and runup elevation, storm overwash duration, beach/berm crest height, dune/berm width and -height (Donnelly, 2007). Severe overwash caused by large storms can be a precursor to marine breaching from the seaward side, initiated by erosion of the beach face and lowering of the berm crest elevation.

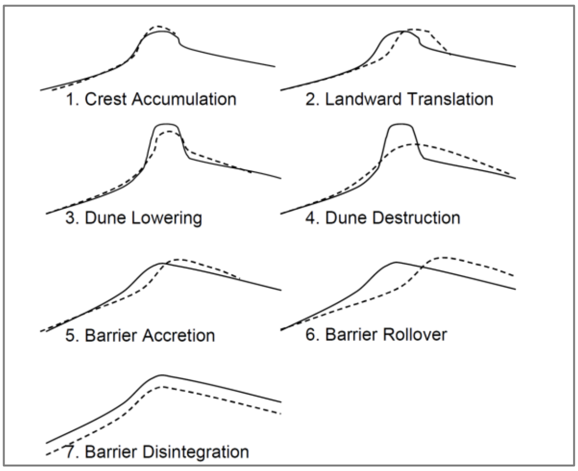


Figure 2-13: Cross-shore profile response to storm overwash. Dotted line indicates post storm profile (Donnelly, 2007)

The focus is often placed on the erosion of coastal barriers subjected to storm overwash, however it is important to distinguish between the morphological response associated with storm overwash and non-storm overwash. The mechanisms of berm growth caused by non-storm overwash are discussed in § 2.6.2.

2.6.1.5. Onshore versus offshore movement of sediment

The dependence of berm crest elevation on wave runup leads to a phenomenon known as the "berm-height paradox". Increased offshore wave height leads to an increase in wave runup and consequently higher berm crest elevations, however the largest waves associated with storm events lead to beach profile erosion which results in a decreased berm crest elevation (Bascom, 1953).

Accurate prediction of the direction of cross-shore sediment transport (onshore vs offshore) is a crucial aspect of understating this paradox. Several studies attributed the relationship between onshore and offshore sediment movement to a critical value based on the deepwater wave steepness and the sediment fall velocity (e.g. Dean, 1973; Kraus *et al.*, 1991).

Kraus *et al*. (1991) evaluated the capability of several popular simple erosion accretion criteria. The results concluded that criteria related to the suspension of sediment through wave energy dissipation (e.g. Dean, 1973) performed reasonably well in predicting the direction of cross-shore sediment transport.

Jackson (1999) evaluated the above-mentioned criteria for predicting erosion and accretion, based on their predictive applicability at an estuarine sandy beach. The results concluded that the methods previously derived from field data fail to predict erosion events on an estuary fronting beach. Jackson suggests that the relation of Dean (1973), based on small scale physical models, provides a more accurate prediction.

The criterion developed by Dean (1973) assumes the majority of sediment transported within the surf zone is as suspended load. The criterion separating offshore (erosion) an onshore (accretion) is defined by Equation 2.3:

$$N_0 = \frac{H_0}{wT} \tag{2.3}$$

where

- H_0 = deep water wave height (m)
- T = wave period (s)
- w = sediment fall velocity (m/s)

The Dean number criterion employed for predicting the direction of cross-shore sediment transport is provided in Table 2-3.

Table 2-3: Dean number criterion for direction of sediment transport (Kraus et al., 1991)

Dean number	Direction of cross-shore sediment transport
$\boldsymbol{N_0 < 2.4}$	Accretion highly probable
$\boldsymbol{N_0 < 3.2}$	Accretion probable
$\boldsymbol{N_0 \geq 3.2}$	Erosion probable
$\boldsymbol{N_0 > 4.0}$	Erosion highly probable

2.6.1.6. Aeolian sediment transport

During prolonged periods of mouth closure, the potential berm crest elevation is not only dependent on the runup and swash zone interactions, but also aeolian sediment transport. Aeolian sediment transport can lead to berm accretion during extended periods of mouth closure.

The wind transports the unconsolidated beach sediment in the direction of the predominant wind direction. Vegetated areas act as a trap for wind-blown sediment. The presence of vegetation near or on the estuarine berm may lead to an increased berm height.

The Klein Estuary is an example of where aeolian sediment transport plays a role in the berm development process. The coastline orientation (northwest - southeast) at the estuary corresponds with the prevailing wind direction, resulting in aeolian sediment movement along the beach.

Artificial stabilisation of the berm/dune has led to the development of a sediment trap in some areas. The gradual build-up of sediment in these areas may prevent or complicate natural- and artificial breaching (Bickerton *et al.*, 1983).

For the purpose of this particular study, the effect of aeolian sediment transport on possible berm crest elevations will not be quantified. However, the effects will be considered in a qualitative sense where applicable.

2.6.2. Mechanisms of berm growth

Hine (1979) identified three mechanism of berm growth based on field observations of a barrier spit complex. The first mechanism describes sediment accumulation at the high tide swash mark during neap tide at areas with ample net longshore transport, resulting in a neap berm. The neap berm is then redistributed over the main berm by means of swash occurring during spring high tide, ultimately resulting in the vertical- and horizontal growth of the main berm. The second mechanism involves the development of an intertidal swash bar which migrates onshore to weld on to the existing berm and beach face, resulting in a widened berm. The third, almost similar, mechanism is caused by the rapid vertical growth of intertidal swash bars. Overtopping of these high bars are inhibited during neap tide, resulting in a steepened seaward slope. Eventual infilling of the landward formed runnel during spring tides subsequently develops a wide berm ridge. All these mechanisms are attributed to the difference in tidal elevations over the spring-neap tidal cycle.

Weir *et al*. (2006) suggests the influence of tide on the maximum berm elevation is not limited to the occurrence of accretion relative to the stage of the spring-neap tidal cycle. Variation in these mechanisms are attributed to the fluctuating longshore sediment availability along the length of the barrier spit where the measurements were taken. Contrarily, berm development at an estuary inlet focusses specifically on the processes at the given location (limited variability along the length of the berm).

A subsequent study conducted by Weir *et al*. (2006) explored the modes of berm development at a closed inlet of a coastal lagoon. The findings are more relevant to this particular study due to the fact that the field experiments were conducted specifically at a closed coastal lagoon inlet on the wave-dominated, micro-tidal New South Wales coastline.

Mechanism 1 - Vertical growth and swash overtopping: This method was documented two weeks after an artificial breaching, just prior to estuary closure. Erosion of the lower beach face is evident, while the upper beach face and berm crest experiences accretion. The boundary between erosion and accretion is most evident during periods of rising and high tide, due to the presence of substantial swash overtopping. A significant deposition of sediment landward of the berm crest is also evident, which is dependent on the overtopping rate of the berm. This mechanism is comparable in certain respects to the mechanisms described by Hine (1979).

The difference is the fact that berm overtopping is not restricted to spring tides, because of the presence of the estuary inlet.

Mechanism 2 - Horizontal growth and absence of swash overtopping: This method was documented during an extended period of inlet closure, with initial berm crest elevation close to the modal elevation. Deepwater wave conditions varied substantially during the measurement campaign. This mechanism involves sediment erosion from the lower swash zone which is deposited in the mid to upper swash zone, resulting in a neap berm below the primary berm. The effect is a steepened beach face along with a more convex beach profile shape, as well as horizontal growth of the principle berm (below crest). Mechanism 2 involves significantly less sediment accumulation rates compared to mechanism 1. This method is consistent with the findings of Hine (1979).

The prevailing mechanism is dependent on the presence or absence of swash overtopping at the berm crest during the time of sediment accumulation. This in turn is a result of the tidal phase, wave runup levels and existing berm crest elevation (Weir *et al.*, 2006).

Matias *et al*. (2010) suggests that the morphological berm response varies according to the relevant washover morphology present during non-storm overwash, i.e., washover plain versus washover lobe. Washover plains are typically associated with wide low-lying denuded areas such as estuarine berms, while washover lobes are features that cut through the dune field. The morphological barrier response on washover plains can be divided into three stages namely: 1) crest accretion during mid flood tide to high tide; 2) onshore sediment transport during high tide; and 3) dynamic equilibrium during high tide to mid ebb tide. The crest accumulation during the first stage is initiated as the runup starts to overtop the existing crest during mid tide. As the tide rises the amount and velocity of overwash increases, leading to an onshore transport of sediment beyond the existing crest, on the leeward side of the berm (stage 2). As the tide starts to drop, a state of dynamic equilibrium is reached and the morphological changes become negligible (stage 3) (Matias *et al.,* 2010).

2.6.3. Previous studies relating to berm height prediction

Early studies on berm development proposed a simple relationship between the berm crest elevation and the offshore wave height (Bagnold, 1940; Bascom, 1953). Bagnold suggested a berm height equal to 1.3 x H_o, based on physical model experiments in a wave channel. This relationship leads to the conclusion that berm crest elevation is higher on exposed beaches, compared to wave protected areas.

Subsequent studies explored the relationship between berm height and wave height, as well as wavelength (or period) (Rector, 1954; Takeda & Sunamura, 1982). There remains some lack of clarity regarding the effect of sediment grain size on berm height from the above mentioned literature.

Okazaki and Sunamura (1995) implemented the use of a reduction factor when calculating berm height, to compensate for the bed roughness and permeability (both functions of sediment grain size). The roughness and permeability of a beach should, to a certain extent, control the berm height.

More recent attempts at predicting the berm crest levels at estuary inlet employ the use of existing parametric runup models (e.g. Nielsen & Hanslow, 1991). The runup models are used as part of larger statistical or process-based parametric models, aimed at quantifying inlet berm crest levels and estuarine mouth functioning (Baldock *et al*., 2007; Wainwright *et al.*, 2013; Behrens *et al.*, 2015).

Processes relating to beach erosion have been well described by numerical models, however the processes involved in beach accretion are mainly excluded from these models (Schoonees & Theron, 1995; Elfrink & Baldock, 2002).

2.6.4. Existing runup and berm height parameterisations

Empirical parameterisations of runup can provide relatively accurate prediction in the absence of comprehensive wave- and hydrodynamic modelling. Empirical models require limited input data and resources, making it an attractive alternative for estimating wave runup along the South African coastline. The prediction of wave runup can potentially identify the landward limit of sediment transport, and therefore the vertical extent of potential sediment deposition.

Additionally, authors have focussed on the development of predictive models of the upper beach profile limits (berm heights) (Swart, 1974; Okazaki & Sunamura, 1995). These models are also based on the assumption that the berm crest is limited to the elevation up to which waves are able to transport sediment.

2.6.4.1. Wave runup predictors

Theron (2016) conducted an evaluation of 13 popular semi-empirical runup prediction methods in order to determine a most accurate and applicable model for the South African coastline. The methods considered for evaluation were: Battjes (1971); Nielsen and Hanslow (1991); Ahrens and Seelig (1997); two models by Ruggiero *et al*. (2001); Erikson *et al.* (2007) version of a third model originally by Ruggiero *et al.* (2001); Guza and Thornton (1982); Mase (1989); two models by Stockdon *et al*. (2006); Priestley's (2013, cited in Theron, 2016) version of Stockdon *et al.* (2006); Diaz-Sanchez *et al.* (2013); and Mather *et al.* (2011).

The models of Battjes (1971) and Ahrens and Seelig (1996) were immediately discarded due to further development by later authors and impractical data requirements respectively. The remaining 11 models were tested against four sets of available data. The data was acquired during a four-day measurement campaign on two separate beaches in the Koeberg-Melkbos area. The beach slopes were 1 in 11 and 1 in 25, indicating intermediate and dissipative conditions respectively (Theron, 2016). The predicted wave runup elevations (2% exceedance) for each of the 11 models were compared to the four data sets. Five of the most accurate models were selected for further testing and validation against field data representing a wider range of South African conditions (Table Bay and KZN). These models were: Nielsen and Hanslow (1991); Ruggiero *et al.* (2001); Stockdon *et al.* (2006); Mather *et al.* (2011); and Diaz-Sanchez *et al.* (2013).

These five models were finally narrowed down to the three best models available for South African application. Models were ranked based on their respective Root Mean Square Error of Prediction (RMSEP) when modelling the above-mentioned datasets, which cover diverse locations and a wide array of conditions. The three best performing models were: Nielsen and Hanslow (1991), Ruggiero *et al*. (2001) and Mather *et al.* (2011). These three models, along with the Stockdon *et al*. (2006) model are further investigated in this section. The Stockdon *et al.* (2006) model was not disregarded, due to its performance reported by Matias *et al.* (2013), for overwash predictions on sandy barriers.

Nielsen and Hanslow (1991)

Nielsen and Hanslow investigated wave runup on 6 sandy beaches on the Coast of New South Wales, with profiles ranging from low gradient (dissipative) to steep gradient (reflective) beaches. The data indicated the Rayleigh distribution to be the most accurate statistical model for the maximum wave runup elevations on a wide range of sandy beaches, ranging in beach face slope from 0.026 to 0.19.

Two runup formulae were developed by Nielsen and Hanslow (1991), with selection dependent on the beach face slope.

For tan(α) > 0.1:

$$R_{2\%} = SWL + 1.98[0.6(H_{0rms}L_0)^{0.5}tan(\alpha)] \tag{2.4}$$

For tan(α) ≤ 0.1:

$$R_{2\%} = SWL + 1.98[0.05(H_{0rms}L_0)^{0.5}] \tag{2.5}$$

where

- SWL = Still Water Level (m)
- H_{0rms} = deep water root mean square wave height (m)
- L_0 = deep water wavelength (m)
- α = beach face slope (measured in the foreshore)

The model implements beach face slope, ensuring easy measurement even under storm conditions. However, the ideal measurement of beach slope required for the calculation of wave runup should account for the slope responsible for the entire wave transformation process in the outer surf zone inwards.

Nielsen and Hanslow concluded that for flatter beach slopes, the runup level is no longer proportionate to the beach slope. The findings of Theron (2016) indicate a lack of accuracy when implementing the Nielsen and Hanslow model on low gradient beach slopes. The threshold value (tan α = 0.1) for determining the applicable formulae is open to further interpretation. The existing value is based on the data available at the time of the study (Nielsen & Hanslow, 1991).

Model evaluations conducted by Theron (2016) concluded that the Nielsen and Hanslow (1991) model was the most consistent performer overall, however the model performed less than satisfactory for beach slopes where (tan α) ≤ 0.06. Theron (2016) affirmed that the model (Eq. 2.6) performance can be improved by modifying the formula coefficient from 0.05 to 0.04. The modified version of the Nielsen and Hanslow (1991) formula for tan(α) ≤ 0.06 is as follows:

$$R_{2\%} = SWL + 1.98[0.04(H_{0rms}L_0)^{0.5}] \tag{2.6}$$

Ruggiero *et al.* (2001) Model 1

Ruggiero *et al.* (2001) developed a runup prediction method based on experiments undertaken on several beaches in order to relate runup elevation to the local beach morphology and deep water wave conditions. Ruggiero *et al.* (2001) comments on the versatility of the model, despite the fact that the application is presented for the dissipative beaches of Oregon. The runup elevation can be estimated according to Equation 2.7:

$$R_{2\%} = 0.5H_0 + 0.22 \tag{2.7}$$

where

- H_0 = deep water significant wave height (m)

Theron (2016) commented on the models accurate prediction for low gradient beach slope (highly dissipative) conditions. This model may be implemented as an alternative to the Nielsen and Hanslow (1991) model, in cases with low gradient beach slopes.

Ruggiero *et al.* (2001) Model 2

Ruggiero *et al.* (2001) developed an additional model, more suitable for reflective beaches. The model is also based on field data and is provided in Equation 2.8.

$$R_{2\%} = 0.27(\tan\alpha\, H_0 L_0)^{\frac{1}{2}} \tag{2.8}$$

where

- α = beach face slope
- H_0 = deep water wave height (m)
- L_0 = deep water wavelength (m)

The models are developed from wave measurements recorded in approximately 68 m water depth, that have been transformed to deep water equivalent wave conditions by using linear wave theory.

Stockdon *et al.* (2006)

Stockdon *et al.* (2006) developed a parameterised runup model by evaluating field observations. A numerical model was implemented to simulate storm driven runup. The simulation results were used to evaluate and improve the parameterisation. The final parameterisation (Eq. 2.9) for the 2% exceedance probability of runup on all natural beach types is:

$$R_{2\%} = 1.1\left(0.35\beta_f(H_0L_0)^{1/2} + \frac{\left[H_0L_0\left(0.563\beta_f^2+0.004\right)\right]^{1/2}}{2}\right) \tag{2.9}$$

where

- β_f = beach face slope (tanα)
- H_0 = deep water significant wave height (m)
- L_0 = deep water wavelength calculated from the peak wave period

Stockdon *et al.* (2006) tested the model performance against the available field measurements, resulting in an average under prediction of 17 cm.

The wave parameters utilised to develop the model were measured in water depths varying from 7 to 20 m, depending on the location. To ensure consistency among locations, the wave recordings were transformed to deep water equivalent wave heights using linear wave theory.

Mather *et al.* (2011)

Mather *et al.* (2011) developed a simple empirical (Eq. 2.10) model for predicting extreme wave runup on natural beaches during severe wave conditions. The model steers clear of traditional runup prediction parameters (beach face slope and Iribarren number), instead opting for the offshore distance to the depth of closure to estimate a nearshore profile slope. The model was tested against data acquired on the South African coastline.

$$\frac{R_x}{H_0} = C\left(\frac{h}{x_h}\right)^{2/3} \tag{2.10}$$

where

- x_h = distance offshore to closure depth (m)
- h = closure depth (15 m in this case)
- H_0 = deepwater significant wave height (m)
- C = dimensionless coefficient

The dimensionless coefficient (C) ranges from 3 to 10 and is used to describe the wave runup on three different coastline types namely: open coastline (C = 7.5), large embayments (C = 5) and small embayments (C = 4) (Mather *et al.* 2011). Thus, the formula for calculating the 2% runup elevation can be written as:

$$R_{2\%} = WL + C\,H_0\left(\frac{15}{x_h}\right)^{2/3} \tag{2.11}$$

It is worth noting that even though this is one of the newer models, there are still significant shortcomings. For instance, the model does account for the lack or presence of dunes on the upper beach slope (Theron, 2016).

Theron (2016) suggested the model to be more applicable along open and exposed coastal areas, generally with steeper beach slopes and reflective conditions. Model evaluation conducted by Theron proposed slightly different "C" coefficients than provided by Mather *et al.* (2011). A coefficient value of 7.5 should be set for open coast locations, as well as semi-exposed locations. A provisional value of 5 should be selected for well sheltered locations (e.g. behind headlands and deep inside bays).

Roux (2015) investigated the relationship between the coefficient (C) and the Iribarren number. Findings suggested that C-values between 3.0 – 5.0 are suitable for low Iribarren numbers (0.25 – 0.4), while C-values between 7 – 10 are suitable for higher Iribarren numbers (0.75 – 0.8). However, these recommendations are based on limited data and may require further investigation.

2.6.4.2. Berm height predictors

The models of Swart (1974) and Okazaki and Sunamura (1995) provide quantitative predictions of berm crest elevations. These models assume that the upper limits of beach profile change (erosion or accretion) occurs at the landward extent of wave runup.

Swart (1974)

Swart (1974) derived a formula to determine the upper limit of the "D-profile" of a beach. The method was derived from laboratory test results and aims to define the upper boundary at which wave action causes sediment transport. The upper limit of the "D-profile" relates well to the berm crest elevation, seeing as both are dependent on the landward limit wave runup. Swart (1974) proposed the following method to determine berm crest elevation (B_c):

$$B_c = D_{50}(7644 - 7706e^{-A}) \tag{2.12}$$

where

$$A = 0.000143\frac{H_0^{0.488}T_p^{0.93}}{D_{50}^{0.786}} \tag{2.13}$$

and

- D_{50} = average grain size of sediment present in berm (m)
- H_0 = deep water significant wave height (m)
- T_p = peak wave period (s)

Equation 2.12 implements the use of the median grain size to estimate the berm crest height, rather than the beach face slope typically used in the previously mentioned runup models. Swart (1974) employs the relationship between beach slope (tanα) and the median grain size (D_{50}) developed by Wiegel (1964). Swart developed the above equations from the results of 75 available model tests and 18 appropriate prototype situations. The applicability of the model is limited to the wave conditions and bed material present in these model- and prototype tests. Thus, the applicability of model has only been established for deep water wave heights between 0.97 m – 4.65 m, wave periods between 7 s and 9.5 s and median grain sizes ranging from 0.11 mm to 0.35 mm.

Larson and Kraus (1989)

Larson and Kraus (1989) presented an empirical model to predict the maximum subaerial elevation (above the SWL) of an equilibrium berm profile at a beach (shown in Eq. 2.14). The empirical relationship was obtained from analysing Large Wave Tank (LWT) data and by implementing the Iribarren number (Eq. 2.1). The LWT experiments implemented monochromatic waves, ensuring the absence of random wave phenomena such as wave grouping and long period wave motion. The absence of these phenomena allowed for a simplified approach which primarily focussed on sediment transported by short-period incident waves. Larson and Kraus reported a coefficient of determination of 75% for the 32 cases evaluated by means of Equation 2.14 (Larson and Kraus, 1989).

$$\frac{B_c}{H_0} = 1.47\left[\frac{tan\alpha}{\sqrt{H_0/L_0}}\right]^{0.79} \tag{2.14}$$

where

- B_c = maximum subaerial elevation of active profile, or maximum berm height (m above SWL)
- H_0 = deep water significant wave height (m)
- L_0 = deep water wavelength (m)
- α = beach face slope

Okazaki and Sunamura (1995)

Okazaki and Sunamura (1995) examined small scale wave flume experimental results to develop quantitative predictors for the position and height of berms in an equilibrium state. The model beach was subjected to monochromatic waves until the profile reached an equilibrium state.

The Okazaki and Sunamura model implemented the breaker type criterion of Galvin (1968) to classify berm behaviour according to only two of the four categories (§ 2.6.1.3): collapsing or surging related berms, as these were responsible for the most significant profile changes in the model (Okazaki and Sunamura, 1995).

Galvin (1968) differentiates between an offshore parameter, $H_0/(L_0m^2)$, and an inshore parameter, $H_b/(gmT^2)$, when classifying breaker type. These dimensionless parameters are based on combinations of beach slope (m), wave period (T), and either deepwater wave height or wave breaker height (H_0 or H_b). The Galvin (1968) wave breaker type criteria is summarised in Table 2-4.

Table 2-4: Galvin (1968) breaker type criteria

	Offshore parameter criteria	**Inshore parameter criteria**
Collapsing/Surging	$\frac{H_0}{L_0m^2} < 0.09$	$\frac{H_b}{gmT^2} < 0.003$
Plunging	$0.09 < \frac{H_0}{L_0m^2} < 4.8$	$0.003 < \frac{H_b}{gmT^2} < 0.068$
Spilling	$\frac{H_0}{L_0m^2} > 4.8$	$\frac{H_b}{gmT^2} > 0.068$

The berm height is parameterised (Eq. 2.15 & 2.16) as a function of wave height and wave period, as well as sediment grain size. A reduction factor (Eq. 2.17) and dimensionless sediment grain size relation (Eq. 2.18) are implemented to account for the roughness and permeability of the beach sediment. Beach profiles associated with collapsing related berms are characterised by a prominent step at the wave breaking point, while surging related berms have a less prominent step (Okazaki & Sunamura, 1995).

Berm height (B_h) for collapsing breaker related berms:

$$\frac{B_h}{(gT^2)^{5/8}H_b^{1/8}{D_{50}}^{1/4}\emptyset} = 0.117 \tag{2.15}$$

For surging breaker related berms:

$$\frac{B_h}{(gT^2)^{5/8}H_b^{1/8}{D_{50}}^{1/4}\emptyset} = 0.067 \tag{2.16}$$

where

- H_b = wave breaker height (m)
- T = wave period (s)
- g = gravitational acceleration (m.s^{-2})
- D_{50} = sediment grain diameter (m)
- $\emptyset$ = reduction factor to compensate for roughness and permeability of sediment

Reduction factor ($\emptyset$) is given by:

$$\emptyset = e^{(-0.04D_*^{0.55})} \tag{2.17}$$

where D_* is the dimensionless sediment grain diameter:

$$D_* = D_{50}\left[\frac{g\left(\frac{\rho_s}{\rho}-1\right)}{\upsilon^2}\right]^{1/3} \tag{2.18}$$

where

- D_{50} = sediment diameter (m)
- ρ_s = sediment density (kg.m^{-3})
- ρ = fluid density of seawater (kg.m^{-3})
- υ = kinematic viscosity of fluid (m^2.s^{-1})

Okazaki and Sunamura (1995) proposed the semi-empirical equation developed by Komar and Gaughan (1973), for the relationship between wave breaker height and deep water wave height.

$$\frac{H_b}{H_0} = 0.563\left(\frac{H_0}{L_0}\right)^{-1/5} \tag{2.19}$$

The applicability of Equation 2.15 and 2.16 were examined by comparing them to prototype scale data. The parameterisations were used to predict the berm height following a period of berm building wave conditions. The mean wave heights recorded in 21 to 28 m depth were transformed using the relationship developed by Komar and Gaughan (1973). The wave breaker height and period of the mean waves were averaged over the entire period of the berm building process. Berm measurements were conducted when near equilibrium profiles were reached. Okazaki and Sunamura proposed an adjustment of the constant in Equation 2.15 and 2.16, to better match the prototype results. The constant for both surging - and collapsing related berms are adjusted to 0.134. The comparison of the parameterisations to the prototype data concluded that no scale effects were involved in the experiment (Okazaki & Sunamura, 1995).

2.7. Conclusions from Literature Review

The literature review provides the relevant background information regarding South African TOCEs, as well as a comprehension of estuary mouth functioning.

It is important to distinguish between processes governing inlet closure and the processes involved in berm growth after closure. Initial closure of an estuary inlet is governed by processes occurring below the high tide level, while longer term variation in berm crest elevation is primarily dependent on wave runup levels and swash zone hydrodynamics. Aeolian sediment transport may also have a significant effect on berm accretion during extended periods of mouth closure. However, wave action and sediment transport are considered primary drivers for inlet closure and berm growth after closure.

The morphological response of berms subjected to wave action, as well as the relevant coastal drivers responsible for berm growth, have been discussed in the review. The morphological response of estuarine berms may vary significantly depending on the hydrodynamic conditions in the nearshore zone and the existing berm geometry.

There are relatively few available methods to predict the morphological response of estuarine berms subjected to wave action. Given the severe data limitations in South Africa, the most suitable methods of predicting berm geometry (crest height) are existing runup parameterisations. The landward extent of potential sediment transport is estimated by predicting the runup elevation at the berm. Suitable runup parameterisations include those developed by: Nielsen and Hanslow (1991); Ruggiero *et al.* (2001); Stockdon *et al.* (2006); and Mather *et al.* (2011). These models are widely known for their relative degree of performance. Additionally, the parametric models of Swart (1974), Larson and Kraus (1989) and Okazaki and Sunamura (1995), predict the upper limit of beach profile change, based on the landward extent of wave runup.

3. Selected South African Estuaries

A total of 20 study locations have been selected along the South African coastline in order to provide a wide sample of berm crest elevations. These include estuaries in the Western Cape, Eastern Cape and KwaZulu-Natal provinces. No Northern Cape estuaries were selected due to the lack of data pertaining to the TOCEs in the region. Selecting estuaries along the entire southern and eastern coastline provides a wide range of coastal variability to the sample sites. Thus, there is a large variation in the relevant hydrodynamic and morphological characteristics at the respective locations, which can aid to identify the relevant drivers and critical variables responsible for variation of berm crest elevations.

Estuaries have been selected primarily on the basis of data availability pertaining to berm levels. The second factor was to evaluate whether the estuary is classified as a TOCE, i.e., whether the estuary would experience periodic closure. A few of the systems considered are classified as coastal/estuarine lakes, however they too experience periodic closure. Permanently open estuaries may also have sand bars or berms present at the inlet, however the berm crest elevation is typically of minor importance due to the continuous open state of the estuary mouth. Historical data pertaining to estuary inlet berm crest elevation include berm/mouth surveys (§ 4.1) conducted by the Council for Scientific and Industrial Research (CSIR), and estuary water level data (§ 4.2) recorded by the Department of Water and Sanitation (DWS). The locations of the selected estuaries are displayed in Figure 3-1.

Figure 3-1: Locations of selected South African TOCEs

The relevant classifications and coordinates are provided in Table 3-1. The estuaries are mentioned in the order they appear when moving along the coastline from Cape Point to Richards Bay.

Table 3-1: Coordinates and classification of selected TOCEs

		Estuary	Estuary mouth coordinates	Classification
1	Western Cape	**Lourens**	34°05'58"S, 18°48'40"E	TOCE
2		**Palmiet**	34°20'37"S, 18°59'45"E	POE/TOCE
3		**Kleinmond**	34°22'06"S, 19°05'56"E	Est Lake
4		**Bot**	34°22'06"S, 19°05'56"E	Est Lake
5		**Onrus**	34°25'07"S, 19°10'47"E	TOCE
6		**Klein**	34°25'24"S, 19°18'13"E	Est Lake
7		**Hartenbos**	34°07'07"S, 22°07'27"E	TOCE
8		**Klein Brak**	34°05'31"S, 22°08'59"E	TOCE
9		**Groot Brak**	34°03'23"S, 22°14'25"E	TOCE
10		**Touw (Wilderness)**	33°59'51"S, 22°34'51"E	Est Lake
11		**Piesang**	34°03'37"S, 23°22'46"E	TOCE
12		**Groot (Natures Valley)**	33°58'52"S, 23°34'17"E	TOCE
13	Eastern Cape	**Tsitsikamma**	34°08'06"S, 24°26'18"E	TOCE
14		**Seekoei**	34°05'11"S, 24°54'30"E	TOCE
15		**West Kleindemonde**	33°32'28"S, 27°02'51"E	TOCE
16		**East Kleinemonde**	33°32'21"S, 27°02'55"E	TOCE
17		**Mngazi**	31°40'32"S, 29°27'40"E	TOCE
18	KZN	**Mhlanga**	29°42'14"S, 31°06'03"E	TOCE
19		**Mdloti**	29°39'07"S, 31°07'43"E	TOCE
20		**Tongati**	29°34'21"S, 31°11'07"E	TOCE

Additional estuaries were considered during the initial phases of this study, however these estuaries were disregarded on the basis of insufficient data availability. These estuaries include: Diep, Kabeljous, Ncera, Quinira, Bulura, Cefane, Mpenjathi, Mbokodweni, Mhlali, Nonoti and Zinkwazi. The collated data pertaining to these estuaries, available at the time of this study, are provided in Appendix B to Appendix E.

A brief qualitative overview of the selected estuaries is presented in § 3.1 through to § 3.3, to provide a basic high-level understanding of each system. This includes site specific details relevant to inlet berm formation and estuarine mouth functioning. By identifying these relevant site specific details, the drivers responsible for berm level variability among different estuaries can be determined.

3.1. Western Cape Estuaries

An overview of the selected Western Cape estuaries is presented in this section.

3.1.1. Lourens

General: The Lourens river flows through Somerset-West and connects to the sea at Strand, in the north-east of False Bay. The Lourens Estuary (Figure 3-2) is classified as a TOCE. However, a semi-closed mouth state is typically exhibited due to its small size and the frequent presence of a shallow, narrow opening which allows water to slowly flow out.

Figure 3-2: Lourens Estuary

Mouth dynamics: The estuary mouth does not breach as frequently as similar small systems. This is due to the presence of the small outflow channel that prevents water build-up in the estuary. However, during periods of high river inflow the water will dam up sufficiently to initiate a breach and scour the existing channel to form a wider inlet. Regular artificial breaching is not practised at this estuary. Previous surveys have measured the beach berm adjacent to the estuary inlet at 2 m high (Bickerton *et al.*, 1983).

Wave exposure: The estuary's location within False Bay results in relative protection from the dominant south-western swell at Cape Point. Nonetheless, the wave energy for the coast at Lourens Estuary was 20% above the mean wave height calculated for ten beaches situated near river mouths in False Bay. Average wave height was estimated at 1.02 m, generally classifying the wave energy as medium to high (Bickerton *et al.*, 1983).

Sediment availability: The inlet is located on an abundantly sandy shoreline, with generally wide beach widths. The vegetated sand dune, east of the inlet, can act as a sediment trap for wind- and wave driven sediment, consequently altering the mouth/berm morphology.

3.1.2. Palmiet

General: The Palmiet Estuary (Figure 3-3) is located between Betty's Bay and Kleinmond, approximately 75 km south-east of Cape Town. Similar to the Lourens Estuary, the Palmiet frequently displays a semi-closed mouth state. In some instances, the Palmiet Estuary is classified as a permanently open estuary, due to its infrequent closure (e.g. Whitfield & Bate, 2007).

Figure 3-3: Palmiet Estuary

Mouth dynamics: A berm extends from the east bank of the estuary mouth, while the west bank is formed by a rocky promontory. The berm extends in a westerly direction due to local westbound longshore sediment transport. During extended periods of low river discharge the mouth will eventually be severely restricted or closed (Bickerton *et al.*, 1983). In the past, natural closure occurred on average once every two to three years, however in recent years' closure has become more likely due to the increase in catchment dams (Whitfield & Baliwe, 2013). Regular artificial breaching is not practised at this estuary.

Wave exposure: The coastline orientation at the estuary mouth only allows waves approaching from between 90° (east) to 270° (west) to reach the mouth. The majority of the waves in the vicinity of the mouth approach from a south-westerly direction.

Sediment availability: The Palmiet Estuary has a limited stretch of sandy beach which is nestled in a rocky coastline. The available sediment in the vicinity and from adjacent beaches is very limited.

3.1.3. Bot/Kleinmond System

General: The Bot/Kleinmond Estuary (Figure 3-4) forms a relatively shallow coastal lake (mean depth of 1.5 m MSL) which is situated between Kleinmond and Hermanus. This large estuary connects to the sea at the Bot River mouth, as well as the Kleinmond mouth. The Bot and Kleinmond Estuaries are connected via a natural channel at 1.7 m above MSL. This natural channel acts as an overflow channel allowing excess flood water to escape the Bot River Estuary and enter the Kleinmond Estuary. This process can prevent natural breaching to occur at the Bot.

Figure 3-4: Kleinmond (A) and Bot (B) Estuaries

Mouth dynamics: The Bot River Estuary is predominantly separated from the sea by an inlet berm and an adjoining coastal dune belt. The dune belt has been stabilised due to deliberate introduction of vegetation, causing a landward shift of sediment and dune growth (Van Niekerk *et al.*, 2005). The dune ridge separating the estuary from the sea is interrupted by several troughs through which marine overwash occurs during periods of high wave energy. Artificial breaching at the Kleinmond mouth has been ceased in the last decade, resulting in more breaching occurences at the Bot River Estuary. At present the Bot mouth is artificially breached approximately once or twice every three years. Natural breaching may also occur under suitable circumstances. Natural breaching at the Kleinmond Estuary mouth is expected to occur at approximately 2.5 m MSL, while natural breaching at the Bot River Estuary mouth is estimated to occur between 2.7 m to 3.0 m MSL (Van Niekerk *et al.*, 2005).

Wave exposure: The highest portion of waves approach the coastline from the south-southwest. The average wave height based on Voluntary Observing Ship (VOS) data along the Bot River Estuary coast was calculated as 2.1 m, classifying it as a high-energy coastline (Bickerton *et al.*, 1983).

Sediment availability: Observations suggest a north-westerly longshore sediment drift at the location (Bickerton *et al.*, 1983). This is evident by the shoreline erosion at the south-easterly beaches near Hawston and accretion near the north-westerly segment.

3.1.4. Onrus

General: The Onrus Estuary (Figure 3-5) is a small TOCE situated between Hawston and Hermanus approximately 87 km south-east of Cape Town.

Figure 3-5: Onrus Estuary

Mouth dynamics: The estuary is generally closed by a large sand bar spanning the mouth. The average crest height, adjacent to the mouth channel, is estimated at 2.8 m above MSL (CSIR, 1991). The estuary frequently exhibits a shallow, narrow channel at the west of the mouth due to slow overtopping at relatively low inflow rates. This considered, the estuary predominantly exhibits a semi-closed mouth state. Limited seawater intrusion takes place through the channel, due to the perched level of the estuary water body. However, marine overwash can be expected during high wave energy conditions. After breaching there is a brief period of tidal fluctuation in the estuary, followed by relatively rapid closure. Regular artificial breaching is not practised at the Onrus Estuary mouth.

Wave exposure: The predominant wave approach direction is from the south-westerly sector (SSW, SW and WSW), with an average significant wave height of 2.4 m (based on Slangkop waverider data). This section of coastline may therefore be considered as a high-energy coastline (Bickerton *et al.*, 1983).

Sediment availability: The estuary mouth is nestled in a small cove on a rocky coastline. Therefore, limited large scale longshore sediment is transported. However, cross shore transport of sediment is evident during erosional wave conditions. Sediment is transported from the mouth to an offshore bar, after which it is transported back toward the mouth during accretive wave conditions (Bickerton *et al.*, 1983).

3.1.5. Klein

General: The Klein River Estuary (Figure 3-6), also known as the Hermanus Lagoon, is situated approximately halfway between Cape Point and Cape Agulhas, in Walker Bay. The Klein Estuary is a large coastal lake that experiences closure for prolonged periods.

Figure 3-6: Klein Estuary

Mouth dynamics: The Klein system is heavily managed with routine artificial breaches. Artificial breaching at inadequately low levels over the past century has caused substantial sediment build-up within the estuary, leading to faster mouth infilling after breaching (Anchor Environmental Consultants, 2015). The estuary is separated from the sea by a large dune belt which is vegetated in some areas.

Wave exposure: Walker Bay is relatively exposed to the prevailing deep sea swell approaching from the south/southwest sector.

Sediment availability: The direction of net longshore sediment movement is considered to be well balanced in Walker Bay. However, occasionally after mouth breaching, additional wave refraction occurs as a result of an offshore bar near the mouth region, leading to a south-easterly movement of sediment toward the inlet (Bickerton *et al.*, 1983).

3.1.6. Hartenbos

General: The Hartenbos Estuary (Figure 3-7) mouth is located in Mossel Bay, approximately 7.5 km north-west of the Mossel Bay harbour. The estuary is classified as a TOCE with a relatively small water area.

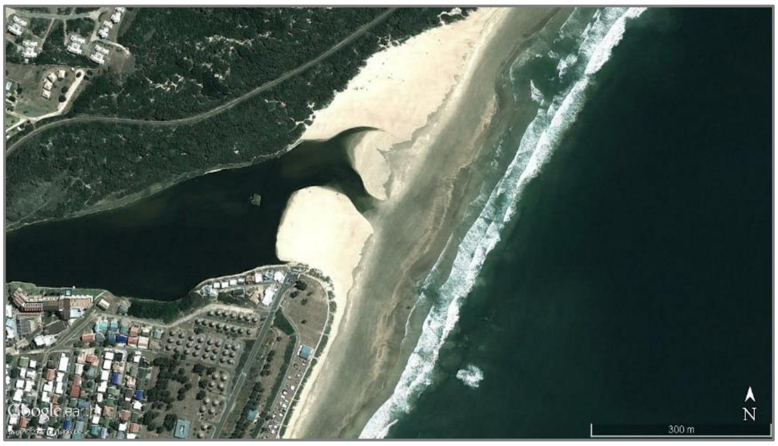


Figure 3-7: Hartenbos Estuary

Mouth dynamics: The estuary mouth is frequently closed off by a large flat sand bar. Regular monitoring and artificial breaching was started in November 2016. Prior to this, preference was given to natural breaching, despite having been subjected to occasional artificial breaching. The mouth sand bar has exhibited dynamic behaviour throughout history, changing not only in berm height, but also width and shape. Frequent breaching occurs throughout the year, followed by closure which typically occurs within a week (Bickerton *et al.*, 1983).

Wave exposure: The promontory of Cape St Blaize (Mossel Bay Point) acts as a barrier to shelter Hartenbos from the prevailing south-westerly swell. The diffracted south-westerly swell reach the estuary as medium energy waves. Theoretically, only deep sea waves from the southeast to east sector can reach the beach undisturbed.

Sediment availability: The west bound longshore sediment transport component is predominant at the estuary mouth. This is attributed to the diffracted wave gradient along the bay, causing westward flowing surf zone currents (Bickerton *et al.*, 1983).

3.1.7. Klein Brak

General: The Klein Brak Estuary (Figure 3-8) is a TOCE located in Mossel Bay, approximately 9.5 km north of the Mossel Bay harbour.

Figure 3-8: Klein Brak Estuary

Mouth dynamics: The estuary is generally in an open state, allowing tidal influence within the estuary. The south-western side of the mouth has a rocky shelf bottom, which restricts the depth of the mouth channel. The position and width of the mouth channel varies over time. Regular artificial breaching is not practised at the Klein Brak Estuary mouth.

Wave exposure: The predominant south westerly swell is diffracted by the Cape St Blaize (Mossel Bay Point) headland, before reaching the estuary inlet.

Sediment availability: The west bound longshore sediment transport component is predominant at the estuary mouth. This is attributed to the diffracted wave gradient along the bay, causing westward flowing surf zone currents (Bickerton *et al.*, 1983).

3.1.8. Groot Brak

General: The Groot Brak Estuary (Figure 3-9) is a TOCE located in Mossel Bay, approximately 16 km northeast of the Mossel Bay harbour. The Groot Brak Estuary is relatively large compared to other South African TOCEs.

Figure 3-9: Groot Brak Estuary

Mouth dynamics: The mouth of the estuary is delimited by a rocky headland on the east and a sandspit on the west side. Regular artificial breaching is practised at the Groot Brak Estuary mouth. Natural breaching is not feasible anymore due to the significant development in the low-lying areas of the estuary. At present, flooding of the lowest properties occur at 2.24 m above MSL (CSIR, 2017). The flow release of the Wolwedans Dam, upstream of the estuary mouth, is also managed to prolong open mouth conditions and maximise sediment flushing. During extended periods of mouth closure the berm can build up to levels exceeding 3.5 m above MSL, as was the case during the 2010/2011 drought (CSIR, 2017).

Wave exposure: The predominant south westerly swell is diffracted by the Cape St Blaize (Mossel Bay Point) headland, before reaching the estuary inlet.

Sediment availability: The west bound longshore sediment transport component is predominant at the estuary mouth. This is attributed to the diffracted wave gradient along the bay, causing westward flowing surf zone currents (Bickerton *et al.*, 1983).

3.1.9. Touw (Wilderness)

General: The Touw Estuary (Figure 3-10) is situated near Wilderness, between the towns of George and Knysna. The Touw can be classified as a TOCE and is connected to the Wilderness Lakes.

Figure 3-10: Touw Estuary

Mouth dynamics: The estuary mouth is artificially breached to prevent flooding of properties. The comparatively low surface area of the estuary causes the water level to rise rapidly during floods, which allows little time for artificial mouth manipulation. The berm crest is often skimmed (lowered) when water levels are high. Under natural conditions the sand berm may reach elevations exceeding 3 m above MSL (Russell, 2013).

Wave exposure: The estuary mouth is located on the exposed high energy Wilderness beach. The prevailing south westerly deep water swell wave can reach the estuary mouth relatively unhindered.

Sediment availability: The dynamic Wilderness beach is relatively wide, and together with the primary dune, contains large volumes of sediment. Thus, the estuary mouth can be closed with relative ease, given favourable wave conditions and low river flow.

3.1.10. Piesang

General: The Piesang Estuary (Figure 3-11) is a small TOCE situated in Plettenberg Bay.

Figure 3-11: Piesang Estuary

Mouth dynamics: The north-east of the estuary mouth is block by a massive sand bar that forms part of the central beach. A shallow channel is usually observed on the opposite end of the mouth, adjacent to the rocks at the landward side of the Beacon Island. This channel acts as an overflow for the estuary and provides access to limited tidal intrusion. During low river flow periods the channel tends to close. The estuary is mostly allowed to breach naturally, however artificial breaches are practised on occasion to flush sediment from the estuary. Breaching leads to a temporary widening of the channel.

Wave exposure: The Piesang Estuary mouth is located in the lee of the Robberg Peninsula, resulting in relatively low wave and current energy in the surf zone.

Sediment availability: The general net longshore current direction in the Bay seems to be towards the north-east, however at the Piesang mouth there is a reversal. The local net longshore sediment transport at this location is in the south-westerly direction (Bickerton *et al.*, 1983).

3.1.11. Groot (Natures Valley)

General: The Groot Estuary (Figure 3-12) is located in the town of Natures Valley, approximately 18 km northwest of Plettenberg Bay. The Groot is classified as a TOCE.

Figure 3-12: Groot (Natures Valley) Estuary

Mouth dynamics: The sand bar formed across the mouth deflects the channel towards the rocky embankment at the eastern side of the mouth. A cyclical migration of the mouth is observed along the sand bar. After breaching a channel is formed to the east. Given enough time, the south-eastern waves cause the channel to migrate west, until the mouth experiences closure (Bickerton *et al.*, 1983). Alternatively, the westward mouth migration may be attributed to the scouring of bends in the outflow channel. Regular artificial breaching is not practised at the Groot Estuary mouth.

Wave exposure: The Natures Valley beach lies relatively exposed to the dominant south westerly deep sea swell.

Sediment availability: The Natures Valley beach is nestled in a rocky coastline. Currents in the mouth region appear to be mainly wave driven. A weak pulsating current moving in an easterly direction was observed during a survey conducted on 2 November 1982 (Bickerton *et al.*, 1983).

3.2. Eastern Cape Estuaries

An overview of the selected Eastern Cape estuaries is presented in this section.

3.2.1. Tsitsikamma

General: The Tsitsikamma Estuary (Figure 3-13) is situated approximately 35 km west from the Cape St Francis headland. The estuary is classified as a small TOCE.

Figure 3-13: Tsitsikamma Estuary

Mouth dynamics: The mouth- and berm dynamics of the estuary is significantly influenced by the presence of sand dunes in the mouth area, specifically to the west of the inlet. The estuary may also exhibit a semi-closed mouth state during periods of low river flow (Whitfield & Bate, 2007). Artificial breaching is not practised at the estuary.

Wave exposure: The inlet is located on the open coastline, exposed to the deep water south-westerly swell.

Sediment availability: The estuary inlet is located on a predominant rocky coastline, which is covered with sand in certain sections. The inlet is constricted by vegetated dunes on either side. These dunes can act as a trap for wind- and wave driven sediment, subsequently altering the inlet morphology.

3.2.2. Seekoei

General: The Seekoei Estuary (Figure 3-14) is situated in St Francis Bay, approximately 5 km south of the town of Jeffreys Bay.

Figure 3-14: Seekoei Estuary

Mouth dynamics: The Seekoei Estuary mouth is seldomly breached as a result of rare, major floods. A northward longshore current is one of the main drivers of sand build up and northward growth of the bar (Bickerton *et al.*, 1983). The Seekoei Estuary is subject to occasional artificial breaching. Prior to 1969, the Seekoei Estuary mouth channel was located at a rocky sill at the north edge of the mouth area. The rocky sill played a crucial part in the hydrodynamic and morphological behaviour of the mouth. The sill reduced the intrusion of marine sediment and prevented complete drainage of the estuary during open mouth state. Construction of a car park and protective embankment (1969), and a swimming pool (1973) to the north of the mouth caused the mouth channel to migrate southward, leading to complete drainage of the estuary during low tide and open mouth state. A poorly designed and -constructed low-level causeway was constructed upstream to mitigate this effect, however it ultimately led to a severely disrupted natural function of the estuary. More recently, an increase in the causeway opening and the halt of artificially maintaining the inlet channel via the concrete channel, has led to an increased open mouth state (James & Harrison, 2010).

Wave exposure: The predominant deep sea wave direction is south-west at St Francis Bay. The Cape St Francis headland provides relative shelter from the predominant swell.

Sediment availability: The net longshore sediment transport within the St Francis bay was estimated to move in a northward direction (Bickerton *et al.*, 1983).

3.2.3. West- and East Kleinemonde

General: The West- and East Kleinemonde Estuaries (Figure 3-15) are two separate systems with neighbouring inlets. These estuaries are situated approximately 15 km north-east of Port Alfred. Both estuaries are classified as small TOCEs.

Figure 3-15: West- and East Kleinemonde Estuary

Mouth dynamics: These estuaries are mainly closed due to low river inflow, only experiencing a few breaches per annum. Neither of these estuaries are subject to regular artificial breaching.

Wave exposure: The inlets are located on a relatively straight section of beach. The predominant south westerly swell reaches the shoreline at an oblique angle. The south westerly waves diffract around the promontory before reaching the inlet.

Sediment availability: The shoreline is relatively rocky, but covered with sediment, towards both directions of the mouth. Large sand dunes are present toward the north-east of the inlets.

3.2.4. Mngazi

General: The Mngazi Estuary is located approximately 10 km south west of Port St Johns (Figure 3-16). The Mngazi is classified as a TOCE, however a semi-closed mouth state is regularly displayed.

Figure 3-16: Mngazi Estuary

Mouth dynamics: The mouth channel forms typically at the eastern side, with the sand berm extending from the western side.

Wave exposure: The inlet is relatively exposed from the south westerly swell, which approaches the shoreline at an oblique angle.

Sediment availability: The beach is nestled between two rocky outcrops/promontories. The berm can grow relatively wide during prolonged periods of semi closed or closed mouth state.

3.3. KwaZulu-Natal Estuaries

An overview of the selected KwaZulu-Natal estuaries is presented in this section.

3.3.1. Mhlanga

General: The Mhlanga Estuary (Figure 3-17) is a perched TOCE located approximately 20 km north-east of the Port of Durban.

Figure 3-17: Mhlanga Estuary

Mouth dynamics: The estuary mouth experiences frequent breaching throughout the year. The estuary has been subject to artificial breaching in the past, however in recent years an effort has been made to let the system breach naturally as far as possible. The estuary is fronted by a mouth barrier spanning an average of approximately 1.5 km. The mouth barrier is topped by forested dunes to the south and lesser vegetation to the north, while the centre section is an unvegetated berm. The mouth channel typically forms in the centre of the barrier after breaching.

Wave exposure: The estuary mouth is situated on a long sandy section of coastline which is exposed to the predominant deep sea waves approaching from between 170° and 220° (CSIR, 2000a).

Sediment availability: The beaches adjacent to the estuary mouth are sandy with underlying rock. The predominant wave direction from the southern quadrant generates a northward longshore movement of sediment (CSIR, 2000a).

3.3.2. Mdloti

General: The Mdloti Estuary (Figure 3-18) is a perched TOCE located approximately 25 km north-east of the Port of Durban.

Figure 3-18: Mdloti Estuary

Mouth dynamics: The estuary mouth is closed for extended periods throughout the year, only breaching occasionally. The estuary has been subject to artificial breaching throughout recorded history, however in recent years an effort has been made to let the system breach naturally as far as possible. The mouth berm is approximately 600 m long, spanning the entire mouth. The berm crest is stabilised by vegetation, especially to the southern half.

Wave exposure: The estuary mouth is situated on a long sandy section of coastline which is exposed to the predominant deep sea waves approaching from between 170° and 220° (CSIR, 2000a).

Sediment availability: The predominant wave direction from the southern quadrant generates a northward longshore movement of sediment (CSIR, 2000a). Frequent overtopping occurs due to the high wave action and steep beach slopes despite the perched estuary berm levels reaching high elevations (Zietsman, 2004).

3.3.3. Tongati

General: The Tongati Estuary (Figure 3-19) is the northern-most estuary in Durban, situated approximately 35 km northeast from the Port of Durban. The Tongati Estuary is a small perched TOCE.

Figure 3-19: Tongati Estuary

Mouth dynamics: A sandy beach with a rocky sill is evident at the adjacent beach to the south of the estuary mouth. The mouth channel is typically located at the south end of the entrance. The inlet berm is approximately 170 m long with a width ranging between 30-70 m, reaching elevations of up to 5 m above MSL at the highest points (Zietsman, 2004). The estuary mouth is generally open, however periodic closure is evident.

Wave exposure: The estuary is located on a straight exposed section of coastline to the north of Durban. There is little protection from the predominant deep sea waves approaching from between 170° and 220°.

Sediment availability: The predominant wave direction from the southern quadrant generates a northward longshore movement of sediment (CSIR, 2000a).

4. Available South African Field Data

The available field data pertaining to berm height at South African estuaries are discussed in this chapter. Available sources of recorded berm heights include estuary mouth/berm surveys and water level recordings within the estuaries. The coastal parameters that describe estuarine berm growth and -functioning are also discussed in this chapter.

4.1. Estuarine Mouth/Berm Surveys

There are limited estuary mouth surveys and berm profile surveys available for South African TOCEs. Therefore, relatively little information is available that could help gain a better understanding of the morphological behaviour of estuarine berms. Several surveys have been conducted, albeit long-term records documented at regular intervals are not readily available.

The Estuarine and Coastal Research Unit (ECRU) was established by the National Research Institute for Oceanology (NRIO) of the CSIR in 1979. The ECRU aimed to gather and collate information of the South African estuaries in order to gain a better understanding of individual system functioning. The relevant work conducted by the ECRU involved occasional mouth surveys and beach sediment sampling at selected estuaries in the Western Cape (CSIR, 2000b) and Eastern Cape (CSIR, 2000c). These surveys were conducted between the years of 1984 and 1999. The number of mouth surveys conducted at individual estuaries vary considerably, ranging from only a single survey, up to approximately 14 surveys conducted over the monitoring period. In some instances, these records provide insight into estuary mouth functioning over time. Unfortunately, not all the surveys are conducted while the estuaries exhibit a closed mouth state. Thus, the desired elevation of the berm crest saddle point (lowest point in the berm crest) could not be obtained. Another difficulty arises from the lack of knowledge of historical estuary breaching events. Without a comprehensive record of breaching events, it is impossible to determine at what stage of the berm building process the survey was conducted. Nevertheless, a few berm crest elevations could be obtained for the selected study locations. The berm elevations at the individual estuaries display considerable variance due to the above-mentioned factors. The mouth survey results of the Seekoei Estuary, conducted on the 20th September 1991, is provided in Figure 4-1 to serve as an example (CSIR, 2000c). The collective berm heights derived from estuary mouth surveys are presented and analysed in Chapter 5.

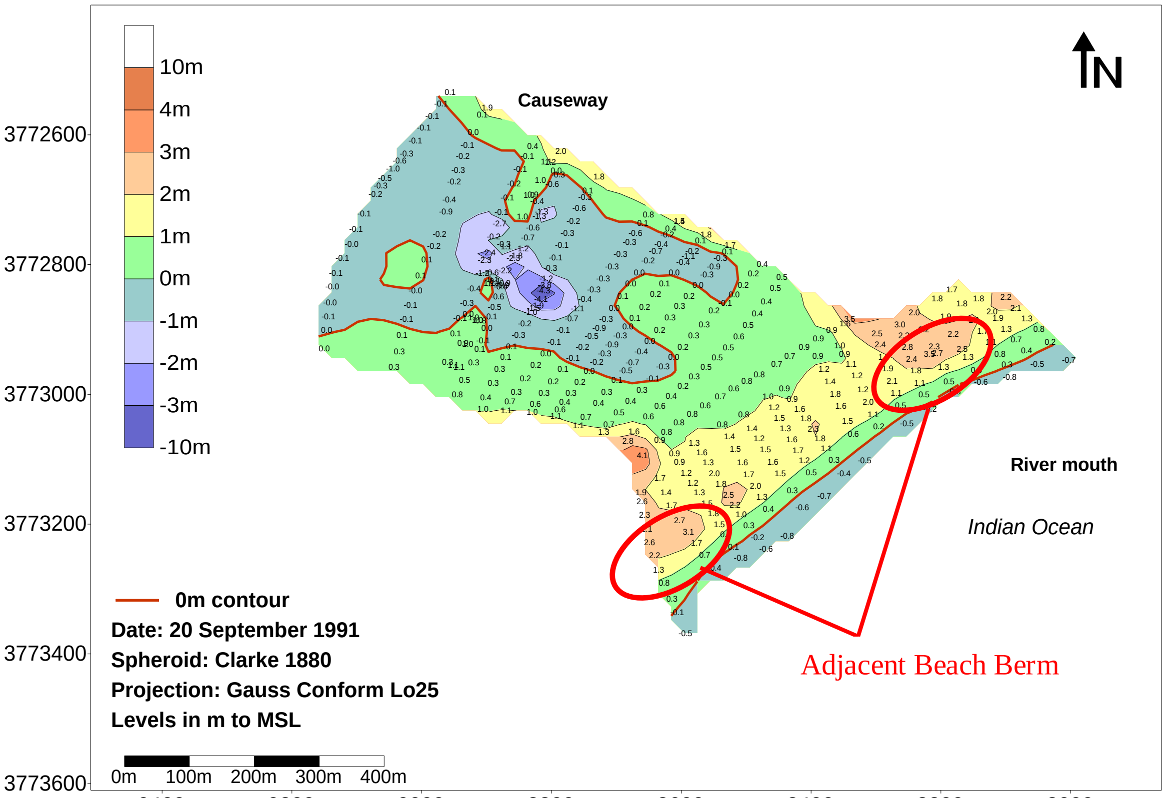


Figure 4-1: Mouth survey of the Seekoei Estuary (CSIR, 2000c)

Alternatively, the berm crest adjacent to the inlet (refer to Figure 4-1) was considered as an indicator of the potential maximum berm crest elevation at the inlet. The reasoning behind this was that given sufficient time at closed mouth state, the berm will eventually build up to that level. This however introduces a certain level of uncertainty regarding the selection of the position of the adjacent beach berm from survey results. The process of identifying the adjacent beach berm crest location from a survey plot, without accidently selecting the crest of a dune or vegetated area, is rather subjective. Satellite imagery from the exact dates of the surveys are not readily available for use as an overlay image, and care should be taking when using imagery from an alternate date, as the estuary mouth morphology is highly variable. The berm crest elevation of the adjacent beach berm is generally higher than the inlet berm crest. Even if the inlet berm is closed for a prolonged period, it may still not reach the level of the adjacent berm. Therefore, the elevations of the adjacent beach berms were not included in the record.

A three-dimensional surface plot of the Seekoei Estuary survey (Figure 4-1) is presented in Figure 4-2, to illustrate the presence of a saddle point(s) at the inlet berm. The surface plot is generated from the raw coordinate data collected during the mouth survey, by using the *Surfer* ® 11 Software package (Golden Software, 2012). The surface plot uses the same coordinate system and vertical scale as illustrated in Figure 4-1.

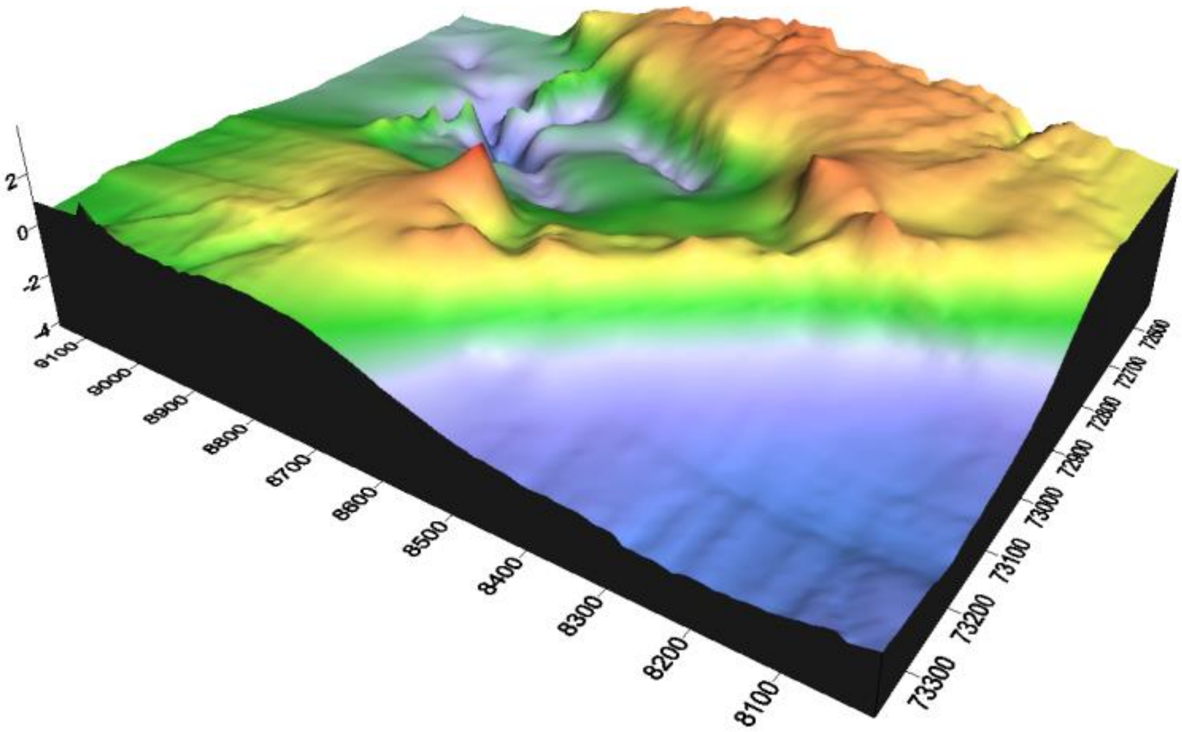


Figure 4-2: Three-dimensional surface model of Seekoei Estuary mouth region

The berm crest elevations cited in literature could potentially be improperly interpreted. The average or maximum beach berm crest elevation derived from mouth survey results are often an improper representation of the potential inlet berm crest elevation. The maximum berm crest elevation in the general mouth/beach region is not of importance, but rather the maximum level the saddle point of the berm at the closed inlet can potentially reach. Breaching will always initiate when the water level reaches the elevation of the berm saddle point, thus making this the important variable. An example of this is the technical report authored by the CSIR (1991), reporting an average crest height of the sand bar between the lagoon and the sea to be 2.8 m above MSL. This value has been derived by considering the average berm crest height along the entire Onrus beach, despite the inlet berm only constituting a minor section of the beachfront, and the fact that the inlet berm is typically lower than the adjacent beach berm at Onrus. Throughout literature this value has repeatedly been used as an estimate (e.g. Massie & Clark, 2016) of the potential inlet berm crest elevation at Onrus. Upon investigation of a 20-year record of water level data recorded in the Onrus Estuary, it was concluded that the inlet berm crest only reaches an absolute maximum elevation of 2.48 m above MSL (§ 5.1). Although this cited value (2.8 m) may be considered as an over conservative estimate, caution should be exercised when using results derived from a single survey.

4.2. Deriving Berm Crest Elevations from Water Level Data

The DWS continuously monitors and records the water level fluctuations within selected estuaries across South Africa. The water level recorders are placed near the estuary inlet in order to register water level fluctuations caused by river inflow and ocean tides. These water level recordings were used to derive berm crest elevations in the absence of comprehensive mouth- or berm survey data.

The elevation of the berm saddle point can be derived by observing the estuary water level at the exact moment of mouth breaching. In theory, the instantaneous estuary water level at the moment of breaching is approximately equal to the berm crest level where the breaching channel forms (berm saddle point). Thus, several recorded berm crest elevations can be obtained by scrutinising the long-term water level recordings of an estuary.

The recorders log the vertical elevation of the water level in 6-minute intervals, providing high frequency readings over extensive periods. The location, reference number and available record length for the water level recorders at the selected locations are provided in Table 4-1. Additionally, the DWS monitors the flow rate of several rivers in South Africa. Stations used to measure the flow rate are typically located upstream from the upper estuary reach. The peaks in river flow rate can help to identify possible estuary breaches caused by the periodic flooding. Care should be taken when considering the location of flow rate gauges. Using a flow rate recorded too far upstream from the estuary, or of a gauge located upstream from a dam is not recommended, as the lag time present between high flow rates and high estuary water level will be substantial. Therefore, only flow gauges located in a relatively near upstream range of the estuaries were considered. The available flow gauges corresponding to the respective estuary water level recorders are also provided in Table 4-1.

As expected, the water level records contain occasional gaps, and recorded periods during device malfunction. Care was taken to avoid any distorted readings in the records.

Table 4-1: Water level recorders and flow gauges at respective estuaries

	Estuary	Water Level Record				Flow Record			
		Tidal station	Coordinates		Water level data record	Flow station	Coordinates		Flow station record
1	**Lourens**	G2T043	-34.10055	18.81527	2004-04-20 to 2013-10-01	G2H044	-34.09619	18.82950	2004-06-22 to 2016-11-30
2	**Palmiet**	G4T009	-34.33611	18.99166	1992-01-21 to 2016-11-24	G4H007	-34.32944	18.98833	1963-03-30 to 2016-11-24
3	**Kleinmond**	G4T012	-34.34138	19.03722	2006-11-01 to 2015-05-21	-	-		-
4	**Bot**	G4T003	-34.32472	19.12166	1979-10-01 to 2015-12-31	G4H014	-34.23527	19.21527	1967-04-13 to 2016-11-23
5	**Onrus**	G4T011	-34.41638	19.17805	1994-01-01 to 2015-05-21	G4H033	-34.35888	19.25388	1977-04-28 to 2016-11-23
6	**Klein**	G4T004	-34.41000	19.34833	1979-01-01 to 2015-12-31	G4H006	-34.40527	19.59611	1963-03-28 to 2016-11-23
7	**Hartenbos**	K1T010	-34.11722	22.11638	1993-10-01 to 2015-11-03	-	-		-
8	**Klein Brak**	K1T020	-34.08750	22.13472	1995-01-01 to 2015-07-01	K1H004	-34.03166	22.05277	1969-03-25 to 2016-11-29
9	**Groot Brak**	K2T004	-34.05138	22.23166	1988-05-02 to 2015-03-11	K2H002	-34.02861	22.22194	1961-05-04 to 2016-11-29
10	**Touw**	K3T006	-33.99361	22.59638	1993-10-01 to 2013-10-01	K3H005	-33.94527	22.61444	1969-04-15 to 2016-12-01
11	**Piesang**	K6T021	-34.06083	23.37277	2010-11-18 to 2015-08-09	-	-		-
12	**Groot**	K7T002	-33.97083	23.56416	2002-09-13 to 2016-12-06	-	-		-
13	**Tsitsikamma**	K8T004	-34.13408	24.44127	1995-01-25 to 2017-01-01	K8H005	-34.09663	24.43911	1995-06-20 to 2017-01-17
14	**Seekoei**	K9T009	-34.08611	24.9028	2002-08-15 to 2013-06-12	-	-		-
15	**West Kleinemonde**	-	-		-	-	-		-
16	**East Kleinemonde**	P4T002	-33.53672	27.04166	2004-12-14 to 2017-01-25	-	-		-
17	**Mngazi**	T7T004	-31.67088	29.45419	2003-12-01 to 2016-12-01	T7H001	-31.55120	29.24380	1970-06-23 to 2017-01-20
18	**Mhlanga**	U3T010	-29.70563	31.09922	2005-09-01 to 2016-08-22	-	-		-
19	**Mdloti**	U3T009	-29.65311	31.12675	2005-09-16 to 2016-08-24	-	-		-
20	**Tongati**	U3T008	-29.57259	31.18436	2005-08-11 to 2014-11-05	-	-		-

4.2.1. Water level corrections

Mean Sea Level (MSL) is used as the vertical datum for the berm crest elevations derived from the water level data, however not all the water level records provided by the DWS are referenced to MSL. The necessary adjustment factors are required to correct the data to MSL. The DWS provided the water level correction factors for certain estuaries, and some of the records are already referenced to MSL (no correction required). Correction factors for the estuary water level records with unknown datums were determined by comparing the recorded sea level with the water level recorded within the estuary during periods of open mouth state. The sea level, recorded at the nearest port, is plotted over the recorded water level within the estuary. The two tidal signals are compared by evaluating the high water – and low water levels during both spring and neap tide. An upward, or downward, adjustment is made to the estuary water level to ensure a match between the crests or troughs of both signals.

Even during open mouth state, most estuaries only experience a reduced tidal range compared to the sea. This reduction is site specific and dependent on factors such as the mouth channel cross sectional area and the river outflow at the estuary mouth. In most cases the neap tidal signal provides a better estimate when comparing the ocean tide and estuary tidal variation. During spring tide the constricting effect of the estuary inlet plays a larger role due to the increased tidal prism.

The recorded sea level variations are obtained from the University of Hawaii Sea Level Centre (UHSLC) database (Caldwell *et al.*, 2015). The UHSLC database contains records of tide gauge data from 641 stations across the world. The UHSLC provides hourly, quality controlled, research quality sea level records.

The time series, over a 6-day period, of the 6-minute interval Seekoei Estuary water level (K9T009) and the hourly sea level recorded at Port Elizbeth are provided in Figure 4-3, to serve as an example. The sea level record is corrected to Land Levelling Datum (LLD) by subtracting a constant of -0.836 m. This correction is provided in the South African tide tables (SANHO, 2017). The dotted red line indicates the estuary water level prior to adjustment. Note that the unreferenced estuary water level (red dotted line) is well above that of the sea level. Therefore, a downward adjustment is required to obtain the elevation relative to MSL. The lag time present between the estuary water level and the sea level curve is due to the constricting effect of the inlet channel. The lag between high tide in the sea and the estuary is relatively small compared to the lag present during ebb tide. This is attributed to the increase in mouth cross sectional area as the water level rises. The estuary water level during low tide is much higher than the sea level, due to the elevation of the channel compared to the sea level.

The average of the lowest neap tide high water levels (sea level) were matched to the estuary water levels during the corresponding tidal cycles. This results in a downward adjustment of 0.24 m to the estuary water level to obtain an elevation relative to MSL.

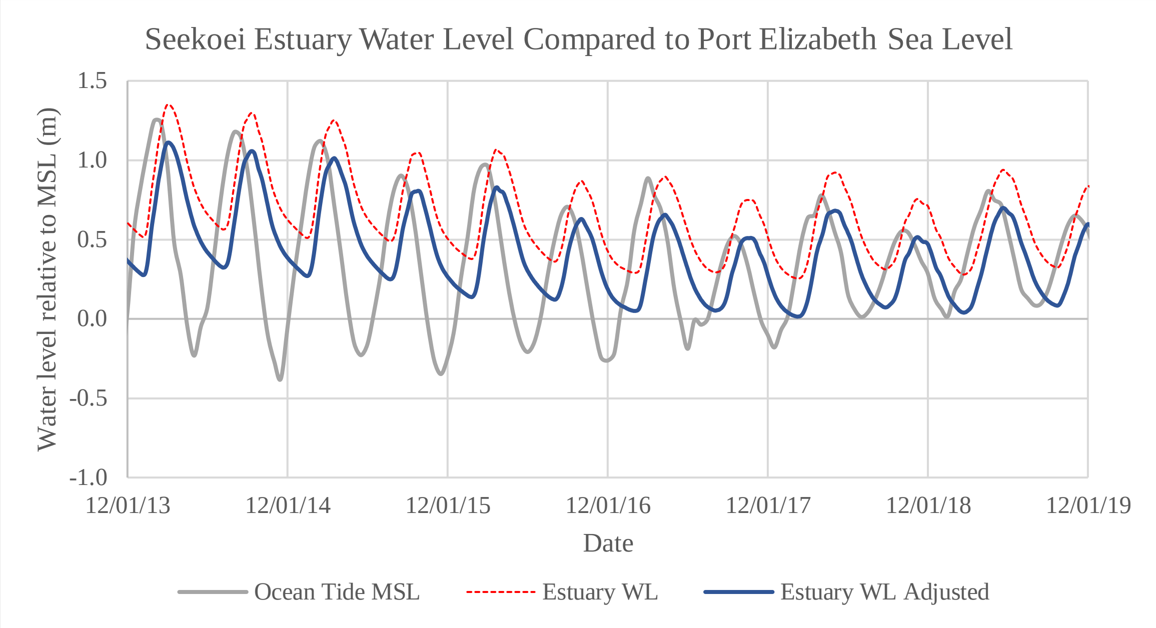


Figure 4-3: Water level correction to Mean Sea Level (MSL) for Seekoei Estuary

A similar process is followed for the remaining estuaries with unknown datums. The ocean tidal station nearest to the individual estuary under consideration were selected for the analysis. The water level correction factors for the respective estuaries, as well as the source of the factors are provided in Table 4-2.

Table 4-2: MSL correction factors for selected estuaries

	Estuary	Water level recorder	MSL correction (m)	Source
1	**Lourens**	G2T043	+0.09	Calculated
2	**Palmiet**	G4T009	-0.67	Calculated
3	**Kleinmond**	G4T012	+0.7	DWS
4	**Bot**	G4T003	-0.6	DWS
5	**Onrus**	G4T011	+0.2	DWS
6	**Klein**	G4T004	-1.47	DWS
7	**Hartenbos**	K1T010	0	DWS
8	**Klein Brak**	K1T020	-0.978	DWS
9	**Groot Brak**	K2T004	0	DWS
10	**Touw**	K3T006	-0.162	DWS
11	**Piesang**	K6T021	0	DWS
12	**Groot**	K7T002	-0.044	DWS
13	**Tsitsikamma**	K8T004	+0.2	Calculated
14	**Seekoei**	K9T009	2002-2006: +0.45	2007-2013: -0.24	Calculated
15	**East-Kleinemonde**	P4T002	-0.09	Calculated
16	**Mngazi**	T7T004	2003-2011: -0.415	2012-2016: -0.59	Calculated
17	**Mhlanga**	U3T010	0	Calculated
18	**Mdloti**	U3T009	2005: 0	2006: -0.33	2007: -1	2008-2013: 0	Calculated
19	**Tongati**	U3T008	0	Calculated

The water level correction process becomes increasingly complicated when evaluating notably perched estuaries, specifically estuaries found in the KwaZulu-Natal province. The estuary water levels are periodically super-elevated above the sea level, while still displaying a significant tidal signal. Perissinotto *et al*. (2010) attributes this effect to the wave action generating a substantial setup as a result of the steep, reflective beach slopes. Essentially, the estuary experiences an elevated sea level, along with its tidal variation, due to wave action.

4.2.2. Identifying estuary breaches

Estuary breaches are identified by examining the time series of water level recorded within the estuary. After mouth closure, the estuary water level will continue to rise due to river inflow and occasional marine overwash. Minor water losses are experienced as result of evaporation and seepage through the inlet berm. The inlet berm generally also grows during this period of mouth closure. Breaching is initiated as soon as the estuary water level exceeds the inlet berm elevation. When observing the water level time series, a breach is typically identified by a rise in water level, followed by a sudden drop (steep gradient). Additionally, a breach is regularly coupled with a spike in river inflow which leads to an increase in water level. A breach is often followed by tidal variations in the estuarine water level due to the open mouth. The tidal range observed in the estuary is often reduced due to the constricting effect of the mouth channel. Figure 4-4 provides a time series for the Bot Estuary water level and river flow rate. A typical breach pattern is displayed, followed by an open mouth allowing tidal fluctuation in the estuary. The approximate berm level and time of mouth closure can be derived by identifying the starting point of a continuous rise in water level which is uninterrupted by tidal variation.

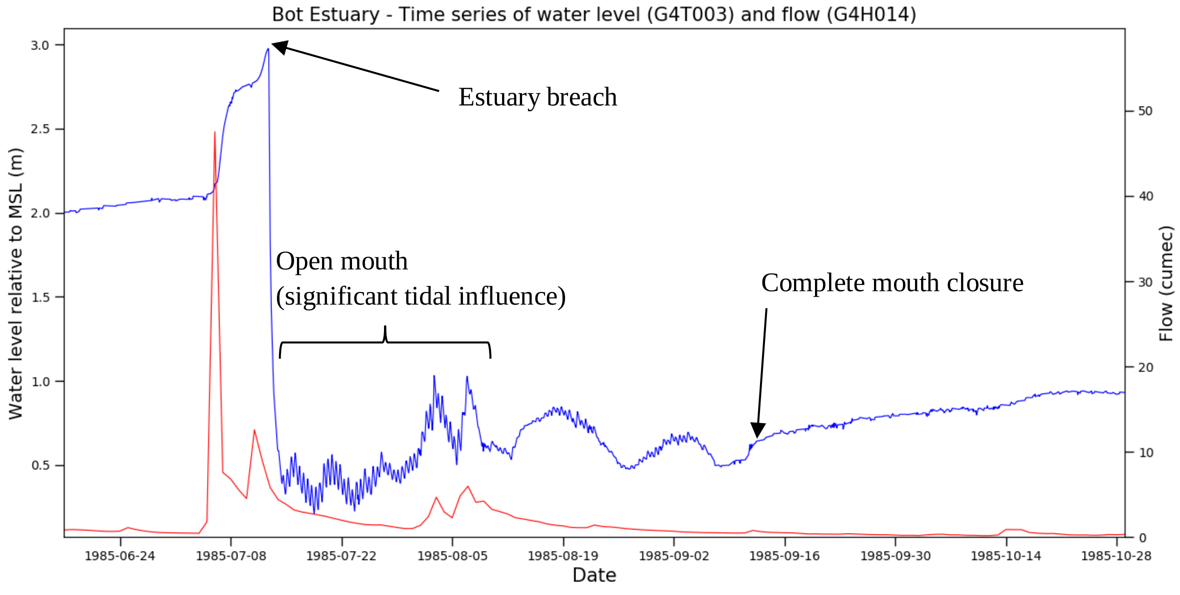


Figure 4-4: Time series of water level (blue) and river flow rate (red) at the Bot Estuary

Estuaries display varying breach patterns in their water level time series. Breaches are not always followed by a period of complete open mouth state and tidal variation. Estuaries with relatively smaller water areas such as Onrus, Hartenbos, Groot and Tsitsikamma, typically exhibit more frequent breaches compared to large estuaries such as Bot and Klein. Included among these frequent breaches are smaller, more insignificant breaches that do not effectively scour the channel. These smaller breaches occur at a lower water level before the berm has grown to a high elevation, thus the hydraulic head is insufficient to scour a prominent mouth channel. Nonetheless, a sudden drop in water level is observed, typically at a slightly lower water level. Smaller breaches are included in the acquired berm crest elevation record, as they also represent an accurate berm crest elevation at a given time. However, significantly smaller breaches and small overtopping events were excluded from the analysis to ensure greater certainty in the data compiled for further analysis. The Onrus Estuary water level time series is provided in Figure 4-5 to illustrate the pattern displayed by smaller breaches.

Higher rates of river inflow can also alter the breaching pattern observed from the water level time series. High river inflow rates are often translated into a sudden spike in water level prior to the breach. Estuaries with smaller water bodies typically exhibit this effect, due to their rapid rate of infilling during floods. The typical breach pattern associated with low- and high river inflow is also displayed in the Onrus water level time series (Figure 4-5). The breaches that transpired during low flow conditions (January to July) exhibit a more gradual rise in water level prior to breaching, compared to the sudden spikes exhibited during the high flow periods (August to December).

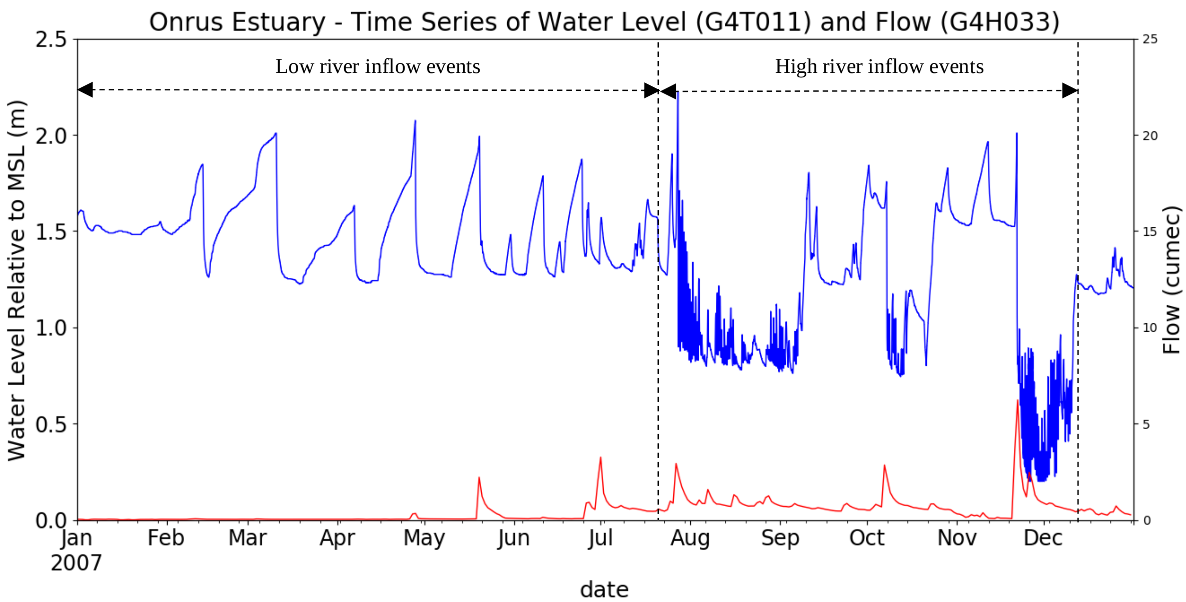


Figure 4-5: Time series of water level (blue) and river flow rate (red) at the Onrus Estuary

The available water level time series of all the estuary study locations have been examined. The collective berm heights derived from estuary water level data are presented in Chapter 5

4.2.3. Potential data interpretation difficulties

Deriving berm crest elevations from water level recordings only provide a vertical crest elevation of the saddle point at the time of the breach. Unfortunately, this provides limited information on the short-term morphological behaviour of inlet berms during the berm building process. The availability of extensive berm monitoring and survey data could potentially lead to a greater comprehension of the dynamics and processes involved in inlet berm growth.

The difficulties associated with the method of deriving berm crest elevations from estuary water level data are discussed in the following subsections.

4.2.3.1. Estuary water body surface gradient

A sudden rise in water level, associated with high river inflow rates, may lead to a slight gradient in the estuary water body surface. In other words, the water level at the upper reaches of the estuary may be slightly higher than the water level at the berm at the moment of breaching. This can lead to a minor difference between the recorded water level (recorded at position upstream of mouth) and the actual water level at the berm. This difference was assumed to be insignificant, due to the relatively small water body size of South African TOCEs, and the fact that the water level recorders are all located relatively near the estuary inlets.

4.2.3.2. Extreme river floods

Extreme floods can also affect the berm crest elevations derived from the water level data. High river inflow rates may lead to a further, minor increase in the water level after breaching is initiated and before the water level drops. Thus, the peak water level prior to the sudden drop can be slightly higher than the berm crest elevation. The flow rate threshold responsible for inducing this effect is dependent on several factors and its determination is subject to extensive field observations. Consequently, this effect was not considered during the analysis of the water level data.

4.2.3.3. Effect of artificial breaching

Several of the South African estuary inlets are artificially manipulated on a routine basis by the relevant governing bodies present in the area. Artificial breaching primarily aims to prevent flooding and ensure suitable ecological functioning of the estuary. The artificial breaching process involves initiating the breach prior to the point at which the water level in the estuary reaches or exceeds the berm crest elevation.

It is impossible to distinguish between a natural breach and an artificial breach when observing the estuary water level records. The only method by which to identify artificial breaches from the water level records is by acquiring a complete record of the artificial breaches conducted at the specific estuary. The relevant authorities are responsible for keeping a record of the artificial breaching times and breaching water levels. Unfortunately, this is not always the case as records are often not readily available, or they do not exist. In recent years, an increased focus has been placed on the effective monitoring and management of estuary inlets. Estuary mouth management plans often dictate regular berm surveys and thorough recording of relevant information during artificial breaches. The ten estuary study locations subject to regular artificial breaching are listed below.

- Kleinmond
- Bot
- Klein
- Hartenbos
- Groot Brak
- Touw
- Seekoei
- Mhlanga
- Mdloti
- Tongati

The artificial breaching records were provided by the relevant parties for the Hartenbos-, Groot Brak- and Touw Estuary. In recent years, it has become best practice to initiate artificial breaching at the maximum estuary water level possible, without leading to flooding and inundation of property. Artificial breaching at water levels similar to that during a natural breach, ensures efficient scouring of the inlet and longer periods of open mouth state. Evidence shows that a larger hydraulic head during breaching is more effective in flushing sediment from the lower reaches of the estuary (Van Niekerk *et al.*, 2012).

The berm level derived from the water level recordings of an estuary subject to frequent artificial breaching, may not be representative of the berm height under natural conditions. The rationale behind using or discarding the above listed artificially manipulated estuaries is based on the following considerations:

- Presence of natural breaches in the estuary water level record. Information regarding natural - and artificial breaches is obtained from the relevant management parties.
- Comparison of the recorded berm crest elevations (derived in this study) to the available information of berm levels and natural breaching levels. Available knowledge includes previous literature and professional opinions.
- Whether artificial breaching is conducted on a regular basis to avoid flooding, or only occasionally to restore natural ecological function to the estuary.
- Specific strategy of artificial manipulation and mouth management.

Considering the above-mentioned factors, the **Touw Estuary** is the only estuary that does not comply. Berm data of the remaining estuaries provide a relatively good representation of the berm height under natural conditions.

The Touw Estuary berm is continually manipulated to avoid flooding of property surrounding the estuary. The sand berm is manipulated by either skimming, or creating a "plug". Analysis of the long-term berm height variability at the Touw Estuary reveals that the berm overtops at water levels ranging from +1.6 m to +2.2 m MSL (§ 5.1). The record of breaching at the Touw Estuary reveals that all breaches since 1991 have been mostly artificial (SANPARKS, 2017). Russel (2013) estimates natural breaching to occur at a water level exceeding +3 m MSL. Consequently, the recorded berm crest elevations at the Touw Estuary are not representative of the berm elevation under natural conditions.

4.3. Data Acquisition of Relevant Coastal Parameters

Site specific data are required to describe the relevant coastal processes involved in estuarine berm formation, and to identify the primary drivers of high berm elevations. Berm growth is dependent on the swash zone interactions and runup elevations, therefore the coastal parameters required include: sediment grain size (D_{50}), beach face slope, offshore and nearshore wave statistics. These parameters are also used as input for the runup and berm crest predictors discussed in Chapter 6. The data acquisition processes of these coastal parameters are discussed in this section.

4.3.1. Field data acquisition

Data pertaining to site specific physical properties such as beach face slope and sediment grain size are not readily available for all the selected study locations. Accordingly, several locations within the Overberg district were selected for field investigations, based on the unavailability of said data. The selected sites include: Kleinmond -, Bot -, Onrus - and Klein Estuary. Surveys of these estuaries were conducted on the 10th of June 2017 in order to obtain the beach face slope, the sediment grain size and the berm crest elevations.

4.3.1.1. Sediment sampling

Surface sediment samples were collected for each of the survey locations to determine the sediment grain size distribution and the median grain size. A composite sample containing surface sediment from along the entire beach face, ranging from the water mark up to the berm crest, was collected for the respective survey locations. By collecting a composite sample, it provides an estimate of the representative cross-shore grain size distribution along the entire beach face. This is a simplified approach compared to collecting several samples along the profile.

A certain degree of variability in sediment grain size is present in the longshore- and cross shore direction of a beach. This variability can be determined by collecting several samples along the beach face. However, for determining a median grain size representative of the entire inlet berm, the composite sample method was deemed adequate.

4.3.1.2. Sediment grain size analysis

The sediment samples were analysed by means of dry sieving, in order to determine the grain size distribution. Generally, the larger the size of the sample, the more accurate the grain size distribution. However, for samples containing only sand and not gravel (particles < 4mm), a minimum sample size of 50 g is sufficient (Poppe *et al.*, 2000). Accordingly, the collected sample sizes varied between 330 g and 1020 g. The Samples were first oven dried overnight at a steady temperature of 100 °C. Dry samples were weighed before starting the sieve analysis. The sieve sizes were selected on the basis of availability and a visual inspection of the grain sizes present. Sediments were passed through the 2 mm, 1.18 mm, 0.6 mm, 0.425 mm, 0.3 mm, 0.15 mm and 0.075 mm sieves. The sieves containing the sample were stacked and vibrated for 15 min before being individually weighed to determine the percentage of sediment retained by each sieve (Figure 4-6).

Figure 4-6: Mechanical vibrating of sediment sample contained in stacked sieves

Consequently, the sediment grain size versus the percentage finer by weight can be plotted on a semi logarithmic scale. From the sediment grain size distribution curve, it is possible to graphically obtain the grain sizes for the 5-, 10-, 16-, 25-, 50-, 75-, 84-, 90- and 95 percentiles. The grain size distribution curve for the sample collected at the Onrus Estuary berm is displayed in Figure 4-7.

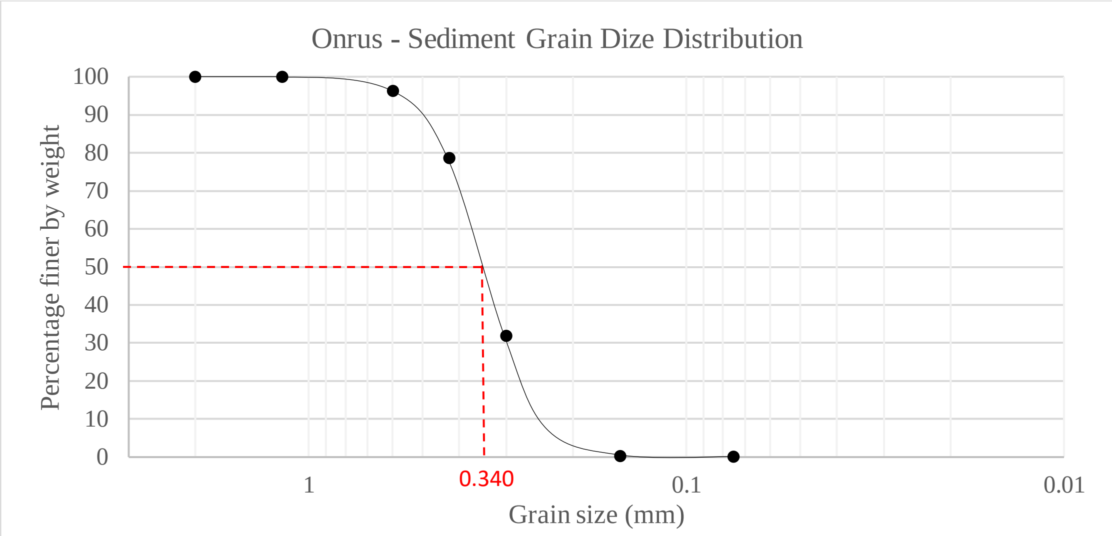


Figure 4-7: Sediment grain size distribution of Onrus berm sample

A summary of grain size distributions derived from the sieve test analysis are provided in Table 4-3. The sieve test analysis results of the survey locations are included in Appendix A.

Table 4-3: Sediment grain size distribution of the selected samples

Sample	**Sediment grain size (mm)**								
	D_5	**D_{10}**	**D_{16}**	**D_{25}**	**D_{50}**	**D_{75}**	**D_{84}**	**D_{90}**	**D_{95}**
Kleinmond	0.200	0.218	0.240	0.260	0.340	0.470	0.535	0.635	0.805
Bot	0.220	0.275	0.340	0.380	0.430	0.455	0.500	0.540	0.602
Onrus	0.195	0.210	0.245	0.280	0.340	0.401	0.440	0.500	0.580
Klein	0.150	0.160	0.175	0.185	0.200	0.235	0.250	0.288	0.360
Touw	0.180	0.203	0.220	0.250	0.280	0.320	0.340	0.370	0.402

The Wentworth Classification is implemented as a descriptive classification scale for sediment grain size (Krumbein & Sloss, 1963). The sieve test analyses concluded that the survey locations all displayed median grain sizes classified as fine to medium sand (between 0.125 mm and 0.5 mm).

Additional statistics of the grain size distribution are implemented to describe the variation from the standard log-normal distribution displayed by the samples. Qualitative descriptors such as the standard deviation (σ_φ), the phi coefficient of skewness (α_φ) and the phi coefficient of kurtosis (β_φ) are implemented.

The methods, descriptions and scale for determining these additional statistical parameters are provided in Appendix A. The additional statistical parameters describing the survey sediment samples are summarised in Table 4-4.

Table 4-4: Summary of qualitative descriptors of sediment grain size distributions

Sample	Std. deviation (sorting)		Coefficient of skewness		Coefficient of kurtosis	
	$\sigma\varphi$	Comment	$\alpha\varphi$	Comment	$\beta\varphi$	Comment
Kleinmond	0.62	Moderately well sorted	-0.18	Coarse skewed	0.96	Mesokurtic (normal peakedness)
Bot	0.38	Well sorted	0.27	Fine skewed	2.29	Very leptokurtic
Onrus	0.47	Well sorted	0.07	Near symmetrical	1.24	Leptokurtic (peaked)
Klein	0.34	Very well sorted	-0.30	Coarse skewed	1.50	Leptokurtic (peaked)
Touw	0.35	Well sorted	0.10	Fine skewed	1.33	Leptokurtic (peaked)

From these results, it is evident that all the samples are relatively well sorted given their small standard deviations, implying that the particle sizes of the respective samples are grouped closely around their typical sizes. The skewness of the respective samples display a fair amount of variation. A coarse skewed value indicates more outliers toward the coarser sediments, while the inverse is true for fine skewed distributions. A near symmetrical distribution, as found in the Onrus, sample is indicative of zero skewness. The kurtosis of the samples varies from normal peakedness (mesokurtic) to very peaked (very leptokurtic), indicating that the majority proportion of the sediments lies in the middle of the distribution.

4.3.1.3. Beach face slope measurements

The beach face slope surveys comprised of surveying several linear transects that start at the sea water mark and spans the entire berm, running perpendicular to the shoreline, and ending at the estuary water body. An effort was made to start the transect in as deep water as possible without damaging the equipment, by scheduling the start of the survey approximately at the time of spring low tide. The location of the survey transects were selected by identifying the location of possible breaching in the berm crest, also known as the saddle point of the berm. On the day of the survey the Onrus Estuary displayed a semi-closed mouth state. The rest of the estuaries displayed closed mouth state at the time of the survey.

A reading was taken approximately every 5 m on the seaward slope and approximately every 10 m on the leeward slope. Smaller increments were selected for the seaward slope to provide a better sense of undulations along the transect, as the seaward slope is the area of focus. The beach profile survey method is illustrated in Figure 4-8.

Figure 4-8: Author conducting beach face slope measurements at Klein Estuary

A Trimble TSC3 Differential Global Positioning System (DGPS) was utilised for the beach face slope measurements at the selected estuarine berms. The apparatus provides relatively good accuracy, combined with rapid storage of surveyed points. The accuracy attained during the survey was 28 mm for the vertical coordinate and 48 mm for the horizontal coordinate. The apparatus was set up to work in the WG19 spatial coordinate system. Real time corrections were made by the apparatus, converting the ellipsoid heights to the height above geoid (Land Levelling Datum). Thus, all the vertical coordinates were provided relative to MSL.

The survey transects locations and the cross-shore profile plots for the Onrus Estuary berm are provided in Figure 4-9 and Figure 4-10 respectively.

A simple linear slope representative of the beach face is calculated from the beach profile plots. The linear upper beach slope is calculated between the 0 m and + 2 m MSL elevations. The upper beach slope is of interest when considering the wave runup and swash zone interactions responsible for berm growth. The simplified slopes for the respective transects are indicated on the respective profile plots.

It is apparent that the slope nearest to the semi-closed mouth channel (L1) displays the lowest gradient, while the slope nearest to the adjacent beach (L3) displays the steepest gradient. This is often the case at estuary fronting beaches where occasional breaching and berm washover occurs.

Onrus Estuary

Figure 4-9: Aerial image of Onrus Estuary indicating the survey transects of the beach profile measurements and the inlet berm (green) in relation to the adjacent beach berm (orange)

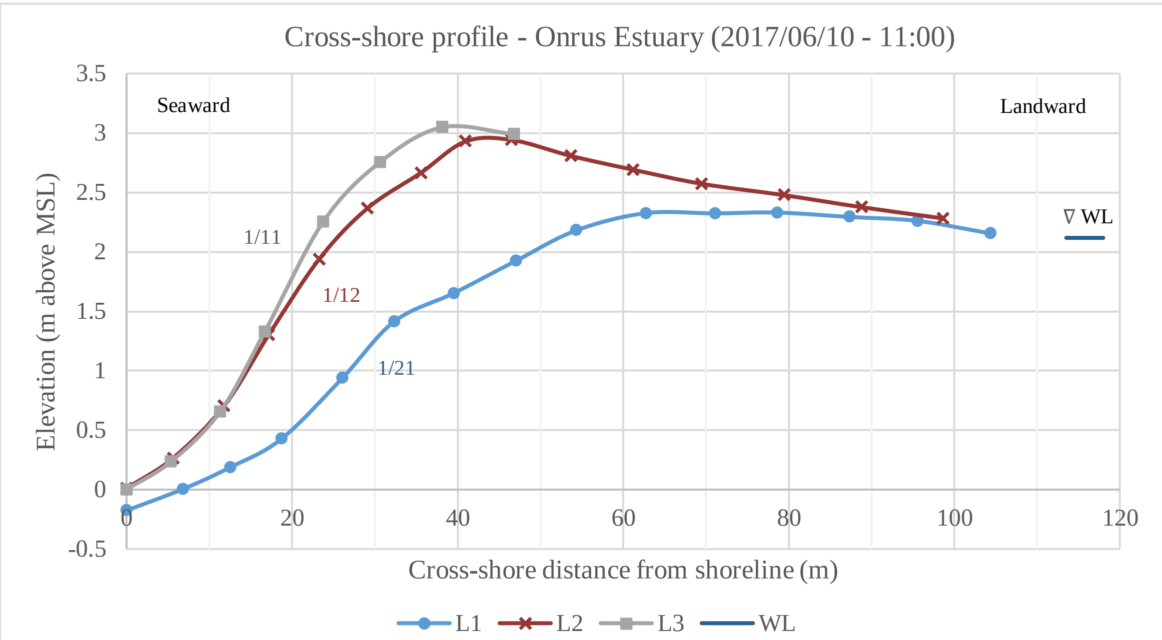


Figure 4-10: Surveyed cross-shore beach profiles at the Onrus Estuary

The crest elevation of the berm directly in front of the estuarine waterbody (refer to Figure 4-9) tends to be lower than the adjacent beach berm. This may be due to the presence of the vegetated dune backing the adjacent beach, acting as a sediment trap. The inlet berm directly in front of the waterbody may be subject to marine overwash, as well as estuary mouth breaching. These processes may cause a lowering of the berm crest, by moving sediment from the crest toward the estuary or sea. From the survey data, it is also evident that the profile displays a steep beach face slope and a gentler sloping landward face. This is attributed to the deposition of marine overwash on the landward face. Higher volumes of overwash present during the initial stages after mouth closure causes a widening of the berm and gentler landward slope.

The cross-shore profile plots, as well as the survey transect locations for the Kleinmond, Bot and Klein Estuaries are provided in Figure 4-11 to Figure 4-16. The remaining beach profile results also indicate a steep seaward face and a gentler landward slope.

Kleinmond Estuary

Figure 4-11: Survey transects of beach face slope measurements at Kleinmond Estuary

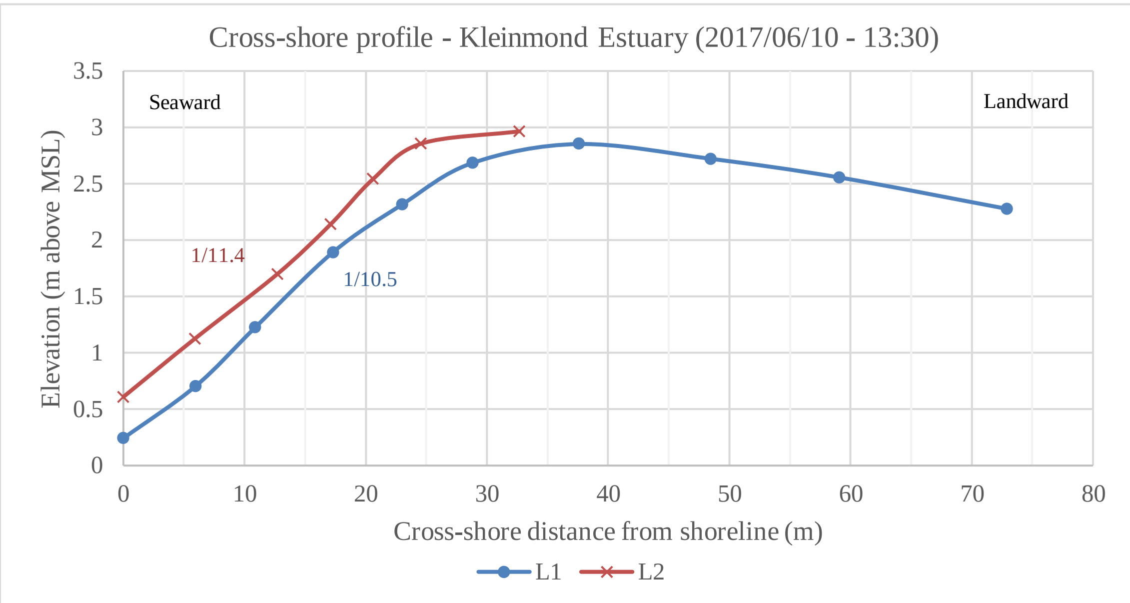


Figure 4-12: Surveyed cross-shore beach profiles at Kleinmond Estuary

Bot Estuary

Figure 4-13: Survey transects of beach face slope measurements at Bot Estuary

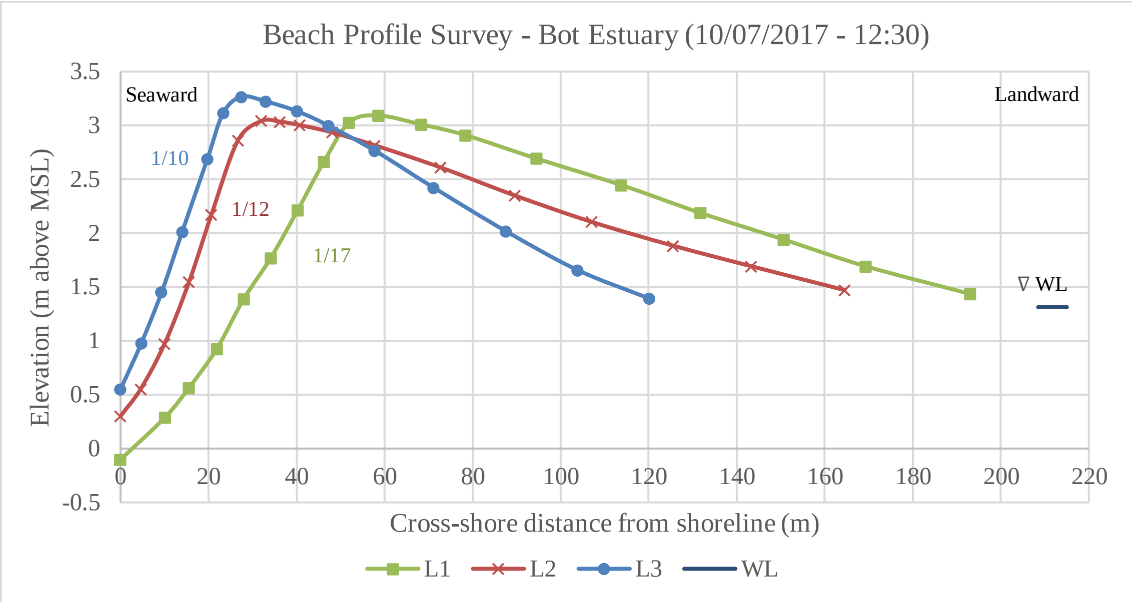


Figure 4-14: Surveyed cross-shore beach profiles at Bot Estuary

Klein Estuary

Figure 4-15: Survey transects of beach face slope measurements at Klein Estuary

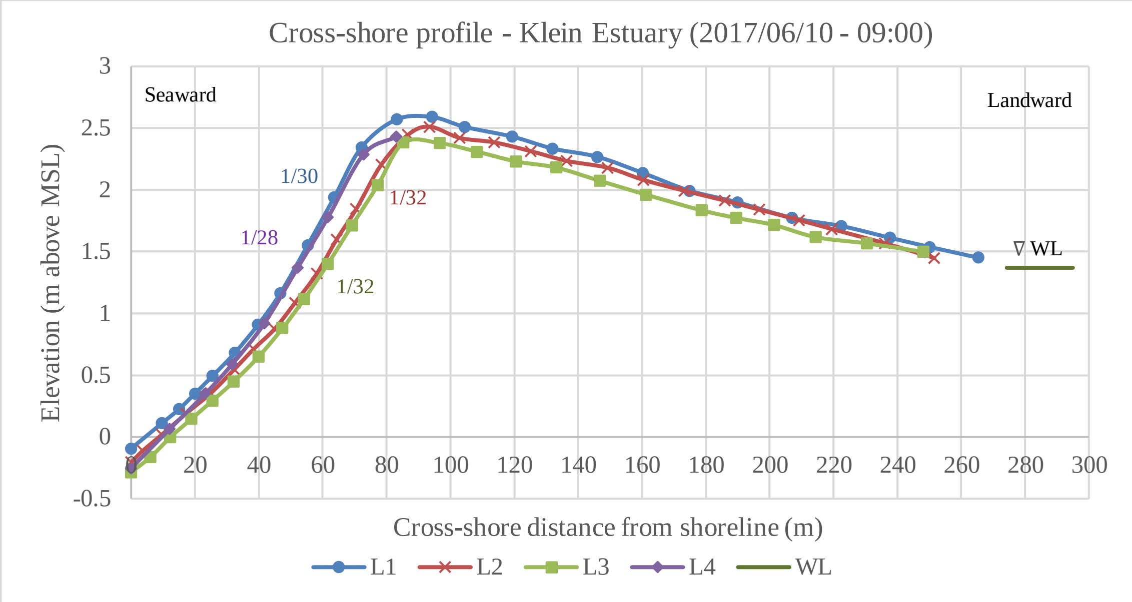


Figure 4-16: Surveyed cross-shore beach profiles at Klein Estuary

4.3.2. Collating available data

The remaining coastal parameters of the selected estuaries were gathered from various sources of available literature.

4.3.2.1. Sediment and beach slope data

The majority of the sediment grain size data were obtained from the CSIR mouth surveys conducted between 1984 and 1999 (CSIR, 2000b; CSIR, 2000c). The sediment samples provided in these reports were collected along the estuarine berms at the time of the estuary mouth surveys.

The majority of the beach face slopes were also derived from the CSIR mouth surveys (CSIR, 2000b; CSIR, 2000c). Only mouth surveys conducted during closed mouth state were considered. A contour plot was generated from the raw survey data (xyz coordinates) with the use of the *Surfer* 11® software package (Golden Software, 2012). From the contour plot it is possible to create a cross-section through the inlet berm and calculate the beach face slope. Several cross-sections were analysed for each respective estuary to obtain an accurate estimate of the typical beach face slopes. Consequently, the average beach face slopes between the 0 m and +2 m MSL elevation were derived from the available survey data.

The distance from the shoreline (at MSL) to the 15 m isobath was collected for the respective estuaries. This horizontal distance is used to calculate a normalised nearshore slope, which is used as input for the Mather *et al.* (2011) runup model. The respective horizontal distances are derived from the inshore bathymetry data used for the nearshore wave modelling conducted by Rossouw *et al.* (2014).

Additional beach profile- and sediment data were gathered from previous CSIR technical reports and Stellenbosch University research papers. The median sediment grain sizes and beach face slopes for the respective estuaries, along with the data sources, are summarised in Appendix C and Appendix D.

4.3.2.2. Nearshore Wave Data

Rossouw *et al.* (2014) determined the inshore wave conditions along the South African coastline by transforming the available offshore wave data through hydrodynamic wave modelling. The NCEP hind cast wave data (NCEP, 2013) from the National Oceanic and Atmospheric Administration (NOAA)/NCEP WAVEWATCH III Global Model (Tolman *et al.*, 2002) were transformed via a medium resolution (0.5 km grid intervals) wave analysis. The analysis comprised 20 models, covering the coastline from Port Nolloth to St Lucia. Each model covered 100 km (approximate) stretch of coastline, with an inshore modelled wave height every 500 m interval. In summation, the model output provides the medium resolution inshore wave climate at 7 m and 15 m water depth, along approximately two-thirds of the South African coastline (Rossouw *et al.*, 2014).

The 1-in-1 year return period modelled wave heights at the 15 m isobath in front of the respective estuary inlets were selected. This is preferred to the longer return period wave heights, as larger waves associated with longer return periods are more probable to initiate breaking at the 15 m isobath. The inshore wave height, prior to the point of breaking, is required for the analysis.

The further calculation/derivation of the required input wave parameters is discussed in § 6.1.1.

4.4. Summary

Berm height data are collected from the mouth/berm surveys (§ 4.1) and estuarine water level recordings (§ 4.2) of the selected estuaries. There are relatively few mouth/berm surveys available for the selected estuaries. The long-term water level data provide several berm crest elevations at the time of breaching, for the respective estuaries.

Ten of the selected estuaries are subject to regular artificial breaching. This could potentially skew the results of the recorded berm crest elevations derived from water level recordings. The breaches included in the water level recording of the Touw Estuary are exclusively artificial, resulting in recorded berm heights that are significantly lower than the berm in its natural state.

Care was taken to ensure the respective berm height recordings of the remaining estuaries, especially the upper range of recordings, are representative of the estuarine berm under natural functioning. The recorded berm crest elevations of the selected TOCEs are analysed and presented in Chapter 5.

The available coastal parameters that describe estuarine berm formation and -functioning are collected (§ 4.3). The parameters include: median sediment grain size of the berm, beach face slope at the berm, nearshore wave data at the estuary inlet. The data was collected by means of field measurements conducted by the author, and from previous field measurements and available literature. The relevant coastal parameters of the selected estuaries are provided in Appendix C through to Appendix E. Additionally, the general estuarine water body characteristics are provided in Appendix B. The coastal parameters are used to identify the primary drivers of high berm elevation, as well as the factors that contribute to variation in berm height among estuaries, in § 5.3 and § 5.4.

Additionally, the coastal parameters are implemented as input to the runup predictions at the respective estuaries in Chapter 6. Further derivation of the necessary wave parameters required for the runup predictions are also discussed in Chapter 6.

5. Recorded Berm Height at South African Temporarily Open/Closed Estuaries

This section provides the recorded berm crest elevations of the selected South African TOCEs. A preliminary grouping of estuaries, based on their respective berm heights is presented. Additionally, the relationship between berm height and the relevant coastal parameters are assessed. Lastly, a method is proposed for estimating the maximum berm height at estuaries, given only a few coastal parameters.

5.1. Analysis of Berm Height

The berm crest elevations derived from the CSIR mouth surveys (§ 4.1) and the DWS estuary water level recordings (§ 4.2) are collated, resulting in a comprehensive record of estuary berm crest elevations. An overview of the total berm crest elevation record is provided in Table 5-1. The total record of berm crest elevations of all the selected estuaries are provided in Appendix G.

Table 5-1: Overview of the recorded berm crest elevation record

	Estuary	Berm survey data points	Breach data points	Water level record length (years)	Average breaches/annum	Combined data points
1	**Lourens**	0	6	9.4	0.6	6
2	**Palmiet**	0	70	24.9	2.8	70
3	**Kleinmond**	0	5	8.6	0.6	5
4	**Bot**	2	19	36.1	0.5	21
5	**Onrus**	0	125	20.3	6.1	125
6	**Klein**	1	35	36.4	1.0	36
7	**Hartenbos**	4	147	22.2	6.6	151
8	**Klein Brak**	3	2	19.6	0.1	5
9	**Groot Brak**	4	80	27.5	2.9	84
10	**Touw**	0	16	10.0	1.6	16
11	**Piesang**	0	11	4.8	2.3	11
12	**Groot**	0	120	14.3	8.4	120
13	**Tsitsikamma**	0	143	21.9	6.5	143
14	**Seekoei**	4	9	10.6	0.9	13
15	**West Kleindemonde**	8	0	-	-	8
16	**East Kleinemonde**	0	31	11.9	2.6	31
17	**Mngazi**	0	21	12.1	1.7	21
18	**Mhlanga**	0	136	11.0	12.4	136
19	**Mdloti**	0	18	11.9	1.5	18
20	**Tongati**	0	34	9.3	3.7	34

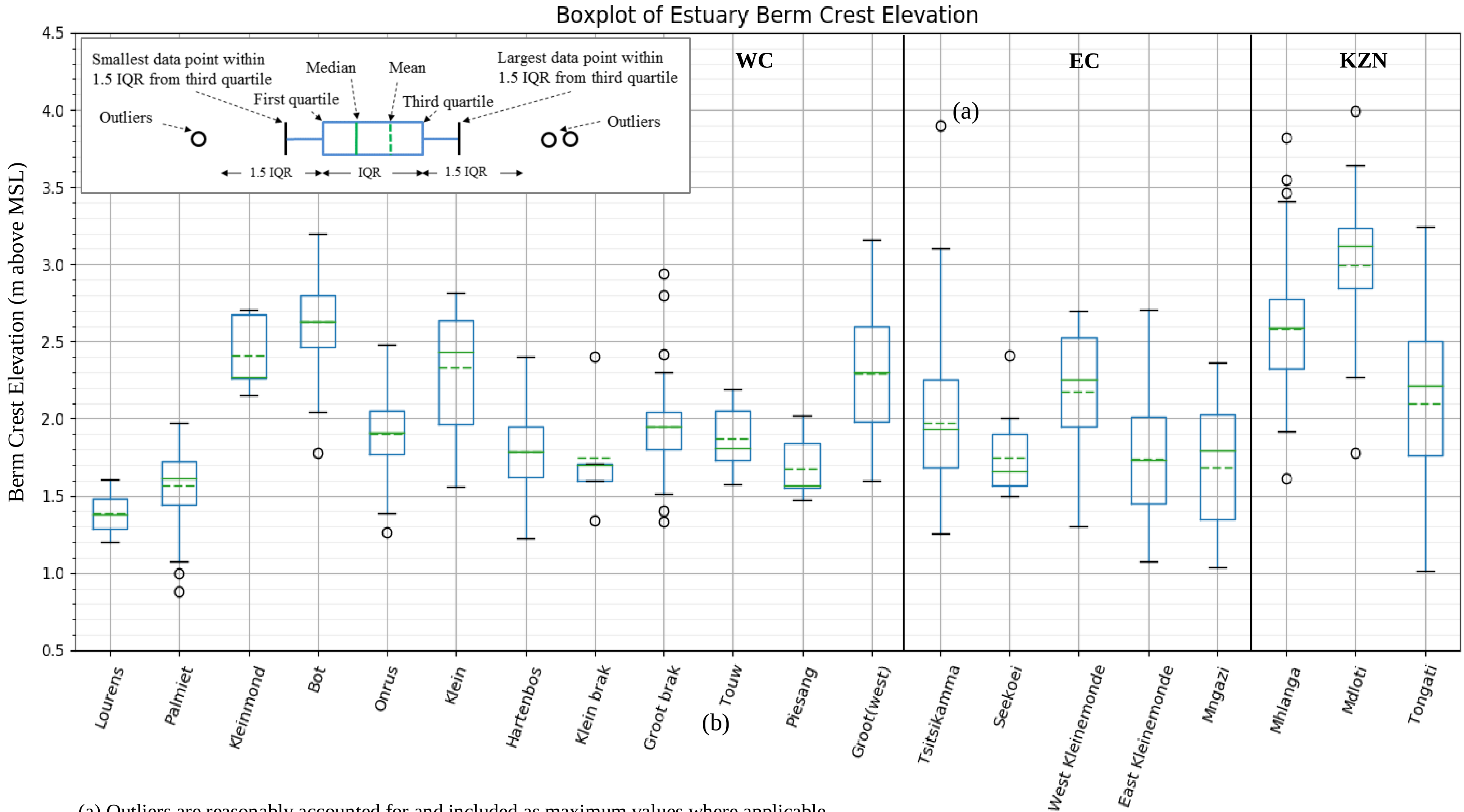
The individual estuary sample sizes (number of data points) vary considerably, mostly due to the variation in the water level record length and the frequency of breaching at the respective estuaries. The length of the respective water level records range between approximately 5 years to 36 years. In most cases a suitable number of recorded berm crest elevations are available. The Lourens-, Kleinmond- and Klein Brak estuaries have relatively fewer data points due to their low rates of average breaches per annum. The West Kleinemonde estuary does not have a water level recorder within the estuary, however there were a total of eight mouth surveys conducted during closed mouth state. Nonetheless, the records with fewer recorded berm crest elevations were not discarded, as they displayed an adequate range of values and compared well to the available information on berm heights at the respective estuaries.

A basic statistical analysis is conducted on the combined records to provide a quantitative sense of potential berm crest elevations and variability at the selected estuaries in South Africa. A boxplot is implemented to graphically display several features of the berm height data set. The boxplot simultaneously provides the data centre (median), mean, spread, range, departure from symmetry and outliers in the data set. A boxplot of the recorded estuary berm crest elevations is provided in Figure 5-1.

This collective record of berm crest elevations provides evidence of the berm height variability and potential maximum berm crest elevations at the estuaries. The sample sizes at the respective estuaries are relatively large, especially compared to previous estimates typically based on limited survey results.

Note that the recorded berm crest elevations for the Touw estuary are not representative of the berm under natural conditions (§ 4.2.3.3). Natural breaching at the estuary occurs at water levels exceeding +3 m MSL (Russel, 2013). The range of berm crest elevations for the Touw estuary provided in Figure 5-1 is merely a representation of the typical water level in the estuary during artificial breaching.

It is possible that the record (Figure 5-1) does not include extreme high levels of berm crest elevation at a given estuary, due to the limited data availability. Nonetheless, due diligence was performed to ensure the provided berm crest elevations are an acceptable representation of the berm crest elevation under natural conditions.



(a) Outliers are reasonably accounted for and included as maximum values where applicable.

(b) Touw Estuary data is not an accurate representation of natural berm levels due to frequent artificial breaching (§ 4.2.3.3).

Figure 5-1: Boxplot providing a statistical summary of the recorded berm crest elevations of the selected South African TOCEs

There are several outliers among the individual estuary records, i.e., high or low values that fall beyond 1.5 Inter Quartile Ranges (IQR) from the first- or third quartile. Low outliers are typically due to a mouth survey conducted during the initial stages of berm recovery, or an artificial breach conducted at an inadequately low water level. Estuaries with smaller water bodies, such as the Onrus estuary, often experience several smaller breaches in succession due to the rapid rate of estuary infilling. Therefore, a low outlier could also be a result of a natural breach during the initial stage berm recovery. High outliers are typically a result of a high berm survey recording, or a natural breach at high water level among a record primarily containing recordings of artificial breaching at lower water levels. Such is the case for the Groot Brak - and Seekoei Estuary. Alternatively, high outliers could merely be a result of favourable hydrodynamic conditions leading to a berm crest elevation associated with low exceedance probability (rare occurrence) included in the record. All outliers in the data set are reasonably accounted for by the natural behaviour of estuary mouth functioning. Therefore, the outlier values are taken as the maximum berm crest elevations for the cases that exhibit high outliers.

The symmetry of the frequency distribution for the individual estuaries can be graphically obtained from the boxplots. Additionally, the skewness coefficient is used to determine the exact deviation from symmetry. There is considerable variation in distribution symmetry among the individual estuaries. None of the locations exhibit perfect symmetrical behaviour, i.e., none conform exactly to the bell shaped (normal or Gaussian) distribution. The Onrus Estuary exhibits a distribution nearest to symmetrical (Figure 5-2), with skewness coefficients of -0.18, nevertheless being slightly negatively skewed (skewed to the left).

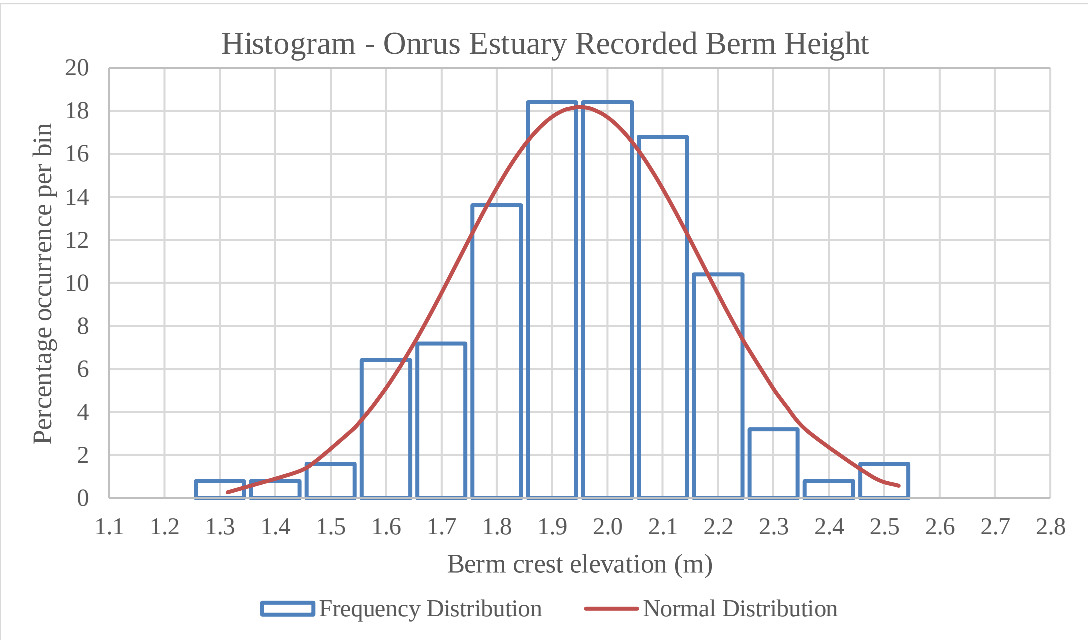


Figure 5-2: Frequency distribution of recorded berm heights at Onrus Estuary, provided to illustrate the distribution symmetry

The estuaries are almost evenly divide between positively- and negatively skewed, with skewness coefficients ranging from -0.88 to 0.96. There is no clear trend associated with the skewness of the frequency distributions among estuaries. Variation of the distribution skewness is most likely attributed to the variation in record length among estuaries, artificial breaching at the estuary, breaching frequency and specific mode of mouth functioning.

It is evident from the boxplot (Figure 5-1) that there is a considerable difference between the mean - and the maximum berm height at the respective estuaries. The upper range of berm crest elevations at an estuary is of greater significance than the mean berm crest elevation. A focus is placed on the potential maximum elevation an estuary berm can reach, rather than the most probable berm crest elevation at the time of breaching. The upper range of recorded berm crest elevations are provided in Figure 5-3. The upper range is presented by means of the 95th percentile, 98th percentile and maximum berm crest elevations.

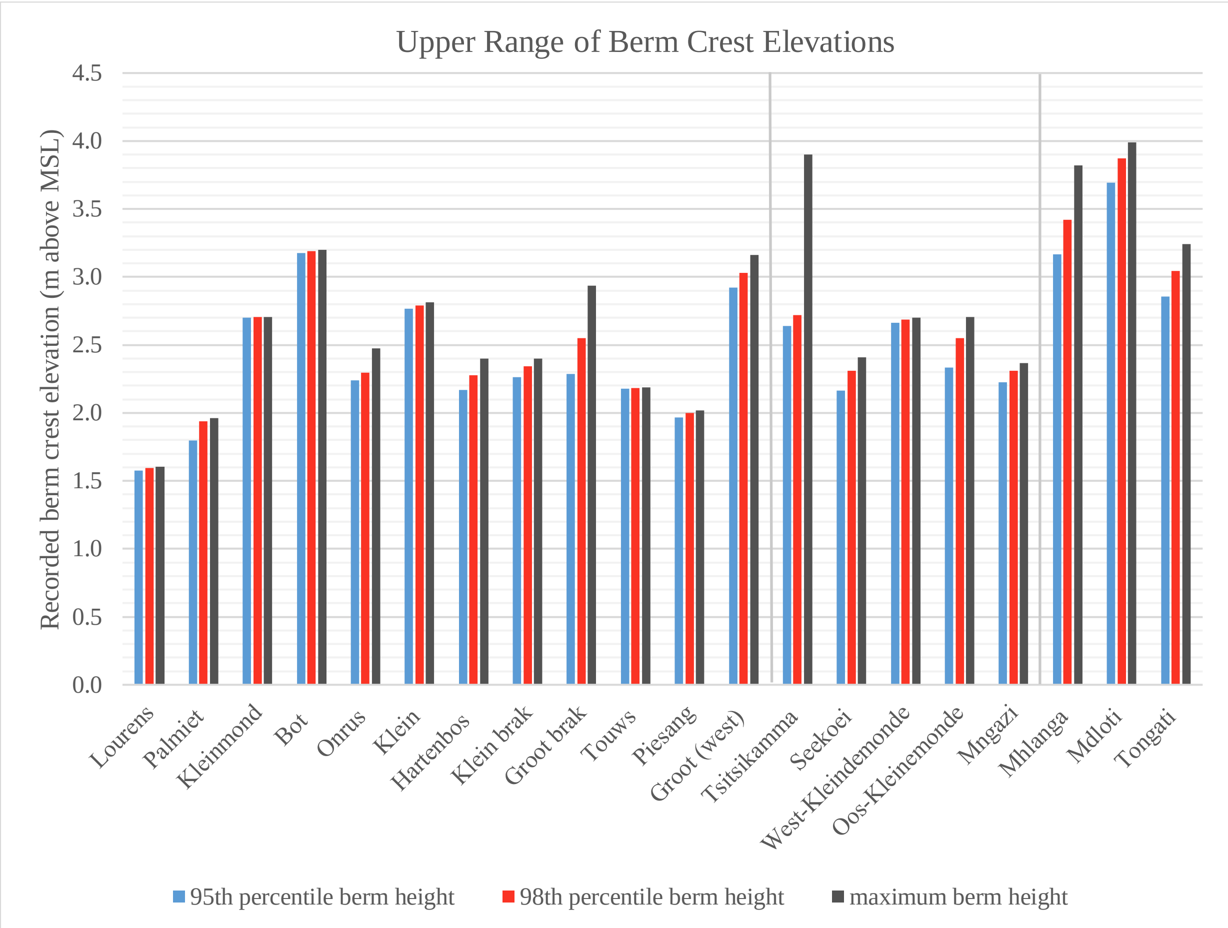


Figure 5-3: Upper range of recorded berm crest elevations of the selected South African TOCEs

The 98th percentile berm height is relatively closely matched with the maximum berm height for all estuaries, except for Tsitsikamma. The large variation at Tsitsikamma is due to a single outlier in the water level record. When scrutinising the record, there was no clear evidence indicating a recording device malfunction. The 98th percentile value can be used as a more probable estimate of the upper range of the berm crest elevation at an estuary.

5.2. Preliminary Classification of Estuaries

Collectively the Western Cape and Eastern Cape estuaries display a similar range of berm height, however the KwaZulu-Natal estuaries display notably higher berms. A preliminary qualitative grouping of estuaries with corresponding ranges of berm crest elevations is provided. The grouping is based on the wave exposure, location, mouth functioning and breaching frequency of the estuaries. A comprehensive analysis of the relationship between berm height and the relevant coastal parameters is provided in § 5.3.

Semi-closed (generally lower berm height)

- Lourens
- Palmiet
- Mngazi
- Onrus

These estuaries are relatively exposed to wave action, however the unique mode of estuary mouth functioning regulates the potential maximum berm crest elevation. These semi-closed estuaries typically exhibit a shallow, narrow opening which allows water to slowly flow out the estuary, while allowing minimal (or zero) tidal exchange. Even during closed mouth state, these estuaries exhibit a very pronounced saddle point with a significantly lower elevation compared to the rest of the berm. Semi-closed mouth state is typically associated with smaller estuaries (Van Niekerk *et al.*, 2002). These smaller estuaries do not require excessive river inflow to fill up and initiate breaching. Thus, breaching often occurs shortly after closure at a lower berm height. There is a considerable difference in berm height when comparing the Lourens - and Palmiet Estuary, to the Mngazi - and Onrus Estuary. However, when comparing one of these semi-closed estuaries to a traditional TOCE with similar wave height, sediment grain size and beach slope, the semi-closed estuary typically displays significantly lower berm height.

High river flow (lower berm height)

- Klein Brak

Relatively small estuaries with high rates of river inflow experience very limited mouth closure. The Klein Brak Estuary only breached twice in the recorded period of approximately 20 years. This is due to the high Mean Annual Runoff (MAR) in the river catchment. Cooper (2001) classified the Klein Brak Estuary as Type F, a barred normally open estuary with MAR exceeding 15×10^6 m^3. However, the estuary does experience occasional closure. Mouth closure is often closely followed by breaching, due to the high river inflow. This leaves very little time for the berm building processes to form a high berm.

Significantly sheltered from wave action (lower berm height)

- Piesang
- Seekoei
- Hartenbos

These estuaries are significantly sheltered from incoming wave action. They are all located within half heart bays, directly in the lee of the headland. The reduced nearshore wave height at the estuary inlet limits the potential height of the berm. The Palmiet Estuary could also be considered for this category, as the estuary inlet is tucked closely behind a rocky promontory, protecting it from incoming waves.

Higher exposure to wave action (medium to high berm height)

- Kleinmond
- Groot Brak
- Klein
- East Kleinemonde
- West Kleinemonde
- Tsitsikamma
- Bot
- Groot (Natures Valley)

These estuaries are generally more exposed to offshore waves, leading to higher runup and potentially higher berms. Variation in berm height among these estuaries is contributed to frequency of breaching and other site specific coastal variables such as sediment grain size and beach face slope. The Bot – and Klein Estuary exhibit infrequent breaching, with approximately one breach per annum. Less frequent breaching extends the potential berm building period, possibly leading to a higher berm.

Perched KwaZulu-Natal estuaries (high berms)

- Mhlanga
- Mdloti
- Tongati

These estuaries are significantly perched, with berms typically displaying a coarser grain sediment and steep beach slopes (reflective conditions). This coupled with relatively high wave energy leads to high levels of wave runup and potential high berm crest elevations. These estuaries also display variation in breaching frequency, which may lead to differences in berm height.

5.3. Relationship Between Berm Height and Coastal Parameters

This section aims to identify the main coastal parameters responsible for high berm crest elevations, as well as berm height variability among South African estuaries.

Berm growth is triggered by the deposition of sediment at the landward extent of wave runup. Existing runup parameterisations are typically a function of wave height (offshore or nearshore), wavelength and beach face slope (e.g. Nielsen & Hanslow, 1991; Stockdon *et al.*, 2006). Berms are depositional features formed on sandy and shingle beaches, therefore the sediment properties should play a role in the potential berm height (Okazaki & Sunamura, 1995). The Dean number is a measure typically used to determine the direction of sediment transport (onshore versus offshore) and is a function of the sediment fall velocity among other variables (Dean, 1973). Since berm growth requires suspended sediment to deposit as the flow velocity of overwash/runup reduces near the existing crest, the grain size of the sediment should control the potential for sediment deposition. Therefore, estuarine berm crest elevation is a function of the following coastal parameters:

- Wave height and - period
- Sediment grain size
- Beach face slope
- Iribarren number

The relationship between estuarine berm crest elevation and the above-mentioned coastal parameters are tested. The coastal parameters are compared to the maximum berm height, as well as the 98th percentile berm height. The berm height of each respective estuary is plotted against the corresponding coastal parameters at the estuary. Linear regression lines are fitted to the data and the coefficient of determination (R^2) is used to assess the linear relationship among variables.

Additionally, the relationship between berm height and these coastal parameters may provide initial insight toward the suitability and performance of the selected berm height predictors (§ 6.2.1).

Wave height

The wave heights at the relevant estuary inlets are evaluated to determine its effect on estuarine berm height. The inshore wave height at the estuary mouth is preferred, as it includes the effect of the nearshore wave transformation processes at the specific location.

The relationship between the 1-in-1 year modelled wave height at 15 m water depth (§ 4.3.2.2) and berm crest elevation of the respective estuaries is presented in Figure 5-4. The relationship of 98th percentile - and maximum berm height are provided.

The data displays only a crude relationship (R^2 = 0.03 and 0.06) between the inshore wave height and the berm crest elevation. The considerable scatter indicates a very weak linear relationship between wave height and berm height. However, this does not signify that berm height is independent of wave height, but rather that wave height alone is not an adequate predictor of berm height.

The wave periods of the 1-in-1 year modelled nearshore wave events were also compared to the corresponding berm height at the respective estuaries. Wave period may influence the direction of sediment transport (onshore vs offshore), however it does not adequately describe the variation in berm height among estuaries.

Sediment grain size

The relationship between the median sediment grain size of the berm and the berm height (98th percentile and maximum) is presented in Figure 5-4. A rather insignificant linear relationship is evident (R^2 = 0.48 and 0.52). Nevertheless, the general trend indicates that coarser grain sediment is associated with higher berms. The three Kwa-Zulu Natal estuaries are clear outliers in the data, although in this case they improve the linear relationship.

Beach slope

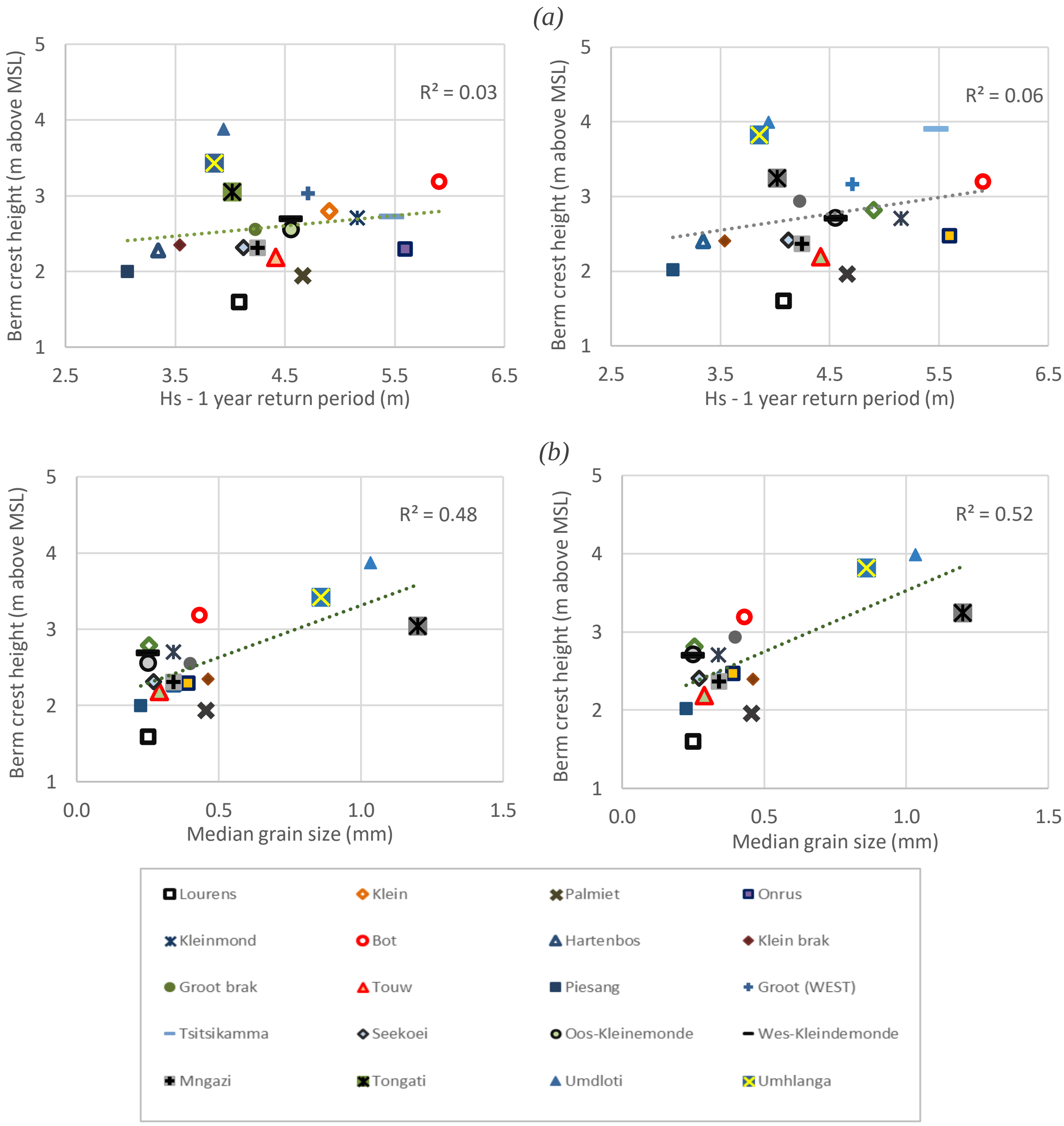
The relationship between the beach face slope at the berm, and the berm height of the estuaries is presented in Figure 5-4. The beach face slope is measured between 0 m and +2 m MSL. A significant linear relationship is displayed between variables (R^2 = 0.72 and 0.69). This suggests that runup prediction models implementing the beach face slope might be superior to models relying purely on wave height for predicting estuary berm crest elevation. Additionally, runup models with greater sensitivity to changes in beach face slope may prove more reliable in predicting berm crest elevations.

The inshore slopes between the 0 m and 15 m isobath, typically used for the Mather *et al.* (2011) runup model, were also evaluated. The results displayed an insignificant relationship between the inshore slope and berm height at the estuaries.

Iribarren number

The nearshore Iribarren numbers for the respective estuaries are calculated by means of the 1-in-1 year modelled nearshore wave height and wave length. The relationship between the nearshore Iribarren numbers and the estuary berm heights is presented in Figure 5-4. The Iribarren number represents the combined effect of wave steepness (H_0/L_0) and beach face slope on the berm height. A significant linear relationship is displayed between variables (R^2 = 0.70 and 0.66). This indicates that higher berms are typically associated with intermediate to reflective conditions, while lower berms are associated with dissipative conditions. The linear relationship of Iribarren number vs. berm height does not provide an improvement on the beach slope vs. berm height relationship.

98th Percentile berm height relationship: **Maximum berm height relationship:**



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98th Percentile berm height relationship: **Maximum berm height relationship:**

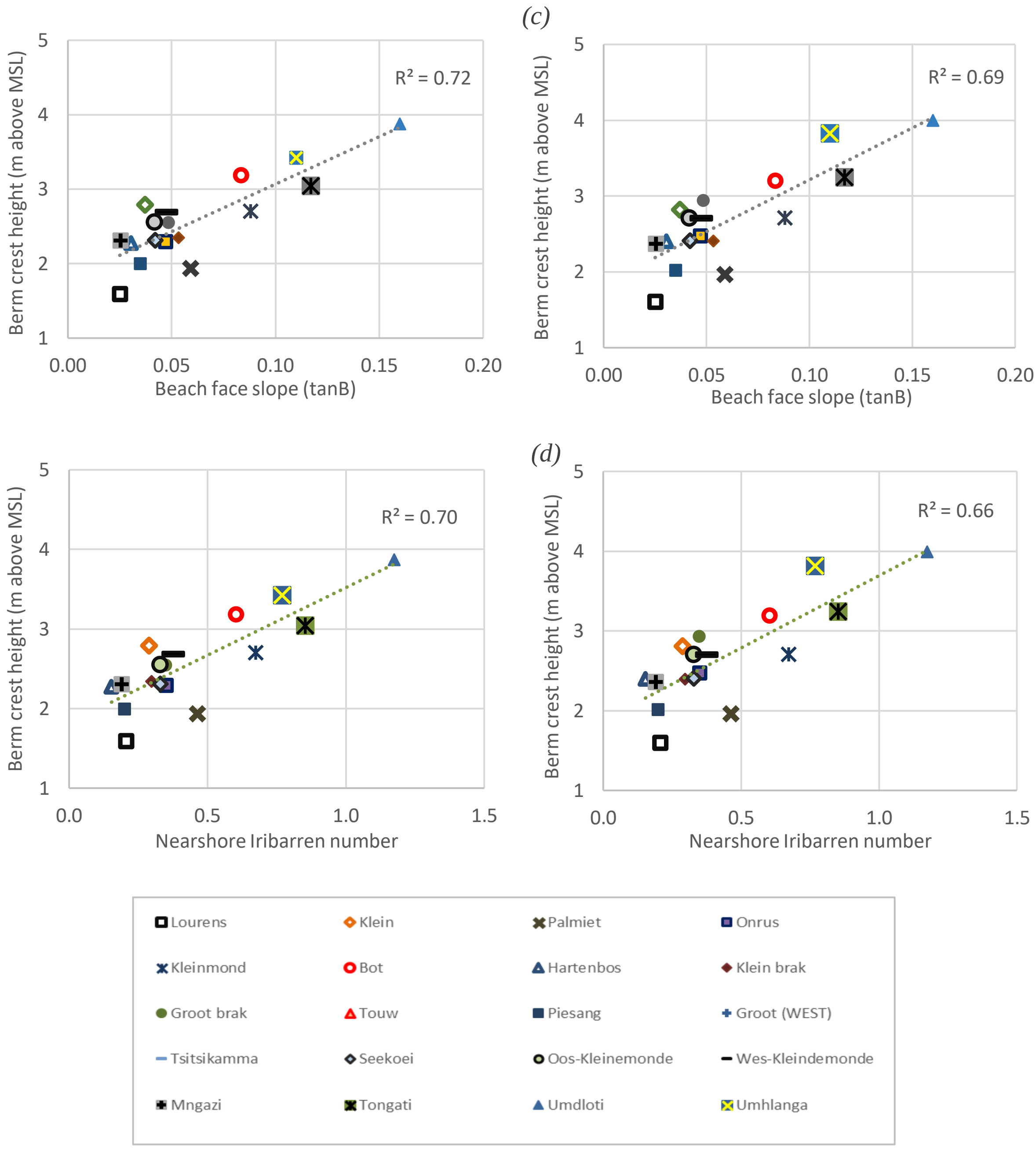

Figure 5-4: 98th percentile ranked berm height (left) and maximum berm height (right) recorded at the respective estuaries, plotted as a function of (a) nearshore wave height, (b) median grain size, (c) beach face slope and (d) nearshore Iribarren number

The relationship between the mean berm heights and the relevant coastal variables were also tested. The results displayed a weaker linear relationship to the coastal parameters, compared to the results from 98th percentile - and maximum berm heights.

The breaching frequency at the respective estuaries were also compared to the berm height. The data shows only a crude relationship with the breaching frequency. The poor correlation indicates the lack of linear relationship between high berms and fewer breaches per annum, however it does not conclude that berm growth is independent of time. Additionally, the estuary water area, tidal volume and MAR were compared to berm height for the respective estuaries, resulting in relatively crude linear relationships.

The linear relationships presented in Figure 5-4 are based on fairly limited data, however the data suggests that higher berms tend to occur on more exposed beaches, with a coarser grain sediment and steeper slopes.

5.4. Berm Crest Elevation Criteria

The relationship between estuarine berm height and the relevant coastal parameters viz., wave height, sediment grain size, beach face slope and nearshore Iribarren number have been discussed in the previous section. However, the combined effect and relative importance of these parameters have not been evaluated.

A multi-criteria analysis, namely the Berm Crest Elevation Criteria, is developed to evaluate the combined effect of the relevant coastal parameters on estuarine berm height. The Berm Crest Elevation Criteria follows a similar basic outline to the criteria developed by Coelho *et al.* (2006). Coelho *et al*. implemented criteria to assess the vulnerability of the coastal zone.

The Berm Crest Elevation Criteria aims to simplify the complex interactions present during the berm building process, in order to estimate the potential maximum berm crest elevation given only a few coastal parameters. This is achieved by evaluating the combined effect of the relevant parameters by means of a weighting system. The method involves calculating a berm crest elevation index from the weighted score of each individual parameter. The relevant criteria are developed from the field data of a wide range of South African estuaries.

5.4.1. Criteria parameters

There are several other available parameters specific to estuarine functioning that contribute to the potential berm height. These parameters include: breaching frequency, estuary water area, tidal volume and MAR. However, the proposed criterion relies on the effect of the coastal parameters, as they provide a more significant relationship when compared to berm height (§ 5.3).

The four relevant coastal parameters implemented in the criteria are presented in Table 5-2. The tidal range is added to the previously mentioned parameters. The tidal level contributes to the maximum level of runup at the estuary berm. The tidal range along the South African coastline remains relatively uniform, resulting in an identical score for all South African estuaries. The tidal range is included mainly to represent the effect it has on the potential berm height. The effect of tidal range on the overall score is more pronounced for locations displaying variability in their tidal ranges. The median grain size (D_{50}) has been included to compensate for the roughness and permeability of the sediment, as well as the effect of grain size on sediment deposition. The beach face slope was selected as it has a relatively good relationship with berm height (§ 5.3). The Iribarren number was omitted, as it does not provide an improved relationship to berm height, compared to the beach face slope. The nearshore modelled significant wave height is included to represent the known effect of wave height on runup and berm height. The relevant score is scaled from 1 to 10, ensuring adequate sensitivity for the respective parameters.

Table 5-2: Individual parameter scoring for Berm Crest Elevation Criteria

Berm Height Index Classification										
Score:	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**
TR – mean tidal range (m)	< 1	1 - 2	2 - 3	3 - 4	4 - 5	5 - 6	6 - 7	7 - 8	8 - 9	> 9
SED – Median grain size, D_{50} (mm) [a]	<0.1	0.1-0.2	0.2-0.3	0.3-0.4	0.4-0.5	0.5-0.6	0.6-0.7	0.7-0.8	0.8-0.9	>0.9
BS - Beach face slope (tanβ) [b]	<0.02	0.02-0.04	0.04-0.06	0.06-0.08	0.08-0.1	0.1-0.12	0.12-0.14	0.14-0.16	0.16-0.18	>0.18
H – Significant wave height [c]	<3	3 - 3.5	3.5 - 4	4 - 4.5	4.5 - 5	5 - 5.5	5.5 - 6	6 - 6.5	6.5 - 7	>7

[a] median sediment grain size sampled from the estuary inlet berm

[b] beach face slope at the berm measured between 0 m and + 2 m MSL

[c] 1-in-1 year nearshore significant wave height measured in 15 m water depth in front of the estuary inlet.

5.4.2. Parameter weighting

All the selected coastal parameters play a crucial role in the formation of berms at estuary inlets. However, their relative degree of importance has not yet been explored. A parameter weighting system is employed to investigate the relative importance of a single variable in combination with the rest. Four different parameter weighting systems have been developed and are presented in Table 5-3. The individual parameter weighting factor is multiplied to the corresponding parameter score. The four individual parameter weighted scores are added together and averaged over the total weighting count. Consequently, a weighted score is assigned to each individual estuary.

Table 5-3: Parameter weighting coefficients

Parameter	Weighting 1	Weighting 2	Weighting 3	Weighting 4
TR - Mean tidal range (m)	1	0.5	1	1
SED - Median sediment grain size (mm)	1	2	3	5
BS - Beach face slope (tanβ)	1	2	4	10
H - Significant Wave Height (m)	1	0.5	2	2
Total	4	5	10	18

Weighting 1 explores the equal weighting of all relevant parameters, i.e., all the parameters play a similarly important role in the process. Weighting 2 relies on the reduced effect of mean tidal range and nearshore significant wave height. This weighting is motivated by the relatively uniform tidal range along the South African coastline, as well as the crude relationship between nearshore wave height and berm height displayed in § 5.3. Weighting 3 is based on a ranking of parameters on a scale from 1 to 4, according to their estimated relative importance. The relative importance of each parameter is motivated by their relationship to berm height as discussed in § 5.3. Weighting 4 is based on assigning a value of relative importance to each variable. The value is scored out of ten and is based on the author's discretion. Note that the weighting criteria are based on the proposed relative importance of parameters at TOCEs in South Africa. For example, the tidal range is not considered an important parameter for describing the variability of berm crest elevations among South African estuaries. However, the tidal range criterion could play a more prominent role when considering estuaries with large differences in tidal range.

5.4.3. Berm Crest Elevation Criteria results

The criteria analysis involves calculating a weighted berm height score (S) for an individual estuary based on the specific input parameters and the selected weighting system. The first step is to calculate the estuary's individual parameter score on a scale of 1 to 10 for **TR**, **SED**, **BS**, **H** (Table 5-2). Secondly, each individual parameter score is multiplied by the corresponding weighting factor provided in Table 5-3. The weighted scores are added together and divided by the relevant weighting total to get the average weighted score. The individual parameter scores, as well as the weighted average scores (S) for the respective estuaries are presented in Table 5-4.

The Touw Estuary is excluded from the analysis due to frequent artificial breaching. The Groot and Tsitsikamma estuaries are also omitted from the analysis, due to the lack of available sediment and beach slope data at the estuaries.

Table 5-4: Berm height classification based on weighted parameter scores

Estuary	Parameter Score				Weighted Berm Height Score (*S*)			
	TR	SED	BS	H	Weighting 1	Weighting 2	Weighting 3	Weighting 4
Lourens	2	3	2	4	2.75	2.60	2.70	2.50
Palmiet	2	5	3	5	3.75	3.90	3.90	3.72
Kleinmond	2	4	5	6	4.25	4.40	4.60	4.67
Bot	2	5	5	7	4.75	4.90	5.10	5.06
Onrus	2	4	3	7	4.00	3.70	4.00	3.67
Klein	2	3	2	5	3.00	2.70	2.90	2.61
Hartenbos	2	4	2	2	2.50	2.80	2.60	2.56
Klein Brak	2	5	3	3	3.25	3.70	3.50	3.50
Groot Brak	2	4	3	4	3.25	3.40	3.40	3.33
Piesang	2	3	2	2	2.25	2.40	2.30	2.28
Seekoei	2	3	3	4	3.00	3.00	3.10	3.06
West Kleinemonde	2	3	3	5	3.25	3.10	3.30	3.17
East Kleinemonde	2	3	3	5	3.25	3.10	3.30	3.17
Mngazi	2	4	2	4	3.00	3.00	3.00	2.78
Mhlanga	2	9	6	3	5.00	6.50	5.90	6.28
Mdloti	2	10	9	3	6.00	8.10	7.40	8.22
Tongati	2	10	6	4	5.50	7.00	6.40	6.67

The ability of the weighted score to predict the maximum berm crest elevation at an estuary was evaluated. Each weighting system was evaluated by comparing the weighted estuary scores (S) to the maximum recorded berm crest elevations at the respective estuary. Simply put, the weighting system that provided the best linear relationship with the relevant recorded maximum estuary berm heights, was considered the most accurate predictor. The performance of the respective weighting systems was tested by means of the coefficient of determination (R^2), the Mean Absolute Error (MAE), and the Root Mean Square Error Predictor (RMSEP) (Table 5-5). The formulae and descriptions of these statistical performance indicators are provided in Appendix F.

Table 5-5: Performance of Berm Crest Elevation Criteria according to the respective weighting systems

Weighting System	R^2	MAE (m)	RMSEP (m)
Weighting 1	0.65	0.28	0.36
Weighting 2	0.68	0.27	0.34
Weighting 3	0.69	0.27	0.34
Weighting 4	0.70	0.26	0.33

The results indicate the best performance from the weighting systems that favour the beach face slope (**BS**) as the dominant parameter, i.e., weighting 3 and -4. Weighting 4 presents the highest R^2-value, and smallest MAE and RMSEP values at 0.7, 0.26 m and 0.33 m respectively.

The weighted criteria scores (Weighting 4) and the corresponding maximum recorded berm crest elevations for the respective estuaries are plotted in Figure 5-5. A linear regression model is presented to provide the relationship between parameters. The prediction - and confidence intervals are based on a 95% confidence level. The regression model (black line) provides the maximum berm crest elevation (B_C) based on the weighted score (S) at a given estuary.

Figure 5-5: Linear regression model indicating the performance of the Berm Crest Elevation Criteria and Weighting 4

The predictive capabilities of the Berm Crest Elevation Criteria can be further improved by manipulating the weighting procedure. The semi-closed estuaries (Lourens, Palmiet, Onrus and Mngazi) typically display lower berm crest elevations than traditional TOCEs with similar coastal parameters. A reduction factor (*rf*) is proposed to alter the weighted scores of semi-closed estuaries. By multiplying an arbitrary factor of 0.5 to the final weighted score of a semi closed estuary, the prediction significantly improves. The reduction factor aims to compensate for the pronounced effect of the shallow, narrow channel at a semi-closed estuary mouth. As previously discussed (§ 5.2), the frequent presence of this semi-closed channel leads to a significantly pronounced saddle point in the berm during mouth closure. The saddle point displays a large difference in elevation when compared to the rest of the berm. Additionally, semi-closed estuaries typically exhibit smaller water bodies which can fill up rapidly after mouth closure and initiate early breaching.

A reduction factor of 0.5 adjusts the criteria score of the four semi-closed estuaries, to better fit the linear trend of the remaining 13 TOCEs, while ensuring accurate predictions. The reduction factor should not be interpreted as a data manipulation technique, but rather as an alteration to the proposed synthesised Berm Crest Elevation Criteria. Reduction factors ranging between 0.2 and 0.8 were tested for the model. The selected value (*rf* = 0.5) provided a superior linear relationship, while retaining the prediction accuracy for semi-closed estuaries.

The Weighting 3 scores, calculated with a semi-closed estuary reduction factor (*rf*) of 0.5, are plotted against the recorded maximum berm heights in Figure 5-6. The reduction factor presents the best results when used in conjunction with Weighting 3. The reduction factor was applied to the scores of all the prominent semi-closed estuaries in the data set, namely Lourens, Palmiet, Onrus and Mngazi.

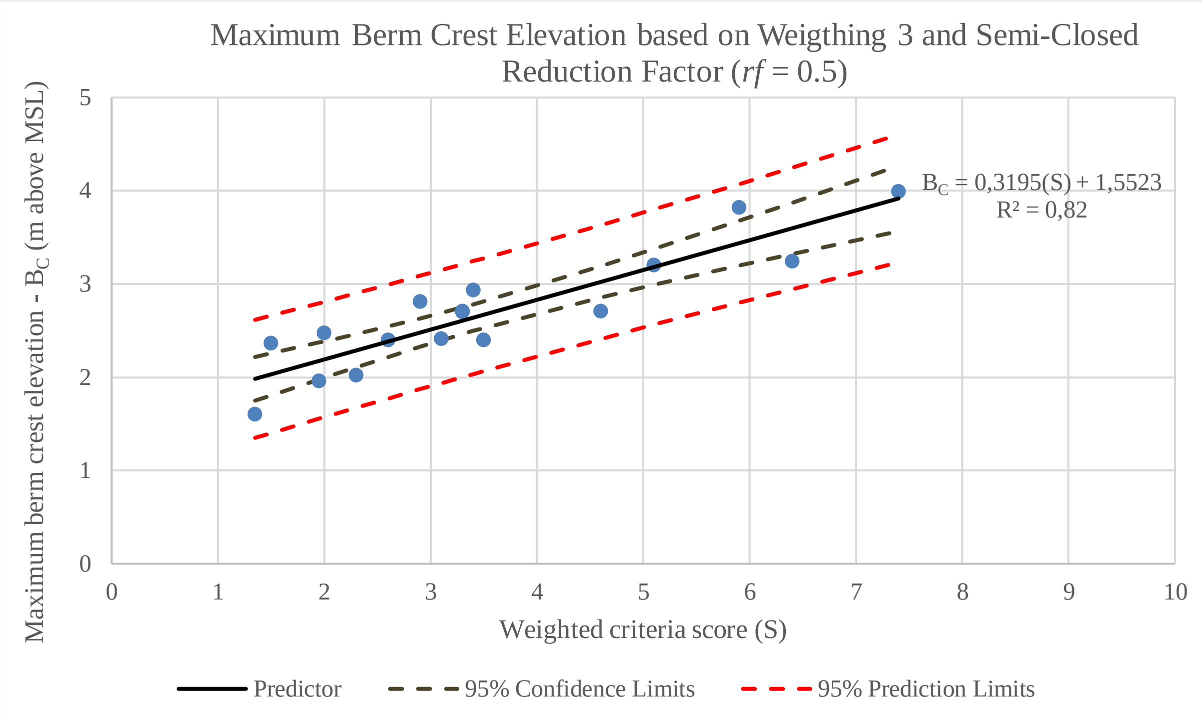

Figure 5-6: Linear regression model indicating the performance of the Berm Crest Elevation Criteria and Weighting 3 with reduction factor (rf = 0.5)

The regression model (black line) provides the maximum berm crest elevation (B_c) based on the Weighting 3 score (S) and a reduction factor (rf) of 0.5, where applicable, at a given estuary. The regression model provides the following relationship:

$$B_c = 0.3195(S) + 1.5523 \tag{5.1}$$

where

- B_c = Maximum berm crest elevation of selected estuary (m above MSL)
- S = Weighted criteria score (per Weighting 3 and rf = 0.5, refer to Table 5-2 & 5-3)

Implementing the reduction factor for semi-closed estuaries has improved the model accuracy to:

- R^2 = 0.82
- MAE = 0.23m
- RMSEP = 0.26 m
- P-value = 7.04 x 10^{-7} (P-value < 0.05 indicates significant relationship for 95% confidence)

The regression model (Eq. 5.1) provides a good first estimate of the maximum berm crest elevation of South African TOCEs. It should be noted that the Berm Crest Elevation Criteria is developed from the data of only 17 estuaries, which is deemed a relatively small data set for linear regression. However, given the data scarcity and the limited previous knowledge of estuarine berm heights in South Africa, the performance of the predictor is deemed adequate.

The 17 estuaries used as input cover a wide range coastal parameters. The range of input parameters of the estuaries used in the model are as follows:

- Median sediment grain size: 0.225 mm – 1.200 mm
- Beach face slope (tanβ): 0.025 – 0.160
- Nearshore significant wave height: 3.1 m – 5.9 m (1-in-1 year, d = 15 m)

5.4.4. Calculation procedure

This section provides an example of the calculation procedure involved in the Berm Crest Elevation Criteria and corresponding regression model (Eq. 5.1). The calculation steps for the maximum berm height prediction of the Quinira Estuary are presented. The relevant coastal parameters (Appendix C to Appendix E) for the Quinira Estuary are as follows:

- **TR** – Mean tidal range: 1.59 m (based on mean spring tidal range)
- **SED** – Median sediment grain size: 0.196 mm (measured at the seaward berm face)
- **BS** – Beach face slope: 0.043 (at the berm between 0 m and +2 m MSL)
- **H** – Nearshore significant wave height: 4.1 m (1-in-1 year at 15 m depth)

Step 1

The individual parameter scores are assigned according to the criteria set out in Table 5-2, resulting in:

- **TR** – score: 2
- **SED** – score: 2
- **BS** – score: 3
- **H** – score: 4

Step 2

The relative importance and combined effect of the selected parameters are established by means of the Weighting 3 coefficients provided in Table 5-3. The individual parameter weighting coefficients are multiplied with the respective scores, resulting in the following weighted parameter scores:

- **TR** – weighted score: 2 x 1 = 2
- **SED** – weighted score: 2 x 3 = 6
- **BS** – weighted score: 3 x 4 = 12
- **H** – weighted score: 4 x 2 = 8

Step 3

The individual weighted scores are added together and divided by the Weighting 3 total (10) provided in Table 5-3, resulting in an average weighted berm height score (*S*) for the Quinira Estuary:

- Average weighted berm height score: (2+6+12+8)/10 = 2.8

The average weighted score is multiplied by the reduction factor (rf = 0.5), in the case of semi-closed estuaries. However, the Quinira Estuary does not display a semi-closed mouth state.

Step 4

The weighted score (S) is used as input for the regression model (Eq. 5.1), resulting in the following maximum berm crest elevation (B_c) prediction:

- $B_c = 0.3195(S) + 1.5523$

  $= 0.3195(2.8) + 1.5523$

  $= 2.45\ m$ above MSL

The Quinira Estuary was excluded from the respective analyses in Chapter 5 due to scanty berm height data. Only four surveys were conducted during closed mouth state, with the maximum recorded berm height (saddle point) of 2.5 m above MSL (CSIR, 2000c). This closely corresponds with the maximum predicted value of 2.45 m.

5.5. Conclusion

The collective record of berm crest elevations (§ 5.1) provides evidence of the berm height variability and potential maximum berm crest elevations at the selected estuaries. The sample sizes at the respective estuaries are relatively large compared to previous estimates, typically based on limited survey results.

The preliminary classification of estuaries (§ 5.2) provides a qualitative grouping based on berm height, wave exposure, location, mouth functioning and breaching frequency.

The relationship between the berm height at the respective estuaries (maximum and 98th percentile) and the corresponding coastal parameters were tested in § 5.3. Beach face slope and nearshore Iribarren number (based on the 1-in-1 year nearshore wave event) provide the best linear relationship when compared to berm height.

The Berm Crest Elevation Criteria (§ 5.4) and corresponding regression model (Eq. 5.1) provides the maximum berm crest elevation at an estuary based on the mean tidal range, median sediment grain size of the berm, beach face slope at the berm and the 1-in-1 year nearshore significant wave height measured in 15 m water depth. The Berm Crest Elevation Criteria provides a realistic estimate (MAE = 0.23 m and RMSEP = 0.26 m) from limited data inputs. The criteria can be used as a relatively accurate first estimate for other, less studied estuaries.

6. Evaluating Berm Height Predictors

This chapter aims to explore the relationship between runup and the maximum berm crest at an estuary. The landward extent of potential sediment deposition at the berm is predicted by evaluating the runup elevation at the berm. The procedure involves the use of existing semi-empirical runup parameterisations to predict the level of runup at an estuary. A proposed methodology to predict the long- and short-term berm crest elevations is presented. Long-term predictions refer to the maximum probable berm crest elevation over a given period (typically several years). Short-term predictions refer to the maximum berm crest elevation on a time scale of individual breaches.

6.1. Data Requirements

The data requirements, as well as the data acquisition process, for the berm prediction methods are discussed in this section. The main input parameters for the runup and profile predictors include: wave height and - period, tidal levels, beach face slope and median sediment grain size. Since the sediment and slope data acquisition has already been discussed in § 4.3, this section will focus primarily on the wave and tidal data.

Estuary mouth functioning is highly dynamic, often exhibiting spatial variations such as mouth migration and other long-term morphological changes. Wave- and tide recordings roughly coinciding with the respective berm height records are required for the analysis. This aims to avoid any long-term variations in mouth morphology, not accounted for in the berm height recordings.

6.1.1. Wave data

The CSIR continuously monitors real time waves at various locations around the South African coastline on behalf of Transnet National Ports Authority (TNPA). The majority of the data are gathered by means of moored wave buoys. A historical database of recorded wave data is available upon special request from TNPA and the CSIR.

6.1.1.1. Data acquisition

The requested wave records were dependent on the availability of data at the desired locations. Wave recording apparatus within the nearest proximity of the respective estuaries were identified. Other factors considered for the selection of wave records include: predominant wave direction at the recorder in relation to the estuary inlet, period and length of available records compared to berm height record, gaps in the record, water depth at wave recorder, and measurement units of recorded waves (e.g. peak wave period or mean wave period).

The recorded wave data from the requested locations are provided in 3 hourly intervals of significant wave height (H_S) and peak wave period (T_P). Inevitably, the wave data contain gaps in the records, primarily due to instrument malfunctions. The 3 hourly interval wave data were converted to hourly intervals by means of linear interpolation. Missing data exceeding a continuous 3-hour period was not filled by interpolation, hence no data for that period was used for the analysis. The selected wave buoys are presented in Table 6-1.

Table 6-1: Properties of selected wave recording devices for respective estuaries

	Estuary	Wave recorder	Recorder type	Depth (m)	Coordinates
1	**Lourens**	Strand	Virtual buoy	16.85	34° 7'12.17"S 18°47'13.21"E
2	**Palmiet**	Cape Point	Directional Waverider buoy	70	34°12'12.00"S 18°17'12.00"E
3	**Kleinmond**				
4	**Bot**				
5	**Onrus**				
6	**Klein**				
7	**Hartenbos**	Mossel Bay	Non-directional Waverider buoy	24	34° 8'7.97"S 22° 9'9.10"E
8	**Klein Brak**				
9	**Groot Brak**				
10	**Piesang**	FA Platform	Radar	113	34°58'12.00"S 22°10'12.00"E
11	**Seekoei**				
12	**West Kleinemonde**	East London	Directional Waverider buoy	27	33° 2'1.78"S 27°56'19.40"E
13	**East Kleinemonde**				
14	**Mngazi**				
15	**Mhlanga**	Durban Bluff	Directional Waverider buoy	30	29°53'2.40"S 31° 4'14.43"E
16	**Mdloti**				
17	**Tongati**				

The wave data for the Lourens Estuary (Strand) is generated by means of a numerical wave transformation model, termed a virtual buoy. The virtual buoy implements the SWAN (Simulating Waves Nearshore) numerical model to propagate the offshore wave forecast into False Bay. The SWAN virtual buoy model is also operated and maintained by the CSIR.

6.1.1.2. Nearshore wave transformation

Nearshore wave transformational processes such as wave shoaling, refraction, diffraction and white capping contribute to the wave conditions at the estuary inlet. Accurate predictions of wave runup at the respective estuary inlets should include these nearshore transformational processes. Ignoring these effects will lead to several estuaries using the same wave conditions as input, as there may only be a single wave buoy within the proximity.

Detailed modelling of the nearshore transformational wave processes was not a viable option, due to the high number of estuaries/locations under consideration. Consequently, a site-specific wave transformation coefficient (K_T) was developed to account for these nearshore processes and provided an approximate nearshore significant wave height.

The wave transformation coefficients were developed by comparing the modelled nearshore significant wave height (15 m water depth) at the estuary inlets (§ 4.3.2.2), to the recorded wave height from the CSIR apparatus (as listed in Table 6-1).

The modelled wave heights (1-in-1 year) at the 15 m isobath in front of the respective estuary inlets were selected and compared to the 1-in-1 year return period wave height from relevant wave buoy records. The 1-in-1 year return period wave height for the respective wave recording devices are presented in Table 6-2.

Table 6-2: 1 Year return period wave heights derived from wave recordings at selected buoys

Wave recorder	1-in-1 Year H_s at recorder (m)	Source
Cape Point	8.30	Cartwright *et al.* (2012)
Mossel Bay	4.00	Clarke (2016)
FA Platform	8.70	Von Saint Ange (2017)
East London	4.86	Rossouw (1989)
Durban Bluff	4.20	Corbella and Stretch (2012)

The modelled wave heights at the respective estuaries include the effect of the nearshore wave transformational processes. The 1-in-1 year nearshore modelled wave height at 15 m water depth is divided by the 1-in-1 year recorded wave height (at the buoy) to get a site specific wave transformation coefficient for each estuary location. The wave transformation coefficients (K_T) for the respective estuaries are presented in Table 6-3.

Table 6-3: Proposed nearshore wave transformation coefficients for selected locations

	Estuary	1-in-1 year modelled H_s at 15 m depth (m)	1-in-1 year H_s at recorder (m)	Wave transformation coefficient (K_T)
1	**Lourens**[a]	-	-	-
2	**Palmiet**	4.664	8.30	0.562
3	**Kleinmond**	5.156	8.30	0.621
4	**Bot**	5.902	8.30	0.711
5	**Onrus**	5.594	8.30	0.674
6	**Klein**	4.904	8.30	0.591
7	**Hartenbos**	3.346	4.00	0.837
8	**Klein Brak**	3.542	4.00	0.886
9	**Great Brak**	4.228	4.00	1.057
10	**Piesang**	3.065	8.70	0.352
11	**Seekoei**	4.122	8.70	0.474
12	**West Kleinemonde**	4.557	4.86	0.938
13	**East Kleinemonde**	4.557	4.86	0.938
14	**Mngazi**	4.249	4.86	0.874
15	**Mhlanga**	3.859	4.20	0.919
16	**Mdloti**	3.942	4.20	0.939
17	**Tongati**	4.019	4.20	0.957

[a] The Lourens Estuary uses the strand virtual wave buoy modelled wave data at d = 16.85 m

Note: The K_T coefficient is multiplied with the recorded wave heights at the relevant buoy, in order to obtain nearshore (d = 15 m) equivalent wave heights.

The coefficient (K_T) acts as a transformation factor, reducing the wave height as it progresses nearshore. The site-specific coefficient compensates for the reduction in wave energy caused mainly by refraction and shoaling and to a lesser extent diffraction. The coefficient values vary considerably among locations. Smaller coefficients are typically associated with wave buoys located in deeper water, while larger coefficient represent buoys located near the estuary inlet. The Groot Brak coefficient causes a minor increase in wave height between the wave buoy and nearshore (d = 15 m). The Mossel Bay wave buoy is located in 24 m water depth, relatively near the Groot Brak Estuary mouth. The recorded wave height at the buoy already includes the majority effect of the nearshore processes. Offshore waves generally exhibit a reduction in height travelling towards the coastline. Nearing the shoreline in shallower water, the wave height could potentially start to increase or decrease as wave shoaling takes effect.

The coefficient provides a basic transformation of the recorded wave heights in deep/transitional water, to nearshore equivalent waves at the 15 m isobath. The 1-in-1 year return period wave heights for both the nearshore models and the offshore records were used due to their availability, and the fact that they represent a comparable value. The method implements the same coefficient at a location, regardless of wave direction or wave period. This simplification is inevitable, given the selected method. Fortunately, the majority of the wave recorders are located in shallow to intermediate water depths (Table 6-1). Therefore, the recorded waves typically include the majority effect of the nearshore wave transformation processes, such as diffraction around headlands and wave refraction. The accuracy of the proposed coefficients may vary, depending on the location of the estuary inlet in relation to the wave recorder. In most cases, the wave transformation coefficients adjust the incident wave height to account for the final wave refraction and -shoaling that takes place in transitional/shallow water depths.

In summation, the recorded wave heights at the respective locations/buoys are transformed to nearshore (d = 15 m) equivalent wave heights at the estuary inlets by means of the respective transformation coefficients (K_T). This is achieved by multiplying the recorded significant wave height at the buoy with the relevant K_T coefficient (Table 6-3).

6.1.1.3. Deep water equivalent waves

The wave data are recorded in varying water depths, ranging from transitional to deep water (17 -113 m). All the selected wave parameterisation require the deep water significant wave height as input, except for the Okazaki and Sunamura (1995) Model, which requires the wave breaker height. The effective deep water, refracted, significant wave heights are calculated for the respective wave records. This ensures a consistent measurement of waves for each estuary. The nearshore (d = 15 m) transformed significant wave heights are reverse shoaled to obtain the deep water equivalent unrefracted wave heights (H_0'). The waves are reverse shoaled using linear wave theory, while assuming a shore normal approach.

The Nielsen and Hanslow (1991) Model requires the characteristics of the root mean square deep water wave height as input. The relationship between the significant wave height (H_s) and the root mean square wave height (H_{rms}), assuming a Rayleigh distribution of wave heights, is as follows (CIRIA, 2007):

$$H_{rms} = 0.706 H_s \tag{6.1}$$

6.1.1.4. Wave breaker height

The Okazaki and Sunamura (1995) Model requires the breaker height and period of mean waves as input. The mean breaker heights and mean wave periods are averaged over the entire berm building period, to use as a single wave height and period input to estimate the equilibrium berm height.

The relationship between significant wave height (H_s) and the mean wave height (H_m), assuming a Rayleigh Distribution of wave heights, is as follow (CIRIA, 2007):

$$H_m = 0.626H_s \tag{6.2}$$

The following relationship was implemented for the transformation of peak period (T_p) to mean period ($T_{m-1,0}$) (CIRIA, 2007).

$$T_p = 1.1T_{m-1.0} \tag{6.3}$$

Okazaki and Sunamura (1995) proposed the relationship developed by Komar and Gaughan (1973), for the transformation of deep water wave height to wave breaker height. The wave breaker height relationship of Komar and Gaughan was provided in Equation 2.18 (§ 2.6.4.2).

6.1.2. Tidal data

The instantaneous tidal elevation contributes towards the elevation of a wave runup event. Berms are often formed in a short period, therefore the short-term variation in tidal elevation is required to predict the water level on a beach at a given time. The tidal gauges within the nearest proximity to the respective estuaries were identified. The hourly recorded inshore sea levels were obtained from the UHSLC database (Caldwell *et al.*, 2015). The respective estuaries and their corresponding tidal gauges are presented in Table 6-4.

Table 6-4: Selected sea level recording devices corresponding to estuary locations

Estuary	Tidal Gauge/Port
Lourens, Palmiet, Kleinmond, Bot, Onrus, Klein	Simons Town
Hartenbos, Klein Brak, Groot Brak, Piesang	Mossel Bay
Seekoei	Port Elizabeth
West – and East Kleinemonde, Mngazi	East London
Mhlanga, Mdloti, Tongati	Durban

The runup prediction procedure requires coinciding tide- and wave records for the selected estuaries. Accordingly, long-term inshore sea level records coinciding with the acquired wave records were obtained.

The recorded tidal elevations from the UHSLC are referenced to Chart Datum (CD). The tidal levels are adjusted to LLD (equal to MSL) as per the correction factors provided by the South African Navy Hydrographic Office (SANHO, 2017). The correction factors are specific to a time period and were used accordingly. The hourly tidal level, relative to MSL, will represent the SWL for that specific hour, upon which the predicted runup level is superimposed.

6.2. Long term – Relationship Between Berm Height and Wave Runup

The primary objective of this section is to determine the relationship between wave runup and the maximum berm crest elevation at an estuary. The approach relies on the evidence that the maximum berm crest height can be estimated by examining the long-term variation in runup levels (Behrens *et al.*, 2015). The maximum berm crest elevations derived from the total berm height record (several years of data), are compared to an overlapping record of predicted runup at the estuary.

Additionally, the analysis aims to identify the most accurate runup parameterisation for the prediction of berm crest height. Runup parameterisations which are well suited to predict runup at typical beach profiles, may not be equally successful in predicting runup at estuarine berms. Berm profiles are inherently different than typical beach profiles. Beach profiles are typically associated with the presence of a scarp or dune in the back-beach area, which may limit wave runup under certain circumstances. Estuarine berms do not exhibit dunes or scarps, allowing high levels of runup to overflow and reach the estuarine water body. This is typically termed overwash, or washover.

6.2.1. Runup parameterisations

The formulae of the selected runup parameterisations are provided in Table 6-5. A detailed description of the input parameters, applicability and the development of each model were provided in § 2.6.4.

The Okazaki and Sunamura (1995) Model and the Larson and Kraus (1989) Model were omitted from the long-term analysis. The Okazaki and Sunamura Model relies on a phase averaged wave height to estimate the berm crest elevation, following a period of accretionary wave events. The Okazaki and Sunamura Model is aimed specifically at predicting berm accretion, and does not account for berm erosion. The proposed methodology involves evaluating several years of wave runup, which includes the presence of accretionary and erosional wave events. Therefore, the use of an average wave height representative of the entire record is unrealistic.

The Okazaki and Sunamura (1994) Model, as well as the Larson and Kraus (1989) Model are better suited for estimating the berm elevation on a shorter time scale, for instance between individual breaches. Therefore, these method will be tested accordingly (§ 6.5).

Table 6-5: Summary of selected runup and berm height parameterisations

Author	Formula
Nielsen & Hanslow (1991) – tanβ>0.1	$R_{2\%} = SWL + 1.98[0.6(H_{0rms}L_0)^{0.5}tan(\alpha)]$
Nielsen & Hanslow (1991) – tanβ<0.1	$R_{2\%} = SWL + 1.98[0.05(H_{0rms}L_0)^{0.5}]$
Ruggiero *et al*. (2001) Model 1	$R_{2\%} = 0.5H_0 + 0.22$
Ruggiero *et al*. (2001) Model 2	$R_{2\%} = 0.27(SH_0L_0)^{0.5}$
Stockdon *et al*. (2006)	$R_{2\%} = 1.1\left(0.35\beta_f(H_0L_0)^{1/2} + \frac{[H_0L_0(0.563\beta_f^2 + 0.004)]^{1/2}}{2}\right)$
Mather *et al*. (2011)	$R_{2\%} = WL + C\,H_0\left(\frac{15}{x_h}\right)^{2/3}$
Swart (1974)	$B_c = D_{50}(7644 - 7706e^{-A})$ $A = 0.000143\frac{H_0^{0.488}T^{0.93}}{D_{50}^{0.786}}$
Larson and Kraus (1989)	$\frac{B_C}{H_0} = 1.47\left[\frac{tan\alpha}{\sqrt{H_0/L_0}}\right]^{0.79}$
Okazaki & Sunamura (1995)	$\frac{B_h}{(gT^2)^{5/8}H_b^{1/8}D^{1/4}\emptyset} = 0.134$ $\emptyset = e^{(-0.04D_*^{0.55})}$ | $D_* = D\left[\frac{g\left(\frac{\rho_s}{\rho}-1\right)}{\upsilon^2}\right]^{1/3}$

6.2.2. Procedure

A long-term record, consisting of several years of predicted wave runup ($R_{2\%}$) is simulated for each estuary. The length of the simulated runup records are dependent on the availability of wave- and tidal data and are selected to coincide with the berm height records of the respective estuaries. The hourly recorded wave- and tidal data, as discussed in § 6.1, are used as input for the simulation.

The runup elevation ($R_{2\%}$) is predicted at an hourly time step, then superimposed on the hourly tidal elevation. The predicted runup is calculated according to each of the selected runup parameterisations. The result of the procedure is a simulated record of hourly predicted runup (superimposed on hourly recorded tide), calculated according to each runup model, for each respective estuary. The simulation accounts for the joint effect of the hourly tide, - wave height and - wave period on the total runup elevation at the berm.

The length and the percentage missing data of the respective simulated records are provided in Table 6-6. The relatively large amount of missing data is due to gaps in both the wave- and tidal data. In some instances, the sea level recordings exhibit large amounts of missing data. Nonetheless, the simulated record length is deemed adequate for the respective estuaries.

Table 6-6: Summary of runup simulation periods for respective estuaries

	Estuary	Simulated runup record length (years)	Percentage complete (%)	Effective record length (years)
1	**Lourens**	9.5	73.5	7.0
2	**Palmiet**	21.9	56.9	12.5
3	**Kleinmond**	9.5	73.3	7.0
4	**Bot**	21.9	56.9	12.5
5	**Onrus**	21.9	56.9	12.5
6	**Klein**	21.9	56.9	12.5
7	**Hartenbos**	9.6	92.9	8.9
8	**Klein Brak**	9.6	92.9	8.9
9	**Groot Brak**	9.6	92.9	8.9
10	**Piesang**	4.8	66.7	3.2
11	**Seekoei**	4.8	64.9	3.1
12	**West Kleinemonde**	24.8	55.5	13.7
13	**East Kleinemonde**	24.8	55.5	13.7
14	**Mngazi**	13.2	73.0	9.6
15	**Mhlanga**	9.4	67.5	6.3
16	**Mdloti**	9.4	67.5	6.3
17	**Tongati**	9.4	67.5	6.3

The following aspects are regarded as constant throughout the entire simulation: beach face slope at the berm, median sediment grain size of the berm, and a closed mouth state. There is very limited available information regarding the variability of the above mentioned constant factors. Knowledge regarding the frequency distribution of mouth state (open or closed) at the respective estuaries can aid in the analysis of berm height. However, this did not form part of the scope of this study.

The assumption regarding the constant closed mouth state was as follow: given a prolonged period, an acceptable distribution of wave runup would occur at the berm during the cumulative closed mouth period. Therefore, the longer the length of the simulated runup record, the safer the assumption becomes. Thus, the longest possible record of simulated runup was generated for each respective estuary.

The selected beach face slopes for the respective locations were measured during closed mouth state. Care was taken not to select a beach face slope that was measured shortly after mouth closure (at a low berm level). Instead, the selected beach face slopes are representative of medium to high berm levels at the respective estuaries.

The long-term runup levels are then compared to the maximum recorded berm crest elevations. The comparison between runup levels and berm height is conducted according to each individual runup parameterisation. The results of the analyses are discussed in the following section.

6.2.3. Results of the wave runup predictions

Initially, the highest value of predicted runup from a record (maximum $R_{2\%}$) was compared to the maximum recorded berm height at an estuary. The highest predicted runup levels exceeded the berm height in all scenarios. This is expected, as runup levels exceeding the berm crest are classified as overwash. Estuarine berms are subject to frequent overwash during the initial stages of berm growth. As the berm grows, overwash becomes less frequent and associated with energetic wave conditions. Most South African TOCEs are subject to marine overwash, however the frequency varies among locations. Thus, the highest level of wave runup (maximum $R_{2\%}$) obtained from several years of predicted runup levels, is a definite overestimate of the maximum berm height.

This considered, the next step was to establish the threshold of runup that corresponds to the berm height. This approach assumes that the berm height is approximately equal to a given percentile ranked runup elevation in the simulated record, i.e., the berm height is governed by a level of runup that is reached a finite number of times within a specified period.

The 98th percentile ranked runup elevation, derived from the hourly simulated runup record, was selected to compare against the maximum recorded berm height at the respective estuaries. The 98th percentile value of the predicted record of hourly $R_{2\%}$, is the runup elevation below which 98% of other observations fall. This estimate excludes the effect of extreme levels of runup, associated with longer return periods. Runup caused by extreme waves and concurrent long waves (infra-gravity waves), typically during storm events, often lead to erosion of beaches and berms.

The 98th percentile ranked predicted runup at each estuary was plotted against the maximum recorded berm height at the corresponding estuary. This is done for each runup parameterisation, in order to identify the most accurate predictor. Each plotted data set comprises the maximum berm heights and 98th percentile ranked $R_{2\%}$ predictions of 17 estuaries.

The accuracy of the predictors was measured by means of the coefficient of determination (R^2), the Mean Absolute Error (MAE) and the Root Means Square Error Predictor (RMSEP).

6.2.3.1. Nielsen and Hanslow (1991) Model

The wave runup levels predicted by the Nielsen and Hanslow (1991) Model were compared to the berm heights at the respective estuaries. Each data point in Figure 6-1 represents the maximum recorded berm height at an estuary, and the 98th percentile ranked $R_{2\%}$ value from the simulated runup record at the same estuary.

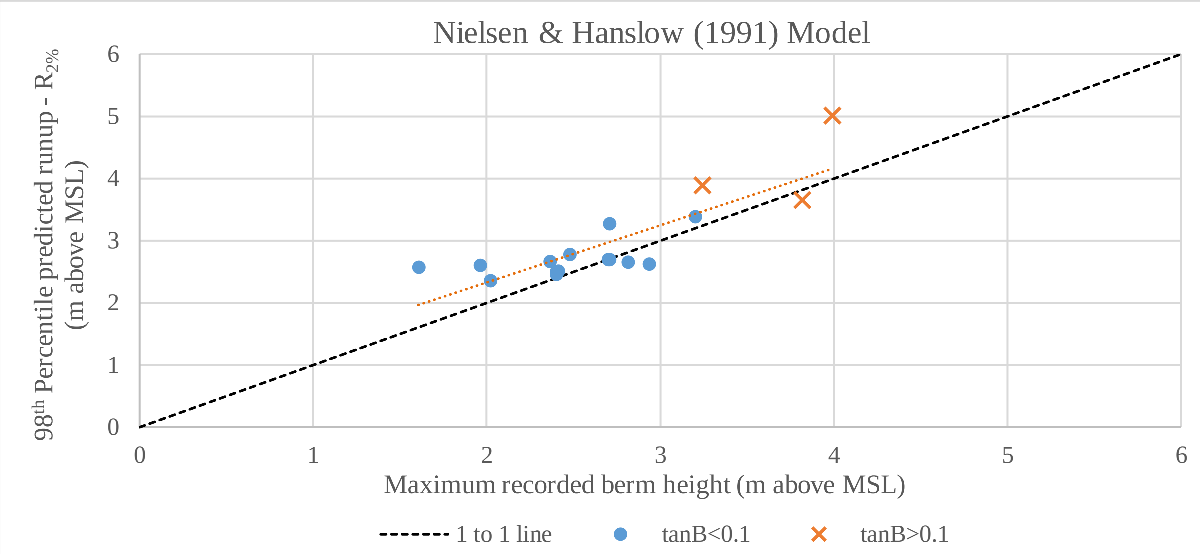

Figure 6-1: Relationship between the maximum recorded berm height and 98th percentile ranked predicted wave runup - Nielsen & Hanslow (1991) Model - for the respective estuaries

The 98th percentile ranked $R_{2\%}$ values predicted by the Nielsen and Hanslow Model compares reasonably well to the maximum berm heights. Low levels of scatter are evident between individual data points (estuaries), which indicates that the 98th percentile value is a reasonable estimate. The data points follow the 1 to 1 line slope reasonably well, suggesting consistent performance of the predictor among the respective estuaries. Overall the Nielsen and Hanslow Model displayed acceptable performance, with MAE and RMSEP values of 0.35 m and 0.46 m respectively.

6.2.3.2. Ruggiero *et al*. (2001) Models

The 98th percentile ranked $R_{2\%}$ values predicted by the Ruggiero *et al.* (2001) Models are compared to the maximum berm heights for each respective estuary. Figure 6-2 presents the results from the combined use of Model 1 and 2. Model 1 is used for dissipative beach types, while Model 2 is used for reflective beaches. The Ruggiero *et al.* Model 1 and 2 perform reasonably well in combination, with MAE and RMSEP values of 0.28 m and 0.34 m respectively. The results display an increasing trend of "underprediction" towards higher berm levels. This may suggest that the 98th percentile ranked $R_{2\%}$ does not correlate well for estuaries with higher berms. Alternatively, it suggests that the Ruggiero *et al.* Model tends to under predict for steeper slopes. The Ruggiero *et al.* Model displays relatively low levels of scatter, indicating consistent performance under a wide range of input parameters.

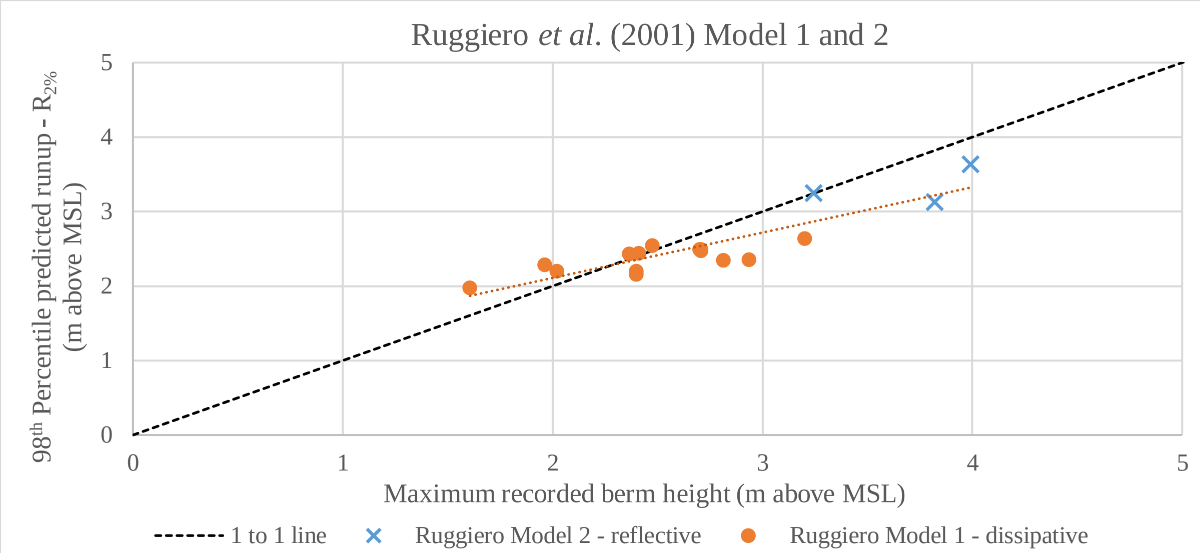

Figure 6-2: Relationship between the maximum recorded berm height and 98th percentile ranked predicted wave runup - Ruggiero et al. (2001) Model 1 and 2 - for the respective estuaries

Figure 6-3 presents the results of the exclusive use of the Ruggiero *et al.* Model 2. The sole use of Model 2 results in MAE and RMSEP values of 0.29 m and 0.36 m respectively. A trend of underpredicting towards the higher berms is evident, similar to the combined use of Models 1 and 2.

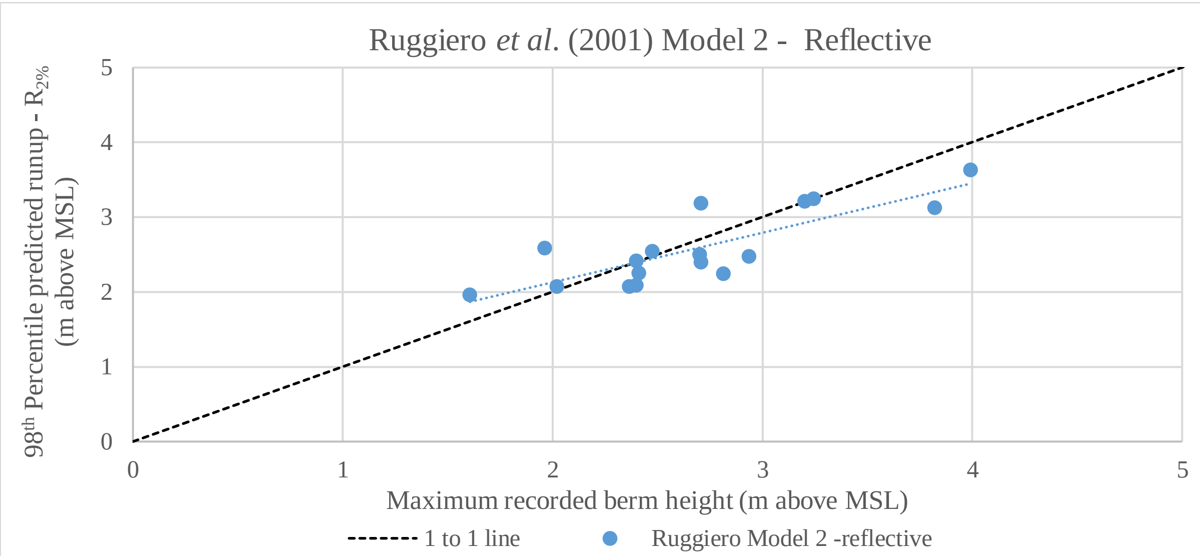

Figure 6-3: Relationship between the maximum recorded berm height and 98th percentile ranked predicted wave runup - Ruggiero et al. (2001) Model 2 - for the respective estuaries

6.2.3.3. Stockdon *et al*. (2006) Model

The 98th percentile ranked $R_{2\%}$ values predicted by the Stockdon *et al.* (2006) Model are compared to the maximum berm heights for each respective estuary (Figure 6-4). A low level of scatter is evident, and predictions closely follow the 1 to 1 prediction line. The model exhibits relatively good performance, with MAE and RMSEP values of 0.31 m and 0.36 m respectively.

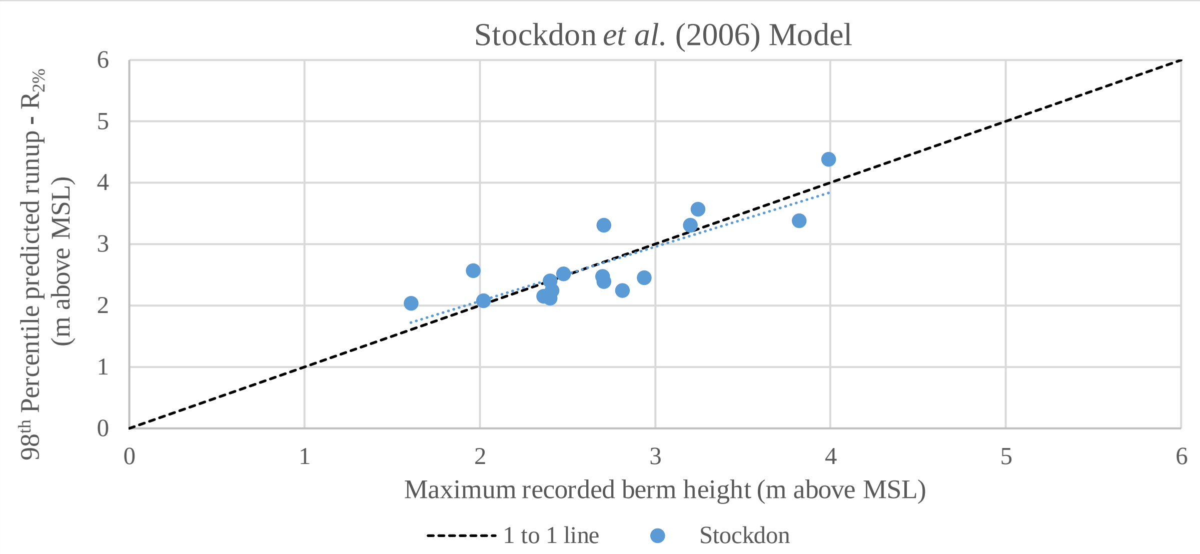

Figure 6-4: Relationship between the maximum recorded berm height and 98th percentile ranked predicted wave runup - Stockdon et al. (2006) Model - for the respective estuaries

6.2.3.4. Mather *et al*. (2011) Model

The 98th percentile ranked $R_{2\%}$ values predicted by the Mather *et al.* (2011) Model are compared to the maximum berm heights for each respective estuary (Figure 6-5). The Mather *et al.* Model under performs compared to the previous models, with MAE and RMSEP values of 0.70 m and 0.83 m respectively. The Mather C-coefficients were used as prescribed in Mather *et al*. (2011). A significant trend of underpredicting is evident, increasing towards higher berm levels.

The Mather *et al*. Model implements the horizontal distance from the shoreline to the 15 m isobath, instead of the beach face slope typically used by other models. As discussed in § 5.2, there is a very weak linear relationship between berm height and the distance to the 15 m isobath among the estuaries. Performance of the Mather *et al.* Model can be improved by calibrating the C-coefficient according to a specific site. This is not a suitable option, given the high number of locations under consideration. Preference is given to a universal model that performs reasonably well across a wide range of input parameters, i.e., a predictive runup model capable of estimating berm height at the majority of TOCEs in South Africa.

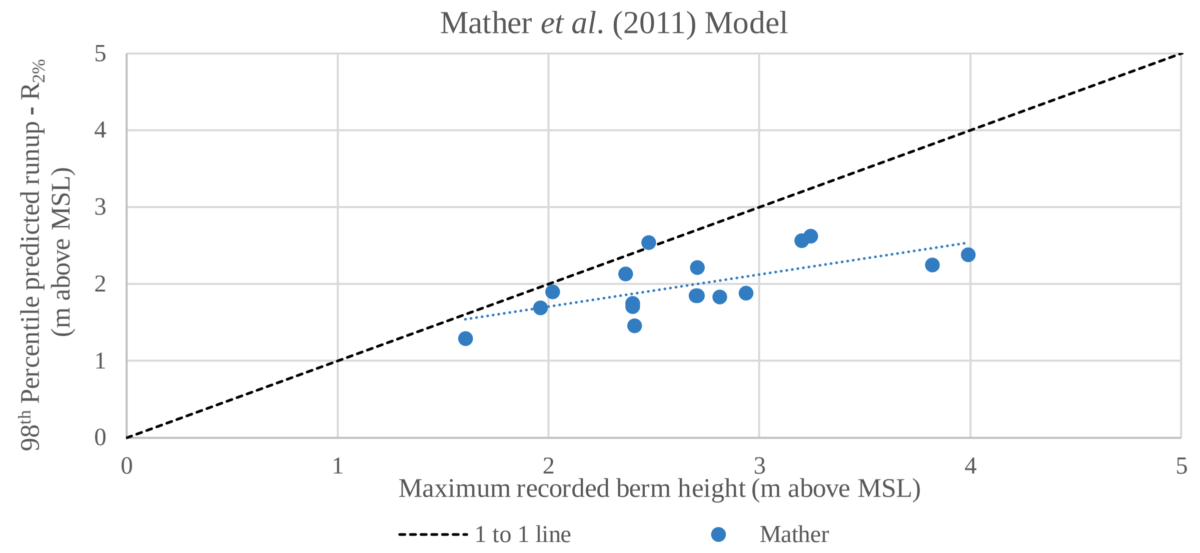

Figure 6-5: Relationship between the maximum recorded berm height and 98th percentile ranked predicted wave runup - Mather et al. (2011) Model - for the respective estuaries

6.2.3.5. Swart (1974) Model

The 98th percentile ranked $R_{2\%}$ values predicted by the Swart (1974) Model are compared to the maximum berm heights for each respective estuary (Figure 6-6). The model results display a relatively high level of scatter, and MAE and RMSEP values of 0.39 m and 0.51 m respectively. The results suggest a slight overprediction, especially at lower berm levels. The Swart Model implements the median sediment grain size, by relying on the relationship between sediment grain size and beach slope (Wiegel, 1964). As discussed in § 5.3, the relationship between the median grain size and berm height is weaker than the relationship between beach face slope and berm height.

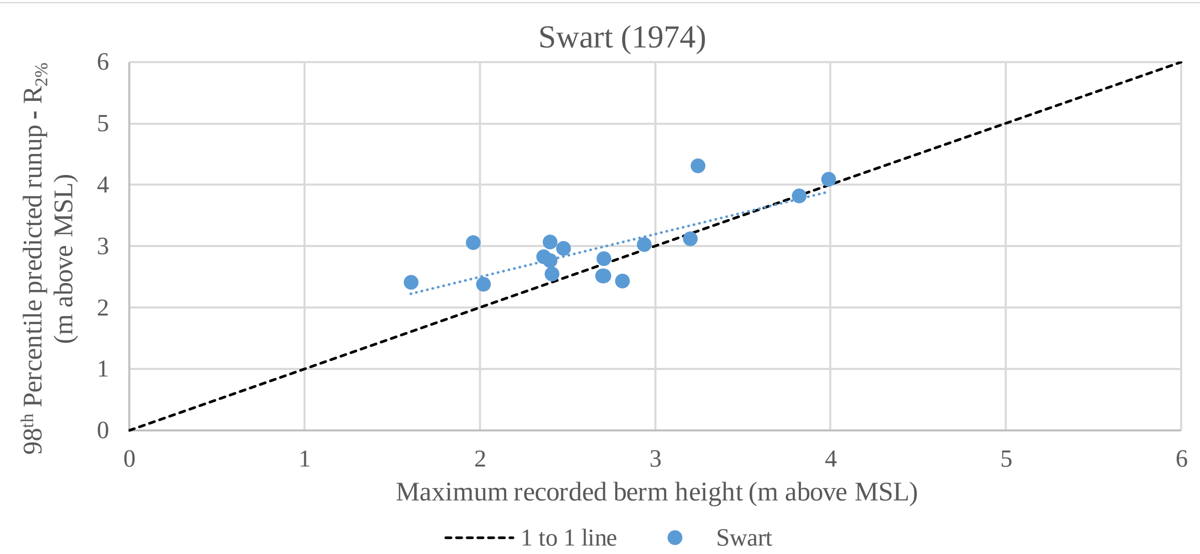

Figure 6-6: Relationship between the maximum recorded berm height and 98th percentile ranked predicted wave runup - Swart (1974) Model - for the respective estuaries

6.2.3.6. Summary of results

The performance of the runup parameterisations are summarised in Table 6-7. The top performing models are those of Ruggiero *et al*. (2001) and Stockdon *et al*. (2006). The combined Ruggiero *et al.* Model 1 and 2 displays the best performance, with R^2, MAE and RMSEP values of 0.78, 0.28 m and 0.34 m respectively. However, the Ruggiero *et al.* Model tends to under predict towards the steeper slopes.

The Stockdon *et al.* Model also performed well, with R^2, MAE and RMSEP values of 0.69, 0.31 m and 0.36 m respectively. The trend of predictions from the Stockdon *et al.* Model closely follows the 1 to 1 line, indicating consistent performance among all estuaries. The prediction trend is considered a crucial performance indicator, especially when using a single parametric model for a range of estuaries and input parameters. A slight under- or overprediction is considered a less significant performance indicator, as the comparison adopts the 98th percentile value as an estimate to predict berm height. A trend of under- or overprediction can be corrected by considering an alternative percentile value. Rather, the comparison aims to identify a suitable predicter, with consistent performance and minor prediction errors across the range of locations.

Table 6-7: Performance of the selected runup parameterisations in predicting long-term variation of estuarine berm height

Author/Model	$R_{2\%}$ (98th percentile value) to predict maximum berm height		
	R^2	MAE (m)	RMSEP (m)
Nielsen & Hanslow (1991)	0.69	0.35	0.46
Ruggiero *et al.* (2001) Model 1 & 2	0.78	0.28	0.34
Ruggiero *et al.* (2001) Model 2	0.67	0.29	0.36
Stockdon *et al.* (2006)	0.69	0.31	0.36
Mather *et al.* (2011)	0.45	0.70	0.83
Swart (1974)	0.55	0.39	0.51

The 98th percentile ranked $R_{2\%}$ values were also tested against the 98th percentile berm height values of the respective estuaries, in a similar fashion as the above-mentioned results. The method displayed marginally improved results compared to Table 6-7. The slight improvement in performance may be attributed to the higher occurrence probability of a 98th percentile berm height, compared to the maximum berm height. Simply put, the 98th percentile $R_{2\%}$ prediction is better correlated with a slightly lower berm height (98th percentile), than with the maximum berm height that only occurs once in an extended period. Notwithstanding, the relation between predicted runup and the maximum berm height is the primary focus, as this section aims to identify suitable methods to predict the highest probable berm height at an estuary.

6.2.4. Conclusion

The primary objective of this section was to determine the relationship between wave runup and the maximum berm crest elevation at an estuary. The analysis provided the following insight regarding the primary objective:

- There is a significant relationship between the level of predicted runup and the maximum recorded berm height at an estuary.

- The maximum predicted runup, derived from the long-term hourly simulated runup record, far exceeds the maximum recorded berm height attained in an overlapping record. This suggests that no significant sediment accretion occurs above the long-term maximum predicted $R_{2\%}$ elevation. Investigation of the occurrence probability of runup at the berm may provide further insight.

- The 98th percentile ranked value of $R_{2\%}$, derived from a record of several years of predicted runup, provides an accurate estimate of the maximum recorded berm height at the estuary. This suggests that the maximum berm height corresponds to a low exceedance probability of runup elevation at the estuary. The occurrence probability of runup events associated with the maximum berm height is explored in § 6.3.

The secondary objective of this section was to identify the most accurate runup parameterisation for the prediction of estuarine berm height. The analysis provided the following insight regarding the secondary objective:

- The most accurate predictors for estimating the berm crest elevations at estuaries are: the combined Ruggiero *et al.* (2001) Model 1 and 2, and the Stockdon *et al.* (2006) Model. The Stockdon *et al.* Model is considered the best suited parametric model, due to the superior trend of predictions, consistent performance and minor prediction errors across the range of estuaries.

- Runup parameterisations relying on beach face slope, as opposed to the sediment grain size (Swart Model) or distance to closure depth (Mather *et al*. Model), display more accurate results when predicting berm height.

It should be noted that the above-mentioned findings are based solely on the long-term predicted runup levels at the respective estuaries. The scope of this study does not involve the calibration/verification of the predicted runup, as measured wave runup data are not readily available for the selected estuaries. The runup parameterisations were used in accordance with the findings and recommendations of previous South African based runup evaluations (e.g. Roux, 2015; Theron, 2016), as to ensure best practice. This particular study primarily focusses on providing initial insight toward the relationship of wave runup and berm height at a wide range of South African estuaries.

6.3. Investigating the Probability of Runup Associated with the Maximum Berm Height

The relationship between the maximum berm height and the 98th percentile ranked $R_{2\%}$ at an estuary has been established in the previous section. A percentile rank provides the percentage of observations less than a given value, however it does not provide any knowledge regarding the probability of said value. Hence, the 98th percentile ranked $R_{2\%}$ does not provide any indication of the occurrence probability of the specific magnitude of runup associated with the maximum berm height.

In this section, an exceedance probability analysis is conducted for the simulated runup records of each estuary. The objective of the analyses is to elucidate the occurrence probability of runup events that govern/produce the maximum berm height at estuaries.

6.3.1. Procedure

Exceedance probability distributions are generated from the simulated runup ($R_{2\%}$) records at each estuary. Two exceedance probability curves are generated for each estuary: one for the hourly simulated runup predicted by the Ruggiero *et al.* (2001) Model 1 and 2, and one for the record predicted by the Stockdon *et al.* (2006) Model. The exceedance probability curves of the simulated runup records will explicate the occurrence frequency of a given magnitude of runup at an estuary.

A Four Parameter Logistic (4PL) regression model is fitted to the respective exceedance probability curves (Rodbard & Frazier, 1975). A description of the 4PL regression model is provided in Appendix F. The regression model enables convenient sampling from the relevant probability distributions.

The exceedance probability curve of the simulated runup ($R_{2\%}$) record (Stockdon *et al.* Model) at Kleinmond Estuary is presented in Figure 6-7. The simulated record comprises approximately 7 years of hourly predicted runup levels, superimposed on the hourly recorded tide. Note that the probability distribution is calculated according to the predicted runup, which is a simulated record of hourly 2% exceedance probability runup elevations. All the selected runup parameterisations provide the $R_{2\%}$ as output.

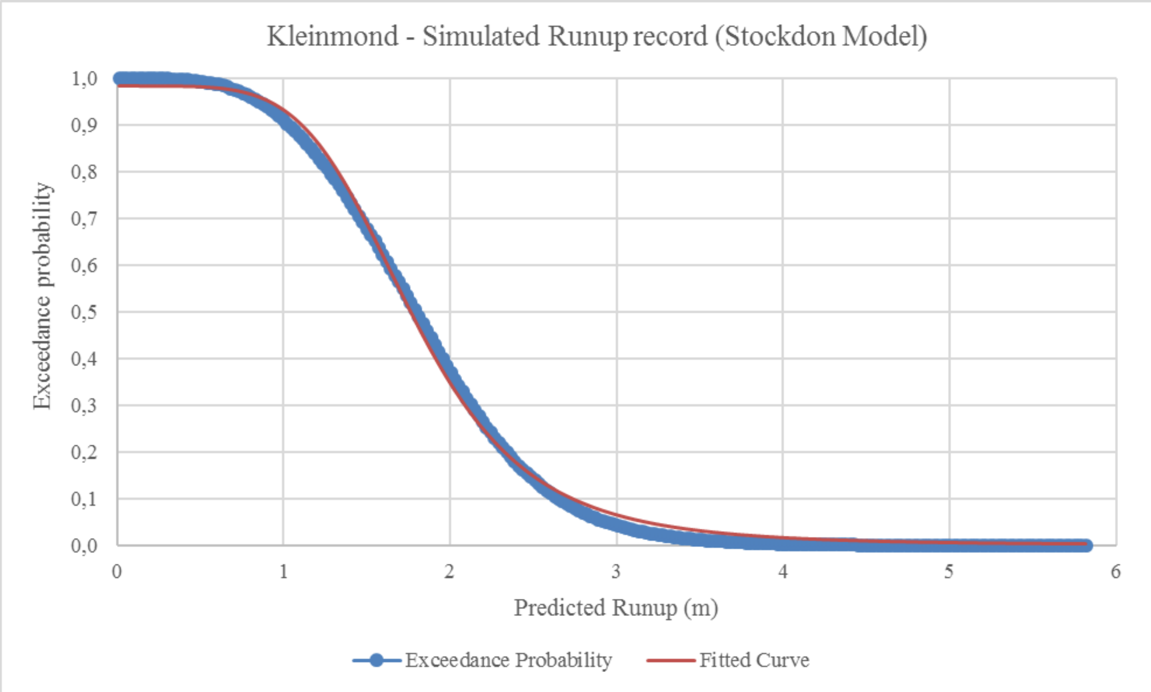

Figure 6-7: Exceedance probability distribution of the simulated wave runup record - Stockdon et al. (2006) Model - at the Kleinmond Estuary

The exceedance probability of the wave runup event equalling the maximum berm height is obtained from the fitted curve in Figure 6-7. The maximum recorded berm crest elevation at the Kleinmond Estuary is 2.7 m. Thus, the exceedance probability of a wave runup event equal to 2.7 m is 0.11 (11%). This indicates that a runup level of 2.7 m has a 11% probability to be equalled or exceeded within the 7-year record.

A similar process as outlined above was followed for the rest of the estuaries (for both the Stockdon *et al.* and Ruggiero *et al.* Model predictions). The results of the analyses are presented in the following section.

6.3.2. Results

The exceedance probability of the predicted runup level equal to the maximum berm height is presented for the respective estuaries in Figure 6-8. The results are based on several years of simulated runup levels at each estuary. The exceedance probabilities vary between 0.01 and 0.15 (1% - 15%). This provides an estimate of the occurrence frequency of the runup elevation that causes/governs the maximum berm height. The average result attained by the Ruggiero *et al*. Model is 3.7% exceedance probability, and the average result attained by the Stockdon *et al*. Model is 5.2%.

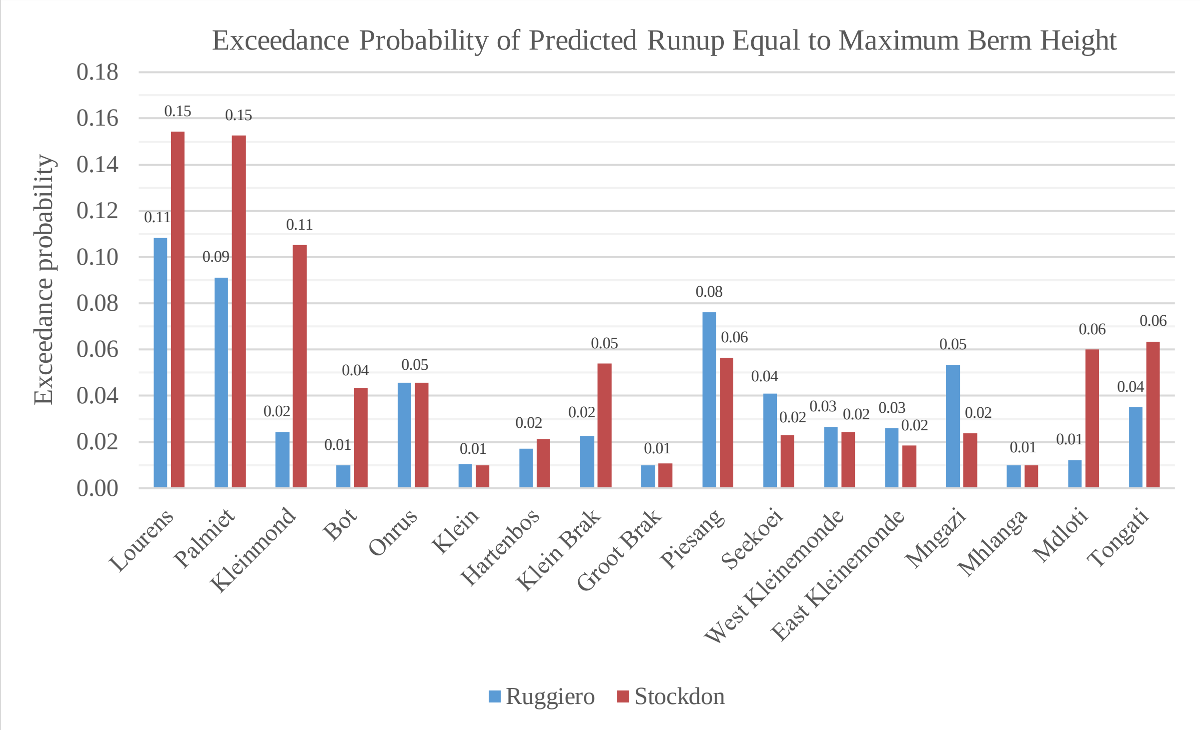

Figure 6-8: Exceedance probabilities of predicted wave runup ($R_{2\%}$) elevation associated with maximum berm height at the respective estuaries

As expected, there are considerable variations among estuaries. Similar to a percentile rank, the exceedance probability is an evaluation of a specific value relative to the rest of the population. Therefore, the variation among estuaries may be attributed to the specific population of the simulated runup record at the estuary. The simulated records exhibit dissimilarities in frequency distribution, specifically with regards to the symmetry of the distributions. These dissimilarities are caused by variation in record length, the sensitivity of the runup parameterisation with regards to the input variables, and gaps in the record (possibly excluding specific storm events). Nonetheless, the results conclude that the maximum berm height is governed by a runup level associated with a generally low (< 15%) exceedance probability.

The general low levels of runup exceedance probability displayed in the foregoing results provide the following insight with regards to the level of runup that contributes to maximum berm height.

Firstly, runup events exceeding the maximum berm height occur only on rare occasions (long return periods), resulting in the inability to build the berm to higher levels. Frequent deposition of sediment beyond the existing crest is required for vertical berm accretion. This is perhaps a more probable rationale for the estuaries in Figure 6-8 displaying relatively lower levels of runup exceedance probability (e.g. < 2%).

Secondly, runup exceeding the maximum berm height is typically associated with energetic wave conditions and sea storms. The results suggest that large runup/overwash events (e.g. exceedance probability < 15%) do not lead to significant vertical accretion of the existing crest. Therefore, these high return period overwash events are more probable to cause berm erosion or landward migration of sediment.

It is essential to distinguish between storm overwash and non-storm overwash. Non-storm overwash is typically exhibited during the initial phase of berm growth/recovery following a breach, when large waves (high runup) are not required to overtop the lowered berm. The initial stages are often associated with rapid vertical growth (Weir *et al.*, 2006). Storm overwash is generated by energetic wave conditions and storm surge, typically at higher berm elevations. Energetic wave conditions associated with sea storms often lead to the erosion of beaches. However, crest accumulation is probable during storm overwash, and is dependent on the berm crest width and the runup level compared to the storm surge level (relative surge height parameter) (Donnelly, 2007). Donnelly *et al.* (2004) state that the probability of berm accretion during storm overwash increases with lower levels of storm surge and higher berm crest width. An increased horizontal berm crest width effectively reduces the velocity of the overwash, leading to increased sediment deposition on the existing crest. The foregoing results (Figure 6-8) suggest that berm accretion is improbable at higher runup levels (exceedance probability < 15%). The findings of Donnelly *et al.* (2004) suggests that the variability displayed in these results may also be attributed to the variation of horizontal crest width at the estuaries.

6.3.3. Conclusion

The results suggest that the maximum berm crest elevation at an estuary can be estimated with a low exceedance probability of wave runup at the berm. The approach relies on several years of data to compare the maximum berm height to runup exceedance values. A theoretical threshold of runup exceedance probability and associated vertical berm growth is presented in Figure 6-9.

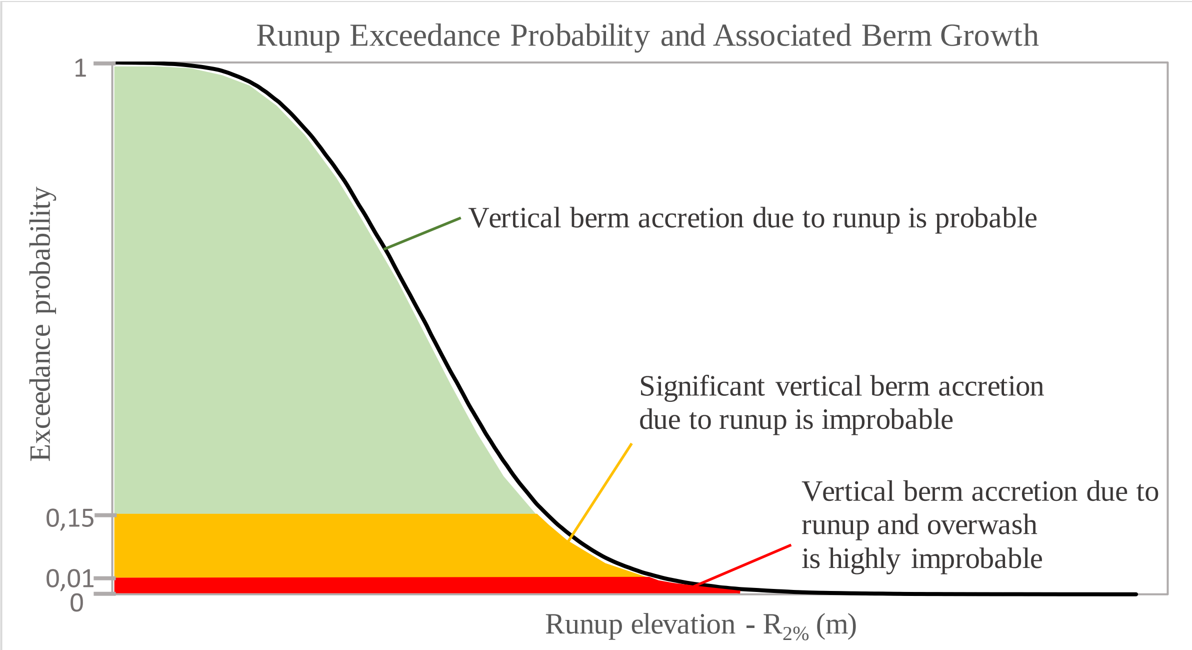

Figure 6-9: Theoretical threshold of berm response associated with exceedance probability of runup event

Berm accretion below the maximum recorded berm height is caused by relatively higher exceedance values of wave runup (green zone). Wave runup with an exceedance probability higher than 15% account for the majority of berm accretion at estuaries.

The estuarine berms rarely grow to elevations higher than the runup level with an exceedance probability between 1% and 15% (orange zone). Therefore, from a long-term perspective, the runup levels associated with exceedance probabilities in the orange zone, rarely account for significant vertical accretion of the berm.

The maximum berm height is governed by the runup level associated with an exceedance probability less than 1% (red zone). Therefore, no significant, regular, vertical berm accretion is associated with runup levels with an exceedance probability in the red zone.

Figure 6-9 attempts to illustrate a methodology to predict the maximum berm crest elevation at an estuary. The threshold values are predicted according to simulated wave runup records, therefore the accuracy is not verified. The process intends only to illustrate the presence of such a threshold, and the predictive capability it has for the maximum berm height.

The method of implementing a specific exceedance probability of wave runup as an estimate of the potential maximum berm crest elevation is discussed in the following section (§ 6.4).

6.4. Predicting Long-Term Variations in Berm Height

This section implements the results and findings of § 6.2 and § 6.3, in order to develop two methods to predict the long-term variation in estuarine berm height at South African TOCEs, with a specific emphasis on the potential maximum berm crest elevation. Method 1 involves correlating the maximum berm crest elevation to a low exceedance probability of total water level on the beach (runup superimposed on tide), as explored in § 6.3. This method requires a long-term record of simulated runup at an estuary. Method 2 is proposed as a simplified, less time-consuming alternative to Method 1. The methodology implements a specific design parameter combination of wave height, wave period and tidal level. The design parameters are used as input for the parametric runup model, in order to attain a singular design runup elevation to predict the vertical extent of berm accretion.

6.4.1. Method 1 – Exceedance probability wave runup

This methodology follows the data acquisition/requirements as set out in § 6.1, in order to simulate several years of hourly runup (as set out in § 6.2) at an estuary. Accordingly, a probability distribution can be generated from the record of simulated runup at an estuary, similar to the process set out in § 6.3. The results of the runup probability analysis in § 6.3 revealed that the maximum berm crest elevation at South African TOCEs can be estimated by a generally low exceedance probability of wave runup (< 15%). The average exceedance probability of wave runup associated with the maximum berm crest elevation, simulated by Stockdon *et al*. (2006) Model, was 5.2% for the respective estuaries (§ 6.3.2). Accordingly, the 5% exceedance probability runup magnitude (rounded to nearest integer for ease of use) is used to estimate the maximum berm crest elevation at the respective estuaries.

The process steps are briefly summarised below:

- Acquire wave and tidal data from the nearest recording devices and convert to hourly intervals (see § 6.1.1.1). Ensure sufficient record length, typically several years, to ensure adequate distribution of wave height and - period.

- Acquire the additional input parameters for the runup parameterisation, such as beach face slope at the estuary berm. The beach face slope is selected as constant throughout the simulation period, primarily due to the lack of information regarding the variability during the berm building period (§ 6.2.2). Care should be taken to select a beach face slope that is representative of a medium to high berm level at an estuary.

- The nearshore wave transformation process can be accounted for by implementing the proposed wave transformation coefficient (K_T in Table 6-3). The nearshore transformed waves should be reverse shoaled to deep water equivalent waves by means of linear wave theory, while assuming a shore normal approach (§ 6.1.1.3).

- The runup elevation is simulated from the recorded wave and tidal data, resulting in several years of hourly predicted total water level on the estuarine berm ($R_{2\%}$ superimposed on tide), as discussed in § 6.2.2. The Stockdon *et al*. (2006) Model is proposed due to its consistent performance and minor errors across the range of estuaries (see Table 6-7).
- The exceedance probability distribution of the simulated runup record is generated. The probability distribution accounts for the combined effect of wave height, wave period and tidal elevation on the elevation of runup (see § 6.3.1).
- The 5% exceedance probability of the simulated runup at the estuary berm is used as an estimate for the potential maximum berm crest elevation.

The 5% exceedance probability of wave runup at the respective estuaries are compared to the corresponding maximum berm crest elevations in Figure 6-10.

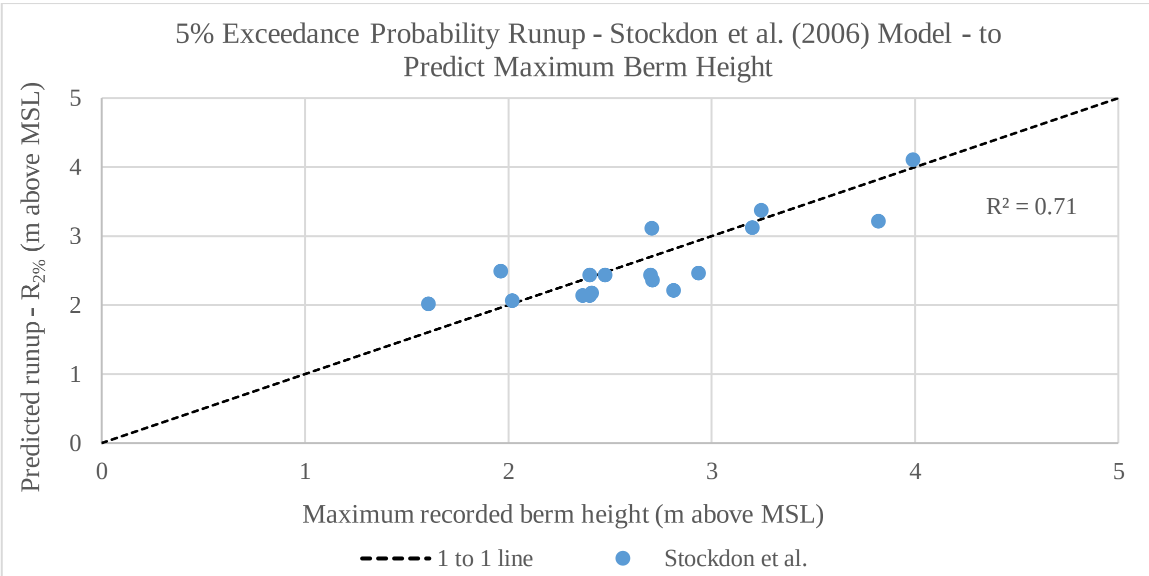

Figure 6-10: Relationship between the maximum recorded berm height and the 5% exceedance probability of wave runup - Stockdon et al. (2006) Model - for the respective estuaries

The proposed method of prediction provides the following accuracy:

- R^2 = 0.71
- MAE = 0.28 m
- RMSEP = 0.34 m

Following the above mentioned method, the potential maximum berm crest elevation can be predicted with relative accuracy.

6.4.2. Method 2 – Specific design parameter combination

Method 2 implements a specific design combination of wave height, wave period and tidal level as input for an existing runup parameterisation. The method delivers a single design runup ($R_{2\%}$) elevation that estimates the maximum extent of sediment deposition at South African TOCE berms, i.e., a runup elevation to predict the maximum berm crest elevation. Method 2 is proposed as a less time consuming methodology to accurately predict maximum berm height. Probability distributions of wave height and - period for the selected wave buoys along the South African coastline are typically available, or easily obtained in practice.

The proposed method implements the 2% exceedance probability significant wave height along with the 50% exceedance probability peak wave period, obtained from the relevant wave records (§ 6.1.1). The design wave height and -period are used to calculate the runup elevation, which is then superimposed on the Mean High Water Spring (MHWS) tidal level. The specific design combination is motivated by the results of the wave runup probability analyses discussed in § 6.3.

The process steps are briefly summarised below:

- Derive the 2% exceedance probability significant wave height from the nearest relevant wave record (e.g. Waverider buoy record).
- Derive the 50% exceedance probability peak wave period from the corresponding wave record.
- Implement the wave transformation coefficient (K_T in Table 6-3) to obtain a nearshore equivalent wave height (d = 15 m). The nearshore transformed waves are reverse shoaled to deep water equivalent waves by means of linear wave theory, while assuming a shore normal approach (§ 6.1.1.3).
- Calculate the predicted runup ($R_{2\%}$) by means of the Stockdon *et al*. (2006) Model.
- The predicted runup ($R_{2\%}$) is then superimposed on the MHWS tidal elevation at the relevant estuary inlet to obtain the runup elevation.

The proposed design parameters for estimating the maximum berm crest elevation at the selected estuaries are presented in Table 6-8. The predicted runup elevations are calculated according to the proposed Method 2, implementing the design parameters provided in Table 6-8. The wave runup scenarios are compared to the maximum recorded berm crest elevations of the respective estuaries in Figure 6-11.

The slightly lower peak wave period (T_p) of the Piesang-, Seekoei-, Mhlanga-, Mdloti- and Tongati Estuary may be attributed to the relatively short available record length of the FA Platform and Durban Bluff wave recording locations (see Table 6-1 and Table 6-6).

The relatively short peak wave period (T_p) of the Durban Bluff wave recordings (Mhlanga, Mdloti and Tongati) may also be attributed to the unique local wave regime. The KwaZulu-Natal wave regime includes a larger effect from the locally generated wind waves (shorter period), especially compared to the Western- and Southern Cape which are dominated by longer period swell waves from the Southern Ocean.

The 2% exceedance probability deep water wave height for the Lourens Estuary appears to be slightly larger than similarly exposed locations such as the Groot Brak - and Seekoei Estuary. The Lourens Estuary is the only location that implements the wave data from the SWAN virtual wave buoy model, managed by the CSIR.

Table 6-8: Selected combination of design parameters for the proposed Method 2 and the resulting predicted runup

	Design Parameters for Proposed Wave Runup Scenario			
Estuary	**H'_0 - 2% Exceedance probability (m)**	**T_p - 50% Exceedance probability (s)**	**MHWS (m above MSL)**	**$R_{2\%}$ Elevation - Stockdon *et al.* (2006) (m above MSL)**
Lourens	3.26	11.12	0.947	2.10
Palmiet	2.96	11.68	0.992	2.63
Kleinmond	3.29	11.76	0.992	3.23
Bot	3.74	11.68	0.992	3.28
Onrus	3.55	11.68	0.992	2.58
Klein	3.11	11.68	0.992	2.34
Hartenbos	2.31	11.69	1.167	2.25
Klein Brak	2.45	11.69	1.167	2.58
Groot Brak	2.92	11.69	1.167	2.63
Piesang	2.16	9.59	1.122	2.02
Seekoei	2.90	9.59	1.024	2.15
West Kleinemonde	3.35	11.33	1.104	2.61
East Kleinemonde	3.35	11.33	1.104	2.52
Mngazi	3.10	11.33	1.104	2.25
Mhlanga	2.96	9.66	1.097	3.16
Mdloti	3.02	9.66	1.097	3.96
Tongati	3.08	9.66	1.097	3.31

Note: Wave heights are calculated according to the methodology set out in this section.

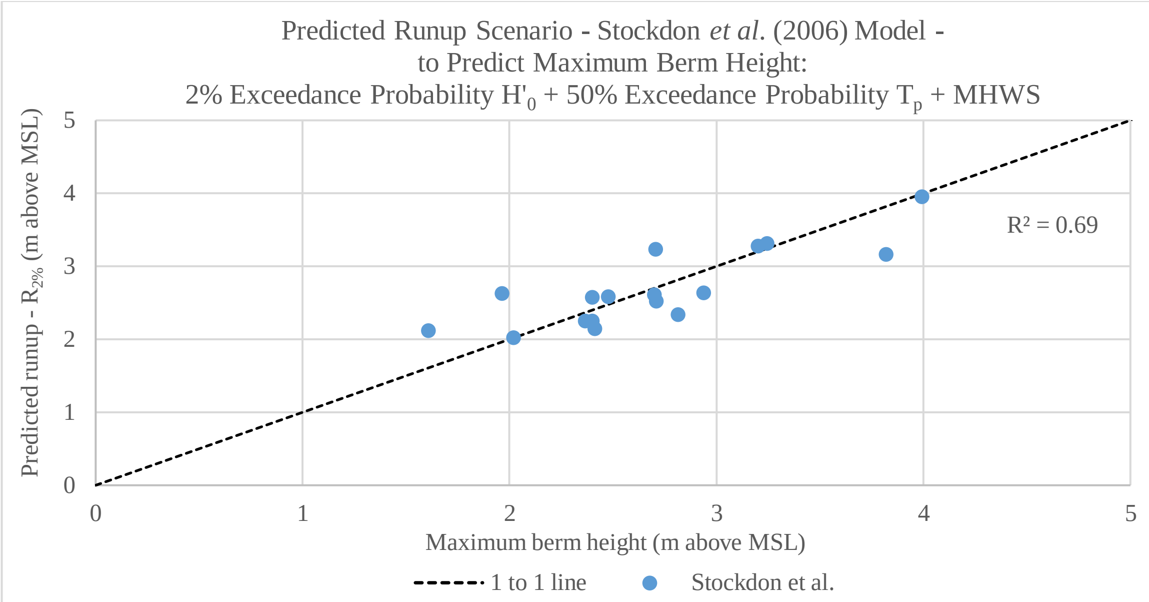

Figure 6-11: Relationship between maximum recorded berm height and proposed wave runup scenario - Stockdon et al. (2006) Model - for respective estuaries

The proposed method of prediction provides the following accuracy:

- R^2 = 0.69
- MAE = 0.26 m
- RMSEP = 0.34 m

The above mentioned performance indicators, as well as the grouping of data points around the 1 to 1 line suggest that the proposed design combination provides a relatively accurate prediction of the maximum berm crest elevation. The trend of predictions slightly deviates from the 1 to 1 line. This deviation is more pronounced than the slight deviation displayed in Figure 6-10.

6.4.3. Conclusion

Method 1 and 2 provide similarly accurate predictions of the maximum potential berm height at South African TOCEs.

Method 1 is considered to be the most robust method of the two. Method 1 considers the combined effect of the hourly tidal level, - wave height and - wave period on the total water level on the berm at the specific time step. Therefore, the simulated record provides a relatively accurate estimate of the wave runup at the estuary berm during closed mouth state. Method 1 provides an estimate of the maximum berm crest elevation at South African TOCEs with a respective MAE and RMSEP of 0.28 m and 0.34 m.

Method 2 provides a simplified, yet similarly accurate approach for estimating the runup elevation above which significant berm accretion is improbable. The proposed design combination of 2% exceedance probability significant wave height, 50% exceedance probability peak wave period and MHWS tidal level is intended for South African TOCEs. The design combination could potentially be less accurate in locations/countries exhibiting a different range of significant wave height and peak wave period. Method 2 provides an estimate of the maximum berm crest elevation at South African TOCEs with a respective MAE and RMSEP of 0.26 m and 0.34 m.

6.5. Short-Term Predictive Methods

Short-term berm crest elevation refers to the maximum achievable berm crest elevation on a time scale of individual breaches.

Predicting berm height on a shorter time scale becomes increasingly complex, as the berm height at the time of a breach is not only dependant on the coastal drivers. The time between estuary closure and mouth breaching governs the potential berm elevation, and is typically dependant on river inflow and the estuary water balance. Additionally, improved predictions regarding the direction and rate of sediment transport are required to estimate the rate of berm growth, given the prevalent coastal drivers.

This section primarily aims to explore plausible predictive methods, considering the limited available coastal parameter data. The focus lies on identifying suitable methods to describe the coastal drivers responsible for short-term berm growth.

6.5.1. Procedure

Four possibilities are considered for estimating berm height on a time scale of individual breaches. Firstly, the selected parametric berm height models are evaluated according to their ability to predict berm height following a period of berm building wave conditions. The Models considered include: the Swart (1974) Model, the Larson and Kraus (1989) Model, and the Okazaki and Sunamura (1995) Model. Secondly, the best performing runup parameterisations (§ 6.2.3.6) are evaluated to determine the typical relationship between runup and berm height on a shorter time scale. Thirdly, the Dean number (Dean, 1973) is evaluated according to its capability to predict the direction, and possibly magnitude, of sediment transport and consequently the berm height. Lastly, a proposed berm growth model is assessed.

The methods were tested for the period starting after mouth closure, up until the point of estuary breaching. Sixteen breaching events were identified from 6 different estuaries. The estuaries were selected based on their typical Iribarren number, to ensure conditions ranging from dissipative to reflective. The selected short-term scenarios are presented in Table 6-9.

Certain scenarios exhibit rapid vertical berm accretion over a relatively short duration. The accumulated height is not always correlated to berm building duration at an estuary, as the berm accretion is dependent on the wave conditions and direction of sediment transport (erosional or accretional).

The recorded waves and – tides were analysed and implemented in a similar method as set out in § 6.1, and used as input for the short-term predictive methods.

Table 6-9: Proposed closure to breach scenarios for the evaluation of short-term berm growth/height

	Estuary	Berm height at closure (m above MSL)	Berm height at breach (m above MSL)	Berm growth (m)	Berm building duration (days)
1	**Onrus**	1.17	1.81	0.65	10
2		1.21	1.90	0.69	9
3		1.25	2.20	0.95	7
4	**Hartenbos**	1.04	1.96	0.92	35
5		1.23	2.24	1.01	65
6		0.93	2.03	1.1	23
7		1.08	1.54	0.46	14
8		1.18	1.95	0.77	32
9	**East Kleinemonde**	0.98	1.5	0.53	7
10		0.67	1.34	0.67	19
11	**Mngazi**	0.6	1.16	0.56	22
12		0.84	2.19	1.34	31
13	**Mhlanga**	1.46	2.39	0.93	8
14		1.09	2.88	1.79	8
15		1.36	2.56	1.20	9
16	**Mdloti**	0.97	3.64	2.67	9

6.5.2. Results

The results and findings of the short-term predictive methods are discussed in this section.

6.5.2.1. Swart (1974) Model

The Swart (1974) Model was developed to predict the upper limit of the "D-profile" of a beach, which relates to the berm crest elevation, as both are dependent on the landward limit of wave runup. The model equation and background is provided in § 2.6.4.2. The Swart Model is considered for both the long-term and short-term predictions, due to the theoretical basis of the model being firmly based on estimating the runup elevation in order to predict the profile height. The Model was tested for the closure to breach scenarios listed in Table 6-9. Several combinations of wave height and period (present during the respective scenarios) were used as input for the Model, resulting in varying degrees of performance. The 75th percentile significant wave height and the 75th percentile peak wave period, calculated from the hourly wave data recorded during the respective scenarios, provided the most accurate prediction of berm height at breaching. Note that the percentile values are calculated from the deep water equivalent unrefracted wave height and the peak wave period (derived in a similar manner as set out in § 6.1). The relationship between the predicted - and recorded berm height at the time of breaching, for the respective scenarios, are presented in Figure 6-12.

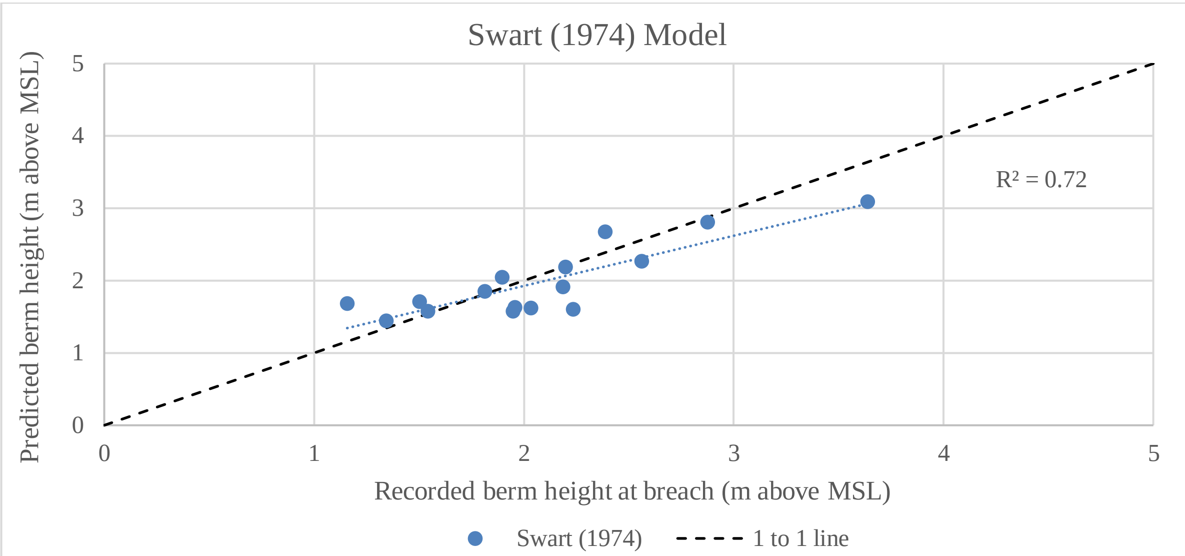

Figure 6-12: Relationship between the predicted berm height - Swart (1974) Model - and the recorded berm height at breaching for respective scenarios

The Model predicts the berm height above MSL, at the time of breaching, for the respective scenarios with the following accuracy:

- R^2 = 0.72
- MAE = 0.33 m
- RMSEP = 0.27 m

The above mentioned performance indicators and the grouping of data points around the 1 to 1 line, suggests that the model provides a satisfactory prediction of berm height at the time of breach, especially considering the complex interactions present during this period. However, the trend of prediction indicates a slight underprediction towards scenarios displaying higher berm heights at breaching.

6.5.2.2. Larson and Kraus (1989) Model

The Larson and Kraus (1989) Model was developed to predict the maximum subaerial elevation (above the SWL) of an equilibrium berm profile at a beach. The model equation and background is provided in § 2.6.4.2. The Model was tested for the closure to breach scenarios listed in Table 6-9. Several combinations of wave height and period (present during the respective scenarios) were used as input for the Model, similar to the evaluation of the Swart (1974) Model (§ 6.5.2.1). The average significant wave height and the average peak wave period, calculated from the hourly wave data recorded during the respective scenarios, provided the most accurate prediction of berm height at breaching. The relationship between the predicted - and recorded berm height at the time of breaching, for the respective scenarios, are presented in Figure 6-13.

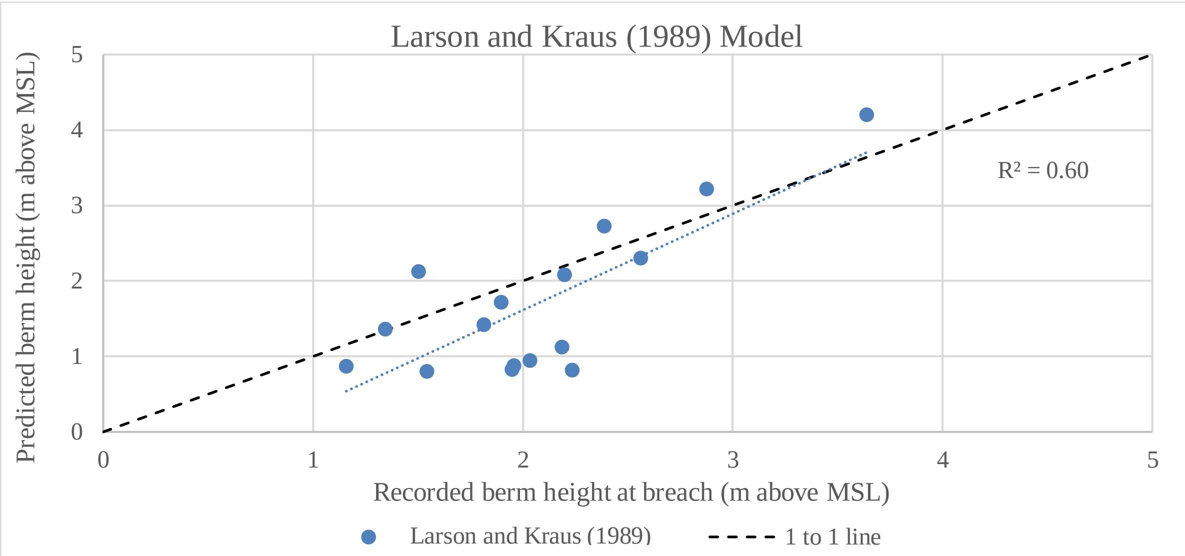

Figure 6-13: Relationship between the predicted berm height - Larson and Kraus (1989) Model - and the recorded berm height at breaching for respective scenarios

The Model predicts the berm height above MSL, at the time of breaching, for the respective scenarios with the following accuracy:

- R^2 = 0.60
- MAE = 0.73 m
- RMSEP = 0.60 m

The Model underperforms, especially compared to the Swart (1974) Model (Figure 6-12). The undesired spread and trend of data points indicates the Model's inability to accurately predict berm height at breaching for the selected scenarios. The Model may be better suited for the prediction of bermed beach profiles in an equilibrium state.

6.5.2.3. Okazaki and Sunamura (1995) Model

The Okazaki and Sunamura (1995) Model was developed to predict the berm height at equilibrium state, following a period of berm recovery (refer to Table 6-5 for model equation). Estuarine berm height is typically governed by the frequency of breaching. Nonetheless, the model was tested for several scenarios from a range of estuaries to assess its ability to describe the variation of typical berm height among different estuaries. The relationship between the predicted - and the recorded berm heights are presented in Figure 6-14.

The model uses an average wave height and – period as input (§ 6.1.1.4), to predict a single berm height representative of an equilibrium profile. The additional, constant, input parameters for the Okazaki and Sunamura Model were selected as follows:

- g = 9.81 m/s^2 (gravitational acceleration)
- ρ = 1025 kg/m^3 (density of sea water)
- ρ_s = 2650 kg/m^3 (density of sediment)
- υ = 1.17 x 10^{-6} m^2/s (kinematic viscosity of sea water)

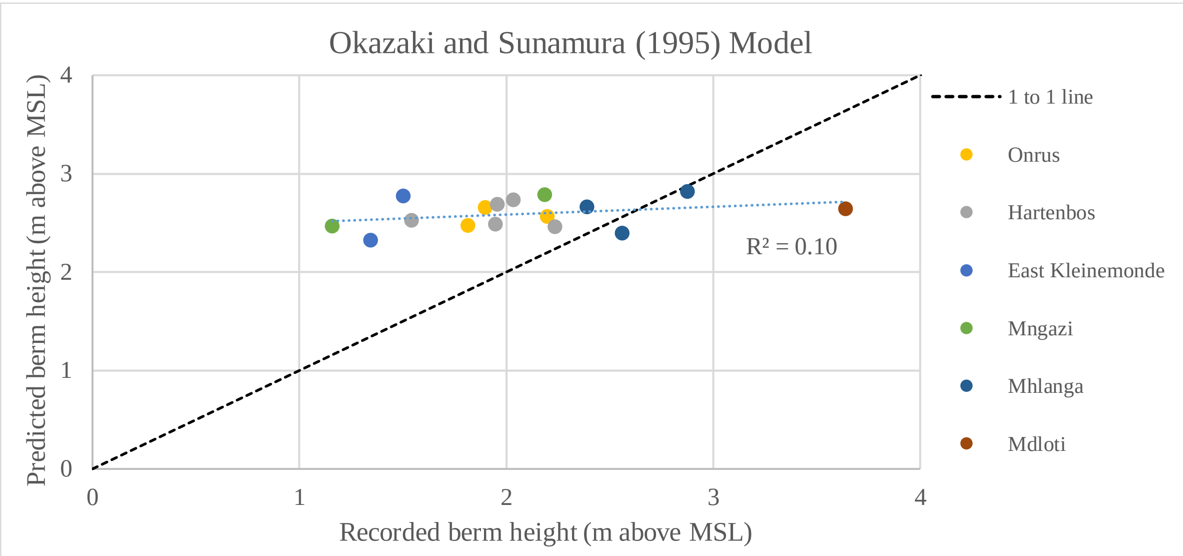

Figure 6-14: Relationship between the predicted berm height - Okazaki and Sunamura (1995) Model - and the recorded berm height at breaching for respective scenarios

The Okazaki and Sunamura Model predicts relatively uniform berm heights (between 2.3 m and 2.8 m) for all scenarios. The results display a trend of overprediction towards the lower recorded berm heights. This may indicate that the berm was breached, prior to the point that an equilibrium state was achieved. However, the Model poorly describes the variation in berm height among the respective scenarios and estuaries. This indicates that given an extended period of berm growth for all scenarios, a low variation in berm height is expected among locations. A sensitivity assessment of the model is conducted to elucidate the weak relationship displayed in Figure 6-14.

Sensitivity Assessment

The sensitivity of the Okazaki and Sunamura (1995) Model was tested with regards to variations of the input parameters. A single parameter is incrementally adjusted while the remaining parameters remain constant. The parameter constants were selected as: H_b = 2 m, T_{mean} = 10 s, and D_{50} = 0.5 mm. These are typical values which can also be found along the South African coastline. The sensitivity of the Okazaki and Sunamura Model with regards to variation in the input parameters are displayed in Figure 6-15 through to Figure 6-17.

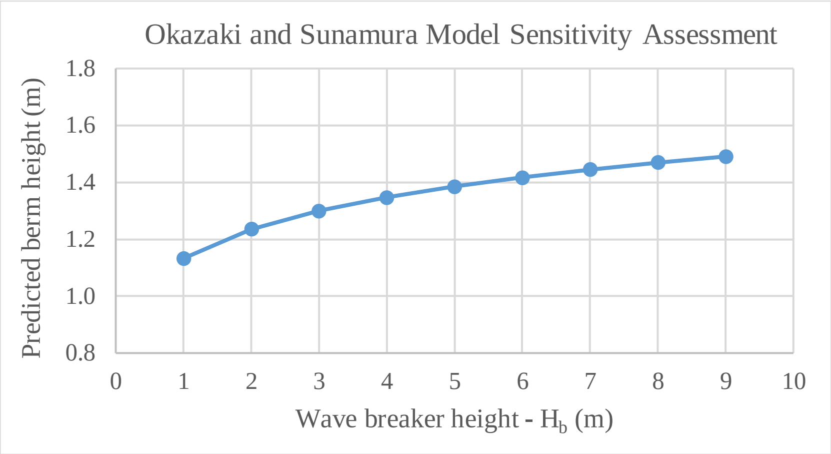

Figure 6-15: Assessment of Okazaki and Sunamura (1995) Model response to variations in wave breaker height

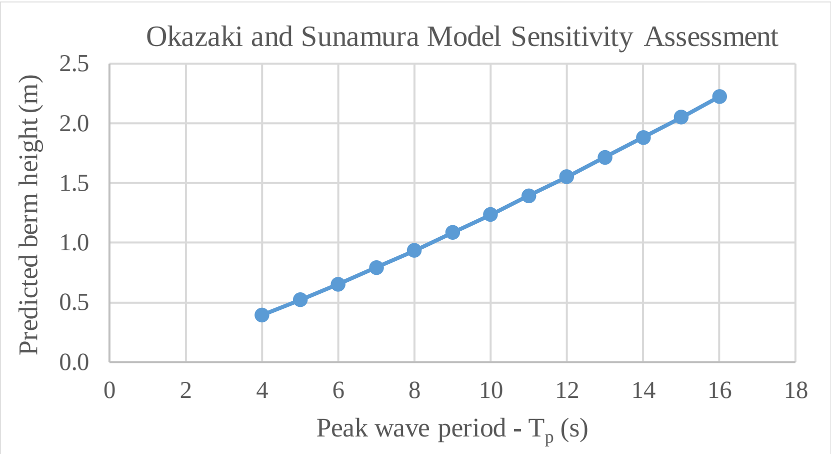

Figure 6-16: Assessment of Okazaki and Sunamura (1995) Model response to variations in wave period

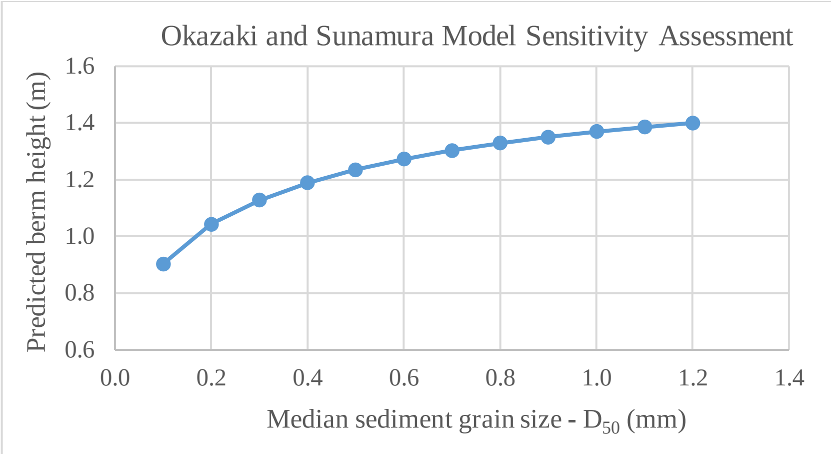

Figure 6-17: Assessment of Okazaki and Sunamura (1995) Model response to variations in median sediment grain size

The model is relatively insensitive to variations in wave height, with sensitivity diminishing towards higher waves.

A near linear response to variation in wave period is exhibited. The response indicates higher berm predictions are expected with increased wave periods. This is related to the response of the Dean number (Dean, 1973) to an increase in wave period, i.e., longer period waves are typically associated with onshore sediment transport and accretion. This high sensitivity to wave period may be useful to compensate for the effect of erosional wave events in the berm recovery period.

The model is relatively insensitive to variations in sediment grain size, with sensitivity diminishing towards coarser sediment.

The lack of performance of the Okazaki and Sunamura Model is attributed to the low sensitivity to variations of wave height and sediment grain size. These parameters contribute to the variation in berm height among estuaries. The wave period may prove a useful coastal parameter to describe the direction of sediment transport (onshore or offshore), however it does not adequately describe the variation of berm height among estuaries

6.5.2.4. Runup models

The Ruggiero *et al.* (2001) Model 1 and 2, as well as the Stockdon *et al.* (2006) Model, were evaluated for the scenarios listed in Table 6-9. The goal was to determine the relationship between wave runup and berm height on a time scale of individual breaches. The runup was predicted in hourly intervals and then superimposed on the hourly recorded sea level, similar to the process outlined in § 6.2.2. The 98th percentile ranked predicted runup was selected to exclude large outlier events in the record. The results for the selected models are presented in Figure 6-18.

The 98th percentile ranked runup elevation, derived from a short-term record, is an over estimate of the berm height at the time of breach (Figure 6-18). The 98th percentile ranked runup elevation, derived from a long-term record, provided a relatively accurate prediction of the maximum berm height at the selected estuaries (see Figure 6-2 and Figure 6-4). This demonstrates the significant effect of the berm building duration on the potential berm height. The maximum berm crest elevation is typically reached during periods of prolonged mouth closure, or during favourable hydrodynamic conditions.

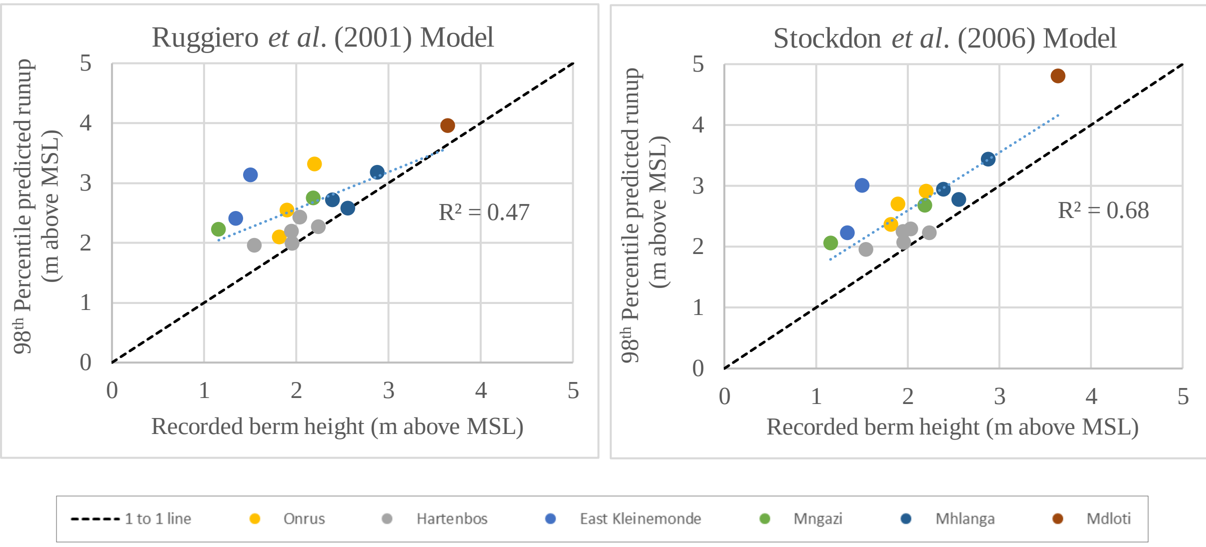

Figure 6-18: Relationship between 98th percentile ranked predicted wave runup and short-term variations in berm height at the respective estuaries – Ruggiero et al. (2001) left and Stockdon et al. (2006) right

Note that by using only the runup elevation as a predictor for berm height at breaching, is an oversimplification. The method does not consider the influence of the following aspects on berm accretion: duration/time; sediment transport rate of runup/overwash; rate of berm accretion; and the direction of sediment transport (onshore or offshore). Figure 6-18 is presented merely to illustrate the relationship between predicted runup and berm height on a shorter time scale. The scatter is due to the significant variation in the magnitude of sediment transport at the berm for the respective scenarios. However, the positive linear correlation displayed in Figure 6-18 illustrates the significant role of the runup elevation on the potential short-term berm height, regardless of the direction and magnitude of sediment transport.

6.5.2.5. Erosion and accretion predictor (Dean Number)

The Dean number (Kraus *et al.*, 1991) is implemented to predict the direction of sediment transport during the berm building phase. The criterion predicts accretion to occur for a Dean number (N_0) < 3.2. The N_0-value was calculated for each hourly wave event of the respective scenario listed in Table

6-9. It was established that accretion had occurred in some scenarios (e.g. scenario 9), despite higher N_0-values (>3.2). It is possible to recalibrate the criterion that distinguishes between erosion and accretion, however the limited data availability makes this difficult.

Alternatively, the N_0-value was used as an indicator for the magnitude of onshore sediment transport, and consequently the amount of accretion at the berm. An average N_0-value, representative of the entire hourly wave record of the scenario was selected to compare to the amount of berm accretion (Figure 6-19 left). The inverse N_0-value is used to normalise the parameter, as well as invert the relationship. This approach assumes insignificant erosion resulting from waves exhibiting higher N_0-values. Instead, a higher N_0-value is expected to generate lower amounts of onshore sediment transport and less accretion at the berm.

The above-mentioned use of the N_0-value still does not account for the possible vertical extent of sediment transport at the berm. The berm height is governed by the elevation to which runup or overwash can transport sediment. By implementing the runup in conjunction with the N_0-value, it provides a vertical deposition scale ($R_{2\%}$) for the proposed magnitude of sediment transport (N_0).

The maximum predicted wave runup ($R_{2\%}$ - Ruggiero *et al.* Model 1 and 2) is multiplied by the inverse of the average N_0-value over the berm building period, resulting in a berm accretion parameter. The proposed method is compared to the recorded berm height at the time of breach, for the respective scenarios (Figure 6-19 right).

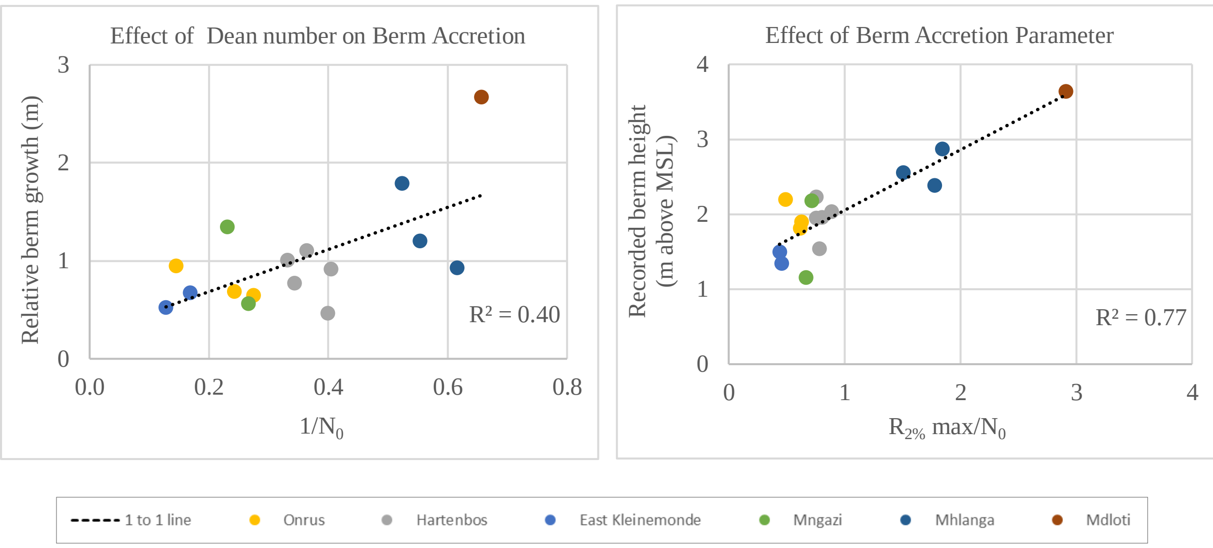

Figure 6-19: Relationship between the berm growth/height and: the inverse Dean number (left); the Berm Accretion Parameter (right)

The results display only a crude linear relationship (R^2=0.40) between the inverse Dean number and the relative berm growth. The presented relationship does not include the effect of the runup elevation on the potential amount of berm accretion.

The berm accretion parameter displays an acceptable linear relationship between variables (R^2=0.77), although the grouping of data points towards lower berm heights is undesirable. This "cone" pattern of data points suggests a potential increase in the variance of observations for data points outside the presented range.

These relationships are presented to illustrate the relationship between the simplified parametric Dean number, and the berm accretion during a recovery period. Wave energy was also considered to use in conjunction with the Dean number as an indicator of the berm growth for the respective scenarios, however it did not provide a significant relationship.

6.5.2.6. Berm growth model

The berm growth model follows the process implemented by Wainwright *et al.* (2013) to statistically model berm height at coastal lagoons. The step wise model involves filtering the offshore waves according to their erosional- and accretional potential, by means of the Dean number (Kraus *et al.*, 1991). Wave events with a Dean number less than 3.2 were considered to be accretionary (onshore sediment transport). The erosional waves are discarded, with the assumption that they cause an insignificant reduction in berm height. The model uses the hourly predicted level of runup (based on the accretionary wave events), along with a berm growth rate, to incrementally adjust the height of the berm. Weir *et al.* (2004, cited in Wainwright *et al.*, 2013) proposed a berm growth rate proportionate to the runup elevation relative to the berm elevation. This difference in elevation, termed overtopping potential, represents the amount of overwash and therefore the potential sediment transport rate at the berm crest. The barrier growth relies on the difference between the predicted runup ($R_{2\%}$) and existing berm height. As the runup exceeds the berm crest, a growth rate proportionate to the overtopping potential is added to the berm (Weir *et al.*, 2007, cited in Wainwright *et al.*, 2013). The growth rate is also dependent on the predefined constant of proportionality. The proposed growth rate is provided in Equation 6.4.

$$\Delta z = c_p \times OP \tag{6.4}$$

where

- Δz = vertical berm growth rate (metres per unit time, hourly in this case)
- c_p = constant of proportionality (determined for individual scenarios/estuaries, dimensionless)
- OP = overtopping potential (predicted runup elevation minus the existing berm height at specific time step, in metres)

Only the recorded berm height at inlet closure and the berm height at the time of breach is available for the respective short-term scenarios. This is due to the data source of the berm elevations, namely the estuarine water level data (§ 4.2). Unfortunately, there are no intermediate berm elevations available for the period between closure and breaching. The output of the berm growth model is based on the following procedure:

- The known berm height at closure is used as the first berm height time step.
- The following hourly berm height is calculated according to the hourly runup elevation ($R_{2\%}$ - Stockdon *et al.* Model), relative to the berm height at the previous time step. The difference in height (*OP*) is multiplied by the constant of proportionality (c_p), in order to obtain the incremental berm growth for that hour.
- The constant of proportionality is not predetermined at the start of the simulation, but rather incrementally adjusted to ensure the modelled berm height conforms to the following outcomes: (1) the final berm height at breach; and (2) to keep the modelled berm height above the recorded estuarine water level up until the point of breaching. These are the only two measurements of the model performance, due to the lack of additional berm measurements between closure and breaching.

The model was implemented for the scenarios listed in Table 6-9. The model output for scenario 14 (Table 6-9) at the Mhlanga Estuary is presented in Figure 6-20 to serve as an example of the procedure. The constant of proportionality was selected as 0.04 for the scenario presented in Figure 6-20. It was noted that the constant of proportionality varies significantly among the individual estuaries, as well as the respective scenarios. The constant of proportionality is dependent on the runup elevation, as well as the direction of sediment transport (Dean's number).

It is suggested that the model be further tested with intermediate berm heights (measured between closure and breach), in order to evaluate the accuracy of the modelled berm height. At this stage, the berm growth model is provided as a suitable model due to its solid theoretical background, limited data input requirements (advantageous for data poor environment in South Africa) and the use of a similar methods in recent studies (Wainwright *et al.*, 2013). The actual performance of the model, i.e., the difference between the modelled berm height and the measured berm height throughout the simulation, cannot be determined at this stage. Further tests with intermediate berm measurements will also elucidate the variation in the proposed constant of proportionality, which may then help to evaluate the model performance and applicability.

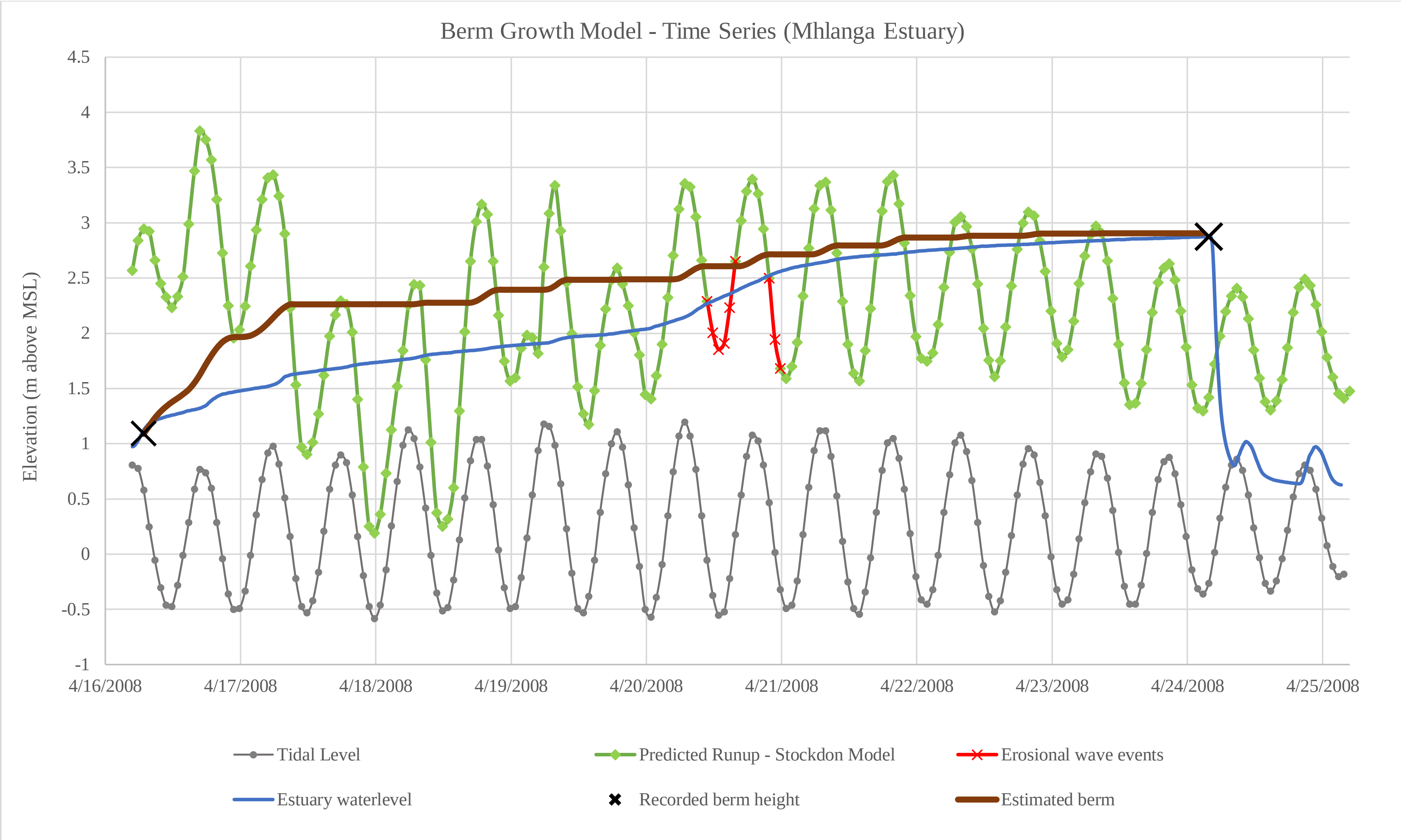

Figure 6-20: Output of the berm growth model used to predict short-term berm growth - Mhlanga Estuary (scenario 14)

Thus, the berm growth model provides a simplified method of predicting berm height, given the lack of appropriate field measurements. The simplified approach makes the following assumptions regarding berm growth:

- Erosional wave events (Dean number > 3.2) do not cause significant erosion of the berm.
- The berm growth rate assumes a positive linear relationship between the overwash potential and the amount of vertical accretion at the berm.
- The constant of proportionality is selected as constant throughout the simulation, when such a value would decrease as the berm grows. The reduction is attributed to the limited sediment transport potential in the upper swash zone (Pritchard & Hogg, 2005).

The model is not recommended for estuaries exhibiting prolonged periods of mouth closure, such as the Bot- or Klein Estuary. The assumption that erosional wave events (Dean number > 3.2) do not cause significant erosion, becomes increasingly questionable as the duration increases. This model could also prove useful in the predictions of estuarine mouth functioning and breaching water levels. The berm growth model, accompanied by predictions of river inflow and estuary water balance, could provide an estimate of the potential breaching level on a short-term time scale. Additionally, the berm growth model could be implemented in the long-term statistical modelling of berm height and estuarine mouth functioning.

6.5.3. Conclusion

The Swart (1974) Model provides a suitable estimate of the berm height at the time of breaching, for the respective scenarios. The Larson and Kraus (1989) Model did not fare well in comparison to the Swart Model. The Okazaki and Sunamura (1995) Model inaccurately describes the variability in berm height among estuaries. This is primarily due to the model’s relative insensitivity to variations in significant wave height and median sediment grain size.

The Dean number and selected runup parameterisations can be implemented to estimate the direction and magnitude of cross shore sediment transport, as well as the vertical extent of sediment deposition on a short-term time scale. These predictors lack high levels of accuracy, however they provide an approximate sense of berm growth.

The berm growth model provides an incremental approach for predicting the morphological response of berms subjected to runup and overwash. The berm growth model employs the hourly predicted runup elevation from constructive waves, in order to incrementally adjust the berm height. Predictions of berm growth/height on a time scale of individual breaches is problematic given the limited knowledge regarding the sediment transport and morphological response of berms during the growth period. The berm growth model is the best suited short-term predictive method, given the limited data availability.

6.5.4. Recommendations

At this stage, there is limited available information regarding the rate and magnitude of accretion caused by runup and overwash events at South African TOCE berms. Local field observations of runup/overwash and corresponding morphological responses could prove to be useful in the prediction of berm height. Field measurements could provide information regarding the morphodynamic response of estuarine berms subjected to runup and overwash. Additional intermediate berm surveys, conducted during a single berm building process, could also help verify the validity of the berm growth model.

Additionally, beach profile modelling could be considered as a method of predicting short-term berm growth/recovery. This could potentially establish the ability of beach profile models to successfully predict accretion at estuarine berms.

7. A Methodology for Predicting Estuarine Berm Height

This chapter primarily acts as a summary of the predictive methods discussed in Chapters 5 and 6. Additionally, this chapter provides an overview of the methodology for predicting the maximum berm height at South African TOCEs. The outline of the methodology takes into consideration the limited data availability and proposes suitable procedures for the relevant berm prediction scenarios.

The methodology is divided into three main components according to the input requirements and the desired outcome. Each individual method/component presents an individual design approach, which is discussed in § 7.1 through to § 7.3.

The selection and aptness of the respective methods are dependent on the available data and the intended application. It is recommended that the methods be used in conjunction, to provide a more confident estimate and range of potential maximum berm height.

7.1. First Estimate of Maximum Berm Height

The first estimate of maximum berm height at a South African TOCE implements the Berm Crest Elevation Criteria and corresponding regression model as set out in § 5.4. The method provides the potential maximum berm height based on only a few coastal parameters namely, nearshore 1-in-1 year significant wave height at 15 m isobath, median sediment grain size, tidal range and beach face slope. The objective of the method is to provide a first order estimate of the maximum berm height at less studied estuaries.

The Berm Crest Elevation Criteria and corresponding linear regression model provides a relatively accurate estimate of the maximum berm crest elevation at South African TOCEs, with a relatively small MAE of 0.23 m and a RMSEP of 0.26 m.

7.2. Predicting Long-Term Variations in Berm Height

This approach consists of two separate methods, both exploring the relationship between the total water level on the berm ($R_{2\%}$ superimposed on tide) and the maximum berm height, as set out in § 6.2 through to § 6.4. The Stockdon *et al*. (2006) Model is proposed due to its consistent performance across the range of input parameters.

Method 1 relies on correlating the maximum berm height to several years of runup data at an estuary (§ 6.4.1). This process is the most robust and comprehensive method described in this study, however it is time consuming compared to the other methods. Additional data requirements include long-term records of wave and tidal data in order to create a simulated record of hourly wave runup at the estuary. This method provides a probability distribution of wave runup from which corresponding maximum berm heights can be derived. The 5% exceedance probability of wave runup elevation ($R_{2\%}$ superimposed on tide) at the estuary berm provides an accurate estimate of the maximum potential berm crest elevation, with a MAE of 0.28 m and RMSEP of 0.34 m.

Method 2 is a less time consuming, yet similarly accurate alternative to Method 1 (§ 6.4.2). The method implements a specific design combination of significant wave height, peak wave period and tidal level to predict the $R_{2\%}$ wave runup elevation beyond which significant berm accretion does not occur. The $R_{2\%}$ wave runup elevation calculated according to the 2% exceedance probability significant wave height, 50% exceedance probability peak wave period and MHWS tidal level provides a relatively accurate estimate of the maximum potential berm crest elevation. Method 2 provides a prediction with a MAE of 0.26 m and RMSEP of 0.34 m.

7.3. Predicting Berm Height Between Individual Breaches

The berm growth model provides an incremental approach to modelling the berm growth/accretion following estuary mouth closure, as set out in § 6.5.2.6. The method requires data inputs similar to the long-term runup simulation, only over a shorter period. The method implements the Dean number and the hourly $R_{2\%}$ wave runup elevation relative to the existing berm height, in order to estimate the incremental berm growth rate. This method involves certain simplifications, however it proves useful for modelling estuarine mouth functioning and berm height on a short-term time scale.

8. Conclusions and Recommendations

The first objective of this study was to determine the potential maximum berm crest elevations of South African Temporarily Open/Closed Estuaries (TOCE), as well as to identify the primary drivers responsible for high berms and variations in berm height among estuaries. Recorded berm crest elevations were acquired from available berm/mouth surveys and long-term estuary water level data. The relationship between the relevant coastal parameters and the maximum berm height at the selected estuaries were evaluated by means of a multi-criteria analysis (Berm Crest Elevation Criteria) and linear regression model.

The second study objective was to identify and evaluate suitable methods for the prediction of berm height, taking into consideration the limited data availability in South Africa. Additionally, this objective aims to elucidate the relationship between wave runup and the maximum berm height at TOCEs. Existing wave runup parameterisations were selected to predict the maximum berm height, due to their limited data inputs, relative accuracy and the significant effect of the runup elevation on berm accretion. Several years of simulated runup was predicted for the respective estuaries, in order to elucidate the relationship between wave runup and estuarine berm height. Additionally, several methods were proposed for the prediction of the berm height (long- and short-term variations) at South African TOCEs.

Detailed conclusions were presented following the respective analyses throughout the report. The following subsections act as a summary to present the conclusions drawn from fulfilling the respective research objectives.

8.1. Berm Height at South African TOCEs

- A relatively large data set of recorded berm crest elevations was derived from the water level data and berm surveys of the respective estuaries. The record length varied considerably among estuaries, ranging between 8 and 151 recorded berm heights. Notwithstanding, the data set provided a suitable range of berm heights for the respective estuaries, especially compared to previous estimates. The recorded berm heights provide an estimate of the potential upper range of berm height at the selected estuaries.

- The relationship between the recorded berm height at the selected estuaries was tested against the following parameters: median sediment grain size of the berm, beach face slope at the berm, 1-in-1 year return period nearshore wave height, and nearshore Iribarren number associated with the 1-in-1 year nearshore wave event. The beach face slope and nearshore Iribarren number provide the most significant linear relationship to maximum berm height at the respective estuaries.

- The proposed Berm Crest Elevation Criteria and corresponding regression model successfully describe the combined relationship the relevant coastal parameters have on berm height. The criteria and model provide a relatively accurate prediction of the maximum berm height at a South African TOCE based on the mean tidal range, beach face slope, median sediment grain size and nearshore significant wave height (1 in 1 year return period at 15 m water depth). Weighting 3 and the semi-closed reduction factor (rf = 0.5) are proposed to attain the most realistic prediction, with a Mean Absolute Error (MAE) of 0.23 m and a Root Mean Square Error Prediction (RMSEP) of 0.26 m.

8.2. Predicting Berm Height at South African TOCEs

- There is a significant relationship between the predicted wave runup elevation and the maximum recorded berm height at an estuary, i.e., predicted runup provides an accurate estimate of the long-term variation of estuarine berm height.

- The maximum predicted elevation of wave runup ($R_{2\%}$), derived from the long-term hourly simulated runup records, far exceed the maximum recorded berm heights attained in an overlapping record. This suggests that no significant sediment accretion occurs above the long-term maximum predicted $R_{2\%}$ elevation.

- The most accurate predictors for estimating the maximum berm crest elevations at estuaries are the combined Ruggiero *et al.* (2001) Model 1 and 2, and the Stockdon *et al*. (2006) Model. These predictors provide consistent performance and minor errors across the entire range of estuaries and input variables. Preference is given to the Stockdon *et al*. Model due to its superior trend of predictions.

- Upon analysis of the occurrence probability of the simulated runup records, it is evident that the maximum berm crest elevation at an estuary can be estimated with a generally low exceedance probability of total water level on the berm ($R_{2\%}$ superimposed on tide). The approach relies on several years of wave- and tidal data to compare the maximum berm height to a specific exceedance probability of predicted runup elevation. The occurrence probability of the wave runup elevation corresponding to the maximum berm height varies among individual estuaries, ranging from 1% up to 15%.

- The average exceedance probability of the predicted wave runup elevation (Stockdon *et al*. Model) corresponding to the maximum berm height among the respective estuaries is 5%. Accordingly, the 5% exceedance probability wave runup elevation is used to estimate the maximum berm crest elevation at the selected TOCEs. The method provides a relatively accurate prediction of the maximum berm height, with a MAE of 0.28 m and a RMSEP of 0.34 m. Therefore, out of a long-term perspective, the runup levels associated with these exceedance probabilities (< 5%), rarely generate regular significant vertical accretion of the berm.

- An alternative method is proposed to estimate the vertical extent of sediment accretion caused by wave runup. The 2% exceedance probability significant wave height, 50% exceedance probability peak wave period and MHWS tidal elevation is used as input for the Stockdon *et al*. (2006) Model, resulting in a single wave runup elevation to predict the maximum berm crest height. The method provides a relatively accurate prediction of the maximum berm height, with a MAE of 0.26 m and a RMSEP of 0.34 m.

- The proposed berm growth model provides an incremental approach for predicting the vertical accretion of berms over a short-term period. The berm growth model employs the hourly predicted runup elevation from waves resulting in accretion, in order to incrementally adjust the berm height. The method is best suited for estuaries exhibiting a relatively rapid rate of closure (e.g. a few days or weeks). Knowledge regarding the short-term morphodynamic response of berms subjected to wave action is required for increased predictive capabilities of berm height/growth on a short-term time scale. Additionally, the Swart (1974) Model provides a suitable prediction of the berm height at the time of breaching (§ 6.5.2.1).

- The method of correlating the maximum berm height at an estuary to a low exceedance probability of wave runup (5%) is the most robust method of prediction explored in this study. The method accounts for the combined effect of the tidal elevation, wave height and wave period on the total water level on the berm ($R_{2\%}$ superimposed on tide).

8.3. Recommendations for Further Research

The results and knowledge acquired during this study indicate that the following aspects may benefit from further investigation in future studies.

The nearshore wave transformational coefficient (K_T) used in this study is a simplified approach, intended to substitute the use of nearshore wave modelling. Additionally, the lack of wave runup measurements at the selected study locations proved it impossible to validate the simulated wave runup records. A similar approach as set out in § 6.1 and 6.2 is proposed, simulating a long-term record of wave runup at a selected TOCE. The process should include numerical wave modelling, as well as runup model verification/calibration by means of runup measurements. The results can be compared to the findings of this study and potentially verify the use of the 5% exceedance probability of runup as a predictor for maximum berm height. Accordingly, the threshold of the runup exceedance probability associated with berm growth may also be verified.

A field observation campaign is proposed to provide information regarding the morphodynamic response of estuarine berms subjected to runup and overwash. The measurements should document the berm recovery/growth following inlet closure, up until the point of breaching. Observations should primarily include high frequency measurements of the berm elevation and geometry, as well as the associated runup elevations and hydrodynamic conditions.

The proposed field observations could potentially establish the relationship between the type of morphological response associated with the hydrodynamic and morphodynamic conditions present. This could potentially result in a qualitative prediction of the cross-shore profile response to wave runup and overwash. The relationship between berm accretion, overwash potential (runup elevation relative to existing berm height) and berm width should be investigated to determine the potential correlation between parameters.

The relationship between berm height and wave runup presented in this study (short- and long-term) can be implemented in larger statistical based modelling approaches of estuarine mouth functioning. A statistical modelling approach – possibly Monte Carlo Simulation – could provide a probabilistic distribution of estuarine berm height.

Beach profile modelling could be considered as a potential predictor of the short-term berm growth/recovery, following breaching or inlet closure.

8.4. Concluding Remarks

The recorded berm crest elevations derived from long-term water levels and survey data provide an accurate estimate of the potential berm height at the selected estuaries. The Berm Crest Elevation Criteria predicts the maximum berm height at South African TOCEs with a relative degree of accuracy. The predictive wave runup methods presented, as well as the acquired knowledge regarding the relationship between wave runup and estuarine berm height, provides suitable fundamentals for the prediction of berm height, given the limited available data. These methods provide increased predictive capabilities for estuarine berm height and consequently estuarine peak flood levels. Additionally, the study provides a comprehensive theoretical background of berm height/growth for future studies.

9. References

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Appendix A Sieve Test Analysis

A.1 Grain Size Parameters

Units of sediment grain size

The recognised units of sediment grain size used in practice includes millimetres (mm) and microns (µm). Alternatively, the *phi unit* (φ) is implemented to aid the classification of sediment grain sizes. A conversion between the phi size and mm is (CEM, 2006: III-1-9):

$$\varphi = -\log_2 D \tag{A.1}$$

$$D = 2^{-\varphi} \tag{A.2}$$

where

- φ = phi size
- D = grain size (mm)

Qualitative descriptors of grain size distributions

Phi standard deviation (σ_φ): This provides a measure of the degree to which the sediment sample is spread out around the mean, i.e., the sorting of the sample. The phi standard deviation is expressed as (CEM, 2006: III-1-10):

$$\sigma_\varphi = \frac{\varphi_{84}-\varphi_{16}}{4} + \frac{\varphi_{95}-\varphi_{5}}{6} \tag{A.3}$$

Phi coefficient of skewness (α_φ): This provides the degree to which the distribution deviates from symmetry. A perfect symmetrical distribution will deliver a skewness coefficient of zero. The phi coefficient of skewness is expressed by (CEM, 2006: III-1-10):

$$\alpha_\varphi = \frac{\varphi_{16}+\varphi_{84}-2(\varphi_{50})}{2(\varphi_{84}-\varphi_{16})} + \frac{\varphi_{5}+\varphi_{95}-2(\varphi_{50})}{2(\varphi_{95}-\varphi_{5})} \tag{A.4}$$

Phi coefficient of kurtosis (β_φ): This provides a measure of the peakedness displayed by the grain size distribution, i.e., to indicate whether the material is grouped around the mean, or spread more toward the tails. The phi coefficient of kurtosis is expressed by (CEM, 2006: III-1-10):

$$\beta_\varphi = \frac{\varphi_{95} - \varphi_5}{2.44(\varphi_{75} - \varphi_{25})} \tag{A.5}$$

Note that the φ_n – value in the above-mentioned formulae refers to the phi size for the relevant distribution percentage. The relative relationship and descriptions for the ranges of standard deviation, skewness and kurtosis are provided in Table A-1.

Table A-1: Qualitative sediment distribution range for standard deviation, skewness, and kurtosis (CEM, 2006: III-1-11)

Table III-1-3
Qualitative Sediment Distribution Ranges for Standard Deviation, Skewness, and Kurtosis

	Standard Deviation
Phi Range	**Description**
<0.35	Very well sorted
0.35-0.50	Well sorted
0.50-0.71	Moderately well sorted
0.71-1.00	Moderately sorted
1.00-2.00	Poorly sorted
2.00-4.00	Very poorly sorted
>4.00	Extremely poorly sorted
	Coefficient of Skewness
<-0.3	Very coarse-skewed
- 0.3 to - 0.1	Coarse-skewed
- 0.1 to +0.1	Near-symmetrical
+0.1 to +0.3	Fine-skewed
>+0.3	Very fine-skewed
	Coefficient of Kurtosis
<0.65	Very platykurtic (flat)
0.65-0.90	Platykurtic
0.90-1.11	Mesokurtic (normal peakedness)
1.11-1.50	Leptokurtic (peaked)
1.50-3.00	Very leptokurtic
>3.00	Extremely leptokurtic

A.2 Sieve Test Results

The sieve test results for the samples acquired during the field measurement campaign are provided in this section.

Table A-2: Sieve test results - Kleinmond Estuary berm

Sample ID:	Kleinmond Estuary		Date Collected:	10/06/2017
Sample Dry Mass (g):	950.59			
Sieve size (mm)	**Mass retained (g)**	**Cumulative mass retained (g)**	**% Retained**	**% Passing**
2	0.37	0.37	0.039	99.961
1.18	2.94	3.31	0.348	99.652
0.6	108.61	111.92	11.774	88.226
0.425	191.1	303.02	31.877	68.123
0.3	340.62	643.64	67.710	32.290
0.15	306.24	949.88	99.926	0.074
0.075	0.66	950.54	99.996	0.004
pan	0.04	950.58	100.000	0.000
Total	950.58			
Percentage loss (%)	0.0011			

Table A-3: Sieve test results - Bot Estuary berm

Sample ID:	Bot Estuary		Date Collected:	10/06/2017
Sample Dry Mass (g):	1025.2			
Sieve size (mm)	**Mass retained (g)**	**Cumulative mass retained (g)**	**% Retained**	**% Passing**
2	0.05	0.05	0.005	99.995
1.18	0.11	0.16	0.016	99.984
0.6	56.46	56.62	5.523	94.477
0.425	506.09	562.71	54.886	45.114
0.3	339.06	901.77	87.958	12.042
0.15	123.3	1025.07	99.984	0.016
0.075	0.14	1025.21	99.998	0.002
pan	0.02	1025.23	100.000	0.000
Total	1025.23			
Percentage loss (%)	-0.0029			

Table A-4: Sieve test results - Onrus Estuary berm

Sample ID:	Onrus Estuary		Date Collected:	10/06/2017
Sample Dry Mass (g):	883.69			
Sieve size (mm)	**Mass retained (g)**	**Cumulative mass retained (g)**	**% Retained**	**% Passing**
2	0.16	0.16	0.018	99.982
1.18	0.3	0.46	0.052	99.948
0.6	33.09	33.55	3.797	96.203
0.425	155.91	189.46	21.441	78.559
0.3	412.35	601.81	68.107	31.893
0.15	280.22	882.03	99.820	0.180
0.075	1.51	883.54	99.991	0.009
pan	0.08	883.62	100.000	0.000
Total	883.62			
Percentage loss (%)	0.0079			

Table A-5: Sieve test results - Klein Estuary berm

Sample ID:	Klein Estuary		Date Collected:	10/06/2017
Sample Dry Mass (g):	885.09			
Sieve size (mm)	**Mass retained (g)**	**Cumulative mass retained (g)**	**% Retained**	**% Passing**
2	0.09	0.09	0.010	99.990
1.18	0.78	0.87	0.098	99.902
0.6	7.24	8.11	0.916	99.084
0.425	15.1	23.21	2.623	97.377
0.3	57.26	80.47	9.093	90.907
0.15	763.21	843.68	95.333	4.667
0.075	41.29	884.97	99.999	0.001
pan	0.01	884.98	100.000	0.000
Total	884.98			
Percentage loss (%)	0.0124			

Table A-6: Sieve test results - Touw Estuary berm

Sample ID:	Touw Estuary		Date Collected:	10/06/2017
Sample Dry Mass (g):	322.13			
Sieve size (mm)	**Mass retained (g)**	**Cumulative mass retained (g)**	**% Retained**	**% Passing**
2	0.03	0.03	0.009	99.991
1.18	0.06	0.09	0.028	99.972
0.6	1.14	1.23	0.382	99.618
0.425	10.81	12.04	3.738	96.262
0.3	122.18	134.22	41.668	58.332
0.212	149.4	283.62	88.048	11.952
0.15	35.88	319.5	99.187	0.813
0.075	2.58	322.08	99.988	0.012
pan	0.04	322.12	100.000	0.000
Total	322.12			
Percentage loss (%)	0.0031			

Appendix B General Information of Selected Estuaries

Table B-1: General characteristics of selected TOCEs (Whitfield and Bate, 2007)

GENERAL INFORMATION								
		Estuary	Coordinates	Classification	Catchment (km^2)	MAR (m^{3}x10^6)	Water Area (ha)	Tidal volume (water area x 1.8) (m^3)
1	Western Cape	**Diep**	33°53'25"S, 18°29'00"E	TOCE	1495	80	100	180
2	Western Cape	**Lourens**	34°05'58"S, 18°48'40"E	TOCE	140	122	2	4
3	Western Cape	**Palmiet**	34°20'37"S, 18°59'45"E	POE/TOCE	535	201	21	39
4	Western Cape	**Kleinmond**	34°22'06"S, 19°05'56"E	Est Lake	26273	66	1500	2700
5	Western Cape	**Bot**	34°22'06"S, 19°05'56"E	Est Lake				
6	Western Cape	**Onrus**	34°25'07"S, 19°10'47"E	TOCE	59	3	1	3
7	Western Cape	**Klein**	34°25'24"S, 19°18'13"E	Est Lake	983	53	741	1334
8	Western Cape	**Hartenbos**	34°07'07"S, 22°07'27"E	TOCE	205	5	16	28
9	Western Cape	**Klein Brak**	34°05'31"S, 22°08'59"E	TOCE	550	60	50	90
10	Western Cape	**Groot Brak**	34°03'23"S, 22°14'25"E	TOCE	190	39	38	69
11	Western Cape	**Touw**	33°59'51"S, 22°34'51"E	Est Lake	89	25	37	66
12	Western Cape	**Piesang**	34°03'37"S, 23°22'46"E	TOCE	96	12	8	15
13	Western Cape	**Groot (West)**	33°58'52"S, 23°34'17"E	TOCE	119	30	28	50
14	Eastern Cape	**Tsitsikamma**	34°08'06"S, 24°26'18"E	TOCE	225	55	5	8
15	Eastern Cape	**Seekoei**	34°05'11"S, 24°54'30"E	TOCE	224	14	83	150
16	Eastern Cape	**Kabeljous**	34°00'17"S, 24°56'13"E	TOCE	276	18	77	139
17	Eastern Cape	**West Kleindemonde**	33°32'28"S, 27°02'51"E	TOCE	94	4	19	35
18	Eastern Cape	**East Kleinemonde**	33°32'21"S, 27°02'55"E	TOCE	46	2	18	32
19	Eastern Cape	**Ncera**	33°10'12"S, 27°40'11"E	TOCE	77	7	17	30
20	Eastern Cape	**Quinira**	32°58'27"S, 27°57'57"E	TOCE	90	9	34	62
21	Eastern Cape	**Bulura**	32°53'28"S, 28°05'38"E	TOCE	47	4	19	35
22	Eastern Cape	**Cefane**	32°48'30"S, 28°08'11"E	TOCE	38	5	23	41
23	Eastern Cape	**Mngazi**	31°40'32"S, 29°27'40"E	TOCE	561	65	13	23
24	Eastern Cape	**Mpenjathi**	30°58'21"S, 30°17'02"E	TOCE	100	26	12	21
25	KwaZulu-Natal	**Mbokodweni**	30°00'29"S, 30°56'12"E	TOCE	283	36	7	13
26	KwaZulu-Natal	**Mhlanga**	29°42'14"S, 31°06'03"E	TOCE	118	26	30	54
27	KwaZulu-Natal	**Mdloti**	29°39'07"S, 31°07'43"E	TOCE	527	117	33	59
28	KwaZulu-Natal	**Tongati**	29°34'21"S, 31°11'07"E	TOCE	17	7	1	3
29	KwaZulu-Natal	**Mhlali**	29°27'40"S, 31°16'39"E	TOCE	304	49	21	38
30	KwaZulu-Natal	**Nonoti**	29°19'01"S, 31°24'29"E	TOCE	-	-	-	-
31	KwaZulu-Natal	**Zinkwazi**	29°16'45"S, 31°26'35"E	TOCE	73	15	20	37

Appendix C Sediment Grain Size of Selected Estuary Berms

Table C-1: Median sediment grain sizes of the inlet berms of selected TOCEs

MEDIAN SEDIMENT GRAIN SIZE OF ESTUARY BERMS				
		Estuary	**Sediment Grain Diameter - D_{50} (mm)**	**Source**
1	Western Cape	**Diep**	0.385	CSIR (2000b)
2		**Lourens**	0.25 | 0.156-0.199	Van der Merwe (2017) | Unpublished data
3		**Palmiet**	0.408/0.356/0.535/0.521	CSIR (2000b)
4		**Kleinmond**	0.34	Field survey (author)
5		**Bot**	0.43	Field survey (author)
6		**Onrus**	0.48-0.53 | 0.33-0.35	CSIR (1991) | Field survey (author)
7		**Klein**	0.255 | 0.199-0.261 | 0.2	CSIR (2000) | CSIR (1991) | Field survey (author)
8		**Hartenbos**	0.341	CSIR (2000b)
9		**Klein Brak**	0.462	CSIR (2000b)
10		**Groot Brak**	0.398 | 0.249 – 0.43	CSIR (2000b) | Unpublished data
11		**Touw**	0.27-0.29	Field survey (author)
12		**Piesang**	0.225-0.260	CSIR (1985)
13		**Groot (West)**	-	-
14	Eastern Cape	**Tsitsikamma**	-	-
15		**Seekoei**	0.271 | 0.281	CSIR (2000c) | Unpublished data
16		**Kabeljous**	-	-
17		**West Kleindemonde**	0.238-0.265	CSIR (2000c)
18		**East Kleinemonde**	0.238-0.265	CSIR (2000c)
19		**Ncera**	-	-
20		**Quinira**	0.196	CSIR (2000c)
21		**Bulura**	0.204	CSIR (2000c)
22		**Cefane**	0.208	CSIR (2000c)
23		**Mngazi**	0.341	CSIR (2000c)
24	KwaZulu-Natal	**Mpenjathi**	0.354	CSIR (1976)
25		**Mbokodweni**	0.888/0.996 | 0.73-0.96	CSIR (1976) | Unpublished data
26		**Mhlanga**	0.38 | 1.12/0.84/0.62	Unpublished data | EMS (2007)
27		**Mdloti**	1.0 | 1.113/0.987	CSIR (2000a) | Unpublished data
28		**Tongati**	1.2 | 0.95	CSIR (2000a) | Unpublished data
29		**Mhlali**	-	-
30		**Nonoti**	-	-
31		**Zinkwazi**	0.528	Unpublished data

Appendix D Beach Face Slopes of Selected Estuary Berms

Table D-1: Beach face slopes measured at the respective inlet berms of the TOCEs and distance from shoreline to 15m isobath

			BEACH FACE SLOPE AT THE ESTUARY BERMS		
		Estuary	Beach face slope (tanα)	Source	Distance to 15m depth contour (m)
1	Western Cape	**Diep**	0,02-0,03	Van der Merwe (2017)	1264
2		**Lourens**	0,042 | 0,025	Van der Merwe (2017) | Hassiem (2016)	3317
3		**Palmiet**	0,059/0,076	CSIR (2000b)	806
4		**Kleinmond**	0,088-0,095	Field survey (author)	900
5		**Bot**	0,057-0,103	Field survey (author)	768
6		**Onrus**	0,022 – 0,047	Field survey (author)	727
7		**Klein**	0,0245-0,0498	Field survey (author)	1258
8		**Hartenbos**	0,0198-0,0409	CSIR (2000b)	829
9		**Klein Brak**	0,0533-0,0741	CSIR (2000b)	796
10		**Groot Brak**	0,04185-0,055	CSIR (2000b)	744
11		**Touw**	0,039	Unpublished data	
12		**Piesang**	0,0349-0,0524	CSIR (1985)	551
13		**Groot (West)**	-	-	616
14	Eastern Cape	**Tsitsikamma**	-	-	931
15		**Seekoei**	0,0183-0,0656	CSIR (2000c)	2524
16		**Kabeljous**	-	-	1732
17		**West Kleindemonde**	0,047619	Whitfield and Bate (2007)	
18		**East Kleinemonde**	0,041667	Whitfield and Bate (2007)	
19		**Ncera**	0,0209/0,0195	CSIR (2000c)	1146
20		**Quinira**	0,0319-0,0538	CSIR (2000c)	954
21		**Bulura**	0,0181-0,0247	CSIR (2000c)	765
22		**Cefane**	0,019 - 0,0244	CSIR (2000c)	947
23		**Mngazi**	0,0147-0,0255	CSIR (2000c)	311 | 960
24	KwaZulu-Natal	**Mpenjathi**	0,0524 - 0,0699	CSIR (1976)	1144
25		**Mbokodweni**	0,1227/0,1405 | 0,091-0,111	CSIR (1976) | Unpublished data	658
26		**Mhlanga**	0,1/0,12	EMS (2007)	796
27		**Mdloti**	0,16	CSIR (2000a)	711
28		**Tongati**	0,108-0,126	Unpublished data	583
29		**Mhlali**	-	-	425
30		**Nonoti**	-	-	
31		**Zinkwazi**	-	-	

Appendix E Nearshore Wave Height at Selected Estuaries

Table E-1: Nearshore modelled wave properties at selected estuary inlets (Rossouw et al., 2014)

	Estuary	Hs – 1 in 1 year return period at d = 15 m (m)	Peak wave period (s)	Wave direction (degrees)
1	**Diep**	3.420	12.93	281
2	**Lourens**	4.078	13.29	208
3	**Palmiet**	4.664	13.61	186
4	**Kleinmond**	5.156	13.91	204
5	**Bot**	5.902	14.06	216
6	**Onrus**	5.594	14.09	205
7	**Klein**	4.904	13.80	220
8	**Hartenbos**	3.346	7.39	108
9	**Klein Brak**	3.542	8.42	125
10	**Groot Brak**	4.228	11.86	166
11	**Touw**	4.418	12.76	177
12	**Piesang**	3.065	8.03	107
13	**Groot (West)**	4.711	13.01	182
14	**Tsitsikamma**	5.476	14.03	217
15	**Seekoei**	4.122	12.75	156
16	**Kabeljous**	2.990	9.05	126
17	**West Kleindemonde**	4.557	13.53	-
18	**East Kleinemonde**	4.557	13.53	-
19	**Ncera**	4.470	13.84	158
20	**Quinira**	4.111	12.10	146
21	**Bulura**	4.418	12.71	146
22	**Cefane**	3.666	11.13	131
23	**Mngazi**	4.249	12.34	156
24	**Mpenjathi**	4.086	12.26	141
25	**Mbokodweni**	3.986	11.80	138
26	**Mhlanga**	3.859	11.00	116
27	**Mdloti**	3.942	11.67	127
28	**Tongati**	4.019	11.69	134
29	**Mhlali**	4.270	11.75	143
30	**Nonoti**	-	-	-
31	**Zinkwazi**	-	-	-

Appendix F Regression Modelling-Additional Information

This appendix provides the formulae and descriptions for the relevant regression models and regression performance indicators.

Mean Absolute Error (MAE)

The MAE is a measure of the average magnitude of the errors of the predictions. The direction of the error (positive or negative) is not considered, as the formula takes the absolute value. The formula for the MAE is:

$$MAE = \frac{1}{n}\sum_{j=1}^{n}\left|\hat{y}_j - y_j\right| \tag{F.1}$$

where

- $\hat{y}_j$ = predicted value from model
- y_j = measured data points
- n = number of data points/observations

The MAE expresses the average model error in the units of interest, e.g. meters. A small value of MAE is preferred, as it indicates good performance of the predictor.

Root Mean Square Error Predictor (RMSEP)

The RMSEP is also an average measure of the error, however the method uses a quadratic scoring rule. The formula for the RMSEP is:

$$RMSEP = \sqrt{\frac{1}{n}\sum_{j=1}^{n}\left(\hat{y}_j - y_j\right)^2} \tag{F.2}$$

where

- $\hat{y}_j$ = predicted value from model
- y_j = measured data points
- n = number of data points/observations

The RMSEP expresses the average model error in the units of interest, e.g. meters. A small value of RMSEP is preferred, as it indicates good performance of the predictor. The RMSEP uses the squared errors, therefore providing relatively high weight to larger errors. This makes the RMSEP helpful when larger errors are particularly unwanted.

Four Parameter Logistic (4PL) Regression Model

The 4PL model is a nonlinear regression model used for fitting a S-shaped curve to a data set. The 4PL model equation is:

$$y = d + \frac{a-d}{a+\left(\frac{x}{c}\right)^{b}} \tag{F.3}$$

where

- *x* = independent variable
- *y* = dependant variable
- *a* = minimum asymptote
- *b* = rate of change of the curve
- *c* = point of inflection
- *d* = maximum asymptote

Appendix G Berm Height Records Derived from Estuarine Water Level Recordings and Berm/Mouth Surveys

This appendix provides the recorded berm crest elevations derived from the Department of Water and Sanitation (DWS) water level recordings and Council for Scientific and Industrial Research (CSIR) mouth/berm surveys. The total record is provided for future reference purposes. The corrected water levels at the time of breach, i.e., the berm crest saddle point elevations, are referenced to Mean Sea Level (MSL) by means of the relevant correction factors provided in Table 4-2. The artificial breaches at the relevant estuaries are also included in the records.

Table G-1: Complete berm height record of selected estuaries

BERM CREST ELEVATIONS DERIVED FROM ESTUARINE WATER LEVEL DATA AND MOUTH/BERM SURVEYS					
Lourens Estuary			**G2T043**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	2005-01-28	09:17	1.28		DWS Estuarine water levels
2	2005-04-11	00:44	1.45		
3	2006-04-22	11:49	1.32		
4	2007-03-04	02:58	1.49		
5	2007-04-11	17:29	1.20		
6	2007-04-26	22:49	1.61		

Palmiet Estuary			**G4H007**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1992-04-10	11:06	1.58		DWS Estuarine water levels
2	1993-04-09	07:52	1.57		
3	1994-05-27	09:24	1.70		
4	1995-03-20	15:46	1.75		
5	1995-04-09	14:02	1.00		
6	1995-04-17	22:52	1.56		
7	1996-03-08	22:30	1.46		
8	1996-04-22	22:15	1.36		
9	1997-03-15	11:24	1.72		
10	1997-04-06	11:58	1.42		
11	1998-03-06	21:41	1.53		
12	1998-03-15	00:56	1.74		
13	1998-04-21	14:41	1.44		
14	1999-02-23	00:05	1.65		
15	1999-03-08	05:18	1.72		
16	1999-03-21	14:00	1.61		
17	1999-04-01	08:36	1.75		
18	2000-03-03	09:55	1.19		
19	2000-05-30	03:18	1.70		
20	2001-01-16	01:35	1.65		
21	2001-03-03	12:14	1.44		
22	2001-03-24	00:51	1.75		
23	2001-05-05	06:12	1.89		

24	2002-03-28	17:34	1.55		
25	2002-04-06	01:14	0.88		
26	2002-04-10	04:35	1.41		
27	2003-01-17	23:33	1.16		
28	2003-02-17	05:48	1.53		
29	2003-12-26	09:21	1.33		
30	2004-02-17	20:02	1.64		
31	2004-04-17	04:17	1.70		
32	2004-06-07	09:59	1.34		
33	2005-02-23	16:26	1.66		
34	2005-04-11	06:29	1.96		
35	2006-02-18	11:16	1.36		
36	2006-03-22	22:24	1.60		
37	2006-04-05	17:13	1.62		
38	2006-04-21	02:51	1.53		
39	2006-04-24	08:32	1.46		
40	2007-02-24	21:42	1.83		
41	2007-03-27	13:56	1.65		
42	2007-04-09	03:05	1.47		
43	2007-04-20	04:33	1.74		
44	2007-04-27	15:56	1.77		
45	2008-05-11	17:32	1.52		
46	2008-05-19	01:48	1.23		
47	2009-03-14	04:39	1.70		
48	2009-04-26	09:11	1.43		
49	2010-02-25	22:18	1.37		
50	2010-03-17	14:30	1.34		
51	2011-03-28	02:03	1.78		
52	2011-04-24	02:25	1.79		
53	2013-02-21	15:50	1.18		
54	2013-04-17	16:42	1.77		
55	2014-12-24	13:19	1.61		
56	2014-12-26	08:05	1.97		
57	2015-02-26	15:10	1.77		
58	2015-03-15	21:27	1.80		
59	2015-03-27	22:40	1.80		
60	2015-04-10	19:47	1.65		
61	2015-04-21	19:47	1.62		
62	2015-05-11	08:27	1.58		
63	2015-05-20	09:56	1.76		
64	2015-05-31	02:53	1.50		
65	2016-01-10	00:39	1.07		
66	2016-02-04	06:00	1.68		
67	2016-02-20	13:38	1.65		
68	2016-02-28	14:56	1.44		
69	2016-03-15	15:50	1.73		
70	2016-03-27	07:53	1.63		

Kleinmond Estuary			**G4H012**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	2007-07-30	06:47	2.71		DWS Estuarine water levels
2	2008-07-13	04:14	2.15		
3	2012-08-11	16:49	2.68		
4	2013-06-27	23:01	2.27		
5	2014-06-14	22:21	2.26		

Bot Estuary			G4H014		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1981-08-12	12:05	2.54		DWS Estuarine water levels
2	1981-10-10	17:53	1.78		
3	1983-06-29	08:34	2.72		
4	1985-07-12	17:34	2.10		
5	1986-08-30	14:18	3.17		
6	1989-06-26	15:02	2.87		
7	1990-07-15	00:12	2.78		
8	1993-04-13	11:15	3.14		
9	1995-08-26	21:53	2.58		
10	1997-11-19			2.80	CSIR survey
11	1998-05-19	01:15	2.52		DWS Estuarine water levels
12	1999-09-30			3.20	CSIR survey
13	2000-09-16	07:09	2.41		DWS Estuarine water levels
14	2003-08-26	12:06	2.80		
15	2006-08-17	01:38	2.62		
16	2008-09-28	23:49	2.68		
17	2009-07-14	10:05	2.40		
18	2012-08-17	22:03	2.96		
19	2013-08-18	20:37	2.59		
20	2014-06-28	09:27	2.05		
21	2015-10-01	10:03	2.47		

Onrus Estuary			G4H011		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1995-03-28	17:19	1.85		DWS Estuarine water levels
2	1995-04-22	20:40	1.79		
3	1995-05-02	05:41	1.81		
4	1995-06-14	15:40	2.17		
5	1995-07-12	21:10	1.99		
6	1995-07-19	14:10	2.12		
7	1995-10-27	03:47	1.83		
8	1996-09-02	23:30	1.97		
9	1996-09-27	14:32	1.98		
10	1996-10-20	11:10	1.62		
11	1997-04-08	11:49	2.00		
12	1997-05-19	02:42	1.95		
13	1997-10-15	11:39	2.04		
14	1997-11-19	09:51	2.05		
15	1998-04-06	22:29	2.05		
16	1998-09-12	22:42	2.10		
17	1998-10-03	13:31	2.05		
18	1998-10-17	18:08	1.83		
19	1998-11-09	02:48	2.07		
20	1998-12-15	10:05	1.91		
21	1999-01-04	10:22	1.67		
22	1999-02-11	08:03	1.49		
23	1999-04-09	09:27	1.91		
24	1999-04-21	15:37	1.80		
25	1999-05-22	12:15	1.75		

26	1999-06-24	19:08	2.02
27	1999-07-06	06:37	1.75
28	1999-07-22	18:46	1.99
29	1999-08-20	10:05	1.96
30	1999-08-31	21:12	1.81
31	1999-11-20	09:20	1.90
32	2000-03-17	21:01	1.84
33	2000-04-30	06:41	1.56
34	2000-05-25	18:06	1.65
35	2000-06-04	23:23	1.70
36	2000-07-16	12:53	1.97
37	2000-07-20	21:36	2.07
38	2000-08-01	18:48	2.01
39	2000-09-06	07:59	2.19
40	2000-10-03	18:56	1.94
41	2000-11-08	14:02	1.51
42	2000-12-09	19:58	1.53
43	2001-07-03	23:38	2.31
44	2001-08-22	11:36	2.43
45	2001-09-21	04:06	1.86
46	2001-10-28	11:25	1.96
47	2001-11-19	01:49	1.42
48	2002-04-29	21:08	2.10
49	2002-06-21	12:27	1.96
50	2002-07-26	13:57	1.80
51	2002-12-10	17:12	2.06
52	2003-02-03	15:27	1.26
53	2003-04-19	13:30	2.14
54	2003-10-28	15:59	1.88
55	2004-06-14	10:21	2.27
56	2004-07-24	00:18	2.01
57	2004-08-20	13:43	1.73
58	2004-09-10	05:54	1.73
59	2004-10-03	20:41	1.80
60	2004-11-25	18:29	1.64
61	2005-04-11	08:11	1.96
62	2005-10-27	09:59	1.39
63	2005-11-12	15:48	1.91
64	2006-04-06	18:18	2.27
65	2006-04-22	13:51	1.80
66	2006-05-03	16:39	1.86
67	2006-05-16	15:36	1.71
68	2006-06-03	00:48	2.06
69	2006-06-16	13:08	1.78
70	2007-02-13	12:41	1.85
71	2007-03-10	21:48	2.01
72	2007-04-06	21:08	1.63
73	2007-04-27	22:11	2.08
74	2007-05-20	02:54	1.99
75	2007-06-11	01:19	1.79
76	2007-11-21	17:17	2.01
77	2008-04-12	17:16	1.74
78	2008-04-20	19:54	1.75
79	2008-05-19	01:24	2.12
80	2009-02-01	13:02	1.58
81	2009-02-15	09:54	1.55
82	2009-03-20	09:17	1.75
83	2009-05-31	17:15	2.09

84	2009-06-24	05:55	2.27		
85	2010-01-26	20:08	1.50		
86	2010-03-01	00:52	1.98		
87	2010-05-16	09:21	2.18		
88	2010-08-15	01:49	2.03		
89	2010-09-15	10:27	1.99		
90	2010-10-15	17:37	1.99		
91	2010-10-28	10:42	1.88		
92	2010-11-09	09:19	1.69		
93	2010-12-30	16:27	1.55		
94	2011-05-04	19:48	2.14		
95	2011-06-05	03:57	2.11		
96	2011-06-19	21:12	2.19		
97	2011-06-27	16:11	2.09		
98	2011-10-08	05:37	2.25		
99	2011-11-13	12:27	1.87		
100	2011-12-12	20:29	1.58		
101	2012-06-08	15:29	2.48		
102	2012-06-25	17:02	1.77		
103	2012-07-07	00:57	1.81		
104	2012-07-11	08:16	1.79		
105	2012-07-23	11:43	1.77		
106	2012-09-21	13:05	1.82		
107	2012-10-19	20:35	1.98		
108	2012-11-18	01:53	1.99		
109	2013-02-14	04:38	1.97		
110	2013-03-30	21:44	2.10		
111	2013-04-19	17:53	2.09		
112	2013-05-05	06:16	1.90		
113	2013-05-19	04:26	1.82		
114	2013-06-26	19:59	2.20		
115	2014-01-06	02:48	1.85		
116	2014-02-16	11:33	1.65		
117	2014-04-02	07:02	1.73		
118	2014-04-25	12:01	1.81		
119	2014-05-22	18:55	1.92		
120	2014-06-03	21:08	2.08		
121	2014-11-08	12:01	1.89		
122	2014-12-26	09:29	2.16		
123	2015-04-05	23:19	2.19		
124	2015-04-26	18:45	1.61		
125	2015-05-12	21:30	1.61		

Klein Estuary			**G4H006**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1980-11-18	13:39	1.71		DWS Estuarine water levels
2	1981-02-06	19:19	1.56		
3	1981-07-25	13:56	1.95		
4	1982-09-05	05:53	1.74		
5	1983-07-01	10:04	1.95		
6	1984-09-09	20:06	2.35		
7	1986-08-30	16:42	2.71		
8	1987-10-04	11:43	2.50		
9	1988-11-09	18:49	1.90		
10	1989-06-24	23:18	1.74		
11	1990-06-11	21:29	2.10		

12	1991-08-03	03:47	2.52		
13	1991-10-31	18:23	1.85		
14	1992-07-18	07:30	1.80		
15	1993-04-17	23:39	2.31		
16	1994-06-29	18:29	2.29		
17	1995-07-28	20:42	2.36		
18	1996-09-27	20:19	2.26		
19	1997-07-02	14:59	2.67		
20	1998-06-04			2.60	CSIR survey
21	1998-06-11	17:35	2.63		DWS Estuarine water levels
22	1998-12-15	12:35	2.39		
23	1999-09-27	14:42	2.63		
24	2000-10-31	07:38	2.48		
25	2001-09-28	17:41	2.55		
26	2003-08-19	19:34	2.67		
27	2005-04-12	06:47	2.47		
28	2006-08-17	03:42	2.74		
29	2007-08-08	23:36	2.70		
30	2008-09-27	01:05	2.26		
31	2009-07-14	14:29	1.97		
32	2011-09-09	14:54	2.78		
33	2012-08-14	05:11	2.81		
34	2013-08-10	23:30	2.70		
35	2014-06-27	17:06	2.63		
36	2015-07-28	13:52	2.77		

Hartenbos Estuary			**K1T010**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1984-04-16			2.30	CSIR survey
2	1988-02-07			2.40	
3	1988-08-08			2.20	
4	1989-07-01			2.00	
5	1993-10-15	17:14	1.65		DWS Estuarine water levels
6	1993-12-17	13:36	1.90		
7	1994-02-28	11:12	1.77		
8	1994-05-01	19:31	1.86		
9	1994-06-08	11:33	1.65		
10	1994-07-12	11:47	1.88		
11	1994-10-27	11:59	1.78		
12	1994-12-24	02:23	1.84		
13	1995-04-13	02:06	1.98		
14	1995-05-11	00:36	1.64		
15	1995-07-19	10:02	1.95		
16	1995-09-08	08:57	1.88		
17	1995-11-29	04:24	1.79		
18	1996-04-09	16:58	1.73		
19	1996-05-02	11:35	1.55		
20	1996-07-06	17:17	1.85		
21	1996-09-15	18:13	1.99		
22	1996-11-21	21:43	1.83		
23	1997-04-12	10:18	1.89		
24	1997-07-01	12:36	1.83		
25	1997-07-11	13:28	1.22		
26	1997-08-03	01:46	1.36		
27	1997-09-10	06:57	1.60		
28	1997-10-30	16:08	1.75		

29	1997-11-29	12:18	1.78
30	1998-01-08	01:41	1.74
31	1998-03-30	12:17	1.88
32	1998-04-23	02:53	1.91
33	1998-07-21	01:54	1.62
34	1998-08-14	12:09	1.57
35	1998-09-03	00:09	1.68
36	1998-09-24	19:00	1.73
37	1999-03-19	10:01	1.99
38	1999-05-12	02:12	1.54
39	1999-08-27	10:17	1.78
40	1999-12-20	09:54	1.79
41	2000-02-29	08:47	1.53
42	2000-03-06	08:17	1.43
43	2000-03-30	08:20	1.48
44	2000-06-08	17:42	1.72
45	2000-07-31	18:15	1.80
46	2000-09-06	12:11	1.79
47	2000-11-08	18:30	1.73
48	2000-11-29	18:31	1.41
49	2001-02-06	10:19	1.67
50	2001-04-03	16:32	1.82
51	2001-04-17	09:34	1.89
52	2001-06-14	19:39	1.79
53	2001-07-30	15:27	1.78
54	2001-09-20	11:20	1.81
55	2001-10-19	10:12	2.03
56	2001-11-29	11:18	1.72
57	2001-12-19	00:40	1.56
58	2002-01-31	23:41	1.73
59	2002-03-20	19:36	1.60
60	2002-04-16	17:48	1.87
61	2002-05-25	03:59	2.28
62	2002-07-18	13:36	1.99
63	2002-08-16	01:33	1.84
64	2002-08-30	04:24	1.55
65	2002-11-05	00:33	2.01
66	2003-03-25	13:29	2.08
67	2003-05-11	01:52	2.00
68	2003-06-07	21:10	1.46
69	2003-06-22	04:17	1.62
70	2003-07-11	11:59	1.52
71	2003-08-24	14:12	1.87
72	2003-10-07	03:52	2.01
73	2003-12-12	09:06	1.71
74	2004-01-09	04:11	1.69
75	2004-04-17	00:01	2.01
76	2004-05-21	14:33	1.96
77	2004-06-17	20:43	1.79
78	2004-07-13	23:14	1.74
79	2004-07-25	16:12	1.40
80	2004-08-09	10:16	1.55
81	2004-08-22	09:50	1.72
82	2004-09-25	09:41	1.64
83	2004-10-07	20:51	1.58
84	2004-12-07	06:11	1.83
85	2005-04-16	00:10	1.51
86	2005-05-12	10:35	1.57

87	2005-06-12	17:52	1.84
88	2005-06-22	11:01	1.61
89	2005-07-14	16:05	1.42
90	2005-08-14	05:29	1.82
91	2005-10-16	17:14	1.97
92	2005-11-16	14:08	1.69
93	2005-12-31	11:48	1.67
94	2006-03-04	11:12	1.79
95	2006-04-03	10:34	1.55
96	2006-05-05	20:07	1.95
97	2006-05-22	23:27	1.72
98	2006-06-04	02:56	1.66
99	2006-06-22	22:11	1.57
100	2006-07-20	17:58	1.90
101	2007-03-05	12:23	2.36
102	2007-06-10	12:19	1.62
103	2007-06-24	14:06	1.58
104	2007-07-01	01:36	1.46
105	2007-07-22	00:55	1.60
106	2007-07-29	05:21	1.97
107	2007-10-03	12:14	1.86
108	2007-11-22	12:53	2.23
109	2008-06-27	10:42	1.76
110	2008-07-11	06:03	1.67
111	2008-08-29	03:39	2.00
112	2008-11-13	18:05	2.18
113	2009-06-13	06:51	2.15
114	2009-08-26	02:59	1.93
115	2009-09-11	21:58	1.73
116	2009-11-21	08:23	1.95
117	2010-05-04	19:40	2.12
118	2010-06-13	02:38	1.90
119	2010-07-27	15:58	1.96
120	2010-10-25	09:28	2.24
121	2011-03-04	15:10	1.64
122	2011-05-30	12:05	1.99
123	2011-11-28	21:34	1.57
124	2012-02-03	16:32	1.98
125	2012-03-19	00:05	1.78
126	2012-04-20	21:51	1.86
127	2012-05-29	14:20	1.87
128	2012-06-17	23:51	1.94
129	2012-07-13	16:00	1.91
130	2013-04-01	18:50	1.87
131	2013-06-04	14:51	2.09
132	2013-08-08	12:19	2.03
133	2013-10-12	10:47	1.54
134	2013-10-28	11:05	1.62
135	2014-03-17	03:34	1.95
136	2014-04-23	05:03	1.55
137	2014-05-23	12:15	1.64
138	2014-06-04	09:45	1.62
139	2014-07-18	13:53	1.69
140	2014-09-15	19:53	2.03
141	2014-10-15	06:28	1.98
142	2014-11-27	11:49	2.03
143	2014-12-31	08:07	2.01
144	2015-02-13	16:24	1.62

145	2015-04-12	02:14	2.01		
146	2015-04-24	23:19	1.62		
147	2015-05-29	08:08	1.90		
148	2015-06-20	13:32	1.43		
149	2015-07-09	23:59	1.45		
150	2015-07-20	18:15	1.54		
151	2015-11-01	15:39	1.84		

Klein Brak Estuary			**K1T020**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1988-08-11			1.60	CSIR survey
2	1989-07-06			1.70	
3	1991-09-26			2.40	
4	2005-09-10	07:38	1.34		DWS Estuarine water levels
5	2006-05-02	05:14	1.71		

Groot Brak Estuary			**K2T004**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1988-06-02	01:17	1.97		DWS Estuarine water levels
2	1988-07-15	14:57	1.97		
3	1988-08-03	19:21	1.91		
4	1988-08-30	10:34	2.11		
5	1988-12-14			1.40	CSIR survey
6	1989-04-14	22:42	1.98		DWS Estuarine water levels
7	1989-07-07			2.80	CSIR survey
8	1989-07-07	22:21	2.08		DWS Estuarine water levels
9	1989-10-03	20:30	2.29		
10	1990-05-23	19:47	2.06		
11	1990-06-09			1.70	CSIR survey
12	1990-06-27			2.30	
13	1990-10-02	18:25	1.76		DWS Estuarine water levels
14	1990-11-30	17:16	2.04		
15	1991-03-14	15:55	2.14		
16	1991-07-09	18:19	1.95		
17	1991-10-30	23:12	2.19		
18	1991-12-18	15:28	2.09		
19	1992-03-17	17:35	2.20		
20	1992-05-06	11:21	2.29		
21	1992-06-26	16:11	2.18		
22	1992-08-10	17:00	2.17		
23	1992-10-16		1.89		
24	1993-05-18	17:02	1.78		
25	1993-06-02	18:08	1.91		
26	1993-07-16	18:34	1.81		
27	1993-09-13	15:54	1.67		
28	1994-02-05	10:29	1.81		
29	1994-05-20	14:42	1.78		
30	1994-07-19	15:12	1.89		
31	1994-08-03	01:36	1.94		
32	1995-07-18	10:57	1.87		
33	1995-09-05	13:59	1.87		
34	1996-10-10	14:34	1.87		
35	1997-07-22	00:37	1.63		

36	1997-08-26	15:48	2.04		
37	1998-03-25	17:14	1.51		
38	1998-04-28	12:08	1.71		
39	1998-09-04	17:00	1.83		
40	1998-11-13	13:47	1.98		
41	1999-03-13	16:47	2.07		
42	1999-06-24	12:06	2.00		
43	1999-09-21	15:54	2.02		
44	1999-11-12	00:21	1.76		
45	2000-03-01	12:38	1.89		
46	2000-09-22	11:47	1.95		
47	2001-01-02	13:29	1.69		
48	2001-02-14	22:06	1.76		
49	2001-09-13	13:29	1.97		
50	2002-05-25	03:53	2.21		
51	2002-07-23	20:38	1.91		
52	2002-08-23	21:27	1.74		
53	2002-09-04	22:35	1.84		
54	2002-11-04	15:45	2.13		
55	2002-12-17	15:38	1.83		
56	2003-02-13	15:02	1.97		
57	2003-03-25	06:09	2.25		
58	2003-08-22	13:05	1.99		
59	2004-04-01	14:36	1.96		
60	2004-05-07	09:35	1.88		
61	2004-09-23	12:48	2.01		
62	2004-10-12	10:13	1.67		
63	2005-09-29	13:48	2.01		
64	2005-10-27	03:09	1.33		
65	2006-08-01	18:24	2.25		
66	2007-02-15	05:20	1.74		
67	2007-09-25	14:23	1.94		
68	2008-09-01	19:01	2.42		
69	2008-11-13	19:23	1.79		
70	2009-03-24	14:11	1.96		
71	2009-07-03	14:56	1.95		
72	2011-02-01	19:37	2.03		
73	2011-06-08	13:41	2.94		
74	2012-07-14	16:18	2.05		
75	2013-08-09	09:13	1.88		
76	2013-08-28	10:54	1.99		
77	2013-09-03	07:25	1.53		
78	2013-10-22	01:27	1.71		
79	2014-09-23	13:42	2.04		
80	2014-11-18	05:02	1.64		
81	2015-04-01	15:17	2.00		
82	2015-06-09	15:52	1.77		
83	2015-06-24	14:47	1.94		
84	2015-07-21	16:27	1.92		

Touw Estuary			**K3T006**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1994-08-03	01:57	2.04		DWS Estuarine water levels
2	1994-12-24	12:36	1.62		
3	1995-11-30	06:41	2.05		
4	1996-10-22	10:35	1.82		

5	1997-10-26	17:11	1.66		
6	1998-03-27	22:44	1.79		
7	1999-04-19	22:37	1.75		
8	1999-10-19	02:57	2.06		
9	2000-01-12	17:51	1.77		
10	2000-03-02	16:24	1.58		
11	2000-11-12	00:53	2.09		
12	2001-03-18	13:07	1.68		
13	2002-01-31	12:30	1.79		
14	2002-05-25	06:28	1.92		
15	2002-09-10	06:28	2.18		
16	2003-03-25	08:41	2.19		

Piesang Estuary			**K6T021**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	2010-12-18	06:09	1.49		DWS Estuarine water levels
2	2011-05-07	17:59	1.91		
3	2011-06-08	09:36	1.57		
4	2012-07-14	09:54	1.57		
5	2013-08-08	07:41	1.54		
6	2014-05-24	17:42	1.62		
7	2014-06-15	18:12	1.91		
8	2014-06-21	09:11	1.48		
9	2014-08-29	04:59	1.77		
10	2014-11-16	15:48	1.56		
11	2015-04-05	06:42	2.02		

Groot Estuary			**K7T002**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	2002-10-14	04:02	2.09		DWS Estuarine water levels
2	2002-11-04	17:50	2.41		
3	2002-11-24	10:40	1.97		
4	2003-02-08	15:00	2.10		
5	2003-03-19	03:03	2.15		
6	2003-06-26	07:25	1.75		
7	2003-07-04	23:47	1.84		
8	2003-07-17	09:54	1.86		
9	2003-07-24	14:58	1.60		
10	2003-08-22	21:48	2.45		
11	2003-09-06	08:53	1.89		
12	2003-12-15	09:28	1.99		
13	2004-01-07	11:47	2.04		
14	2004-01-29	11:00	1.82		
15	2004-02-24	20:02	2.42		
16	2004-07-13	08:18	2.35		
17	2004-08-21	04:31	2.38		
18	2004-09-03	11:34	2.08		
19	2004-12-06	01:25	1.95		
20	2005-05-08	04:41	2.06		
21	2005-06-01	02:34	1.96		
22	2005-06-14	22:16	2.48		
23	2005-07-07	17:13	2.02		
24	2005-08-25	11:46	2.41		
25	2005-10-01	10:36	2.22		

26	2005-11-06	16:35	2.39
27	2005-12-08	03:23	2.03
28	2005-12-14	10:37	1.83
29	2005-12-22	02:10	1.90
30	2006-02-28	05:22	1.79
31	2006-07-24	06:53	2.10
32	2006-09-26	10:12	2.00
33	2006-11-20	20:41	1.74
34	2006-12-04	11:08	1.76
35	2007-02-25	00:25	2.10
36	2007-03-05	14:28	2.43
37	2007-05-21	05:47	2.39
38	2007-07-23	01:45	2.04
39	2007-07-31	15:59	2.73
40	2007-08-17	09:25	1.98
41	2007-08-24	04:21	1.63
42	2007-09-12	18:34	2.02
43	2007-10-01	00:40	2.04
44	2008-05-24	21:28	2.30
45	2008-06-08	18:54	2.30
46	2008-06-26	17:55	2.37
47	2008-07-28	09:18	2.70
48	2008-08-16	23:39	2.80
49	2008-09-29	21:49	2.73
50	2008-10-08	14:53	2.93
51	2009-02-18	00:45	2.57
52	2009-04-07	20:23	2.53
53	2009-04-25	12:40	2.41
54	2009-06-05	16:29	2.74
55	2009-07-23	19:08	2.87
56	2009-08-27	01:01	2.68
57	2009-09-25	17:58	2.80
58	2009-10-13	10:22	2.51
59	2009-12-09	13:13	2.75
60	2010-02-25	10:05	2.92
61	2010-04-23	08:32	2.91
62	2010-05-11	03:54	3.16
63	2010-08-19	11:19	2.83
64	2010-09-04	02:36	2.30
65	2010-10-14	15:12	3.15
66	2011-01-17	23:01	2.17
67	2011-03-18	15:48	2.39
68	2011-04-09	13:49	2.30
69	2011-05-03	14:40	2.70
70	2011-06-08	06:54	2.59
71	2011-07-24	20:31	2.39
72	2011-09-20	03:59	2.62
73	2011-10-02	16:07	2.94
74	2011-11-16	10:08	2.47
75	2011-11-24	11:37	2.29
76	2012-02-15	07:27	2.49
77	2012-02-29	14:41	2.59
78	2012-03-31	02:48	3.05
79	2012-06-13	19:16	2.87
80	2012-07-11	22:27	3.00
81	2012-09-22	20:34	2.54
82	2012-10-11	07:38	2.78
83	2013-01-17	00:10	2.49

84	2013-01-31	11:12	2.42		
85	2013-06-09	18:10	2.80		
86	2013-06-27	23:11	2.72		
87	2013-08-08	16:10	2.73		
88	2013-08-21	03:13	2.86		
89	2013-09-19	20:52	2.72		
90	2013-10-06	09:10	2.81		
91	2014-04-03	08:26	1.95		
92	2014-05-15	17:30	1.86		
93	2014-06-06	13:52	2.41		
94	2014-06-28	23:58	2.13		
95	2014-07-23	18:04	2.39		
96	2014-08-11	01:12	1.89		
97	2014-09-02	15:52	2.12		
98	2014-09-29	14:29	2.14		
99	2014-11-04	23:55	1.71		
100	2014-11-19	21:57	1.90		
101	2014-11-26	04:07	1.80		
102	2014-12-26	08:12	2.24		
103	2015-02-06	00:21	2.12		
104	2015-03-17	11:22	2.07		
105	2015-06-18	02:48	1.92		
106	2015-06-28	06:40	1.70		
107	2015-07-09	12:13	1.90		
108	2015-09-01	06:24	2.77		
109	2015-11-01	21:06	2.07		
110	2016-01-07	11:55	1.73		
111	2016-02-03	01:28	1.65		
112	2016-03-11	11:43	2.25		
113	2016-04-15	13:46	1.93		
114	2016-05-02	01:13	2.12		
115	2016-05-19	06:57	1.75		
116	2016-06-20	13:48	2.31		
117	2016-07-21	14:14	2.32		
118	2016-08-11	07:54	1.82		
119	2016-09-03	07:10	2.60		
120	2016-11-08	18:24	1.71		

Tsitsikamma Estuary			**K8T004**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1995-02-04	13:42	1.48		DWS Estuarine water levels
2	1995-02-17	13:18	1.42		
3	1995-02-23	19:12	1.46		
4	1995-06-24	23:11	1.66		
5	1995-07-01	12:58	1.37		
6	1995-07-11	15:53	2.02		
7	1995-07-27	18:35	2.12		
8	1995-08-22	21:23	1.69		
9	1995-09-29	18:10	1.63		
10	1995-10-14	13:49	1.91		
11	1995-11-30	13:21	1.76		
12	1996-01-17	17:57	1.71		
13	1996-02-05	01:17	1.58		
14	1996-03-21	16:34	1.73		
15	1996-04-22	06:11	2.02		
16	1996-05-26	16:06	1.40		

17	1996-07-16	01:43	2.66
18	1996-08-06	14:06	1.88
19	1996-09-14	04:33	2.28
20	1996-10-22	14:36	1.87
21	1997-02-14	17:06	1.71
22	1997-02-27	23:47	1.41
23	1997-05-27	18:20	1.69
24	1997-11-24	09:46	2.51
25	1998-03-21	07:27	1.99
26	1998-04-03	18:54	1.41
27	1998-05-13	00:03	2.25
28	1988-06-09	02:46	1.83
29	1998-07-15	12:05	2.27
30	1998-07-21	08:11	1.53
31	1998-07-30	00:36	1.48
32	1998-09-10	11:59	1.94
33	1998-09-28	17:08	2.12
34	1998-11-08	11:54	1.94
35	1998-11-25	09:06	1.62
36	1999-01-13	12:57	2.12
37	1999-06-20	04:45	2.13
38	1999-07-30	03:17	2.64
39	1999-08-05	23:54	2.03
40	1999-09-01	14:49	2.16
41	1999-09-18	18:59	2.34
42	1999-09-27	20:19	1.96
43	1999-10-13	08:48	1.28
44	2000-03-05	08:49	2.20
45	2000-03-15	02:45	1.28
46	2000-03-26	19:08	1.31
47	2000-07-31	22:38	2.67
48	2000-09-18	17:10	2.43
49	2000-11-05	17:59	2.11
50	2001-01-16	02:06	2.20
51	2001-03-22	09:25	1.69
52	2001-07-08	00:16	2.49
53	2001-07-26	04:25	1.94
54	2001-08-03	08:30	2.65
55	2001-11-21	11:42	1.60
56	2002-05-21	08:17	2.11
57	2002-06-27	07:48	2.33
58	2002-07-11	20:35	2.36
59	2002-10-10	09:18	2.06
60	2002-11-08	15:49	2.30
61	2002-11-24	00:18	1.78
62	2003-03-25	21:50	2.44
63	2003-05-10	18:17	1.68
64	2003-08-03	08:19	1.57
65	2003-10-17	11:25	1.37
66	2004-01-14	07:51	1.91
67	2004-05-06	03:45	2.50
68	2004-06-10	13:05	2.04
69	2004-07-05	21:11	2.26
70	2004-09-03	23:35	2.31
71	2004-09-20	03:07	1.74
72	2004-11-03	04:40	1.53
73	2004-12-08	07:12	1.92
74	2004-12-22	13:37	1.38

75	2005-04-29	11:02	1.59
76	2005-06-20	01:31	2.43
77	2005-09-13	14:34	2.71
78	2006-01-10	04:22	2.50
79	2006-04-24	02:23	2.16
80	2006-08-02	19:22	2.28
81	2006-12-01	14:36	3.90
82	2007-05-01	23:12	1.77
83	2007-05-21	04:58	2.16
84	2007-06-28	17:51	1.62
85	2007-07-13	19:24	1.85
86	2007-11-23	12:03	2.15
87	2008-01-26	16:04	1.25
88	2008-02-14	01:03	1.53
89	2008-03-17	09:19	1.72
90	2008-06-02	10:09	1.90
91	2008-08-16	12:41	2.37
92	2008-09-14	00:42	2.29
93	2008-10-10	03:42	2.17
94	2008-11-15	04:57	2.78
95	2009-06-26	01:20	3.10
96	2009-07-24	12:15	1.90
97	2009-10-13	10:39	2.61
98	2010-06-15	11:13	2.55
99	2010-06-28	04:49	1.47
100	2010-07-12	05:12	2.12
101	2010-08-12	23:29	1.81
102	2010-10-28	10:10	2.27
103	2010-12-18	08:13	1.58
104	2011-03-20	15:36	2.02
105	2011-05-07	18:35	2.11
106	2011-05-25	19:07	1.54
107	2011-11-02	23:06	1.86
108	2011-11-16	06:03	1.93
109	2012-04-12	12:46	2.41
110	2012-05-29	20:52	2.09
111	2013-01-07	12:55	1.74
112	2013-03-25	12:13	2.02
113	2013-05-06	05:55	1.94
114	2013-06-10	15:40	2.06
115	2013-07-03	22:29	2.27
116	2013-08-09	02:05	2.29
117	2013-09-13	20:50	1.84
118	2013-10-05	12:14	2.45
119	2014-04-04	16:27	2.32
120	2014-04-27	20:53	1.67
121	2014-05-18	21:14	1.76
122	2014-06-06	14:14	2.41
123	2014-07-01	19:55	1.54
124	2014-07-09	14:21	1.75
125	2014-08-31	01:45	2.07
126	2014-09-25	18:21	2.00
127	2014-11-10	20:39	1.68
128	2014-11-26	05:45	1.61
129	2015-02-15	22:46	1.87
130	2015-03-04	01:40	1.63
131	2015-04-05	11:36	1.91
132	2015-05-10	05:48	1.73

133	2015-06-01	17:35	1.93		
134	2016-03-11	22:11	1.82		
135	2016-04-04	07:52	2.22		
136	2016-05-03	09:37	2.19		
137	2016-07-03	00:03	2.39		
138	2016-07-09	06:04	1.84		
139	2016-07-24	20:11	1.78		
140	2016-08-19	11:26	1.54		
141	2016-09-04	03:15	1.60		
142	2016-10-02	22:37	1.93		
143	2016-11-22	21:47	1.97		

Seekoei Estuary			**K9T009**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1988-07-15			1.70	CSIR survey
2	1989-07-02			1.60	
3	1991-09-20			1.60	
4	1996-07-25			1.90	
5	2003-08-27	13:42	1.57		DWS Estuarine water levels
6	2004-12-26	10:18	1.92		
7	2005-05-11	04:06	2.00		
8	2006-05-30	19:33	1.50		
9	2006-08-02	19:35	1.79		
10	2007-03-05	20:43	1.52		
11	2009-06-27	23:16	1.66		
12	2009-08-28	12:37	1.52		
13	2011-06-09	03:24	2.41		

West Kleinemonde Estuary			**NA (Only surveys)**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	1986-02-28			1.80	CSIR survey
2	1986-11-17			1.30	
3	1988-01-31			2.00	
4	1988-07-07			2.10	
5	1991-09-17			2.40	
6	1992-10-01			2.60	
7	1996-07-30			2.50	
8	1999-03-08			2.70	

East Kleinemonde Estuary			**P4T002**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	2005-04-11	09:23	1.43		DWS Estuarine water levels
2	2005-05-03	08:33	1.10		
3	2005-11-08	04:57	2.22		
4	2006-06-23	12:04	2.16		
5	2006-08-03	17:30	1.60		
6	2006-09-27	08:47	1.50		
7	2006-12-10	18:38	1.34		
8	2007-03-19	09:07	2.04		
9	2007-05-23	11:43	1.81		

10	2007-06-18	22:28	1.73		
11	2008-09-01	17:36	2.71		
12	2011-05-10	10:53	2.45		
13	2011-05-25	22:43	1.41		
14	2011-07-05	06:17	1.58		
15	2011-11-28	06:23	1.77		
16	2011-12-17	06:05	1.17		
17	2011-12-30	02:41	1.08		
18	2012-02-04	06:12	1.51		
19	2012-07-14	03:45	2.08		
20	2012-08-12	06:11	1.86		
21	2012-09-06	12:58	2.03		
22	2013-08-18	05:54	2.00		
23	2013-10-26	12:51	2.16		
24	2013-11-17	03:01	1.41		
25	2013-12-21	09:50	1.47		
26	2014-04-24	03:15	1.88		
27	2014-05-24	04:20	1.24		
28	2015-06-02	02:00	1.97		
29	2015-06-18	10:19	1.65		
30	2015-07-25	14:11	1.80		
31	2015-10-01	11:06	1.67		

Mngazi Estuary			**T7T004**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	2004-01-14	20:54	1.13		DWS Estuarine water levels
2	2004-01-24	15:04	1.05		
3	2004-03-24	01:16	1.18		
4	2004-08-26	14:15	1.27		
5	2005-06-27	22:27	1.35		
6	2005-11-06	11:54	1.94		
7	2006-07-19	20:29	1.43		
8	2007-07-07	03:18	1.40		
9	2007-08-09	08:22	2.19		
10	2007-10-05	07:41	1.84		
11	2007-11-22	04:32	1.69		
12	2007-12-08	08:15	1.04		
13	2008-10-10	01:28	1.54		
14	2009-06-29	08:37	2.37		
15	2011-11-17	23:30	1.79		
16	2012-07-16	21:05	1.96		
17	2012-08-08	06:28	2.22		
18	2014-08-12	07:38	2.03		
19	2014-12-12	20:18	1.81		
20	2015-08-21	11:10	2.08		
21	2015-12-09	19:29	2.15		

Mhlanga Estuary			**U3T010**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	2005-09-18	15:53	2.77		DWS Estuarine water levels
2	2005-10-09	16:11	2.45		
3	2005-10-19	09:05	2.06		
4	2005-10-26	18:47	1.61		
5	2005-11-06	16:23	2.56		

6	2005-11-13	06:35	2.48
7	2005-12-16	10:54	2.59
8	2005-12-25	19:38	2.49
9	2006-01-02	19:47	2.40
10	2006-01-21	17:29	2.71
11	2006-02-09	20:31	2.93
12	2006-02-23	13:17	2.72
13	2006-03-20	21:29	2.79
14	2006-04-23	00:03	2.87
15	2006-05-20	06:06	3.00
16	2006-06-02	19:11	2.62
17	2006-07-21	22:07	2.84
18	2006-08-20	16:52	2.84
19	2006-08-27	03:53	2.44
20	2006-09-19	12:52	2.97
21	2006-09-29	23:47	2.96
22	2006-10-15	12:32	2.26
23	2006-10-19	11:17	2.33
24	2006-11-10	22:41	2.65
25	2006-12-10	09:35	3.03
26	2007-01-10	15:04	2.28
27	2007-01-19	12:34	2.14
28	2007-02-10	22:47	2.44
29	2007-03-16	17:36	3.35
30	2007-03-30	07:14	2.48
31	2007-04-07	14:24	2.39
32	2007-04-19	01:30	2.67
33	2007-05-05	14:20	2.67
34	2007-05-19	23:56	2.58
35	2007-06-01	22:41	2.70
36	2007-06-10	18:25	2.21
37	2007-06-27	14:59	2.74
38	2007-07-09	18:23	2.49
39	2007-07-21	08:21	2.46
40	2007-08-19	02:13	2.86
41	2007-09-14	22:32	3.09
42	2007-12-30	16:16	2.62
43	2008-01-24	00:36	2.11
44	2008-03-14	13:00	2.80
45	2008-04-01	08:23	2.69
46	2008-04-24	23:12	2.88
47	2008-05-27	00:27	2.06
48	2008-06-13	18:47	2.39
49	2008-06-27	19:24	2.09
50	2008-07-06	00:43	2.32
51	2008-08-24	02:26	2.53
52	2008-09-21	08:49	2.77
53	2008-10-21	21:46	2.56
54	2008-11-15	10:00	2.64
55	2008-12-28	04:24	2.61
56	2009-01-08	05:16	2.14
57	2009-01-29	03:16	2.48
58	2009-02-28	15:22	2.38
59	2009-03-16	10:31	2.70
60	2009-04-03	12:41	2.04
61	2009-05-01	19:00	2.56
62	2009-06-02	20:28	2.63
63	2009-06-15	05:11	2.16

64	2009-07-06	17:18	2.57
65	2009-08-09	03:31	2.09
66	2009-08-30	13:29	2.85
67	2009-09-22	16:03	2.08
68	2010-01-22	12:19	2.75
69	2010-05-10	05:36	2.64
70	2010-06-02	22:06	2.06
71	2010-06-21	17:48	2.47
72	2010-07-28	11:10	2.80
73	2010-08-16	00:32	2.31
74	2010-08-27	14:02	2.22
75	2010-09-24	04:52	2.83
76	2010-10-20	16:04	2.88
77	2010-10-31	01:41	2.13
78	2010-11-11	02:49	2.70
79	2010-11-28	11:43	2.59
80	2011-02-26	18:53	2.60
81	2011-03-12	03:41	2.26
82	2011-03-20	21:32	2.13
83	2011-05-06	19:28	2.34
84	2011-06-08	16:28	2.32
85	2011-06-29	22:42	3.17
86	2011-07-11	22:38	3.08
87	2011-07-19	02:41	2.60
88	2011-07-24	17:29	2.40
89	2011-08-15	02:40	3.27
90	2011-09-10	07:26	3.82
91	2011-09-22	21:35	3.41
92	2011-10-03	01:40	3.55
93	2011-10-17	11:31	3.46
94	2012-01-18	21:40	2.71
95	2012-01-28	03:29	3.15
96	2012-02-28	00:09	2.92
97	2012-04-15	11:40	2.12
98	2012-05-11	01:11	2.97
99	2012-06-22	12:22	2.07
100	2012-07-07	14:16	2.50
101	2012-07-24	23:09	2.62
102	2012-08-07	11:03	2.34
103	2014-06-25	09:16	2.15
104	2014-07-17	21:41	2.31
105	2014-09-27	23:33	2.37
106	2014-10-17	06:53	2.98
107	2014-10-27	23:07	2.65
108	2014-11-12	13:01	2.63
109	2014-11-26	05:51	2.75
110	2014-12-10	22:35	2.20
111	2015-01-17	21:10	2.09
112	2015-01-28	00:56	1.92
113	2015-02-07	07:50	2.51
114	2015-02-21	01:30	2.51
115	2015-02-24	15:00	2.18
116	2015-03-27	12:14	3.17
117	2015-05-08	21:04	3.14
118	2015-06-14	18:04	2.61
119	2015-07-08	10:29	2.78
120	2015-07-23	01:36	3.15
121	2015-08-17	02:16	2.57

122	2015-09-16	19:12	2.38		
123	2015-09-28	18:59	2.28		
124	2015-10-15	18:46	2.42		
125	2015-10-30	22:44	2.48		
126	2016-02-12	01:32	2.71		
127	2016-02-17	22:13	1.94		
128	2016-03-02	05:59	2.76		
129	2016-03-15	06:29	2.74		
130	2016-04-06	06:52	2.09		
131	2016-04-25	22:19	2.50		
132	2016-05-08	10:37	3.11		
133	2016-05-27	22:55	2.71		
134	2016-07-01	17:46	2.73		
135	2016-07-14	01:30	2.10		
136	2016-08-11	00:34	2.60		

Mdloti Estuary			**U3T009**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	2006-05-02	05:47	2.51		
2	2006-05-21	02:14	2.98		
3	2006-10-09	02:13	3.23		
4	2006-10-14	08:28	2.32		
5	2006-10-19	09:23	1.78		
6	2006-11-15	00:23	2.80		
7	2006-12-10	16:36	3.10		
8	2006-12-21	22:31	2.27		
9	2007-03-19	19:17	2.98		DWS Estuarine water levels
10	2012-04-02	04:54	3.21		
11	2012-09-07	06:00	3.99		
12	2012-10-02	04:01	3.14		
13	2012-11-27	01:02	3.04		
14	2013-02-19	00:29	3.24		
15	2013-03-27	14:47	3.25		
16	2013-04-20	20:11	3.24		
17	2013-05-12	21:53	3.64		
18	2013-07-11	02:46	3.16		

Tongati Estuary			**U3T008**		
#	Date of breach/survey	Time of breach	Corrected water level at breach/berm height **(m above MSL)**	Berm saddle point from survey **(m above MSL)**	Source
1	2005-08-16	12:53	2.28		
2	2005-09-16	13:41	2.62		
3	2005-10-13	20:53	2.36		
4	2005-11-07	15:00	2.56		
5	2005-11-21	16:29	2.12		
6	2005-12-21	21:00	2.52		
7	2006-08-17	00:31	2.59		
8	2006-09-15	18:05	2.00		DWS Estuarine water levels
9	2006-11-08	21:41	2.23		
10	2007-02-19	23:05	2.53		
11	2007-03-08	04:05	2.94		
12	2007-03-19	05:30	3.24		
13	2007-05-26	13:19	2.00		
14	2007-09-07	14:17	2.20		
15	2007-09-14	21:24	1.92		

16	2008-09-13	18:11	2.28
17	2008-09-20	23:36	2.28
18	2008-09-28	18:05	1.97
19	2009-07-03	11:05	2.53
20	2009-07-29	03:00	2.23
21	2010-04-29	01:38	2.01
22	2010-05-15	22:20	1.71
23	2010-05-31	00:30	1.41
24	2010-08-07	03:22	1.26
25	2010-08-18	00:51	1.24
26	2010-09-04	16:16	1.01
27	2010-10-17	10:02	1.67
28	2010-10-29	16:41	1.19
29	2010-11-10	13:11	1.26
30	2013-07-29	07:51	1.49
31	2014-03-04	04:11	2.81
32	2014-08-31	02:05	2.44
33	2014-09-03	03:50	2.33
34	2014-09-29	16:01	2.15